DAM BREAK ANALYSIS OF NALGAD HYDROPOWER PROJECT IN NEPAL USING HEC-RAS

A DISSERTATION

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By

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CANDIDATE'S DECLARATION

I hereby certify that the works which is being presented in this dissertation entitled "DAM BREAK ANALYSIS OF NALGAD HYDRO POWER PROJECT" is in partial fulfillment of the requirement for the award of the degree of Master of Technology in Water Resources Development (Civil) and submitted to the Department of Water Resources Development and Management (WRD&M), Indian Institute of Technology Roorkee. This is an authentic record of my own work carried out during the period July, 2018 to May, 2019 under the supervision and guidance of Mr. J. P. Patra, Scientist-"C", National Institute of Hydrology, Roorkee, India and Dr. S. K. Mishra, Professor, WRD&M, IIT Roorkee, Uttrakhand, India.

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

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CERTIFICATION

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

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ABSTRACT

This study mainly focuses on dam breach analysis of Nalgad Hydro-Power dam. Nalgad Hydro-Power Dam is a concrete faced rock-fill dam with clay core of 200 m height and 546 m length and designed to generate hydroelectric power, which has crucial part in development of Nepal. The results of dam breach analysis are useful in minimizing the loss of life and properties due to likely dam break flood in the event of failure. In Nepal, there is no law existing and practice of studying dam break floods, and thus, this study is perhaps the first for Nepal. This work focusses on predicting the breach outflow hydrograph of Nalgad Hydro dam and its routing through downstream valley employing a computer software for determination of consequences of dam failure. The computer program of U.S. Army Corps of Engineers HEC-RAS 5.05 software models such floods in two ways namely 1-D and 2-D. In 1-D dam breach flood analysis, the essential parameters involved in reservoir and river routing techniques were estimated manually externally. Dam breach parameters include time of failure of breach, side slope of breach, bottom breach width, manning roughness coefficient, shape of breach and boundary condition. The unsteady hydraulics of the dam breach due to overtopping failure mode was modeled using 1-D approach. The model results show a peak flow of 140350.40 m^3/s at the dam site for overtopping mode of failure and it is 39.84 times greater than PMF of 3523 m^3/s. The attenuation of discharge from head to tail is 27.07% and the peak discharge reaches the tail end, at 47.66 km, after 1.1 hour of attaining peak discharge at distance 0.51 km d/s of dam.

To route the downstream valley both hydrologic and hydraulic routings were undertaken. Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end whereas hydraulic routing employs the continuity equation and both energy and momentum balances to calculate open channel flow profiles. 1-D dam break scenario is further compared with two other 1-D scenarios namely natural flow, i.e. no dam case and without dam break case. All the results computed by dam break case is higher than other two cases but time of arrival of peak discharge. The arrival time of peak natural flood computed by HEC-RAS 1-D model is 6 hour, 6.10 hour, 6.30 hour, 6.4 hour, 6.4 hour, 7.30 hour, 8.40 hour, 9.20 hour, 10.22 hour and 11 hour at the selected distance 0.51 km, 1..98 km, 11.31 km, 12.84 km, 14.37 km, 22.91 km, 31.63 km, 35.59 km,

43.12 km and 47.66 km d/s respectively. Similarly, the arrival time of dam break peak flood is 7.2 hour, 7.2 hour, 7.3 hour, 7.4 hour, 7.4 hour, 7.5 hour, 8 hour, 8.10, hour, 8.20 hour 8.30 at the same selected distance. This shows that the arrival time computed in natural flood case is earlier than the dam break flood case. up to distance 22.91 km (at station no. 24875.10). After this distance, the arrival time of dam break flood is earlier than the natural flood case.

Flood plain mapping for the downstream reach of Nalgad Hydro Dam was performed using maximum water surface elevations on the XS cut lines, within the limits of the bounding polygon and flow affected settlement area is found with help of remote sensing and GIS. The sensitivity analysis was performed using time of failure, bottom breach width, side slope of beach and relative effect of one parameter on the resulting peak discharge. The model results show that breach formation time is more sensitive than bottom breach width and side slope of breach.In 2-D analysis, the peak hydrograph of the first station from dam axis computed by 1-D is given as u/s boundary condition and friction slope of tail end reach is given as d/s boundary condition and then model is allowed to simulate unsteady flow. The computed peak discharge varies from 140001 m^3/s to 102600 m^3/s, i.e. the attenuation of discharge from head to tail is 26.71% and the peak discharge reaches the tail end, i.e. at 47.66 km after 2 hours of attaining peak discharge at distance 0.51 km d/s.

When river flows in steep slope up to distance 14.36 km, the flow affected settlement area computed by 1-D is slightly higher than 2-D and also water depth is decreasing gradually but after 14.36 km, when river meets flatter slope, the flow affected settlement area computed by 2-D is higher than 1-D. The total flood affected settlement area computed by 1-D is 3.16 km^2, and by 2-D it is 4.3 km^2 which is 35.13 % higher than computed by 1-D. The depth of water computed by 2-D is abruptly increasing in flatter slope at distance 22.91 km and 47.66 km from dam axis which is not reliable. Hence, for steep reaches 1-D model performs better than 2-D. Steep streams are gravity driven and have small overbanks. It means rivers and flood plains where the dominant flow directions and forces follow the general river flow path. Such type of nature of river is obtained in hilly zone and in such cases 1-D modeling is preferable.

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LIST OF SYMBOLS AND ABBREVIATIONS

1-D	One dimensional.
2-D	Two dimensional.
A	The active flow area, m^2
A_o	The inactive storage area, m^2
B	Breach Width
B_W	Bottom breach width.
B_t	Top breach width.
B_{avg}	Average Breach Width..
BEED	Breach Erosion of Embankment Dams Model.
C_b	Offset factor as a function of reservoir storage in Von Thun and Gillette equation, m
C_r	Courant Number
Cumecs	Cubic meter per second
D/S	Down Stream
EG	Energy Gradient
F_r	Froude Number
Ft	feet
g	Gravitational acceleration
h	Water surface elevation
h_a	Hectare
h_b	Height of breach, m
h_d	Height of dam
h_w	Hydraulic depth of water at dam at failure above breach bottom, m
H	Breach Height in m
HEC-RAS	Hydrologic Engineering Center- River Analysis System.
K_c	The core wall correction factor (0.6 if dam contains a core wall; 1.0
K_o	Froehlich Constant, 1.4 for overtopping, 1.0 for piping.
M	Meter.
MCM	Million Cubic Meter
MWL	Maximum Water Level
N	Manning’s roughness coefficient

NWS	National Weather Service of United States
PMF	Probable Maximum Flood, m^3/sec.
Q	Discharge, m^3/sec.
Q_p	Peak Outflow, m^3 /sec.
q	Lateral Out flow, m^3 /sec
S	Storage, m^3
S_c	The expansion/contraction slope terms in Saint-Venant's Equation
S_f	The channel/flood plain boundary friction slope.
t	The time, hours
t_f	Time of failure (breach formation time), hours.
T_r	Time of rise of the hydrograph to be routed, seconds.
u/s	Upstream
V_{er}	Volume of eroded embankment materials, m^3
V_{out}	Volume of water discharged through breach, m^3
V_w	Flood wave speed, m/s.
WSEL	Water Surface Elevation, m.
X	The longitudinal distance along the waterway, m
Z	Breach side slope factor.
Dx	Distance between cross-sections.
Dt	Computational time step, seconds.

CHAPTER-1

INTRODUCTION

1.1 Background

For optimum utilization of water, there must be balance between demand and supply of water in every water resource sector. This is achieved only through construction of water storage projects. Hence, dams are hydraulic structures constructed to store the water in the u/s of reservoir formed to release water as per necessities. The objective of construction of dam may be different and different like for domestic and industrial water supply, irrigation, hydro-electric power, navigation, for development fish & wild life, Recreation and so on. The main function of dam is to collect & store water in rainy season and supply of this stored water in dry season as per necessities. The functions of dams are not only to store water but also to control flood. Hence, dams play very vital role to the society in sense of providing water during lean period but dams are also very dangers to the society downstream river reach in sense of flood wave generated after dam break. When failure of dam occurs, loss of properties is common but loss of human life may vary in a very rapid way as per the extent of inundation area, size of community at risk and how much warning time is available (Bureau of Reclamation, 1998).As per International Commission on Large Dams (ICOLD) there are 36000 large dams are in the world and stills others large dams are under construction. In Nepal, only one major is in existence and Nalgad Hydro-Dam is only one large dam which is being constructed in very near future.

From different and different dam failure case studies, it is seen that there is very low probability of dam failure but loss associated with this dam failure is very high. Due to human interrupted activities in catchment, surface runoff picks up maximum silt from catchment area and ultimately, silts are deposited in the reservoirs. Due to which, capacity and useful life of reservoir is decreasing year to year. So, when high flood enters in the reservoir, no much space in reservoir remains to accommodate the flood due to which excess flood overtops the dam ultimately leading to the failure of dam that result huge mass of flood wave passes downstream of dam causing loss of life and properties in the vicinity of downstream river reach. From different case studies, it is seen that 40% of dam failure is due to the inadequate spillway capacity ATALLAH

(2002). Due to lack of proper hydrological investigation technique, it is seen that the most of the older dams are designed without considering hydrological investigation. So, failure rate of such older dams are also high. There are so many reasons for dam failure among of them overtopping due inadequate spillway capacity, seepage, earthquake, Landslide, Construction defect, Use of low grade construction material are major. The comparison of the nature of flood wave after dam break and natural flood is completely different and can't be matched. Dam break flood attains peak flow in a very short interval of time but in same locations natural flood attains the peak flow comparatively loner period of time. The flood wave safely passing through spillway has no such real meaning to potential hazards. It is seen that there is high probability of risk of dam failure in the beginnings of the year (8-9 years) after construction but as the dam becomes older, there is low probability of risk of failure. There are several hydraulic models that have been developed so far to simulate dam breach flood downstream of the dam to identify inundation area so that potential hazards can be minimized. Whenever, failure of dam occurs huge amount of water is released suddenly due to which large flood wave is produced. Depending upon the quantity of water available in the reservoir and the height of the dam, the flood waves have much more tendency to damage the nearby settlement and infrastructure in the downstream reach (Singh, 1996).

There are two main tasks that should be done in the analysis of dam failure. One of them is prediction of outflow hydrograph and the other is routing of that hydrograph in the downstream of river valley so that consequences of dam failure is determined. The flood should be modeled with sufficient details so that both the spatial and temporal progressions of the flood event can be captured so that mitigation of impact of flood can be minimized with fair degree of accuracy. The flood wave generated after dam break is affected by so many factors like rainfall in the catchment and resulting runoff, area of reservoir, characteristics of reservoir and dam, quality of construction material, Earthquake, type of dam and so on. Prediction of breach parameters breach width, breach depth, breach Side slope, breach formation time, mode of failure, condition of flow, progression of breach, use of construction material, type and geometry of dam greatly affect the prediction of outflow hydrographs and these parameters have varied influenced on the peak flow in downstream location. Since, failure of dam results catastrophic disaster to the downstream reaches, dam failure cannot be prevented but losses can be minimized preparing emergency action plan for evacuation of life and properties for critical location after dam break analysis.

History of dam construction shows that the construction of dam dates back to 2900 B.C. It is believed that the first dam constructed in Egypt, Wadi el- Garawi, which is 30 Km from Cairo in the south direction (Singh, 1996**)**. Alicante Dam in Spain is the highest dam in the world for 300 years but in the present moments, The Nurek Dam of Tajikistan is the highest but Chinese Three Gorges Dams are the largest dam in the world. Dams are designed considering low probability of failure but it is seen that dams are frequently fail and dam failure may be either partially or complete. After dam break, huge amount of uncontrolled flood wave generated and this wave behave as catastrophic disaster causing damage to the life and properties in the area of influence. As per Cost (1985**),** major reasons behind failure of dams are overtopping (34%), seepage (28%) and defects in foundation (40%). In the year of 1963, the Vaiont arch (Italy) failed due to the overtopping circumstances and the estimated height of flood wave was 100 m. The result of failure causes death of 2600 people. The death of 14 people and loss of $1 billion property happened after failure of Teton dam located at Idaho US and failure mode was piping in the year 1976. Similarly many more dam failure could be seen in the US in the 1970's, they are Buffalo Creek coal waste dam (West Virginia, 1972), Laurel Run Dam and Sandy Run Dam (Pennsylvania, 1977), and Kelly Barnes Dam (Georgia, 1977). From these data, it is seen that construction of dam is not only major part of the storage project implementation but focus on security of the people and infrastructure downstream of dam is also another major part. So, "dam break analysis and its impact" must be one of the major parts of the report. Considering this fact, after failure of Malpasset dam in France in 1959, the law on dam failure was made and constituted in France in 1968 following this failure. Similarly, other countries also made laws compulsory on dam safety after this event.

Machhu II dam in India failed due to the overtopping mode of failure because of inadequate spillway capacity in 1979 that causes death of 2000 people. Although, many others dams are also failed in India but the law on Dam Safety is made by Indain Government and a committee is formed naming Dam Safety Organization in Central Water Commission (CWC) after failure of this Machhu II dam dated May 1979. The role of this organization is to develop suitable dam safety practices. (CWC-DSO, 1986).

Nepal is highly potential in water resource and the water resource can be developed for hydro-power, water supply and irrigation is direct runoff system. There is only one storage project constructed up to now but there is no provision of dam break analysis and this Nalgad-Hydro Project is going to construction in near future but there is no dam break analysis for this project. Government will realize importance of dam break analysis only after failure of dam but in Nepal, there is only one storage project dam and Government of Nepal has not faced dam break disaster up to now and has not realized the importance of dam break analysis so GoN has not made any law & provision on dam break analysis on dam projects.

The ultimate objective of dam break analysis is to predict what may be the flooding condition and impact of flooding condition in terms of loss of life and properties. The flooding condition refers to that area which comes under the influence of flood said to be inundation area. The flooding condition and its impact can be analyzed by dam break modeling tool that predicts the flood inundation. The dam breach and flood routing models like National Weather Service DAMBRK or FLDWAV model, the NWS Simplified Dam-Break Flood Forecasting Model (SMPDBK), MIKE 21, CCHE FLOOD the HEC-RAS model from the U.S. Army Corps of Engineers & several other models having similar capacity developed after since 1990. HEC-RAS is one of the most powerful computers based mathematical simulation tool that can be modeled for one dimensional and two-dimensional analyses. Modeler uses first HEC-RAS for reservoir routing of inflow hydrograph before breaching the dam by any method of unsteady flood routing using full Saint Venant Equations, unsteady flood routing using full Saint Venant Equations or Diffusion wave and level pool routing. The output of one dimensional analysis after breaching the dam are generally arrival time of peak discharge, volume of peak discharge, Stage of peak discharge, water surface elevation, velocity flow, froud no. etc. in downstream reach where as two dimensional analysis gives the output about water spread area, velocity of flow in two dimension (direction) and also water surface elevation. The results of these software is much more influenced by given input. So, while inputting the data in the software very much care should be given while making assumption and make correlate the data similar historical dam failure.

1.2. Objectives

The purpose of this study is to model dam in Nalgad River of the Karnali river basin to conduct dam break analysis using HEC-RAS (Hydrologic Engineering Center –River Analysis System) model. The specific objectives of the study are:

1. To prepare dam break model setup for Nalgad-Hydro dam using HEC-RAS.
2. To estimate dam breach parameters using various empirical models.
3. To compare flood inundation maps prepared using 1-D and 2-D HEC RAS model.

1.3 Dissertation outline

This dissertation has been categorized in seven chapters organized as follows.

- **Chapter One** deals with the dam, its importance in modern society,
- **Chapter Two** describes the Reviewed literature related to the study.
- **Chapter Three** deals with the Description of Methodologies.
- **Chapter Four** describes about Study Area.
- **Chapter Five** describes the Results and Discussion.
- **Chapter Six** describes the Sensitivity Analysis
- **Chapter Seven** describes the Conclusion and Recommendation

CHAPTER-2
LITURATURE REVIEW

2.1 General description

Since demand of water is increasing day by day in every sector of water resource including water supply, irrigation hydro-power and so on and in the same manner water resource crisis is also increasing day by day. To maintain the balance between these two, it is very necessary to implement multi-purpose water storage project by construction of appropriate dam. Although, Dams are designed by analyzing with different angles with high factor of safety but there is always risk of dam failure due to various reasons. Though, Probability of dam failure is very low but the result of loss life and properties is very high. Whenever dam failures occur, it causes the catastrophic disaster in the vicinity downstream river reach causing huge loss of life and properties. Dam break study and analysis is conducted not only for identifying the potential loss of life and properties but also to study the behavior of generated flood wave in different critical location that falls under inundation flooding so that potential damage can be minimized. The dam break simulation can be done by using the several hydraulic modeling programs developed by different investigators in different time. It is not well known the actual failure mechanism of dam for any type of dam still up to now. In older practice dam breaching and flooding to downstream is done by making the only assumption of dam failure instantaneously and completely (Abinet, 2010). Dam failure may be partial or complete as result of which unexpected uncontrolled huge amount of water flowing downstream (Fread, 1993). As result of release of uncontrolled water downstream, there is high probability of loss of life and properties that forces the dam engineers to conduct the dam break analysis for dam projects so that appropriate mitigation measures can be applied to reduce the effect of flood impact. That is why, it is very necessary to have sufficient details to run the flood model so that spatial and temporal evolutions of the flood event can be captured (FEMA, 2004).

2.2 General procedures for dam break analysis

There are much more physical and mathematical models are developed so far for the analysis of dam breach and simulation of breached flow. The widespread and most commonly used some of the models are: the National Weather Service (NWS) Dam-

Break Flood Forecasting Model (DAMBRK); the U.S. Army Corps of Engineers Hydrologic Engineering Center Flood Hydrograph Package, HEC-5 (Hydrologic Engineering Center, 2015); the NWS Simplified Dam-Break Flood Forecasting Model, SMPDBK (Wetmore and Fread, 1983), MIKE 11 and MIKE 21 developed by Danish Hydraulic Institute etc. All these models are capable to route the breached flood with higher detail than the real breaching phenomenon.

There are two major steps that must be taken in the analysis of dam breach are the estimation of breached out flow hydrograph and routing of that breached outflow hydrographs via downstream reaches. The prediction of outflow hydrograph can be done only after prediction of breached parameters characteristics like location, Bottom width, bottom elevation, shape, Time of failure. Then, routing of inflow and storage before breaching the dam is done and in models like HEC-RAS, this can be done by any of the three methods by unsteady flood routing using full Saint Venant Equations, unsteady flood routing using full Saint Venant Equations or Diffusion wave and level pool routing. In one dimension analysis generally unsteady flood routing using full Saint Venant Equations gives the better out flow hydrograph and in two dimension analysis unsteady flood routing using full Saint Venant Equations or Diffusion wave gives the better out flow hydrograph and models are capable to route the flood downstream valley automatically so that the peak discharge, its arrival time, elevation, velocity etc. can be easily known at downstream valley. The dam break analysis methods are categorized into four methods (Reclamation (1988)).

a. Physically based methods

The erosion model is used for the Prediction and progression of breach formed on dam body causes water to pass through the breach said to be breach outflow. Such type of model is based on the engineering principle of hydraulics, sediment transport and geotechnical concepts.

b. Parametric models

Failure time and final breach geometry are estimated using case study information and then assumption is made for breach growth is that the breach growth occurs linearly depending upon the time for simulation of breach growth using engineering principles of hydraulics, breach out flow is computed.

c. Predictor equations

Different researchers have developed empirical equation from the case study data of historically dam failure for the estimation of peak discharge. Such peak discharge is estimate using such empirical equation and assumption of appropriate and reasonable shape of outflow hydrograph is done.

d. Comparative analysis

Selection breach parameter and outflow hydrograph is done by comparison with the dam having very similar parameters that have been failed in the past, if well documentation of failure is available.

2.3 Dam breaching mechanism

MacDonald & Langridge-Monopolis (1984) had conducted study for breaching mechanism for dam failure. As per their study, it is found that the development of breach is due to either sudden removal of all or a portion of dam or by the erosion of dam material. Breaching of dam may be due to various reasons but reasons for breaching in dam is mainly modelled as overtopping mode and piping mode.

2.3.1 Overtopping mode of failures

Incoming flow in the reservoir is in such a way that the reservoir stores this flow up to pool level even after pool level, flow is incoming continuously and excess flow above pool level is discharged by side & emergency spillway but when excess flow becomes beyond the capacity of spillway due to heavy rainfall in the catchment area, then, pool level in the reservoir increases continuously and then finally overtopping of embankment occurs causing the failure of dam In most of the dam it is seen that the major reason of overtopping mode of failure is due to the inadequate spillway capacity. It is believed that about fifty per cent dam in the world are failed due to overtopping causing most of the loss of life (ICOLD, 1997). As per research of Foster et al (2000), overtopping is responsible for 46 % of embankment dam failures. The possible of reasons behind the overtopping events may be due to inadequate spillway capacity; a situation may come when spillway gate may be partly or fully blocked, increase of dead storage of reservoir and may due to earthquake.

The hydraulic properties of dam crest behave as broad crested when water starts to overtop the dam crest causing the formation of small head cut on the downstream side. As the water continuously flows over the dam, this head cut formed progressed continuously backward towards centre and crest of dam by continuous erosion of dam material. When the head cut starts at the crest of the dam and width of crest gets reduced and dam behave as sharp crested weir and the moment of breach initiation starts when this head cut reaches to the upstream face covering whole crest width causing failure of upstream crest in mass. Since initiation of breach starts from dam crest and reaches to the final stage when the downstream cut reaches the natural river bed level. There may be three cases under which progression of breach may be stopped. The cases are when there is no water in the reservoir or available water in the reservoir is unable to erode the dam or head cut reaches the natural river bed level (Gee, 2009).

2.3.2 Piping mode of failures

When the seepage gradient line does not go below the downstream end and lies in body of dam in such cases water via seepage path emerges from downstream face of dam. The flow under such condition is pressurized and the emerging water generates force on the soil particles such that the soil particles gets displaced from their position at the outlet point and as soil particle displaced, the flow of water increases generating more forces on remaining soil particles and so the erosion started at the outlet point moves progressively back ward towards upstream finally resulting the cavity. It should be noted that during the piping flow the head cutting and sloughing of material go with together. The flow of seepage water through cavity increases, by removing more and more soil particles on the flow path and resulting larger and larger size of cavity. Then, the size of cavity become such that the weight of particles above the cavity become too high causing mass caving incident and the resultant out flow will be very high and behave as catalyst for progressing the breaching process. It should be very important to note that the pressure flow changes into open air weir type flow at this point. After this, remaining process is similar to overtopping type of failure. It should be very important to note that this type of failure does not have any relation with rainfall event and spillway capacity so it can be also known as “sunny day” event.

2.4 Description of breach shape

The main target of this task is the selection of appropriate breach parameter so that shape and size breach, height of breach, Side slope of breach, average width and Bottom width of breach, location of breach and Time of failure can be estimated so that the prediction and verification of outflow hydrograph can be done. As per case studies of many historic dam failures, many researchers have developed the regression equation for the estimation of breach parameter. The regression equation developed by researchers provide the average breach width which is lies in mid breach height except the regression equation developed by MacDonald & Langridge-Monopolis (1984).The average breach width calculated by the these regression equation is projected to bottom and top of the dam with the help of Side slope and dam height. The top width of breach must be less than the length of the crest. The centre of location should be centre line of river and bottom elevation is the bottom of dam i.e. average river bed level of natural channel. In HEC-RAS model, the selection of breach parameter is done by taking the average value of parameter calculated by Froehlich (1995), Froehlich (2008), MacDonald & Langridge-Monopolis (1984), Von Thun and Gillette (1990) & Xu and Zhang (2009). However, following relationship in the table 2.1 is generally followed for the selection of dam breach parameter.

Table 2. 1: Breach Parameter Characteristics for Earthen Dam

Dam Type	Average Breach Width (Bave)	Breach Side slope (H:V)	Failures time (hours)	Agency
Earthen/ Rock fill	(0.5 to 3)XHD	0 to 1	0.5 to 4	USACE 1980
	(1 to 5)X HD	0 to 1	0.1 to 1	FERC
	(2 to 5)X HD	0 to 1 (slightly larger)	0.1 to 1	NWS

Note: HD- Height of Dam
USACE- United States Army corps of Engineering.
FERC-Federal Energy Regulatory Commission
NWS- National Weather Service

Analysis and prediction of dam breach parameter is very difficult task for the selection of appropriate shape but generally approximation of breach shape is done considering the geometric shape like trapezoidal, rectangular, triangular etc. The progression of breach is in “V” shape at the starting phase and top width of the breach is 3-4 times more than depth of breach but start of progression of Bottom width starts only when depth of breach reaches the original river bed level Johnson and Illes (1976).

Sine the documentation on Case study data of dam failure is available in negligible amount so selection of breach parameter on the basis of case study data is not so reliable even though dam to construct and dam which is failed is in similar nature. So, selection of dam failure analysis and method should be physically based models and the for this model of dam breach analysis breach geometries are breach Bottom width, depth, Side slope, time for initiation of breach and breach formation time (Wahl, 1998). Breach initiation and formation time indicates that the dam failure is not generally instantaneously. It means that the dam failure occurs in two stages. Whenever there is breach in the dam at the stage of breach initiation time, the dam is not in failure mode and the breach out flow is in negligible amount. At this stage, the dam may keep it up to last if out flow through breach is checked. Identification of such breach initiation in the dam shows that there is enough warning time available to warn and evacuate the people of the downstream valley if outflow through breach cannot be stopped. When the breach in the dam is at the stage of breach formation time, the outflow rate through breach is so high it cannot be checked and ultimately failure of dam occurs. There is no provision to enter the breach time initiation time in any dam break models and it is very difficult to distinguish between breach initiation and breach formation time even for the highly trained observer. Following are some definitions related to breach geometry.

a. Breach Depth

The distance between the crest of the dam and invert of the breach. Generally, breach invert is taken as the top level of crest minus height of the dam but some researchers refers the reservoir head in place of dam height.

b. Breach Width

The width of opening at the bottom and top of the breach through which outflow discharge passes. Different researchers have developed the different regression equation with suitable Side slope for average breach width which lies in the mid height of breach. This average width can be projected to the bottom and top of breach to calculate bottom and top width with help of height and Side slope of breach. This is key factor in breach geometry which highly affects the peak discharge as well as downstream inundation.

c. Side slope of Breach

This factor governs the shape of breach geometry whether rectangular or trapezoidal. If Side slope factor is taken under consideration the breach geometry is trapezoidal otherwise rectangular. Different researchers have given different suitable Side slope. This factor depends upon the top & Bottom width and depth of breach.

d. Breach initiation time

In a dam, a condition may arise at any moment i.e. when the first flow occurs by overtopping or piping the dam body until breach formation stage starts. The time elapsed under this condition is said to be breach initiation time. It means time duration under which downstream head cut formed progressed up to the crest of the dam.

e. Breach Formation Time

In case of overtopping mode of dam failure, breach formation time is that time which is elapsed in such a way that the downstream portion of dam is eroded away and the developed head cut moves backwards and reaches upstream face via across dam crest and this initiation of breaching on the crest causes until complete breach. When upstream face of crest is eroded, then initiation of breach occurs and flow via dam is also increased. Then, Progression of this breach from upstream to downstream occur in uncontrolled manner and failure of dam occurs.

f. Failure Rate

The failure rate indicates what the nature of breach progression is. It means breach width and depth is increasing either in slowly or rapidly. If slower the breach progression rate, slower the rate of failure and the higher the breach progression rate higher the failure rate. The breach progression rate may be either linear or nonlinear i.e. Sine wave progression. The failure rate depends upon the shape, size, Side slope, and Time of failure, location of breach and pool elevation of reservoir. In HEC-RAS model, there is a tool named as Inline structure inside which there is breach plan data where all the estimated breach parameter should be inputted. The accuracy of outflow hydrograph and area under inundation depends upon the selection of breach parameter. So, prediction and analysis of breach parameter is very important and serious task.

2.5 Predictor Equations for Breach Parameters

Many researchers have developed regression equation for the estimation of dam breach parameter from the case study of historic dam failure data, some of them are out lined here.

Johnson and Illes (1976)

Johnson and Illes (1976) were first who study and provide contribution on the categorization on the shape of failure breach geometry for earth, gravity, and arch dams after study of failure of 300 dams in worldwide. As per their conclusion of analysis on the earthen dam, the shape of breach may changes from **"V"** i.e. triangular shape at the initiation to trapezoidal shape at the final. They have given the relationship of varying breach width with the depth. As per their relationship, breach width may vary from 0.5 h_d to 3 h_d for earth fill dams. Now, it should be noted that h_d is the dam height but as per study conducted by Wahl (1998), breach shape is trapezoidal for earthen dam not varying from triangular to trapezoidal as concluded by Johnson and Illes.

Singh and Scarlatos (1988)

Singh and Snorrason (1982) carried out research on the variation of breach width with depth and corresponding failure time from inception of breach to completion of breach and also depth of water above the crest just before the failure. The research work is done on the case study data of 20 dam failure by plotting the graph width verses height. As per the conclusion of their research, the range of variation of breach width is 2 to 5 times the dam height. In similar manner, failure time varies from 15 minute to 1 hour form initiation of breach to completion of breach respectively. The major interesting part of their research is that they have found depth of water just before the dam is failure is 0.15 m to 0.61 m above the crest.

MacDonald and Langridge-Monopolis (1984)

The research intervention of MacDonald and Langridge-Monopolis (1984) developed the breach formation factor by utilizing the 42 case study dam failure data of earth fill dams, earth fill with clay core and rock fill dam. As per their conclusion "breach formation factor" is the product of volume of water coming out of dam and the depth of water above the crest of dam. Here, volume of water coming out of dam also includes the initial storage of reservoir and coming inflow at that time. They further developed the relationship of breach formation factor with the volume of material eroded from dam body. The breach geometry shape is triangular initially and finally trapezoidal when apex reaches the natural bed of river. For this, Side slope is assumed as 1H: 2V in most of the cases. The following are the relation of volume of material eroded and breach formation time developed by the MacDonald and

Langridge-Monopolis as per the report of Wahl (1998) .

$V_{er} = 0.0261(V_{out} h_w)^{0.769}$ for earth fill dam (2.1)

$V_{er} = 0.00348(V_{out} h_w)^{0.852}$ for rock fill dam (2.2)

$t_f = 0.0179 (V_{er})^{0.364}$ (2.3)

Where

V_{out} = Volume of water escaping from dam.

h_w = Water height at failure above the final level of breach bottom.

t_f = Breach formation time (hrs.)

Froehlich (1995)

As result of research conducted by Dr.Froehlich on 63 earthen dam, earthen zoned dam, earthen dam with clay core and rock fill dam give the a set of regression equation used for computation of average breach width, Side slope and failure time. The range of data used for the development of regression equation by Dr.Froehlich for the height of the damis 3.66 to 92.26 m and in the same manner volume of water for the breaching moment is 0.00130 to 660x106 m^3 (with 87% < $25x10^6$ m^3 , & 76 %<$15x10^6$ m^3). By using these data range, Dr.Froehlich developed the following regression equation for the average breach width and failure time using Side slope 1.4H: 1V for overtopping failure and 0.9H: 1V for piping failure case.

$B_{av} = 0.1803 K_o V_w^{0.32} h_b^{0.19}$ (2.4)

$t_f = 0.00254 V_w^{0.53} h_b^{(-0.90)}$ (2.5)

Ko = 1.4 for overtopping; 1.0 otherwise

where

B_{av} =Average breach width

t_f = Breach formation time.

Ko = constant having value 1.4 for overtopping and 1 for seepage.

V_w = Volume of water in the reservoir at the breaching moment.

h_b = Water height at failure above the final level of breach bottom.

Froehlich (2008)

Dr.Froehlich conducted further research to update his regression equation of average breach width and breach formation time using 74 case study data including 11 no of case study in the year 2008 and remaining case studies data in the previous year for earthen dam, earthen with a clay core wall and rock fill dam in the year 2008. The range of data used to update regression equation by Froehlich for the height of the dam is 3.05 to 92.26 m (with 93% < 30 m, & 81% < 15m) and in the same manner volume of water for the breaching moment .is same for as previous year by using Side slope 1H: 1V for overtopping mode of failure and 0.7H: 1V for piping failure case.

$B_{av} = 0.27\ K_o\ V_w^{\ 0.32}\ h_b^{0.04}$ (2.6)

$t_f = 63.2\ (Vw/gh_b^{2})^{0.5}$ (2.7)

Ko = 1.3 for overtopping; 1.0 otherwise

Where

B_{av} =Average breach width

t_f = Breach formation time.

Ko = constant having value 1.4 for overtopping and 1 for seepage.

V_w = Volume of water in the reservoir at the breaching moment.

h_b = Water height at failure above the final level of breach bottom.

Von Thun and Gillette (1990)

Von Thun and Gillette (1990) develop their regression equation by using the case study data of 57 dams. They developed their regression equation by using the data of both Froehlich (1987) and MacDonald and Langridge-Monopolis (1984). The range of data used for the development of regression equation by Von Thun and Gillette for the height of the dam is 3.66 to 92.96 m (with 89% < 30m, & 75% < 15 m) in the same manner volume of water for the breaching moment is 0.027 to 660x106 m^3 (with 89% < $25x10^6$ m^3 , & 84 %<$15x10^6$ m^3) by using Side slope of 1H: 1V except for dam with cohesive soils and for cohesive soils and for cohesive soils the suitable range of Side slope may vary from 1H: 2V or 1H: 3V (Wahl, 1998).

The following is the regression equation developed by Von Thun and Gillette for average breach width is.

$B_{av}= 2.5h_w +C_b$ (2.8)

Where

B_{av} = average breach width (meters)

h_w = Depth of water at the reservoir at the failure time.

C_b = Coefficient taken as reservoir function storage mentioned in the following table 2.2.

Table 2.2: Value of C_b as per reservoir size

S.N.	Size of reservoir, m^3	C_b, m	Size of reservoir , acre-feet	C_b, feet
1	$< 1.23*10^6$	6.1	< 1,000	20
2	$1.23*10^6$- $6.17*10^6$	18.30	1,000-5,000	60
3	$6.17*10^6$- $1.23*10^7$	42.7	5,000-10,000	140
4	$> 1.23*10^7$	54.90	>10,000	180

Many other researchers developed only one equation for breach formation time but Von Thun and Gillette developed two set of equation for breach formation time. There are two equations in one set of equation. One equation is for erosion resistance dam materials and other equation is for easily erodible dam materials which are separately described below.

The first set of equation for breach formation time totally depends upon the water depth above the breach invert.

$$t_f = 0.02h_w + 0.25 \quad \text{(Erosion Resistant)} \quad (2.9)$$

$$t_f = 0.15h_w \quad \text{(Easily Erodible)} \quad (2.10)$$

Where t_f is in hours and h_w is in meters.

The second set of equation for breach formation time depends upon two variable namely average breach widths and the water depth above the breach invert and developed by observation of lateral erosion rate with depth of water. The term lateral erosion rate explains how the final bottom breach width changes with breach development time.

$$t_f = B_{av}/(4h_w) \quad \text{(Erosion Resistant)} \quad (2.11)$$

$$t_f = B_{av}/(4h_w + 61) \quad \text{(Easily Erodible)} \quad (2.12)$$

Xu and Zhang (2009)

Xu and Zhang conducted research to develop the regression equation of breach width, Time of failure and even also for slope for earthen and rock fill dam. They used data of 182 dam from USA and China but 50% dams are greater than 15 m. Other researchers assume the numerical value of Side slope but Xu and Zhang developed the equation for Side slope and Regression developed by them can be used in the analysis and prediction of breach parameter but Time of failure cannot be used in HEC-RAS model because it gives extremely high value.

2.6 Prediction of breach out flow hydrograph

The estimation and selection of proper breach parameter is very important part of dam break analysis procedure because breached flow passes downstream through the breach. The effect of dam breached flood on life and property and flood inundation very much depends on the breached-out flow magnitude. So, prediction of out flow via breach is another important part of the dam breach procedure. In modern day, context although the numerical model like HEC-RAS are developed which computes the breach out flow but some researchers have also developed empirical relationship for routing of incoming flow and reservoir storage via breach. The peak out flow equations are developed from the case study data of earthen, zoned earthen, earthen with clay core and rock fill dams only and relating the peak flow to the dam height, reservoir storage volume or combination of both. Wahl (1998), carried out the analysis of the breach out flow from theoretical and practical aspect and found great variation in between two. The purpose of computing out flow hydro graph using empirical relation is only to compare with the out flow hydrograph using models like HEC-RAS (USACE, 2014) but data should be similar for both dams.

Kirkpatrick (1977)

Kirkpatrick developed the peak outflow best relation with the depth of water in the reservoir at the Time of failure using 13 actual dam failures data and 6 hypothetical failures.

$$Q_p = 1.268\ (h_w+0.3)^{1.24} \qquad (2.13)$$

Where, h_w = Depth of water above the breach invert

In this analysis the failure data of concrete dam namely St. Francis located at California is also used. Initially it is believed that this failure is due to piping through right abutment but recent study shows that the main reason of failure is combined effect of the overturning of dam section and land slide failure at right abutment because of this using this failure data is not appropriate **(**Rogers and McMahon, 1993).

Soil Conservation Service (1981)

The Soil Conservation Service (SCS) study the data used by Kirkpatrick excluding the hypothetical failure data and as a result of their study, they propose the relation of peak out flow with depth of water in reservoir at the failure time.

$Q_p = 16.6\ h_w^{1.85}$ (2.14)

Where, h_w = Depth of water above the breach invert & Q_p = Peak Discharge

Reclamation (1982) just elaborates the SCS model and developed similar envelope equation as developed by SCS using 21case studies data (Wahl, 1998).

Singh and Snorrason (1982 and 1984)

Singh and Snorrason (1982 and 1984) developed the peak outflow relation as function of dam height and volume of water stored in the reservoir by carrying out simulation of eight dam failures using tool DAMBRK and HEC-1. Their research of 1984 cited only storage-peak outflow relation.

$Q_p = f(S)$: $Q_p = f(h_d)$ (2.15)

Where, S = Reservoir Storage, h_d = Dam Height & Q_p = Peak Discharge

MacDonald and Langridge-Monopolis (1984)

As per research conducted by MacDonald and Langridge-Monopolis concluded that breach formation factor is responsible for the peak outflow from breached earth fill dams developing the best fit and envelope curves for this.

$Q_p = 3.85(V_w{*}h_w)^{0.411}$ (2.16)

Where

Q_p = Peak Discharge,

h_w = Depth of water above the breach invert

Froehlich (1995)

Froehlich conducted research on the 22 case study of dam failure and able to develop the relationship among the peak out flow, reservoir volume and head by developing the regression equation. There are prediction error and uncertainty to all type of peak flow estimation equation. He did the modification of Side slope to 0.9H: 1V for seepage failure and develop the following equation for peak discharge in the year 1955b.

$Q_p = 0.607\ V_w\ 0.295\ h_w^{1.24}$ (2.17)

Where

Q_p = Peak Discharge

V_w = Volume of water in reservoir at the Time of failure

h_w = Depth of water above the breach invert

Costa (1985)

Costa carried out research on the failure of all type of constructed and natural dams and summarized for discharge resulting from the respective failures. As a result of the study, he developed the both envelope curve and regression equation defining the peak out flow is the function of water volume in the reservoir or product of dam height and reservoir volume at the Time of failure.

$$Q_p = 1.122\ (S)^{0.57} \tag{2.18}$$

$$Q_p = 1.776\ (S\ h_d)^{0.44} \tag{2.19}$$

Where Q_p = Peak Discharge & S = Reservoir Storage

2.7 Dam break analysis models

Dam break analysis can be done by using the models and there are numbers of models developed so far. These models can be used for one- dimensional, two-dimensional analysis or combination of both in dam break analysis. Two- dimensional model gives better result than one dimensional model under certain condition. Similarly, 1-D model can give better results than 2-D under certain condition. 1-D modelling can produce better results than 2-D modelling when length to width ratio is more than 3:1 (Desktop Review of 2D Hydraulic Modelling Packages, UK Environment Agency, 2009) and steep stream which are highly gravity driven having small overbanks. It means rivers and flood plains where the dominant flow directions and forces follow the general river flow path. Such type of nature of river is obtained in hilly zone. Similarly, 2-D modelling can produce better results than 1-D modeling when length to width ratio is less than 3:1 (Desktop Review of 2D Hydraulic Modelling Packages, UK Environment Agency, 2009) and river flowing in very wide and flat flood plains having meandering nature with braided nature. Such type of nature of river is obtained in alluvial zone.The main objective of using such model is to develop breach hydrograph and routing of that hydrograph in downstream valley so that the dam break analysis and inundation mapping can be done. Followings are some models.

Cristiano (1965)

The initiation of development of model is first done by Cristiano. This model operates by relating the erosion rate of breach with the discharge through breach using tractive force equation (Wahl, 1998). The breach geometry used for this model is trapezoidal shape of constant Bottom width; Side slope depends upon the angle of repose material, the bed slope of breach channel was equal to the internal angle of friction of material. There is doubt for the reliability of an empirical coefficient for the model's performance (Fread, 1988).

Harris and Wagner (1967)

Harris and Wagner do not have their own developed empirical equation but they use Schoklitsch sediment transport equation for the prediction of dam breach flow. The assumption made for this model is that the breach erosion starts immediately after the first over topping the dam crest by water and this erosion proceed continue till the breaches reach the bottom of dam (Wahl, 1998). On the basis of Harris and Wagner's study, Brown and Rogers (1977, 1981) developed a breach model, BRDAM applicable to both overtopping and piping induced breaches.

Lou (1981) and Ponce and Tsivoglou (1981)

Lou and Ponce and Tsivoglou developed the model that operates by relating the Meyer-Peter and Müller sediment transport equation to the one-dimensional differential equations of unsteady flow and also silting and scouring phenomena of silt particles i.e. sediment conservation. The simulation of model occurs on the basis of solving differential equation four-point implicit finite difference method but computation is very complex and highly prone to crashing of model (Wahl, 1988). Since, the model uses the sediment transport equation, so there should be consideration of flow resistance through the breach length that was done by using manning's n. The relationship between breach width to the discharge via breach is empirical. The reservoir storage depletion is taken as upstream boundary condition

NWS DAMBRK Model

NWS DAMBRK model simulates the dam failure and gives the out flow hydrograph. The model contains the simulation routine in such a way that the initiation of breach is started from top of the dam and progress in uniform manner outward downstream till the final bottom breach width is reached at specified breach formation time (Dr. Fread (1977). This model is further improved by Dr. Fread since 1988 naming "BREACH

Model" and applicable for both overtopping and piping. The model operates by using the sediment equation developed by the Meyer-Peter and Müller sediment transport as modified by Smart (1984) for steep channels. The model understands the embankment material in three different ways. The inner most core of impervious material, outer portion and protective layer riprap, vegetation cover exposed to open in downstream side and uses the simple weir orifice formula for flow calculation in the breach section where as downstream flow is computed by quasi-steady uniform flow and manning's n value is selected by Strickler equation. The model is also capable for computing the spillway flow including tail water effect. The mechanism behind the formation of breach is slope stability factor of breach Side slope and probable collapse of upper part of dam by shear and sliding. As the discharge passes through the final breach geometry in specified time, the model computes the out flow hydrograph as well as routing of that out flow hydrograph in the downstream reach is done. The model is capable for doing the simulation of flood wave generated after dam break and provides the information about time of arrival of peak flood, stage of peak flood, volume etc.in the downstream reach.

BEED Model

Singh and Scarlatos (1985) introduced the physically based BEED (Breach Erosion of Embankment Dams) model that is capable of simulating breach progression, sediment and flood routing. Einstein-Brown and Bagnold equations are used for computing the Erosion and sediment transport. Further Singh and Quiroga (1988) noticed that for the use these equations need to assume such value beyond the original value assumed by Einstein-Brown and Bagnold.

SMPDBK Model

SMPDBK is one of the models invented by National Weather Services (NWS), USA and this model is developed as simplified version of DAMBRK and this model is very simple and easy to handle though it is very easy to handle than DAMBRK but as per comparison of the results, both model give similar results under similar case. Another advantage of using this model is that it requires less parameter than DAMBRK and able to simulate for dam break scenario so that emergency action plan (EAP) can be prepared providing information related to the downstream inundation area. This model is efficient to categorize the dam and also the potential risk associated with the dam break.

HEC-RAS Model

Hydraulic Engineering Centre River Analysis System (HEC-RAS) is developed by US Army Corps of Engineers was able to simulate not only the one-dimensional flow but also two dimensional flow. The application of two-dimensional flows is started from 2014. The model operates following the principles of conservation of mass and conservation of momentum for unsteady flow but for steady flow only the conservation of energy using the full Saint Venant equation. The main breaching parameters required for the dam breach are final Bottom width of breach, Side slope of breach & breach formation time, rate of breach progression, channel roughness factor i.e. manning's n, coefficient of contraction & expansion (Brunner 2012).The model is capable of developing out flow hydrograph and routing of that flow hydrograph downstream so that inundation area can be easily identified.

2.8 Application of HEC-RAS Model

Hydraulic Engineering Centre River Analysis System (HEC-RAS) is easy to handle, easy and freely available world wise used numerical model for hydro-dynamic modelling of river.The application of HEC-RAS for dam break by different researchers analysis are described below.

Salajeghah et al (2009) conducted study on Polasjan river basin in Iran central plateu for floodplain analysis using HEC-RAS and GIS. They presented direct approach for processing output of HEC-RAS in ArcGIS platform and identified from the results obtained that GIS is an effective tool for floodplain mapping and analysis.

Xiong (2011) conducted dam break study using HEC-RAS numerical model on 100 ft high and 1.3 mile length comprehensive flood control Foster Joseph Sayers dam situated cross Susquehanna river based on the available geometry data for 3 scenario such as "no dam break", "dam break" and "without dam" he advocated that the dam break he advocated that the dam break is complicated & comprehensive process and actual failure mechanics are very hard to understood, neither physical model nor empirical models could fully depict the dam break scenarios and impacts of the dam break

Timbadiya et al (2012) developed hydrodynamic model of lower Tapi river using HEC-RAS for Unsteady flow condition by using surveyed data for stream and appropriate downstream and upstream boundary condition. Observed and Simulated flood are compared with the stage hydrograph at different stations on the river and found that the simulated flood flows are relatively similar to observed flows. Hence validated the model developed for flood forecasting in lower Tapi river. Yerramilli (2012) developed hybrid hydrodynamic model to the city of Jackson, MS using HEC-RAS for identifying flood hazard and assessing the vulnerability of the region by integrating the numerical model with GIS. Combination of hydraulic model using HEC-RAS one-dimensional flood simulation model and GIS tool indicates the capability of simulating flood events and spatially depicting the degree of exposure or vulnerability of the region towards a hazard event in terms of inundation extent, water levels and depth. HEC-RAS model simulation results gave same result as that of the observed inundation depth record at that location

Yang et al (2006) Developed direct processing approach to river floodplain delineation on part of south nation river system in Ottawa, Ontario of Canada. Floodplain mapping is done by integrating geographic information system (GIS) with HEC-RAS. Numerical model simulations are performed to generate water surface profile for six different design storm events. Geo-referenced maps are integrated with digital elevation model (DEM) data to develop triangular irregular network (TIN) terrain model and floodplain zones for six design storm events were reproduced by overlaying the integrated terrain model with the corresponding water surface. They concluded that HEC-RAS river network model provides upgraded simulations with better computational routine, supports import and export of GIS data and allows to view the river reach and cross-section data. This study focuses on integrating the hydraulic data with GIS map for inundation floodplain zones.

Yochum et al (2008) developed a one-dimensional unsteady numerical model using HEC-RAS to simulate Big Bay dam embankment failure which happened on March 12, 2004 and to predict the behaviour of flood wave generated caused by dam breach through the downstream valley. The model was developed using observed breach geometry and the HEC-RAS model gives relatively accurate result comparing with the water marks measured immediately after the failure.

CHAPTER-3

STUDY AREA

3.1 Project location

Nepal lies on the lap of the highest mountain ranges of the Himalaya. It extends between the Tibetan plateau of the People's Republic of China in the North and Republic of India in the South and it is roughly rectangular in shape. Nepal embraces in itself a unique variety of geographical settings ranging from the southern lowland to the high mountain in the North. There are basically three richly varied regions - the Terai Region, the middle Region and the Himalayan Region.

Nalgad Storage Hydroelectric Project is located in Jajarkot District in the mid-western development region of Nepal between Longitude 82017’15”E - 82017’55”E and Latitude 28047’28.8”N-28054’15”N. Nalgad is a tributary of the Bheri River in the Karnali Basin. The dam site of the project is located approximately 9.25 km upstream from the confluence of the Nalgad and the Bheri-River and the powerhouse is located on the left bank of Nalgad River approximately 500 m upstream from the suspension bridge at Dali VDC. The powerhouse was originally proposed at the right bank Bheri-River. Due to the adverse geological condition, the powerhouse location was shifted to the left Bank of Nalgad River. The powerhouse is proposed to be underground and is founded on the solid rock base.

The Nalgad River is comprised of one major river followed by other small tributaries. The main-stem Nalgad Gad River drains the southern flank of the Middle Mountain range and the central part of the catchment. Upstream of dam site, Nalgad River meets many small tributaries, out of which the major tributaries are described here. From the dam site, about 9 km upstream at Bayata two rivers named Ranga Gad and Daha Gad merges and become Barekot Khola, which also called Nalgad Gad River. The river gradient from dam site to this confluence point is 1:90. The river gradient is not much flatter. The river almost flows north to south. These are the two major tributaries of the Nalgad.

3.2 Location map

Nalgad Storage Hydroelectric Project is located in Jajarkot District in the mid-western development region of Nepal between Longitude 82˚ 17’15”E - 82˚ 17’55”E and Latitude 28047’28.8”N-28054’15”N.The district headquarter is Khalanga bazar having road connectivity with other neighbouring district. The following fig.3.1 shows the location of the district. The proposed dam is located at 82˚ 17’ 35.05”E and 28˚ 47’ 5.54”.

Fig. 3. 1: Location of dam axis at out let point of watershed area.

3.3 Data collection

The data required for dam break analysis is collected from Nepal Hydro Corporation Private Limited (NHCPL) under the water resource, energy and irrigation ministry of Government of Nepal. The prefeasibility study of the projects is conducted by Nepal Electric Association (NEA) since 2012 and the project is going to implement in near future by NHCPL

3.3.1 Data required for the dam

Salient feature of the project, Inflow hydrograph, Elevation area capacity curve, falls under this category. These data are taken from detail project report of the project as provided by the NHCPL.

a. Salient feature

This portion of dam characteristics includes the type of the dam, height of the dam, crest length, width of the crest, bed elevation of the stream, crest elevation, spillway no. and no of gates in spillway, pond level of spillway and other design parameter of dam and formed reservoir normally pond level of reservoir, area of reservoir etc. The following table 3.1 shows the details of design parameter of dam and reservoir.

Table 3. 1: Design parameter of dam and reservoir for this study.

S.N.	Dam and Reservoir Description.	Type/size
1	**Design Parameters of Dam**	
1.1	Types of Dam	Rock fill with impervious core
1.2	Height of Dam (m)	200
1.3	Crest Elevation (m)	1580.30
1.4	Length of Crest (m)	545
1.5	Width of Crest (m)	10
1.6	Height of Spillway (m)	14.7
1.7	Maximum Spillway Capacity (m^3/s)	2978
1.8	Width of Spillway (m)	24
1.9	Shape of Spillway	Ogee
2	**Design Parameters of Reservoir**	
2.1	Maximum Water surface elevation (m)	1570
2.2	Minimum Operating Level (MOL, m)	1498
2.3	Total Storage Volume (Million m^3)	419.6
2.4	Live Storage Volume (Million m^3)	296.30

b. Inflow hydrograph

Nepal Hydro Corporation Private Limited (NHCPL)) developed the inflow design flow (IDF) for the dam site for the design of dam parameters and reservoir. This inflow hydrograph is developed considering the probable maximum flood.. The data for hydrograph is given in table 3.2 and hydrograph is also shown in fig. 3.2.

Table 3.2: Inflow hydrograph for dam break and natural flow cases

Time (hrs)	0	1	2	3	4	5	6	7	8	9	10
Discharge(m^3/s)	57	57	211	1072	2605	3448	3523	3492	3169	2698	2419

Time (hrs)	11	12	13	14	15	16	17	18	19	20	21
Discharge(m^3/s)	2179	1945	1787	1645	1456	1220	1047	900	710	525	404

Time (hrs)	22	23	24	25	26	27	28	29	30
Discharge(m^3/s)	301	220	176	140	110	92	77	66	57

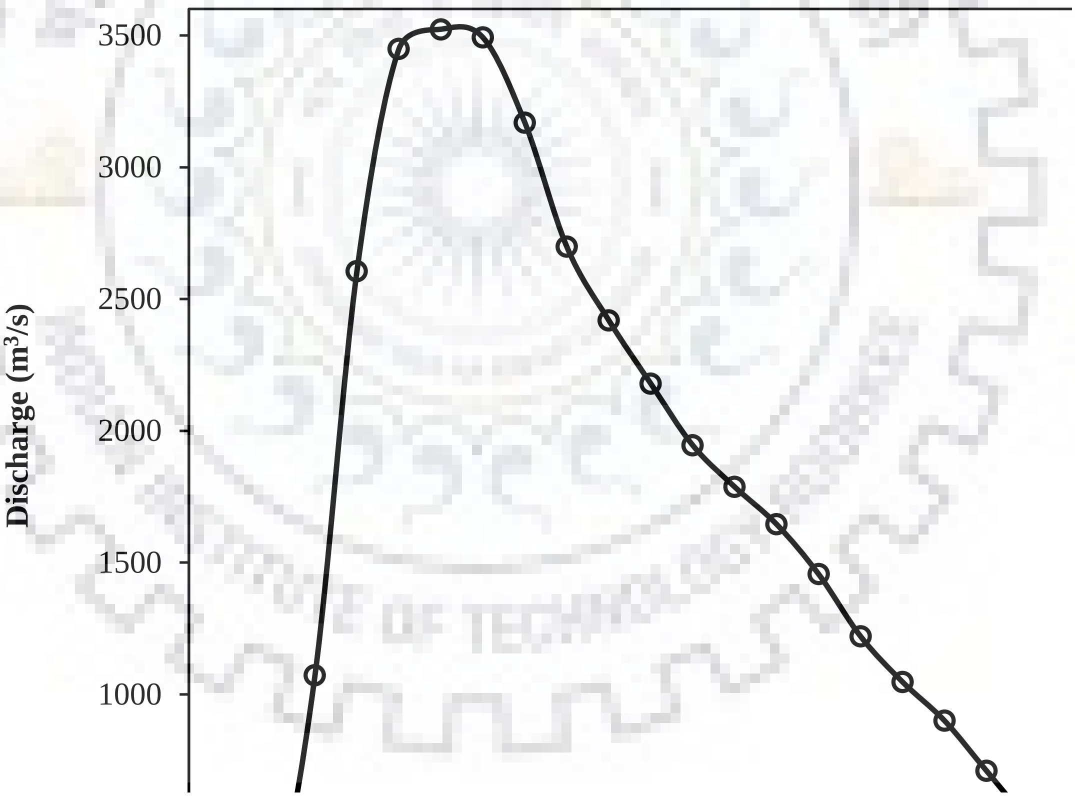


Fig. 3.2 Inflow hydrograph

c. Elevation and capacity curve

This curve must be developed in any storage project to define the geometric characteristics of reservoir. The basis of development of this curve depends upon the design parameters of dam. Elevation, capacity and area curve is the representation of reservoir storage capacity. This curve defines the storage capacity of reservoir in HEC-RAS Model. The Nepal Hydro Development Association developed elevation curve for this project is given in table 3.3 below and elevation capacity curve is shown in fig.3.3 also.

Table 3. 3: Elevation and capacity curve data

Elevation(m)	1380.3	1385.3	1390.3	1400	1410	1420	1440	1458	1470
Capacity (Mm^3)	0	0.5	0.95	1.20	2	3	20	40	60

Elevation(m)	1480	1490	1498	1505	1512	1519	1524	1530	1534
Capacity (Mm^3)	80	100	120	140	160	180	200	220	240

Elevation(m)	1539	1542.99	1547.55	1550.97	1555	1560	1562.92	1568
Capacity (Mm^3)	260	280	300	320	340	360	380	400

Elevation(m)	1570	1574	1578	1579.8	1583	1591.2	1595	1598.5	1600
Capacity (Mm^3)	420	440	460	480	500	540	560	580	600

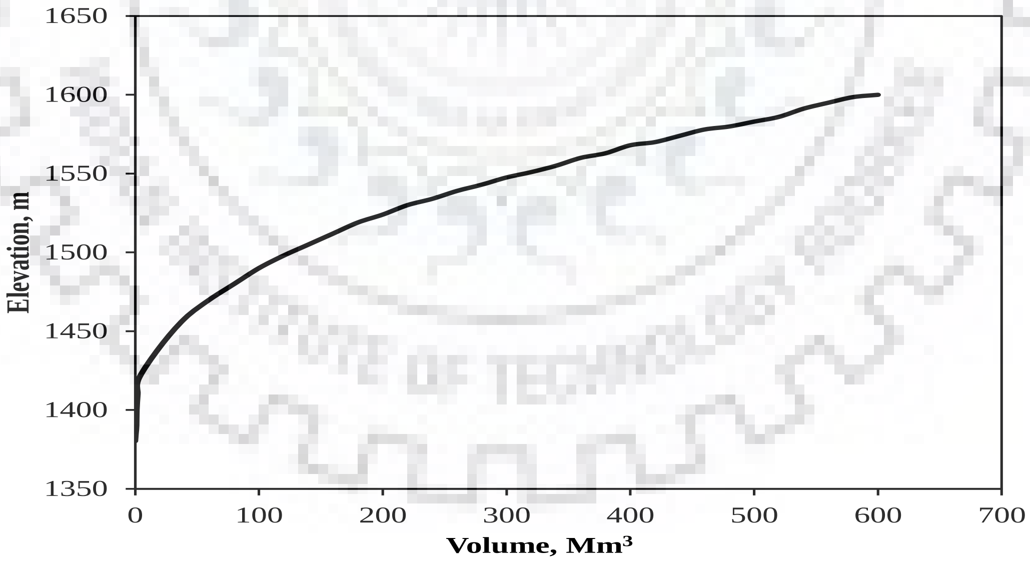


Fig. 3.3 Elevation and capacity curve for Nalgad Hydro Project

3.3.2. River characteristics

The bathymetric survey data for the X-section of river is up to 10 km downstream of dam axis is available. The remaining X-section data after 10 km are not available. The "SRTM" digital elevation model DEM is used to collect additional X-section data. The DEM of the river is shown in fig. 3.4 below.

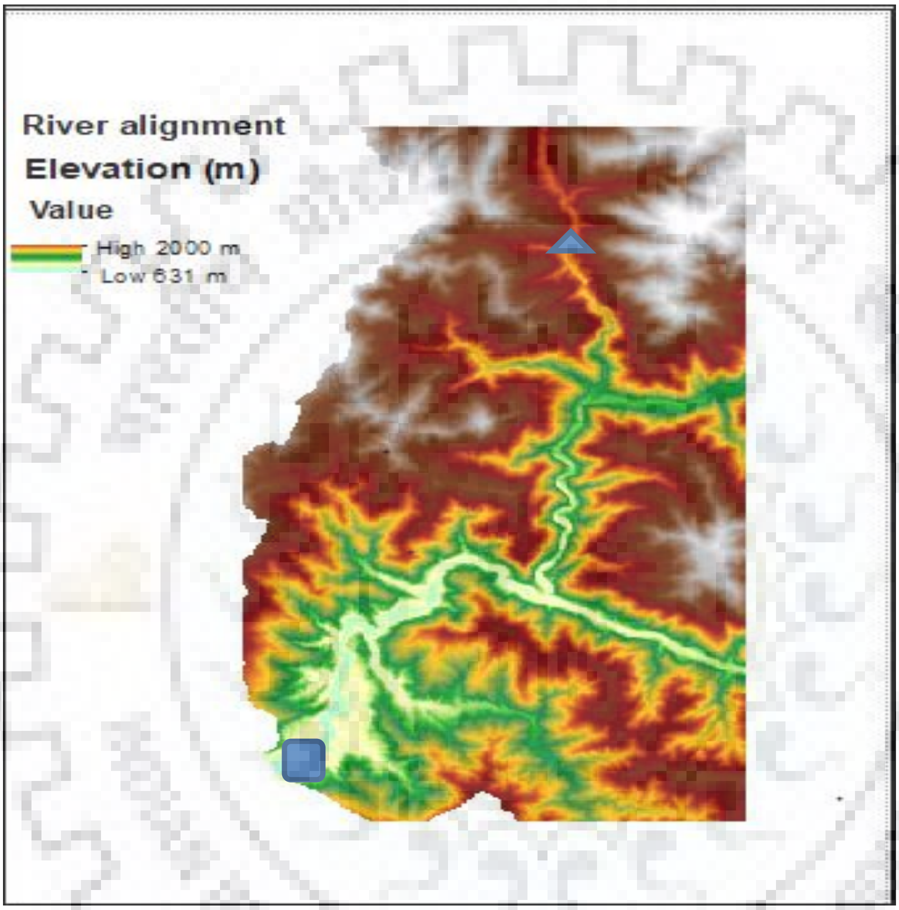


Fig. 3.4: River alignment with △ and □ symbol for dam axis and tail end.

CHAPTER-4

METHODOLOGY

4.1 Introduction

The flow chart describing various steps to carry out the dam break analysis of Nalgad-Hydro Dam is shown in Figure 4.1. The required data can be grouped in two category viz. dam characteristics and river characteristics. The salient feature, inflow hydrograph, elevation capacity and boundary condition etc. comes under the dam characteristics. The various breach parameter like shape, size, location, breach parameter and Time of failure etc. are selected based on the dam characteristics. The cross-section of the river with bank station, roughness etc. comes under the river characteristics. Further bathymetry along the river is also required for flood inundation mapping.

Three case are analysed using HEC- RAS simulation viz. (a) No Dam – Natural flow in the river reach without presence of any inline structure in the river; (b) Dam Break – Dam is represented by in line structure with is storage capacity and the dam fails during peak of inflow hydrograph; (c) Without Dam Break – It is similar to previous case expect that the dam doesn't fail and the inflow hydrograph is routed through spillway using Modified Puls method, the dam is allowed to overtop in water level rises beyond the top of dam.

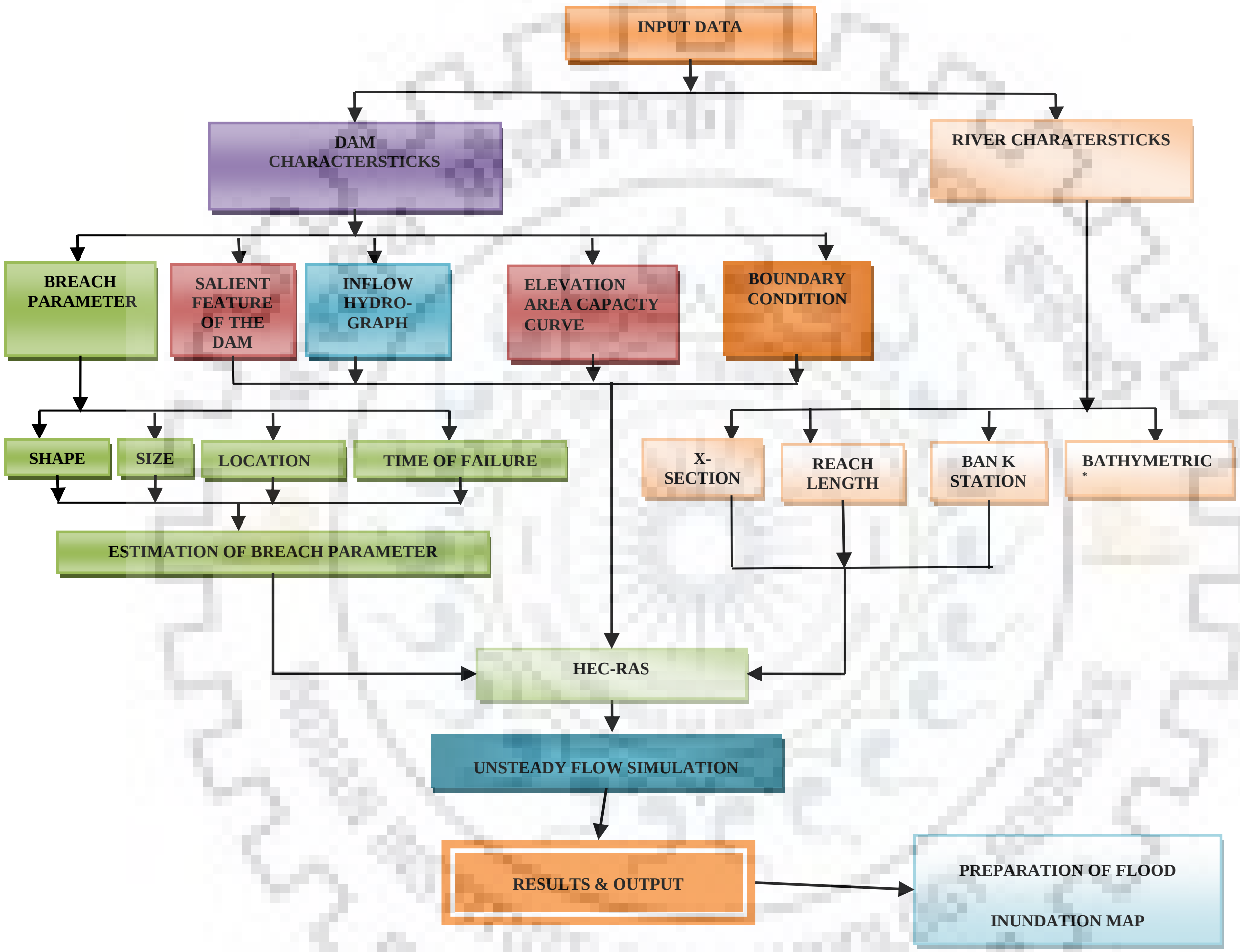


Fig. 4.1 Flow chart for dam break modelling

4.2 Data input

The selection of breach geometry is very complex and important task including breach formation time also. Different researchers have given different regression equations for prediction of breach parameter but HEC-RAS model use the following regression equation and selection of breach parameter is done by manual calculation using following equation.

Froehlich (1995)

As result of research conducted by Dr.Froehlich on 63 earthen dam, earthen zoned dam, earthen dam with clay core and rock fill dam give the a set of regression equation used for computation of average breach width, Side slope and failure time. The range of data used for the development of regression equation by Froehlich for the height of the dam is 3.66 to 92.26 m and in the same manner volume of water for the breaching moment is 0.00130 to 660x106 m^3 (with 87% < 25x10^6 m^3 , & 76 %<15x10^6 m^3). By using these data range, Froehlich developed the following regression equation for the average breach width and failure time using Side slope 1.4H: 1V for overtopping failure and 0.9H: 1V for piping failure case.

$$B_{av} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19} \tag{4.1}$$

$$t_f = 0.00254\ V_w^{0.53}\ h_b^{(-0.90)} \tag{4.2}$$

Ko = 1.4 for overtopping; 1.0 otherwise

Where

B_{av} =Average breach width

t_f = Breach formation time.

Ko = constant having value 1.4 for overtopping and 1 for seepage.

V_w = Volume of water in the reservoir at the breaching moment.

h_b = Water height at failure above the final level of breach bottom.

Froehlich (2008)

Dr.Froehlich conducted further research to update his regression equation of average breach width and breach formation time using 74 case study data including 11 no of case study in the year 2008 and remaining case studies data in the previous year for earthen dam, earthen with a clay core wall and rock fill dam in the year 2008. The range of data used to update regression equation by Dr.Froehlich for the height of the dam is 3.05 to

92.26 m (with 93% < 30 m, & 81% < 15m) and in the same manner volume of water for the breaching moment .is same for as previous year by using Side slope 1H: 1V for overtopping mode of failure and 0.7H: 1V for piping failure case.

$B_{av} = 0.27\ K_o\ V_w^{\ 0.32}\ h_b^{0.04}$ (4.3)

$t_f\ = 63.2\ (Vw/gh_b^2\ )^{0.5}$ (4.4)

Ko = 1.3 for overtopping; 1.0 otherwise

Where

B_{av} =Average breach width

t_f = Breach formation time.

Ko = constant having value 1.4 for overtopping and 1 for seepage.

V_w = Volume of water in the reservoir at the breaching moment.

h_b = Water height at failure above the final level of breach bottom.

MacDonald and Langridge-Monopolis (1984)

The research intervention of MacDonald and Langridge-Monopolis (1984) developed the breach formation factor by utilizing the 42 case-study dam failure data of earth fill dams, earth fill with clay core and rock fill dam. As per their conclusion “breach formation factor” is the product of volume of water coming out of dam and the depth of water above the crest of dam. Here, volume of water coming out of dam also includes the initial storage of reservoir and coming inflow at that time. They further developed the relationship of breach formation factor with the volume of material eroded from dam body. The breach geometry shape is triangular initially and finally trapezoidal when apex reaches the natural bed of river. For this, Side slope is assumed as 1H: 2V in most of the cases. The following are the relation of volume of material eroded and breach formation time developed by the MacDonald and Langridge-Monopolis as per the report of Wahl (1998) .

$V_{er} = 0.0261(\ V_{out}\ h_w)^{0.769}$ for earth fill dam (4.5)

$V_{er} = 0.00348(\ V_{out}\ h_w)^{0.852}$ for rock fill dam (4.6)

$t_f\ =0.0179\ \ (V_{er}\ )^{0.364}$ (4.7)

Where

V_{out} = Volume of water escaping from dam.

h_w = Water height at failure above the final level of breach bottom.

t_f = Breach formation time (hrs.)

Von Thun and Gillette (1990)

Von Thun and Gillette (1990) develop their regression equation by using the case study data of 57 dams. They developed their regression equation by using the data of both Dr.Froehlich (1987) and MacDonald and Langridge-Monopolis (1984). The range of data used for the development of regression equation by Von Thun and Gillette for the height of the dam is 3.66 to 92.96 m (with 89% < 30m, & 75% < 15 m) in the same manner volume of water for the breaching moment is 0.027 to 660x106 m^3 (with 89% < $25x10^6$ m^3 , & 84 %<$15x10^6$ m^3) by using Side slope of 1H: 1V except for dam with cohesive soils and for cohesive soils and for cohesive soils the suitable range of Side slope may vary from 1H: 2V or 1H: 3V (Wahl, 1998).

The following is the regression equation developed by Von Thun and Gillette for average breach width is.

$B_{av}= 2.5h_w +C_b$ (4.8)

Where

Bav = average breach width (meters)

h_w = Depth of water at the reservoir at the failure time.

C_b = Coefficient for as reservoir function storage given in table 4.1 below.

Table 4.1value of C_b as per size of reservoir

S.N.	Size of reservoir, m^3	C_b, m	Size of reservoir , acre-feet	C_b, feet
1	$< 1.23*10^6$	6.1	< 1,000	20
2	$1.23*10^6$- $6.17*10^6$	18.30	1,000-5,000	60
3	$6.17*10^6$- $1.23*10^7$	42.7	5,000-10,000	140
4	$> 1.23*10^7$	54.90	>10,000	180

Many other researchers developed only one equation for breach formation time but Von Thun and Gillette developed two set of equation for breach formation time. There are two equations in one set of equation. One equation is for erosion resistance dam materials and other equation is for easily erodible dam materials which are separately described below.

The first set of equation for breach formation time totally depends upon the water depth above the breach invert.

$t_f = 0.02h_w + 0.25$ (Erosion Resistant) (4.9)

$t_f = 0.15h_w$ (Easily Erodible) (4.10)

Where t_f is in hours and h_w is in meters.

The second set of equation for breach formation time depends upon two variable namely average breach widths and the water depth above the breach invert and developed by observation of lateral erosion rate with depth of water. The term lateral erosion rate explains how the final bottom breach width changes with breach development time.

$t_f = B_{av} /(4h_w)$ (Erosion Resistant) (4.11)

$t_f = B_{av} /(4h_w + 61)$ (Easily Erodible) (4.12)

Xu and Zhang (2009)

Xu and Zhang conducted research to develop the regression equation of breach width, Time of failure and even also for slope for earthen and rock fill dam. They used data of 182 dam from USA and China but 50% dams are greater than 15 m. Other researchers assume the numerical value of Side slope but Xu and Zhang developed the equation for Side slope and Regression developed by them can be used in the analysis and prediction of breach parameter but Time of failure cannot be used in HEC-RAS model because it gives extremely high value.

Xu and Zhang regression equation is, $(B_{ave}/h_b) = 0.787*(h_d/h_r)^{0.133}*(V_W^{1/3}/h_w)^{0.652}*e^{B_3}$

Where

B_{ave}= Average Breach Width

V_w= Reservoir Volume at the Time of failure

h_b= Height of final breach

h_d= Height of dam

h_r= 15m is reference height for distinguishing dam

h_w=Height of water above breach bottom

B_3= $b_3+b_4+b_5$ coefficient that is function of dam properties

b_3= -0.041, 0.026 & -0.226 dams with core walls, concrete faced dams and homogeneous/ zoned fills dam respectively.

b_4= 0.149 and -0.389 for overtopping and seepage events respectively.

b_5 = 0.291,-0.14 & -0.391 for high, medium and low durability of dam erodibility of dam respectively.

Xu and Zhang do not give directly the numerical value of Side slope but they developed the equation for top width of breach and Side slope.

Top width of breach is given by relation, $(B_t/h_b) = 1.062*(h_d/h_r)^{0.092}*(V_w^{1/3}/h_w)^{0.508}*e^{B_2}$

Where

$B_2= b_3+b_4+b_5$ coefficient that is function of dam properties

b_3= 0.061, 0.088 and -0.089 dams with core walls, concrete faced dams and homogeneous/ zoned fills dam respectively.

b_4= 0.299 and -0.239 for overtopping and seepage events respectively

b_5 = 0.411, -0.062 and -0.289 for high, medium and low durability of dam erodibility of dam respectively

Slope for overtopping $(Z) = (B_t-B_{ave})/h_b$ (H: 1V) (4.13)

Time of failure $(t_f) = 0.304*t_r * (h_d/h_r)^{0.707} *(V_w^{1/3}/h_w)^{1.228} *e^{B_5}$ (4.14)

Where

$t_r =$ 1 hour (unit duration).

$B_5 = b_3+b_4+b_5$ coefficient that is function of dam properties.

b_3= - 0.327, -0.674 and -0.189 dams with core walls, concrete faced dams and homogeneous/ zoned fills dam respectively.

b_4= -0.579 and -0.611 for overtopping and seepage events respectively

b_5 = -1.205, -0.564 and 0.579 for high, medium and low durability of dam erodibility of dam respectively

In HEC-RAS model, this Time of failure is not used because it gives higher value than other regression equation. The breach parameters are estimated using above five regression equation.

4.3 HEC-RAS model setup

The reason behind the selecting the HEC-RAS model for this study is that this model is used world widely and the level of accuracy is very fair, easy to handle and easily available. The dam break analysis is mainly categorized in to two sections. The first part is to breach the dam & second part is to route the resulting flood wave in downstream valley. The HEC-RAS model uses the principle of conservation of mass and conservation of momentum for the analysis of unsteady flow solving the St, Venant's equation for conservation of continuity and momentum where as for the analysis of steady flow, the principle of conservation of energy is used on the basis of the flow condition.

4.3.1 One dimensional model (1-D) setup

The main data required for the dam breaching is the dam characteristics, reservoir characteristics and breach parameters. The data related to the dam characteristics and reservoir characteristics are taken from design report of the project but selection of breach parameters is done using the regression equation as mentioned above in this chapter. The main input data required for the flood routing is cross-section of the river downstream the dam. The data related to the X-sections are obtained from river characteristics by bathymetric survey but width of cross-section should be so wide to cover the flood inundation which is not possible using the bathymetric survey data. So, "SRTM" digital elevation model is used for X-section data. the data After collecting regarding, dam characteristics, reservoir characteristics and river characteristics, the DEM is exported in RAS mapper, a GIS tool in HEC-RAS model. Then, with help of available coordinate, dam axis is identified and from dam axis, center of line of river, bank lines and flow paths are drawn up to the tail end. Then, cross section lines are drawn from left to right at the spacing of 25 m and 10 m and additional cross-sections are drawn using interpolation tools so that models will be stable. The boundary condition is given and then, dam is inline using inline tool in HEC-RAS 1-D model. The manning's n value is taken as 0.15 for steep reach and for flat reach 0.075 and then. Then, computation time interval 2 second and hydrograph output interval 10 minute model is simulated. The important factors considered for 1-D HEC-RAS modeling are the given below.

- Cross-sections spacing and hydraulic properties of section.
- Manning's roughness coefficient
- Computational time step
- Modeling steep streams.
- Theta weighing factor
- Inline structure and lateral structure stability factors.

a. Cross-sections spacing and hydraulic properties of section

Cross-sections are drawn in such a way that it covers the normal flow as well as high flood flow geometry predicted by the dam breaching scenario. The cross-section line must be perpendicular to the flow lines and river lines and should be placed where the river geometry changes. Additional cross-section should be drawn for the reaches in the river where contraction and expansion of river, river bending, sudden changes in bed slope, and changes in roughness are faced. The additional cross-sections are also drawn at the location where Inline structure like dam, levees, bridge and culverts etc. are placed. The velocity of flow is obviously high in steep reaches so there should be closer spacing of cross-section. The spacing of cross-section depends upon the experience of modeler, however at the beginning rough estimate of spacing can be done by using the empirical equation developed by Fread (1993) and Samuels (1989) but the spacing of cross-section should be neither to close nor far apart and spacing is done such that courant number should be equal to or less than one so that model instability problems are not encountered, HEC-RAS manual (2010).

The layout of river reach of about 47.66 km in HEC RAS is shown in Figure 4.2. River cross-section at dam site and up to 14.36 km downstream are collected from the Nepal Hydro Corporation Private Limited (NHCPL) under the Ministry of Energy, Irrigation and Water resource. The SRTM Digital Elevation Model is used for extraction of cross-section data in the downstream reach after 14.36 km. The river cross-section data extracted from SRTM data are corrected at some locations as per the observed error in the upstream reaches of 14.36 km. The longitudinal profile of the river is shown in Figure 4.3. It may be observed that the initial section of river up to about 14.36 km is having very high slope of 40 m/ km. Hence in this section cross-section spaced at distance of 10 m and in rest of the section cross-section spacing are at distance of 25 m. A typical river cross-section is shown in Figure 4.4.

b. Manning's roughness coefficient

The velocity of flow in the channel or flood plain very much depends upon the selection of roughness factor because manning's roughness coefficient is the representation of the resistance to flow in the channel. As per the empirical velocity relation that is developed by manning, smaller the value of "n" higher the velocity or vice versa i.e. super critical and Sub-critical flow respectively. So, model stability very much depends up on the selection of roughness coefficient. The value of "n" 0.15 is taken for steep reach and 0.075 for comparatively flatter reach for this study.

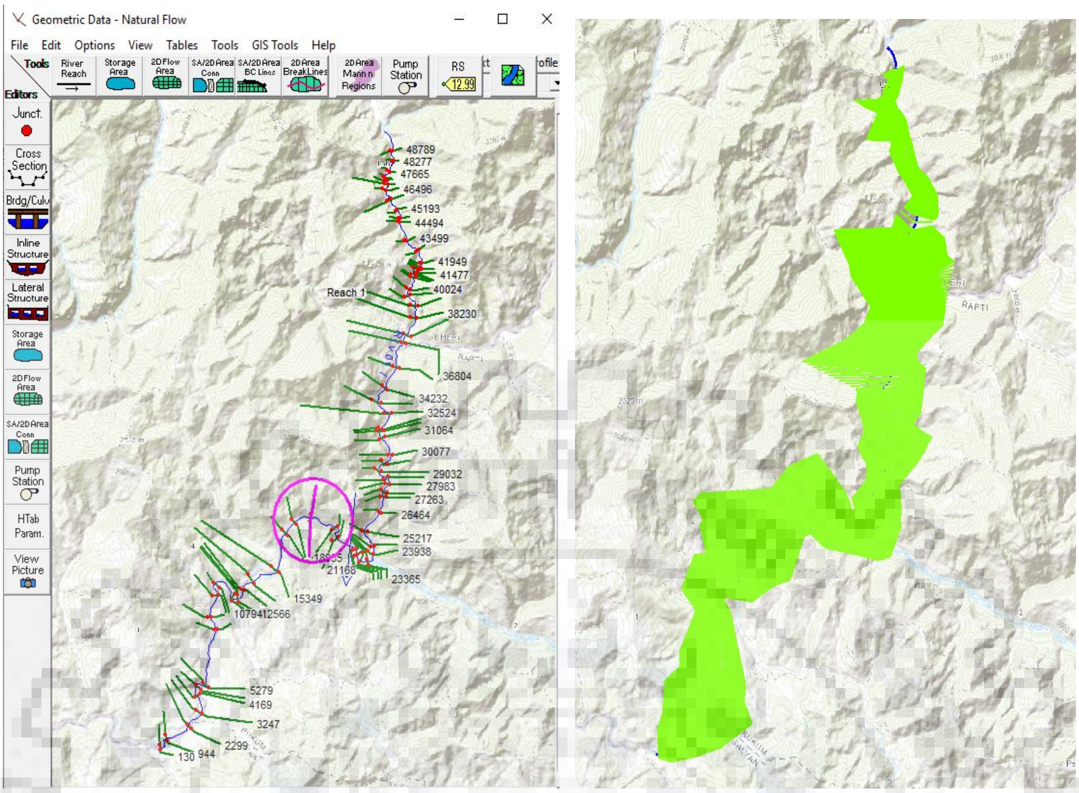


Fig. 4.2: Layout of river network

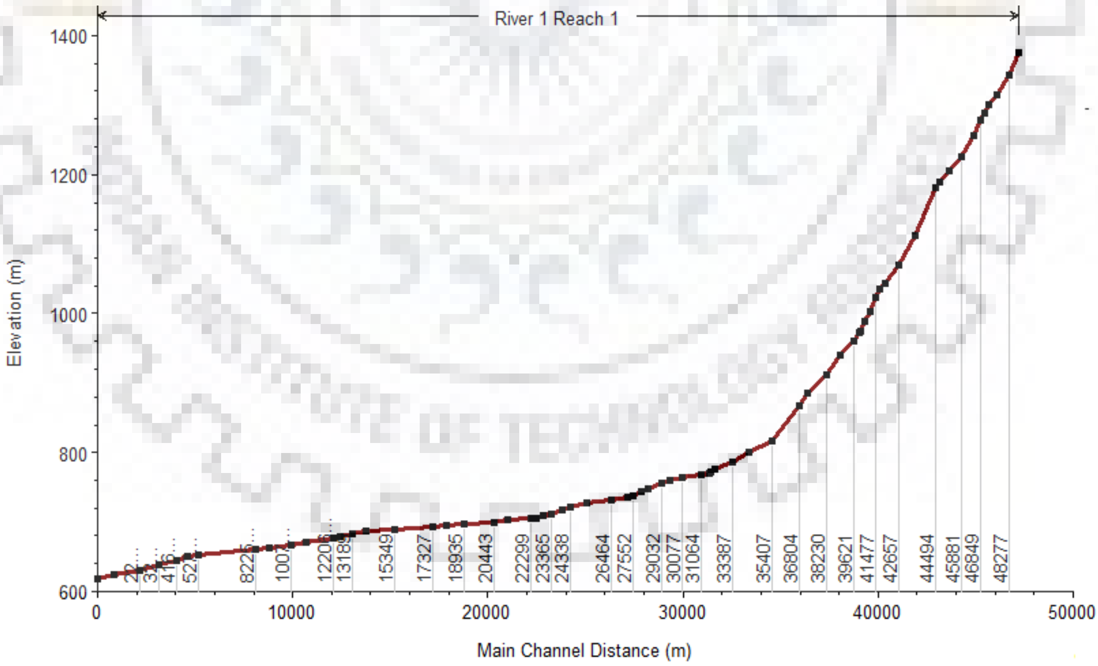


Fig. 4.3: Longitudinal profile of river

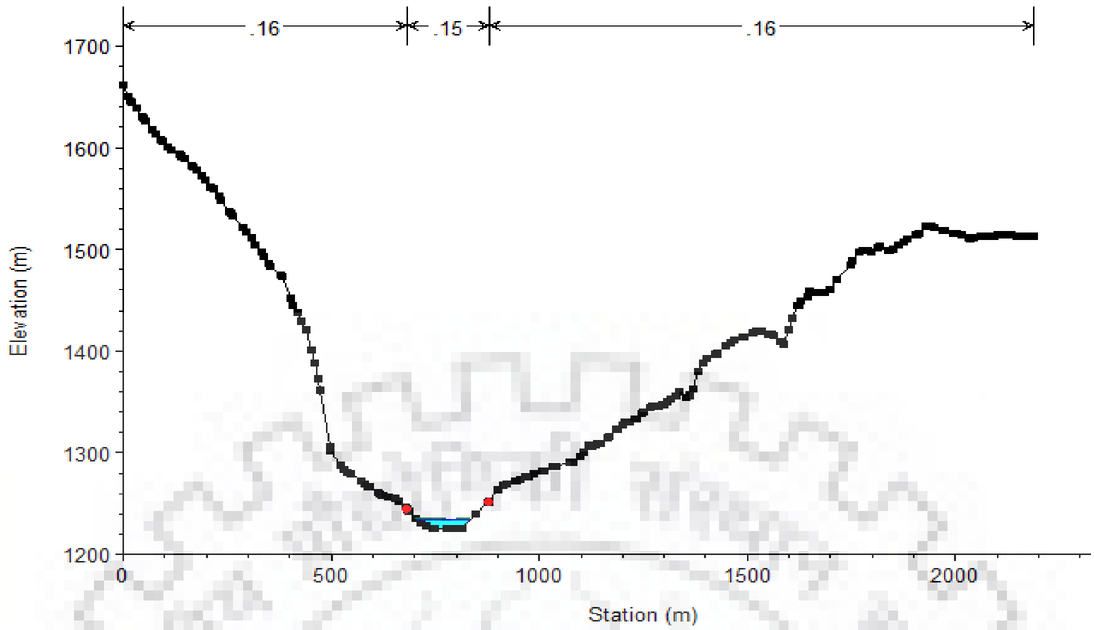


Fig. 4.4: Typical river cross-section

4.3.2 Dam characteristics

The dam in modelled in HEC RAS as an inline structure as shown in Figure 4.5. The salient features are obtained from the detail project report describing the type of the dam, crest length, width of the crest, crest and bottom elevation, the shape and capacity of spillway, width and height of the spillway etc. Similarly, the total storage is represented by a storage area. The area as per reservoir formed is taken from the elevation-area-capacity (EAC) curve. The alignment of dam with two spillway gate-opening in HEC-RAS model is shown in fig. 4.6

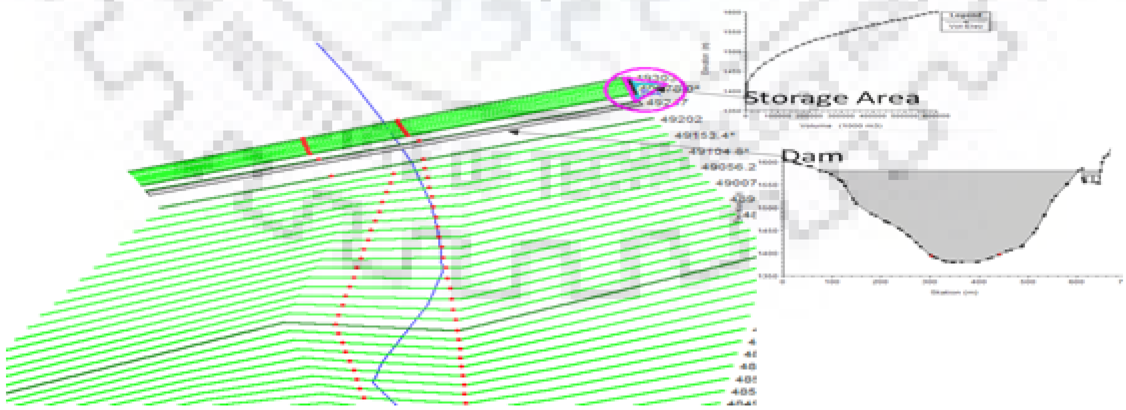


Fig. 4.5: Cross-section at dam axis with spillways

4.3.2 Boundary condition

The boundary condition is the process in which data corresponding to stage and flow hydrograph, normal depth, rating curve and normal depth are provided at various locations of river in the HEC-RAS model. The boundary condition is categorized as external boundary condition and internal boundary condition. The boundary condition given at the open ends (first and the last river station) is said to be external boundary condition and the boundary condition given in between these stations is called internal boundary condition. Normally, flow hydrograph and normal depth can be given as external boundary condition in the first and last station of river respectively and other boundary condition can be given in between these two as per requirement in the river system. For this study, inflow hydrograph is provided as the upstream boundary condition and normal depth represented by bed slope at tail end is given downstream boundary condition. Time series of spillway gate opening is given as internal boundary condition for inline structure. Initial flow condition provided to avoid dry bed situations and stability of the model. Initial reservoir elevation is also given at the spillway crest level to represent storage volume of the reservoir. The initial river station elevation is defined as top of dam crest for the cross section just upstream of inline structure.

a. Computational time step

For the stability and accuracy of model, the selection of computational time step is very important task especially for dam break model. The time step longer than the suggest by courant condition may cause model instability especially in steep reach because in steep reach, there is a rapid change in hydraulic properties of the cross-section and large time step given for such reach may cause over estimation of time based derivatives leading to model instability. So, the solution for such steep reach is to decrease the time step. When the model is stable but the given time step is longer and that will cause numerical reduction in the peak of the hydrograph which cannot be related physically. So, computational time step is selected as per the condition of courant. The selection of too small time step than the suggested by courant condition will make model to run longer time than necessary and finally, model stability problems are faced, HEC-RAS manual (2010). Hence, it is very necessary and best way to select the proper computational time step satisfying the courant condition. The courant condition is described in following way.

$C = V_w * (dt/dx) \leq 1$ (4.16)

So,

$dt \leq (dx/V_w)$ (4.17)

Where

C= Courant Number Value equal to or less than one.

V_w = Speed of flood wave.

dx = Cross-section spacing.

dt = Computational time step

The speed of flood wave generated after dam break can be estimated by following simple relation of discharge, area and velocity.

$V_w = dQ/dA$ (4.18)

Where, dQ is the change in discharge ($Q_2 - Q_1$) over the change in area ($A_2 - A_1$).

Practically, the courant condition is applied by taking maximum average velocity from HEC- RAS and this velocity is multiply by a factor 1.5 for natural channel. However following table 4.2 describes the ratio of speed of flood wave to average velocity.

Table 4. 2: Ratio of flood wave speed to average velocity as per shape of channel

S.n.	**Shape of Channel**	**Ratio of Speed** (V_w/V_a)
1	Triangular	**1.33**
2	Wide Rectangular	**1.67**
3	Wide Parabolic	**1.44**
4	Natural	**1.5**

(Source: HEC-RAS manual, 2010)

The time step selected by above method may be very large or very small causing the model instability. The practical aspect of selection of time step obtained by courant condition is further modified by following relation.

$dt \leq T_r/20$ for medium to large rivers (4.19)

Note: T_r is the time of rise of hydrograph.

For dam break study, practical time step range is in between one to sixty second because of very fast wave speed and very short time of raise of flood wave, HEC-RAS manual (2010). So, time step selected from courant condition is further reduced by above relation.

b. Modeling steep streams

Steep streams are generally located in hilly zone with steep slope and higher velocity with smaller depth. So, probably, the flow is super critical. So, for the dams located in the hilly area where the slope of the stream is steep, the flow is super critical and it requires the good judgement for the estimation of the value of manning's **"**n**"** value. For such steep reach, the smaller value of causes the higher velocity and smaller depth as well as rapid change in value of "n" causes model stability problems**.** For such steep streams, Dr. Robert Jarrett **(**Jarrett, 1984**)** developed the regression equation for the computation of **"**n**"** value, taking the data of 75 events at 21 locations for each event (HEC-RAS manual, 2010).

$n= 0.39\, S^{0.38}\, R^{-0.16}$

Where,

n= Manning's roughness coefficient the main channel.

S= Energy slope.

R= Hydraulic radius of the main channel.

From these experiment, the maximum value of **"**n**"** that can be taken up to 0.2 for such steep stream.

c. Theta weighing factor

The range of value of this factor ranges from 0.6 to 1. The model becomes the most stable near value of 1 but the results are comparatively less accurate but trend of model is more instable but the results are comparatively more accurate. This theta weighing factor is used to finite difference approximation for the solving of unsteady flow equation. So, the value of 0.75 is provided for this study.

d. Inline structure and lateral structure stability factors

For the stability of the model, inline stability factor is chosen three and lateral stability factor is chosen as two.

4.3.3 Two-dimensional (2-D) model Set up

Hydrologic Engineering Centre River Analysis System (HEC-RAS) added new function for two-dimensional flow analysis (released in 2014). The fundamental concept underlying 2D modeling is to discretize the river and adjacent floodplain areas into a collection of individual cells called grid cells, 2D flow cells, or computational grid cells. Each grid cell contains elevation and roughness data to represent the ground surface elevation and friction effects along the ground surface. HEC-RAS uses a combination of a finite-difference and a finite-volume method to compute water elevation at the center of each computational grid cell for each computational time step. 2D modeling features in HEC-RAS allow a user to create computational mesh. In the Geometric Data Editor, the modeler can define the limits of the computational mesh that envelopes the channel itself plus any adjacent floodplain areas. Spatial details describing the polygon can be defined with 2D Flow Area Editor button. Spatial details include the size of the individual 2D flow cells as well as Manning's roughness values for each cell. It needs to add break-lines. The break lines are any high ground that users want to align the mesh faces along such as levees and roads and we can add these lines manually. HEC-RAS will keep water out of the "dry" side of a break line until the water surface elevation is greater than the elevation of the break line. The 2D Area Break Lines button can be used add these lines into the terrain background. After the spatial details of the computational mesh and after any break lines have been defined, then detailed information describing the computational grid including hydraulic property table can be generated. There are tolerance input boxes that allow the user to have some control of the 2D grid. Finally, boundary conditions at the upstream and downstream ends using must be defined using 2D Area BC Lines. The 2-D geometry is consists of computational mesh and each mesh is consists of interconnected cells of varying shape and size. The maximum cell faces (sides) is limited to eight and the cell face behave as cross-section in 1-D and computing flow between the cells except outer boundaries of mesh. The benefit of using 2-D model is that it produces a continuous floodplain as it represents the topography as a series of mesh elements and able to model flow in both lateral and longitudinal direction resulting in a more accurate representation of the flood plain. For this study, computation point spacing for longitudinal and lateral direction is 25 m taken so that computed maximum and minimum cell size 1209.30 m^2 and 501.65 m^2 respectively whereas average cell size is 625.93 m^2. The following fig. 4.7 shows the mesh computation for 2-D model of HEC-RAS and fig.4.8 shows u/s boundary condition for 2-D model.

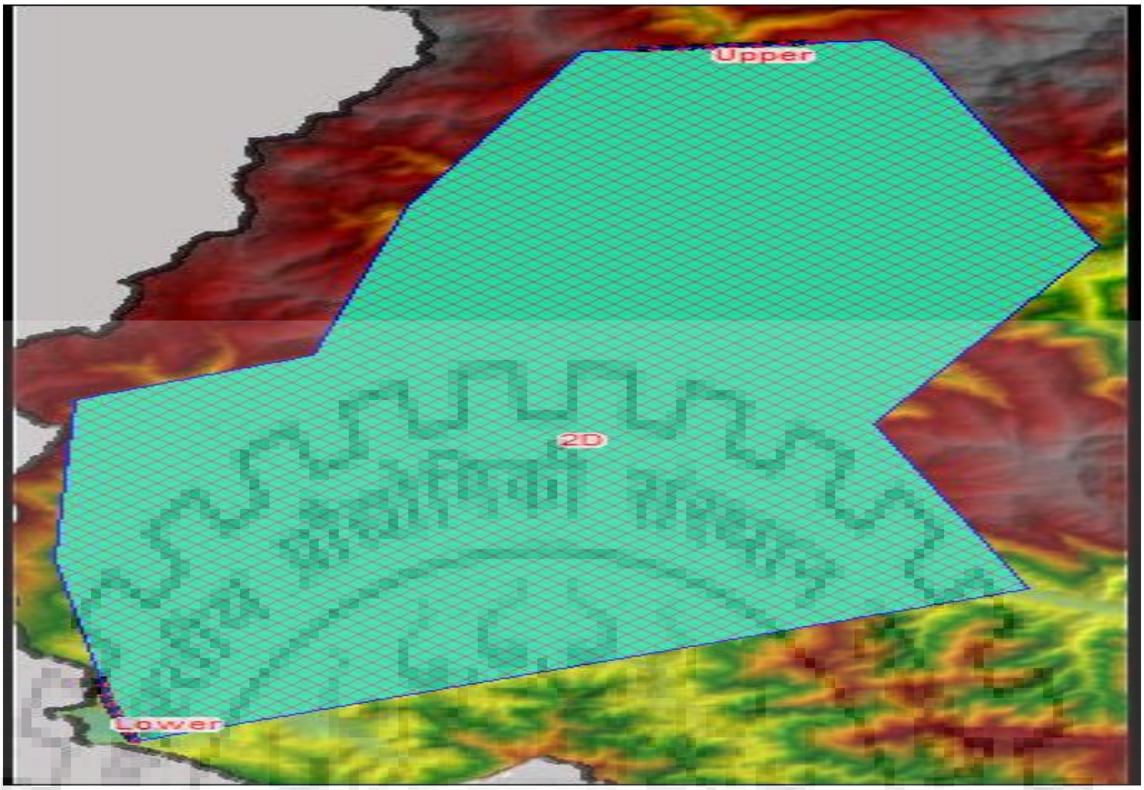


Fig. 4.6: Mesh computation for HEC-RAS 2-D model

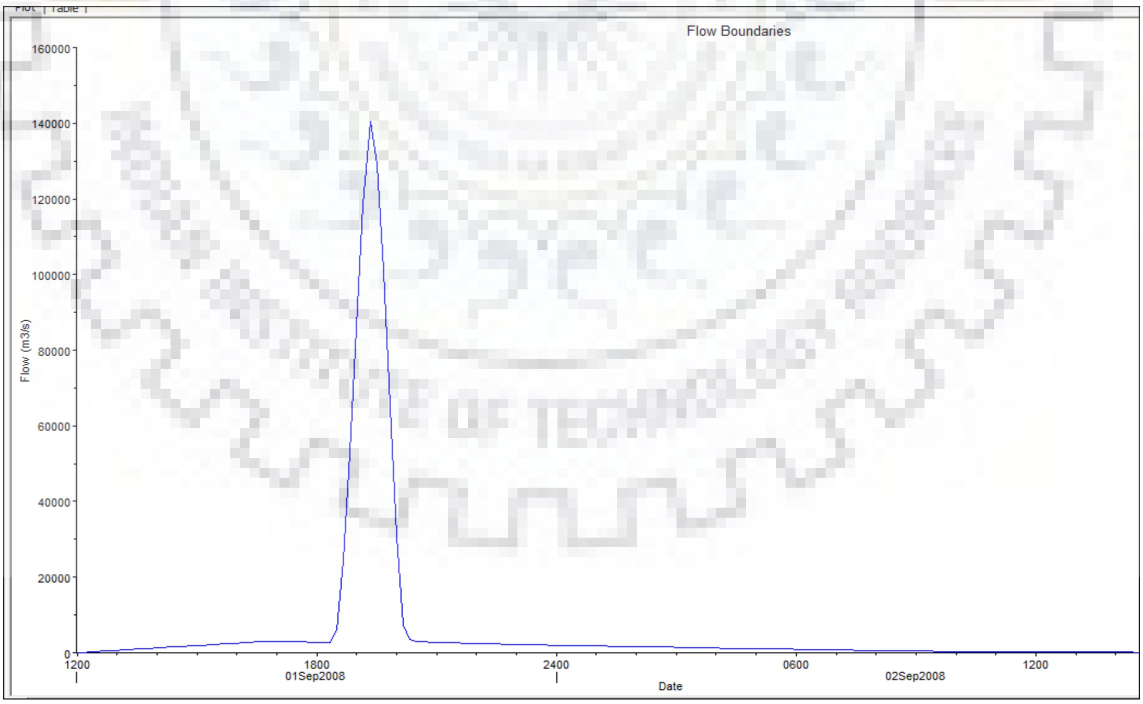


Fig. 4.7: Hydrograph for u/s boundary condition for 2-D analysis

CHAPTER-5

RESULTS AND DISCUSSION

5.1 Analysis of dam breach parameter

The breach geometry and breach formation time are selected using five regression equation as mentioned in methodology is given in in table 5.1. Generally, the breach parameter is selected by taking the average value as computed in above table that should be fit with site condition. Here, in this study the length of Nalgad hydro dam is 546 m and dam height is 200 m. It is seen that crest length is smaller so that for such high dam the bottom with is obviously small. The estimated average value of bottom breach width, Side slope are given in table 5.1. Corresponding breach is plotted along with the cross-section of dam as shown in fig. 5.1. Practically such conditions may not occur practically. So, the final breach geometry is selected as trapezoidal shape having Bottom width 42.5 m, Side slope 0.6: 1 (H: V) such that top breach width does not exceeds the crest length of the dam fitting with site condition but the breach formation time is taken as average value of failure time 2.26 hour as computed in table 5.1. The adopted final breach formation and its progress with time is shown in Figure 5.2

The dam break analysis of Nalgad Hydro Dam is done using one dimensional flow analysis in HEC-RAS model and the velocity, maximum water surface elevation, peak discharge and arrival time computed at the specified location is further compared with two dimensional flow analyses in HEC-RAS model setup. Similarly, flow inundation mapping is also compared computed by 1-D and 2-D.

Table 5.1: Breach parameter calculated by different regression equation

S.N	Regression Equation	Average Breach Width (m)		Breach Bottom Width (m)		Time of Failure (Hrs)		Side Slope (H:1V)		Remarks	
		Over flow	Seepage	Over flow	Seepage	Over flow	Seepage	Over flow	Seepage	Over flow	Seepage
1	Froehlich (1995)	414	283	134	103	0.86	0.80	1.4	0.9		
2	Froehlich (2008)	260	192	60	52	0.61	0.57	1	0.7		
3	Von Thun and Gillette (1990)	555	529	455	429	4.25	4.04	0.5	0.5	erosion resistant	erosion resistant
						3.00	2.85			easily erodible	easily erodible
						0.69	0.70			erosion resistant	erosion resistant
						0.64	0.65			easily erodible	easily erodible
4	Mac Donald and Langridge-Monopolis (1984):	-	-	34	19	5.81	5.48	0.5	0.5	Equation gives directly bottom width	
5	Xu and Zhang (2009):	519	304.00	310.00	180.00	Time of failures can not be used due to high value					
	Average Value	437	327	198.6	156.6	2.26	2.16	0.85	0.65		

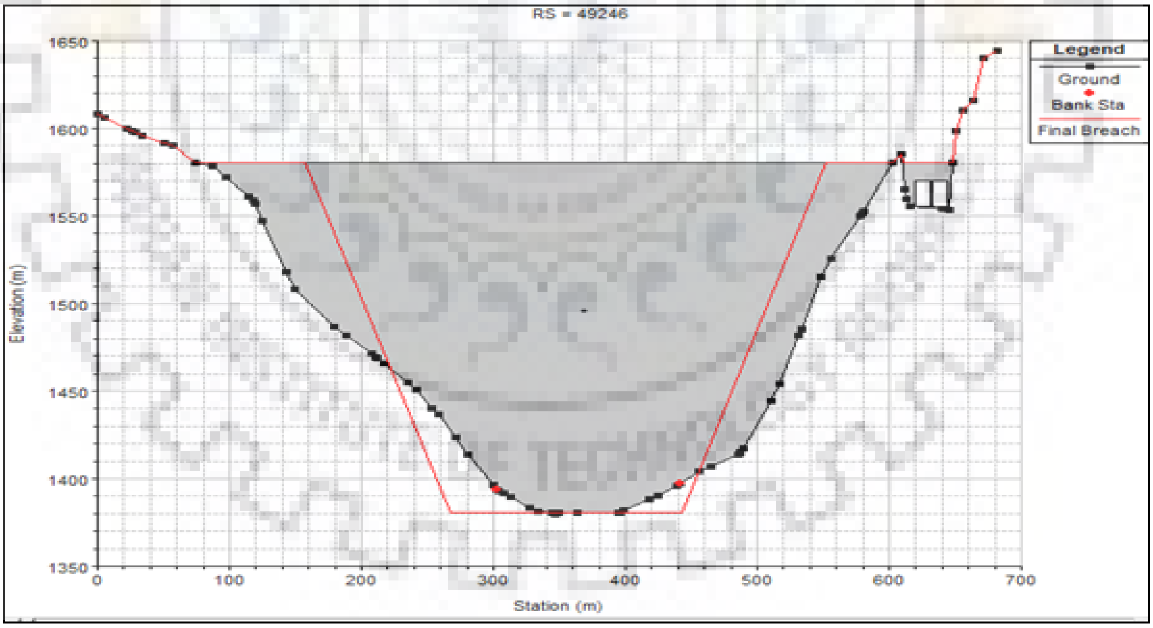


Fig. 5.1 Average breach width not matching with site condition.

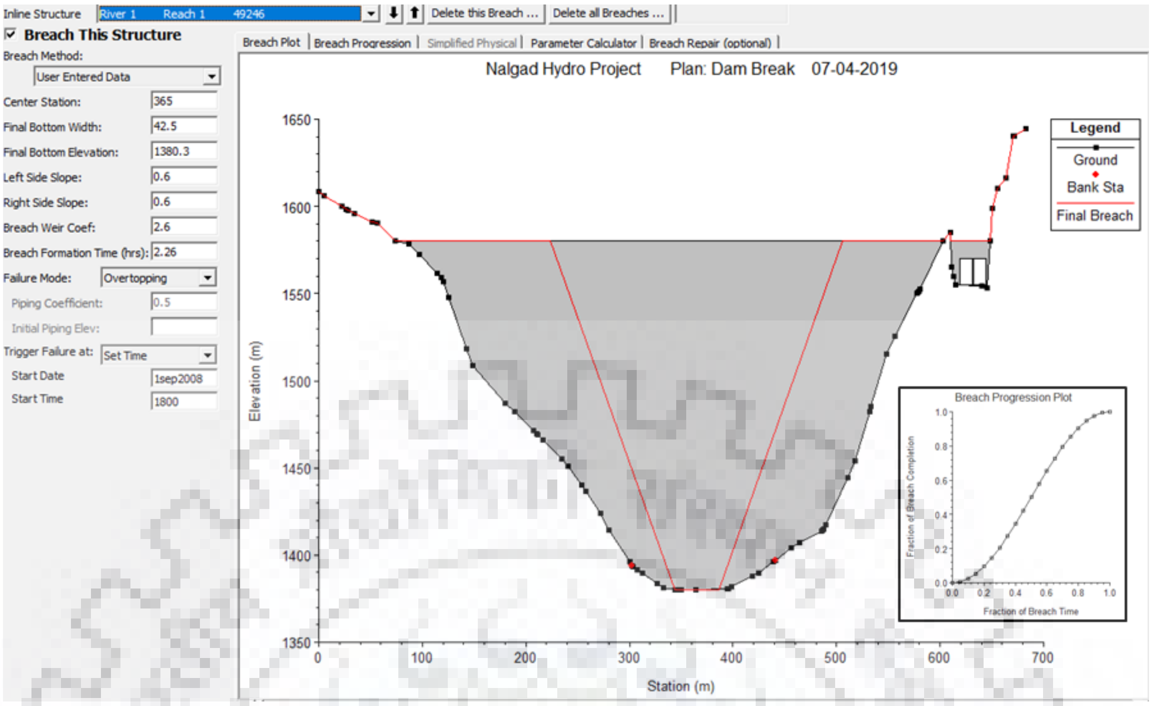


Fig.5.2 Plot of final breach and its progression in time duration

5.2 One dimensional flow analysis

Whenever spillway capacity is inadequate to safely pass the incoming excess flow in such a case overtopping of dam crest by water is began and ultimately, leading to the failure of the dam and the failure associated with such case is said to be overtopping failure. The spillway capacity of Nalgad Hydro Project is 2880 m3/s and incoming PMF is 3523 m3/s and when the PMF of such or greater magnitude enters into the reservoir, the spillway is not able to pass such huge magnitude of flood causing overtopping mode of failure.

The 1-D HEC-RAS model is allow to run and breach the dam such that the resulting breach hydrograph after dam break in 1-D analysis and routing of that hydrograph at specified locations down streams are shown in dam break case. The maximum dam break flood computed is 140350.4 m^3/s and it is seen that which is 39.84 times higher than the PMF of 3523 m^3/s in dam break case. This 1-D dam break scenario is further compared with two other 1-D scenarios namely natural flow i.e. no dam case and without dam break case. The results like hydrograph and routing of that hydrograph downstream of dam and maximum water surface elevation, velocity, peak discharge and arrival time are computed by natural flow case, dam break case and without dam break case are described separately in following way.

a. Natural flow case

The PMF hydrograph is used for the simulation of natural flow i.e. No dam case. The computed hydrograph at proposed dam axis and routing of that hydrograph is shown in fig. 5.3 below and similarly the water surface elevation, velocity, peak discharge, arrival time etc. shown in table 5.2 below.

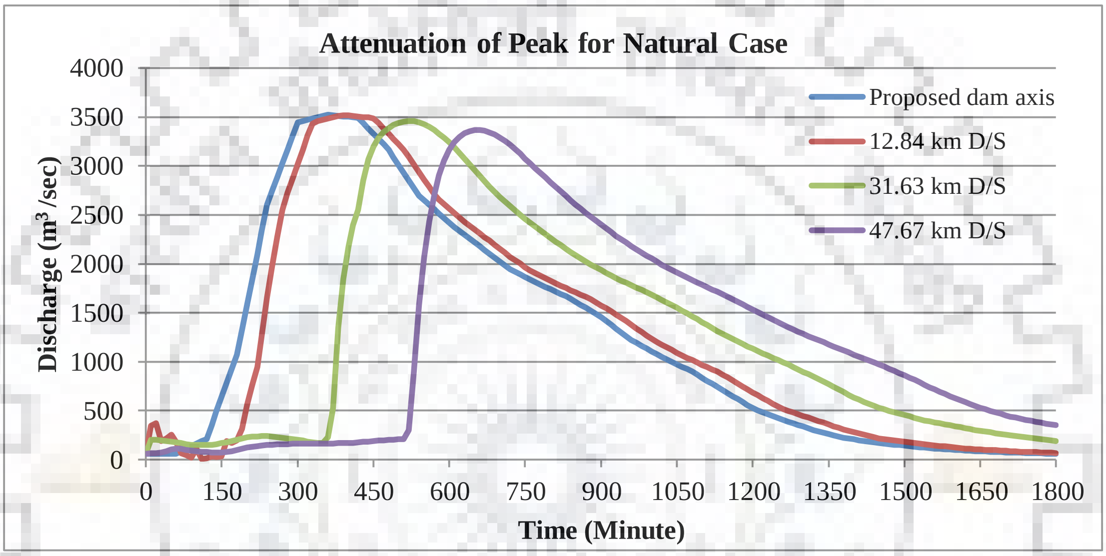


Fig. 5.3: Hydrographs and its routing downstream for natural flow case

Table 5. 2: Summary output tables by profile for natural flow case i.e. no dam

Dist. From Dam	River Station	Dis. (Q)	Min. chnl elev.	W.S. elev	E.G. elev.	Time of arrival	Chnl. vel.
Km		(m^3/s)	(m)	(m)	(m)	(Hour)	(m/s)
	49246						
0.51	48740.2	3522.96	1373.09	1384.13	1385.84	6.00	5.8
1.98	47265	3522.27	1300.12	1309.05	1309.34	6.10	2.45
11.31	37217	3521.11	885.88	895.64	897.05	6.30	5.27
12.84	35681.4	3520.93	827.02	839.26	840.49	6.30	4.9
14.36	34157.4	3519.76	799.03	818.57	818.57	6.40	4.4
22.91	24875.1	3512.06	725.69	740.2	740.64	7.30	2.93
31.63	16164.9	3446.35	690.92	704.71	704.77	8.40	1.06
35.59	12206	3416.65	677.36	688.4	688.5	9.20	1.36
43.12	4673	3384.86	650.08	659.31	659.55	10.22	2.18
47.66	130	3370.53	619.12	631.63	631.71	11.00	1.25

The reduction in discharge from 3522.96 m^3/s to 3370.53 m^3/s occurs from distance 0.52 km d/s to 47.66 km at tail end i.e. attenuation of discharge by 4.33 % .The water surface elevation varies from 1384.13 m to 631.63 m and similarly, velocity of flow varies from 1.06 m/s to 5.8 m/s and time of arrival also varies from 6 hour to 11 hour i.e. after 5 hour reaches the peak discharge at tail end.

b. Dam break case

After selecting breach parameter, the inflow hydrograph is used as u/s boundary condition and slope of last reach is used as d/s condition, area elevation curve is used to define reservoir and dam is in line at station. no. 49246, after inputting breach data, model is run for the simulation of the unsteady flow simulation. The breach hydrograph and routing of that hydrograph at selected location is shown in fig. 5.4 and water surface elevation is shown in fig. 5.5 below. Similarly, X-section area holding peak flood is shown in fig.5.6 and flood inundation computed by 1-D is shown in fig. 5.7 Longitudinal profile showing maximum water surface elevation is shown in fig.5.8 and similarly, peak discharge, water surface elevation, velocity of flow and arrival time is shown in table 5.3 below.

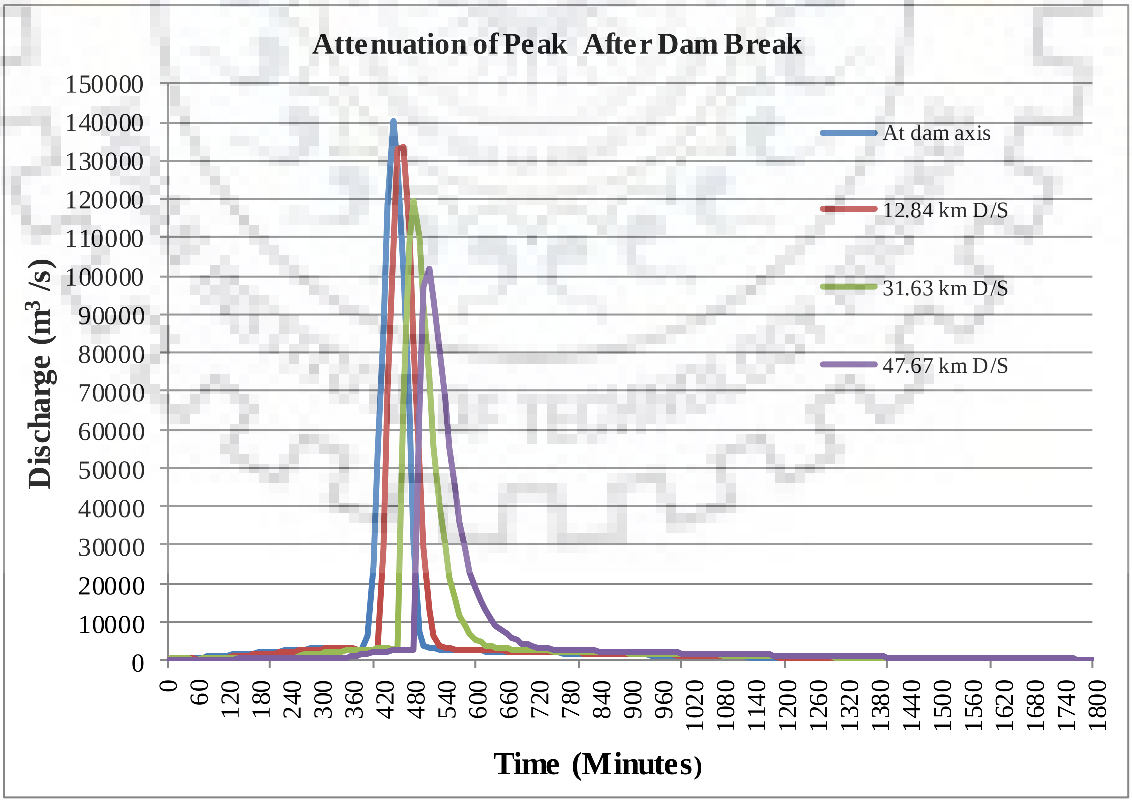
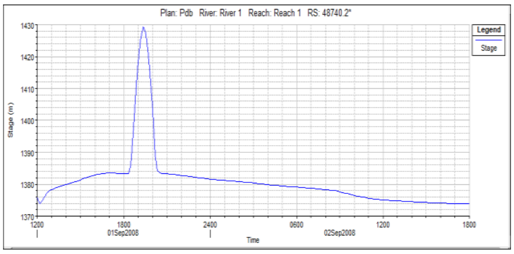
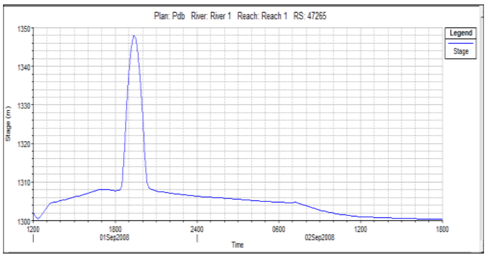


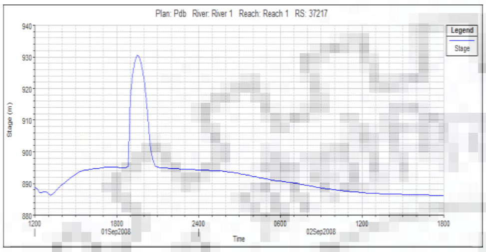
Fig. 5.4 Breach hydrograph and its routing downstream for dam break case



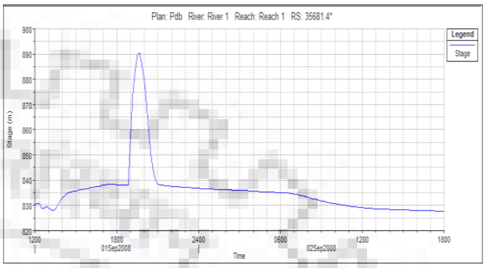
At distance 0.51 km



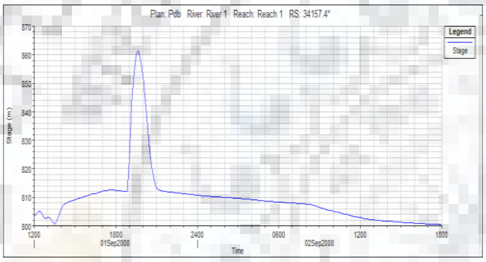
At distance 1.98 km



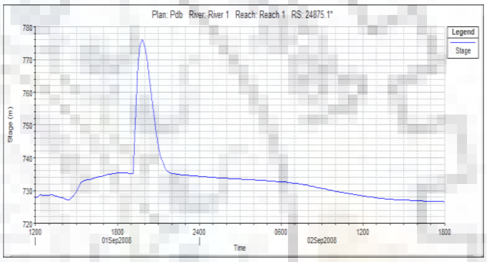
At distance 11.98 km



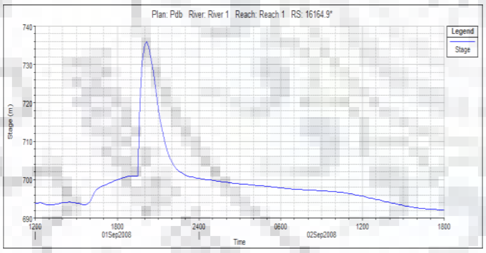
At distance 12.84 km



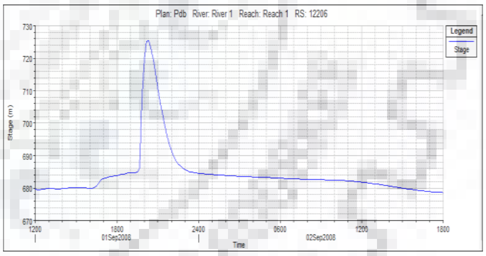
At distance 14.36 km



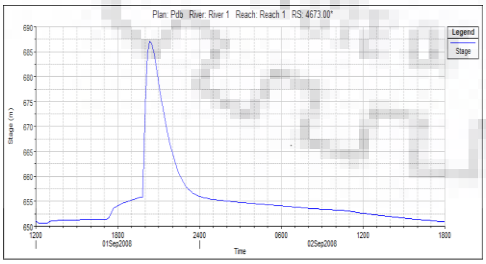
At distance 22.91 km



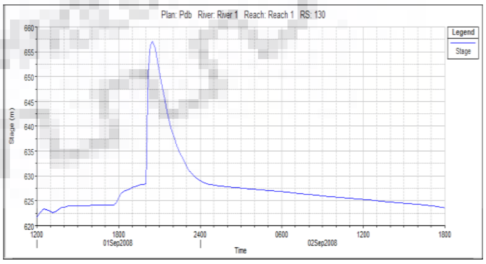
At distance 31.63 km



At distance 35.59 km

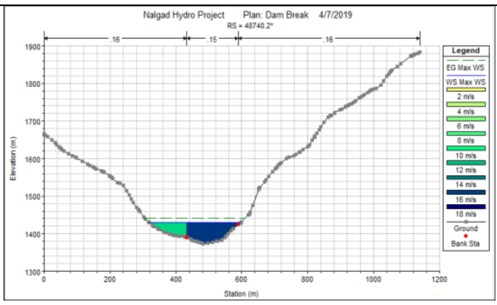


At distance 43.12 km

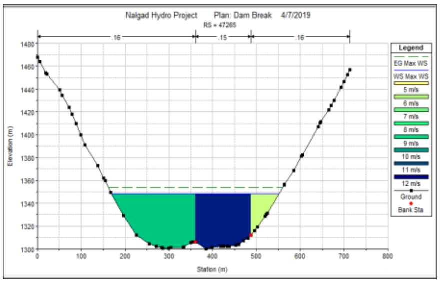


At distance 47.66 km

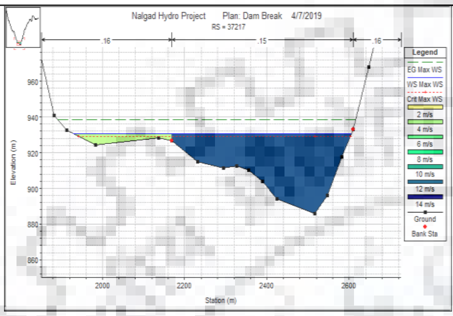
Fig. 5.5 Stage hydrograph at selected 10 locations.



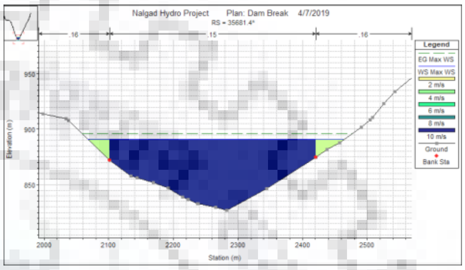
At distance 0.51 km



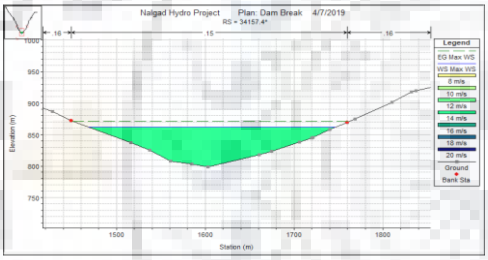
At distance 1.98 km



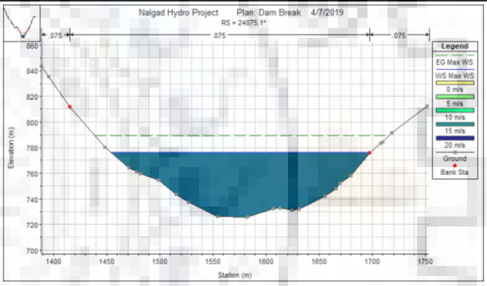
At distance 11.98 km



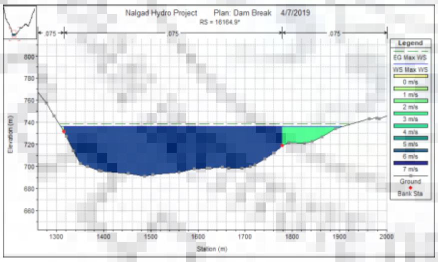
At distance 12.84 km



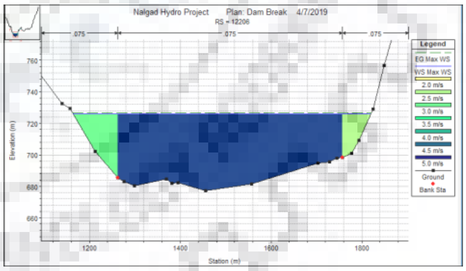
At distance 14.36 km



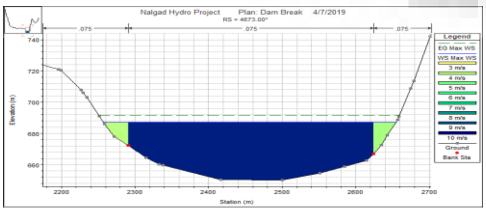
At distance 22.91 km



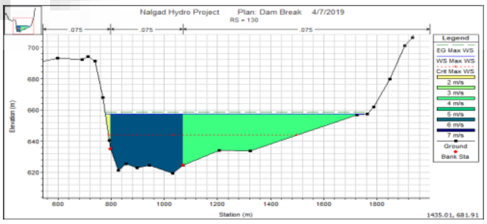
At distance 31.63 km



At distance 35.59 km



At distance 43.12 km



At distance 47.66 km

Fig. 5.6: Cross-section holding peak flood at 10 selected locations.

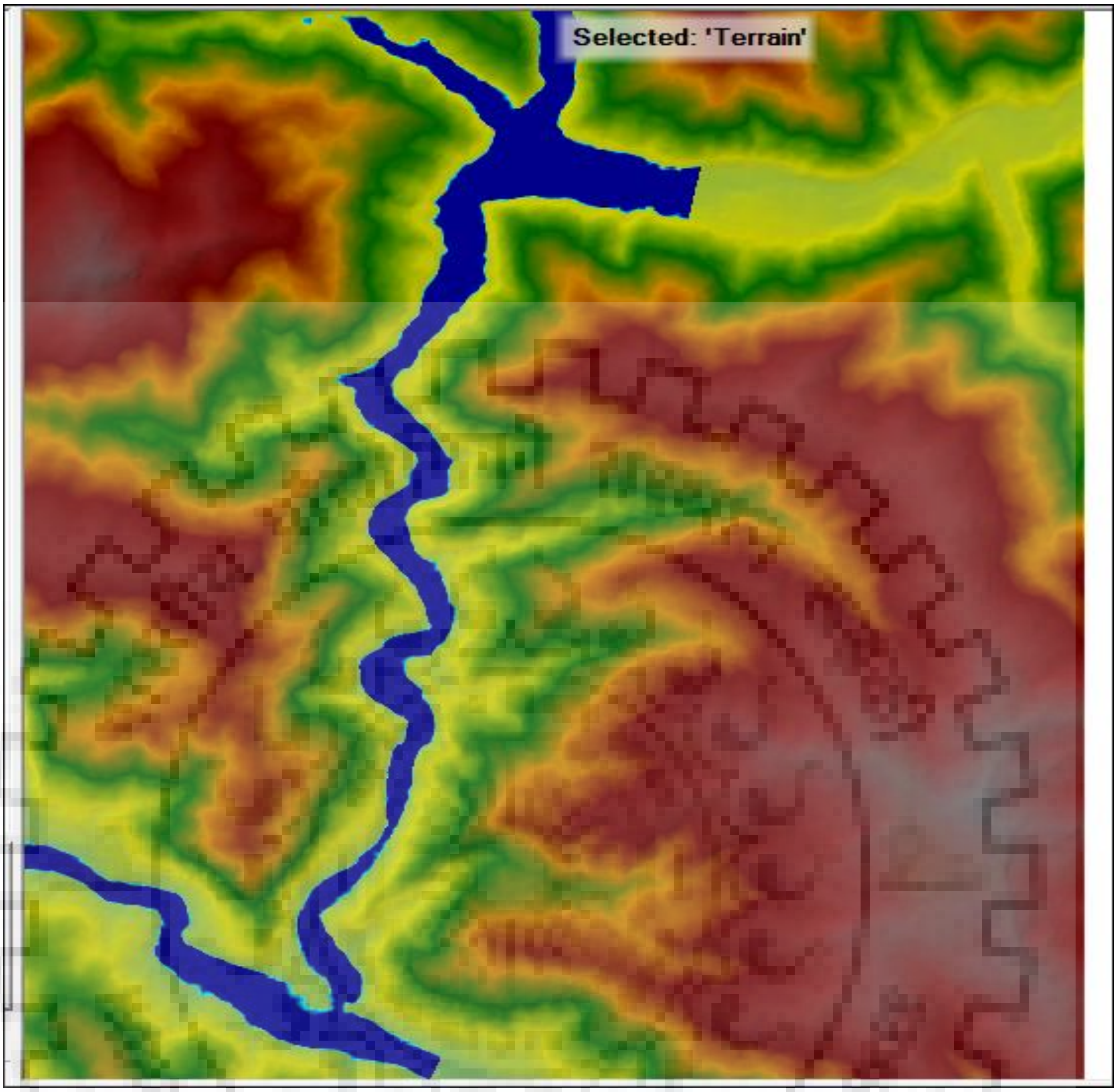


Fig. 5.7 Flood inundation map computed by 1-D

.

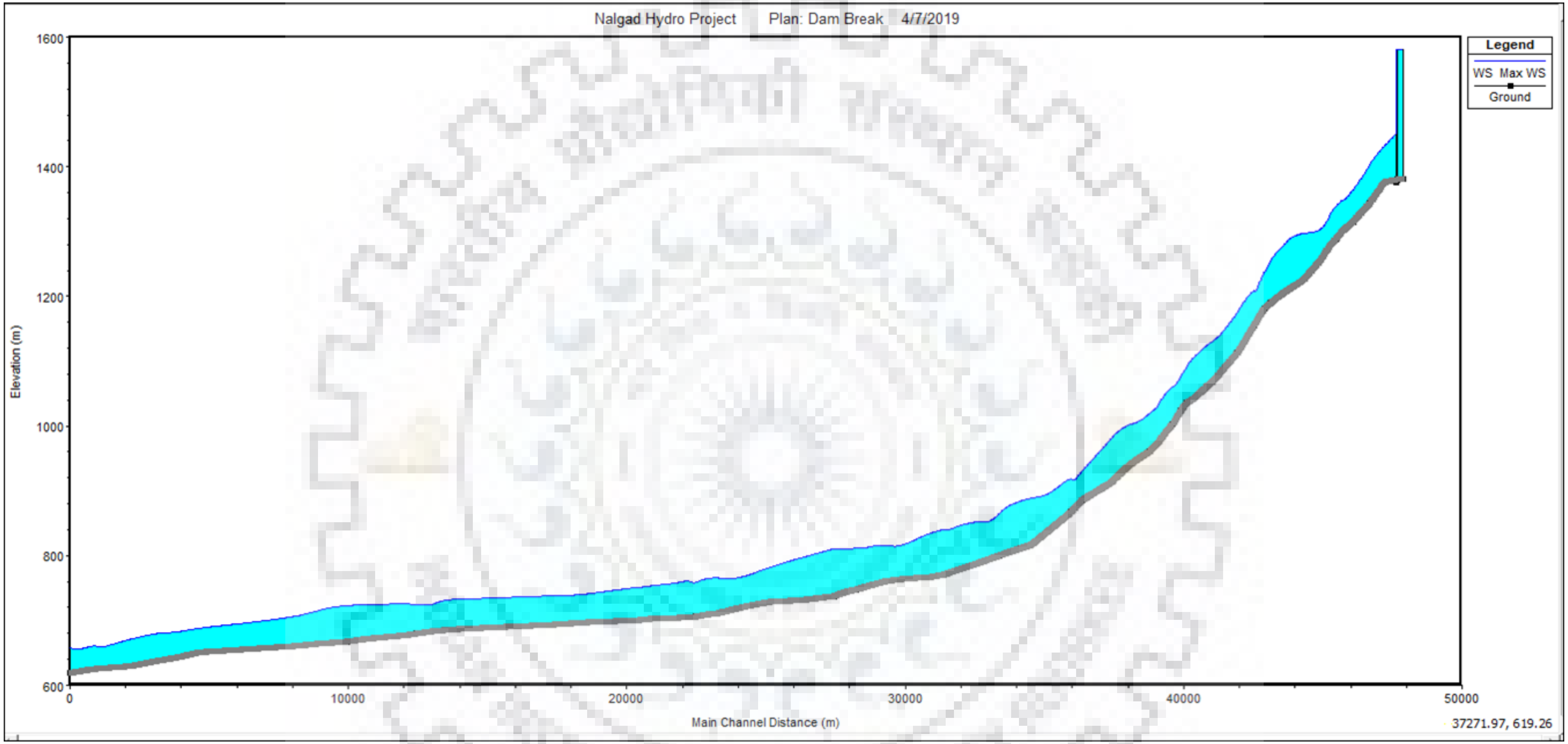


Fig. 5.8 Longitudinal profile showing maximum water surface elevation

Table 5.3: Summary output tables by profile for dam break case

Dist. From Dam	River Station	Discharge (Q)	Min. chnl. Ele.	W.S. ele.	E.G. ele.	Time of arrival	Chnl vel.
Km		(m^3/s)	(m)	(m)	(m)	(Hour)	(m/s)
49246							
0.51	48740.2*	140032.5	1373.09	1429.43	1441.29	7.20	16.29
1.98	47265	139801.4	1300.12	1348.51	1353.94	7.20	11.62
11.31	37217	138258.5	885.88	930.74	938.52	7.30	12.5
12.84	35681.4*	136928.8	827.02	890.91	895.72	7.40	9.77
14.36	34157.4*	136616.1	799.03	861.63	871.4	7.40	13.85
22.91	24875.1*	130862	725.69	776.04	789.61	7.50	16.31
31.63	16164.9*	112231.3	690.92	736.01	738.02	8.00	6.37
35.59	12206	105330.3	677.36	725.73	726.81	8.10	4.74
43.12	4673.00*	103186.7	650.08	687.24	691.83	8.20	9.59
47.66	130	102121.3	619.12	657.21	658.65	8.30	6.2

The fig. 5.4 represents the attenuation of hydrograph from dam axis to tail end for dam break case which is representation of profile output tables of HEC-RAS model setup for this project and fig. 5.5 shows the water surface elevations at selected reach. The water surface elevation varies from 1429.29 m to 657.21 m from first selected location to tail end location and velocity of flow also varies from 4.74 m/s to 16.29. Similarly, discharge varies from 140032.5 m^3/s to 102121.3 m^3/s i.e. attenuation by 27.08 % and time of arrival varies from 7.20 hour to 8.30 hour i.e. peak flood reaches after 1.1 hour at tail end. The stage and flow hydrographs developed at specified locations downstream of dam axis as shown in above fig. 5.5 shows that the stage of water is in decreasing order as we move downstream from the dam axis. As per fig.5.6, the drawn x-section across the river is sufficient to hold peak flood. The flood inundation map (Fig. 5.7) shows that there is sufficient discharge is released after dam break and L-section containing peak flood i.e. fig.5.8 shows that the velocity is higher steep reach and as the slope becomes comparatively flat velocity of flow gradually decreasing.

c. Without dam break Case

This case is taken to pass the flood water through spillway gate and dam is not in failure mode. For this case **s**eparate hydrograph for spillway is developed by routing the inflow hydrograph using the "Modified Puls" method. For this stored water above sill level to full supply level of spillway is calculated using elevation area capacity curve. Then, outflow discharge above sill of spillway is calculated using simple discharge through orifice formulae. Then, by adding stored water with outflow discharge i.e. Q; total volume is obtained i.e. $(S+Q\Delta t/2)$ at each level. Since, inflow hydrograph is hourly hydrograph, so, Δt is in hour. Then, two graphs elevation versus discharge and elevation versus $(S+Q\Delta t/2)$ is plotted. Then, average inflow volume is calculated for each interval of time and for first interval of time, $(S-Q\Delta t/2)_1$ is calculated using first stored S value above sill of spillway, Q is first outflow discharge for same interval of time. Then, $(S+Q\Delta t/2)$ is obtained by adding value obtained from $(S-Q\Delta t/2)_1$ and average inflow volume of water. With the help of graph elevation versus $(S+Q\Delta t/2)$, for the known value of $((S+Q\Delta t/2)$, corresponding elevation is selected and with this known elevation, corresponding discharge is selected from graph elevation versus discharge. The value of discharge for each time interval calculated following same procedure and hydrograph for without dam break case is prepared shown in table 5.4 below and fig.5.9 below.

Table 5.4: Hydrograph for without dam break case

Time (hr)	0	1	2	3	4	5	6	7	8	9	10
Discharge (m^3/s)	46.7	47	95	195	680	1095	1710	2250	2800	2880	2798

Time (hr)	11	12	13	14	15	16	17	18	19	20	21
Discharge (m^3/s)	2600	2350	2175	2020	1870	1730	1600	1460	1300	1160	1000

Time (hr)	22	23	24	25	26	27	28	29	30
Discharge (m^3/s)	860	740	620	530	420	320	235	140	60

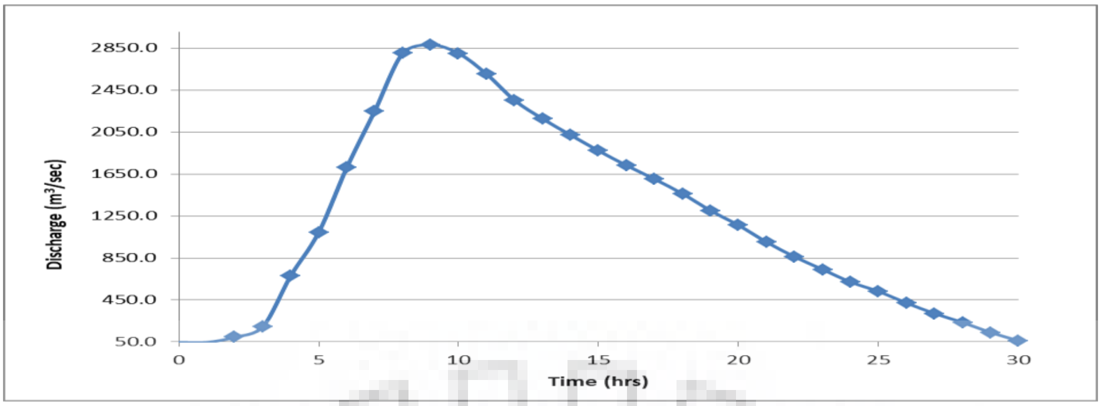


Fig. 5.9 Hydrograph for the case without dam break

Now, using this hydrograph as u/s boundary condition, HEC-RAS model is run and attenuation of peak of hydrograph at selected location is computed as shown in fig.5.11 and similarly, maximum water surface elevation, peak discharge and arrival time is shown in table 5.5 below.

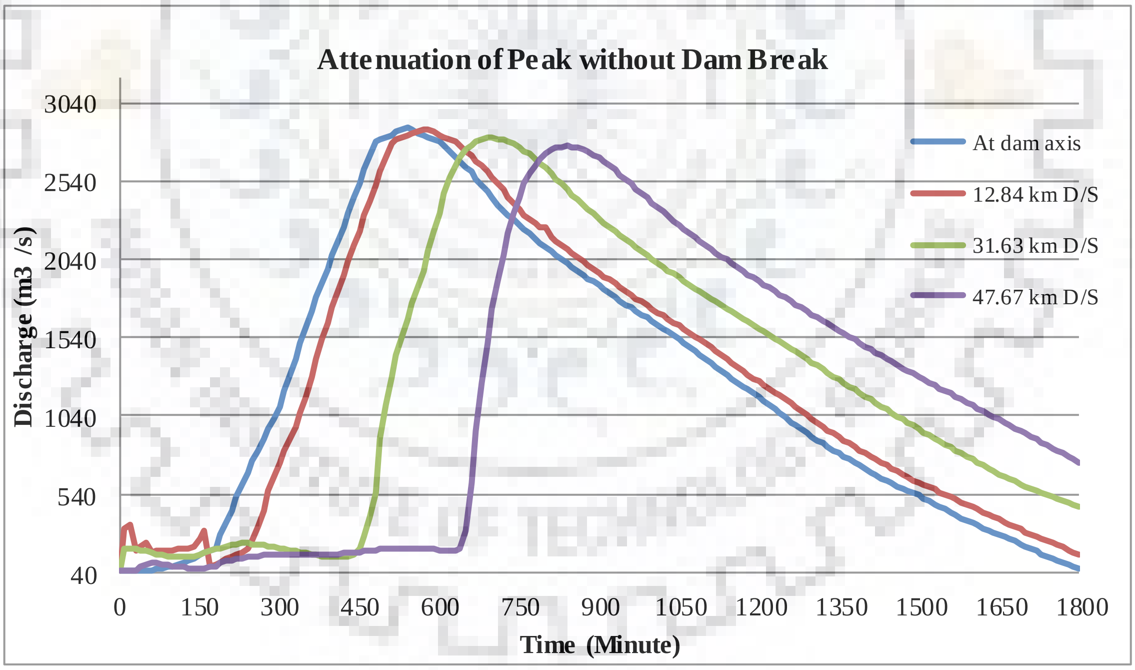


Fig. 5.10 Hydrograph and its routing downstream for without dam break case

Table 5.5: Summary output tables by profile for without dam break case

Dist. From Dam	River station	Discharge (Q)	Min. chnl Elev.	W.S. elev.	E.G. elev.	Time of arrival	Chnl. vel.
Km		(m^3/s)	(m)	(m)	(m)	(Hour)	(m/s)
49246 Dam Axis							
0.51	48740.2*	2879.87	1373.09	1383.38	1384.83	9.00	5.34
1.98	47265	2878.98	1300.12	1307.88	1308.17	9.00	2.47
11.31	37217	2876.78	885.88	895.03	896.23	9.30	4.85
12.84	35681.4*	2876.54	827.02	838.08	839.27	9.30	4.83
14.36	34157.4*	2874.93	799.03	812.29	813.26	9.40	4.36
22.91	24875.1*	2862.24	725.69	739.18	739.55	10.20	2.68
31.63	16164.9*	2810.23	690.92	703.69	703.74	11.40	0.99
35.59	12206	2794.28	677.36	687.48	687.57	12.20	1.29
43.12	4673.00*	2771.72	650.08	658.33	658.56	13.20	2.11
47.66	130	2764.61	619.12	630.75	630.82	14.00	1.17

The discharge varies from 2879.87 m^3/s to 2764.61 m^3/s from distance 0.51 km to distance 47.66at tail end i.e. attenuation by 4.02%. The water surface elevation varies from 1383.38 m to 630.75 m and similarly, velocity of flow varies from 0.99 m/s to 5.34 m/s and time of arrival also varies from 9 hour to 14 hour i.e. peak discharge at tail end reaches after 5 hours.

5.3 Comparison of natural flow, dam break and without dam break case

The three scenarios as mentioned above for 1-D flow analysis are “No Dam” i.e. Natural Flood, “Dam Break” and “without Dam Break” and peak discharge, time of arrival, maximum water surface elevation and the maximum velocity computed for each scenario are compared with each other at specified location for the detection of the more critical flood induced disaster among of three case. The table 5.6 shows the velocity and water surface elevation whereas table 5.7 shows the peak discharge and arrival time. Similarly, fig.5.12 shows water surface elevation of three cases.

Table 5.6: Max. water surface elevation and velocity at specified location for three cases

S.N.	Station No.	D/S distance from dam Axis (Km)	Natural flood		Dam break		Without dam break	
			W.S. elev (m)	Vel chnl (m/s)	W.S. elev (m)	Vel chnl (m/s)	W.S. elev. (m)	Vel chnl (m/s)
	49246	Dam Axis						
1	48740.2	0.51	1384.13	5.8	1429.43	16.29	1383.38	5.34
2	47265	1.98	1309.05	2.45	1348.51	11.62	1307.88	2.47
3	37217	11.31	895.64	5.27	930.74	12.5	895.03	4.85
4	35681.4	12.84	839.26	4.9	890.91	9.77	838.08	4.83
5	34157.4	14.37	818.57	4.4	861.63	13.85	812.29	4.36
6	24875.1	22.91	740.2	2.93	776.04	16.31	739.18	2.68
7	16164.9	31.63	704.71	1.06	736.01	6.37	703.69	0.99
8	12206	35.59	688.4	1.36	725.73	4.74	687.48	1.29
9	4673	43.12	659.31	2.18	687.24	9.59	658.33	2.11
10	130	47.67	631.63	1.25	657.21	6.2	630.75	1.17

Table 5. 7: The peak discharge and arrival time at specified location for three cases.

S.N.	Station No.	D/S distance from dam (Km)	Natural flood		Dam break		Without dam break	
			Peak discharge (m^3/s.)	Time of arrival (hrs.)	Peak discharge (m^3/s)	Time of arrival (hrs.)	Peak discharge (m^3/s.)	Time of arrival (hrs.)
	49246	Dam Axis						
1	48740.2	0.51	3522.96	6.00	140032.50	7.20	2879.87	9.00
2	47265	1.98	3522.27	6.10	139801.40	7.20	2878.98	9.00
3	37217	11.31	3521.11	6.30	138258.50	7.30	2876.78	9.30
4	35681.4	12.84	3520.93	6.30	136928.80	7.40	2876.54	9.30
5	34157.4	14.37	3519.76	6.40	136616.10	7.40	2874.93	9.40
6	24875.1	22.91	3512.06	7.30	130862.00	7.50	2862.24	10.20
7	16164.9	31.63	3446.35	8.40	112231.30	8.00	2810.23	11.40
8	12206	35.59	3416.65	9.20	105330.30	8.10	2794.28	12.20
9	4673	43.12	3384.86	10.22	103186.70	8.20	2771.72	13.20
10	130	47.67	3370.53	11.00	102121.30	8.30	2764.61	14.00

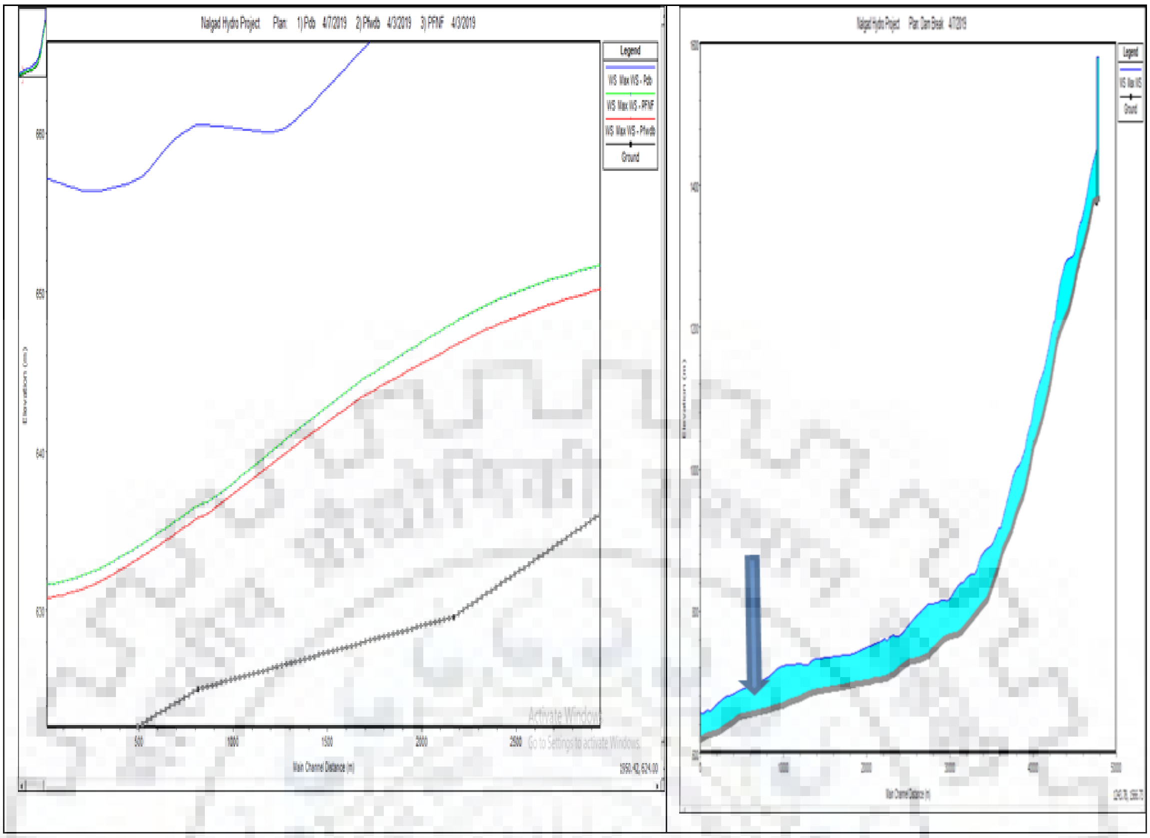


Fig. 5.11 Water surface elevation of three cases

Note,

WS Max WS- Pdb= Maximum water surface elevation for dam break case blue line

WS Max WS- PFNF= Maximum water surface elevation for natural flood case green line

WS Max WS- Pfwdb= Maximum water surface elevation without dam break case for red line

The peak discharge and arrival time computed from HEC-RAS 1-D model is presented in the above table 5.7 for the cases of natural flood, dam break and without dam break and it is seen that the maximum flood discharge after dam break is much more higher than the discharge resulting from without natural flooding case and without dam break case. The arrival time of dam break flood is earlier than the without dam break case in all selected location downstream of the dam axis but arrival time of natural flood is earlier than the dam break flood up to distance 22.91 km at station no. 24875.10. After this station no. , the arrival time of dam break flood is earlier than the natural flood case. Hence, upper reaches are more prone to the risk associated with the natural flood disaster but the flood affected area may be comparatively lower than the dam break flood and the downstream reaches are more prone to risk associated with the dam break flood disaster. However, flood affected area is much more higher caused by dam break flood than the natural flood case and without dam break case. Similarly, arrival time of dam break case is earlier than the arrival time of flood resulting from without dam break case at any station.

Hence, the settlements in the vicinity of the river reach in the upper reach have high level of risk associated with the disaster induced by either dam break flood or natural flood because of lesser warning time available.

The water surface elevation of dam break case is also very high than the natural flooding case and without dam break case. As per table 5.6, water surface elevation of natural flooding case is slightly higher than the water surface elevation resulting from without dam break flood but fig. 5.13 shows the water surface elevation of natural flooding and without dam break cases are looking like coinciding because fig. was not zoomed.

The velocity of flow is also very high in dam break flooding case than other two cases by3-4 times and velocity of flow in natural case is slightly higher than the without dam break case.

5.4 Two-dimensional analysis

Two dimensional 2-D, HEC-RAS model is setup for the comparison of results computed by 1-D and 2-D model and also to identify the condition at which either model givers better results. The attenuation of peak of hydrograph computed by 2-D is shown in fig. 5.12 below and also the water surface elevation and velocity of flow computed by 2-D listed in table 5.8 below and peak discharge and arrival time computed from 2-D are listed in the same table 5.8 below and also fig.5.13 shows the cross-section containing peak flood.

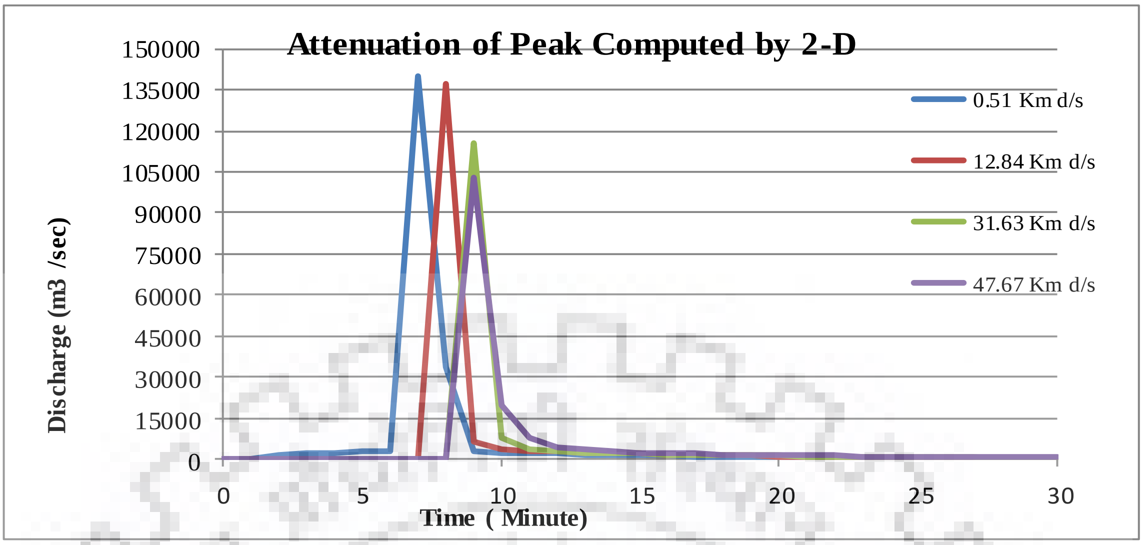
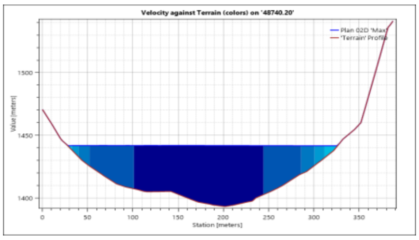


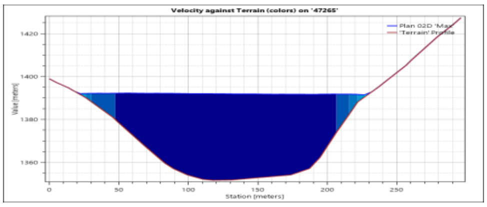
Fig. 5. 12 Attenuation of peak of hydrograph d/S of dam axis computed by 2-D

Table 5.8: Max. water surface elevation, velocity, peak discharge and arrival time computed by 2-D

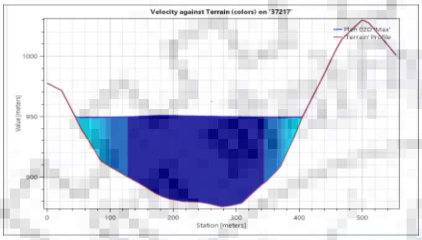
Station No.	D/S distance from dam axis (Km)	2-D			
		W.S. elev (m)	Vel chnl (m/s)	Peak discharge (m^3/s)	Time of arrival (hrs.)
49246		Dam Axis			
48740.2	0.51	1442.32	18.13	140000.60	7.00
47265	1.98	1392.03	15.23	139900.12	7.00
37217	11.31	950.88	18.12	138500.65	7.00
35681.4	12.84	888.65	10.47	137266.45	8.00
34157.4	14.37	887.11	16.12	136900.81	8.00
24875.1	22.91	870.01	16.5	132300.32	8.00
16164.9	31.63	773.42	8.75	115264.94	9.00
12206	35.59	768.68	7.21	106100.2	9.00
4673	43.12	736.09	9.94	103900.5	9.00
130	47.67	732.50	6.92	102600.40	9.00



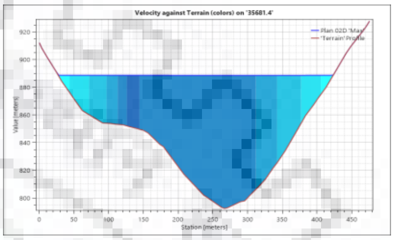
At distance 0.51 km



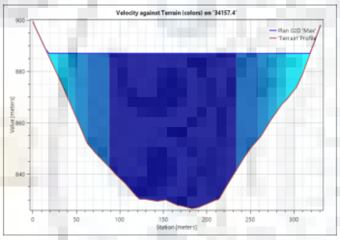
At distance 1.98 km



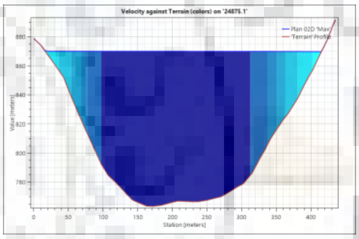
At distance 11.98 km



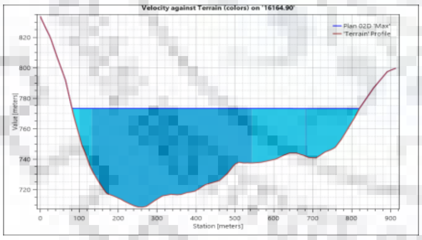
At distance 12.84 km



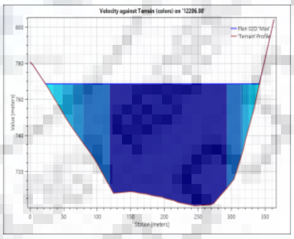
At distance 14.36 km



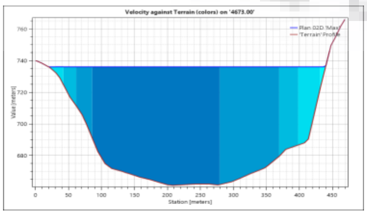
At distance 22.91 km



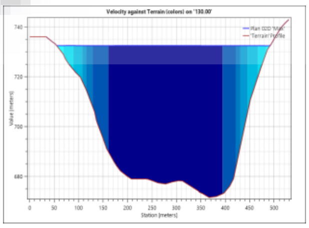
At distance 31.63 km



At distance 35.59 km



At distance 43.12 km



At distance 47.66 km

Fig. 5. 13 Cross-section holding peak flood computed by 2-D

The fig. 5.14, 5.15 and 5.16 below shows the flood inundation mapping computed by 2-D and also fig. 5.17, 5.18 and 5.19 shows the flood affected settlement area computed by 2-D.

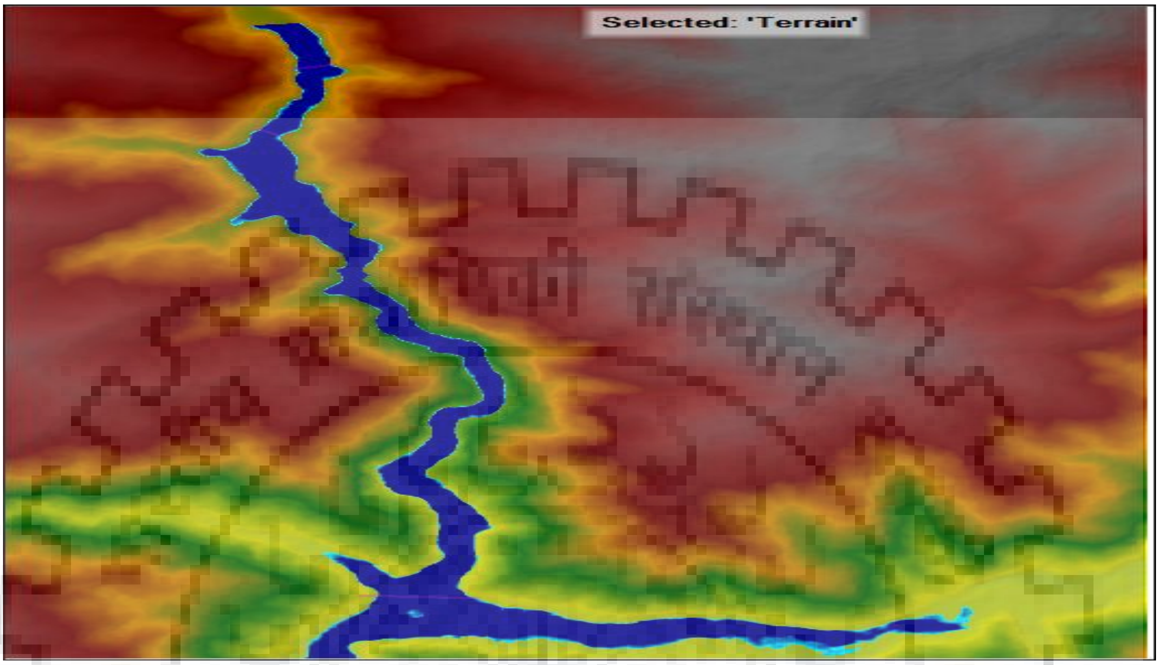


Fig. 5. 14: Flood inundation mapping at upper reach computed by 2-D

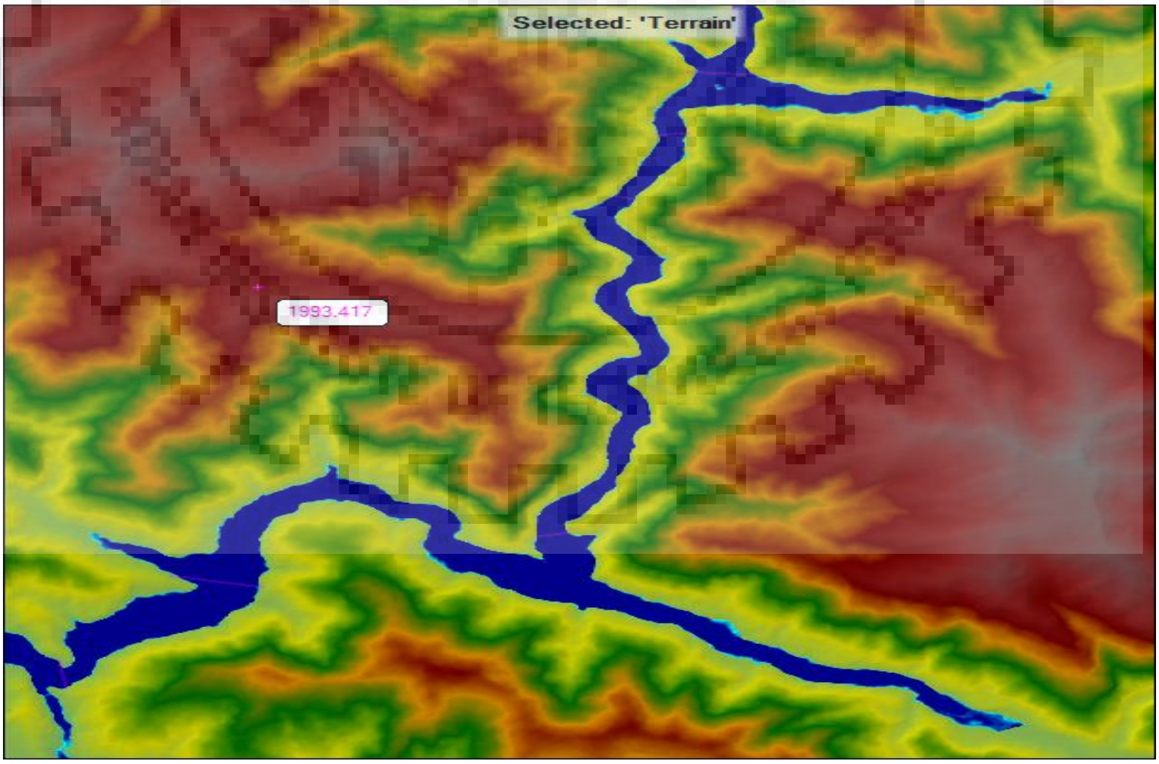


Fig. 5.15: Flood inundation mapping at middle reach computed by 2-D

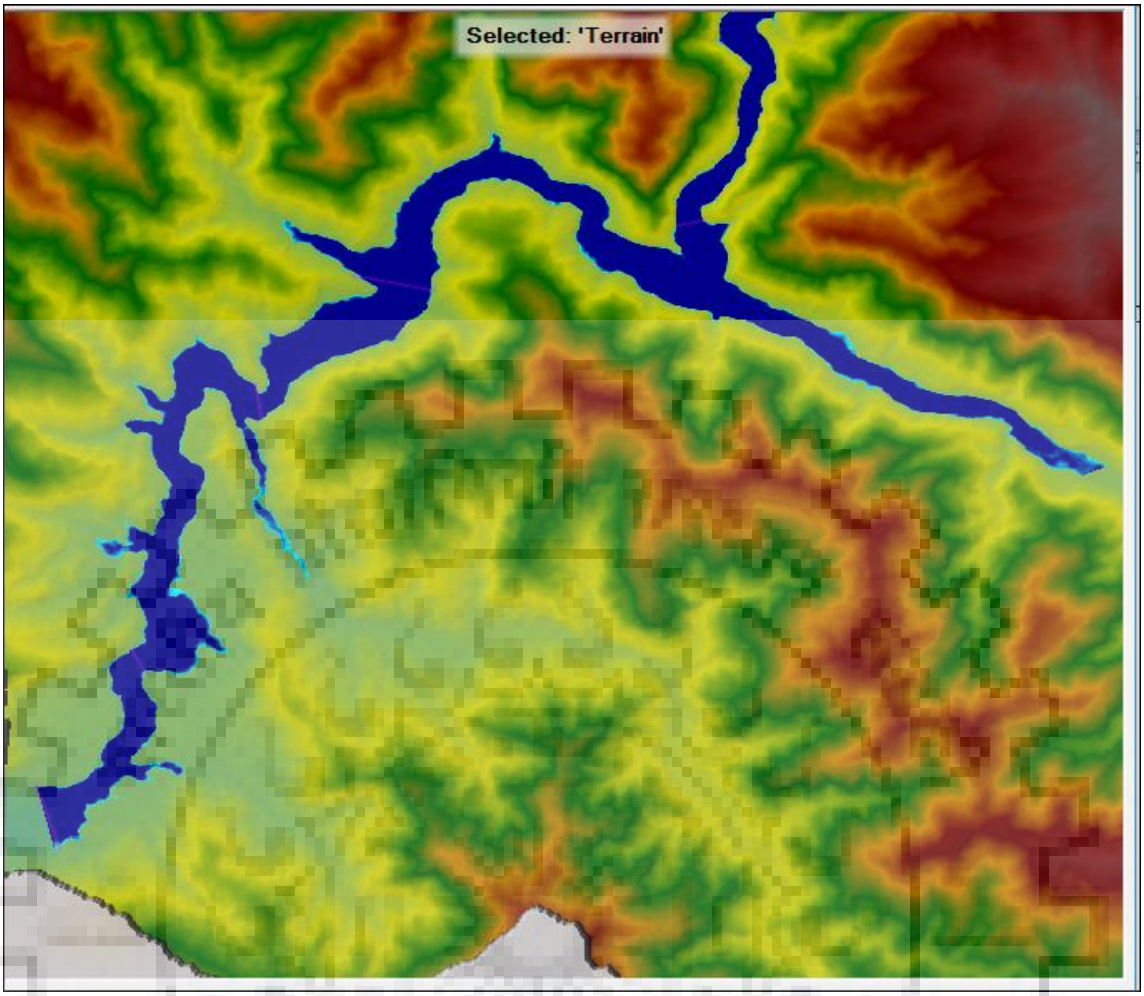


Fig. 5. 16 Flood inundation mapping at tail reach computed by 2-D

Fig. 5. 17 Flood inundation and flow affected settlement area**.**upper reach

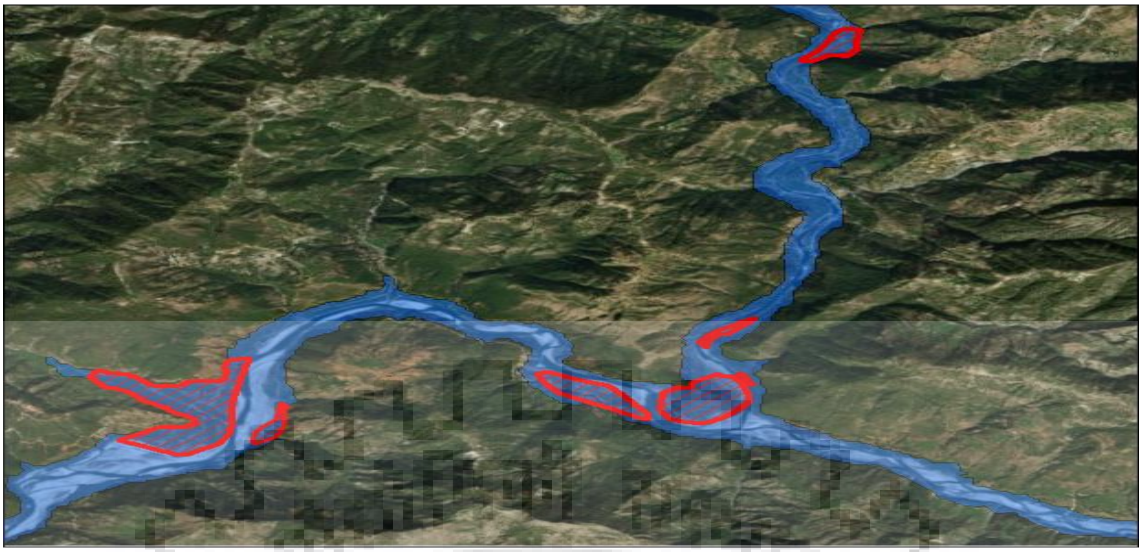


Fig. 5. 18 Flood inundation and flow affected settlement area middle reach

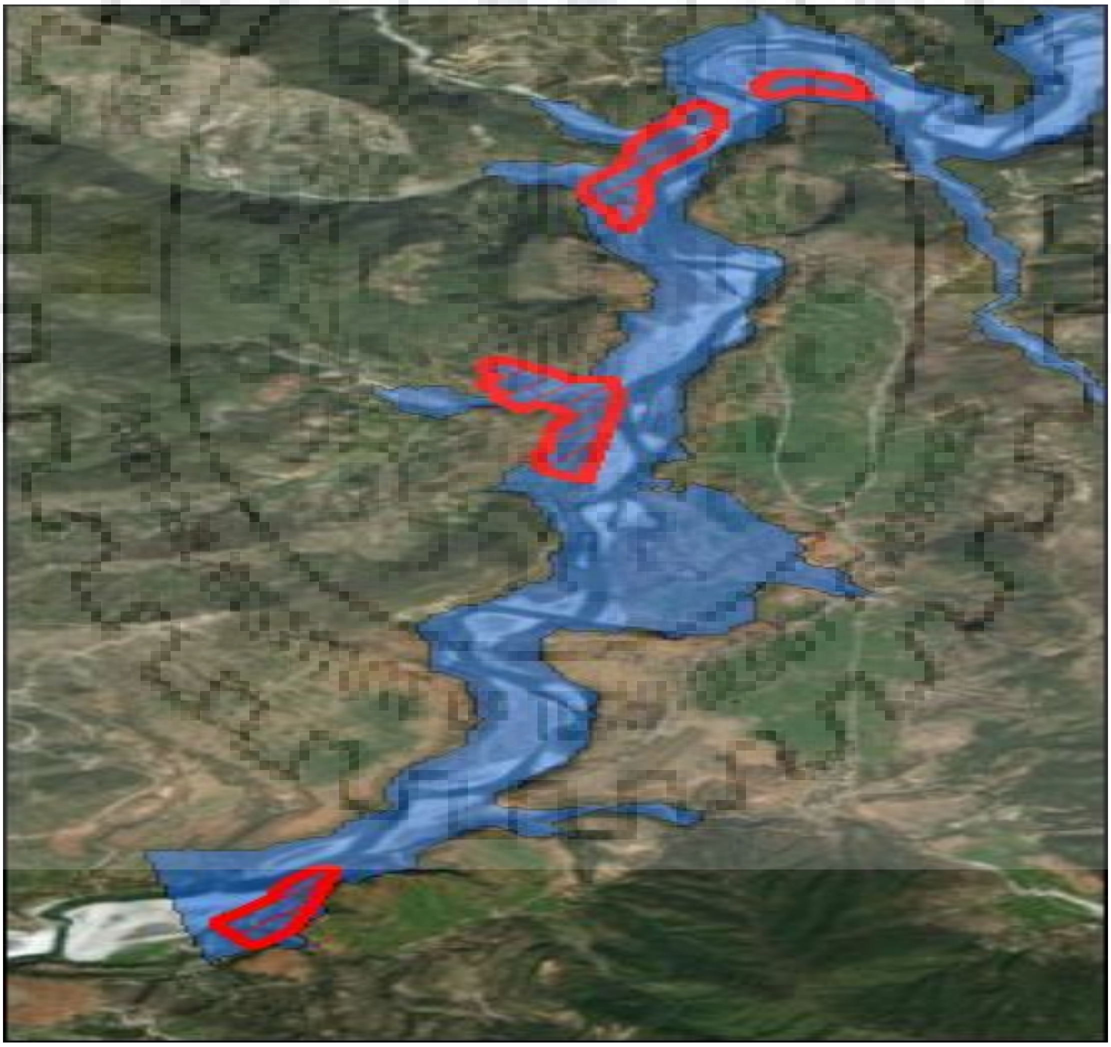


Fig. 5. 19 Flood inundation and flow affected settlement area lower reach

From above table 5.8 it is seen that the water surface elevation changes from 1442.32 m to 732.50 from head to tail of river and velocity of flow ranges from 18.13 m/s to 6.92 m/s. The peak discharge ranges from 140000.60 m^3 /s to 102600.40 m^3 /s i.e. attenuation of peak discharge by 26.71% and time of arrival ranges from 7 hour to 9.00 hour from head to tail i.e. after 2 hours. The flood inundation map shows that the flood inundation is comparatively lower in steep reach than flatter reach and so the flow affected settlement area is also lower in steep reach than flatter reach.

5.5 Comparison of 1-D and 2-D model Results

The main objective of dam break analysis is to compute the maximum water surface elevation, velocity, peak discharge and arrival time of flood to downstream the reaches of dam so that the loss of life from flow affected area may be minimized. The comparison of both models is done to identify the conditions at which, either 1-D or 2-D model gives better results and up to which limits, the results computed by 1-D and 2-D matches. The following tables 5.9 show the water surface elevation and velocity computed by 1-D and 2-D whereas table 5.11 shows the peak discharge and arrival time computed by 1-D and 2-D. Similarly, flow affected settlement area is shown in fig.5.16 (a) and 5.20 below and table 5.21 shows the comparison of flow affected area computed by 1-D and 2-D.

Table 5.9: Max. water depth and velocity computed from 1-D and 2-D.

Cross section	**D/S distance from dam Axis (Km)**	**1-D**		**2-D**	
		Depth (m)	**Vel chnl (m/s)**	**Depth (m)**	**Vel chnl (m/s)**
49246	Dam axis				
48740.2	0.51	56.34	16.29	69.23	18.13
47265	1.98	48.39	11.62	91.91	15.23
37217	11.31	44.86	12.5	65	18
35681.4	12.84	63.89	9.77	61.63	10.47
34157.4	14.37	62.6	13.85	88.08	16.12
24875.1	22.91	50.35	16.31	144.32	16.5
16164.9	31.63	45.09	6.37	82.50	8.75
12206	35.59	48.37	4.74	91.32	7.21
4673	43.12	37.16	9.59	86.01	9.94
130	47.66	38.09	6.2	113.38	6.92

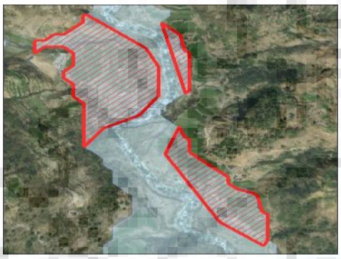
Table 5.10: Peak discharge, arrival time computed from 1-D and 2-D.

Cross section	D/S distance from dam Axis (Km)	1-D		2-D	
		Peak aischarge (m^3/sec.)	Time of arrival (hrs.)	Peak discharge (m^3/sec.)	Time of arrival (hrs.)
49246	Dam axis				
48740.2	0.51	140032.50	7.20	140000.60	7.00
47265	1.98	139801.40	7.20	139900.12	7.00
37217	11.31	138258.50	7.30	138500.65	7.00
35681.4	12.84	136928.80	7.40	137266.45	8.00
34157.4	14.37	136616.10	7.40	136900.81	8.00
24875.1	22.91	130862.00	7.50	132300.32	8.00
16164.9	31.63	112231.30	8.00	115264.94	9.00
12206	35.59	105330.30	8.10	106100.20	9.00
4673	43.12	103186.70	8.20	103900.5	9.00
130	47.67	102121.30	8.30	102600.40	9.00

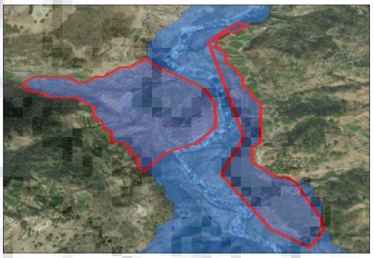
From above table, it is seen that discharge is computed by 2-D at specified location is relatively higher than 1-D expect at station no 48740.2.and also arrival time of peak discharge is in 2-D analysis is earlier than 1-D up to the distance 1.98 & 11.31 Km by 20 minutes 30 minutes respectively and after 11.31 km, arrival time of peak discharge computed by 1-D is earlier than 2-D up to distance 14.37 km and 22.91 km by 20 minutes and 10 minutes respectively. After 22.91 Km, the arrival time of peak discharge computed by 1-D is earlier than 2-D up to the distance 31.63 km, 35.59 km, 43.12 km and 47.67 km by the time 1 hour, 50 minute, 40 minute and 30 minute respectively. Hence, for the settlements up to the distance 11.31 km, it is better and safe to provide warning time based on 2-D computation and after 11.31 km, it is better and safe to provide warning time based on 1-D computation. The water surface elevation and velocity of flow computed by 2-D is more than the computed by 1-D except at the station 48740.2, 35681.4 water surface and velocity of flow is slightly higher in 2-D than 1-D.

At distance 0.51 km computed by 1-D

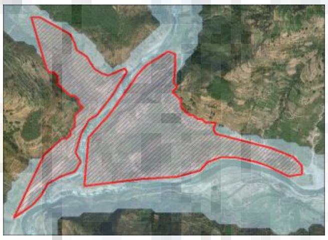
At distance 0.51 km computed by 2-D



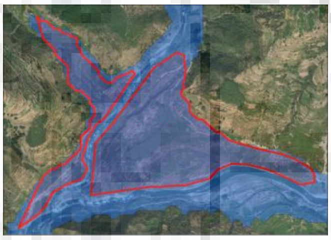
At distance 1.98 km computed by 1-D



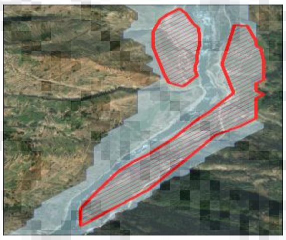
At distance 1.98 km computed by 2-D



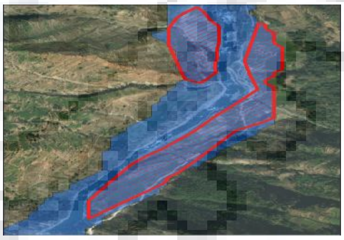
At distance 11.98 km computed by 1-D



At distance 11.98 km computed by 2-D

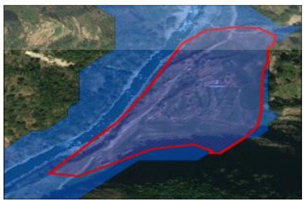


At distance 12.84 km computed by 1-D



At distance 12.84 km computed by 2-D

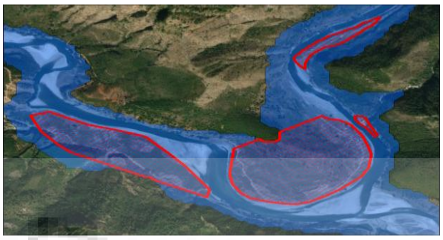
At distance 14.36 km computed by 1-D



At distance 14.36 km computed by 2-D

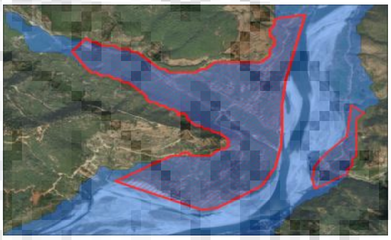
Fig. 5. 20 Flow affected settlement area computed by 1-D and 2-D

At distance 22.91 km computed by 1-D



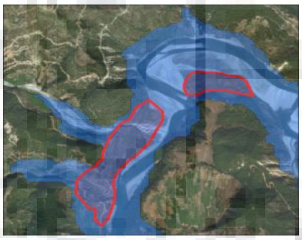
At distance 22.91 km computed by 1-D

At distance 31.63 km computed by 1-D

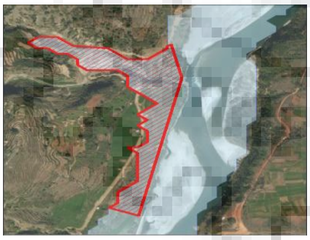


At distance 31.63 km computed by 2-D

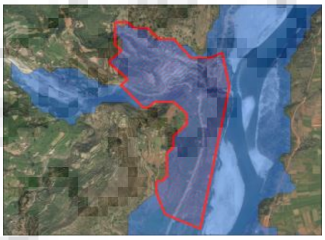
At distance 35.59 km computed by 1-D



At distance 35.59 km computed by 2-D



At distance 43.12 km computed by 1-D



At distance 43.12 km computed by 2-D

Fig. 5. 21 Flow affected settlement area computed by 1-D and 2-D

Table 5.11: Station-wise summary of flow affected Area

S.N.	Station no.	Flow Affected Area (Km^2)		Percentage (%) Higher		Rem.
		Computed by 1-D	Computed by 2-D	Computed by 1-D	Computed by 2-D	
1	48740.20	0.01	0.01	1.12	-	
2	47265	0.18	0.32	-	77.80	
3	37217	0.92	0.88	4.55	-	
4	35681.40	0.31	0.29	6.90	-	
5	34157.40	0.14	0.15	-	7.10	
6	24875.10	0.6	0.83	-	38.30	
7	16164.90	0.65	1.11	-	70.80	
8	12206	0.22	0.28	-	27.30	
9	4673	0.10	0.27	-	170	
10	130	0.03	0.13	-	333.30	
	Total	**3.16**	**4.3**			

From above table 5.11, it is seen that the flow affected settlement area computed by 1-D is more than computed by 2-D at station no.48740.20, 37217 and 35681.40 by 99.08 sqm, 0.04 sqkm and 0.02 sqkm respectively. Similarly, the flow affected settlement area comuped by 2-D is more than computed by 2-D at station no. 47265, 34157.4, 24875.10, 16164.90, 12206, 4673 and 130 by 0.14 sq. km, 0.23 sq. km, 0.46 sq. km, 0.06 sq. km, 0.17 sq. km and 0.1 sq. km respectively. The flow affected area computed by 1-D is more than computed by 2-D up to station no. 35681.40 where river is steep and flow does not spread significantly and flow pattern is unidirectional except at the station no. 47265 where local slope is flat at that particular station. Whenever the river slope becomes flat and flow is in both lateral and longitudinal direction after station no.35681.40, then, flood affected area computed by 2-D is more than computed by 1-D.

CHAPTER-6

SENSITYVITY ANALYSIS

6.1 General description

The purpose of carrying out sensitivity analysis is to identify how the independent variable value will influence a particular dependent variable under the given consideration so that analysis of how sensitive the output can be done. This process is done by changing one input value whereas keeping other input values same. The parameter considered for the sensitivity analysis in 1-D study is considers only the breach parameters. The independent variables are taken are breach formation time, breach Bottom width and breach Side slope and expected dependent variables are taken only water surface elevation and velocity and comparison of water surface elevation and velocity computed by without adjustment with Adjusted full formulation time, adjusted breach Bottom width and adjusted Side slope by increasing and decreasing the value by 20 %, 50 % and 75 % respectively. The input value of breach parameters described below in sense of sensitivity.

a. Breach formation time (t_f)

b. Breach Bottom width (B_w)

c. Breach Side slope.

a. Breach formation time (t_f)

Keeping the input value of breach Bottom width and breach Side slope constant, the breach formation time is changes by ± 20 %, ± 50 % and ± 75 % to determine the impact of changing formation time on water surface elevation and velocity of flow for dam break case. The details of impact is given on table 6.1, table 6.2 and table 6.3 below for ± 20 %, ± 50 % and ± 75 % changing on the breach formation time respectively.

b. Breach Bottom width (B_w)

Keeping the input value of breach formation time and breach Side slope constant, the breach Bottom width is changes by ± 20 %, ± 50 % and ± 75 % to determine the impact of changing breach Bottom width on water surface elevation and velocity of flow for dam break case. The details of impact is given on table 6.1, table 6.2 and table 6.3 below for ± 20 %, ± 50 % and ± 75 % changing on the breach Bottom width respectively.

c. Breach Side slope

Keeping the input value of breach formation time and breach Bottom width constant, the breach Side slope is changes by ± 20 %, ± 50 % and ± 75 % to determine the impact of changing breach Side slope on water surface elevation and velocity of flow for dam break case. The details of impact is given on table 6.1, table 6.2 and table 6.3 below for ± 20 %, ± 50 % and ± 75 % changing on the breach Side slope respectively.

Table 6.1: Dam break sensitivity analysis 1 (±20%).

Station no	D/S distance from dam axis(Km)	Break without adjustment		Adjusted full formulation time			
				Increase 20%		Decrease 20%	
		W.S. elev (m)	Vel chnl (m/s)	W.S. elev (m)	Vel chnl (m/s)	W.S. elev (m)	Vel chnl (m/s)
49246	**Dam Axis**						
48740.2	0.51	1429.43	16.29	1425.94	16.12	1433.68	16.74
47265	1.98	1348.51	11.62	1345.43	11.33	1351.84	11.92
37217	11.31	930.74	12.5	928.94	12.29	933.1	12.78
35681.4	12.84	890.91	9.77	887.19	9.51	895.46	9.98
34157.4	14.37	861.63	13.85	858.22	13.66	865.92	14.11
24875.1	22.91	776.04	16.31	773.46	16.11	779.05	16.65
16164.9	31.63	736.01	6.36	734.07	6.13	738.16	6.51
12206	35.59	725.73	4.73	723.68	4.61	727.89	4.85
4673	43.12	687.23	9.59	685.76	9.28	688.82	9.92
130	47.67	657.21	6.2	655.84	6.02	658.56	6.36

Table 6. 2: Dam break sensitivity analysis 1 (±20%).

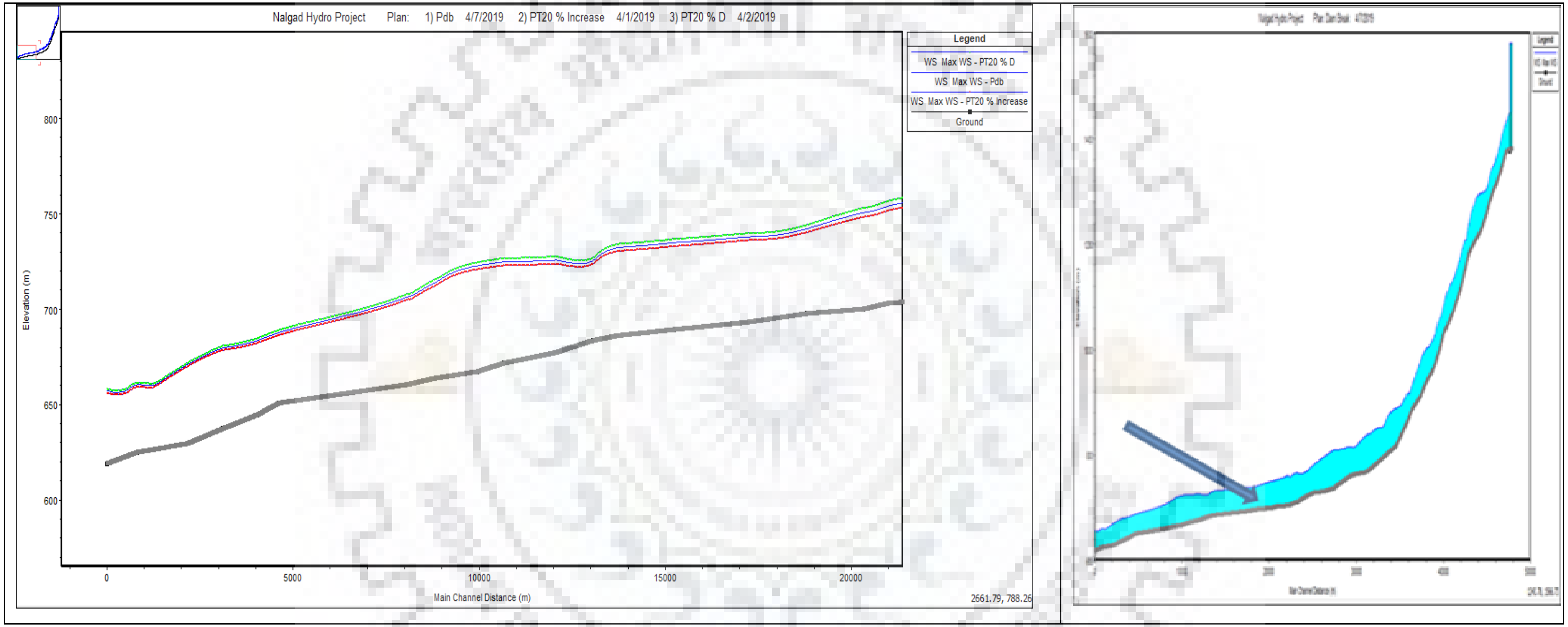
Statin no	D/S distance from dam (Km)	Adjusted breach width (m)				Adjustment Side slope			
		Increase 20%		Decrease 20%		Increase 20%		Decrease 20%	
		W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)
49246									
48740.2	0.51	1429.79	16.37	1429.06	16.22	1429.87	16.38	1428.88	16.18
47265	1.98	1348.81	11.7	1348.17	11.53	1348.87	11.71	1348.01	11.49
37217	11.31	930.95	12.56	930.54	12.44	930.97	12.57	930.46	12.41
35681.4	12.84	891.32	9.82	890.53	9.72	891.35	9.83	890.37	9.7
34157.4	14.37	862	13.91	861.27	13.79	862.03	13.91	861.13	13.76
24875.1	22.91	776.33	16.43	775.78	16.22	776.33	16.43	775.7	16.19
16164.9	31.63	736.23	6.48	735.78	6.4	736.19	6.36	735.77	6.4
12206	35.59	725.96	4.75	725.48	4.72	725.91	4.74	725.49	7.72
4673	43.12	687.4	9.63	687.06	9.55	687.36	9.62	687.07	9.55
130	47.67	657.36	6.23	657.05	6.17	657.32	6.22	657.05	6.17

The above table 6.1 provides the dam break sensitivity analysis-1 (± 20 %) and three scenarios compared at ten different specified locations. The changes in input value of breach Bottom width and Side slope have very minor i.e. less influence on the value of water surface elevation and velocity of flow. So, there is no significant difference among the break "without adjustment", "adjusted breach Bottom width" and "adjusted Side slope" but the changes in input value of breach formation time have significant influence on the value of water surface elevation and velocity of flow. When breach formation time is increases by 20% keeping the other input value constant, the water surface elevation decreases on the value of Break without adjustments by 3.49 m, 3.08 m, 1.8 m, 3.72 m, 3.41 m, 2.58 m, 1.94 m, 2.05 m, 1.47 m and 1.37 at the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. Similarly, velocity of flow decreases by, 0.17 m/s, 0.29 m/s, 0.21 m/s, 0.26 m/s, 0.19 m/s, 0.20 m/s, 0.23 m/s, 0.12 m/s, 0.31 m/s , 0.18 m/s on same station as mentioned above changes in water surface elevation.

Similarly, when breach formation time decreases by 20 % keeping the other input value constant, the water surface elevation and velocity flow increases on the value of Break without adjustment by 4.25m, 3.33 m, 2.36 m, 4.55 m, 4.29 m, 3.01 m, 2.15 m, 2.16 m , 1.59 m, 1.35 m and 0.45 m/s, 0.3 m/s, 0.28 m/s, 0.21 m/s, 0.21 m/s, 0.26 m/s, 0.34 m/s, 0.15 m/s, 0.12 m/s, 0.33 m/s and 0.16 m/s respectively on the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. It is seen from these data that more the change in breach formation time, more changes in value of water surface elevation and velocity of flow. So, changes in breach formation time is significantly affecting to the value of Break without adjustment.

The graphical representation of changing in water surface elevation for the condition of ± 20 % changes in input values of breach formation time only, keeping breach width and Side slope constant are shown in fig.6.1 below.

Fig. 6. 1 Maximum water surface elevation for without adjustment and ± 20 % adjustment on the breach formation time only



WS max. WS PT20%D = Maximum water surface profile at breach formation time decreases by 20 %.
WS max. WS Pdb = Maximum water surface profile without adjustment.
WS max. WS PT20% I = Maximum water surface profile at breach formation time increases by 20 %.

Table 6. 3: Dam break sensitivity analysis 2 (±50%).

Statin no	**D/S distance from dam (Km)**	**Break without adjustment**		**Adjusted full formulation time**			
				Increase 50%		**Decrease 50%**	
		W.S. elev (m)	**Vel chnl (m/s.)**	**W.S. elev (m)**	**Vel chnl (m/s.)**	**W.S. elev (m)**	**Vel chnl (m/s.)**
4924		Dam Axis					
48740.2	0.51	1429.43	16.29	1421.97	15.72	1443.12	16.98
47265	1.98	1348.51	11.62	1342.14	11.12	1359.42	12.15
37217	11.31	930.74	12.5	926.95	11.76	937.92	13.31
35681.4	12.84	890.91	9.77	882.82	9.31	904.92	10.25
34157.4	14.37	861.63	13.85	853.98	13.14	874.44	14.62
24875.1	22.91	776.04	16.31	770.2	15.97	784.54	17.12
16164.9	31.63	736.01	6.36	731.5	6.12	741.21	6.65
12206	35.59	725.73	4.73	720.87	4.51	730.82	4.97
4673	43.12	687.23	9.59	683.64	9.44	690.83	9.91
130	47.67	657.21	6.2	653.98	5.89	660.12	6.54

Table 6.4: Dam break sensitivity analysis 2 (±50%).

Statin no.	**D/S distance from dam (Km)**	**Adjusted breach width (m)**				**Adjustment Side slope**			
		Increase 50%		**Decrease 50%**		**Increase 50%**		**Decrease 50%**	
		W.S. elev (m)	**Vel chnl (m/s.)**	**W.S. elev (m)**	**Vel chnl (m/s.)**	**W.S. elev (m)**	**Vel chnl (m/s.)**	**W.S. elev (m)**	**Vel chnl (m/s.)**
49246		Dam Axis							
48740.2	0.51	1430.45	16.5	1428.46	16.09	1430.42	15.72	1427.86	16.98
47265	1.98	1349.29	11.86	1347.65	11.4	1349.26	11.12	1347.13	12.15
37217	11.31	931.3	12.67	930.23	12.33	931.26	11.76	929.92	13.31
35681.4	12.84	892.02	9.91	889.91	9.65	891.92	9.31	889.27	10.25
34157.4	14.37	862..65	14.01	860.71	13.69	862.54	13.14	860.16	14.62
24875.1	22.91	776.77	16.56	775.33	16.03	776.65	15.97	774.95	17.12
16164.9	31.63	736.59	6.51	735.39	6.36	736.42	6.12	735.28	6.65
12206	35.59	726.3	4.78	725.06	4.66	726.13	4.51	724.99	4.97
4673	43.12	687.63	9.68	686.76	9.48	687.51	9.44	686.72	9.91
130	47.67	657.57	6.26	656.75	6.12	657.45	5.89	656.71	6.54

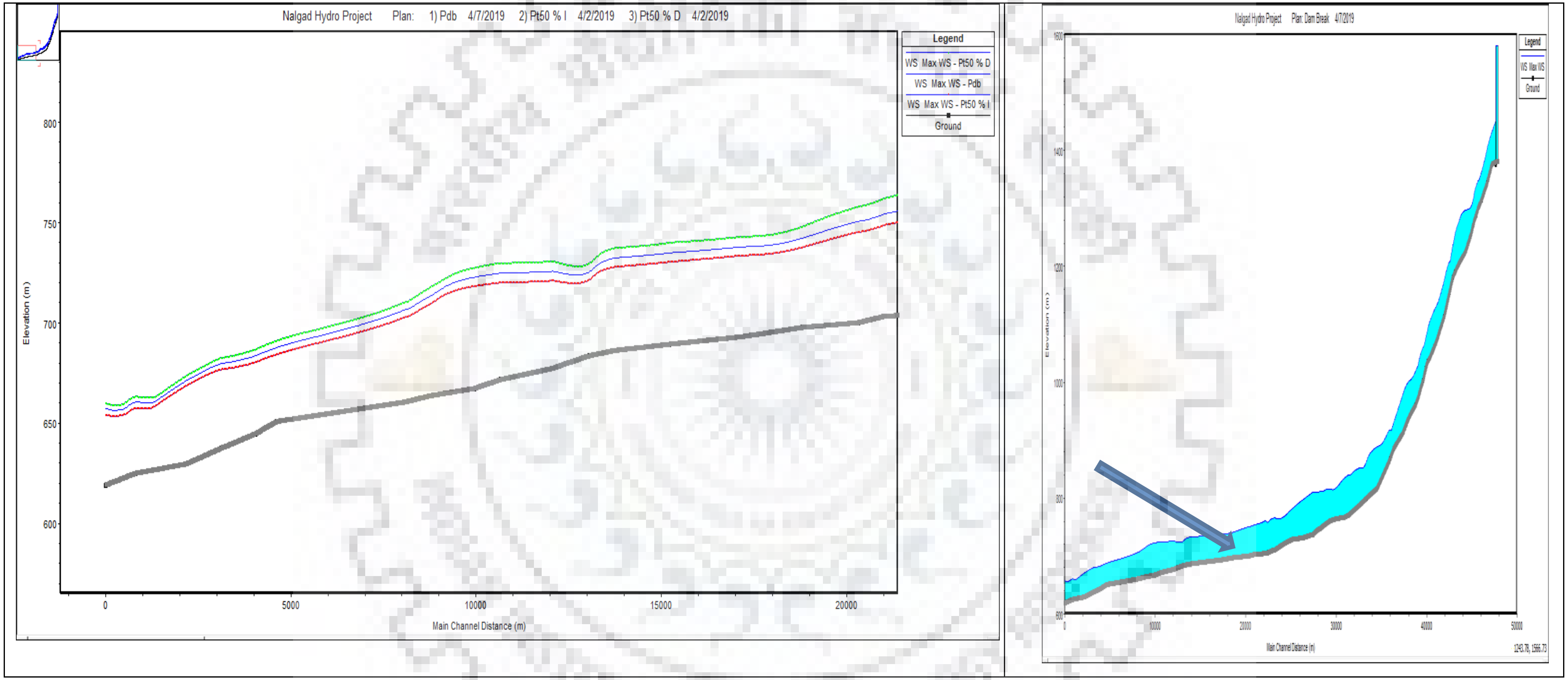
The above table 6.2 provides the dam break sensitivity analysis-2 (± 50 %) and three scenarios compared at ten different specified locations. The changes in input value of breach Bottom width and Side slope have very minor i.e. less than influence on the value of water surface elevation and velocity of flow except station 48740.2, 35681.4 &

34157.4 where the water surface elevation is about 1 m slightly higher or lower than the value obtained in the Break without adjustment and So, there is no significant difference among the break “without adjustment”, “adjusted breach Bottom width” and “adjusted Side slope” but the changes in input value of breach formation time have significant influence on the value of water surface elevation and velocity of flow. When breach formation time is increases by 50% keeping the other input value constant, the water surface elevation decreases on the value of Break without adjustments by 7.46 m, 6.37 m, 3.79 m, 8.09 m, 7.65 m, 5.84 m, 4.51 m, 4.86 m, 3.59 m and 3.23 at the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. Similarly, velocity of flow decreases by 0.57 m/s, 0.5 m/s, 0.74 m/s, 0.46 m/s, 0.71 m/s, 0.34 m/s, 0.24 m/s, 0.22 m/s, 0.15 m/s, and 0.31m/s on same station as mentioned above changes in water surface elevation.

Similarly, when breach formation time decreases by 50 % keeping the other input value constant, the water surface elevation and velocity flow increases on the value of Break without adjustment by 13.69 m, 10.91 m, 7.18 m, 14.01 m, 12.81 m, 8.5 m, 5.2 m, 5.09 m , 3.6 m, 2.91 m and 0.69 m/s, 0.53 m/s, 0.81 m/s, 0.48 m/s, 0.77 m/s, 0.81 m/s, 0.29 m/s, 0.24 m/s, 0.32 m/s, 0.34 m/s respectively on the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. It is seen from these data that more the change in breach formation time, more changes in value of water surface elevation and velocity of flow. So, changes in breach formation time is significantly affecting to the value of Break without adjustment.

The graphical representation of changing in water surface elevation for the condition of ± 50 % changes in input values of breach formation time only, keeping breach width and Side slope constant are shown in fig. 6.2.

Fig. 6. 2 Maximum Water surface elevation for without adjustment and ± 50 % adjustment on the breach formation time only



WS max. WS PT50%D = Maximum water surface profile at breach formation time decreases by 50 %.
WS max. WS Pdb = Maximum water surface profile without adjustment.
WS max.WSPT50%I = Maximum water surface profile at breach format ion time Increases by 50 %

Table 6.5: Dam break sensitivity analysis 3 (±75%).

Statin no.	D/S distance from dam (Km)	Break without adjustment		Adjusted full formulation time			
				Increase 75%		Decrease 75%	
		W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)
49246		Dam Axis					
48740.2	0.51	1429.43	16.29	1419.3	15.23	1456.08	17.44
47265	1.98	1348.51	11.62	1339.79	10.89	1369.57	12.41
37217	11.31	930.74	12.5	925.3	11.74	944.5	13.37
35681.4	12.84	890.91	9.77	879.76	9.12	915.67	10.42
34157.4	14.37	861.63	13.85	851.2	13.02	883.86	14.81
24875.1	22.91	776.04	16.31	768.01	15.57	788.91	17.34
16164.9	31.63	736.01	6.36	729.65	6.09	742.93	6.69
12206	35.59	725.73	4.73	718.79	4.47	732.13	4.97
4673	43.12	687.23	9.59	682.16	9.13	691.72	10.11
130	47.67	657.21	6.2	652.65	5.94	660.8	6.62

Table 6. 6: Dam break sensitivity analysis 3 (±75%).

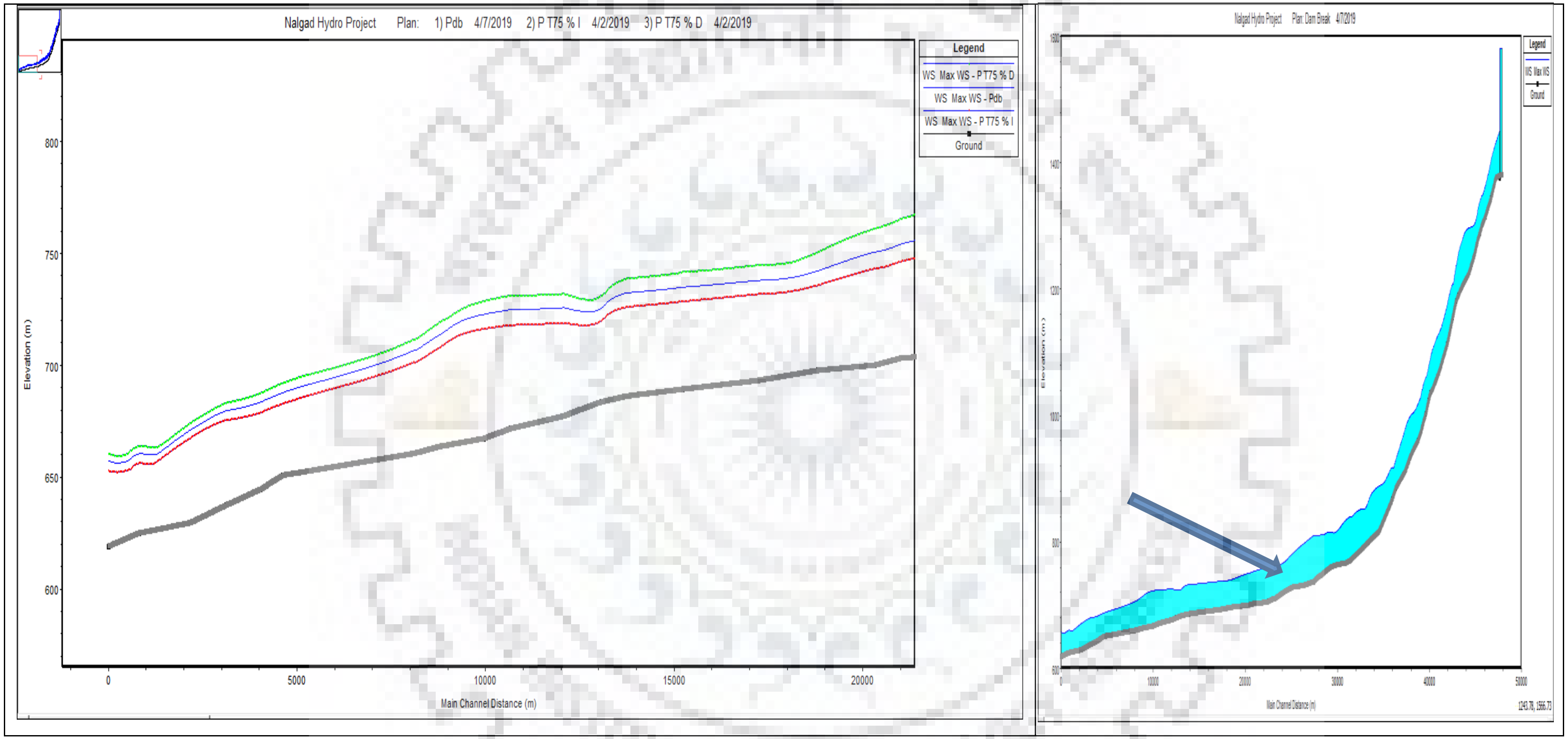
Statin no.	D/S distance from dam (Km)	Adjusted breach width (m)				Adjustment Side slope			
		Increase 75%		Decrease 75%		Increase 75%		Decrease 75%	
		W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)	W.S. elev (m)	Vel chnl (m/s.)
49246		Dam Axis							
48740.2	0.51	1431.13	16.65	1428.02	16	1430.8	16.58	1426.61	15.69
47265	1.98	1349.74	12.05	1347.24	11.31	1349.52	11.96	1346.01	11
37217	11.31	931.64	12.77	929.96	12.24	931.45	12.71	929.29	11.98
35681.4	12.84	892.75	9.99	889.37	9.59	892.3	9.94	887.94	9.43
34157.4	14.37	863.34	14.1	860.24	13.6	862.89	14.06	858.95	13.39
24875.1	22.91	777.23	16.71	774.92	15.99	776.87	16.61	774.05	15.69
16164.9	31.63	736.87	6.47	735.02	6.3	736.57	6.51	734.67	6.22
12206	35.59	726.6	4.81	724.66	4.65	726.26	4.77	724.35	4.63
4673	43.12	687.84	9.73	686.46	9.42	687.6	9.67	686.26	9.38
130	47.67	657.74	6.27	656.46	6.09	657.53	6.25	656.28	6.07

The above table 6.3 provides the dam break sensitivity analysis-3 (± 75 %) and three scenarios compared at ten different specified locations. The changes in input value of breach Bottom width and Side slope have very minor i.e. less than influence on the value of water surface elevation and velocity of flow, only change in the water surface elevation is about 1 m slightly higher or lower than the value obtained in the Break without adjustment and So, there is no significant difference among the break "without adjustment”, “adjusted breach Bottom width” and “adjusted Side slope” but the changes in input value of breach formation time have significant influence on the value of water surface elevation and velocity of flow. When breach formation time is increases by 75% keeping the other input value constant, the water surface elevation decreases on the value of Break without adjustments by 10.13 m, 8.72 m, 5.44 m, 11.15 m, 10.43 m, 8.03 m, 6.36 m, 6.94 m, 5.07 m and 4.56 m at the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. Similarly, velocity of flow decreases by 1.06 m/s, 0.73 m/s, 0.76 m/s, 0.65 m/s, 0.83 m/s, 0.74 m/s, 0.27 m/s, 0.26 m/s, 0.46 m/s, and 0.26 m/s on same station as mentioned above changes in water surface elevation.

Similarly, when breach formation time decreases by 75 % keeping the other input value constant, the water surface elevation and velocity flow increases on the value of Break without adjustment by 26.65 m, 21.06 m, 13.76 m, 24.76 m, 22.23 m, 12.87 m, 6.92 m, 6.4 m , 4.49 m, 3.59 m and 1.15 m/s, 0.79 m/s, 0.87 m/s, 0.65 m/s, 0.96 m/s, 1.03 m/s, 0.33 m/s, 0.24 m/s, 0.52 m/s, 0.42 m/s respectively on the stations 48740.2, 47265, 37217, 35681.4, 34751.4, 24875.1, 16164.90, 4673 & 130 respectively. It is seen from these data that more the change in breach formation time, more changes in value of water surface elevation and velocity of flow. So, changes in breach formation time is significantly affecting to the value of Break without adjustment.

The graphical representation of changing in water surface elevation for the condition of ±75 % changes in input values of breach formation time only, keeping breach width and Side slope constant are shown in fig.6.3 below.

Fig. 6. 3 Maximum water surface elevation for without adjustment and ± 75 % adjustment on the breach formation time only.



WS max. WS PT75%D = Maximum water surface profile at breach formation time decreases by 75 %.
WS max. WS Pdb = Maximum water surface profile without adjustment.
WS max.WSPT75% I = Maximum water surface profile at breach formation time increases by 75 %.

6.2 Sensitiveness of input parameters

The adjustment of input value of breach parameter breach bottom Time of failure, breach Bottom width, and breach Side slope are done one by one and using these adjusted value in HEC-RAS model, water surface elevation and velocity are computed and these results are compared with Break without adjustment. The sensitiveness of these values is mentioned in table 6.4 below for Time of failure and breach width and table 6.5 below shows the sensitiveness of Side slope for (±20 %) and for Time of failure and bottom breach width for (±50 %). Similarly, the table 6.6 shows sensitiveness of Side slope for (±50 %) and sensitiveness of Time of failure and breach Bottom width for (±75 %) and table 6.7shows the sensitiveness of Side slope.

Table 6.7: The sensitiveness of Side slope for (±20 %)

Station no.	Adjusted parameter and d/s distance	Water surface elevation (m)		Velocity (m/s)	
		Higher (%)	Lower (%)	Higher (%)	Lower (%)
	Time of failure				
48740.2	0.51 km d/s	0.30	0.24	2.76	1.04
47265	1.98 km d/s	0.25	0.23	2.58	2.50
37217	11.31 km d/s	0.25	0.19	2.24	1.68
35681.4	12.84 km d/s	0.51	0.42	2.15	2.66
34157.4	14.37 km d/s	0.50	0.40	1.88	1.37
24875.1	22.91 km d/s	0.39	0.33	2.08	1.23
16164.9	31.63 km d/s	0.29	0.26	2.36	3.62
12206	35.59 km d/s	0.30	0.28	2.54	2.54
4673	43.12 km d/s	0.23	0.21	3.44	3.23
130	47.67 km d/s	0.21	0.21	2.58	2.90
	Bottom width				
48740.2	0.51 km d/s	0.025	0.026	0.49	0.43
47265	1.98 km d/s	0.022	0.025	0.69	0.77
37217	11.31 km d/s	0.023	0.021	0.48	0.48
35681.4	12.84 km d/s	0.046	0.043	0.51	0.51
34157.4	14.37 km d/s	0.043	0.042	0.43	0.43
24875.1	22.91 km d/s	0.037	0.034	0.74	0.55
16164.9	31.63 km d/s	0.030	0.031	1.89	0.63
12206	35.59 km d/s	0.032	0.034	0.42	0.21
4673	43.12 km d/s	0.025	0.025	0.42	0.42
130	47.67 km d/s	0.023	0.024	0.48	0.48

Table 6.8: The sensitiveness of Side slope for (±20 %) and time of failure and bottom breach width for ((±50%)

Station no.	Adjusted parameter and d/s distance	Water surface elevation (m)		Velocity (m/s)	
		Higher (%)	Lower (%)	Higher (%)	Lower (%)
	Side slope				
48740.2	0.51 km d/s	0.03	0.04	0.55	0.68
47265	1.98 km d/s	0.03	0.04	0.77	1.12
37217	11.31 km d/s	0.02	0.03	0.56	0.72
35681.4	12.84 km d/s	0.05	0.06	0.61	0.72
34157.4	14.37 km d/s	0.05	0.06	0.43	0.65
24875.1	22.91 km d/s	0.04	0.04	0.74	0.74
16164.9	31.63 km d/s	0.02	0.03	0.01	0.63
12206	35.59 km d/s	0.02	0.03	0.21	0.63
4673	43.12 km d/s	0.02	0.02	0.31	0.42
130	47.67 km d/s	0.02	0.02	0.32	0.48
The sensitiveness of time of failure for (±50 %)					
48740.2	0.51 km d/s	0.96	0.52	4.24	3.50
47265	1.98 km d/s	0.81	0.47	4.56	4.30
37217	11.31 km d/s	0.77	0.41	6.48	5.92
35681.4	12.84 km d/s	1.57	0.91	4.91	4.71
34157.4	14.37 km d/s	1.49	0.89	5.56	5.13
24875.1	22.91 km d/s	1.10	0.75	4.97	2.08
16164.9	31.63 km d/s	0.71	0.61	4.56	3.77
12206	35.59 km d/s	0.70	0.67	5.07	4.65
4673	43.12 km d/s	0.52	0.52	3.34	1.56
130	47.67 km d/s	0.44	0.49	5.48	5.00
The sensitiveness of bottom breach width for (±50 %)					
48740.2	0.51 km d/s	0.071	0.068	1.29	1.23
47265	1.98 km d/s	0.058	0.064	2.07	1.89
37217	11.31 km d/s	0.060	0.055	1.36	1.36
35681.4	12.84 km d/s	0.125	0.112	1.43	1.23
34157.4	14.37 km d/s	0.118	0.107	1.16	1.16
24875.1	22.91 km d/s	0.094	0.091	1.53	1.72
16164.9	31.63 km d/s	0.079	0.084	2.36	0.00
12206	35.59 km d/s	0.079	0.092	1.06	1.48
4673	43.12 km d/s	0.058	0.068	0.94	1.15
130	47.67 km d/s	0.055	0.070	0.97	1.29

Table 6.9: The sensitiveness of Side slope for (±50 %) and time of failure and bottom breach width for ((±75%)

Station no.	Adjusted parameter	Water surface elevation (m)		Velocity (m/s)	
	Side slope	Higher (%)	Lower (%)	Higher (%)	Lower (%)
48740.2	0.51 km d/s	0.07	0.11	1.29	2.03
47265	1.98 km d/s	0.06	0.10	1.98	2.93
37217	11.31 km d/s	0.06	0.09	1.28	2.24
35681.4	12.84 km d/s	0.11	0.18	1.23	1.94
34157.4	14.37 km d/s	0.11	0.17	1.08	1.95
24875.1	22.91 km d/s	0.08	0.14	1.59	2.02
16164.9	31.63 km d/s	0.06	0.10	2.83	0.94
12206	35.59 km d/s	0.06	0.10	0.42	1.48
4673	43.12 km d/s	0.04	0.07	0.63	1.25
130	47.67 km d/s	0.04	0.08	0.81	1.45
	The sensitiveness of failure time for (±75 %)				
48740.2	0.51 km d/s	1.86	0.71	7.06	6.51
47265	1.98 km d/s	1.56	0.65	6.80	6.28
37217	11.31 km d/s	1.48	0.58	6.96	6.08
35681.4	12.84 km d/s	2.78	1.25	6.65	6.65
34157.4	14.37 km d/s	2.58	1.21	6.93	5.99
24875.1	22.91 km d/s	1.66	1.03	6.32	4.54
16164.9	31.63 km d/s	0.94	0.86	5.19	4.25
12206	35.59 km d/s	0.88	0.96	5.07	5.50
4673	43.12 km d/s	0.65	0.74	5.42	4.80
130	47.67 km d/s	0.55	0.69	6.77	4.19
	The sensitiveness of bottom breach width for (±75 %)				
48740.2	0.51 km d/s	0.119	0.099	2.21	1.78
47265	1.98 km d/s	0.091	0.094	3.70	2.67
37217	11.31 km d/s	0.097	0.084	2.16	2.08
35681.4	12.84 km d/s	0.207	0.173	2.25	1.84
34157.4	14.37 km d/s	0.198	0.161	1.81	1.81
24875.1	22.91 km d/s	0.153	0.144	2.45	1.96
16164.9	31.63 km d/s	0.117	0.135	1.73	0.94
12206	35.59 km d/s	0.120	0.147	1.69	1.69
4673	43.12 km d/s	0.089	0.112	1.46	1.77
130	47.67 km d/s	0.081	0.114	1.13	1.77

Table 6.10: The sensitiveness of Side slope for (±75 %)

Station no.	Adjusted parameter	Water surface elevation (m)		Velocity (m/s)	
	Side slope	Higher (%)	Lower (%)	Higher (%)	Lower (%)
48740.2	0.51 km d/s	0.10	0.20	1.78	3.68
47265	1.98 km d/s	0.07	0.19	2.93	5.34
37217	11.31 km d/s	0.08	0.16	1.68	4.16
35681.4	12.84 km d/s	0.16	0.33	1.74	3.48
34157.4	14.37 km d/s	0.15	0.31	1.52	3.32
24875.1	22.91 km d/s	0.11	0.26	1.84	3.80
16164.9	31.63 km d/s	0.08	0.18	2.36	2.20
12206	35.59 km d/s	0.07	0.19	0.85	2.11
4673	43.12 km d/s	0.05	0.14	0.83	2.19
130	47.67 km d/s	0.05	0.14	0.81	2.10

As per table 6.7 and table 6.8 when value of Time of failure, breach Bottom width and Side slope is increased by 20 %, the water surface elevation is 0.21 % to 0.42 % lower, 0.022 % to 0.042 % higher and 0.02% to 0.05% higher than the value of Break without adjustment and also velocity of flow is 1.04 % to 3.23 % lower, 0.42 % to 1.89 % higher and 0.01 % to 0.61 % higher than the value of Break without adjustment respectively. Similarly, when value of Time of failure, breach Bottom width and Side slope is decreased by 20 %, the water surface elevation is 0.21 % to 0.51 % higher, 0.021 % to 0.043 % lower and 0.02% to 0.06 % lower than the value of Break without adjustment respectively and also velocity of flow is 1.88% to 3.44% higher, 0.21 % to 0.63 % lower and 0.42 % to 1.12 % lower than the value of Break without adjustment respectively.

As per table 6.8 and table 6.9 when value of Time of failure, breach Bottom width and Side slope is increased by 50 %, the water surface elevation is 0.41 % to 0.91 % lower, 0.058 % to 0.125 % higher and 0.04% to 0.11% higher than the value of Break without adjustment and also velocity of flow is 1.56 % to 5.92 % lower, 0.94 % to 2.36 % higher and 0.42 % to 1.98 % higher than the value of Break without adjustment respectively. Similarly, when value of Time of failure, breach Bottom width and Side slope is decreased by 50 %, the water surface elevation is 0.44 % to 1.57 % higher, 0.055 % to 0.112 % lower and 0.07% to 0.18 % lower than the value of Break without adjustment respectively and also velocity of flow is 3.34% to 6.48 % higher, 0.11 % to 1.89 % lower and 0.94 % to 2.93 % lower than the value of Break without adjustment respectively

As per table 6.9 and table 6.10 when value of Time of failure, breach Bottom width and Side slope is increased by 75 %, the water surface elevation is 0.58 % to 1.25% lower, 0.081 % to 0.207% higher and 0.05% to 0.16% higher than the value of Break without adjustment and also velocity of flow is 4.19 % to 6.65 % lower, 1.13 % to 3.7% higher and 0.8 % to 2.93% higher than the value of Break without adjustment respectively. Similarly, when value of Time of failure, breach Bottom width and Side slope is decreased by 75 %, the water surface elevation is 0.55% to 2.78% higher, 0.084% to 0.147% lower and 0.20% to 0.33% lower than the value of Break without adjustment respectively and also velocity of flow is 5.07% to 7.06% higher, 0.94% to 2.67% lower and 2.10% to 5.34 % lower than the value of Break without adjustment respectively

It is seen from above results that sensitiveness of breach Bottom width and Side slope has very minor or say no influence on the water surface elevation and velocity but Time of failure has some minor influence on the output value of Break without adjustment.

CHAPTER-7

SUMMARY AND CONCLUSION

Dams play a vital role in water resources management in a river basin by storage of water during high flow seasons and meeting various demands throughout the year. Large dams are often constructed with multiple purposes viz. flood control, water supply, hydropower, navigation etc. Though, dams provide benefits to the society as well as the damages caused by dam failure ranges from loss of human life to loss of properties in tune of millions to billions. The loss due to such event can be minimized with dam break analysis studies and preparation of emergency action plans. The behavior of the high flood generated from the dam break can be analyzed in dam break simulation studies and the area under inundation can be mapped in advance.

Kulekhani is only one storage project constructed in the Nepal and Nalgad Hydro-Power Project is being planned as next storage project for hydropower generation. However, dam break studies have not been carried for any of the project in Nepal. The Nalgad Hydro-Power Project is proposed to construct in Nalgad River of Karnali river basin with a storage capacity of 480 MCM. This is a concrete faced rockfill dam with clay core of 200 m height. This study is carried out with major objective of simulating dam breach flood due to failure of Nalgad Hydro-Power dam. The HEC-RAS model is used for dam break analysis. The flood wave generated due to bam breach is routed using both 1-D and 2-D modules of HEC-RAS. The initial section of river up to about 14.36 km is having very high slope of 40 m/ km. Hence in this section cross-section spaced at distance of 10 m and in rest of the section cross-section are interpolated at distance of 25 m. The PMF inflow hydrograph, salient feature of the project, elevation area capacity curve, river cross-sections etc. are obtained from NHCPL and GON. SRTM digital elevation model is used for extracting additional cross-section and flood inundation modelling.

The breach geometry and breach formation time are estimated using five regression equations. Initially, breach parameters are selected by taking the average value. However, it is observed that the length of dam is smaller than estimated top width of breach. Hence, the final breach geometry is selected as trapezoidal shape having Bottom width 42.5 m, Side slope 0.6: 1 (H: V) such that top breach width does not exceeds the length of the dam as per site condition. Breach formation time is taken as average value of failure time i.e. 2.26 hour. Further, sensitivity analyses of breach parameters are also carried out.

Three case are analysed using HEC- RAS 1-D simulation viz. (a) No dam – natural flow in the river reach without presence of any inline structure in the river; (b) Dam break – Dam is represented by in line structure with is storage capacity and the dam fails during peak of inflow hydrograph; (c) Without dam break – It is similar to previous case expect that the dam doesn't fail and the inflow hydrograph is routed through spillway using Modified Puls method, the dam is allowed to overtop in water level rises beyond the top of dam. The reservoir in HEC RAS is defined as a storage area with volume of storage represented by elevation area capacity curve. The reservoir is connected with dam using lateral structure. Due to very high flow and high discharge, the unsteady flow simulation is carried out with computational time step of 2 second and results are stored at interval of 10 minute.

For the simulation of unsteady flow by 2-D model, the mesh is drawn with computation point spacing for longitudinal and lateral direction as 25 m. The computed maximum and minimum cell size is 1209.30 m^2 and 501.65 m^2 respectively whereas average cell size is 625.93 m^2. The hydrograph of first station from dam axis computed by 1-D is given as u/s boundary condition, friction slope of tail end reach is given as d/s boundary condition, and model is allowed to simulate unsteady flow taking computational time step of 5 second and hydrograph output interval 1 hour.

Based on the study following specific conclusions are drawn

- The peak discharge at selected section varies from 140032.50 m^3 /s to 102121.30 m^3 /s for dam break case from first to last section i.e. attenuation of 27.07 %, for the without dam break case, the peak discharge ranges from 2879.87m3/s to 2764.61 m^3 /s from first to last section i.e. attenuation of 4.02 % and similarly for the natural flood case the peak discharge varies from 3522.96 m^3 /s to 3370.53 m^3 /s from first to last section i.e. attenuation of 4.32 %.

- The depth of peak flood for dam break case varies from 75.13 m to 38.09 from head to tail, for the without dam break case, the depth of flow varies from 15.31 m to 11.63 m from head to tail and similarly for the natural flood case, the depth of flow varies from 17.44 m to 12.51 m from head to tail end.
- The time of arrival of peak flood for dam break case varies from 7.2 hour to 8.3 hour from first to lase section i. e reaching tail end after 1.1 hour, for the without dam break case., the time of arrival varies from 9 hour to 14 hour i. e reaching tail end after 5 hour end and similarly for the natural flood case, the time of arrival varies from 6 hour to 11 hour i.e. reaching tail end after 5 hour.
- The sensitivity analysis shows that the output results after making adjustment on the breach width and Side slope have no significant effect but changing of failure time influences the results minorly. So, the sensitivity analysis shows that the output of the model is reliable for the selected time step and distance.
- Two dimensional flow analyses is carried out and computed peak discharge varies from 140000.60 m^3/s to 102600.40 m^3/s from first to last section i.e. attenuation of 26.71 %, and velocity varies from 18.13 m/s to 6.92 m/s and arrival time varies from 7 hour to 9 hour from first to lase section i. e. reaching tail end after 2 hour.
- The total flow affected settlement area computed by 1-D is 3.16 Km^2 and total flow affected settlement area computed by 2-D is 4.16 Km^2. Hence, flow affected area computed by 2-d is 35.13 % higher than computed by 1-D.
- The peak discharge, velocity and time of arrival computed by 1-D and 2-D are almost matching except water surface elevation. The water surface elevation computed by 1-D and 2-D in steep portion of river up to 14.36 km are almost matching except at distance 1.98 Km where local slope is flat. When river meets flatter slope after 14.36 km, the water surface elevation computed by 2-D is relatively higher than computed by 1-D.
- The depth of water computed by 1-D is gradually varying from 56.34 m to 44.86 m up to distance 11.31 km and from 11.31 km to 12.5 km., the water depth become 63.89 m. Then, from 12.5 km to 47.67 km i.e. tail end, the depth of water gradually decreasing to the depth 38.09 m at tail end. Hence, 1-D modeling is producing reliable results

- In this case the 1-D modeling produced better results than 2-D modeling. It is also reported (UK Environment Agency, ((2009)), of 2D Hydraulic Modeling Packages) that when length to width ratio is more than 3:1 and steep stream, which are highly gravity driven having small overbanks are better simulated in 1-D model. For this study, results obtained and as per river flowing condition 1-D model is preferable than 2-D modeling.

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