RISK ASSESSMENT ANALYSIS OF TEHRI DAM

A DISSERTATION

Submitted in partial fulfilment of the

Requirements for the award of the degree of

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i*n*

HYDROLOGY

(With Specialization in Surface Water Hydrology)

By

AMAN KUMAR SINGH

DEPARTMENT OF HYDROLOGY

INDIAN INSTITUTE OF TECHNOLOGY

ROORKEE-247667 (INDIA)

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this report entitled, '**RISK ASSESSMENT ANALYSIS OF TEHRI DAM'**, in partial fulfilment of the requirements for award of the degree of **Master of Technology in Hydrology** submitted to the Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out under the guidance of **Dr N.K. Goel, Professor, Department of Hydrology, IIT Roorkee** during the period from July 2018 to June 2019.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Date: 26 June, 2019

AMAN KUMAR SINGH

(17537002)

CERTIFICATE

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

Dr N. K. Goel,

Professor,

Department of Hydrology,

Indian Institute of Technology Roorkee,

Roorkee – 247 667 (Uttarakhand), India.

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ABSTRACT

In India and worldwide large population lives near and downstream of dams. The dam failure results in large economic losses and loss of human lives living in the downstream areas of dams. In the present study, risk assessment analysis of Tehri dam has been carried out. Tehri dam is located in Uttarakhand, India. All the data required for Risk assessment analysis have been collected from THDC office, Rishikesh (Uttarakhand), India. Digital Elevation Model data is downloaded from USGS earth explorer for analysis. HEC-RAS is used for model development of Tehri dam. Risk assessment analysis is done by performing level pool routing for different gate conditions and initial reservoir level of Tehri dam in HEC-RAS model. Tehri dam has five gated spillways and two ungated spillways. Mechanical and electrical failure of gates is possible and hence its failure probability has been considered for risk analysis. Initial reservoir levels for this analysis are taken as 810m, 815m, 820m, 825m, 826m, 827m, 828m, 829m and 830m. Level pool routing has been performed for different gate conditions i.e. zero gate working, one gate working, two gates working, three gates working, four gates working and all the five gates working. For level pool routing probable maximum flood is taken as the inflow to the reservoir of Tehri dam. Level pool routing is performed for different cases to find out the downstream consequences and overtopping height of water above the dam body. Failure probability is calculated by using Quantitative risk model architecture.

Based on the level pool routing, the situations under which overtopping of dam is possible have been worked out. The results indicate that if all the gates are functional during the incoming flood, the dam is able to safely pass the PMF without overtopping. However, if one or more gates of the dam are not functional then overtopping of the dam is possible.

Flood inundation mapping has been carried out for different scenarios using HEC-RAS 2-D model and the inundated areas have been computed.

Dam breach analysis has also been carried in the dissertation and the downstream inundated areas has been computed.

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LIST OF NOTATIONS AND ABBREVIATIONS

ASTER	The Advanced Spaceborne Thermal Emission And Reflection Radiometer
B_{avg}	Average breach width
CWC	Central water commission
DEM	Digital Elevation Model
DHARMA	Dam Health Rehabilitation And Improvement Project
E	Explicit method
FERC	Federal Energy Regulatory Commission
FRL	Full Reservoir Level
g	Acceleration due to gravity (9.80665 meter per second squared)
GIS	Geographical Information System
H_b	Height of final breach (in meter)
HEC-RAS	Hydrologic Engineering Centre River analysis system
HMS	Hydrologic Modelling System
I	Implicit Method
I(t)	Inflow at any time t
ICOLD	International Commission On Large Dam
IRL	Initial Reservoir Level
K	Storage Time constant
K_0	Constant
L	Length of Series
LBSS	Left bank shaft spillway
m	Number of gates are working properly
MCM	Million Cubic Metre-cube
MOC	Method Of Characteristics

MRL	Maximum Reservoir Level
MWL	Maximum Water Level
P(m)	Probability that m number of gates are working properly
PMF	Probabale maximum flood
PMP	Probabale maximum precipitation
PE_r	Probability Of Exceedance
QRA	Quantitative Risk Assessment
Q(t)	Outflow at any time t
r	Individual reliability of Gates
RBSS	Right bank shaft spillway
S	Storage
S_0	Channel bed slope
S_f	Slope of energy line
SPANCOLD	Spanish National commission on large dams
t_f	Breach formation time (in seconds)
Δt	Routing Interval
t	Total number of gates of spillway
THDCIL	Tehri Hydro Development Corporation of India Limited
USACE	United States Army Corps of Engineers
USBR	United States Bureau Of Reclamation
USA	United States Of America
USGS	United States Geological Survey
v	Velocity of flow
V_w	Reservoir Volume at time of failure (in m^3)
x	Weighing Factor

CHAPTER 1
INTRODUCTION

1.1 GENERAL

Dams serve as necessary frameworks for economic development and flood assurance worldwide. Dam failures in the past have caused high financial and social losses. As indicated by (ICOLD 2017), from around 36, 000 large dams recorded in the World Register of Dams, there have been around 300 revealed mishaps. It makes the general mishap rate of dams to approximately 1%. At present, there are 5264 finished substantial dams, and 437 huge dams are under development in India. Maharashtra, Madhya Pradesh, Gujarat, Chhattisgarh, Karnataka, Rajasthan, Odisha, Telangana, Andhra Pradesh, and Uttar Pradesh have 1845, 906, 666, 258, 231, 211, 204, 182, 153 and 130 number of dams. Till date, 36 dam mishaps have been recorded in India, where the general mishap rate is approximately 0.685%, which is slightly lower than the world average of 1%. Due to high population density in India, a portion of these dam failures have had high consequences downstream. India is a thickly populated nation, and the majority of the dams come under high class of potential peril grouping. If any disaster occurs, then there will be an expansive death toll, and financial outcomes. So today, society requests an expansion in the well-being and dependability dimensions of such frameworks considered as essential. The best way to react emphatically to these dam failure scenarios is to coordinate the dam's plan, development, and activity in such a manner that it guarantees successful alleviations of characteristic and human-centered dangers. In the recent years, several advancements have been made in the field of dam safety. Endeavors completed to execute them efficiently incorporate viewpoints, for example, supportability, flexibility, and public cooperation. A period related investigation demonstrates dam failure has been diminished by a factor of at least four throughout the most recent forty years, principally because of the upgrades in dam structure engineering, investigation systems, and dam safety management.

1.2 ADVANTAGES OF RISK ASSESSMENT ANALYSIS

Risk assessment involves consideration of all factors that are responsible for dam failure beforehand. A sound risk assessment may prove advantageous in a number of ways as listed below:

- Risk Assessment and Management gives a sound, straightforward and deliberate procedure to illuminate basic dam safety leadership.
- Risk Assessment additionally gives benefits in the task and observation of present dams.
- Risk Assessment likewise gives benefits during the arrangement of the plan of new dams.
- Risk appraisal and management gives a better comprehension of the dam's material science and its association with the establishment, and provides suggestions to improve the plan.
- Changes during the plan will be more conservative and effective than future rehabilitation works.
- A prior estimate of any future hazard helps in the regulation of dam in accordance with the areas affected downstream.
- A few instruments are being utilized for this reason, including the most widely recognized one of the appraisals of dangers to downstream regions. These instruments help to comprehend the possible consequences of any failure situation, and accordingly help in mitigation measures.

1.3 DAM FAILURES IN INDIA AND FAILURE MODES

Dam failure cases in India and failure modes are presented in Table 1-1 and 1-2.

Table 1-1 Major Dam Failures in India

Name of Dam/reservoir system,	Location	Type of dam	Construction year	Failure year	Dam failure reason
Kaddam Project Dam	Adilabad, Andhra Pradesh	A composite structure, earth fill, rock fill, and gravity dam	1957	August 1958	Overtopping
Kaila Dam	Kutch, Gujarat	Earth-fill dam	1952-55	1959	Failure of energy dissipating device and then failure of embankments
Kodaganar Dam	Tamilnadu	Earthen Dam	1977	1979	Failure of shutters during floods because of power failure and resulting overtopping
Machhu II dam	Rajkot, Gujarat	Earthen embankment	1972	1979	Overtopping due insufficient spillway capacity
Nanak Sagar dam	Punjab	Earthen Embankment	1962	1967	Piping failure
Panshet Dam	Ambi, Maharashtra	Earthen Embankment	1961(under construction)	1961	Inappropriate arrangement of the outlet facility during an emergency which resulted in the breakdown of the structure over the outlets.
Khadakwasla Dam	Mutha, Maharashtra	Masonry gravity dam	1879	1961	Dam failure due to high release of water from a upstream dam
Tigra dam	Sank, Madhya Pradesh	Masonry gravity dam	1915	1917	Sliding

Table 1-2 Modes of Dam Failure

Failure mode	**Concrete gravity**	**Earthen/ embankment**	**Concrete arch**	**Concrete buttress**	**Concrete multi-arch**
Overtopping	✓	✓	✓	✓	✓
Piping/Seepage	✓	✓	✓	✓	✓
Sliding	✓	✓	☒	✓	☒
Foundation defects	✓	✓	✓	✓	✓
Overturning	✓	☒	✓	☒	☒
Cracking	✓	✓	✓	✓	✓
Equipment failure	✓	✓	✓	✓	✓

1.4 OBJECTIVES OF THE STUDY

The objectives of the present study are listed below:

i. To find out possible types of failure modes in Tehri dam,

ii. To classification of various types of failure modes for Tehri Dam,

iii. To develop risk model architecture for Quantitative risk assessment analysis of Tehri Dam,

iv. To develop graph between previous pool levels Vs Exceedance probability,

v. To find out spillway reliability,

vi. To perform reservoir flood routing by passing PMF from Tehri reservoir for different gate conditions and initial reservoir revels;

vii. To carry out flood routing analysis and prepare flood inundation maps and area covered under submergence for different conditions including dam failure case.

1.5 ORGANIZATION OF THE THESIS

The thesis is presented in seven chapters. Chapter 1 on Introduction gives brief introduction of Risk assessment analysis of dams and its advantages and objectives of the study. The chapter 2 on Review of literature presents the details of risk assessment studies for Bhadra dam, which has been recently carried out by Central Water Commission (CWC). This chapter also gives the details of risk assessment studies of Green and Red dams of USA carried out by U.S. Bureau of Reclamation and presented in Central Water

Commission manual on “Guidelines for Assessing and Managing Risks Associated with Dams.” HEC-RAS application on dam break studies is also explained in this chapter.

Risk assessment methodology and its procedure is presented in Chapter 3 using flowcharts, graphs, and tables. Chapter 4 presents the study area. Chapter 5 presents the results of questionnaire based risk assessment analysis of Tehri Dam and the probability of working and non-working of gates along with combined annual probabilities. The results of reservoir and channel flood routing for different gate and reservoir level conditions and dam break condition are presented and discussed in Chapter 6. This chapter also presents the results of flood inundation mapping using HEC-RAS 2 D software. Chapter 7 presents the conclusions drawn from the study, limitations of the study and future scope of work.

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

The present study is based on the CWC manual on risk assessment analysis and risk assessment reports for Bhadra dam in India and Red and Green dams in US. Based on these reports risk assessment analysis of Tehri dam has been carried out. Risk assessment analysis of Tehri dam has been carried out using HEC-RAS software and the design parameters of Tehri dam have been collected from previous reports pertaining to Tehri dam. The details of risk assessment of Bhadra dam, Red and Green dams and HEC-RAS applications are presented in the following sections.

2.1.1 BHADRA DAM

The Bhadra Dam Project is situated on the River Bhadra, a tributary of the River Tungabhadra in the District of Chikmagalur in the state of Karnataka. The dam was constructed in 1962, and is utilized for irrigation, water supply, and hydropower. The dam has an absolute repository limit of 2026 hm^3. For Bhadra Dam, the assessed populace in danger, in view of the archive Flood-Inundation Maps for Bhadra Dam (July 2017), is more than one lakh occupants (evaluated populace in danger inside the assumed settlement limits is 5,72,572 occupants). Thus, this project comes under the most elevated hazard class meant as 'Catastrophic'. The Bhadra dam spillway is designed for hydrological capacity of 3021 m^3/s. Estimated PMF for Bhadra dam is 7,544 m^3/s after 38 hours from the start of rainfall.

Bhadra, dam leakage has been reported along the non-overflow section of the main dam, and several drainage holes in non-working condition and uneven settlement on the upstream face was found on saddle dam. After the technical site visit and evaluating all the information by the risk analysis team, ten possible failure modes, and its classification were given by the risk assessment team. Whole analysis on failure mode depends on the classification category in which it lies. Subsequently, quantitative and semi-quantitative risk assessment was performed on Bhadra dam. For quantitative risk assessment analysis, the first step was to prepare risk model architecture. Risk model architecture preparation is the central part of Quantitative Risk assessment. In this analysis, the input is essential because the output depends on the input and it changes when we change the input. So for proper risk assessment, a credible analysis of the input is necessary.

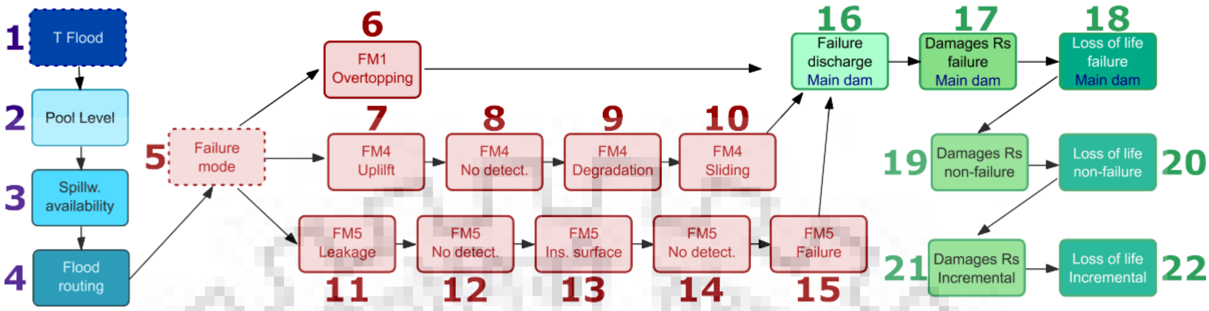

Figure 2-1 Risk model architecture for Bhadra dam

Fig.2-1 shows the risk model architecture involved in the quantitative risk assessment analysis of Bhadra Dam. Node 1 introduces the range of flood hydrographs for a range of return periods from 2.33 to 10,000 years (21 cases). Node 2 introduces the pool level probability (12 cases). Node 3 introduces the probability of gates working correctly (5 cases). Node 6 probability is obtained by fragility curve. Node 7, 8, 9 probability is obtained by expert judgment. Node 10 probability is obtained by Monte Carlo analysis. Node 11,12,13,14 and 15 probability is obtained by expert judgment. At Node 16 failure hydrograph is introduced and economic risk and life loss risks are found out. At Nodes 17, 19 and 21, economic consequences are obtained by depth damage curve given by Huizinga. Node 18, 20 and 22 loss of life is estimated by using Graham (1999) and SUFRI (I.Escuder-Bueno et al. 2012). Taking everything into account, the accompanying table (Table 1) demonstrates a synopsis of the outcome estimation figures regarding financial expense for private structures and horticultural land. Also, the mean water depth (in m) is introduced for every situation.

These outcomes demonstrate that potential expenses for private land use are impressively higher than the potential expenses of agrarian land harm in the event of flooding because of the failure of Bhadra Dam. Likewise, there is an increased probability of increased costs associated with potential harms with increase in reservoir level. For Case C (supply level 1 m above dam peak level) the normal financial misfortunes are 10% higher than for Case A (store level at MOL).

Table 2-1 Figures regarding financial expense for private structures and horticulture land

	Case A(MOL)	Case B(Crest)	Case C (Crest+1m)
Agricultural Estimated cost (Rs Crores)	55.38	59.70	61.90
Residential Estimated cost (Rs Crores)	4624.18	4851.78	5062.55
Total Flood Estimated cost (Rs Crores)	4679.56	4911.48	5124.46
Reconstruction estimated cost(Rs Crores)	2456.33	2456.33	2456.33
No failure estimated cost (Rs. Crores)	4679.56	4911.48	5124.46
Failure estimated cost (Rs. Crores)	7135.89	7367.81	7580.78

After fruition of information for hazard estimation, and once joined in the hazard model architecture, societal and monetary dangers were obtained. Incremental hazard is obtained as the part of hazard solely because of dam failure. It is acquired by subtracting from the results because of dam failure the ones that would have happened even if there should arise an occurrence of non-failure. Results for the Base Case for Bhadra Dam appear in the table underneath.

Table 2-2 Risk and Probability associated with different failure modes of Bhadra Dam

Failure Mode	Failure probability (1/year)	Societal risk (lives/year)	Economic risk (Rs Crores/year)
Failure mode 1: Overtopping	5.700E-04	4.470E-01	4.217E+00
Failure mode 4: Sliding dam foundation	6.031E-05	4.457E-02	4.321E-.01
Failure mode 5: Sliding dam body	9.519E-08	6.325E-05	6.338E-04
Total	**6.304E-04**	**4.916E-01**	**4.650E+00**

Results demonstrate that the dominating failure mode is overtopping, unmistakably higher than 10^{-4}. This outcome mirrors the significance of current vulnerability about precipitation information considered for hydrological investigation.

2.1.2 GREEN AND RED DAM

Green and Red dams are situated on Blue River in the state of Colorado, U.S.A. Green dam is an earthen embankment dam, and red dam is a Gravity Dam. In this Risk Assessment, it has been decided to dissect the two dams in the framework (Green Dam and Red Dam) inside a similar hazard model, since they are worked together and the two suppliers have comparative extent, so the failure of the upstream dam (Green Dam) could deliver (or not) the failure of the downstream dam (Red Dam).

The development of Green Dam was completed in 1933 and the development of Red Dam was completed in 1989. In 1992, this dam was restored because of the noteworthy spillage through the dam's body because of the low quality of its soil. The Red Dam is a homogenous dike with the straight plant. The Red Dam has an ungated spillway that is joined to the body of the dam, on its accurate projection. It additionally has a base channel and a water intake. There have not been real restorations in the Red Dam since it was built in 1989. At last, as indicated by the Potential Hazard Classification made in January 2014 made by CCCC, the Green Dam and the Red Dam were named Catastrophic, because of the high populace found downstream in the floodplain territory. This populace is higher than 1, 60,000 individuals.

For this dam, possible failure modes and its classification were made after the site visit by the technical team in accordance with the possibility of occurrence. In this red dam had six possible failure modes after its six possible failure mode classified in failure mode classification. The green dam had four possible failure modes after its four possible failure mode classified in failure mode classification. Thereafter, quantitative and semi-quantitative risk assessment analysis was performed.

Then for quantitative risk assessment analysis, first step was to prepare risk model architecture. Risk model architecture was prepared based on the possible failure mode. Risk model architecture for Red and Green dams are shown below.

Figure 2-2 Risk model Architecture for Red and Green Dam

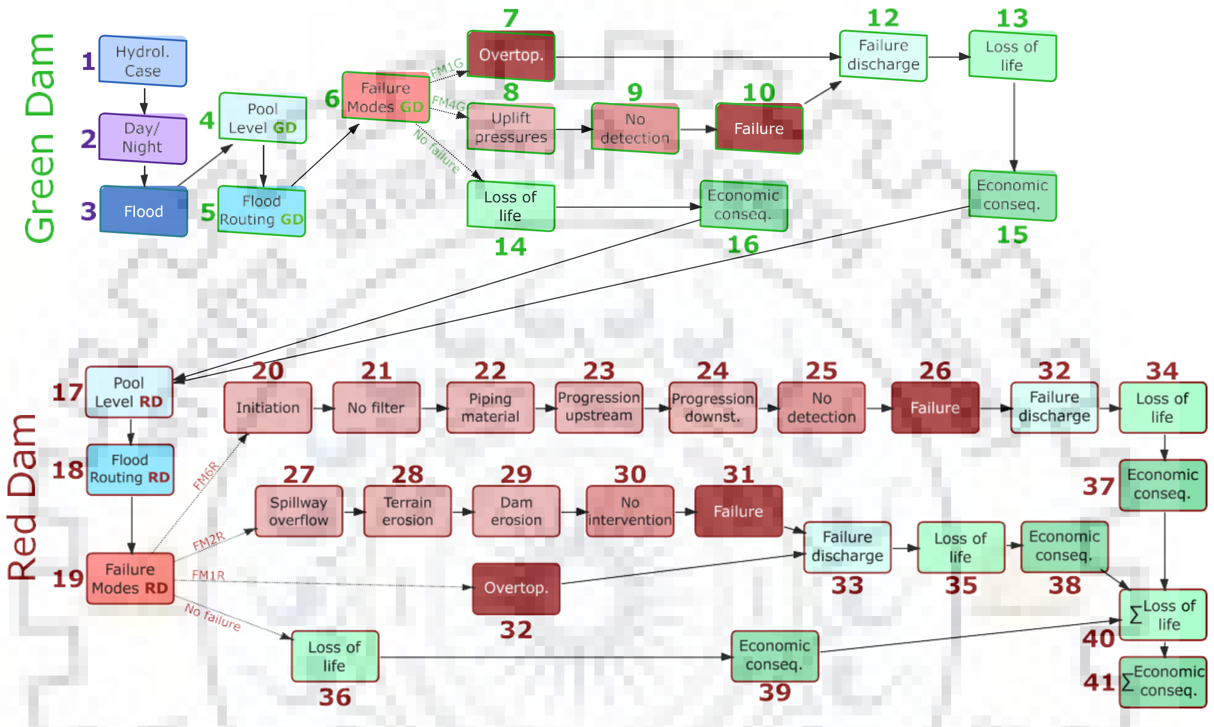

Table 2-3 Risk and Probability associated with different failure modes of Red and Green Dams

Failure Mode	Failure Probability (1/year)	Societal Risk (lives/year)	Economic Risk (Rs. Crores/Year)
Green Dam			
FM1G:Overtopping	1.108E-03	1.810E-04	4.003E-01
FM4G:Sliding	1.106E-04	1.498E-05	3.926E-02
Total	**1.219E-03**	**1.960E-04**	**4.396E-01**

Red dam			
FM1R:Overtopping	3.960E-07	2.199E-05	4.003E-03
FM2R:Insufficient spillway capacity	1.598E-07	8.889E-06	1.620E-03
FM6R: Internal erosion	5.026E-07	5.816E-05	3.738E-03
Total	**1.058E-06**	**8.904E-05**	**9.360E-03**

2.2 HEC-RAS APPLICATION IN DAM BREAK STUDIES

The development of HEC-RAS (Hydrologic Engineering Center's (HEC), River Analysis System) hydraulic model requires an accurate portrayal of the territory information and the hydrologic sources of information utilized as boundary conditions. Also, suitable model parameters for landscape terrain roughness and hydraulic structures must be evaluated and afterwards calibrated to believe in the model outcomes. The rules in this archive are centered on the improvement and utilization of unsteady flow models for dam-break studies. In general, a full unsteady stream routing will be progressively exact for both, with and without breach situations.

The estimation of a dam break location, dimension, and advancement time are critical in any appraisal of a dam's potential risk. This is particularly valid in risk evaluation where dams will be positioned based on the potential for loss of life and property. The break parameters will straightforwardly influence the peak flow leaving the dam, just as any conceivable warning time accessible to downstream areas. Sadly, the break location, size, and development time are regularly the most uncertain pieces of data in a dam disappointment examination.

The accompanying regression equations, Table 2-4, have been utilized for a few dam safety studies found in the literature. Dam breach weir and piping coefficients for a different type of dams are given in Table 2-5.

Table 2-4 Breach parameter regression equations used by different researchers

Reference	Number of case studies	Relation Proposed (S.I. units, meters, m^3/s, hours)
Johnson and Illes (1976)		$.5h_d \le B \le 3h_d$ for earth fill dams
Singh and Snorrason	20	$2h_d \le B \le h_d$ 0.15 meters $\le d_{overtop} \le$ 0.61meters 0.25 hours $\le t_f \le$ 1hours
MacDonald and Langridge-Monopolis (1984)	42	Earth fill dams: $V_{cr} = 0.0261\ (V_{out}*h_w)^{0.769}$ (best-fit) $t_f = 0.0179\ (V_{cr})^{0.564}$ (upper envelope) Non-earth fill dams: $V_{cr} = 0.00348\ (V_{out}*h_w)^{0.852}$ (best-fit)
FERC (1987)		B is normally 2-4 times h_d B can range from 1-5 times h_d Z= 0.25 to 1.0 (engineered, compacted dams) Z= 1 to 2 (Non-engineered, slag or refuse dams) t_f =0.1-1 hours (engineered compacted earth dams) t_f=0.1-0.5 hours (non-engineered, poorly compacted)
Froehlich (1987)	43	$B = 0.47K_o(S^*)^{0.25}$ K_0=1.4 overtopping; 1.0 otherwise $Z = 0.75\ K_c(h_w^*)^{1.57}(W^*)^{0.73}$ Kc=0.6 with corewall, 1.0 without a corewall $t_f^* = 79(S^*)^{0.47}$
Reclamation (1988)		$B = (3)h_w$ $t_f = (0.011)B$
Singh and Scarlatos (1988)	52	Breach geometry and time of failure tendencies B_{top}/B_{bottom} averages 1.29
Von Thun and Gillette (1990)	57	B, Z, t_f guidance
Dowey and Gillette (1993)	57	Breach initiation model ; B, Z, t_f guidance
Froehlich (1995b)	63	$B = 0.1803K_OV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.55}h_b^{(-0.90)}$ K_0 =1.4 for overtopping; 1.0 otherwise

Table 2-5 Dam breach weir and piping coefficients for a different type of dams

Dam Type	**Overflow/Weir Coefficients**	**Piping/ Pressure Flow Coefficients**
Earthen Clay or Clay Core	2.6-3.3	0.5-0.6
Earthen Sand and gravel	2.6-3.0	0.5-0.6
Concrete Arch	3.1-3.3	0.5-0.6
Concrete Gravity	2.6-3.0	0.5-0.6

The equations given in Table 2-4 and 2-5 are utilized for the selection of parameters in HEC-RAS application for Tehri dam for risk assessment. The details of the study area and the results of the risk assessment analysis are given in the subsequent chapters.

CHAPTER 3

RISK ASSESSMENT METHODOLOGY AND PROCEDURE

3.1 GENERAL

Risk assessment study involves the following three steps:

a) Initial risk-based screening
b) Identification of failure mode
c) Quantitative risk assessment

The details various steps of risk assessment study are given in subsequent sections:

3.2 INITIAL RISK-BASED SCREENING

Initial risk-based screening tells about the order to be followed in Risk assessment analysis. It starts with a collection of all the information about the dam. Central Water Commission (CWC) has prepared standardized data book format, sample checklist and Performa for periodical inspection of dams. In addition to this, CWC uses the web-based tool “DHARMA (Dam Health Rehabilitation & Monitoring Programme)” for collection of data of large dams. DHARMA planned and created to upgrade the limit of people and associations all through India to deal with their dam resources deductively and expertly in order to support points of interest of dams (water system and water supply, flood control, hydro-control). There are three key factors for initial risk-based screening as follows:

i. The population at risk
ii. Main Dam and reservoir features and emergency preparedness
iii. Hydrological and Seismic adequacy.

The population at risk can be estimated by dam break analysis, which is used for Hazard classification. Hazard classification is done according to CWC guidelines that are shown in Table 3-1.

Table 3-1 Potential hazard classification for dams

Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
Low	None expected (non-permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (no urban developments and no more than a small number of inhabitable structures)	Appreciable (notable agriculture, industry or structures)
High	More than few	Excessive (extensive community, industry or agriculture)

3.3 IDENTIFICATION OF FAILURE MODE

A failure mode is a particular order of events that can lead to dam failure. This succession of events must be connected to a loading scenario and will have a rational arrangement beginning with an initiating event, at least one of the dynamic failure and will finish with dam failure or mission interruption of the dam-reservoir framework as shown in the figure 3-1.

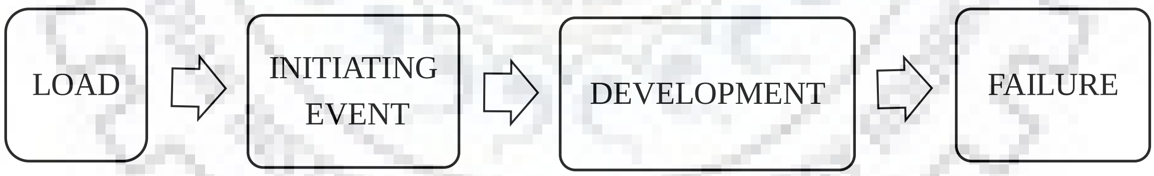

Figure 3-1 Sequence of propagation of failure mode

3.3.1 THE PROCESS OF IDENTIFICATION OF FAILURE MODES

The prescribed procedure for identification of failure modes is outlined in Figure 3-2. This procedure is commonly made through a collective work of a few specialists and professionals, including a far-reaching re-perspective on accessible data, a technical visit to the dam and gathering extensive assessment about the present condition of the dam.

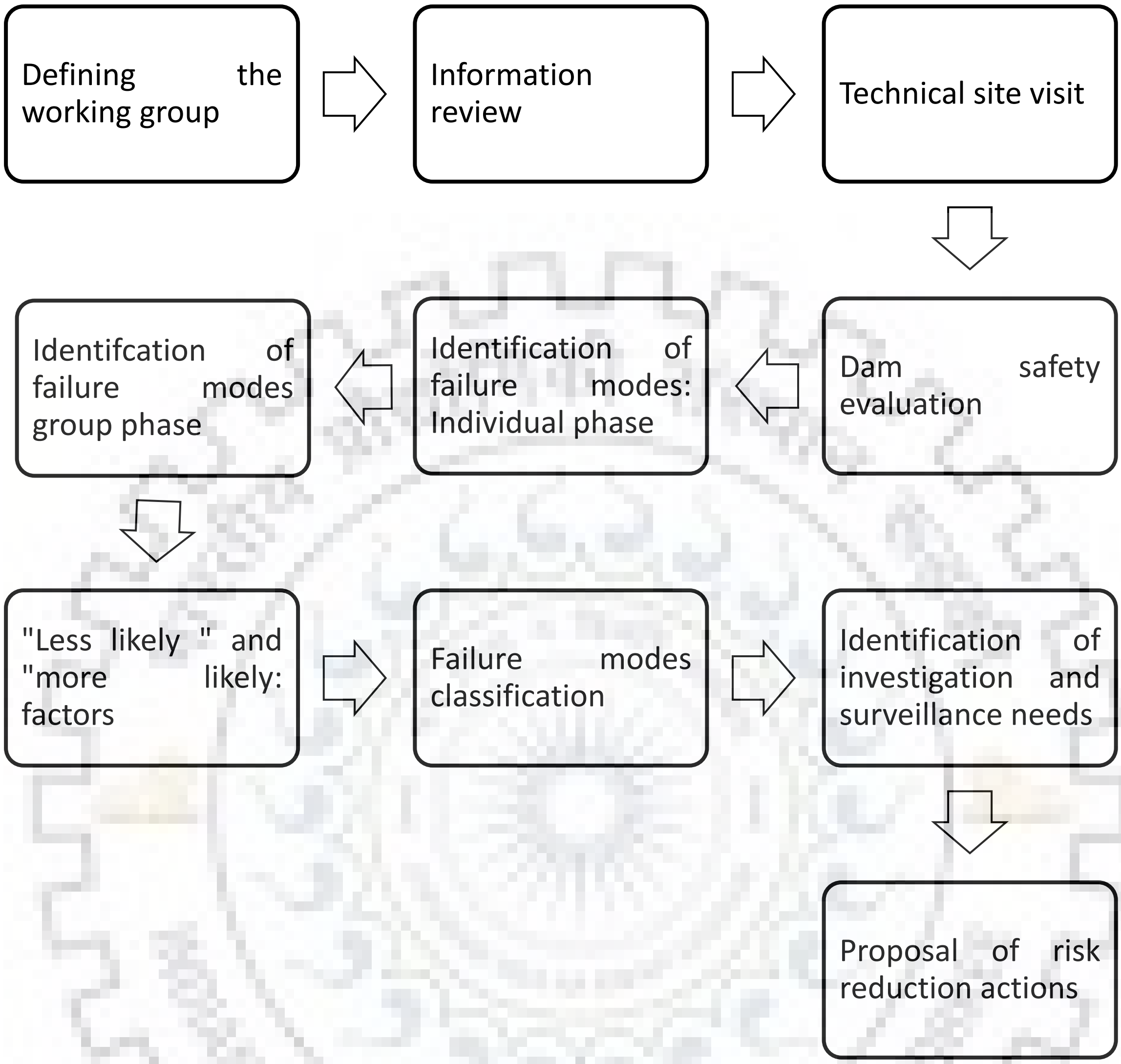

Figure 3-2 General Procedure for Identification of Failure modes

3.3.1.1 FAILURE MODE CLASSIFICATION

Failure modes are characterized by the Failure Mode Credibility. Based on their severity, failure modes are classified into four categories, Class A, Class B, Class C, and Class D. A brief description about each failure mode classification is given in subsequent sections.

Class A:

Failure is in progress or imminent. Exceptionally urgent rehabilitation measures and/or emergency actions are needed the need for urgent rehabilitations can also be identified during technical inspections. Failure Modes should only be classified as A in very exceptional cases when failure seems imminent in the short term. These actions should be carried out as soon as possible, without waiting for risk assessment results.

Class B:

Failure mode is credible and available information is enough for a Quantitative Risk Assessment. All the Class B failure modes are introduced within a quantitative risk model to compute risk in the dam. This risk is evaluated and if needed, potential risk reductions are proposed and prioritized.

Class C:

These potential failure modes, have to some degree lacked information to allow a confident judgment of significance. Hence, available information is not enough for a Quantitative Risk Assessment. In these cases, a Semi-Quantitative Risk Analysis is used to prioritize the studies and instrumentation needed to reduce the uncertainty on these failure modes.

Class D:

Failure mode is not credible or its consequences are very low. These potential failure modes can be ruled out because the physical possibility does not exist, or existing information shows that the potential failure mode is clearly extremely remote. They should be documented and reviewed in the following updates of the Risk Assessment process.

3.4 QUANTITATIVE RISK ASSESSMENT

Complete quantitative risk assessment tries to count the dangers in terms of probability and consequences in quantitative terms. This quantitative assessment is prescribed for Class B failure modes, which are failure modes that are viewed as credible and with enough available data for this sort of examination. Quantitative risk assessment involves the following steps:

i. Estimation of pool level probability
ii. Performance evaluation of gates
iii. Flood routing analysis
iv. Failure probability estimation
v. Loss of life estimation
vi. Estimation of economic consequences

The details are given in subsequent sections.

3.4.1 ESTIMATION OF POOL LEVEL PROBABILITY

The investigation of past pool levels is carried out to examine the probability of finding a specific pool level in the reservoir at the entry of a flood or for the occurrence of any seismic activity. The connection between probability and pool levels can be acquired by utilizing the register of chronicled pool levels. It is important to check with a register that is long enough and adequately illustrative of the present circumstance. Along these lines, the likelihood of exceedance of each pool level is given by the accompanying equation, given by Spanish Commission on large dams, 2012:

$$PE_r = 1 - \frac{i_r - 1}{L - 1}$$

Where PE_r is the probability of exceedance for a pool level is, i_r is the number of the order of pool level r within the series of ordered pool levels, and L is the length of the series.

3.4.2 PERFORMANCE EVALUATION OF GATES

Outlet works and spillway reliability is of great importance to dam safety and has assumed a basic job in numerous disastrous failures. Despite its slow significance, the gates reliability assessment has always remained difficult when it comes to its incorporation into conventional hydrologic adequacy analysis. Consequently, it has been typically treated independently. With regards to Risk Analysis, this viewpoint gets coordinated into the risk model and its effect on safety is taken into account.

At the point when execution is assessed, the investigation of the causes that must lead to gates failure cannot be constrained to a mechanical failure, as experience demonstrates failure can be because of unique reasons. At the point when the entire framework is examined, it is found that several obvious causes may instigate failure:

- Human failure (either on account of the requirement for opening an entryway is not recognized or because the request is not transmitted or because the individual responsible for working a door commits an error, and so forth.).
- Absence of access to the manoeuver chamber (e.g., because of snow).
- Mechanical failure (breakage of a piece, blockage, and so on.).
- The mechanical malfunctioning of the common works (that could render the outlet works or spillway pointless).
- Electrical failure (either in the supply or in the parts of the outlet works or spillway themselves).
- Blockage of the outlet works or spillway (e.g., because of the nearness of logs and debris).
- Fault in the product controlling the gate or the valve (on the off chance that it exists).

In the fundamental examination, singular reliability of gates can be evaluated straightforwardly for each gate in the wake of dissecting these perspectives. (Altarejos Garcia et al. 2014) gives direction to this individual reliability estimation:

- 95%: When the outlet is new or has been very much kept up.
- 85%: When the outlet is very much kept up, however, had some minor issues.
- 75%: When the outlet has a few issues.
- 50%: When the outlet is inconsistent for the flood routing.
- 0%: When the outlet is not dependable at all, or it has never been utilized.

In this sort of essential investigation, entryways can be viewed as independent. Along these lines, the probability of every accessibility gates case can be assessed with a binomial distribution and the accompanying equation:

$$P(m) = \frac{t!}{m!\,(t-m)!} * r^{m} * (1-r)^{t-m}$$

Where, P (m) is the probability that m number of gates are working properly, r is individual reliability of gates, and t is total number of gates.

3.4.3 FLOOD ROUTING ANALYSIS

While confronting a hydrologic situation, it is important to complete a flood routing investigation with the target of assessing the reaction of the dam-reservoir framework when stood up to by hydrologic loads. This is carried out through the estimation of a function that relates the emptied releases downstream of the dam and the pool levels came to in the reservoir along time.

The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing. In these applications two broad categories of routing can be recognized. These are

a) Reservoir routing
b) Channel routing

The details of flood routing analysis are available in a number of text books like Subramanya (2013) and Bras (1990). The details of the flood routing methods, provided in subsequent sections have been taken from Subramanya (2013). These are being provided here for completeness sake in Annexure 1. The concepts given herein helped in the selection of the methods and the parameters in HEC-RAS application to Tehri dam.

Software:

A large number of software, both in commercial and in common domain, are available for flood flow analysis in natural channels. Among these, the HEC-RAS of U.S. Army Corps of Engineers, FLDWAV of US National Weather Service, and MIKE-II of DHI, Denmark and FLO-2D of USA are very popular.

The fundamental information required to complete a flood routing estimation are:

- Hydrograph entering the repository (inflow hydrograph).
- Past pool level.
- Discharge curves of the outlet works and spillways.
- Gates activity rules.

In the hazard models, flood routing outcomes ought to be presented for every mix of:

- Flood, including every one of the floods and their return period acquired from the probabilistic hydrological investigation.

- Past pool level, including all the pool levels in which the exceedance curve has been discretized
- Gates accessibility case.

For each case, the outflow from the store and water pool levels are acquired. Commonly, results presented in the hazard model from these flood directing calculations are:

- Maximum water level came to in the reservoir.
- Maximum overtopping height.
- Peak release outflow discharge by the dam.
- Time of overtopping.

In this thesis work reservoir routing is performed by using modified Pul's method in HEC-RAS model. Channel routing is performed by using Hydraulic method of flood routing and HEC-RAS software has been used. In HEC-RAS there are two options available for performing channel routing first one is by using full diffusive wave computation method and other one is by using 2D Saint Venant Full Momentum Computational Method. In this thesis work full diffusive wave computation method has been used for performing channel routing, i.e. by neglecting 2^{nd} and 3^{rd} terms in the LHS of equation 3.10.

In this thesis work, 2-D flow area has been used for performing channel routing and generation of flood inundation map. The details of 2-D flow area are given in technical manual of HEC-RAS. The same are also given in Annexure-II of this report for completeness sake. In the present work, 2D Diffusive Wave computational method has been used.

3.4.4 FAILURE PROBABILITY ESTIMATION

When failure modes have been separated in well-defined failure systems, the likelihood of every case is assessed. The investigation of failure probabilities is one of the key elements of the hazard model, and it is legitimately founded on the identification of failure modes' results. The principal devices to assess these probabilities are reliability tools, expert judgment, and specific procedures to appraise failure probabilities for certain sorts of failure modes like internal disintegration or overtopping.

3.4.4.1 RELIABILITY TECHNIQUES

Any likelihood of a failure mode that can be demonstrated by a deterministic numerical model is a potential possibility to be numerically evaluated all through reliability techniques. Reliability procedures comprise of spreading the vulnerabilities of the input of a model until achieving an outcome so that a likelihood is acquired rather than a deterministic value (like a factor of safety). Along these lines, a fragility curve is acquired that gives disappointment likelihood to each loading case, has appeared in Figure.

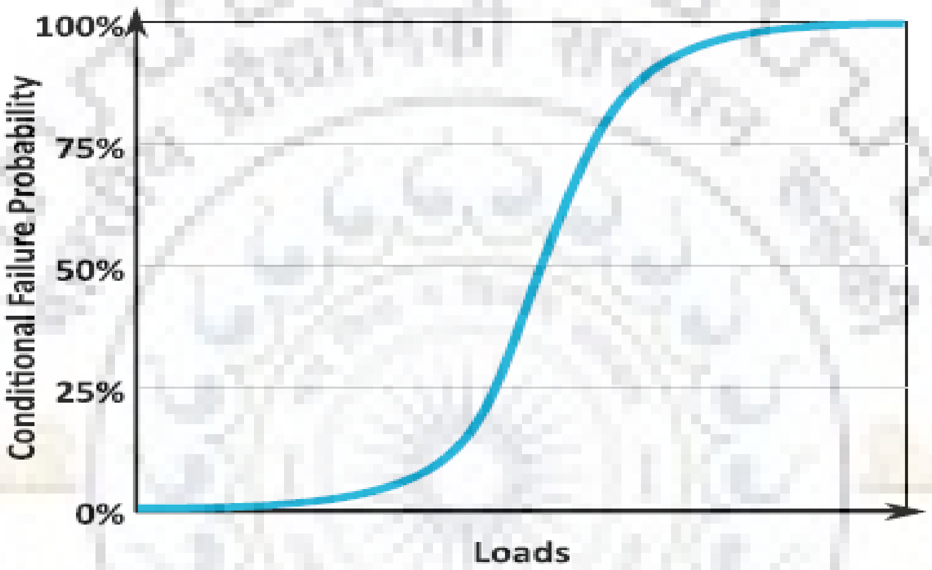

Figure 3-3 Fragility curve for reliability Estimation

3.4.4.2 EXPERT JUDGEMENT

The estimation of probabilities by experts comprises of record the supposition a subject has about the credibility of an occasion. To provide further robustness to this estimation, the average of the estimations of several people is continuously suggested.

The tables of verbal descriptors are useful to control the procedure of articulation of probabilities.

Table 3-2 Table showing probability based on verbal description

Expression	Single-number probability, % (median of responses)	Specified range, % (median upper and lower bounds)
Almost impossible	2	0-5
Very improbable	5	1-15
Very unlikely	10	2-15
Unlikely	10	5-15
Low chance	15	5-20
Medium chance	15	10-25
possible	20	10-20
Very low chance	40	40-70
Improbable	50	40-60
Probable	50	45-55
Likely	70	60-75
Very probable	70	65-85
Even chance	80	70-87.5
Very possible	80	75-92
High Chance	80	80-92
Very likely	85	75-90
Very high chance	90	85-99
Almost certain	90	90-99.5

3.4.4.3 RECOMMENDATIONS FOR SPECIFIC FAILURE MODES

Overtopping

Fragility curves for overtopping failures are prescribed by the dam typology. These curves appear in Figure 3-4.

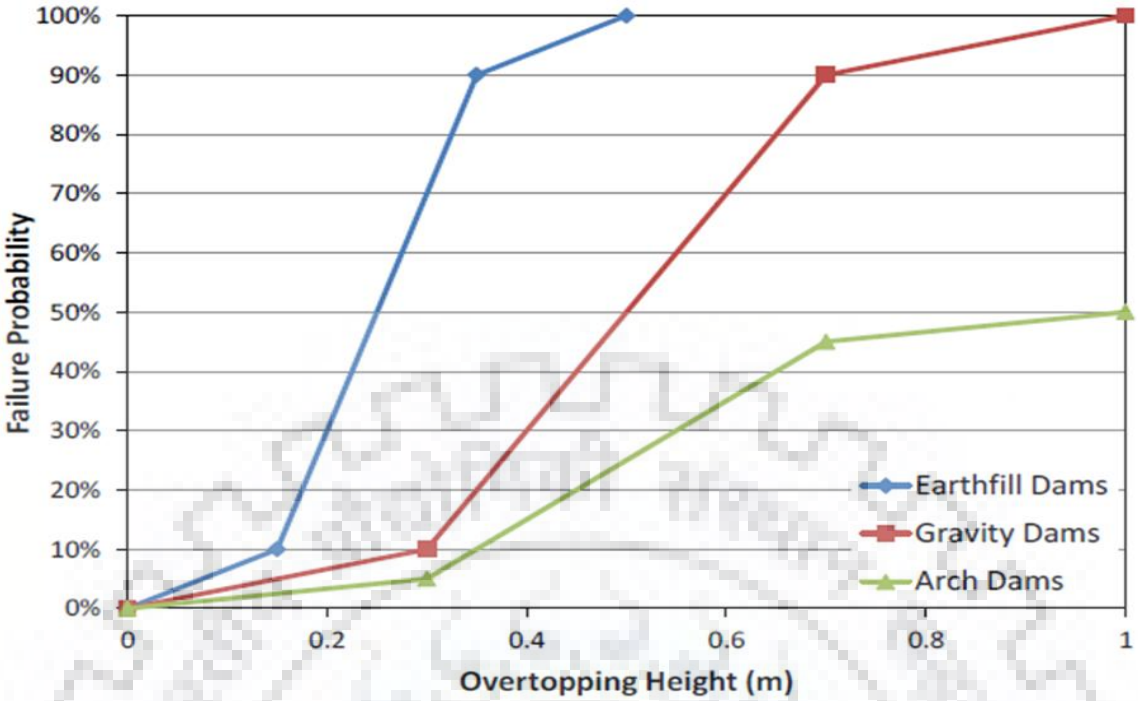

Figure 3-4 Fragility curve for estimation of overtopping failure probability

3.4.5 LOSS OF LIFE ESTIMATION

At the point when a flood because of a dam failure produces death toll, this is plainly the most genuine result and the one that causes the most significant effect on the open view of the catastrophe. Like this, this result has been the object of most outcomes studies, and along these lines, the one which more calculation strategies are accessible.

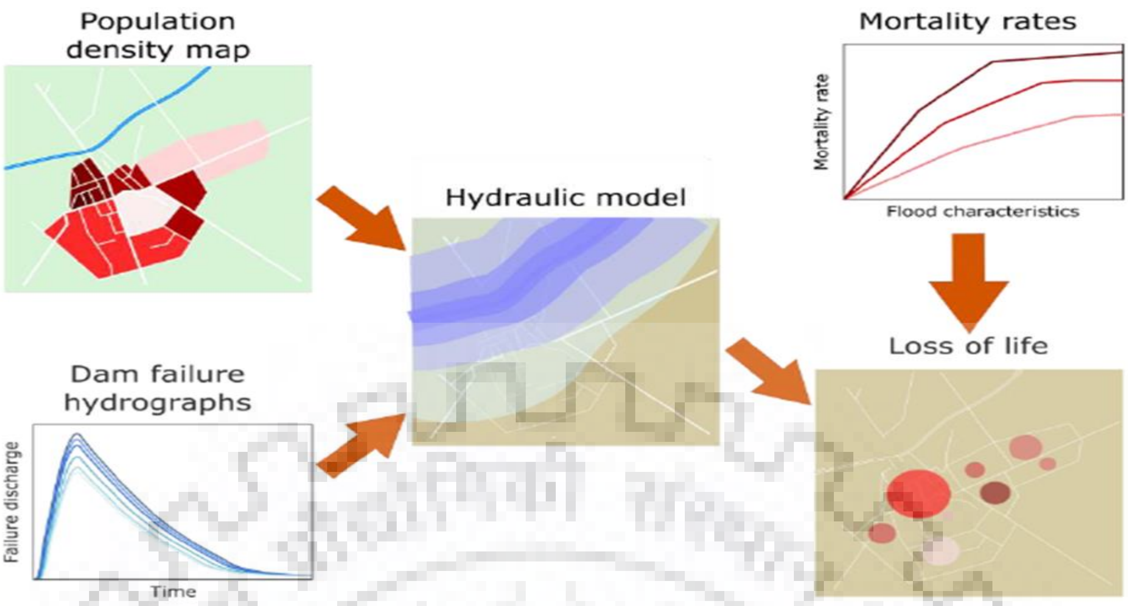

Figure 3-5 General procedure for estimation of loss of life

The most widely recognized technique to gauge the death toll is the one proposed by Graham (1999). Graham's strategy proposes fixed casualty rates that are connected to the populace in the areas overflowed because of a dam failure. These casualty rates rely upon:

- The severity of the flood
- Warning time
- Understanding the severity of the flood

In this strategy, the populace in danger is defined as the populace inside the inundated region when the dam fails, like this, it does not consider unequivocally the clearing procedures. To do it, it is critical to think about the populace situated in every one of the areas influenced by such a catastrophic situation. It is conceivable to fall back on registration information and to population studies performed by open institutions that reflect regular varieties. Other information, for example, the number of working individuals in the business and modern territories alongside their inception, is additionally critical to estimate the everyday varieties of the populace.

The technique proposed by the SUFRI task proposes a grouping of ten categories for the considered populace, as per the presence of warning system, coordination between the emergency system and the nearby specialists, broad communications, training of the populace, and so on. Every class is identified with some referential fatalities rates, contingent upon the warning time and the level of seriousness of the flood, which depend on the investigations done by Graham in 1999. SUFRI has put attention on the influenced populace, expected to act legitimately to lessen potential results of flooding. In this unique situation, effective risk communication assumes a noteworthy job to start, support, and keep up the information about flood decreasing measures and satisfactory behavior.

3.4.6 ESTIMATION OF ECONOMIC CONSEQUENCES

Large dam's failure can deliver extremely high outcomes in downstream zones. These monetary results are commonly isolated into two categories:

- **Direct consequences**: Damages delivered straightforwardly by the flood wave.
- **Indirect consequences**: Due to the monetary interruption that the flood delivers in the downstream zone and the misfortunes made by the nonattendance of the repository.

As a rule, current techniques used to gauge direct financial results comprise of two stages:

- Assessing the absolute estimation of land use, that is, the thing that would be the expenses if each structure and yield existing in the downstream areas were pulverized due to the flood.
- Applying those expenses to the curve depth-damages, that relates the greatest depth of a flood with a related estimation of destruction. Like this, by multiplying the level of decimation by the monetary expense of a maximal demolition, it is conceivable to estimate the financial outcomes of the important flood.

Therefore, to apply this philosophy, it is important to use as a beginning stage land-use maps and flood maps which demonstrate the depth of water at every area has appeared in Figure 3-6.

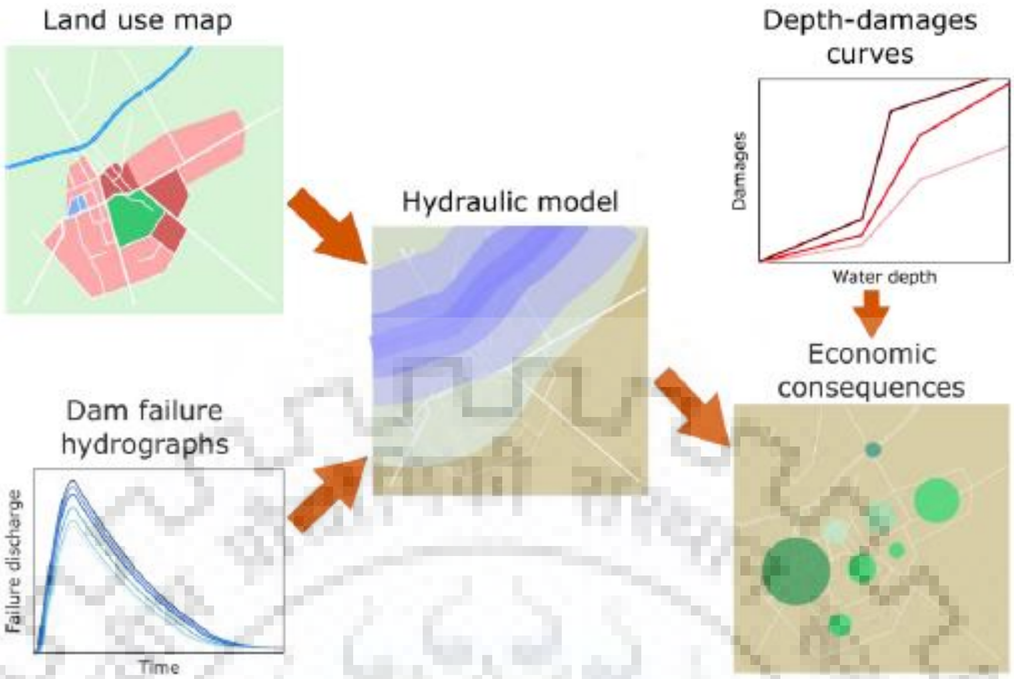

Figure 3-6 Procedure for estimation of Direct Economic Consequences

In the present analysis the following analysis have been carried out

1) Initial risk-based screening
2) Identification of failure modes
3) Quantitative risk assessment
    I. Preparation of risk model architecture
    II. Estimation of Pool level probability
    III. Performance Evaluation Of gates
    IV. Flood routing analysis
        a) Performed Level pool routing (By using Modified pul's method in HEC-RAS) for finding outflow discharge from reservoir.
        b) Performed Channel routing (By using hydraulic routing method in HEC-RAS) for generation of flood inundation map.

CHAPTER 4

STUDY AREA

4.1 GENERAL

In the present study, risk analysis for Tehri dam is performed. Various details of the dam are provided in the subsequent sections.

4.2 STUDY AREA

Tehri dam is located in Tehri Garhwal district of Uttarakhand. Tehri dam is a high rock and earth-fill embankment dam which has a height of 260.5m. The length of Tehri dam at the top is 575 meters. The width at top is 25.5 meters, which is flared to 30.5 meters at abutments and base width of the dam is 1,128 meters. The other parameters of the dam are listed below:

Dam Slope	
Upstream	1:2.5
Downstream	1:2
Spillways	
Chute Spillways	
Crest Level	EI. 815.0 m
Waterway	3 bays each of 10.5m width
Design Discharge	5480 Cumecs
Type and no. of gates	Radial Gates, 3 Nos.
Right bank shaft spillways	
Type &Nos.	Ungated, 2 Nos.
Crest Level	EI. 830.2m
Diameter of shaft	12m
Design Discharge	3880 Cumecs
Left bank shaft spillways	
Type	Gated 2Nos.
Crest Level	EI. 815m
Diameter of shaft	12m
Design Discharge	3680 Cumecs
Types of Gates	Radial Gates

Reservoir	
Full reservoir level (FRL)	EI. 830 m
Maximum Water level (MWL)	EI. 835 m
Dead storage level	EI. 740 m
Gross storage	3540 MCM
Dead Storage	925 MCM
Live Storage	2615 MCM
Water Spread at Full Reservoir level	44 sq. km
Water spread at Dead storage level	18 sq.km
Hydrology	
Normal Annual rainfall	1016 to 2630 mm
Maximum recorded flood discharge	3800 Cumecs
Adopted Maximum Flood for diversion during Monsoon period	8120 Cumecs
Probable Maximum Flood	15540 Cumecs
Routed Flood	13040 Cumecs

4.3 MAP FOR DOWNSTREAM AREAS OF TEHRI DAM

Whenever the inflow exceeds the water requirement for storage in the dam, for instance, in flood situations, the excess water is released to the downstream side by means of spillway. Thus, there is every chance of downstream areas to be flooded whenever significant amount of flow is discharged through the spillway. So downstream region from where a flood has to be passed plays a crucial role in risk assessment analysis. All the damage caused by flood is in the downstream regions, whether it causes damages in the form of loss of lives and economic losses.

ASTER DEM (digital elevation model) 30 M data downloaded from USGS earth explorer site have been used to carry out the analysis of flood risk mapping. Different tiles of the data were mosaicked in Arc GIS to get one single DEM. This DEM is given as input terrain for all HEC-RAS analysis and on the basis of this the inundation maps due to the flood discharges released from Tehri dam have been prepared.

CHAPTER 5

RISK ASSESSSMENT ANALYSIS ON TEHRI DAM

5.1 INITIAL RISK-BASED SCREENING RELATED TO TEHRI DAM

Tehri dam is an earth and rock fill dam which has a gross storage capacity of 3540 MCM, and a height of 260.5m. Thus, it comes under the category of major dams. The major cities in the downstream of Tehri Dam are Devprayag, Rishikesh, Haridwar, Bijnor, Garhmukteshwar etc. The areas downstream of Devprayag are densely populated. Therefore, if any failure occurs, then high population is at risk of flooding. Thus, Tehri dam comes under high potential hazard category. Accordingly, risk assessment is performed. In initial risk-based screening, all the available information pertaining to Tehri dam, as discussed in previous chapter, were collected.

5.2 IDENTIFICATION OF FAILURE MODES FOR TEHRI DAM

For identification of failure mode, a comprehensive analysis of the dam is carried out. In this, sound understanding about the type of dam, the material used for construction of dam, the spillway operation, the geology of that area, the hydrological aspects of that area, and the topography of that area is needed since all these factors play a vital role in the understanding and mitigation of failure in dam. Technical site visit is an important part of failure mode identification. The information was collected as per the questionnaire, prepared using CWC guidelines for a technical site visit. The questions for the technical site visit and the replies of the questions by the design office of THDCIL are listed below:

Question **What is the design flood for Tehri dam? How is it obtained?**

Answer The PMF for Tehri dam is 15,540 Cumecs, which is obtained using the hydro-meteorological approach.

Question **Has the hydrological adequacy of Tehri dam been checked? When? What hypothesis has been followed on this check?**

Answer Yes, hydrological adequacy of Tehri dam has been checked. Physical model tests on scale ranging from 1:30 to 1:80 were conducted to test the spillways and ILO of Tehri dam to accomplish the most improved water hydraulic design of each segment and to watch their conduct in various circumstances.

Question **Are there seasonal freeboard requirement for flood routing?**

Answer A freeboard of 9.5 m above the full reservoir is provided in Tehri dam.

Question **What are the gates operation rules?**

Answer In Tehri dam, radial gates are given in ILO, chute spillway and LBSS to control/manage the excess flood water. In contrast to vertical gates, radial gates control the progression of water in a better way. In ILO, there is one regulating radial gate with an exact opening of 4.5m x 6.0m. This opening is utilized during starting of filling of the dam. One fixed wheel emergency door with an opening of 4.5m x 7.5m has also been provided. The admission structure of Left bank shaft spillway (LBSS) is outfitted with a radial gate of size 10.5m x 15.50m.

Question **How was the hydraulic behaviour of the gates-spillway systems during previous floods?**

Answer In September-2010, Tehri dam received a flood of enormous volume with peak flow as 3500 m^3/s, and every one of the spillways had to be operated to pass this flood. Spillways of Tehri dam passed the whole flood satisfactorily. There was no real harm to the spillway structures aside from some minor scouring on the glacis of chute spillway and at some development joints which were effectively fixed after the passage of the flood.

Question **What is the current maintenance state of gates and also of other electro-mechanical equipment?**

Answer The gates are consistently checked for security viewpoint before and after the monsoon for appropriate maintenance. There is fixed timetable and agenda for the examination of the gates which is strictly pursued, observed and monitored at the highest level in THDCIL.

Question **What are the power supply options?**

Answer There is one substitute power source other than the primary power source. In the case of complete power failure, there is also an arrangement of manual operation of gates to cater to any emergency.

Question **What is the precautionary measure taken to prevent toe erosion? Moreover, for the safety of the stilling basin?**

Answer The chute spillway of Tehri dam is located adjacent to the dam and water drops from a head of 225 m; The same may lead to toe erosion, when the spillway is operated. Satisfactory consideration has been taken by giving a stilling basin, with a dead water pool of 140m x50m x 22m, intended to scatter the high velocity because of the high head through hydraulic jump formation. To avoid the Tehri dam from toe disintegration, the following provisions have been made:

High Training Walls – The preparation dividers/control dividers of the stilling basin for the chute spillway were built with foundation level 12m beneath the basin floor and top dimension 13m over the greatest water level relating to PMF. These highlights were considered as uncommonly safe to avoid dam toe from backflow of water.

Efficient Hydraulic Design to Guide the Flood Releases – The left channel of chute spillway was planned to such an extent that it manages and adjusts the outpourings from chute spillway and outlets of RBSS to the normal course of waterway Bhagirathi through model testing.

Extra Safety Provisions – Downstream slopes of the dam are secured by a 10 m thick layer of stones and phyletic rocks of sizes up to over 600mm; not more than 25-30% material smaller than 300 mm size with minimum size limited to 150 mm. The downstream dyke toe is likewise ensured by expanding an extra 150m long zone with fill.

Question **Is regular monitoring done?**

Answer Yes, regular monitoring is done.

Question **What are the current state of monitoring data and the state of monitoring system?**

Answer **Efficient monitoring scheme**: A well–arranged instrumentation system has been provided in the dam to have a constant appraisal about its performance. task. Around 353 number of instruments have been introduced in the dam body and its foundation, with a game plan for programmed information securing and recovery, for estimation of pore pressures, horizontal and vertical stresses, deformations and settlements in the body of the dam and at its surface, the drainage through dam body and abutments.

Continuous monitoring: Broad periodical reviews (post-storm and pre-rainstorm) assume the real job in boosting the certainty level that seepage is not happening. The checklist for assessments in case of Tehri dam has been prepared utilizing national and universal rules set down in safety manuals of ICOLD, USBR (Safety Evaluation of Existing Dams) and CWC, India. Standard monitoring and examination of drainage, settlement, cracks, temperature, and stresses are done through the instruments introduced in dam body. Arrangement of galleries has likewise been made at various dimensions inside the dam body for inspection reason.

Question **How is slope stability of rock near Tehri dam maintained?**

Answer The reservoir spread of Tehri dam at FRL is around 42 km^2. The failure of slopes around this territory can cause an abrupt increment in supply level. Normal observations of slopes is being done to evaluate the likelihood of any significant landslide. The slope around the region of Tehri dam has been treated by cleaning and straightening (scaling) and shotcreting to maintain a strategic distance from any harm to the dam and its appurtenant structure because of the landslide.

Question **How high is the seismic hazard in the dam area?**

Answer Tehri dam is viewed as inclined to high seismic dangers because the site is situated in Zone-IV of the five zones in the seismic zoning map of India which compares to an intensity rating of VIII on Modified Mercalli (MM) scale.

Question **Has Tehri dam stability been checked for seismic events? When?**

Answer The Department of Earthquake Engineering, Indian Institute of Technology Roorkee has contemplated the material properties of Tehri dam, completing field tests like the Block Vibration Test, Wave Propagation Test, and Vertical Dynamic Plate Load tests. Tehri dam has been tested for a peak ground acceleration up to 0.5g for a magnitude 8.0 earthquake and observed to be protected. Dam area was checked through Hydro Project Institute, Moscow utilizing real accelerogram of Gazli (Karakyr) tremor, which had an abnormal state of peak ground acceleration up of 1.36g vertical and 0.72g horizontal, both acting at the same time, and the dam was observed to be safe.

After a technical site visit and dam safety evaluation, some failure modes and its corresponding classification were identified that are shown in Table 5-1.

Table 5-1 Types & classification of failure mode of the Tehri dam

S.N.	TYPES OF FAILURE MODE	CLASSIFICATION OF FAILURE MODE
1	Overtopping failure due to hydrologic event	B
2	Overtopping failure due to sudden landslide	C
3	Piping failure in the main body of the dam	B
4	Piping failure in the foundation	C
5	Sliding in a dam along the dam foundation surface	B
6	Sliding in a seismic event in the main dam	C
7	Overtopping in a seismic event in the dam	C
8	Failure due to liquefaction in a seismic event in the dam	C
9	Failure due to slope instability in a seismic event in the dam	C

5.3 QUANTITATIVE RISK ASSESSMENT FOR TEHRI DAM

In the study, only the overtopping failure mode has been analysed. The subsequent analysis has been carried out as per the following steps:

i. Assume the inflow hydrograph as PMF;
ii. The pool level probabilities are computed;
iii. Gate reliability of spillway is calculated;
iv. Flood routing analysis is performed for different pool levels and different gate conditions. The ungated spillway of morning glory has been assumed to be working above 830m level.
v. Flood routing analysis has also been carried out for the dam failure due to overtopping.

5.3.1 ESTIMATION OF POOL LEVEL PROBABILITY OF TEHRI DAM RESERVOIR

For estimation of pool level probability, pool level register data of Tehri dam is used. For estimation of pool level probability, the data from 1, January, 2015 to 31, December, 2017 is used. The pool level probability curve is plotted in Figure 5-2.

5.3.2 ESTIMATION OF SPILLWAY RELIABILITY

Tehri dam gates are new with regular monitoring and maintenance. Hence the reliability of gates is considered as 95%. Tehri dam has five gated spillways. The reliability of the gates for different combinations is computed using binomial distribution as follows:

$$P(x) = \frac{n!}{(x!\,(n-x)!)} * p^{x} * (1-p)^{n-x} ----------- 5.1$$

Where,

P(x) is the reliability of x gates working; n represents the total number of gates;

x represents the number of gates working; p represents the reliability of individual gate;

(1-p) represents the probability of failure of individual gate;

The probabilities are shown in Table 5-2.

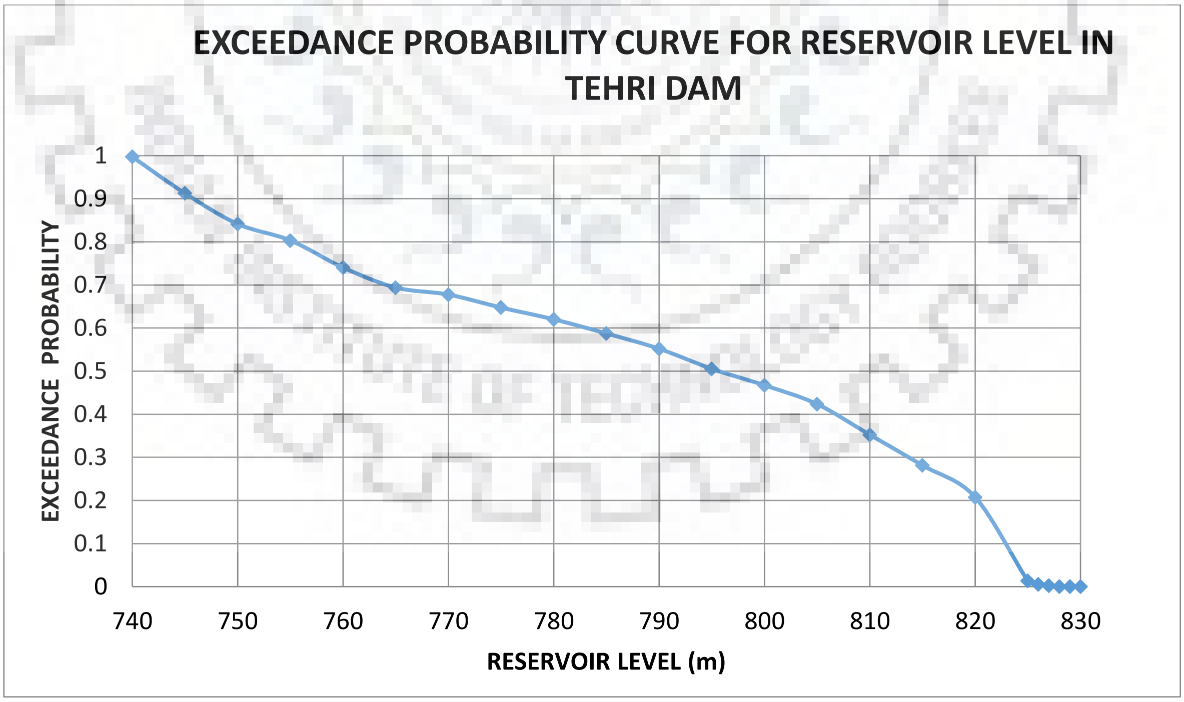

Figure 5-1 Exceedance probability curve for Tehri reservoir

Table 5-2 Probability of working of gates for different gate combinations

No. of gates working properly	Probability (%)
0	3.1×10^{-5}
1	2.97×10^{-3}
2	0.11
3	2.14
4	20.36
5	77.37

In this report, flood routing analysis has been performed for risk assessment analysis of dam body and for finding out downstream consequences using HEC-RAS software. Complete procedure of model preparation in HEC-RAS for flood routing, reservoir routing as well as channel routing, and its results have been explained in the next chapter.

5.4 ANNUAL PROBABILITY ESTIMATION

In this report, annual probabilities have been estimated for different combination of gates and for different initial reservoir levels by following sequence of annual probability estimation that is shown in Figure 5-2. Estimated annual probabilities have been shown in Table 5-3.

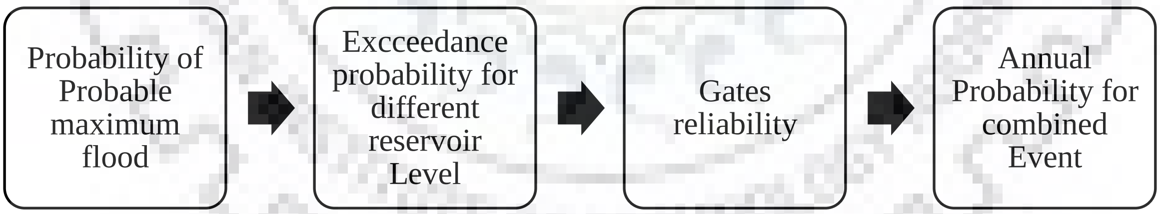

Figure 5-2 Sequence for annual probability estimation

All of these events i.e. PMF, initial reservoir level and availability of gates are independent events. So the combined probability of happening of all of these events at the same time will follow multiplicative rule.

Example: If probable maximum flood enters into a reservoir and that time initial reservoir level is at 810 m and all the gates are in working condition than what is the annual probability of happening of this event.

Answer: Probable maximum flood has a probability of 1/10000. The probability of initial reservoir level 810 m when flood enters is 0.352459. And probability of all gates working is 0.7733.

Probability of Probable maximum flood → Excceedance probability for different reservoir Level → Gates reliability → Annual Probability for combined Event

$$1/10000 \times 0.352459 \times 0.7733 = 2.72698 \times 10^{-05} = 0.0027\ \%$$

The annual probabilities for other conditions i.e. all the gates working for different reservoir levels are tabulated in Table 5-3. The results of the case when some of the gates have been assumed to be non-functional are shown in Table 5.4.

Table 5-3 Annual probabilities for various initial reservoir levels with all the gates working

Initial reservoir level	Gates availability	Annual probability	Remarks
810m	All the gates are working	2.72698×10^{-05}	No Overtopping
815m	All the gates are working	2.17735×10^{-05}	No Overtopping
820m	All the gates are working	1.60659×10^{-05}	No Overtopping
825m	All the gates are working	1.05695×10^{-06}	No Overtopping
826m	All the gates are working	4.2244×10^{-07}	No Overtopping
827m	All the gates are working	2.1122×10^{-07}	No Overtopping
828m	All the gates are working	7.737×10^{-08}	No Overtopping
829m	All the gates are working	7.737×10^{-08}	No Overtopping
830m	All the gates are working	7.737×10^{-08}	No Overtopping

Table 5-4 Annual probabilities for various initial reservoir levels with some of the gates not working

Initial reservoir level	**Gates availability**	Annual probability	Remarks
820m	One gate not working	4.2275×10^{-06}	No overtopping
	Two gates not working	4.4507×10^{-07}	No overtopping
	Three gates not working	2.3268×10^{-08}	Overtopping
	Four gates not working	6.1464×10^{-10}	Overtopping
	Five gates not working	6.4896×10^{-12}	Overtopping
825m	One gate not working	2.7818×10^{-07}	No overtopping
	Two gates not working	2.928×10^{-08}	Overtopping
	Three gates not working	1.530×10^{-09}	Overtopping
	Four gates not working	4.0436×10^{-11}	Overtopping
	Five gates not working	4.2690×10^{-13}	Overtopping
830m	One gates not working	2.036×10^{-08}	Overtopping
	Two gates not working	2.1434×10^{-09}	Overtopping
	Three gates not working	1.12×10^{-10}	Overtopping
	Four gates not working	2.96×10^{-12}	Overtopping
	Five gates not working	3.125×10^{-14}	Overtopping

Table 5-4 shows the importance of the working of the gates. The working of the gates is to be ensured. As long as the gates are functional, the dam will remain safe under all the eventualities. The probabilities of the overtopping are very less but not negligible.

Different scenarios of non-overtopping and overtopping cases have been created for the situation when PMF enters the reservoir at different initial levels and the outflow hydrographs at the dam site have been computed using Modified Pul’s method in HEC-RAS framework, though the same could be done otherwise also. This is followed by flood risk mapping of the downstream areas under two illustrative cases. The results are presented in the next chapter.

CHAPTER 6

RESERVOIR FLOOD ROUTING ANALYSIS, DAM BREAK ANALYSIS AND FLOOD INUNDATION MAPPING

6.1 FLOOD ROUTING ANALYSIS ON TEHRI DAM

Flood routing analysis for Tehri dam was performed with different combination of gates and initial reservoir levels. For reservoir and channel routing studies, PMF hydrograph has been considered as input at different pool levels and different gate working conditions. For the channel section downstream of Tehri dam, a number of initial conditions may emerge. In the present study the average monsoon discharge has been assumed to be existing in the channel as an initial discharge. For River Alaknanda at Devprayag also the average monsoon discharge has been assumed. The results of the inundation mapping are valid only for this condition. If the initial discharge in the channel is more than this discharge, then more area will be inundated. Hence the results of the inundation mapping are just indicative.

The analysis has been performed using HEC-RAS model as per the following details/ steps:

Step 1: Geometry development for HEC-RAS model

For geometry development, Aster global DEM 30 M from the USGS site are downloaded for the study area. Afterwards, a coordinate file is developed using ArcGIS. The terrain file is developed using RAS MAPPER. The terrain file is imported in the Geometric Editor of HEC-RAS and model geometry is developed. The details of other steps are given in the following section:

1. 2-D flow area Development

For 2-D flow area development open HEC-RAS 2-D flow area tool has been used. 2-D flow area is drawn for the downstream of Tehri Dam to Narora (Aligarh), which is about 376 km from Tehri. 2D flow area defines the extent of cross sections to be considered during the propagation of the flood wave in HEC-RAS. Alaknanda River meets with Bhagirathi River at Devprayag and hence about 26 Km of Alaknanda River is also included in 2-D flow area. For 2 D mesh creation, cell size is taken as 200m×200m.

Manning’s roughness coefficient of 0.04 has been taken for the channel and the overland floodplain. Mesh properties of 2D flow area are shown in Fig. 6-1.

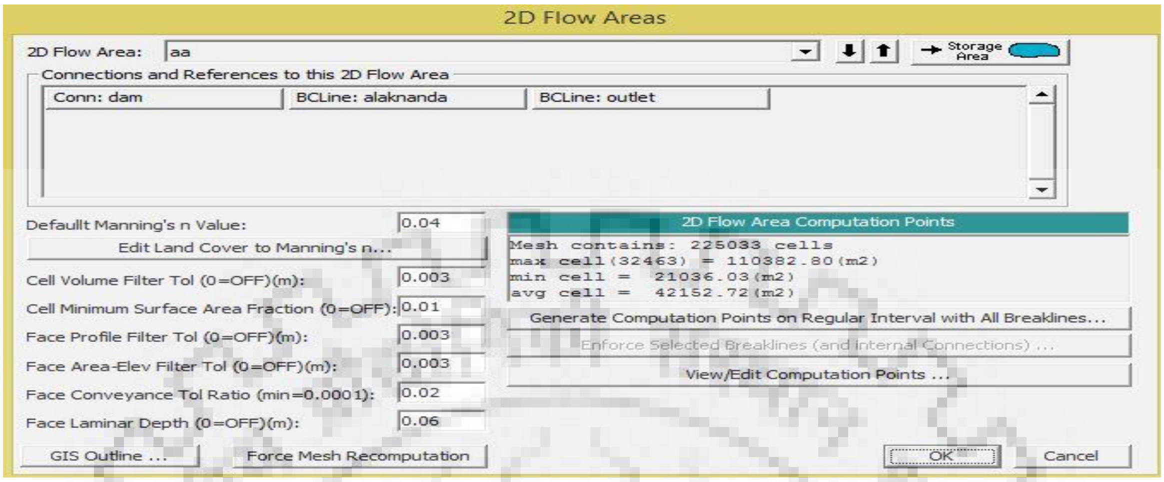

Figure 6-1 Showing Mesh Properties for 2-D flow area

2. Development of HEC-RAS model for Tehri Dam reservoir

For development of Tehri reservoir in HEC-RAS Open storage area tool has been used along with reservoir elevation capacity curve. The elevation capacity curve of Tehri dam is tabulated in Table 6-1 and plotted in Fig. 6-2.

Table 6-1 Elevation Vs Volume curve for Tehri Resrvoir

Elevation (m)	Volume (MCM)	Elevation (m)	Volume (MCM)
607.5	0	690	270
608	0.2	695	310
608.5	0.4	700	350
609	0.5	705	400
610.5	0.7	710	450
611	1	715	520
612	1.4	720	590
612.6	1.8	725	662.5
613.2	2	730	735
613.5	2.5	735	830

614	3	740	925
614.8	3.5	745	1012.5
615.2	4	750	1100
615.8	4.5	755	1207.5
616	5	760	1315
617.3	5.5	765	1432.5
618	6	770	1550
618.6	6.5	775	1685
619	7	780	1820
620	8	785	1965
625	11.5	790	2110
630	15	795	2265
635	20	800	2420
640	25	805	2600
645	37.5	810	2780
650	50	815	2965
655	67.5	820	3150
660	85	825	3345
665	112.5	830	3540
670	140	835	3770
675	170	840	4000
685	235		

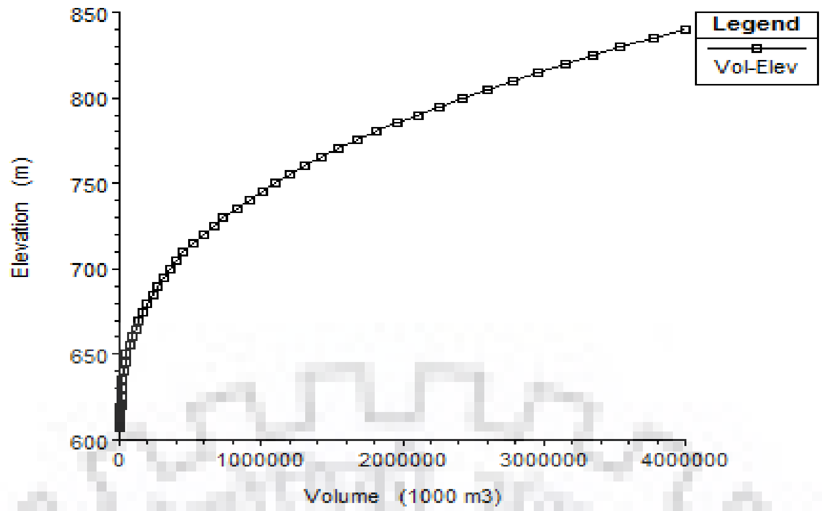

Figure 6-2 Elevation vs Volume curve for Tehri Reservoir

3. Development of connection between 2-D flow area and reservoir in HEC-RAS model using the spillway

For connecting the reservoir and the 2-D flow area in HEC-RAS model, gates are provided. In Tehri dam, there is chute spillway which has three gates of dimension (10.5×15.5) each. The gates are radial. The capacity of all gates working at full reservoir level is 5481 Cumecs, each gate having a capacity of 1827 Cumecs. Tehri dam has also LBSS (Left bank shaft spillway) which has two radial gates of size 10.5×15.5 each. The total capacity of LBSS is 3880 Cumecs. Tehri dam also has RBSS (right bank shaft spillway), which is ungated, having a capacity of 3680 Cumecs. The operating level of chute spillway is El. 815m, LBSS is El. 815m and RBSS are El. 830.2m.

In HEC-RAS, there is no provision of providing shaft spillway, but Tehri dam has a shaft spillway. For this shaft spillway, dimensions and discharge is converted in terms of chute spillway. So in HEC-RAS model, five radial gates are provided.

For Connection of 2-D flow area and reservoir in HEC-RAS model, all gates related data is entered in the SA/2D Connection Editor Toolbar.

For performing flood routing analysis of the dam, its outflow characteristics curve and gates boundary conditions must be known. Outflow characteristics curves for the chute spillway for one radial gate opening is shown in Figure 6-3. The outflow characteristics curve for the ungated spillway is shown in Figure 6-4.

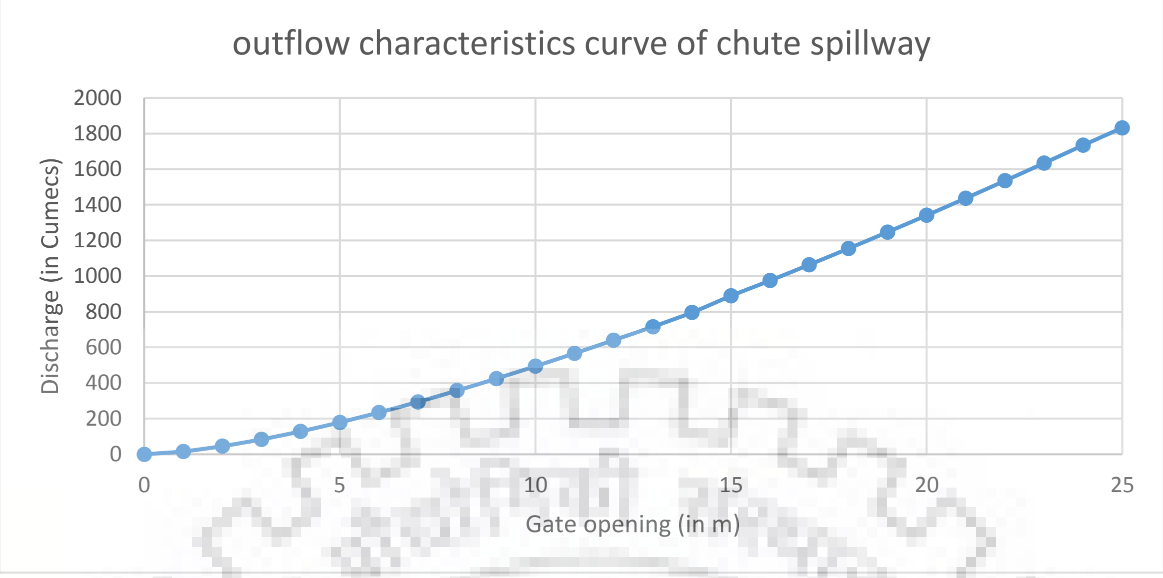

Figure 6-3 outflow characteristics curve for chute spillway

Tehri dam also have 2 ungated spillways and its outflow for various reservoir elevation have been shown in figure number 6-4.

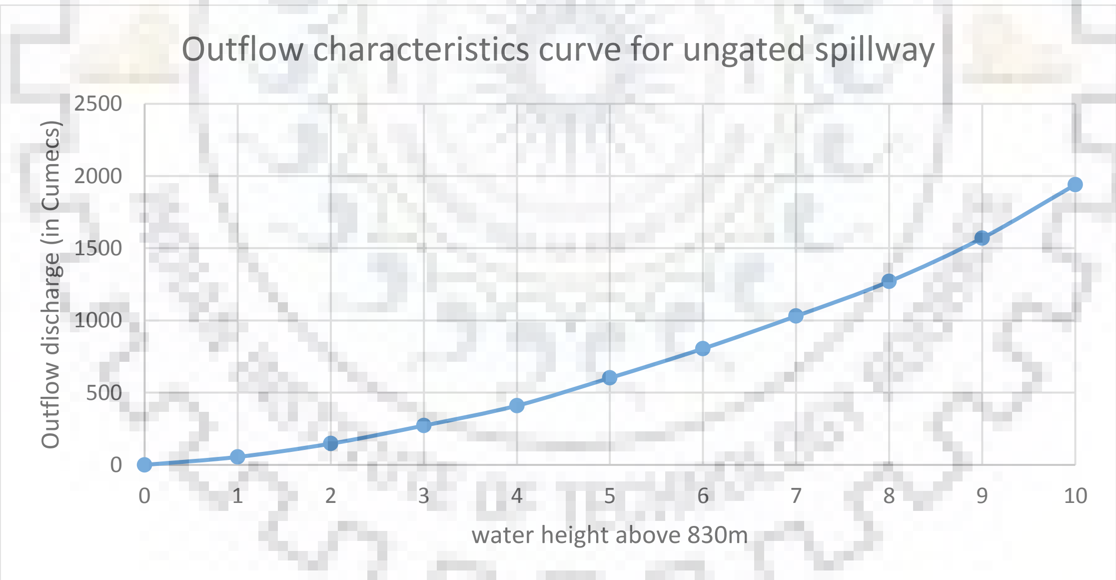

Figure 6-4 Outflow characteristics curve for ungated spillway

When all gated and ungated spillways of Tehri dam are open, the outflow characteristics curve for water surface elevation has been shown in Figure 6-5.

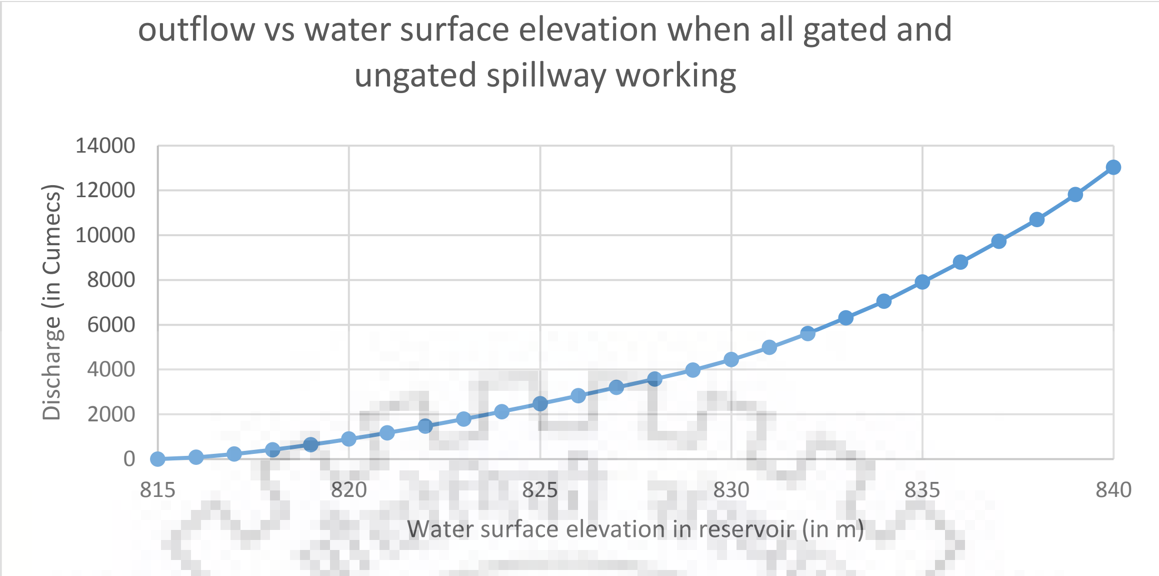

Figure 6-5 Outflow characteristics curve for Tehri dam when all gated and ungated spillways are working

Outflow characteristics curves of Tehri dams for availability of different gates are shown in Figure 6-6.

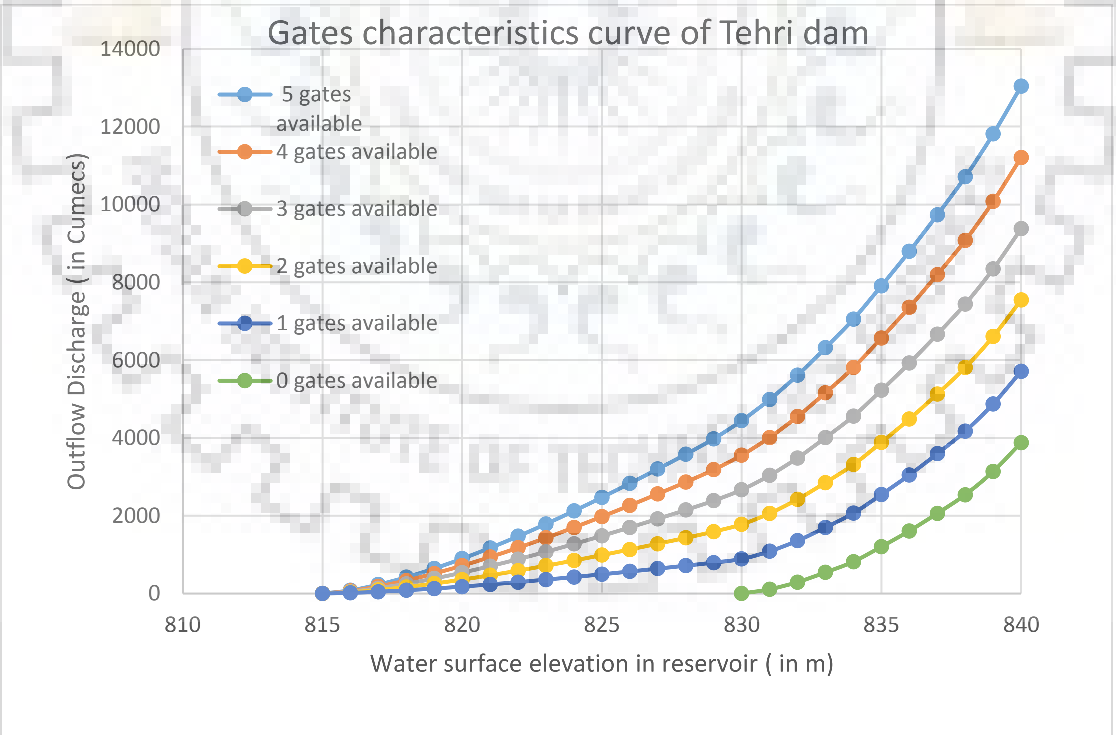

Figure 6-6 Outflow characteristics curve for Tehri dam for availability of number of operational gates

The boundary condition of gates used in this report for initial reservoir level of 810 m, 815 m, and 820 m have been shown in Table 6-2.

Table 6-2 Boundary conditions used for chute spillways for initial reservoir levels of 810 m, 815 m and 820 m.

Elevated controlled gates	
Type of spillway	Chute spillway
Reference	Based on upstream water surface elevation
Upstream water surface elevation at which gates begin to open	820 m
Upstream water surface elevation at which gates begin to close	815 m
Maximum gate opening	25 m

The boundary conditions of gates used for initial reservoir level more than 820m have been shown in Table 6-3.

Table 6-3 Showing boundary condition used for chute spillways when initial reservoir is more than 820 m.

Elevated controlled gates	
Type of spillway	Chute spillway
Reference	Based on upstream water surface elevation
Upstream water surface elevation at which gates begins to open	830 m
Upstream water surface elevation at which gates begins to close	815 m
Maximum gate opening	25 m

Step2: Reservoir routing analysis

Reservoir routing studies have been carried out using Modified Pul’s method with PMF as input flood hydrograph for various initial reservoir levels and different gate conditions. Conditions for which the runs were taken are listed in Table 6-4. The inflow hydrograph is plotted in Fig. 6-7. The maximum water level achieved in each case is also shown in this table. Inflow and outflow hydrographs and the reservoir levels were plotted for all the cases. In the report, some of these graphs have been shown. The figure number of such graphs are shown in the last column of this table. Flood routing results when initial reservoir level is at 820 m and all gates of spillway are working is given in Annexure-III.

Table 6-4 Various initial reservoir level and different gate conditions considered in the study

Initial reservoir level	Gate conditions	Max. water level achieved (in m)	Max. outflow discharge (in Cumecs)	Inference drawn	Fig. No.
810 m	All the gates are working	834.96	8320	No overtopping	
815 m	All the gates are working	836.52	9633	No overtopping	
820 m	All the gates are working	837.82	10361	No overtopping	6-8
825 m	All the gates are working	839.09	11119	No overtopping	6-9
826 m	All the gates are working	839.21	11480	No overtopping	
827 m	All the gates are working	839.32	11890	No overtopping	
828 m	All the gates are working	839.48	12180	No overtopping	
829 m	All the gates are working	839.61	12410	No overtopping	
830 m	All the gates are working	839.72	12770	No overtopping	6-10
820 m	One gate not working	838.43	8966.2	No overtopping	
	Two gates not working	839.26	8401	No overtopping	
	Three gates not working	839.91	7398	No overtopping	
	Four gates not working	840.18	6245	Overtopping	
	Five gates not working	840.39	4814	Overtopping	

825 m	One gate not working	839.84	10654	No overtopping	
	Two gates not working	840.22	9600	Overtopping	
	Three gates not working	840.62	9216	Overtopping	
	Four gates not working	840.89	9121	Overtopping	
	Five gates not working	841.16	9245	Overtopping	
830 m	One gates not working	840.40	12076	Overtopping	6-11
	Two gates not working	840.88	12701	Overtopping	6-12
	Three gates not working	841.43	14197	Overtopping	6-13
	Four gates not working	841.85	15740	Overtopping	
	Five gates not working	842.06	16205	Overtopping	

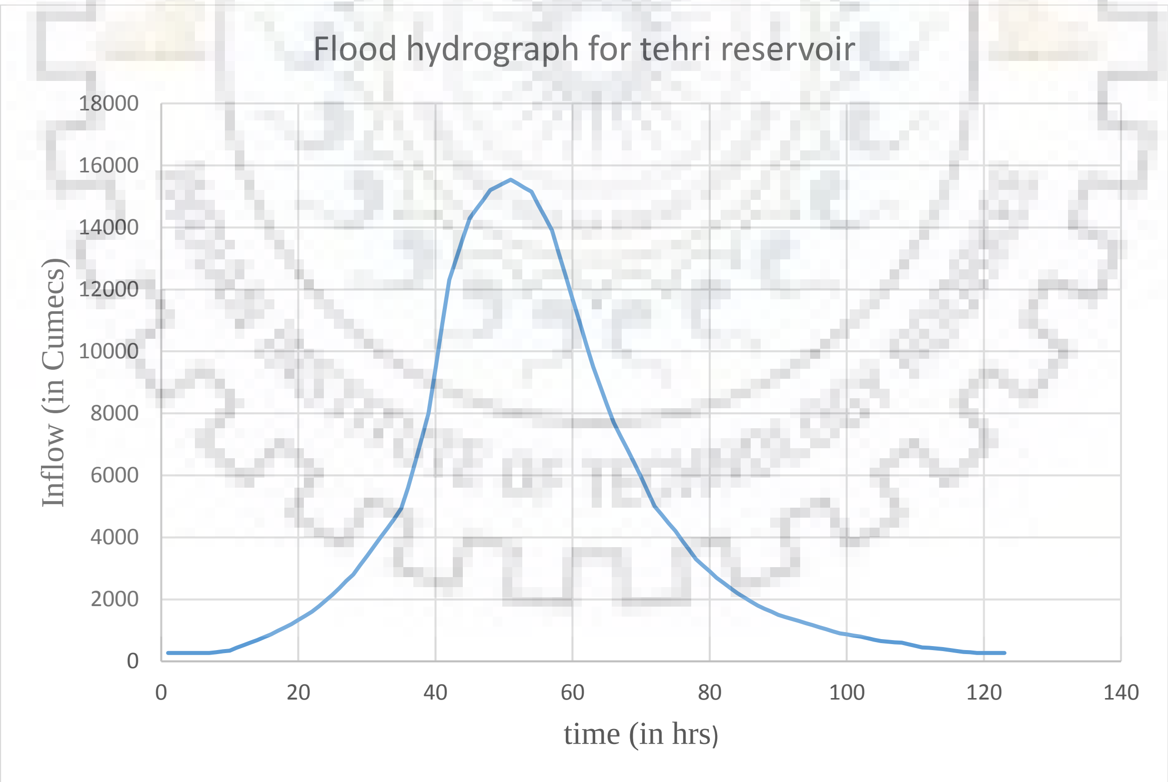

Figure 6-7 PMF Inflow Hydrograph

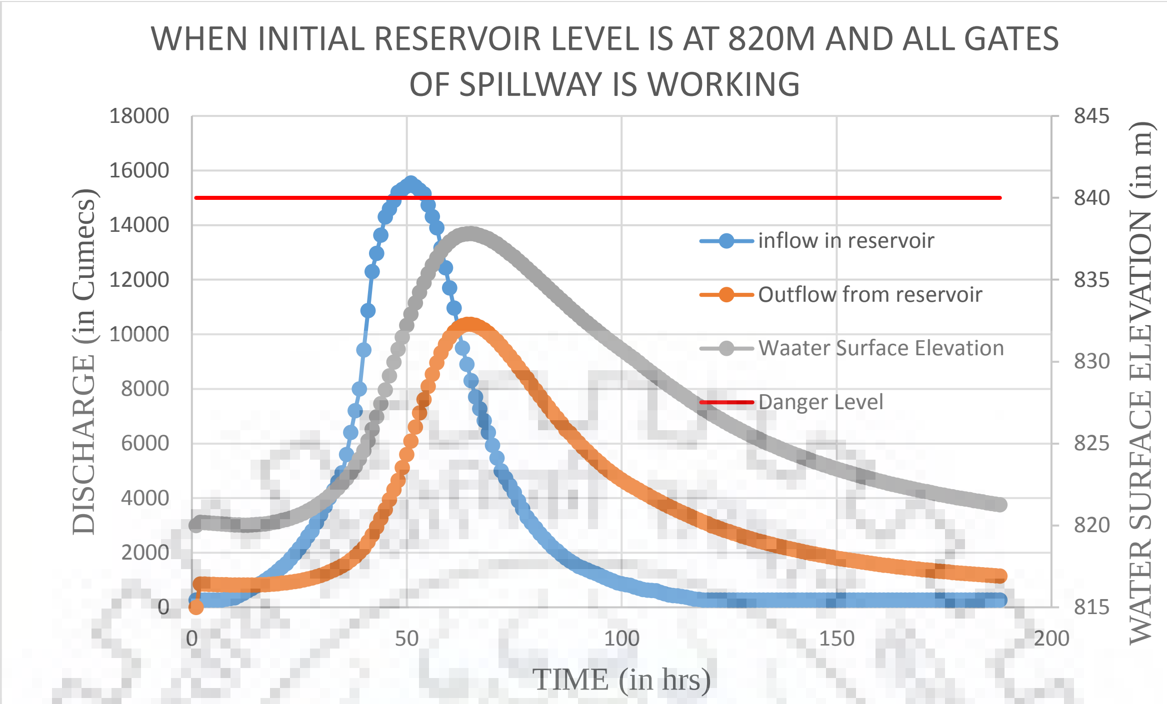

Figure 6-8 Reservoir routing results when all the gates are working with initial reservoir level as 820m

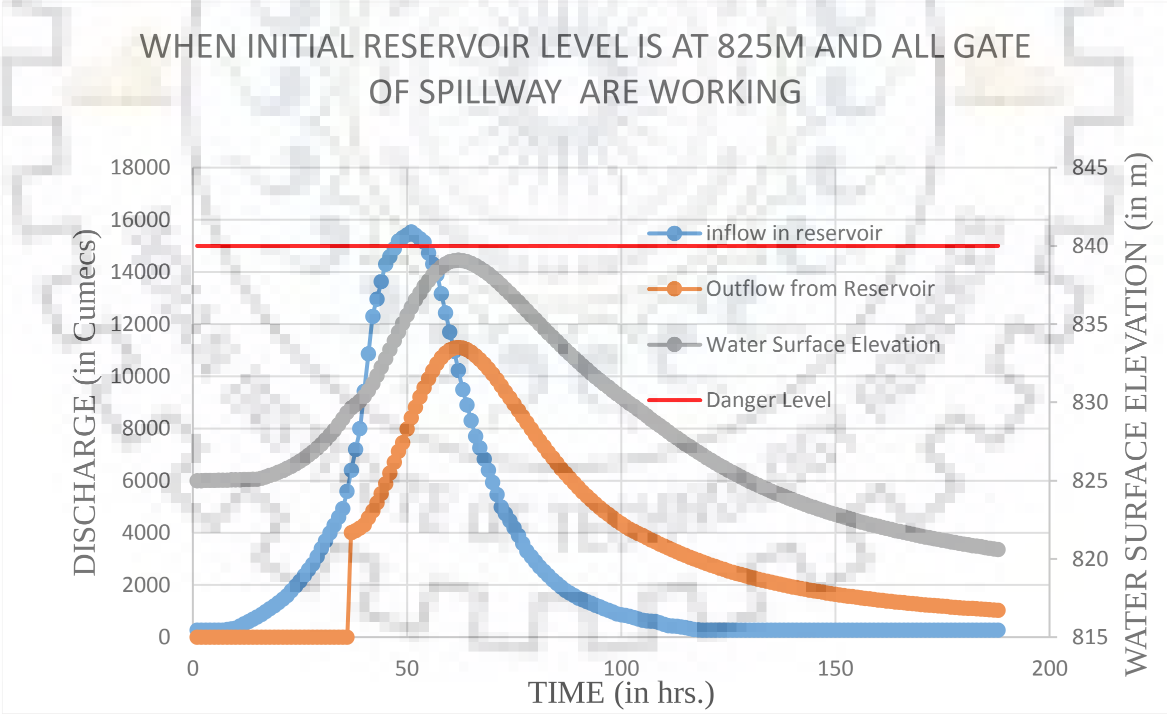

Figure 6-9 Reservoir routing results when all the gates are working with initial reservoir level as 825m

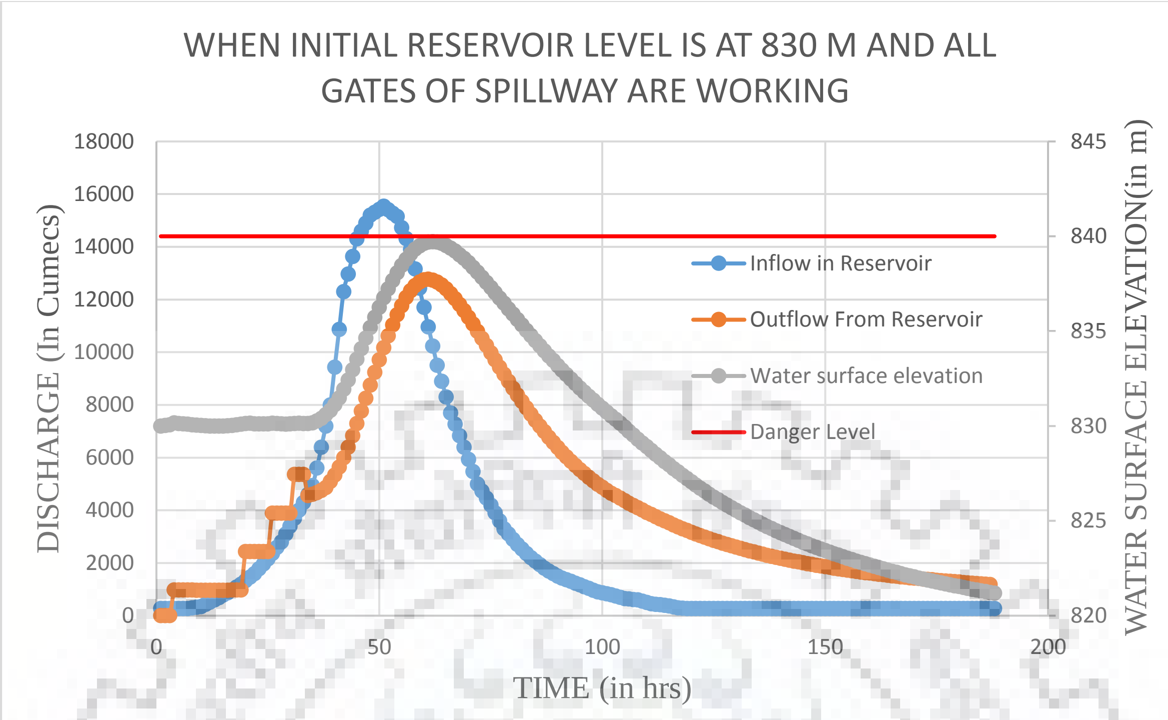

Figure 6-10 Reservoir routing results when all the gates are working with initial reservoir level as 830m

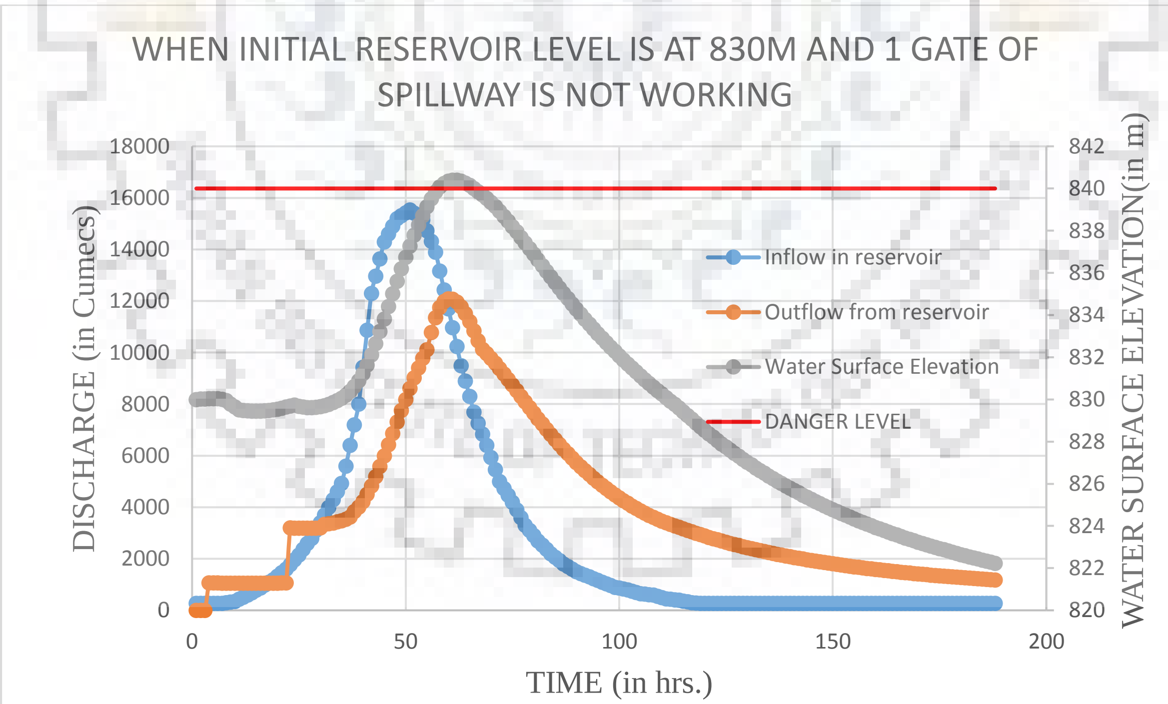

Figure 6-11 Reservoir routing results when the initial reservoir level is at 830 m and one gates is not functioning

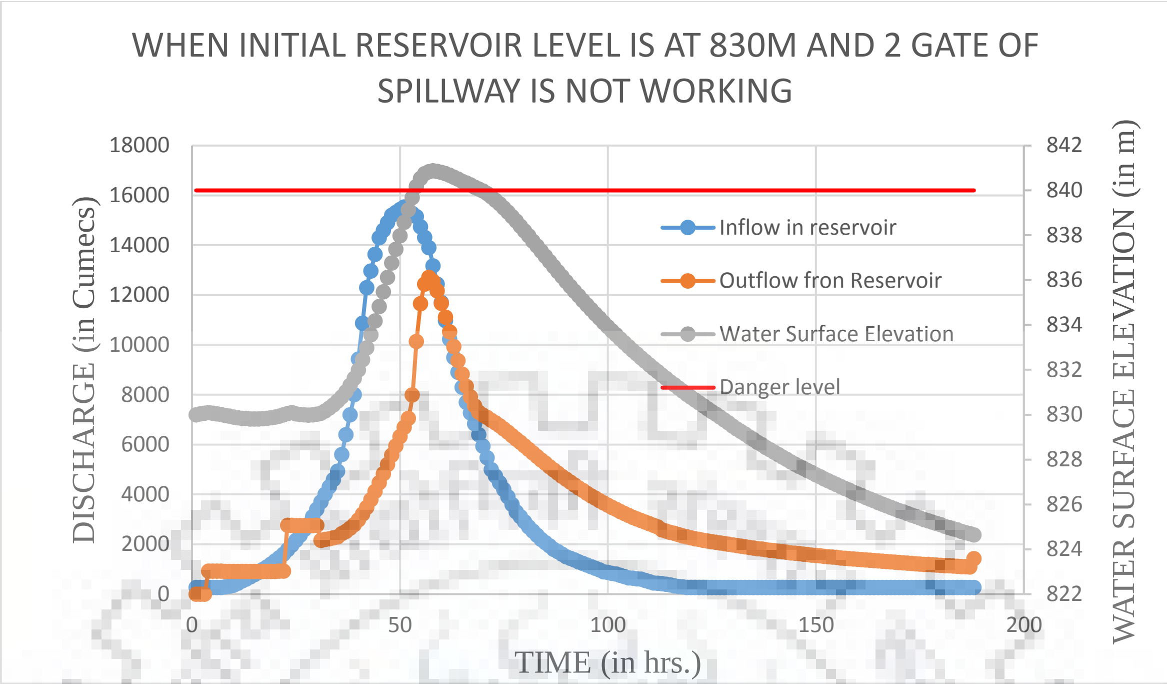

Figure 6-12 Reservoir routing results when the initial reservoir level is at 830 m and two gates are not functioning

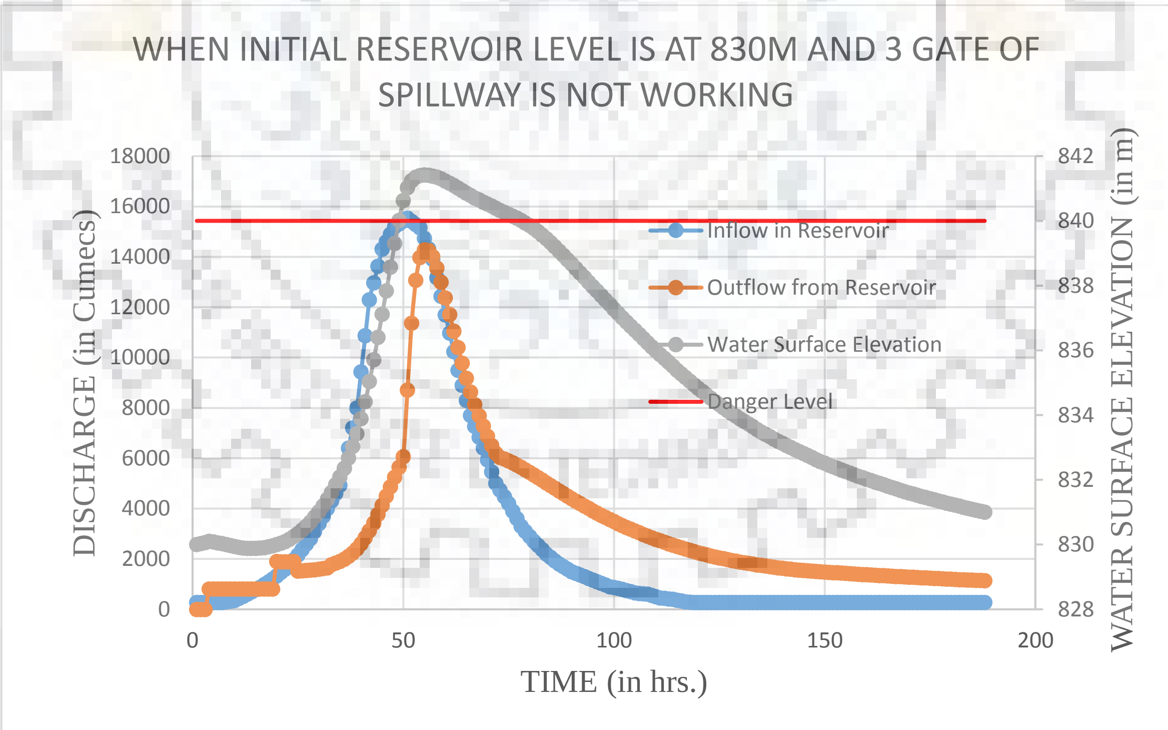

Figure 6-13 Reservoir routing results when the initial reservoir level is at 830 m and three gates are not functioning

6.2 FLOOD ROUTING RESULTS WHEN DAM FAILS/ BREAKS IN OVERTOPPING

In the case of proper functioning of the gates during the flood season, the spillways are able to safely pass the PMF even at an initial reservoir level of 830 m. However, the overtopping of the dam is possible in case of non-functioning of some of the gates under different initial reservoir levels (Table 6-4). The dam break analysis and its consequence on the downstream area has been performed using HEC-RAS model.

Dam break analysis requires breach depth, breach width, breach initiation time and breach formation time and side slope angles as per following details:

Breach depth: This is the vertical extent of the breach, measured from the dam crest down to the invert of the breach.

Breach width: This is average of top and bottom width.

Breach initiation time: The breach initiation time should begin with the first flow over or through a dam that is of enough significance to warrant warning, evacuation, or heightened awareness of the potential for dam failure.

Breach formation time: The breach formation phase begins at the point at which the breach begins to compromise the reservoir volume; flow rates and erosion rates increase dramatically as the breach is enlarged. For failures caused by overtopping, the breach formation time can be assumed to begin at the instant at which erosion progresses back through the upstream edge of the crest.
Side Slope Angles: are normally assumed 1.0 H: 1V for overtopping failure and 0.7H: 1V otherwise for piping and seepage failure.

For dam break analysis, the first requirement is to fix the breach parameter. Breach parameters used in the present have been by computed using Froehlich (2008) regression equations as follows:

$$B_{avg} = 0.27 K_0 V_W^{0.32} H_b^{0.04} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .6.1$$

$$t_f = 63.2\sqrt{V_w/gH_b^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 6.2$$

Where,

B_{avg} = Average breach bottom Width (in meters).

K_0 = constant (1.3 for overtopping failure, 1.0 for piping).

V_w= Reservoir volume at time of failure (in m^3).

H_b = height of the final breach (in meters).

g = gravitational acceleration (9.80665 meters per second squared)

t_f = breach formation time (in seconds).

Average side slope should be (1.0 H: 1V for overtopping failure, 0.7H: 1V otherwise (i.e. piping/seepage))

The breach parameters are shown in Table 6-5.

Table 6-5 Breach parameters for Overtopping failure

Overtopping failure				
Breach bottom width	Side slopes	Breach development time	Breach weir coefficient	Trigger Failure
435m	1:1	7.09 hrs.	2.6	At set time

The generated dam break flood hydrographs at the dam site are shown in Figure number 6-14 and 6-15 for two initial reservoir levels of 825 and 830 m. It may be seen from these figures that the outflow hydrograph peaks remain almost the same in case of dam break floods generated in case of these two initial reservoir levels. Inundation mapping of the downstream areas due to these dam break floods was done using HEC-RAS 2D modelling. The results are presented in next section.

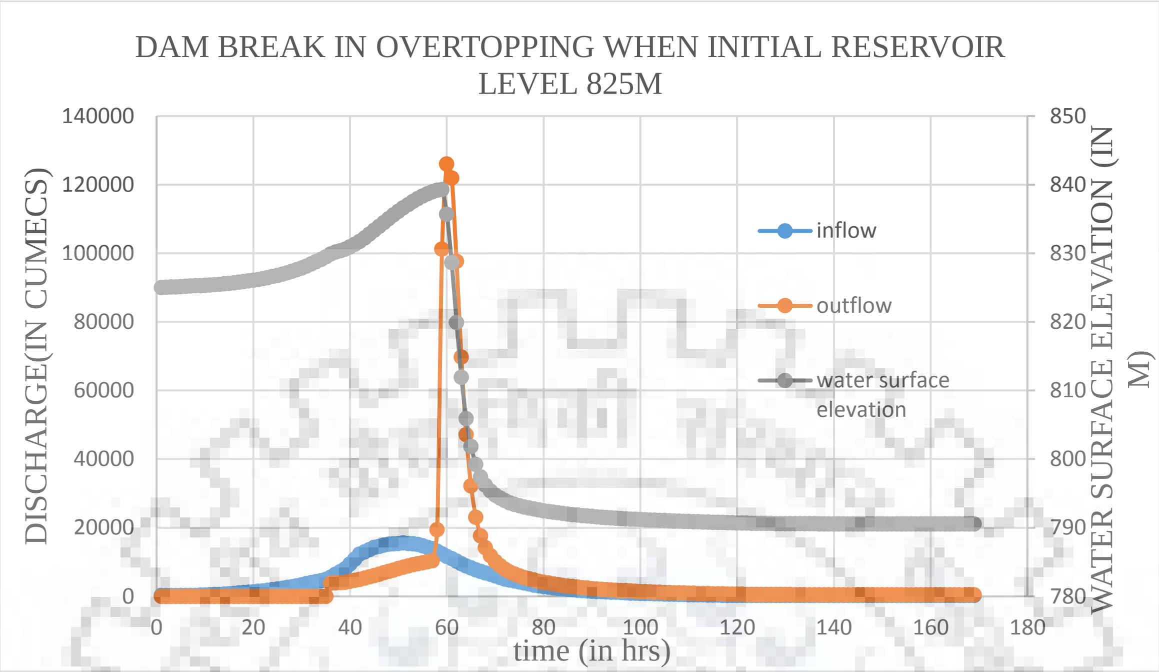

Figure 6-14 Dam break results in overtopping when the initial reservoir level is 825m

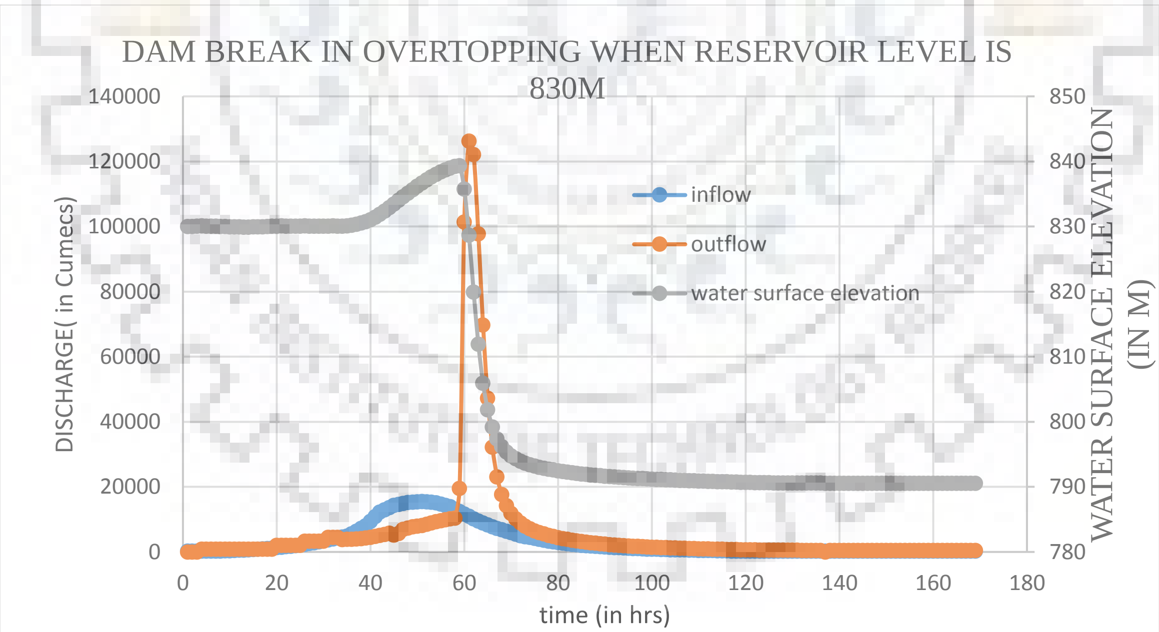

Figure 6-15 Dam break results in overtopping when the initial reservoir level is 830m

6.3 DOWNSTREAM INUNDATION MAPPING

For finding downstream consequences, flood inundation maps are generated for various cases used in this report. Flood inundation map are generated for 376 km stretch from Tehri to Narora, by using HEC-RAS 2-D modelling. The diffusive wave equation is used for performing channel routing in HEC-RAS 2-D model. Flood inundation map file such as depth, velocity and depth are generated in RAS Mapper. Inundation area is calculated in Arc-GIS. Inundated areas for various cases used in this report are tabulated in Table 6-6, 6-7 and 6-8, as per following details:

Table 6.6: Inundated areas when all the gates are working for different initial reservoir levels

Table 6-6: Inundated Areas for different initial reservoir levels and availability pf different number of gates

Table 6-7: Inundated Areas under dam break in overtopping for two different initial reservoir levels

Critical flood inundation maps for initial reservoir level 830 under all gates are in working condition and for dam break are shown in Figure 6-16 and 6-17. It may be seen from the two graphs that in case of dam break the submerged area is increased to 2090 km^2 from 1443 km^2 i.e. an increase of 45%.

Table 6-8 Inundated areas when all the gates are working for different initial reservoir levels

Initial reservoir level	810m	815m	820m	825m	826m
Total inundated area in Downstream of Tehri Dam in Km^2	931	1074	1220	1374	1386
Initial reservoir level	827m	828m	829m	830m	
Total inundated area in Downstream of Tehri Dam in Km^2	1397	1410	1428	1443	

Table 6-9 Inundated Areas for different initial reservoir levels and availability pf different number of gates

Initial reservoir level 820m					
Availability of gates	4	3	2	1	0
Inundated area in km^2	1140	1105	1030	916	824
Initial reservoir level 825m					
Availability of gates	4	3	2	1	0
Inundated area in km^2	1307	1240	1219	1213	1225
Initial reservoir level 830m					
Availability of gates	4	3	2	1	0
Inundated area in km^2	1418	1440	1545	1598	1620

Table 6-10 Inundated Areas under dam break in overtopping for two different initial reservoir levels

Dam breaks in overtopping	
Initial reservoir level	Inundated area in km^2
825	2023
830	2090

Depth **Velocity**

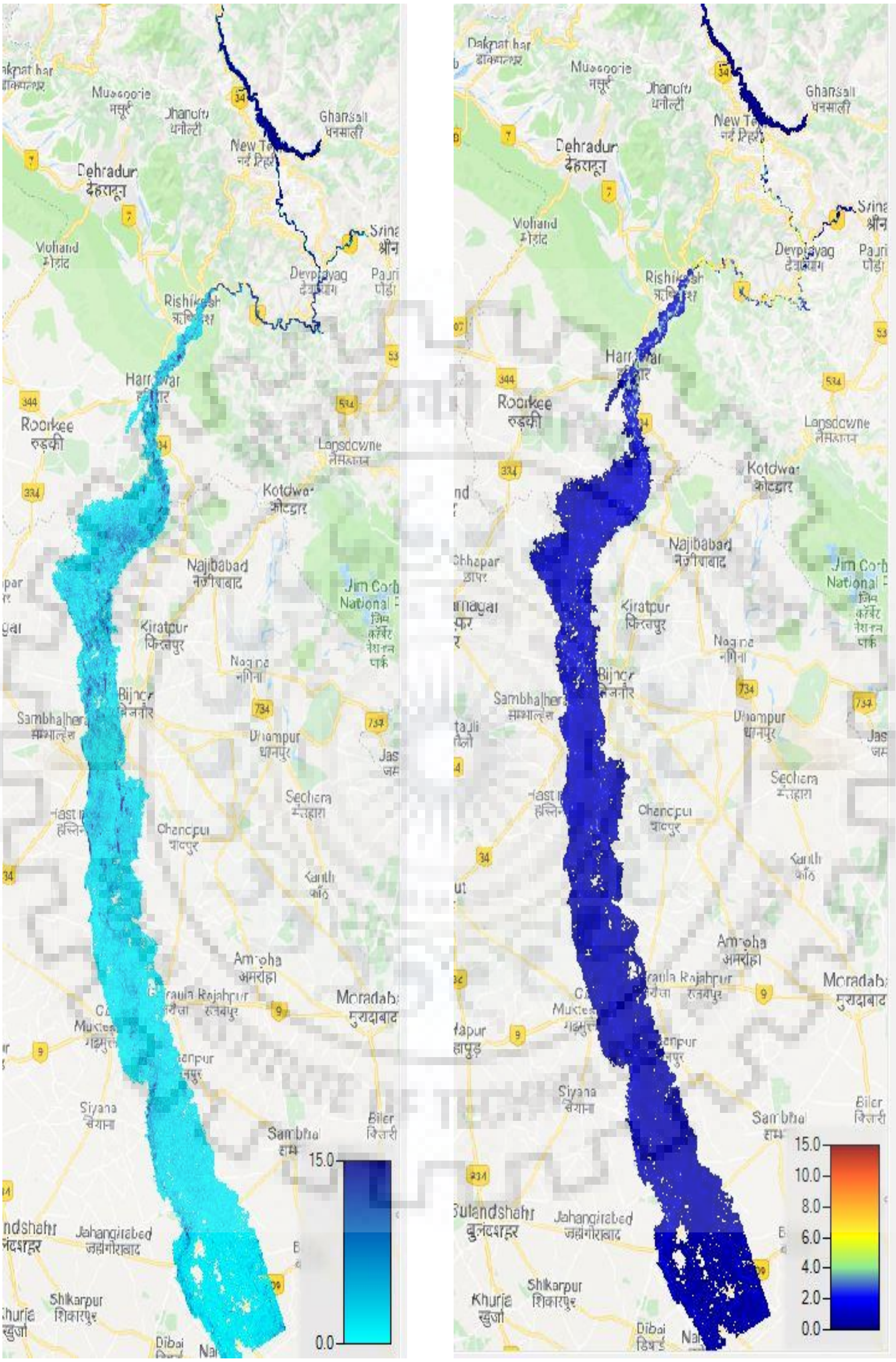

Figure 6-16 Flood inundation map when initial reservoir level is at 830 m and all gates of spillway are open submerged area 1443 km^2

Depth **Velocity**

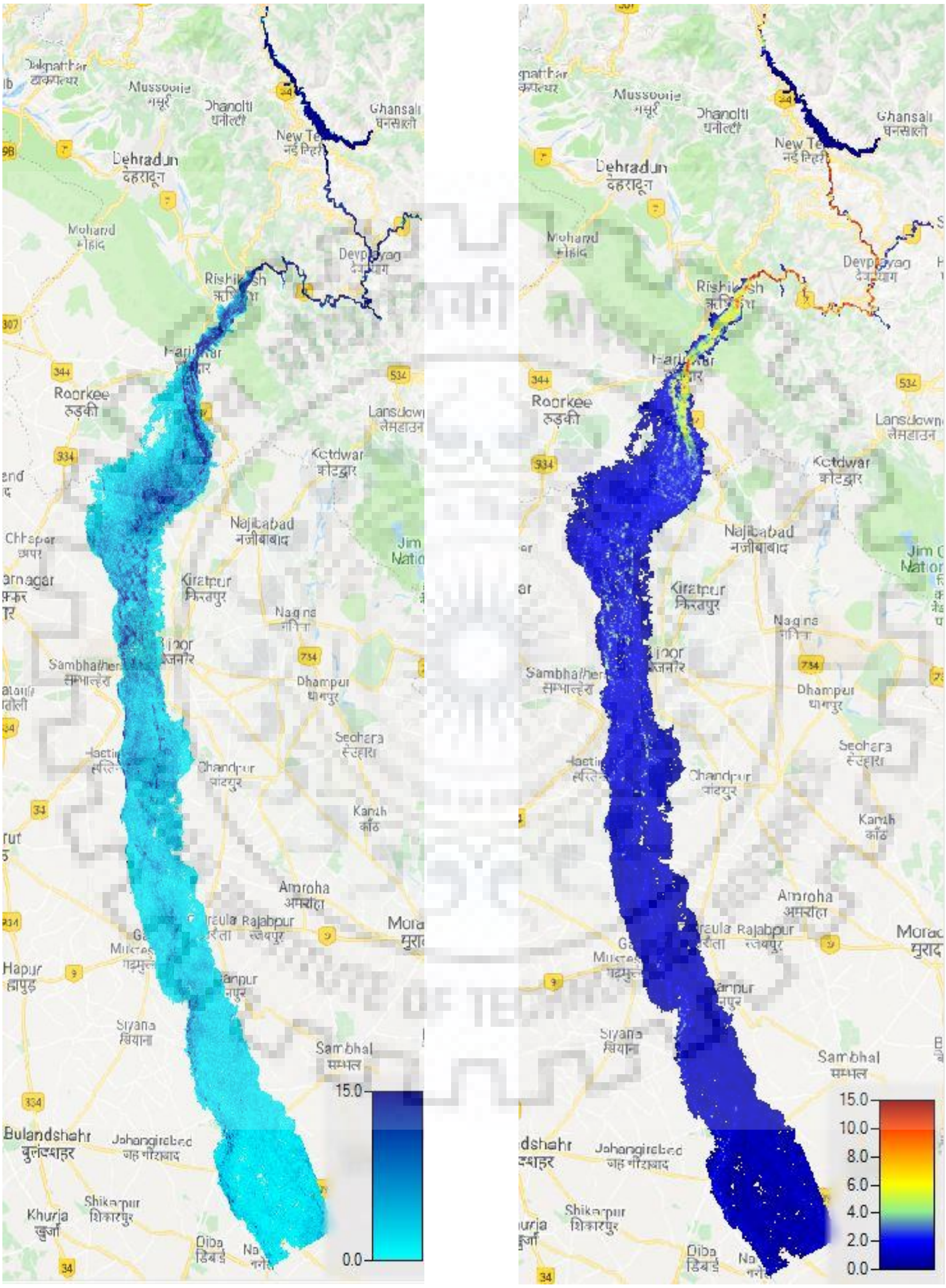

Figure 6-17 Flood inundation map when dam breaks in overtopping submerged area 2090 km^2

CHAPTER 7

CONCLUSIONS, LIMITATIONS AND SCOPE OF FUTURE WORK

7.1 GENERAL

In the present study, risk assessment analysis of Tehri dam has been carried out. Tehri dam has five gated spillways and two ungated spillways. Mechanical and electrical failure of gates is possible and hence risk assessment analysis is done by performing level pool routing for different gate conditions and initial reservoir levels of Tehri dam in HEC-RAS model. Flood risk mapping of the downstream areas under different scenarios has also been attempted. The conclusions drawn from the study, limitations of the study and scope for the future work are presented in this chapter:

7.2 CONCLUSIONS

The following conclusions can be drawn from the study:

1. Major failure mode for Tehri dam is overtopping, as the dam is an earthen rock-fill dam.
2. The Tehri Dam is able to pass the probable maximum flood (PMF) under proper functioning of all of its gates, even when the initial reservoir level reaches up to maximum operating level of 830 m.
3. With initial level reservoir level as 810 or less, the dam is able to accommodate the entire PMF
4. With initial level as 820 m or more, the dam may face overtopping of flood water, if some of the gates are not working.
5. The probability of PMF, initial reservoir levels and probability of the non-functioning of the gates and their combined probabilities have been worked out in the thesis and are documented in Table 5.4.
6. Maximum water levels reached in the reservoir during the occurrence of PMF for various initial reservoir levels and different gate conditions are documented in Table 6.4.

7. The flood inundation area works out to be 1443 km^2 under extreme condition of PMF occurring at reservoir level of 830m. In case of dam break flood, this area works out to be 2090 km^2.
8. The probabilities of dam failure due to overtopping are close to Nil.

7.3 LIMITATIONS OF THE STUDY

1. In this study, HEC-RAS is used for the analysis which does not have the provision for shaft spillway. However, the Tehri dam consists of three gates of chute spillways, 2 gates of left bank shaft spillways, two ungated right bank shaft spillway. HEC-RAS has the provision of chute spillway only. In this study, the flow through the left and right bank shaft spillway in the Tehri dam is simulated by using the chute spillway option in HEC-RAS which may not be completely accurate, but still gives satisfactory results.
2. For channel routing single value of Manning's roughness coefficient has been taken for channel and flood plain and also the same value for hilly and flatter terrains. In reality Manning's value will be different for channel and flood plain for hilly and plain terrains. Different values of Manning's roughness coefficients will result in different flood inundation maps.
3. The channels cross sections have been developed using the ASTER DEM 30 M data and these channel cross sections are very approximate.

7.4 FUTURE SCOPE OF WORK

1. The results of flood inundation mapping need to be refined further for different values of Manning's roughness coefficients and different DEM.
2. At few locations, the actual river cross sections should be used.
3. The results presented in the thesis are just indicative. For precise computation of inundated areas, the actual data of the longitudinal profile and the cross sections at close interval will be needed.

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Annexure 1

FLOOD ROUTING THEORY

Reservoir routing:

In reservoir routing, the effect of flood wave entering a reservoir is studied. Knowing the volume-elevation characteristics of the reservoir and outflow-elevation relationship for the spillways and other outlet structures in the reservoir, the effect of flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time.

Channel routing:

In channel routing, the change in the shape of hydrograph as it travels down a channel is studied. By considering a channel reach and input hydrograph at the upstream end, this form of routing aims to predict the flood hydrograph at various sections of the reach. Information on the flood peak attenuation and duration of high water levels obtained by channel routing is of utmost importance in flood forecasting operations and flood-protections works.

The routing methods can be broadly classified into two categories as:

a) Hydrologic routing
b) Hydraulic routing

Hydrologic routing method:

Hydrologic routing methods employ essentially the equation of continuity. The equation of continuity used in all hydrologic routing as the primary equation states that the difference between the inflow and outflow rate is equal to the rate of change of storage, i.e.

$$\frac{dS}{dt} = I(t) - Q(t) - - - - - - - - - - - - 3.1$$

Where *I(t)* = inflow rate, *Q(t)* = outflow rate and *S* = storage

Hydrologic routing method is used for both reservoir routing and channel routing and its brief explanation is shown in subsequent sections:

a) Hydrologic Reservoir routing method

There are a variety of methods available for routing of floods through reservoir all of them use continuity equation (3.1) but in rearranged manners. As the horizontal water surface is assumed in the reservoir, the reservoir routing is also known as Level Pool Routing. Two commonly used semi graphical and a numerical methods are described below.

Modified Pul's method:

Equation (3.1) is rearranged as

$$\frac{(I_1+I_2)}{2}\Delta t + \left(S_1 - \frac{O_1\Delta t}{2}\right) = (S_2 + \frac{O_2\Delta t}{2})\text{-----------------------------}3.2$$

At the starting of flood routing, the initial storage and outflow discharges are known. In equation (3.2) all the terms in the left-hand side are known at the beginning a time step Δt. Hence, the value of function ($S_2+Q_2\Delta t/2$) at the end of the time step is calculated by equation (3.2). Since the relation S=S(h) and Q=Q (h) are known, $(S+Q\Delta t/2)_2$ will enable one to determine the reservoir elevation and hence the discharge at the end of time step. The procedure is repeated to cover the full inflow hydrograph.

Goodrich method:

Another popular method of hydrologic reservoir routing, known as Goodrich method utilizes equation (3.1) rearranged as

$$(I_1 + I_2) + (\frac{2S_1}{\Delta t} - Q_1) = (\frac{2S_2}{\Delta t} - Q_2) \quad \text{--------------------------------}3.3$$

For a given time- step, the left-hand side of equation 3.3 is known and the term $(\frac{2S_1}{\Delta t} + Q)_2$ is determined by using equation (3.3). From the known storage-elevation-discharge data, the function $(\frac{2S_1}{\Delta t} + Q)_2$ is established as a function of elevation. Hence, the discharge, elevation and storage at the end of the time step are obtained.

The Pul's method and Goodrich method of level pool routing are essentially semi graphical methods. While they can be used for writing programs for use in a computer, a more efficient computation procedure can be achieved by use of any of the Runge-Kutta methods. The standard fourth-order Runge-Kutta method is most accurate one.

b) Hydrologic channel routing method:

In reservoir routing presented in previous sections, the storage was unique function of the outflow discharge, S = f(Q). However, in channel routing the storage is function of both outflow and inflow discharges and hence different routing method is needed. The flow in a river during a flood belongs to category of gradually varied unsteady flow. The water surface in a channel reach not only not parallel to channel bottom but also varies with time. Considering a channel reach having a flood flow, the total volume in storage can be considered under two categories as:

Prism storage:

It is the volume that would exist if the uniform flow occurred at the downstream depth, i.e. the volume formed by an imaginary plane parallel to the channel bottom drawn at outflow section water surface.

Wedge storage:

It is the wedge like volume formed between the actual water surface profile and the top surface of the prism storage.

At a fixed depth at a downstream section of river reach, the prism storage is constant while the wedge storage changes from a positive value at an advancing flood to a negative value during a receding flood. The prism storage S_p is similar to a reservoir and can be expressed as function of the outflow discharge, S_p = f(Q). The wedge storage can be accounted for by expressing it as S_w = f(I). The total storage in the channel reach can then be expressed as

$$S = K(xI^m + (1-x)Q^m) \text{--} 3.4$$

Where k and x are coefficients and m= a constant exponent. It has been found that the value of m varies from 0.6 for rectangular channels to a value of about 1.0 for natural channels.

Muskingum Equation

Using m=1.0, equation (3.4) reduces to a linear relationship for Sin terms of I and Q as

$$S = K(xI + (1-x)Q) - - - - - - - - - - - - 3.5$$

And this relationship is known as the Muskingum Equation. In this, parameter x is known as weighting factor and takes a value between 0 and 0.5. When x=0, obviously the storage is function of discharge only and equation 3.5 reduces to

$$S = KQ -------------------3.6$$

Such a storage is known as linear storage or linear reservoir. When x=0.5 both the inflow and outflow are equally important in determining the storage.

The coefficient K is known as storage time constant and has the dimension of time. It is approximately equal to the time of travel of a flood wave through the channel reach.

In this thesis work, modified Pul's method is used for performing level pool routing for finding outflow discharge from spillway and maximum water surface elevation in reservoir.

For a given channel reach by selecting a routing interval Δt and using the Muskingum equation, the change in storage is

$$S_2 - S_1 = K\big(x(I_2 - I_1) + (1 - x)(Q_2 - Q_1)\big) ----- 3.7$$

Where suffix 1 and 2 refer to conditions before and after the time interval Δt. The continuity equation for the reach is

$$S_2 - S_1 = \frac{(I_2 + I_1)\Delta t}{2} - \frac{(Q_2 + Q_1)\Delta t}{2} ---------- 3.8$$

From Equation (3.7) and (3.8), Q_2 is evaluated as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1 ---------------- 3.9$$

Where $$C_0 = \frac{-Kx+0.5\Delta t}{K-Kx+0.5\Delta t} ------------------3.9a$$

$$C_1 = \frac{Kx+0.5\Delta t}{K-Kx+0.5\Delta t} ------------------3.9b$$

$$C_2 = \frac{K-Kx+0.5\Delta t}{K-Kx+0.5\Delta t} ------------------3.9c$$

Note that $C_0+C_1+C_2=1.0$ can be written in a general form for the nth time step as

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} ------------3.9A$$

Equation (3.9) is known as Muskingum Routing Equation and provides a simple linear equation for channel routing. It has been found that for best results the routing interval Δt should be so chosen that K>Δt>2Kx. If Δt <2Kx, the coefficient C_0 will be negative. Generally, negative values of coefficients are avoided by choosing appropriate value of Δt.

Hydraulic method of flood routing

The hydraulic method of flood routing is essentially a solution of the basic St. Venant Equations (3.10).

$$\frac{\partial y}{\partial x}+\frac{V}{g}\frac{\partial V}{\partial x}+\frac{1}{g}\frac{\partial V}{\partial t}=S_0-S_f ------------ 3.10$$

Where V= velocity of flow at any section, S_0 = channel bed slope and S_f = slope of energy line.

These equation is simultaneous, quasi-linear, first-order, partial differential equation of hyperbolic type and is not amenable to general analytical solutions. Only for highly simplified cases one can obtain the analytical solution of these equations. The development of modern, high-speed digital computers during the past two decades has given rise to the evolution of many sophisticated numerical techniques. The various numerical methods for solving Saint Venant equations can be broadly classified into two categories:

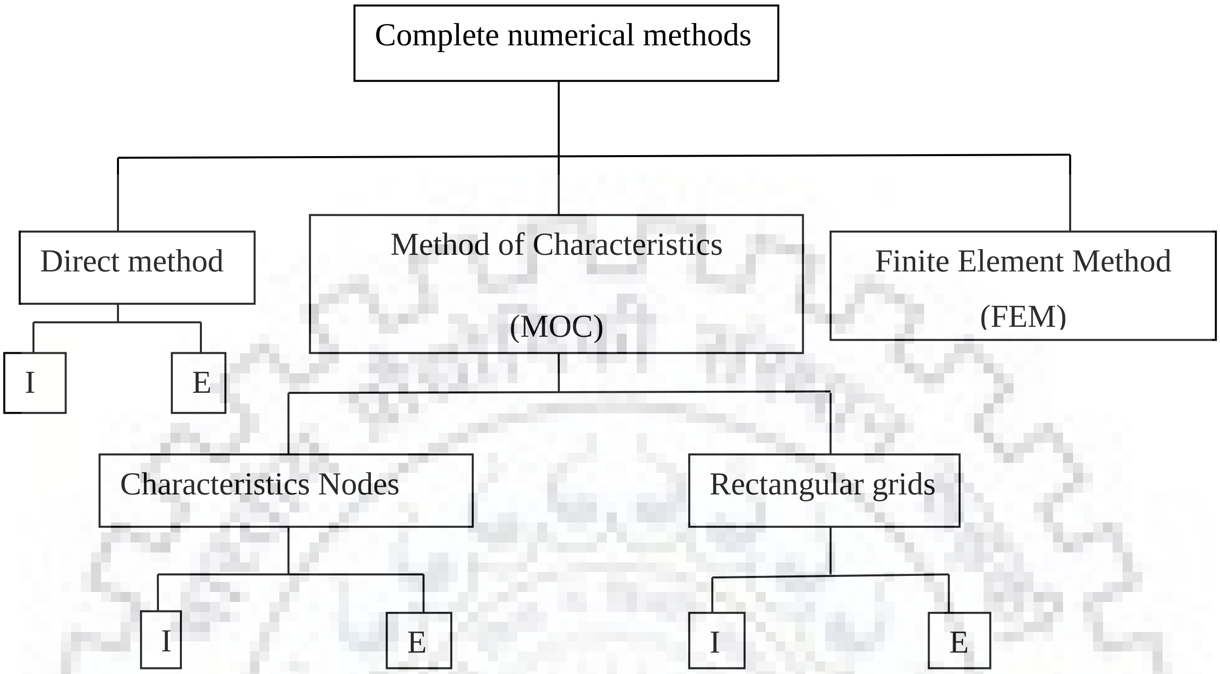
a) Approximate methods
b) Complete numerical methods

Approximate methods

These are based on the equation of continuity only or on a drastically curtailed equation of motion. The hydrological method of storage routing and Muskingum channel routing discussed earlier belongs to this category. Other methods in this category are diffusion analogy and kinematic wave models.

Complete Numerical Methods

These are the essence of the hydraulic method of routing and classified into many categories as mentioned below:



I = Implicit method, E = Explicit method

In the direct method, the partial derivatives are replaced by finite differences and the resulting algebraic equations are than solved. In the method of characteristics (MOC), St. Venant equations are converted into pair of ordinary differential equations (i.e. characteristics forms) and then solved by finite difference techniques. In the finite element method (FEM) the system is divided into a number of elements and partial differential equations are integrated at the nodal point of elements.

The numerical schemes are further classified into explicit and implicit methods. In the explicit method the algebraic equations are linear and dependent variables are extracted explicitly at the end of each time step. In the implicit method, the dependent variables occur implicitly and the equation are non-linear. Each of these two methods have a host of finite-differencing schemes to choose from.

Annexure II

DETAILS OF 2-D FLOW AREA AS PER TECHNICAL MANUAL OF HEC-RAS

HEC-RAS 2D flow modeling can be used in a variety of different situations:

i. Detailed 2D channel and floodplain modeling
ii. Combined 1D channel flow with 2D floodplain flow areas
iii. Combined 1D channel and overbank flow with 2D flow areas behind levees
iv. Simplified to detailed dam failure (i.e., dam breach) analyses
v. Simplified to detailed levee failure (i.e., levee breach) analyses
vi. 1D flow that suddenly expands laterally into the floodplain overbank area
vii. Flow outside of well-defined single channel
viii. Interconnected or braided streams, meanders, loops, and
ix. Alluvial fans and estuaries etc.

a) Methods used for solving 2-D flow area in HEC-RAS

HEC-RAS provides two methods for computing the flow field in a 2D mesh

i. 2D Diffusive Wave Computational Method
ii. 2D Saint Venant Full Momentum Computational Method

2D Diffusion Wave Computational Method:

The 2D Diffusion Wave computational method is the default solver and allows the computations to run faster and with greater stability. Most 2D modeling situations, such as flood modelling, can be accurately modeled using this solver, where inertial forces tend to dominate frictional and other forces.

The Diffusion Wave computational method can be used in the following situations:

i. Flow is mainly driven by gravity and friction
ii. Fluid acceleration is monotonic and smooth (i.e., no waves)
iii. Compute rough global estimates (i.e., flood extents)
iv. Assess interior flooding (i.e., levee breach)
v. Quick estimate for using the Full Momentum computational method

2D Saint Venant Full Momentum Computational Method:

The 2D Full Momentum computational method, often referred to as the Saint Venant equations for shallow flow, can account for turbulence and Coriolis effects, making it applicable to a wider set of conditions. However, solving the 2D Saint Venant flow equations requires more computational power and thereby results in longer run times. In addition, the 2D Saint Venant flow equations can become numerically unstable in regions of the 2D mesh where the water surface profile or flow direction is changing rapidly. To avoid an unstable model, a finer mesh and a corresponding smaller time step will need to be used.

The Full Momentum computational method should be used in the following situations:

i. Dynamic flood waves (i.e., dam failure, rapid rise and fall).
ii. Sudden expansion or contraction of flow with high velocity changes.
iii. Detailed flow solutions around hydraulic structures and obstacles (i.e., bridge openings, piers and abutments).
iv. Detailed mixed flow regime (i.e., hydraulic jumps, critical flow, etc.).
v. Wave propagation (i.e., waves reflecting off walls and objects).
vi. Tidal boundary conditions (i.e., upstream wave propagation).
vii. Super elevation around river bends.

Algorithm used by HEC-RAS to solved 2-D flow area

Both the 2D Diffusion Wave and 2D Saint Venant solvers use an Implicit Finite Volume solution algorithm. The implicit solution method allows for larger computational time steps than explicit solution methods. In addition, the finite volume method provides a greater degree of stability and robustness over traditional finite difference and finite element methodologies.

The 2D computational process is as follows:

Starting depth

↓

Movement of water between cells calculated based on starting depth and cell face rating curve

New cell volume calculated based on flux over computational time

↓

Depth in each cell extracted from volume-depth rating curve

Assumption used in 2-D flow area modelling in HEC-RAS

i. Vertical fluid motion is negligible
ii. Velocity is vertically averaged at the cell center (depth averaged flow)
iii. Energy head is computed at the cell center
iv. Manning's roughness assigned on cell face using roughness value at cell face center
v. Manning's roughness assumed constant across each cell face, although each cell face can have its own value
vi. Rain on grid is applied uniformly to all cells of the 2D flow area
vii. Rainfall initial abstraction and other losses need to be accounted for prior to assigning precipitation data
viii. At least one external boundary condition must exist on the 2D mesh
ix. Time step selection should consider cell size and wave speed.

ANNEXURE III

FLOOD ROUTING RESULTS WHEN INITIAL RESERVOIR LEVEL IS AT 820 M AND ALL GATES OF SPILLWAY ARE WORKING

Table 8-1 Flood routing results when initial reservoir level is at 820 m and all gates of spillway are working

Time (in hrs.)	Inflow in reservoir(in Cumecs)	Outflow from reservoir (in Cumecs)	Water surface elevation in reservoir In meter
1	270	0	820.00
2	270	853	820.19
3	270	848	820.17
4	270	843	820.15
5	270	838	820.13
6	270	833	820.11
7	270	828	820.09
8	297	823	820.07
9	323	819	820.05
10	350	815	820.04
11	433	812	820.03
12	517	811	820.02
13	600	811	820.02
14	690	814	820.03
15	780	818	820.05
16	870	823	820.07
17	980	831	820.10
18	1090	840	820.14
19	1200	852	820.19
20	1333	865	820.24
21	1467	881	820.30
22	1600	899	820.37
23	1783	920	820.45
24	1967	944	820.54
25	2150	972	820.65
26	2367	1003	820.76
27	2593	1039	820.90
28	2800	1079	821.04
29	3100	1123	821.20
30	3400	1174	821.38
31	3700	1231	821.58
32	4000	1294	821.80

33	4300	1364	822.04
34	4600	1440	822.29
35	4933	1523	822.57
36	5600	1616	822.87
37	6400	1726	823.21
38	7200	1856	823.61
39	8000	2006	824.06
40	9433	2181	824.57
41	10867	2395	825.18
42	12300	2649	825.87
43	12967	2934	826.63
44	13633	3249	827.44
45	14300	3585	828.27
46	14600	3941	829.13
47	14900	4308	829.98
48	15200	4648	830.75
49	15313	5119	831.49
50	15427	5595	832.21
51	15540	6095	832.91
52	15410	6606	833.58
53	15280	7114	834.22
54	15150	7612	834.82
55	14733	8094	835.38
56	14316	8542	835.88
57	13900	8953	836.34
58	13167	9321	836.73
59	12433	9630	837.06
60	11700	9882	837.32
61	10967	10080	837.53
62	10233	10224	837.68
63	9500	10317	837.77
64	8900	10360	837.81
65	8300	10362	837.82
66	7700	10325	837.78
67	7267	10256	837.71
68	6833	10163	837.61
69	6400	10049	837.50
70	5933	9914	837.36
71	5467	9758	837.19
72	5000	9583	837.01
73	4733	9393	836.81
74	4467	9199	836.60
75	4200	9002	836.39
76	3900	8800	836.17

77	3600	8592	835.94
78	3300	8381	835.70
79	3100	8166	835.46
80	2900	7953	835.22
81	2700	7743	834.97
82	2533	7535	834.73
83	2367	7331	834.48
84	2200	7130	834.24
85	2067	6934	834.00
86	1933	6742	833.76
87	1800	6515	833.52
88	1700	6377	833.28
89	1600	6201	833.05
90	1500	6032	832.82
91	1433	5868	832.59
92	1367	5710	832.37
93	1300	5560	832.15
94	1233	5415	831.94
95	1167	5276	831.73
96	1100	5143	831.53
97	1033	5016	831.32
98	967	4894	831.12
99	900	4778	830.92
100	867	4668	830.73
101	833	4565	830.54
102	800	4471	830.35
103	750	4386	830.16
104	700	4294	829.95
105	650	4204	829.74
106	633	4115	829.53
107	617	4029	829.33
108	600	3945	829.13
109	550	3863	828.94
110	500	3783	828.75
111	450	3703	828.55
112	433	3625	828.37
113	417	3549	828.18
114	400	3475	828.00
115	367	3403	827.82
116	333	3332	827.64
117	300	3263	827.47
118	290	3194	827.30
119	270	3128	827.13
120	270	3064	826.96

121	270	2980	826.80
122	270	2940	826.64
123	270	2881	826.49
124	270	2825	826.34
125	270	2769	826.19
126	270	2716	826.05
127	270	2664	825.91
128	270	2614	825.78
129	270	2565	825.64
130	270	2517	825.51
131	270	2471	825.39
132	270	2427	825.26
133	270	2383	825.14
134	270	2341	825.02
135	270	2300	824.91
136	270	2260	824.80
137	270	2222	824.69
138	270	2184	824.58
139	270	2148	824.47
140	270	2112	824.37
141	270	2078	824.27
142	270	2044	824.17
143	270	2011	824.07
144	270	1979	823.98
145	270	1948	823.89
146	270	1918	823.80
147	270	1889	823.71
148	270	1860	823.62
149	270	1833	823.54
150	270	1805	823.45
151	270	1779	823.37
152	270	1753	823.29
153	270	1728	823.22
154	270	1704	823.14
155	270	1680	823.07
156	270	1657	822.99
157	270	1634	822.92
158	270	1612	822.85
159	270	1590	822.78
160	270	1569	822.71
161	270	1549	822.65
162	270	1529	822.58
163	270	1509	822.52
164	270	1490	822.46

165	270	1472	822.40
166	270	1453	822.34
167	270	1436	822.28
168	270	1418	822.22
169	270	1401	822.16