Guidelines for Selecting and Accommodating the Inflow Design Floods for Dams

Doc. No. CDSO_GUD_DS_06_v1.0

June 2021

Dam Rehabilitation & Improvement Project

Central Water Commission
Ministry of Jal Shakti
Department of Water Resources,
River Development & Ganga Rejuvenation
Government of India

Front Cover Photograph: Srisailam Dam, which spans the Krishna River on the border of Mahabubnagar District, Telangana and Kurnool District, Andhra Pradesh, is the second largest capacity hydroelectric plant in the country.

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Guidelines for Selecting and Accommodating Inflow Design Floods for Dams was published in July 2017 and is one of a series of dam safety guidelines being developed under the Dam Rehabilitation and Improvement Project (DRIP).

Disclaimer

Selecting and accommodating the Inflow Design Flood (IDF) for a dam is a key decision in ensuring its hydrological safety. Whilst these guidelines help in taking appropriate assumptions and actions, the Central Dam Safety Organization or the Central Water Commission cannot be held responsible for the efficacy of the IDF adopted based on these guidelines. Appropriate discretion may be exercised while selecting and accommodating the IDF.

Readers of this document are cautioned to use sound engineering judgment when applying the guidelines herein. This publication is intended solely for use by professional personnel who are competent to evaluate the significance and limitations of the information provided herein, and who will accept total responsibility for the application of this information. Anyone making use of this information assumes all liability from such use.

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MESSAGE

In the series of various Guidelines and Manuals, published by Central Water Commission under the World Bank assisted Dam Rehabilitation and Improvement Project (DRIP), this guideline titled "***Guidelines for Selecting and Accommodating Inflow Design Floods for Dams***" is a new addition. There are existing BIS codes for arriving at IDF during the planning stage of new dam projects. In case of an existing dam, there are potential limitations to accommodate the revised design flood, if increase is significant. This Guideline also addresses the challenges of the accommodation of an increased IDF in a given dam by through structural and non-structural measures. This Guideline is not intended to review the methodology or prevailing practices for IDF estimation, or any replacement of current methodologies or guidelines of BIS/CWC/Indian Meteorological Department. Rather, it is a step forward adding new dimension to the existing practices and makes it more scientific.

This publication acknowledges the vast improvement in data availability for precipitation, terrain, land use and land cover, census data etc. This data is one of the important input to facilitate detailed consequence assessment and risk analysis in selecting an IDF. Further, risk due to climate change for hydrological safety has been considered by using most critical design storm, significant change in land use and land cover leading to reduction in infiltration and time of concentration with associated increase in flood peak, including rapid development of habitations downstream of existing dams have been factored.

All above described elements emphasize the importance of the framework described in this Guideline. This is one step forward from hydrologic perspective to switch over for a risk-informed dam safety management program in the country. In a risk-informed hydrologic hazard assessment, it is crucial to evaluate a full range (frequency of occurrence) of hydrologic loading conditions and possible dam failure mechanisms and not only a single "design flood" scenario. The traditional approach uses a single upper bound SPF or PMF (as the case may be). The Guidelines are very descriptive and include detailed examples.

I hope, these guidelines will help our dam owners to manage their existing water infrastructure more efficiently by prioritizing available limited financial resources. The guideline is applicable for new as well as existing dams. The dams having higher consequences could be assigned higher priority, accordingly prioritize financial resources to effectively implement hydrologic safety measures. I firmly believe, this document will prove useful to Indian dam authorities in coming time.

(S K Haldar)
Chairman
Central Water Commission

New Delhi
June 2021

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FOREWORD

Amongst the various dam safety concerns, evaluation and addressing hydrological safety in a new or existing dam is one of the most important safety concerns. In India, there are well established methodologies for hydrological assessment in terms of determining an Inflow Design Flood(IDF), routing of the flood through reservoirs and downstream channels, evaluation of various feasible options to safely pass the extreme floods etc. This Guidelines titled "***Guidelines for Selecting and Accommodating Inflow Design Floods for Dams***" proposes a risk based tiered framework for selection of IDF, which is in line with the need of various dam owners to overcome the existing limitations in the existing procedures. This document intends to infuse global best practices in the area of hydrological assessment. This approach may prove valuable in selecting an IDF which is more in alignment with the level of risk generally accepted by society and ensure the efficient use of available economic resources.

The constantly refined techniques and the ever more efficient applied technologies doubtlessly allow significant quality progress in the hydrological field. The automation of data collection, the improvement of the dependability of data transmission, the electronic processing of interminable series of numbers, considerably lighten the tedious work of the hydrologist and speed up the quantitative analyses. They also considerably contribute to raising the overall quality of the basis material – the long lists of numbers and statistics expressing the reality of water flow in a river, for instance – on which hydrological models are based.

The document presented herein, aim to support dam officials and experts throughout the process of reasoning, interpretation and sound engineering judgment required in the selection and accommodation of the IDF for their respective dams, which will also support in taking further decisions related to dam safety management.

Finally, I compliment all the individuals and organisations involved in the preparation of this Guideline. This Guidelines under DRIP, is a forward-looking step to ensure vibrant dam safety management in India at par with global practices. I also acknowledge the efforts made by members of Review Committee as well as CPMU experts in finalising this document.

(Dr R K Gupta)
Member (D&R)
Central Water Commission

New Delhi
June 2021

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PREFACE

Currently the world is experiencing continuous changes in a fast pace, and the Hydrology, like other sciences is not the exception to this rule. Hydrology applied to dam safety has evolved over the period of time, but at an amazingly faster pace, and despite the fundamental principles and the main conceptual axes are widely accepted, this science is still subject to local variations and interpretations not only between different countries, but also within the borders of the same nation.

Consequently, the purpose of these "*Guidelines for Selecting and Accommodating the Inflow Design Flood for dams"* is to describe an overarching approach based on the latest and internationally accepted practices for selecting an inflow design flood (IDF), without discussing the fundamental principles of Hydrology or IDF estimation methods. The proposed framework is comprised by a tiered approach, which makes it flexible and scalable to implement at any portfolio level, and with an IDF, which is technically defensible, as the main outcome.

The Tier-I for low hazard dams and new dams, correspond to the basic level and make use of the consequences-based hazard classification as prescriptive selection method. Subsequent Tier-II approach, which uses the incremental consequences analysis, is meant for significant hazard dams by discerning the iterative process needed for an optimal IDF. Finally, a Tier-III analysis is proposed for high hazard dams where a risk-informed hydrologic hazard analysis is necessary in order to make the dam hydrologically safe under the tolerable risk levels. The associated risks should be estimated, managed, and minimised.

Furthermore, the wide variety of dams and watersheds in India require a variety of approaches to accommodate the selected IDFs that can achieve a reasonable balance of public protection, efficiency of evaluation, and efficiency of project operation. The presented Guidelines aim to achieve this important objective, through all its chapters.

Guidelines Review Committee

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List of Abbreviations

The following acronyms are used in this publication:	
AEP	Annual Exceedance Probability
AFP	Annualised Failure Probability
ALL	Annualised Loss of Life
BIS	Bureau of Indian Standards
CDSO	Central Dam Safety Organisation
CWC	Central Water Commission
DAD	Depth-Area-Duration
DEM	Digital Elevation Model
FRL	Full Reservoir Level
HHA	Hydrologic Hazard Analysis
HHC	Hydrologic Hazard Curves
IDF	Inflow Design Flood
IMD	India Meteorological Department
MWL	Maximum Water Level
O&M	Operation and Maintenance
PFM	Potential Failure Modes
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SPF	Standard Project Flood

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Chapter 1. THE HYDROLOGIC SAFETY OF DAMS

1.1 Introduction

A well-designed, constructed, and operated dam can reduce the risk of flooding in downstream areas by temporarily impounding flood waters and attenuating observed peak flood flows in vulnerable low lying areas, even if the dam is not specifically designed for flood mitigation.

However, impounding the water behind a dam also creates a risk to the downstream areas because of the potential for an uncontrolled release of the reservoir pool caused by dam failure which could result in a peak flow discharge that exceeds the greatest flow of any possible natural flood. There are several potential causes of dam failure, including hydrologic, hydraulic, geologic, seismic, structural, mechanical, and operational.

This guideline considers the selection of the inflow design flood (IDF) for the hydrologic design of a dam to reduce risks to the public.

One of the most common causes of dam failures is the inability to pass flood flows safely. Failures resulting from hydrologic conditions can range from sudden failure, with complete breaching or collapse of the dam, to gradual failure, with progressive erosion and partial breaching. The most common potential failure modes associated with hydrologic conditions include overtopping erosion, erosion of spillways, internal erosion (seepage and piping) at high reservoir levels and overstressing the structural components of the dam.

The IDF is the flood used to design and/or modify a specific dam and its appurtenant structures or works, particularly for sizing the spillways and outlet works, and for determining surcharge storage and the required dam height. It is the flood used for the design of a safe structure.

Selecting an IDF for the hydrologic safety design of a dam requires balancing the likelihood of failure by overtopping against the consequences of dam failure. Consequences of failure include the loss of life and social, environmental, and economic impacts. The inability to accurately define flood probabilities for rare events, and to accurately assess the potential loss of life and economic impact of failure when it would occur, dictate the use of procedures that offer some latitude to meet site-specific conditions in selecting the IDF.

The primary goal of the Central Dam Safety Organisation (CDSO) of the Central Water Commission (CWC) is to encourage and facilitate dam safety practices that will help ensure operation of dams to their full capacities and intended purposes, and also to reduce the risk to lives and property from the consequences of both structural and operational dam incidents and failures. Although most dam owners have a high level of confidence in the structures they own and are confident their dams will not fail, history has shown that on occasion dams do fail and that often these failures cause extensive damage to property, and sometimes loss of life. Dam owners are responsible for keeping these threats to acceptable levels.

Existing guidelines for evaluating the hydrologic safety of dams were written a couple of decades ago or earlier. However, significant technological and analytical advances have since led to better watershed and rainfall information, improvements in the analysis of extreme floods, greater sophistication in means to quantify incremental dam failure consequences, and tools for evaluating hydrologic events in a risk-based context. Lead agencies and professionals in the nation's dam safety community recognize the need for updated guidelines for evaluating the hydrologic safety of dams and, in particular, for selecting an appropriate Inflow Design Flood (IDF).

The IDF is the flood hydrograph entering a reservoir that is used to design and/or modify a specific dam and its appurtenant works, particularly for sizing the spillways and outlet works, and for evaluating maximum storage, the height of the dam, and freeboard requirements.

Cognizant of the requirement of the day, in September 2012 the Central Water Commission initiated the development of this document as part of the project titled: Dam Rehabilitation and Improvement Project (DRIP). The objective of this effort was to develop and publish a guidance document for the assessment of the hydrologic safety of dams, including guidelines for selecting the IDF for new and existing dams that could be applied nationwide.

1.2 Dam Safety Management Program and the Inflow Design Flood

The approach of selecting adequate inflow design floods (IDFs) should focus in maximizing the benefits from the storage project while ensuring the safety of dam and its appurtenant works through optimization of sizes of spillways and other outlet works. However, for verifying the existing or implementing designing acceptable hydrologic safety for dams a more comprehensive assessment is required, which needs to be embedded in the Dam safety Management Program in the country (Figure 1.1)

For instance, from a hydrologic perspective, a Risk-informed Dam Safety Management Program, requires an evaluation of a full range (frequency of occurrence) of hydrologic loading conditions and possible dam failure mechanisms and not only a single "design flood" scenario. This risk approach contrasts with the traditional approach in India of using a single upper bound (SPF, PMF). In the context of probabilistic hydrologic loadings, a deterministic maximum event such as the PMF is just one flood outcome amongst a collection of flood peaks, volumes and hydrograph shapes. Typically, different flood durations are tested with the risk model through a sensitivity analysis to check the

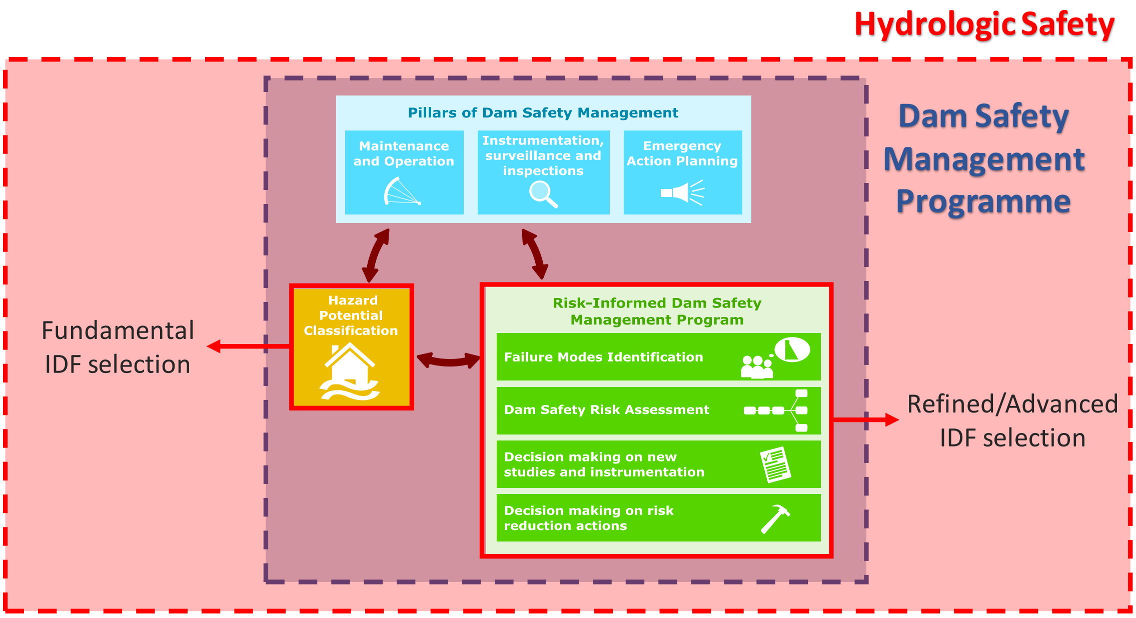

Figure 1.1.- Relationship between the Dam Safety Management Program and the Inflow Design Flood Selection

most critical one for the reservoir.

In addition to the selection of a design flood, the hydrologic design of a new reservoir or the evaluation of an existing project involves consideration of observed performance capabilities and whether improvements are necessary to ensure safety. The reservoir regulation plans, water control management plan, and data information systems should be periodically reviewed for safely deficiencies and potential for mis-operation during both severe flood events and normal conditions. Necessary corrections should be made as soon as practicable.

1.3 Purpose and Scope of Guidelines

Considering the expected differences in dam safety standards, this document is intended to provide an overarching framework for the selection and accommodation of IDFs according to the different goals each dam is expected to serve. The guidelines herein are neither intended to be a mandate for uniformity nor to provide a complete manual of all procedures available for estimating or accommodating IDFs. The basic philosophy and principles are described in enough detail to promote a reasonable degree of consistency and uniformity among state and central agencies in the design and evaluation of dams from the standpoint of hydrologic safety.

Over the past few decades, prescriptive hydrologic guidelines have been accepted and used by both state and central dam safety agencies. While this guidance document provides for such an approach, it also acknowledges the vast improvement in available precipitation, terrain, land use and census data, and in analysis tools that facilitate detailed consequence assessment and risk analysis in selecting an IDF.

This document is not intended to either promote or discourage the use of newer methods, such as incremental consequence analysis or risk assessment. It does, however, recognize that the cost of these advanced approaches may prove valuable in selecting an IDF which is more in alignment with the level of risk generally accepted by society and ensure the efficient use of available economic resources.

The main goals of these guidelines are to recommend appropriate procedures for selecting and accommodating IDFs based on current and accepted practices and promote common and/or compatible approaches among state and central agencies. Selection of the IDF is the first step in evaluating and designing a dam to address hydrologic potential failure modes and reduce risks to the public.

1.4 How to use this Guideline

The following basic sections of this guideline help to understand not only the basic principles of selecting and accommodating the inflow design flood but also a proposed approach for India:

- ***Chapter 2*** describe de current methodology followed in India to estimate/compute the IDF (PMF and SPF) in large dams by the Hydro-meteorological approach.
- ***Chapter 3*** gives an overview and notional background of the different approaches available in the process of selecting the inflow design flood for dams.
- ***Chapter 4*** presents a comparison of current international practices in the selection of the inflow design flood. This chapter lays the base to propose a suitable approach for India.
- ***Chapter 5*** discusses in detail the tiered proposed framework for a preliminary selection of the Inflow Design Flood in India based on the hazard potential classification, and in a more advanced stage (and if required) using a risk-informed hydrologic assessment. Framework is applicable either for existing dams or dams under project stage.

- ***Chap*ter 6** describe the general recommendations for accommodating the selected IDF in new dams. Also, discusses the challenge of accommodating an increased inflow design flood in existing dams, as well as proposes a general framework to guide dam owners in the hydrologic safety evaluation making use of other guidelines of the same series.

1.5 Relationship with other Guidelines and Policies

This guideline provides technical advice and guidance on the selection and accommodation of the inflow design flood for dams in India, and the same should be read in conjunction with:

- *Bureau of Indian Standards (BIS) in IS: 11223-1985 "Guidelines for fixing spillway capacity."* Current guidelines for selecting design floods for dams in India are given by this document. In these guidelines, dams are classified by size using the hydraulic head and the gross water storage capacity of the reservoir.

- *Guidelines for Classifying the Hazard Potential of Dams* describes in detail the proposed and new approach for Dam Hazard Classification in India. In contrast to the approach of the BIS IS: 11223-1985 this guidelines classify the dams based on the assessment of the potential consequence-es of a failure scenario and presents a justification on why the method of using solely the dam characteristics for the hazard classification should be discontinued.

- *Dam Safety Bill, 2019.* This bill, already ap-proved by the Parliament's lower house (Lock Sabha), mandates that dam's owner of a specified dam shall make or cause to be made comprehensive dam safety evaluation, which shall consists of, among others, general assessment of hydrologic and hydraulic conditions with mandatory review of design floods as specified by the regulations.

- *Guidelines for Assessing and Managing Risks Associated with Dams.* This document provides a global framework where all aspects related with dam safety are integrated to improve decision making. One of the several aspects this document influences to is how hydrological studies are used to analyse overtopping and other hydrologic failure modes, which can be used to evaluate the risk among established tolerability guidelines. On the other hand, risk assessment can also evaluate the effect of hydrological data uncertainty on dam safety.

- *Manual on Assessing Hydraulic Safety of Dams.* It introduces a methodology that allows the engineers involved in the Dam Safety Evaluation process to assess, from a hydraulic perspective, the safety of the dam, identifying vulnerabilities and associated failure modes. Also, describes in detail potential alternatives of rehabilitation measures that reduce or eliminate the detected hydraulic / hydrologic vulnerabilities in any component and develop enough information and knowledge to initiate a specific design for the rehabilitation work selected option.

Figure 1.2 illustrates the relationship between the documents and regulatory framework mentioned above

1.6 Publication and Contact Information

This document along with the template EAP or dams is available on the CWC website

- http://www.cwc.gov.in

and the Dam Rehabilitation and Improvement Project (DRIP) website

- http://www.damsafety.in

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1.7 Acknowledgments

In preparing these guidelines, material has been drawn liberally from the work of others from India, the United States, Australia and elsewhere. Grateful appreciation is extended to the following organisations and persons whose publications and websites provided valuable information regarding hydrologic safety:

- Federal Emergency Management Agency, U.S. Department of Home-land Security
- Federal Energy Regulatory Commission (FERC)
- S. Samuel Lin, Civil Engineer, FERC
- Bureau of Reclamation, U.S. Department of the Interior
- U.S. Army Corps of Engineers
- Washington State U.S., Department of Ecology
- Engineers Australia (EA)

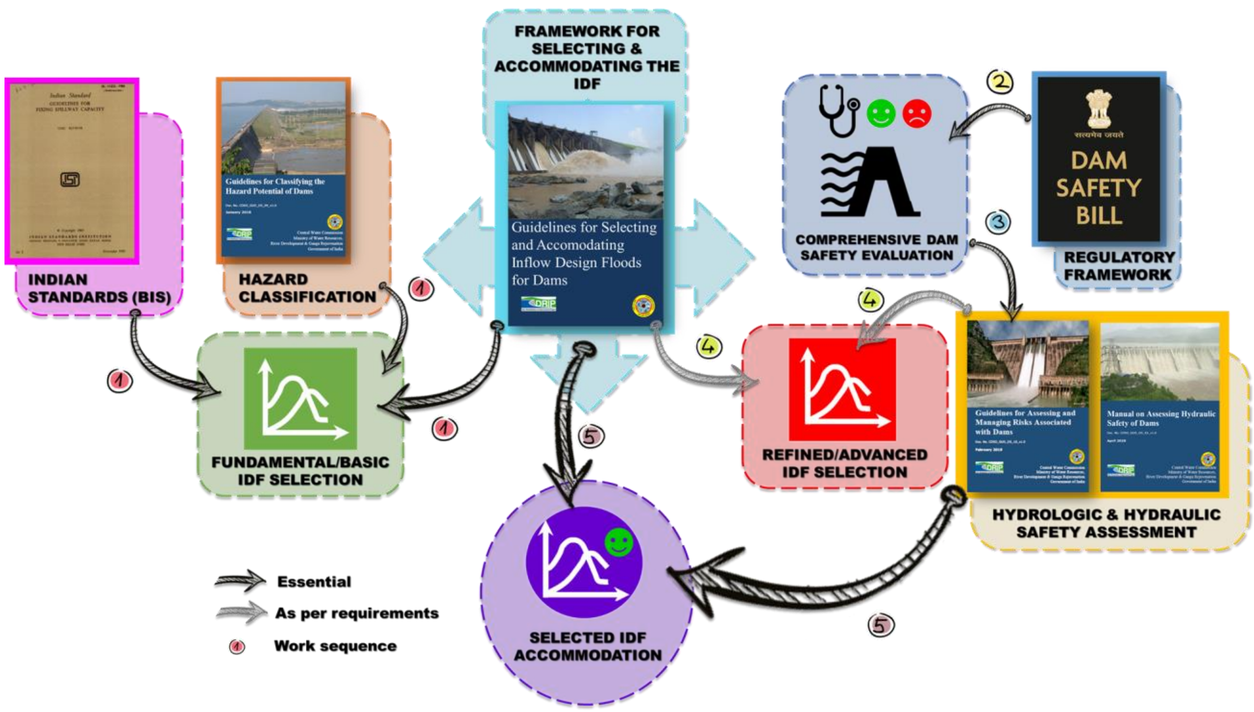

Figure 1.2.- Relationship to other Guidelines and Policies in the Country

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Chapter 2. Hydro-Meteorological Approach for Design Flood Estimation

2.1 Hydro-Meteorological Approach

In the hydro-meteorological method, attempt is made to analyse the causative factors responsible for production of severe floods. Even though many of the components elude precise physical definition, the method is found to be very convenient and sufficiently accurate for practical purposes. The design flood computation mainly involves estimation of a design storm hyetograph, and derivation of catchment response function. The various steps involved in the method are indicated by flow chart given (Figure 2.1). The catchment response function used can be either a lumped system model or a distributed lumped system model. In the former, a unit hydrograph is assumed to represent the entire catchment area. In the distributed model, the catchment is divided into smaller sub-regions, and the unit hydrographs of each sub-regions applied with channel and/or reservoir routing will define the catchment response. The main advantage of the hydro-meteorological approach is that it gives a complete flood hydrograph, and this allows making a realistic determination of its moderating effect while passing through a reservoir or a river reach. This approach however is subjected to certain limitations such as

(i) Requirement of long-term hydro-meteorological data for estimation of design storm parameters

(ii) The knowledge of rainfall process as available today has severe limitations and therefore, physical modelling of rainfall to compute PMP is still not attempted.

(iii) Maximisation of historical storms for possible maximum favourable conditions is presently done on the basis of surface dew point data. Surface dew point data may not strictly represent moisture availability in the upper atmosphere.

(iv) Availability of SRRG data for historical storms is too poor.

(v) Many of the assumptions in the UG theory are not satisfied in practice.

(vi) Many times, data of good quality and adequate quantity is not available for derivation of UG.

2.2 General

The design storm for a project or at a location in the river basin can be defined as an estimate of the amount of rainfall and its temporal distribution over the catchment under considerations used in determining the design flood. This could be a Probable Maximum Storm (PMS), Standard Project Storm (SPS) or a storm of a specified return period, which are respectively used in deriving the Probable Maximum Flood (PMF), Standard Project Flood (SPF) or a return period flood.

The Probable Maximum Storm is an estimate of the physical upper limit to storm rainfall over the catchment and is obtained by studying all the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions. The Standard Project Storm is the one, which is reasonably capable of occurring over the basin under consideration, and is generally heaviest rainstorm, which has occurred in the region of the basin during the period of rainfall records. It is not maximized for the most critical atmospheric conditions, but it may be transposed from an adjacent region to the catchment under consideration.

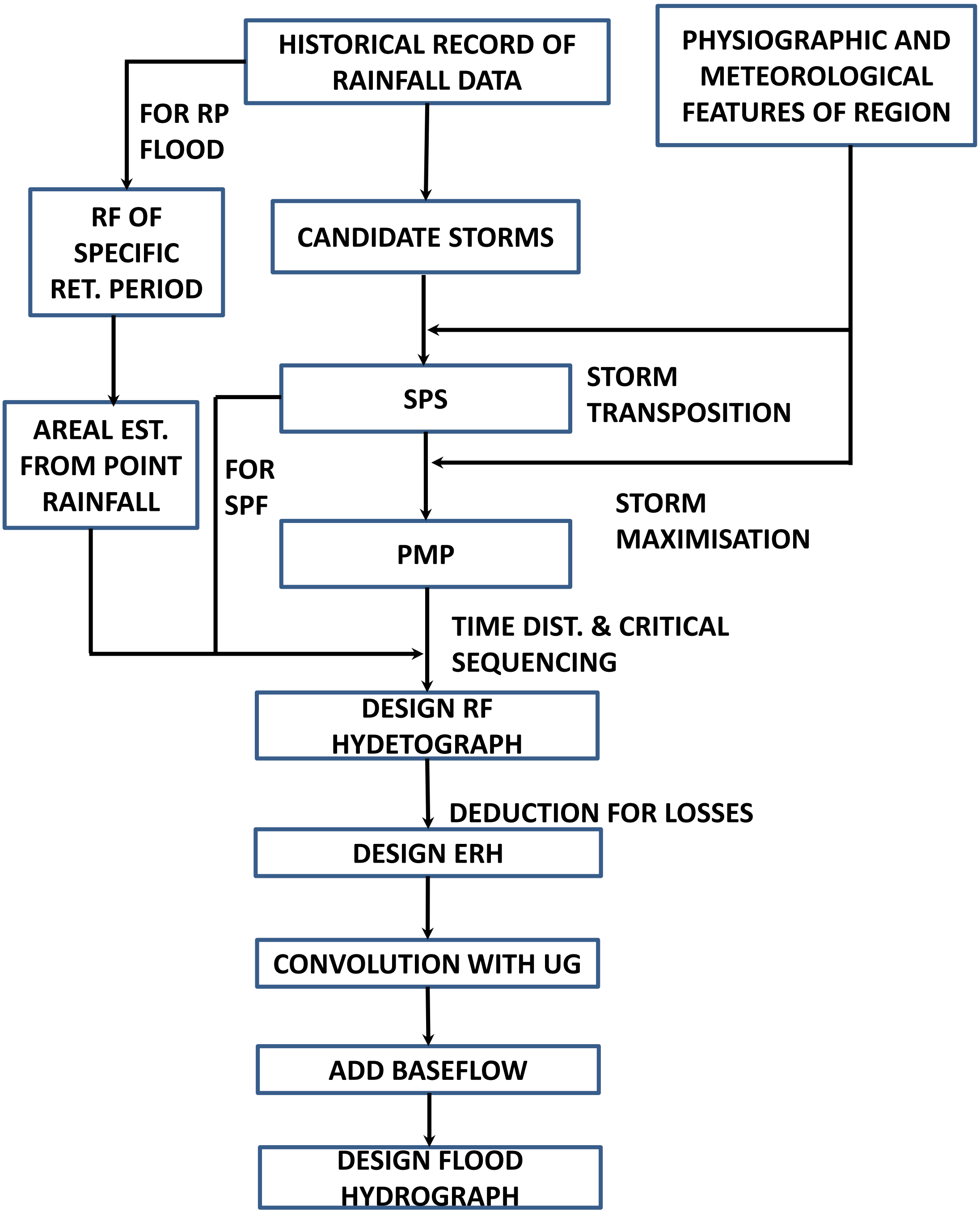


Figure 2.1.-Hydro-meteorological Approach for Design Flood Estimation

2.3 Causes of Heavy Rainfall in India

The weather of India is affected by tropical as well as extra-tropical disturbances, because the country stretches from the equatorial to the northern temperate latitudes. While the extra-tropical disturbances normally affect the tropical parts of the country during the period from December to April, the influence of the tropical disturbances is felt all over the country including the northernmost parts, during the period from June to November.

The extra tropical disturbances, otherwise known as *'Western disturbances'* originate near the Mediterranean Sea and travel eastwards; their tracks being north of latitude 45° N but shifting to southern latitudes near 30° N during the winter period. They generally cause light rain of the order of 25 to 50 mm per month (some place 75 to 100 mm) over mountains. Most of the precipitation in the Himalayas is in the form of snow.

The monsoons are the seasonal trade winds, which originate in the southern hemisphere and move across the equator northward to the Tropic of Cancer. This wind stream influences Pakistan, India, Myanmar, Singapore, Nepal and Thailand lying in this belt India comes under their influence from June to October. The Western Ghats along the West Coast of India are the first to intercept these monsoons in the last week of May or beginning of June. The monsoon then divides itself into two branches: the Arabian Sea and the Bay of Bengal branch. The Arabian Sea branch moves as a weak current along the Western Ghats northwards towards the Gujarat coast The Bay of Bengal branch continues to move over the Indian peninsula and the Bay in Southwest direction. The latter strikes the north-eastern spurs of the Himalayas viz.; the Naga and Patkoi Bum ranges including the Garo, the Khasi and the Jaintia hills of Meghalaya. It is then turned eastwardly along the Himalayas. Both the Western Ghats and the northeastern hills accordingly receive heavy rainfall owing to orographic lifting. The heaviest rainfall in rest of the country is the result of westward or northwestward movements of depressions that form in the Bay of Bengal. When the depressions form and move across India, heavy rain commences over the coastal areas and the rain belt shifts westwards along with the depression.

Generally the Southwest quadrants of these depressions are the zones of heaviest precipitation. However, when a depression recurves, the zone of depression shifts to right forward sector. Nearly eighty percent of the precipitation over the Indian mainland is associated with these depressions. The frequency of these depressions is 1 to 2 in June and 2 to 4 per month from July to September. A few depressions form during October and these tend to cause the heavy floods.

The mechanism for greater volume of precipitation in northern regions is conditioned by two meteorological factors when there is a temporary withdrawal of the monsoon. Firstly, the non-tropical continental but slightly cooler air mass moves over the sea and develops instability. Secondly, the zonal westerlies come down to slightly lower latitudes and influence movement of the depressions, which recurves and move in a north-easterly direction and cause very heavy precipitation in the catchment areas of the Himalayas. One or two depressions also form per year in the Arabian sea, which generally move northwards and strike the Gujarat – Saurashtra Coast.

The big rivers of Central India become swollen due to rain associated with monsoon depressions. The convergence between the Bay of Bengal branch and Arabian Sea branch of the monsoon sometimes becomes significant and causes heavy precipitation in the region of convergence. The Tapi, Nannada and Mahanadi rivers flood owing to this effect. The floods in these rivers pass to the ocean within seven to eight days, for a length of stream of about 1200 kms. The peninsular rivers such as the Godavari and the Krishna are in spate towards the end of September or October when the monsoon depressions move in a more southerly direction, covering their catchment area. The monsoon withdraws hereafter and turns north-easterly and causes heavy rainfall in the extreme southern peninsula including Tamil Nadu as Northeast monsoon in the months of December-January. From January to March, the weather over most parts of India becomes dry except in the north where western disturbances have an effect

2.4 Design Storm Duration

Important parameters for deciding Critical Storm Duration are size and shape of the catchment, travel time/base period of Unit Hydrograph (UH} and the direction the storm movement with reference to the direction of river flow. The following criteria are recommended for determining the storm duration. Theoretically, the larger the duration of the storm, better would be the estimated the flood hydrograph. However, for the purpose *of* estimation of the flood peak it would not be advantageous to go for a considerably large duration. Rather it would increase scanning of voluminous rainfall data as also analytical work in design flood synthesis.

For all practical purposes, the U.G. Base governs the duration of the storm depth. For unit hydrograph having a base of 24 hours or less, the design storm *of* one day is considered appropriate and sufficient In case the U.G. Base is more than 24 hours but less than 48 hours, 2-day design storm should be considered. For basins having U.G. Base of more than 48 hours. A design storm duration of 3- day duration should in general be adopted. Storms of periods exceeding 72 hours are not generally required to be considered in the design flood estimation.

2.5 Estimation of PMP by Physical Approach

2.5.1 Details about PMP

Introduction

It has been recognized that there is a physical upper limit to the amount of precipitation that can fall over a specified area in a given time. This precipitation, associated with the physical upper limit, is known as the Probable Maximum Precipitation (PMP). The dams which require a high degree of safety are needed to be designed to pass the flood resulting from the upper limit to precipitation. WMO (2009) defined PMP as the depth of precipitation for a given duration that is meteorologically possible for a design watershed or a given size storm area at a particular location at a certain time of year. Such is the conceptual definition of PMP. The values derived as PMP under this definition with no allowance made for long-term climatic trends are subject to change as knowledge of the physics of atmospheric processes increases. They are also subject to change with long-term climatic variations.

Probable maximum precipitation (PMP) was once known as Maximum Possible Precipitation (MPP), and this latter term is found in most reports on estimates of extreme precipitation made prior to about 1950. The main reason for the name change to PMP was that MPP carried a stronger implication of physical upper limit of precipitation than does PMP, which is preferred because of the uncertainty surrounding any estimate of maximum precipitation. Procedures for estimating PMP, whether physical or statistical, are admittedly inexact, and the results are approximations.

Theoretical Background of Physical Method

There are two main approaches to estimate the PMP. The first is the physical or meteorological approach, which involves the identification of the maximum rainfall produced by severe rainstorms over the specific catchment and from other neighbouring area. Having obtained a suitable severe storm database, the process of PMP estimation involves working out Depth Area Duration (DAD), Depth Duration (DD), envelope/transposed depth, their temporal and areal distribution, and moisture adjustment parameters. The second is the statistical approach where the estimates of PMP at a particular location or point are determined from the frequency analysis of annual maximum rainfall data. This method is useful when the meteorological data for moisture maximization are not available but where there is a large amount of rainfall data.

The main assumption in the physical method is that the PMP will result from a rainstorm in which there is the optimum combination of the available moisture in the atmosphere and the efficiency of the rainstorm mechanism. Factors, which influence the rainstorm efficiency, include horizontal mass convergence, topography, induced lifting, vertical motion and the rate of condensation.

Efficiency is the ability of a rainstorm to convert moisture into precipitation. The extreme observed rainfall values are then maximized by Moisture Maximization Factor (MMF) to estimate PMP. When the record of rainstorm rainfall over relatively plain area catchment is not adequate, severe rainstorms are transposed from other areas, which are meteorologically homogeneous.

Methods for Estimating PMP

As per WMO (2009), there are six methods of PMP estimation currently being used:

(a) The local method (local storm maximization or local model);

(b) The transposition method (storm transposition or transposition model);

(c) The combination method (temporal and spatial maximization of storm or storm combination or combination model);

(d) The inferential method (theoretical model or ratiocination model);

(e) The generalized method (generalized estimation);

(f) The statistical method (statistical estimation).

Most of these methods can be used in medium- or low-latitude areas. In the current study, primarily two methods; 1) Generalized estimation and 2) Statistical estimation have been used. The sections below give the details of the generalized method. The other associated details (storm moisture maximization, rainstorm transposition adjustments, persisting dew point temperature and maximum persisting dew temperature) with generalized method are also included in the subsequent sections.

PMP by Generalized Method

The generalized method is used to estimate PMP for a large, meteorologically homogeneous region. The procedure involves compilation and use of observed storm rainfalls for various major events over the catchment or region. It also includes adjustments for moisture availability and topographic effects. The storm rain depths are enveloped by smoothing over a range of areas and durations. This method requires a large amount of long-term data obtained by rainfall self-recorders in the study area. This is a time-consuming and expensive process. However, the method can lead to high accuracy and easy application of PMP results. This method is applicable to watersheds under 13,000 sq. km in orographic regions and 52,000 sq. km in non-orographic regions, and rainfall durations up to 72 hours or less (WMO 2009).

Approach followed in the current study is given in Figure 2.2. The procedure starts with the acquisition of daily rainfall for various rain gauge stations in and around basin/region under consideration. The rainfall data is processed to develop the relational database to facilitate queries, look ups and joining with the other data bases. The comprehensive search for medium and major storms is conducted based on the defined threshold. This gives the initial list of the storms. The storm list is finalized based on initial spatial overview, review of historical records and other factors. All the listed storms are subjected to rainstorm analysis, which includes isohyets generation and Depth-Area-Duration (DAD) analysis. In the next step, the dew point temperature data for listed storms is acquired and the storm wise persisting dew point temperatures are estimated. Using the persisting storm dew point, maximum persisting dew point, barrier elevations, a Moisture Maximization Factors (MMF) are estimated all the listed storms.

This study provides the PMP estimates for 1) various catchments in the basin and 2) grid point placed at regular intervals. These estimates are provided for the range of areas in the catchment and at the grid points for 1day, 2day and 3day durations.

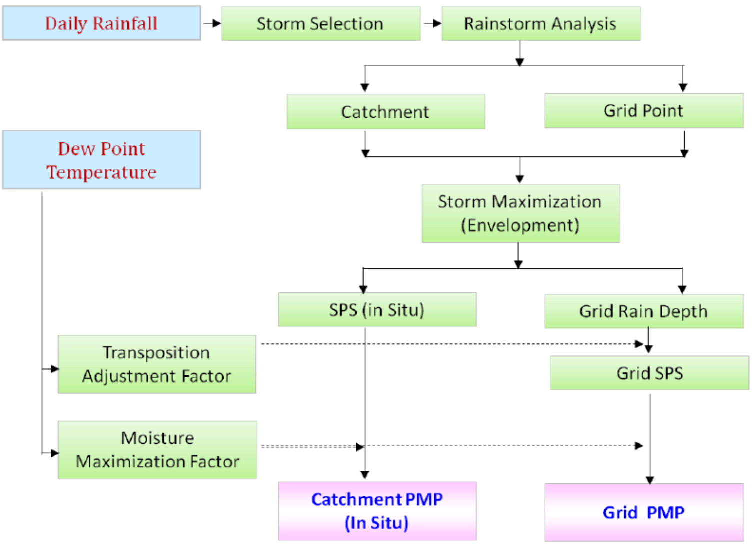


Figure 2.2.- PMP by generalised method

In case of PMP estimation for various catchments pertaining to non-orographic areas, the in-situ rainstorm analysis using DAD approach and moisture maximization is applied. The rainstorm maximization includes the estimation of envelope rain depth for range of areas from all listed storms in and around the catchment. These envelope rain depths are considered as in-situ Standard Project Storm (SPS) values. The SPS values are then multiplied by the MMF of storm contributing to envelope curve for the respective area and duration to the get the PMP estimate. In case of the orographic areas, the rainstorm analysis is done using Depth-Duration (DD) analysis instead of DAD analysis before the moisture maximization.

In case of grid point PMP estimation, a grid system is constructed at 1-Degree resolution of latitudes and longitudes for non-orographic areas. In case of orographic areas, the grid system of 0.25-0.50 degrees is considered. The rainstorm maximization is done by using the DAD analysis to estimate the enveloping rain depths at each grid point for range of the areas and durations (1-3 days). Since, a rainstorm transposition is assumed in this case, the SPS values are estimated by multiplying the grid rain depths with the Transposition Adjustment Factors, TAF, (which considered the impact of location adjustment and barrier adjustments). Finally, the PMP estimates at each grid point are derived by multiplication of grid SPS and MMF.

Estimation of Atmospheric Moisture Persisting Dew Points

Since many of the extreme, or major, recorded storms occurred before extensive networks of upper air temperature and humidity soundings had been established, any index of atmospheric moisture must be obtainable from surface observations. Even today, current upper-air observational networks are too sparse to adequately define the moisture inflow into many storms, especially those limited to areas of the size considered in this manual. Fortunately, the moisture in the lower layers of the atmosphere is the most important for producing precipitation, both because most atmospheric moisture is in the lower layers and because it is distributed upward through the storm early in the rainfall process (Schwarz, 1967; United States Weather

Bureau, 1960). Theoretical computations show that, in the case of extreme rains, ascensional rates in the storm must be so great that air originally near the surface has reached the top of the layer from which precipitation is falling within an hour or so. In the case of severe thunderstorm rainfall, surface air may reach the top in a matter of minutes. The most realistic assumption seems to be that the air ascends dry-adiabatically to the saturation level and thence moist-adiabatically. For a given surface dew point, the lower the level at which the air reaches saturation, the more moisture a column of air will contain. The greatest perceptible moisture occurs when this level is at the ground. For these reasons, hydro-meteorologists generally postulate a saturated pseudo-adiabatic atmosphere for extreme storms.

Moisture maximization of a storm requires identification of two saturation adiabats. One typifies the vertical temperature distribution that occurred in the storm to be maximized. The other is the warmest saturation adiabat to be expected at the same time of year and place as the storm. It is necessary to identify these two saturation adiabats with an indicator. The conventional label in meteorology for saturation adiabats is the wet bulb potential temperature, which corresponds to the dew point at 1000 hPa. Tests have shown that storm and extreme values of precipitable water may be approximated by estimates based on surface dew points, when saturation and pseudo-adiabatic conditions are assumed (Miller, 1963; United States Weather Bureau, 1960).

In order to obtain the storm moisture, dew points in respect of stations located in the warm air flowing into the storm are identified from the surface weather map. While selecting the stations, care should be taken such that the storm centre should invariably fall close to the stations. In addition, dew points between the rain area and moisture source have to be given prime consideration. Dew points in the rain area may be too high because of precipitation, but they need not be discarded if they appear to agree with dew points outside the rain area. Of course, one station is inside the heavy rainfall area and the other three in the path of moisture inflow. The storm dew point is then determined by averaging these four stations. Before averaging, the dew point values are reduced pseudo-adiabatically to the 1,000 hPa level, so that dew points for stations at different elevations are comparable. The amount of moisture in the air can be obtained from the single observation of dew point temperature, but these have certain synoptic limitations and are also susceptible to observational error. The moisture itself must be such that it persists for a period of several hours rather

Table 2.1.- Dew Point Temperature at 24°C

Details	**Day 1**		**Day 2**		**Day 3**		**Day 4**	
Time (IST)	0830	1730	0830	1730	0830	1730	0830	1730
Dew point (°C)	22	22	23	24	26	24	20	21
Persisting dew point (°C) at 0830 hrs			22		23		20	
Maximum Persisting dew point (°C) at 0830 hrs	23							
Persisting dew point (°C) at 1730 hrs			22		24		21	
Maximum Persisting dew point (°C) at 1730 hrs	24							
Maximum 24 hr Persisting dew point (°C)	24							

than minutes. Thus, the dew point values used to estimate moisture during the storm as well as over the specific area is based on three or more consecutive dew point values over a reasonable time interval. The value of the dew point so obtained is called the persisting dew point. Various meteorological departments worldwide publish the daily dew point data for meteorological stations.

The daily dew point data during a storm period can be collected from such publications. The general practice is to use the 12-hour or 24-hour persisting dew point. The maximum persisting 24-hour dew point can be obtained from the series of dew point temperatures on days as shown in Figure 2.2.

In determining the persisting dew point, the consecutive dew points during a 24-hour period are examined for their reliability and the lowest of these values is selected. The highest persisting 24-hour dew point for the above series is 24°C.

Storm Moisture Maximization

The PMP for different durations over an area is derived by maximizing the highest rainfalls obtained from major historical rainstorms that have occurred over the area under study. This maximization consists of simply multiplying the highest rainfall values by the moisture maximization factor (MMF). The MMF for a rainstorm in a place is the ratio of precipitable water corresponding to the maximum persisting dew point temperature on record at the original location of the rainstorm in the same fortnight of the month in which the rainstorm occurred to the precipitable water corresponding to the maximum persisting dew point temperature of rainstorm. The objective of maximization is to determine the physical upper limit of rainfall, which would result if the moisture available to the storm were at maximum. Obviously, the most important factor in the moisture maximization is the estimation of moisture or precipitable water available in the atmosphere. U.S. Weather Bureau (USWB) (1960), now National Weather Service (NWS) and Reitan (1963) showed that the precipitable water or moisture in the air mass from which large precipitation occurs can be estimated from the surface dew point temperatures when saturation and pseudo-adiabatic conditions are assumed. The MMF, therefore, is determined based on 12-hour or 24-hour persisting storm dew point temperature and the maximum ever-recorded persisting dew point temperature for the area under study. The meaning and the method of determining the 12-hour or 24-hour persisting dew point temperature have been discussed in another section. Both storm dew point temperature and dew point

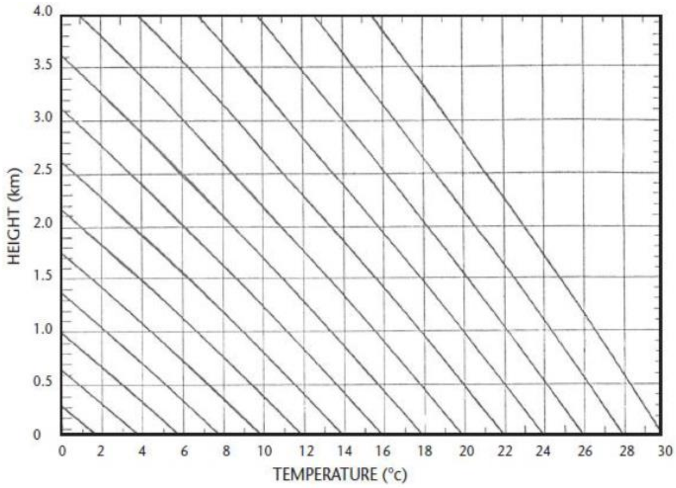
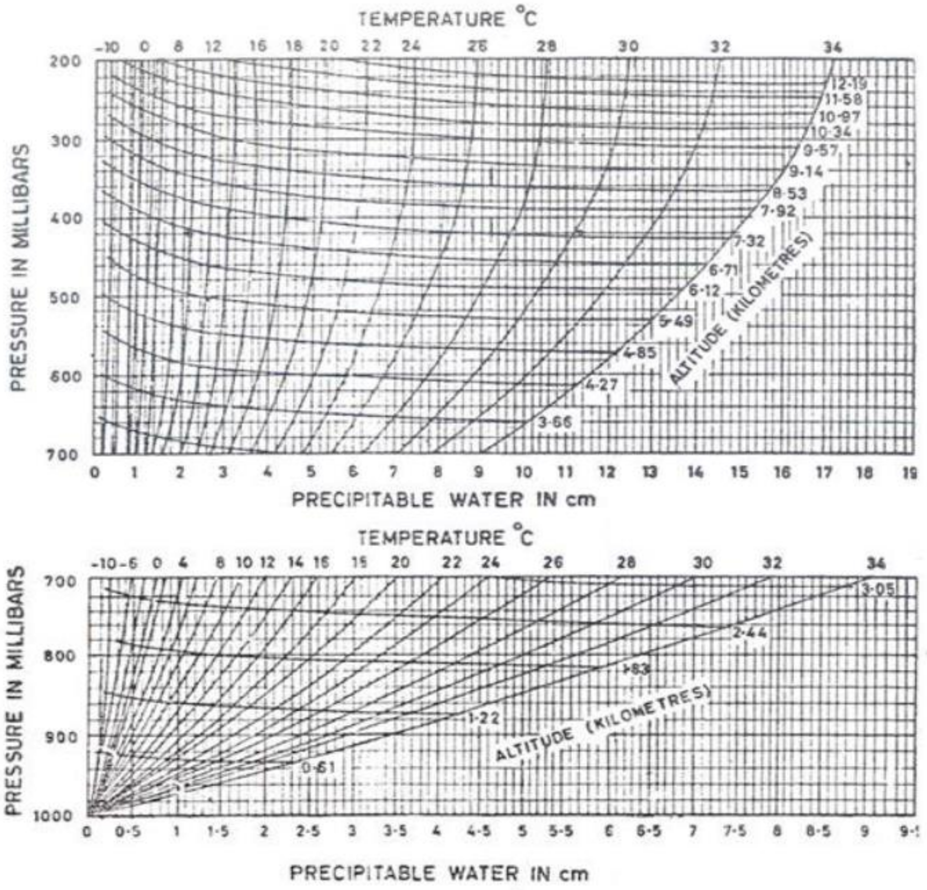


Figure 2.3.- Pseudo-adiabatic diagram for dew point reduction to 1,000 hPa



(Source – National Weather Service (NWS) and Reitan (1963))

Figure 2.4.- Precipitable water above 1,000 hPa in saturated air mass knowing surface dew

temperature are reduced to the 1,000 hPa level (mean sea level) by using Figure 2.3, so that dew points obtained at different elevations are comparable. Figure below gives values of precipitable water (mm) between 1,000 hPa surface and various pressure levels up to 200 hPa in a saturated pseudo-adiabatic atmosphere as a function of the 1,000 hPa dew point. The dew points are converted to precipitable water by the use of Figure 2.4 such as those given in WMO (2009).

The moisture maximization factor can be computed by

$$\text{MMF} = \frac{(W_2)_{h_1}}{(W_1)_{h1}}$$

Where,

h_1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre.

W_1 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to the representative persisting storm dew point temperature (d_1).

W_2 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_2) on record at the location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

Rainstorm Transposition Adjustments

The main purpose of rainstorm transposition is to increase rainstorm experience of an area by considering not only the rainstorm, which have occurred over and near the area in the past, but also those rainstorms, which have resulted in heavy rainfall on adjacent areas that are meteorologically homogeneous. Rainstorm transposition technique is generally applied to such areas, which have markedly irregular shapes or peculiar orientation. Before applying the storm transposition technique, following corrections wherever required are to be considered along with the transposition guidelines as given below.

When no severe rainstorm has occurred in the study area then the nearest available storm from a meteorologically homogeneous region has to be physically moved to the area under study. This movement is called transposition of the storm.

Location Adjustment Factor (LAF)

Outstanding rainstorms in a meteorologically homogeneous region surrounding a project basin are often transposed to the basin to frame PMP estimates for the project basin. The transposition of the rainstorm necessitates application of two adjustments for location and barrier. The location adjustment factor (LAF) is estimated by

$$\mathrm{LAF} = \frac{(W_3)_{h_1}}{(W_2)_{h1}}$$

Where,

h_1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre.

W_2 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_2) on record at the location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

W_3 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_3) on record at the transposed location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

Barrier Adjustment Factor (BAF)

The barrier adjustment factor (BAF) is estimated by

$$\mathrm{BAF} = \frac{(W_3)_{h_2}}{(W_3)_{h1}}$$

Where,

h_1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre.

h_2 is mean crest elevation of the barrier between the original location of rainstorm and the transposed location with mean crest elevation higher than mean elevation of original and transposed locations of rainstorm.

W_3 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_3) on record at the transposed location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

Transposition Adjustment Factor (TAF)

The combined effect of location adjustment factor (LAF) and barrier adjustment factors (BAF) is called the transposition adjustment factor (TAF), thus

$$\mathrm{TAF} = \mathrm{LAF}\ \mathrm{X}\ \mathrm{BAF}$$

$$\mathrm{TAF} = \frac{(W_3)_{h_1}}{(W_2)_{h1}} \mathrm{X} \frac{(W_3)_{h_2}}{(W_3)_{h1}}$$

$$\mathrm{TAF} = \frac{(W_3)_{h_2}}{(W_2)_{h1}}$$

Where,

h_1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre.

h_2 is mean crest elevation of the barrier between the original location of rainstorm and the transposed location with mean crest elevation higher than mean elevation of original and transposed locations of rainstorm.

W_2 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_2) on record at the location of the rainstorm in the same fortnight of the month in which the rainstorm occurred

W_3 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_3) on record at the transposed location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

Moisture Adjustment Factor (MAF)

The combined effect of MMF, LAF and BAF is expressed by a single term known as Moisture Adjustment Factor (MAF) and is expressed by the following relation

$$\mathrm{MAF} = \mathrm{MMF}\ \mathrm{X}\ \mathrm{LAF}\ \mathrm{X}\ \mathrm{BAF}$$

$$\mathrm{MAF} = \frac{(W_2)_{h_1}}{(W_1)_{h1}} \mathrm{X} \frac{(W_3)_{h_1}}{(W_2)_{h1}} \mathrm{X} \frac{(W_3)_{h_2}}{(W_3)_{h1}}$$

$$\mathrm{MAF} = \frac{(W_3)_{h_2}}{(W_1)_{h1}}$$

$$\mathrm{MAF} = \frac{\text{Precipitable water corresponding to maximum persisting dew point on record at transposed location}}{\text{Precipitable water corresponding to persisting storm dew point at rainstorm location}}$$

MAF can be computed directly by using the following formula:

Where,

h_1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre.

h_2 is mean crest elevation of the barrier between the original location of rainstorm and the transposed location with mean crest elevation higher than mean elevation of original and transposed locations of rainstorm.

W_1 is precipitable water in an atmospheric column between 1,000 and 300 hPa levels, corresponding to the representative persisting storm dew point temperature (d_1).

W_3 is precipitable water in an atmospheric column at 1,000 and 300 hPa levels, corresponding to maximum persisting dew point temperature (d_3) on record at the transposed location of the rainstorm in the same fortnight of the month in which the rainstorm occurred.

Various aspects of elevation barriers h_1 and h_2 can be understood from Figure 2.5.

Limit of Transposition

The transfer of storm parameters identified at their places of occurrence to the places where they could occur is known as storm transposition. The storm transposition is limited to meteorologically homogeneous regions. There are many areas that have not experienced severe storms as observed in the vicinity and hence transposition of severe storms is done to supplement the inadequate records.

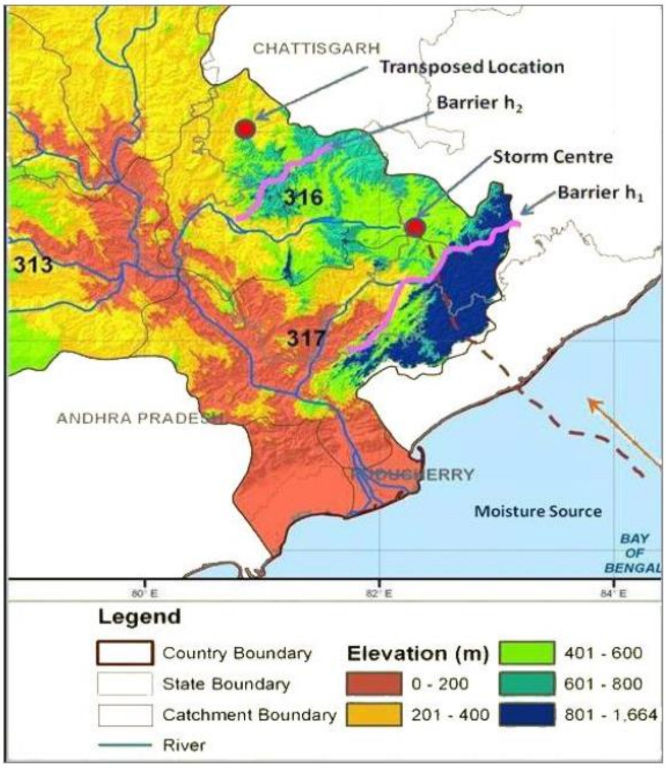


Figure 2.5.- Movement of storm through barrier elevations h1 and h2

Fixing limits to storms for their transposition is one of the most important aspects in a design storm study. The guide to hydro-meteorological practice (WMO-332) suggests that:

i. The area within the transposable limits may have similar, but not identical, topographic and climatic characteristics throughout (Para 2.5.1).

ii. It is essential to determine maximum limits of seasonal transposition along with geographical limits since the storm mechanism may be changing beyond 15 days on either side of the storm period (Para 2.3.1).

iii. In temperate latitudes several lakh sq. km. can be meteorologically homogeneous. Contiguous homogeneity of such large areas are not possible in tropical regions (Para 6.1.5).

iv. Series of depths duration values over a catchment for a long period may form part of the historical evidence to avoid unrealistic exposure to certain parts of the catchments and help in obtaining realistic estimates of SPS/PMS for large catchments.

v. Transposition involving elevation differences of more than 800 m are generally avoided. Regardless an elevation adjustment is used (Para 2.6.2 and 2.6.3) because of their dynamic influences on storms.

vi. Limitation is placed on the rotation of displacement of an isohyetal pattern (Para 2.11.2).

The area for transposition is considered as 2° latitude x 2° longitude for a nearly flat region and 1/2° x 1/2° to 1° x 1° in a mountainous region WMO (1986). Mohile, et al. (1983), while studying the 1982 storm of coastal Odisha, had also verified it to be adequate.

Coastal storms (storms having centered west of the Western Ghat and east of the Eastern Ghat) were not transposed to catchments located far inland. Storms that occurred in Orographic regions were not transposed to plain areas and vice versa.

Transposition to catchment larger than 50,000 sq.km may result in unrealistically excessive PMP estimates. This may result in a severe unrealistic situation of flooding that is not experienced by the catchment.

Thus, transposability and limits of transposition of an individual rainstorm are governed by the meteorological conditions associated with the rainstorm, climatic, topographic characteristic depending upon the type of weather systems which cause rainstorm, and the direction of movement of these systems. Historical records show the characteristic behaviour of these weather systems.

2.6 Selection of major storms

The daily records of rainfall stations within the homogenous region including the project basin are examined to select the dates of occurrence of historical major storms. When there are few rainfall stations within the region the records of each are examined. In areas with relatively dense networks, stations may be selected at spacing such that high rainfalls of limited areal extent (such as intense short lived thunderstorms) will be avoided but all significant rain storms with an areal coverage dose to or larger than the project basin will be detected. Generally four to five such storms are selected for further analysis for their severity.

2.7 Design Storm Depth for Small Catchments

Point SPS/PMP is recommended for catchments up to 50 sq. km or catchments whose basin lags are less than two hours. No reduction of point PMP for the area is required. If the elongation ratio is less than 1.5, point rainfall values can be applied to catchments up to 1 00 sq. km and depth-area- duration values can be used for catchments up to 500 sq. km. For other catchments storm transposition is recommended.

For reducing point SPS/PMP to areal values where necessary, DAD curves of severe storms in the region shall be used to find the appropriate reduction factor. Where an existing dam intercepts the project catchment storm transposition is preferred. If this is not possible then necessary adjustment shall be made to the DAD values for the parts and the full catchment with the assumption that the storm is centred in the intercepted or the free catchment at different times and the combination of storm depths which yields higher flood is used.

2.8 Time adjustment of design storm and its critical sequencing

The design hyetograph should be arranged in two bells (peak) per day. The combination of the bell arrangement and the arrangement of the rainfall increments within each of the bell shaped spells will be representing the maximum flood producing characteristics.

The critical arrangement of increment in each bell should minimize the sudden hill or sluggishness and maximizing the flood peak. Hence, the arrangement is to be such that the time lay between peak intensities of two spells may be minimum. The cumulative pattern of all the increments in the order of their positioning should resemble the natural mass curve pattern as observed by a Self

Recording Rain gauge (SRRG) of the project region.

2.8.1 The Critical Sequence of the Design Rainfall

For this purpose, the severe-most storm 'ever recorded' on the catchment responsible for the highest flood ever recorded is taken up for analysis. If data of the same is not available, the data of a transposed severe-most storm is used for the purpose. For reliable estimates data of recording rain gauges (automatic type) are required.

Following steps are involved in this analysis.

1. The point rainfall data of the storm under consideration is used to compute its average hyetograph which is the plot of rainfall intensity vs time in the form of a histogram. The time step for it is kept equal to the 'unit duration' of the design unit hydrograph'.

2. The loss index is applied to this rainfall hyetograph. This way the rainfall excess hyetograph is computed.

3. For arriving at the critical sequences, the following steps are involves:

    a. The ordinates of the design unit hydrograph are written in a column.

    b. The highest rainfall excess is written against the peak ordinate of the design unit hydrograph. The second highest rainfall excess ordinate is written against the second highest ordinate of the unit hydrograph which is next to the peak ordinate and so on. If all the ordinates of unit hydrographs are not covered by the rainfall excesses zeros are put against them.

    c. The column of the rainfall excess sequence so generated in step (b) above is reversed i.e. the last value of the rainfall excess is put as the first, the 'last but one' is written at the place of the second, and so on. This arrangement of the rainfall excesses is called the 'critical sequences'. In order to compute the design flood these critical sequences of rainfall are now convoluted upon the 'design unit hydrograph'.

2.9 Unit Hydrographs

One of the important aspects of the hydro meteorological approach is to determine the storm-rainfall runoff relation, through an ·appropriate response function of the catchment or the basin. Depending upon the size, shape and other features of the basin, the hydro-meteorological characteristics, and the purpose for which it is required, the response function may be represented either in a comprehensive way by a catchment model or in the simplest form by a unit hydrograph. Unit hydrograph, previously known as unit graph is a simple but the most commonly used tool for estimation of the design flood hydrograph.

The unit hydrograph (UG) of a drainage basin is defined as the direct runoff (outflow)· hydrograph resulting from one unit of effective rainfall which is uniformly distributed over the basin at a uniform rate during the specified period of time known as unit time or unit duration. The unit quantity of effective rainfall is generally taken as 1 mm or 1 cm and the outflow hydrograph is expressed by discharges in cumecs. The unit duration may be 1 hour, 2 hours, 3 hours or so depending upon the size of the catchment and storm characteristics. However, the unit duration cannot be more than the time of concentration, basin lag or the period of rise.

2.9.1 Assumptions

The following assumptions are made while using the unit hydrograph principle:

1. Effective rainfall should be uniformly distributed over the basin, i.e., if there are five rain gauges in the basin which represent the areal distribution of rainfall over· the basin, then all the five rain gauges record almost same amount of rainfall during the specified time.

2. Effective rainfall is at a uniform rate during the unit time.

3. The base or the time duration of the hydrograph of the direct runoff due to effective rainfall of unit duration is constant.

4. The ordinates of the direct runoff hydrograph of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph. This is known as principle of linearity, superposition and proportionality.

5. For a given drainage basin, the hydrograph of runoff due to given period of rainfall reflects all the combined physical characteristic of the basin. The unit hydrograph theory assumes the principle of time invariance. This means that tt1e direct runoff hydrograph from a given .drainage basin due to a particular pattern of effective rainfall will always be the same irrespective of time.

2.9.2 Limitations

Under the natural conditions of rainfall and drainage basins, the above conditions cannot be satisfied perfectly. However, when the hydrologic data used in the unit hydrograph analysis are carefully selected so that they meet the above assumptions closely, the results obtained by the unit hydrograph theory have been found acceptable for all practical purposes. The unit hydrograph theory is not applicable to runoff originating from snow or ice and to the condition having duration of effective rainfall greater than the time of concentration.

In theory, the principle of unit hydrograph is applicable to a basin of any size. However, in practice, to meet the basic assumption in the derivation of the Unit hydrograph as closely as possible, it is essential to use storms, which are uniformly distributed, over the basin and producing rainfall excess at uniform rate. Such storms rarely occur on large areas. The size of the catchment is, therefore, limited although detention, valley storage and infiltration all tend to minimize the effect of rainfall variability. The limit is generally considered to be about 5000 Sq.km. beyond which the reliability of the unit hydrograph method diminishes. When the basin area exceeds this limit it has to be divided into sub-basins and the unit hydrograph is developed for each sub-basin. The flood discharge at the basin outlet is then estimated by combining the sub-basin floods, using flood routing procedures.

2.9.3 Derivation of Unit Hydrograph

The unit hydrograph is best derived from the observed hydrograph resulting from a storm which fulfills the two basic conditions i.e. the rainfall is more or less uniformly distributed over the basin and has a reasonably uniform intensity. Such a hydrograph will generally form a single and sharp peak. In case, such a hydrograph is not available then the unit hydrograph has to be derived from the analysis of an observed complex event. When observed discharge and rainfall data at short interval are not available, then synthetic unit hydrographs are derived with the help of basin characteristics.

Synthetic Unit Hydrograph

To develop unit hydrographs to a catchment, detailed information about the rainfall and the resulting flood hydrograph are needed. However, such information would be available only at a few locations and in a majority of catchments, especially those which are at remote locations; the data would normally be very scanty. In order to construct unit hydrographs for such areas, empirical equations of regional validity which relate the salient hydrograph characteristics to the basin characteristics are available. Unit hydrographs derived from such relationships are known as synthetic-unit hydrographs. A number of methods for developing synthetic-unit hydrographs are reported in literature and for Indian Scenario, they have published by Central Water Commission (CWC) in its Flood Estimation Reports (FER). It should, however, be re-numbered that these methods being based on empirical correlation's are applicable only to the specific regions in which they were developed and could not be considered as general relationships for use in all regions.

Flood Estimation Reports of CWC

The Central Water Commission (CWC) in association with India Meteorological Department (IMD), Ministry of Railway and Ministry of Surface Transport has prepared Flood Estimation Reports for small and medium catchments. However, these reports are finding use even for large catchments. In such a case the large catchment is subdivided into sub catchments and these reports can be conveniently used for each sub- catchment individually and the total effect of the entire catchment can be studied along with other principles of Hydrology such as channel routing etc.

For this purpose of publication of these reports, the country has been divided into 26 hydro-meteorologically-homogenous sub-zones. Theoretically the subzones are considered to be hydro-meteorologically homogeneous but some of the parameters such as the slope of the river, land use etc vary with in these subzones. In each of the sub-zones, data at sufficient number of sites is available. The data at these stations are helpful in studying the variability. of hydrologic and physiographic properties within the catchment. Formulae have been developed which correlates identified parameters of the unit hydrograph with some specific features of the basin. These formulae give more dependable and reliable unit hydrograph because they are based on the data of the same region and properly account for the physiographic variability within the region.

The basic approach used is as follows.

1) Derive the representative unit hydrographs for various catchments in the region for which data are available.

2) Find relations between some defined parameters of these unit hydrographs and the catchment characteristics, and

3) Use the relation to estimate the parameters of the unit hydrograph for catchments which have no records of stream flow but for which the catchment characteristics can be derived from topographical maps.

The catchment characteristics, which are used for derivation of synthetic unit hydrograph·, are:

a. Catchment area, A

b. Length of longest main stream along the river course in km, L

c. Length of the longest main stream from a point opposite to centroid of the catchment area to the gauging site along the main stream in Km, Lc

d. Equivalent stream slope in m/km, S

The formula for deriving synthetic unit hydrographs for various sub-zones has been derived with data in respect of relatively smaller catchments. Further, in some cases the equations have been developed with very limited data. Therefore, due care should be taken while adopting these formulae. The general recommendations of the applicability of the Flood estimation reports are as follows:

i) In case the physiographic parameters of the ungauged catchment matches with any gauged catchment the unit hydrograph parameters of the gauged catchment can be proportionately transposed to the ungauged catchment and the unit hydrograph so derived be adjusted.

ii) While identifying the gauged catchment having physiographic characteristics similar to ungauged catchment preference is given to the catchment close to the location of the ungauged catchment.

iii) When no such catchment is found suitable for transposing the unit hydrograph to the ungauged location the recommended relations are used to develop synthetic unit hydrograph.

2.10 Convolution

The UH Convolution formula is

$$Q_n = \sum_{i=1}^{n} P_i U_{n-i+1} = P_n U_1 + P_{n-1} U_2 + \ldots + P_1 U_n$$

Where n is the time, P_i is the rainfall excess at time increment i and U_i is the unit hydrograph ordinate at time increment i.

For each of the blocks of hyetograph of critical sequences, the depth of rainfall excess (i.e. d_1, d_2, d_3 …. cm) cm is worked out. As shown in the table, corresponding to each depth of rainfall excess (dj) its direct runoff hydrograph is computed and written in columns (3), (4) …. by given suitable lags keeping in view the starting time of the rainfall excess (dj) (At the 'Starting time' of the rainfall excess the zero of its DRH is placed). The total direct runoff is obtained by summing the rows. The base flows are added to the total DRH to obtain the total 'design flood'. The area under the design flood hydrograph gives the total design flood volume.

The methodology which has been explained from point 2.4 to point 2.10 of this Chapter has been used in projects whose reports are tagged in Appendix D and Appendix E

Sl.No.	Δt-hour Design U-H (m^3/s)	Direct Runoff Hydrograph Ordinates for Rainfall Excess (d_j) d_1	d_2	d_3 ……….	Total DRG (Σ)
0	0	0			0
Δt	U_1	d_1U_1	0		d_1U_1
2Δt	U_2	d_1U_2	d_2U_1	0	d_1U_2 x d_2V_1
3Δt	U_3	d_1U_3	d_2U_1	d_3U_1	$d_1U_3+d_2U_2+d_1U_1$
4Δt	U_4	..	…	d_3U_2	….
…	….	…	…	d_3U_3	…
…	….	…	…	0	…
…	….	…	…	…	…

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Chapter 3. SELECTING AN INFLOW DESIGN FLOOD. OVERVIEW

3.1 Different Approaches Same Objective

In any design scenario, it is important to consider the full range of hydrologic events to which a dam will be subjected. When contemplating modifications to a dam to increase the conveyance capacity to pass extreme hydrologic events, some changes, like widening the spillway or lowering the crest of the spillway, may increase the risk to the downstream public by increasing the spillway flows during large floods. Other modifications, such as raising the dam to increase the spillway capacity, can increase the downstream consequences should the dam fail during an extreme flood event by creating a larger dam breach flood wave. The goal of selecting the IDF should be to balance the risks of a hydrologic failure of a dam with the potential downstream consequences and the benefits derived from the dam.

Selection of an IDF can involve trade-offs in trying to satisfy multiple objectives including:

- Providing adequate safety to the public.
- Effectively applying the resources of the dam owner.
- Supporting the credibility of the regulator in representing the interest of the public.
- Assessing the desire of the public for the benefits of a dam in exchange for the inherent risks that come from living downstream of a dam.

No single approach for choosing an IDF is adequate for the unique situations of thousands of existing or planned dams. The following alternative ways (**Figure 3.1**) of deciding on an IDF are recommended to accommodate the wide variety of situations, available resources, and conditions which

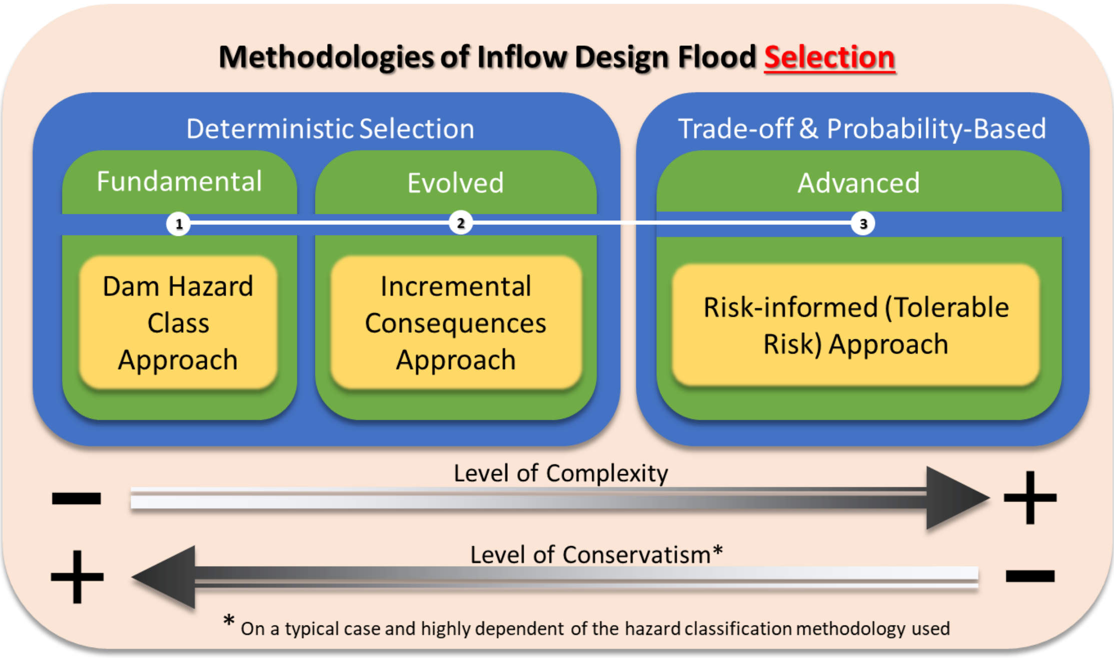


Figure 3.1.- Methodologies of Inflow Design Flood Selection

might be encountered in practice:

3.1.1 Prescriptive (Dam Hazard Class) Approach

In this initial phase, a planned dam is designed, or an existing dam is evaluated for a prescribed standard based on the hazard classification of the dam, which normally deal with relatively simpler dam breach assumptions where the most conservative parameters resulting in a worst downstream inundation scenario are considered. This approach is intended to be overly cautious to allow for efficiency of resource use while providing reasonable assurance of the safety of the public. It does not guarantee that there is an economical marginal benefit from designing for a conservative IDF.

Limitations of a simple prescriptive approach also need to be recognized and considered if such approach is used to select an IDF and require modifications to a dam to satisfy these criteria. In some cases, modifications to a dam to accommodate a prescriptive IDF, such as increasing dam height or spillway capacity, can increase potential consequences or introduce potential failure modes that could significantly increase rather than decrease risks to the public.

Consequently, a prescriptive approach to IDF selection and implementation should be applied judiciously with full consideration of its overall risk and public safety implications.

An example of a basic prescriptive approach for the IDF selection is illustrated in Table 3.1. As can be seen, the IDF selected through this methodology is highly dependent of the hazard potential classification methodology used. For further information in the hazard classification process please refer to the *Guidelines for Classifying the Hazard Potential of Dams*. Also, a brief summary of the above-mentioned guidelines is presented in Section 3.3 (Dam Classification System) of this document.

Currently, the prescriptive approach in India relies upon determination of a PMF for high hazard dams which requires assessment of the PMP. The most common sources of the PMP information are the regional PMP Atlases published by the CWC and the IMD. These reports provide generalized rainfall values that are basin-specific and tend to represent the largest PMP values within broad regions. Most of these reports

Table 3.1.- Illustrative Example of a Prescriptive Approach for IDF selection

Dam Hazard Potential Classification	Definition of Hazard Potential Classification	Inflow Design Flood
High	Probable loss of life due to dam failure or misoperation (economic loss, environmental damage, or disruption of lifeline facilities may also be probable, but are not necessary for this classification)	PMF
Significant	No probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities due to dam failure or misoperation	0.1% Annual Chance Exceedance Flood (1,000-year Flood)
Low	No probable loss of human life and low economic and/or environmental losses due to dam failure or misoperation	1% Annual Chance Exceedance Flood (100-year Flood) or a smaller flood justified by rationale

have been updated to reflect current state-of-the-art knowledge and technology. A site - specific study of the PMP/PMF using current techniques can result in a more appropriate estimate of the PMF for consideration as the IDF as it would have the advantage of using the current/ updated available rainfall data as well.

3.1.2 Incremental Consequence Analysis

The inflow design flood (IDF) selection using the incremental consequences analysis can be defined as the flood above which there is none or negligible increase in downstream inundation consequences (i.e. depth, flow velocity, loss of life or economic consequences) due to failure of the dam when compared to the same flood without dam failure. (**Figure 3.2**)

Typically, incremental consequence analysis considers the potential for loss of human life and economic loss or property damage. The analysis could also consider consequences such as lifeline disruption and environmental impacts.

If incremental consequence analysis is used to define the IDF, upstream and downstream conditions and development should be periodically reviewed to ensure that changes in prospective consequences do not lead to a different recommendation for the IDF.

It is also important to understand that once a dam is constructed, the downstream hydrologic regime may change, particularly during flood events. The change in hydrologic regime could alter land use patterns to encroach on a floodplain that would otherwise not be developed without the dam. Consequently, evaluation of the consequences of dam failure must be based on the dam being in place and should compare the impacts of with-failure and without-failure conditions on existing development and known and prospective future development. Comparisons between existing downstream conditions with and without the dam are not recommended when analysing incremental consequences.

A hypothetical dam failure should be estimated using conservative yet realistic

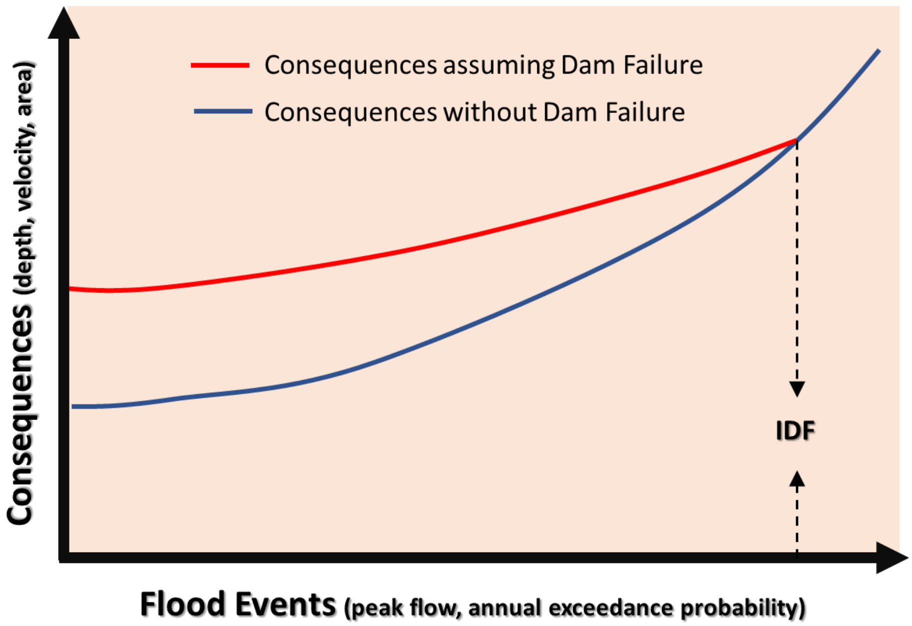


Figure 3.2.- Schematic Illustration of Incremental Consequence Analysis for the Selection of IDF

dam breach parameters. If it can be shown that the PMF dam failure event would not cause additional loss of life or significant property damages greater than the PMF non-failure event, a flood of lesser magnitude can be analysed in the same comparative manner. This process is continued until the flood of greatest magnitude that causes incremental consequences is identified. **Figure 3.3** illustrates this iterative process of the incremental consequence analysis.

It worth to mention that the Incremental Consequences Analysis is proven to be more meaningful when the reservoir's volume may be small in comparison to the volume of the hydrologic event to which they may be subjected. In general terms, its application in exceptionally large reservoirs would lead to the selection of the maximum theoretical event (e.g. PMF) as the suitable IDF, aspect that could be inferred beforehand only using engineering judgment.

Additionally, exist a lot of debate regarding what should be considered as a "significant incremental consequences". In this regard, criteria, and methodologies described in the "*Guidelines for Mapping Risk Associated with Dams*" and "*Guidelines for Classifying the Hazard Potential of Dams*" (CWC, 2020) should be followed for assessing the incremental consequences in the entire floodplain or study area, and not only in a single location. Such criteria should not be considered as absolute decision-making thresholds. Rather, sensitivity analysis and engineering judgment must be applied.

Generally, "acceptable" or "negligible" incremental consequences exist when the evaluation of the affected area indicates one of the following:

- There are no human habitations or major infrastructure, commercial, or industrial developments within the dam failure inundation area.

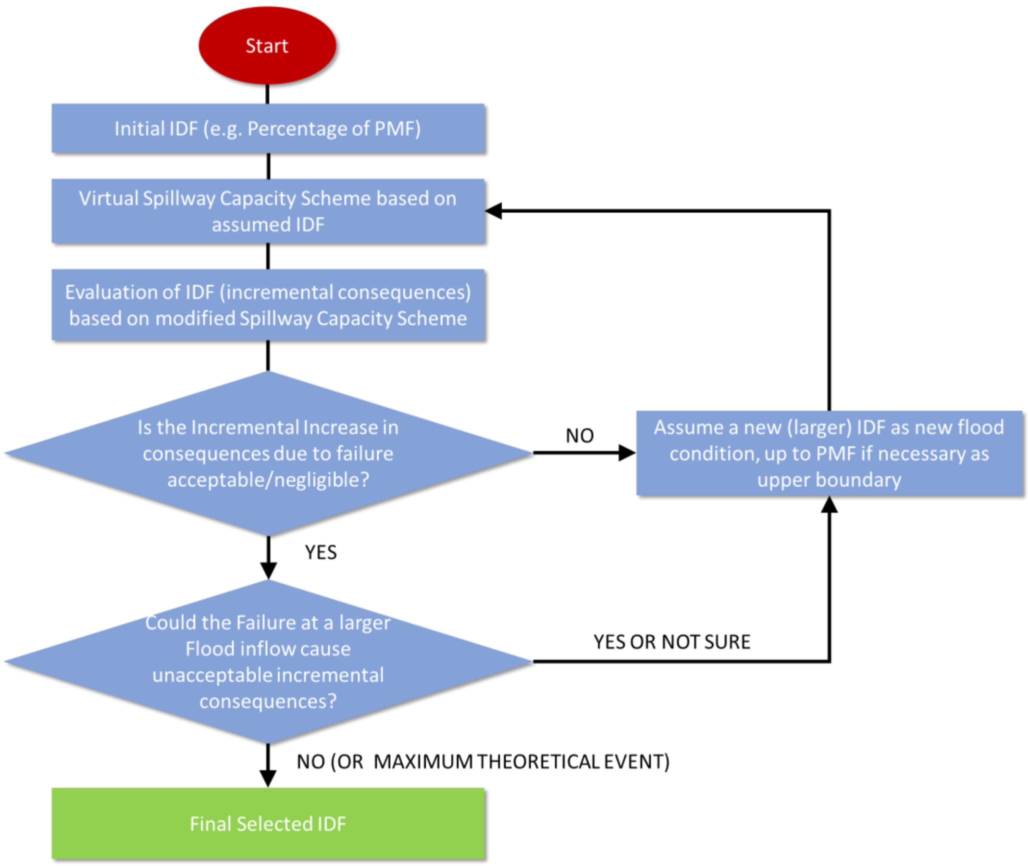


Figure 3.3.- Iterative process of the Incremental Consequence Approach

- There are human habitations or major infrastructure, commercial, or industrial developments within the dam failure inundation area, but there would be no significant incremental increase in the threat to life or property.

3.1.3 Risk-informed Hydrologic Hazard Analysis

The risk-informed hydrologic hazard analysis is a decision-making process that includes a site-specific evaluation of the probabilities of a full range of extreme hydrological events (i.e. Hydrologic Hazard Curves) and performance of the dam during those events, and evaluates in more detail the social, economic, and environmental consequences of failure. In short, risk-informed approach is a tool for evaluating hydrologic events in a risk-based context and the level of effort is proportioned to safety issues.

In this method, the IDF is selected as the design flood below which the consequences risk due to failure of a dam does not exceed a given level of "tolerable risk". For instance, an international practice is to use two tolerable risk indices to justify the IDF selection such as averaged annual failure probability (AFP) of a dam and a resulting averaged annual life loss (ALL). For India, the proposed tolerability guidelines are described in the "*Guidelines for Assessing and Managing Risks Associated with Dams*" (CWC, 2109) and are presented in **Figure 3.5**. Examples, and case studies of this approach are also included in the above-mentioned guideline.

The strengths of this approach include providing dam owners and regulators the ability to assess the marginal value of increasing levels of flood protection, balancing capital investment in risk reduction across a number of different failure modes, and prioritizing risk mitigation actions.

When using risk analysis to select the IDF for a dam, the uncertainty associated with the analysis needs to be considered (Please see Appendix C). If the results are sensitive to assumed or extrapolated values that have significant uncertainty, conservative

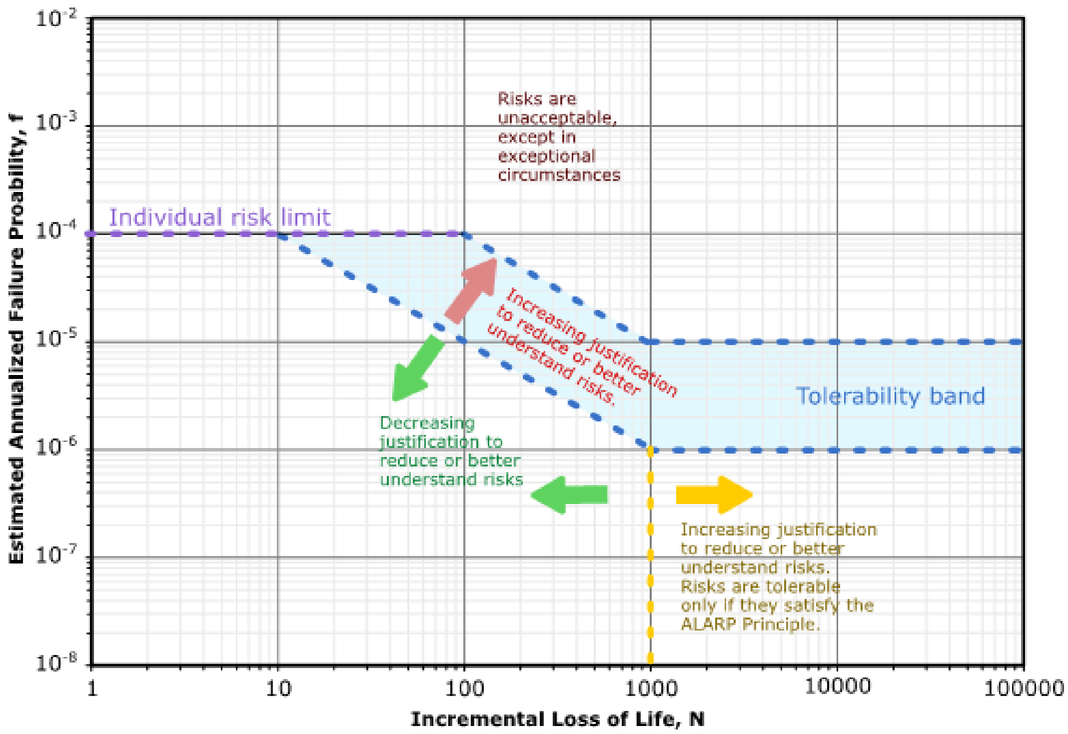


Figure 3.5.- Proposal of Risk Tolerability Guidelines for India (CWC, 2019)

assumptions within the confidence limits of the analysis should be used to select the IDF. Because risk may change with time, a periodic review of conditions at the dam as well as upstream and downstream conditions and development should be performed to ensure the validity of the analysis.

3.2 Holistic Comparison of IDF Selection Methodologies

The selection methodologies of both deterministic and probability-based approaches, shown in **Figure 3.1**, are appropriately being used to facilitate dam safety risk management for evaluating hydrologic safety of dams. The deterministic approach includes the prescriptive approach based solely on a dam's hazard potential class, and the Incremental Consequences Analysis is based on the incremental upstream and downstream inundation situations. The more advanced probability-based approach is a quantitative oriented risk-informed decision-making process to meet a defined tolerable risk level.

To illustrate the merits and limitations of the deterministic and probabilistic approaches for the IDF selection process, **Table 3.2** describes the advantages and disadvantages of each methodology from a practical perspective on how each approach could fulfil different aspects and categories not only in the IDF slection but also

3.3 Dam Classification System

All state and central agencies should use some type of dam classification system to categorize dams according to the probable damages or adverse consequences caused by a dam failure. Under a prescriptive approach, the IDF is often specified based solely on the dam classification system. Given the limited resources of many states and central agencies and the fact that they have hundreds or thousands of dams under their jurisdictions, use of a generalized dam classification system based on the hazard to select the IDF is both practical and reasonable.

The *Guidelines for Classifying the Hazard Potential* of dams, published as part of the same series, describe in detail the proposed approach for hazard potential classification of dams in India (**Table 3.3**). This hazard potential classification system for dams is simple, clear, concise, and adaptable to any agency's current system. The intent of this classification system is to provide straightforward definitions that can be applied consistently and uniformly by all federal and state dam safety agencies and can be readily understood by the public.

It should be understood that the "hazard potential" is the possible adverse incremental consequences that result from the release of water or stored contents due to failure or mis-operation of the dam. Incremental consequences are defined as the impacts that would occur due to failure or mis-operation of the dam over those that would have taken place without failure or mis-operation of the dam. The hazard potential assigned to a dam is based on consideration of the incremental adverse effects of failure during both normal and flood flow conditions. Hazard potential does not indicate the structural integrity of the dam itself, but rather the consequences

This dam classification system is recommended to be used with the understanding that the failure of any dam, no matter how small, could represent a danger to downstream life and property. It is recommended that the hazard potential classification system is used as the basis for IDF selection guidelines and that use of any classification scheme based on the size (height or storage volume) of a dam for IDF selection be gradually discontinued with the availability of data and development of consensus on framework.

The size of a dam has historically been used as a simple indicator for estimating hazard potential because of the unavailability of data or inability to conduct more detailed analyses. It is recognized that size classification has been helpful in reducing the number of dams impacted by risk or hazard creep; however, a formal assessment of future development should also be sufficient to select an appropriate level of design while limiting the potential for risk creep. Data and analytical approaches are now available that are economical to perform and provide a more precise assessment of the hazard potential. By using a classification system that is based on the size of the dam, a small dam located in a densely populated area may, in fact, be subject to less stringent requirements than a substantially larger large dam in a remote location where the downstream development is sparse.

Therefore, dam size is not always indicative of potential consequences due to failure and should not be the basis for evaluating hydrologic design requirements.

Table 3.2.- Comparison of Methodologies for the IDF selection

Item	Sub-Item	Prescriptive Approach	Incremental Consequences Analysis	Risk-informed Hydrologic Hazard Analysis
Dam Failure mechanism Rationalization	Consideration and Evaluation of Potential Failure Modes	None or highly dependent of the type of hazard classification approach. In the best case, a single and most conservative potential failure mode is considered without describing the failure mode's sequential physical process	Subjective, conservative overall assumptions usually made on one single failure mode without specifically describing the failure mode's sequential physical process	More realistically sequential occurrence probabilities estimated as needed based on a flood event tree risk model in which loading, response and consequence of dam failure are represented by levels of branching
	Dam Breach Size	Single size causing a worst inundation scenario. Usually no sensitivity analysis is done	Single size causing a worst inundation scenario through a sensitivity analysis which may be overly conservative	Varied cases of multiple assumed sizes with individual probabilities while actual size remains uncertain
	Uncertainties on structural component functions	Not considered. Dam structural component functions usually assumed as designed without flexibility	Not considered. Dam structural component functions usually assumed as designed without flexibility	Considered. Gate reliability, spillway debris plugging, etc. driven by probabilities based on historical records
Consequences Estimation	Consequences model of life and property loss estimates	None or highly dependent of the type of hazard classification approach used. In the best case, Life and property loss estimates as a lump sum figure	Life and property loss estimate as a lump sum figure	Life and property loss estimates associated with each end node of the probability-based event trees. Annualized Loss of Life is obtained
Justification of	Sensitivity Analysis for structural and Non-	none	Explores the effects of adjusting parameters of dam	Explores not only the effects of modifying dam structure,

Item	Sub-Item	Prescriptive Approach	Incremental Consequences Analysis	Risk-informed Hydrologic Hazard Analysis
IDF selection	structural measures to reduce dam failure hazard potential		structure breach rather than including evacuation effectiveness	but also adjusting the evacuation effectiveness and improving dam performance/operation
	Final Justification	Hazard potential class	Analysis result of insignificant inundation incremental rise	Tolerable risk levels such as Annual Probability of Failure and Annual Loss of Life to be satisfied
Justification for IDF Accommodation	Measures for an inadequate spillway system	Only structural measures as a common approach. Normally, limited options available	Only structural measures as a common approach. Several options aiming to reduce incremental consequences	Both structural/non-structural measures such as improving evacuation effectiveness
	Risk Reduction Assessment	In general, only the required IDF based spillway capacity upgrading as the solution	In general, only the required IDF based spillway capacity upgrading as the solution	Quantitative risk reduction measures allowing uncertainties judgment and flexibility of IDF by tolerability guidelines. Uses ALARP (As low as reasonable practicable) principle to evaluate the strength (i.e., adequacy and degree) of justification of risk reduction options

Table 3.3.- . Recommended Dam Classification System Based on Hazard Potential (CWC,2020)

Hazard Potential Class	Hazard	Consequences Categories			
		Capital Value of Project	Potential for Loss of Life	Potential for Property Damage	Potential for Environmental and Cultural Impact
Class I	**Low**	Low	None. Occasional or no incremental population at risk, no potential loss of life is expected. No inhabited structures.	Minimal. Limited economic and agricultural development.	None
Class II	**Intermediate**	Average	Minimal or low population at risk. No potential loss of life is expected even during the worst-case scenario of emergency management	Notable agriculture or economic activities. States highways and/or rail lines.	Minimal incremental damage. Short-Term or reversible impact (less than 2 years)
Class III	**High**	Significant	Considerable. several inhabited developments. Potential for loss of life highly dependent of the adequacy of warning and rescue operations.	Significant industry, commercial and economic developments. National and state highways and rail lines.	Limited. Impact have a mid-term duration (less than 10 years) with high probability of total recovery after mitigation measures
Class IV	**Extreme**	Critical	Extreme. High density populated areas. Potential for loss of life is too high even during the best scenario of emergency management	Highly developed area in terms of industry, property, transportation, and lifeline features	Severe. long-term impact/effects in the protected areas or cultural heritage sites with low probability of recovery.

Chapter 4. THE INTERNATIONAL EXPERIENCE

4.1 Introduction

There is not a single international standard for the Selection and Accommodation of Inflow Design Floods. In fact, in several countries (Australia, Canada, and the USA), there are broad national guidelines, and each of the states or provinces is free to either adopt them completely, adopt partial aspects, or altogether ignore the guidelines. After all, these are guidelines, not legal requirements. Some other countries (e.g., Germany, Portugal, Spain) the "guidelines" are requirements that must be followed by national law. A fairly comprehensive review of the international guidelines is contained in Chapter 3 of the International Committee of Large Dams (ICOLD) Bulletin 170, although many of the guidelines described there have now been superseded by more recent ones. This current section borrows liberally from the ICOLD reference, and the descriptions of those guidelines that have been superseded are based on original documents from each of the countries

Some countries have guidelines on dam classification based exclusively on the physical characteristics of the dam and/or reservoir, such as dam height, dam crown length, reservoir volume, or on empirical combinations of several of these characteristics. In preparing this review, it was clear that countries, organizations or states/provinces that have updated the guidelines are moving away from physical dam/reservoir characteristics to guidelines that include some explicit description of the risk involved in dam failure. These modifications include doing away altogether with the physical characteristics of the dam as the source of dam classification to having a mixed classification that involves both downstream risk and physical features.

4.2 Inflow Design Flood Currently Followed in India

Current guidelines for selecting design floods for dams in India are given by the Bureau of Indian Standards (BIS) in IS: 11223-1985 Guidelines for fixing spillway capacity. In this publication, a prescriptive approach is recommended (Table 4.1), where dams are classified by size using the hydraulic head and the gross water storage capacity of the impoundment at the full reservoir level - whichever leads to higher hazard class.

The hydraulic head is the difference between the maximum water level in the reservoir and the annual average flood level on the downstream side. Since this involves pre-selection of design flood for assessment of the MWL, an alternate definition (vide Amendment No. 2, Sep 1991) has been presented which considers the difference between the FRL and the minimum tail water level downstream of the dam as the

Table 4.1.- Existing Dam Classification for Inflow Design Flood Selection (IS 11223)

Class	Gross storage capacity (Mm^3)	Hydraulic head (m)	Inflow Design Flood (IDF)
Small	0.5 to 10	7.5 to 12	100-year flood[a]
Intermediate	10 to 60	12 to 30	Standard Project Flood (SPF)
Large	> 60	> 30	Probable Maximum Flood (PMF)

hydraulic head.

4.3 Australia (Federal)

Australia guidelines for *Selection of an Acceptable Flood Capacity for Dams* were published in 2000 and developed by the Australian National Committee on Large Dams (ANCOLD).

In Australian Guidelines risk assessment is integrated in the determination of design flood. The Acceptable Flood Capacity, AFC, for a specific dam is defined as "the overall flood capacity, including freeboard as relevant, which provides an appropriate level of safety against a flood initiated dam failure to protect the community and environment, to acceptable overall risk levels, within the total context of overall dam safety from all load cases".

According with Australian guidelines three approaches are defined to evaluate the AFC. First, one approach is described for AFC determination in small dams based exclusively in the population at risk. A second approach is based on the hazard category and use **Table 4.2** and **Table 4.3** to classify dams based on 1) population at risk (PAR), and 2) potential loss of life (PLL), both in combination to the severity of damage or loss. The damage or loss can be to health and social, environmental, infrastructure, and business cost, and has a scale of four qualitative values: Minor, Medium, Major, and Catastrophic

Finally, a third risk-informed approach is also recommended. This comprehensive approach is based on the ALARP principle which requires that risks should be as low as reasonably practicable and as minimum within the limits of tolerability in the country (See **Figure 4.1**). The methodology for demonstrating risks is to be applied to all assessments where the risk assessment procedure is used for determining the Acceptable Flood capacity (AFC).

In the **Figure 4.1**, the Y-axis corresponds to the probability of occurrence and the X-axis is the number of fatalities. The red zone corresponds to risks that are unacceptable, except in exceptional circumstances. The yellow zone shows risks that are tolerable if they are as low as reasonably practicable (ALARP). The Green Zone shows where the risk should be monitored.

The design flows corresponding to those categories are shown on the **Table 4.4**. Worth to mention that several states in Australia, including Victoria, New South Wales, Queensland, Tasmania, and the Australian Capital Territory, have their own guidelines.

Table 4.2.- Dam Hazard Potential Classification based on Severity of Damage vs Population at Risk (ANCOLD, 2012)

Population at Risk (PAR)	SEVERITY OF DAMAGE AND LOSS (e.g. health and social, environment, infrastructure and business cost)			
<1	Very Low	Low	Significant	High C
≥ 1 to 10	Significant *(Note 2)*	Significant *(Note 2)*	High C	High B
≥ 10 to < 100	High C	High C	High B	High A
≥ 100 to <1,000	*(Note 1)*	High B	High A	Extreme
≥ 1,000		*(Note 1)*	Extreme	Extreme

Note 1: With a PAR in excess of 100, it is unlikely damage will be minor. Similarly with a PAR in excess of 1,000 it is unlikely damage will be classified as medium.

Note 2: Change to "High C" where there is the potential of one or more lives being lost.

Table 4.3.- Dam Hazard Potential Classification based on Severity of Damage vs Potential Loss of Life (ANCOLD, 2012)

Potential Loss of Life (PLL)	SEVERITY OF DAMAGE AND LOSS (e.g. health and social, environment, infrastructure and business cost)			
<0.1	Very Low	Low	Significant	High C
≥ 0.1 to 1	Significant	Significant	High C	High B
≥ 1 to < 5	*(Note 1)*	High C	High B	High A
≥ 5 to <50		High A	High A	Extreme
≥ 50		*(Note 1)*	Extreme	Extreme
Note 1: With a PLL equal to or greater than one (1), it is unlikely damage will be minor. Similarly with a PLL in excess of 50 it is unlikely damage will be classified as medium.				

Table 4.4.- Required range of acceptable flood capacities for different hazard categories (ANCOLD, 2000)

Incremental Flood Hazard Category	Flood Annual Exceedance Probability
Extreme	PMF
High A	PMP* design flood
High B	Smaller between PMP design flood and 10^{-6}
High C	Smaller between PMP design flood and 10^{-5}
Significant	5×10^{-4} to 10^{-4}
Low/Very Low	Upto 5×10^{-4}

Note: probability of the probable maximum precipitation (PMP) design flood is a function of the catchment area

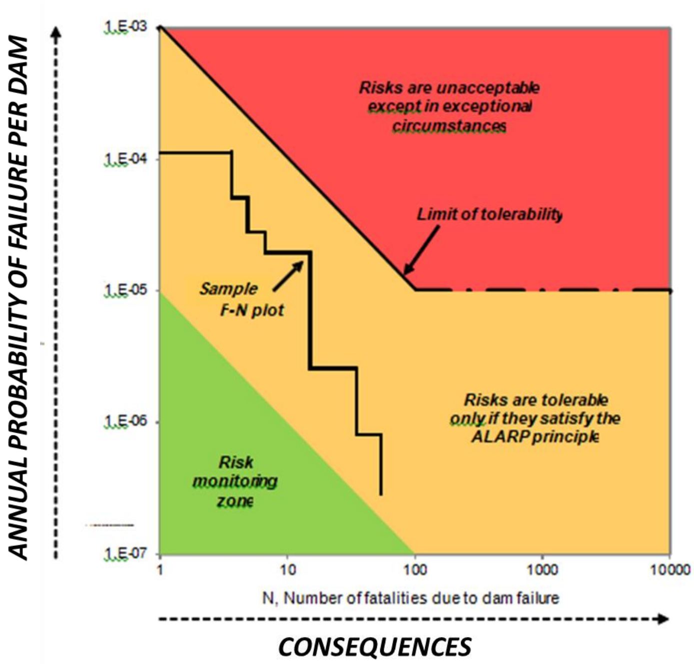


Figure 4.1.- Limits of Risk Tolerability (Australia Federal Guidelines, ANCOLD 2012)

4.4 China

China classifies dams into 5 categories ("I" being the highest and "V" the lowest), based both on physical characteristics of the project, and on the downstream risk associated with a dam failure.

China considers seven criteria in its dams' classification. Each criterion has five different thresholds. The criterion with the highest threshold sets the class for the project, and, therefore, the magnitude of the IDF. **Table 4.5** shows the thresholds for each of the criteria. For instance, a project that is design to provide irrigation to more than 1000 ha will have a Category I classification, even if the other criteria are below their respective thresholds.

China additionally assigns different IDF for parts of the project, based on the project classification. **Table 4.6** shows for each project rank what grade main structures, less important structures and temporary structures, (used during construction only), will have. The Grade is directly related to the IDF, as shown in Table 4.7

For Chinese dams, then, the process is:

i. Classify the project according to the highest criterion found in Table 4.5;
ii. Define the grade for associated structures, both permanent and temporary, and find the corresponding grade from Table 4.6, and
iii. Find on **Table 4.7** the flood return period for each grade.

There are additional caveats for classification and IDF selection, and the reader is referred to the ICOLD Bulletin 170 for additional information.

Table 4.5.- China Dam Classification Table (ICOLD, Bulletin 170)

Rank of Project	Storage Capacity (hm3)	Flood Prevention		Water Logging	Irrigation	Water Supply	Water Power
		Cities and Industrial Areas	Farmland (10^3 ha)	Logged Areas (10^3 ha)	Area (10^3 ha)	Cities and Mines	Installed Capacity (MW)
I	> 1000	Very Important	> 333	>133.3	> 100	Very Important	>750
II	100 – 1000	Important	67 - 333	40 – 133.3	33.3 - 100	Important	250 - 750
III	10 - 100	Moderately Important	20 - 67	10 - 40	3.3 – 33.3	Moderately Important	25 - 250
IV	1 - 10	Less Important	3.3 - 20	2.0 - 10	0.3 – 3.3	Less Important	0.5 - 25
V	< 1		< 3.3	< 2	< 0.3		< 0.5

Table 4.6.-. Classification of Hydraulic Structures in China (ICOLD, Bulletin 170)

Rank of Project	Grade of Permanent Structures		Grade of Temporary Structures
	Main Structures	Less important ones	
I	1	3	4
II	2	3	4
III	3	4	5
IV	4	5	5
V	5	5	-

Table 4.7.- Inflow Design Flow in China (ICOLD Bulletin 170)

Return Period of Flood		Grade of Hydraulic Structures				
		1	2	3	4	5
Design Flood		500	100	50	30	20
Check Flood	Embankment	10000 or PMF	2000	1000	500	200
	Concrete	5000	1000	500	200	100

4.5 United States

In the United States, the states have, by law, considerable control on the water resources. Federal guidelines provide a framework upon which the states may develop their own guidelines, but, as guidelines, they are not mandatory.

There are several guidelines in the United States. Federal Agencies such as the Federal Emergency Management Agency (FEMA), the Federal Energy Regulatory Commission, the US Bureau of Reclamation, the US Park Service, the US Department of Agriculture, and the US Army Corps of Engineers have their own guidelines. Furthermore, several states have their own norms or guidelines, or no guidelines at all. In the following sections one Federal and one State guidelines are presented.

3.5.1 FEMA

FEMA, the Federal Emergency Management Agency, has the main role of managing accidents, including preparing for unavoidable natural disasters, such as hurricanes, to providing monetary and other types of assistance to affected parties.

FEMA has provided several publications that deal with dam safety. For the purpose of the Inflow Design Flood, Publication FEMA 333, which includes the classification of dams with respect to the risk they pose, and FEMA 94, which provides guidelines on the selection and accommodation of inflow design flows are the most relevant. As is the case with Australia, FEMA's dam classification is based solely on risk

Table 4.8.- FEMA Classification of Dams

Hazard Potential Classification	Loss of Human Life	Economic, Environmental & Lifeline losses
Low	None expected	Low and generally limited to owner
Significant	None Expected	Yes
High	Probable. One or more expected	Yes, but not necessary for this classification

FEMA's guidelines provide three approaches to compute the IDF for new dams, or dams undergoing significant modifications: Prescriptive, Incremental, and Risk-Informed Hydrologic Hazard Analysis. For existing dams, FEMA warns against "grandfathering" existing dams without consideration to assessing the risk downstream. That risk may have changed since a dam was built, due to a number of issues such as downstream development, updates in the hydrology due to additional years of record, due to updated hydrologic practices, or due to changed hydrologic regime.

The prescriptive approach IDF is summarized below (**Table 4.9.-** FEMA's Prescriptive IDF)

Table 4.9.- FEMA's Prescriptive IDF

Hazard Potential Classification	IDF
Low	1% Annual Exceedance Probability (100-yr flood)
Significant	0.1% Annual Exceedance Probability (1000-yr flood)
High	PMF

The incremental consequence approach defines the IDF as the inflow above which the incremental consequence of a dam breach is negligible. In other words, at inflows above that one, the damage caused by the natural runoff will not be incremented by the dam breach. *Chapter 3.1.2* of this Guideline (Incremental Consequence Analysis) includes a more detailed description of this approach.

The third technique is considerably more involved. It is the Risk-Informed Hydrologic Analysis. The analysis includes

i. Characterization of Hydrologic Hazard Curve (Swain et al. 2006)
ii. Development of the corresponding hydrologic loads
iii. Identification of the potential failure modes
iv. Assessment of the potential failure mode probabilities
v. Quantification of consequences of dam failures
vi. Quantification of dam safety risk
vii. Selection of the IDF based on public risk tolerance and risk guidelines

3.5.2 Washington State

The State of Washington bases its classification on the FEMA guidelines, but expands the conditions to which the classification applies. Calculation of the IDF for Washington State is unique in which the downstream consequences are assessed as a point value. The IDF is then a function of the point value. **Table 4.10** shows the Dam Classification for Washington State. **Table 4.11** shows the guidance for assigning point

Table 4.10.- Washington State Dam Classification

Downstream Hazard Potential	Downstream Hazard Category	Population at Risk	Economic Loss	Environmental Damage
Low	3	0	Minimal. No inhabited structures. Limited agricultural development.	No deleterious material in reservoir contents.
Significant	2	1-6	Appreciable. 1 or 2 inhabited structures. Notable agriculture or work sites. Secondary highway and/or rail lines.	Limited water quality degradation from reservoir contents and only short-term consequences
High	1C	7-30	Major. 3 to 10 inhabited structures. Low density suburban area with some industry and work sites. Primary highways and rail lines.	Severe water quality degradation potential from reservoir contents and long-term effects on aquatic and human life.
High	1B	31-300	Extreme. 11 to 100 inhabited structures. Medium density suburban or urban area with associated industry, property, and transportation features.	
High	1A	More than 300	Extreme. More than 100 inhabited structures. Highly developed, densely populated suburban or urban area with associated industry, property, transportation, and community lifeline features.	

Table 4.12.- Washington State IDF Steps Options as a Function of the Downstream Hazard Classification

Downstream Hazard Classification	3	2	1C	1B	1A
Typical Design Steps	1-2	3-4	3-6	4-8	8

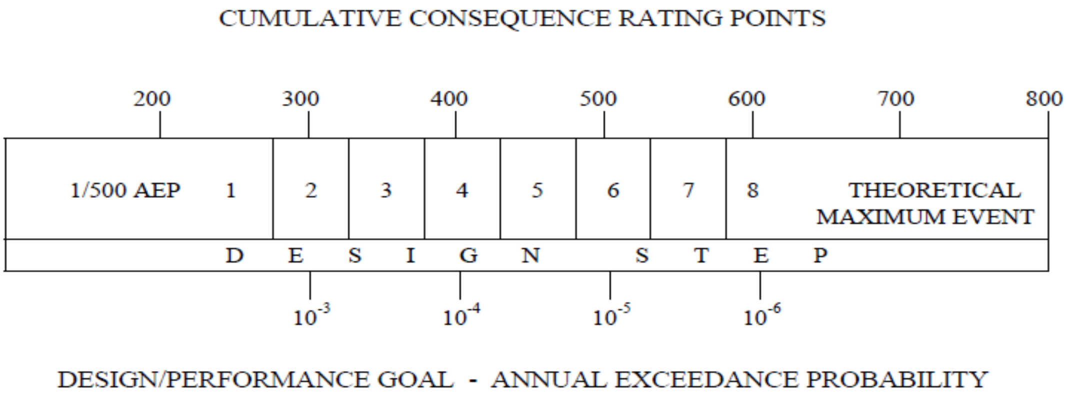


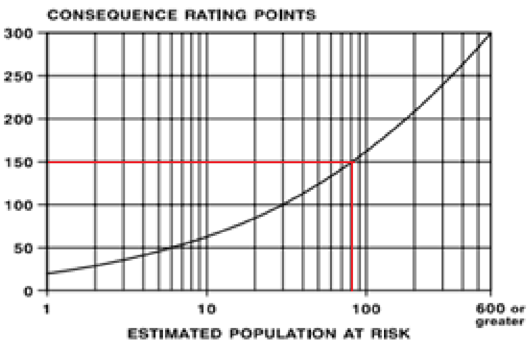
Figure 4.2.- Washington State. Inflow Design Flood and Design Step as a function of Cumulative Consequence Rating Points

Table 4.11.- Washington State Consequences Point Values

Consequence Categories	Consequence Rating Points	Indicator Parameter	Considerations
Capital Value of Project	0 -150	Dam Height	Capital Value of Dam
	0 - 75	Project Benefits	Revenue Generation or Value of Reservoir Contents
Potential for Loss of Life	0 - 75	Catastrophic index	Ratio of dam peak breach discharge to 100-yr flood
	0 - 300	Population at risk	Population at risk potential for future development
	0 - 100	Adequacy of warning	Likely adequacy of warning in the event of dam failure
Potential for Property Damage	0 -250	Items damaged or services disrupted	Residential and Commercial Property Roads, bridges, transportation facilities Lifeline facilities community services Environmental degradation from reservoir contents (Tailings, wastes, etc)

values. **Figure 4.2** defines the IDF and corresponding annual probability of exceedance as a function of the cumulative points, and **Table 4.12** shows the design steps that should be considered for each Dam Classification category.

To select the consequence points, the State of Washington Dam Safety Guidelines (Technical Note 2) provides a series of graphs and tables that help the engineer assign a point value to each of the consequences (Table 4.11). For instance, Figure 5 in Technical Note 2 (below), shows that a population at risk of 80 corresponds to a rating point of 150. The process is repeated for each of the consequences categories and the point values are computed.



Once the points are computed, the cumulative value is entered into **Figure 4.2**, which can be used as a quick estimation of the design steps. For instance, if the cumulative points add up to 500, then the IDF Probability is $5x10^{-5}$.

4.6 Canada

In Canada, Parks Canada Agency (PCA) owns and operates in every province (with the exception of Prince Edward Island) 255 dams and water-retaining structures most of which are managed by three Field Units. A consistent approach for the design, construction, inspection, and maintenance of the dams has been developed. This approach accounts for specific existing provincial dam safety requirements and is regarded as PCA standard. The standard contains many unique aspects, such as de-coupling of the direct relationship between classification and the dam safety parameter selection process and the specific recognition of lock structures.

Dams are classified based on five key concepts that include: incremental consequences; normal and flood failure scenario; potential for dam failure; transient losses; and third-party losses. The classification system uses a hazard-based system which allows for consequence calculations to better define appropriate design parameters. Dam safety evaluation criteria include life safety- persons at risk;

Table 4.13.- Parks Canada Agency (PCA) IDF Selection Table (Donelly et al., 2009)

Hazard Potential Classification	**Range of Inflow Design Floods for Life Safety Hazards[1]**			**Range of Inflow Design Floods for All Other Hazards[1]**
	Expected Loss of Life [1]	Transient Population at Risk	Range of Inflow Design Floods for Life Safety Hazards	
Very low	0	For major flood events, transient use would not be expected		25-yr flood to 100-yr flood
Low				100-yr flood
Significant				100-yr flood to 1,000-yr flood
High A	10 or less		1/3 between 1:1,000-yr flood and PMF	1,000-yr flood
High B	11 to 100		2/3 between 1:1,000 years and PMF or the 10,000-yr flood whichever is greater	
High C	>100		Incremental analysis or PMF	

[1] In general, transient persons that could be reasonably be expected to be subjected to incremental life safety hazards as a result of a dam breach to a random event such as an earthquake are treated as permanent population at risk

economic losses; infrastructure and public utilities; and environmental losses and cultural losses. Dam safety engineering parameters include inflow design flood and design basis earthquake. Another basic principle is that the standard of care applied to manage the safety of a dam should be commensurate with the potential consequences that would impact population and the environment at large, should a dam fail. The PCA hazard classification for dam safety management encompasses: (1) very low; (2) low; (3) significant; and (4) high. The PCA high hazard category is subdivided into three classes based on expected incremental loss of life, thus eliminating the direct relationship between dam classification and engineering parameter selection. The IDF is selected, as shown in **Table 4.13.-** Parks Canada Agency (PCA) IDF Selection Table.

4.7 Spain

Spain bases it classification of dams on the risk presented downstream of the dam. The guidelines apply to dams that fall in the category of "Large Dams," which, in the Spanish case is dams of over 15 m in height, or between 10 and 15 meters, with a crown length of 500 m and a spillway capacity of over 2000 m^3/s.

A key aspect of the Spanish classification is that it is entirely qualitative, putting considerable responsibility on the judgment of the engineer in charge of the classification. There are three categories A, B and C.

- Class A: Dam failure may seriously affect at least one urban nucleus or equivalent number of houses, or pose a risk to a number of human lives, or essential services of the community, or result in severe economic or environmental damages.
- Class B: Dam failure may affect a limited number of homes but does not pose a serious threat to any urban nucleus, nor does it result in significant economic damages or serious disruptions to any of the essential services to the community.
- Class C: Failure of dams by breaching causes incidental loss of human life; moderate damage

There are four components to be considered when assigning qualitative categories, A, B or C, namely:

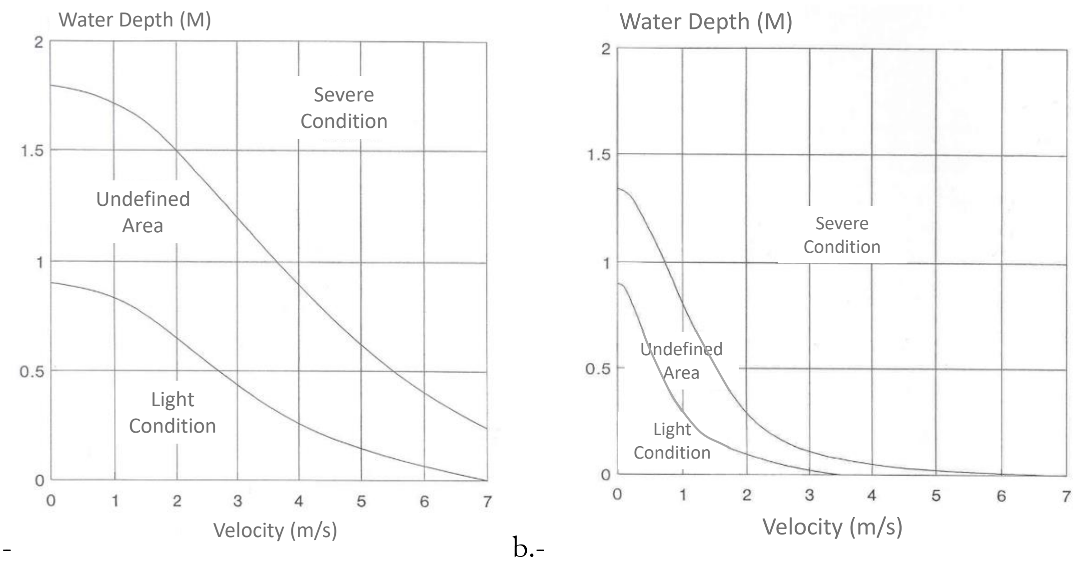
- Potential risk to human life. People at risk
- Impact to essential services
- Damage to property
- Environmental damage

Analysis is done for each of those four components individually, without considering any possible combination among them. The classification of the dam will be identical to the highest category determined for each of the components. So, if all of those components except one fall in Class C, but the exception is Class A, the classification of the dam will be the highest among the four, in this example, Class A.

To help decide the category for potential risk to human life, the Spanish Guidelines for Classiffying the Hazard Potential of Dams includes two charts (**Figure 4.3.a** and **Figure 4.3.b**) that estimate the qualitative risk to human life as a function of water depth and water velocity.

For the purpose of classification, "Essential Services" are those public services (water, energy, sewage treatment, communications and transportation) that are supplied to more than 10,000 people.

Damage to property is assessed with the help of Table II-1, which considers the following elements: Industry and rural properties; non-irrigated agriculture; irrigated agriculture; roads; and railroads.



a.- b.-

Figure 4.3.- Qualitative risk to human life as a function of water depth and water velocity (a.- urban areas and b.- open fields). (Adapted from DGOP, 1996)

Finally, environmental damage considers both impact to purely environmental aspects as to cultural heritage sites.

Spanish guidelines (SPANCOLD, recommend inflow design floods of 1,000, 500 and 100 years of return period for Class A, B, and C dams, respectively.. It also recommends Maximum Probable Floods of 10,000, 5,000 and 1,000 years of return periods for Class A, B and C dams of loose materials, and of 5,000; 1,000 and 500 years of return period for solid dams (concrete, masonry).

4.8 France

France does not explicitly use a risk approach for the classification of dams. It has a four level classification system (A through D), which is based on the height of the dam, and a heuristic factor which is a function of the square of the height in metres and the square root of the volume in hm3. This heuristic value is used as a surrogate for the downstream risk.

Table 4.14. Dam's Classification System in France

Class	A	B	C	D
H (m) and V (hm^3)	H ≥ 20	H ≥ 10 and $H^2\sqrt{V} \geq 200$	H ≥ 5 and $H^2\sqrt{V} \geq 20$	H ≥ 2

The French guidelines propose a double approach to the IDF for all dams above 50,000 m^3:

- Exceptional flood, in which the dam still has enough freeboard that protects the dam from waves, but that freeboard is smaller than under normal circumstances. The dam must meet all safety conditions regarding stability, erosion, etc.

- Extreme flood. This is reached at the point when the stability of the dam can't be guaranteed.

Existing dams are reviewed as to their condition regarding Extreme Floods, in order to assess the need to improve spillway capacity and/or reliability, or other measures. For exceptional situations, the guidelines treat dams differently regarding the type of dam: rigid or embankment.

The review of dams under exceptional floods should consider:

- Routing through the reservoir and upstream structures, with a reservoir level at its normal operating conditions
- Spillway operating under no restrictions
- The return period for the IDF for the Exceptional Case, for each dam type is:

Table 4.15. France's IDF for Exceptional floods

Dam Class	Rigid Dams	Embankment Dams
A	1000 to 3000	10000
B	1000	3000
C	300	1000
D	100	300

Class-A Rigid dams have some flexibility: existing dams need to meet the lowest value, and new dams the higher value.

The review of dams under the Extreme Flood case should consider:

- Normal flood routing through the reservoir and upstream structures, with the reservoir at the maximum operating level, normal spillway discharge
- Malfunctioning spillway
- The probabilities of exceedance for Extreme floods are shown below (Table 4.16):

Table 4.16. France's Probability of Exceedance for Extreme Floods

Dam Class	Annual overrun probability
A	10^{-5}
B	3×10^{-5}
C	10^{-4}
D	10^{-3}

4.9 Germany

The German norm is specified within the German Institute for Standardization (Deutsches Institut für Normung) DIN 19700 part 10 and part 11. German norms do not explicitly consider downstream risks, but one of the design floods indirectly takes those risks under consideration. These norms have a fairly complete set of procedures to assess the safety of dams. For additional details, please refer to the Bulletin 170 (ICOLD, 2018)

The DIN norm defines two classes of dams:

- Class 1, dams of more than 15 m height, or more than 1 hm^3 of storage.
- Class 2 dams, the rest.

There are three design floods:

1. Design flood 1, (BHQ1), for normal spillway design and safety, has a return period of 1000 years for Class 1 dams, and 500 years for Class 2 dams.
2. Design flood 2 (BHQ2), to make sure that structural safety is maintained, although it allows for damages to structure components, operating or measuring mechanisms
3. Design flood 3 (BHQ3) applies to the regular flood storage capacity. It is defined based on the downstream protection requirements

Application of the German norm requires the consideration of the following factors (See ICOLD Bulletin 170 for details):

- Design flood discharge
- Effect of retention
- Water level at the beginning of flood event
- Freeboard
- Pre-release (usually at the start of filling the exclusive flood control volume)
- Simultaneous release (after reaching the spillway crown)
- Flood alleviation (through the spillway)
- Emergency alleviation
- Resulting water level

4.10 Ireland

Ireland does not have a national norm or guidelines. Instead, dam users select what criteria to use. the Electricity Supply Board is a government utility that owns 10 of the 16 large dams in the country, and it follows its own standard.

The top priority is the safe operation of the dam to avoid any risk or safety of the dam. It has two dam categories: Category A, for those dams for which a breach will present danger to lives, and Category B for dams whose breach will not present a danger to human life.

The IDFs for Category A dams are:

- Ability to pass the 10000 year flood without overtopping the dam, with all the gates operating, and
- Ability to pass the 1000 year flood with one spillway gate unavailable and freeboard allowance for wave runup.

Category B dams must be able to pass the 1000 year flood with one spillway gate unavailable and with freeboard allowance for wave run-up.

All associated works (headrace canal, embankments, etc. need to meet those criteria.

To meet those standards, there were improvements to the spillways and/or downstream channel protection works.

4.11 Italy

Italy's guidelines require that spillways be dimensioned to pass the 1000-yr flood for concrete dams, and the 3000-yr flood for embankment dams. The regulating effect of the reservoir is taken into consideration. In addition, there is a freeboard requirement depending on the dam type and height

The freeboard for dams of intermediate height are computed by interpolation.

The magnitude of the flood is computed by standard probabilistic hydrologic techniques, using complete rainfall and runoff observations. In those dams for which there is no information, the calculation may be made using standard hydrologic transposition methods from neighbouring, hydro-climatologically similar basins.

There is also a requirement to estimate the return period of the flood that would leave no freeboard. In addition, the verification has to include flows with return periods of 50, 100, 200 and 500 years.

4.12 Japan

Japan classifies dams in two classes: Only those over 15 m are subject the Structural Standards for River Protective Facilities. The standards separate concrete from embankment dams, and considers freeboards due to wind, earthquake and type of gate operation.

Embankment dams require design floods 20% higher than those of concrete dams at the same location. The freeboard requirement for embankment dams is also increased between 0 and 1 m when compared to those required for concrete dams.

Table 4.17. - Japan - Estimation of freeboard (Hf). (Adapted from ICOLD, 2018)

Reservoir Water Level	Concrete Gravity Dam/Arch Dam	Embankment Dam
Normal high water level	$H_f = h_w + h_e + h_a$ $Hf \geq 2$	$H_f = h_w + h_e + h_a +1$ $Hf \geq 3$
Surcharge water level	$H_f = h_w + h_e /2 + h_a$ $Hf \geq 2$	$H_f = h_w + h_e /2 + h_a +1$ $Hf \geq 3$
Design flood water level	$H_f = h_w + h_a$ $Hf \geq 1$	$H_f = h_w + h_a$ $Hf \geq 2$

Where

H_f : Freeboard (m)
h_w : wave height due to wind
h_e : wave height due to earthquake
h_a : Allowance for gate operation (m). With gate 0.5 m. Without gate, 0 m

The IDF for concrete dams is calculated as the largest of these three criteria:

- 200-year flood at dam site
- Maximum flood discharge observed at dam site
- Maximum flood that can be expected based on basins with similar characteristics, computed according to Creager's formula.

For an embankment dam, the design flood will be 1.2 times the discharge values for a concrete dam.

Calculation of the maximum flood that can be expected, based on Creager's formula for specific discharge, is shown in Figure 4.4.- Japan - Specific flood discharge vs area (Adapted from ICOLD,2018)

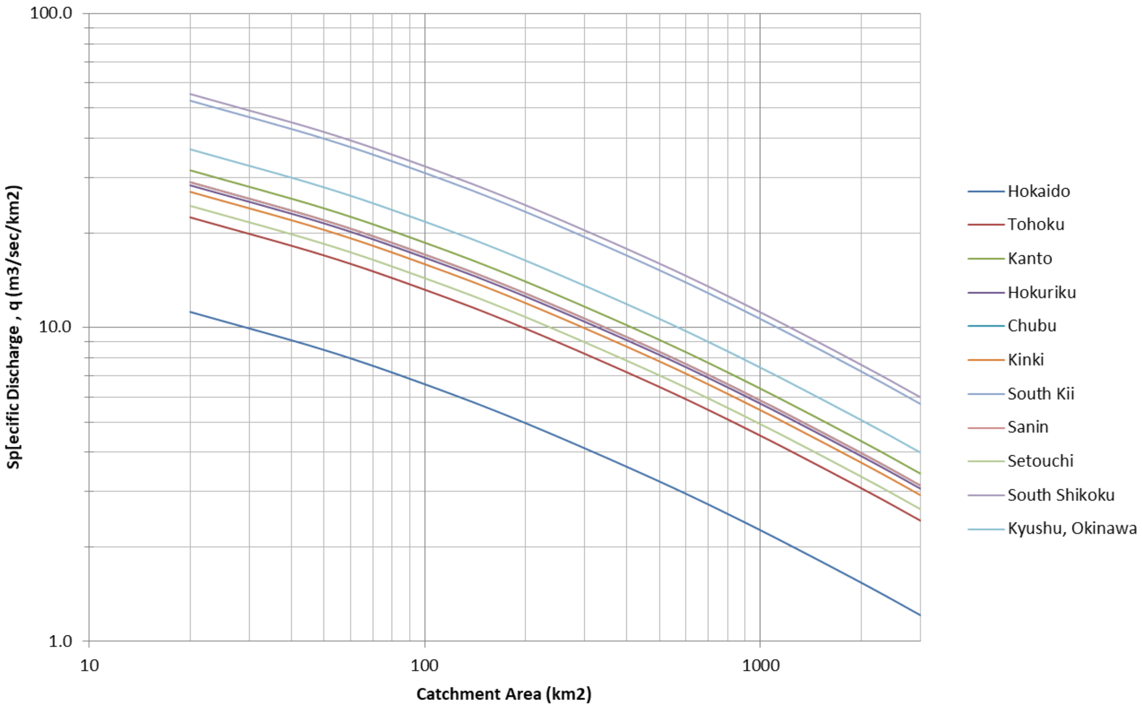


Figure 4.4.- Japan - Specific flood discharge vs area (Adapted from ICOLD,2018)

4.13 New Zealand

The New Zealand guidelines are intended for dams where the potential impacts of failure include loss of life and damages beyond the owner's property. Dams which would be classified in the very low category are generally outside the scope of these guidelines. The dam classification is based on the potential incremental consequence of a dam failure, i.e. on the number of fatalities and the socio-economic, financial and environmental impact.

The dam height and reservoir volume parameters, while useful for an initial screening of potential impact classification, should not control the potential impact classification where the consequences of a dam failure are not consistent with such an initial screening. For example, a 10 - 15-metre high dam whose failure can lead to fatalities should be classified with a high potential impact. Similarly, a 25-metre high dam whose failure would not cause fatalities and where damages are moderate can be classified as Low Potential Impact.

Table 4.18. - New Zealand Dam Classification

Potential Impact Category	Potential Incremental Consequences of Failure		IDF
	Life	Socio-Economic, Financial & Environmental	
High	Fatalities	Catastrophic damages	Between 10^{-4} Annual Exceedance Probability and PMF
Medium	A few Fatalities are possible	Major damages	Between 10^{-3} and 10^{-4}
Low	No fatalities are expected	Moderate damages	Between 10^{-2} and 10^{-3}
Very Low	No fatalities	Minimal damages beyond the dam owner's property	No requirement

4.14 Norway

Norway uses a risk-based approach for dam classification, except for those dams with very low height and volume. The classification and corresponding IDF requirements are shown on the table below. It uses an Inflow Design Flow, and a Safety Check Flood

4.15 Portugal

Portuguese Guidelines are specified in two Decree-Laws[1] [2]. Dam classification divides dams into Large Dams (Classes I, II and III), based on both physical characteristics of the dam and reservoir, and the potential impact downstream. It defines two variables, X and Y, where $X = H^2\sqrt{V}$ with H being the height of the dam in meters, and

Table 4.19. - Norway Dam Classification and IDF

Dam Class	Classification Criteria	Inflow Design Flood	Safety Check
0	$H < 2m$; $V < 10\,000\ m^3$ minimal consequence	200 year flood	Not Applicable
1	Low consequence (No permanent dwelling)	500-year flood	PMF or 1.5 x 500-yr flood
2	Medium consequence (1 to 20 dwellings)	1000 year flood	PMF or 1.5 x 1000-yr flood
3	High consequence (21 to 150 dwellings)	1000-yr flood	PMF
4	Very High Consequence (More than 150 dwellings)	1000-yr flood	PMF

V the volume of the reservoir in hm^3.

The IDF for small dams corresponds to a return period of 500 years, unless the volume of the reservoir is less than 100,000 m3. In that case, the IDF would be 100 years.

Table 4.20. - Large Dam Classification in Portugal

Class	Dam risk and potential damages
I	$Y \geq 10$ and $X \geq 1000$
II	$Y \geq 10$ and $X < 1000$ Or $0 < Y < 10$ independently of the value of X Or Impact to infrastructure, facilities, and important environmental assets
III	$Y = 0$, independently of the value of X

Table 4.21. - United Kingdom Dam Classification and IDF

Category	Consequence of a dam breach	Normal design standard	Minimum standard if overtopping tolerated	Initial reservoir condition	Minimum wave speed and minimum wave surcharge
A	Endangers lives in a community (more than 10 persons)	PMF	10000-yr flood	Spilling long-term average inflow	Mean annual maximum wind speed Minimum 0.6 m wave surcharge
B	Endangers lives of individuals or causes extensive damage	10000-yr flood	1000-yr flood	Full to spillway crest (no spill)	As Category A
C	Negligible risk to life and limited damage	1000-yr flood	150-yr flood	Full to spillway crest (no spill)	Mean annual maximum wind speed Minimum 0.4 m wave surcharge
D	No risk to life and very limited additional flood damage	150-yr flood	150-yr flood	Spilling long-term average inflow	Average annual maximum wind speed Minimum 0.3 m wave surcharge

4.16 United Kingdom

The UK norm defines four categories of dams, based on the consequences of a dam failure. Each category has both a normal design flood, and a minimum design flood if the dam characteristics allow for overtopping.

In addition, gated spillways must have a minimum of two gates. If one of them is not operating, the remaining gates must be able to release the 150-yr flood. All gates of Class A must be automated.

4.17 Selection of the IDF. International Comparison

In Bulletin 170 of ICOLD, an example was prepared to compare the various approaches used in different countries for the evaluation of the IDF. The proposed example (Project β) is representative of a large dam, but not large enough to make the IDF the maximum expected in each country. (ICOLD, 2018)

Based on the system's characteristics of the proposed example and the consequence of a dam failure, the IDF was estimated for each country, the inflow design floods vary from the 1:100-yr flood to the PMF. **Figure 4.5** illustrates the results obtained for each country. The figure shows that the majority of the IDF values for the majority of the countries stands between the 1:1.000-yr flood to the 1:10.000-yr flood. However, there is no clear trend in the results, since there is almost the same number of countries for which the IDF will be the 1:1.000-yr flood, the 1:10.000-yr flood or the PMF. About 10% of the countries will consider an IDF lower than the 1:1 000-yr flood and about 20% of the countries will use the PMF.

It should also be noted that the IDF is not necessarily the only design parameter considered in the design. For example, in South Africa, the IDF correspond to the 1:100-yr flood, but the Safety Evaluation Discharge (SED) is also used to evaluate the adequacy of the spillway system of a new or existing dam under extreme flood conditions. Substantial damages may result from the occurrence of the SED, but the design must be such that the dam will not fail. In this case, the SED will exceed the 1:1000-yr flood.

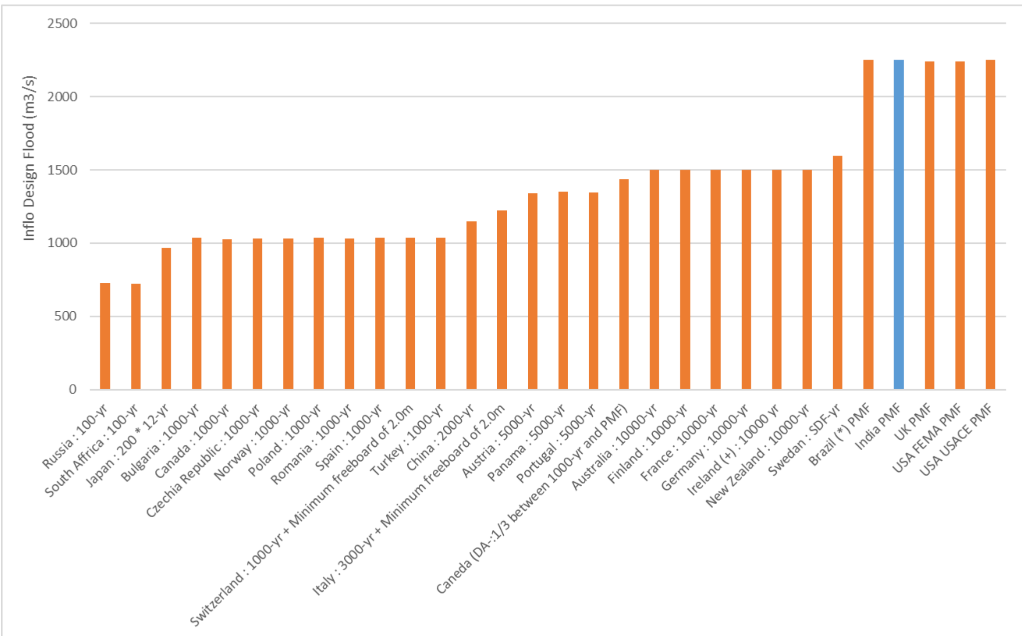


Figure 4.5.- Project β - Inflow Design Flood Comparison - Base case (Adapted from ICOLD,2018)

Table 4.22. - Comparison of the Characteristics Considered To Evaluate The Design Flood (Adapted from ICOLD, 2018

Country	System characteristics				Consequences of dam failure						Design flood		Check flood	Free board
	Height	Volume	Type of dam	Perma-nent/ temporary	LOL	PAR	Eco-nomic	Social	Environ-ment	Flooded area	Min	Max		
Australia						X	X				100-yr	PMF		
Austria	X	X										5 000-yr		
Brazil	X	X			X						1 000-yr	PMF		
Bulgaria			X	X							33-yr	10 000-yr		
Canada					X		X		X		100-yr	PMF		
Canada-Quebec						X	X				100-yr	PMF		X
China	X	X	X	X			X				100-yr	10 000-yr	X	
Czech Republic					X	X	X		X		20-yr	10 000-yr		
Finland					X	X			X		100-yr	10 000-yr		
France	X	X	X								1 000-yr	10 000-yr		
Germany	X	X									1 000-yr	10 000-yr		X
India	X	X									100-yr	PMF		
Ireland					X						1 000-yr	10 000-yr		
Italy			X								1 000-yr	3 000-yr		X
Japan			X								200-yr	1 000-yr		X
New Zealand					X		X				100-yr	10 000-yr		
Norway						X					500-yr	1 000-yr	X	X
Panama					X		X				100-yr	5 000-yr		
Poland	X	X				X				X	200-yr	1 000-yr		
Portugal	X	X			X		X					1 000-yr		
Romania	X	X					X				100-yr	10 000-yr		
Russia	X	X	X				X				20-yr	1 000-yr	X	
South Africa	X				X		X				1200-yr	6 000-yr	X	
Spain						X	X	X			100-yr	1 000-yr	X	
Sweden						X	X		X		100-yr	SDF		
Switzerland	X	X	X								1 000-yr	1.5x1 000-yr	X	X
Turkey	X	X	X								500-yr	PMF	X	X
UK					X	X	X				150-yr	PMF		
USA/FEMA					X		X		X		100-yr	PMF		
USA/USBR					X						100 yr	PMF		

A summary table of the criteria for each country and the corresponding PMF for the Project β is presented in **Table 4.22**

4.18 Discussion

Even though all the countries' regulations examined in this Guidelines are considered prescriptive approach (based on dam hazard classification), the examination of the international practices reveals that with few exceptions, countries updating their guidelines for selection and accommodation of the IDF are now moving away from physical characteristics alone, to consider also the risk posed by the failure of the dam.

In particular, the guidelines of Washington State have a feature that is different from the rest. In addition to having an explicit consideration of the risk posed by a dam failure, those guidelines follow a point approach to define the IDF. That approach solves a problem of selecting an IDF based on a discrete number of thresholds for each category, which causes considerable discontinuities in the definition of the IDF. To illustrate this point, let's look at the Chinese guidelines.

Let's assume we have two projects, A and B, both concrete dams. Project A has 900 hm^3 of storage capacity, with "moderately important" cities and industrial areas downstream, 20,000 ha of flood protection, 2,000 ha of water logged areas, 100,000 ha of irrigated land, less important water supply to cities and mines, and one installed capacity of 25MW.

Project B, on the other hand, has 990 hm^3 of storage capacity, provides flood protection for important cities and industrial areas and for 330,000 ha of land, 133,000 ha of water logged areas, 99,000 ha of irrigated land, supplies water to important cities and mines and has an installed capacity of 740 MW.

According to the Chinese scheme, Project A would be in Class I, because it provides irrigation to 100,000 ha of land. Project B would be classified as Class II, because none of the criteria reaches the threshold set for Class I, even though all its criteria exceed the corresponding criteria of Project A, and is lower only on the irrigated land. Therefore, Project A would have an IDF with a return period of 5,000 years (or PMF), and project B would have an IDF with a return period of 1000 years, despite having clearly considerably a more serious impact than project A.

Chapter 5. Selecting the Inflow Design Flood. The Proposed Framework

5.1 Overview

The guidelines presented here are intended to provide a balance between the objectives mentioned in *Section 3.1 (Different Approaches Same Objective*). Where that balance is obvious, a straightforward and efficient prescriptive approach based on the hazard potential classification may suffice. For dams for which there are significant trade-offs between the consequences of failure and the cost of designing to the recommended prescriptive standard, the guidelines provide alternatives for more rigorous and detailed analytical investigations to evaluate the potential for selecting a lower IDF while reducing risks to the public. In other words, an alternative to the simplified prescriptive approach is appropriate where an investment in more precisely understanding the trade-offs in the selection of an IDF can result in better use of resources. Advanced methodologies such as incremental consequence analysis, or risk-informed hydrologic hazards analysis are described in *Chapter 3 and Chapter 2* and should be used at the discretion and judgment of dam safety regulators and owners. This approach includes such provisions in an effort to strike a balance between what is theoretically desirable and what is possible with existing technologies

5.2 Framework for IDF Selection in New Dams

Traditionally, the first step in the selection of the IDF for a new project has been to classify the project among a number of classes. Originally, these classes were exclusively based on physical characteristics of the project, typically dimension of the dam and, in some cases, reservoir volume, regardless of the consequences downstream. The thinking was that large dams will cause heavy damage and, therefore, the higher the class, the higher the IDF. As was shown in the review of international guidelines, agencies in charge of developing guidelines started to include the risk associated with a project. In fact, projects that in the past, would not qualify for the top class, based solely on its physical characteristics, could now, based on the risk associated with a major failure, be required to pass a much larger IDF. Conversely, a large dam and reservoir that pose little damage, should it fail, could have a reduced IDF.

Consider, for example the case of two projects. Assume that there is a guideline that classifies projects using only dam height, and that the cut-off between class I and class II is 50 m. Dams with 50.5 m height will be class I, and dams of 49.5 m will be class II. Assume also that class I requires an IDF with an annual exceedance probability, AEP = 10^{-4}, and that Class II requires an IDF with an AEP = 10^{-3}. Without looking at the risk associated with a dam failure, using the physical-alone approach, the cut-off between class I and class II implies one order of magnitude difference in the AEP of the IDF for only 1 m difference in dam height.

Therefore, in addition to the physical size of the dam, these guidelines incorporate downstream risks and hazard potential into the calculation of the IDF. The approach is similar to the one adopted by the State of Washington in the US, in the sense that it uses a system of assigning points to all conditions to eliminate the jump in calculating the IDF among different classes and also includes topics pertinent to the conditions in India, and discards those not relevant to India.

For a detailed description of the hazard categories included in the additive weighting scheme developed for India, as well as the step-wise procedure in how to estimate the potential consequences index (PCI), please refer to the "*Guidelines for Classifying the*

Hazard Potential of Dams", published as part of the same series. **Table 5.1** in page 56 (reproduced from the above-mentioned Guideline) summarises the Hazard Classes and corresponding potential consequences index (PCI) thresholds.

Based on the hazard classification categories and potential implications described in the **Table 5.1**, five principles were established in order to select appropriate IDF values for each hazard category:

I. Especial emphasis was given in reducing the flood risk to the public and developed areas located downstream of the dam.

II. The PMF or maximum theoretical event may be adopted as the IDF in those situations where the consequences attributable to dam failure due to floods conditions less severe than the PMF are unacceptable. The determination of unacceptability clearly exists when the area prone to be affected is evaluated and indicates there is a potential for loss of life and/or extensive property damage, even in the best scenario of emergency management.

III. A flood event less stringent than the PMF or maximum theoretical event may be adopted as the IDF in those situations where the consequences of dam failure due to flood conditions much more severe than the selected IDF are acceptable. In other words, when the hazard potential and associated studies indicate that the risk is mainly associated to the dam owners' facilities.

IV. Consideration of whether a dam provides vital community services such as municipal water supply or energy. Therefore, a higher degree of protection may be required against failure to ensure those services are continued during and following extreme flood conditions when alternate services are unavailable. If the economic risk of losing such services is acceptable, the IDF can be less conservative.

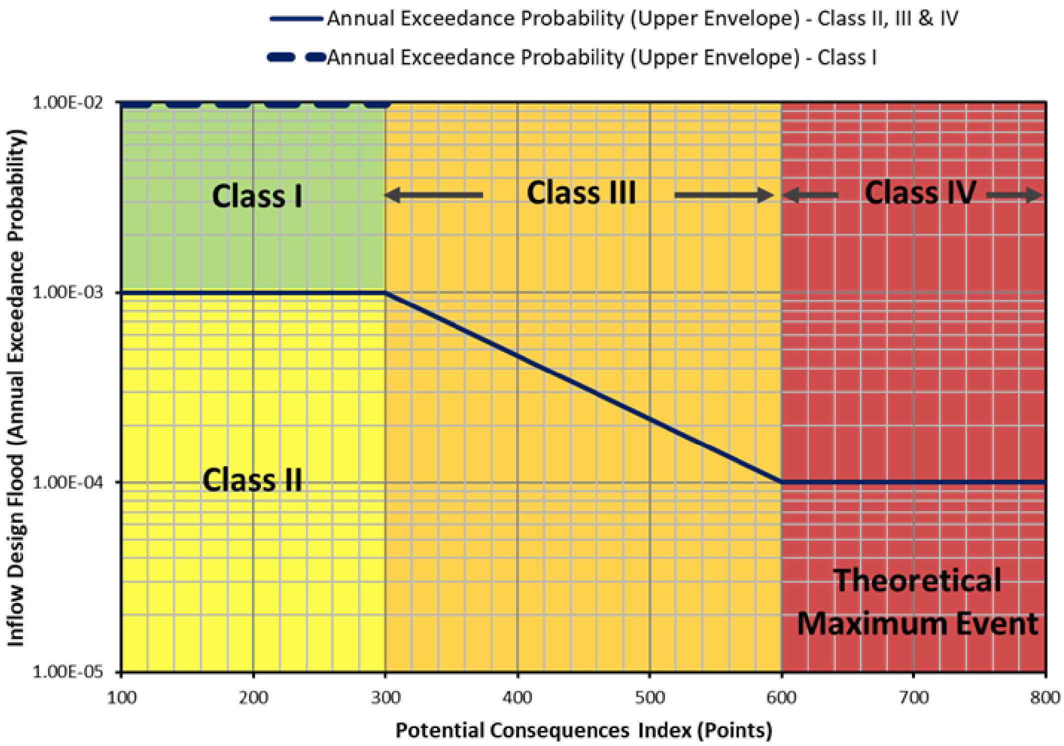


Figure 5.1.- Recommended maximum Annual Exceedance Probability of the Inflow Design Flow using only the Dam Hazard Potential Classification as selection approach.

Table 5.1.- Proposed Dam Classification based on the Additive weighting Scheme (Potential Consequences Index)

Hazard Potential Class	**Potential Consequences Index (P_{CI})***	**Consequences Categories**			
		Capital Value of Project	Potential for Loss of Life	Potential for Property Damage	Potential for Environmental and Cultural Impact
Class I	< 300	Low	None. Occasional or no incremental population at risk, no potential loss of life is expected. No inhabited structures.	Minimal. Limited economic and agricultural development.	None
Class II	< 300	Average	Minimal or low population at risk. No potential loss of life is expected even during the worst-case scenario of emergency management	Notable agriculture or economic activities. States highways and/or rail lines.	Minimal incremental damage. Short-Term or reversible impact (less than 2 years)
Class III	$300 < P_{CI} < 600$	Significant	Considerable. several inhabited developments. Potential for loss of life highly dependent of the adequacy of warning and rescue operations.	Significant industry, commercial and economic developments. National and state highways and rail lines.	Limited. Impact have a mid-term duration (less than 10 years) with high probability of total recovery after mitigation measures
Class IV	> 600	Critical	Extreme. High density populated areas. Potential for loss of life is too high even during the best scenario of emergency management	Highly developed area in terms of industry, property, transportation and lifeline features	Severe. long-term impact/effects in the protected areas or cultural heritage sites with low probability of recovery.

* Disclaimer: Dams with total consequences index near the boundaries between two classes (+/- 50 points) warrant a comprehensive assessment and additional engineering judgment to determine the actual hazard classification.

V. IDF values between hazard categories should follow a continued pattern and avoiding jumps of more than one order of magnitude between classes' thresholds. This principle would prevent radical differences in design standards among dams with similar characteristics but contiguous hazard classes.

These five principles were used to develop the recommendations summarised in **Figure 5.1** for selecting the annual exceedance probability of the IDF. The IDF's recommended return period can either be estimated graphically using **Figure 5.1** or analytically by using equation given below

$$F = \begin{cases} 10^{-2}, if\ p\ \leq 300\ and\ Class\ I \\ 10^{-3}, if\ p\ \leq 300\ and\ Class\ II \\ 10^{-(p/300)-2},\ \ if\ 300 < p < 600 \\ 10^{-4}, if\ p \geq 600 \end{cases}$$

where:

$p =$ potential consequences index (PCI) or total hazard score for the project, after the hazard classification process

$F =$ Recommended (maximum) annual probability of exceedance (APE) for the IDF.

Table 5.1 below intents to illustrate the application of the prescriptive (hazard classification-based) approach into the selection of the annual exceedance probability of four hypothetical new projects with different hazard classes. **It worth to mention that when only the prescriptive approach based on dam hazard classification is used to select the IDF, recommended values should be considered as the "upper bounds", which could be crossed towards less stringent values only when are supported by more advance selection techniques/studies such as the incremental consequences analysis and/or a risk-informed decision-making process (See *Chapter 3*)**

In principle, a selection approach based solely on a dam hazard classification may result sufficient for new projects under design stage. This is because the design standards as consequence of a higher IDF could be updated and met as part of the design process itself, where it can be easier to satisfy or balance the multiple project's objectives at the same time.

However, and as discussed in previous chapters, no single approach for the IDF selection is 100% adequate for the infinite situations of thousands of dams under planned stage. Therefore, in those cases where the financial or physical limitations of the project made impossible to satisfy 100% the prescriptive standards given by the hazard classification, an equivalent or risk tolerable IDF could be selected through a more rigorous analysis. In that context, the estimation of the annual exceedance probabilities through a proper frequency analysis would serve as key inputs for a subsequent risk-informed hydrologic safety assessment.

Table 5.2.- Sample estimation of the annual exceedance probability for the IDF of 4 hypothetical Dams with different hazard classes

Dam Name	Hazard Class	Potential Consequences Index (PCI)	Recommended Annual Exceedance Probability for the IDF
Blue Dam	Class I (no potential for loss of life)	150	10^{-2}
Red Dam	Class II	230	10^{-3}
Orange Dam	Class III	490	$10^{-3.6}$
Green Dam	Class IV	800	10^{-4} (or PMF)

5.3 Framework for IDF Assessment in Existing Dams

Many existing dams were constructed prior to the development of existing guidelines and/or regulations for safely passing a suitable IDF. Additionally, many of these dams were designed using hydrologic information or technologies that differ from those that are now available. For these reasons, existing dams often do not meet current regulatory IDF requirements.

In order to equitably address public safety, which is the primary goal of establishing IDF guidelines, the same criteria for selection of IDF should be applied on all the dams across the country, irrespective of its date of construction or years of service already provided. In other words, the concept of "grandfathering" (i.e. evaluate the dam safety using obsolete standards accepting less stringent design parameters just because the dam is old) should not be considered at any form, especially when no downstream risk assessment encompass such decision.

Guidelines and/or regulations should include considerations of safety and risk (e.g. hazard potential and incremental risk reductions to be achieved) when determining whether dam owners are required to upgrade existing dams to comply with updated regulatory requirements. Even if a regulator decides not to require upgrades to a dam to fully meet new conditions, there may be cost-effective alternatives for partially upgrading the dam and lowering the risk exposure of downstream populations below commonly accepted levels of risk tolerance which should be considered.

When dam owners have made a good faith effort to accommodate an appropriate IDF based on the applicable engineering practice, hydrologic data, and regulatory guidelines in place at the time the dam was designed and constructed or rehabilitated, a new regulatory guideline related to spillway discharge capacity or new hydrologic information may not be sufficient by itself to require the dam owner to modify their dam to meet the revised regulatory guideline. Several principles should be considered before requiring a dam owner to apply updated guidelines to an existing dam.

- If significant modifications are otherwise required to the dam and appurtenant structures, the IDF should be updated to reflect the new guidelines and/or hydrologic data.

- If the IDF for an existing dam is not in accordance with current guidelines and hydrologic data, consideration should be given to the risk exposure of the population downstream of the dam. If the risk exposure exceeds commonly accepted levels of risk tolerance, a new (or equivalent) IDF should be established and the dam should be modified to accommodate the new IDF.

- If the IDF for an existing dam is shown to be inadequate to address the guidelines, known hydrologic conditions, or commonly accepted engineering practices in place at the time of design and construction of the dam, the IDF should be revised to meet current guidelines and the dam should be modified as necessary to accommodate the new IDF.

Based on these principles a tiered framework is being proposed in this section to guide dam's authorities in the selection of the revised IDF for existing dams. This tiered framework is at same time based on a progressive refinement of the rigorousness, level of detail and resources required for the IDF selection process.

The level of analysis of the tiered framework presented here (Table 5.3) correlates the sophistication and accuracy of the analysis with the scale, complexity and public safety required for the dam and downstream area under investigation.

Table 5.3.- Tired Framework to assess and select the revised IDF in existing Dams

Tier Level	IDF Selection Methodology	Applications	Advantages	Challenges	Inputs required	Outcome
Tier 1 – Basic and fundamental level	Hazard Classification Based Approach	• All type of dams as screening level, irrespective of their size and hazard class • Generally acceptable and sufficient for low/intermediate hazard dams (Class I and II) if the revised IDF can be easily accommodated. • When the potential consequences index (PCI) is clearly defined within a specific hazard class with not many uncertainties	• Simple and efficient • Effectiveness of resource utilization • Easily accepted by the public due for its conservative result	• Conservative outcomes which, in some cases, are not workable • Dam failure assumptions are based on the worst downstream inundation scenario rather than actual physical conditions. • Uncertainty not considered	• Potential Consequences Index (PCI) given by the hazard classification process	Conservative Inflow Design Flood
Tier 2 – Evolved and intermediary level	Incremental Consequences Analysis	• Potential Consequence Index (PCI) fall within the threshold's tolerance between two classes (+/- 50 points) • Small Reservoir's volume compared to the inflow hydrograph volume • Dams that could not afford prescriptive standards given by the hazard classification-based approach, mainly due to financial or site-specific constraints	• Evaluates incremental consequences more precisely • Marginal reductions of flood risk without spending limited resources on conservative designs • Wide range of extreme flood magnitudes considered	• Additional investigations and more advanced analytical tools and methods required • A comprehensive iterative process is not usually preformed • Uncertainty associated with the analysis is not quantified • Generally, not an enlightening and efficient approach for large reservoirs	• Iterative Dam break analysis scenarios • Evaluation of consequences for the entire floodplain for each iteration.	Inflow Design Flood with marginal risk reduction and usually lower than IDF selected by prescriptive approach, especially in small reservoirs
Tier 3 – Advanced and Rigorous level	Risk-informed Hydrologic Hazard Analysis	• All type of dams as part of the risk-informed dam safety management programme in the country • Usually required for Intermediate/High Hazard Dams (Class III and IV) as priority • Dams that could not afford prescriptive standards given by the hazard classification-based approach, mainly due to financial or site-specific constraints	• A range of hydrologic loading conditions and their corresponding risk • Uncertainty can be quantified • Emergency preparedness can be considered to reduce risk	• Recourses and time consuming • Some dam's authorities may face technical challenges such as: lack of reliable data, limited expertise available, deterministic mindset • Could be subjective in several aspects, which required good engineering judgment from experts	• Potential Failure mode Analysis • Quantitative Risk Analysis • Tolerability Guidelines	Prioritisation of Risk reduction measures which make the dam Hydrologically safe under the tolerable risk levels in the Country. Generally, an "equivalent" IDF is obtained which is much lower than the prescriptive standards

As part of the Dam Rehabilitation and Improvement Project (DRIP), more than 223 existing dams have been hydrologically assessed and updated inflow design floods have been obtained following the prescriptive approach given by the Bureau of Indian Standards (BIS) in IS: 11223-1985, which, as mentioned in *0*, is based only in the dam's characteristics (height and volume). As result of the updated hydrological studies, significant differences between the revised and original flood were encountered among the portfolio of dams. As can be seen in

Table 5.4 and **Figure 5.2- Ratio "Revised/Original" IDF for DRIP dams, more than 85% of dams under scrutiny increased their design floods on 100% in average (ratio revised IDF/ original IDF = 2**). As can be inferred, these increments represent a challenge for many of dam's owners since would be virtually impossible to several of them to adjust the dam's conditions to meet the new design standards.

Furthermore, the revised IDF under DRIP was carried out using the current prescriptive approach (dam's characteristics based). When a consequences and hazard-based approach was used in selected 23 Dams, some differences were obtained in few dams (See **Figure 5.3** and **Figure 5.4**).

Figure 5.2- Ratio "Revised/Original" IDF for DRIP dams (Adapted from *Manual for Assessing Hydraulic Safety)*

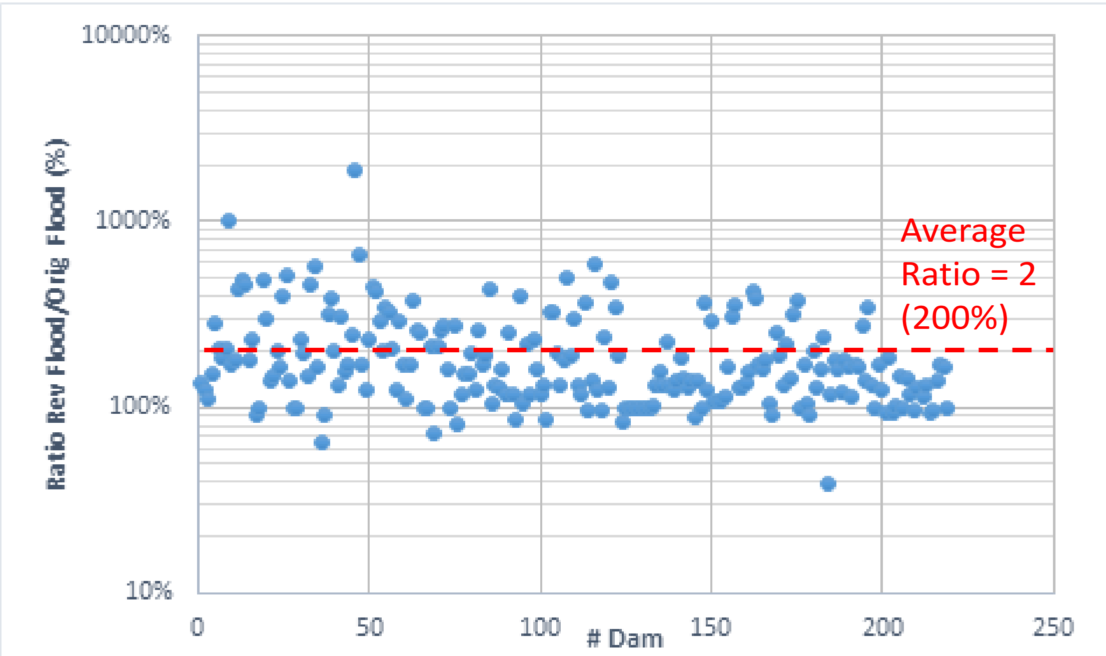


Table 5.4.- Inflow Design Flood Increment (% Percentage) in DRIP dams

	> 5 times (400% increase)	> 4 times (300% increase)	> 3 times (200% increase)	> 2 times (100% increase)	> 1.5 times (50% increase)
No. of Dams	7	18	37	68	115
Dams Percentage	4%	10%	20%	37%	63%

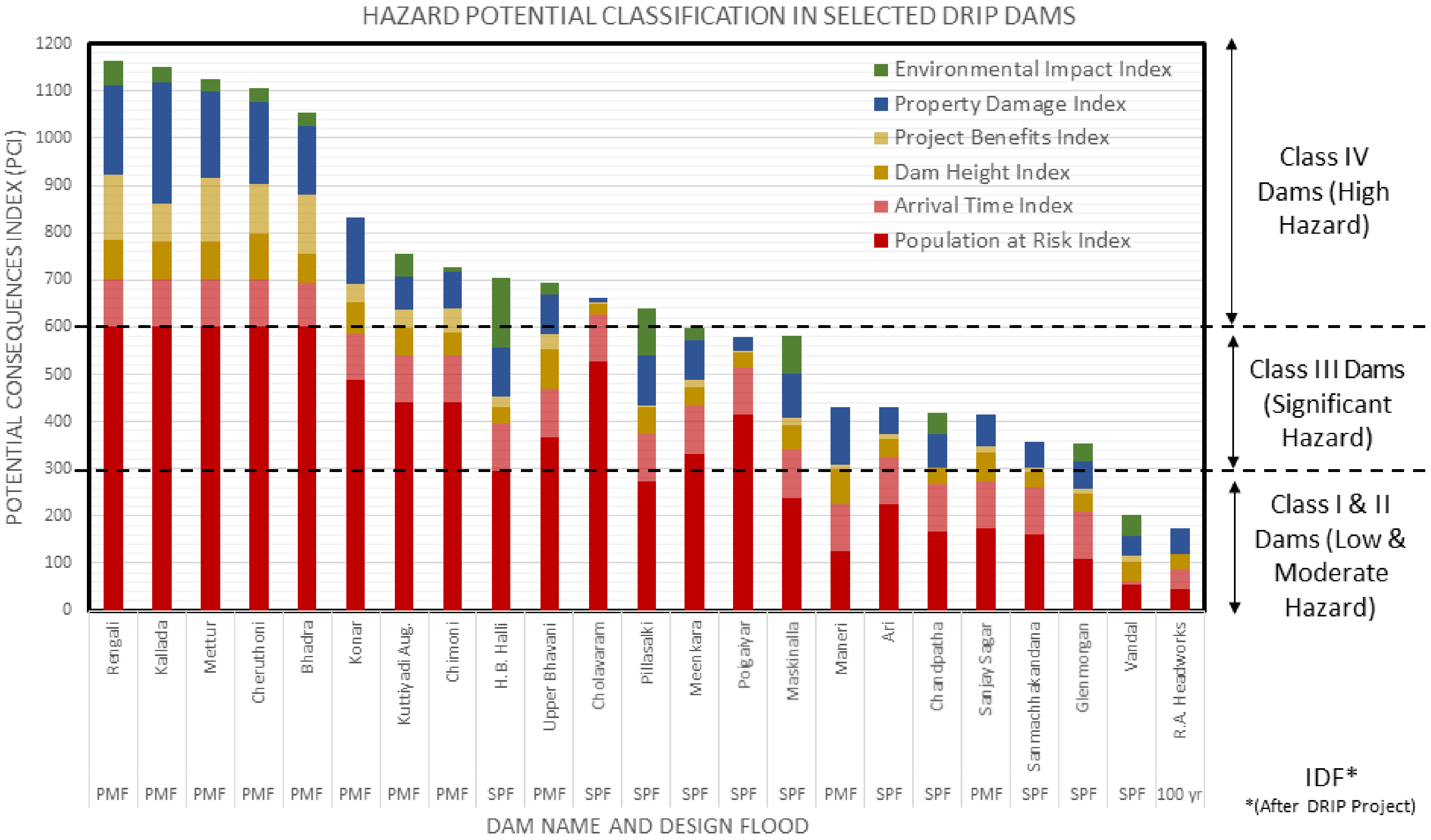
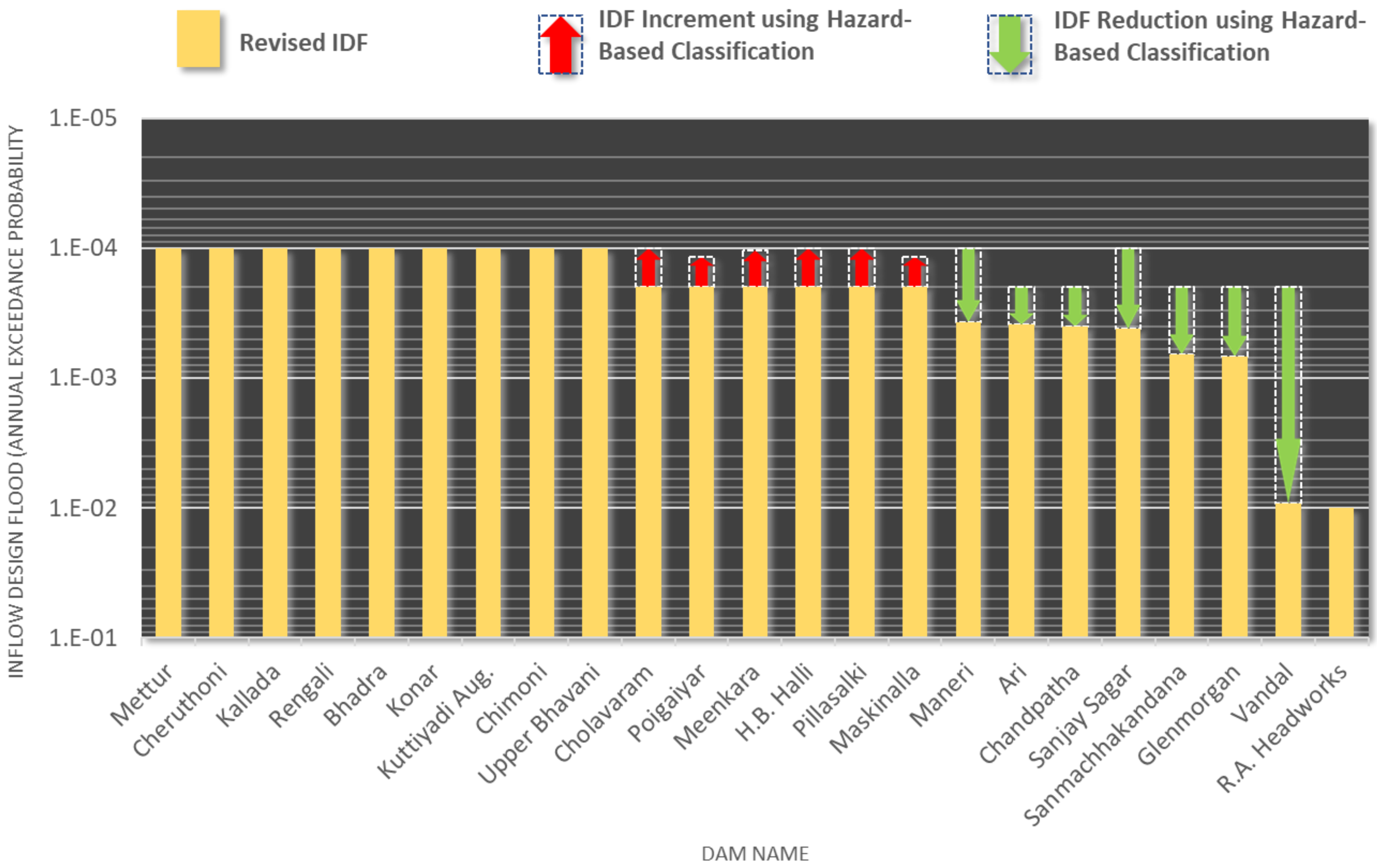


Figure 5.3.- Total Potential Consequences Index (P_{CI}) and Hazard Classification in selected 23 DRIP Dams (Adapted from Hazard Classification Guideline)



Note: Only for comparison purposes, PMF and SPF were assumed to have an annual exceedance probability equal to $1x10^{-4}$ (10,000 years return period) and $2x10^{-4}$ (5,000 years return period), respectively.

Figure 5.4.- Selection of IDF's annual exceedance probability in selected 23 DRIP Dams using the proposed Hazard-based Classification.

The limited case studies shown in Figure 5.4 give grounds for the following conclusions:

- In a preliminary and screening stage (Tier-I), a prescriptive approach, either dam's characteristics-based or consequences-based, could be considered sufficient for IDF selection in evident and unquestionable high-hazard, and exceptionally large dams. Further refinement in the IDF selection may be explored only through a risk-informed hydrologic hazard assessment and when the dam-specific conditions make unfeasible to upgrade the dam to the new loading conditions.

- Consequences-based IDF selection method (Hazard Classification) demonstrates to be a more suitable approach over the current prescriptive approach currently followed in India. By using a prescriptive approach, dams with high hazard potential are designed with less stringent standards only because their size (e.g. Cholavaram dam, Tamil Nadu); on the other hand, Dams with low hazard potential are designed with unjustifiable stringer standards due to its increased size (e.g. Vandal Weir, Tamil Nadu)

If the IDF selected by any of the approaches described in Table 5.3 are bigger that the current spillway's capacity, either the inflow volume has to be accommodated within the reservoir, or it has to be passed safely downstream. This would imply the implementation of structural and non-structural measures in the dam. The most common of these measures are discussed next in *Chapter 6* of this guideline

Usually, while using a Tier-I or Tier-II approach, these measures aims to fully comply with the new design standards by matching the spillway's capacity with the selected IDF. However, using a Tier-III approach, it allows dam owners to implement a trade-off process where a balance between flood risk reduction measures, residual risk and capital investment is found. Generally, the main outcome of this trade-off process is an "equivalent" IDF (fraction of the prescriptive IDF), which when implemented along with others risk reduction measures, are considered enough to lower the dam's risk within the tolerability thresholds.

5.4 Consideration of Future Development

Selection of an IDF for a new dam or a dam undergoing significant modifications should take into account both current conditions and reasonably anticipated future development. This is especially important when the hazard classification of the dam is low or significant and when the IDF selection is based on methods that are dependent upon the magnitude of the downstream consequences.

Primary sources of information for future development are local land use planning organizations. Upstream development in the watershed should be considered to the extent that it could alter the hydrologic characteristics used in determining the IDF for the dam. Downstream development should be considered to the extent that it could alter dam hazard classification and/or estimates of risk that may indicate a need for a larger IDF.

If there are uncertainties about future development, the dam owner may wish to consider designing the dam to include provisions for accommodating a larger IDF with minimal additional investment. Choosing designs that can substantially increase storage or discharge capacity with minimal investment may result in considerable savings if a larger IDF becomes necessary at a later date.

Chapter 6. Accommodating The Selected Inflow Design Flood

6.1 Introduction

In addition to selecting an inflow design flood, the accommodation of the IDF is critical to the hydrologic safety of a dam. IDF accommodation may include the consideration of structural measures such as spillway and outlets design, provision of parapet walls (freeboard criteria), overtopping protection designs, heightening of the dam, raise of gate's height, among others. Spillways and flood outlets should be designed to safely convey major floods to the watercourse downstream of the dam. They are selected for a specific dam and reservoir on the basis of dam safety, dam type and purpose, release requirements, topography, geology, project economics, and other possible factors.

Non-structural measures could be also considered along with measures mentioned above

It is recognized that the procedures and design criteria for IDF accommodation vary significantly based on these and other factors. The guidelines contained in this document are not intended to provide a complete manual of all such criteria but are intended to act as a basis on which dam safety agencies can develop criteria suitable to their varied objectives, jurisdictions, and resources. The underlying philosophy and principles are described in sufficient detail to promote a reasonable degree of consistency and uniformity among state and central agencies in the design or evaluation of dams from the standpoint of hydrologic safety. **For further details in how to assess the hydraulic and hydrologic safety in dams as well as recommended flood risk mitigation measures, following Guidelines and Manuals should be examined in parallel with this document:**

- Manual for Assessing the Hydraulic Safety of Dams (all chapters)
- Guidelines for Assessing and Managing Risks Associated with Dams (all chapters)
- Manual for Assessing the Structural Safety of Existing Dams (Chapter 2, 3, and 8).
- Manual for Rehabilitation of Large Dams (Chapter 7)

Due to the importance of safely accommodating the IDF and floods of lesser magnitude, all designs and analyses should be performed, or directed and reviewed by an engineer experienced in hydrology and hydraulics.

6.2 Increased IDF in Existing Dams

Many of the large reservoirs in the country were planned and constructed immediately after the independence in 1947. These dams, representing a vast majority of dams in the country, are more than fifty years old today. In those times, the availability of hydrological data was limited. The methods available for computation were also crude. Worldwide, the understanding about the meteorological and hydrological processes that lead to floods had undergone tremendous advancement in the intervening period, as more and more recorded information about the flood occurrences became available. The computational capabilities also progressed in leaps and bounds

These aging dams, however, are integral and indispensable components for the achievement of water security of the country, flood control, and hydropower generation. Consequently, the need to re-estimate the design floods of these existing dams following the current practices arises to ensure the safety and security of the population who are living in the downstream areas of these dams. As was explained in *Section 5.3* of this document, the Dam Rehabilitation and Improvement Project's (DRIP) experience has shown that the re-estimated design flood, more often than not, increases beyond its initial design value – sometimes even by a number of times. It poses a difficult challenge to accommodate this increased flood peak in an existing dam, as construction of an additional spillway or more spillways to augment the capacity of spillage is generally associated with a number of problems:

- Geologically competent location for placing an additional spillway is difficult to find.
- Allocation of area for the construction of channel downstream of the spillway to carry the discharge back to the stream often requires significant rehabilitation and resettlement.
- The construction calls for emptying/ lowering of the reservoir –forcing a compromise of water supply.
- The demand for capital outlay is often huge.
- The increase of spillway discharge capacity from an upstream dam is sometimes coupled with an increase in chance of bank overflow and flooding downstream, as the dwindling carrying capacity of the river and the ever-increasing trend of occupation work together to constrict the flow channel over the years.

However, the security of life for those residing downstream of a dam must be safeguarded without fail, matching with the standards being followed worldwide. This would call for judicious selection and application from a number of available strategies, or a combination thereof. The current chapter attempts to present a discussion on different possibilities. **The final choice for a particular dam has to be made on a case-by-case basis, as each dam is envisaged to pose a challenge that is as unique as the dam itself.**

In our country, many of the floods causing high devastation take place during the end of monsoon season. Moreover, for many of the reservoirs there is no dedicated flood storage space. As most of the reservoirs are planned for within the year-storage, the end of monsoon is also the time when the reservoirs are at their full capacity. So, the mega flood, more probably than not, will impinge on the full reservoir level. **The hydrological safety of the dam against breach may be secured through adoption of either structural measures (e.g. constructing additional spillway) or non-structural measures (e.g. implementing effective flood warning system) or a combination of both**. Some of these measures are further discussed in this chapter.

6.3 Routing the Inflow Design Flood

Site-specific considerations should be used to assign flood routing criteria for each dam and reservoir. The criteria for routing any flood should be consistent with the reservoir regulation procedure to be followed in actual operation. The general guidelines to be used to set flood routing criteria are presented below and should be used if applicable.

6.3.1 Guidance for Initial Reservoir Elevation

In general, if there is no allocated or planned flood control storage, the flood routing begins with the reservoir at the "normal maximum pool elevation" or "full reservoir level" (FRL). If regulation studies show that pool levels should be lower than the normal maximum pool elevation during the critical IDF season, then the results of those specific regulation studies should be analysed to select the appropriate initial pool level for routing the IDF.

However, enforcement of lower maximum pool level throughout the flood season when floods of high magnitudes are anticipated is more challenging to implement on the ground (See ***Section 6.6.2.- "Optimising the Reservoir Rule Curve").*** This is because of consequent loss in power generation (in case of dams with hydropower generation as the primary purpose) and loss of area / depth of water supply for irrigated agriculture (in case of dams with irrigation as the primary purpose), since such extreme floods take place more often towards the very end of the monsoon season, which is also the time for storage of water to the maximum possible extent. In certain cases, it may be required to change/modify turbines to compensate for fall of efficiency following reduction of the effective head for power generation into inadmissible zones. Similarly, change of crop types/change of irrigation type from gravity to lift from canals may be mandated to cope up with the reduced irrigation water availability.

6.3.2 Reservoir Constraints

Flood routing criteria should recognize constraints that may exist on the maximum desirable water surface elevation. A limit or maximum water surface reached during a routing of the IDF can be achieved by providing spillways and outlet works with adequate discharge capacity. Backwater effects of flood flow into the reservoir must specifically be considered when constraints on water surface elevation are evaluated. Reservoir constraints may include the following:

- The topography of the reservoir rim which makes the construction of saddle-dike economically unviable.
- Public works around the reservoir rim, which are not to be relocated, such as water supply facilities and sewage treatment plants.
- Dwellings, factories, and other development around the reservoir rim, which are not to be relocated.
- Sediment deposits in reservoir headwater areas which may build up a delta and can increase flooding in that area, as well as reduce flood storage capacity.
- Geologic features that may become unstable when inundated and result in landslides, which would threaten the safety of the dam, domestic and/or other developments, or displace reservoir storage capacity.
- Floodplain management plans and objectives established under central, state, or municipal regulations.

6.3.3 Reservoir Regulation Requirements

To ensure the hydrologic safety of a dam, several reservoir regulation requirements need to be followed. For example, largest and smallest regulated releases from a dam should be specified. The largest regulated release rate should be specified to prevent flooding or erosion of downstream areas and control the rate of reservoir drawdown. The smallest regulated release capacity eases the recovery of flood control storage for use in regulating later floods. It is also important to allow for the evacuation of the reservoir in case of an emergency and for performing inspections, maintenance, and repairs.

Spillways, outlet works, penstocks for power plants, and navigation locks are sized to satisfy project requirements and must be operated in accordance with specific instructions if these project works are relied upon to make flood releases. These are subject to the following conditions and limitations in deciding whether to assume release facilities are operational:

- Structural competence and availability for use
- Availability and reliability of generating units for flood release during major floods
- Availability of a source of auxiliary power for gate operation
- Effects of reservoir debris on operability and discharge capacity of gates and other facilities
- Accessibility of controls
- Design limits on operating head
- Reliability of road network for access to the site
- Availability of operating personnel at the site during floods
- Any other condition or situation that limits the operation of facilities at design capacity

Only those release facilities which can be expected to operate reliably under the assumed flood condition should be assumed to be operational for flood routing.

A positive way of making releases to the natural watercourse by use of a bypass or waste way must be available if canal outlets are to be considered available for making flood releases. Bypass outlets for generating units may be used if they are or can be isolated from the turbines by gates or valves.

In flood routing, assumed releases are limited by several factors including project uses, availability of outlet works, tailwater conditions including effects of downstream tributary inflows and wind tides, and downstream non-damaging discharge capacities until the specified storage elevations are exceeded. When a reservoir's capacity to regulate outflow is exceeded, other factors including the safety of the dam will govern releases.

During flood routing, the rate of outflow from the reservoir should not exceed the maximum projected rate of inflow, to the extent possible, until the outflow approaches the maximum project discharge capacity, nor should the maximum rate of increase of outflow exceed the maximum rate of increase of inflow to the extent possible. This is to prevent flooding impacts downstream of the dam from being more severe than they would have been had the same flood occurred prior to the dam's existence. This only applies to the rising limb of a flood hydrograph. Once inflows and downstream flows have receded, the dam must release the water it has stored which will result in outflow exceeding inflow. Another exception to the above would be those uncommon cases where reliable streamflow forecasts are available and sufficient time exists for pre-flood releases to reduce reservoir levels to provide appreciable storage for flood flows.

IS: 7323-1994 (Operation of Reservoirs-Guidelines) classifies flood time reservoir operations into either of the four categories listed below:

1. Effective use of available flood control storage: Reduces flood damages at downstream locations through lower outflows. This is used for low floods, allowing water to store up in the reservoir.

2. Control of reservoir design flood: Full storage capacity is utilized only when the flood develops into the reservoir design flood. Maximum possible releases are made to ensure the safety of the dam.

3. The combination of principles 1 and 2: Principle 1 is followed for the earlier portion of the flood while the lower portion of the flood reserve gets filled up. Thereafter Principle 2 is followed.

4. Flood control in emergencies: Followed under extreme conditions. Ensuring safety of the dam against failure is the concern.

Successful implementation of any of these principles will depend much on the available lead time and accuracy of the meteorological and hydrological forecast. While development of flood forecasting system for a large dam with large catchment area (and consequently, long flood travel time) has little difficulty, putting the same into service for a dam intercepting small and steep catchment is challenging. In such cases, mandatory lowering of the water level during the entire flood season through rule curve may be the acceptable solution, if there are chances of significant damage to downstream property and loss of life in case of a failure.

6.3.4 Evaluation of Failure of Dams in a Series

If one or more dams exist downstream of the site under review, the flood wave that could result from failure of the specified dam should be routed to evaluate if any of the downstream dams on the same river would potentially breach one after the other. The flood routing of flows entering the dam being reviewed may be either dynamic or level pool depending on site-specific conditions. For instance, a dynamic pool routing should be used in cases with significant backwater effect, flat channel slope, and rapidly rising inflow hydrographs in long and narrow reservoirs or irregularly shaped reservoirs.

In the other hand, the routing through all subsequent downstream reservoirs should be only dynamic. Tailwater elevations should consider the effect of backwater from downstream constrictions. **If the failure of the new or existing dam being reviewed could contribute to the failure of a downstream dam, the hazard classification of the upstream dam should at least be the same as (or higher than) the classification of the downstream dam**.

It is worth to mention that the cascade failure should be analysed judiciously and based on the *Guidelines for Mapping Flood Risk Associated with Dams* and the *Guidelines for Classifying the Hazard Potential of Dams* published as part of the same series under DRIP. These guidelines describe with more detail how and when the cascade failure of dams might be assessed.

As part of the proposed framework presented in this Guideline, the IDF return period will be assigned for new dams after the hazard classification process is completed. Therefore, the IDF return period for a specified dam will depend upon the potential consequences of the worst-case scenario of failure of that dam, including a potential cascade failure, which needs to be either corroborated or discarded after proper hydraulic modelling.

As mentioned before, if the failure of the upstream dam is considered to threaten the integrity of the downstream dam, after an appropriate dam break analysis, the upstream dam will be classified with the same or higher hazard class of the downstream dam, and therefore; corresponding IDF will be computed for the dams, which either may be similar for both dams or higher for the upstream dam depending on their class.

In general terms, and without entering in the details of a proper risk assessment, is very unlikely that the case of a low storage dam located upstream would be able to threaten the safety of a much bigger dam/reservoir located downstream and, therefore; an stringent IDF return period for the small dam is not justified. In this case, the hazard class for the upstream dam would be lower than the downstream dam classification, leading to a smaller return period IDF for the upstream dam.

Furthermore, it is important remarking that, when only the hazard classification based approach is used, while assessing the IDF and potential hazard class of a new or existing downstream dam, none of the upstream dams' potential failures should be considered, irrespective of the size or reservoir's volume of the upstream dams. However, a proposed new dam should not increase the hazard potential classification of an existing upstream dam and must be designed to eliminate any increase in the upstream dam's hazard potential class.

6.4 Structural Measures for Flood Accommodation

6.4.1 Principal Spillway

Controlled versus Uncontrolled Spillways

By definition, an uncontrolled spillway releases water whenever the reservoir elevation exceeds the spillway crest level. Conversely, a controlled spillway can regulate releases over a range of water levels through the use of gates and/or valves. Each of these spillway types has specific design implications which should be considered when designing a spillway.

The selection of a controlled or uncontrolled type of spillway for a specific dam depends on site conditions, project purposes, the magnitude of the IDF, economic factors, costs of operation and maintenance, and other considerations. The following considerations influence the use of either a controlled or uncontrolled spillway:

- Discharge capacity – For a given spillway crest length and maximum allowable water surface elevation, a controlled spillway can be designed to release higher discharges than an uncontrolled spillway if the spillway crest elevation is lower than the normal reservoir storage level. This can impact spillway design selection when there are limitations on spillway crest length or maximum water surface elevation.

- Project objectives and flexibility – Controlled spillways permit a wide range of releases and have the capability for pre-flood drawdown.

- Operation and maintenance – Uncontrolled spillways are typically more reliable and self- maintaining than controlled spillways. Controlled spillways may experience more operational problems and are more expensive to construct and maintain than uncontrolled spillways. Constant-attendance or several inspections per day by an operator during high water levels is highly desirable for reservoirs with controlled spillways, even when automatic or remote controls are provided. However, access to the dam during a major flood event might be difficult or even impossible. Controlled spillways require regular maintenance and periodic testing of gate operations.

- Reliability – The nature of uncontrolled spillways reduces dam failure potential associated with improper operation and maintenance. Where forecasting capability is unreliable, or where the time from the beginning of runoff to peak inflow is only a few hours, uncontrolled spillways are more reliable, particularly for high hazard potential structures. Consequences of failure of operating equipment or errors in operation can be severe for controlled spillways. Susceptibility to plugging due to debris can also impact the reliability of both controlled and uncontrolled spillways.

- Data and control requirements – Operational decisions for controlled spillways should be based upon real-time hydrologic and meteorological data to make proper regulation possible. These should be carried out on the basis of dam-specific rule curves.

- Emergency drawdown – Typical uncontrolled spillways cannot be used to evacuate a reservoir during emergencies. The capability of controlled spillways to draw down pools from the top of the gates to the spillway crest can be an advantage when rapid reduction of load on the dam is required.

- Economics – Economic considerations often influence whether controlled or uncontrolled spillways are selected. Controlled spillways are typically more expensive than uncontrolled spillways.

The choice of a combination of more than one type of spillway is also a possibility. Final selection of the type of crest control should be based on a comprehensive analysis of all pertinent factors.

6.4.2 Additional Spillways

Dams and their appurtenant structures should be designed to give satisfactory performance. In addition to distinguishing between controlled and uncontrolled spillways, these guidelines identify three specific types of spillways: 1) service or principal spillways, 2) auxiliary spillways, and 3) emergency spillways. Outlet works can also be used to lower reservoir levels in anticipation of a flood event or to pass floodwaters.

- Service spillways should be designed for frequent use and should safely convey releases from a reservoir to the natural watercourse downstream of the dam. A service spillway should have excellent performance characteristics for frequent and sustained flows, such as up to the 100-year flood event. In general, service spillways should pass design flows without sustaining any damage.

- Auxiliary spillways are usually designed for infrequent use. It is acceptable for an auxiliary spillway to sustain limited damage during the passage of the IDF if it does not jeopardize the structural integrity of the dam or the function of the spillway. Reference to these spillways as “emergency spillways” should be stopped. Media references to flow through “emergency spillways” often leads to a misconception by the public that an emergency condition exists at a dam when the dam is safely functioning as designed.

- Emergency Spillways are not intended to be used for the routing of the IDF. They are provided where there is a desire to protect against a malfunction

Figure 6.1. Additional Spillway in Sanamachhakandana Dam (Odisha). Works finished under DRIP

of another component of the dam designed to safely pass the IDF.

6.4.3 Performance Improvement for Control and Conveyance Structures

One of the possibilities to adjust the dam to the new hydrologic conditions and overhauling its ability to pass larger flood values is to optimize the coefficient of discharge of the weir and conveyance structures as well as to improve the effective length of the weir.

- Local flow conditions can be adjusted with training or guide works upstream of the reservoir and streamlining of the flow. This will improve the coefficient of discharge "C", due to better flow conditions and more effective length of weir. Also, surface repairs avoid concrete irregularities and helps to improve flow conditions.
- Increasing the length of weir is a good option if there is space to do it or if type of weir can be modified. For cases in which limited length is available, there are options of a different kind of layout that can be adapted to available space (e.g. Labyrinth spillways, Piano Key weirs, fuse gates, etc), These options and their design insights are described in detail in the *Manual for Assessing the Hydraulic Safety of Dams*.

It is worth to mention that the **discharge capacity of the conveyance structure must remain equal to the discharge capacity from the upstream control section**. In other words, capacity of the spillways should not be modified with any change or modification in the conveyance structure since it only conveys what the weir allows to pass.

Furthermore, measures to ensure satisfactory functioning of chute or tunnels are required to be taken on a periodical basis in order to guarantee the hydraulic behaviour, integrity and stability of the conveyance structures. Some of the most common hydraulic actions over these types of structures and their potential adverse response are also summarised in the *Manual for Assessing the Hydraulic Safety of Dams*. The manual also compiles the recommended rehabilitation measures for each case.

6.4.4 Parapet Walls (Freeboard Requirements)

The capacity of spillways and outlets at a dam need to be large enough to pass the IDF without overtopping or breaching of the embankment. Freeboard, i.e. the difference between the maximum water level in the reservoir during the passage of the IDF and the lowest crest level of the dam, must also be able to withstand the wave action without overtopping and needs to include an allowance for the predicted long-term settlement of the embankment and foundation.

Freeboard gives a margin of safety against overtopping failure of dams. It is not necessary to prevent splashing or occasional overtopping of a dam by waves under extreme hydrologic conditions. However, the number and duration of such occurrences should not threaten the structural integrity of the dam, interfere with project operation, or create hazards to personnel.

Freeboard should be evaluated on a case-by-case basis considering many factors including the magnitude of the selected IDF; the predicted duration of high water levels during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave run-up based on the roughness and slope of the upstream side of the dam; the potential for debris plugging and/or mis-operation of a spillway; and the ability of the dam to resist erosion from overtopping waves.

Freeboard allowance for settlement should be applied to account for consolidation of foundation and embankment materials when uncertainties existing in computational methods or data used yield uncertain values for camber design. Freeboard allowance for settlement is not necessary when an exact estimate of settlement can be made and is accounted for with camber. Freeboard allowance for embankment dams for estimated earthquake-generated movement, resulting in standing waves, and permanent embankment displacements or deformations should be considered if a dam is in an area with potential for seismic activity. Reduction of freeboard allowances on embankment dams may be proper for small fetches, obstructions that impede wave generation, special slope and crest protection, and other factors.

Freeboard for concrete and masonry dams can be less than for embankment dams because of their resistance to wave damage or erosion. If studies show that dams can withstand the IDF while overtopped without significant erosion of foundation or abutment material, then no freeboard should be required for the IDF condition. Special consideration may be needed in cases where a power plant or other feature of national importance is located near the toe of the dam.

Estimating Wave Run-up

One of the key aspects in the estimation of the freeboard requirements is the estimation of the parapet wall, which is only intended to replace the portion of the freeboard needed to prevent overtopping from wind effects and waves run-up.

Various methods may be used for calculating freeboard. Some of these methods are as under:

- Stevenson's Formula as modified by Molitor.
- Guidelines as per USBR – Design of
- Small Dams (1987).
- USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and Guidelines for computing Freeboard allowances for storage dams (1981).
- USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and Guidelines for computing Freeboard allowances for storage dams (1992) USBR – Design Standards No.13 – Embankment Dams, Chapter 6: Freeboard, September 2012.
- IS-10635 - Free board requirements in Embankments dam and IS 6512 – Design of Solid Gravity dams.

All these methodologies are described in detail in the *Manual for Assessing Structural Safety of Existing Dams* on its *Chapter 3*.

6.4.5 Heightening of the Dam's Crest

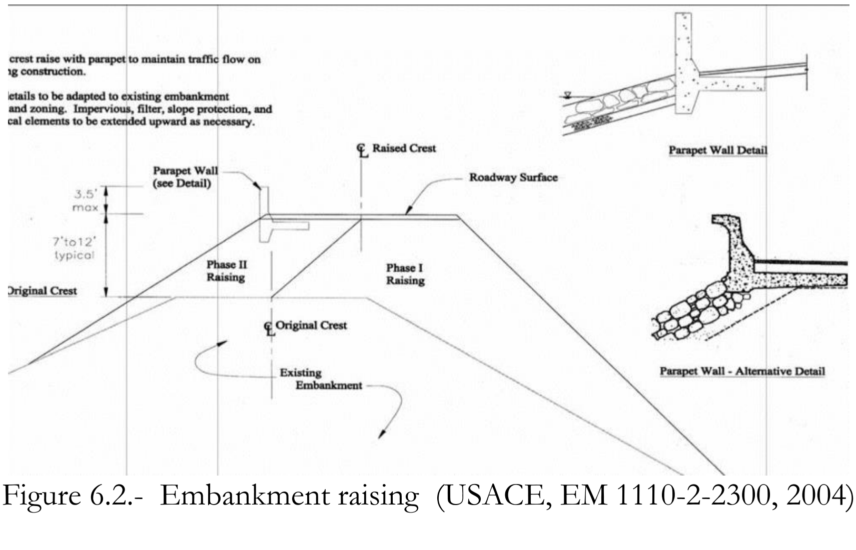


Figure 6.2.- Embankment raising (USACE, EM 1110-2-2300, 2004)

Despite the raise of the dam heigh may be considered as the logical and typical option to increase the available freeboard to accommodate the revised IDF in existing dams, it is worth to emphasise that the implementation of this alternative will have implications not only in the stability of the dam's structure - whether is concrete or earthen dam - but also can increase the potential consequences or introduce

potential failure modes that could significantly increase rather than decrease risks to the public.

In that sense, it is recommended that the raising of the dam should be considered as a structural measure only when is evaluated as part of a risk-informed decision-making process.

Design aspects and implications related to the implementation of this type of structural measure are further described in detail in the *Manual for Assessing Structural Safety of Existing Dams.*

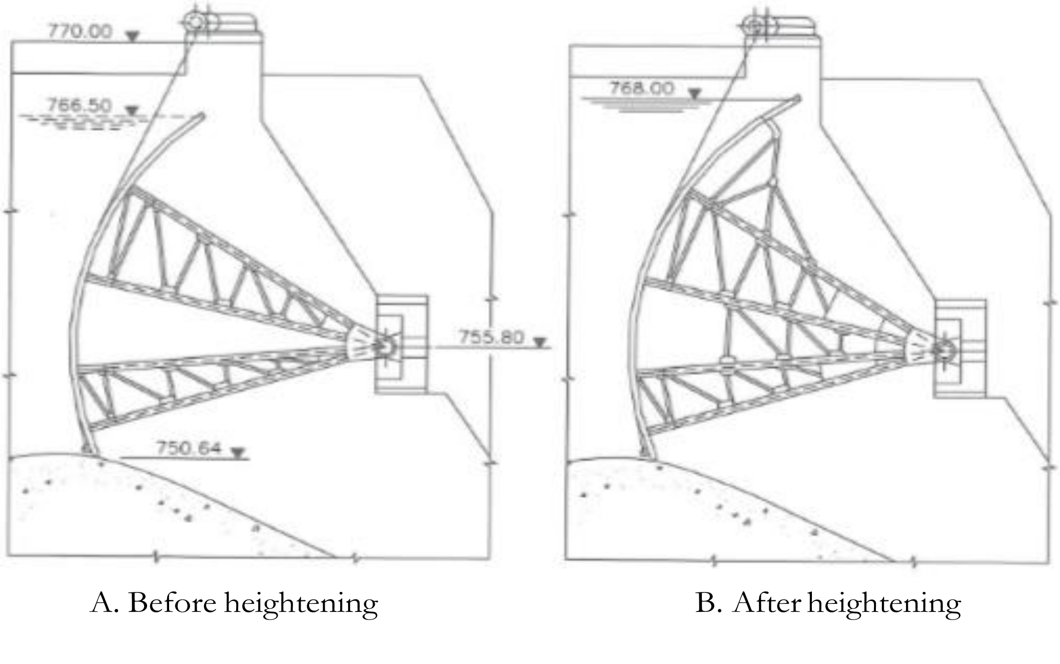
6.4.6 Heightening of the Existing Gates

Need for increasing the storage volume of reservoir for accommodating a new maximum water level (MWL) may be accomplished by raising the height of spillway's radial gates, which represents an advantage for projects where radial gates are used. The associated hydrostatic pressure on the submerged gates, however, increases and it is necessary to recheck the design of the gates and their hoists, because the hydrostatic force increases accordingly. In that case, there are several options: Substitute the existing gates by new higher gates, Heighten the existing gates, install new gates above the existing ones or even place new gates on top of the dam i.e. over spillway crest.

The different options to carry out a heightening of the existing gates along with its design considerations, advantages and disadvantages are further discussed in the *Manual on Assessing the Hydraulic Safety of Dams.*

6.4.7 Lowering the Spillway's Crest

One effective option to increase the spillway capacity for a given reservoir elevation is through lowering the spillway's crest, which



A. Before heightening B. After heightening

Figure 6.3.- Top heightening of the Furnas Dam' radial gates, Brazil.

would add extra hydraulic head and therefore, increase the discharge for a given flood magnitude. However, this option would reduce the reservoir storage leading to lower reservoir elevations and may actually increase the risk to the downstream public by increasing the spillway flows during hydrologic events that are more likely to occur.

From another perspective, lowering the spillway's crest is particularly effective when the downstream incremental consequences are being evaluated. Because the increased discharge would result in higher downstream water surface elevations in the river, the differential head between the reservoir and the tailwater would be reduced

Figure 6.4.- Placement of cable-tied mats over a geotextile on the downstream face of Strahl Lake Dam, Indiana (Photo by Contech Construction Products, Inc.)

significantly. Therefore, the incremental rise

Figure 6.5.- Overflow Protection in abutments (Gipson Dam, US)

in the downstream flood elevation due to a dam failure scenario would be also reduced.

6.4.8 Overflow Protection

While choosing an alternative to accommodate the increased design flood in an existing dam, those alternatives that avoid flow over the dam crest has been the traditional and safest approach. However, providing project-specific protection during dam overtopping (**Figure 6.4**, **Figure 6.5** and **Figure 6.6**) can be a viable method in some instances to safely convey larges flows downstream of the dam. A major concern with overtopping protection is that if protection fails during a flood event and the underlying material of the embankment/ abutments becomes exposed, erosion and head cutting may progress very rapidly and eventually lead to failure of the dam.

A decision to use dam overtopping protection rather than improving the service spillway, constructing an auxiliary spillway, raising the dam crest elevation, or imposing a reservoir restriction should be made with careful considerations of all potential impacts. Further design considerations for these type of rehabilitation measures are described in detail in the *Manual for Assessing the Hydraulic Safety of Dams*

6.5 Non-Structural Measures for Flood Accommodation and Flood Risk mitigation

An important driver of the increasing losses from floods is the accumulation of assets in flood-prone areas. By 2030, developing countries are expected to have a large share of vulnerability to flooding because of more rapid urbanisation in high-frequency flood zones (OCED, 2016). Floodplain regulatory management assumes the critical role to provide risk reduction in a sustainable manner. A host of literature is available on the subject. A few important ones that merit consideration include AEMI (2013), Santato et al., (2013), FEMA P-259 (2012), NFIP (2011), PPS25 (2010), AIDR-21 (1999) and

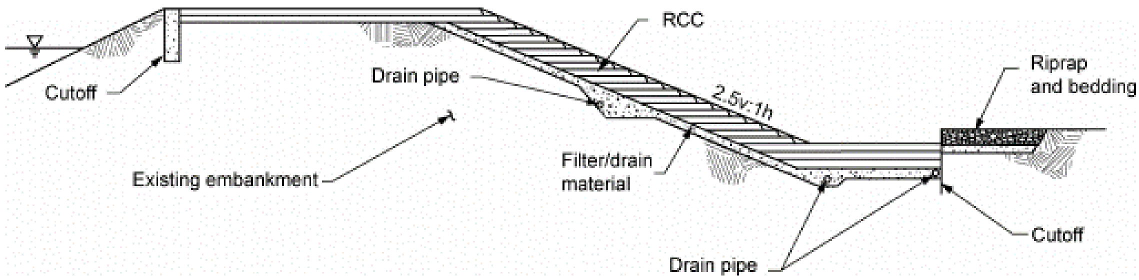


Figure 6.6.- Typical Section, RCC overtopping protection in Embankment Dams (Adapted from PCA, 2002)

INCID (1993). Some other measures to reduce flood risk at a community scale may include the development of flood forecasting and warning system, preparing community-scale emergency response plans, increasing community preparedness, and developing community recovery plans.

6.5.1 Real-Time Inflow Forecasting

Real-time forecasting of inflows to a reservoir is an extremely important factor in the optimal use of the water resource, as it is impossible to operate reservoirs in an optimal manner without any forecasting of reservoir inflows. The information and techniques used in forecasting inflows for reservoir operations are identical to those used for flood forecasting, including floods resulting from dam failures. Therefore, real-time hydrologic forecasting provides a non-structural measure to mitigate to some extent, the downstream impact of flooding, by 1) advising dam operators about the magnitude and timing reservoir releases to mitigate the impact downstream, and 2) providing advanced warning to government officials, emergency managers and the population, in general, about the impending risk from flooding, either natural, or accidental, resulting from a dam failure.

Flow forecasting, by itself, cannot eliminate the risk posed to a dam by incoming

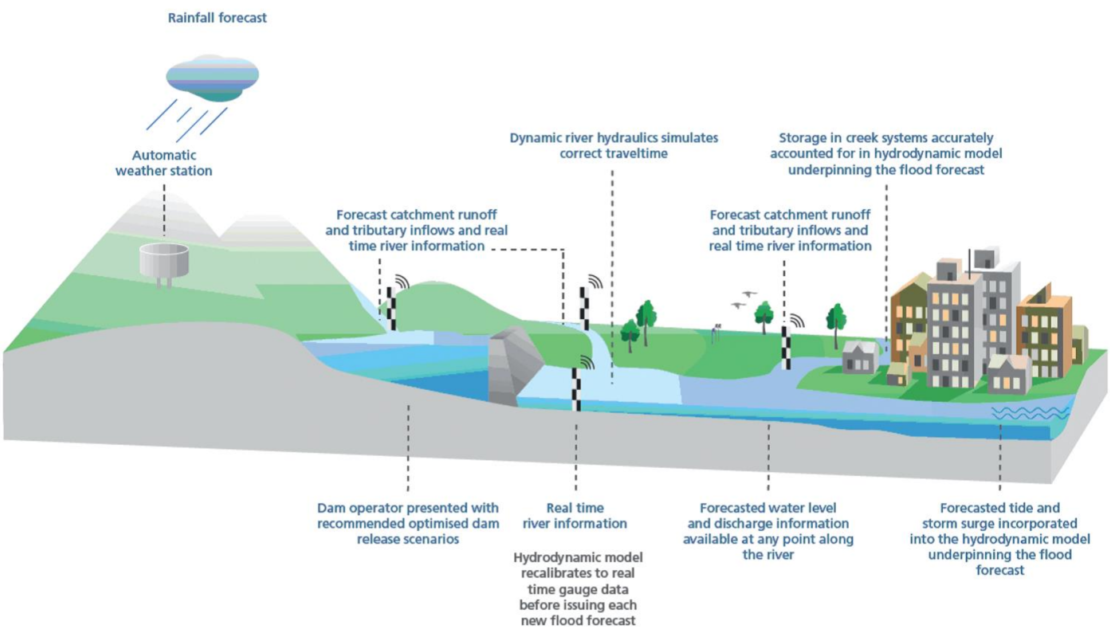


Figure 6.7.- Concept Diagram of a real-time Inflow Forecasting System (Adapted from Boulton et al, 2011)

exceptional flooding to a reservoir, but it can provide the required information to issue a timely warning to help reduce the impact on lives and property downstream of the dam. Therefore, not having an adequate system of warnings in place may require using a more conservative IDF, and, therefore, more expensive structural measures.

This section covers some of the basic functions of a real-time forecasting system needed for reservoir operation and flood warning from normal and emergency operations, and avoiding the failure of a major dam component.

Why including flood forecasting? Assume that you have two different projects whereby all conditions are identical, and both dams have the same classification based on the conventional approach of using the dam height and reservoir volume for IDF selection. The only difference among the projects is that one project (”A”) doesn’t have any type of flood forecasting and elements to provide dam-break or even flood warnings. Project ”B” has available a good system of forecasting and disseminating a flood warning to the downstream population. Project “B” should have a lower IDF than project “A”, because by being able to forecast the dam failure and advise the population of the impending risk, thus saving lives and protecting property (that is, property that can be moved).

Flow Forecasting in India

The Central Water Commission currently performs inflow forecasts for about 120 reservoirs (**Figure 6.8**). These forecasts are made in regional offices. CWC uses two different techniques: statistical analysis (correlation) among observations upstream of the forecasting point and observed precipitation as independent variables and forecasted flow as the dependent variable. There is no input of forecast precipitation or any other component that may affect the hydrologic cycle, such as temperature. The other method of forecasting is based on the Mike model of the Danish Hydraulic Institute.

Recently, the India Meteorological Department (IMD) started to issue flash flood forecasts, and shares these forecasts with CWC. Flash Flood Forecasts are really intended to provide early warning of impeding flash floods and their routing is, therefore, somewhat limited. However,

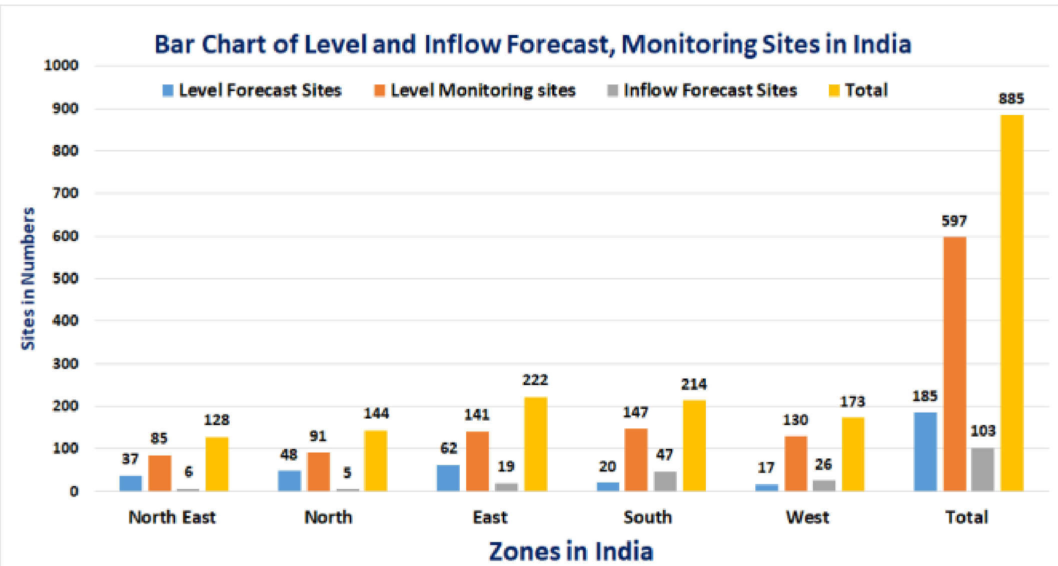


Figure 6.8.- Level and Inflow Forecast/Monitoring Sites in India 2018 (Adapted from SANDRP, 2018)

having this capability in India does open the possibility of expanding the hydrological services of CWC in the future, expanding, in coordination with IMD a comprehensive forecasting system for all types of flooding, and input to all, or, at least, the majority, of India's reservoirs, besides forecasting for flooding condition.

Static Data

Static observations are those that remain constant or quasi-constant for a long period of time. Among them are topography, bathymetry, soil types and properties, and land use.

- ✓ Topography

With the availability of the Shuttle Radar Topography Mission Data the world counts with free access to a database covering entire world from -60 to +60 degrees of latitude at a 30 m horizontal resolution. The data are available for free download from the USGS Eros Data Center.

- ✓ Bathymetry

Strictly speaking, bathymetry is not a type of static data. Bathymetry, the measurement of the bottom of bodies of water, changes frequently in rivers as a result of scouring and deposition. Accurate implementation of hydraulic models for routing and flood inundation mapping requires the use of current bathymetry data.

Bathymetry acquisition used to be a very expensive task, due to the need to perform surveys with traditional optical surveying equipment. The availability of GPS positioning systems, and ultrasonic depth and current sensors has lowered the cost of acquisition of bathymetry. For fairly clean rivers and shallow reservoirs there is yet another option, namely the use of LIDAR-equipped drones.

- ✓ Soils and Land Use

Soils and land use play a fundamental role in the generation of runoff. The Food and Agricultural Organization of the UN has

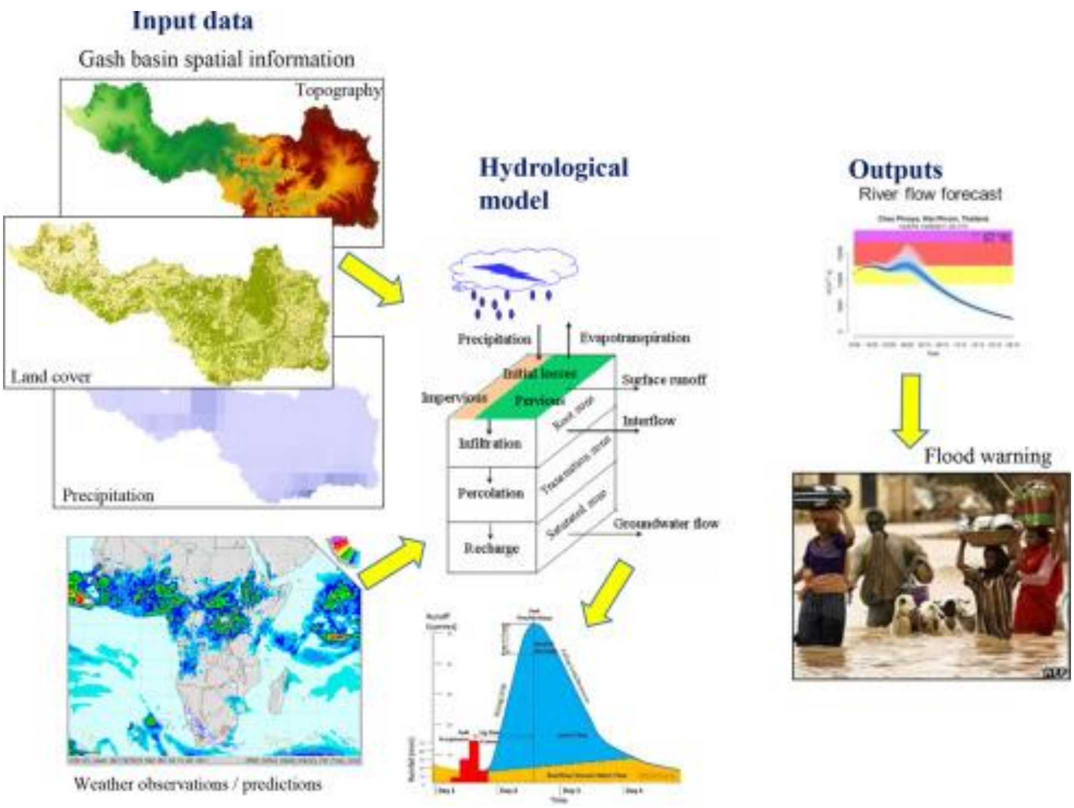


Figure 6.9.- Overall elements of an Inflow Forecasting System (adapted from G. Amarnath et al, 2016)

developed the harmonized World Soil Database of the entire world. This information can be used as a source for parameter estimation in rainfall-runoff models.

Soil types do not change over the course of generations. However, land use does change. There is urban expansion, changes in crop types, hopefully reforestation, but, more likely, deforestation. All of those changes in land use have an effect on the quantitative generation of runoff. It is important, thereof, to update the land use parameters on a periodic basis

Dynamic Data

Dynamic data changes continuously with time. Some of the data, such as temperature, precipitation, evaporation, river stage, etc. are gathered from point sensors, and others from spatial sensors

✓ Data Collection

Point Data: The absolute minimum data required for the simplest real-time hydrologic forecasting are precipitation data collected from pluviometres, and river flow estimates from stream-gauges. Other data that may be required depending on the models are temperature, potential evapotranspiration, wind speed, solar radiation, dew point, snowfall amount, snow depth, snow water equivalent, soil moisture. With the exception of streamflow observations, point data requires to be converted into spatial area by means of a number of techniques to be used by hydrologic models. Streamflow is not observed directly. Estimates of streamflow are obtained from direct observations of river stages and conversion into flow estimates using a rating curve. Alternatively, streamflow values can be estimated by using Acoustic Data Collection Platforms (ACDP) and developing a relationship between water velocity measured at a single point and total flow at the cross section (the Index method). Periodic preventive maintenance of sensors is a requirement. In addition, updating of cross section bathymetry at gauging sites, and river bathymetry at those river stretches for which hydrodynamic models provide water-surface elevation and river flow forecasts, need to be revised on a periodic basis, and every time after major flooding.

Spatial Data: As the name implies, dynamic spatial data are collected simultaneously over a large area. Several international agencies (NASA, NOAA, JAXA, ESA) make space-based observations freely available. These include, among others, precipitation estimates from the GPM mission, as well as estimates of soil moisture, snow cover area, snow water equivalent, wind speed and direction, and soil surface temperature.

✓ Data Transmission

The ability to reliably transmit observations as soon as they are received lies at the heart of a timely forecast. There are a number of possible real-time data transmission technologies. The broad availability of cellular networks, and the affordability of the technology for data transmission makes it an ideal way to transmit data whenever there is mobile phone coverage. For those places without cell-phone coverage, typical of headwaters in mountainous areas, other options include satellite-radio communications, satellite-phone communications, and even meteor-burst communications. These technologies have essentially replaced line-of-sight radio communications because of higher reliability and/or lower maintenance cost.

✓ Data Quality Control

There are a number of sources of errors in data. Ground-based rain gauges may be obstructed by dirt or bird droppings; stream gauge floats may get stuck; ice and ice dams may also affect river surface elevation and with the corresponding estimates of river flow. Spatial observations also need to be quality controlled. In most cases, that information contains biases. Precipitation observations from space and ground-based radars are de-biased by using rain-gauges located under the coverage area of the spatial observation sensor. Other observations, such as soil moisture and, snow water equivalent, use passive microwave sensors which are not as accurate as active observations from radar, and have a coarser horizontal resolution. They also need to be corrected with ground-based observations.

It is essential, thus, for quality-control observations. In many cases, automated procedures can be used to flag, or even correct, suspect data, by using neighbouring sensors. Sometimes it is impossible for automated systems to resolve erroneous readings, as is the case with ice dams, in which erroneous river elevation would yield exceptionally high river flows. Knowledge of the basin and river characteristics and comparison with streamflow forecasts from well-calibrated models are good indicators of when icing is the culprit of erroneous river stage observations.

Forecasting System

A hydrologic forecasting system consists of 1) a forecasting framework; and 2) hydrologic and hydraulic models; and 3) utilities to interact with the forecasts, visualize input data and forecasts, and allows the forecasts to be disseminated

✓ Forecasting Framework

The Forecasting Framework takes care of managing the communication between the database, the models, and the human forecasters. It makes sure that input data (historical and real-time observations, static data and model parameters) is available to the models, and provides a graphical user interface for forecasters to interact with the models, perform quality control on observations and forecasts, and issue and approve forecasts. For additional flexibility, the forecasting system may be designed in a client-server configuration whereby responsibilities for the different operations are distributed among servers and clients. For instance, one server takes the role of managing the database, another one executes the models, another one manages the communications, and the clients are used by the forecasters to run the models, verify performance, make correction to observations or model states, and visualize model output in line graphs, or 2-dimensional displays of input data, flooded areas, etc. The client-server architecture allows the entire system to run on a single machine, a useful feature that permits the system to operate from a single computer in the case of emergency when the main system is unavailable. It also allows for the system to be geographically distributed, with servers in one location, and clients in one or several different locations, all communicating via a wide area network.

✓ Hydrologic Models

Hydrologic models comprise models of the movement of soil moisture within the soil column, over the surface as runoff, and as groundwater. Snow accumulation and melt models are also members of the family of hydrologic models. Finally, hydrologic routing models are a subset of routing models that work either with simplified de Saint Venant or empirical equations to model the movement of water from upstream to downstream. All those models can be classified as Black Box, conceptual or Physically based.

Black Box Models: are a class of models that do not attempt to describe the physical processes connecting the components of the hydrologic cycle. Typically, these models use mathematical equations or regression analyses to link streamflow at the point of forecast, with observations and forecasts of precipitation, temperature, and other variables.

Conceptual Models: approximate the physical processes of the hydrologic cycle with empirical equations. These models have parameters that may have initial values derived from physical properties, such as hydraulic conductivity, porosity, slope, aspect, etc. but require calibration to be able to obtain the best performance.

Conceptual models can be lumped, when there is one model over the entire catchment, or distributed, when there is a regular grid distributed over the catchment, and each grid element has its own set of parameters. Some people call a model composed of a series of lumped models for each of the sub-catchments a "semi-distributed" model and restrict the use of the "distributed" qualifier for those models based on geometric grids.

Physically Based or Mechanistic Models use the governing equations with either minor or no simplifications. With few exceptions, parameters are directly observable from available databases, and, in theory, would go into the model without modification or calibration. However, it is not uncommon that some of those parameters that are directly observable need to be adjusted (i.e. "calibrated"), in addition to those parameters not directly observable. In theory, these models have the advantage of allowing changes in the watershed (land use, for instance) to be easily represented by changing the corresponding model parameters, and the resulting simulation values should reflect the observed streamflow. In practice, however, physically based models do not outperform conceptual models in actual forecasting operations

✓ Hydraulic Models

Hydraulic models apply the full de Saint Venant equations to reflect the movement of water, either within the river channel, or on the flood plain. Most commonly used hydraulic models are one-dimensional, following the river channel, and approximating the bathymetry by using cross section information. It often is the case that during flooding conditions, the river flow ceases to be one-dimensional and

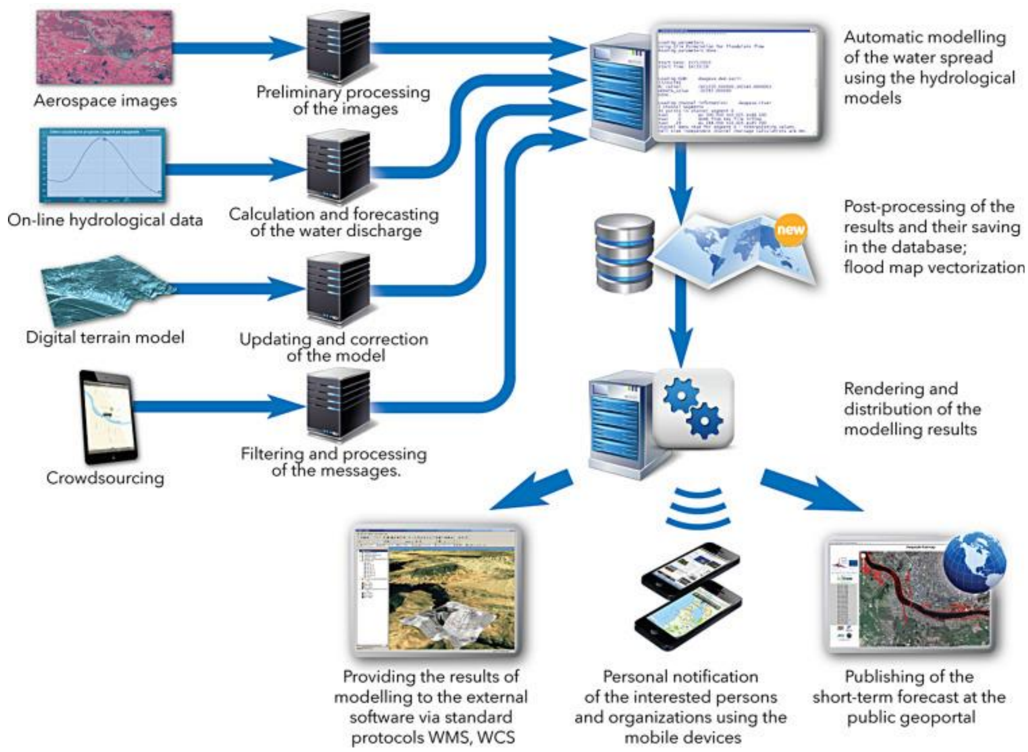


Figure 6.10.- Schematic Processing and Dissemination of Data in an Inflow Forecasting System (adapted from G. Merkuryeva et al. 2015)

takes many paths over the flood plain. In those cases, it is possible to use multiple one-dimensional models, or, alternatively, use two dimensional models.

Hydraulic model routing presents several advantages over hydrologic routing, being able to consider explicitly downstream conditions, something that hydrologic routing models cannot do. Furthermore, these models compute estimates of river flow and stage at every cross section (for one-dimensional models) and at every computational element (for two-dimensional models), which allows the estimation of water levels and flows at a very fine resolution. In comparison, hydrologic models can only produce estimates of river flow and stage at rating curve locations. A down point of these models, especially for the two-dimensional models, is the computational burden.

- ✓ Probabilistic and Deterministic Forecasting

Hydrologic forecasting can be either deterministic, or probabilistic. Deterministic forecasts do not take into consideration the uncertainties inherent in the input data (meteorological observations and forecasts, model parameters, and observed streamflow), and model structure. Probabilistic models consider at least one or more of the sources of uncertainty to arrive at an approximation of the probability distribution of forecasted flows. A probability distribution of the flow forecasts is a direct measure of the uncertainty surrounding the forecast.

- ✓ Error Correction

Forecasts will never be perfect because of the forecast uncertainty described above. It is important, therefore, to prevent the accumulation of errors in a forecast. Correction of the forecasts can be done with a manual intervention by the forecaster, or by automated methods. Manual corrections by skilled forecasters tend to produce forecast flows more acceptable to stakeholders. Automated methods often require that the model be perfectly calibrated and/or pose heavier demands on computer processing.

Output Dissemination

The final step in the hydrologic forecasting process is the dissemination of the information. The system should include automated procedures for the forecasts to be distributed to stakeholders in an automated fashion. Stakeholders include reservoir operators in the same catchment, both upstream and downstream, emergency managers, and the public at large. In the context of forecasting within a system of cascading reservoirs makes it critical that the communication among reservoir operators be very active, since releases from upstream dams affect inflows to downstream dams. For extreme flooding conditions, good communication and coordination among operators of reservoirs in cascade may help reduce total inflow to the downstream reservoir by filling upstream reservoirs, thus reducing the risk of dam failure.

6.5.2 Optimising the Reservoir Rule Curve

A rule curve defines the storage to be maintained as close to the desired storage as possible for meeting various needs during different times of the year. The rule curve does not specify the magnitude of water release which depends on the inflow to the reservoir and demands for various purposes. Different rule curves can be developed for different purposes, such as flood control, hydropower generation, irrigation, water supply, and tourism and recreation.

Figure 6.11 in next page shows the schematic rule curves associated with a typical multi-purpose reservoir, in which the calendar year is divided into various within-year time periods, herein two seasons and two transition periods: Non-flood season, flood season, drawdown period, and refill period. In non-flood season, water supply curves guide the release with conservation water storage (S_{cons}) and dead storage (S_{dead}) as the upper and lower limits. During normal-inflow years, all planned demands are met 100% and reservoir storage is kept above the target storage curve ($S_{target,t}$), but during drought years when storage falls below the target storage curve or even firm storage curve ($S_{firm,t}$), the release is reduced. In flood season, reservoir storage is constrained by flood limited water storage (S_{flood}). The maximum storage capacity (Smax) is the upper bound for flood routing. In the transition periods, the reservoir should be lowered to S_{flood} at the beginning of flood season and refilled to S_{cons} by the end of flood season. The curves are established at the planning stage and usually kept unchanged during the lifespan of reservoirs.

As reflected in the conventional rule curves of **Figure 6.11** , water supply shall be small if the current water level/availability is low. For instance, if the beginning reservoir storage is in *Zone 1*, all planned demand is met; but if the storage is in *Zone 2 or 3*, different degree of release curtailment is needed; and the lower the storage zone, the higher the reduction of water supply.

Traditionally, rule curves are derived by reservoir operation studies using historical or synthetically generated flows. Traditional reservoir operations have relied on fixed rule curves to control reservoir levels and/or reservoir releases. The basic idea behind rule curves for reservoir levels was to try to capture as much runoff in those reservoirs whose inflow was markedly seasonal, such as those relying on snowmelt or monsoonal flows, and release those stored flows during the dry season. Reservoirs whose objective has been flood control have had a curve that instructed the operators to keep the reservoir level below given monthly targets. Reservoirs for irrigation water supply try to keep as much volume in storage in order to meet the varying irrigation demand during the growth season.

Derivation of a rule curve depends on the type of reservoir and the purpose to be served. A reservoir may be seasonal or multi-annual. For the seasonal reservoir, the storage is carried over from the wet season to the dry season, whereas for a multi-year reservoir the storage is carried over from the wet period to a subsequent dry period which may occur several years later. By plotting the

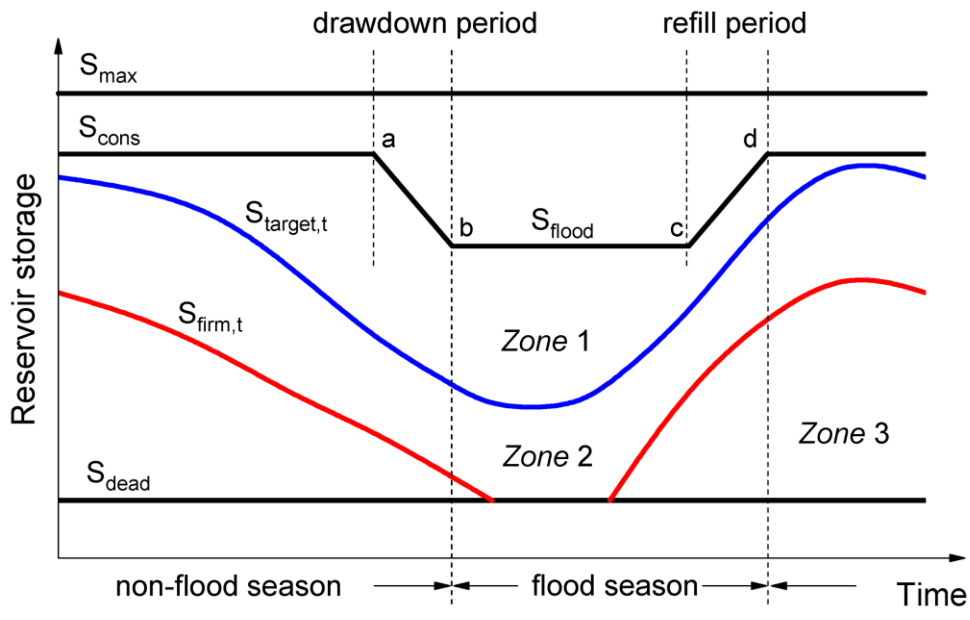


Figure 6.11.- Schematic illustration of multipurpose reservoir rule curves, specifying the storage targets and the release given particular current storage and time of within-year period.

mass curve of inflow and the mass curve of demand, usually on a monthly basis for reservoirs with more-than-seasonal regulating capacity, the difference between the two curves provides the storage as a function of month which is the rule curve. Other approaches have included using computer simulation or optimization models. Historically, the rule curve is constructed for the flow pattern in a critical year, where it assumed that if the reservoir can satisfy demand during the critical year, it can satisfy demand during non-critical years. Often it is desirable to plot the rule curves for other near-critical years on the same graph. These curves may cross each other at several places so a smooth enveloping curve is plotted which is the required rule curve.

The problem with the enveloping curve used in the traditional approach is that it may lead to operations that are too conservative, with the corresponding sub-utilization of the water resource, giving the rigidity of a fixed rule curve. For instance, if an irrigation rule curve based on a critical year or on an envelope curve is used during a wet year a reservoir may end up spilling water that won't be available later on during the higher demand months. A similar problem will affect hydropower generation reservoirs.

There are several ways to resolve the problem rule curve inflexibility. One more modern approach consists on using stochastic optimization techniques where the uncertainty in the forecasted inflows. In those places where it is impossible to obtain a reliable probabilistic forecast with a skill higher than that of climatology, it is possible to eliminate the use the rule curve altogether at an acceptable probability level. Another approach consists on developing a rule curve based on a hedging approach, whereby there is a lot of flexibility incorporated on the rule curve to allow for variation on the expected reservoir inflows.

There are cases of irrigation reservoirs that were developed and had been operated under a single operating purpose, and, therefore, had a single rule-curve. One such example is the case of "El Carrizal" reservoir in Mendoza, Argentina: when the area downstream of the reservoir suffered catastrophic flooding downstream, the purpose of the reservoir was modified to include also flood protection, and, therefore, a second rule curve.

In the case of reservoirs for flood control, the objective is to keep the reservoir as low as possible all the time. The main objective of an irrigation supply reservoir is to capture as much runoff as possible to deliver it during the crop-growing season according to a time-changing demand curve, which requires getting the reservoir as high as possible to save water for the high-demand season. Reservoirs for hydropower generation have an additional complication: not only water spills are to be avoided, but also reservoir level can't be lowered more than necessary so that hydraulic head, a critical component of power generation, be reduced unnecessarily.

It is clear then that irrigation supply and hydropower generation on one hand, and flood control on the other, are conflicting purposes. It is possible to have a rule curve for irrigation and/or hydropower generation that will make it impossible to develop any feasible flood control rule curve, and vice versa. Clearly, a reservoir rule curve that dictates that the reservoir should be permanently empty would meet the flood control purpose but would be disastrous for either irrigation and/or hydropower generation. It is also possible to foresee that there are cases in which, for a given level of protection against flooding, it is possible to design an irrigation rule curve that will meet a given level of satisfying the irrigation demand. The key here is to develop a set of two rule curves that will provide a chosen level of meeting the irrigation demand, while, at the same time, meeting the flood protection objective at a given level of reliability. For a given level of flood protection it is possible to stepwise increase the level of satisfying the irrigation and/or hydropower demand while still meeting the desired level of flood protection. There is a point, however, beyond which it is impossible to improve irrigation and/or hydropower satisfaction without lowering the flood protection reliability. This point lies on the so-called Pareto curve or efficiency curve. This curve marks the frontier at which it is possible meet both conflicting objectives such that improving one objective must come at the cost of lowering the other objective. The operation of the reservoir at any pair of points lying over the Pareto curve leads then to a pair of rule curves, one for flood protection and the other one for irrigation supply and/or hydropower.

Classical rule-curves for long-term operations are being replaced by a host of techniques, among them risk aversion (to improve the worst-case performance, see Figure 6.12), or probabilistic and hedging approaches (to maximise the average performance of the system).

Rule curves for short-term reservoir operations are becoming in disuse, due to the availability and skill of modern flow forecasting techniques, which, together with real-time information on flows, precipitation, and other weather parameters, allow for real-time optimization of reservoir operations.

A classic set of rule curves for an irrigation and flood control reservoir can be constructed by solving a Linear Programming optimization problem, for which there are a number of solvers, both commercial and freely available, including the Excel solver. The following equations are a fairly simplified set, in which losses to evaporation and seepage are ignored:

$$\boldsymbol{Min\{D^*\}} \quad (1)$$

Subject to:

$$\boldsymbol{S_{t+1} = S_t + I_t - Q_t - W_t, \forall t} \quad (2)$$

$$\boldsymbol{D^* \geq \frac{Q_t}{D_t}, \forall t} \quad 3)$$

$$\boldsymbol{W_t \leq F, \forall t} \quad 4)$$

$$\boldsymbol{S_t \leq S_{max} \forall t} \quad 5)$$

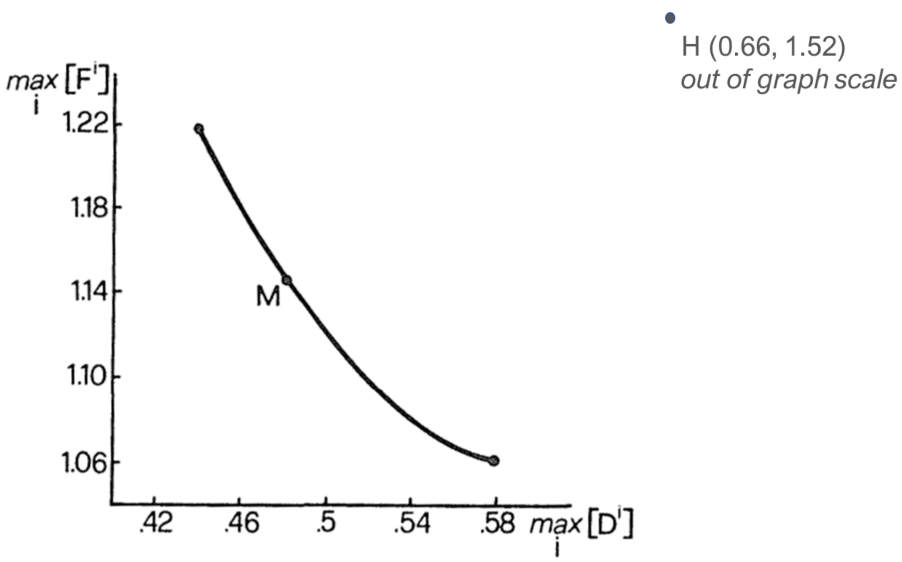


Figure 6.12.- Pareto boundary in the risk aversion space for a reference set of 28 years, Carrizal reservoir, Argentina (Adapted form C. Gandolfi et al. 1989). Where D^i = water shortage indicator (maximum rate of unsatisfaction of the demand) , F^i = flood indicator (maximum rate of unsatisfaction of the flood release limit), H = historical performance, M = improvement target (trade-off between objectives attained by the historical management)

Where,

D^* is the maximum relative demand deficit,

S_t is the reservoir storage volume at time t, with S_o set by the user

I_t is the historical or synthetic inflow volume at time t, set by the user,

Q_t is the regulated discharge volume at time t,

W_t is the spilled discharge volume at time t,

D_t is the demand at time t, set by the user

F is the flood level at which protection is desired,

S_{max} is the maximum reservoir level.

Optimizing the system of equations (1) through (5) using monthly time steps with a historical inflow time series will yield time series of discharges, spills, and more importantly for the objective of obtaining a rule curve, reservoir values. Appendix F of this Guideline incorporates an example calculation of reservoir rule curves for a hypothetical dam, using the methodology described.

Notice also that using historical monthly values the rule curves obtained with this approach will only serve to guarantee that the goals of flood protection and irrigation supply are met as long as the sequence of future flows stay within the bounds of the historical flows. Using synthetic series could give better assurances that the derived rule curves will provide the level of protection and irrigation demand coverage sought by the users.

Examining then the resulting end-of-month reservoir storage values the user obtains the rule curves. The flood protection rule curve for January is derived by looking at all the Januaries storage values and selecting the lowest value. Similarly, the irrigation rule curve for January is obtained by looking at the resulting end-of-month storage values and selecting the highest one among all the years. The process is repeated for February, March, etc.

Notice that, typically, it is only one

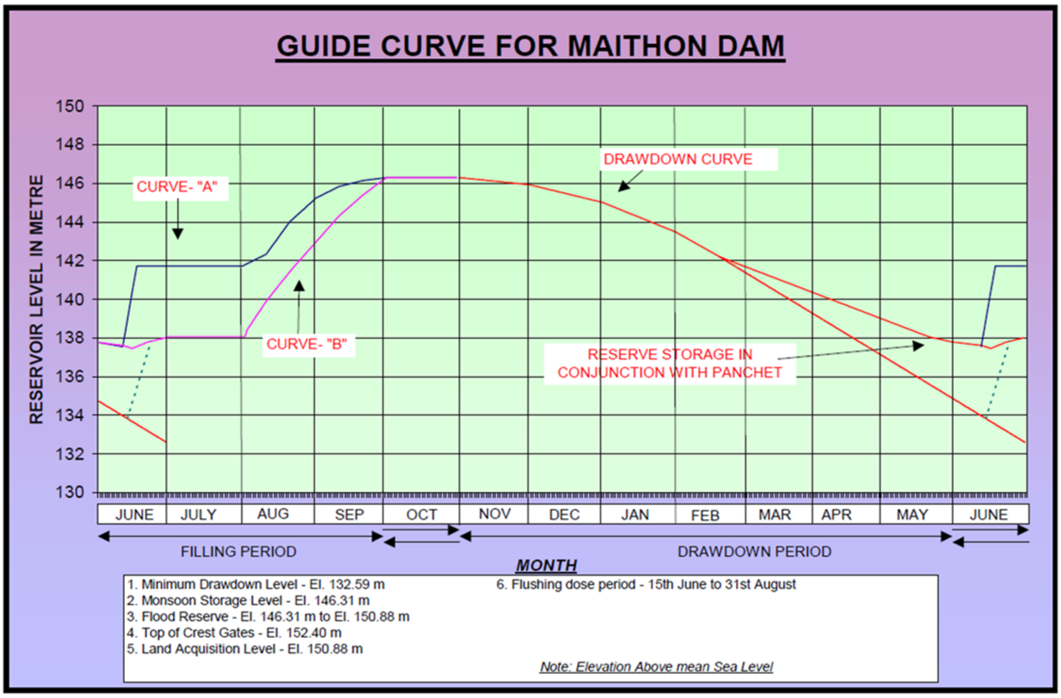


Figure 6.13.- Reservoir Rule Curve for Maithon Dam (Jharkhand, India).

particular year, among the entire historical series, that is the one that sets either the flood protection or irrigation coverage rule curves. As we explained before, rule curves derived with this approach are somewhat inefficient since with this approach for deriving rule curves it is the storage value for a month of a single year the one that sets the curve. Storage value for other months could be above (in the case of flood protection) or below (in the case of irrigation supply) of the rule curve without causing flooding or irrigation deficits above the chosen points.

6.5.3 Improving the Gates reliability

Outlet works and spillway reliability is of great relevance to dam safety and has played a fundamental role in many catastrophic failures of hydrologic nature (i.e flood management). Despite its manifest importance, gate reliability has remained an aspect of difficult integration into traditional hydrologic analysis and thereby, has been usually treated separately.

In the context of a risk-informed approach, when performance is evaluated, the analysis of the causes that must lead to gates failure cannot be limited to a mechanical failure, as experience shows, failures can be due to very disparate reasons. When the whole system is analysed it becomes apparent there are several causes that might induce failure:

- Human failure (either because the need of opening a gate is not identified or because the order is not transmitted or because the person in charge of operating a gate makes a mistake, etc.).
- Lack of access to the manoeuvre chamber (e.g., due to snow, excessive rain).
- Mechanical failure (breakage of a piece, blockage, etc.).
- Mechanical failure of the civil works (that could render the outlet works or spillway useless)
- Electrical failure (either in the supply or in the components of the outlet works or spillway themselves).
- Blockage of the outlet works or spillway (e.g., due to the presence of logs and debris).
- Failure in the software controlling the gate or the valve (in case it exists).

Therefore, all these assumptions must be considered to estimate the gate failure probability in order to render appropriate remedial and/or preventive measures to improve the gates reliability.

Further details about the gates reliability analysis can be found in the *Guidelines for Assessing and Managing Risks Associated with Dams*

6.5.4 Improving the Emergency Preparedness

Despite the emergency preparedness is not considered a measure to safely accommodate the inflow design flood, is certainly an option to lower the flood risk associated with hydrological and no-hydrological scenarios. As mentioned in previous chapters, **emergency preparedness should be considered as non-structural measure only when a risk-informed hydrologic hazard analysis is carried out**, simply because is the only method to quantify its actual effectiveness and impact in the flood risk mitigation, by ensuring an overall risk below the tolerability thresholds in the country.

In this context, is strongly recommended do not considered the emergency preparedness as a non-structural measure to safely cater the inflow design flood, especially when only a prescriptive approach is used to select the IDF

Improving the emergency preparedness in the dam's organisation involves the engagement of several stakeholders in the different stages of the process:

- Flood Hazard Understanding
- Public Education
- Warning Systems
- Emergency Management

Flood Hazard Understanding

A proper consequences assessment constitutes the core of any emergency preparedness plan. In order to plan customized remedial actions, it is important to recognize the actual consequences of each potential emergency scenario.

Damages produced by a dam failure are usually very severe, leading to high economic impacts and in many cases loss of life. Looking forward to reducing these impacts and especially those over the human lives, consequences must be estimated for several dam failure scenarios in order to plan and carry out suitable zoning and territorial planning. Finally, in order to be able to work with incremental risks, no-failure consequences must also be estimated. All this can be accomplished through dam break analyses and inundation mapping.

An inundation map is used to depict areas that could flood if a dam fails, and must be included in the Emergency Action Plan (EAP). Inundation maps should also show the time to flood (the time from the breach to the time that critical structures are flooded) and the time to peak flow. The inundation maps represent the most important tool to identify the main and alternative evacuation routes, as well as to prioritize rescue operations.

For further details in the preparation of the inundation maps and emergency action plans please refer to the *Guidelines for Mapping Risk Associated with Dams* and *Guidelines for Developing Emergency Action Plans for Dams*.

Public Education

Public education programmes can reduce flood risk considerably. A better knowledge of the existing risk, emergency management practices, sources of risk, protective measures and procedures in case of flooding can reduce potential flood consequences. An increase in public awareness can be either considered in the analysis as a better flood severity understanding, or as a reduction on population at risk or the percentage of people exposed to the flood due to a more effective response and more rapid evacuation processes.

Even though flooding and dam breaks are in general unexpected events, both represent the most predictable natural disasters if are compared with earthquakes, tsunamis, hurricanes, and eruptions, whose consequences are normally very difficult to estimate and predict; thus we can (and should) manage the risks involved by reducing the expected downstream consequences. The most efficient way to achieve this purpose is through a suitable dissemination plan among the population, which must develop each of the following aspects:

- Sensitize each affected community making them aware about their own risk. This can be accomplished by dissemination of pamphlets, triptychs, brochures and carrying out community workshops.
- Give them instructions how to act previous, during and after a flood.
- Educate them regarding the different evacuation routes available
- Educate them regarding the different types of sirens messages/sounds and levels of alert
- Encourage the creation of "Community Committees for Emergency Management"

Figure 6.14.- Stakeholders' Consultation Meeting. Kundah Basin, the Nilgiris District, Tamil Nadu, India.

Warning systems

The purpose of a flood system is to provide warning on impending flooding and help disaster management agencies and the members of flood-prone communities to understand the nature of developing floods so that they can take action to mitigate the flood's effects. A flood warning system is made up of a number of components which must be integrated. These components include (AEMS, 2009):

- Monitoring of rainfall and river flows that may lead to flooding (See *Section 6.5.1*)
- Prediction of flood severity and the time of onset,
- Interpretation of the prediction to determine flood impacts,
- Construction of warning messages describing what is happening and will happen, the expected impact and what actions should be taken,
- Dissemination of warning messages,
- Response to the warnings by involved agencies and community members, and
- Review of the warning system after flood events.

The improvement of the effectiveness of existent warning systems or the implementation of advance systems can increase the available warning time and the percentage of people who receive the message during the flood event. In addition, the improvement of emergency management plans can reduce considerably potential consequences.

On the other hand, the warning system to the population must fulfil the following criteria (see **Figure 6.15**):

- It must cover, as a minimum, the population residing in the areas with less than 1 hour of warning time (wave arrival time), being preferable the areas with less than 2 hours of warning.
- Must be permanently operational, even during adverse conditions. Features like

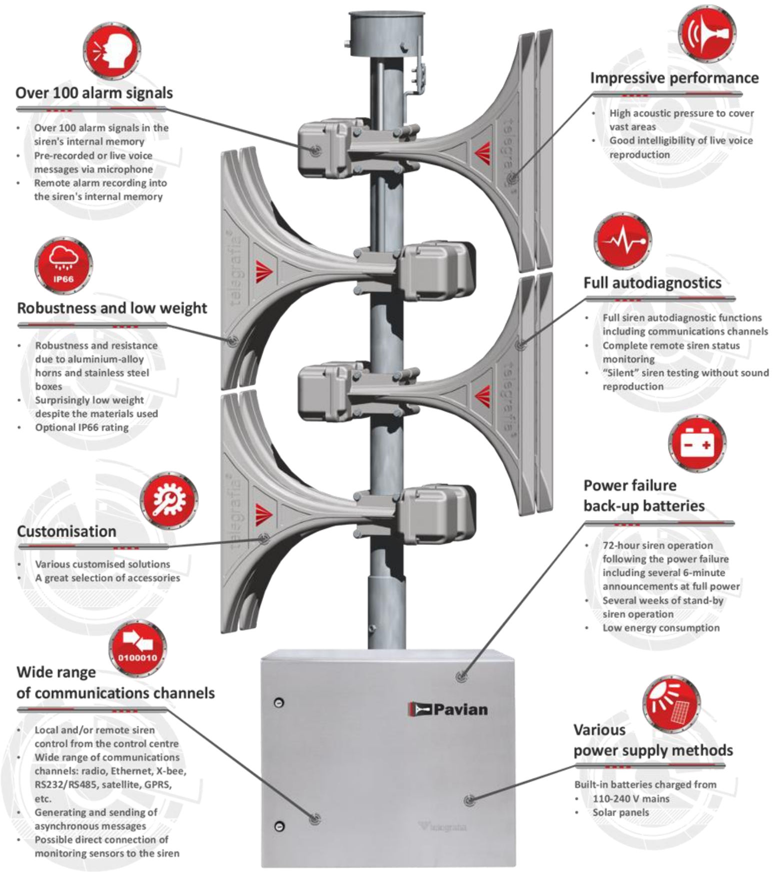


Figure 6.15.- Desirable features of a suitable sirens system within the floodplain (photo by telegrafia.eu)

robustness, resistance, redundant power supply and easy maintenance should be taken into account.

- Remote Access/communication
- There must be no false alarms
- Sirens with pre-recorded or live messages are the best option.

Emergency Management

A high level of coordination between emergency agencies and authorities will increase the effectiveness. This will result in prompt responses, larger warning times and efficient evacuating procedures providing shelter and assistance of flood emergency management.

Also, risk communication is the basis for an effective flood risk management. The combination of public education and risk communication will provide information to the public, increasing risk awareness and decreasing vulnerability.

Through a continuous risk-based approach, by planning for a dimensioning scenario and capability building for the emergency management organisation(s) we can reduce the losses from dam failure. **It is the process of continuous planning that will develop the organisations capability.**

6.5.5 Land Acquisition

One of the most common policies toward controlling floods has been focused primarily in structural measures not only in the dam but also in the floodplain through the construction of structures such floodwalls, embankments and levees along the banks of the downstream river and/or in the reservoir's rim.

While this structural approach undoubtedly reduced the severity of flooding in many communities, it also destroyed the natural capacity of floodplains to attenuate floods. Dams, floodwalls and levees may give people a false sense of security that previously flood-prone areas were safe for development (White 1945; Burby et al. 1985; Burby et al. 1988). In addition, these structural controls are normally expensive and is common to see that despite hundreds of crore rupees are spent on flood control measures, flood losses continue to occur, as more people and property become exposed to flooding.

For this reason, in some cases the implementation of land acquisition programs/solutions may represent an attractive and suitable option to accommodate an increased design flood by acquiring affected lands and reducing the potential consequences; ergo, a flood risk reduction. However, is also important to consider all the aspects that could affect the decision-making process of a particular community or homeowner about whether to participate (or not) in the buyout. A host of social, economic, cultural and political factors including the extent of the flood damage, the likelihood of future flooding and homeowner's ties to the neighbourhood could play an important role in the negotiation.

In general, inhabitants of a particular flood-prone area, would agree to participate in an acquisition process when:

1. They are well aware of the flood risk;
2. They believe they will benefit personally; and,
3. They have a low attachment to the community

Other factors include the acquisition price, availability of equivalently priced houses out of the floodplain, severity of the flood, and flood frequency (Smith and Handmer 1986; Handmer 1985)

6.5.6 Integrated River Basin Approach

Experience has shown that effective measures for flood prevention and protection must be taken in the level of river basins and that it is necessary to take into account interdependence and interaction of effects of individual measures implemented along watercourses.

It is absolutely necessary to organise the water management systems and improve forecasting, flood defence measures and crisis management on a river basin basis, cutting across regional boundaries and the country borders. This should be done in co-operation with the relevant organisations in the fields of hydrology (Central Water Commission) and meteorology (India Meteorological Department), mitigation planning (National Disaster Management Authority), river control (Water Resources Departments), civil protection and crisis management units (National Response forces, Police Departments, etc.).

Promoting the inter-agency co-operation among the different dam's owners/ operators within a basin, certainly can reduce the flood risk. Actions like developing a comprehensive flood management plan for the entire basin, irrespective of the dam's ownership, along with common control and command centres to receive all important data, and to effectively operate the integrated flood forecasting system during any extreme flood represents the state of the art in this field.

Integrated Flood Management

Integrated Flood Management is a process that promotes an integrated – rather than fragmented – approach to flood management. It integrates land and water resources development in a river basin, within the context of integrated water resources, and aims at maximizing the net benefits from the use of floodplains and minimizing loss of life from flooding (WMO, 2009)

Integrated Flood Management recognizes the river basin as a dynamic system in which there are many interactions and flux between land and water bodies (Figure 6.16). In an integrated flood management (IFM) the starting point is a vision of what the river basin should be. Incorporating a sustainable livelihood perspective means looking for ways of working towards identifying opportunities to enhance the performance of the system as a whole. The flows of water, sediment and pollutants from the upper catchments of the river into the coastal zone (ridge to reef) – often taken to extend dozens of kilometres inland and to cover much of the river basin – can have significant consequences. As estuaries embrace both the river basin and the coastal zone, it is important to integrate coastal zone management into IFM. Figure 6.17 depicts an IFM model.

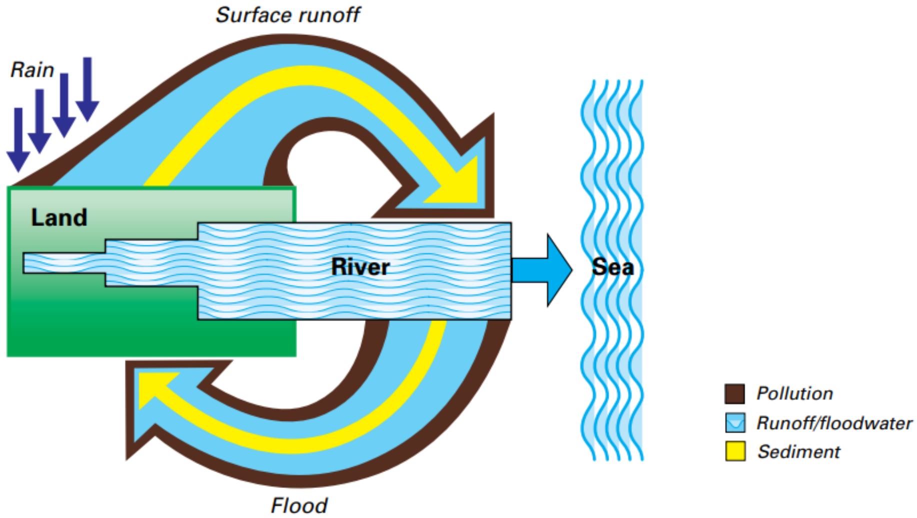


Figure 6.16.- Interaction between land and water (adapted from WMO, 2009).

Elements of Integrated Flood Management

Integrated Flood Management takes a participatory, cross-sectoral and transparent approach to decision-making. The defining characteristic of IFM is integration, expressed simultaneously in different forms: an appropriate mix of strategies, carefully selected points of interventions, and appropriate types of interventions (structural or non-structural, short- or long-term).

An Integrated Flood Management plan should address the following six key elements that follow logically for managing floods in the context of an IWRM approach:

- Manage the water cycle as a whole;
- Integrate land and water management.
- Manage risk and uncertainty.
- Adopt a best mix of strategies.
- Ensure a participatory approach; and
- Adopt integrated hazard management approaches

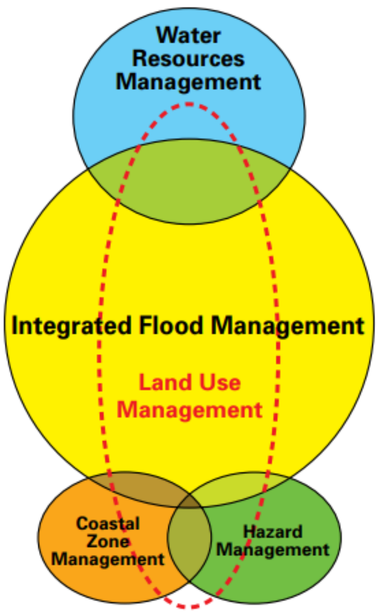


Figure 6.17.- Integrated flood management model (adapted from WMO, 2009)

Appendix A . CASE STUDY FOR SELECTING IDF BY INCREMENTAL CONSEQUENCES ANALYSIS METHOD

Disclaimer: The following Case Study was developed with the limited information available in the public domain and the salient features available in the Central Project Management Unit of DRIP project. Therefore, the report presented herein along with the results obtained are meant to provide an illustrative example of incremental consequences analysis method for IDF selection. Procedure and results are not intended to replace the actual design flood review report and corresponding conclusions of the specified dam.

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DEVARABILLIKERE DAM (Karnataka)

Introduction

Devarabelakere dam is an irrigation project situated in Devarabelakere village in Harihara Tq, of Davangere District at a Latitude of 14^{0} 24` 0`` N and Longitude of 750 500 00 E. The pickup is at a distance of 10.00 Kms from Malebennur. The project is providing assured irrigation to 4310 ha. In Khariff & 1942 Ha in Rabi season. This is a minor Irrigation project, completed in the year 1985.

Devarabelakere dam and reservoir are owned and operated by Karnataka Neeravari Nigam Limited. The construction commenced during the year 1978-79 and completed in 1986-87. It is constructed across a local stream formed after merging of two local streams Shyagale Halla and Shantisagara Halla, which are tributaries of Tungabhadra River flowing in Krishna basin near Devarabelakere village in Harihara taluk of Davanagere district. The stream across which the dam is constructed has a total catchment area of 2286.08 sq km (893 sq miles) upto the dam site. Average annual rainfall in the catchment is 609 mm (24 inches). The average annual yield of 216.34 MCM(7.64TMC) at 75% dependability available from the catchment out of which 45.56MCM (1.609 TMC) is utilized for irrigating an atchkat of 4280 ha (10576 acres).

The dam is an earthen dam with zonal section and central spillway having a total length of 1245 m. Length of left bank earthen dam is 940 m and right bank earthen dam is149.60 m long. The length of central concrete spillway is 155.40 m. The maximum height of the earthen dam is 17.37 m (RL 549.55 m – RL 532.18 m) above lowest foundation level and 17.07 m (RL 549.55 m – RL 532.48 m) above deepest river bed level for the earthen dam section.

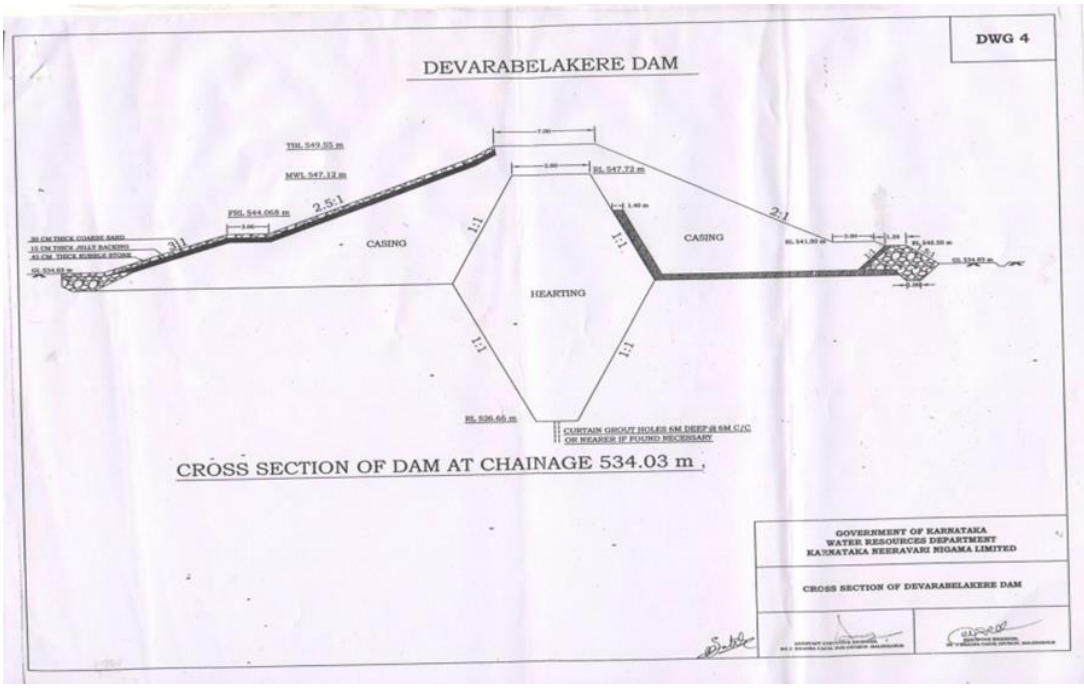


Figure A.1.- Typical Cross Section of Devarabelakere Dam

The sanctioned project envisaged construction of outlets arrangements consisting of a concrete ogee spillway having length of 155.40 m designed to discharge a flood of 1699 Cumec under a spillage depth of 3.06 m over its crest kept at RL 544.07 m. However later it was proposed to provide an RCC road bridge to create communication facility for vehicular movement between villages and towns situated on either sides of the valley. Accordingly, a roadway was formed over the spillway in the year 1986 by providing 14 number of piers each of 0.90 m thickness, thereby reducing the linear water way to 142.80 m. Subsequently from security point of view it was decided not to allow public traffic over the dam for which a new bridge had been constructed in the year 2002 across the main valley downstream of the spillway draft channel connected to the existing roads on either banks by providing a diversion road. In the year 2014-15, 30 Nos of Godbole type automatic radial gates of size 2 x 4.435 x 0.60 m have been installed on the crest of the spillway to increase storage capacity of the reservoir from 2.62 MCM to 5.23 MCM. A maximum flood discharge of 1361.72 cumec (48089 cusecs) has passed over the spillway in the year 1992.

Table 1.- Dam and Reservoir Salient Features

Sl. No.	Item	Details
a.	Full Reservoir Level (m)	544.66
b.	Original Maximum Water Level (m)	547.12
c.	Gross Reservoir Storage Capacity at FRL (Mm3)	5.23
d.	Live Storage Capacity (Mm3)	4.97
e.	Revised Live Storage Capacity, if any (Mm3)	4.97
f.	Date of bathymetric survey, if any	Nil
g.	Dam Type	Earthen dam with concrete Spillway
h.	Length of Dam at Top (m) i) Total length of the main dam ii) Length of embankment dam iii) Length of masonry/concrete dam	 1245 1089.6 155.40
i.	Number and length of dykes (No. & m)	Nil
j.	Top of dam (El. in m.)	549.50
k.	Top Level of Upstream Parapet Wall of main dam (El. in m.)	-
l.	Height of Dam (m) i) Embankment dam – above river bed level (up to dam top without camber) ii) Concrete/Masonry dam – above deepest foundation level (up to dam top)	 15.25 22.89
m	Top width of main dam (m)	7.0
n.	Spillway details	
	Location	Right Flank
	Type of spillway	Ogee
	Length of spillway (m)	155.40
	Spillway crest level (m)	544.07

Sl. No.	Item	Details
	Type of Gate	God bole type automatic radial gates
	Number and size of gates (no. and m. x m.)	15, 4.435 X 2
	Number and thickness of piers (no. and m. x m.)	13, 1.55 m thick
o.	Outlet/Sluice details i) In Embankment dam • Number • Size (Width (m). x Height (m).) • Location • Invert level El. (m) • Discharging capacity (m3/s) ii) In Concrete/Masonry dam • Number • Size (m. x m.) • Location • Invert level El. (m) • Discharging capacity (m3/s)	 2, RBC and LBC 2.40 x 1.20 and 2.0 X 1.20 Right flank and Left Flank 542.54 and 542.54 2.78 and 1.51 1 0.60 m dia. vent sluice. Middle of the Dam 539.50 1.12

Hydrology

The original design flood for Devarabillikere dam was 1,699 cumecs with corresponding MWL of 547.12 m. Under the Dam Rehabilitation and Improvement Project (DRIP) a revised design flood was estimated as part of the hydrologic safety review. The criteria of selection of the inflow design flood for safety of dam used under DRIP is given as per IS: 11223-1985 (Indian standard on guidelines for fixing spillway capacity), which was adopted by the then Indian Standards Institution (now BIS) on 13 February 1985 and reaffirmed in 1995

The revised design flood (SPF) under DRIP worked out to be 6,377 cumecs, which is 3.7 times the original design flood (Table 2). Flood routing study carried out by CPMU as part of Dam Break Analysis (DBA) indicates that the MWL for the revised design flood will surpass the TBL by 2.3 m. Due to this increment of 3.7 times the original design flood, Dam's authorities were instructed to determine mitigation measures to accommodate the revised design flood, which are still awaited.

Table 2.- Design Flood Review carried out under DRIP project

Sl. No.	Item	Original Value	Revised Value	Remarks
a.	Inflow Design Flood (m^3/s)	1,699	6,377	SPF
b.	Routed Outflow for the IDF (m^3/s)	1,699	6,251	Flood routing study by CPMU as part of DBA
c.	Maximum Water Level (m)	547.11	551.85	TBL = 549.55

Incremental Consequences Analysis

Considering the reduced volume of Devarabillikere's reservoir (5.23 MCM) when compared to the revised design flood hydrograph (413 MCM), the incremental consequences analysis may result appropriate to guide dam's authorities into the selection of a less stringent equivalent design flood, which will be a reflection of the actual hydrologic hazard posed by the dam.

For the incremental consequence analysis of Devarabillikere Dam an iterative process for different inflow magnitudes was carried out, with the purpose of defining the flood above which there is none or negligible increase in downstream inundation consequences due to failure of the dam when compared to the same flood without dam failure. Since no frequency analysis has been carried out for this project, the use of flood hydrograph for different return periods or exceedance probabilities was not possible in the present study. Instead, inflow design flood alternatives were analysed as a fraction of the current standard project flood (SPF).

Alternative flood magnitudes represented as the 90, 75, 60, 50, 35 and 20% of the SPF volume were evaluated (Figure 2 and Table 3). For each alternative, consequences in terms of population at risk, affected agricultural, residential, and industrial areas were estimated using the flood severity approach (depth x velocity) described in Figure 3 and Table 4. For population at risk and agricultural losses, hazard vulnerability classes from H3 to H6 were considered, while for economic consequences (residential and industrial areas) only H5 and H6 were counted.

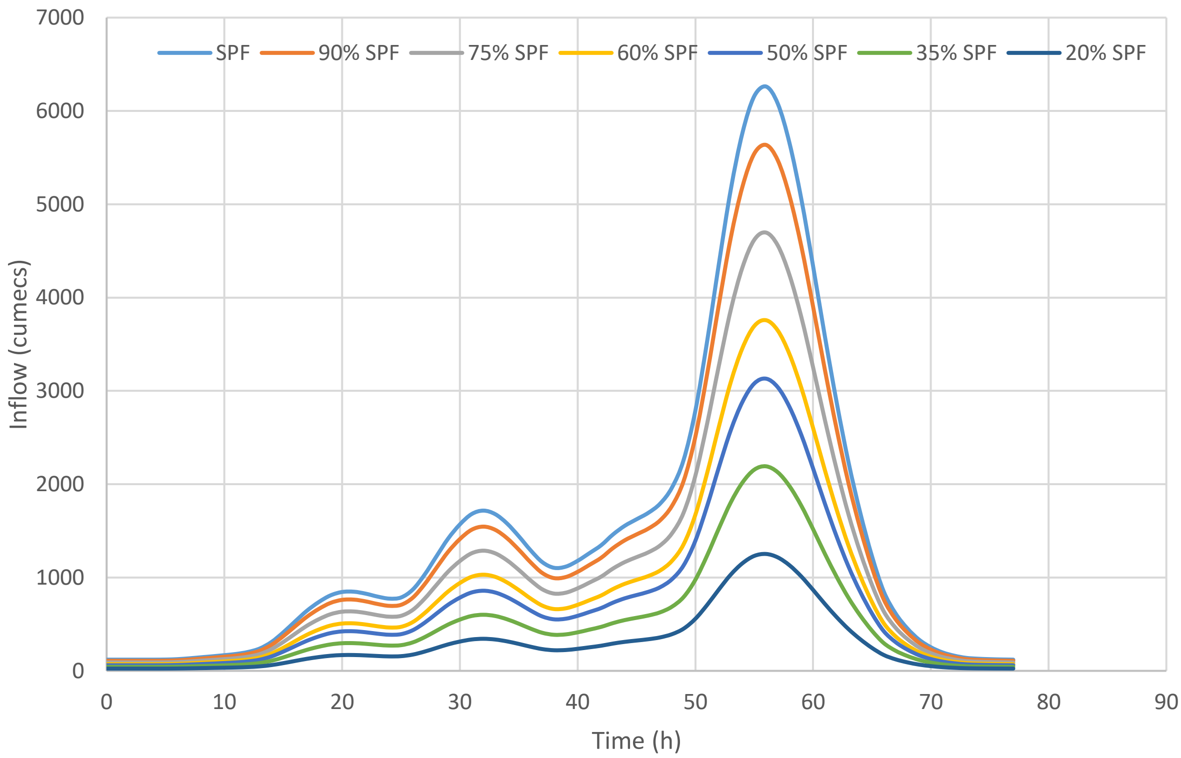


Figure A.2.- Inflow Design Flood Hydrographs.

Table 3.- Inflow Design Flood Peaks.

SPF	90% SPF	75% SPF	60% SPF	50% SPF	35% SPF	20% SPF
6,262	5,636	4,697	3,757	3,131	2,192	1,252

Table 4.- Vulnerability thresholds classification limits [a]

Hazard Vulnerability Classification	Description	Classification Limit (Depth * Velocity)	Limiting Water Depth (m)	Limiting Velocity (m/s)
H1	Generally safe for vehicles, people and buildings.	D*V ≤ 0.3	0.3	2.0
H2	Unsafe for small vehicles.	D*V ≤ 0.6	0.5	2.0
H3	Unsafe for vehicles, children and the elderly.	D*V ≤ 0.6	1.2	2.0
H4	Unsafe for vehicles and people.	D*V ≤ 1.0	2.0	2.0
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure.	D*V ≤ 4.0	4.0	4.0
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.	D*V > 4.0	-	-
[a] Combined Hazard – Vulnerability Classification (Smith et al., 2014)				

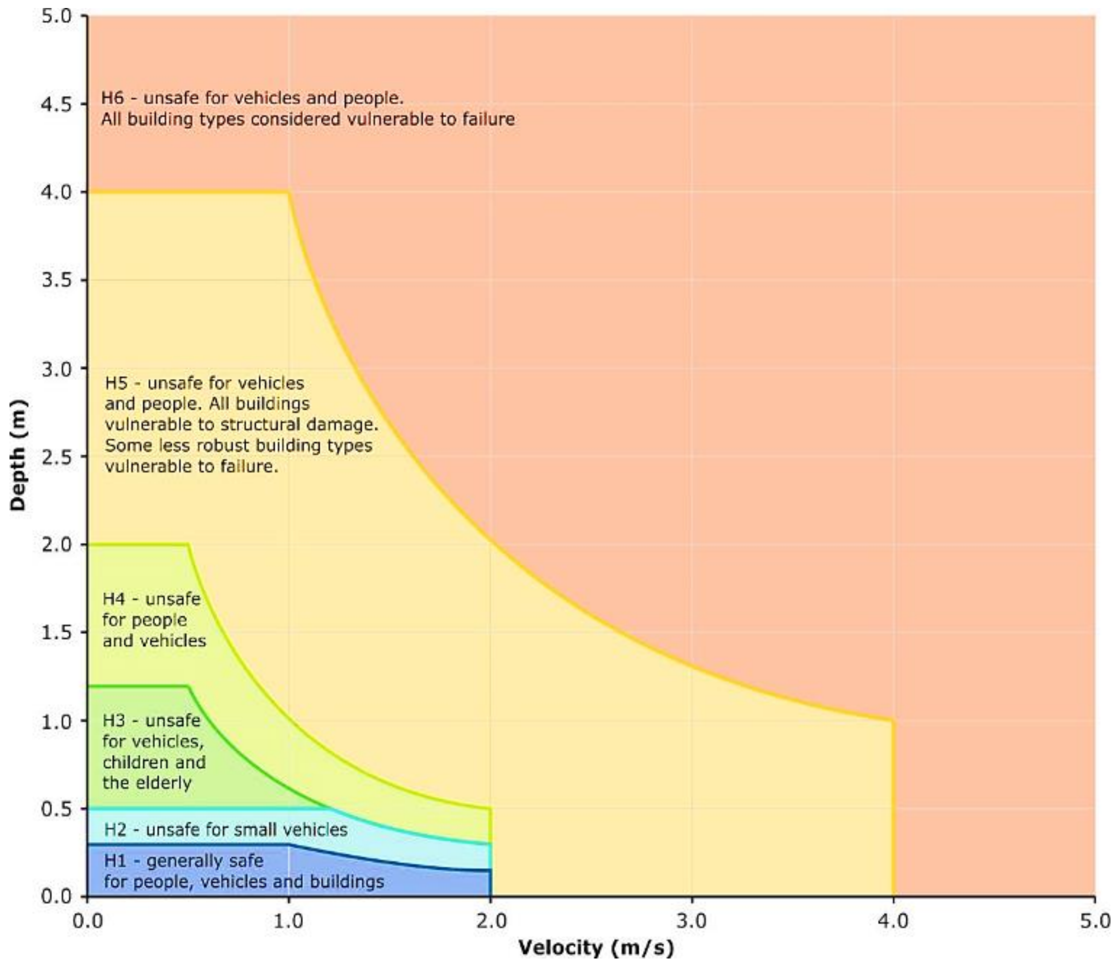


Figure A.3.- Flood Hazard Vulnerability Curves (Smith et al. 2014)

It worth to mention that for each flood alternative/iteration, spillway's discharge capacity was upgraded in order to maintain the original MWL (El 547.11 m). Therefore, all failure scenarios were triggered using the same reservoir maximum water level and dam breach parameters (Table 4). Results obtained after the iterative process of the analysis of the incremental consequences are shown in Figures 4 to 6, and the summary inundation map located at the end of this appendix.

Table 4.- Trapezoidal Dam Breach Model Parameters [a]

Breach parameter	Units	Dam Failure Mode Flood-induced
Breach Height	m	17.4
Bottom width	m	143
Average side slope (horz : vert)	--	1:1
Formation time	Hrs.	1.87

[a] Parameters of the trapezoidal dam breach model used in HEC-RAS (Brunner 2016)

Inflow Design Flood Selection

After analysing the incremental consequences (population at risk and economic impact) for the different inflow scenarios at Devarabillikere Dam, it can be concluded that the inflow design flood imposed by the prescriptive approach of the Bureau of Indian Standards (IS: 11223-1985), which is based solely on the physical characteristics of the dam (Reservoir volume and height), it leads to a more stringent IDF when compared to the value obtained after considering the incremental impact in the entire floodplain. As shown in Figures 4 to 6, an IDF of about 70-75% of the SPF (Between 4,380 and 4,690 cumecs) could be considered an acceptable design flood, and a higher IDF would not be justified or in accordance with the actual hazard potential of the dam.

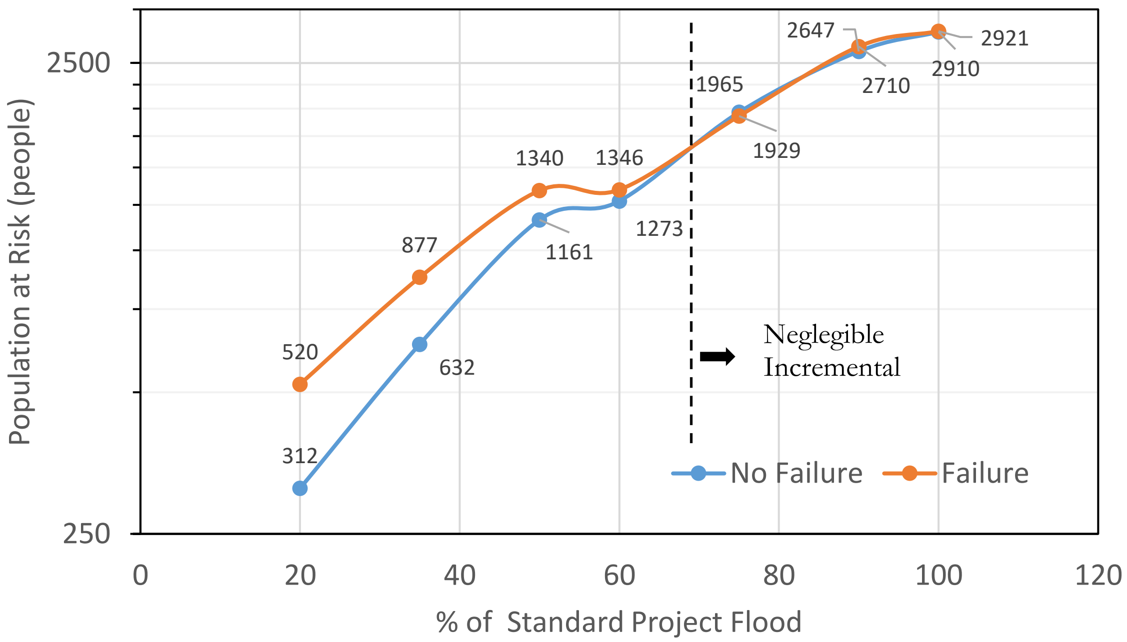


Figure A.4.- Population at Risk for different routed inflow hydrographs (as % of the revised SPF) with and without failure.

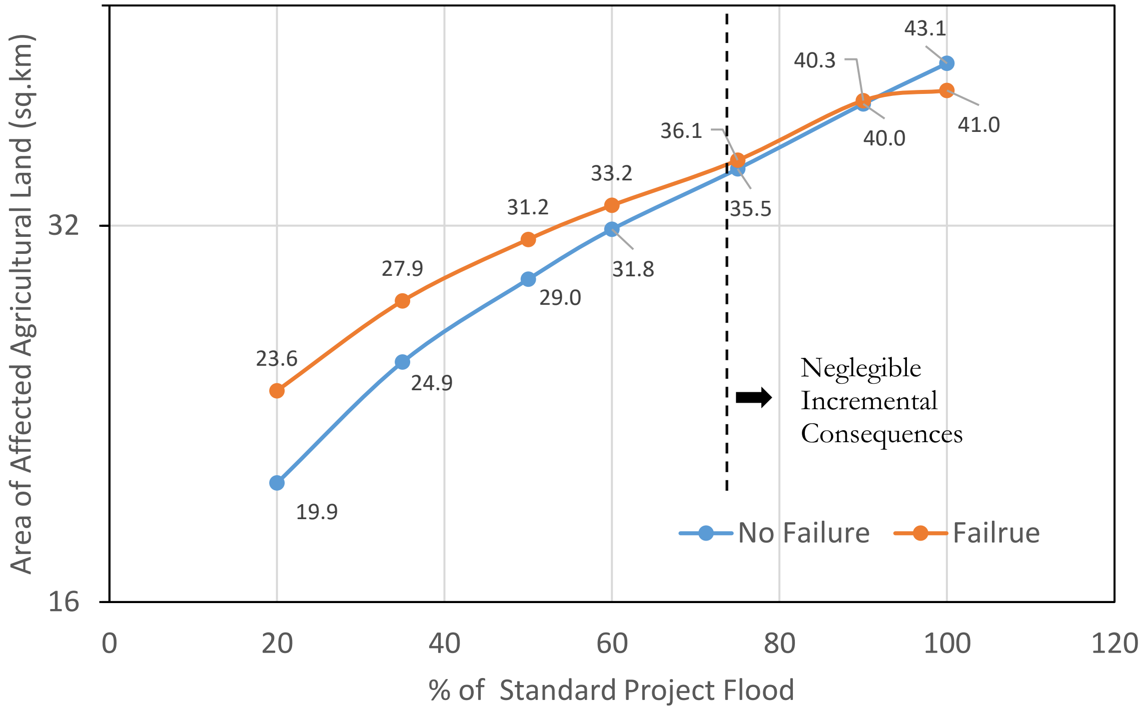


Figure A.5.- Affected Agricultural Land for different routed inflow hydrographs (as % of the revised SPF) with and without failure.

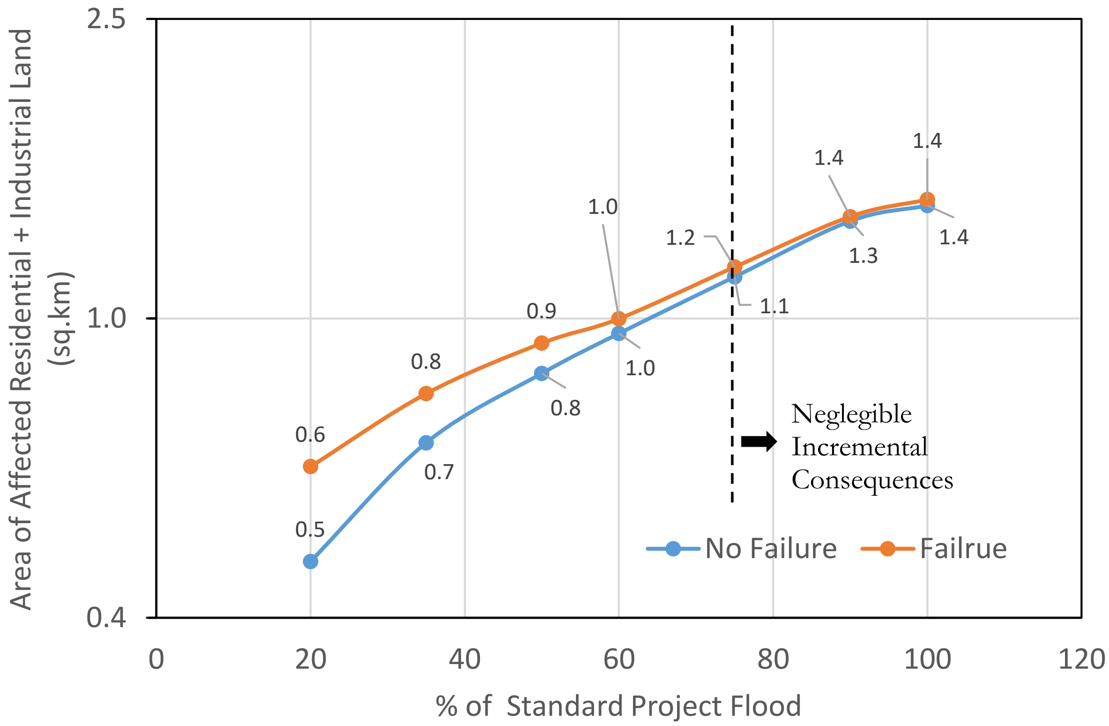
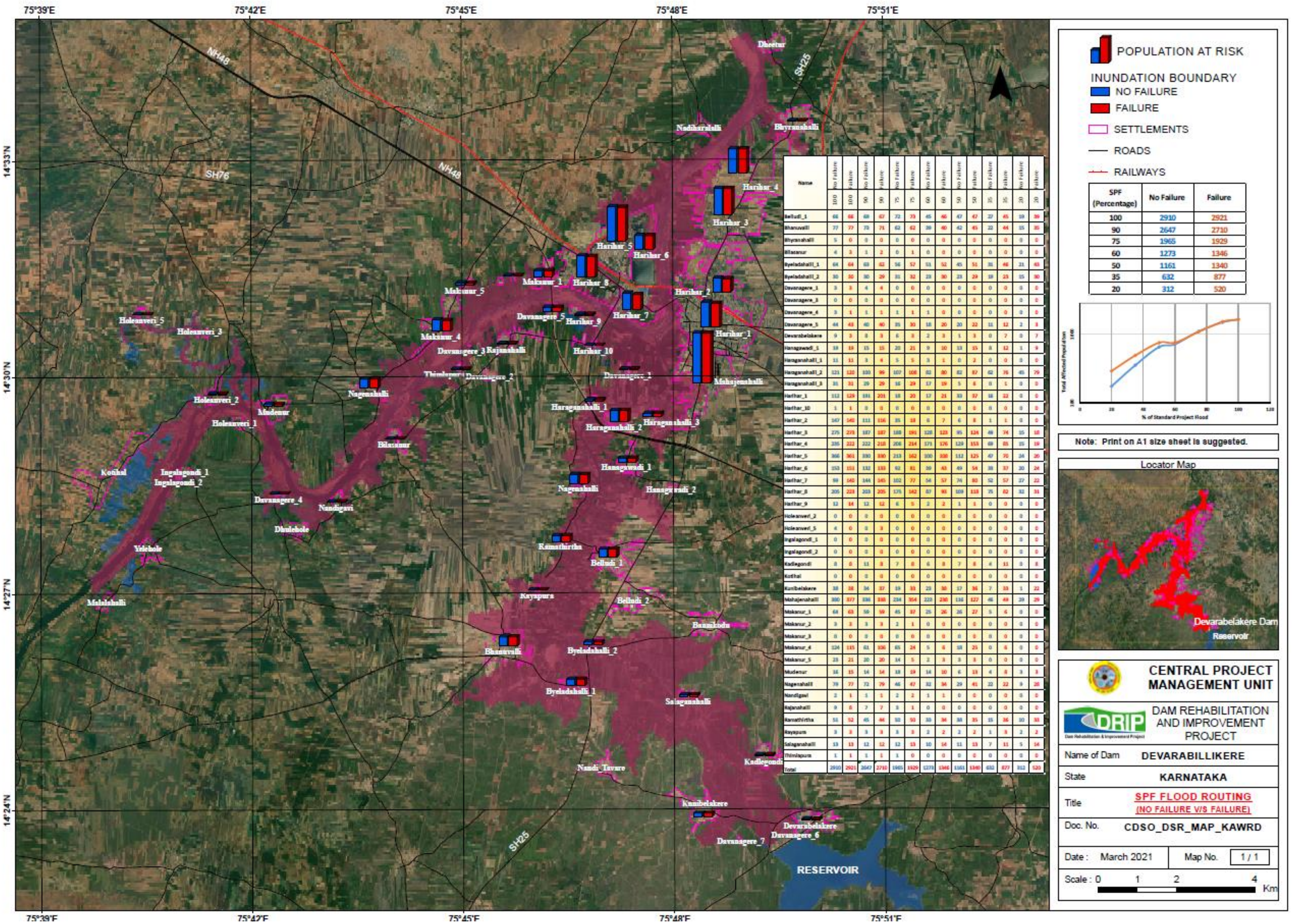
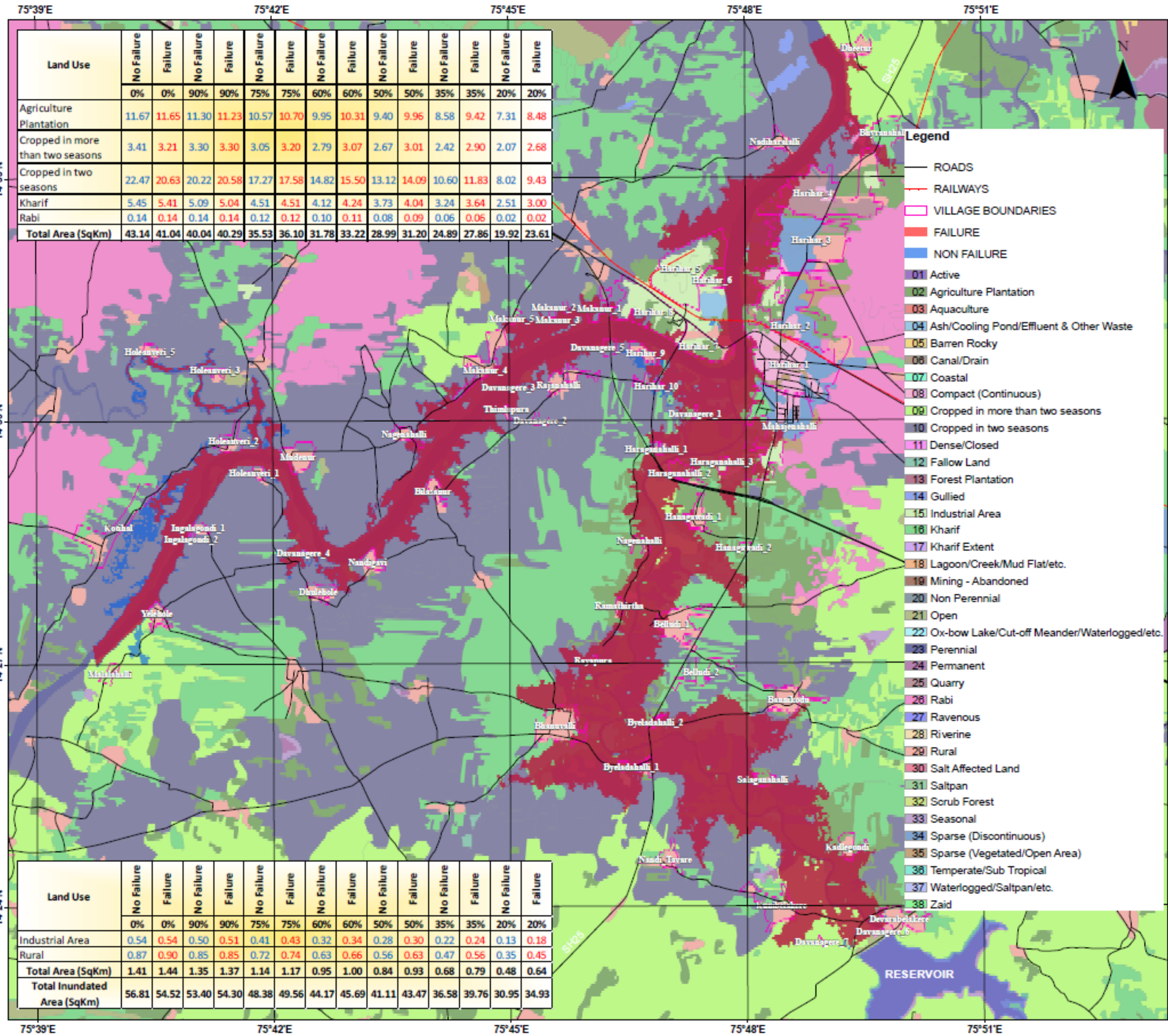


Figure A.6.- Affected Residential and Industrial Land for different routed inflow hydrographs (as % of the revised SPF) considering failure and no failure of the dam.

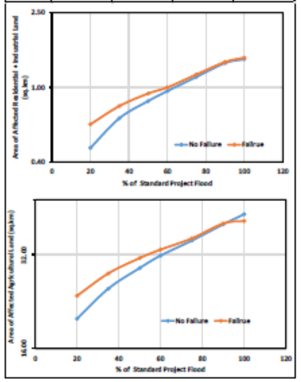

75°39'E
75°42'E
75°45'E
75°48'E
75°51'E
14°33'N
14°30'N
14°27'N
14°24'N
NH48
SH76
SH25
RESERVOIR
Harihar_1
Harihar_2
Harihar_3
Harihar_4
Harihar_5
Harihar_6
Harihar_7
Harihar_8
Harihar_9
Harihar_10
Makanur_1
Makanur_4
Makanur_5
Davanagere_1
Davanagere_2
Davanagere_3
Davanagere_4
Davanagere_5
Davanagere_6
Davanagere_7
Holeanveri_1
Holeanveri_2
Holeanveri_3
Holeanveri_5
Ingalagondi_1
Ingalagondi_2
Haraganahalli_1
Haraganahalli_2
Haraganahalli_3
Hanagawadi_1
Hanagawadi_2
Byeladahalli_1
Byeladahalli_2
Belludi_1
Belludi_2
Mahajenahalli
Nagenahalli
Bhanuvalli
Kunibelakere
Devarabelakere
Salaganahalli
Kadlegondi
Ramathirtha
Rajanahalli
Nandigavi
Dhulehole
Yelehole
Malalahalli
Kotihal
Mudenur
Bilasanur
Thimlapura
Keyapura
Bannikodu
Nandi_Tavare
Nadiharalalli
Bhyranahalli
Dhoolar
POPULATION AT RISK
INUNDATION BOUNDARY
NO FAILURE
FAILURE
SETTLEMENTS
ROADS
RAILWAYS
SPF (Percentage) | No Failure | Failure
100 | 2910 | 2921
90 | 2647 | 2710
75 | 1965 | 1929
60 | 1273 | 1346
50 | 1161 | 1340
35 | 632 | 877
20 | 312 | 520
Total Affected Population
% of Standard Project Flood
Note: Print on A1 size sheet is suggested.
Locator Map
Devarabelakere Dam
Reservoir
CENTRAL PROJECT MANAGEMENT UNIT
DRIP
DAM REHABILITATION AND IMPROVEMENT PROJECT
Name of Dam DEVARABILLIKERE
State KARNATAKA
Title SPF FLOOD ROUTING (NO FAILURE V/S FAILURE)
Doc. No. CDSO_DSR_MAP_KAWRD
Date : March 2021
Map No. 1 / 1
Scale : 0 1 2 4 Km

Land Use	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure
	0%	0%	90%	90%	75%	75%	60%	60%	50%	50%	35%	35%	20%	20%
Agriculture Plantation	11.67	11.65	11.30	11.23	10.57	10.70	9.95	10.31	9.40	9.96	8.58	9.42	7.31	8.48
Cropped in more than two seasons	3.41	3.21	3.30	3.30	3.05	3.20	2.79	3.07	2.67	3.01	2.42	2.90	2.07	2.68
Cropped in two seasons	22.47	20.63	20.22	20.58	17.27	17.58	14.82	15.50	13.12	14.09	10.60	11.83	8.02	9.43
Kharif	5.45	5.41	5.09	5.04	4.51	4.51	4.12	4.24	3.73	4.04	3.24	3.64	2.51	3.00
Rabi	0.14	0.14	0.14	0.14	0.12	0.12	0.10	0.11	0.08	0.09	0.06	0.06	0.02	0.02
Total Area (SqKm)	**43.14**	**41.04**	**40.04**	**40.29**	**35.53**	**36.10**	**31.78**	**33.22**	**28.99**	**31.20**	**24.89**	**27.86**	**19.92**	**23.61**



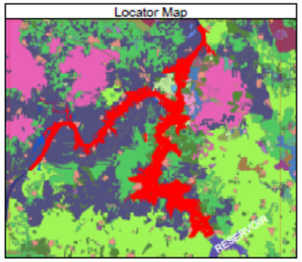
Land Use	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure	No Failure	Failure
	0%	0%	90%	90%	75%	75%	60%	60%	50%	50%	35%	35%	20%	20%
Industrial Area	0.54	0.54	0.50	0.51	0.41	0.43	0.32	0.34	0.28	0.30	0.22	0.24	0.13	0.18
Rural	0.87	0.90	0.85	0.85	0.72	0.74	0.63	0.66	0.56	0.63	0.47	0.56	0.35	0.45
Total Area (SqKm)	**1.41**	**1.44**	**1.35**	**1.37**	**1.14**	**1.17**	**0.95**	**1.00**	**0.84**	**0.93**	**0.68**	**0.79**	**0.48**	**0.64**
Total Inundated Area (SqKm)	**56.81**	**54.52**	**53.40**	**54.30**	**48.38**	**49.56**	**44.17**	**45.69**	**41.11**	**43.47**	**36.58**	**39.76**	**30.95**	**34.93**

	Agriculture		Residential +Industrial Area	
	No Failure	Failrue	No Failure	Failrue
100	43.14	41.04	1.41	1.44
90	40.04	40.29	1.35	1.37
75	35.53	36.10	1.14	1.17
60	31.78	33.22	0.95	1.00
50	28.99	31.20	0.84	0.93
35	24.89	27.86	0.68	0.79
20	19.92	23.61	0.48	0.64



Note: Print on A1 size sheet is suggested.

Locator Map



CENTRAL PROJECT MANAGEMENT UNIT

DRIP

DAM REHABILITATION AND IMPROVEMENT PROJECT

Name of Dam	DEVARABILLIKERE
State	KARNATAKA
Title	SPF FLOOD ROUTING (NO FAILURE V/S FAILURE)
Doc. No.	CDSO_DSR_MAP_KAWRD
Date : March 2021	Map No. 1 / 1
Scale : 0 1 2 4 Km	

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Appendix B . CASE STUDY FOR SELECTING IDF BY RISK-INFORMED HYDROLOGIC HAZARD ANALYSIS

Disclaimer: The following case study is solely a hypothetical but educational exercise and does not reflect an actual assessment. Although the case study is based on an actual Indian reservoir (Chickkahole Dam, Karnataka), it was developed only to illustrate a risk-informed hydrologic hazard analysis. Input data, events, assumptions, and results have been modified to fulfil and illustrate the procedures described in these guidelines. Procedure and results are not intended to replace the actual hydrologic hazard analysis and corresponding risk mitigation measures for the specific dam.

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CHIKKAHOLE DAM (Karnataka)

Introduction

The Chikkahole Reservoir Project is one of the Medium Irrigation Projects taken up by the Government of Karnataka in the Cauvery Basin (Figure B.1). To the South of Chamarajanagar and close to Karnataka and Tamilnadu borders lies Thalamale and Kongalli betta range of Hills. Out of the number of streamflows from it, Dodda Halla and Haalubidda Halla take their origin at an elevation of 1100 m above MSL. Dodda Halla and Chikahole Halla have their source at Kongalli betta hills. Initially, the dam was constructed in 1969, having a catchment area of 157.00 sq. miles. The annual rainfall ranges from 25" to 90" per annum in the catchment. The dam was constructed by using surki mortar in Masonry. The total length of the dam is 674.50 m.

This Dam was functioning well up to December 1972, the date after which a heavy cyclonic storm occurred in the catchment area, and due to flash floods, the dam breached on the right flank (from Chainage 1965 to 2092). The breach occurred in the non-overflow section of the dam, and the lowest level of breach was at RL 741.50 m.

During this flood, a total of 2,182 houses were washed away, 13 human lives lost, 1,282 heads of cattle were washed away, and 9,341 acres of agricultural land and crops were damaged causing a total loss of about 2.56 crores approximately.

Based on the recommendations of a panel of experts constituted by the Government of Karnataka, the re-construction of Chikkahole dam was taken-up and all works completed during 1983-1984 at a revised cost of 350 Lakhs Rupees (Figure B.2 and Figure B.3). After the re-construction, utilization of water in Cauvery basin is now restricted to 274 TMC from previous 464 TMC. The gross utilization of Chikkahole Reservoir Project is 1.219 TMC, and the project provides irrigation facilities to the extent of 4,076 acres. The Gross storage capacity of Chikkahole Reservoir Project is 10.64 Mm3, and live storage of 10.53 Mm3. (Table B.1)

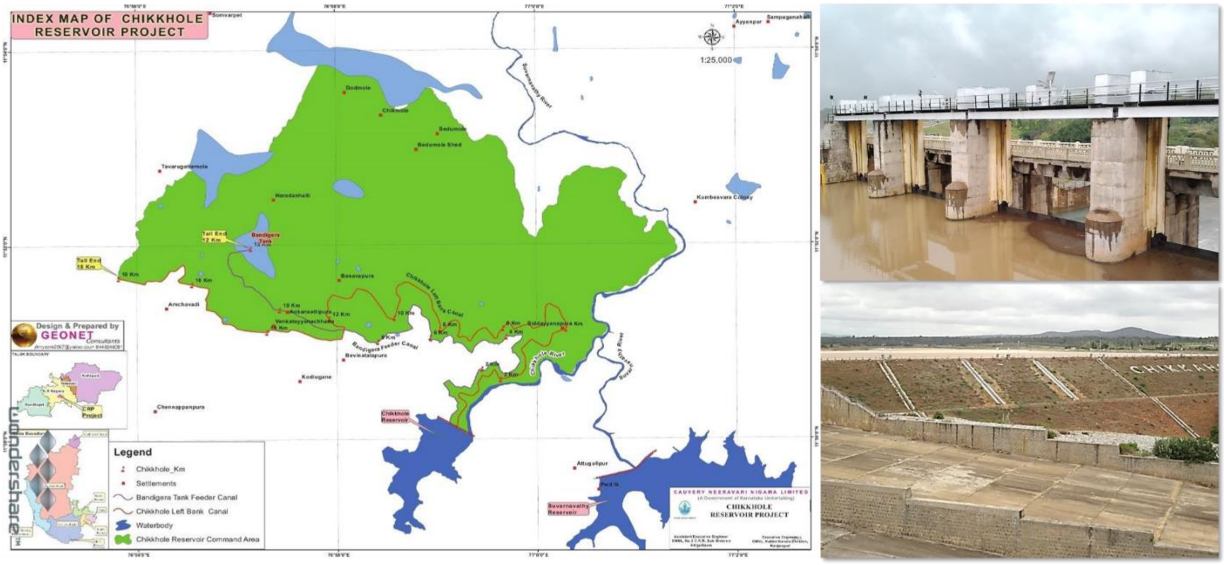


Figure B.1- General location and command area of Chikkahole Dam

Features and Current State of the Dam

The maximum design flood at Chikkahole Dam site, after its reconstruction, was fixed at 2,407 m^3/s. However, the current discharge capacity of all its surplus arrangements is 1,274 m^3/s at MWL (755.84 m), with contribution from service gated spillway (792.86 m3/s) and ungated side-channel spillway (481.38 m^3/s). See Table B.1 below for further details on salient features of Chikahhole Dam.

Table B.1.- General location and command area of Chikkahole Dam

Sl. No.	Item	Details
a.	Full Reservoir Level (m)	754.07
b.	Original Maximum Water Level (m)	755.83
c.	Gross Reservoir Storage Capacity at FRL (Mm^3)	10.64
d.	Live Storage Capacity (Mm^3)	10.53
g.	Dam Type	Earthfill Cum Masonry Dam
h.	Length of Dam at Top (m) i) Total length of the main dam ii) Length of embankment dam iii) Length of masonry/concrete dam	 750 633.6 116.4 (44.5 + 121.9, both spillways)
i.	Number and length of dykes (No. & m)	Nil
j.	Top of dam (El. in m.)	757.4
k.	Top Level of Upstream Parapet Wall of main dam (El. in m.)	-
l.	Height of Dam (m) i) Embankment dam – above river bed level (up to dam top without camber) ii) Concrete/Masonry dam – above deepest foundation level (up to dam top)	 24.1 30.5
m	Top width of main dam (m)	6.1
n.	Spillway details	
	i) Location	Right Bank
	ii) Type of spillway	Ogee (main & side channel)
	iii) Length of spillway (m)	44.5 (main), 121.9 (side channel)
	iv) Spillway crest level (m)	751.0 (main), 754.07 (side channel)
	v) Type of Gate	Vertical Lift (main), Ungated (side)
	vi) Number and size of gates (no. and m. x m.)	4 gates, 9.75 X 3.05 (main)
	vii) Number and thickness of piers (no. and m.	3, each 1.8 m wide
	viii) Discharge Capacity (m^3/s)	792.86 (main) + 481.38 (side) = 1,274 (combined)
o.	Outlet/Sluice details i) In Embankment dam • Number • Size (Width (m). x Height (m). • Location • Invert level El. (m) • Discharging capacity (m^3/s)	 2 Nos. 0.9 x 1.8 and 1.8 x 1.21 CH 225 m and CH 380 m 745.85 and 737 3.4 and 1.98

Under the Dam Rehabilitation and Improvement Project (DRIP), a design flood review was carried out by project authorities and the revised inflow design flood (SPF or standard project flood) worked out to be 4,654 m^3/s against its original value of 2,407 m3/s. Since the current discharge capacity (1,274 m^3/s) represents only 27% of revised IDF, flood routing studies were carried out by the SPMU. These studies revealed that the revised MWL is at EL 759.583 m. which is 2.44 m above the TBL (EL 757.39 m). Hence, in order to guarantee the hydrologic safety of the dam, it was recommended to consider structural and non-structural measures to accommodate the revised design flood, which are yet to be taken up by project authorities.

It worth to mention, that Chikkahole dam was reconstructed by encasing the old masonry dam with earth material, as shown in Figure B.2 and Figure B.3. As a result, some areas of the downstream slope of the earthen dam lay in the former stilling basin area, which is below the existing bed level. This design has created a condition of potential high pore pressures due to the lack of proper natural drainage, creating potential stability concerns for the downstream slope of the embankment dam. Under DRIP, this condition was addressed by performing stability analysis, geotechnical investigations of the downstream slope and installation of piezometers in different locations of the embankment for continuous monitoring through an automated datalogger (Figure B.4)

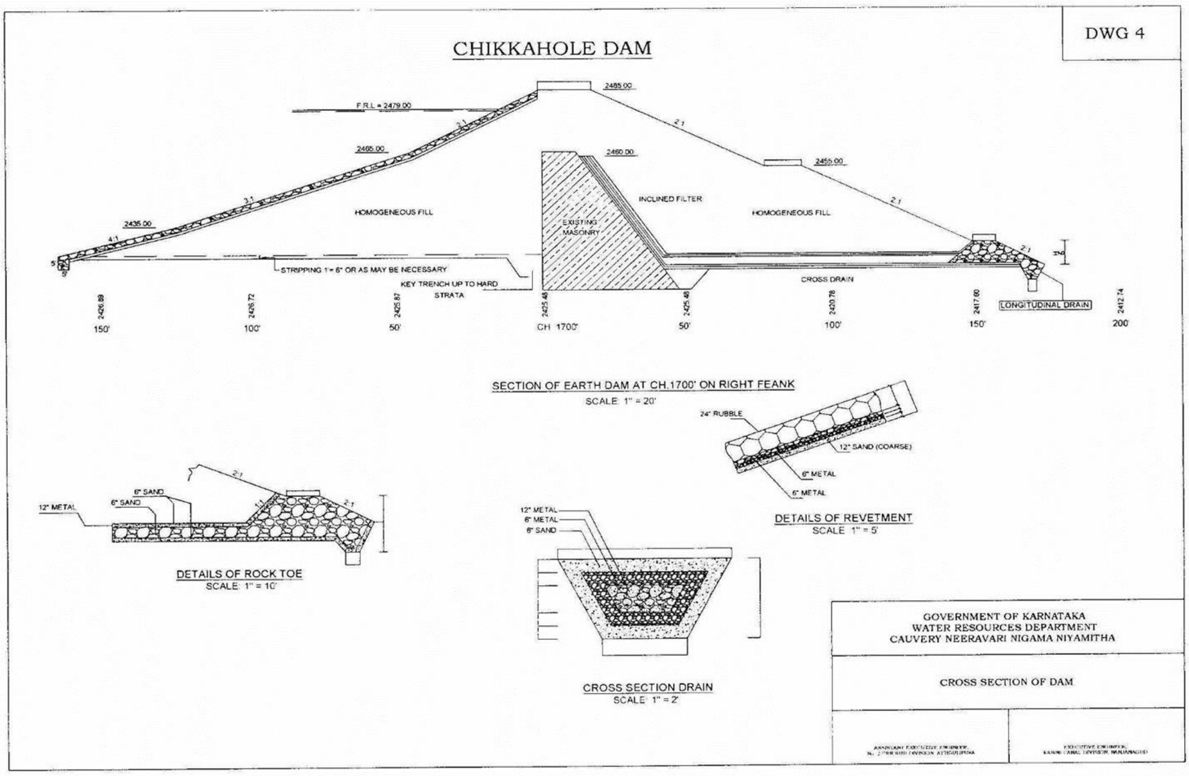


Figure B.2.- Section of earth dam after re-construction, using old masonry as impervious core

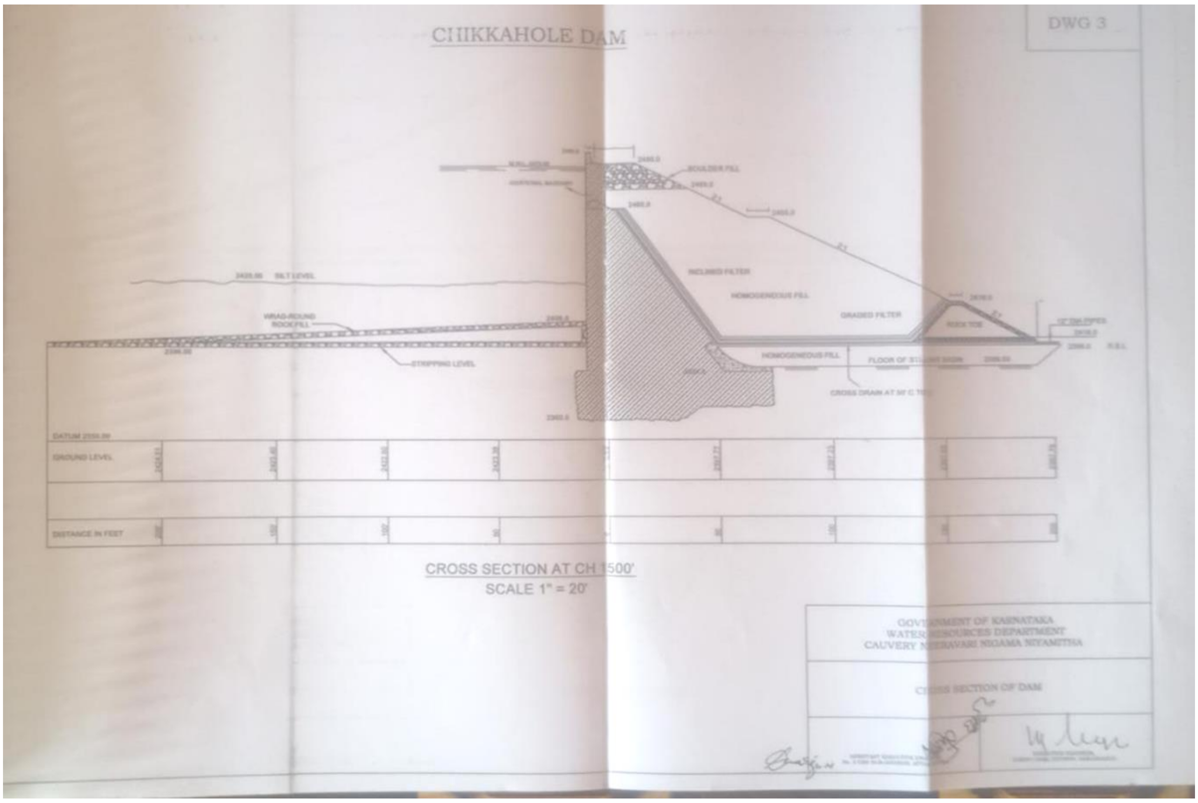


Figure B.3.- Section of earth dam after re-construction, used as backed material of old masonry dam

Figure B.4.- Installation of Pizometers in Embakment Dam.

Failure Modes Identification

A working group or panel of experts was established for analysing all available information, performing site visits and extensively debate on the current situation of the dam. As recommended in the published "*Guidelines for Assessing and Managing Risk Associated with Dams"* (CWC, 2019), the working group was constituted with a total of 15 officials from dam's organisation and several sessions were conducted, including individual and group phases. The working group proceeded with the failure mode identification and analysis.

Since the main objective of this risk evaluation was to evaluate hydrologic safety, it worth mentioning that only flood-induced failure modes were discussed in the working sessions. The remaining failure modes, related to normal operation or seismic events, will be discussed in the subsequent annual dam safety risk assessment report comprehensively.

"Most-likely" and "less-likely" aspects were evaluated and debated in the group sessions, and credible flood-induced failure modes identified by the working group are described in Table B.2:

Table B.2.- Summary of Failures modes identification process for Chikkahole Dam

Failure Mode No.	Failure Mode Description	Failure Mode Classification[a]	Remarks[b]
1	Overtopping of main dam crest and erosion of earth embankment during a flood event	Class B	Dam already breached in 1972 and revised IDF higher than current discharge capacity
2	Overtopping of training walls of main spillway's chute with further backwards erosion of earth embankment	Class C	Hydraulic performance of spillway outlet channel under revised design flood and other extreme floods is uncertain due to the lack of a hydraulic analysis
3	Internal erosion in the contact between overflow and non-overflow section during a flood event	Class C	Lack of adequate drawings and lack of instrumentation in this area of the dam
4	Sliding of embankment downstream slope due to high pore pressure caused by a long-duration flood event	Class B	After the dam breached in 1972, the old masonry portion was backed by the current embankment, which is located in the former stilling basin area

[a] As per Guidelines for Assessing and Managing risk Associated with Dams (CWC, 2019)
[b] Key remark after a comprehensive evaluation of "most-likely" and "less-likely" factors. This column does not represent the entire set of arguments used to classify the failure mode.

In summary, the working group members expressed concern on aspects like overtopping and sliding stability of the embankment dam under hydrologic loads, the emergency preparedness, hydrologic adequacy of outlet work, the effectiveness of drainage and reliability of instrumentation and monitoring system. In this line, failure modes No. 1 and No. 4 were classified as "Class B" for further risk quantification. Failure modes No. 2 and No.3 were not discarded by the working group, but further information is required in order to include them in full quantitative risk analysis.

Risk Model Architecture

The architecture of the risk model has been developed in an MS Excel spreadsheet using an event tree and its influence diagram, which allowed calculating the flood-induced risk estimates. This influence diagram links the failure modes identified by the working group (i.e. Overtopping and sliding of the embankment) with the probability of high-water levels in the dam (hydrological scenario) and the dam failure consequences. In addition, it also allows estimating the risk in the cases of non-failure of the dam. Figure B.5 and Figure B.6 show the simplified architecture of the risk model developed for the Chikkahole dam and the event tree developed in the spreadsheet, respectively.

For more details about how the probability of failure for FM No.1 and No.4 was estimated, please also refer to the "*Guidelines for Assessing and Managing Risk Associated with Dams*" (CWC, 2019)

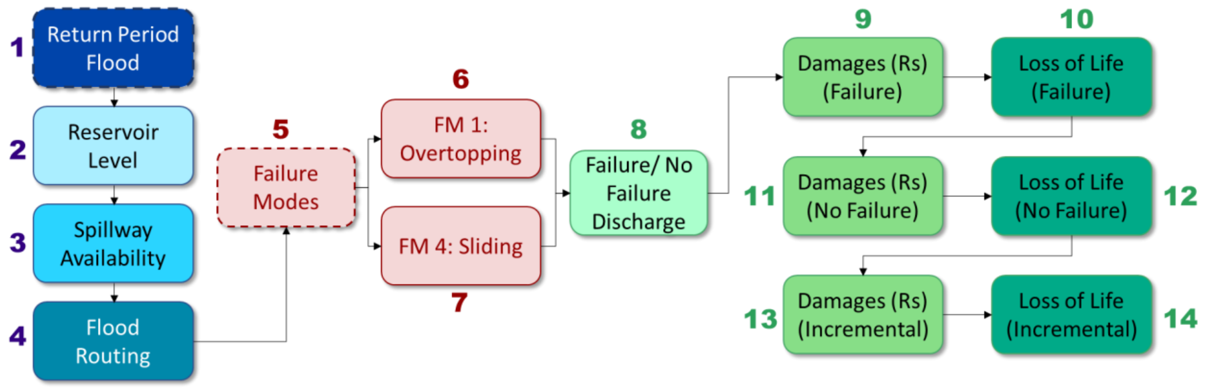
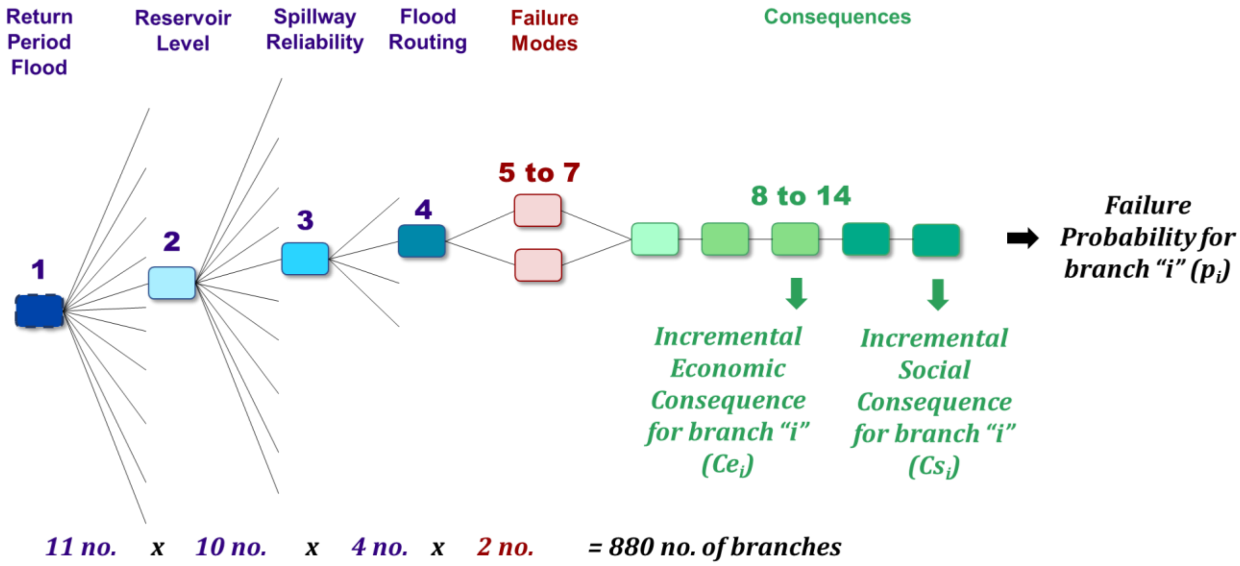


Figure B.5.- Architecture of the risk model of Chikkahole dam



$$Failure\ Probability = \sum_{i=0}^{n=880} p_i \qquad Economic\ Risk = \sum_{i=0}^{n=880} p_i.Ce_i \qquad Societal\ Risk = \sum_{i=0}^{n=880} p_i.Cs_i$$

Figure B.6.- Structured Event Tree used to estimate the Hydrologic Risk at Chikkahole Dam

Input Data: Loads

In this stage of the risk-informed hydrologic hazard assessment, all input data to build the risk model was collected and analysed. As key input, probabilistic flood hazard curves were estimated through a rainfall-based frequency analysis (Figure B.7 and Figure B.8)

For Chikkahole Dam, the hydrologic-hydraulic performance of the reservoir was analysed for different scenarios combinations using as input:

- Eleven (11) flood hydrographs with different return periods (Figure B.9)
- Ten (10) different reservoir initial pool levels (Figure B.10)
- Four (4) scenarios of gates' reliability (50%, 75%, 85%, 95%) (Table B.4)
- Two (2) failure modes

It worth mentioning that further scenarios considering climate change, rainfall/storm events duration, failure modes, surplus configuration, etc., can also be considered in subsequent and more detailed risk assessments for Chikkahole Dam. In total, 880 flood routing combinations (Figure B.6) were computed, obtaining the results of maximum water levels and peak outflows in the reservoir (Figure B.11). Hence, it was possible to characterise the hydraulic performance of the dam-reservoir system as a function of different variables and, thus, to be able to observe the influence of these situations in the results.

As an example of the flood routing calculations performed for each of the combinations, Figure B.11 presents the results obtained for the routing of the SPF, with a reservoir impinging level of 753 m, and considering the current discharge capacity of 1,274 m3/s (combined gated and ungated spillway.

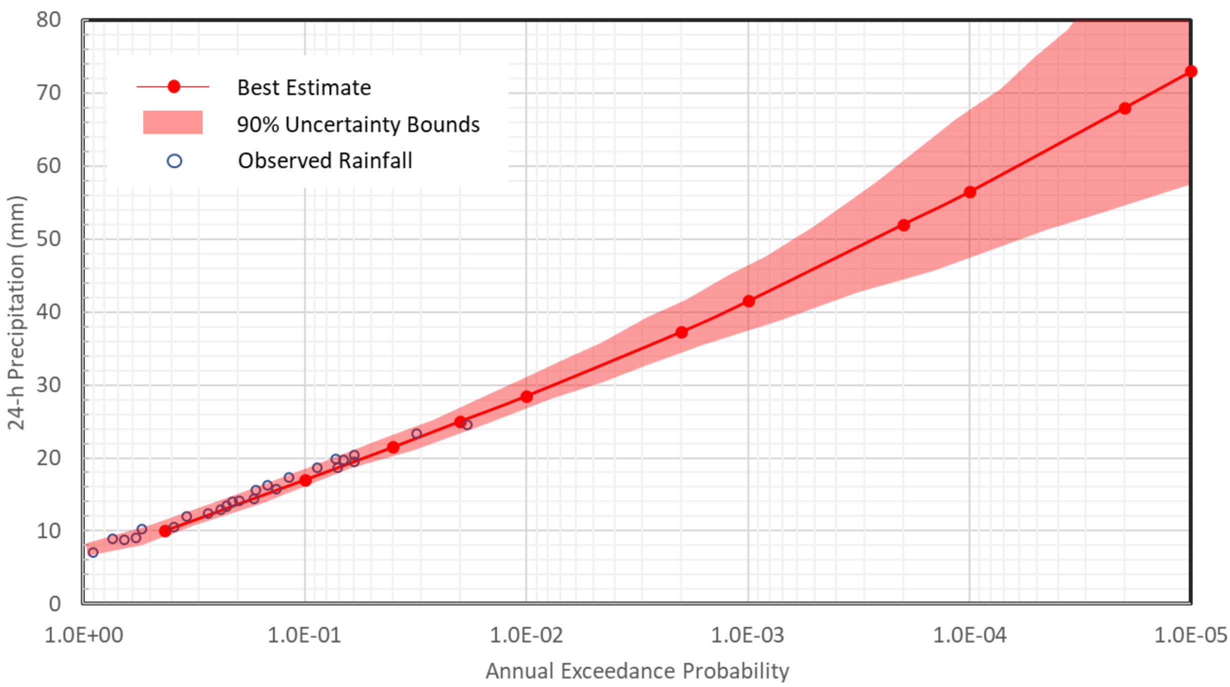


Figure B.7.- Precipitation-frequency analysis for Chikkahole Catchment

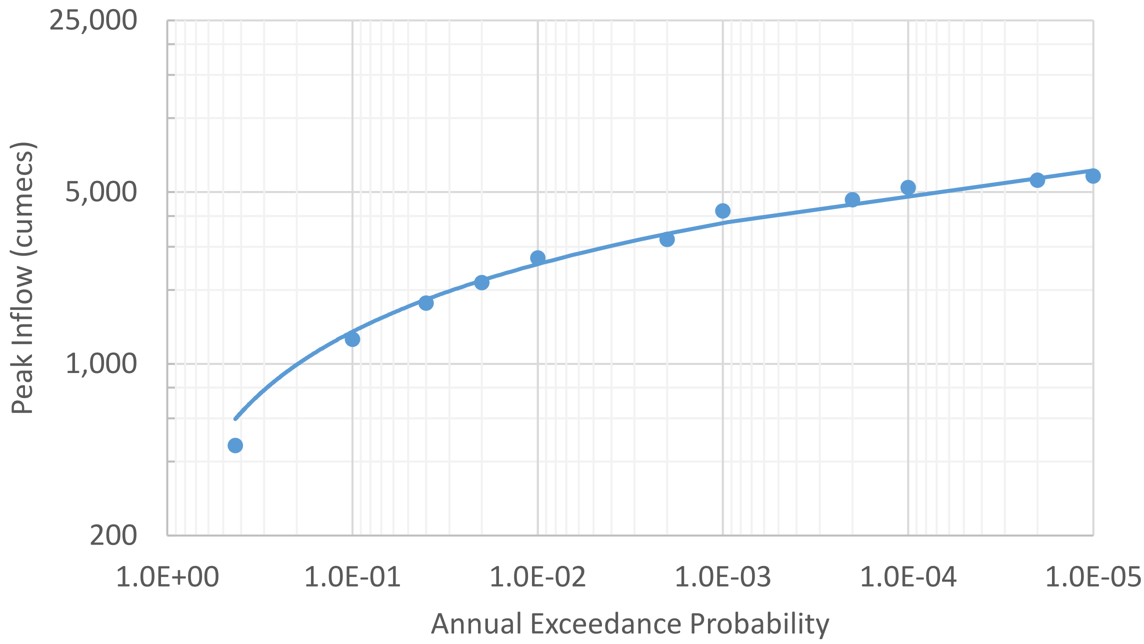


Figure B.8.- Probabilistic Flood Hazard Curve (Peak Inflow).

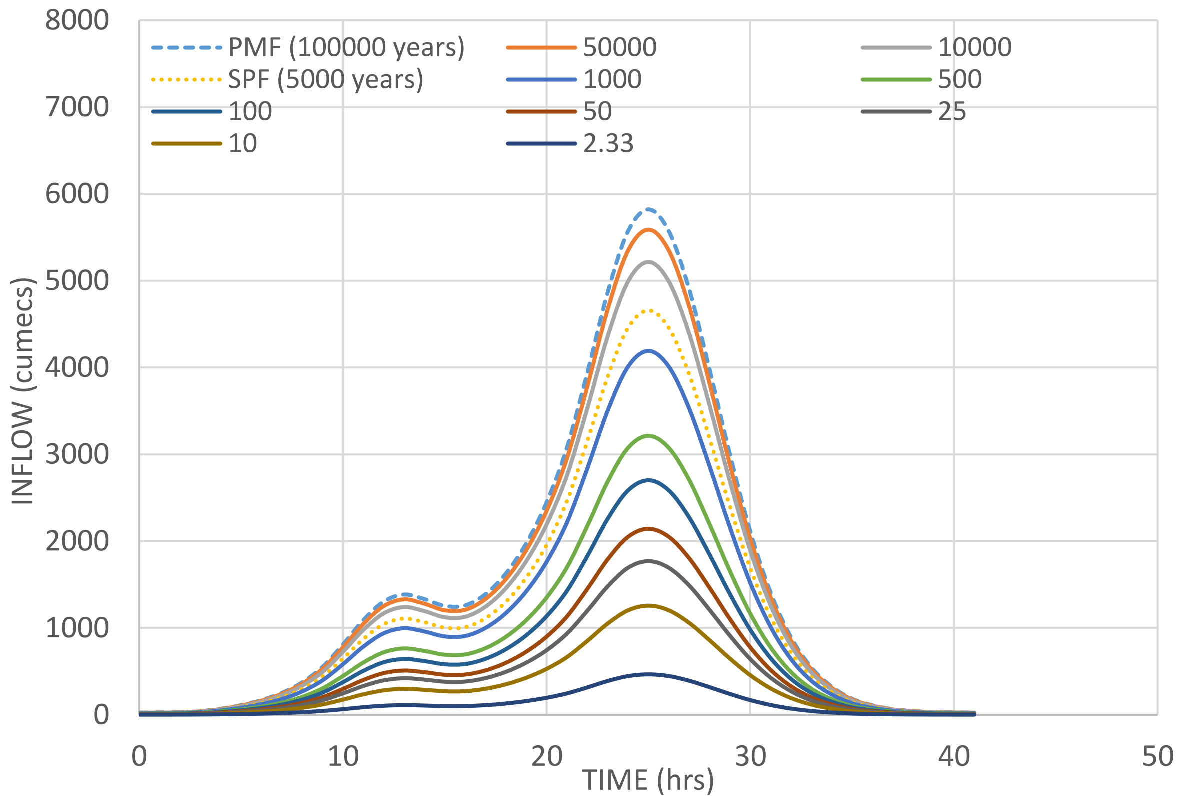


Figure B.9.- Flood hydrographs used in the flood routing Analysis.

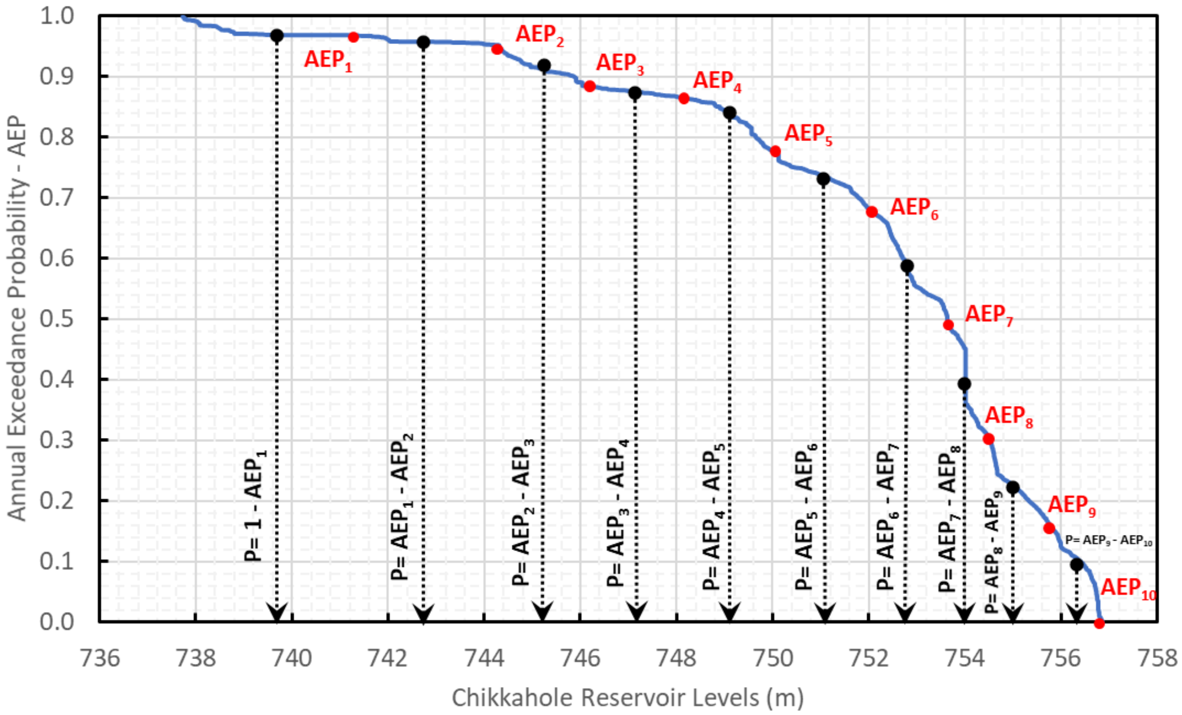


Figure B.10.- Discretization of water pool level-exceedance probability curve of Chikkahole Dam

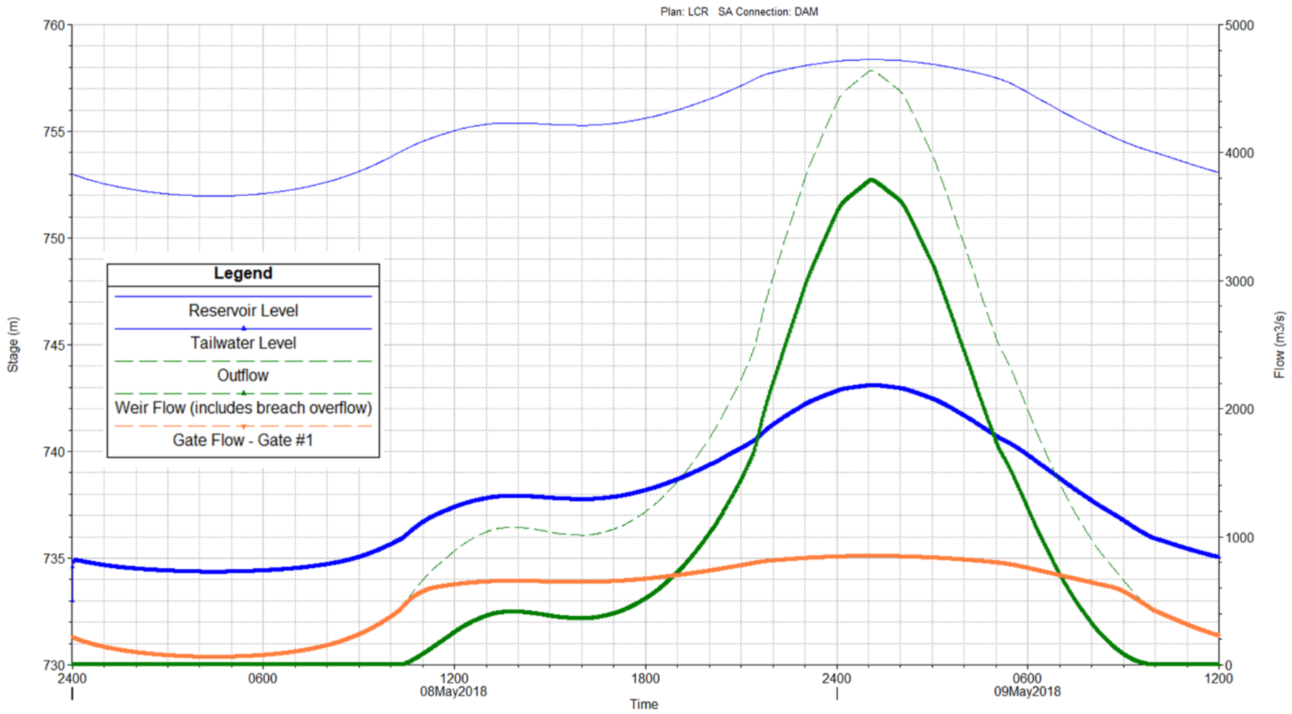


Figure B.11.- Flood Routing results of the SPF with an impinging level of 753 m, and with the current configuration of gated and ungated spillways

Input data from gated spillway availability should be included in the risk model before the nodes that include results of the flood routing analysis since this depends on which outlet works can be used during the flood event.

Therefore, information included in this node (node 3 in Figure B.5) refers to the probability that each gate can be used for that purpose, that is, the probability that at the moment in which the flood arrives, each component can be used or not for flood routing. In this case, the objective of this node is to introduce the probability of spillway availability. The individual reliability value has been assigned according to the following recommended values:

Table B.3.- General recommendation for individual gates' reliability (SPANCOLD, 2012)

Gate Reliability (%)	Description
95	When the outlet is new or has been very well maintained
85	When the outlet is well maintained but has had some minor problems
75	When the outlet has some problems
50	When the outlet is unreliable for flood routing
0	When the outlet is not reliable at all, or it is not used.

For Chikkahole dam, gate reliability of 95% was considered for individual gates since all gates and hoisting system are regularly maintained, and no problems have been reported by the dam's operator to date. It was assumed that each gate operates independently. Consequently, once the individual reliability of each gate was established, a binomial distribution has been used to calculate the probabilities of each case of gate availability, as shown in the following equation:

$$\boldsymbol{p(X=x)=\binom{n}{x}r^{x}(1-r)^{n-x}}$$

Where $\boldsymbol{x}$ is the number of gates that can be used for flood routing, $\boldsymbol{n}$ is the total number of gates, and $\boldsymbol{r}$ is the individual reliability. Therefore, the following data for gates performance probability is introduced in the risk model:

Table B.4.- Estimated Probabilities for Gates' availability at Chikkahole Dam

No. Gates working properly	Probability (%)
1	0.05
2	1.40
3	18.55
4	99.9

Input Data: System response

This stage of the risk analysis corresponds to nodes 5 to 7 of the risk model (Figure B.5 and Figure B.6). These nodes introduce the probability that the dam will fail by any of the failure modes considered for Chikkahole dam (FM no.1 and FM no.4) as a function of the reservoir levels. For this purpose, published fragility curves have been used to develop a customised curve for overtopping failure mode of Chikkahole Dam (Figure B.12), starting from the top bound level (TBL) of 757.4 m.

In the case of FM no.4. (Sliding in downstream slope) probabilities of failure were estimated through expert judgment sessions based on the results obtained from the stability analysis and flood routing studies carried out in previous stages.

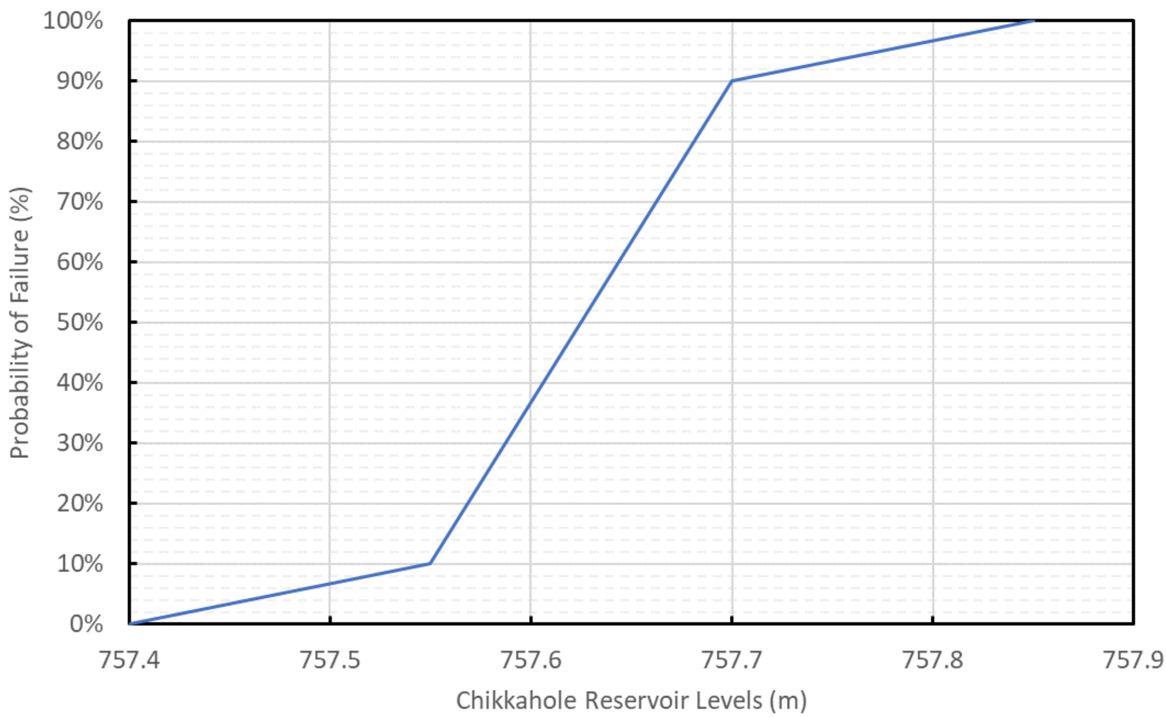


Figure B.12.- Fragility Curves for Overtopping Failure Mode (Chikkahole Dam).

Input Data: Consequences

This stage of the risk analysis corresponds to nodes 8 to 14 of the risk model (Figure B.5 and Figure B.6). As input for this phase, the dam break model developed by the Central Project Management Unit (CPMU) under the DRIP project was employed to quantify the consequences in the downstream floodplain for different scenarios of failure and non-failure at different reservoir levels (Figure B.14 and Figure B.15). Dam breach parameters and further details in the assumptions used in hydraulic modelling can be found in the above-mentioned report (Figure B.13)

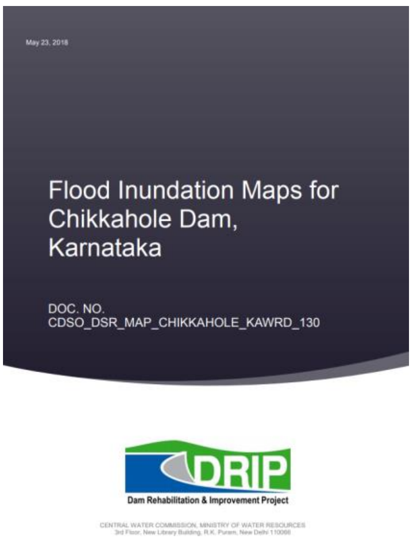


Figure B.13.- DBA and Inundation Maps report (Chikkahole Dam).

Dam failure hydrographs were obtained as a first step for the consequence analysis in order to relate the reservoir maximum water levels and peak flow discharges to downstream areas when the failure occurs. Required data from the breach outflow hydrographs were categorised in two parts:

- Curves that relate the maximum level in the reservoir with the peak flow discharge for each failure mode. These curves are introduced in the risk model
- Full dam failure hydrographs (not only peak flow discharge). These hydrographs are not included directly in the risk model but are used to perform hydraulic modelling of failure events and obtain potential consequences in downstream areas. Outcomes from con-sequence estimation are then related to peak flow discharges of each flood event, which are those used in the risk model

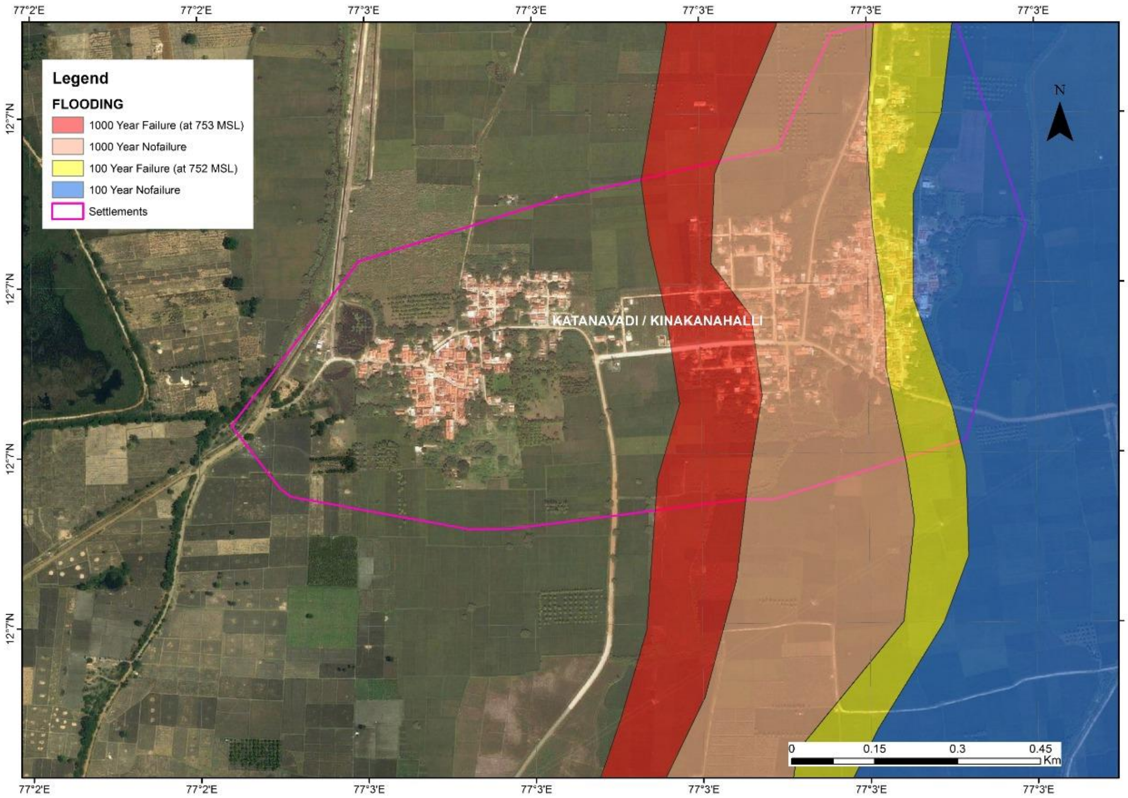


Figure B.14.- Flooding extents at Katanavadi Village for different failure and non-failure scenarios

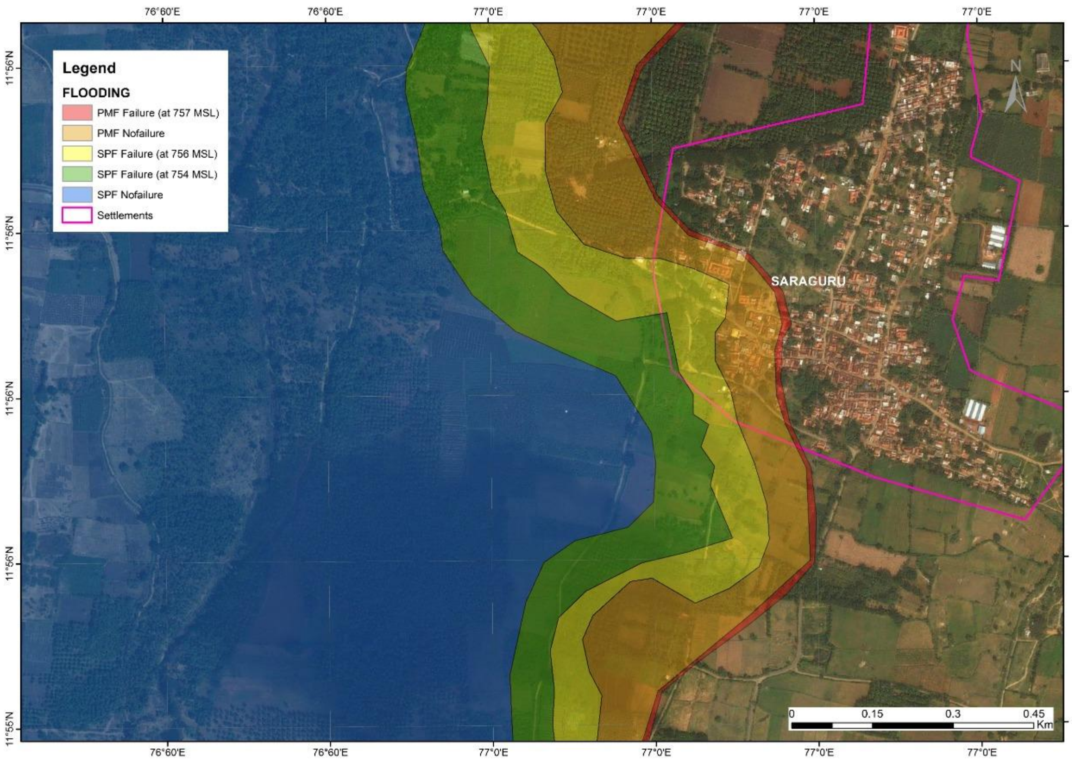


Figure B.15.- Flooding extents at Saraguru Village for different failure and non-failure scenarios

Based on the results of the hydraulic modelling and dam breach analysis for different scenarios, the magnitude of potential consequences was estimated for failure and non-failure scenarios in terms of the potential loss of life and economic consequences.

For the estimations of the downstream economic consequences of Chikkahole Dam, depth-damage reference curves shown in Figure B.16 and Figure B.17 were used, which has been adapted from the Global Flood Depth-Damage functions technical report (Huizinga et al., 2017). These values are incorporated into the risk model to estimated economic risk. For establishing the non-damage economic consequences (no failure), normal routed outflow hydrograph from Chikkahole spillway were used without considering the failure of the dam. Incremental consequences were then computed for each branch of the risk model by subtracting consequences estimates in failure and non-failure scenarios.

Incremental consequences in terms of the potential loss of life were estimated using recommended fatality rates for different scenarios of emergency preparedness and flood severity. Fatality rates developed within the European project SUFRI (I. Escuder-Bueno et al., 2012) were used for the case of Chikkahole Dam and are replicated in Table B.5. Two main categories fo emergency preparedness were assumed for Chikkahole Dam; for the baseline case (current situation), a poor emergency plan implementation was considered (i.e. lack of public education, EAP prepared but not implemented, no coordination among stakeholders), while for the improved situation a 95% effectiveness in the emergency panning was considered (i.e. early warning system along with a decision-support system implemented in the catchment, public education, EAP prepared, implemented, and tested, high coordination between stakeholders)

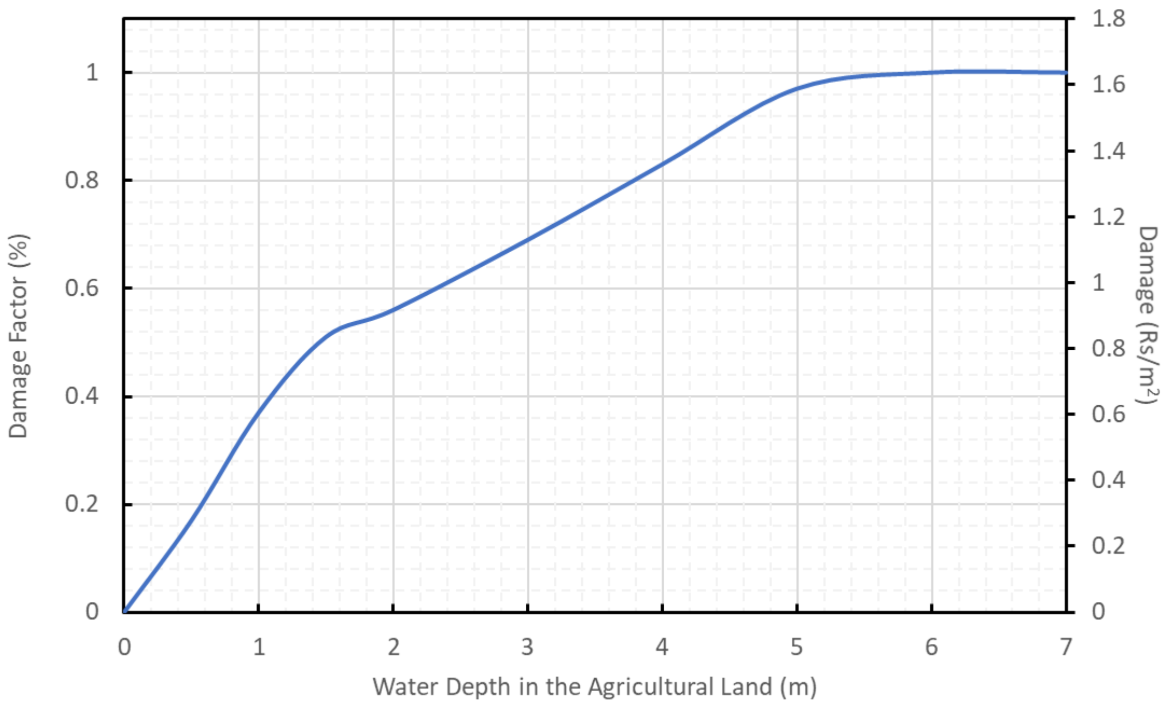


Figure B.16.- Relative average damage-depth function for agricultural land-use (adapted from Huizinga et al., 2017)

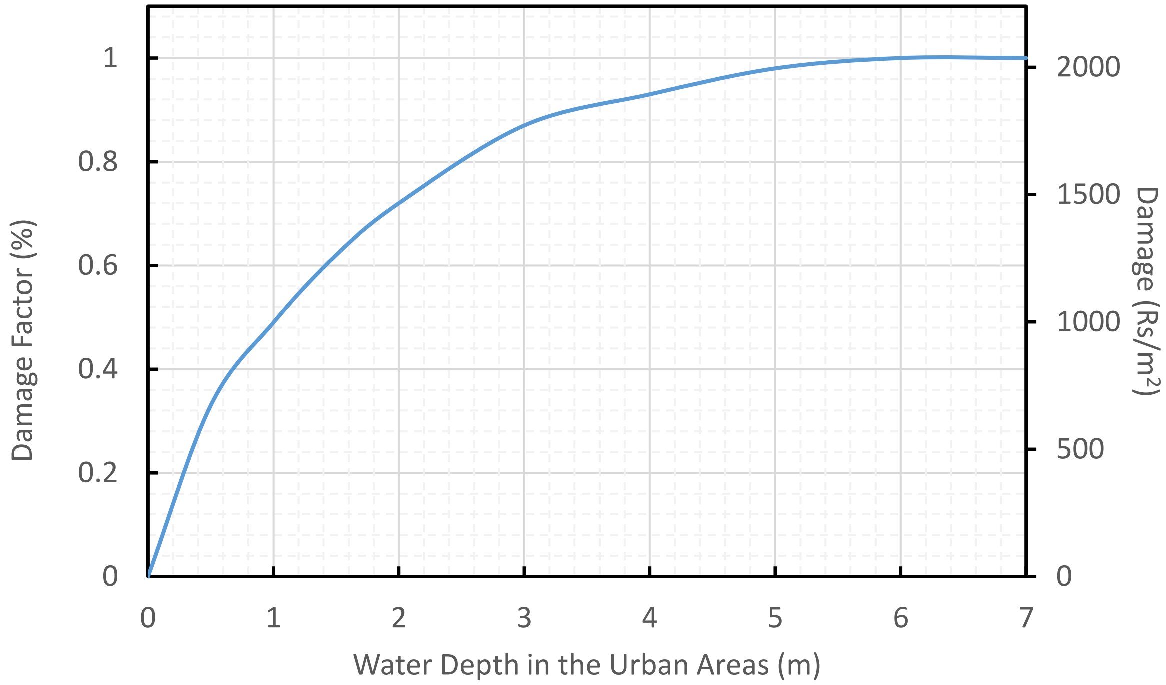


Figure B.17.- Relative average damage-depth function for residential land-use (adapted from Huizinga,2017)

Table B.5.- Fatality rates used to estimated potential loss of life in downstream floodplain

Category for the Case Study	Warning Time (h)	Flood Severity		
		High	Medium	Low
BASE LINE CASE (Current Situation) • There is no public education on flood risk terms. • There is EAP, but it has not been applied yet. • Some coordination between emergency agencies and authorities (but protocols are not established). • No communication mechanisms to the public.	0	0.9	0.3	0.02
	0.25	0.85	0.2	0.015
	0.625	0.6	0.07	0.012
	1	---	0.05	0.0005
	1.5	---	0.0002	0.0002
	24	---	0.0002	0.0001
IMPROVED SITUATION • Public education. • EAP is already applied. It has been proved or used previously. • High coordination between emergency agencies and authorities (there are proto-cols established). • Communication mechanisms to the public.	0	0.9	0.3	0.02
	0.25	0.55	0.06	0.006
	0.625	0.35	0.008	0.0015
	1	---	0.004	0.000125
	1.5	---	0.0002	0.0001
	24	---	0.0002	0.0001

Baseline Risk Characterisation (Current Situation)

After completion of input data for risk calculation, and once incorporated in the risk model architecture, societal and economic risks were obtained. For Chikkahole Dam, only incremental risk was computed, which is considered the fraction of total risk exclusively to dam failure. Incremental risk was obtained by subtracting from the consequences due to dam failure the ones that would have happened even in case of non-failure. In the following sections, this type of risk is compared with tolerability recommendations given in the guidelines published by CWC, and is later used to prioritize risk reduction actions. Results for the current situation of Chikkahole Dam are shown in Table B.6 and Figure B.18

Table B.6.- Incremental Risk Results for Chikkahole Dam (Current Situation)

Failure Mode	Annual Probability of Failure (1/year)	Societal Risk (lives/year)	Economic Risk (Crore Rs. /year)
FM 1. Overtopping	3.0×10^{-03}	1.07	1.86
FM 4. Sliding Dam Body	5.6×10^{-04}	1.94×10^{-01}	1.8
Total	$\mathbf{3.56 \times 10^{-03}}$	**1.27**	**3.66**

Results show that the current risk due to a hydrologic scenario lies in the "unacceptable risk" region of the fN chart. It can also be established that overtopping is the predominant failure mode contributing to the total incremental risk value, which is clearly higher than the sliding failure mode. These results reflect the importance of current rainfall data uncertainty for the hydrologic hazard analysis, and the insufficient discharge capacity of Chikkahole dam to withstand extreme flood events, as well as the importance of risk mitigation measures in line with this predominant failure mode.

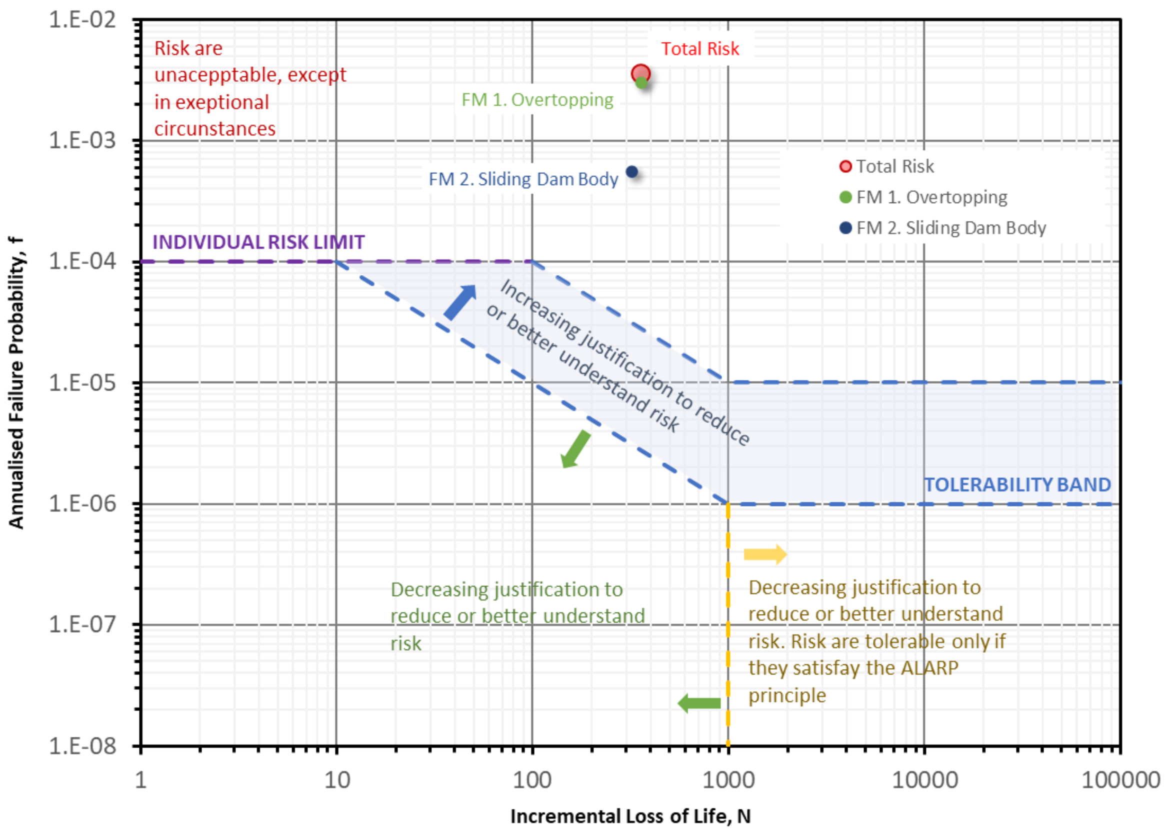


Figure B.18.- Individual and Societal Risk Evaluation (Current Situation)

Proposed Risk Mitigation Alternatives and its Evaluation

Based on the recommendations derived from technical inspections, the failure mode identification phase, and the results obtained from the risk analysis of the current situation, a total of six (5) mitigation measures, apart from the current situation, have been identified as alternatives for risk mitigation and further design flood selection

Alternative A. Current Situation

As the name indicates, this alternative represents the baseline case described in the previous paragraph and is only useful for comparison purposes. This alternative does not imply any structural or non-structural intervention.

Alternative B. Improvement of Emergency Preparedness/Response

Implementation of the Emergency Action Plan (EAP), including improved flood forecasting and analysis systems, results in better procedures in case of emergency, improved communication, warning issues and response for conducting evacuation of population downstream. Consequently, potential fatalities in case of dam failure decrease due to larger available warning times and better emergency procedures. This plan has been already developed, but it is still not implemented.

Alternative C. Enhance Discharge Capacity of side-channel spillway using a piano-key layout

This measure analyses the effect of a potential improvement in the current lateral spillway's discharge coefficients by overhauling the spillway's crest using a piano-key profile rather than the current and straight profile (ogee shape). After preliminary design (Figure B.19), it could be observed that a piano-key spillway can increase the discharge capacity 1.5 times, on average, the original capacity.

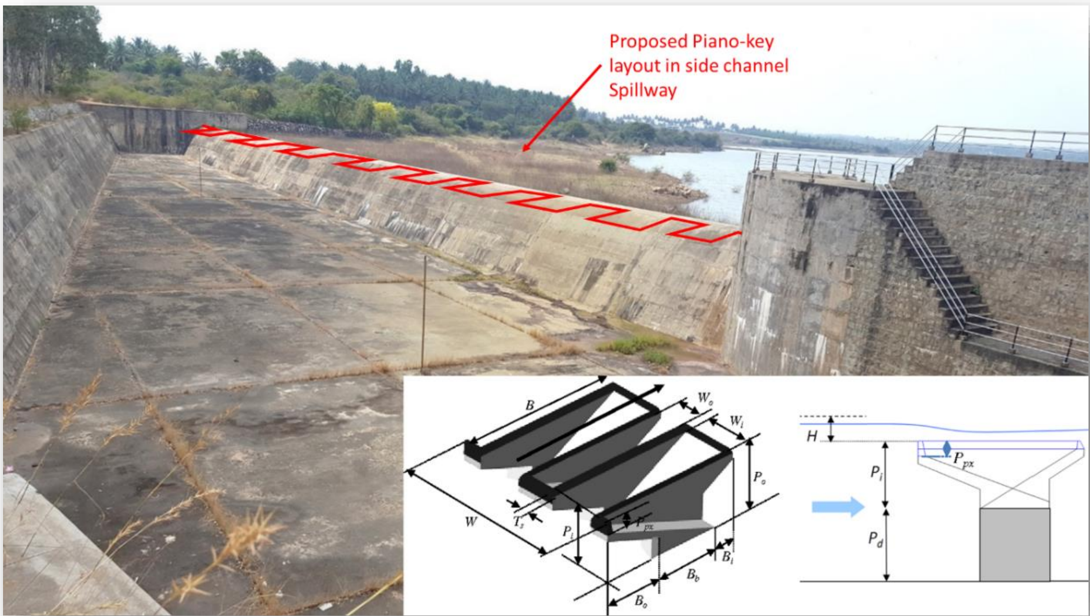


Figure B.19.- Schematic design for risk mitigation measure "Alternative C"

Alternative D. Add Fuse plug in Right flank of Reservoir's rim

The proposal of providing a breaching section (fuse plug) with a capacity of about 520 cumecs was also analysed. It was considered that a breaching section of bigger capacity could increase the population at risk within the potentially flooded area, which could adversely increase the associated risk rather than contribute to its mitigation. Figure B.20 and Figure B.21 show the hypothetical location of the proposed breaching section, based on the topography suitability (contour lines), drainage conditions, and social development.

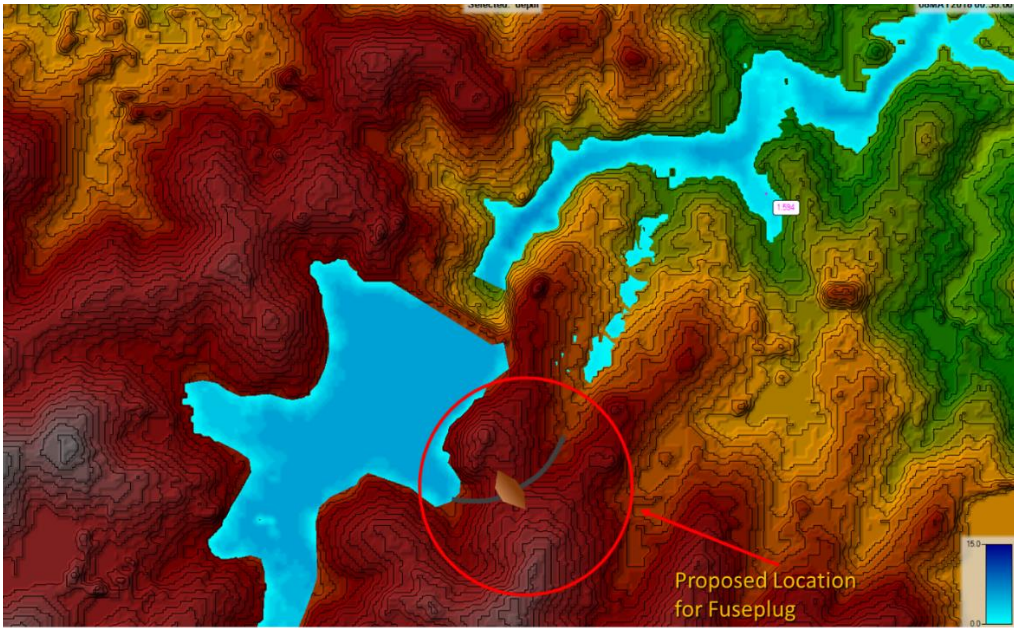


Figure B.20.- Topographic contour lines in Chikkahole Reservoir's area and proposed fuse plug location (Alternative D).

Figure B.21.- Sattelite imagery of Chikkahole Reservoir's area and proposed fuse plug location (Alternative D).

Alternative E. Armor Embankment to allow overtopping

Considering the inadequate capacity of Chikkahole dam to accommodate the revised design flood (4,654 m^3/s), which is 3.6 times the current discharge capacity of all its surplus arrangements (1,274 m^3/s), a proposal to design and construct an overtopping protection in the embankment section was also considered as a viable alternative. Overtopping protection was only considered because of its potential of low annual probability of operation under extreme events, and no major physical or environmental constraints seems to influence the design and implementation. However, considering the total dam length, this alternative might imply a prohibitive cost when compared with other flood protection alternatives.

Fo Chikkahole Dam, a Roller-compacted Concrete (RCC) overflow protection was considered for the entire length of the embankment dam by armouring the crest and downstream face with RCC. (Figure B.22)

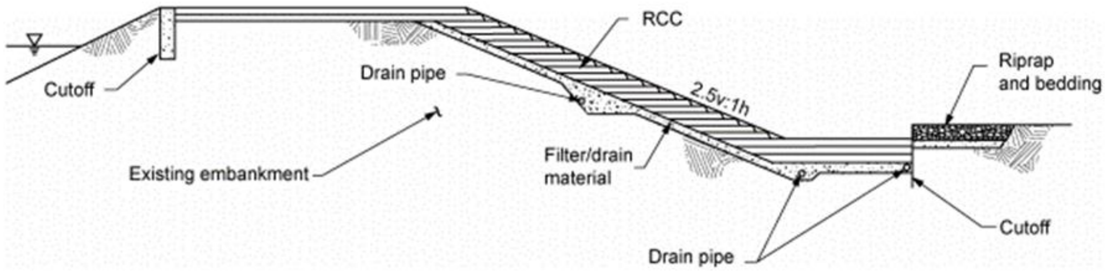


Figure B.22.- Schematic design for proposed RCC overflow protection at Chikkahole embankment dam (Alternative E).

Alternative F. Lower service Spillway's crest and add new gates

This alternative explores the scenario of increasing the hydraulic head, and therefore, the discharge capacity by lowering the service spillway crest and raising the gates' height (Figure B.23). Despite this structural measure may actually increase the risk to the downstream public by increasing the spillway flows during hydrologic events that are more likely to occur, special attention was retained during the preliminary design to avoid a significant rise in the incremental consequences. Figure B.23 below shows a schematic design of this alternative.

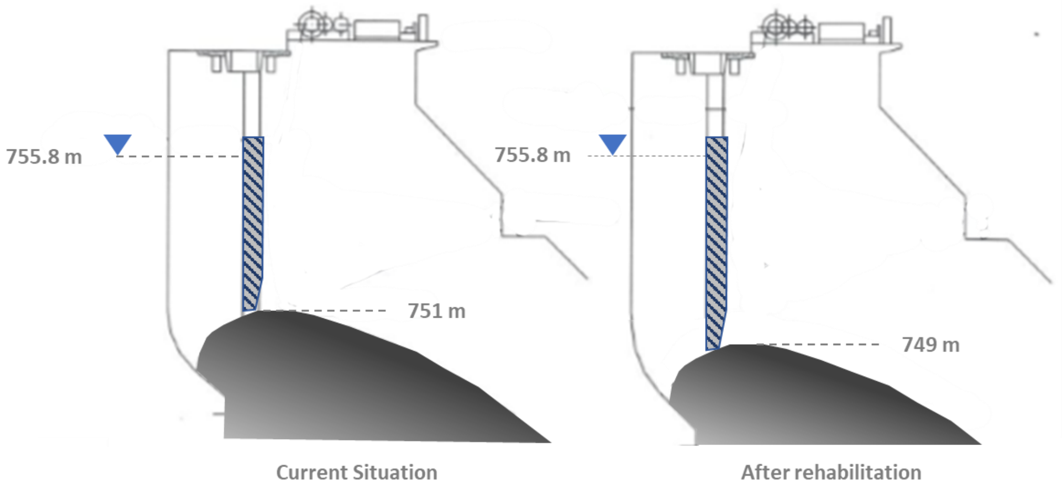


Figure B.23.- Schematic design for lowering spillway crest (Alternative F)

Finally, for all five (5) risk mitigation measures described above, individual and societal risks were estimated and evaluated following the tolerability recommendations from the *Guidelines for Assessing and Managing Risks Associated with Dams* published by CWC in 2018e. The effect of each of the measures in the risk model and incremental risk is summarised in Table B.7 and

Figure B.24

Risk-informed Inflow Design Flood Selection

After evaluating the results obtained from the risk model for all the risk mitigation alternatives (

Figure B.24), it can be concluded that no single intervention is enough to reduce incremental risk to tolerable levels. For instance, it has been shown that the implementation of an effective emergency action plan and early warning system for Chikkahole dam, with no other measure, results insufficient to mitigate the overall hydrologic risk.

Furthermore, and despite alternative "E" (i.e. armour embankment for overflow protection) is able to considerably reduce risk and meets all tolerability recommendations, its implementation cost and technical challenges involved in a proper design and construction phase may result in considering this alternative disadvantageous when compared to other risk reduction measures that also meet tolerability guidelines, such as Alternative "F" (i.e. lower service spillway's crest and adding new gates for increasing discharge head +Alt. D)

The total incremental risk for Alternative "D" (i.e. piano key weir layout in side-channel spillway + fuse plug) lays down within the tolerability band, which is still considered non-tolerable, but depending on uncertainty analysis results, recommended actions could be more focused on understanding these risks better and reducing its uncertainty (with new studies or new instrumentation) rather than investing in new risk reduction actions. If, after new studies, risks are still in this non-tolerable area, new risk reduction measures should be implemented.

Based on this analysis, the Chikkahole Dam's authority may considerer, in principle, reduce the uncertainties linked to Alternative "D", especially those associated with the design and location of the fuse plug, to better understand the risk and determine if the same could be further reduced to tolerable levels. If, after reducing the uncertainty, risk values cannot be lowered enough, the Alternative "F" should be implemented. These two structural interventions (Alt. D or Alt. F) would give to Chikkahole dam an enhanced discharge capacity between 2,035 and 2.420 m^3/s (43% and 52% of current IDF, respectively), depending upon which alternative is finally chosen, which can be considered enough to categorise the dam as hydrologically safe.

Table B.7.- Comparison of Risk Results and Total Discharge Capacity for the five (5) proposed risk reduction measures

Alt.	Description	Discharge Capacity (m^3/s)	% of revised SPF	Risk Results[1]		
				Estimated Total Annual Probability of Failure (1/year)	Societal Risk (lives/year)	Economic Risk (Crore Rs. /year)
A	Current Situation (do nothing)	1,274	27%	3.56 x 10^{-03}	1.27	3.66
B	Only improve Emergency preparedness (95% of warning and evacuation effectiveness)	1,274	27%	3.56 x 10^{-03}	1.59 x 10^{-01}	3.66
C[2]	Enhance Discharge Capacity of Lateral Spillway by using Piano-key layout	1,515	32%	9.78 x 10^{-04}	4.37 x 10^{-02}	9.78 x 10^{-01}
D[2]	Add Fuse plug on right flank + Alt. C	2,035	43%	2.43 x 10^{-05}	1.76 x 10^{-02}	3.16 x 10^{-02}

Alt.	Description	Discharge Capacity (m³/s)	% of revised SPF	Risk Results[1]		
				Estimated Total Annual Probability of Failure (1/year)	Societal Risk (lives/year)	Economic Risk (Crore Rs. /year)
E[2]	Armor embankment to allow overtopping	1,274	27%	1.39×10^{-07}	6.20×10^{-06}	2.35×10^{-04}
F[2]	Lower main spillway crest and add new gates for increasing discharge head + Alt. D	2,420	52%	5.42×10^{-06}	4.13×10^{-04}	7.05×10^{-03}

[1]Total Risk for all failure modes considered (i.e. FM.1 and FM 4)

[2]Alternative B was also considered (i.e. improvement of Emergency Preparedness) in all structural interventions

(red)	Does not meet tolerability guidelines. Risk is unacceptable
(yellow)	Within tolerability band. Increase justification to reduce uncertainty and better understand risk rather than additional mitigation measures
(green)	Meets tolerability guidelines

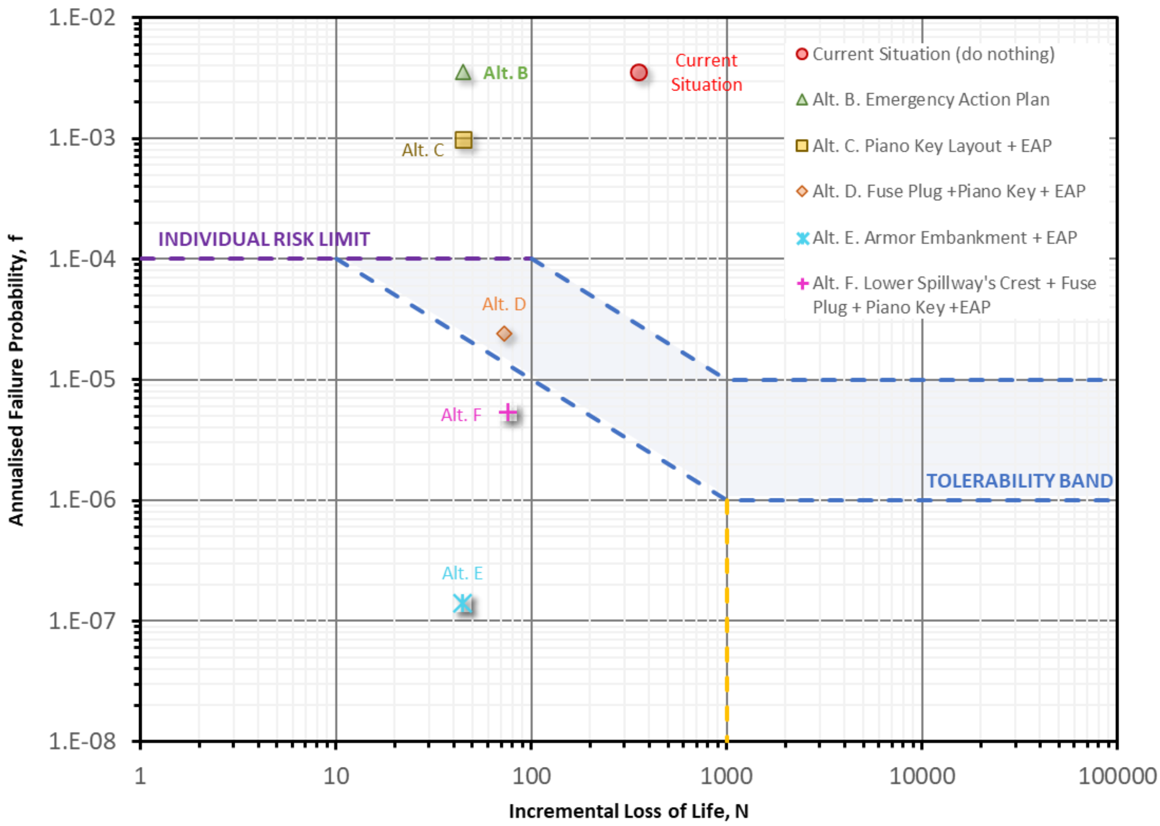


Figure B.24.- Individual and societal risk evaluation for proposed risk reduction actions

Appendix C . FREQUENCY ANALYSIS AS PART OF THE HYDROLOGIC HAZARD ANALYSIS. DATA EXTRAPOLATION LIMITS AND UNCERTAINTY ASPECTS

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FREQUENCY ANALYSIS AS PART OF THE HYDROLOGIC HAZARD ANALYSIS. DATA EXTRAPOLATION LIMITS AND UNCERTAINTY ASPECTS

C.1. INTRODUCTION

Risk informed decision making is used to assess the safety of dams and levees, recommend safety improvements, and prioritize expenditures. Risk estimates, from a hydrologic perspective, require estimation of the full range of hydrologic loading conditions to evaluate Potential Failure Modes (PFMs) tied to consequences of the failure mode of interest. These hydrologic loading conditions, for static (normal operation) and hydrologic/hydraulic scenarios can be evaluated through hydrologic hazard curves, which are developed from a hydrologic hazard assessment.

An hydrologic hazard curve is a graph of reservoir elevations vs annual exceedance probabilities (**Figure C.1**). In some situations, peak inflows/outflows, flood volumes (for a specified duration), or stage durations versus annual exceedance probabilities (AEPs) are also employed. The range of AEPs that is displayed will depend on the data available for the study location, the PFMs under consideration (such as static, seismic, or hydrologic), the type of risk-informed decision, and the needs of the risk team and dam's authority.

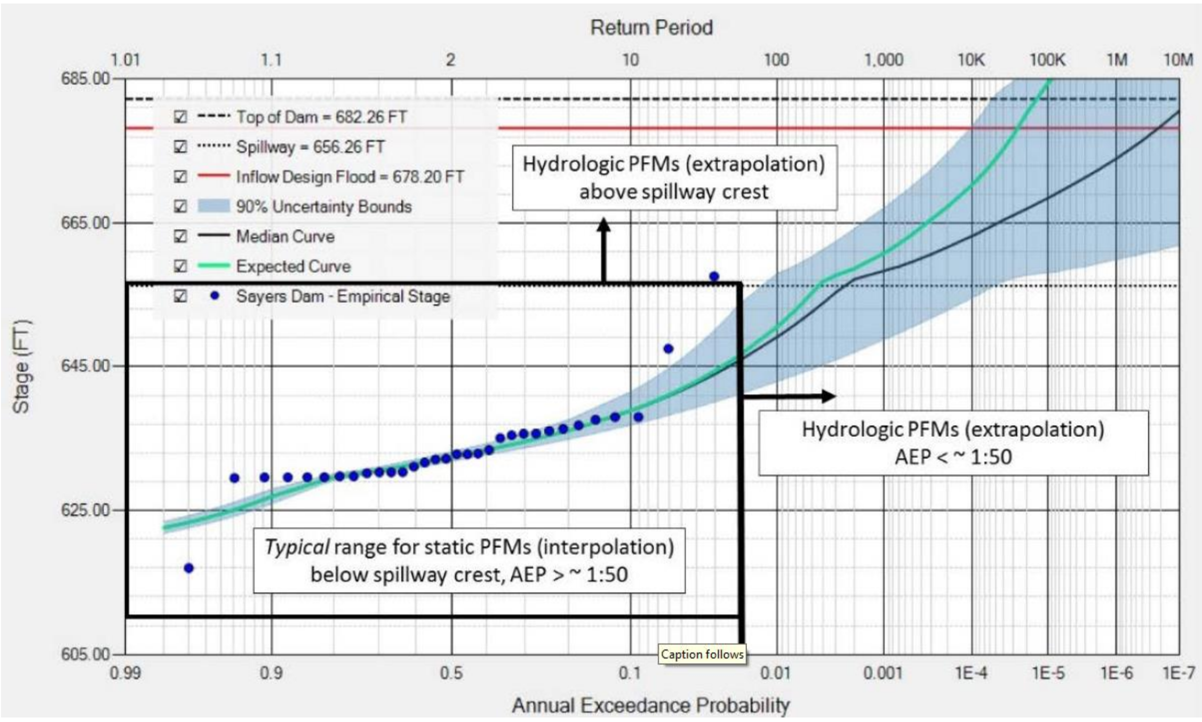

Figure C.1.- Example of Hydrologic Hazard Curve (adopted from USBR & USACE, 2019)

Nowadays, models with varying levels of details and complexity are available for the purpose of flood modelling. However, in our country, adequate data on soil and land use – that exhibit high variability in both space and time - poses great challenge to model rainfall-runoff in a realistic way. The simpler approaches for estimation of design flood may be broadly categorised into (CWC, 2001)

a. Use of empirical flood formulae and enveloping curves

b. Flood frequency analysis
c. Hydro-meteorological approach

Use of empirical flood formulae (e.g., the Dicken's formula for north Indian catchments and Ryve's formula for south Indian catchments) have been widely practised in the past. However, as they depend solely on catchment area and engineering judgement, disregarding the available information on parameters like rainfall pattern and catchment shape, their use is discouraged in the present. Information about envelope curves is available in textbooks on hydrology and literature like CWC (2001). The frequency analysis approach for the estimation of design flood is discussed in the present appendix as key element in the development of hydrologic hazard curves discussed in previous paragraph. The hydro-meteorological approach has been dealt with in *Chapter 2* of this document.

Flood frequency analyses, and the hydrologic hazard curves (HHCs) developed from them, provide magnitudes and probabilities for the entire ranges of peak flow, flood volume (hydrograph), and reservoir elevations, and do not focus on a single event. Reservoir elevation curves can be used to assess the probability of overtopping, and probabilities of water levels in spillway crests or crest structures to assess erosion, chute wall overtopping, or other potential failure modes. Inflow and outflow hydrographs for various water levels provides peaks, volumes, and durations of loadings. To satisfy current India risk guidance for dam safety risk assessments, HHCs of high hazard dams need to extend beyond AEPs of 1 x 10-4 (1 in 10,000), and have involvement by the flood hydrologist that performed the analysis.

C.2. DATA REQUIREMENTS AND INFLUENCING FACTORS

For arriving at reliable estimates of hydrologic hazard curves, and especially to floods with high return periods, the data should be of sufficient length. Normally, frequency analysis is carried out only when the data is available for a period of 30 years or more. Data on flood peak magnitude to be used for frequency analysis is assumed to be independent (i.e., data points should not be inter-correlated) and identically distributed (i.e., data points are homogenous, being part of the same population, which implies that no changes should take place in the upstream catchment and recording mechanism over the data period).

In reality, it is rarely the case where there are no changes in the upstream catchment over the years of data observation. Sometimes, the gauging stations are also shifted upstream or downstream, and the gauging equipment and methodology are changed. The effects of such changes should be removed, before attempting to carry out flood frequency analysis. Once this is done, the statistical tests to check for randomness and presence of trend may be applied to ensure that the data is reasonably fit for analysis. If found otherwise, corrections are to be applied before frequency analysis is taken up.

Developing hydrologic hazard curves for risk assessment uses the length of record and type of data to determine the extrapolation limits for flood frequency analysis. Extrapolation beyond the data is often necessary to provide information needed for dam safety risk assessments. The sources of information used for flood hazard analyses include streamflow and precipitation records and paleoflood data.

C.2.1. Streamflow records

Streamflow records consist of data collected at established gaging stations and indirect measurements of streamflow at other sites. Streamflow data can include estimates of peak discharge as well as average or mean discharge for various time periods.

C.2.2. Precipitation and Weather Data

Precipitation and weather data used in hydrologic models can include rainfall, snowfall, snow water equivalent, temperature, solar radiation, and wind speed and direction. Some of these types of data (i.e., snowfall, snow water equivalent, solar radiation, and wind) are limited to record lengths of less than about 30 years; rainfall and temperature data are available for some stations for up to 150 years, but in most cases are limited to less than 100 years.

C.2.3. Paleoflood Data

Paleoflood hydrology is the study of past or ancient flood events which occurred before the time of human observation or direct measurement by modern hydrological procedures (Baker, 1987). Unlike historical data, paleoflood data do not involve direct human observation of the flood events. Instead, the paleoflood investigator studies geomorphic and stratigraphic records (various indicators) of past floods, as well as the evidence of past floods and streamflow derived from historical, archeological, dendrochronologic, or other sources. The advantage of paleoflood data is that it is often possible to develop records that are 10 to 100 times longer than conventional or historical records from other data sources. Paleoflood data generally include records of the largest floods, or commonly, the limits on the stages of the largest floods over long time periods.

C.2.4. Key Hydrologic Hazard Analysis Factors

Some of the major flood hydrology-related factors that affect the hydrologic hazard curves estimates can be summarised in the figure below:

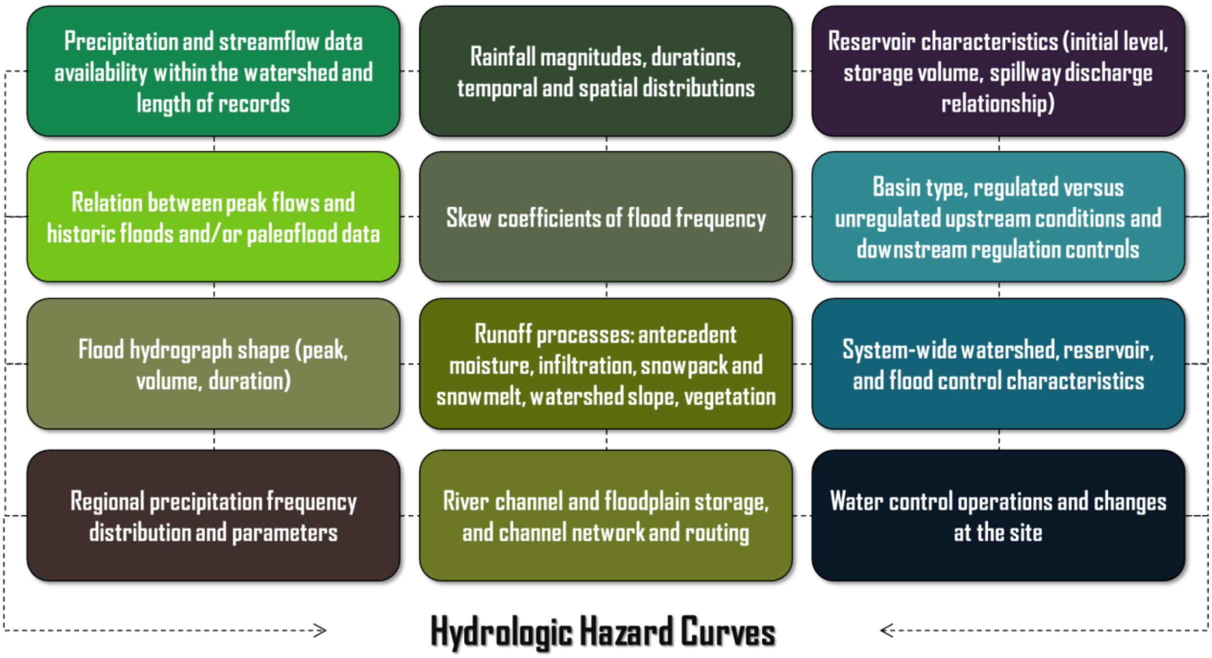

Figure C.2.- Flood hydrology-related factors that affect the hydrologic hazard curves estimates

C.3. COMMONLY USED FREQUENCY DISTRIBUTIONS

Earlier in India it was most common to use Gumbel's Extreme Value Type I distribution for estimation of the magnitude of flood peak because of its simplicity of computations. A few other distributions in use for the purpose worldwide are lognormal distribution, Pearson Type III distribution, Log Pearson Type III distribution, Extreme Value Type II distribution, Extreme Value Type III distribution, Gamma distribution, Weibull distribution, Wakeby distribution, Log-logistic distribution, Generalised logistic distribution. A few other distributions in use in the country were the lognormal distribution, Pearson type III distribution, Log Pearson type III distribution (CWC, 2001).

With the wide availability of computational power, use of more complicated distributions can now be attempted without significant difficulty. For the purpose of extreme value analysis, the following additional distributions merit consideration: Generalised extreme value, generalised Pareto, generalised logistic, Weibull distribution, Exponential distribution (Hosking and Wallis, 1997).

C.4. SUGGESTED EXTRAPOLATION LIMITS OF BASED ON DATA TYPE

It may be appreciated that estimation of flood with very high return period based on short period of discharge observations taken at the site may not be expected to yield reliable results. The type of data and the length of record used for the analysis governs the limit to which credible extrapolation of flood estimates can be made. Credible estimates of extreme floods with very high return periods can be prepared by combining regional data from multiple sources. Such approaches include pooling of data and information from regional precipitation, regional stream flow, and regional paleo-flood. Typical limits of extrapolation for different data types has been adopted from UBBR (2004) and presented in **Table C.1**. In many cases, the limits of credible extrapolation may actually be less than optimal, depending on the availability of data length for that particular location (e.g., for a station record length of 50 years, the credible extrapolation might be limited to a 100 year flood). **Approaches that consider both the flood peak and the flood volume are expected to yield better results.**

Floods can be categorized, according to the "*Australian Rainfall and Runoff: A Guide to Flood Estimation"* (Nathan and Weinmann, 2001), as large, rare, and extreme. These flood categories are shown in **Figure C.3**. Large floods generally encompass events for which direct observations and measurements are available. Rare floods represent events located in the range between direct observations and the credible limit of extrapolation from the data. Extreme floods generally have very small AEPs, which are beyond the credible limit of extrapolation but are still needed for dam safety risk assessments. In general, and considering the high population density in India, the level of protection that should be evaluated within a typical risk-informed hydrologic assessment in the country would range between "very rare" and "extreme" events with AEP between 1 in 100 to 1 in 10,000,000.

Extreme floods border on the unknowable. Uncertainty is very large and unquantifiable. Since data cannot support flood estimates in this AEP range, hydrologists and engineers must rely on their knowledge and understanding of hydrologic processes to estimate extreme floods. Oftentimes, these floods may result from unforeseen and unusual combinations of hydrologic parameters generally not represented in the flood history at a particular location. One potential upper bound to the largest flood at a particular site of interest is the PMF. **If peak flows or volumes calculated using probability or statistically based hydrology methods exceed**

those of the PMF, then the PMF is used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. If the PMF has been properly developed, it represents the upper limit to runoff that can physically occur at a particular site.

Table C.1.- Guidance on Hydro-meteorological data categories and corresponding extrapolation limits for flood frequency analysis (adopted from USBR, 2004)

Type of data used for flood frequency analysis	Limit of credible extrapolation for annual exceedance probability[1]	
	Typical[2]	Optimal[3]
At-site stream flow data	1 in 100	1 in 200
Regional stream flow data	1 in 500	1 in 1,000
At-site stream flow and at-site paleo flood data	1 in 4,000	1 in 10,000
Regional precipitation data	1 in 2,000	1 in 10,000
Regional stream flow and regional paleo flood data	1 in 15,000	1 in 40,000
Combinations of regional data sets and extrapolation	1 in 40,000	1 in 100,000

[1] Many factors can affect the equivalent independent record length for the optimal case
[2] Typical limits are based on the combination(s) of data that are commonly available and analysed for most sites
[3] Optimal Limits are based on the best combination(s) of data foreseen for a particular site in the foreseeable future

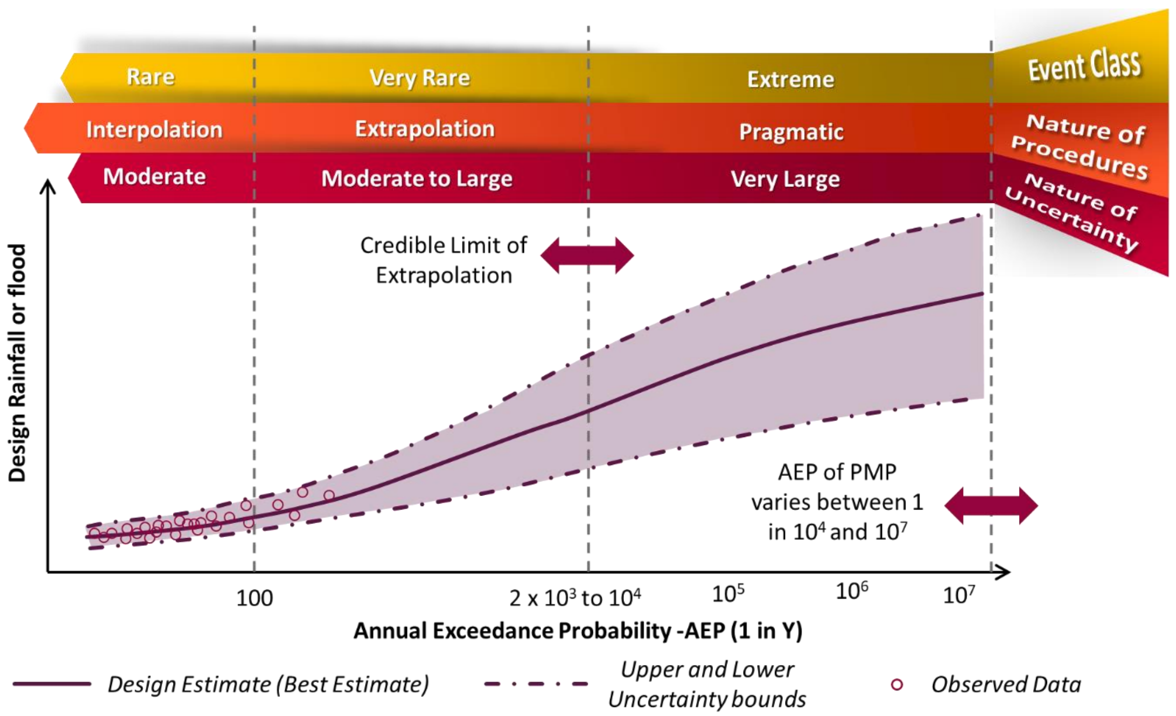

Figure C.3.- Design Characteristics of notional event classes (adapted from Nathan and Weinmann, 2001)

In summary, the greatest gains to be made in providing credible estimates of extreme floods can be achieved by combining regional data from multiple sources Thus, analysis approaches that pool data and information from regional precipitation, regional streamflow, and regional paleoflood sources should provide the highest assurance of credible characterization of low AEP floods

C.5. UNCERTAINTY MANAGEMENT

Historically in India, dam design and hydrologic analysis methods have focused on selecting a level of protection based on a single maximum flood given by a prescriptive approach. For high and intermediate hazard dams the probable maximum flood (PMF), and the standard project flood (SPF), respectively, are traditionally the two main protection levels used. However, as mentioned earlier, an entire range of peak inflows, volumes (hydrographs), and reservoir elevations with different (high and extremely low) annual exceedance probabilities should be considered for a proper risk -informed hydrologic hazard analysis. This aspect inserts a new hurdle to the hydrologic hazard assessment, which is the uncertainty management.

Figure C.4 shows example ranges of hydrograph shapes and variations in peak flows (six hydrographs) that have the same 1/10,000 AEP flood volume. In this scenario, all the hydrographs need to be included in design and risk analysis to properly characterize the flood loading. Maximum reservoir water surface elevations are also caused by combinations of peak, volume, and initial reservoir level, as shown in **Table C.2**. Because these estimates are being used in a risk assessment, best estimates are recommended, with numerical estimates of confidence bounds or upper and lower limits based on sensitivity analysis or uncertainty bounds. Quantifying uncertainty, identifying key factors of uncertainty, and performing an elicitation on those key factors, are also recommended.

Furthermore, while considering any given rainfall flood event upstream from a dam; the rainfall, peak discharge, volume, and resulting pool elevation would all have frequency estimates associated with the measured or estimated values. For most storms, it is unlikely the frequency estimates for these four observations would agree and may span an order of magnitude or more based on the assumptions made. This may be the result of varying antecedent conditions (previous rainfall, infiltration, runoff, starting pool, etc.), the mechanisms contributing to runoff generation (snowmelt, rainfall intensity and distribution, storm types, storm location, storm duration, vegetation changes, etc.), and operational releases would impact observed data and frequency calculations based on that data. In fact, at some dams, similar inflows have resulted in significantly different pool elevation from operational differences based on different downstream flow conditions.

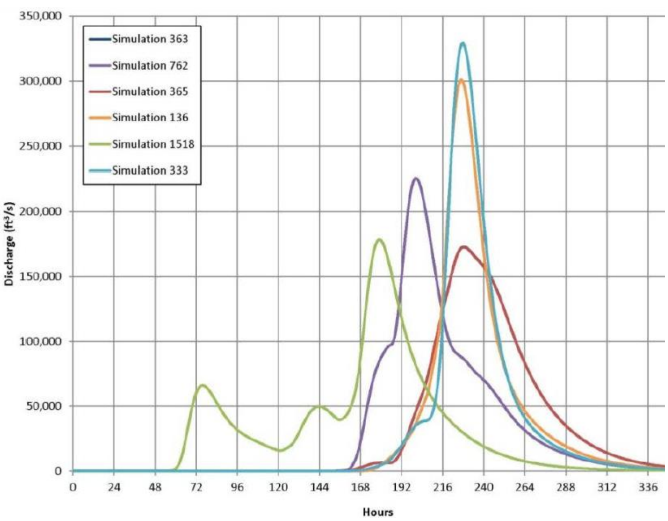

Figure C.4.- Example of reservoir inflow frequency hydrograph variations, based on 1 in 10,000 years Annual Exceedance Probability Volume

There are a number of methods that can be used to extend frequency curves and characterise the uncertainty, depending on the scale of the analysis. Some methods may be used for screening level analyses while other methods, with additional cost, time, and data requirements, are better suited for more detailed analyses. These studies typically involve precipitation and extreme storm frequency analysis and modelling using stochastic analysis (Monte-Carlo approaches), and more in-depth paleoflood studies. Hydrologic hazard curves from these types of studies provide ranges on peaks, volumes, hydrographs, and reservoir levels, and most importantly, include uncertainty.

There are several methods available to estimate magnitudes and annual exceedance probabilities of extreme flood events and hydrologic loadings for dam safety studies. Methods can generally either be classified as streamflow-based statistical analysis or rainfall-based with statistical analysis on the generated runoff. Methods that principally use streamflow data are presented in **Table C.4**; and rainfall-runoff methods are listed in **Table C.3**.

Table C.2.- Example of variations in Peak inflow and Initial Reservoir Levels for a similar Maximum Water Surface Elevation

AEP (1 in Y)	Maximum Water Level (MWL) (m)	Initial Reservoir Level (m)	Inflow peak (m^3/s)	Inflow Volume (M m^3)
1 in 220	479.44	467.40	9,191	1,908.2
1 in 180	479.42	472.43	9,064	1,059.5
1 in 150	479.42	475.16	9,024	1,983.4

Table C.3.- Rainfall-runoff approaches for a Risk-informed Hydrologic Hazard Analysis and Uncertainty management (adapted from USBR & USACE, 2019)

Approach	Inputs	Assumptions	Hydrologic Hazard Curve Product	Advantages	Level of Effort[1]
Rainfall-runoff methods	Probable Maximum Precipitation (PMP) design storm, rainfall frequency; watershed parameters	Exceedance probability of PMP, average watershed parameters values, runoff frequency same as rainfall frequency	Peak flow and hydrographs, based on rainfall frequency and PMP	Similar runoff model as PMP/PMF, familiar design concepts	Moderate to High
Stochastic event-based precipitation runoff modelling	Rainfall gages/detailed regional rainfall frequency, watershed parameters, snowpack, reservoir data	Main inputs defined by distributions; unit hydrograph; rainfall frequency analysis using recommended distributions	Peak flow frequency; hydrographs; volume frequency; reservoir elevation frequency	Monte-Carlo simulations to sample input distributions	High
Stochastic Watershed analysis by coupling rainfall-runoff, river routing, and reservoir operations models for system-wide basin flood risk studies	Can be regional extreme storm data (Depth-Area-Duration curves) or meteorologic extreme storm data, watershed parameters, snowpack	Main inputs defined by distributions; unit hydrograph; rainfall frequency analysis using recommended distributions or weather generator	Monte Carlo inputs and resampling; Reservoir elevation (pool) frequency curves, flood volumes, and hydrographs	Flexible framework for system-wide flood modelling with coupled components	High
[1]Low: less than 20 staff days; Moderate: 21-100 staff days; High: more than 100 staff days					

Table C.4.- Streamflow approaches for a Risk-informed Hydrologic Hazard Analysis and Uncertainty management (adapted from USBR & USACE, 2019)

Approach	Inputs	Assumptions	Hydrologic Hazard Curve Product	Advantages	Level of Effort[1]
Peak-flow and Volume frequency analysis with historical/paleo-flood data	Peak flow, historical data, paleoflood data, regional skews	Various flood frequency distributions with moments and regional skew and/or likelihood	Peak flow frequency and confidence intervals, volume frequency	Uses historical and paleoflood data when available	Low to moderate
Bayesian peak-flow frequency analysis with historical/paleoflood data	Peak flow, detailed paleofloods	Various flood frequency distributions with likelihood	Peak flow frequency and confidence intervals,	Detailed paleoflood data available; need FFA confidence intervals, choice of distribution	Low to Moderate
Balanced Hydrographs and Pattern Scaling (England 2003, Smith and Fleming 2018)	Hydrographs and volumes	Hydrographs represent extreme flood response; requires FFA for scaling	Hydrographs represent extreme flood response; requires FFA for scaling	Ratios of the IDF hydrograph and statistically based balanced and patterned hydrographs	Low
Streamflow Volume Stochastic Modeling with reservoir routing	Volume frequency, hydrographs, flood season, initial reservoir stage	Inputs defined by distributions, volume-frequency, observed hydrographs, and pool duration frequency	Reservoir elevation and confidence intervals	Monte-Carlo methods to sample inputs; combine inflows and routing, quantify uncertainty	Low to Moderate
Coupled Streamflow Volume Stochastic-Modelling and flood risk modelling	Pool duration, volumes, and Hydrographs	Inputs defined by distributions, volume-frequency observed hydrographs, and pool duration frequency	Reservoir elevation and confidence intervals	Monte-Carlo methods to sample inputs; quantify uncertainty; system/downstream effects with coincident frequency	High

[1]Low: less than 20 staff days; Moderate: 21-100 staff days; High: more than 100 staff days

As can be inferred from Tables above, no single approach can provide the needed characterization of extreme floods over the full range of AEPs required for risk analysis. Results from several methods and sources of data should be combined to yield a proper hydrologic hazard curve product. Therefore, the recommended approach is to combine streamflow peak or volume-based frequency analysis with stochastic rainfall-runoff models. Ideal situations would utilize multiple methods to estimate HHCs due to the significant extrapolation of the flood frequency relationships and the uncertainties involved in the analysis. When multiple methods have been used to determine the hydrologic hazard, sound physical and scientific reasoning for weighting or combining results is needed. Clearly, a measure of judgment is required to ensure that appropriate information is included in the dam safety decision making process. The selection is based on the experiences of the team members and the assumptions used in each of the analyses.

When methods for quantifying uncertainty are not possible, it is required by the hydrologist/hydraulic engineer to make a strong effort in characterizing the possible uncertainty to the risk team so that the uncertainty is taken into account during risk analysis.

While the extension of the hydrologic loading curve (**Figure C.1** show previously) will result in an AEP estimate for the Inflow Design Flood (IDF), assigning a frequency to the IDF maximum reservoir elevation (MWL) should be done as first stating the range for the IDF AEP based on the uncertainty and then stating the AEP based on the best or expected probability estimate. The intent of the hydrologic loading curve is to extrapolate as accurately as possible out through the 1/1,000 to 1/10,000 AEP. This is typically the portion of the loading curve that drives risk calculations when combined with the probability of failure and consequence estimates. Extrapolation past this AEP needs to include quantitative uncertainty with expected probability estimates; that uncertainty should be communicated in the risk assessment.

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Appendix D . CASE STUDY. PROBABLE MAXIMUM FLOOD (PMF) ESTIMATION FOR DAM "A", UTTAR PRADESH

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DESIGN FLOOD REVIEW STUDY FOR DAM A, UTTAR PRADESH

1.0 Introduction

The Dam A is located in Mirzapur district of Uttar Pradesh State. The Latitude and longitude of Dam A site is at $25^0$02'47" N and $82^0$56'54" E respectively. The catchment area of river up-to the dam site is 388 sq km. The catchment is mainly rainfed and has mixed land use pattern. The project consists of an earthen dam of length 8400 m and maximum height of dam is 29.9 m above river bed level. The hydraulic head is estimated as 28.1m. The top of the dam is at EL 100 m, full reservoir level at EL 98.2 m and Maximum water level at EL 98.5 m and minimum drawdown level at EL 86 m above MSL. The spillway discharge capacity is 1994 cumecs and its crest level is at EL 93.59 m. It has 12 Nos. of steel gate of size 4.6m. The reservoir has gross storage capacity of 150.85 MCM (5.33 TMC) and live storage capacity of 147.45 MCM (5.21 TMC). The project was completed in the year 1958.

2.0 Earlier design flood study

Neither any record of earlier flood study nor any flood hydrograph of earlier study has been made available for Dam A Project by the Project Authorities. As per information submitted by Project Authorities design discharge capacity for the project is 1994 m3/s.

3.0 Data availability

The concurrent catchment representative short term rainfall and runoff data is not available, hence for estimating the synthetic unit hydrograph on the basis of physiographic parameters of the catchment, Flood Estimation Report of CWC for Sone sub zone 1(d) has been used. Further, the PMP Atlas of Ganga River Basin published by CWC in June 2015 is also available, in which grid SPS and PMP values and isohyets of all severe storms occurred so far in and around the basin are available. The time distribution coefficient to estimate the hourly rainfall is also available in the PMP Atlas.

4.0 Design flood approach

The criteria of selection of inflow design flood for safety of dam as per IS: 11223-1985 is given below in Table -1. IS: 11223-1985 is Indian standard on guidelines for fixing spillway capacity, which was adopted by the then Indian Standards Institution (now BIS) on 13 February 1985 and reaffirmed in 1995.

Table-1: Criterion for Selection of Design Flood

Classification	Gross storage	Hydraulic Head	Inflow design flood for safety of dam
Small	Between 0.5 and 10 MCM	Between 7.5 m and 12 m	100 year flood
Intermediate	Between 10 and 60 MCM	Between 12 m and 30 m	Standard Project Flood
Large	Greater than 60 MCM	Greater than 30 m	Probable Maximum Flood

Since the gross storage capacity of Dam A is more than 60 MCM and hydraulic head is between 12m and 30m. Therefore, as per BIS criteria the Dam A qualifies for Probable Maximum Flood (PMF) as its design flood.

5.0 Physiographic parameters

The physiographic parameters of the river catchment at project site have been estimated by GIS processing of STRM 30 m DEM. As per GIS mapping the catchment area at Dam A project site is about 388 sq.km. The catchment parameters viz. catchment area, longest flow path, centroidal longest flow path, equivalent stream slope as obtained from GIS processing are given in Table-2. The catchment area plan of the project is shown in Figure-1.

Table-2: Physiographic parameters of the catchment

Catchment Area (sq.km)	Longest flow path L (km)	Centroidal longest flow path Lc (km)	Slopes (m/km)
388	35.9	13.4	3.30

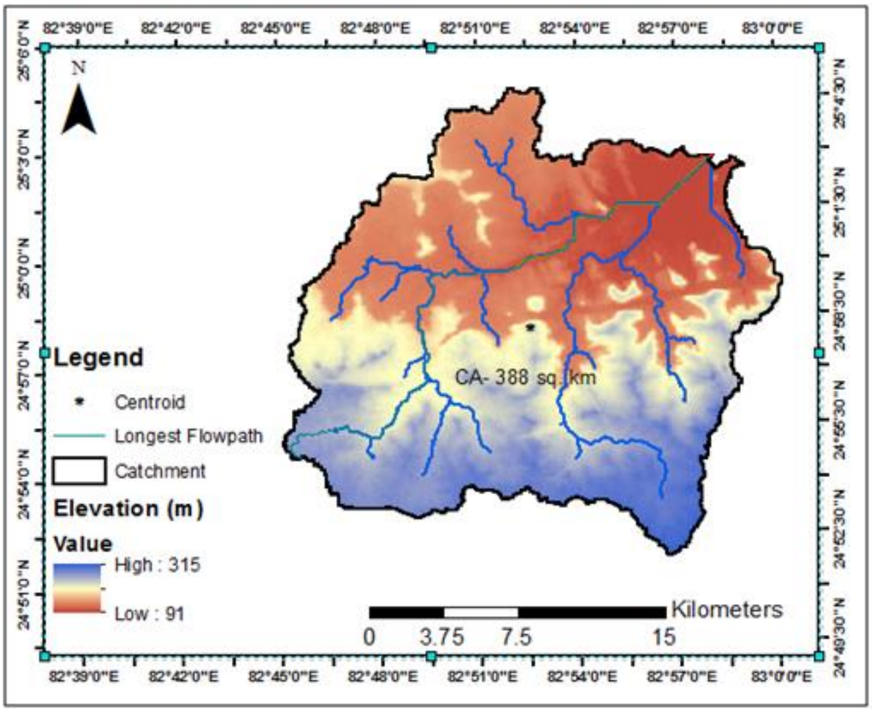

Figure- 1: Catchment plan of Dam A

6.0 Assessment of Unit hydrograph (UH) Ordinates

In absence of short interval observed discharge and concurrent rainfall data; the unit hydrograph of one hour duration has been derived using Flood Estimation Report for Sone subzone 1(d). The estimated UH parameters for catchments are given in Table-3. The unit hydrograph ordinates as assessed for the unit hydrograph of catchment are given in Table-4. Smoothened /adjusted synthetic Plot for the same is presented in Figure-2.

Table-3: Unit Hydrograph Parameters

Parameter	Unit	Value
Time from the centre of effective rainfall duration to the UH peak tp = 0.314(L /s^0.5)^1.102	hr	6.43 (Rounded off to 6.5)
Peak discharge of unit hydrograph per unit area qp =1.664/(tp)^0.965 qp	m3/sec/sq. km	0.27
Width of the UH measured at 50% of peak discharge ordinate W50 = 2.534/(qp)^0.976	hr	8.99
Width of the UH measured at 75% of peak discharge ordinate W75 = 1.478/(qp)^00860 W75	hr	4.51
Width of the rising limb of UH measured at 50% of peak discharge ordinate WR50 = 1.091/(qp)^0.750	hr	2.89
Width of the rising limb of UH measured at 75% of peak discharge ordinate WR75 = 0.672/(qp)^0.719	hr	1.71
Base width of UH TB = 5.526*(tp)0.8 66	hr	27.95 (Rounded off to 28)
Peak Discharge of UH Qp = qp x A	m3/sec	106.05
Unit duration of unit hydrograph tr	hr	1
Time from the start of rise to the peak of the UH Tm=tp+tr/2	hr	7.0
Q theoretical = A*d/0.36*tr here d= 1 cm depth and tr = 1 hr	m3/sec	1077.78

Table-4: 1 hr Unit Hydrograph Ordinates

Time (hr)	**Unit Hydrograph Ordinates (m3/sec)**
0	0
1	5
2	13
3	27
4	49
5	72
6	95
7	106
8	100
9	89
10	79
11	70
12	62
13	54
14	46
15	39
16	33
17	28
18	24
19	20

Time (hr)	Unit Hydrograph Ordinates (m3/sec)
20	17
21	14
22	11
23	9
24	7
25	5
26	3
27	1
28	0

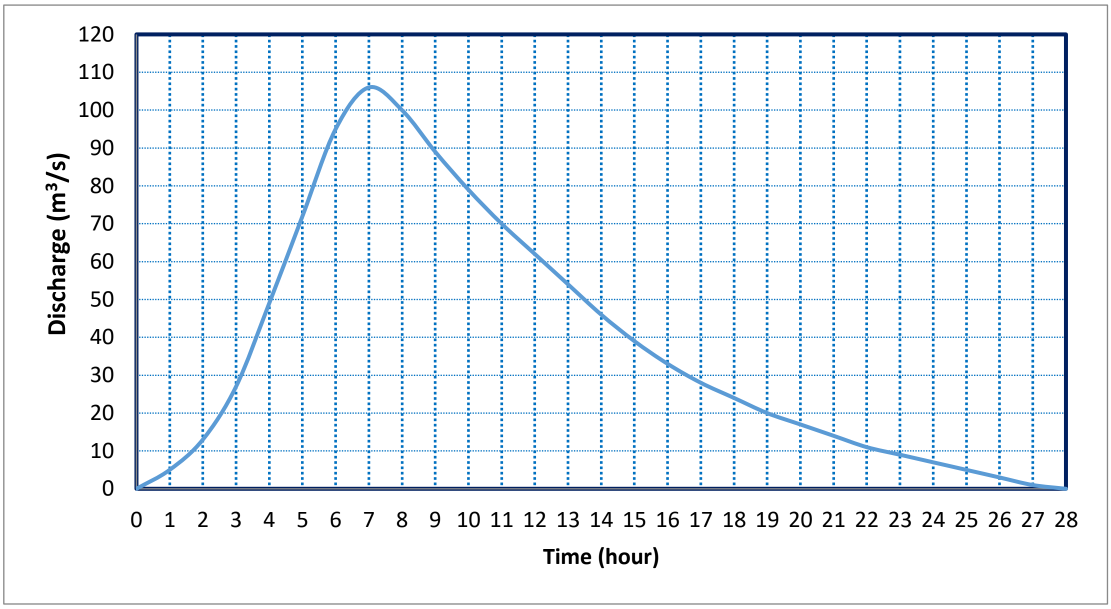

Figure- 2: 1 hr Synthetic Unit Hydrograph of catchment

7.0 Design Storm

Dam A project lies in PMP Atlas of Ganga River Basin published by CWC in June 2015. The project qualifies for 1-day storm for design rainfall depths computations. Further, from the PMP Atlas it has been found that the most critical 1-day storm for the Dam A project catchment is the storm dated 22 June 1916 with eye of storm at Meja and recorded 1-Day peak rainfall depth as 512 mm at its eye location. For comprehensive output, grid rain depth value has been taken from PMP Atlas for Ganga Basin. 1–day PMP value of 55.76 cm (grid rain depth of 42.24cm, TAF of 1.00 and MMF of 1.32), has been assessed from PMP Atlas for Ganga Basin and same has been adopted in estimating PMF. 1 day value is increased by 15% to convert the same into 24 hours value, up-to a maximum value of 50 mm. It is calculated to be as 60.76 cm. The 24 hour rainfall has been converted into one 12 hourly rainfall bells by using the time distribution as per PMP atlas for Ganga Basin. The 12 hr coefficient for 24 hr rainspell is 83.0% for the catchment. Hourly distribution of rainfall normalized distribution coefficient has been worked out for bell of

12 hours each using the hourly distribution coefficient. The hourly distribution coefficients of 12 hour rainfall are given in Table-5.

Table-5: Hourly distribution coefficient of 24 hour rainfall and normalized distribution coefficient for 12 hour bell

Time (hr)	Distribution coefficient for 24 hour rainfall (%)	Normalised Distribution coefficient for 12 hour bell (%)
1	19.1	23.01
2	29.3	35.30
3	36.4	43.86
4	44.0	53.01
5	48.9	58.92
6	53.9	64.94
7	59.1	71.20
8	62.6	75.42
9	67.7	81.57
10	74.3	89.52
11	78.6	94.70
12	83.0	100.00
13	86.4	
14	88.8	
15	90.3	
16	91.2	
17	92.0	
18	93.3	
19	95.7	
20	96.8	
21	98.5	
22	99.6	
23	99.8	
24	100.0	

8.0 Design Loss rate and Base Flow

The average loss rate has been adopted by looking the bridge position nearer to project location and catchment area as given in Table-4 of Flood Estimation Report for Sone sub zone 1 (d). An average loss rate has been worked out in the order of 0.75 cm/hour and the same has been adopted for the study.

As recommended by CWC subzone 1(d) report following base flow rate has been adopted:
Base flow / km2 of drainage area = 0.045
Using the above formula, the computed base flow for the catchment area is 17.5 m3/sec.

9.0 Hourly distribution and critical sequencing of rainfall

The hourly distribution coefficient of 12 hour rainfall is done using Table-5 and given in Table-6. Critical sequencing of hourly effective rainfall is given in Table-7. The reverse of critically sequenced effective rainfall has been used for convolution with ordinates of unit hydrograph to get Probable Maximum Flood hydrograph.

Table-6: 12 Hourly distribution of rainfall

1day PMP rainfall as per PMP atlas	55.76 cm
24 hr PMP rainfall as per PMP atlas (with 15% clock hour correction restricted to maximum 50 mm)	60.76 cm
Depth 1st 12 hr bell (0.83x 60.76)	50.43 cm
Depth 2nd 12 hr bell (0.17x 60.76)	10.33 cm

Time	Distn coeff	Nor-malised coeff	Cumulative rainfall depth		Incremental rainfall depths		Loss rate	Effective rainfall depths	
			1st 12 hr bell	2nd 12 hr bell	1st 12 hr bell	2nd 12 hr bell		1st 12 hr bell	2nd 12 hr bell
	(%)	(%)	(cm)	(cm)	(cm)	(cm)	(cm/hr)	(cm)	(cm)
1	19.1	23.01	11.61	2.38	11.61	2.38	0.75	10.86	1.63
2	29.3	35.30	17.80	3.65	6.20	1.27	0.75	5.45	0.52
3	36.4	43.86	22.12	4.53	4.31	0.88	0.75	3.56	0.13
4	44.0	53.01	26.73	5.48	4.62	0.95	0.75	3.87	0.20
5	48.9	58.92	29.71	6.09	2.98	0.61	0.75	2.23	0.00
6	53.9	64.94	32.75	6.71	3.04	0.62	0.75	2.29	0.00
7	59.1	71.20	35.91	7.35	3.16	0.65	0.75	2.41	0.00
8	62.6	75.42	38.04	7.79	2.13	0.44	0.75	1.38	0.00
9	67.7	81.57	41.13	8.43	3.10	0.63	0.75	2.35	0.00
10	74.3	89.52	45.14	9.25	4.01	0.82	0.75	3.26	0.07
11	78.6	94.70	47.76	9.78	2.61	0.54	0.75	1.86	0.00
12	83.0	100.00	50.43	10.33	2.67	0.55	0.75	1.92	0.00

Table-7: Critical sequencing for Effective hourly rainfall

Time	Unit Hydrograph Ordinates	Critical arrangement of rainfall increments		Reversed sequence		Critically sequenced effective rainfall (B2-B1)
				B1	B2	
(hr)	(cumecs)	(cm)	(cm)	(cm)	(cm)	(cm)
0	0					0.00
1	5					0.00
2	13					0.00
3	27					0.00
4	49	1.92	0.00	1.38	0.00	0.00

Time	Unit Hydrograph Ordinates	Critical arrangement of rainfall increments		Reversed sequence		Critically sequenced effective rainfall (B2-B1)
				B1	B2	
(hr)	(cumecs)	(cm)	(cm)	(cm)	(cm)	(cm)
5	72	2.41	0.00	1.86	0.00	0.07
6	95	3.87	0.20	2.23	0.00	0.13
7	106	10.86	1.63	2.29	0.00	0.52
8	100	5.45	0.52	2.35	0.00	1.63
9	89	3.56	0.13	3.26	0.07	0.20
10	79	3.26	0.07	3.56	0.13	0.00
11	70	2.35	0.00	5.45	0.52	0.00
12	62	2.29	0.00	10.86	1.63	1.38
13	54	2.23	0.00	3.87	0.20	1.86
14	46	1.86	0.00	2.41	0.00	2.23
15	39	1.38	0.00	1.92	0.00	2.29
16	33					2.35
17	28					3.26
18	24					3.56
19	20					5.45
20	17					10.86
21	14					3.87
22	11					2.41
23	9					1.92
24	7					
25	5					
26	3					
27	1					
28	0					

10.0 Probable Maximum Flood (PMF) for Dam A project

The reverse sequence of hourly effective rainfall (B2-B1) as given in Table-7 has been convoluted with ordinates of unit hydrograph to get PMF direct runoff hydrograph as shown in Annexure-I. The base flow contribution has been added to get the PMF hydrograph at Dam A. The estimated PMF is 3537 m^3/sec. The PMF hydrograph ordinates are given in Table-8. A plot of the same is given in Figure- 3. The same may be utilized for the dam safety review of the project.

Table-8: PMF hydrograph for Dam A

Time(hr)	Design Flood Ordinates for PMF (m^3/s)	Time(hr)	Design Flood Ordinates for PMF (m^3/s)
0	**17**	**28**	**3406**

Time(hr)	Design Flood Ordinates for PMF (m^3/s)	Time(hr)	Design Flood Ordinates for PMF (m^3/s)
1	17	29	3143
2	17	30	2825
3	17	31	2493
4	17	32	2180
5	17	33	1889
6	18	34	1619
7	19	35	1378
8	24	36	1168
9	39	37	987
10	65	38	831
11	106	39	692
12	160	40	572
13	222	41	465
14	288	42	370
15	351	43	292
16	421	44	223
17	529	45	162
18	695	46	109
19	908	47	65
20	1157	48	38
21	1461	49	26
22	1809	50	19
23	2221	51	17
24	2682		
25	3104		
26	3427		
27	3537		

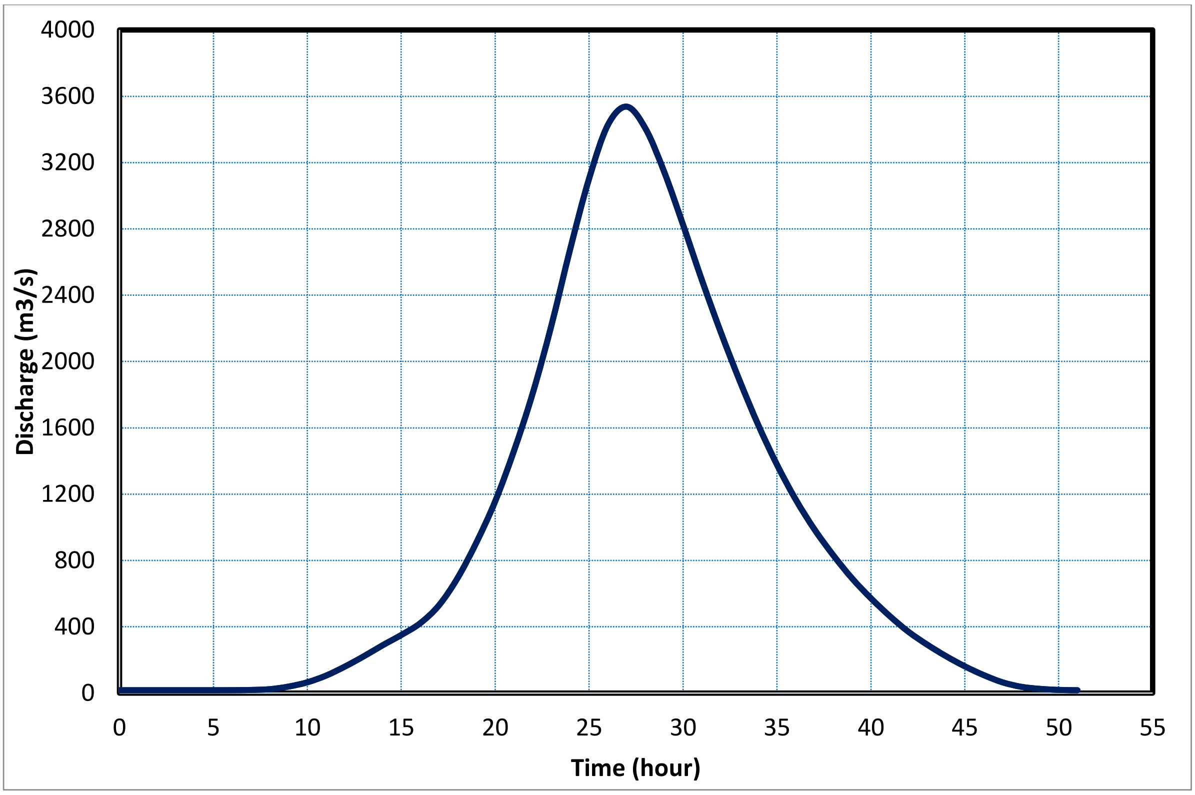

Figure - 3: PMF hydrograph for Dam A

Annexure-I

Time (hr)	UH Ord. (m3/s)	Effective Rainfall (cm)																								DRH (m3/s)	Baseflow (m3/s)	PMF Ord (m3/s)
		0.00	0.00	0.00	0.00	0.00	0.07	0.13	0.52	1.63	0.20	0.00	0.00	1.38	1.86	2.23	2.29	2.35	3.26	3.56	5.45	10.86	3.87	2.41	1.92			
0	0	0.0																								0.0	17.5	17
1	5	0.0	0.0																							0.0	17.5	17
2	13	0.0	0.0	0.0																						0.0	17.5	17
3	27	0.0	0.0	0.0	0.0																					0.0	17.5	17
4	49	0.0	0.0	0.0	0.0	0.0																				0.0	17.5	17
5	72	0.0	0.0	0.0	0.0	0.0	0.0																			0.0	17.5	17
6	95	0.0	0.0	0.0	0.0	0.0	0.4	0.0																		0.4	17.5	18
7	106	0.0	0.0	0.0	0.0	0.0	0.9	0.7	0.0																	1.6	17.5	19
8	100	0.0	0.0	0.0	0.0	0.0	1.9	1.7	2.6	0.0																6.3	17.5	24
9	89	0.0	0.0	0.0	0.0	0.0	3.5	3.6	6.8	8.1	0.0															22.0	17.5	39
10	79	0.0	0.0	0.0	0.0	0.0	5.1	6.5	14.0	21.2	1.0	0.0														47.8	17.5	65
11	70	0.0	0.0	0.0	0.0	0.0	6.8	9.6	25.4	43.9	2.5	0.0	0.0													88.3	17.5	106
12	62	0.0	0.0	0.0	0.0	0.0	7.6	12.7	37.4	79.7	5.3	0.0	0.0	0.0												142.7	17.5	160
13	54	0.0	0.0	0.0	0.0	0.0	7.1	14.2	49.3	117.1	9.6	0.0	0.0	6.9	0.0											204.3	17.5	222
14	46	0.0	0.0	0.0	0.0	0.0	6.4	13.4	55.1	154.6	14.1	0.0	0.0	17.9	9.3	0.0										270.6	17.5	288
15	39	0.0	0.0	0.0	0.0	0.0	5.6	11.9	51.9	172.5	18.6	0.0	0.0	37.2	24.2	11.1	0.0									333.0	17.5	351
16	33	0.0	0.0	0.0	0.0	0.0	5.0	10.6	46.2	162.7	20.8	0.0	0.0	67.5	50.3	29.0	11.4	0.0								403.4	17.5	421
17	28	0.0	0.0	0.0	0.0	0.0	4.4	9.4	41.0	144.8	19.6	0.0	0.0	99.1	91.3	60.1	29.7	11.7	0.0							511.2	17.5	529
18	24	0.0	0.0	0.0	0.0	0.0	3.9	8.3	36.4	128.5	17.4	0.0	0.0	130.8	134.1	109.1	61.8	30.5	16.3	0.0						677.1	17.5	695
19	20	0.0	0.0	0.0	0.0	0.0	3.3	7.2	32.2	113.9	15.5	0.0	0.0	145.9	177.0	160.4	112.1	63.4	42.4	17.8	0.0					891.0	17.5	908
20	17	0.0	0.0	0.0	0.0	0.0	2.8	6.1	28.0	100.9	13.7	0.0	0.0	137.7	197.4	211.6	164.7	115.1	88.0	46.3	27.2	0.0				1139.7	17.5	1157
21	14	0.0	0.0	0.0	0.0	0.0	2.4	5.2	23.9	87.9	12.1	0.0	0.0	122.5	186.3	236.1	217.4	169.1	159.7	96.2	70.8	54.3	0.0			1443.9	17.5	1461
22	11	0.0	0.0	0.0	0.0	0.0	2.0	4.4	20.3	74.8	10.6	0.0	0.0	108.8	165.8	222.7	242.5	223.1	234.7	174.6	147.1	141.1	19.3	0.0		1791.9	17.5	1809
23	9	0.0	0.0	0.0	0.0	0.0	1.7	3.7	17.1	63.5	9.0	0.0	0.0	96.4	147.2	198.2	228.8	249.0	309.7	256.6	266.9	293.1	50.3	12.0	0.0	2203.2	17.5	2221
24	7	0.0	0.0	0.0	0.0	0.0	1.4	3.2	14.5	53.7	7.6	0.0	0.0	85.3	130.4	176.0	203.6	234.9	345.6	338.6	392.2	531.9	104.4	31.3	9.6	2664.3	17.5	2682
25	5	0.0	0.0	0.0	0.0	0.0	1.2	2.7	12.5	45.6	6.5	0.0	0.0	74.3	115.5	155.9	180.8	209.0	326.0	377.8	517.5	781.6	189.5	65.1	25.0	3086.4	17.5	3104
26	3	0.0	0.0	0.0	0.0	0.0	1.0	2.3	10.4	39.0	5.5	0.0	0.0	63.3	100.6	138.1	160.2	185.6	290.2	356.4	577.4	1031.2	278.5	118.1	51.9	3409.6	17.5	3427

Time (hr)	UH Ord. (m3/s)	Effective Rainfall (cm)																								DRH (m3/s)	Baseflow (m3/s)	PMF Ord (m3/s)
27	1	0.0	0.0	0.0	0.0	0.0	0.8	1.9	8.8	32.5	4.7	0.0	0.0	53.7	85.7	120.3	141.9	164.4	257.6	317.2	544.8	1150.6	367.4	173.5	94.2	3519.9	17.5	3537
28	0	0.0	0.0	0.0	0.0	0.0	0.6	1.5	7.3	27.7	3.9	0.0	0.0	45.4	72.6	102.5	123.6	145.6	228.2	281.6	484.8	1085.5	410.0	228.9	138.5	3388.1	17.5	3406
			0.0	0.0	0.0	0.0	0.5	1.2	5.7	22.8	3.3	0.0	0.0	38.5	61.5	86.9	105.2	126.8	202.1	249.5	430.4	966.1	386.8	255.4	182.7	3125.5	17.5	3143
				0.0	0.0	0.0	0.4	0.9	4.7	17.9	2.7	0.0	0.0	33.0	52.2	73.5	89.2	108.0	176.0	221.0	381.3	857.6	344.2	241.0	203.9	2807.5	17.5	2825
					0.0	0.0	0.2	0.7	3.6	14.6	2.2	0.0	0.0	27.5	44.7	62.4	75.5	91.6	150.0	192.5	337.7	759.9	305.6	214.4	192.3	2475.4	17.5	2493
						0.0	0.1	0.4	2.6	11.4	1.8	0.0	0.0	23.4	37.3	53.5	64.1	77.5	127.1	163.9	294.2	673.0	270.7	190.4	171.2	2162.5	17.5	2180
							0.0	0.1	1.6	8.1	1.4	0.0	0.0	19.3	31.7	44.5	54.9	65.8	107.6	139.0	250.6	586.2	239.8	168.7	152.0	1871.1	17.5	1889
								0.0	0.5	4.9	1.0	0.0	0.0	15.1	26.1	37.9	45.8	56.4	91.3	117.6	212.5	499.3	208.9	149.4	134.6	1601.2	17.5	1619
									0.0	1.6	0.6	0.0	0.0	12.4	20.5	31.2	38.9	47.0	78.2	99.8	179.8	423.4	177.9	130.1	119.3	1360.6	17.5	1378
										0.0	0.2	0.0	0.0	9.6	16.8	24.5	32.0	39.9	65.2	85.5	152.5	358.2	150.8	110.8	103.9	1150.1	17.5	1168
											0.0	0.0	0.0	6.9	13.0	20.0	25.2	32.9	55.4	71.3	130.7	303.9	127.6	94.0	88.5	969.5	17.5	987
												0.0	0.0	4.1	9.3	15.6	20.6	25.8	45.6	60.6	109.0	260.5	108.3	79.5	75.0	814.0	17.5	831
													0.0	1.4	5.6	11.1	16.0	21.1	35.9	49.9	92.6	217.1	92.8	67.5	63.5	674.5	17.5	692
														0.0	1.9	6.7	11.4	16.4	29.3	39.2	76.3	184.5	77.4	57.8	53.9	554.8	17.5	572
															0.0	2.2	6.9	11.7	22.8	32.1	59.9	152.0	65.8	48.2	46.2	447.7	17.5	465
																0.0	2.3	7.0	16.3	24.9	49.0	119.4	54.1	41.0	38.5	352.6	17.5	370
																	0.0	2.3	9.8	17.8	38.1	97.7	42.5	33.7	32.7	274.8	17.5	292
																		0.0	3.3	10.7	27.2	76.0	34.8	26.5	26.9	205.4	17.5	223
																			0.0	3.6	16.3	54.3	27.1	21.7	21.2	144.1	17.5	162
																				0.0	5.4	32.6	19.3	16.9	17.3	91.5	17.5	109
																					0.0	10.9	11.6	12.0	13.5	48.0	17.5	65
																						0.0	3.9	7.2	9.6	20.7	17.5	38
																							0.0	2.4	5.8	8.2	17.5	26
																								0.0	1.9	1.9	17.5	19
																									0.0	0.0	17.5	17

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Appendix E . CASE STUDY. PROBABLE MAXIMUM FLOOD (PMF) ESTIMATION FOR DAM "B", MAHARASHTRA BY HYDRO-METEOROLOGICAL APPROACH

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DESIGN FLOOD REVIEW STUDY FOR DAM B, MAHARASHTRA

1.0 INTRODUCTION

The Dam B Project, a major irrigation Project, is located in Aurangabad district of Maharashtra State. The Latitude and longitude of Dam B site is $19^0$35'23.05" N and $76^0$57'23.79" E respectively. The catchment area of river up-to the dam site is 7848 sq km. The project consist composite earthern dam and maximum height of dam is 38.10 m. The top of the dam is located at EL 417.27 m, full reservoir level at EL 413.00 m and Maximum water level at EL 414.83 m and minimum drawdown level at EL 410.26 m above MSL. The spillway discharge capacity is 10789 cumecs and its crest level is 408.74 m. It has 14 Nos. of gates of size 4.26m x 12.19m. The reservoir has gross storage capacity of 250.36 MCM. The project was completed in the year 1968.

2.0 EARLIER DESIGN FLOOD STUDY

Neither any record of earlier flood study nor any flood hydrograph of earlier study has been available for Dam B Project. As per information submitted by Project Authorities design discharge capacity for the project is 9910 m^3/s.

3.0 DATA AVAILABILITY

The concurrent catchment representative short term rainfall and runoff data is not available, hence for estimating the synthetic unit hydrograph on the basis of physiographic parameters of the catchment, Flood Estimation Report of CWC for Upper Godavari sub zone 3(b) has been used. Further, the PMP Atlas of Godavari Basin published by CWC in November 2014 is also available, in which grid SPS and PMP values and isohyets of all severe storms occurred so far in and around the basin are available. The time distribution coefficient to estimate the hourly rainfall is also available in the PMP Atlas.

4.0 DESIGN FLOOD APPROACH

The criteria of selection of inflow design flood for safety of dam as per IS: 11223-1985 is given below in Table -1. IS: 11223-1985 is Indian standard on guidelines for fixing spillway capacity, which was adopted by the then Indian Standards Institution (now BIS) on 13 February 1985 and reaffirmed in 1995.

Table-1: Criterion for Selection of Design Flood

Classification	Gross storage	Hydraulic Head	Inflow design flood for safety of dam
Small	Between 0.5 and 10 MCM	Between 7.5 m and 12 m	100 year flood
Intermediate	Between 10 and 60 MCM	Between 12 m and 30 m	Standard Project Flood
Large	Greater than 60 MCM	Greater than 30 m	Probable Maximum Flood

Since the gross storage capacity of Dam B is 250.36 MCM which is more than 60 MCM, therefore, as per BIS criteria the Dam B qualifies for Probable Maximum Flood (PMF) as its design flood.

5.0 PHYSIOGRAPHIC PARAMETERS

The physiographic parameters of the river catchment at project site have been estimated by GIS processing of SRTM 30m DEM. As per GIS mapping the catchment area at Dam B project site is about 7848 sq.km. For the present study the catchment area at the project site has been divided into three sub catchments and design flood computations have been carried out by making a quasi distributed hydrological model on HEC-HMS. The sub catchments parameters viz. catchment area of each sub catchment, longest flow path, centroidal longest flow path, equivalent stream slope of each sub catchment/sub basin as obtained from GIS processing are given in Table-2. The catchment area plan of the project along with adopted sub-catchments is shown in Figure-1.

Table-2: Physiographic parameters of the sub catchments

S.No.	Sub-catchment	Area (sq.km)	Longest flow path L (km)	Centroidal longest flow path Lc (km)	Slopes (m/km)
1.	Sub-Basin 1	2562	103	60.6	1.56
2.	Sub-Basin 2	2461	120	41.8	1.43
3.	Sub-Basin 3	2825	196	111.0	0.84
	Total	**7848**			

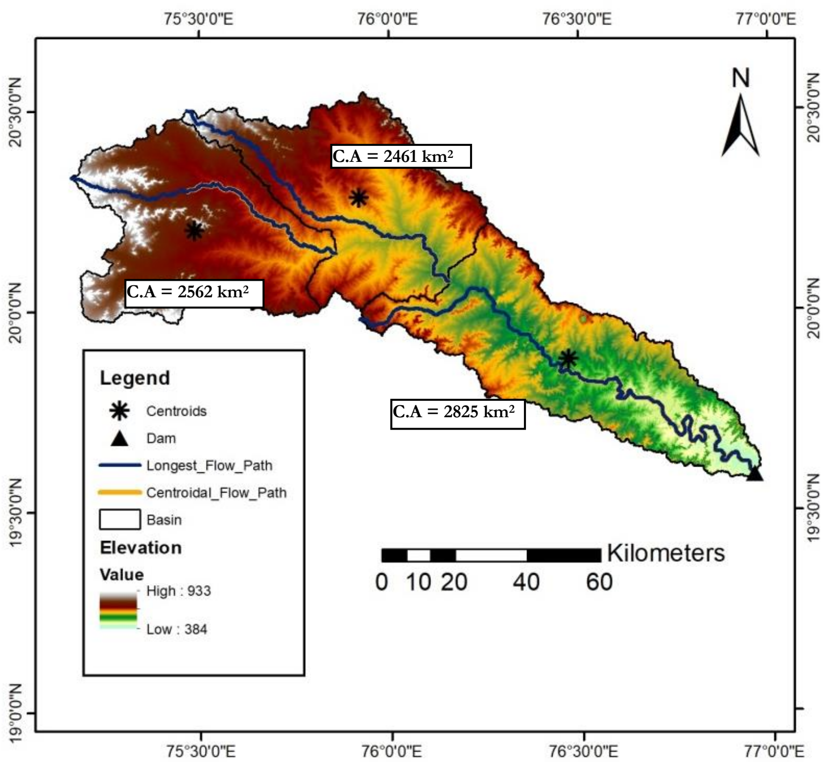


Figure- 1: Catchment plan of Dam B

6.0 ASSESSMENT OF UNIT HYDROGRAPH (UH) ORDINATES

In absence of short interval observed discharge and concurrent rainfall data; the unit hydrograph of one hour duration has been derived using Flood Estimation Report for Upper Godavari subzone 3(e). The estimated UH parameters for 3 sub-catchments are given in Table-3. The Unit Hydrograph thus generated is presented in Figure - 2

Table-3: Unit Hydrograph Parameters

Parameter	Unit	Sub-Basin 1	Sub-Basin 2	Sub-Basin 3
Time from the centre of effective rainfall duration to the UH peak tp = 0.727(L /(s0.5))0.59	hr	9.81 (rounded to 9.5)	11.04 (rounded to 11.5)	17.22 (rounded to 17.5)
Peak discharge of unit hydrograph per unit area qp =2.020/(tp)0.88 qp	m3/sec/sq. km	0.28	0.24	0.16
Width of the UH measured at 50% of peak discharge ordinate W50 = 2.228/(qp)1.04	hr	8.42	10.03	14.72
Width of the UH measured at 75% of peak discharge ordinate W75 = 1.301/(qp)0.96 W75	hr	4.44	5.21	7.43
Width of the rising limb of UH measured at 50% of peak discharge ordinate WR50 = 0.880/(qp)1.01	hr	3.20	3.79	5.51
Width of the rising limb of UH measured at 75% of	hr	1.84	2.16	3.09

peak discharge ordinate WR75 = 0.540/(qp)0.96				
Base width of UH TB = 5.485*(tp)0.73	hr	28.37 (rounded to 29)	32.62 (rounded to 33)	44.32 (rounded to 45)
Peak Discharge of UH Qp = qp x A	m3/sec	713.73	579.73	459.89
Unit duration of unit hydrograph tr	hr	1	1	1
Time from the start of rise to the peak of the UH Tm=tp+tr/2	hr	10	12	18
Q theoretical = A*d/0.36*tr here d= 1 cm depth and tr = 1 hr	m3/sec	7117	6839	7850

Table-4: 1cm-1 hr Unit Hydrograph Ordinates

Time (hr)	Ordinates of UH (SubBasin 3)	Ordinates of UH (SubBasin 2)	Ordinates of UH (SubBasin 1)
0	0	0	0
1	7	19	24
2	16	39	54
3	28	61	90
4	40	86	135
5	54	115	196
6	70	154	275
7	87	204	375
8	106	274	520
9	127	350	650
10	150	450	714
11	177	540	665
12	208	580	580
13	252	550	500
14	300	500	430
15	355	439	365
16	410	380	303
17	450	330	256
18	460	294	216
19	450	257	183
20	422	224	150
21	390	193	123
22	357	168	99
23	327	143	77
24	300	120	56
25	275	100	39
26	252	82	24
27	232	63	13
28	212	49	5
29	193	35	0
30	175	22	
31	158	13	

Time (hr)	Ordinates of UH (SubBasin 3)	Ordinates of UH (SubBasin 2)	Ordinates of UH (SubBasin 1)
32	143	5	
33	126	0	
34	109		
35	95		
36	80		
37	68		
38	55		
39	44		
40	34		
41	25		
42	17		
43	10		
44	4		
45	0		

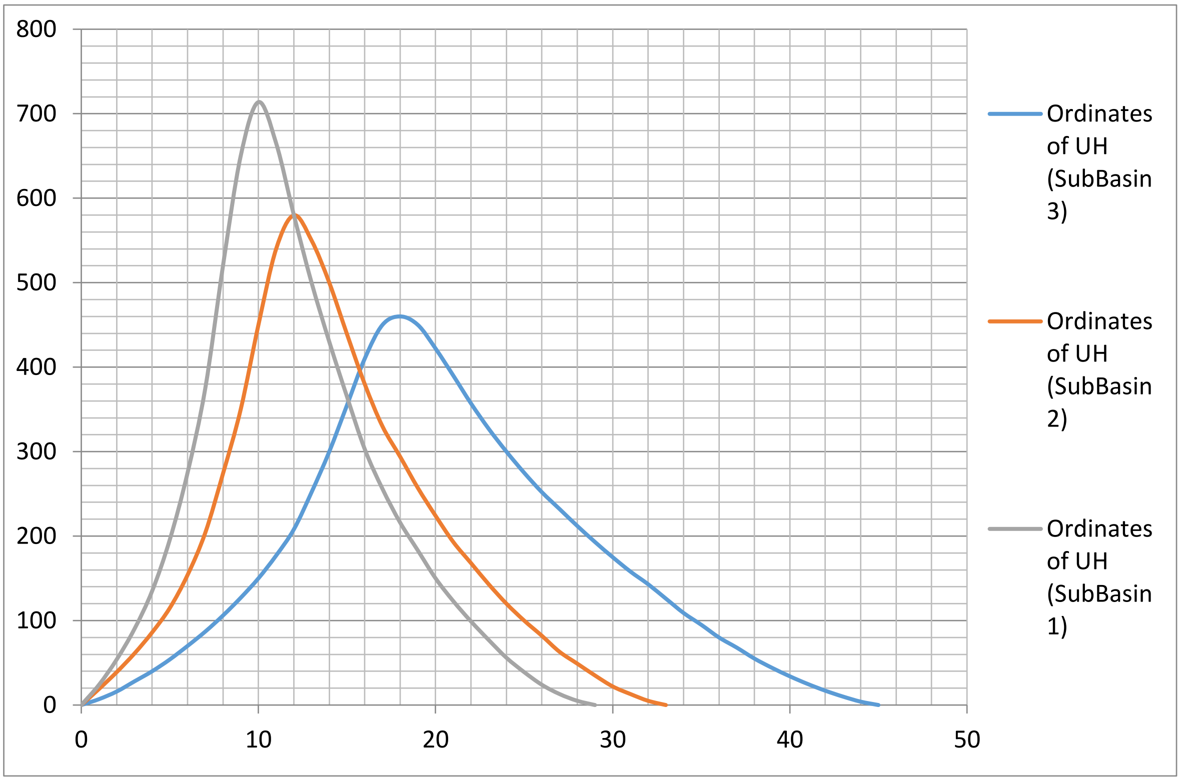


Figure- 2: 1 hr Synthetic Unit Hydrograph of 3 sub-catchments

7.0 DESIGN STORM

Dam B Project lies in PMP Atlas of Godavari River Basin published by CWC in November 2014. The project qualifies for 2-day storm for design rainfall depths computations. Further, from the PMP Atlas it has been found that the most critical 2-day storm for the Dam B catchment is the storm dated 14-15 Sept 1959 with eye of storm at Telhara, Akola and recorded 2-Day peak rainfall depth as 606 mm at its eye location. The corresponding 1-Day peak rainfall depth with eye at same location is of 392 mm at its eye location.

For the present study, the catchment area of Dam B site has been sub-divided into 3 sub-catchments. Based on the critical transposition of the storm isohyetal-pattern of above two storms, it has been found that the overall two day and one day storm depth in the drainage area (7848 sq.km) of the Dam B is 382.15 mm for 2-day storm, and 230.22 mm for 1-day storm. The storm isohyets for 2-day and 1-day storm after transposition to project catchment are presented in Fig.3 and Fig.4 respectively.

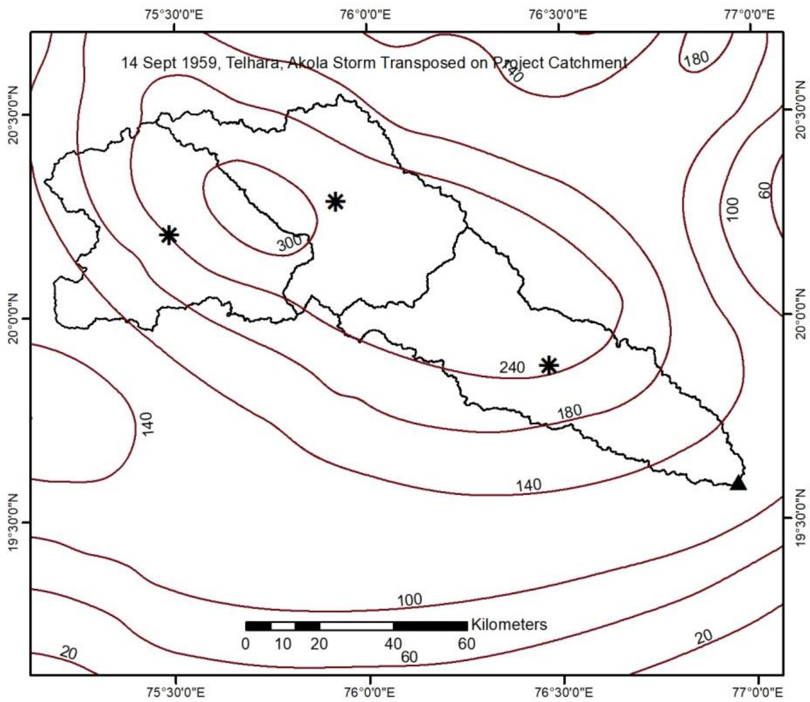


Fig. 3: 14 Sept 1959 Storm transposed at Dam B catchment

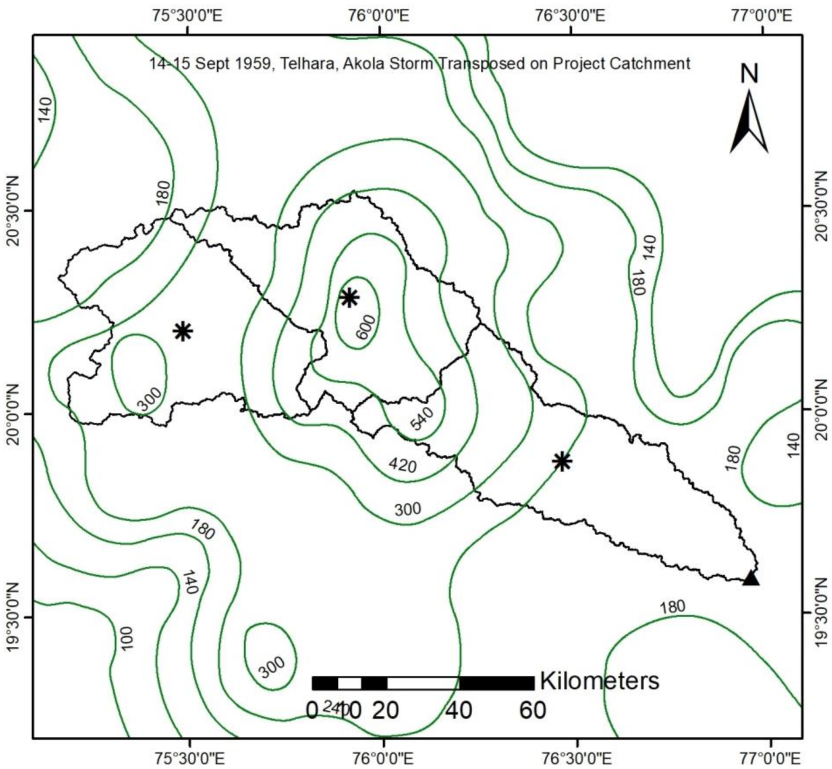


Fig. 4: 14-15 Sept, 1959 Storm transposed at Dam B catchment

With the transposed storm isohyets presented in Fig.3 and Fig.4, the realized storm depths for different sub-catchments have been multiplied with Transport Adjustment Factor (TAF) to get Standard Projected Storm (SPS) and the Moisture Adjustment Factor (MAF) to get the Probable Maximum Precipitation (PMP) depths for that sub-catchment. For computation fo TAF and MAF, tables 8-2 and table 8-3 of PMP atlas published by CWC during 2014-2015 have been used. For this project, TAF and MAF have been calculated as 0.8125 and 0.9483 respectively using the parameters h_1, h_2, d_1, d_2, d_3 which have been explained earlier. The computed Transposed depth and PMP depths for each sub-catchment are shown in Table-5.

Table-5: Design Storm depths for catchment of Dam B

Sub Basin	Catchment	1 day and 2 day storm depths					
	Area (km2)	Transposed Depth (mm)		SPS (mm)		PMP (mm)	
		1-day	2-day	1-day	2-day	1-day	2-day
SB-1	2562	22.67	29.22	18.42	23.74	21.50	27.71
SB-2	2461	26.97	49.34	21.91	40.09	25.58	46.79
SB-3	2825	19.90	36.68	16.50	30.41	18.87	25.59

The 24 hour rainfall has been converted into two 12 hourly rainfall bells by using the time distribution as per PMP atlas for Godavari Basin. For hourly distribution of rainfall normalized distribution coefficient has been worked out for bell of 12 hours each using the hourly distribution coefficient. The hourly distribution coefficient of 24 hour rainfall and normalized distribution coefficient for 12 hour bell are given in Table-6.

Table-6: Hourly distribution coefficient of 24 hour rainfall and normalized distribution coefficient for 12 hour bell

Time (hr)	Disn coeff	Normalised dist coeff
1	18.3	27.03
2	25.4	37.52
3	30.2	44.61
4	34.2	50.52
5	42.5	62.78
6	47.5	70.16
7	50.9	75.18
8	55.4	81.83
9	59.2	87.44
10	62.1	91.73
11	64.7	95.57
12	67.7	100.00
	70.6	
	74.1	
	77.5	
	81.8	
	85.1	
	89.1	
	91.8	
	92.5	
	94.0	
	96.5	
	98.7	
	100.0	

As 2day PMP has been used for estimating the design flood, the incremental daily PMP for each sub catchment has been worked out as follows:

$$2^{nd} \text{ day PMP} = (2 \text{ day PMP}) - (1 \text{ day PMP})$$

The 1st day, 2nd day PMP along with PMP depths for each bell is given in Table-7.

Table-7: PMP depth for 12 hour bells

Sub basin	1-Day	2-Day	1st Day	2nd Day PMP	24 hr max	1st bell	2nd bell	3rd bell	4th bell
	PMP	PMP	PMP	col(2)-col(1)		B1	B2	B3	B4
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
SB-1	**215**	**277**	**215**	**62.1**	**265**	**145.5**	**69.4**	**42.1**	**20.1**

SB-2	**256**	**468**	**256**	**212**	**306**	**173.1**	**82.6**	**143.6**	**68.5**
SB-3	**189**	**256**	**189**	**67.2**	**239**	**127.8**	**61**	**45.5**	**21.5**
Since sum of bell B1 and B3 is more than 24 hr rainfall in 2 sub-catchments, hence, Bell sequence used for convolution is : B1-B2-B3-B4									

8.0 DESIGN LOSS RATE AND BASE FLOW

The average loss rate has been adopted from Flood Estimation Report for Upper Godavari sub zone 3 (e). An average loss rate of 0.25 cm/hour has been used in the study.

As recommended by CWC subzone 3(b) report following base flow rate has been adopted:

Base flow / km2 of drainage area = 0.122*(A-0.304)

Using the above formula, the computed base flow for different sub-catchments are given in Table-8.

Table-8: Base flow for sub-catchments

Sub-Basin	Base flow (cumec)
SB-1	28.75
SB-2	28
SB-2	30.78

9.0 HOURLY DISTRIBUTION AND CRITICAL SEQUENCING OF RAINFALL

The hourly distribution of rainfall of each bell has been carried out taking the normalized distribution coefficients of Table-6. The hourly distribution of rainfall for sub basins SB1, SB-2 and SB-3 are given in Table-9, Table-10 and Table-11 respectively. In order to keep the rainfall sequence in proper order the critical sequencing of the hourly rainfall excess of the bells need to be carried out with respect to UH of one of the sub catchment preferably the central sub catchment. For the present case the critical sequencing of hourly rainfall excess of the bells of all the sub-catchments have been carried out with respect to UH of sub-catchment SB2. The critical sequencing of hourly effective rainfall for sub catchments SB1, SB-2 and SB-3 are given in Table-12, Table-13 and Table-14 respectively.

Table-9: Hourly distribution of rainfall of SB1

Bell	PMP depth (cm)
1st 12 hr bell	14.55
2nd 12 hr bell	6.94
3rd 12 hr bell	4.21
4th 12 hr bell	2.01

			Cumulative rainfall depth	Incremental rainfall depth		Effective rainfall depth

Time (hr)	Disn coef f	Normalised dist coeff	1st 12 hr bell	2nd 12 hr bell	3rd 12 hr bell	4th 12 hr bell	1st bell	2nd bell	3rd bell	4th bell	Loss rate 0.25 cm/hr	1st bell	2nd bell	3rd bell	4th bell
			cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm
1	18.3	27.03	3.93	1.88	1.14	0.54	3.93	1.88	1.14	0.54	0.25	3.68	1.63	0.89	0.29
2	25.4	37.52	5.46	2.61	1.58	0.75	1.53	0.73	0.44	0.21	0.25	1.28	0.48	0.19	0.00
3	30.2	44.61	6.49	3.10	1.88	0.89	1.03	0.49	0.30	0.14	0.25	0.78	0.24	0.05	0.00
4	34.2	50.52	7.35	3.51	2.12	1.01	0.86	0.41	0.25	0.12	0.25	0.61	0.16	0.00	0.00
5	42.5	62.78	9.14	4.36	2.64	1.26	1.78	0.85	0.52	0.25	0.25	1.53	0.60	0.27	0.00
6	47.5	70.16	10.21	4.87	2.95	1.41	1.07	0.51	0.31	0.15	0.25	0.82	0.26	0.06	0.00
7	50.9	75.18	10.94	5.22	3.16	1.51	0.73	0.35	0.21	0.10	0.25	0.48	0.10	0.00	0.00
8	55.4	81.83	11.91	5.68	3.44	1.64	0.97	0.46	0.28	0.13	0.25	0.72	0.21	0.03	0.00
9	59.2	87.44	12.73	6.07	3.68	1.75	0.82	0.39	0.24	0.11	0.25	0.57	0.14	0.00	0.00
10	62.1	91.73	13.35	6.37	3.86	1.84	0.62	0.30	0.18	0.09	0.25	0.37	0.05	0.00	0.00
11	64.7	95.57	13.91	6.64	4.02	1.92	0.56	0.27	0.16	0.08	0.25	0.31	0.02	0.00	0.00
12	67.7	100.00	14.55	6.94	4.21	2.01	0.64	0.31	0.19	0.09	0.25	0.39	0.06	0.00	0.00
						Sum	14.55	6.94	4.21	2.01	3.00	11.55	3.94	1.48	0.29

Table-10: Hourly distribution of rainfall of SB2

Bell	PMP depth (cm)
1st 12 hr bell	17.31
2nd 12 hr bell	8.26
3rd 12 hr bell	14.36
4th 12 hr bell	6.85

			Cumulative rainfall depth				Incremental rainfall depth					Effective rainfall depth			
Time (hr)	Disn coef f	Normalised dist coeff	1st bell	2nd bell	3rd bell	4th bell	1st bell	2nd bell	3rd bell	4th bell	Loss rate 0.25 cm/hr	1st bell	2nd bell	3rd bell	4th bell
			cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm
1	18.3	27.03	4.68	2.23	3.88	1.85	4.68	2.23	3.88	1.85	0.25	4.43	1.98	3.63	1.60
2	25.4	37.52	6.50	3.10	5.39	2.57	1.82	0.87	1.51	0.72	0.25	1.57	0.62	1.26	0.47
3	30.2	44.61	7.72	3.69	6.41	3.06	1.23	0.59	1.02	0.49	0.25	0.98	0.34	0.77	0.24
4	34.2	50.52	8.75	4.17	7.26	3.46	1.02	0.49	0.85	0.40	0.25	0.77	0.24	0.60	0.15
5	42.5	62.78	10.87	5.19	9.02	4.30	2.12	1.01	1.76	0.84	0.25	1.87	0.76	1.51	0.59
6	47.5	70.16	12.15	5.80	10.08	4.81	1.28	0.61	1.06	0.51	0.25	1.03	0.36	0.81	0.26
7	50.9	75.18	13.02	6.21	10.80	5.15	0.87	0.41	0.72	0.34	0.25	0.62	0.16	0.47	0.09
8	55.4	81.83	14.17	6.76	11.75	5.61	1.15	0.55	0.95	0.46	0.25	0.90	0.30	0.70	0.21
9	59.2	87.44	15.14	7.22	12.56	5.99	0.97	0.46	0.81	0.38	0.25	0.72	0.21	0.56	0.13
10	62.1	91.73	15.88	7.58	13.17	6.29	0.74	0.35	0.62	0.29	0.25	0.49	0.10	0.37	0.04
11	64.7	95.57	16.55	7.89	13.73	6.55	0.66	0.32	0.55	0.26	0.25	0.41	0.07	0.30	0.01
12	67.7	100.00	17.31	8.26	14.36	6.85	0.77	0.37	0.64	0.30	0.25	0.52	0.12	0.39	0.05
						Sum	17.31	8.26	14.36	6.85	3.00	14.31	5.26	11.36	3.85

Table-11: Hourly distribution of rainfall of SB3

Bell	PMP depth (cm)
1st 12 hr bell	12.78
2nd 12 hr bell	6.10
3rd 12 hr bell	4.55
4th 12 hr bell	2.17

			Cumulative rainfall depth				Incremental rainfall depth					Effective rainfall depth			
Time (hr)	Disn coeff	Normalised dist coeff	1st bell	2nd bell	3rd bell	4th bell	1st bell	2nd bell	3rd bell	4th bell	Loss rate 0.25 cm/hr	1st bell	2nd bell	3rd bell	4th bell
			cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm
1	18.3	27.03	3.45	1.65	1.23	0.59	3.45	1.65	1.23	0.59	0.25	3.20	1.40	0.98	0.34
2	25.4	37.52	4.79	2.29	1.71	0.81	1.34	0.64	0.48	0.23	0.25	1.09	0.39	0.23	0.00
3	30.2	44.61	5.70	2.72	2.03	0.97	0.91	0.43	0.32	0.15	0.25	0.66	0.18	0.07	0.00
4	34.2	50.52	6.45	3.08	2.30	1.10	0.75	0.36	0.27	0.13	0.25	0.50	0.11	0.02	0.00
5	42.5	62.78	8.02	3.83	2.86	1.36	1.57	0.75	0.56	0.27	0.25	1.32	0.50	0.31	0.02
6	47.5	70.16	8.96	4.28	3.19	1.52	0.94	0.45	0.34	0.16	0.25	0.69	0.20	0.09	0.00
7	50.9	75.18	9.61	4.58	3.42	1.63	0.64	0.31	0.23	0.11	0.25	0.39	0.06	0.00	0.00
8	55.4	81.83	10.45	4.99	3.72	1.78	0.85	0.41	0.30	0.14	0.25	0.60	0.16	0.05	0.00
9	59.2	87.44	11.17	5.33	3.98	1.90	0.72	0.34	0.26	0.12	0.25	0.47	0.09	0.01	0.00
10	62.1	91.73	11.72	5.59	4.18	1.99	0.55	0.26	0.19	0.09	0.25	0.30	0.01	0.00	0.00
11	64.7	95.57	12.21	5.83	4.35	2.08	0.49	0.23	0.17	0.08	0.25	0.24	0.00	0.00	0.00
12	67.7	100.00	12.78	6.10	4.55	2.17	0.57	0.27	0.20	0.10	0.25	0.32	0.02	0.00	0.00
						Sum	12.78	6.10	4.55	2.17	3.00	9.78	3.11	1.75	0.35

Table-12: Critical sequencing for hourly rainfall of SB1with respect to UH of SB2

Duration (Hrs)	Ordinates of UH (cumecs)	Critical seq	Critical seq	Critical seq	Critical seq	Reversed sequence	Reversed sequence	Reversed sequence	Reversed sequence	Conv Rainfall
						B1	B2	B3	B4	cm
0	0									0.31
1	19									0.39
2	39									0.48
3	61									0.61
4	86									0.72
5	115									0.82
6	154									1.53
7	204									3.68
8	274	0.37	0.05	0.00	0.00	0.31	0.02	0.00	0.00	1.28
9	350	0.57	0.14	0.00	0.00	0.39	0.06	0.00	0.00	0.78
10	450	0.78	0.24	0.05	0.00	0.48	0.10	0.00	0.00	0.57
11	540	1.28	0.48	0.19	0.00	0.61	0.16	0.00	0.00	0.37
12	580	3.68	1.63	0.89	0.29	0.72	0.21	0.03	0.00	0.02
13	550	1.53	0.60	0.27	0.00	0.82	0.26	0.06	0.00	0.06

Duration (Hrs)	**Ordinates of UH (cumecs)**	**Critical seq**	**Critical seq**	**Critical seq**	**Critical seq**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Conv Rainfall**
						B1	B2	B3	B4	cm
14	500	0.82	0.26	0.06	0.00	1.53	0.60	0.27	0.00	0.10
15	439	0.72	0.21	0.03	0.00	3.68	1.63	0.89	0.29	0.16
16	380	0.61	0.16	0.00	0.00	1.28	0.48	0.19	0.00	0.21
17	330	0.48	0.10	0.00	0.00	0.78	0.24	0.05	0.00	0.26
18	294	0.39	0.06	0.00	0.00	0.57	0.14	0.00	0.00	0.60
19	257	0.31	0.02	0.00	0.00	0.37	0.05	0.00	0.00	1.63
20	224									0.48
21	193									0.24
22	168									0.14
23	143									0.05
24	120									0.00
25	100									0.00
26	82									0.00
27	63									0.00
28	49									0.03
29	35									0.06
30	22									0.27
31	13									0.89
32	5									0.19
33	0									0.05
										0.00
										0.00
										0.00
										0.00
										0.00
										0.00
										0.00
										0.00
										0.00
										0.29
										0.00
										0.00
										0.00
										0.00

Reversed critical sequence used for convolution with UH of SB1 : B1-B2-B3-B4

Table-13: Critical sequencing for hourly rainfall of SB2with respect to UH of SB2

										B1-B2-B3-B4
Duration (Hrs)	**Ordinates of UH (cumecs)**	**Critical seq**	**Critical seq**	**Critical seq**	**Critical seq**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Conv Rainfall**
						B1	**B2**	**B3**	**B4**	**cm**
0	0									0.41
1	19									0.52
2	39									0.62
3	61									0.77
4	86									0.90
5	115									1.03
6	154									1.87
7	204									4.43
8	274	0.49	0.10	0.37	0.04	0.41	0.07	0.30	0.01	1.57
9	350	0.72	0.21	0.56	0.13	0.52	0.12	0.39	0.05	0.98
10	450	0.98	0.34	0.77	0.24	0.62	0.16	0.47	0.09	0.72
11	540	1.57	0.62	1.26	0.47	0.77	0.24	0.60	0.15	0.49
12	580	4.43	1.98	3.63	1.60	0.90	0.30	0.70	0.21	0.07
13	550	1.87	0.76	1.51	0.59	1.03	0.36	0.81	0.26	0.12
14	500	1.03	0.36	0.81	0.26	1.87	0.76	1.51	0.59	0.16
15	439	0.90	0.30	0.70	0.21	4.43	1.98	3.63	1.60	0.24
16	380	0.77	0.24	0.60	0.15	1.57	0.62	1.26	0.47	0.30
17	330	0.62	0.16	0.47	0.09	0.98	0.34	0.77	0.24	0.36
18	294	0.52	0.12	0.39	0.05	0.72	0.21	0.56	0.13	0.76
19	257	0.41	0.07	0.30	0.01	0.49	0.10	0.37	0.04	1.98
20	224									0.62
21	193									0.34
22	168									0.21
23	143									0.10
24	120									0.30
25	100									0.39
26	82									0.47
27	63									0.60
28	49									0.70
29	35									0.81
30	22									1.51
31	13									3.63
32	5									1.26
33	0									0.77
										0.56
										0.37
										0.01
										0.05

0.09
0.15
0.21
0.26
0.59
1.60
0.47
0.24
0.13
0.04

Reversed critical sequence used for convolution with UH of SB2 : B1-B2-B3-B4

Table-14: Critical sequencing for hourly rainfall of SB3 with respect to UH of SB2

										B1-B2-B3-B4
Duration (Hrs)	**Ordinates of UH (cumecs)**	**Critical seq**	**Critical seq**	**Critical seq**	**Critical seq**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Reversed sequence**	**Conv Rainfall**
						B1	**B2**	**B3**	**B4**	**cm**
0	0									0.24
1	19									0.32
2	39									0.39
3	61									0.50
4	86									0.60
5	115									0.69
6	154									1.32
7	204									3.20
8	274	0.30	0.01	0.00	0.00	0.24	0.00	0.00	0.00	1.09
9	350	0.47	0.09	0.01	0.00	0.32	0.02	0.00	0.00	0.66
10	450	0.66	0.18	0.07	0.00	0.39	0.06	0.00	0.00	0.47
11	540	1.09	0.39	0.23	0.00	0.50	0.11	0.02	0.00	0.30
12	580	3.20	1.40	0.98	0.34	0.60	0.16	0.05	0.00	0.00
13	550	1.32	0.50	0.31	0.02	0.69	0.20	0.09	0.00	0.02
14	500	0.69	0.20	0.09	0.00	1.32	0.50	0.31	0.02	0.06
15	439	0.60	0.16	0.05	0.00	3.20	1.40	0.98	0.34	0.11
16	380	0.50	0.11	0.02	0.00	1.09	0.39	0.23	0.00	0.16
17	330	0.39	0.06	0.00	0.00	0.66	0.18	0.07	0.00	0.20
18	294	0.32	0.02	0.00	0.00	0.47	0.09	0.01	0.00	0.50
19	257	0.24	0.00	0.00	0.00	0.30	0.01	0.00	0.00	1.40
20	224									0.39
21	193									0.18
22	168									0.09
23	143									0.01
24	120									0.00

25	100
26	82
27	63
28	49
29	35
30	22
31	13
32	5
33	0

0.00
0.00
0.02
0.05
0.09
0.31
0.98
0.23
0.07
0.01
0.00
0.00
0.00
0.00
0.00
0.00
0.00
0.02
0.34
0.00
0.00
0.00
0.00

10.0 CONVOLUTION AND CHANNEL ROUTING OF FLOOD HYDROGRAPHS

- The convolution and channel routing of flood hydrographs of different sub catchments have been carried out on HEC-HMS hydrological model. The HEC-HMS model set up is shown in **Fig.5**. The reversed critical sequence of hourly effective rainfall of each bell of the respective sub-catchments as given in **Table-12, Table-13 and Table-14** respectively, have been convoluted with UH of that sub catchment. The base-flow contributions of the sub-catchments have been added to get the total response function viz. flood hydrograph at the outlet of that sub-catchment.

- Further, as shown in **Fig.5**, flood hydrograph of Subbasin-1 sub-catchment coming at outlet (Junction-1) has been channel routed through river reach Reach-1 and Reach-2 to get its response function at Dam. Flood Hydrograph of Subbasin-2 sub catchment coming at Junction -2 has been channel routed through river reach Reach-2 to get its response function at Dam. At Dam, both the flood hydrograph of subbasin-3 and the routed response of sub-basin Subbasin-1 and Subbasin-2 are added to get the design flood hydrograph at Dam B.

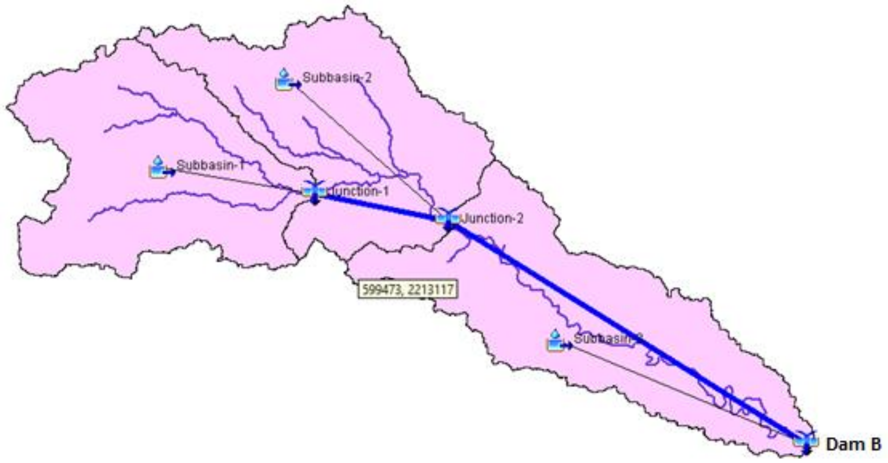


Fig.5: HEC-HMS model set up for PMF computation of Dam B

The channel routings through the river reaches as per the model set up have been carried out using Muskingum method. Since Δt > 2KX for stability of Muskingum routing algorithm, hence channel routing in river reaches have been carried out in steps by dividing the routing reach in 3 sub reaches for Reach between Junction-1 & Junction-2 and in 5 sub-reaches for Reach between Junction-2 and Dam. The Muskingum K and X parameters used for the Reaches routing are given below:

Reach between	**K**	**X**
Junction 1 & Junction -2	**10**	**0.15**
Junction -2 & Dam	**15**	**0.15**

1.0 PROBABLE MAXIMUM FLOOD (PMF) FOR DAM B

Based on the methodology discussed above the estimated Probable Maximum Flood (PMF) for Dam B Project is 11803 cumec. The same is recommended as design flood for the project. The PMF hydrograph is given in Table-15. A plot of the same is presented in Fig.6. The HEC-HMS simulation plots viz. hydrograph of each sub catchment, Reach and dam plots are presented in Annexure-I.

Table-14: PMF hydrograph for Dam B

Time (hrs)	PMF Ordinate
0	87.5
1	89.2
2	93.6
3	102.1

Time (hrs)	PMF Ordinate
49	10049.1
50	9871.6
51	9683.6
52	9491

Time (hrs)	PMF Ordinate
98	130.3
99	121.6
100	114.6
101	109

Time (hrs)	PMF Ordinate
4	115.8
5	136.4
6	165.8
7	209.4
8	283.2
9	379.7
10	499.4
11	637.7
12	800.5
13	991.6
14	1214.7
15	1479.6
16	1794.8
17	2166.5
18	2600
19	3103.3
20	3697.2
21	4356.8
22	5076.5
23	5820.5
24	6533.2
25	7165.4
26	7725.6
27	8221.6
28	8684.1
29	9126.3
30	9561.4
31	9989.2
32	10413.5
33	10812
34	11175.3
35	11475.5
36	11687.3
37	11792.7
38	11802.8
39	11731.5
40	11597.6
41	11424.2
42	11234.6
43	11035.5
44	10849.3
45	10670.4

Time (hrs)	PMF Ordinate
53	9296.6
54	9091.6
55	8866.4
56	8623.6
57	8357.3
58	8074.4
59	7775.6
60	7462.1
61	7134.3
62	6799.1
63	6458.8
64	6116.8
65	5776.5
66	5436.1
67	5092.6
68	4748.4
69	4402.5
70	4060.1
71	3722.6
72	3393.8
73	3076.9
74	2774.8
75	2490.3
76	2224.9
77	1981.5
78	1757.9
79	1552.3
80	1365.3
81	1195
82	1041.9
83	904.9
84	783.1
85	675.6
86	581.5
87	499.8
88	429.9
89	371.2
90	321.4
91	279.4
92	244.2
93	214.9
94	190.7

Time (hrs)	PMF Ordinate
102	104.5
103	100.9
104	98
105	95.8
106	94
107	92.6
108	91.5
109	90.6
110	89.9
111	89.4
112	89
113	88.6
114	88.4
115	88.2
116	88
117	87.9
118	87.8
119	87.8
120	87.7
121	87.7
122	87.6
123	87.6
124	87.6
125	87.6
126	87.6
127	87.6
128	87.6
129	87.5
130	87.5
131	87.5
132	87.5
133	87.5
134	87.5
135	87.5
136	87.5
137	87.5
138	87.5
139	87.5
140	87.5
141	87.5
142	87.5
143	87.5

Time (hrs)	PMF Ordinate
46	10512.3
47	10364.7
48	10214.9

Time (hrs)	PMF Ordinate
95	170.8
96	154.4
97	141.1

Time (hrs)	PMF Ordinate
144	87.5

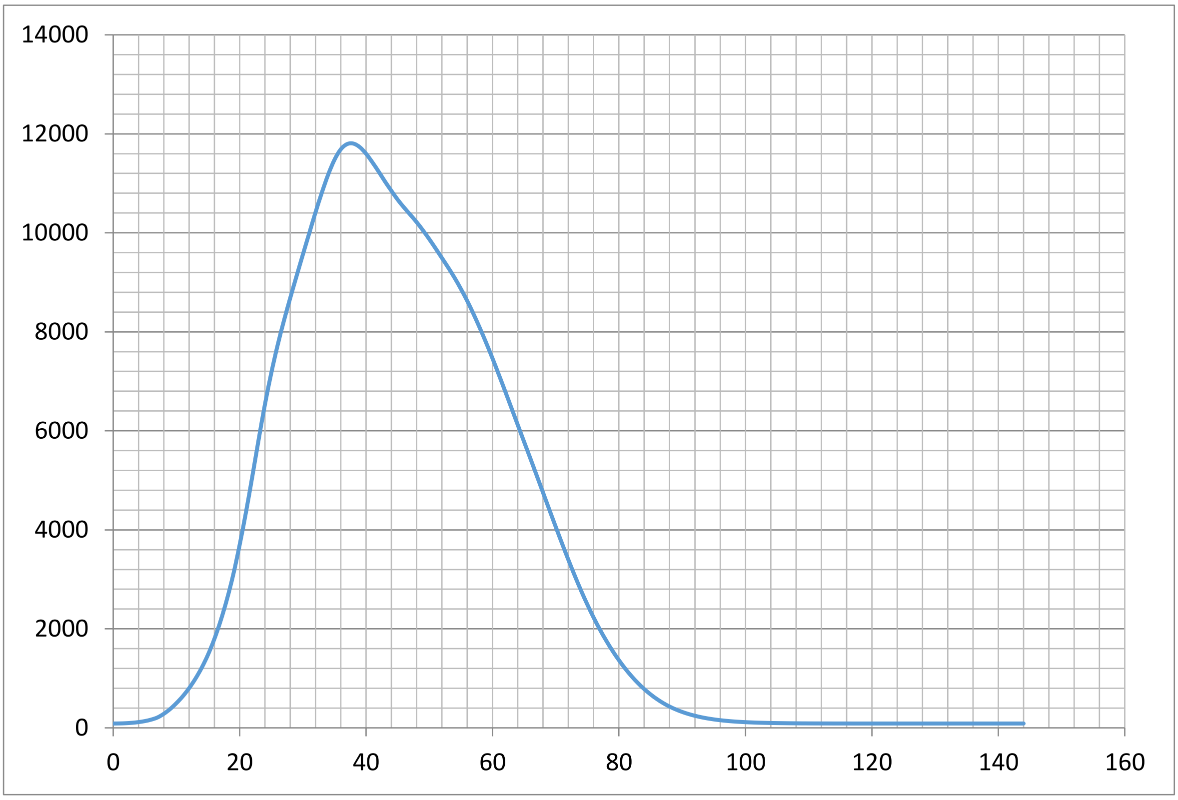
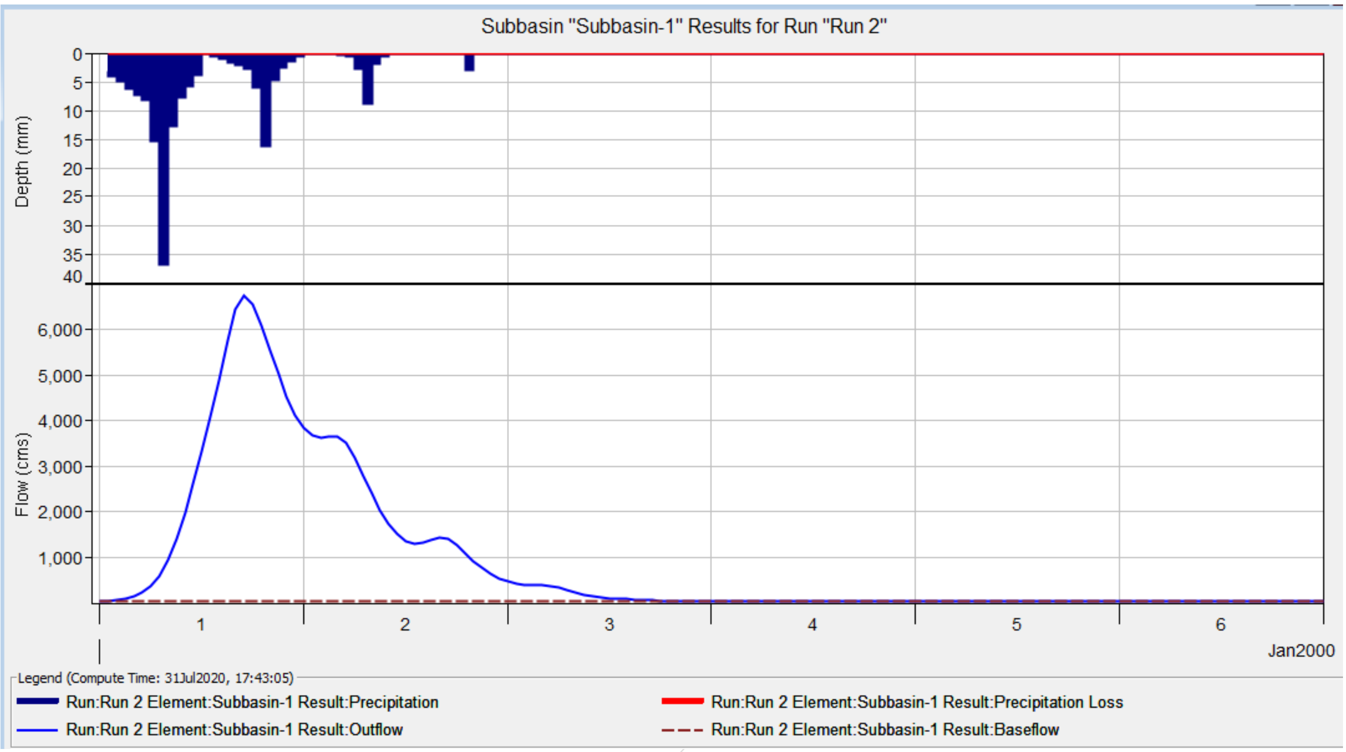
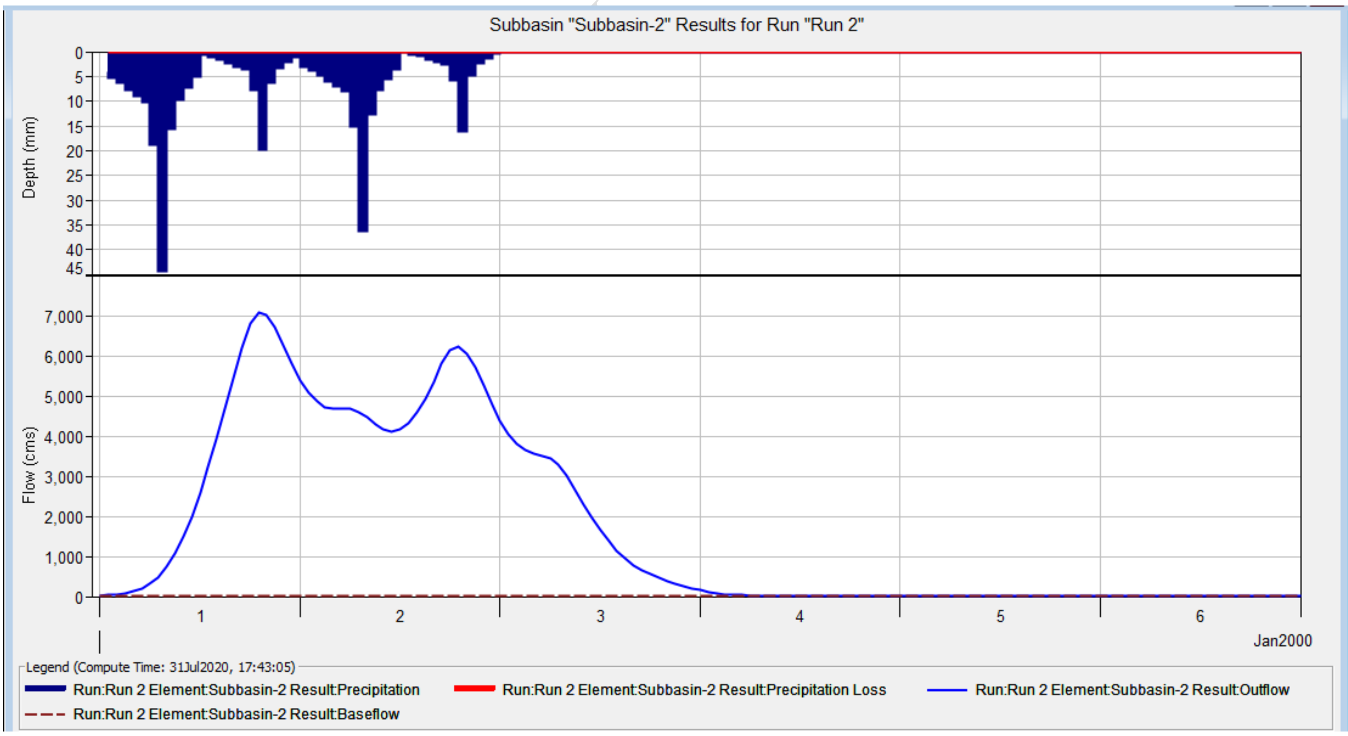


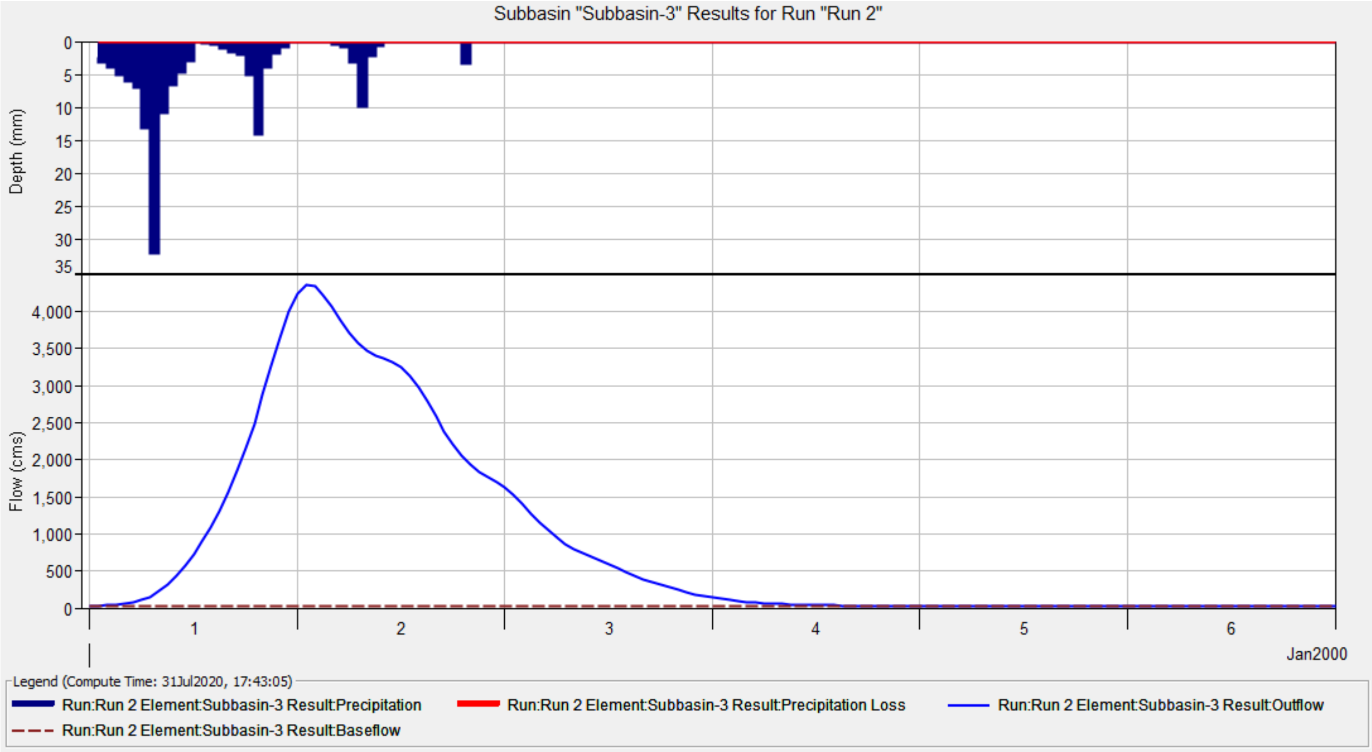
Figure - 6: PMF hydrograph for Dam B Project

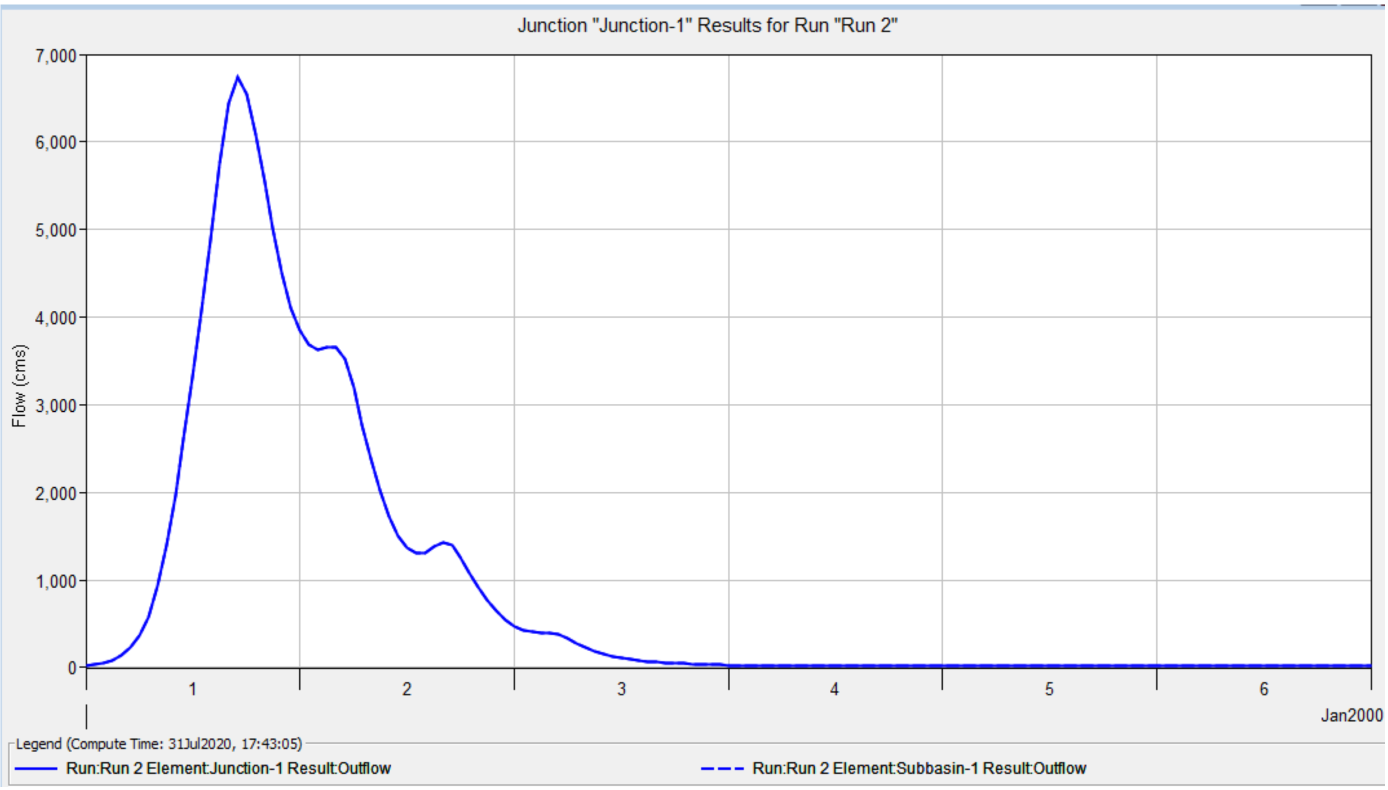
ANNEXURE-I

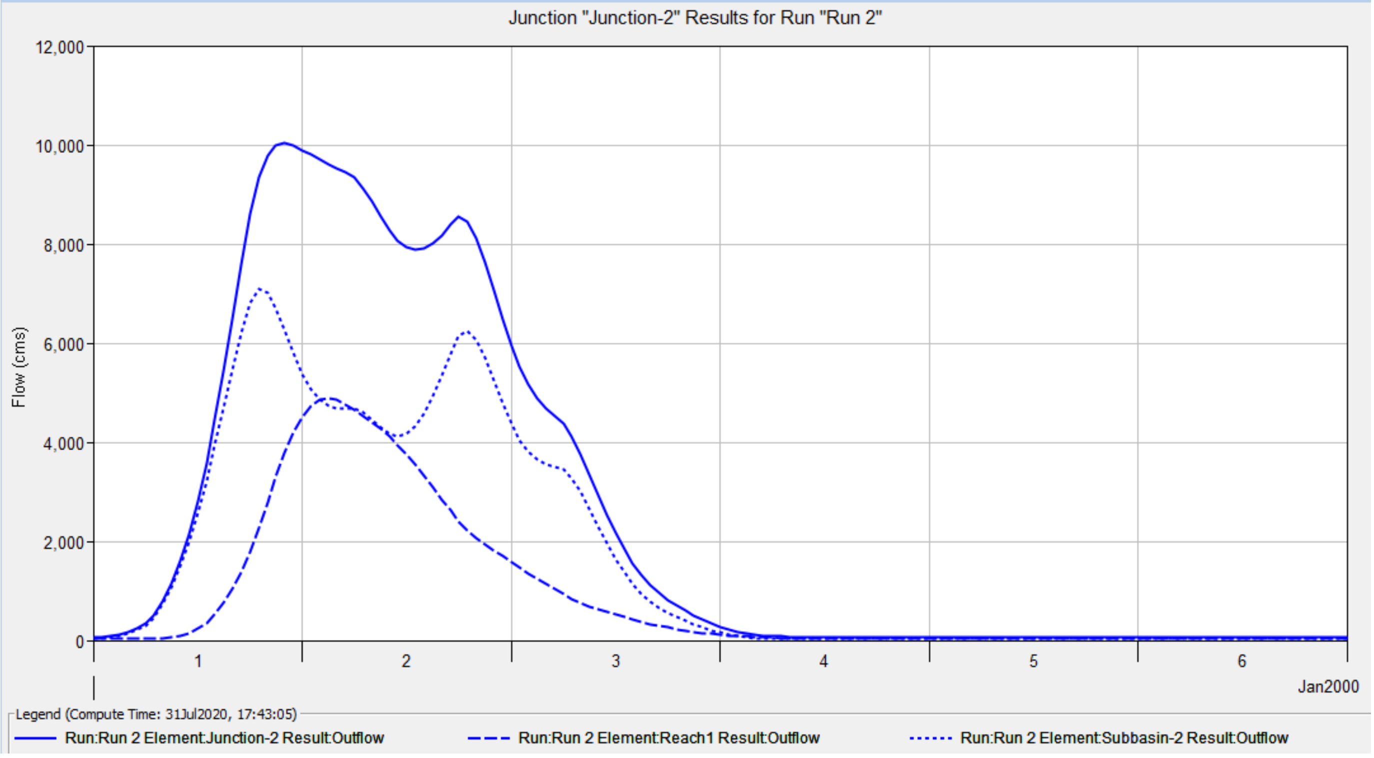
Note: The dates shown on time axis of the plots are not the actual dates but arbitrary dates used for model simulation only

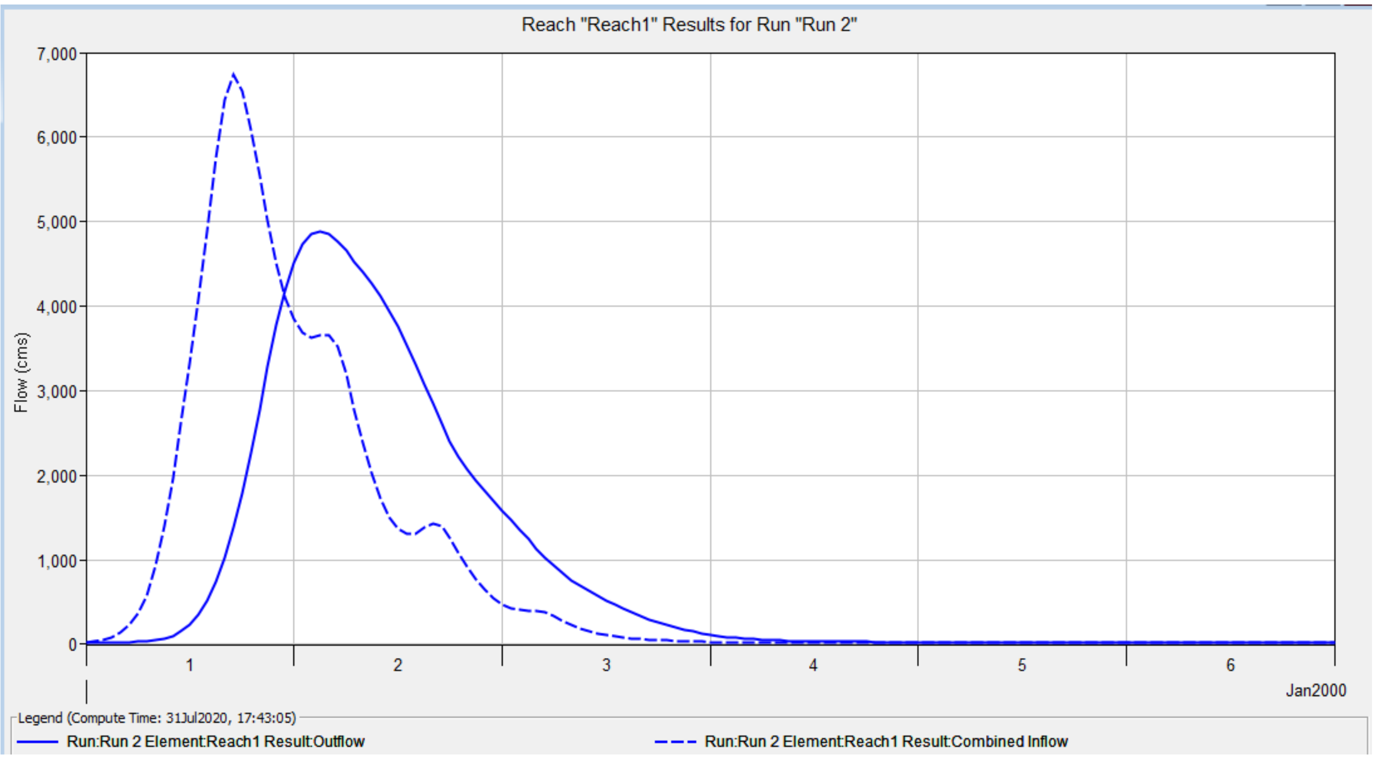


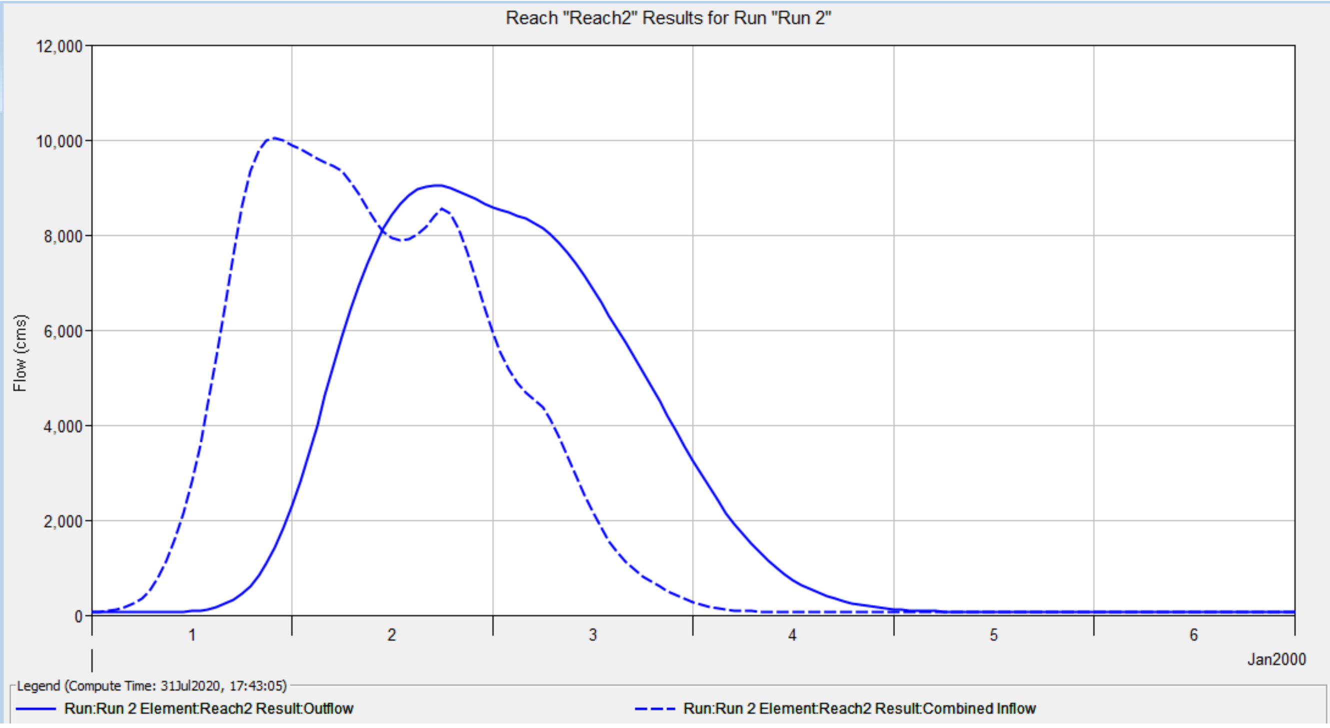


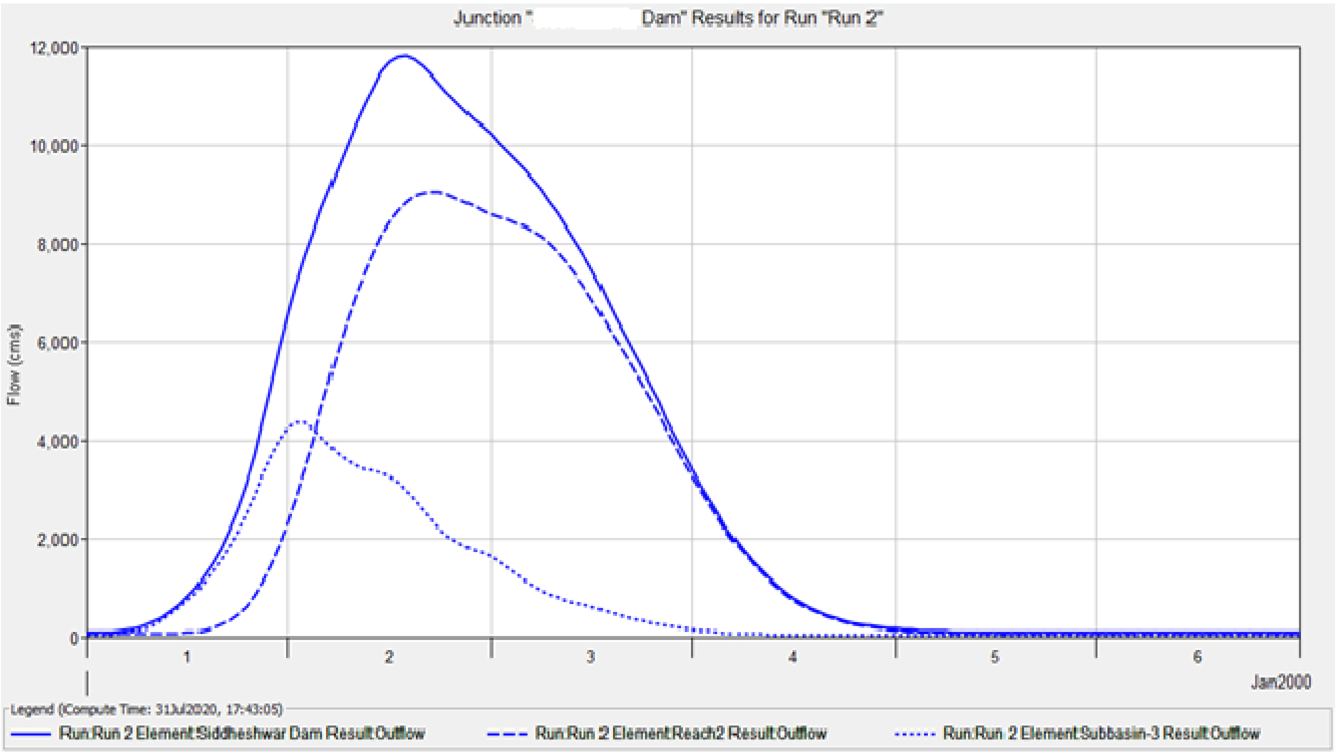

Subbasin "Subbasin-3" Results for Run "Run 2"
Depth (mm)
0
5
10
15
20
25
30
35
Flow (cms)
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Subbasin-3 Result:Precipitation
Run:Run 2 Element:Subbasin-3 Result:Precipitation Loss
Run:Run 2 Element:Subbasin-3 Result:Outflow
Run:Run 2 Element:Subbasin-3 Result:Baseflow


Junction "Junction-1" Results for Run "Run 2"
Flow (cms)
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Junction-1 Result:Outflow
Run:Run 2 Element:Subbasin-1 Result:Outflow


Junction "Junction-2" Results for Run "Run 2"
Flow (cms)
12,000
10,000
8,000
6,000
4,000
2,000
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Junction-2 Result:Outflow
Run:Run 2 Element:Reach1 Result:Outflow
Run:Run 2 Element:Subbasin-2 Result:Outflow


Reach "Reach1" Results for Run "Run 2"
Flow (cms)
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Reach1 Result:Outflow
Run:Run 2 Element:Reach1 Result:Combined Inflow


Reach "Reach2" Results for Run "Run 2"
Flow (cms)
12,000
10,000
8,000
6,000
4,000
2,000
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Reach2 Result:Outflow
Run:Run 2 Element:Reach2 Result:Combined Inflow


Junction " Dam" Results for Run "Run 2"
Flow (cms)
12,000
10,000
8,000
6,000
4,000
2,000
0
1
2
3
4
5
6
Jan2000
Legend (Compute Time: 31Jul2020, 17:43:05)
Run:Run 2 Element:Siddheshwar Dam Result:Outflow
Run:Run 2 Element:Reach2 Result:Outflow
Run:Run 2 Element:Subbasin-3 Result:Outflow

Appendix F . EXAMPLE OF RULE CURVE DETERMINATION

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EXAMPLE OF RULE CURVE DETERMINATION

The example of application of the determination of rule curves for multipurpose reservoir presented herein uses a hypothetical dam. The flows and the fraction of the annual irrigation demand corresponding to each month, however, are the real historical values of flow and irrigation demand fraction at the Konar Dam's reservoir (Damodar Valley Corporation). All other characteristics in this example are artificial.

As mentioned in the Section 6.5.2 "Optimising the Reservoir Rule Curve", there are some simplifications in the calculations carried out in this example. These simplifications are:

1. Maximum discharge through the power plant is assumed constant. In reality, it is a function of the net hydraulic head at the outlet works.
2. The discharge through the spillway is assumed independent of the reservoir volume. In reality, it is a function of the hydraulic head over the spillway sill, the gates, and the spillway hydraulic characteristics.
3. Evapotranspiration and seepage losses are ignored. In reality evaporation loses are a function of the reservoir area and the potential evapotranspiration demand. Seepage losses are a function of the type of dam, the conditions of the dam, and the reservoir level.

All those simplifications should be removed for a real-life rule-curve determination.

For the current example, we used the following data:

Number of Years of data	**57**
Maximum Monthly Flow (HM)	**20000**
Maximum Volume (HM)	**40000**
Annual Irrigation Demand (HM)	**40000**

The Irrigation Demand Degree of Reliability (α) varies from a 0.4 to a maximum of 0.95 in increases of 0.05. The minimum irrigation annual demand then would be 0.4*40000 = 16000, and the maximum would be 38000

The Flood protection factor (β) varies from 0.6 to 1.4 in increments of 0.05. The allowed monthly releases would be from 12000 (least flooding, best protection) to 28000 (worst flooding, worst protection)

Monthly Irrigation Demand as a fraction of the annual demand:

Monthly Distribution of the Annual Demand

Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May
0.0483	0	0	0.0004	0.046	0.0604	0.0748	0.0862	0.1033	0.1226	0.2238	0.2342

Monthly Flows by Water Year:

Monthly Inflows per Water Year (HM)												
Water Yr	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May
1961	11758	16026	25921	15721	15368	1636	549	472	499	474	500	2649
1962	2692	7017	7268	7463	2942	482	290	170	249	412	523	923
1963	3733	4113	10001	14823	20294	2133	621	352	304	90	289	1520
1964	3549	9519	7693	6648	1436	72	97	1	89	10	95	525
1965	982	8785	5931	7702	1489	194	50	186	86	109	82	111
1966	2331	1882	6416	1752	300	67	90	173	16	248	141	204
1967	1435	4243	19698	7843	1237	329	591	462	318	360	720	766
1968	8618	11827	9692	1045	1419	206	186	347	262	132	372	872
1969	3224	4353	12305	5025	262	131	262	197	250	161	340	656
1970	4065	3912	6184	25271	3231	68	24	132	2	487	601	1991
1971	7087	26159	28154	17256	3974	1187	273	351	498	274	967	826
1972	958	4365	14751	10556	1105	398	299	116	160	399	640	864
1973	3522	5276	9172	16234	12904	1362	59	776	384	306	407	518
1974	1023	8855	17944	4356	2051	183	285	673	438	858	180	278
1975	2172	17667	8613	9783	3804	468	464	90	108	58	486	974
1976	1774	4807	11715	21454	1027	307	165	166	332	227	867	1193
1977	8069	20285	13971	7938	3227	1148	547	384	644	327	432	376
1978	5534	4583	16387	18474	9970	1217	370	763	695	54	246	243
1979	2440	8868	4084	1712	1116	130	205	174	37	6	60	114
1980	4459	9608	13358	8468	931	117	333	102	143	422	530	714
1981	1287	7261	6155	1801	88	93	242	36	19	546	248	326
1982	2501	2773	11242	5078	1046	738	423	249	359	153	746	1052
1983	1151	6167	6570	8162	3779	112	168	497	449	0	398	535
1984	16153	13571	18983	13134	651	56	24	298	0	35	43	252
1985	1435	10172	9297	11077	8122	499	267	250	279	156	279	295
1986	11305	19114	8602	4331	7474	1283	964	387	213	149	271	403
1987	652	13698	21739	22884	1673	1128	277	107	49	840	311	383
1988	8365	6902	12565	3748	2074	288	38	196	32	274	281	515
1989	2530	15711	6238	4603	3061	262	576	38	563	213	75	944
1990	2326	19147	13487	10644	12010	831	370	496	320	630	241	734
1991	5453	10241	14340	13952	1316	207	410	492	223	80	381	869
1992	3348	3488	9940	2333	752	631	181	286	227	170	120	94
1993	2174	3637	1516	17855	1543	277	92	68	283	24	13	71
1994	6384	15588	11818	4621	8677	90	38	10	0	30	93	43
1995	964	4442	9417	17174	3905	4439	367	780	256	115	0	95
1996	9192	5745	17658	6888	657	51	15	46	444	483	336	168

Monthly Inflows per Water Year (HM)												
Water Yr	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May
1997	3382	16463	21576	10173	1358	240	211	230	354	563	178	1330
1998	2739	6336	7977	12050	5071	313	9	32	68	215	241	451
1999	6530	12551	17451	13017	5392	459	781	1042	1150	1203	1457	1610
2000	4039	4657	6831	18747	2898	1569	634	1065	1508	1336	888	610
2001	5954	10076	4481	6205	1779	899	1493	1902	1500	1186	734	663
2002	2627	4289	3217	5514	2827	1490	1781	1043	2260	1786	1377	1860
2003	3504	6581	5027	7580	11063	735	859	1136	639	1299	2385	2893
2004	3751	4024	11058	7761	5099	784	2133	1703	1344	1550	1269	1682
2005	2873	4873	7948	4693	1392	1293	1948	2249	1743	1999	1898	3364
2006	7031	8713	10069	17622	3255	1239	1078	725	860	800	378	1822
2007	3391	17847	14601	18530	4527	2984	3311	3151	2934	3224	2535	1812
2008	7429	12114	13047	6163	2092	376	953	1040	771	1003	1184	1622
2009	2613	4020	4587	16890	3328	1156	725	1334	1088	1183	1153	982
2010	2479	2403	4063	3682	2003	1141	1178	1152	1026	904	1113	1105
2011	6809	5060	21746	9793	3212	1129	923	1121	895	693	1789	2407
2012	2296	5490	8275	4069	2549	1978	1064	791	827	851	807	815
2013	5281	3385	8057	2620	11225	1554	1174	1182	1101	1001	848	1166
2014	3093	9051	17037	8706	5007	1077	1089	921	684	801	992	3091
2015	3655	12014	17153	2472	1275	1108	1147	906	982	1248	725	888
2016	1558	5813	20253	16369	9995	1856	1640	2278	1609	1308	542	572
2017	1785	32823	9848	5492	4681	1176	1238	310	201	507	747	1060

Figure 6.5.1 shows the rule curves corresponding to values of α from 0.4 to 0.95. As expected, for low reliability in the irrigation supply (40% of the demand), the reservoir can stay at lower levels. As α is increased, the reservoir needs to be kept at higher levels.

Figure 6.5.2 shows the rule curves corresponding to values of β ranging from 0.65 to 1.35. Notice that the program that generates the rule curves was executed for β ranging from 0.6 to 1.4. The β =0.6 does not appear on the rule curve figure because it is non-feasible. In other words, using the specified hydrology and the example reservoir characteristics, it was impossible for the system to find initial reservoir values at each month such that the outflow would be below or equal to 12000 HM (60% of the nominal flow of 20000 HM). The curves for β greater than 1.35 are identical (full reservoir), so they are not shown in the figure.

Each of those sets of curves works well for single-purpose reservoirs. But for multipurpose reservoirs, such as the current example, there is a need to select a set of two curves that allow the releases from the reservoir to meet pair of values of α and β. For each value of α is necessary to find values of β such that for every time period, the maximum value of the α curves is lower than the values of β for the same month. There could be several values of β that meet that requirement, but the best value is the one for which its rule curve that is closest to the α rule curve.

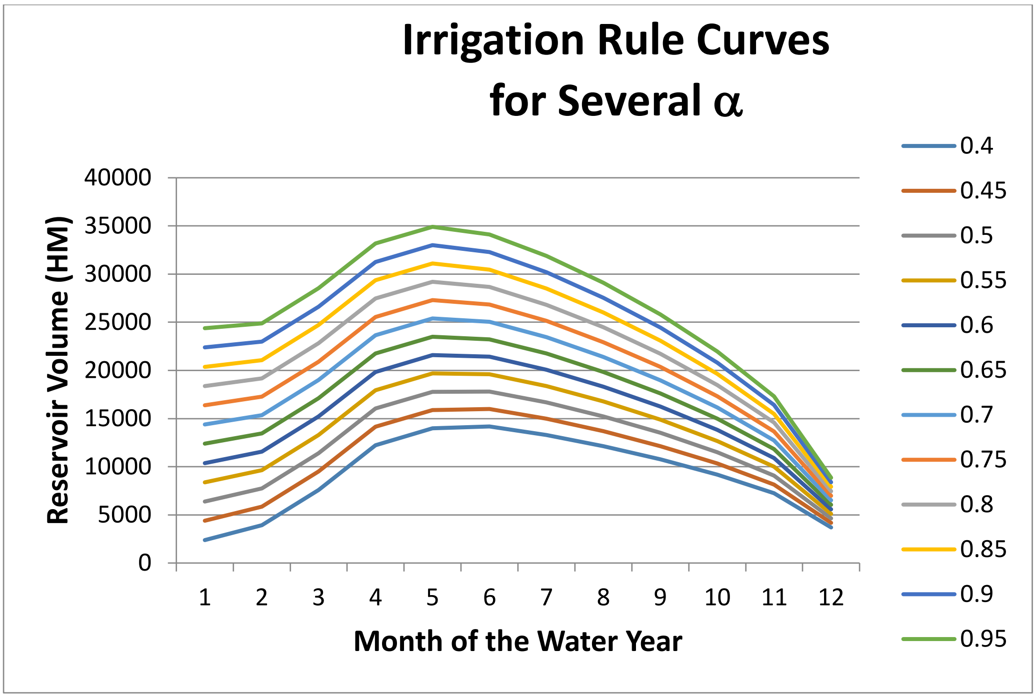

Figure 6.5.1. Irrigation Rule Curves for Several Reliability Values

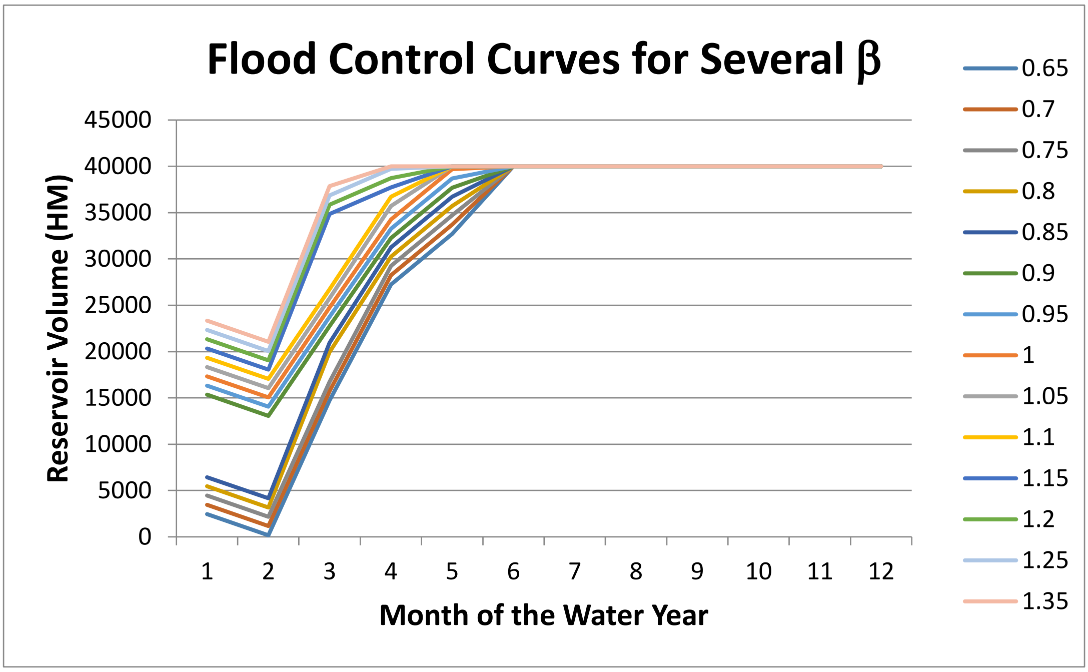

Figure 6.5.2. Rule Curves for Several Levels of Flood Protection

Figure 6.5.3 shows a feasible region (in Green) that will meet both α = 0.5 (poor coverage of irrigation demand) and β = 1.2 (poor protection against downstream flooding). So, although the region is feasible, it is said that is inefficient, since is clear that it should be possible to improve both irrigation demand coverage (increase α) and reduce downstream flooding (decrease β). For each α there is a β such that all reservoir values for the β rule curve are above the corresponding values for α. For each value of α there is a value of β such that the rule curve values for β are the closest to the corresponding values of α. This is called the efficiency curve, or the Pareto curve. Points along that curve are such that it is impossible to improve irrigation (increase α) without improving flood protection (decrease β).

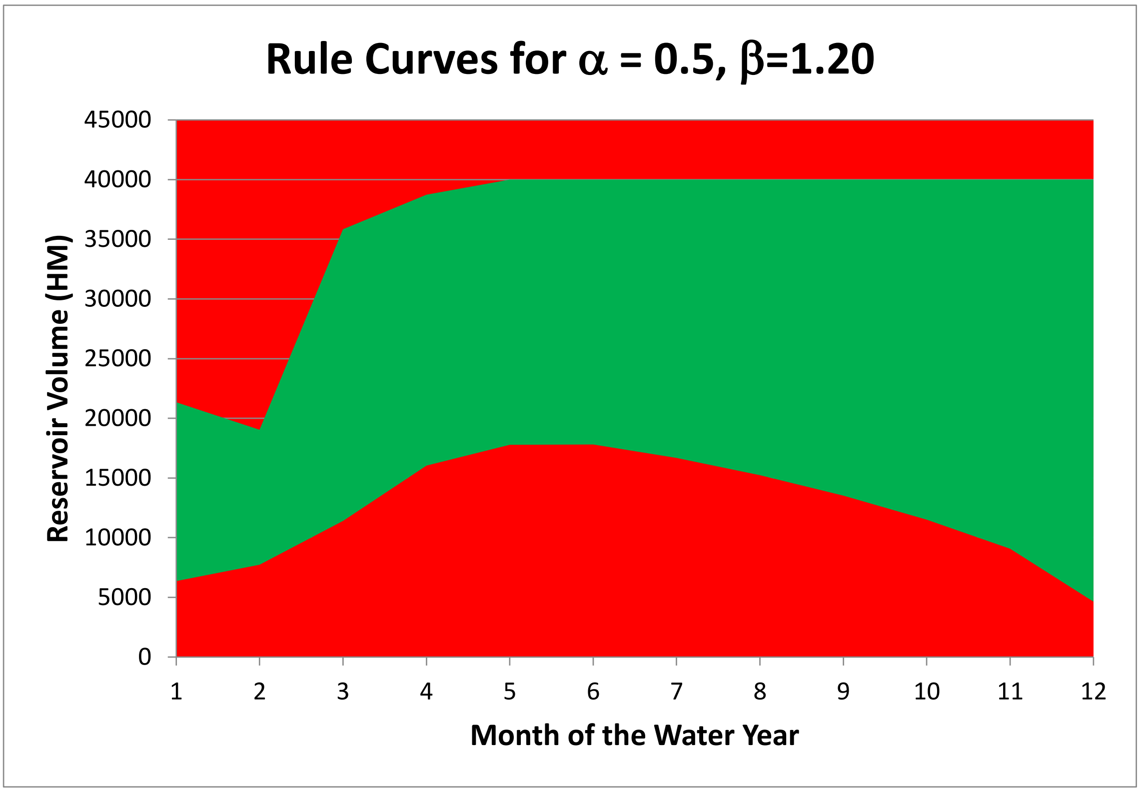

Figure 6.5.3. Feasible Region (Green) for α = 0.5 and β = 1.2

Figure 6.5.4 shows the Pareto curve for the current example. The curve values are shown in Table.

Notice that for an irrigation coverage of α=0.5, the best flood protection corresponds to β = 0.9. Notice how the rule curve for flood protection is lower when compared to the curve for β = 1.2 shown in Figure 6.5.3. Feasible Region (Green) for α = 0.5 and β = 1.2Figure 6.5.3.

A more realistic value for a reservoir whose primary goal is irrigation support, the reliability factor should be α=0.95. From the Pareto curve we see that the corresponding value for flood protection is β = 1.45. The corresponding rule curves are shown in Figure 6.5.6. On the other hand, a reservoir with a primary objective of flood protection should have a lower β. From the Pareto curve we find that for a β = 1.05, the corresponding irrigation reliability would be α=0.70. The resulting rule curves are shown in Figure 6.5.7

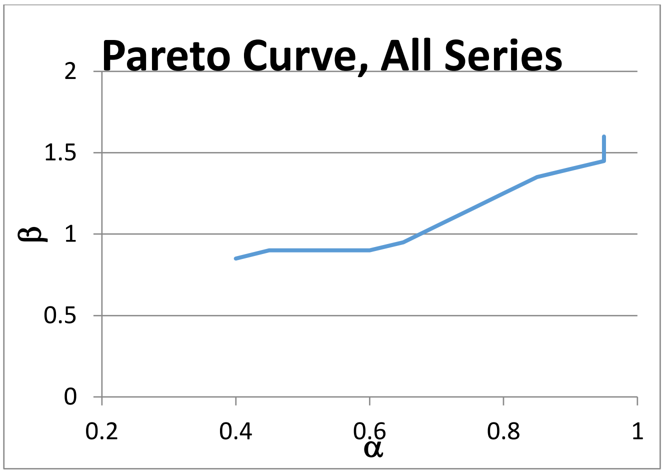

Figure 6.5.4. Pareto Curve

Table 25. Pareto Curve Table

Pareto Curve	
α	β
0.40	0.85
0.45	0.90
0.50	0.90
0.55	0.90
0.60	0.90
0.65	0.95
0.70	1.05
0.75	1.15
0.80	1.25
0.85	1.35
0.90	1.40
0.95	1.45
0.95	1.60

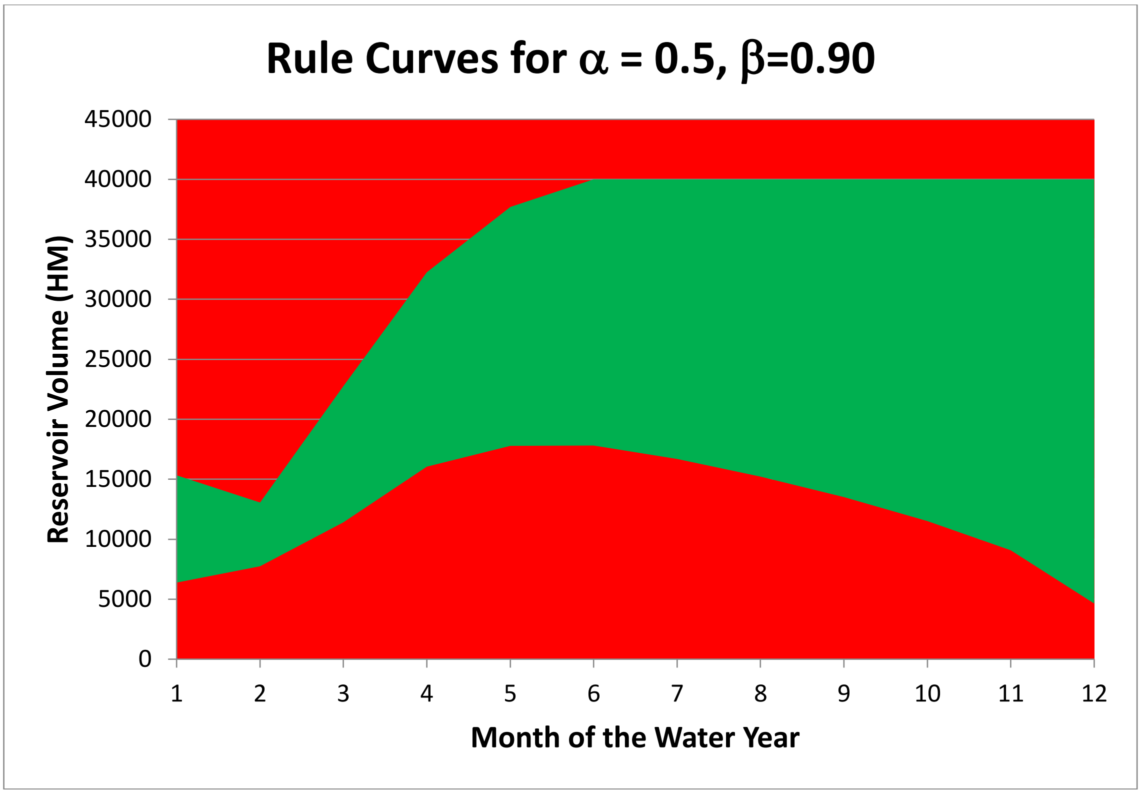

Figure 6.5.5. Rule Curves for α = 0.5, β=0.

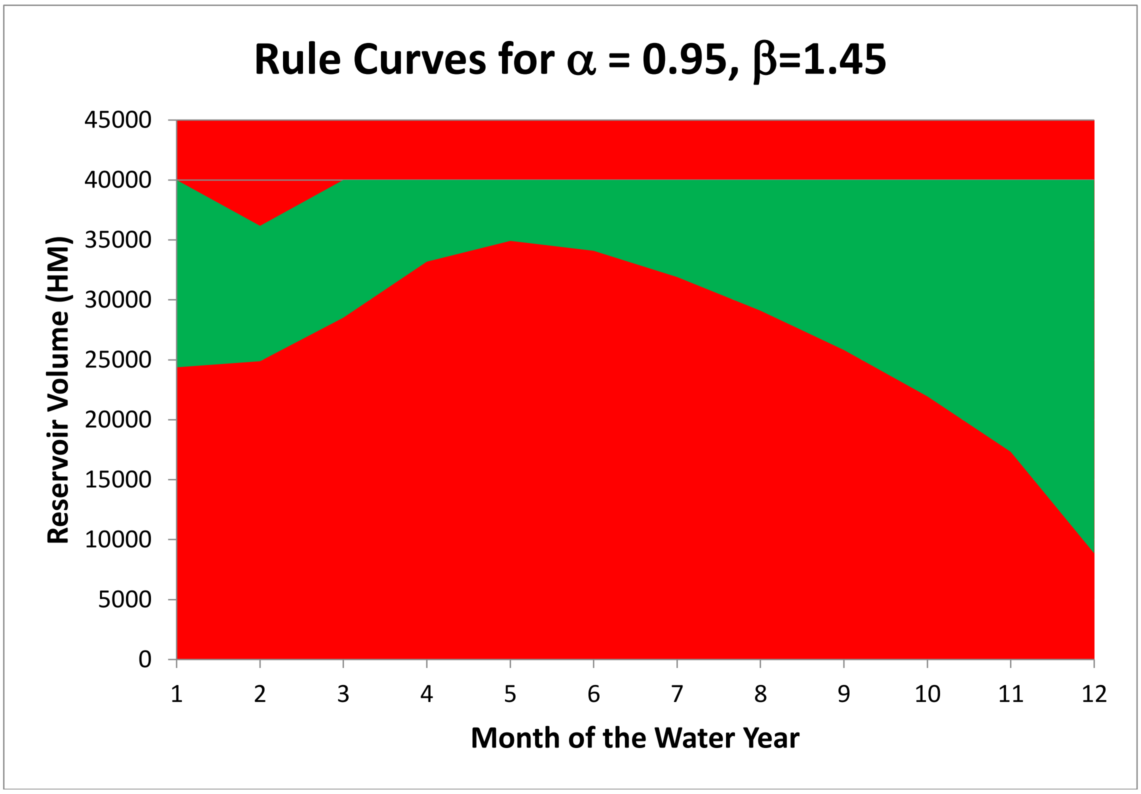

Figure 6.5.6. Rule Curves for α = 0.95 and β = 1.45

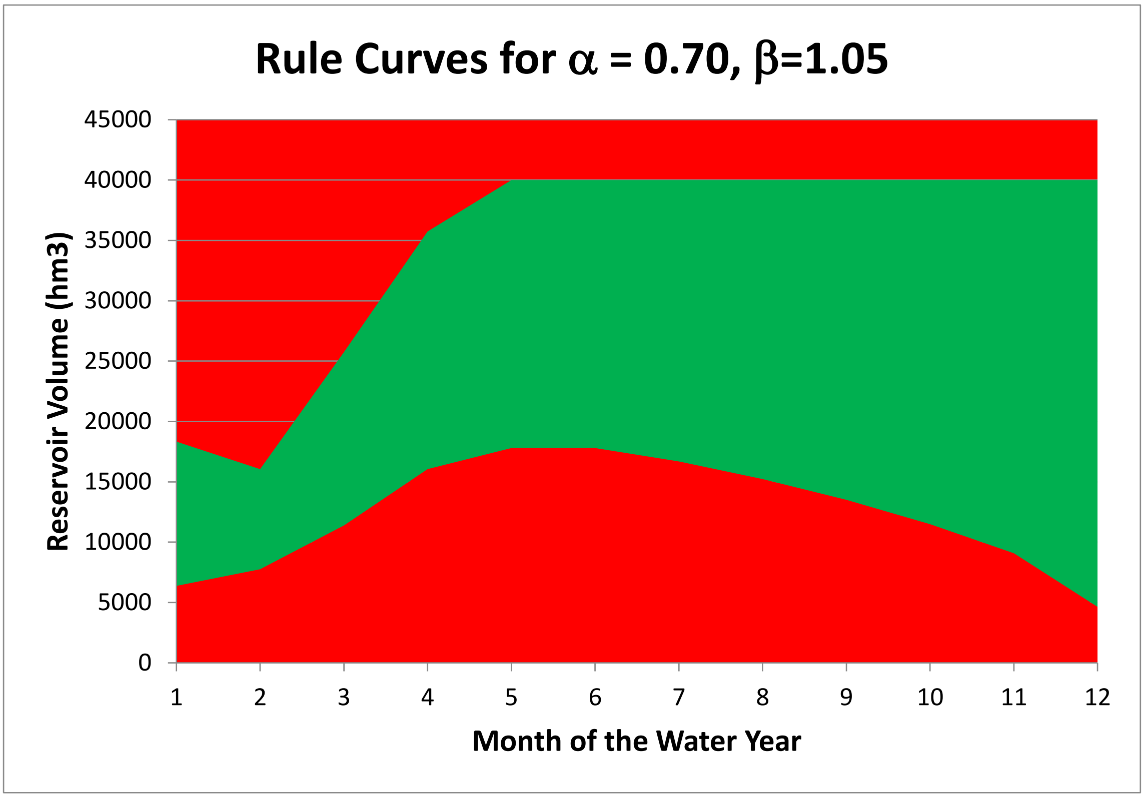

Figure 6.5.7. Rule Curves for $\alpha = 0.70$ and $\beta = 1.05$

As mentioned in Section 5.5, selecting extreme inflow time series of inflows produces, not surprisingly, extreme rule curves for irrigation reliability and flood protection. One such extreme series is the one composed such that the flows on every month correspond to the minimum flows for that particular month among all the historical series. That would correspond to an unprecedented drought. Similarly the wet counterpart would be to form a series in which the flow in each month would be the highest historical flow in that particular month, among all historical series.

Figure 6.5.8 shows the irrigation curves for several reliability values. Comparing Figure 6.5.8 with Figure 6.5.1 it is clear how much higher the reservoir needs to be in the extreme inflows case. Figure 6.5.9 shows the rule curves for several flood protection values. Notice that only four values of b were feasible. That means, that, under the selected conditions (maximum volume and exceptionally wet time series), the computer model could not find rule curves that would provide better flood protection. Comparing Figure 6.5.9 with Figure 6.5.2 it is clear in the extreme cases how much lower the reservoirs need to be during the first months of the water year to provide the same level of protection

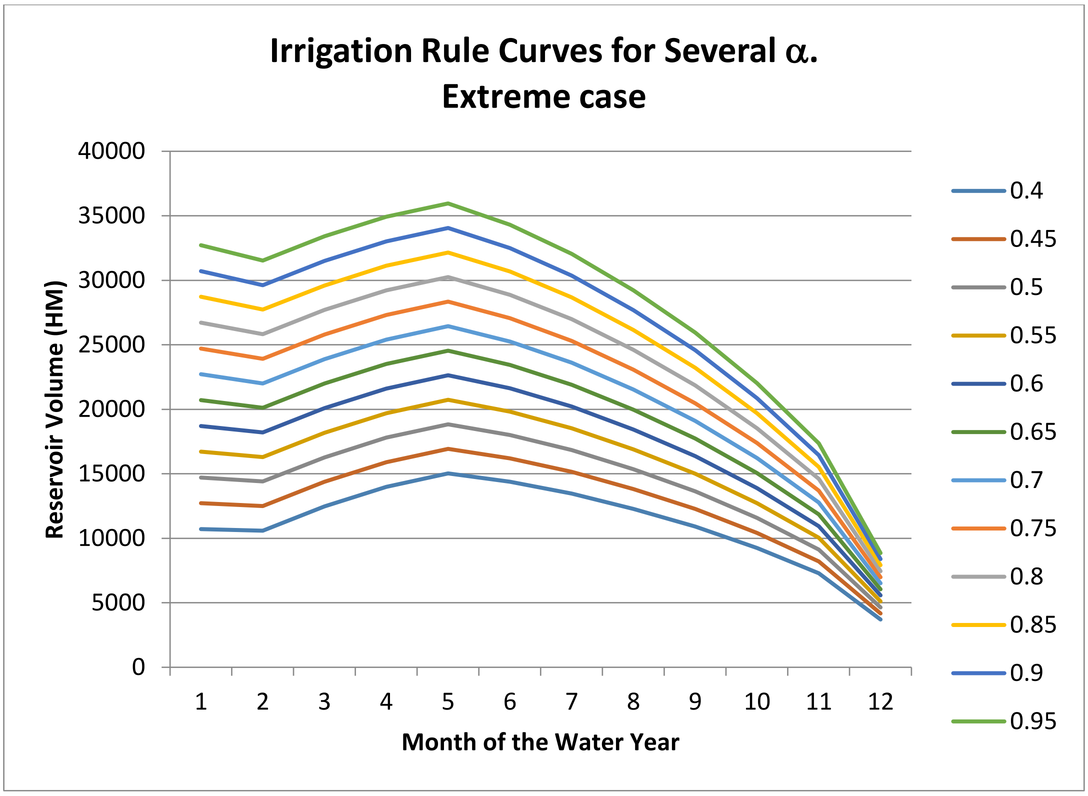

Figure 6.5.8. Rule Curves for Irrigation Reliability for Several α. Extreme case

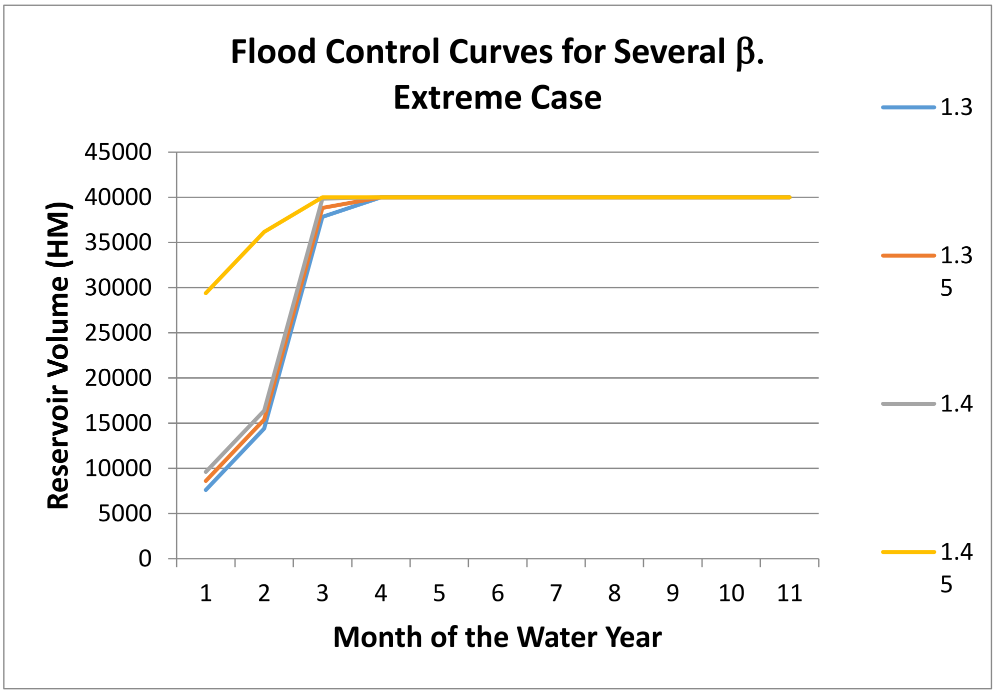

Figure 6.5.9. Rule Curves for Flood Protection for Several β. Extreme Case

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