REVIEW OF HYDROLOGY, HYDRAULIC DESIGN OF STILLING BASIN AND ENERGY DISSIPATION ARRANGEMENT OF ERACH DAM

A DISSERTATION

Submitted in the partial fulfillment of the requirements for award of the degree

of

MASTER OF TECHNOLOGY

in

WATER RESOURCE DEVELOPMENT

By

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MAY, 2018

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this thesis report entitled **"REVIEW OF HYDROLOGY, HYDRAULIC DESIGN OF SPILLWAY AND ENERGY DISSIPATION ARRANGEMENT OF ERACH DAM"** in partial fulfilment of the requirement for the award of the degree of Master of Technology with specialization in Water Resource Development, submitted to the department of Water Resource Development And Management, Indian Institute of Technology, Roorkee, India, is an authentic record of my work carried out under the supervision and guidance of **Dr. Deepak Khare** and co-guidance of **Mr. A.C.Pandey** and **Er. G.P.S Bhati.** The matter embodied in this dissertation report has not been submitted by me for the award of any other degree or diploma.

Date: 9th May 2018

Place: **Roorkee**

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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ACKNOWLEDGEMENT

I wish to express my deep sense of gratitude and sincere thanks to my guide **Dr. Deepak Khare,** Professor of Water Resources Development and Management department, IIT Roorkee and Co-guides **Mr. A.C.Pandey**, Former Research Officer (Incharge), IRI and **Er. G.P.S Bhati**, Retired Executive Engineer, U.P Irrigation Department for being helpful and a great source of inspiration. I would like to thank them for providing me with an opportunity to work on this excellent and innovative field of research. Their keen interest and constant encouragement gave me the confidence to complete my work. I wish to thank them for their constant guidance and suggestions without which I could not have successfully completed my thesis evaluation report.

I would also like to thank **Dr. S. K. Mishra**, HOD WRDM, IIT Roorkee for his constant support during my study. Also, I would like to thank all the teaching and non-teaching staff members of the department who have contributed directly or indirectly in successful completion of my thesis.

I am very much thankful to Central Water Commission for giving me this opportunity to study in this esteemed Institute for Master degree program.

I wish to extend my gratefulness to my parents, family members and friends who stood behind me and whose firm belief in my capabilities and morale boosting support. I also express my gratitude and love to my wife **Ruchita** for her motivation and inspiration with high mental support.

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ABBREVIATIONS

BIS	Bureau of Indian Standards
CCA	Culturable Command Area
CEA	Central Electricity Authority
CFD	Computational Fluid Dynamics
CWC	Central Water Commission
D/S	Downstream
EMPP	Erach Multipurpose Project
FFA	Flood Frequency Analysis
F.no.	Froude Number
FRL	Full Reservoir Level
G&D	Gauge and Discharge
GIS	Geographic Information System
HEC-RAS	Hydrologic Engineering Centre's River Analysis System
IS	Indian Standards
KE	Kinetic Energy
MDDL	Minimum Draw Down Level
MSL	Mean Sea Level
PE	Potential Energy
PRVs	Pressure Relieving Valves
SD	Standard Deviation
TRC	Tail Rating Curve
TWL	Tail Water Level
TD	Ten Daily
U/S	Upstream
USBR	United States Department of the Bureau of Reclamation

ABSTRACT

A case study of Erach Multipurpose Project (EMPP) has been taken to review the hydraulic design of stilling basin, energy dissipation arrangement and hydrological studies (Flood Frequency Analysis). As during construction of this project, the Tail Water Level (TWL) changed so in this study an attempt has been made to optimize the Energy Dissipaters without changing the design parameters.

Conventional methods are used for Flood frequency analysis of the Project. HEC-RAS Model is being used to analyse the hydraulics of the structure. Based on the results of hydraulic model, the calculations are made in different conditions to find out basin parameters and its optimization. The design of stilling basin should consider three interrelated parameters that are jump position, jump type and TWL. The hydraulic jump can be controlled by adding structure in the stilling basin or by stilling basin modification.

The hydraulic jump is used as an energy dissipation device for many spillways, outlet works, and canal structures**.** Performance of hydraulic jump in a basin is related to tail rating curve of the river. In situations where the tail depth is higher than the jump, no clear jump is expected and entering jet oscillates from bottom to surface resulting in pulsating action. In such cases the wavy action last in long reach downstream of spillway. Computation of basin parameters require setting of basins at very high almost infeasible level but after isolating the basin width by providing divide walls in basin the discharge intensity in the separated compartment is increased and good jump can take place even at comparatively lower seated basin floor. Thus, a good and stable flow regime of the hydraulic jump may be achieved in the divided stilling basin for a large range of frequently occurring low flow rates. The height of divide walls is decided by water depth in the basin for lower range of discharge up to 30% of the peak flood. The present thesis describes an arrangement in stilling basin with divide walls which can specially be applied for optimization of energy moderators, where tail water depth is quite high than the required jump depth.

Keywords: Hydraulic Jump, Energy Dissipation, Energy Moderators, Spillway, Tail rating curve, Discharge Intensity, Stilling Basin, Flood Frequency Analysis.

CHAPTER-I

1 INTRODUCTION

1.1 General

Multipurpose Water Resources Projects are planned for various purposes like irrigation, hydro power generation, water supply for drinking and industrial purpose, flood control, navigation, tourism etc. Projects which serves more than one purpose are called as multipurpose projects. Generally, majority of multipurpose projects are combination of irrigation and hydro-power. In some of the areas where water is very scarcely available during long duration of the year, storage projects are also planned for supplying the drinking water. Water resource projects involves a huge public investment, it is, therefore, very necessary to carry out the detailed hydrological and hydraulic studies to assess design parameters rationally. Hydrological and hydraulic inputs play a vital role in planning, designing and successful completion of a water resources project. The economics and sizing of every project is dependent on these parameters.

In hydrological study there are two main design parameters i) Assessment of the resource potential to design a water resources development project ii) Design flood estimation for the safety of any hydraulic structure. Design Flood Studies are important for safety of structure, flood control works, drainage works, diversion works, locating structures and outlets in vicinity of river bank/ reservoir, tail water rating curve. These studies shall require a long terms hydrological and hydro-meteorological data of good quality and sufficient quantity. The accuracy of hydrological assessments depends a lot on the quantity and quality of data availability. Output of any hydrological studies is as good as the input to the studies. Typical data requirements for hydrological studies are

a) Precipitation (rainfall, snowfall etc)
b) Gauges and discharges
c) Evaporation
d) Ground water levels/potential
e) Annual maximum discharge series
f) Minimum flows
g) Short interval (hourly) concurrent rainfall (SRRG) and discharges
h) Design storm (PMP, SPS, return period storm)
i) Time distribution of design storm -PMP, SPS at shorter time interval
j) Silt load (Suspended and bed load)
k) Shape and size of the reservoir

As per Guidelines for preparation of DPR of Irrigation and Multipurpose Project, MOWR, 2010 R, the minimum length of hydrological observations required is 10 years and frequency required is daily records for low flow (at 08 hrs) and thrice daily during high flow season (at 0800, 1300 and 1800 hrs). This much data is required for properly carrying out Hydrological studies.

The main problem being come to simulate flow in an ungauged site or poorly gauged streams or catchment. Absence of any hydrological data records or mere inadequacy of whatever information is available for catchments is generally caused by lack of foresight of planners to have potential sites gauged for harnessing the water resource of such catchments in the future. Apart from the headwater regions, many potential sites even in the downstream reaches in India also suffer from insufficiency of site-specific records of data. In our country wide networks of G&D stations are maintained by central and state water resource departments, but sometime data do not exist at locations where these are most needed. Lack of data often required alternate methods for accessing hydrology of ungauged site. With the growing demand to harness untapped potential of river water resources in many parts of the world, the need to devise new approaches and methodologies for assessment of water resources from these sources is also increasing. In the present thesis a case study of Erach Multipurpose project site of Betwa River was taken to analyses its hydrology. One of the important hydrological analysis is to calculate the design flood. Therefore, the analysis of design flood is very necessary to be carried out including the testing of goodness of fit for evaluating the suitable probability distribution which is used in the frequency analysis.

Statistics is the main discipline enabling the extraction of needed information from data and the derivation of conclusions about the characteristics of hydrologic random variables Statistical estimates are numerical properties of samples. They are necessary in statistical modelling, or for direct use in hydrology. To be effective in application of statistics in hydrology the civil engineer or hydrologist must understand the fundamentals of statistical methods which are employed in existing hydrologic techniques.

There are so many hydraulic parameters which need to be consider during Hydro Projects design. Of so many other hydraulic parameters, one important factor need to be consider for Hydro Projects is the proper Energy Dissipation arrangement below the structure. The kinetic energy (KE) gushing down the structure needs to be dissipated to prevent the fears of scouring of downstream river bed and the undermining of foundation which may cause failure of spillway and dam. For this purpose, energy dissipaters are used to perform the energy reduction by converting the kinetic energy into turbulence and finally into heat. Depending upon the site and the flow requisites the choice of energy dissipation device is meet. Stilling Basin, where the high energy loss occurs in a hydraulic jump has led to its universal adoption as a part of energy dissipater system below a structure. The formation of hydraulic jump in the stilling basin leads to dissipation of excess energy. In the stilling basin, the exiting supercritical flow from the spillway is reduced to subcritical flow by a hydraulic jump.

The flood discharge passing over the high spillway crest is associated with high amount of potential energy (PE), which is converted into kinetic energy (KE) as the flow glides over the spillway crest. This high energy of flow must be taken care of, before the discharge is returned to the main river channel in downstream, to check the unwanted erosion at the toe of the overflow structure. The objective is achieved through suitable arrangement provided in the downstream of spillways called 'Energy Moderators'. The choice of energy moderator below a spillway largely depends upon the problem at hand. The conversion of supercritical to subcritical flow with the provision of stilling basin is not an easy phenomenon but needs a detail consideration of several hydraulic parameters such as basin length, basin elevation and basin appurtenant.

Tail water level (TRC) plays a significant role in the formation of hydraulic jump. If the tail water depth is less than the sequent depth or the jump depth, the jump sweeps beyond the basin without any appreciable dissipation of energy but if the tail water depth is higher than the sequent depth, then the submerged or drowned jump will form producing wave actions and surface turbulence in the basin without much dissipation of energy. The Froude number plays a vital role in hydraulic jump theory. It is a dimensionless number which is directly proportional to the ratio of velocity V and depth D. Best results of Hydraulic jump are obtained when 1) tail depth is close to jump depth 2) The Froude Number of the flow in basin is in between 4.5-9.0.

1.2 Background of the Study

Erach Dam is proposed to be constructed on Betwa River near village-Jhujharpura of tehsil-Garotha of district-Jhansi (U.P.) about 41 Km downstream of Parichha Wier. The River Betwa is one of the main tributary of River Yamuna. The Erach Dam is a multipurpose project proposed for irrigation, drinking water supply & a little Power generation. The 1290-metre-long earthen dam has 492.5-metre-long overflow section with 24 vents of 18 m each. The spillway crest is at El. 148.7 m above MSL equipped with 12.3 m high vertical gates to facilitate ponding in the upstream. A small power house is also proposed at Dam toe having capacity of 1.8 MW power generation by two units of 0.9 MW each. Total irrigated CCA will be 1850 Ha after construction of dam. The Salient Features of the Project are as shown in Table 1.1.

Table 1.1. Salient Features of the Project

Name of the project	Erach Multipurpose Project (Jhujharpura Dam Phase-I)
Type of the project	Multipurpose with Irrigation, Drinking water supply & Power generation.
River	Betwa
Nearest Village/Tehsil/District/State	Jhujharpura Village, Garautha Tehsil, Jhansi District, Uttar Pradesh
Latitude	25°43'57.45"N
Longitude	79° 3'23.99"E
River diversion Type	Ring Coffer Dam in two phases
Spillway Type	Gated ogee crest supressed Spillway
Head over Crest	12.3 m
Length of Overflow including piers	492.50 m
No. of Spillway Gates	24
Design Discharge	18500 m^3/s
Non-overflow	365.5m long left bank earthen embankment 432 m long right bank earthen embankment
Power Block	20 m Bay Dam Toe Powerhouse
Top of Dam	El. 158.50 m
Top width of Dam to provide road over it	6.60 m
FRL	El. 156.50 m
MDDL	El. 148.70 m
Gross Storage	62.00 Mm^3
Live Storage	56.25 Mm^3
Storage for Drinking Water	29.89 Mm^3
Storage for Irrigation	9.55 Mm^3
Type of Power House	Dam toe
Installed Capacity	1.8 MW
No. & size of Units	2 units of 0.9 MW
Normal Tail Water Level	El. 138.50 m
CCA	1850 Ha

1.3 Research Gaps

The concept of stilling basin has been extensively researched and very definite guidelines are provided in literature. Most of the research workers have studied optimisation of basin and flow parameters and design guidelines before the construction of projects. Sometimes problems occur during the construction of project. These problems may be hydraulic, hydrological or geological and may call for alternative remedial measures. In case of stilling basin below a structure, the flow energy is negotiated by formation of "hydraulic jump". The hydraulic jump can be controlled by different methods.

In "A Hybrid Approach to Improve the Design of Stilling Basin" (Abdelazim 2010) it is stated that the function of these methods is to ensure the formation of a jump within the stilling basin and to control its position under all probable operating conditions. In other words, "to control" means to force the occurrence of the jump and to control its position, hence, reducing the risk of bed scour after the hydraulic structures. The design of such controlling structures should consider three interrelated parameters:

a) Position of jump
b) TWL
c) Type of jump

Mainly, the hydraulic jump can be controlled by these two categories:

a) By adding structures in the stilling basin
b) By stilling basin modifications.

Formation of a hydraulic jump in stilling basin depends upon the relation between jump depth and available water depth. Research workers {Patrika (1984), USBR (1973), Varshne (2014)} have proposed several appurtenant like chute blocks, end sill etc to help formation of jump in case, available water depth is less than the jump height which ultimately causes sweep out of the jump/ stilling basin. Stilling Basin Sweep out is a phenomenon in which "The tail water is insufficient to allow a hydraulic jump to develop or to be maintained" (United States department of Reclamation, 1984). The failure to preserve a hydraulic jump can cause one of the two failure modes of the Stilling Basin. The first starts with downstream erosion which leads to degradation of foundation material. The Second failure mode occurs when the tail water has overtopped the spillway and surrounded the stilling basin. In addition to this failure of the stilling basin sometimes can also transpires due to "floatation of stilling basin due to uplift pressure" (Unites States Department of Reclamation, 1984).

In another condition which is opposite to above, the tail water depth sometimes higher than the jump depth at some or most of the discharges. Under very exceptional conditions, the tail water curve is higher than the required jump depth curve for full range of discharges producing very low Froude Numbers in the stilling basin. For the formation of good hydraulic jump, the basin invert needs to be placed at very high, sometimes at infeasible elevation. Research workers advice to compromise with turbulent and wavy flow in the basin which may travel up to kilometres in the downstream. No published work is available to modify the stilling basin with some additional construction to facilitate formation of hydraulic jump up to certain range of frequently occurring discharges. An attempt has been made in the thesis to resolve the issue of high tail water depth by comparting the stilling basin with divide walls to insure hydraulic jump within the basin for some range of inflow floods.

1.4 Objectives of the Study

This study overall aims to review the hydraulic design of overflow structure, appurtenant works of Erach Dam under extraordinary situation where Tail Water level is raised remarkably during the construction stage. As the TWL changed afterwards so to review the hydrology is the initial requirement. The change scenario is causing additional submergence of the spillway crest and more submergence of the hydraulic jump even at low discharges. When no change in design is possible, then how to handle situation to get optimal possible hydraulic condition. Keeping in view of the above, the Objective of the study is as follow:

a) Estimating the floods of different return period.
b) To find the Rating of storage structure up to maximum discharge to pass different floods with revised Tail Rating Curve (TRC) through mathematical modelling.
c) To develop spillway gate Rating Curves at FRL and other operating levels without 3D modelling.
d) To optimize Stilling Basin geometry under New Tail Water Rating Curve at least for frequently occurring flow ranges.
e) To find out/calculate various hydraulic parameters such as discharge coefficient, downstream flow energy etc for different flow discharges.

1.5 Major Contribution of the Studies and Issues:

The work or study tries to handle any condition where any governing hydraulic factor like design discharge, tail water level (TWL) or water bays change. If the construction in the various parts of the project have already been started.

It became more difficult to encounter such a revised situation. In the present case when the construction of the overflow structure had reach to advance stages, the tail rating curve was authentically raised up to 4 m at the design discharge. Under such a situation neither crest level could be lowered to avoid the excess submergence, nor it was possible to increase the number of vents or their width. Similarly, the performance of proposed stilling basin was also adversely affected. Therefore,

with the help of this study a guideline is produced or prepared indicated that if it is not possible to optimize stilling basin or other appurtenant for full range of discharges then it is better to optimize the structure for frequent occurring floods. To encounter the raised upstream water level for passing the maximum discharge. The embankments may be raised if required with adequate cushion as free board.

CHAPTER-II

2 LITERATURE REVIEW

2.1 General

Whenever a project is conceived, the first and most important task is to establish a G&D site and start taking data which shall be very helpful in hydrological studies even of short duration. Even when the construction activities started at site, it was desirable to take gauge and discharge data at the proposed site. It was noticed during observations (during construction stage) that the tail rating curve of the river is raised. As per CWC and CEA guidelines (2010 R & 2015 R), in any hydro project if any additional data is observed even after the start of the work, it would be necessary to consider if possible keeping in mind the construction issues. There are so many studies carried on hydrological and hydraulics of the projects.

2.1.1 Studies Carried to Estimate Design Flood

Kpttegoda, (1980) evaluated that the consistency of hydrological data is evaluated with the help of t-test (t-statistics) and f-test (f-statics).

Guidelines for design flood estimation (USWRC 1981) stated that flood frequency analysis is the important parameter to determine the extent of flooding for the different return period. Traditionally, flood flow frequency determination focused on the analysis of flood observations Q recorded in every year Y at continuous-record stream gauges, which could be represented as point data. The description of flood and streamflow data for frequency analysis, and knowledge of the statistical characteristics of the data, have changed over the time. A generalized representation is needed to capture what is known about annual peak flows in a given year, or over a range of years n. This includes information about specific annual floods that are known to be within a range of values, or above or below an estimated perception threshold. Also, there may be information over a range of years in which it is known that no flood occurred above a known perception threshold. There may be sites where multiple perception thresholds are needed to represent different segments of the sample data across the historical period. Representations of peak-flow observations are now generalized to include concepts such as: flow intervals, exceedances, non-exceedances, and multiple perception thresholds. These concepts are described in this section to provide a generalized data representation for flood frequency.

Generally, the instantaneous peak discharge of river at the various recorded location are taken from long term data and the maximum flood of each year is extracted (Chow 1988). Then the data is processed for the outliers and consistency test. The outliers are those data points which depart significantly from the trend of the remaining data.

The conventional approaches for estimating the design flood are the flood frequency analysis are i) Formulae Approach ii) Statistical approach, commonly known as Flood Frequency Approach. iii) Hydro-meteorological approach, commonly known as the Unit Hydrograph Approach. (Chow 1988, Stedinger 1993). The conventional approaches for estimating the design flood are the flood frequency analysis have provided the estimation of design floods based on the regional frequency analysis (Hosking 1990). The application of the FFA methods is widely recognized by the various researchers in the water resource field. There are several sorts of frequency analysis distribution that have been successfully applied to the hydrological data.

Kwon, et al., (2007) describes that the design flood estimations are routinely required for water resources engineering purposes. The design flood is required for the planning and operation measures, the structural design, and the safety and risk analysis of the existing structures.

Guru, et al., (2015) confirmed some of the extreme value probability distributions usually used for hydrological analysis such as Normal Distribution, Log-Normal Distribution, Gumbel-Weibull Distribution, and Log Pearson Type III Distribution.

Radevski, et al., (2016) evaluated that the frequency analysis is usually used in hydrology for the possibility and scale of discharge extremes, especially low flows or high Floods.

In hydrological analysis one of the important aspect is to calculate the design flood. Therefore, the analysis of design flood is very important to be carried out including the testing of goodness of fit for evaluating the suitable probability distribution which is used in the frequency analysis. In addition, for this evaluation suitable G&D data or rainfall data also needed as the reference of flooding recorder.

2.1.2 Studies Carried on the Hydraulics of Project

Rhone, et al., (1940) contributed in the major development to standardize energy dissipaters at the Saint Anthony Falls (SAF) hydraulic laboratory at the University of Minnesota

USBR (1950 to 1960) initiated an extensive research program with the objective of developing standard designs for energy dissipaters. The product of this program was Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators."

Chow (1959) presented studies that for energy dissipation in stilling basins, maximum energy dissipation occurs when a clear hydraulic jump forms inside basin. A clear jump indicates non-submerged and non-swept jump inside stilling basin with its front located near toe of spillway.

Ashraf, et al., (1962) deliberated that the basin should be designed in such a way that the elevation of tail water depth in the downstream channel not be much less than the elevation of conjugate depth of jump. Otherwise sweep out of the jump from the basin takes place but if the conjugate depth is too low than the tail water depth the jump will be drowned. As a result, it will lose its function as an energy dissipater.

Rajaratnam, et al., (1966) describes that the length of apron depends upon the length and location of jump (for design discharge condition) which in turn depends upon the pre-jump depth and the relative magnitudes of required post jump depth and available tail water depth.

Peterka (1984) describes that in case of varying discharges, energy dissipaters are not efficient as the location of hydraulic jump tends to shift on apron. This would result in percentage reduction in energy dissipation.

The U.S. Bureau of Reclamation [USBR (1973)] and Basco (1984) confirmed that the optimum design for a stilling basin without baffles would have an apron elevation such that the jump curve defining the required D2 depth would superimpose on the tailwater curve for the full range of discharge.

Tokyay, et al., (1990) initiated a study to develop generalized criteria for the design of low Froude number hydraulic-jump stilling basins. The criteria and guidelines from previous studies were combined with the results of this study to formulate the design guidelines recommended for low Froude number stilling basins. However, it should be noted that a hydraulic-jump stilling basin is not an efficient energy dissipater at low Froude numbers; that is, the efficiency of a hydraulic-jump basin is less than 50 percent in this Froude number range or a downed jump is formed. Alternative energy dissipaters, such as the baffled apron chute or spillway, should be considered for these conditions.

The Federal Highway Administration (2006) describes that the tailwater depth must be equal to or some greater than full conjugate depth.

Ashiq, et al., (2010) discussed on Optimization of Energy Dissipation Works for Nai Gaj Dam Project in Western Sindh. In general, the paper concludes that energy dissipaters are important to be designed to minimize the damages in the downstream vicinity of the dam that results from high velocity flows.

Demetriou, et al., (2015) describes that if an optimally designed basin is defined as “a basin with minimum cost in which the requirements of hydraulic performance are satisfied”, the parameters affecting the optimal design of stilling basin may be counted as: 1) width of basin, 2) its length, 3) its height of walls, 4) level of floor of basin, 5) position of baffle blocks and 6) dimensions of chute blocks, baffle blocks and end sills.

Gavhane, et al., (2017) discussed the design of hydraulic jump type stilling basin for the overflow Spillway at Gunjwani dam. A model study is carried out by applying Froude’s model.

The preliminary design of a selected energy moderator is fixed through available empirical formulae of the above studies and are further refined by testing a scale model of the structure in a hydraulic model testing laboratory, but during construction, sometimes, unseen conditions may arise which call for change the hydraulic design in short span of time in an economical way. Such situations generally arise due to change in design discharge or change in tail rating curve. In this study, attempts are made to deal a situation where tail water curve rises during construction. With the raised tail rating curve, the upstream water levels for different floods shall also change due to increased submergence of the crest. The above studies are used for all the calculations which were made for hydraulic parameters of jump and stilling basin with revised tail water levels.

2.2 Case Studies

There are so many case studies in which due to wrong input data used in design, resulting in the failure of the dam or to make changes in structure to keep it safe after construction. Some of the similar case studies in which impacts of Tail Water change on the Design of Several Stilling Basins are given below:

2.2.1 Case Study-I El Guapo Dam (United States Department of Reclamation, 2014)

The El Guapo Dam, constructed between 1975 and 1980 near El Guapo, Venezuela, was designed to provide safe drinking water, flood mitigation, and irrigation water to the surrounding area. The dam was fashioned to hold a volume of about 40 lakh cubic-metre, and originally included an uncontrolled ogee crest spillway ending in a concrete hydraulic jump stilling basin. The spillway had a width of 12.2 metres, a length of 282 metres, and a design discharge of 102 m3/sec, an additional tunnel spillway 250 metre from the original was added after the spillway overtopped during construction. The design of the dam and primary spillway were based on a hydrologic study of a basin like the Guapo, but not on the Guapo basin itself. The overtopping of the initial spillway led to a new flood study, which resulted in the tunnel spillway's construction.

Breach of the El Guapo Dam during December 1999, the water elevation of the reservoir rose considerably, peaking on December 15th with a water elevation 20 metre below the dam crest. The spillway chute walls began to overtop near spillway crest, and the cities in the perceived flood path were evacuated at this time. The danger due to flooding was determined to have ceased, with the dam's water elevation cresting at 0.75 metre below the dam crest. However, by that afternoon, the water had rose considerably and quickly. The spillway began to overtop, which led to a sweepout of the stilling basin.

This sweepout, concurrent with overtopping in the spillway, eroded the soil foundation of the stilling basin, spillway chute, and crest structure, and within an hour head cutting had progressed to the reservoir itself, causing a breach and simultaneous failure of the El Guapo Dam.

Though overtopping of the spillway chute was the main cause of the erosion and subsequent failure, this overtopping was primarily the result of the stilling basin being improperly designed. The stilling basin was not designed to withstand the capacity of flow it experienced during the rainfall event, which not only caused the sweepout of the stilling basin, but the backlogging of water in the spillway chute, Sweepout of the El Guapo stilling basin which then overtopped and accelerated the failure of the dam. This could have been prevented by properly examining the El Guapo basin and determining the maximum discharges the stilling basin may experience.

The El Guapo basin itself was studied during construction due to the spillway overtopping, the study seems to have focused primarily on the spillway discharge capacity, rather than the stilling basin discharge capacity. This is evidenced in that the spillway was the structure to fail amidst construction, and in that the eventual result of the study was a supplemental tunnel spillway, which increased the discharge of the spillway with no apparent complementary addition to the stilling basin discharge. This resulted in a stilling basin with more water flowing into it than it was designed to discharge. Had a full and comprehensive hydrologic study of the El Guapo basin been conducted, the incommensurate discharge of the stilling basin may have been detected, and suitable alterations may have been made to prevent the catastrophic failure at the El Guapo Dam.

In this case study following are the main causes of failure:

a) Initial hydrologic studies based on similar basin but not Rio Guapo Basin
b) During construction, spillway chute walls were overtopped which prompted addition hydrologic studies and addition of a tunnel spillway
c) Basin design calculations on inaccurate data resulted in a stilling basin and spillway system that could not handle the volume of water necessary.

2.2.2 Case Study: Bullock Pen Dam (Feimster, et al., 2016)

The Bullock Pen Lake Dam is in Crittenden, Kentucky. It retains Bullock Pen Lake, which is used for recreation and water supply for the town of Crittenden. This Dam was constructed in 1953. After few years of construction and shortly after initial filling of the reservoir, erosion of the shale in the excavated rock spillway channel was observed.

The proposed structure will consist of a reinforced concrete labyrinth spillway over the current dam embankment. The spillway will discharge into a stepped chute and then into a USBR Type -1 stilling basin located at the toe of the embankment. Flows from the stilling basin will discharge into a riprap-lined outlet channel. Under normal conditions, the tailwater will be located 0.3 metre above the low stage of the stilling basin and 0.9 metre below the second stage of the stilling basin. During the 10-year storm, the tailwater will be located at the elevation of the lowest step of the chute. During the Design, the tailwater will be located 0.27 metre above the spillway slab, submerging the stepped section of the spillway chute. Due to the site complications, both a CFD model and conventional methods were used to analyse the energy dissipation at this site. The approach was to design the stilling basin using the conventional methods and use the CFD model to check the results of the conventional methods for reasonableness. A piecemeal approach was used to design the stilling basin. The selected stilling basin was a USBR type 1 basin, which is a concrete apron. The CFD model showed that the hydraulic jump occurred over the stepped chute due to the high tailwater. The energy dissipation was calculated to be less than the models; however, the velocity of flow at the exit of the stilling basin was lower than predicted. This is most likely due to the high tailwater at the site that the predictive models do not consider. The results of the CFD model confirmed that the spillway layout designed using the conventional methods is appropriate for the site conditions. The design methods for a project can be limited by external factors such as time, funding, property boundaries, or a combination of factors due to these constraints physical model studies was not carried out. In these cases, consideration should be given to the complexities of the site and structures that may not be considered with the method or methods selected. Conveying the unknown effects of these factors should be discussed with the project authorities so that they understand the risk they are taking on with the selected design. In addition, the owner should be made aware of measures that could be taken in advance of, during, or after a major flooding event when the design limits are tested.

From the above case studies, it is quite clear that a due consideration should be given on the inputs used for the design of Dams and Tail Rating curve, Energy dissipation arrangement and stilling basin design should be given due weightage.

CHAPTER-III

3 STUDY AREA AND GEOLOGY

3.1 Location of Study Area

There are many operational projects in the upstream of Erach. Some of them are Rajghat Dam, Matatila Dam, Dhukwan Dam and Parichha Dam. Out of these Parichha Dam is the nearest about 41 km upstream of Erach. The Latitude and Longitude of the Erach Dam site are 25°43'57.45"N and 79° 3'23.99"E respectively. The Study Area Map and catchment area of EMPP shown in Figure 3.1.

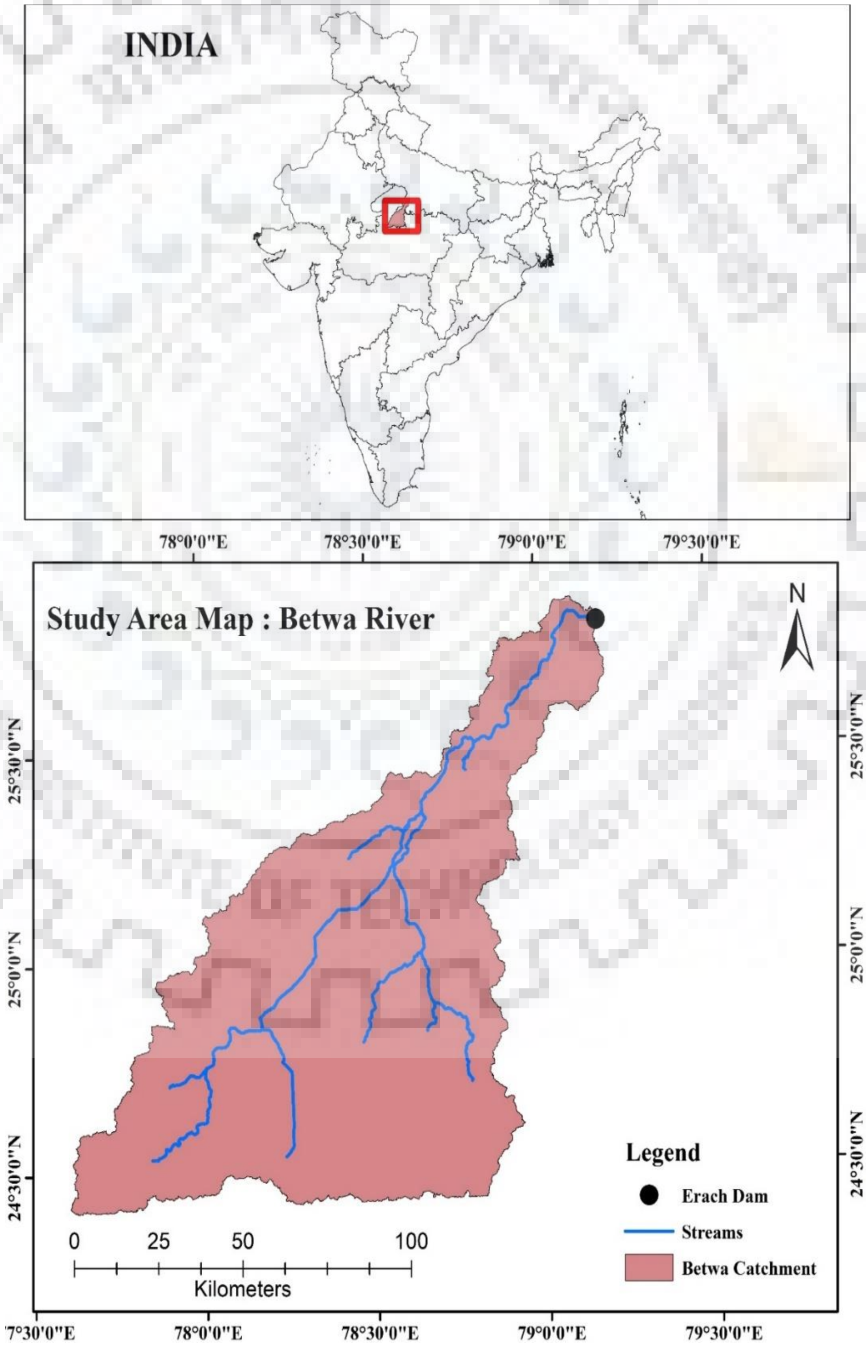

Figure 3.1 Study Area Map and Catchment Area Map of EMPP

The Erach site is an ungauged site and G&D site is available at about 41 km downstream of our study location. The Latitude and Longitude of Mohana G&D site are 25°42'00"N and 79° 42'00"E respectively. The Mohanna G&D site data is used in this thesis for Hydrological studies. The location of Mohanna site with respect to Erach Dam is shown in Figure 3.2.

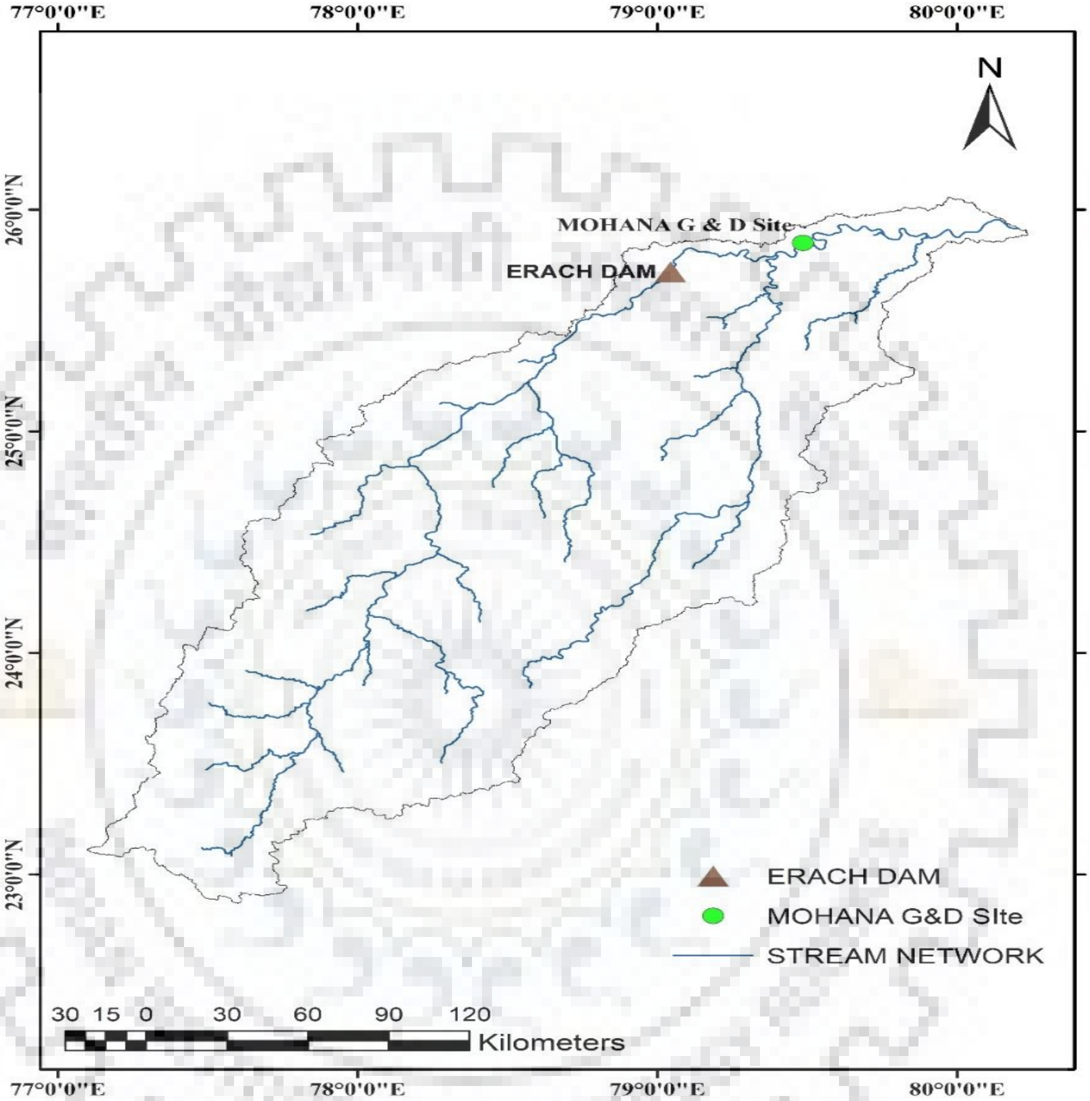

Figure 3.2. Location Map of Erach dam and Mohana G&D Site.

3.2 Geology of Area

Erach Multipurpose Project (EMPP) envisages construction of a dam across the River Betwa. Betwa River in northern India, rising in the Vindhya Range just north of Hoshangabad, Madhya Pradesh. It flows generally northeast through Madhya Pradesh and Uttar Pradesh states and empties into the Yamuna River just east of Hamirpur after a 610 km course. Nearly half of its course, which is not navigable, runs over the Malwa Plateau before it breaks into the upland of Bundelkhand. The Jamni and Dhasan rivers are the main tributaries. Bundelkhand lies between the Indo-Gangetic Plain to the north and the Vindhyan Range to the south. It is a gently sloping upland, distinguished by barren hilly terrain with sparse vegetation, although it was historically forested. The plains of Bundelkhand are intersected by three mountain ranges, the Vindhyan, Fauna and Bhander chains, the highest elevation not exceeding 600 meters above mean sea-level.

3.3 Climate of the region

The land is suitable for species of citrus fruit and crops include wheat, pulses, peas and oilseeds. The region relies heavily on monsoon rains for irrigation purpose. As the area is on rocky plateau, it experiences extreme temperatures. Winter begins in October with the Southwest Monsoon and peaks in mid-December. The minimum temperature in winters is 4 degree and maximum is 21 degree. Spring arrives by the end of February and is a short-lived phase of transition. Summer begins by April and summer temperatures can peak at 47 degrees in May. The rainy season starts by the third week of June (it varies year to year). Monsoon rains gradually weaken in September and the season ends by the last week of September. In the rainy season, the average daily high temperature is around 36 degrees Celsius with high humidity. The average rainfall for the region is about 900 mm per year. In summer the region experiences temperatures as high as 45 to 47 degrees.

CHAPTER-IV

4 DATA ACQUISITION AND PROCESSING

4.1 Data for Hydrological studies

As Erach is an ungauged site, secondary data are needed for this study. The gauge and Discharge site is available at 31 km downstream of our study area. The daily discharge data (1985-2011) is collected for this study. In this study basic statistical analysis has been done for the discharge data for the period 1984-85 to 2011-12 for the Mohana Gauge and Discharge (G&D) site in the Betwa river. The Mohana G&D site is maintained by CWC, Lower Yamuna Division, Agra.

4.2 Data for Hydraulic studies

The Basic data required for hydraulic model is the river profile and cross-sections of the river about 2 km upstream and 1.2 km downstream of the river respectively. The cross-sections are extracted from the contour map taken from project authorities. The revised tail rating curve and other project features required for model building are also taken from project authorities. The revised tail Rating curve and old tail rating curve is given in Figure 4.1.

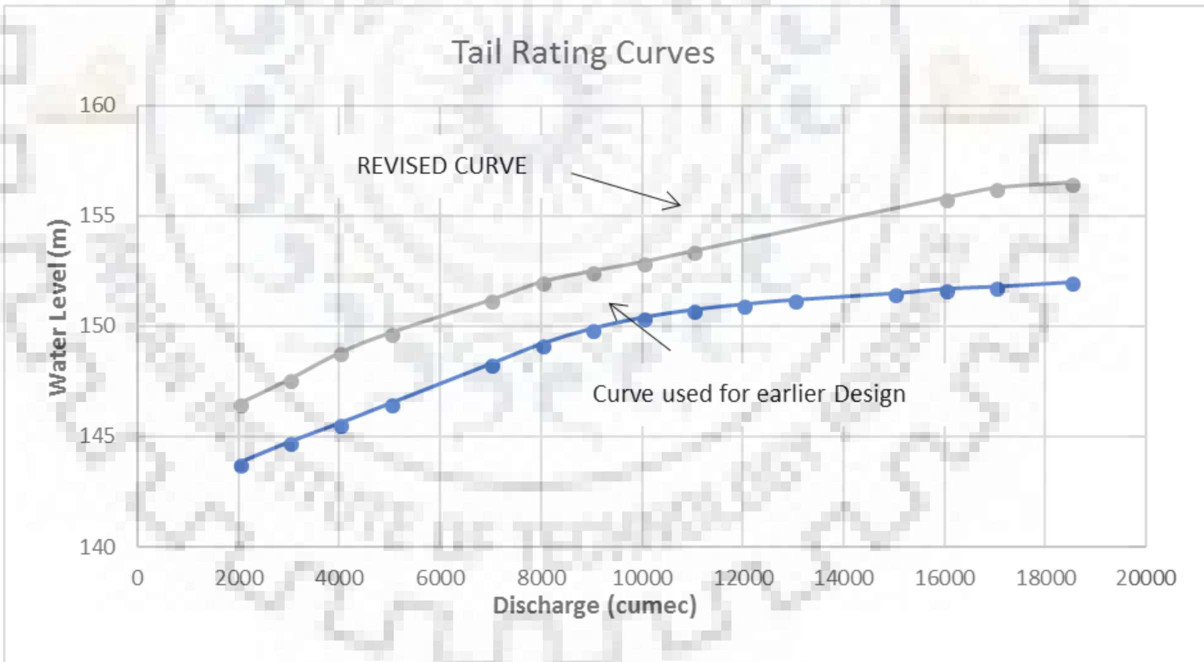

Figure 4.1. Revised Tail rating curve and curve used for earlier design

CHAPTER-V

5 METHODOLOGY AND ANALYSIS

The main aim of this study is to review Hydrology and Hydraulic of the Erach Dam.

5.1 Steps of Study for Hydrological studies

a) Daily/10-daily data was initially converted to annual flow series, before analysing the frequency and dependable flow.

b) Hydrological data are mostly available as samples of limited size. The consistency of data is evaluated with the help of t-test (t-statistics) and F-test (F-statics).

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{\frac{s_1^2}{n_1} - \frac{s_2^2}{n_2}}} \quad \text{……………………………… (5.1)}$$

where

$\bar{x}_1$ = Mean of the first data set,

$\bar{x}_2$ = Mean of the second dataset,

S_1 = Standard deviation of the first dataset,

S_2 = Standard deviation of the second dataset,

n_1 = Size of first dataset,

n_2 = Size of second dataset

$$F_{value} = \frac{\sigma 1^2}{\sigma 2^2} \quad \text{………………………… (5.2)}$$

Where, σ2 = variance

c) Using statistical principles, the needed information (such as mean, standard deviation, skewness co-efficient, kurtosis co-efficient etc.) are extracted from the available sample data and are assumed to be the characteristics of the population. Whatever be the length of the sample, it cannot exhaust all possible elements of a variable. Therefore, the sample is assumed to be representative of the population. In the application of the statistical analysis methods, it is assumed that the occurrences are individual events independent of each other i.e., they are assumed to be evolved from a purely random process. A time series of events is said to be homogeneous if it does not exhibit systematic variation in time (e.g., cyclic variation, an increasing or a decreasing trend or a jump). The factors which affect the homogeneity of peak flows are the developments in the catchment over time such as deforestation, urbanisation, flood control works, earthquake etc.

d) It may be difficult to find data which conform to all the requirements. As a preliminary step, the basic data should be properly processed, screened and adjusted to remove, as far as possible, any non-conformities that may exist. The data series is to be checked for the randomness and presence of trends, jump and outliers.

e) After arriving at the series, the series is analysed to test as to which of the known probability distribution provide best fit.

5.2 Probability Distribution Methods

The description of various Probability Distribution methods and Goodness of fit tests used in this study are:

5.2.1 Normal Distribution

The Normal distribution or Normal curve is also mentioned as the Gauss distribution. The formula for calculating the estimation value with the return period of T (Xt) is as follow:

$$X_T = X + K_T * S \quad \text{.......................................} \; (5.3)$$

Where,

XT: estimation of value which is hoped to be happened by the return period of T

X: mean

S: deviation standard

KT: factor of frequency which is as the function of probability or return period and as the type of mathematical modelling of the probability distribution that is used for the probability analysis

5.2.2 Log Normal Distribution

The formula of Log Normal distribution is the same as the Normal distribution, but the data must be transformed into log.

$$XT = X + KT * S \quad \text{.......................................} \; (5.4)$$

Where,

XT: estimation of value which is hoped to be happened by the return period of T (in the log)

X: mean (in the log)

S: deviation standard (in the log)

KT: factor of frequency which is as the function of probability or return period and as the type of mathematical modelling of the probability distribution that is used for the probability analysis.

5.2.3 Gumbel Distribution

The formula of the Gumbel distribution that is used for estimating the value which is hoped to be happened with the return period of T (Xt) is as follow:

$$\mathrm{Xt} = \mathrm{X} + \mathrm{K} \times S_x \quad \text{...} \; (5.5)$$

Where,

Xt = estimation of value which is hoped to be happened by the return period of T

X = mean of the observed result.

K= Frequency factor for Gumbel.

Sx = standard deviation.

5.2.4 Log Pearson Type III distribution

To use the Log Pearson Type III, the data must be transformed into the Log form. The formula of Log Pearson Type III with the return period of T (Xt) is as follow:

$$Log\ XT = LogX + K.S \quad \text{...} \; (5.6)$$

Where,

Log XT: estimation of value (in the Log from) which is hoped to be happened by the return period of T

X: mean (in the Log form)

S: deviation standard (in the log form)

KT : factor of frequency which is as the function of probability or return period and as the type of mathematical modelling of the probability distribution that is used for the probability analysis.

5.2.5 Goodness of Fit Tests

i) Testing of Goodness of Fit by Using Smirnov-Kolmogorov Test

Testing of goodness of fit by using Smirnov-Kolmogorov test is carried out by comparing the probability of every variant between the empirical and theoretical probability and then the maximum deviation is compared with the deviation of table.

If the Δ max on the probability paper is less than Δ critic for a level of significance and the number of certain variant, so it can be concluded that the deviation which is happened is caused by the accidental error. The steps for carrying out the test are as follow:

a. To rank the data (from small to big or big to small) and to calculate the probability each of the data as the empirical probability.
b. To determine the value each of the theoretical probability
c. To find the maximum deviation between the empirical and theoretical probability.
d. Then, the maximum deviation is compared with the critical value with a level of significant from the Smirnov-Kolmogorov table.

ii) Testing of Goodness of Fit by Using the Chi- Square Distribution

Test of chi square distribution evaluates the difference between the sample data and the probability distribution. The formula of chi square is as follow:

$$X2 = \sum_{i=1}^{n} \left(\frac{Ei - Oi}{Oi}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.7)$$

Where

X2 = chi-square calculated value;

Ei = frequency that is hoped regarding to the class division;

Oi= frequency on the same class;

N = number of class.

The value of Ei can be found with the formula as Ei=n/N

Where, n = number, N = number of class.

5.2.6 Limitations of Probability Distribution Methods

Fitting of various probability distributions can be carried out either mathematically or graphically. Computer programmes are also available. However, the statistical analysis method has certain limitations as:

a) It yields only the peak, not volume or shape of the hydrograph.
b) Correct inference about the distribution, which fits the sample data for a site as different distribution fitted to same data results in different estimated values especially in the extrapolated range and poses for the planners for economic problem appraisal of the project.
c) Sufficiently long data length to allow reliable estimation of population parameters from the sample data.

Elements of risk and uncertainty are inherent in any flood frequency.

5.3 Flood Frequency Analysis

5.3.1 Analysis of Discharge Data

In the hydrological analysis, the peak yearly discharge data of Mohana is extracted from the data available and is increased by 15% to get instantaneous peak. The annual instantaneous flood peaks are shown in Table 5.1. and Figure 5.1. Annual instantaneous flood peaks & 10-year moving average also shown in Figure 5.2.

Table 5.1. Peak Yearly Discharge at Mohana

SL. No.	Year	Date	Daily Flood Peak (m^3/s)	Daily value increased by 15% for Instantaneous peak
1	1984	23 August 1984	2310.00	2656.50
2	1985	10 October 1985	16546.00	19027.90
3	1986	24 July 1986	12124.00	13942.60
4	1987	31 August 1987	16816.00	19338.40
5	1988	06 August 1988	6273.00	7213.95
6	1989	14 August 1989	4326.00	4974.90
7	1999	06 September 1999	13701.00	15756.15
8	2000	22 July 2000	11100.00	12765.00
9	2001	17 August 2001	5400.00	6210.00
10	2002	07 September 2002	3162.00	3636.30
11	2003	11 September 2003	10758.00	12371.70
12	2004	24 August 2004	5422.00	6235.30
13	2005	06 July 2005	7852.00	9029.80
14	2006	03 September 2006	10374.00	11930.10
15	2007	25 August 2007	120.20	138.23
16	2008	12 August 2008	7306.00	8401.90
17	2009	12 September 2009	1163.00	1337.45
18	2010	28 July 2010	630.60	725.19
19	2011	24 July 2011	7813.00	8984.95

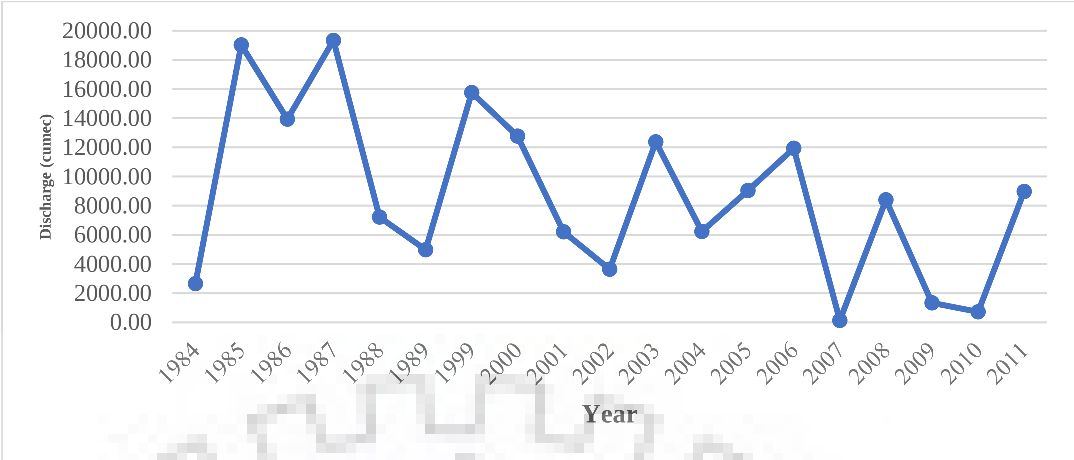

Figure 5.1. Instantaneous Flood Peaks

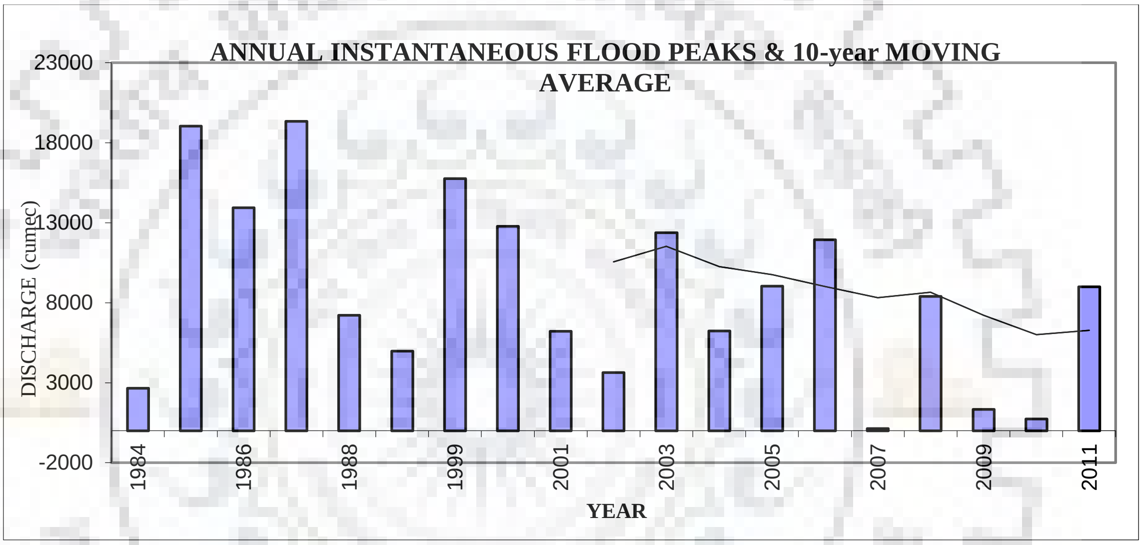

Figure 5.2. Annual instantaneous flood peaks & 10-year moving average

5.3.2 Tests of Significance:

The test of significance is used to assess the strength of the evidence against the null hypothesis (the hypothesis that there is no significant difference between specified populations, any observed difference being due to sampling or experimental error). The significance level α for a given hypothesis test is a value for which a P-value less than or equal to α is considered statistically significant. Typical values for α are 0.1, 0.05, and 0.01. For these tests we have taken α=0.05. These values correspond to the probability of observing such an extreme value by chance. In the below t- test, the P-value is 0.05, which should be equal to or less than α= 0.05 value, and the result is significant at the 0.05 level. Results of this consistency test are shown in Table 5.2

Table 5.2. t-test Two-Sample Assuming Unequal Variances

t-test Parameters	Variance 1	Variance 2
Mean	12899.55	6813.83
Variance	43807385.33	20640975.30
Observations	5.00	12.00
Hypothesized Mean Difference	0.00	0.00
Df	6.00	6.00
t Stat	1.88	1.88
P(T<=t) one-tail	0.05	0.05
t Critical one-tail	1.94	1.94
P(T<=t) two-tail	0.10	0.10
t Critical two-tail	2.45	2.45

In ‘F’ test first we need to read the p-value first. If the p-value is less than or equal to (α=0.05) in one tail, you can accept the null hypothesis. After that consider the f-value, it is less than the F-critical then we can reject the null hypothesis (Table 5.3.) ‘F’ test results at 5% significant level are accepted.

Table 5.3. F-Test Two-Sample for Variances

F-test Parameters	Variable 1	Variable 2
Mean	11192.38	7501.70
Variance	52532587.15	25072059.15
Observations	6.00	13.00
Df	5.00	12.00
F	2.10	2.10
P(F<=f) one-tail	0.14	0.14
F Critical one-tail	3.11	3.11

5.3.3 Randomness

Randomness tests (or tests for randomness), in data evaluation, are used to analyse the distribution of a set of data to see if it is random (pattern less, Table 5.4.). The nonparametric Turning point Test and difference sign tests are performed (as shown in Table 5.5.) for randomness and it was found that flood values are not of random nature.

Table 5.4. Randomness Tests

SL. No.	Instantaneous flood peaks	Peaks	Troughs	Sign
1	2656.50	-	-	-
2	19027.90	1	0	1
3	13942.60	0	1	0
4	19338.40	1	0	1
5	7213.95	0	0	0
6	4974.90	0	1	0
7	15756.15	1	0	1
8	12765.00	0	0	0
9	6210.00	0	0	0
10	3636.30	0	1	0
11	12371.70	1	0	1
12	6235.30	0	1	0
13	9029.80	0	0	1
14	11930.10	1	0	1
15	138.23	0	1	0
16	8401.90	1	0	1
17	1337.45	0	0	0
18	725.19	0	1	0
19	8984.95	-	-	1
	Score	**6**	**6**	

Table 5.5. Results of Randomness tests

Turning Point Test		**Difference Sign Test**	
N	19.00	No. of Positive signs, Np	8
P	12.00	No. of Negetive signs, Nn	10
E(P)	11.33	Score = Max (Np, Nn)	10
Var(P)	3.06	N	19
Z	0.38	Mean = (N-1)/2	9
As -1.96<0.38<1.96 Flood values are not of random nature		Variance = (N + 1)/ 12	1.67
		Standard Deviation = sqrt(Variance) =	1.29
		Test Statistic = (Score - Mean)/SD	0.77
		As -1.64<0.77<1.64 Flood values are not of random nature	

5.3.4 Outlier Removal and FFA

The outliers are removed so that the modified values follow the general trend as indicated by the rest of the data as shown in Table 5.6.

Table 5.6. Check for outliers and FFA

SL. No.	Year	Instantaneous flood peaks	Log of peaks
1	1984	2656.50	3.42
2	1985	19027.90	4.28
3	1986	13942.60	4.14
4	1987	19338.40	4.29
5	1988	7213.95	3.86
6	1989	4974.90	3.70
7	1999	15756.15	4.20
8	2000	12765.00	4.11
9	2001	6210.00	3.79
10	2002	3636.30	3.56
11	2003	12371.70	4.09
12	2004	6235.30	3.79
13	2005	9029.80	3.96
14	2006	11930.10	4.08
15	2007	138.23	2.14
16	2008	8401.90	3.92
17	2009	1337.45	3.13
18	2010	725.19	2.86
19	2011	8984.95	3.95

From calculations we get, Mean of Log X value= 3.75, SD=0.544 and Skew=-1.77

Low outlier =293.53 and High outlier =108295

As skew is less than +0.4 so test for either one outlier was preferred, and skew value is less than -0.4 so test for lower outlier needs to be done. Data after removal of outliers confirm the actual observation of outlier values as shown in Table 5.7.

Table 5.7. Data after removal of errors

SL. No.	Year	Instantaneous flood peaks	Log of peaks	log(Xi-Xo)
1	1984	2656.50	3.42	3.29
2	1985	19027.90	4.28	4.26
3	1986	13942.60	4.14	4.12
4	1987	19338.40	4.29	4.27
5	1988	7213.95	3.86	3.81
6	1989	4974.90	3.70	3.63

7	1999	15756.15	4.20	4.18
8	2000	12765.00	4.11	4.08
9	2001	6210.00	3.79	3.74
10	2002	3636.30	3.56	3.46
11	2003	12371.70	4.09	4.07
12	2004	6235.30	3.79	3.74
13	2005	9029.80	3.96	3.92
14	2006	11930.10	4.08	4.05
15	2007		Deleted	
16	2008	8401.90	3.92	3.89
17	2009	1337.45	3.13	2.79
18	2010	725.19	2.86	
19	2011	8984.95	3.95	3.92

The statistical characteristics the mean, standard deviation and coefficient of skewness etc are computed of the data (Excel functions: Average, Stdev, Skew etc) and are given in Table 5.8.

Table 5.8. Statistical characteristics of the flow series:

Parameters	**Flood Peaks**	**Log of Peak**	**Log (Xi-Xo)**
Sample size	19	19	19
Mean	1999.16	3.84	3.84
Standard Deviation	5649.892	0.959	0.383
Skew ness	0.33	-1.24	-1.41
Kutrosis	-0.73	1.24	2.36
Maximum	19338.40		
Minimum	725.19		
Cs/6=k	0.0544	-0.2061	-0.2357
Return Period	1000		

As per goodness of fit tests, Chi square and Kolmogorov-Smirnov Test, (attached in Appendix 5.1) and normal statically characteristics of the flow series, the hypothesis is that the normal distribution, Gumbel Extreme Value distribution and Pearson-III fits the data. Since the value of the variate for the required return period determined by different methods can have errors due to limited years of data available, an estimate of the confidence limit of the estimate is desirable. As the skew coefficient is between -1 and +1 so Gumbel distribution can be used in this case. So Extreme Value distribution or Gumbel distribution is used for FFA in this study as given below in Table 5.9, 5.10 and 5.11.

Table 5.9. Flow Series for Gumbel distribution

S. No.	**Year**	**Peak Discharge (m3/s)**	**Rearranged Discharge (m3/s)**	**Order Number ('M')**	**Recurrence Interval ('T')**	**Frequency ('f')**
(1)	(2)	(3)	(4)	(7)	(8)	(9)
1	1984	2656.5	19338.4	18	1.06	94.74
2	1985	19027.9	19027.9	17	1.12	89.47
3	1986	13942.6	15756.15	16	1.19	84.21
4	1987	19338.4	13942.6	15	1.27	78.95
5	1988	7213.95	12765	14	1.36	73.68
6	1989	4974.9	12371.7	13	1.46	68.42
7	1999	15756.15	11930.1	12	1.58	63.16
8	2000	12765	9029.8	11	1.73	57.89
9	2001	6210	8984.95	10	1.90	52.63
10	2002	3636.3	8401.9	9	2.11	47.37
11	2003	12371.7	7213.95	8	2.38	42.11
12	2004	6235.3	6235.3	7	2.71	36.84
13	2005	9029.8	6210	6	3.17	31.58
14	2006	11930.1	4974.9	5	3.80	26.32
15	2008	8401.9	3636.3	4	4.75	21.05
16	2009	1337.45	2656.5	3	6.33	15.79
17	2010	725.19	1337.45	2	9.50	10.53
18	2011	8984.95	725.19	1	19.00	5.26

Table 5.10. Statistical Characteristics of Flow Series

<table>
<tr><td>(1)</td><td>Standard Deviation σ_{n-1}</td><td>5649.89</td><td></td></tr>
<tr><td>(2)</td><td>Mean</td><td></td><td></td></tr>
<tr><td>(3)</td><td colspan="2">Total number of events in the given data series is 18</td><td rowspan="3">From Gumbel's Table (P.no 311 of K Subramanya)</td></tr>
<tr><td>(4)</td><td>$\bar{y}_n$</td><td>0.5202</td></tr>
<tr><td>(5)</td><td>σ_n</td><td>1.0493</td></tr>
</table>

Table 5.11. Calculations for Standard Flood and Probability Error

Exceedence Probability Limits	**Return Period (T)**	**Gumbels Frequency Factor (K)**	**A factor derived from Gumbel's frequency factor (T)**	**Standard Normal Variate for 90%**	**Standard Flood**	$b = \sqrt{1 + 1.3(K) + 1.1(K)^2}$	**Probable Error S_e**	**90% Confidence Limit** +	-
0.10	10	1.6489	2.25	1.645	18456.98	2.48	5425.62	23882.60	13031.36
0.04	25	2.5525	3.20	1.645	23562.33	3.39	7423.96	30986.29	16138.38
0.02	50	3.2229	3.90	1.645	27349.77	4.08	8929.39	36279.16	18420.37
0.01	100	3.8883	4.60	1.645	31109.24	4.76	10433.75	41542.99	20675.49
0.00	500	5.4259	6.21	1.645	39796.81	6.36	13930.47	53727.28	25866.34
0.00	1000	6.0870	6.91	1.645	43531.71	7.05	15438.82	58970.53	28092.90
0.00	10000	8.28	9.21	1.645	55932.26	9.34	20457.91	76390.17	35474.36

Table 5.12. Chi Square Test for Gumbel Distribution

Class =5			**Ej=3.6**				
Exceedence Probability Limits		**Discharge Limits**		**Oj**	**Ej**	**$(Oj-Ej)^2$**	**X^2 Com**
Higher	Lower	Lower	Upper				
1	0.8	0	3777.65	4	3.6	0.16	0.04
0.8	0.6	3777.65	6807.21	3	3.6	0.36	0.1
0.6	0.4	6807.21	9956.89	4	3.6	0.16	0.04
0.4	0.2	9956.89	14416.35	4	3.6	0.16	0.04
0.2	0	14416.35	Infinity	3	3.6	0.36	0.1

Chi-square Calculated Value, $X^2com = (Oj - Ej)^2/Ej$ =0.33

α%= 5 %

degree of freedom = K-h-1= 2

Chi-square Critics, $X^2critical = X^2_{0.95,5-2-1}$=5.99

Since $X^2com < X^2critical$ (accepted)

Therefore, the distribution is fitting and the plot between Standard flood, confidence limit and return period is shown in Fig.5.3. below:

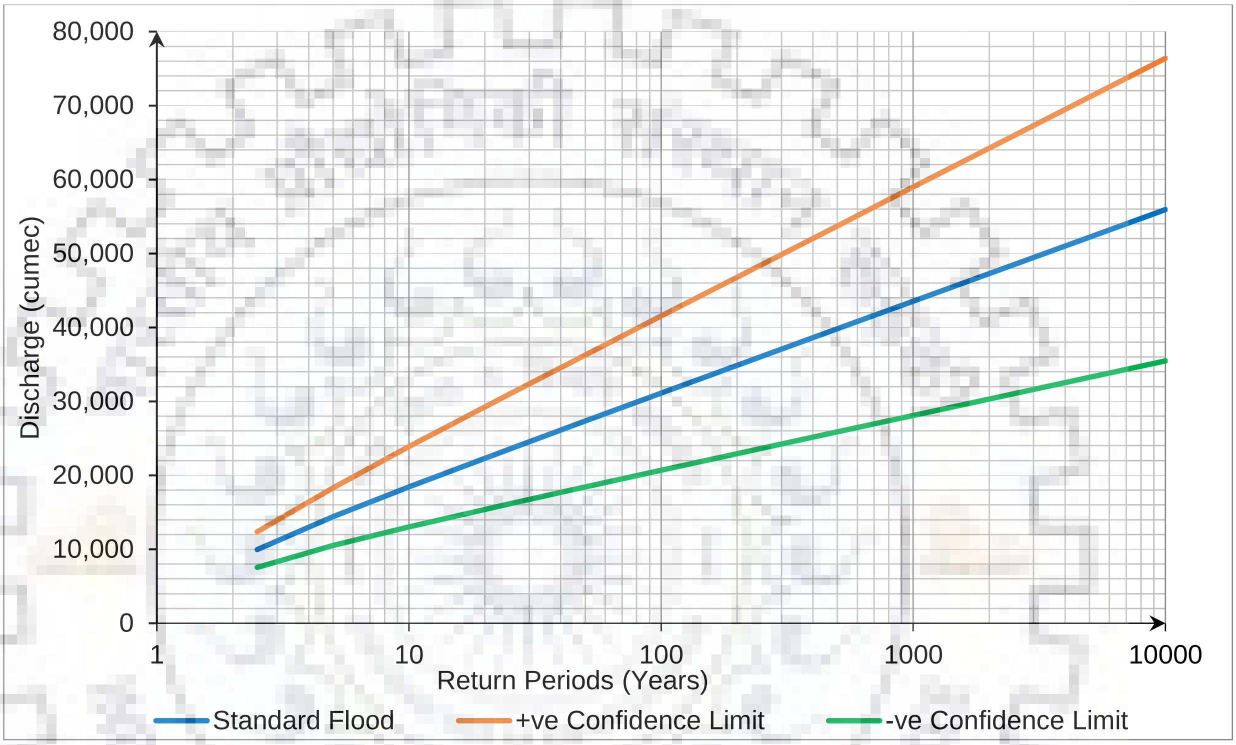

Fig.5.3. Plot between Return Period and Standard flood , Confidence Limit

The FFA results we get from Gumbel distribution method should be transpose to Erach dam site by using Dicken's formula. The catchment area of Mohanna G&d site get from CWC report on "*Details of Hydrological observation stations as on 2012*" and the catchment area of Erach site we get from ArcGIS software by making shape file of the catchment of Erach Dam site. The results of the Design flood for Erach dam site are given below in Table 5.13.

a) Catchment Area at Mohanna site = 41054 km^2
b) Catchment Area at Erach site = 9000 km^2

Table 5.13. Catchment transpose by Dicken's formula $Q = CA^{3/4}$

Sl. No.	Return Period	Design Flood at Mohana as per Gumbel dist Method	Design Flood at Erach
	(yrs.)	(m^3/s)	(m^3/s)
1	25	30986.2874	9,927.38
2	50	36279.16462	11,623.12
3	100	41542.99371	13,309.54
4	500	53727.27982	17,213.14
5	1000	58970.52803	18,892.97

The Design flood at Erach site as per above method is 18892 cumec which is very near to 18500 cumec taken by project authorities for the design of Erach Dam. So, the design flood is acceptable for the raised Tail rating curve (TRC) and further hydraulic design of Energy Dissipation arrangement.

5.4 Steps of Study for Hydraulic Studies

5.4.1 General

The basic aim of hydraulics is to understand the occurrence, movement and use of water, whether it is in lakes, rivers, pipes, drains, percolating through soils or pounding the coastline as destructive waves. Natural river channels, whose cross-sectional areas and bed roughness vary significantly from place to place on the river, are simply too complex for the theory that is available due to additional forces appear on the fluid motion. So, Coefficient of discharge (Cd) and friction factors should be considered in these cases. The computer software/models can simulate the hydraulic behaviour of whole river basins and simulating dam failure scenarios.

In this chapter the data collected for hydraulic studies like Cross-sections, L-sections and tail rating curve are processed. After that a mathematical model is selected for hydraulic study of the Betwa river for Erach Dam. The model is then analysed in three different conditions i.e in virgin condition (without dam), free flow condition and ponded conditions (with dam). The results we get from the model are used to find hydraulic parameters of Stilling basin and Energy dissipation arrangement using conventional methods. Based on the different results the Stilling Basin is then optimized for different flows. The methodology applied for the present study is presented through flow chart in Figure 5.4.

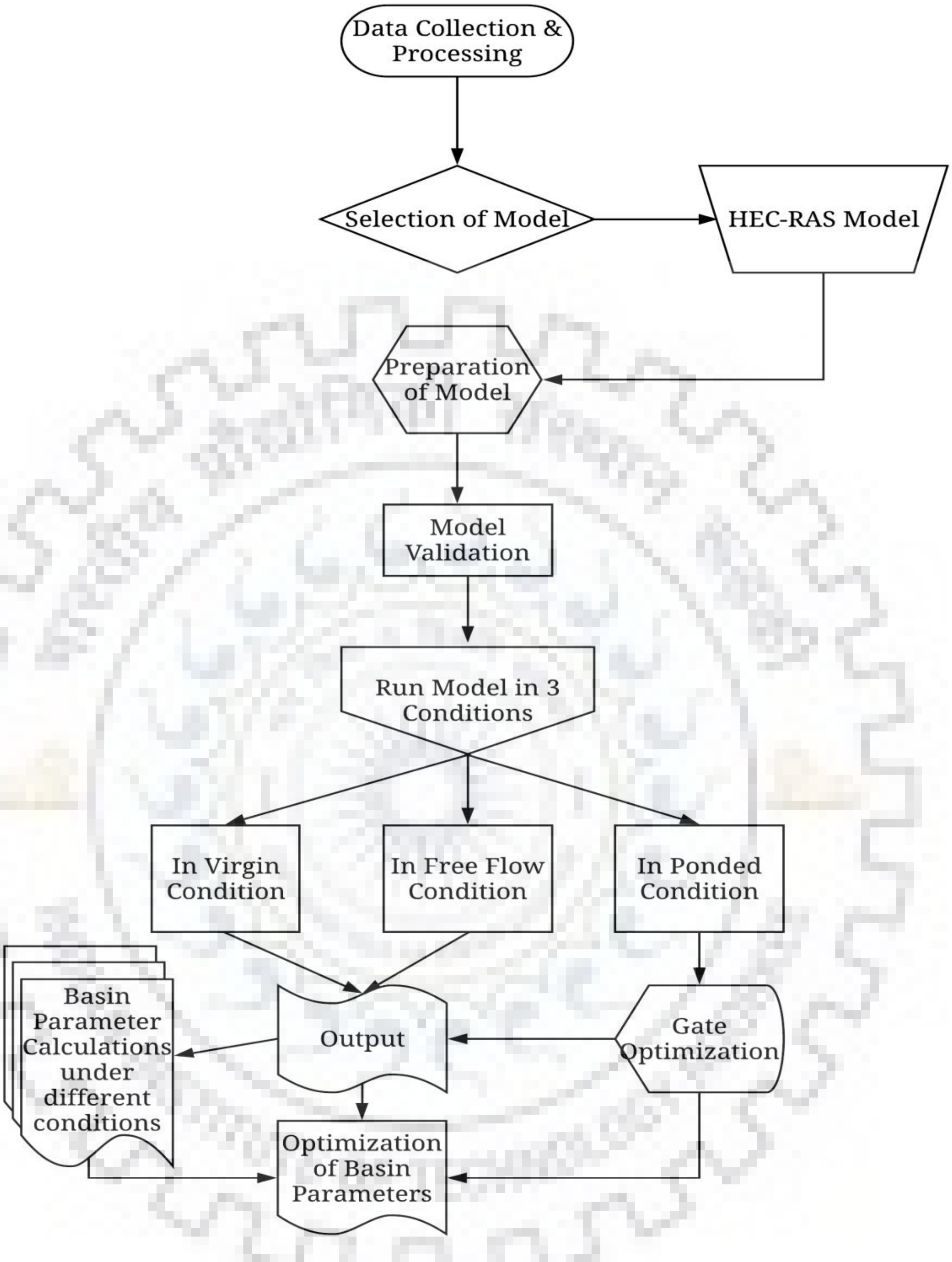

Figure 5.4. Flow Chart of Methodology for Hydraulic Studies

5.4.2 Selection of Mathematical Model

As the problem in hand is a unique problem where design parameters are changed while construction is on and no 3D physical modelling is possible due to time constraint, it was decided to understand the hydraulic behaviour of the river and the conditions on the upstream side of the Dam through the mathematical model study.

A popular and well-known model, namely, Hydrologic Engineering Center's River Analysis System (HEC-RAS) is used for Hydraulic investigations and river morphological study purposes. This model is developed by the U.S. Army corps of Engineers and it allows to perform one-dimensional steady, unsteady flow hydraulics, sediment transport/mobile bed computations for quantifying the effects of new structures and their operation in the river. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system is comprised of a graphical user interface (GUIC), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The HEC-RAS system contains many one-dimensional river analyses including the components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment and transport computations. A key element is that all the components use common geometric data representation and common geometric and hydraulic computation routines. In addition to the four river analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed. The version 5.03 of HEC-RAS is used in this study as it supports steady and unsteady flow water surface profile calculations and sediment transport/mobile bed computations simultaneously.

5.4.3 Hydraulic Capability of HEC-RAS

General Capability of the HEC-RAS is to calculate the water surface profiles for steady and gradually varied flow. The system can handle a single river reach or a full network of channels. The steady flow component is capable of modelling subcritical, supercritical, and mixed flow regime water surface profiles. The computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include hydraulics of Dams and evaluating profiles at river confluences (stream junctions). The effects of various obstructions such as inline structure and other structures on the flood plain are considered in the computations. The steady flow system is designed for application in floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements, and reservoirs.

5.4.4 HEC RAS in Structure Scour

HEC-RAS, is one of the mostly widely used computer program for Dams, bridges and other structure scour one dimensional hydraulic analyses program with scour estimation modules. It predicts scour at structure crossing reasonably well for simple regular channel. It provides predictive scour-depth computations using parameters from a one-dimensional hydraulic analysis. Field observations show that structure scour predicted by HEC-RAS generally overestimated the actual scour depth. One of the reasons is that scour prediction equations used in HEC-RAS was developed based on scaling up the laboratory results, which are difficult to satisfy both the hydraulic and hydrodynamic similitude. The assumption of one dimensional flow is another potential source of over estimation. The basic computational procedure in HEC-RAS is based on solving the one-dimensional energy equation. Energy losses are accounted for by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions). Scour occurring at Dam crossing generally include three components: 1) Long-term aggradation and degradation of the river bed, 2) general scour at Structure (including contraction scour and other general scour), and 3) local scour at the piers or abutments.

Based on the existence of sediment transportation, scour is classified as clear-water scour and live-bed scour. Two of the scour prediction formulas (i.e., Froehlich equation) and HEC-18 equation (CSU equation) are available in HEC-RAS. Depending upon the angle of flow towards the bridge, HEC has capability to calculate scours along each cross-section downstream and upstream of the inline structure.

5.4.5 Preparation of the Mathematical Model

The HEC-RAS mathematical model of Betwa River was set up by providing, as inputs, the cross sectional geometric data at the longitudinal spacing of about 200 m. The channel cross sections (Appendix-5.2) were fed from the contour map of the area. The cross sections were provided up to El.160 m to accommodate flood of 18500 cumec. The value of Manning’s n = 0.03 was used uniformly for the channel and flood plains on both sides (Design of small Dams, USBR, P.no 577 and VEN TE CHOW - Open Channel Hydraulics, P.no 109). Various other parameter values like contraction coefficient of 0.1 and expansion coefficient = 0.3, value of computational step length etc. also was specified. The file thus produced is called *.geo file and governs the topography and GAD of the project. The view of model in virgin condition is shown in Figure 5.5. The discharge data are created in two files namely steady flow data and quasi unsteady flow data. As no proto type discharge data was provided, an arbitrary single peak flood hydrograph (Figure 5.6) was developed up to 18500 cumec. The time duration of arbitrary flood hydrograph was kept 1 year. The quasi unsteady flow file (*.flowq) requires upstream boundary for main river, downstream boundary condition, flow change condition (if any). The upstream boundary conditions for river Betwa was defined at the upstream most cross section simulated in the model. The upstream boundaries were defined as flow series providing flow and flow duration (in hours) for each of the profile to be run. The computational interval for updating the results is also set as 24 hours. The downstream boundary is defined at the downstream most end of the model and is generally opted as normal depth controlled by general slope of the river.

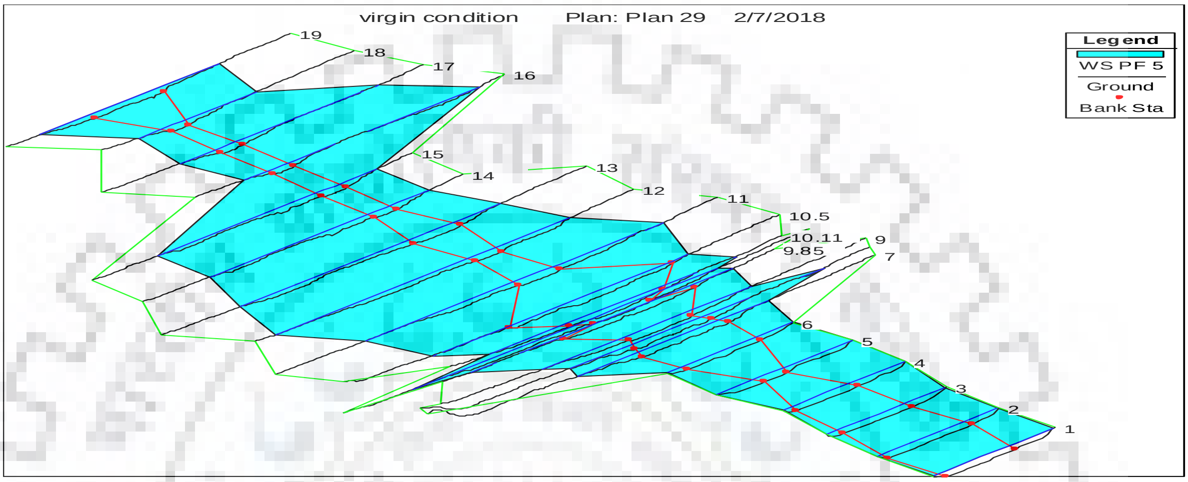

Figure 5.5. HEC-RAS Model in virgin Condition.

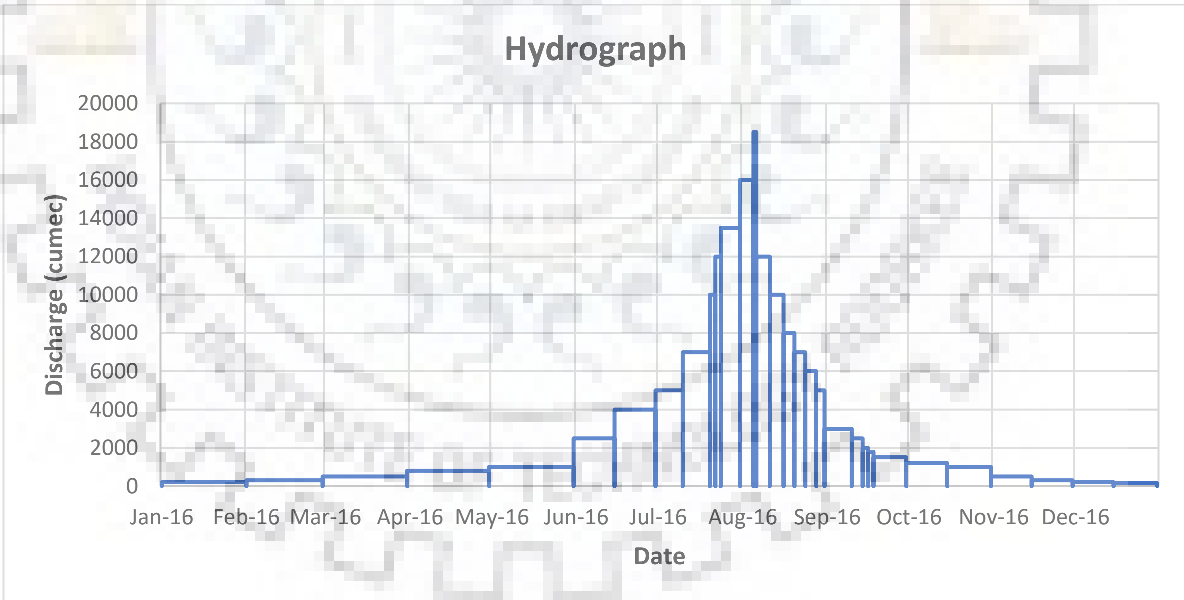

Figure 5.6. Single Peak Hydrograph as used in Mathematical Model

5.4.6 Approach to Model Validation

The model of River Betwa was validated by developing Rating Curves for different discharges up to maximum discharge of 18500 cumec at 200 m downstream of the dam. The developed Curve was then compared with revised Tail Rating Curve of the project as shown in Figure.5.7.

As both curves almost, tallies with each other so the model was considered validated and good for further work.

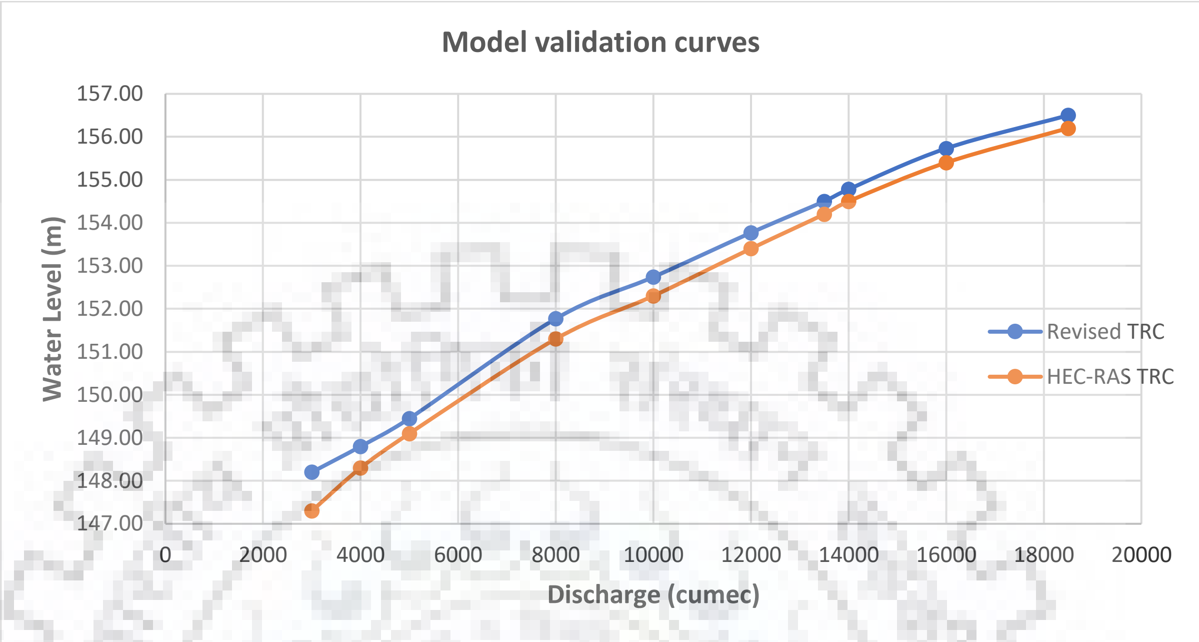

Figure 5.7. Tail Rating Curve at 200 m downstream for model validation.

CHAPTER-VI

6 RESULTS AND DISCUSSIONS:

6.1 Model Running Under Virgin Condition

After validating the model, the river hydraulics was analysed under virgin condition to observe various hydraulic parameters. Following curves were developed: -

a) Rating curve of the river at downstream (Figure 6.1.) and upstream of the dam site location (Figure 6.2.).

b) Water Surface profiles at different discharge profiles (Figure 6.3).

c) Velocity distribution near dam sites (Figure 6.4).

d) Dam axis profile change due to sediment (Figure 6.5.)

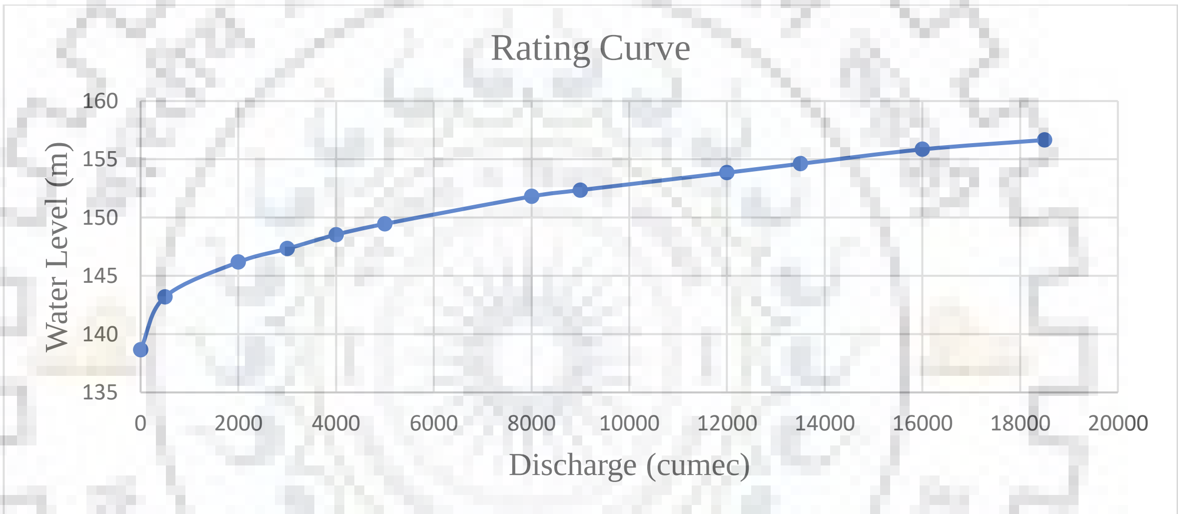

Figure 6.1. Rating Curve 50 m upstream of Dam under virgin condition

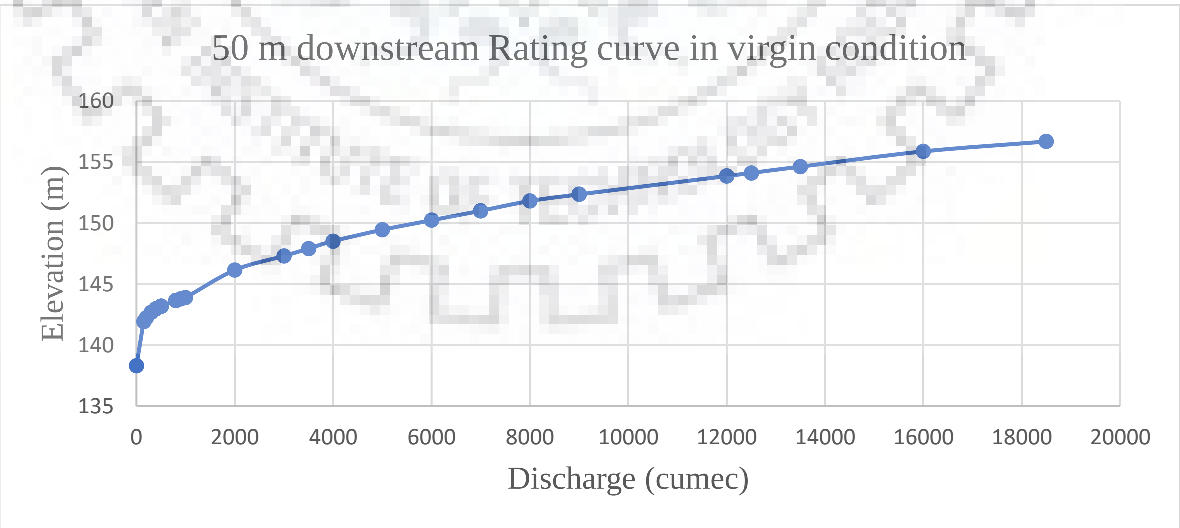

Figure 6.2. Rating Curve 50 m down stream of Dam under virgin condition

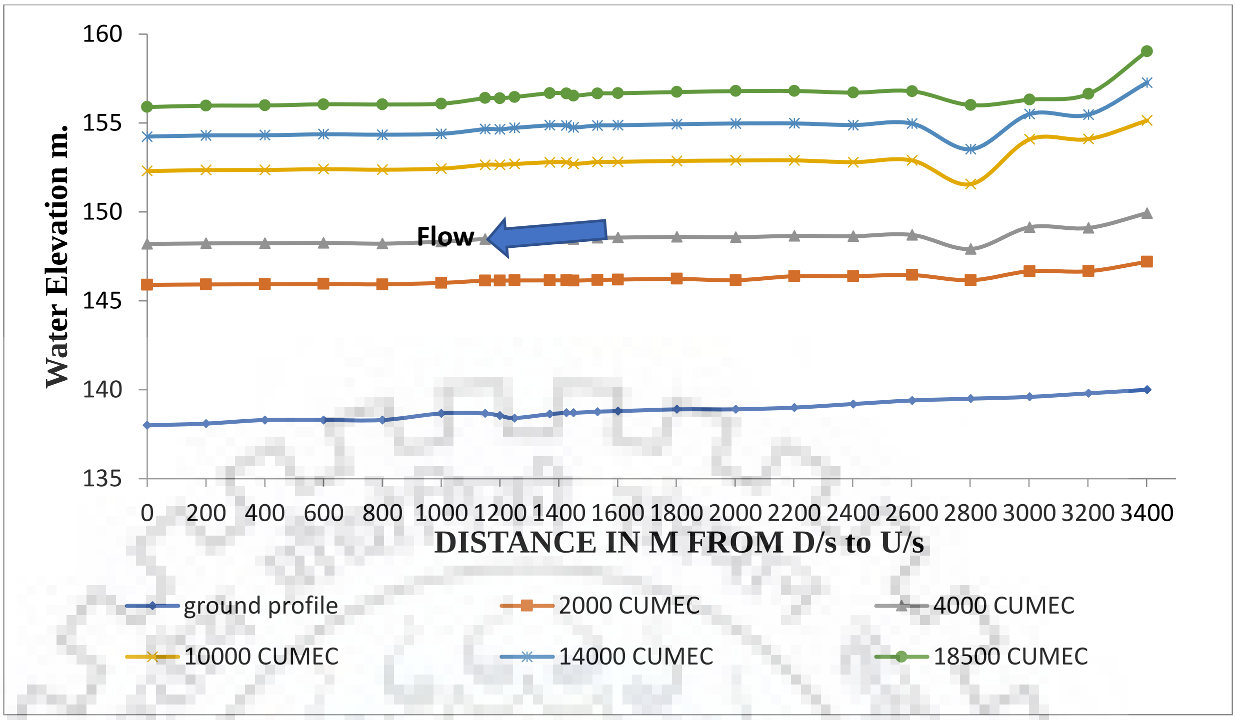

Figure 6.3. Water Surface Profile under Virgin Condition

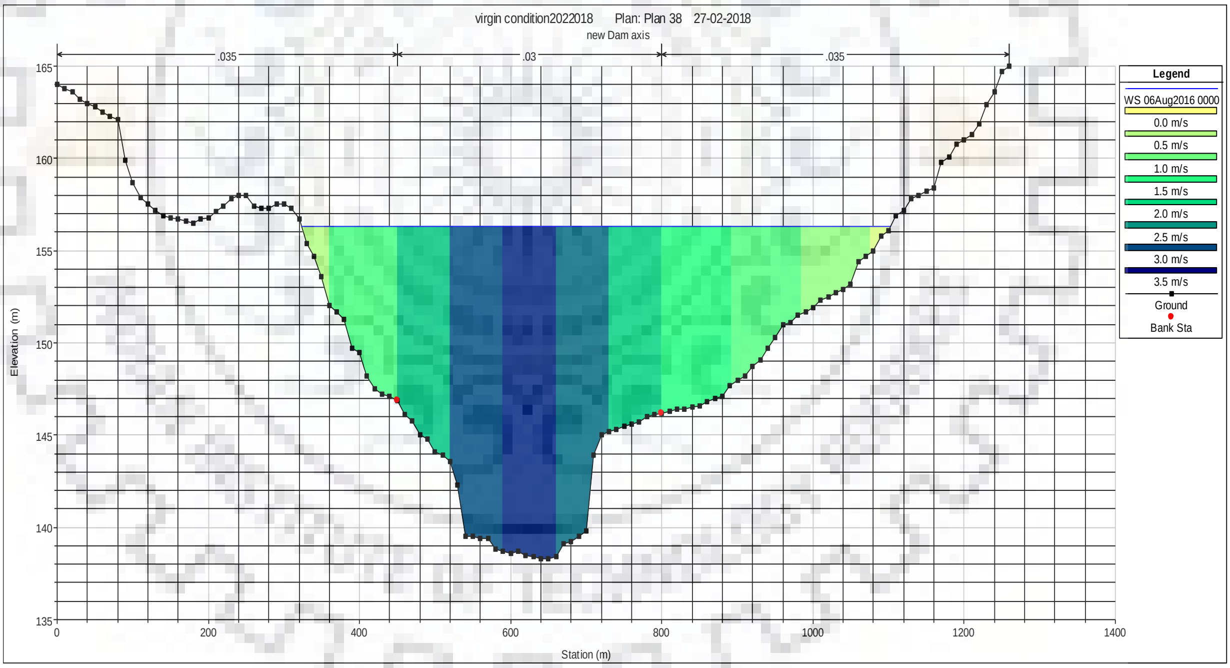

Figure 6.4. Velocity distribution at Dam axis under Virgin condition

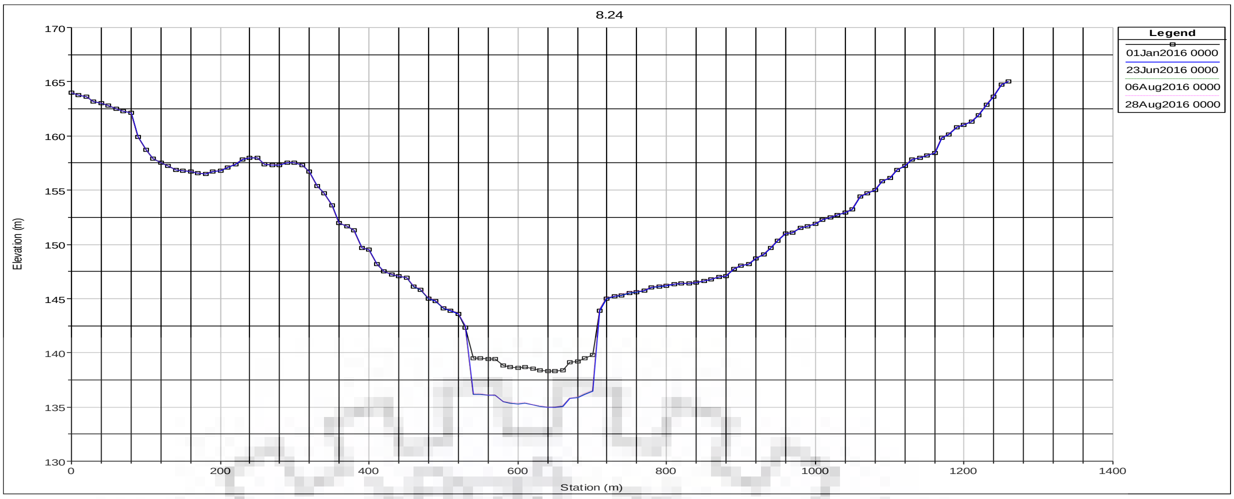

Figure 6.5. Dam axis profile change due to sediment

6.2 Simulation of Proposed Dam, Spillway and Appurtenant

6.2.1 Incorporation of Spillway in Model

After running the model under virgin conditions, inline Structure as being executed at site was incorporated on model by simulating all the bays, piers, gates etc. Input required to simulate gates and inline structure such as Spillway approach height, weir crest shape, downstream and upstream embankment side slope, gate parameters like height, width, type and invert were taken as per salient features of the project described in Table 1.1. A view of proposed structure as simulated on the model is shown in Figure 6.6.

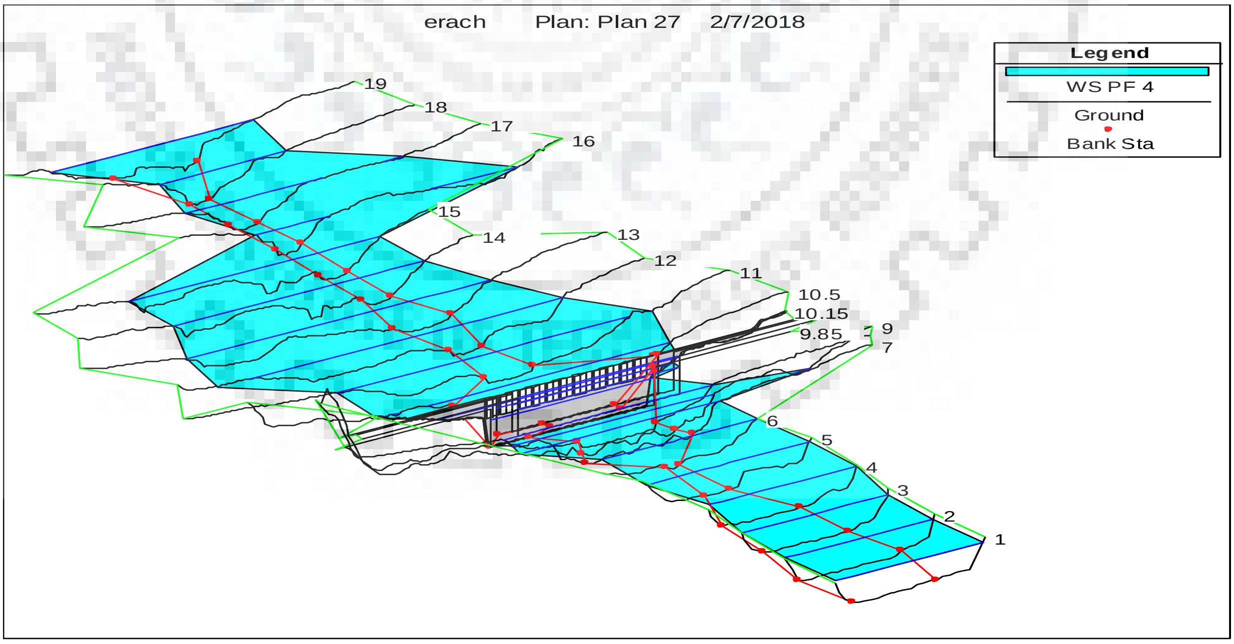

Figure 6.6. HEC-RAS Model with inline structure

6.2.2 Analysis with Proposed Structure at Site

After incorporating the inline structure and other hydraulically related works, the model was rerun for full range of discharges as per hydrograph in Figure 5.6 of chapter 5. Following observations were made after running the mode:

i) Rating of Spillway Crest under Free Flow Condition

A curve was developed for various discharges passing over the spillway under free flow conditions against different water levels required in the upstream to pass the corresponding discharge. The rating curve thus obtained is shown in Figure 6.7.

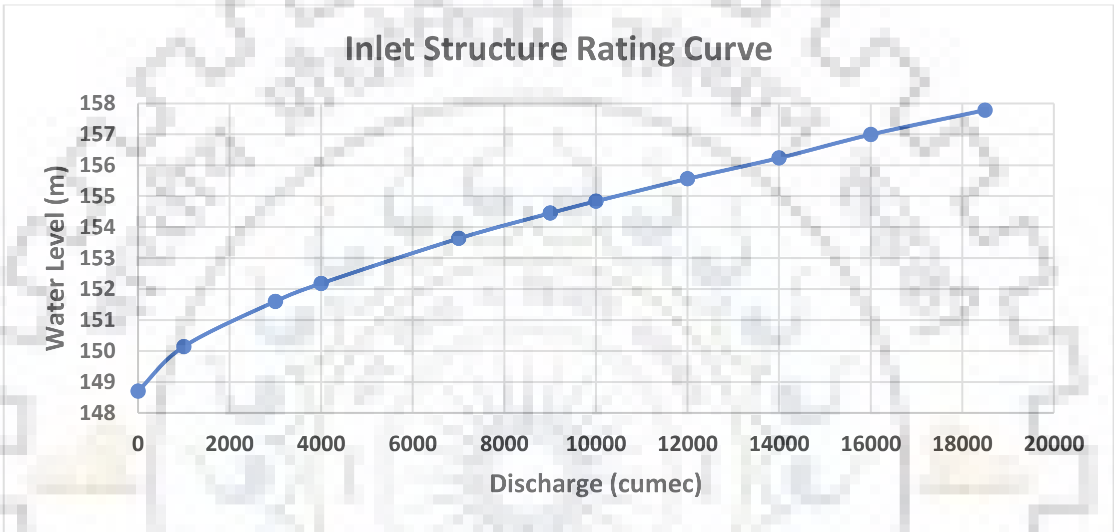

Figure 6.7. Rating Curve of Spillway (Free Flowing Conditions)

Water Surface Profiles for different discharges flowing freely over the spillway are shown below in Figure 6.8.

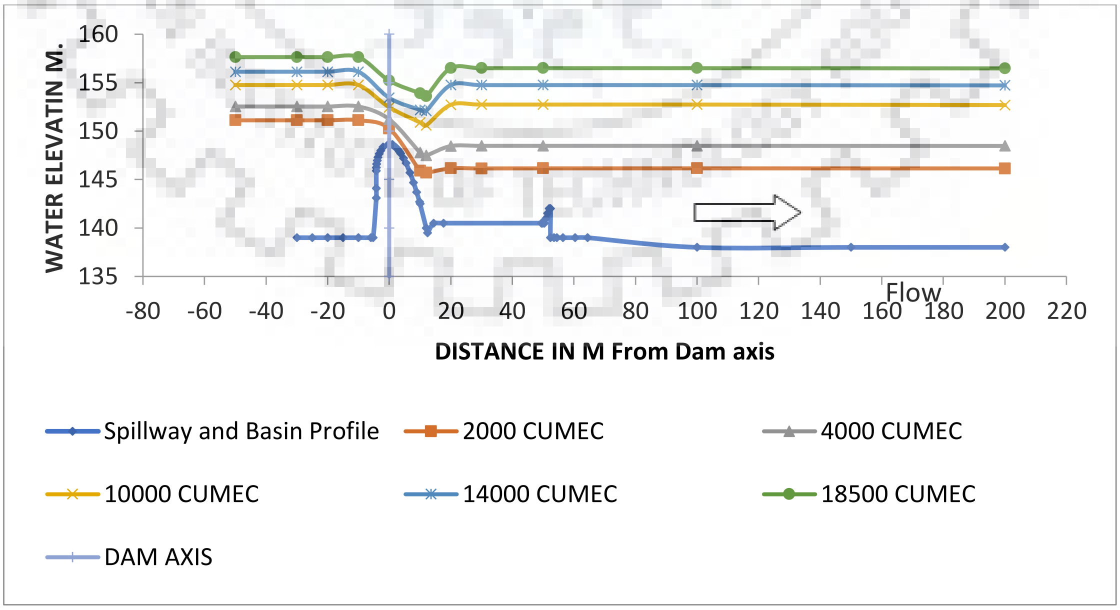

Figure 6.8. Water Surface Profile under Free Flow Condition

iii) Gate Rating curve of spillway were developed for full reservoir level i.e 156.5 m, 154.0 m and at pond level of 151 m. These curves are developed with all gates opened equally. The obtained rating curves for all three reservoir levels are shown below in Figure 6.9.

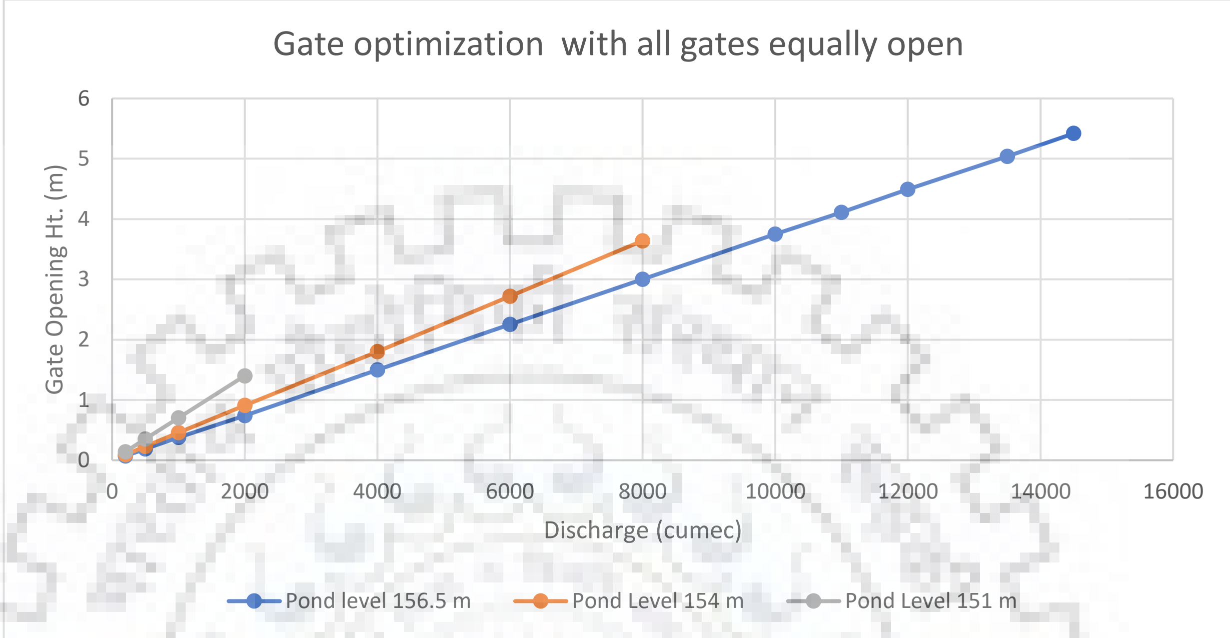

Figure 6.9. Rating Curve of Spillway (Ponded Condition)

Water Surface profiles for different discharges under ponded condition over the spillway are shown below in Figure 6.10.

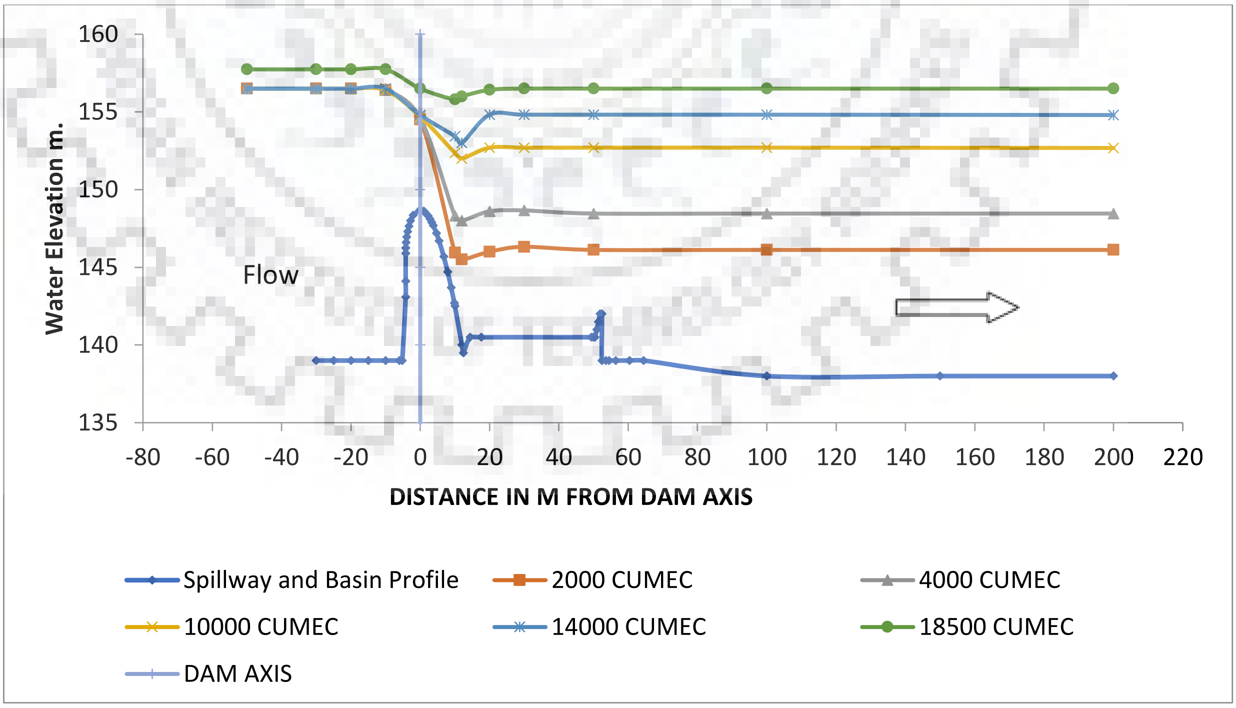

Figure 6.10 Water Surface Profile under Ponded Condition

6.3 Discharge Coefficient and Its Variation with Head over the Crest

The Cd of the crest was obtained under free-flowing condition for different range of discharges up to design discharge. The value of discharge coefficient varies from 1.68 at 2000 cumec and to 1.55 at 18500 cumec. The variation of Cd with discharge and head is shown in Figure 6.11.

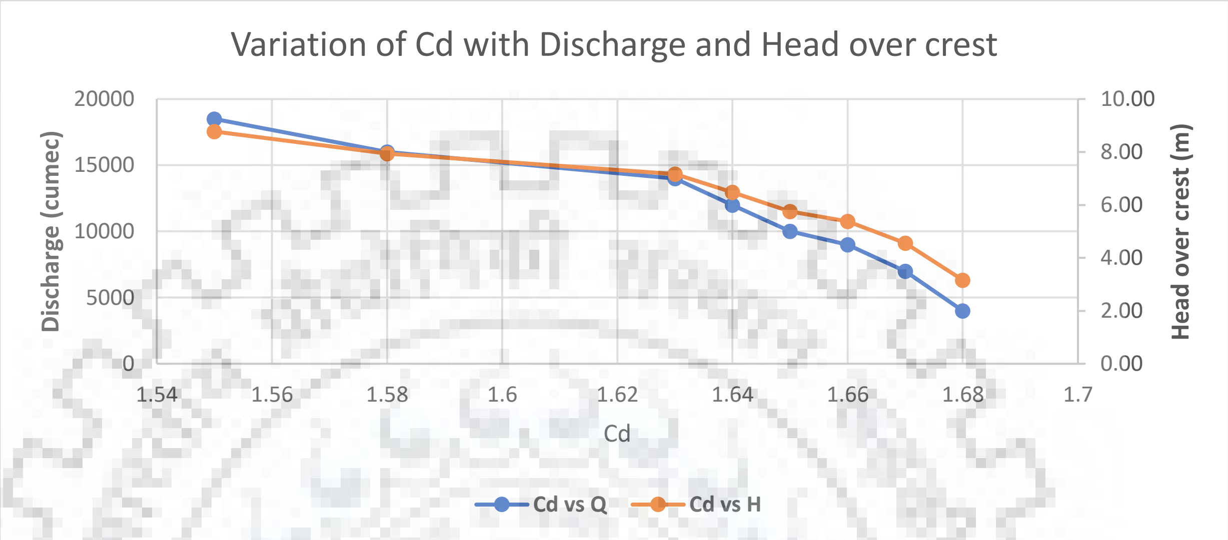

Figure 6.11 Variation of Cd with Discharge and Head over Crest

The coefficient of discharge was also computed under Ponded conditions at FRL. The Cd under these conditions was found to vary from 0.5 to 0.62.

6.4 Velocity Observations in the Upstream and Downstream of Dam Axis

Velocity observations were also recorded through mathematical models at different cross-sections under virgin condition of the river and after incorporated the dam on the axis. The observations at different cross-sections under different conditions are plotted in Figure 6.12, 6.13 and 6.14. It can be seen that the velocity in the reservoir vary from 0.5 to 2.5 m/sec at maximum discharge of 18500 passing free over the spillway (Figure 6.12.) against 0.5 to 2.8 m/sec passing in the river under virgin condition (Figure 6.13.) at 50 m upstream of the dam. With revised Tail water level, the FRL of 156.5 can only be achieved up to a discharge of 14800 cumec for which the velocities in the reservoir are of the order of 0.43 to 2.27 m/sec at 50 m upstream (Figure 6.14.).

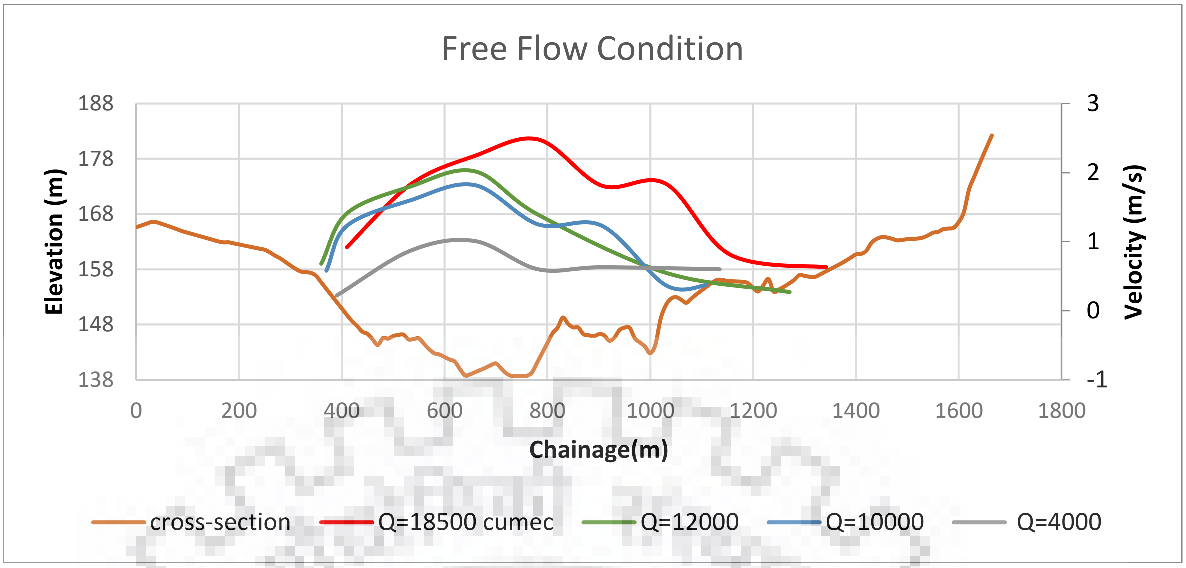

Figure 6.12. Free Flow Condition Velocity Profiles at 50 m U/s of Spillway.

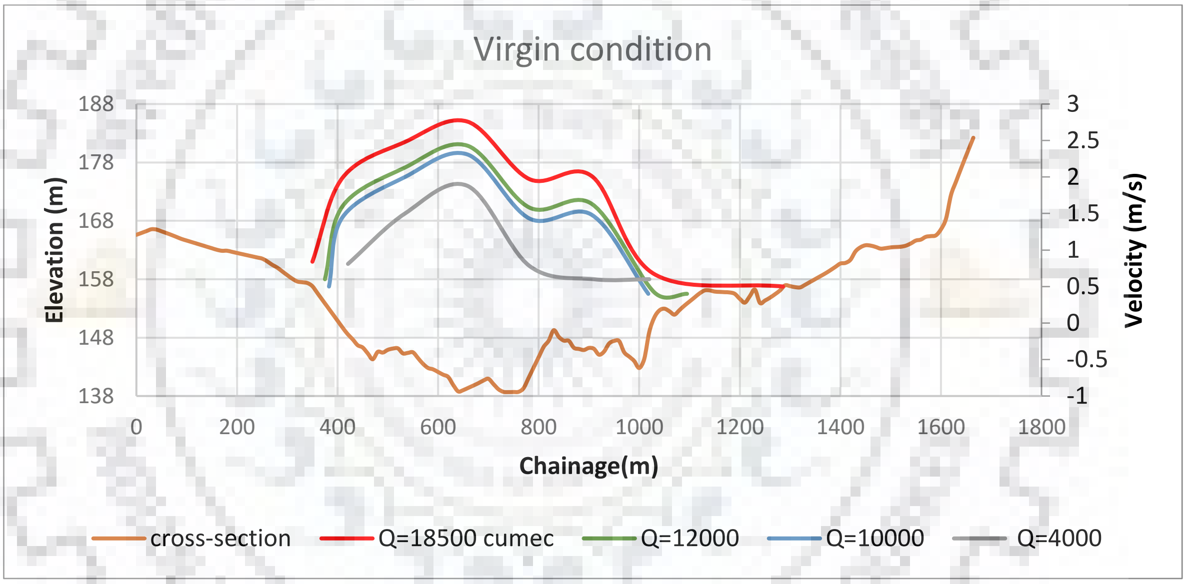

Figure 6.13. Virgin Condition Velocity Profiles at 50 m U/s of Spillway.

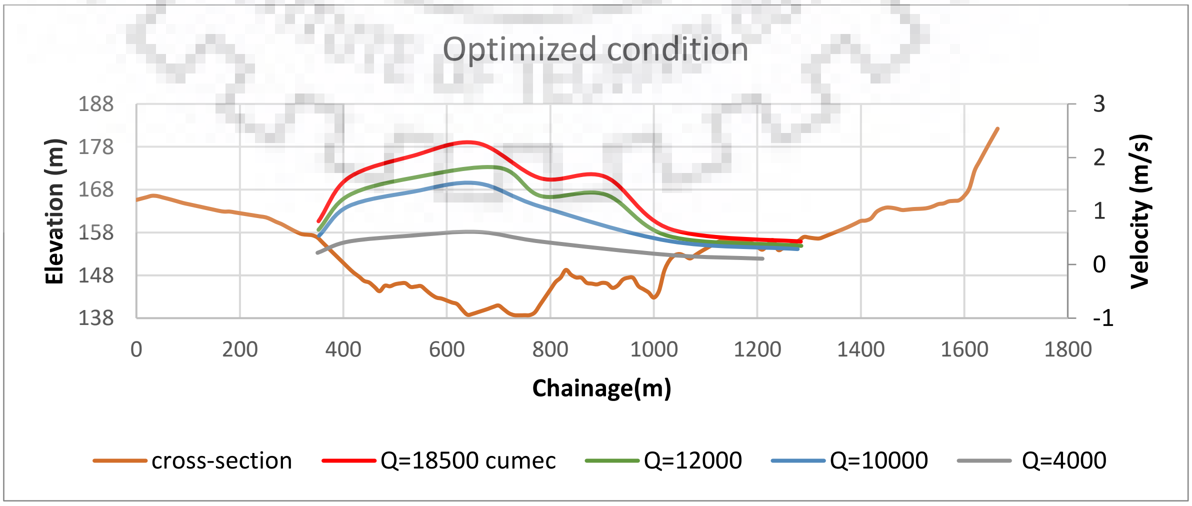

Figure 6.14. Optimized Condition Velocity Profiles at 50 m U/S of Spillway.

6.5 Optimization of Basin Parameters

Since HEC-RAS is a 1d model and does not simulate hydraulic jump condition in a stilling basin, therefore, mathematical calculations based upon conventional formula were used to find out basin parameters and its optimization. Based on these calculations jump heights obtained under different operating conditions were plotted against Tail water depth and is shown below (Figure 6.15).

It is clear from the Figure 6.15 that Tail water depth is always higher than the jump depth throughout the range of discharges. Even under the ponded condition, the tail depth is higher than the required jump depth. Calculations of these basin parameters are shown in Table 6.1 & 6.2 for free flow and ponded condition respectively. The computation indicated requirement of a higher seated basin even for the formation of submerged jump at higher discharges. Provision of basin at such a higher elevation is neither feasible nor possible.

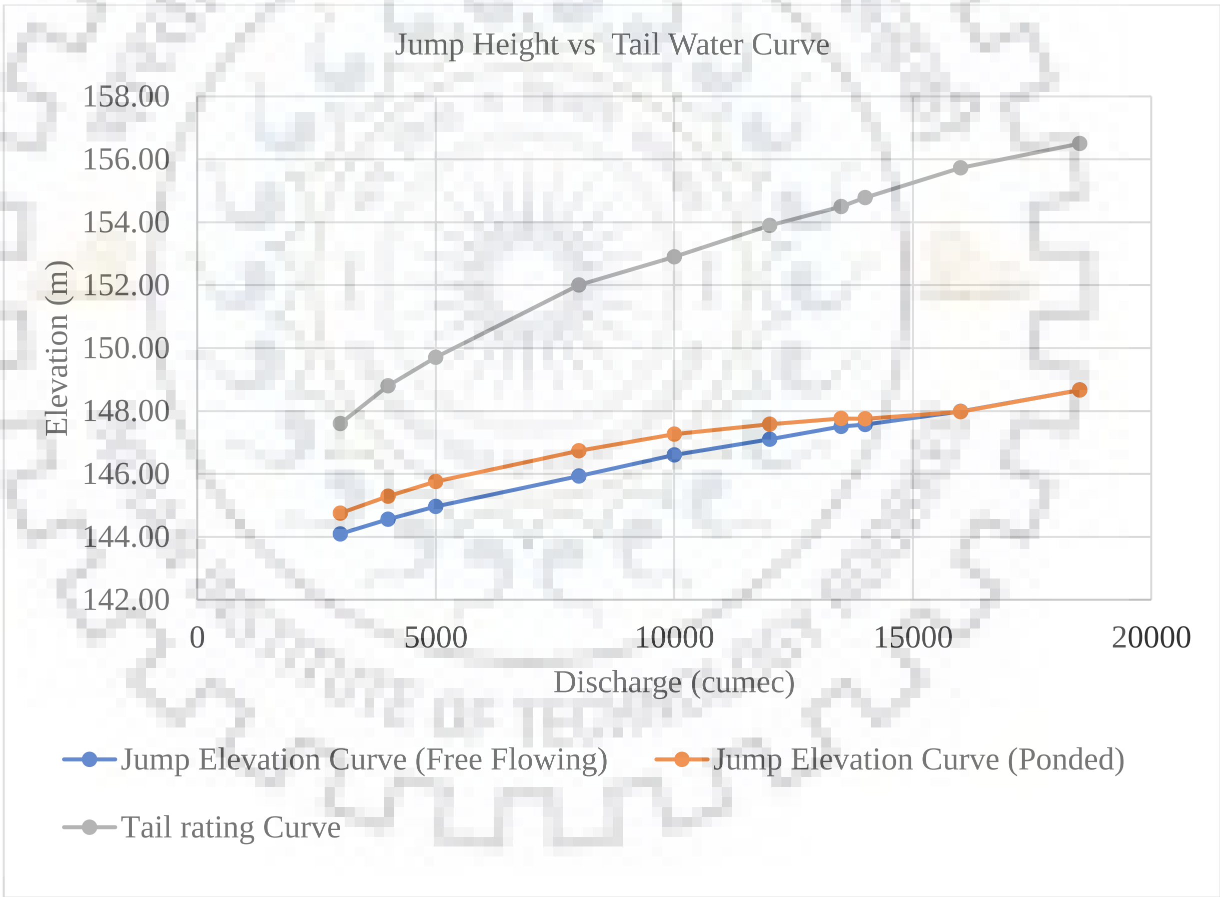

Figure 6.15. Revised Tail Rating Curve Vs Jump Height Curve (Stilling Basin at El.140.5m).

TABLE 6.1.

Calculation of Hydraulic Parameters for Stilling Basin

(*Theory and Design of Irrigation Structures,* Vol.-II, By Varshney , Gupta and Gupta)

(Free Flow Conditions, Total Basin width =492.50 m)

Sl. No.	Q cumec	US WATER LEVEL (M)	TWL m	q (cumec/m)	U/S TEL (m)	D/S TEL (m)	HL (m)	D1 (m)	D2 (m)	Ef2	Basin El. (m)	Basin Length (m)	Froude no.	Dc (m)
1	18500	157.78	156.5	37.56	157.82	156.56	1.26	3.12	8.16	9.24	147.32	25.22	2.18	4.10
2	18000	157.62	156.04	36.55	157.66	156.10	1.56	2.94	8.27	9.26	146.84	26.66	2.32	4.03
3	16000	156.99	155.74	32.49	157.03	155.80	1.23	2.80	7.48	8.44	147.36	23.43	2.22	3.73
4	14000	156.23	154.78	28.43	156.2695	154.84	1.43	2.45	7.07	7.89	146.94	23.12	2.37	3.41
5	13500	156.08	154.5	27.41	156.12	154.56	1.56	2.34	7.01	7.79	146.77	23.38	2.45	3.33
6	12000	155.56	153.9	24.37	155.60	153.96	1.64	2.10	6.60	7.30	146.66	22.50	2.55	3.08
7	10000	154.84	152.9	20.3	154.88	152.96	1.92	1.75	6.10	6.66	146.29	21.73	2.79	2.73
8	8000	154.04	152	16.24	154.07	152.06	2.02	1.44	5.43	5.88	146.17	19.92	2.99	2.36
9	5000	152.66	149.7	10.15	152.69	149.75	2.94	0.88	4.47	4.73	145.02	17.94	3.93	1.73
10	4000	152.18	148.80	8.12	152.21	148.85	3.36	0.70	4.06	4.26	144.59	16.81	4.46	1.49
11	3000	151.6	147.6	6.09	151.63	147.65	3.97	0.51	3.59	3.74	143.91	15.41	5.30	1.23

TABLE 6.2.

Calculation of Hydraulic Parameters for Stilling Basin

(Pond Conditions, Total Basin width =492.50 m)

Sl. No.	Q cumec	US WATER LEVEL (M)	TWL m	q (cumec/m)	U/S TEL (m)	D/S TEL (m)	HL (m)	D1 (m)	D2 (m)	Ef2	Basin El. (m)	Basin Length (m)	Froude no.	Dc (m)
1	18500	157.78	156.50	37.56	157.82	156.56	1.26	3.12	8.16	9.24	147.32	25.22	2.18	4.10
2	16000	156.99	155.74	32.49	157.03	155.80	1.23	2.80	7.48	8.44	147.36	23.43	2.22	3.73
3	14000	156.5	154.78	28.43	156.5389	154.84	1.70	2.36	7.25	8.03	146.81	24.42	2.50	3.41
4	13500	156.5	154.5	27.41	156.54	154.56	1.98	2.22	7.26	7.99	146.57	25.19	2.64	3.33
5	12000	156.5	153.9	24.37	156.54	153.96	2.58	1.90	7.08	7.68	146.27	25.90	2.97	3.08
6	10000	156.5	152.9	20.30	156.53	152.96	3.58	1.50	6.76	7.22	145.73	26.30	3.52	2.73
7	8000	156.5	152	16.24	156.53	152.06	4.48	1.17	6.23	6.58	145.47	25.34	4.12	2.36
8	5000	156.5	149.7	10.15	156.53	149.75	6.77	0.67	5.25	5.44	144.31	22.89	5.85	1.73
9	4000	156.50	148.80	8.12	156.53	148.85	7.67	0.53	4.79	4.94	143.92	21.32	6.77	1.49
10	3000	156.50	147.60	6.09	156.52	147.65	8.87	0.38	4.25	4.36	143.30	19.34	8.18	1.23

Since at this advance stage of construction of the project nothing much was possible in design alteration, therefore, an alternative and compromising solution to negotiate the energy for frequently occurring discharges was considered in the proposal, the stilling basin was proposed to be divided into four compartments with the provision of divide wall after every 6 bays of the spillway. It is thought that if lower discharge say up to 4000-5000 cumec are passed in isolation (through one compartment of basin), the discharge intensity of the flow in the partitioned basin increases which subsequently increases conjugate depth also. This phenomenon may result in reduction of jump submergence up to certain frequently occurring discharges and thereby reducing the basin elevation up to some extent. Computations for the partitioned basin parameter were also made and are compared in Table 6.3. The plot of Tail Rating Curve and jump height in the partitioned basin (Figure 6.16.) reveals that with the provision of divide walls, the jump start submerging after a discharge of 6000 cumec.

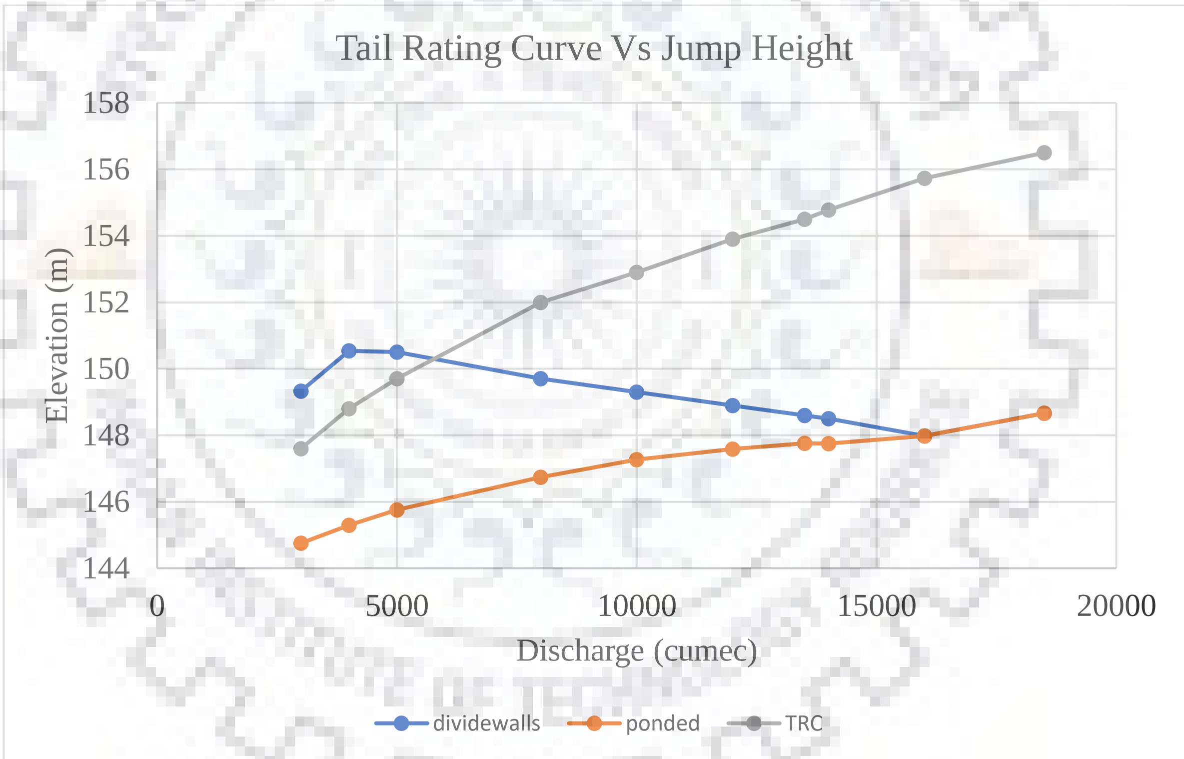

Figure 6.16. Revised Tail Rating Curve Vs Jump Height Curve (With and Without Divide Walls in basin) Stilling Basin at El.140.5m.

TABLE 6.3.

Calculation of Hydraulic Parameters for Stilling Basin – Pond Conditions (without and with Divide Walls)

(With divide walls in basin after every 6th bay)

Q m³/sec	**TWL**	**q (m³/sec)**		**U/S TEL (m)**		**D/s TEL (m)**		**H_L (m)**		**D1 (m)**		**D2 (m)**		**EF2**		**Basin elev. (m)**	
		w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall	w/o divide wall	divide wall
18500	156.5	37.6	37.6	157.8	157.8	156.6	156.6	1.3	1.3	3.1	3.1	8.2	8.2	9.2	9.2	147.3	147.3
16000	155.7	32.5	32.5	157.0	157.0	155.8	155.8	1.2	1.2	2.8	2.8	7.5	7.5	8.4	8.4	147.4	147.4
14000	154.8	28.4	28.4	156.5	156.5	154.8	154.8	1.7	1.7	2.4	2.4	7.2	7.2	8.0	8.0	146.8	146.8
13500	154.5	27.4	27.4	156.5	156.5	154.6	154.6	2.0	2.0	2.2	2.2	7.3	7.3	8.0	8.0	146.6	146.6
12000	153.9	24.4	32.5	156.5	156.5	154.0	154.0	2.6	2.6	1.9	2.4	7.1	8.3	7.7	9.1	146.3	144.9
10000	152.9	20.3	27.1	156.5	156.5	153.0	153.0	3.6	3.6	1.5	1.9	6.8	7.9	7.2	8.5	145.7	144.4
8000	152.0	16.2	32.5	156.5	156.5	152.1	152.1	4.5	4.5	1.2	2.1	6.2	9.1	6.6	9.8	145.5	142.3
5000	149.7	10.2	20.3	156.5	156.5	149.8	149.8	6.8	6.8	0.7	1.3	5.3	7.6	5.4	8.0	144.3	141.8
4000	148.8	8.1	32.5	156.5	156.5	148.9	148.9	7.7	7.7	0.5	1.8	4.8	10.0	4.9	10.6	143.9	138.3
3000	147.6	6.1	24.4	156.5	156.5	147.7	147.7	8.9	8.8	0.4	1.3	4.3	8.8	4.4	9.2	143.3	138.5

Remarks: Up to 4000 cumec - only 123 m width of the basin is effective. (only one cluster of 6 bays to be operated)

Up to 8000 cumec - 246 m width of the basin is effective. (Two cluster of 6 bays each to be operated)

Up to 12000 cumec - 369 m width of the basin is effective. (Three cluster of 6 bays each to be operated). After 12000 cumec open all gates.

6.6 Discussions

The analysis of mathematical model with raised TWL 156.5 m indicated that the maximum discharge of 18500 cumec can pass through the provided spillway at U/S water level of 157.8 m against 156.5 m with earlier reported TWL. Therefore, the embankment upstream of the dam are recommended to be raised with adequate Free board.

Since sill of the overflow structure, vent parameters and gate dimensions are already under construction therefore, no alteration is possible in the design parameters. Raising of TWL, therefore, caused greater submergence of spillway crest and resulted in higher heads to pass the design discharge. This resulted in reduction of the discharge coefficient also shown in Figure 6.11 of chapter 6. Even under ponded condition the value of Cd was found to reduce as expected.

The Tail Water Depth even before the revision of TRC, was higher than the conjugate depth for a wide range of discharges which caused formation of submerge jump even at the lower discharge of 3000-4000 cumec also. This Figure 6.15 also indicates that after the revision of TRC that the available water depth is higher than jump depth at all discharges. Under free conditions, it was observed that the available depth downstream of glacis is too high and is almost 200% of conjugate water depth at higher discharges. Under these conditions no clear jump is expected, and the entering jet shall oscillate from bottom to surface resulting in pulsating action. Else, the requirement of basin floor is at quite high elevation which is neither feasible nor possible at site (Table 6.1). Even for the ponded condition, the computation for basin parameters reveal that due to high availability of water depth in downstream the basin elevation requirement is quite high even for low discharges. These computations are made with all gates equally open to pass floods in downstream. Raising of the floor, to that extent was not considered feasible (Table 6.2).

Generally, it is advised to pass the low floods in ponded conditions because if it is passed in free flow conditions upstream level shall come down below FRL and it may take several days to fill reservoir up-to FRL again.

The plot (Figure 6.15) of Tail Water Depth and jump depth under free flow conditions and under ponded condition show that, although jump depths under ponded conditions are above than the free-flowing depths, still the tail water depth is quite higher than the required conjugate depths, therefore possibilities of good hydraulic jump are not seen even under ponded condition.

After considering all above conditions, it was decided to optimize stilling basin at least for lower discharges for producing a satisfactory jump under ponded conditions since most of the time the spillway shall be operated under ponded conditions. It is reported that a discharge of 4000 cumec to 5000 cumec frequently occurs at site, therefore it was thought better to optimize stilling basin floor at such an elevation which can produce an acceptable hydraulic jump at frequently occurring discharges. As such, a proposal for stilling basin with floor elevation at 140.50 m with 3 divide walls one after every 6th bay in the basin was considered. This elevation is supposed to be quite feasible to obtain at site under construction against the earlier recommended basin elevation of 137 m. Jump elevation curve with divide walls after every 6th bay, was also computed and plotted against tail rating curve in ponded conditions, with and without divide walls in the basin. It may be seen from the Figure 6.16 that with the provision of divide walls in the basin jump height curve remains above the tail rating curve up to a discharge of about 5000 cumec and becomes submerged after that.

The proposal is expected to help in increasing the discharge intensity in the basin and subsequently increases conjugate depth “d2” and ultimately shall reduce the submergence of the jump up to certain frequently occurring discharge. This phenomenon brings the required basin elevation to a desired feasible elevation. The Table 6.3 of the report shows computations of basin parameters for this arrangement under ponded conditions. It is indicated from the table that the floor provided at elevation of 140.5 m may function satisfactorily for formation of jump up to a flood of 4000 to 5000 cumec which the flood of annual occurrence.

. For discharges exceeding 5000 cumec, the jump is submerged reducing Froude number below 4 and therefore no clear jump is expected at higher discharges only a weak drowned jump may form and the jet oscillating from bottom to surface resulting in pulsating action. Wavy action is expected in a long reach downstream of spillway in such cases at higher discharges.

To reduce the pulsating action of flow in the basin at higher discharges i) a row of baffle blocks is recommended ii). It is also recommended to provide C.C. blocks in sufficient reach in the downstream as per Indian Standard code. These blocks shall also help in checking any churning action during asymmetric operation of the basin. iii) The height of the divide walls in the basin proposed up to El. 152.0 m which will compartmentalize the flood up to a discharge of 6000 to 8000 cumec. It is suggested that spillway gates should be open cluster wise that is up to 4000 cumec only one cluster of 6 gates should be open to pass the flood so that the active basin width is only one fourth of the total basin width and the procedure should be followed as mentioned in the foot note of Table 6.3.

CHAPTER-VII

7 CONCLUSION AND FUTURE SCOPE

7.1 General

Based on the statistical analysis of the hydrological studies, it can be concluded that the Gumbel distribution can be accepted for the data available of 18 years. The hydrological inputs play a very vital role in hydraulic and structural design. Inappropriate hydrological inputs lead to improper results. As so many latest models and software are available in the market to access the hydrology of the ungauged stations and basin but still there are few limitations in each method.

The Hydraulic jump type stilling basin are taken to be most effective and dependable means of catering the wide range of flow energy over a controlled/ uncontrolled structure. Alternate arrangement of Buckets as Energy dissipaters is sometime not suitable due to heavy sedimentation, geology of the area and cost of the excavation.

The performance of hydraulic jump type stilling basin is always TRC dependent. The best jump formation in the basin is expected when Tail water depth is almost equal or near to jump depth. Where the available water depth is more than the required sequent depth for a wide range of floods, a basin at quite higher level is required for the formation of good jump. In the case study of Erach dam even for moderate discharge of 3000 cumec, the basin elevation works out to be 143.0 m to 144.0 m and 148.0 m at the design flood. (against natural bed level of 138.0m). Provision of such high basin invert is not feasible at site as the construction has already been started and is in advance stage. The TRC has been somehow revised after design of all parameters and during ongoing construction.

7.2 Conclusion

Based on the above studies following conclusion arises:

a) A good jump can be obtained for frequently occurring annual flood, if basin floor is divided with help of divide walls up to optimum heights. Thus, the discharge in the basin shall be isolated in the fractions of 4000-5000 cumec under ponded conditions.

b) This isolation of discharge in the basin increases the “q” in the basin resulting in the increase of sequent depth ‘d2. Consequently, desired basin elevation is reduced.

c) This optimization scheme is expected to be effective and feasible, and it would also provide a reference for future design of such works. The top of the divide walls in the basin should be kept above the TWL for frequently occurring discharges.

d) Emphasises should be given for proper hydrologic and meteorological sites in the remote locations of the rivers where still lot of water resource projects needs to be designed.

7.3 Scope for Further Study

The problem and its remedial measure for Erach dam has indicated need of further following studies/works to provide guidelines.

a) Study the design parameters specially of incoming floods and Tail Rating Curve which are somehow change during the advance stage of construction.

b) Provision and optimization of divide walls with respect to its shape and height in stilling basin to facilitate hydraulic jump formation in situations where Tail Rating Curve is quite High.

c) The function of divide walls extended in the basin against the uplift pressure needs to be studied at large vis-a-vis functioning of PRVs.

d) The operation of Controlled structure in the upstream should carefully be studied in case of a portioned stilling basin in hand.

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APPENDIX 5.1

CHI SQUARE TEST

S.no	Class Interval	upper limit	Ni	(zi) of upper limit	table-2.6 patra	F(zi)	pi=F(zi)-F(zi-1)	Relative frequency	Ei=npi	$(Ni-Ei)^2/Ei$	χ^2
1	0-3000	3000	3	-1.087	0.362	0.138	0.138	0.167	2.482	0.108	0.108
2	3000-6000	6000	2	-0.556	0.212	0.288	0.150	0.111	2.696	0.180	0.180
3	6000-9000	9000	5	-0.025	0.012	0.488	0.200	0.278	3.605	0.539	0.539
4	9000-12000	12000	2	0.506	0.212	0.712	0.224	0.111	4.037	1.028	1.028
5	12000-15000	15000	3	1.037	0.351	0.851	0.139	0.167	2.493	0.103	0.103
6	15000-18000	18000	1	1.568	0.442	0.942	0.091	0.056	1.638	0.249	0.249
7	18000-21000	21000	2	2.099	0.482	0.982	0.040	0.111	0.725	2.240	2.240
			18					1.000		4.447	4.447

No. of class interval=7, No. of Parameter=2, degree of freedom v=4, Significance level α=5%, X^2=4.447, $X^2_{0.95,4}$= 9.48 (Table 2.25, Patra)

Since $X^2 < X^2_{0.95,4}$ therefore the hypothesis is that the Normal distribution fits the data can be accepted at 95 % confidence level.

Kolmogorov-Smirnov Test

i	x	Descending order	Rank (m)	P=m/n+1	$P(x_i)$=1-p	Z_i	from table 2.6 patra	$F(Z_i)$	ABS($P(x_i)$-F(Zi))
1	2656.50	19338.40	1	0.05	0.95	1.80	0.4641	0.964	0.02
2	19027.90	19027.90	2	0.11	0.89	1.75	0.4599	0.960	0.07
3	13942.60	15756.15	3	0.16	0.84	1.17	0.3790	0.879	0.04
4	19338.40	13942.60	4	0.21	0.79	0.85	0.3023	0.802	0.01
5	7213.95	12765.00	5	0.26	0.74	0.64	0.2389	0.739	0.00
6	4974.90	12371.70	6	0.32	0.68	0.57	0.2157	0.716	0.03
7	15756.15	11930.10	7	0.37	0.63	0.49	0.1879	0.688	0.06
8	12765.00	9029.80	8	0.42	0.58	-0.02	0.0080	0.492	0.09
9	6210.00	8984.95	9	0.47	0.53	-0.03	0.0120	0.488	0.04
10	3636.30	8401.90	10	0.53	0.47	-0.13	0.0517	0.448	0.03
11	12371.70	7213.95	11	0.58	0.42	-0.34	0.1331	0.367	0.05
12	6235.30	6235.30	12	0.63	0.37	-0.51	0.1950	0.305	0.06
13	9029.80	6210.00	13	0.68	0.32	-0.52	0.1985	0.302	0.01

14	11930.10	4974.90	14	0.74	0.26	-0.74	0.2704	0.230	0.03
15	8401.90	3636.30	15	0.79	0.21	-0.97	0.3340	0.166	0.04
16	1337.45	2656.50	16	0.84	0.16	-1.15	0.3749	0.125	0.03
17	725.19	1337.45	17	0.89	0.11	-1.38	0.4162	0.084	0.02
18	8984.95	725.19	18	0.95	0.05	-1.49	0.4319	0.068	0.02
								MAX=	**0.09**

Significance Level =5%, Test statics value from table=0.317, Since 0.09< 0.317. The hypothesis that the normal distribution fits the data and can be accepted at 5 % significance Level.

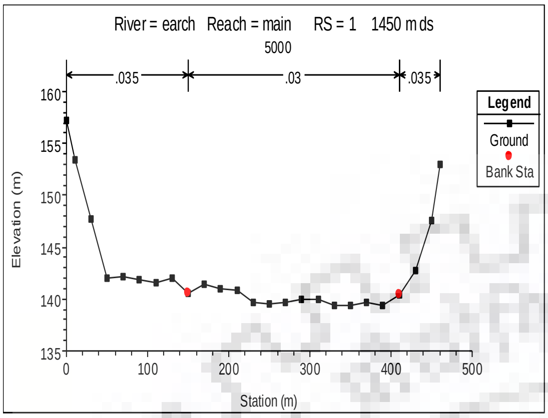
K Values	**As per Return Period**		
Return Period	**K Normal**	**K Gumbel**	**K Pearson III**
50	2.0537	2.5923	2.2247
100	2.3263	3.1367	2.5644
200	2.5758	3.6791	2.8832
500	2.8782	4.3947	3.2798
1000	3.0902	4.9355	3.5649
2000	3.2905	5.4762	3.8394
10000	3.7190	6.7312	4.4439

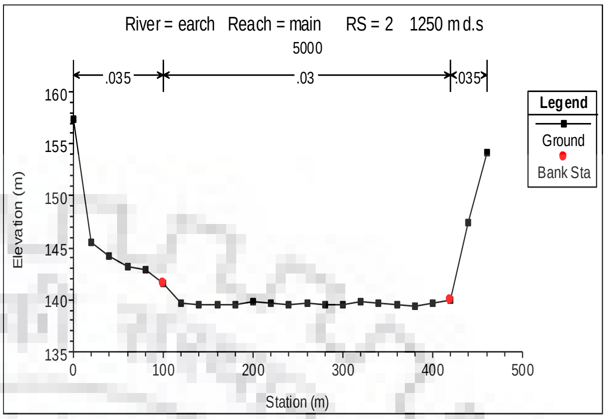
Flood Peaks (cumec)

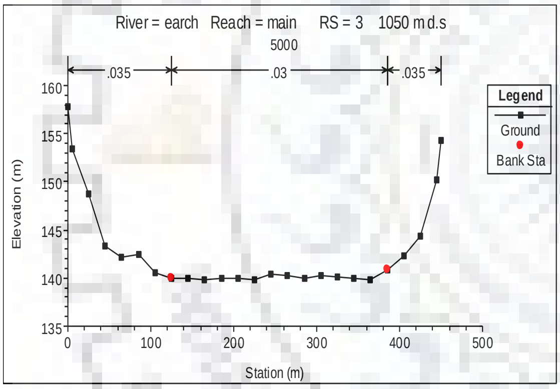
Return Period	**Dischrge Normal**	**Dischrge Gumbel**	**Dischrge Pearson III**
50	13602.62	16645.32	14568.62
100	15142.77	19721.10	16487.54
200	16552.31	22785.64	18288.70
500	18260.46	26828.73	20529.79
1000	19458.64	29884.41	22140.44
2000	20590.28	32938.97	23691.23
10000	23011.20	40029.78	27106.59

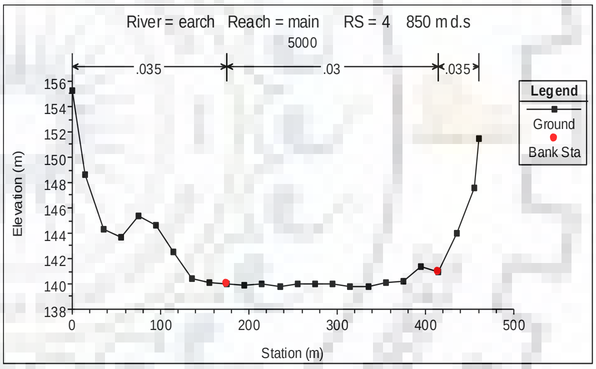
APPENDIX 5.2

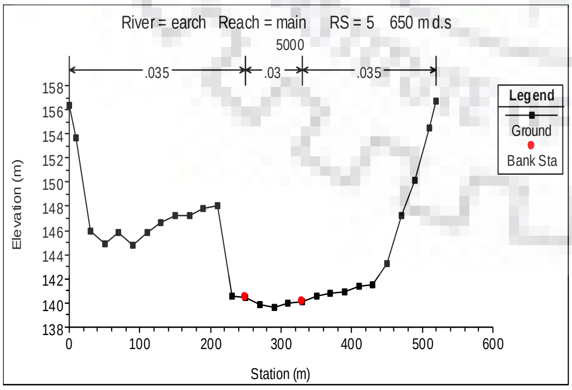
Cross-sections @ 200 m interval from Dam axis (2 km U/S & 1.4 km D/S of the Dam axis)

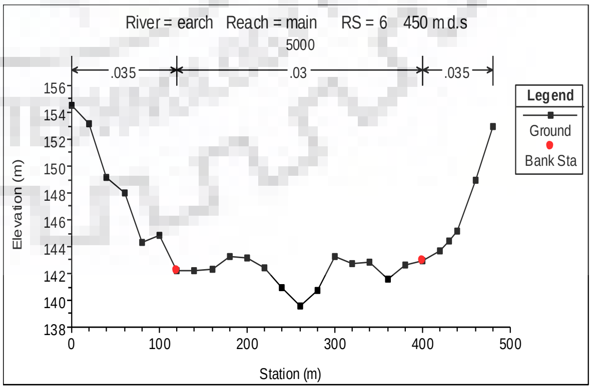


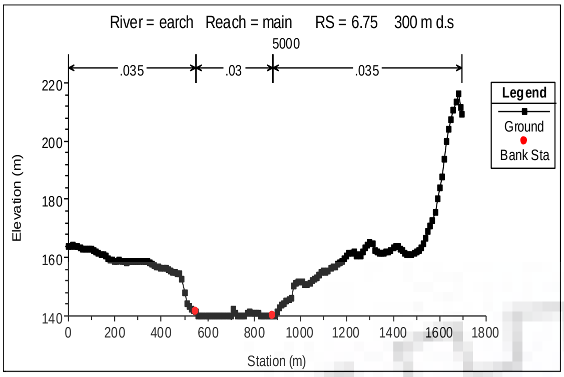








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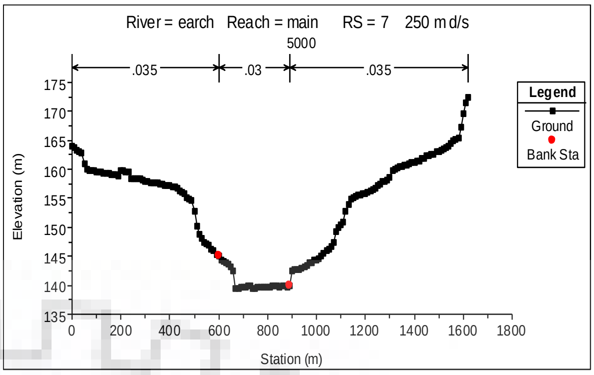
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)
140
160
180
200
220
0 200 400 600 800 1000 1200 1400 1600 1800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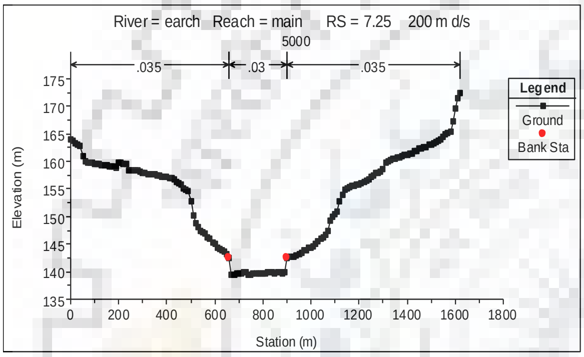
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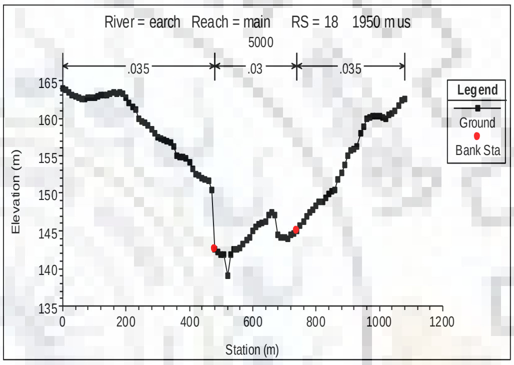
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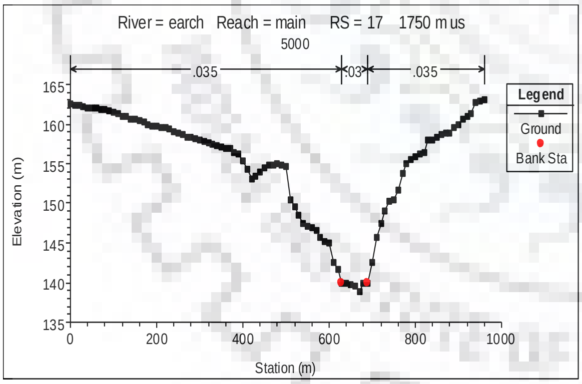
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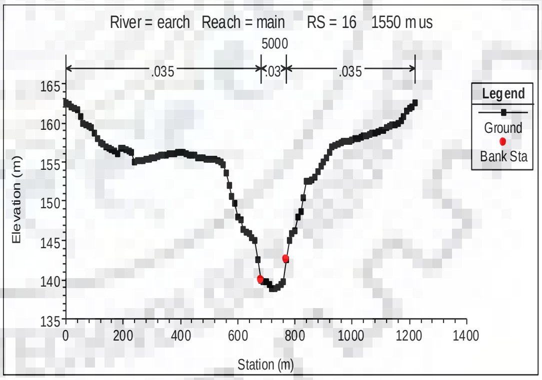
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Elevation (m)
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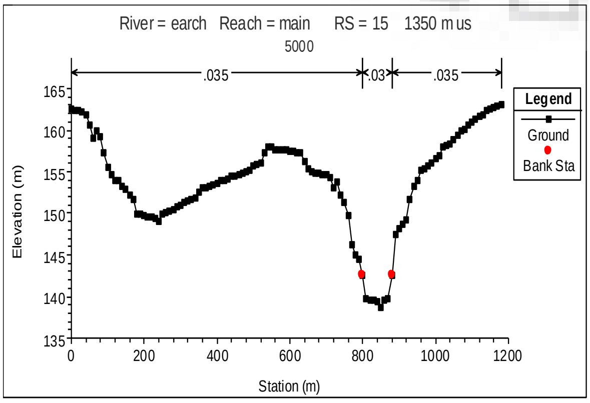
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Bank Sta
Elevation (m)
Station (m)
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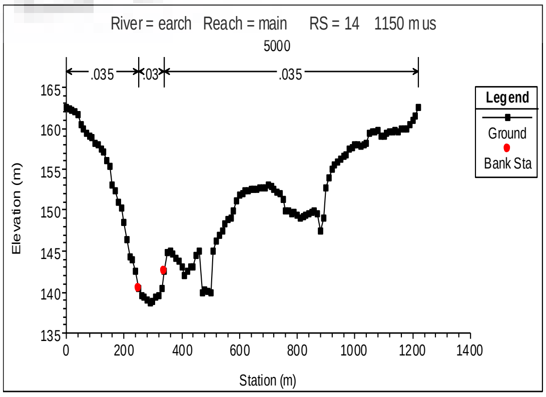
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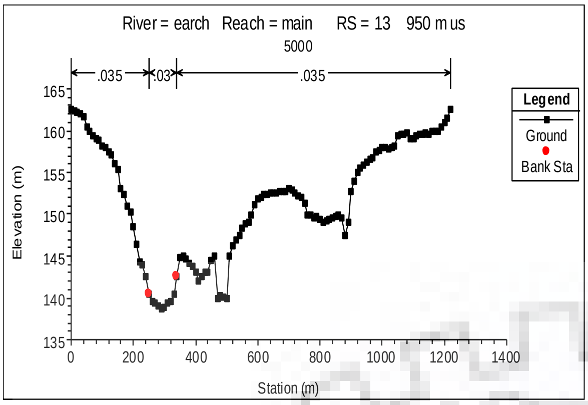
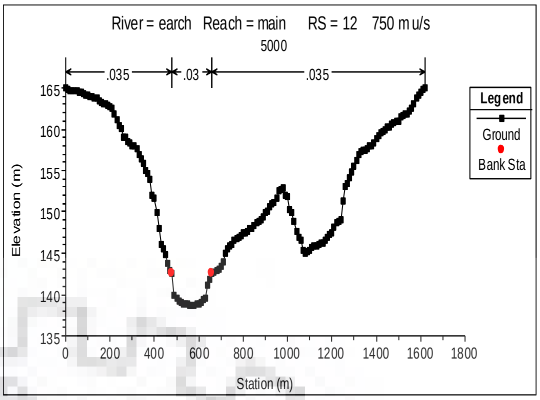
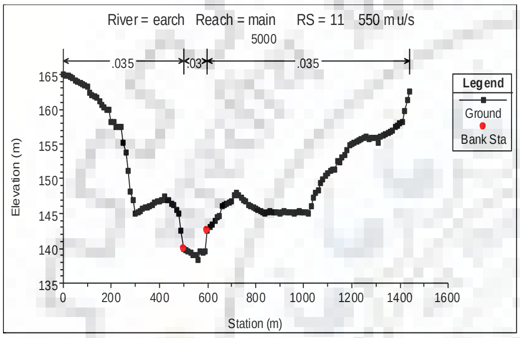
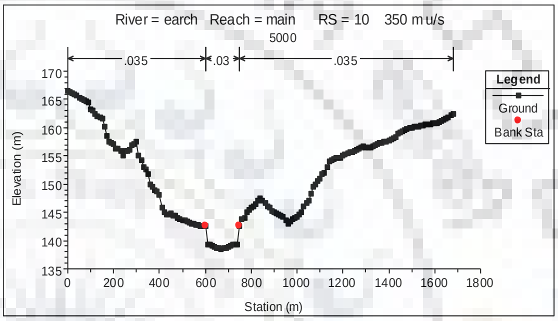
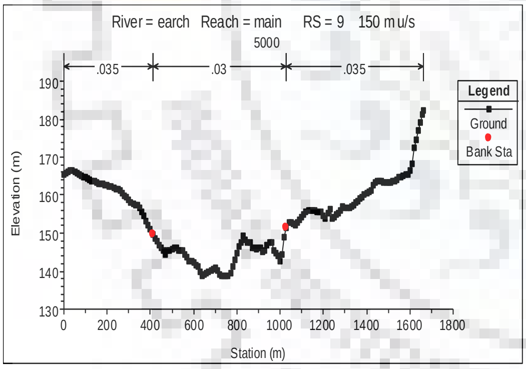
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)
135 140 145 150 155 160 165
0 200 400 600 800 1000 1200 1400

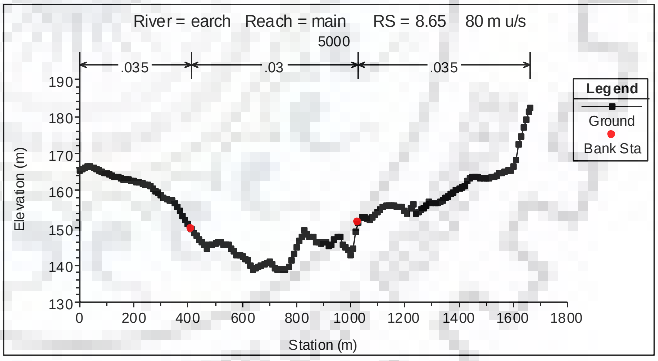

River = earch Reach = main RS = 13 950 m us
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)

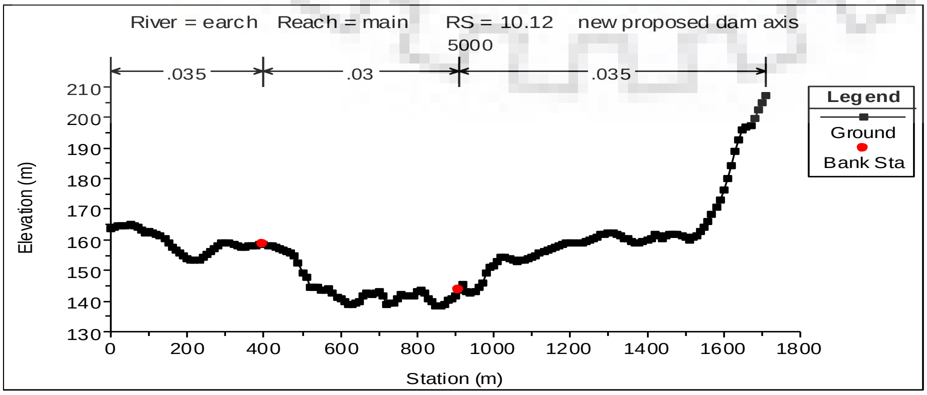

River = earch Reach = main RS = 12 750 m u/s
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)


River = earch Reach = main RS = 11 550 m u/s
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)


River = earch Reach = main RS = 10 350 m u/s
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)


River = earch Reach = main RS = 9 150 m u/s
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)


River = earch Reach = main RS = 8.65 80 m u/s
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)


River = earch Reach = main RS = 10.12 new proposed dam axis
5000
.035
.03
.035
Legend
Ground
Bank Sta
Elevation (m)
Station (m)

Few Photographs of the Erach Dam under Construction

PUBLICATION

S.no	Title of Paper	Journal and Publishing Details	Remarks
1	Optimization of Stilling Basin with High Tail Water Depth	"WATER AND ENERGY INTERNATIONAL" journal published by "CENTRAL BOARD OF IRRIGATION & POWER" in VOLUME 61/RNI, NO. 1 ISSN:0974-4711 pp 62-67	Published in April 2018 edition