DAM BREAK ANALYSIS USING MIKE11 FOR LOWER NAGAVALI DAM AND RUKURA DAM

THIS THESIS IS PRESENTED AS PART OF THE REQUIREMENTS FOR THE AWARD OF THE MASTER IN TECHNOLOGY DEGREE

In

CIVIL ENGINEERING

By

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CERTIFICATE

This is to certify that the thesis entitled, "**DAM BREAK ANALYSIS USING MIKE 11 FOR LOWER NAGAVALI DAM AND RUKURA DAM"** submitted by **Mr. SACHIN** a part of requirements for the award of **Master of Technology** Degree in Civil Engineering at the National Institute of Technology, Rourkela is an authentic work carried out by him under our supervision and guidance.

To the best of our knowledge, the matter embodied in the thesis has not been submitted to any other University/ Institute for the award of any Degree or Diploma.

Date: 30-05-2014

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LIST OF ABBREVIATIONS

Particular	Description
1-D	One Dimensional
2-D	Two Dimensional
BT	Breach Time
BW	Breach Width
Ch.	Chainage
DB	Dam Break
DHI	Danish Hydraulic Institute
d/s	Downstream
DEM	Digital Elevation Model
DSL	Dead Storage Level
FRL	Full Reservoir Level
HD	Hydro Dynamic
HEC	Hydrologic Engineering Center
Km	Killometer
LN	Lower Nagavali
m	Meter
Max.	Maximum
min.	Minutes
MWL	Maximum Water Level
NWS	National Weather Services
PMF	Probable Maximum Flood
Q	Discharge
Q-h	Discharge-Stage
s	Seconds
SCS	Soil Conservation Service
UK	United Kingdom

ABSTRACT

The society gets benefited in many ways from the dams but what if dam fails? The consequences are devastating to the society; causes extensive damage to properties and loss of human life due to short warning time available. So, the safety of downstream area is one of the most important aspects during the planning and designing of dam. It is always assumed that large magnitude of flood wave is generated due to failure of dam and inundates large area along the downstream portion of river.

This Thesis mainly provides an overview of the methods used to predict the breach outflow hydrographs with a detailed case study of hypothetical breach failure of two dams “Lower Nagavali Dam” and “Rukura Dam” using Mike 11 software. The two Dam breaks are analyzed for failure with comparison of the hydrographs at different downstream locations by changing its breach parameter using Mike 11. The parameters describing a breach are typically taken to be the breach depth, width, side slope and breach formation time. Wahl (1998) and Wahl (2004) and Froehlich (2008) have found them to be very significant, especially the time parameter.

The results are able to provide information for preparation of Emergency Response plan. It has been concluded that for Lower Nagavali Dam the downstream area from 12 Km to 17 km is more flooded. Rukura Dam break contribute 16018 m^3/s of flood into the Brahmini River. Beside the dam break analysis the sensitivity analysis for various parameters which will affect the maximum discharge and maximum water level has been analysed.

Chapter 1

INTRODUCTION

1.1 Background

There are thousands of dams have been constructed over many centuries around the world for different purposes: flood control (the most common purpose), irrigation, electricity generation, water supply, recreation, etc. But also, hundreds of dams have failed and every year many dikes breach due to high flows in the rivers, sea storm surges, etc. often leading to catastrophic consequences. In India the worst dam disaster occurred in Machhu II (Irrigation Scheme) Dam, Gujarat (1972 - 1979). This dam was constructed to serve an irrigation scheme. The dam failed on August 1, 1979, because of abnormal floods and inadequate spillway capacity and due to overtopping of water from the embankment caused a loss of 2000 lives. Kaddam Project Dam, Andhra Pradesh, failed in August 1958. The main cause of dam failure was overtopping of water above the crest by 46 cm and due to it 137.2 m of breach width has been developed on the left bank. Kaila Dam, Gujarat (1955-59) earth fill dam with a height of 23.08 m above the river bed and a crest length of 213.36 m. The embankment break due to the weak foundation bed made of shale in 1959. Kodaganar Dam, Tamil Nadu (1977) failed due to overtopping by flood waters which flowed over the downstream slopes caused a huge loss of property in downstream area. There is still large number of dam failures occurs in past few years in India. By far the world's worst dam failure "Banqiao Dam and the Shimantan Dam" occurred due to the overtopping caused by torrential rains in August 1975, in China. About 85,000 people died from flooding. In France Malpasset concrete dam failed in 1959 which takes life of 433 person and after that France introduce the dam safety legislation. In Italy October 1963, Vaiont reservoir fails when a landslide fell into it creating a flood wave some 100 m high that overtopped the dam and flooded into the downstream valley and about 2000 people died. More recently, in May 1999, a dam failed in Southern Germany causing 4 deaths and over 1 billion Euro of damage. In Spain 1997, failure of a dam on the Guadalquivir

River, caused immense ecological damage from the release of polluted sediments into the river valley. As we know climate is continuously changing and which has introduced uncertainty in flow within the life span of dams. Many dams previously considered safe are now exhibit uncertainty in maximum flows which cause overtopping during high flood events leading to safety concerns. If a dam fails, loss of life and economic damage are direct consequences of such an event, depending on the magnitude of water depth and velocity, warning time, and presence of population at the time of the event. Early warning is crucial for saving lives in flood prone areas. The construction of dams leads people to believe that the floods are fully controlled, and therefore an increased urban and industrial development in the floodplains usually takes place. Hence, if the structure fails, the damage caused by flooding might be much greater than it would have been without the presence of it. Having the historical failures of structures in mind as discussed above, one might pose the question what can be done in order to reduce the risk posed from a dam failure event.

1.2 General

Dams provide benefits to the society in terms of fulfilling their basic needs such as drinking water, irrigation water, electricity and flood protection etc. In advent of knowledge on engineering construction technology has helped the engineers to construct dams with more suitable design and factor of safety, but the nature is more powerful. USACE Hydrologic Engineering Center is (HEC) Research document 13 lists causes of failure as follows: 1.Earthquake, 2.Landslide, 3.Extreme storm, 4. Piping, 5.Equipment malfunction, 6.Structure damage, 7. Foundation failure, 8.Sabotage. But what if above mentioned cause of dam failure occurs, huge volume of water with high speed travel along a downstream valley. The high flood wave generated from dam break is sufficient to destroy the developed areas there infrastructure, roads, railways, bridges and more important if advance warning and evacuation were not done than with loss of life of people the disaster becomes more painful to the society. As no program for preventing failure can ever be certain so to mitigate the risk associated with dam break the pre analysis is carried out. Dam break analyses include three distinct analysis parts; Estimation of the dam-break outflow hydrograph, Routing of the dam-break hydrograph through the downstream valley, Estimation of inundation levels and damages to downstream structures. For the analysis of dam break lot of hydraulic software has been developed in the past few year such as DAMBRK, HEC-RAS and MIKE 11 etc.

1.3 About Mike 11 Software

Danish Hydraulic Institute (DHI) has introduce Mike 11 software for the simulation of flow which includes the following modules, Hydrodynamics, Rainfall-Runoff, Structure Operation, Dam Break, Advection Dispersion, and Water Quality. Hydrodynamic module (HD) is an implicit, finite difference model in Mike 11is the main functional unit which is capable of simulating unsteady flows in a network of open channels. The results obtained from the HD simulation consist of time series of water levels and discharges. For Open channel flow Mike 11 uses Saint Venant equations (1D) continuity equation and momentum equation.

Few assumptions in Mike 11 software are:

1. Water is incompressible and homogeneous,
2. Bottom slope is small,
3. Flow everywhere is parallel to the bottom (i.e. wave lengths are large compared with water depths).

Flow description:

The flow is described according to the number of terms used in momentum equations.

1. Dynamic wave (full Saint Venant equations)

2. Diffusive wave (backwater analysis)

3. Kinematic wave (relatively steep rivers without backwater effects)

Solution scheme:

Implicit finite difference scheme is used in which equations are transformed into the set of Implicit finite difference equations over a computational grid alternating Q and H points, where Q and H are computed at each time step. Numerical scheme used in the software is 6 point Abbott-Ionescu scheme

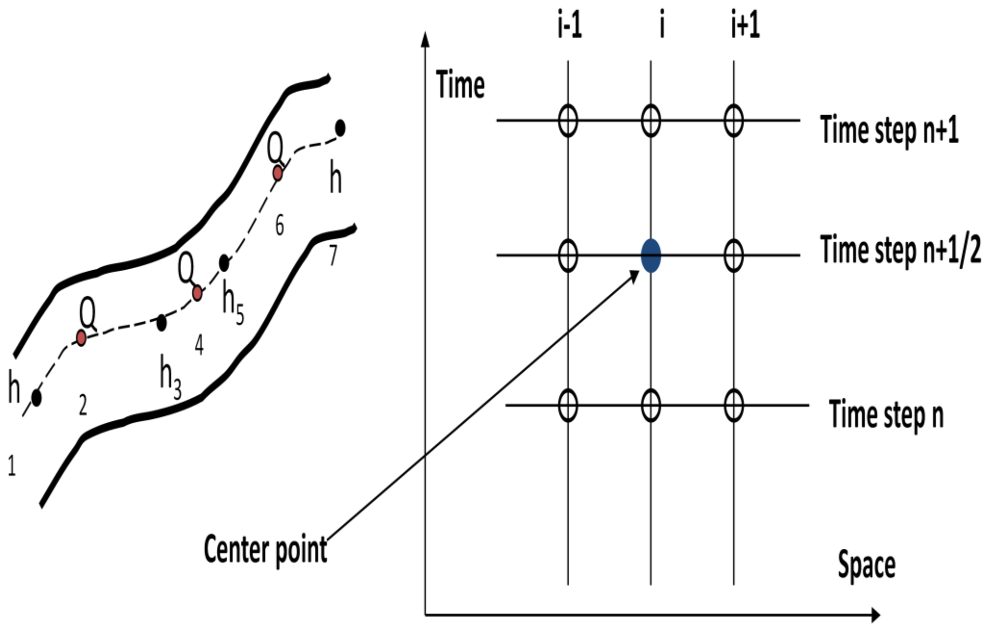

Fig. 1.1 Six (6) point Abbott-Ionescu scheme and implicit scheme used in Mike 11 hydrodynamic model

Boundary conditions

Mike 11 software includes two boundary conditions external boundary condition and internal boundary condition. External boundary conditions are for upstream and downstream of the river. Internal boundary conditions are for hydraulic structures (here Saint Venant equation are not applicable). Some typical upstream boundary conditions are also used which are useful for dam break analysis constant discharge from a reservoir, inflow hydrograph of a specific event (like PMF). Some typical downstream boundary conditions are also used which are constant water level, time series of water level, a reliable rating curve.

Initial condition

Time is assumed to be zero as initial condition

River Branches:

In Mike 11 hydrodynamic model the river branches are denoted and discretized as reach node. Fig 1.2 shows actual stream corridor and concept of representing the stream corridor in Mike 11.

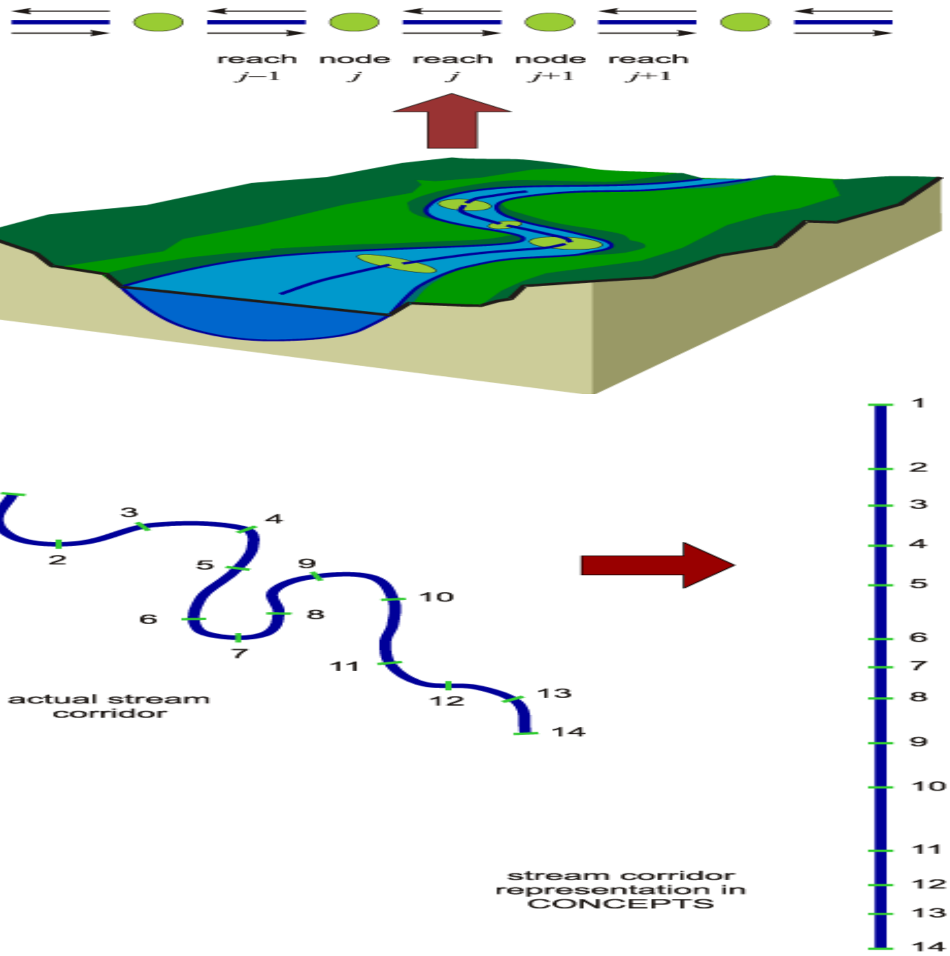

Fig.1.2 Discretization of river branch in the Mike 11 hydrodynamic model

Representation of cross sections

River cross sections are represented in the river network as X & Z coordinate system. X coordinate denotes the width of river and Z coordinate denotes the vertical distance of x coordinate. River cross sections are required to be represented accurately so that the flow changes, bed slope, shape, flow resistance characteristics etc are accurately define in Mike 11.

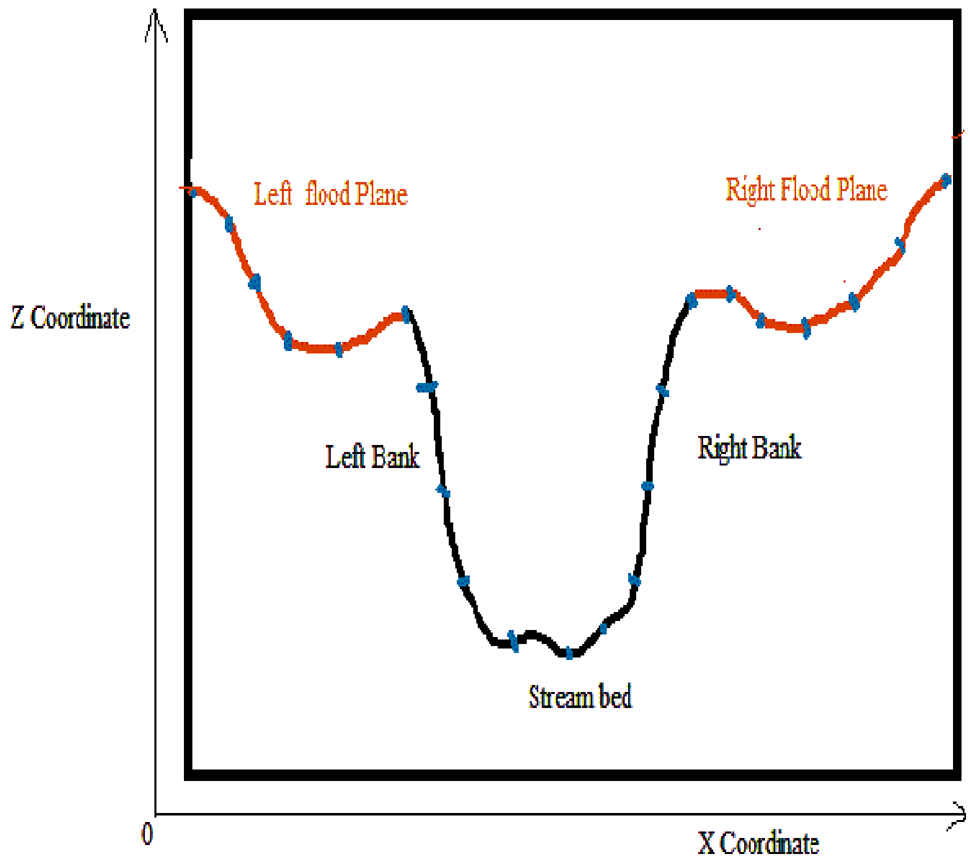

Fig.1.3 Discretization of cross section of river in the Mike 11

1.4 Dam Break Modeling

Generally, Dam Break (DB) Modeling can be carried out by either 1) scaled physical hydraulic models, or 2) mathematical simulation using computer. In mathematical modeling of dam break floods either 1-D analysis or 2-D analyses can be carried. In 1-D analysis, the time series of discharge and water level and velocity of flow through breach are obtained in the direction of flow. In case of 2-D analyses, with the results of 1-D extra and important information about the flood inundated map, variation of surface elevation and velocities in two directions can be analysed. Many investigators have proposed simplified methods for determining peak outflow from a breached dam. SCS (1981), MacDonald and Langridge-Monopolis (1984), Costa (1985), and Froehlich (1995) develop equations for predicting peak-flow from breached dam but none of these equations include material erodibility. Xu and Zhang (2009) include the erodbility effect. Walder and O'Connor (1997) uses analytical approach that predicts peak outflow by knowing the various dam and reservoir parameters, as he developed the relation from analysis of huge number of case study data from the available data of past dam failures. In past time many researchers developed regression model for prediction of breach parameters by utilizing the real case study data from dam failures. The breach parameters are breach depth, breach width, side slope and breach formation time. For dam break analysis the important aspect is to predict the accurate breach parameters. Breach width (BW) and breach time (BT) are the most important parameter for the study of dam break analysis and for predicting these two parameters many investigators have developed the regression models. NWS breach model (Fread 1988) is most widely used model around the world.

1.5 Scope of Thesis

Developing the dam break model and risk assessments due to flood produced from the dam break models for already constructed dams and dikes is becoming a necessity for a variety of reasons such as decreasing human casualties and economic damage. In this thesis, instead of focusing on already built hydraulic structures, we propose the analysis on two proposed medium dams by prediction of outflow hydrograph due to dam breach and it's routing through the downstream valley to get the maximum water level and discharge along with time of travel at different locations of the river. For carry out the analysis Mike 11 Dam Break Model is used for two different proposed dams namely Rukura Irrigation Dam and Lower Nagavali Dam. Model is used to Estimate the consequences of Dam Break for downstream areas in terms of water level, travel time of flood waves, flow velocity etc. that cope up with hazards caused by structural failure events by decreasing their consequences. We consider events, though not likely to happen in any given year, if occurring is extremely catastrophic and have enormous socio–economic impact.

CHAPTER 2

LITERATURE REVIEW

Johnson and Illes (1976) describe a failure shapes for earthen dams, gravity dams, and arch dams. For earthen dams, he describes that mostly developed trapezoidal breach shape with few of triangular breach shapes.

Singh and Snorrason (1982) conclude in his study of 20 dam failure that the variation of breach width was vary from 2 to 5 times the height of dam. The time of complete failure of dam, was generally 0.25 to 1 hour. There results also show that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.15 to 0.61 meters.

MacDonald and Langridge-Monopolis (1984) proposed a breach formation factor, defined as the product of the volume of breach outflow and the depth of water above the breach invert at the time of failure. They related the volume of embankment material removed to this factor for both earth fill and non-earth fill dams (e.g., rock fill, or earth fill with erosion-resistant core). Further, they concluded from analysis of the 42 case studies cited in their paper that the breach side slopes could be assumed to be 1h:2v in most cases; the breach shape was triangular or trapezoidal, depending on whether the breach reached the base of the dam. An envelope curve for the breach formation time as a function of the volume of eroded material was also presented for earthfill dams; for non-earthfill dams the time to failure was unpredictable, perhaps because, in some cases, failure may have been caused by structural instabilities rather than progressive erosion.

Singh and Snorrason (1984):

Singh and Snorrason compare the results of DAMBRK and HEC-1 for eight hypothetical breached dams. By varying the breach parameters he predicted the peak outflows using both the models. In his results he shows for large reservoirs the change in BW produces larger changes (35-87%) in peak outflow and for small reservoirs the change is smaller in peak outflow (6-50%).

Petra check and Sadler (1984):

Petra check and Sadler demonstrated the sensitivity of discharge, inundation levels, and flood arrival time with the change in breach width and breach formation time. For locations near the dam, both parameters can have a dramatic influence. For locations well downstream from the dam, the timing of the flood wave peak can be altered significantly by changes in breach formation time, but the peak discharge and inundation levels are insensitive to changes in breach parameters.

Froehlich (1987) developed non dimensional prediction equations for estimating average breach width, average side-slope factor, and breach formation time. The predictions were based on characteristics of the dam, including reservoir volume, height of water above the breach bottom, height of breach, width of the embankment at the dam crest and breach bottom, and coefficients that account for overtopping vs. non-overtopping failures and the presence or absence of a core wall. Froehlich also concluded that, all other factors being equal, breaches caused by overtopping are wider and erode laterally at a faster rate than
breaches caused by other means.

Wurbs (1987):

Wurbs concluded that breach simulation contains the greatest uncertainty of all aspects of dam-breach flood wave modeling. The importance of different parameters varies with reservoir size. In large reservoirs, the peak discharge occurs when the breach reaches its maximum depth and width. Changes in reservoir head are relatively slight during the breach formation period. In these cases, accurate prediction of breach geometry is most critical. For small reservoirs, there is significant change in reservoir level during the formation of the breach, and as a result, the peak outflow occurs before the breach has fully developed. For these cases, the breach formation rate is the crucial parameter.

Singh and Scarlatos (1988):

documented breach geometry characteristics and time of failure tendencies from a survey of 52 case studies. They found that the ratio of top and bottom breach widths, *Btop/Bbottom,* ranged from 1.06 to 1.74, with an average value of 1.29 and standard deviation of 0.180. The ratio of the top breach width to dam height was widelyscattered. The breach side slopes were inclined 10-50° from vertical in most cases. Also, most failure times were less than 3 hours, and 50 percent of the failure times were less than 1.5 hours.

Von Thun and Gillette (1990) and Dewey and Gillette (1993):

used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (h:v) may be more appropriate.

Tony L. Wahl (July 1998), "Prediction of Embankment Dam Breach Parameters" U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office, July 1998.

CHAPTER 3

METHODOLOGY

3.1 Dam Structure

The dam is represented as a structure in the river setup when the dam break structure is located the momentum equation is replaced by the broad crested weir flow equation which describe the flow through the structure. This flow may be either critical or subcritical.

3.2 Failure Moment

Four ways of failure are described in Mike 11.

- A given number of hours after start of the simulation.
- At a specified time(year, month, day, hour, minute)
- Overtopping Failure
- At a specified reservoir level

3.3 Failure Mode

The way the dam starts to breach can be specified as one of the following failure modes

- Instantaneous Failure
- Linear Failure i.e. the increase in breach dimension is assumed to occur linearly over a given time(the time of breach development)
- Erosion Based Failure i.e. the increase in the breach Depth is calculated from classical sediment transport formulation. The increase in width is calculated as the increase in breach depth multiplied by a side index.

3.4 Breach Formulation

Breach description for the study of dam break must be accurate because the development of breach will determine the reservoir outflow hydrograph. Earth fill dams never break instantaneously first breach is developed and then it increases gradually. The breach time may vary from few minutes up to few hours, depending upon the dam geometry and construction material. The breach may be rectangular, triangular or trapezoidal in shape. In case of an instantaneous or linear failure the breach formulation is straight forward i.e. only the start shape, end shape & development time has to be given

A dam break structure is a dam in which a breach can develop. The flow through a dam breach may be described in MIKE 11 through the use of the energy equation or alternatively a calculation method as implemented in National Weather Services (NWS) DAMBRK program.

3.4.1 Energy equation based dam breach modeling:

The flow at the dam break structure is quite similar to a broad crested weir, but there are two differences. First the shape of the dam changes with time, i.e. the breach increases and the dam crest is shortened. As a consequence the critical flow characteristics (Q-h) relationship of the crest and of the breach cannot be calculated beforehand. Second the Q-h relationship for the dam crest and the breach are different therefore the flow over the crest and the flow through the breach are calculated separately.

Initial breach development:

Using the standard dam breach methods the breach is initiated either as a trapezoidal breach or if the erosion based method is used as a circular piping failure.

1. Trapezoidal Breach Geometry:

During the development of the breach the trapezoid increases in size and changes shape. The initial breach shape is described by three parameters as shown in Figure.

1 level of the breach bottom (HB)

2 width of the breach bottom (WB)

3 side slope of the breach (SS) (horizontal: vertical). The left side slope and the right side slope are equal. The development of the breach can either be specified as a known function of time, or it can be simulated from the sediment transport capacity of the breach flow.

3.4.2 NWS DAMBRK dam-breach method:

The NWS DAMBRK method comes in two failures. Breach failure uses a weir type equation to determine the flow through the breach and Piping failure which is based on an orifice type equation

Breach failure

$$\boxed{Q = c_v k_s[c_{weir} b\sqrt{g(h-h_b)}(h-h_b) + c_{slope} S\sqrt{g(h-h_b)}(h-h_b)^2]}$$

Where,

b is the width of the breach bottom, g is acceleration due to gravity, h is upstream water level (reservoir water level), h_b is level of breach bottom, S denotes side slope of breach, c_{weir} denotes weir coefficient for horizontal part (=0.546430), c_{slope} is weir coefficient for slope part (=0.431856), c_v correction coefficient for approach sections (This coefficient compensates for the loss in energy due to the inflow contraction), and k_s correction coefficient due to submergence.

The weir coefficients have been made non-dimensional e.g.

$$c_{weir}\sqrt{g} = 0{,}546430\sqrt{9{,}81(\mathrm{m/s^2})} = 1{,}7115(\mathrm{m^{1/2}/s}) = 3{,}1(\mathrm{ft^{1/2}/s})$$

The correction coefficient for the approach section is determined through

$$c_v = 1 + \frac{c_B Q^2}{g W_R^2 (h - h_{b,\,term})^2 (h - h_b)}$$

Where,

C_B Non-dimensional coefficient (= 0.740256) termed the Brater coefficient

W_R Reservoir width given by the undestroyed crest length

$h_{b,term}$ The terminal level of the breach bottom. The minimum level in the time series file.

The submergence correction is determined through

$$k_s = \max\left(1 - 27{,}8\left(\frac{(h_{ds} - h_b)}{(h - h_b)} - 0{,}67\right)^3, 0\right)$$

Where,

h_{ds} is the downstream water level

Piping failure

The flow through a piping failure is given by

$$Q = C_{orifice} A \sqrt{2g(h - \max(hp, hds))}$$

Where, $C_{orifice}$ Orifice coefficient (= 0.599769), A is Flow area in pipe = b (h_{pt} – h_b) + S (h_{pt} - h_b)^2, h_{pt} is top of pipe, h_b is bottom of pipe and hp centerline of pipe = (h_{pt} +h_b)/2

The pipe may collapse either due to the top of the pipe reaching the crest level or if the water level upstream isn't high enough to maintain pipe flow. The criteria for the latter is given by

$$h < 3/2(hpt - hb) + hb$$

Once the pipe has collapsed the flow is calculated based on the breach flow equations.

3.4.3 Erosion Based Breach Development using the energy equation

If this mode is chosen the initial and the final breach shape must be specified. The England-Hansen sediment transport formula is used to calculate the sediment transport in the breach. The sediment transport rate, qt, calculated from the Engelund-Hansen formula is in terms of m^2/ s per meter-width of pure sediment only and this must then be related to a change in bed (i.e. breach) level. It is assumed that the breach remains horizontal. From the given upstream and downstream slopes, the length of the breach in the flow direction, L_b, may be calculated. By application of the sediment continuity equation in the breach, the change in breach level dH_b in a time interval dt is given as:

$$dH_b/dt = q_t / L_b(1 - \varepsilon)$$

Where,

H_b is the breach level

q_t is the sediment transport rate m^2/s

ε is the porosity of the sediment

L_b is the breach length in the direction of flow

t is time

CHAPTER 4

DAM BREAK MODEL SETUP IN MIKE 11

4.1 Introduction

There will be two types of arrangements of dam-structure. One is of dam and river network after the d/s of dam. Other is dam with spillway and river network after the d/s of the dam as shown in Fig. 4.1. For setting up and running MIKE11 dambreak model to we have to create MIKE11 simulation file. MIKE11 simulation file consists of network file, x-section file, bondary file and hydrodynamic file. So 1st step will be creating network file and then create branch for reservoir, river d/s of and spillway (if there is gated spillway) for digitizing we have to add point and define branch tools. After finishing network part create x-section. We need x-section for reservoir branch, spillway, and river d/s of dam. Reservoir is storage so area-elevation curve is required for defining the reservoir. The 1st Chainage X-section in the reservoir branch should be treated as storage for reservoir. After completing X-section create boundary file. In creating boundary file the inflow at the u/s end of reservoir and water level or Q-h at the d/s end is required. Now make Time series for discharge and water level. After that create HD parameters. After completing 4 editors run the model. For running model we have to create simulation editor.

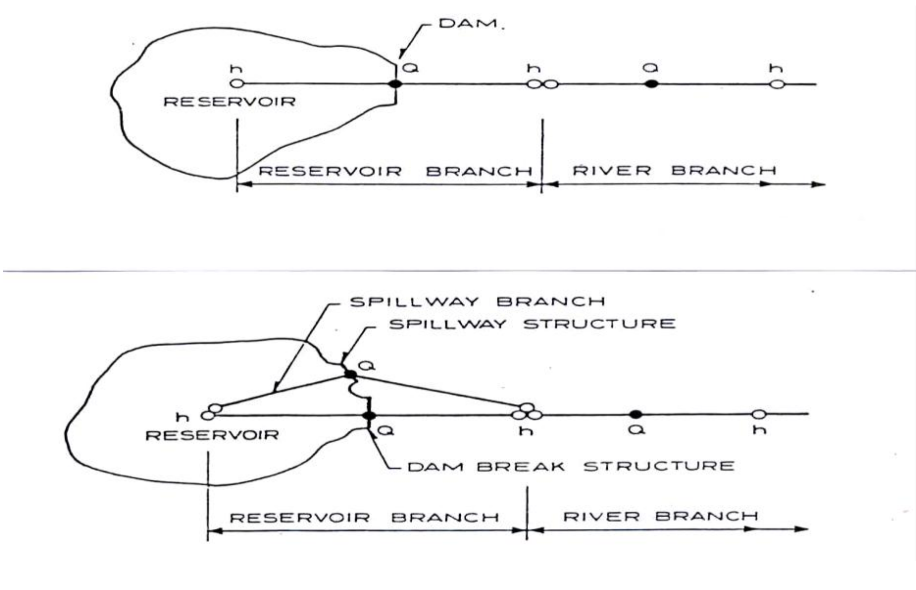

Fig. 4.1: Arrangement of Dam Strucure with Spillway in Mike-11

4.2 Model setup for Lower Nagavali Dam

For setting up hydrodynamic model for dam break analysis as per the requirement, different components of the project have been represented in the model as follow.

4.2.1 Nagavali River

In Hydrodynamic model setup the first step is creating the Nagavali River in network editor. Nagavali River is shown with 20 Km length in network editor with 38 cross sections. The dam break structure is defined at chainage point 2550 m from the starting Chainage point. Downstream of dam site the river is defined with 36 cross sections equally divided at every 500 m throughout the river network as shown in Fig. 4.2. As dam break flood is highly unstable and unsteady in nature so it is necessary that river geometry must be close to the real world condition. In the present study the river is traced with the help of Mike 11 GIS software using ASTER DEM of that location. The river cross sections are auto generated in the software and with the use of survey data of cross sections, the river network is modelled with more accuracy.

4.2.2 Reservoir

The Reservoir is normally modelled in Mike 11 as a Level-Area-Capacity curve at Chainage point “0” m of the modelled lower Nagavali River. Table 1 shows the Level-Area-Capacity data for reservoir.

4.2.3 Upstream Boundary Condition

Probable Maximum Flood (PMF) is considered as upstream boundary condition for the Mike 11 dam break simulation model and it has been considered as lateral inflow to the reservoir. Table 2 shows the value for PMF.

4.2.4 Downstream Boundary Condition

Chainage point “20000” m is the point where the downstream boundary conditions is defined as level(h)-discharge(Q) auto generated from the Manning’s formula employing the normal slope of the river at the downstream. Table 3 shows the Q-h data for Downstream

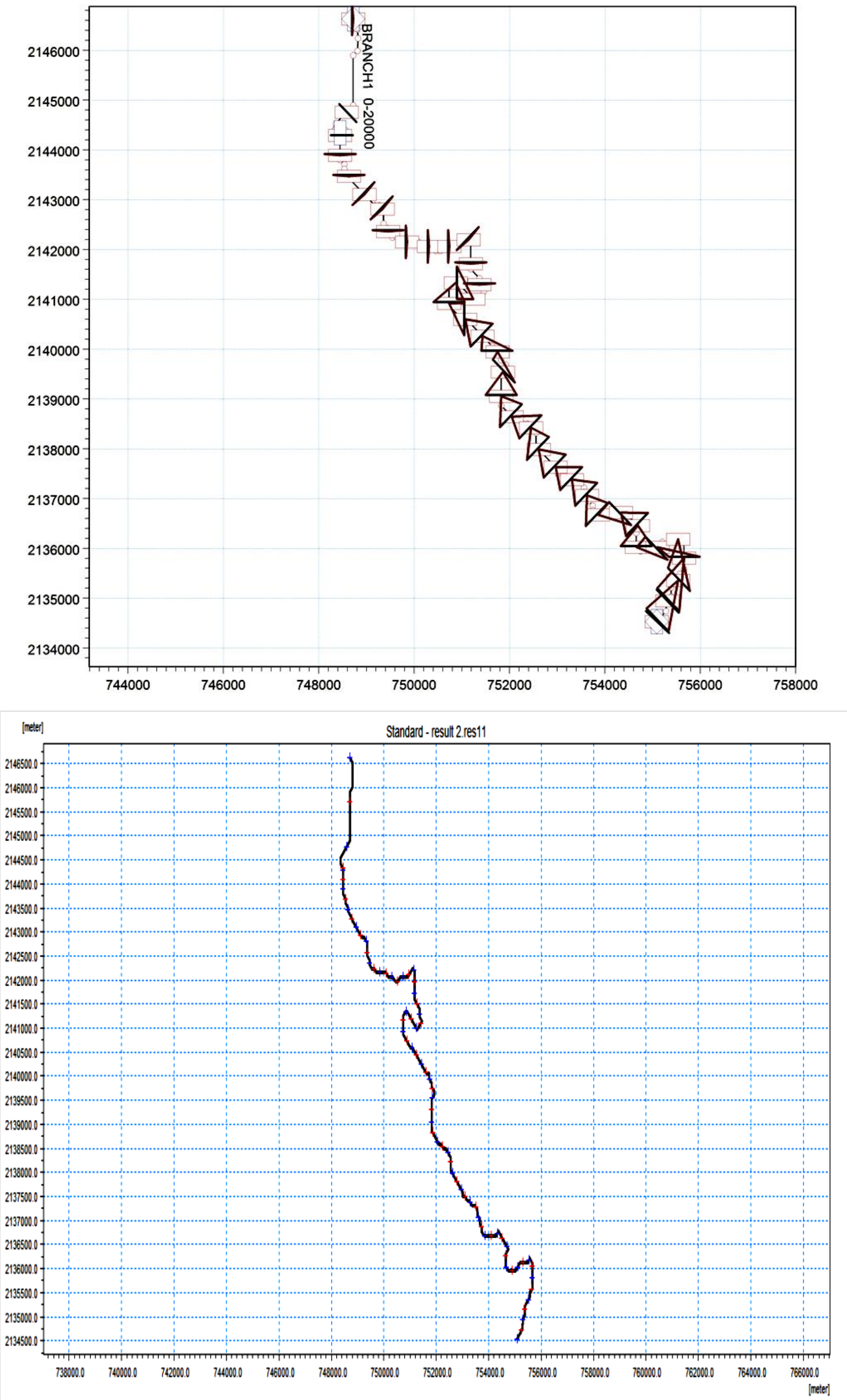

Fig. 4.2: River Network for lower Nagavali river in Mike-1

Table 1: Stage-Area –Capacity for lower Nagavali Reservoir

S.No.	Stage (m)	Area (m^2)	Capacity (m^3)
1	252	100	9000
2	260	479000	125000
3	262	775000	168000
4	264	1167000	211000
5	266	1615000	247000
6	268	2161000	295000
7	270	2814000	355000
8	272	3582000	412000
9	274	4456000	457000
10	276	5425000	511000
11	278	6511000	578000
12	280	7755000	646000
13	282	9294000	822000
14	284	11160000	1002000
15	286	13349000	1187000
16	288	16628000	1392000
17	290	18906000	1584000
18	292	22322000	1806000
19	294	26291000	2159000
20	296	31634000	2523000
21	298	37109000	2952000
22	300	43749000	3688000
23	302	48765000	4235000
24	304	55732000	4506000

Table 2: PMF for Lower Nagavali River

Time (hr)	Inflow m^3/S	Time (hr)	Inflow m^3/S
0	59	32	7846
1	62	33	7148
2	72	34	6492
3	94	35	5851
4	121	36	5120
5	157	37	4470
6	202	38	3894
7	267	39	3314
8	364	40	2834
9	510	41	2382
10	697	42	2014
11	920	43	1717
12	1177	44	1451
13	1454	45	1193
14	1748	46	965
15	2068	47	766
16	2432	48	600
17	2865	49	472
18	3331	50	369
19	3766	51	272
20	4229	52	196
21	4726	53	149
22	5314	54	117
23	6045	55	95
24	6936	56	81
25	7765	57	72
26	8446	58	66
27	8998	59	61
28	9196	60	59
29	9115	61	59
30	8819	62	59
31	8373	63	59

Table 3: Stage Discharge for Lower Nagavali river

Level (h) in m	Discharge (Q) m^3/s	Level (h) in m	Discharge (Q) m^3/s
227.85	0.00	236.60	1089.73
229.81	6.86	236.63	1102.00
229.82	7.03	236.87	1197.62
229.87	7.44	237.28	1375.69
230.05	9.63	237.28	1378.53
230.74	24.51	237.29	1382.60
230.77	25.65	239.10	2378.65
231.18	40.70	240.31	3214.48
231.50	57.13	241.23	3931.08
231.75	73.99	242.15	4727.16
231.76	74.45	243.98	6562.12
232.67	164.20	245.82	8704.96
233.58	294.59	247.65	11140.42
234.05	378.20	249.49	13855.05
234.13	392.97	251.32	16840.04
234.25	417.82	253.15	20090.73
235.12	620.18	254.99	23606.37
235.58	746.66	256.82	27386.27
235.91	846.15	258.66	31430.00
236.23	954.77	262.33	40309.76
236.28	970.42	265.99	50226.34
236.31	982.06	269.66	61141.66

4.3 Model setup for Rukura Dam

4.3.1 Rukura Nala

The model is prepared for a length of 7 km from the dam site has been represented in the model by 70 cross sections at about 50 m and 100 m intervals. The chainage point 2883 of the river has been connected to a storage area representing the reservoir. The Manning's roughness coefficient for the reach of Rukura river has been taken as 0.033 considering the rocky river beds. For this type of river Chow (1959) suggested its range between 0.03 and 0.05.

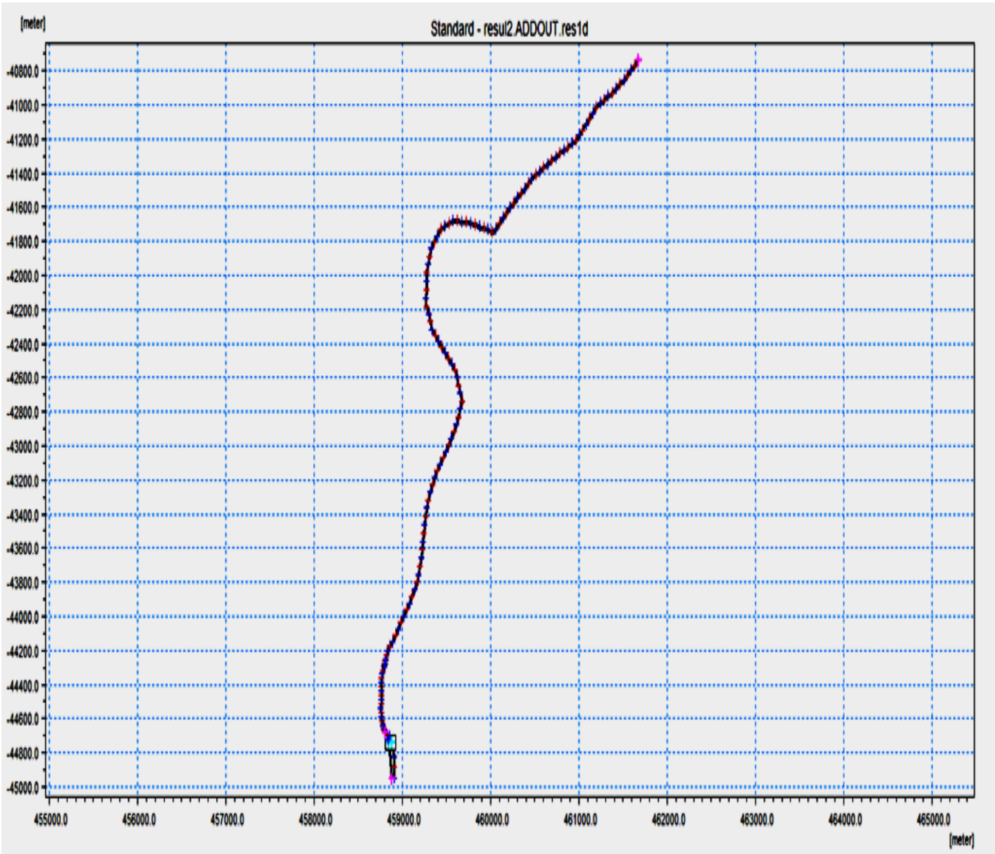

Fig 4.3: River Network for Rukura river in Mike-11

4.3.2 Reservoir

The reservoir has been represented in the model by reservoir stage- area-volume relationship.

Table 4: Stage-Area-Capacity Of Rukura Reservoir

Stage (m)	Area (ha)	Area (m^2)	Cumulative Capacity (ha.m)	Cumulative Capacity (m^3)
164	0	0	0	0
165	2.06	20600	0.6868	6868
166	4.43	44300	3.857	38570
167	8.06	80600	10.0121	100121
168	14.75	147500	43.7257	437257
169	27.71	277100	64.6179	646179
170	37.65	376500	47.1711	471711
171	53.64	536400	142.5808	1425808
172	73.4	734000	205.8431	2058431
173	123.36	1233600	303.1483	3031483
174	146.41	1464100	437.8683	4378683
175	166.6	1666000	594.2651	5942651
176	198.97	1989700	776.8107	7768107
177	220.63	2206300	986.5174	9865174
178	251.74	2517400	1222.5559	12225559
179	289.16	2891600	1431.8373	14318373
180	316.32	3163200	1793.5131	17935131
181	354.51	3545100	2128.7468	21287468
182	395.88	3958800	2503.7516	25037516
183	433.79	4337900	2918.4422	29184422
184	475.43	4754300	3372.8932	33728932
185	511.11	5111100	3866.0556	38660556
186	545.3	5453000	4394.3069	43943069
187	582.54	5825400	4958.266	49582660
188	614.77	6147700	5556.8479	55568479
189	649.11	6491100	6188.7109	61887109
190	690.94	6909400	6358.6217	63586217

4.3.3 Upstream Boundary

For the Rukura dam break model simulation, the Standard Probable Flood has been considered as a lateral inflow to the reservoir.

Table 5: Standard Probable Flood for Rukura Dam

Sl. No.	Time	Discharge (m^3/s)	Sl. No.	Time	Discharge (m^3/s)
1	0	8.19	29	28	812.63
2	1	10.01	30	29	731.64
3	2	11.83	31	30	654.29
4	3	18.2	32	31	581.49
5	4	27.3	33	32	509.6
6	5	42.7	34	33	439.53
7	6	65.52	35	34	374.01
8	7	99.19	36	35	314.86
9	8	148.33	37	36	259.35
10	9	218.4	38	37	211.12
11	10	318.5	39	38	168.35
12	11	440.44	40	39	133.77
13	12	581.49	41	40	105.56
14	13	741.65	42	41	83.72
15	14	920.92	43	42	66.43
16	15	1116.57	44	43	51.87
17	16	1305.85	45	44	40.95
18	17	1459.64	46	45	31.85
19	18	1543.36	47	46	24.57
20	19	1534.26	48	47	20.02
21	20	1470.56	49	48	15.47
22	21	1394.12	50	49	12.74
23	22	1314.95	51	50	10.92
24	23	1234.87	52	51	10.01
25	24	1154.79	53	52	9.1
26	25	1071.07	54	53	9.1
27	26	980.98	55	54	8.19
28	27	894.53			

4.3.4 Downstream Boundary

The study-state stage-discharge relationship described by the Manning's formula, which is auto generated in the mike 11 software.

Table 6: Stage- Discharge for Rukura River

Level (h) in m	Discharge (Q) m^3/s	Level (h) in m	Discharge (Q) m^3/s
147.1	6	152.78	14726
147.30	36	153.45	20982
147.66	179	154.12	28033
148.03	439	154.54	32226
148.57	1071	154.96	36838
149.1	1940	155.38	41843
149.63	3051	155.80	47204
150.17	4453	156.22	53013
150.70	5999	156.63	59309
151.22	7819	157.05	65975
151.45	8386	157.47	73122
151.58	8496	157.76	76302
151.70	8713	157.87	77410
151.84	9077	158.08	80469
151.97	9535	158.35	83954
152.01	9756	158.61	87958
152.20	10870	158.70	89647
152.39	12068	158.74	90542
152.58	13353	159.30	105752

4.4 Manning's Roughness

For the whole river course a constant Manning's Roughness Coefficient is assumed. As the dam breach flood levels far exceed the normal flood level marks and the flood spreads beyond the normal river course so the manning's roughness coefficient is assumed to be little more than usually used in other hydrodynamic model. For selecting the manning's roughness coefficient for Nagavali River and Rukura Nala course which has rocky river beds with grassy banks usually steep, trees and brush along banks submerged has been taken as 0.0333 (Chow(1959) suggested the range for this type of bed surface in between the range of 0.03 to 0.05).

4.5 Breach Parameter Selection

The breach parameter selection is more important for carry out the dam break study. As we have already discuss the breach formulation and about the breach selection procedures. In Chapter 6 first we have consider and analysed the Ideal Dam break scenario which has most probability of occurrence. As earthen dam are assumed to be taken more time for its complete failure compare to the concrete gravity dam. According to the NWS (Fread, 2006) guidelines, earthen dams take 0.1 to 1.0 hour failure time and concrete gravity dam takes 0.1 to 0.2 hours failure time. The UK Dam Break Guidelines and U.S. Federal Energy Regulatory Commission (FERC) Guidelines are shown in Table 7.

As NWS Guidelines are most accepted in the world so for the present study the NWS (Fread, 2006) Earth fill dam guidelines are used which are, breach width range is in between (2.0 to 5.0) x Height of Dam (HD), horizontal component of breach side slope(H) is 0 to 1.0 (slightly larger) and failure time in hours is in between 0.1 to 1.0 hours

Table 7: UK Dam Break Guidelines and U.S. Federal Energy Regulatory Commission (FERC) Guidelines

Dam Type	Average Breach width	Failure Time hrs	Breach Side Slope H:1V	Agency
Earthen/ Rockfill	(0.5 to5.0) x HD (1.0 to 5.0) x HD (2.0 to 5.0) x HD	0.5 to 4.0 0.1 to 1.0 0.1 to 1.0	0 to 1.0 0 to 1.0 0 to 1.0	USACE (2007) FERC (1988) NWS(Fread, 2006)
Concrete Gravity	Multiple Monoliths Usually ≤ 0.5 L Usually ≤ 0.5 L	0.1 to 0.5 0.1 to 0.3 0.1 to 0.2	Vertical Vertical Vertical	USACE (2007) FERC NWS (Fread, 2006

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CHAPTER 5

STUDY AREA

5.1 Lower Nagavali

Lower Nagavali Irrigation Project is a reservoir project proposed in Nagavali Basin on river Nagavali, at village Bheja in Kalyanasinghpur Block of Rayagada District of Odisha. The project envisages construction of a 508 m long earth dam having maximum height of 51.49 m besides a central spillway proposed at the centre of river gap.

Salient Features for Nagavali Dam

1. Location

a.	State	: Orissa
b.	District	: Rayagada
c.	River	: Nagavali
d.	Latitude & Longitude	: 19^0 – 23’ N & 830 – 21’ – 45” E

2. Hydrology

a.	Catchment area	: 1176 Sq. Km
b.	Max. Annual monsoon rainfall	: 2098.6 mm
c.	Min. Annual monsoon rainfall	: 772.8 mm
d.	Net 75% dependable yield	: 17677.46 HaM
e.	Design Flood Discharge	: 9196 Cumec
f.	Average Normal rainfall	: 1313.1 mm

3. Reservoir

a.	Gross Storage Capacity	: 4374.9 HaM
b.	Live Storage Capacity	: 3148.9 HaM
c.	Dead Storage Capacity	: 1226 HaM
d.	Full Reservoir Level	: 300.0 M
e.	Dead Storage Level	: 285.0 M
f.	Top Bank Level	: 303.0 M

4. **Dam**

a.	Type of Dam	: Homogeneous Earth Fill
b.	Total length	: 508 M
c.	Max. Height	: 51.49 M
d.	Top Width	: 6.00 M

5. **Spillway**

a.	Type Crested	: Centrally located Ogee
b.	Effective Length	: 120.0 m
c.	Crest Level	: 288.00 m
d.	Spillway Capacity	: 9196 Cumec
e.	No. of Bays	: 10
f.	Size of Radial Gates	: 14.0 m x 16.0 m

5.2 Rukura Dam

Rukura dam project is located in Sundargarh District, Odisha.is one of the medium irrigation project envisages construction of an Earth dam of 1185 m length including a central spillway of 52 m length & one head-regulator across Rukura River, a tributary of river Brahmani which shall create a reservoir of 3800.42ham. of live Storage capacity from the catchment area of 171.00sqkm.

Salient Features for Rukura Dam

1. **Location**

a.	State	Orissa
b.	District	Sundargarh
c.	Sub-Division	Bonai
d.	Village	Mushaposh
e.	River	Rukura Nallah
f.	Latitude	21^0 47’-50” N
g.	Longitude	84^0 50’-50“ E

2. Reservoir

a.	Gross storage at FRL	4394.307 Ham.
b.	Dead storage capacity	594.265 Ham.
c.	Live storage capacity	3800.042 Ham.
d.	Full reservoir level	186.00 M.
e.	Maximum water level	186.00 M.
f.	Top bank level	189.00 M.
g.	Submerged area at FRL/MWL	668.45 Ha.
h.	Dead storage level	175.00 M.
i.	Deepest bed level	163.00 M.
j.	Submergence at DSL	166.60 Ha.

3. Dam

a.	Type	Homogeneous earth fill dam
b.	Length (Earth Dam)	1185 M
c.	Maximum height	26.00 M
d.	Top width	6.00 M

4. Spillway

a.	Location & type	Centrally located ogee shaped & Gated
b.	Length	52.00 M
c.	Crest level of spillway	177.00 M
d.	Size of gate	10 M x 9 M
e.	Number of bays	4 Nos.

CHAPTER 6

RESULT AND ANALYSIS

This Chapter is divided into two sections, Section A and Section B. Section A discuss the Results for Lower Nagavali Dam as a Dam Break in detail and Section B discuss the results of Rukura Dam as a Dam Break.

SECTION A: Lower Nagavali Dam

The most critical situation for the dam break is the condition when the reservoir is at full reservoir level and then peak of the most severe flood (PMF) impinges over the reservoir. As the spillway capacity is 9196 cumec which is similar to the peak Value of PMF. So it is obvious that spillway will discharge the peak of PMF without overtopping the dam crest level. For this study it is assumed that due to improper timing of gate opening at the time of PMF, the dam is just slightly overtopped by PMF and than dam is failed due to breaching. Since the dam is of earthen type the time of breach is assumed to be 50 minutes. The breach width of 3*HD (154.47 m) is assumed. The Water Level of reservoir at the time when breach started is 303.05 m and breach will continue up to 252 m water level.

6. A.1 Dam Breach Statistics

Dam breach is started at 19.267 hour from the start of PMF as at that time PMF is just overtopped and attain the water level of 303.05 m. The maximum discharge flows out from the breached dam is 53334.90 m^3/ s which is 5.8 times greater than the PMF. The max discharge is attained at 45.78 min from the start of dam break and the water is coming out with the velocity of 9.38 m/s. The breach parameters at the time of max. discharge are breach bottom width is 142.12 m, breach width at crest is 235.95 m, breach depth is 32.56 m and breach level is 256.08 m. The Maximum velocity is 9.47 m/s at the time of 42.18 min. from the starting time of dam break. The dam breach statistics are shown in table 9.

Table 8: Dam Breach Statistics for Lower Nagavali Dam

Time (h)	Q in Breach (m^3/s)	V in Breach (m/s)	Reservoir Water Level (m)	Level of Breach (m)	Depth in breach (m)	Breach Bottom Width (m)	Breach width at crest (m)
19.28	7.1	1.67	303.06	302.03	1.02	3.19	5.13
19.37	646.3	4.09	303.27	296.92	6.33	18.62	30.78
19.4	1323.6	4.72	303.33	294.88	8.44	24.8	41.04
19.43	2301.6	5.27	303.38	292.84	10.54	30.97	51.29
19.57	9622.3	6.98	303.22	284.67	18.57	55.67	92.33
19.6	12333.9	7.33	303.06	282.63	20.45	61.85	102.59
19.63	15380.5	7.65	302.83	280.59	22.27	68.02	112.85
19.67	18737.3	7.95	302.52	278.55	24.01	74.2	123.11
19.8	34425.2	8.91	300.27	270.38	29.99	98.9	164.14
19.83	38563.2	9.09	299.37	268.34	31.15	105.07	174.4
19.87	42528.8	9.24	298.26	266.29	32.11	111.25	184.66
20	53087.8	9.46	291	258.13	33.19	135.95	225.69
20.03	53334.9	9.38	288.29	256.08	32.56	142.12	235.95
20.13	33757	7.38	278.49	252	26.92	154.47	256.47
20.17	25696.8	6.52	275.41	252	23.78	154.47	256.47

6. A.2 Routing of Flood Hydrograph

Routing of flood hydrograph is analysed at the four Chainage points 2.45 Km, 7.45 Km, 12.45 Km, and 16.95 Km downstream of the dam. Fig 6 shows the flood hydrographs for different Chainage points. At the dam site the peak discharge of 53370 m^3/ s is flows out in 47 min from the starting time of dam break. At 2.45 Km d/s location, the peak flood discharge is about 52367 m^3/ s which is 1.8 % less than the peak discharge coming out from the breached dam. The arrival time of flood is just 9 minute from the start of flood from the breached dam and in about 47 min. the peak flood is arrived in this region. It means in 38 min. the peak flood is arrived from the start of flood in this location. This flood reaches 7.45 Km in 28 minutes and the peak discharge of about 49055 m^3/s takes 27 min from the arrival time of the flood. It means the total time of 55 minutes is taken by flood to flow with its full capacity. So, we conclude that about 28 minutes is the time to deal with the flood at 7.45 Km d/s of the dam. After the arrival of flood still authority will get about 27 minutes to minimize the disaster from peak flood. Now, if we further goes downstream of the dam then we see the arrival time of dam break flood in 12.45 Km d/s is 43 minutes and peak discharge of 46272 m^3/s will start flowing in 19 min from the arrival time of flood. The total of 62 min is taken by peak flood to flow over this region from the time of start of dam break. After this region the peak discharge start decreasing rapidly and at 17 Km d/s it comes down to 24569 m^3/s, still it is sufficiently large to do the disaster d/s of this region. The time of arrival of flood for this region is 57 minutes and peak .discharge will arrived in 6 min. There is huge fluctuation and large decrease in the peak value of discharge at this location is observed. This can be predicted that maximum flood water is spill over the flood banks in the region from 13 Km to 17 Km. So, in this thesis this region is seems to be most critical region for flooding and we conclude results in terms of arrival time of peak flood in downstream valleys of the river Nagavali from dam site. The data is further analysed with the longitudinal bed profile, water level graphs, and cross-sections of the river and flood map.

6. A.3 Longitudinal Bed Profile

Fig.7 shows the longitudinal bed profile of river Nagavali, minimum bank Level, maximum water level reached due to dam break in the Nagavali River downstream of the dam site. As we analysed from the longitudinal profile and from the study of topography of the area situated near the Nagavali River that the from the dam site about 1.5 Km to 3 Km d/s the flooded water will enter the flood plains. Fig 8 to Fig 11 shows the Cross sections of river at 1.45 Km, 2.45 Km, and 9.45 Km and with maximum water level and the time of occurrence of the maximum water level

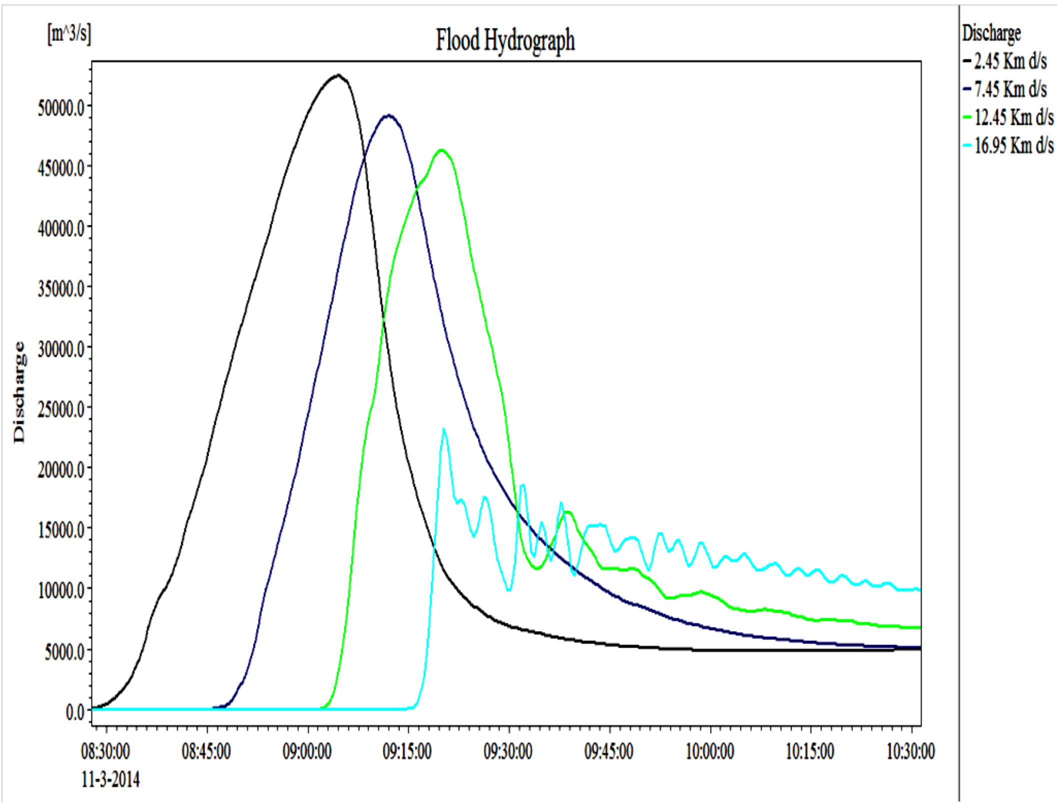

Fig. 6.1: Flood Hydrographs for 2.45 Km, 7.45 Km, 12.45 Km and 16.95 Km d/s of the Lower Nagavali dam

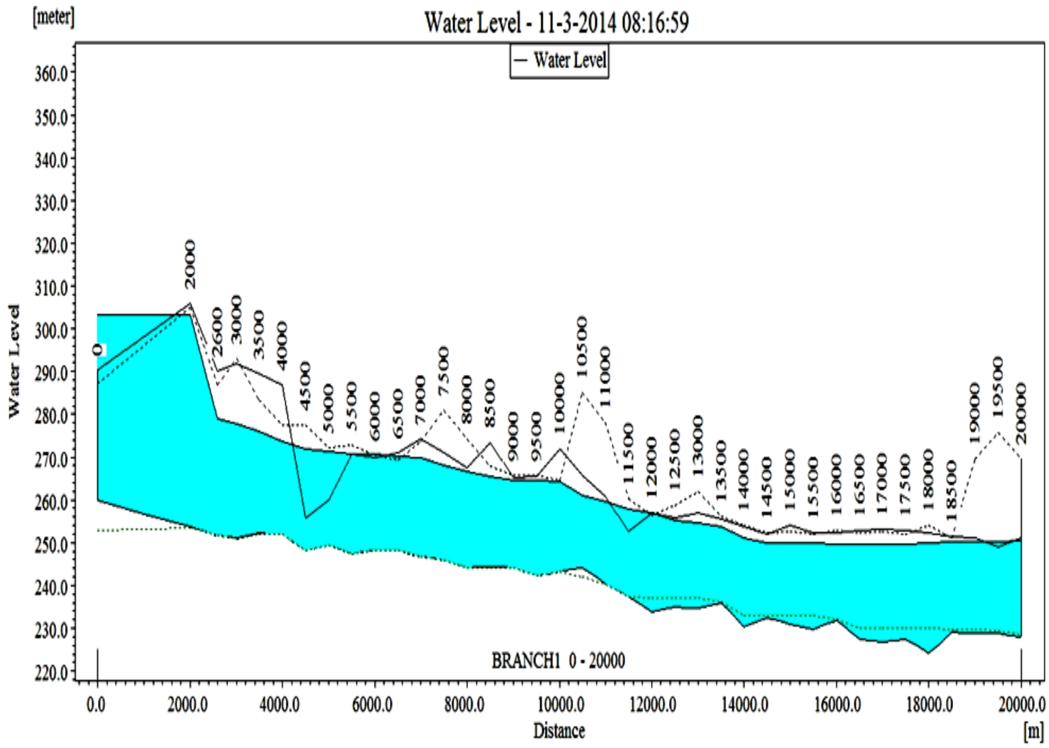

Fig.6.2: Longitudinal bed profile of Nagavali River showing maximum water levels

6. A.4 Routing of Water Level Hydrograph

Water level scenario for four Chainage points (2.45 Km, 7.45 Km, 12.45 Km and 16.45 Km) are explained in Table 9 and Fig. 8. Cross sections with maximum water level for the four Chainage points (1.45 Km. 2.45 Km, 9.45 Km and 16.45 Km d/s from the dam) are shown in Fig. 9, Fig. 10, Fig. 11 and Fig. 12.

Table 9: Max. WL and Arrival Time of flood of Lower Nagavali River

Distance d/s of dam (Km)	Max. W.L (m)	Arrival time of flood (min)	Max. W.L time after the arrival time of flood (min)	Max. W.L time from the start of D.B (min)
2.45	271.2	9	41	50
7.45	264	28	27	55
12.45	249.8	43	35	78
16.45	250.36	57	12	69

Time of dam break is 08:17:00 am in the model, W.L denotes water level, d/s denotes downstream, D.B denotes dam break

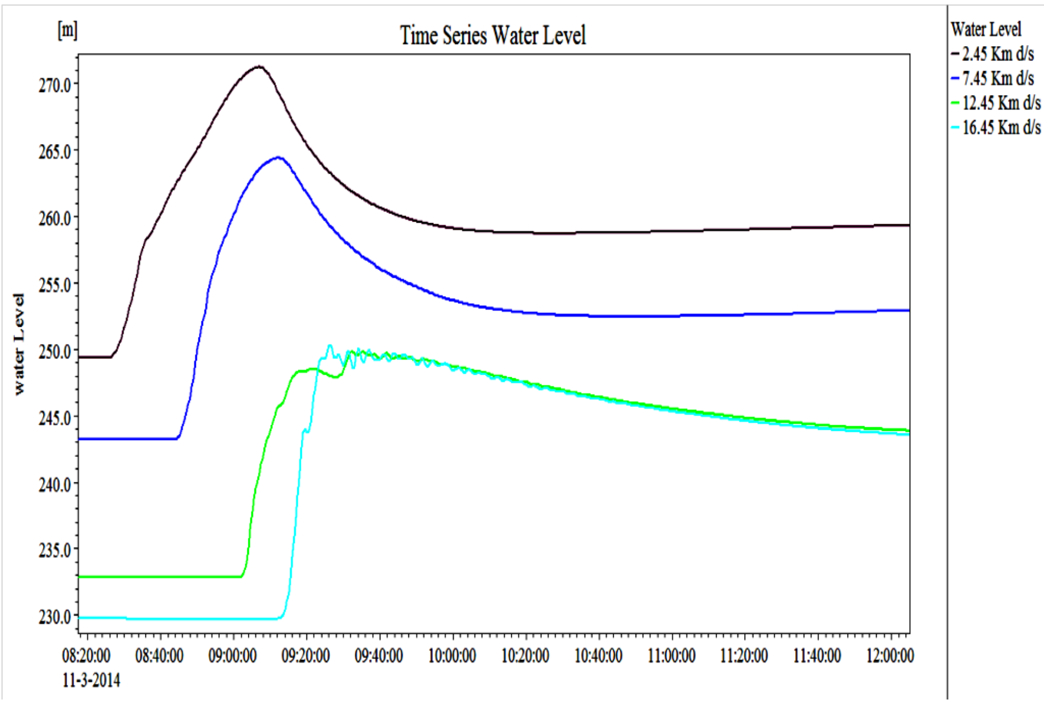

Fig. 6.3: WL for 2.45 Km, 7.45 Km, 12.45 Km and 16.45 Km d/s of the LN dam

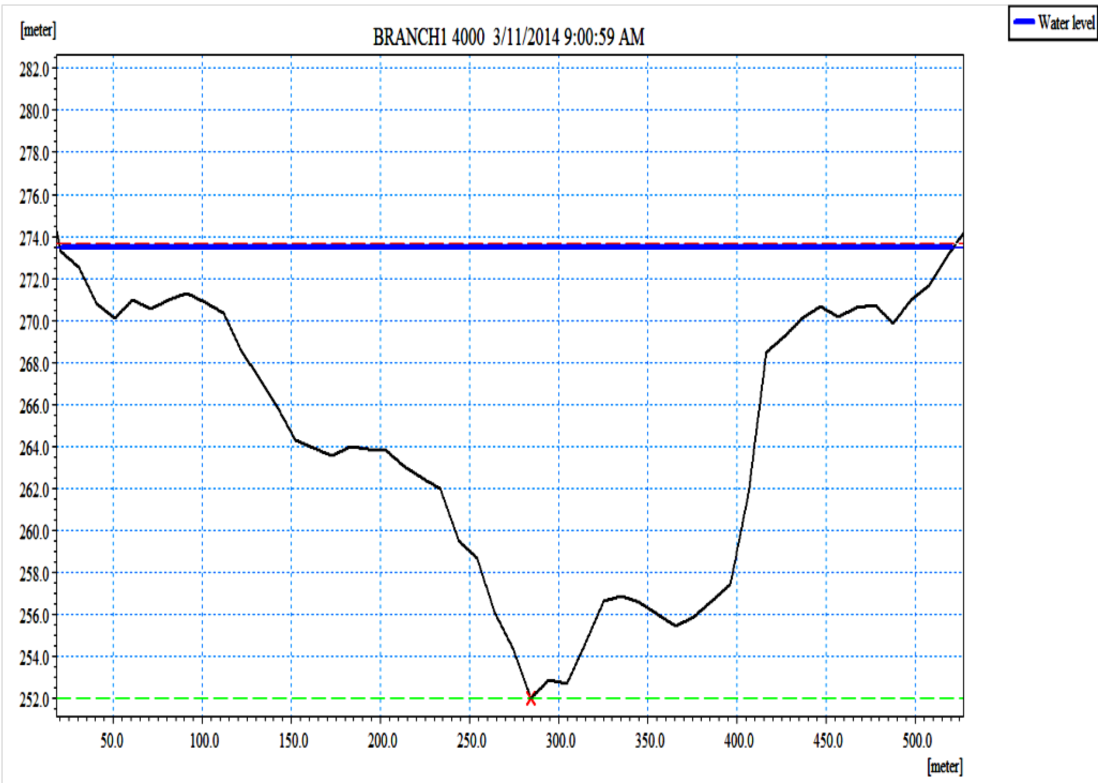

Fig. 6.4: River cross section at 1.45 Km d/s from the Lower Nagavali dam

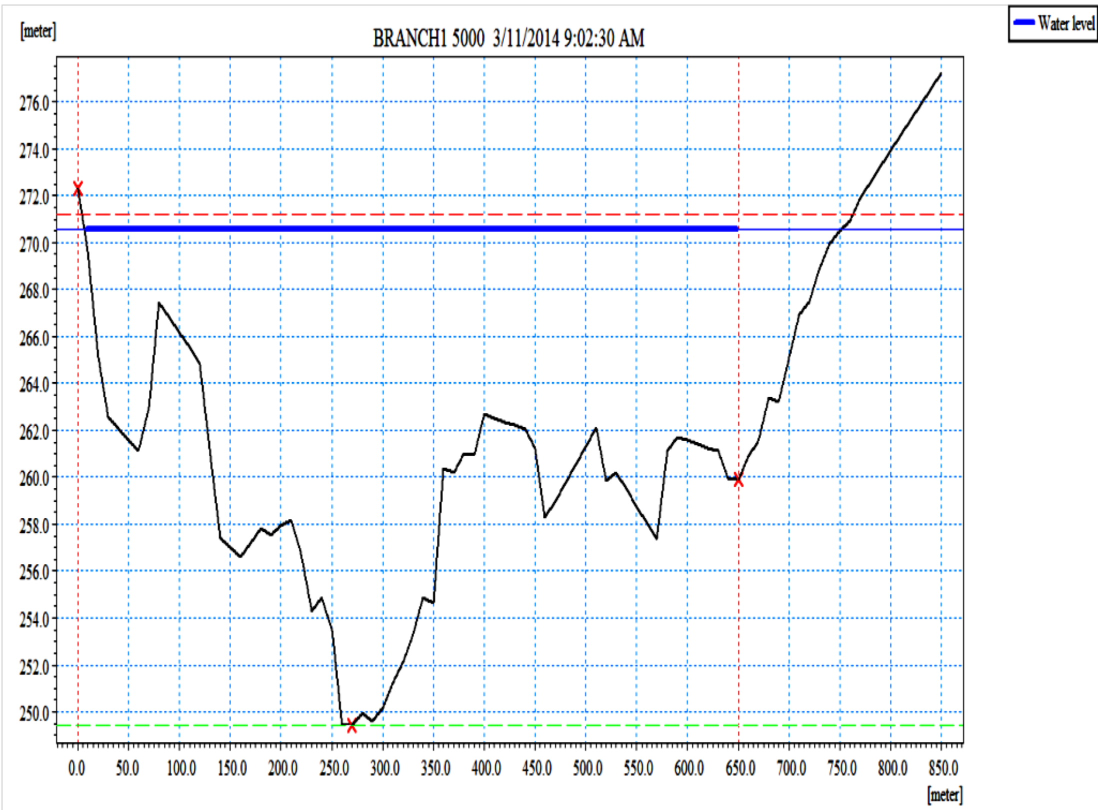

Fig. 6.5: River cross section at 2.45 Km d/s from the Lower Nagavali dam

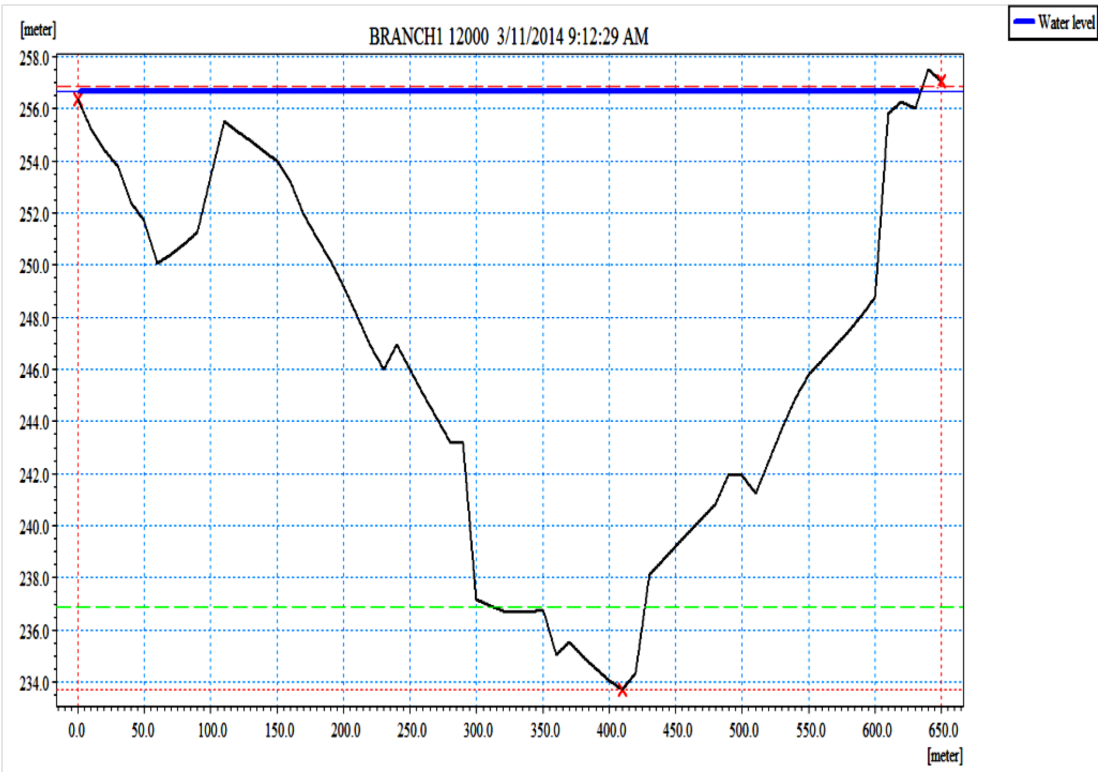

Fig. 6.6: River cross section at 9.45 Km d/s from the Lower Nagavali dam

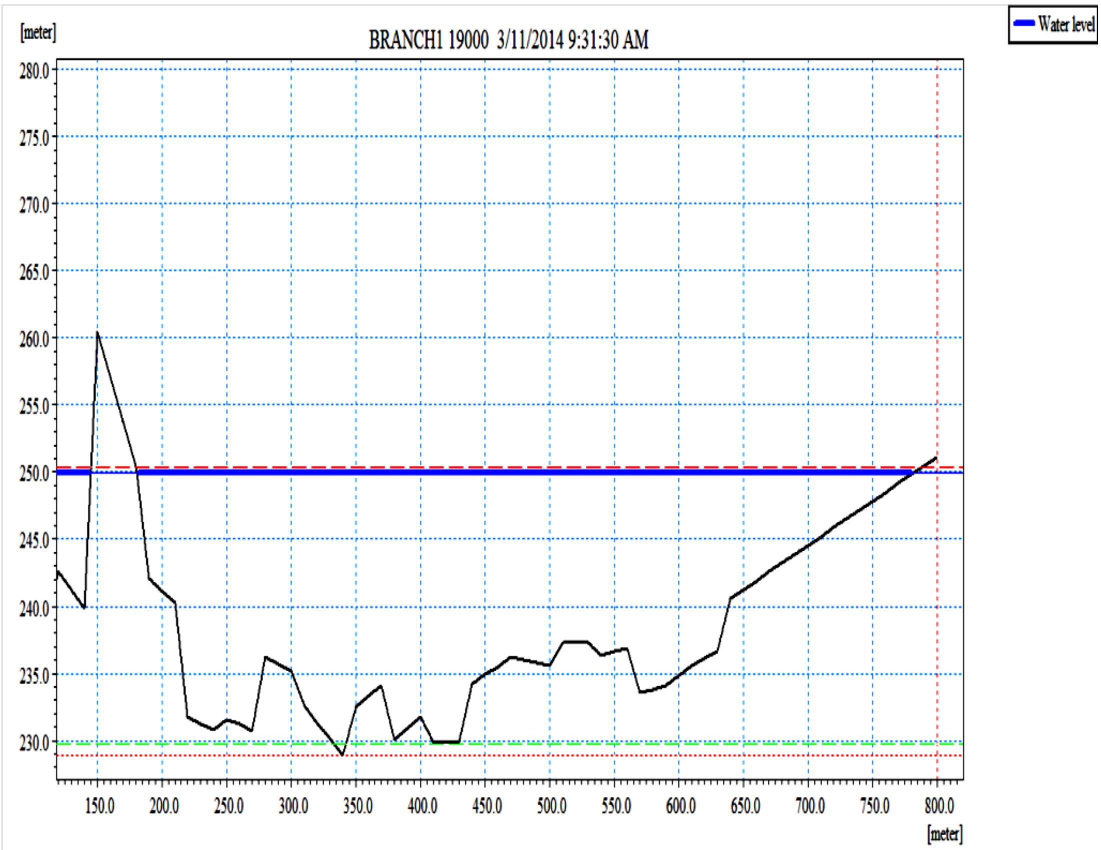

Fig. 6.7: River cross section 16.45 Km d/s from the Lower Nagavali dam

Flood Map for Lower Nagavali:

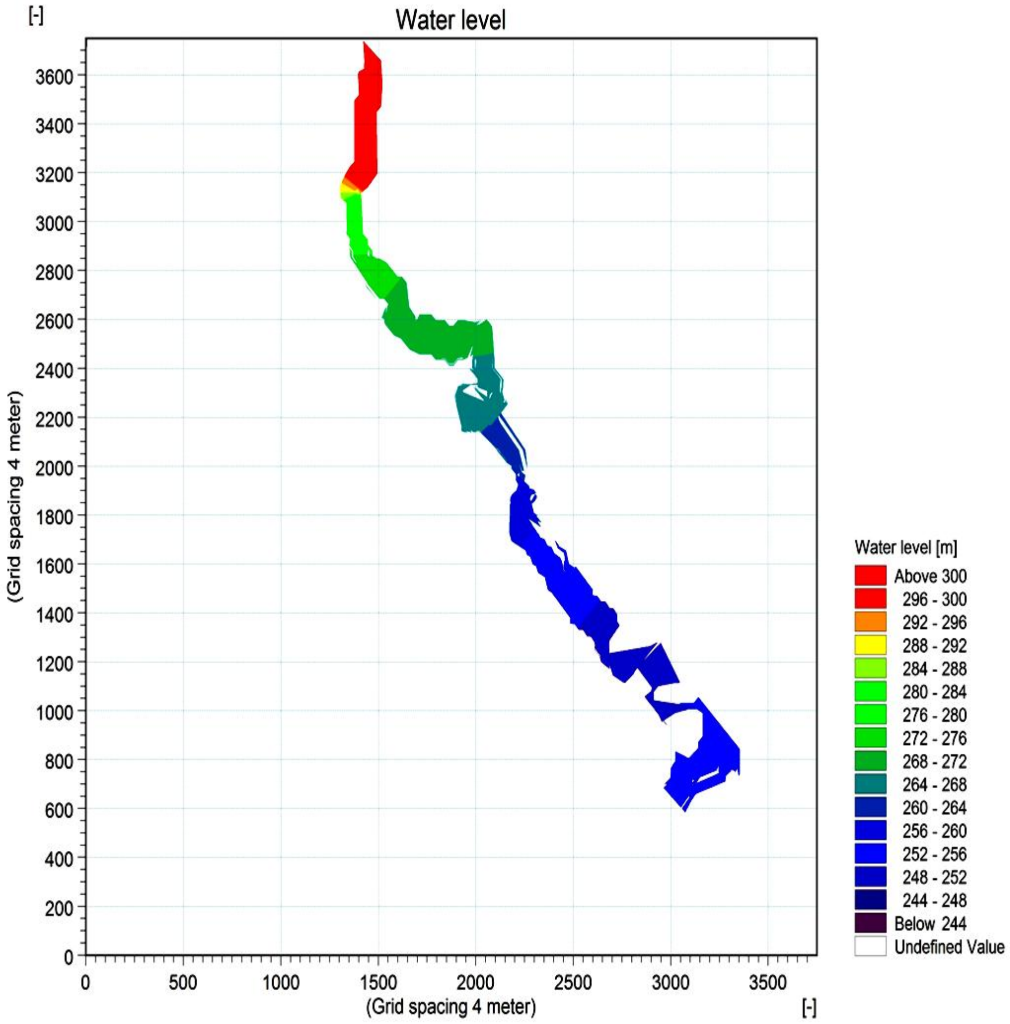

Fig. 6.8: Flood Map of Lower Nagavali river

6. A.5 Sensitivity analysis for various inputs to the model setup in terms of peak discharge and Water Levels

As we know the selection of input parameters for the dam break model are very important to do the analysis. If we change the values of these input parameters to the model setup then what is the effect on discharge values and water levels is analysed and this analysis part is known as sensitivity analysis. So Input parameters which are considered for the sensitivity analysis are:

a) Breach Time
b) Breach Width
c) Side slope
d) Manning's roughness
e) Inflow hydrograph

For the full study of Lower Nagavali Dam break the results are obtained, analysed and compared with different dam break scenarios as explained in Table 11. Further the whole analysis is done on the different scenarios as explained bellow

a) Effect of Breach Time

In this section Setup 2, Setup 10, Setup 14, Setup 18, Setup 22 are compared with the Setup 6

Flood Hydrographs

Fig. 14 shows the Flood Hydrograph coming out from the breached dam for different breach time. The sensitivity of discharge is analysed by changing the breach time parameters which are explained with the setups explained in Table 6.3. Breach time has more impact on discharge than the other breach parameters. When the breach width is constant (154.47 m) as for the present study then with the 20% increase in breach time there was decrease in peak discharge by 12.47% at the dam site and with 20% decrease in breach time there was increase in peak discharge by 17.03%. Similarly, the effect of 40%, 60%, and 80% decrease in breach time increases the peak discharge by 39.5%, 64.5% and 89.8% respectively.

Water Levels

Breach time is the time of development of breach fully in the dam structure and we know that earthen dams are assumed to be breaches gradually. When the breach time is decreased means less time the breach will take to develop fully. As with the decrease in time less volume of runoff will pass through the breach into the downstream channel. So in the reservoir the surface water elevation is still high and as the breach is fully develop in short time the depth of flow is more over the crest of the breach for long time leading to more increase in the magnitude of peak discharge passing jointly through these breach. The more magnitude of peak discharge will result in more peak of water level in the downstream. This phenomenon is shown in the Fig. 15 which shows the maximum water level for downstream distance from the dam for setup 2, setup 6, setup 10, setup 14, setup 18 and setup 22. Further the increase in water level in 2.45 Km, 7.45 Km and 15.45 Km downstream locations as with the decrease in breach time is shown in Fig. 16 (a) to Fig 16 (e).

b) Effect of Breach Width

The setup 5, setup 6, setup 7 and setup 8 shows the change of breach width by making breach time constant and the results obtained from these setup is analysed as how much the breach width will affect the peak discharge and water level downstream the valley. When breach width is increased from 3* HD to 4*HD there is 3.9 % increase in the peak discharge is noticed and when breach width is increased to 5* HD then7.1 % increase in peak discharge is noticed. So, with the change of breach width there is slightly increase in peak discharge from the breach dam and almost same peak water level along the downstream location is observed. Fig.6.17, Fig.6.18, Fig.6.19, Fig.6.20, Fig.6.21, and Fig.6.22 shows the effect of breach width on discharge and water level.

c) Effect of Side Slope

The side slope is the lateral slope of trapezoid of the breach section. The model is test for the side slopes of 0.25, 0.5 and 0.75. Results obtained from these models shows not much change in the value of maximum water level and discharge for the downstream location. So, we conclude that sensitivity of this parameter has insignificant effect on the peak values of water level and discharge.

Table 10: DB Modelling for different Breach Parameters of LN River

Scenario	Breach Time (min)	Breach Width (m)	Breach Slope
Setup 1	60	2*HD	1H:1V
Setup 2	60	3*HD	1H:1V
Setup 3	60	4*HD	1H:1V
Setup 4	60	5*HD	1H:1V
Setup 5	50	2*HD	1H:1V
Setup 6	50	3*HD	1H:1V
Setup 7	50	4*HD	1H:1V
Setup 8	50	5*HD	1H:1V
Setup 9	40	2*HD	1H:1V
Setup 10	40	3*HD	1H:1V
Setup 11	40	4*HD	1H:1V
Setup 12	40	5*HD	1H:1V
Setup 13	30	2*HD	1H:1V
Setup 14	30	3*HD	1H:1V
Setup 15	30	4*HD	1H:1V
Setup 16	30	5*HD	1H:1V
Setup 17	20	2*HD	1H:1V
Setup 18	20	3*HD	1H:1V
Setup 19	20	4*HD	1H:1V
Setup 20	20	5*HD	1H:1V
Setup 21	10	2*HD	1H:1V
Setup 22	10	3*HD	1H:1V
Setup 24	10	4*HD	1H:1V
Setup 25	10	5*HD	1H:1V

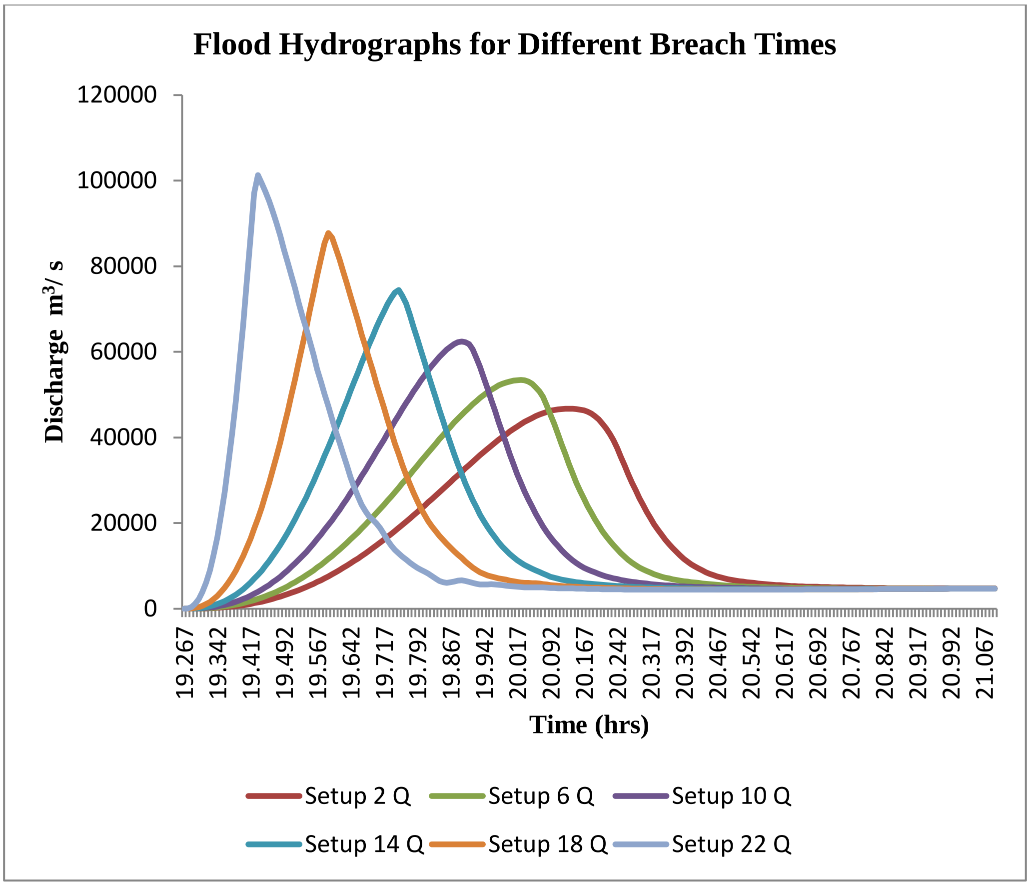

Fig.6.9: Sensitivity of BT on Flood Hydrograph of Lower Nagavali breached dam

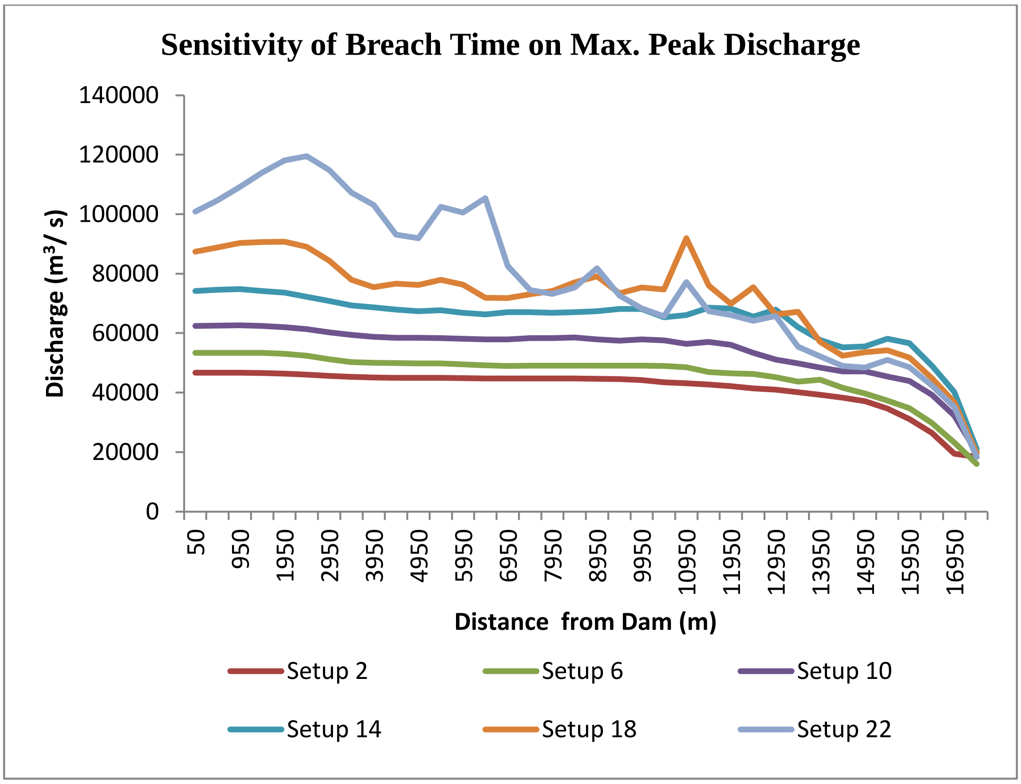

Fig.6.10: Sensitivity of breach time on Max. Peak Discharge of Lower Nagavali River

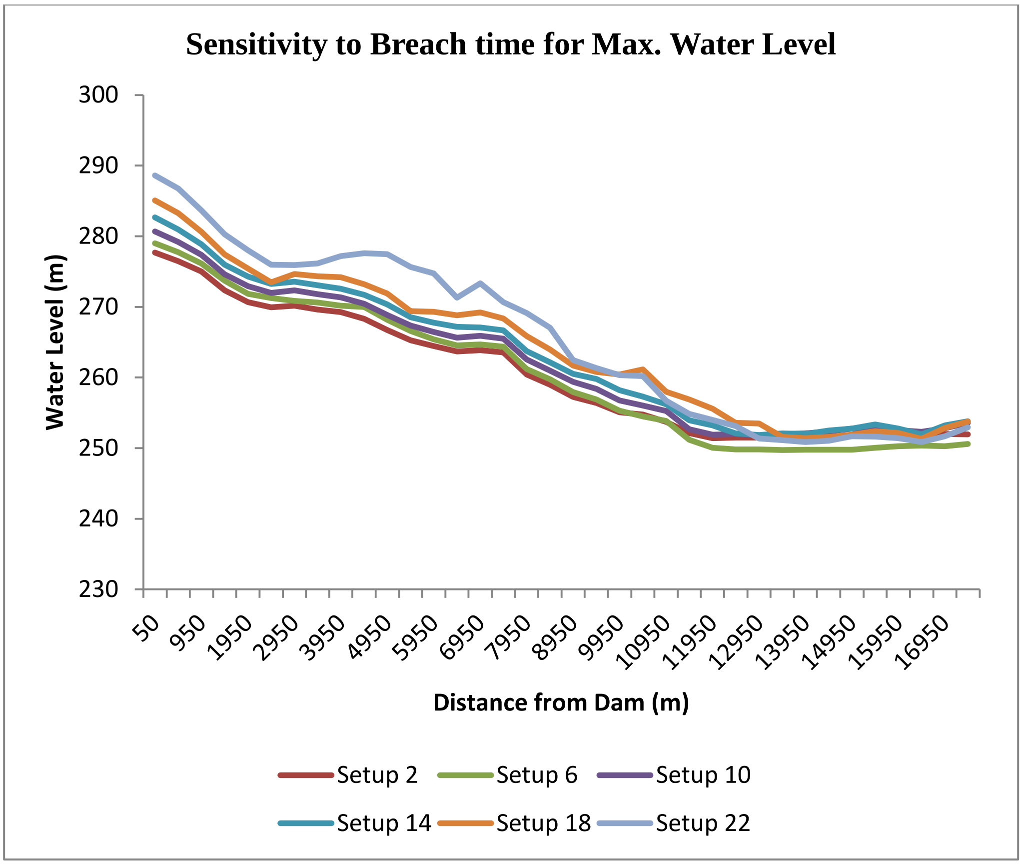

Fig.6.11: Sensitivity of Breach Time on Max. WL of Lower Nagavali river

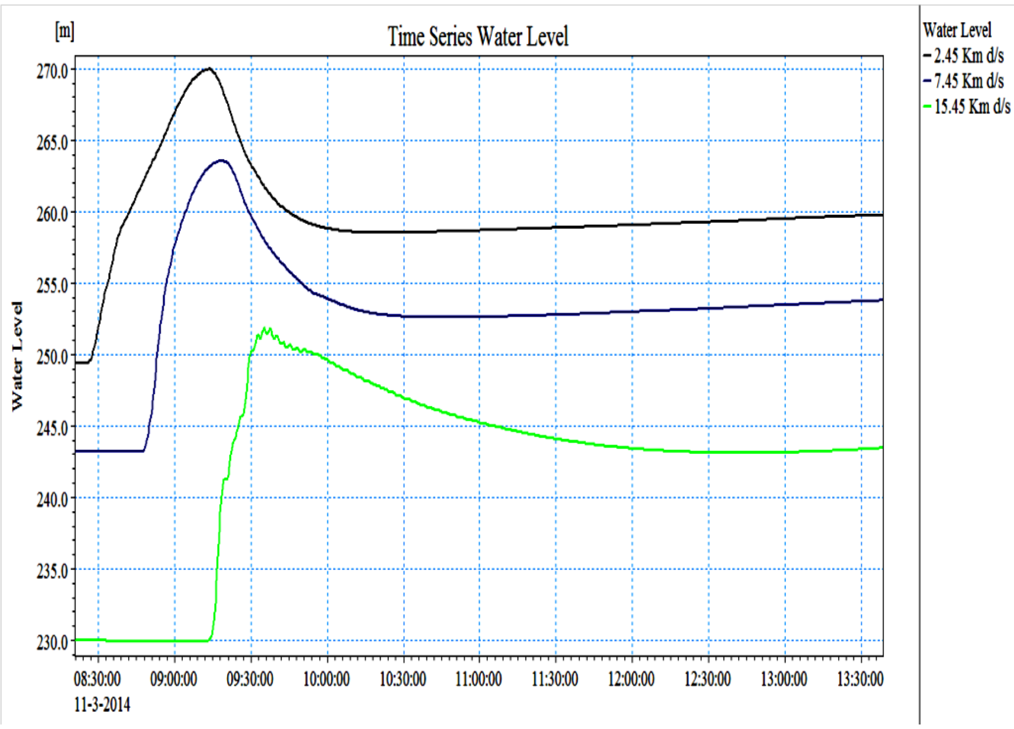

Fig.6.12: WL Hydrograph for setup 2 at selected locations of Lower Nagavali River

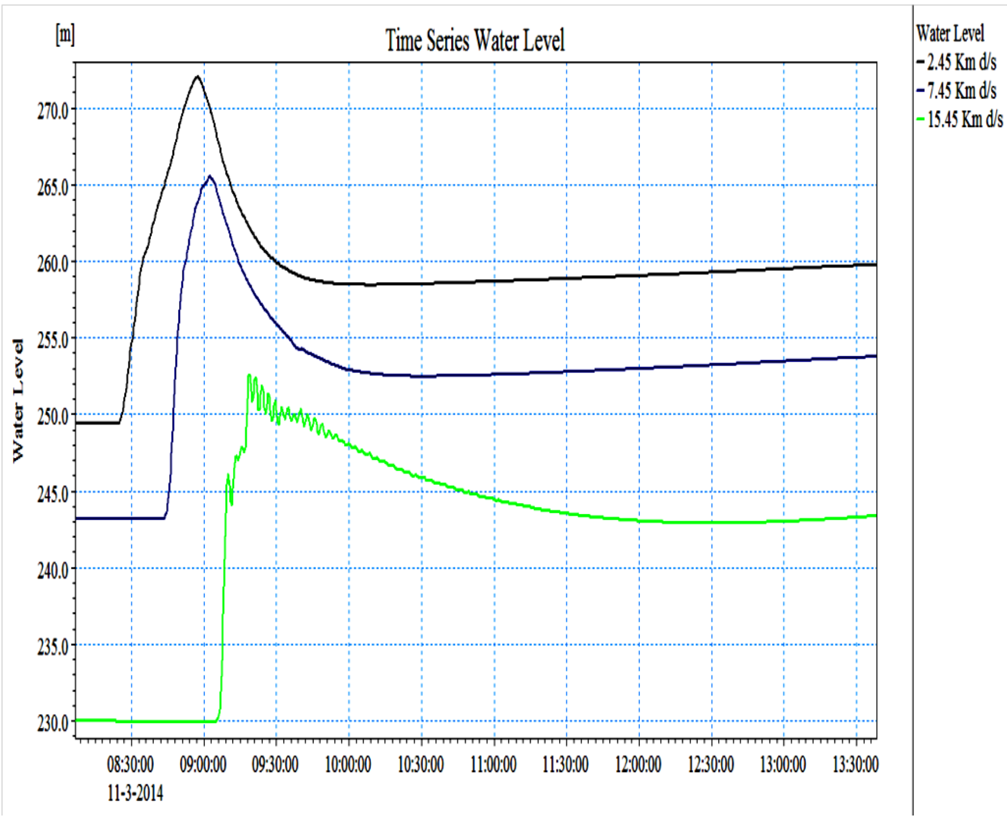

Fig.6.13: WL Hydrograph for setup 10 at selected locations of Lower Nagavali River

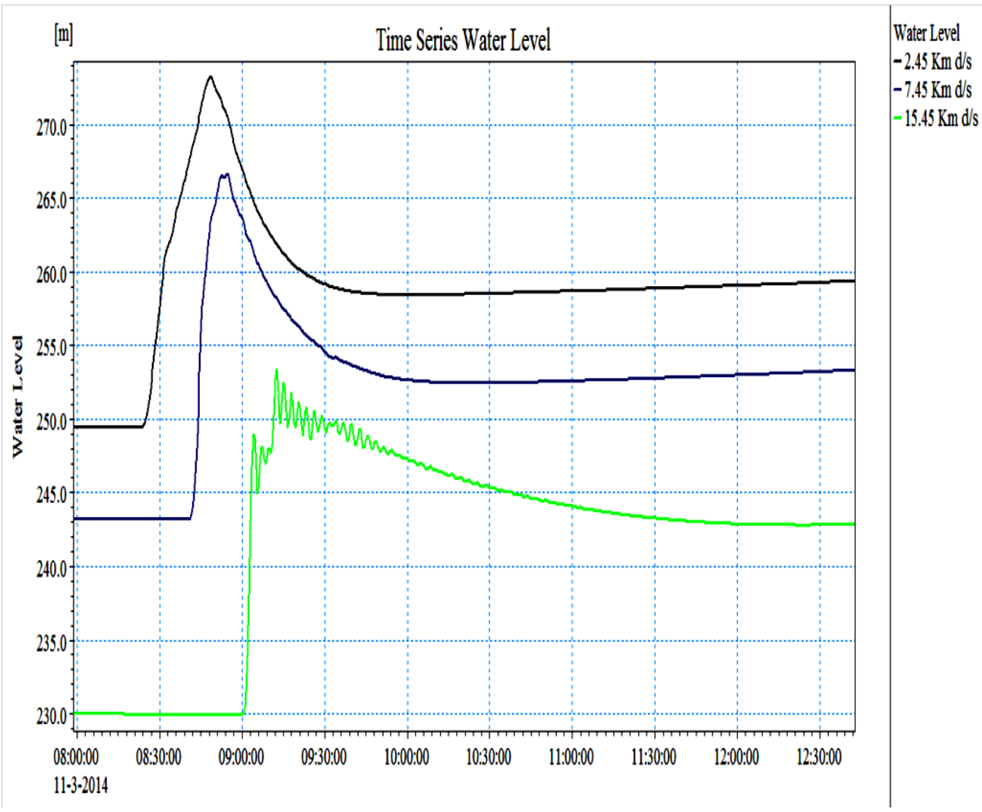

Fig.6.14: WL Hydrograph for setup 14 at selected locations of Lower Nagavali River

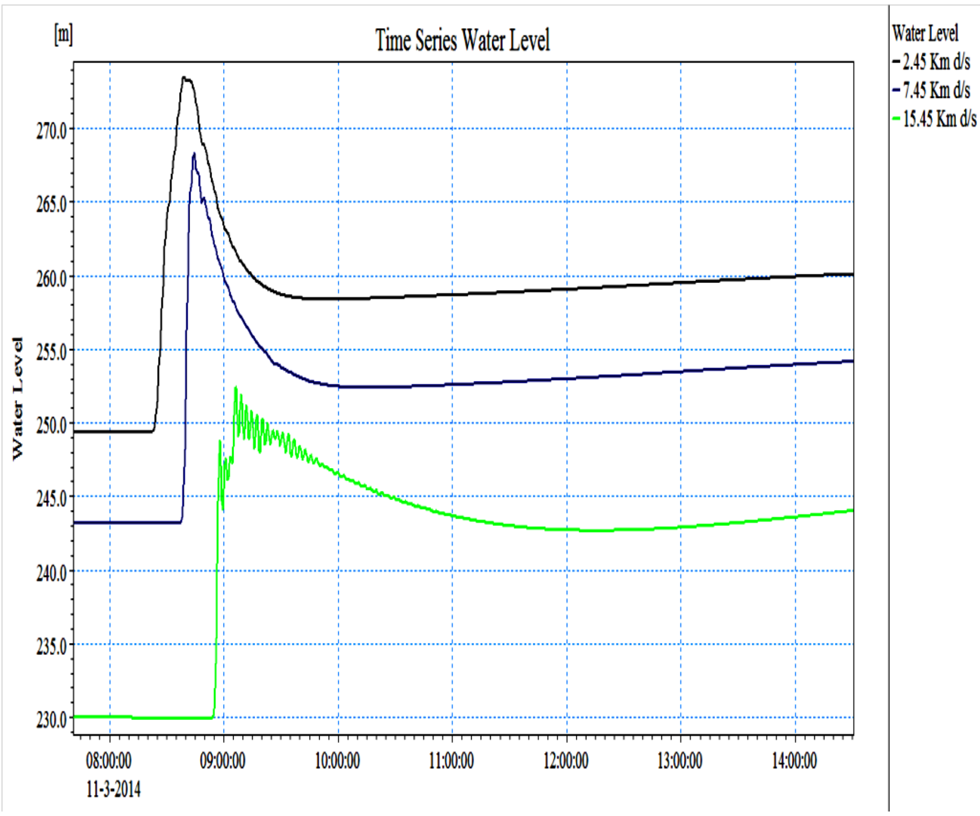

Fig.6.15: WL Hydrograph for setup 18 for at selected locations of Lower Nagavali River

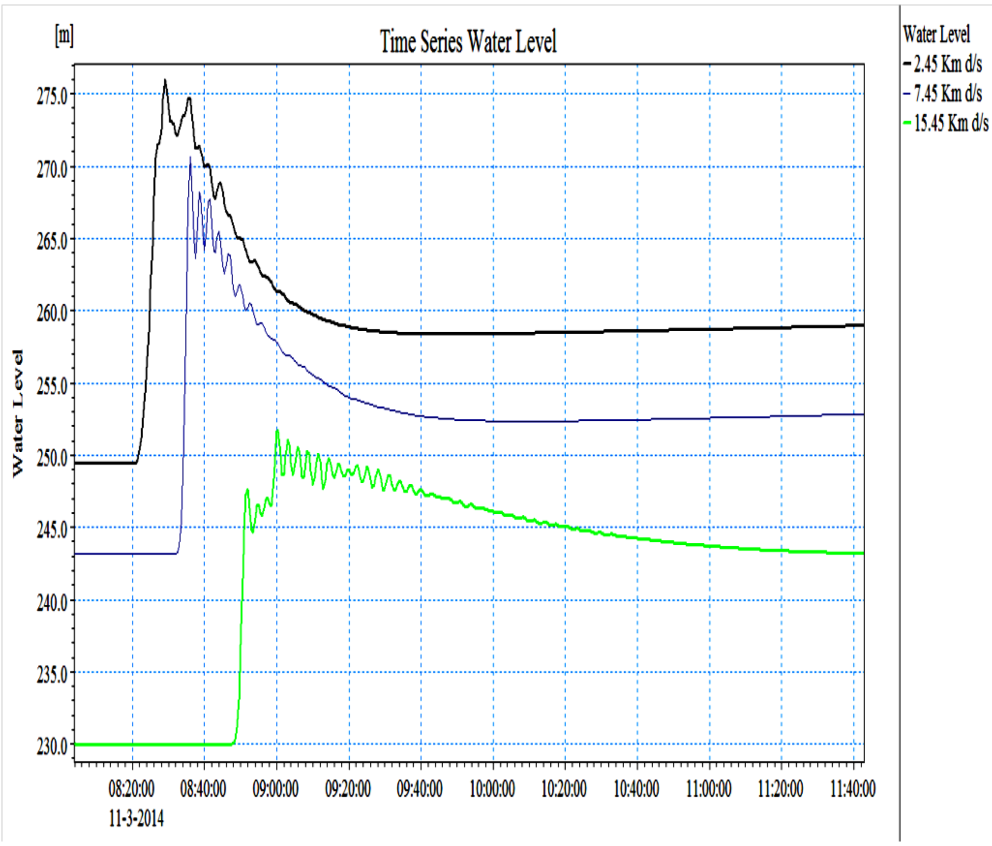

Fig.6.16: WL Hydrograph for setup 22 at selected locations of Lower Nagavali River.

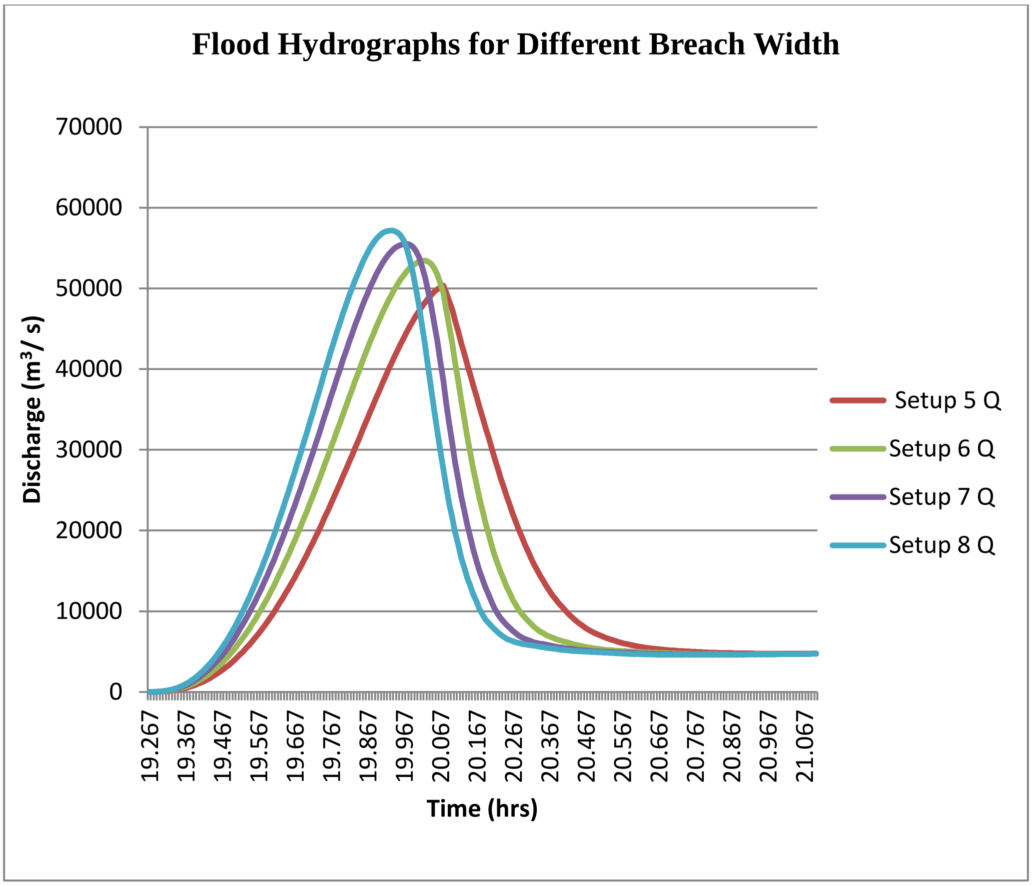

Fig.6.17: Sensitivity of BW on Flood Hydrographs of Lower Nagavali breached dam

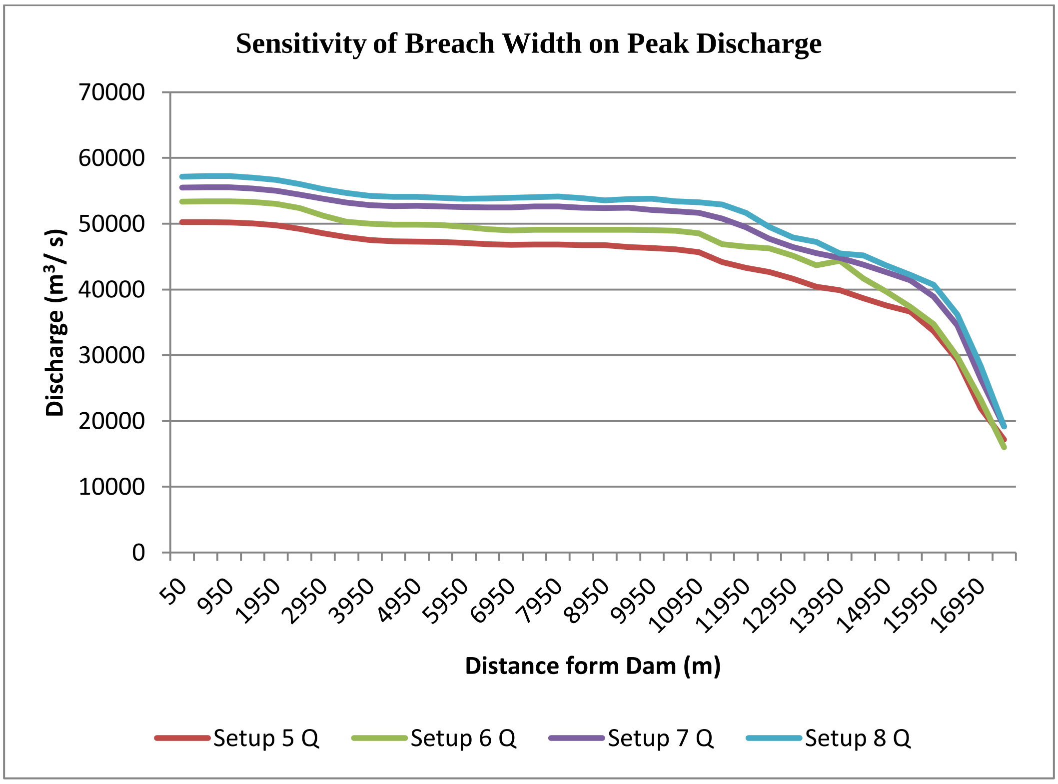

Fig.6.18: Sensitivity of Breach Width on Peak Discharge of Lower Nagavali river.

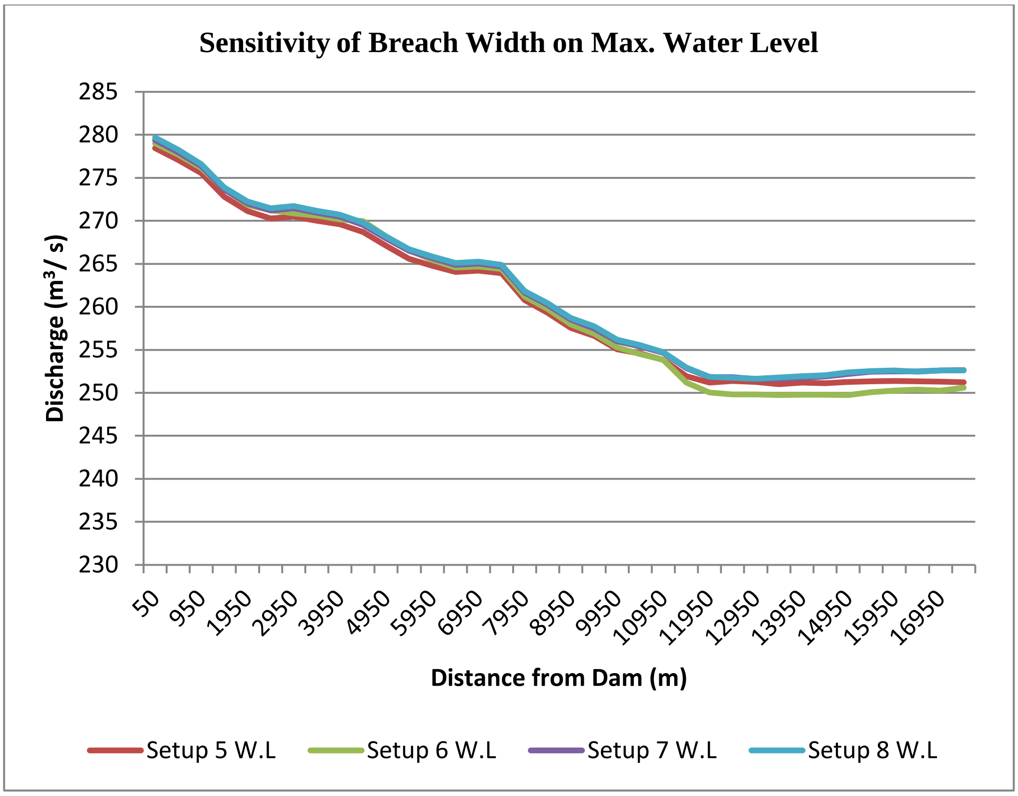

Fig.6.19: Sensitivity of Breach Width on Max. Water Level of Lower Nagavali river

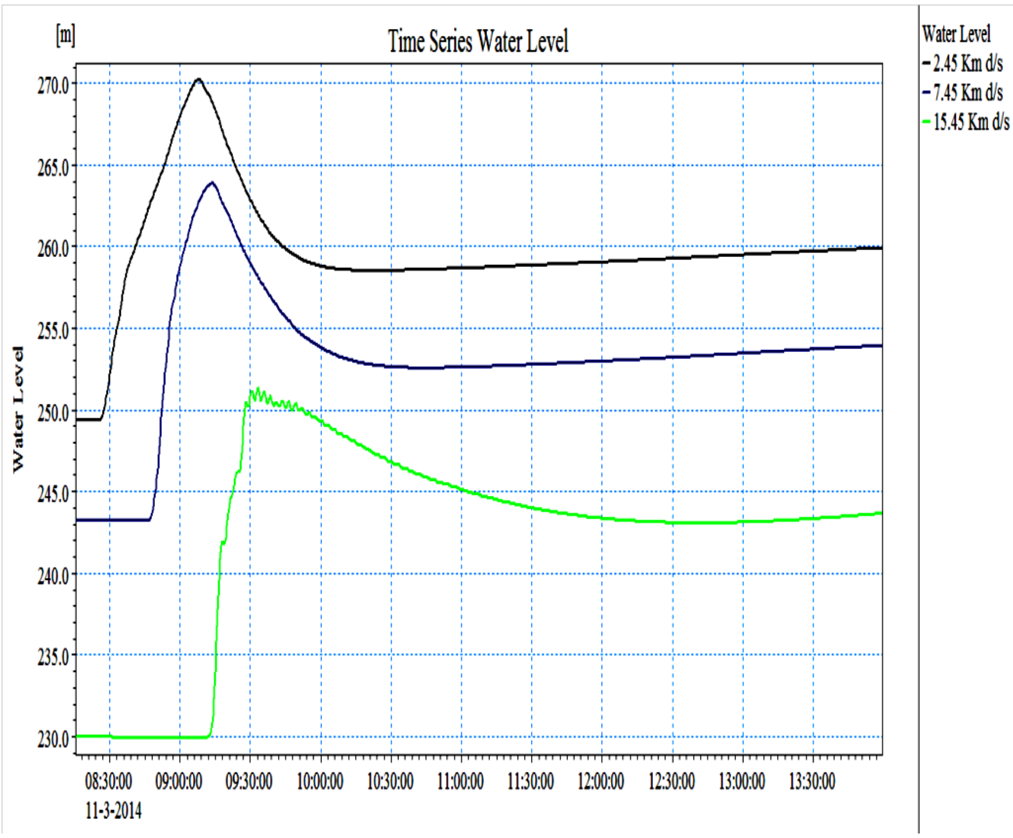

Fig.6.20: WL Hydrograph for Setup 5 at selected locations of Lower Nagavali River

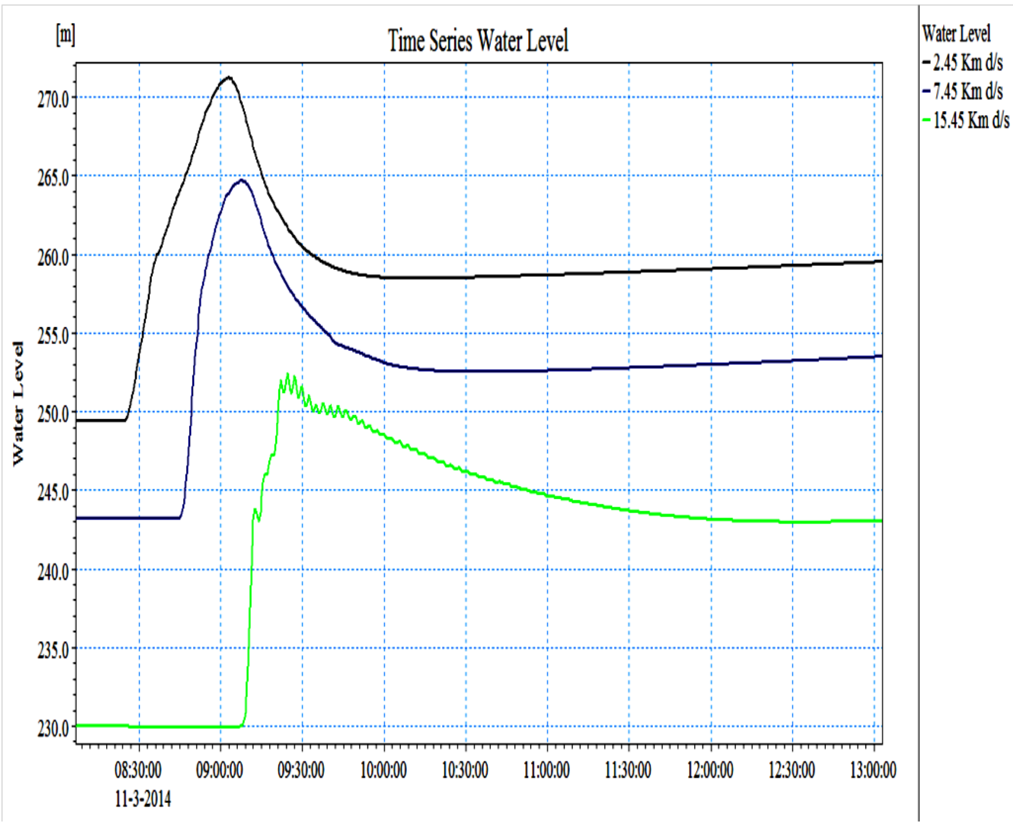

Fig.6.21: WL Hydrograph for Setup 7 at selected locations of Lower Nagavali River

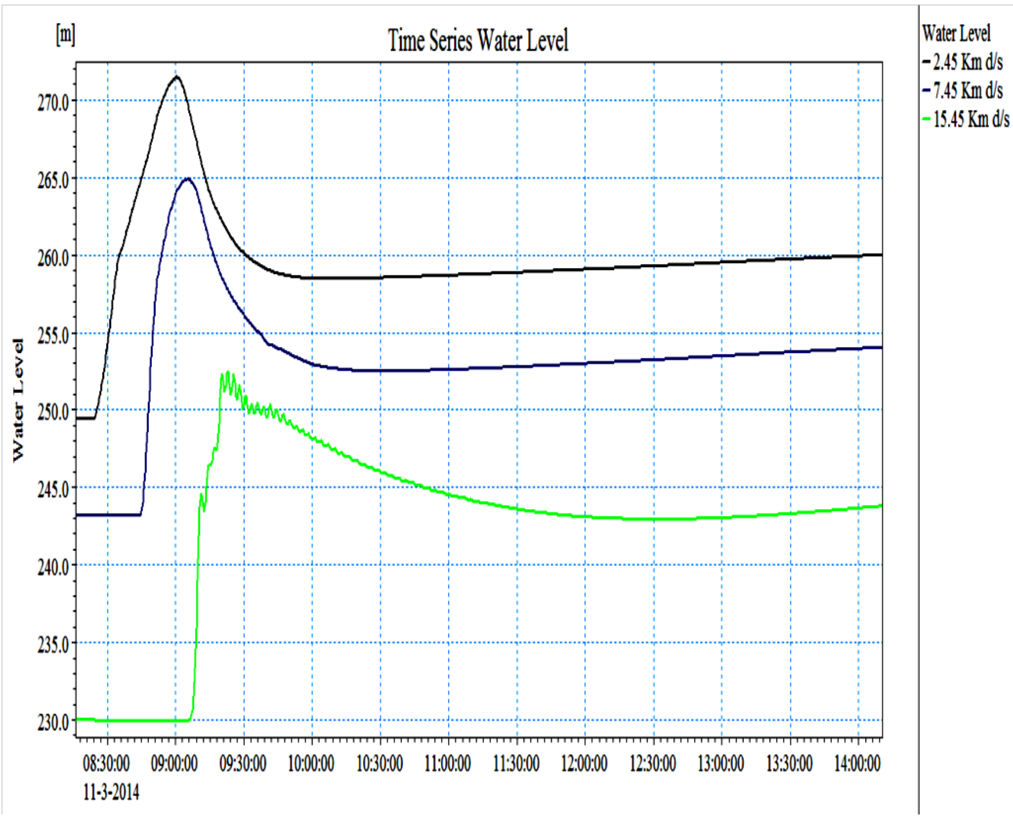

Fig.6.22: WL Hydrograph for Setup 8 at selected locations of Lower Nagavali River

d) Effect of Manning's roughness (N)

As we Know, when the Manning's Roughness Coefficient (N) increases there is loss of energy which will affect the wave speed. This loss of energy is dissipated in the atmosphere through the bounding walls of the channel or the water surface. Chow, 1959 has been suggested us the value of Manning's N in the range of 0.03 to 0.05 for the regions showing gravels, cobbles and few boulders at the bottom with no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage as discussed earlier. As expected the velocities reduce with increase in Manning's N, and vice versa. This will affect the maximum water level and discharge value also. As from Table 12 it has been noticed that there is increase in max. water level at nearby regions from dam but there is very slightly increase of water level for the far distance from the dam locations. For Peak Flood Manning's N plays crucial role and it is shown in Fig. 18 (a) and Fig. 18 (b).

Table 11: Sensitivity of Manning's Roughness

Location from dam (Km)	Discharge (m^3/ s)			Max. Water Level (m)			Velocity (m/s)		
	N= 0.033	N= 0.4	N= 0.45	N= 0.033	N= 0.4	N= 0.45	N= 0.033	N= 0.4	N= 0.45
2.45	52367	51498	50618	271.3	272.2	272.8	6.62	5.85	5.36
7.45	49055	45021	41408	264.3	264.8	264.7	12.81	11.06	10.29
15.45	37356	31616	26528	250	249.2	48.8	1.7	1.62	1.6

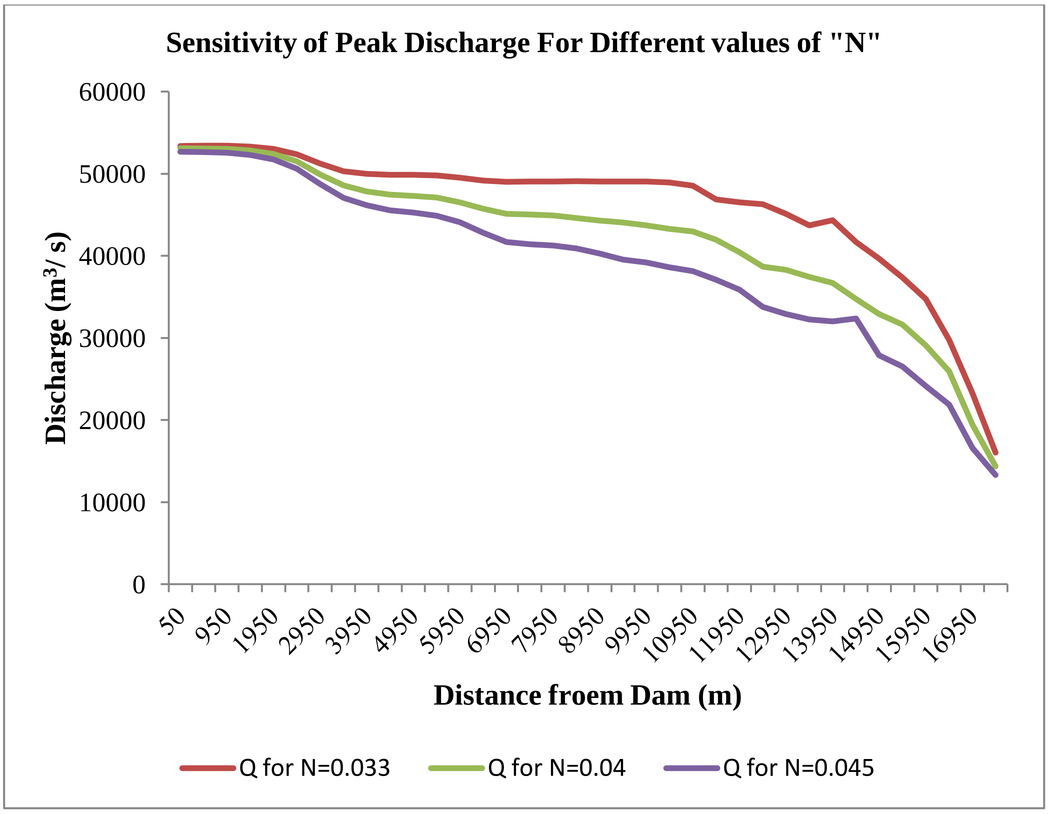

Fig.6.23: Sensitivity of "N" on Peak Discharge for LN River

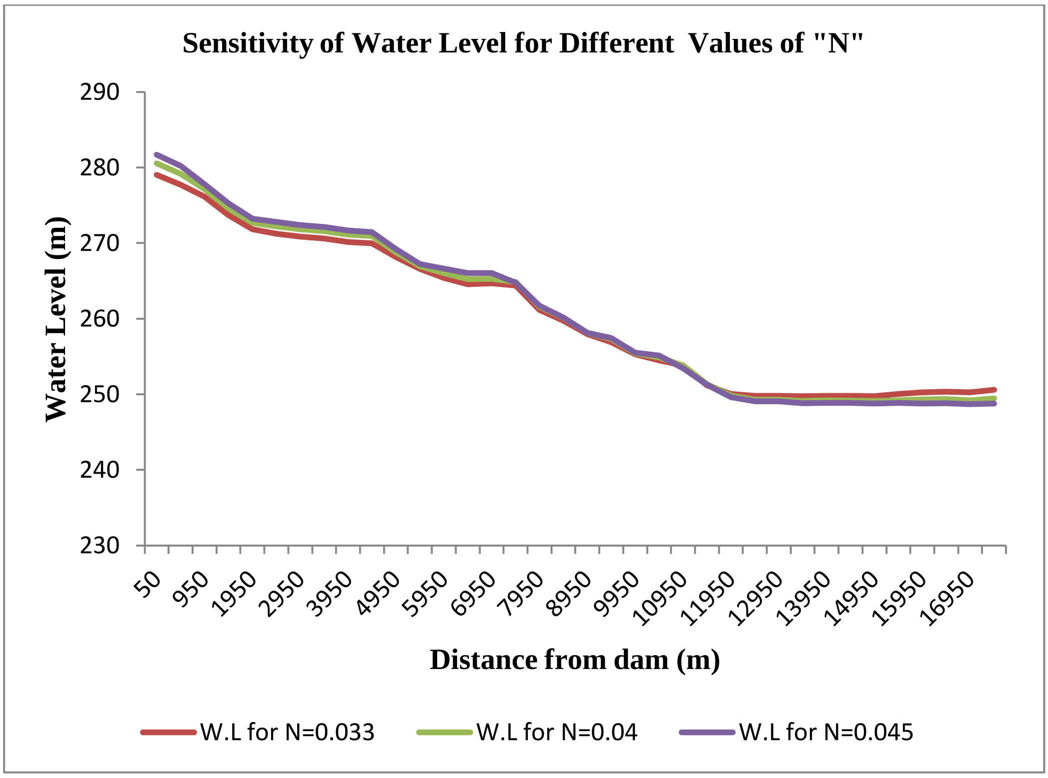

Fig.6.24: Sensitivity of "N" on Max. Water Level for LN River

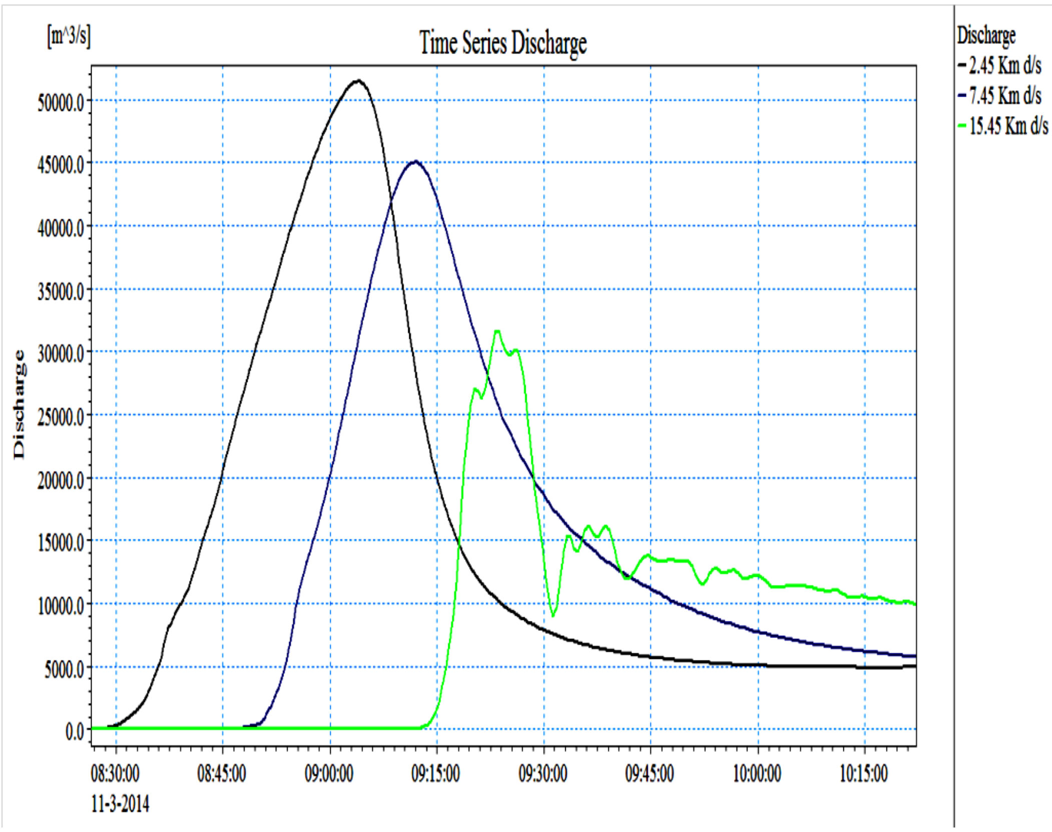

Fig.6.25 Flood hydrographs for N=0.04 at selected locations of LN River

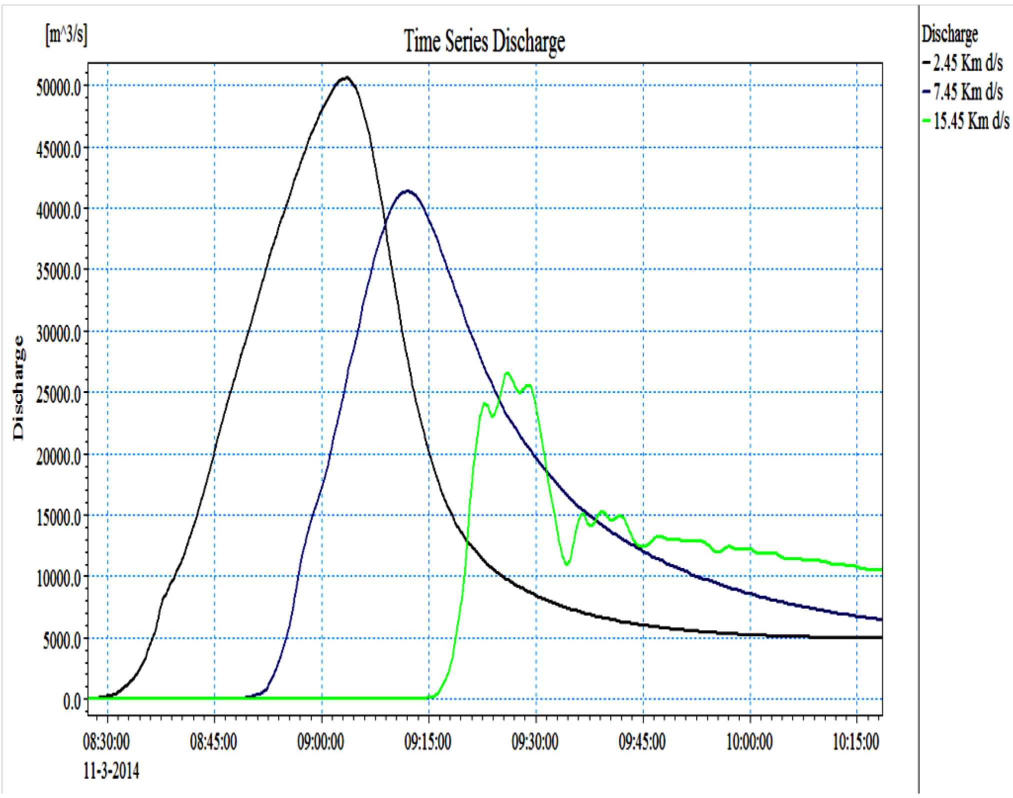

Fig.6.26: Flood hydrographs for N=0.045 at selected locations of LN River

e) Inflow Hydrograph

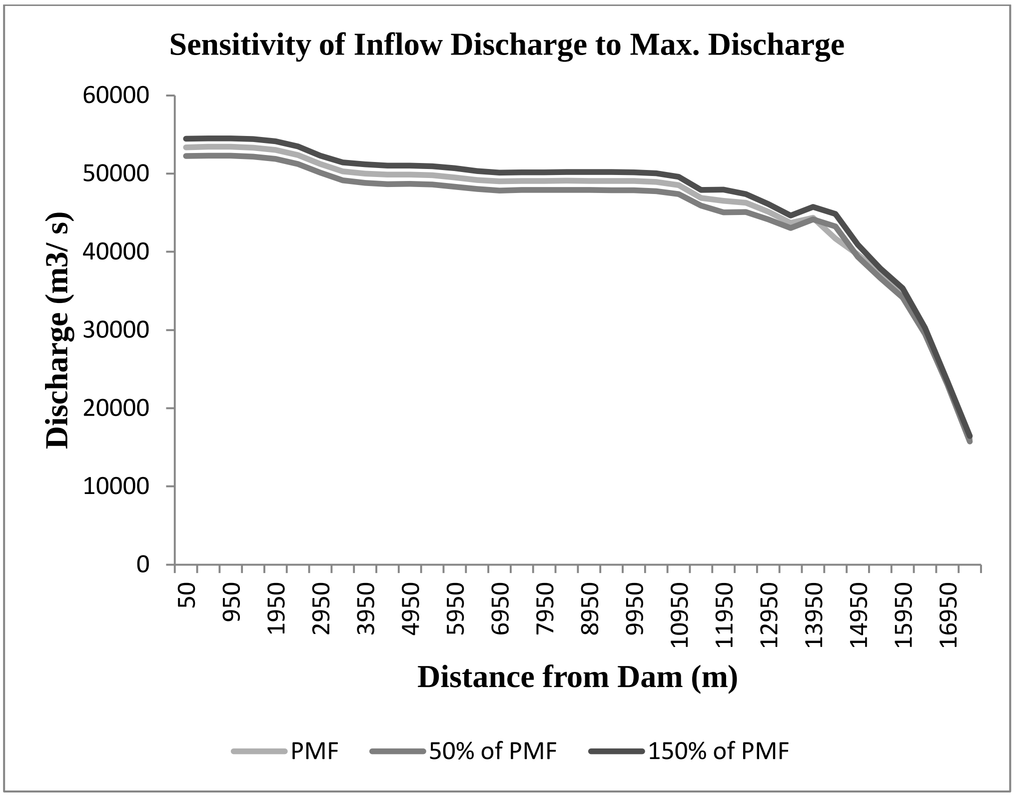

Fig.6.27: Sensitivity of Inflow Discharge to Max. Discharge for Lower Nagavali river

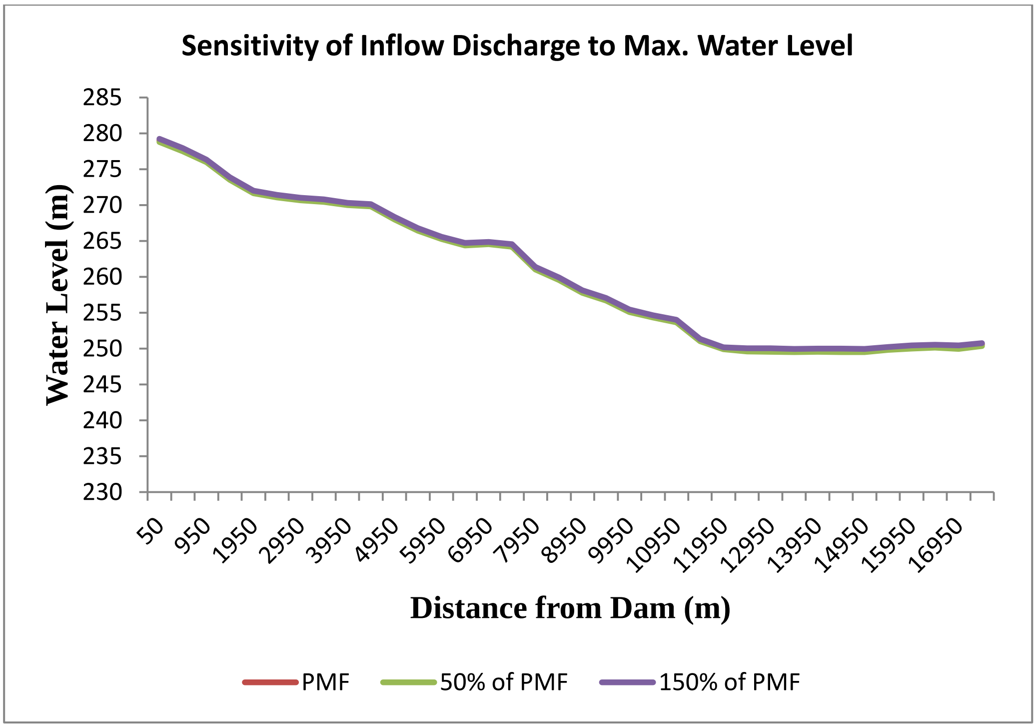

Fig.6.28: Sensitivity of Inflow Discharge to Max. Water Level for Lower Nagavali river

6. A.6 Dam Break Analysis for different Breach Parameters

When we conduct further analysis for different breach parameters which are denoted in this thesis as different breach scenarios as explained in Table 11, we observe that when breach time is 10 minute and breach width is 5 times the height of dam then the peak discharge of 123452 m^3/ s is flow out of the breached dam which is 13.4 times greater than the PMF. This amount of peak discharge is seriously a danger for downstream locations. So, for this scenario i.e. setup 25 further analyses is required and which is done in next section. As we see in the Table 12 the peak discharge values for different breach parameters, the minimum discharge of 44475 m^3/ s was came out when breach time is maximum and breach width is small. But duration of flood for this setup 1 is long than the other breach setups. Further all these setups are explained in sensitivity analysis.

Table 13 shows the percentage increase in peak discharge from the peak discharge of 2*HD of the same breach time. We analyse that for maximum breach time the variation in percentage of peak discharge for different breach width is less than the other breach parameters. For breach time 10 min 30 % increase of discharge is noticed when breach width is changes from 2*HD to 3*HD. Further increase in breach width from 2*HD to 4*HD than about 48 % increase in peak discharge and for 5*HD it was 58 %. For breach time 20 min the increase in peak discharge of 24%, 36%, and 43% was noticed.

Table 12 Peak Discharge (m^3/s) for Different Dam Breach Conditions at Dam Location chainage 2550 m

Breach width	Breach time 10 min	Breach time 20 min	Breach time 30 min	Breach time 40 min	Breach time 50 min	Breach time 60 min
2*HD	77813	70371	63166	56621	50348	**44475**
3*HD	101289	87724	74433	62437	53432	46725
4*HD	115268	96314	78488	65304	55422	48461
5*HD	**123452**	100943	81720	65627	57159	49838

Table 13 Percentage Increase in Peak Discharge from the Peak Discharge of 2*HD of same breach time.

Breach width / Breach time	3*HD	4*HD	5*HD
10 min	30	48	58
20 min	24	36	43
30 min	17	24	29
40 min	10	15	19
50 min	6	10	13
60 min	5	9	12

Analysis for Setup 25 (BT 10 minute & BW 5*HD)

The maximum discharge of 123452 m3/ s was flow out, when the breach width was fully developed i.e. at breach time 10 min. The arrival time of flood at Chainage 20000 m is 25 min and peak flood reaches at 20000 m chainage in just 28 minutes. So we can imagine that disaster caused by this type of flood is so large than the other breach conditions. Fig. 29, Fig 30 and Fig 31 shows the peak discharge, peak water level and flood map for the Lower Nagavali River.

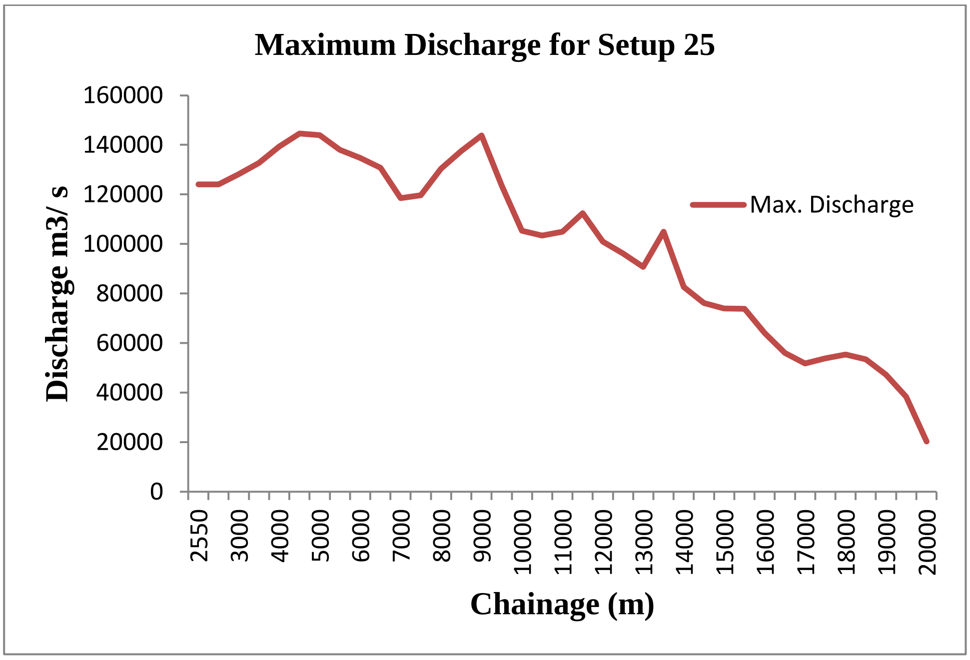

Fig. 6.29 Maximum Discharge for Setup 25 at different Chainages

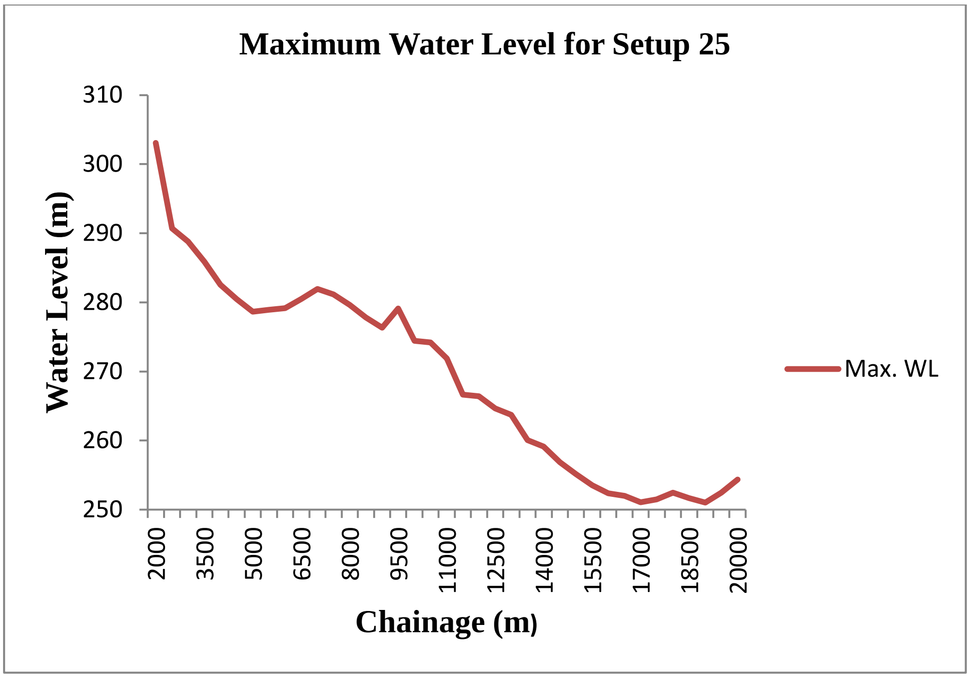

Fig. 6.30 Maximum Water Level for Setup 25 at different Chainages

Flood Map for Setup 25

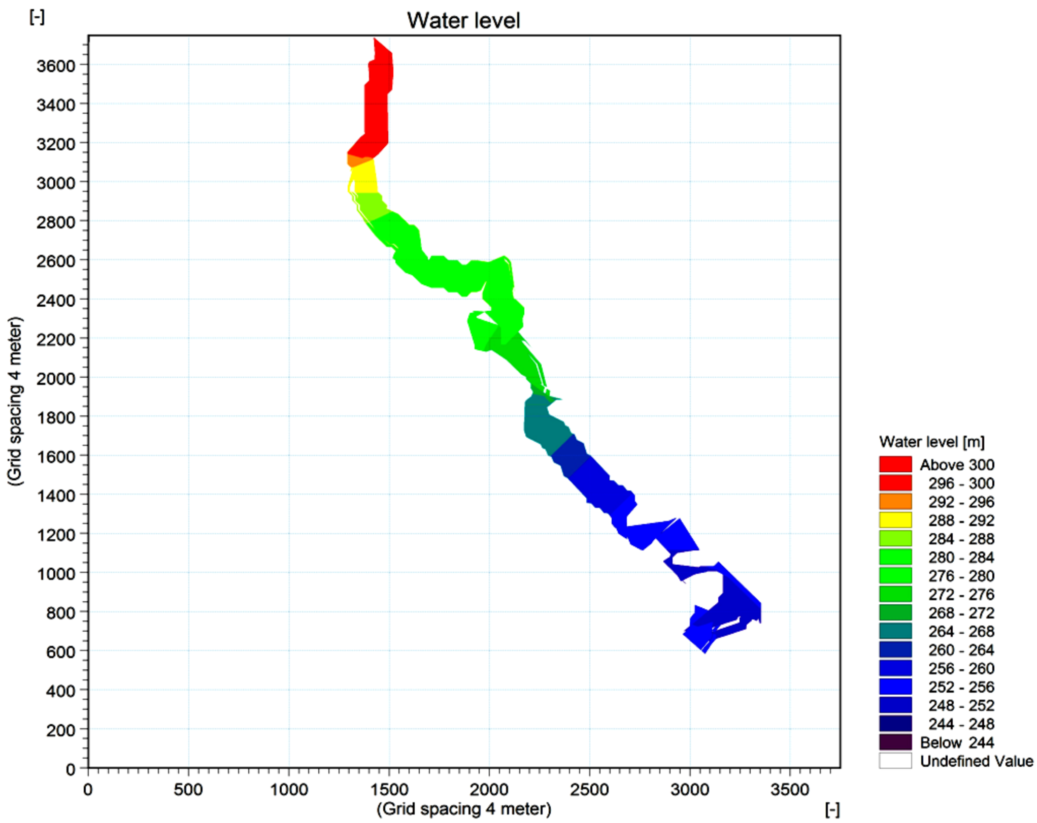

Fig. 6.31 Flood Map for Setup 25 of Lower Nagavali river

SECTION B: Rukura Dam

In Rukura Dam Break study the initial water level of reservoir is assumed to be at dead storage level and it has been assumed that spillway gates are closed while PMF comes so that reservoir gets completely fills and overtop the dam. These assumptions are made so that we can study the dam failure flood flow consequences to the downstream region and how much peak flood water has enter into the Brahmini River. Dam will take around 15.38 hrs to fill it up to the water level of 189 m and at time 15.417 hrs dam will start breaching and continue till 50 minutes. The maximum discharge of 17740 m^3/ s is released from the breached Rukura dam which is around 11.5 times greater than the Standard Probable flood. Dam Breach statistics are shown in Table 14

6. B.1 Routing of Flood Hydrograph and Water Level

Routing of flood hydrograph is analysed at the three Chainage points 4600 m, 6100 m, and 9150 m of the river network. Fig 6.32 shows the flood hydrographs at selected Chainage points. At the dam site the peak discharge of 17740 m^3/ s is flows out in 50 minute from the starting time of dam break. At Chainage 4600 m the peak flood discharge is about 16658 m^3/ s and the arrival time of peak flood is 52 minute from the start of dam break flood. Dam break flood reaches 6100 m in 17 minutes and the peak discharge of about 16232 m^3/ s takes 41 minute from the arrival time of the flood. It means that total time of 58 minutes is taken by flood to flow with its full capacity. Now, for Chainage 9150 m the arrival time of flood is 42 minutes and peak discharge of 15983 m^3/ s will start flowing in 24 min from the arrival time of flood. The total of 66 min is taken by peak flood to flow over this region from the time of start of dam break. It has been concluded that about 15983 m^3/ s is being discharge into the Brahmini River which may further cause disaster to the downstream valley. Table 13 shows the value of maximum discharge, maximum water level, and maximum velocity with their arrival times. Fig.6.33 shows the water level hydrograph and Fig.6.34, Fig.6.35, Fig.6.36, shows the cross section of Chainage 4600 m, 6100 m, and 9100 m of the river with maximum and minimum water level.

Table 14: Dam Breach Statistics for Rukura River

Time h	Q in Breach m3/s	V in Breach m/s	Reserv. Water Level m	Level of Breach m	Depth in Breach m	Breach Bottom Width m	Breach Width at crest m
15.417	0	0.266	189	189	0.003	0.1	0.1
15.458	11.7	1.815	189.03	187.79	1.235	3.995	6.405
15.483	39.5	2.322	189.04	187.01	2.034	6.332	10.308
15.517	111.2	2.861	189.06	185.968	3.099	9.448	15.512
15.533	164.4	3.095	189.07	185.446	3.63	11.006	18.114
15.567	310.3	3.516	189.09	184.402	4.691	14.122	23.318
15.592	457.3	3.8	189.10	183.619	5.485	16.459	27.221
15.633	781.3	4.23	189.11	182.314	6.802	20.354	33.726
15.642	858.6	4.311	189.11	182.053	7.064	21.133	35.027
15.683	1311.5	4.691	189.11	180.748	8.371	25.028	41.532
15.708	1638.2	4.904	189.11	179.965	9.149	27.365	45.435
15.742	2140.8	5.172	189.09	178.921	10.18	30.481	50.639
15.758	2421.5	5.3	189.08	178.399	10.692	32.039	53.241
15.792	3042.8	5.546	189.05	177.355	11.707	35.155	58.445
15.817	3561.9	5.722	189.02	176.572	12.462	37.492	62.348
15.858	4528.9	6	188.96	175.267	13.702	41.387	68.853
15.867	4737.5	6.053	188.94	175.006	13.947	42.166	70.154
15.908	5856.6	6.311	188.84	173.701	15.159	46.061	76.659
15.933	6588	6.458	188.778	172.918	15.872	48.398	80.562
15.967	7631.4	6.646	188.66	171.874	16.807	51.514	85.766
15.983	8181.6	6.737	188.60	171.352	17.266	53.072	88.368
16.017	9337	6.911	188.45	170.308	18.169	56.188	93.572
16.042	10249.8	7.037	188.33	169.525	18.83	58.525	97.475
16.083	11853.3	7.236	188.09	168.22	19.9	62.42	103.98
16.092	12185.6	7.274	188.04	167.959	20.108	63.199	105.281
16.133	13899.9	7.458	187.74	166.654	21.125	67.094	111.786
16.158	14966.9	7.563	187.55	165.871	21.713	69.431	115.689
16.192	16427.9	7.696	187.25	164.827	22.467	72.547	120.893
16.208	17172.5	7.759	187.09	164.305	22.831	74.105	123.495
16.233	17677	7.777	186.84	163.522	23.362	76.442	127.398
16.25	17739.5	7.756	186.66	163	23.711	78	130
16.258	17624.9	7.742	186.58	163	23.624	78	130

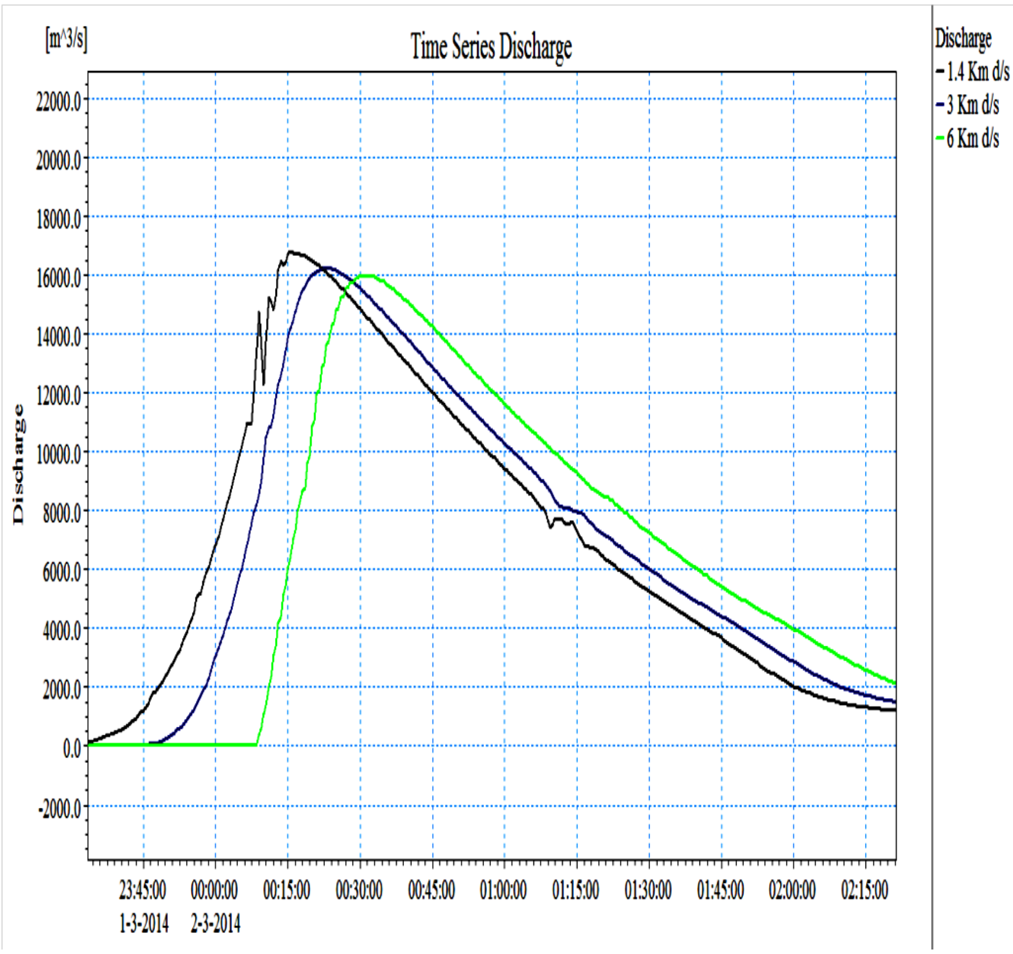

Fig.6.32: Flood Hydrograph for Rukura Dam Break

Table 15: Max. Discharge, Max. Water Level, Max. Velocity and their time of occurrence at selected locations of Rukura River

Chainage (m)	Max. Q m^3/s	Time for Max. Q (min)	Max. W.L (m)	Time for Max W.L (min)	Max. V (m/s)	Time for Max. V (min)
4600	16658	52	171.8	57	2.14	31
6100	16232	58	166.9	58	5.33	57
9150	15983	66	152.9	66	6.77	66

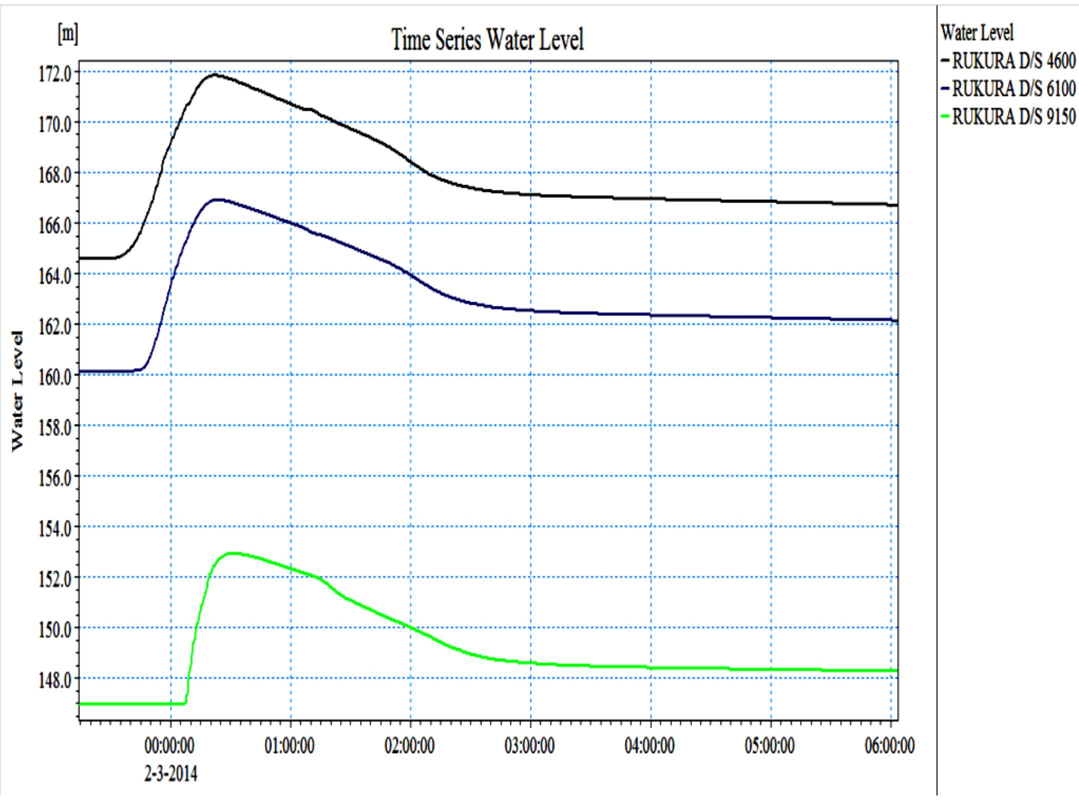

Fig.6.33: water level at 4600m, 6100m and 9150m d/s from Rukura Dam

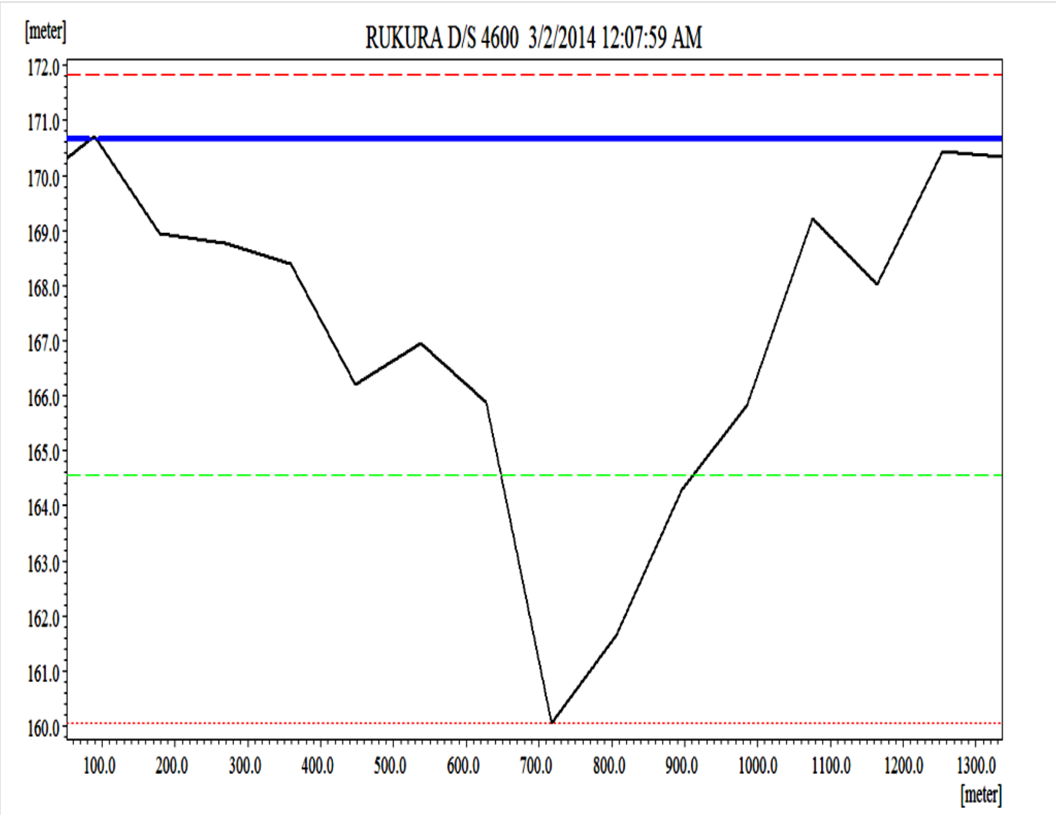

Fig.6.34 River Cross section at 4600mt d/s from the Rukura Dam axis

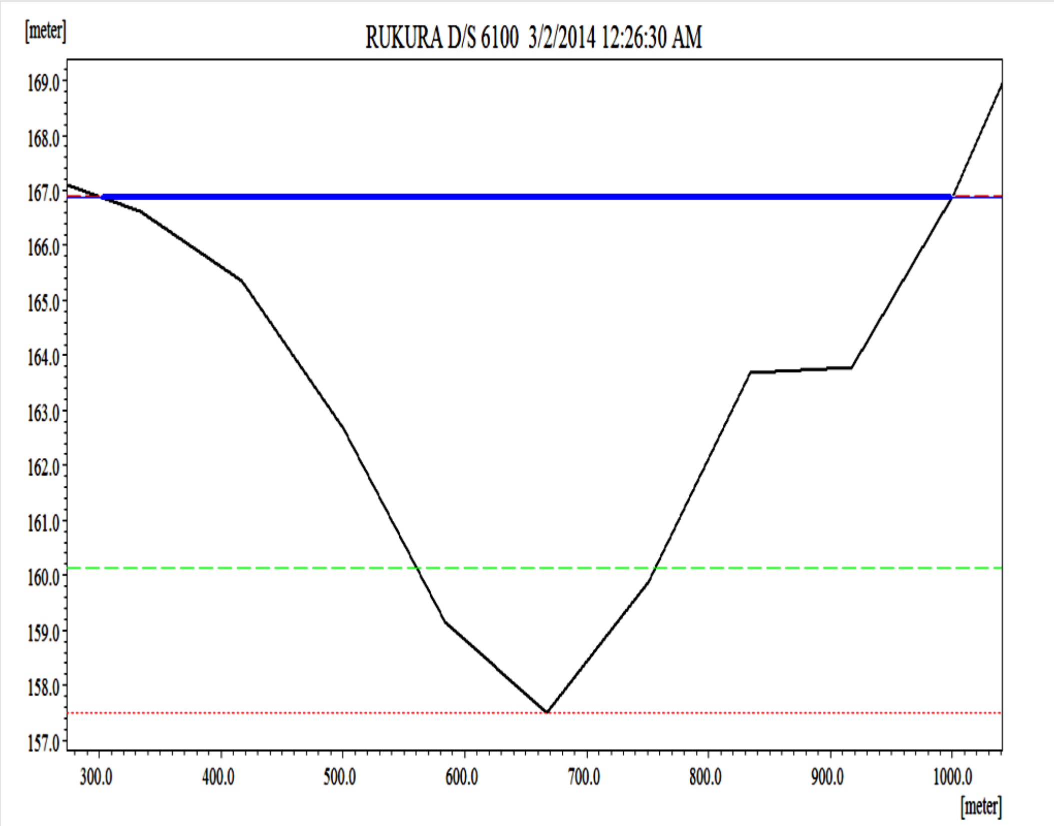

Fig.6.35 River Cross section at 6100mt d/s from the Rukura Dam axis

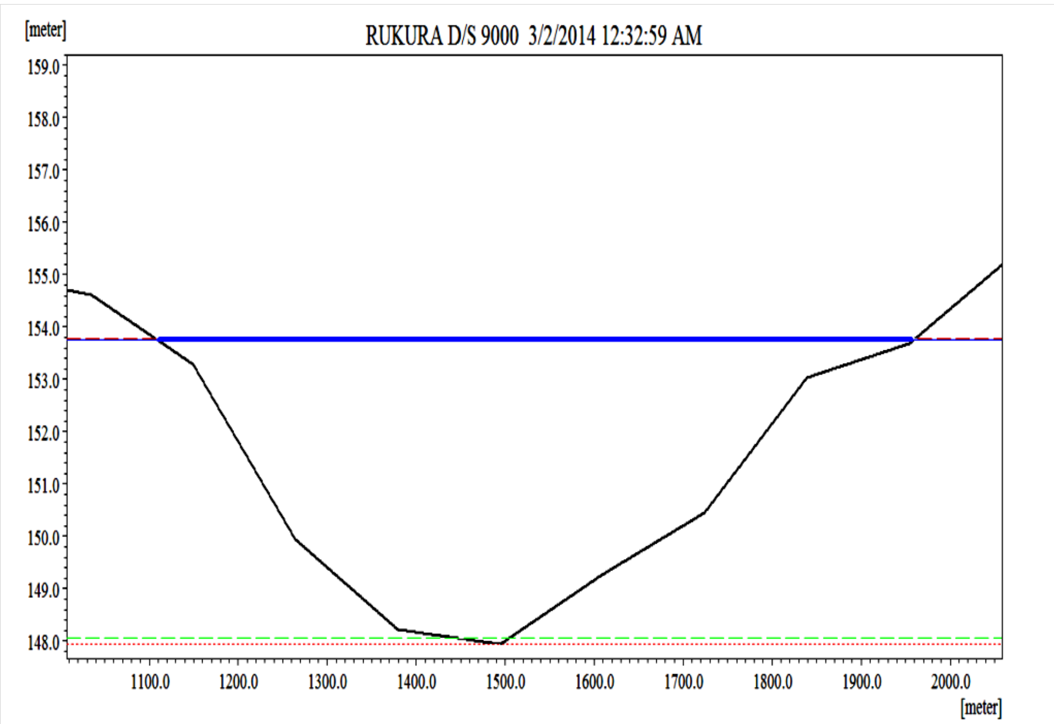

Fig.6.36 River Cross section at 9000mt d/s from the Rukura Dam axis

Rukura Flood Map

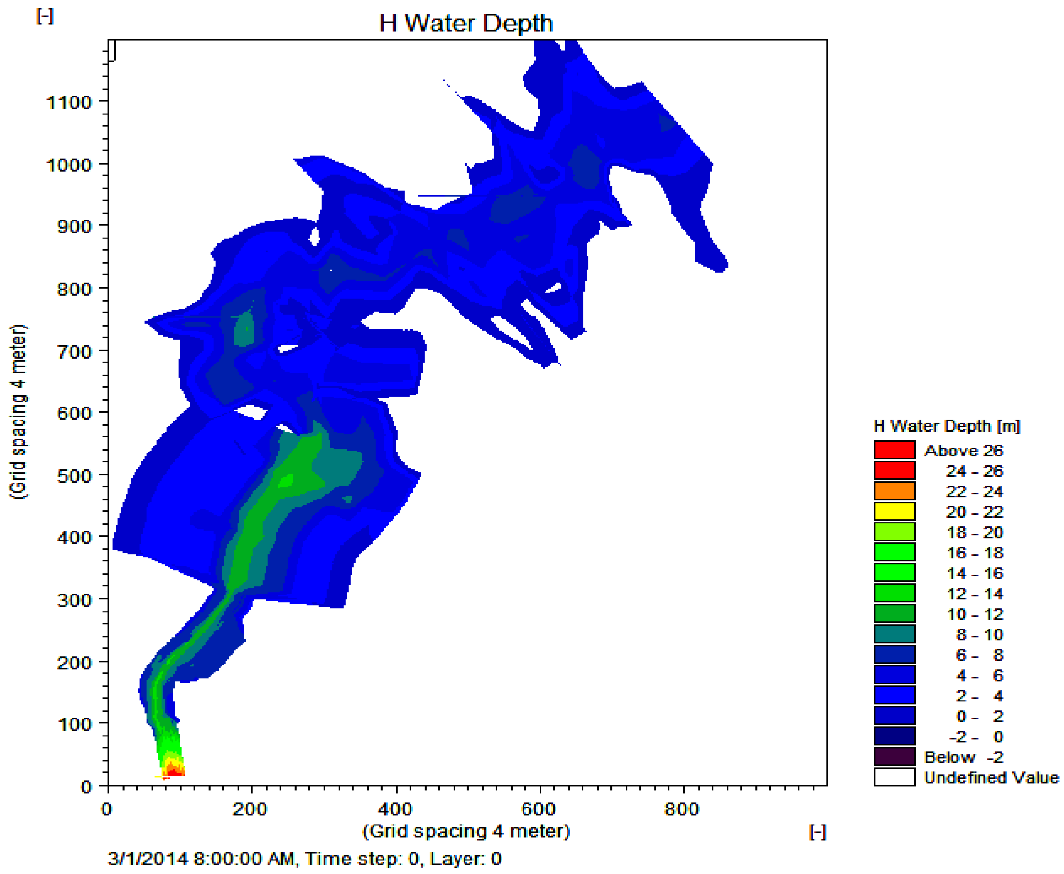

Fig.6.37 Flood Map of Rukura river after Dam Break

6. B.2 Dam Break Scenarios

a) Analysis for Breach Time 10 minute and Breach Width 130 m

The peak of the dam break flood is 30494 m^3/ s and the peak discharge of the standard probable maximum flood is 1543 m^3/ s. Thus the former is about 19.7 times of the later. When dam breaks in critical condition (i.e expected breach width, breach time is maximum and breach slope is 1H : 1V) then at dam location the maximum velocity achieved is 8.141 m/s which is quite high. At Ch. point 3575 m the approximate rise in water level of 10 m is being noticed. At Ch.6200 m rise in water level comes down to 8 m. The flooded water enters the Brahmini River with 5 m rise in water level. Fig 6.39 shows the water level increase with time at four selected locations. Ch. 5500 m to Ch. 7000 m flooded water enters into the flood plain on one side as other side elevation is high enough that flooded water will not enter into the flood plain. Fig.6.38 shows the flood hydrograph for three d/s Chainages.

b) Analysis for Breach Width 78 m and Breach Time 10 minute

The peak of the dam break flood is 20263 m^3/ s and the peak discharge of the probable maximum flood is 1543 m^3/s. Thus the former is about 13 times of the later. When dam breaks then at dam location the maximum velocity achieved is 8.04 m/s which are quite high covering a 6 km d/s it will take 12.5 min. in ideal conditions. Fig. 6.40 and Fig.6.41 shows the flood hydrograph and water level hydrograph for selected locations of Rukura River.

If we compare the above two scenarios, reservoir water level fall down quickly in 1st scenario as in less time the maximum breach width is developed so, more water is released from break dam causing more flood and high water level in the downstream valley.

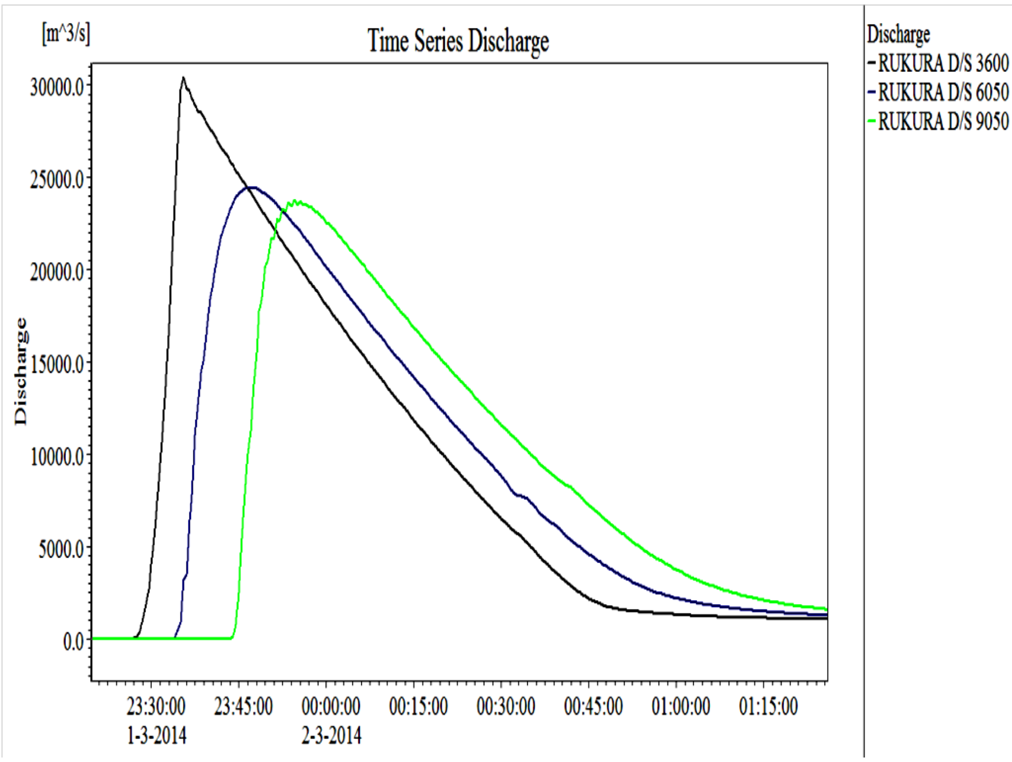

Fig.6.38 Rukura DB Flood Hydrograph for Critical Breach Condition at 3 Locations

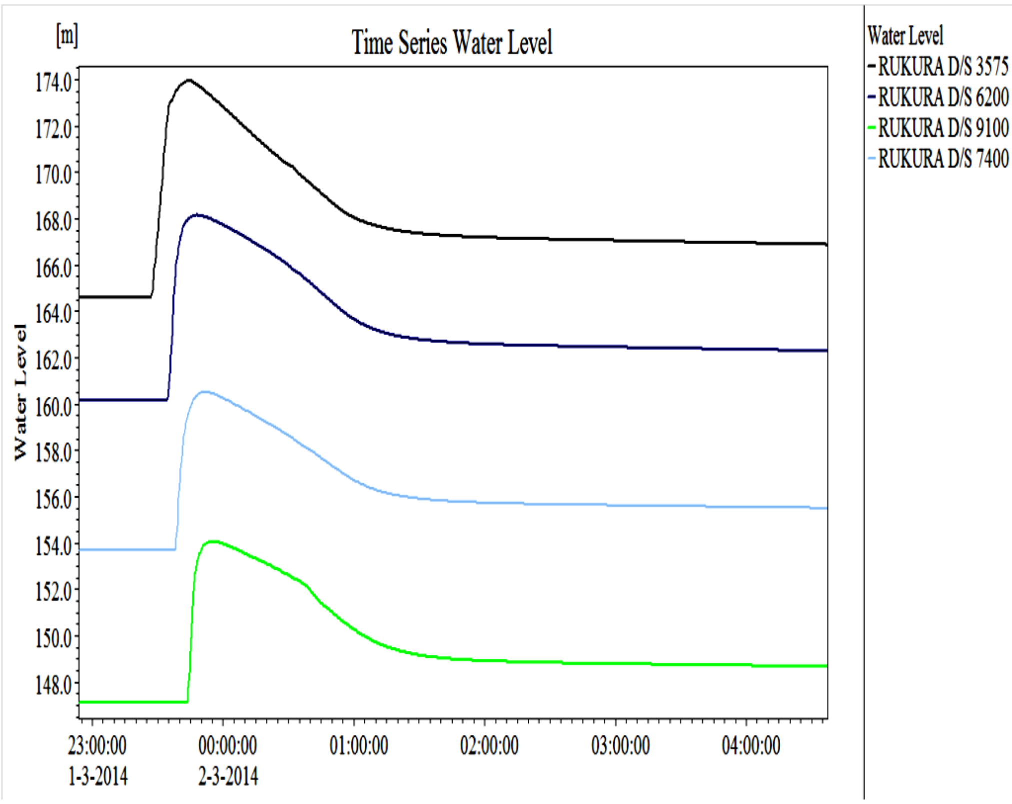

Fig.6.39 WL Hydrograph for Critical Breach Condition at Four d/s Locations

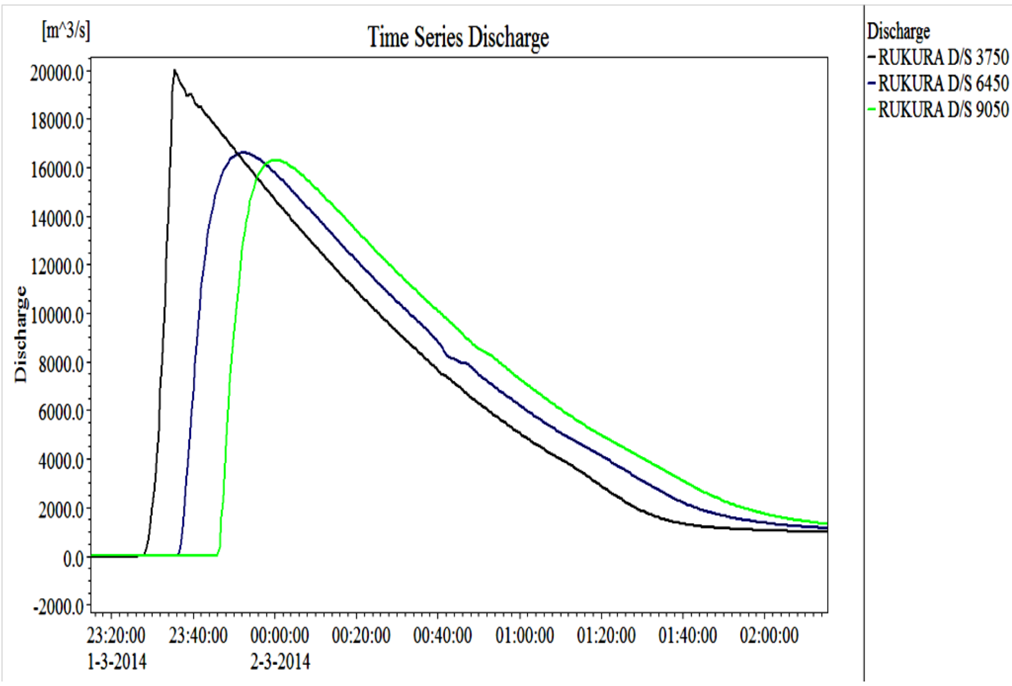

Fig.6.40 DB Flood Hydrograph for BW 78 m and BT 10 min at selected Locations of Rukura

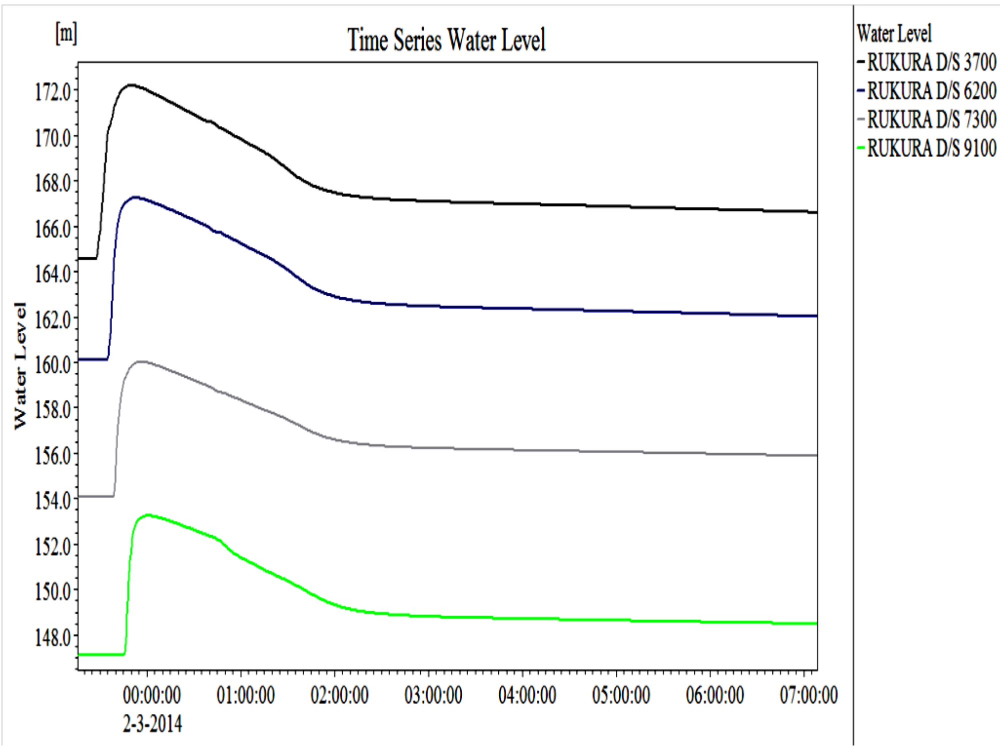

Fig.6.41 WL Hydrograph for BW 78 m and BT 10 min. at selected Location of Rukura

6. B.3 Sensitivity analysis for various inputs to the model setup in terms of peak discharge and Water levels

a) Effect of Breach Time on Discharge and Water Level

For breach time 10 min the maximum discharge coming out from the breached dam is 20263 m^3/ s and for breach time 50 min it is 17739 m^3/ s. So with the decrease in breach time the maximum discharge value is increases and for Rukura dam it is 14.2%. Further the Table 14 shows the breached dam data. Fig 6.42 shows the flood hydrograph coming out from breached dam and Fig 6.43 shows the variation of maximum discharge for different breach time having constant breach width of 78 m at different Chainages of Rukura river network. In Rukura dam break model when sensitivity of breach time on water level is examined then we found that water levels for different downstream locations have not much difference in their peak values. The reason behind this is that discharge coming out from the breached dam has not much difference in their peak values. Fig. 6.44 shows the result for different breach time in terms of water level for whole river network.

Table 16: Sensitivity of Breach Time on Max. Discharge, Max. Velocity and Reservoir WL

Breach Time (min.)	Max. Discharge (m^3/s)	Max. Velocity (m/s)	Reservoir WL (m)
10	20263	8.04	188.5
20	19611	7.97	188.05
30	18971	7.91	187.58
40	18348	7.8	187.12
50	17739	7.79	186.66
60	17143	7.72	186.21

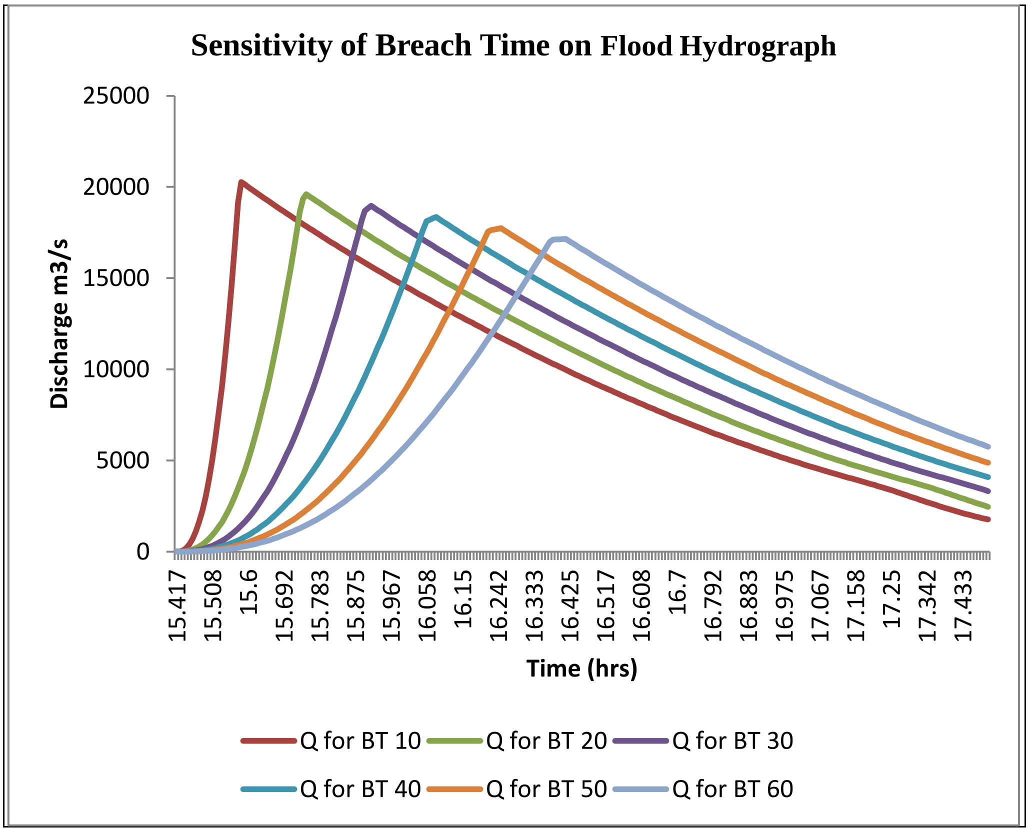

Fig. 6.42 Sensitivity of BT on Flood Hydrograph of Rukura breached dam

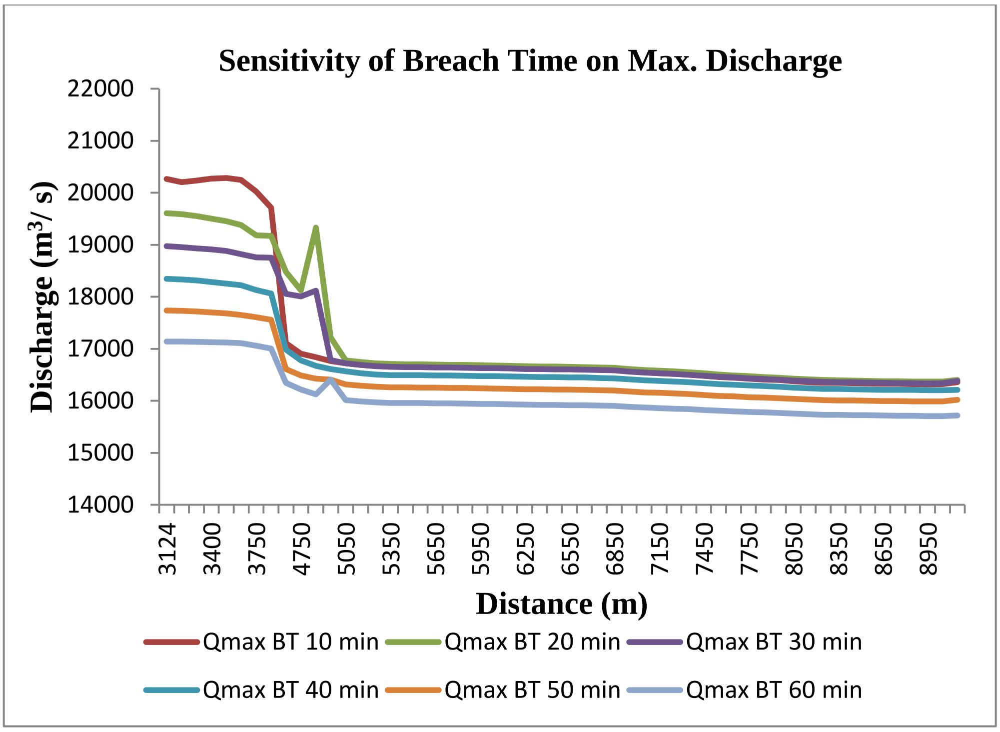

Fig. 6.43 Sensitivity of BT on Max. Discharge for Rukura River Chainages

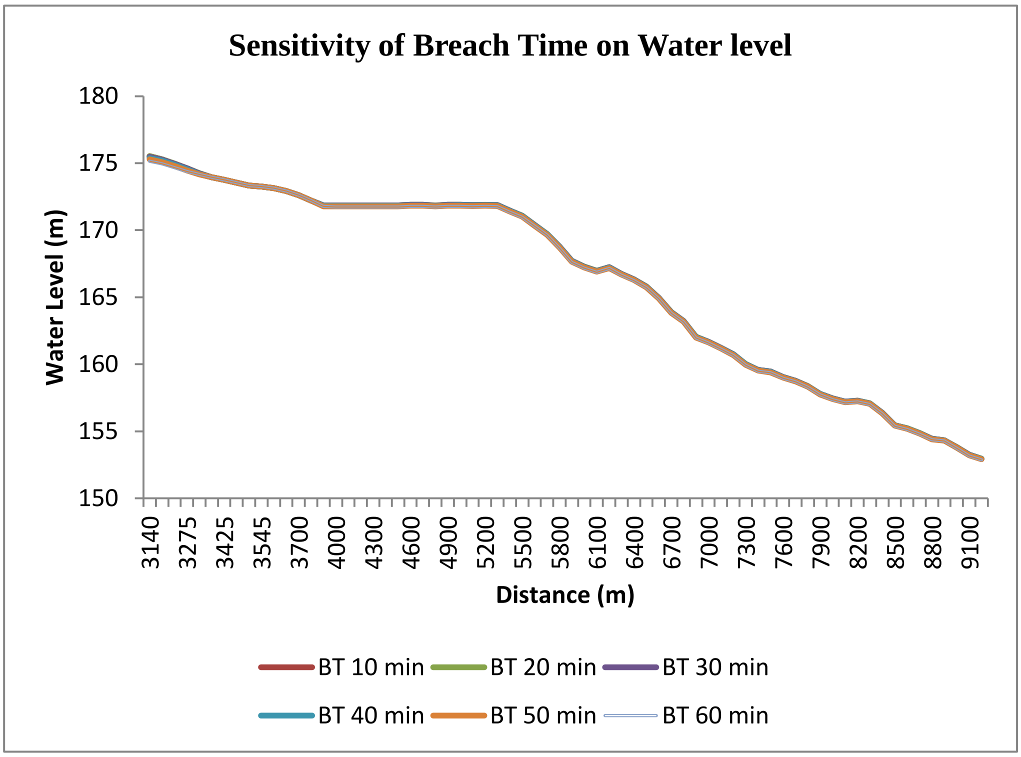

Fig. 6.44 Sensitivity of Breach Time on Water Level of Rukura dam break model

b) Effect of Breach Width on Discharge and Water Level

As we see in Fig 6.45 with the increase in breach width the peak of discharge increases. For breach width 3*HD the peak discharge is 17739 m^3/s and when breach width is increased to 5*HD the peak discharge of 24694 m3/s is being noticed. It means about 39% of increase in discharge when the breach width is increased from 78 m to 130 m. similarly when breach width is decreased to 2*HD than about 23 % decrease in Peak discharge is being noticed. So, there is great impact on dam break flood hydrograph with the change of breach width during the dam failure. There is very slight increase in maximum water level has been noticed with the increase in breach width. Fig 6.46 shows the sensitivity of breach width on maximum water level at every Chainage point of Rukura River.

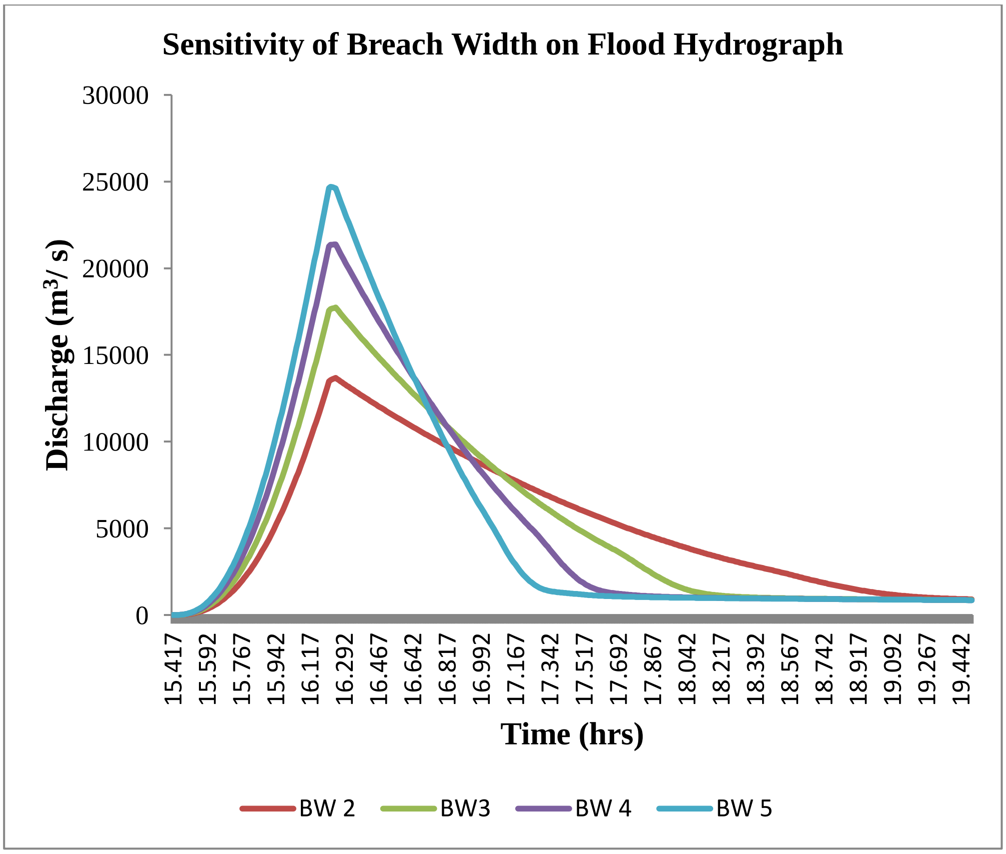

Fig. 6.45 Sensitivity of BW on Flood Hydrograph for Rukura Dam

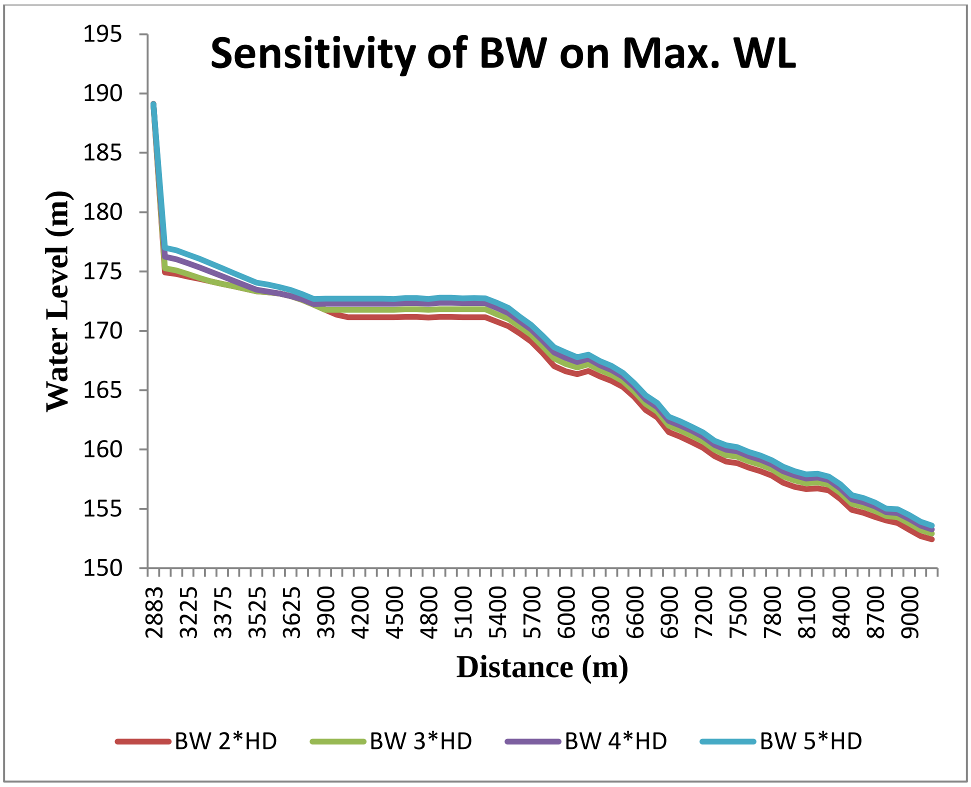

Fig. 6.46 Sensitivity of BW on Max. Water Level of Rukura River

c) Effect of Inflow on Discharge

Sensitivity of inflow is examined by increasing it with 150% and 200%. As we see in Fig. 6.47 there is shift in flood hydrograph of dam break as when the inflow is increases. When inflow is increased it means dam reservoir will fill earlier than the actual time so according to our assumptions dam will break much earlier when the inflow is increased by 150% and 200%. This implies that evacuation time for human life is also short when high inflow is being noticed than the actual inflow. Further when Inflow to the reservoir is increased the peak discharge is also increased which is clearly shown in Fig. 6.48.

d) Effect of Manning's N on Discharge

.As expected that the velocities reduces with the increase in Manning's N, and vice versa. This will affect the maximum water level and discharge value also. Fig. 6.49 shows the sensitivity of manning's N on discharge values of Ch. 9125 m of the Rukura River.

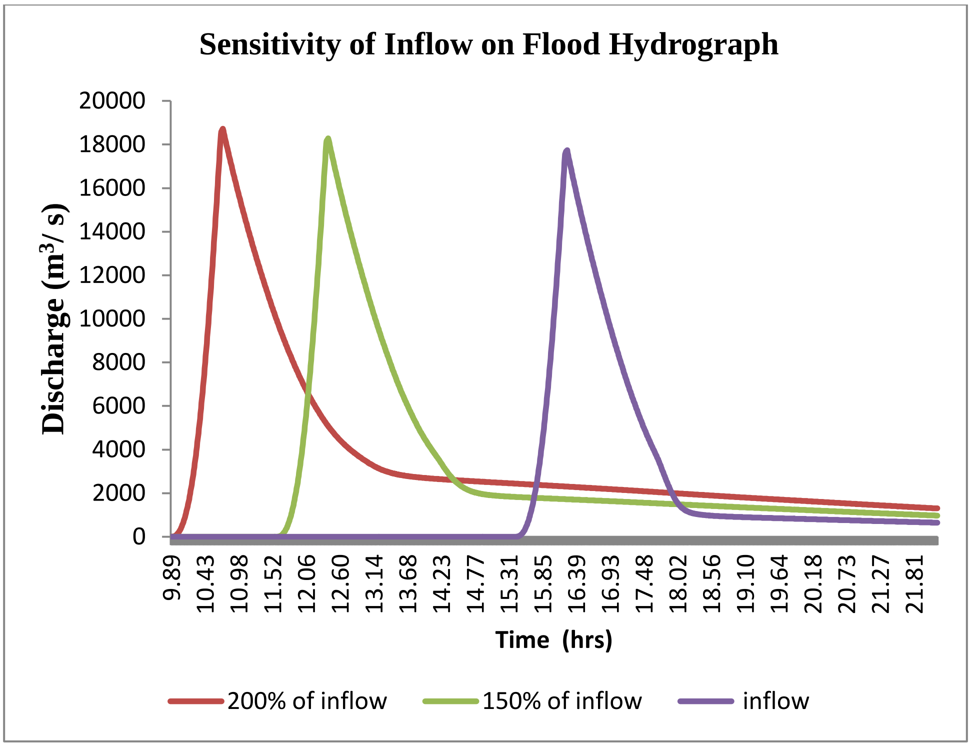

Fig. 6.47 Sensitivity of Inflow on Flood Hydrograph for Rukura Dam

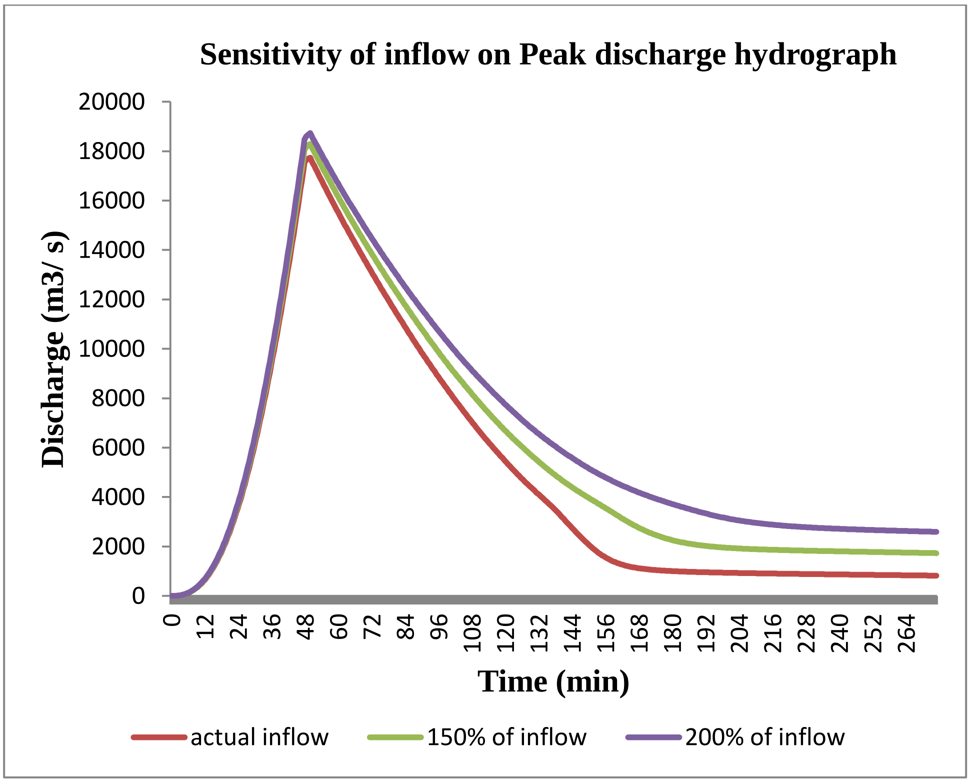

Fig. 6.48 Sensitivity of Inflow on Peak discharge of Rukura Dam Break

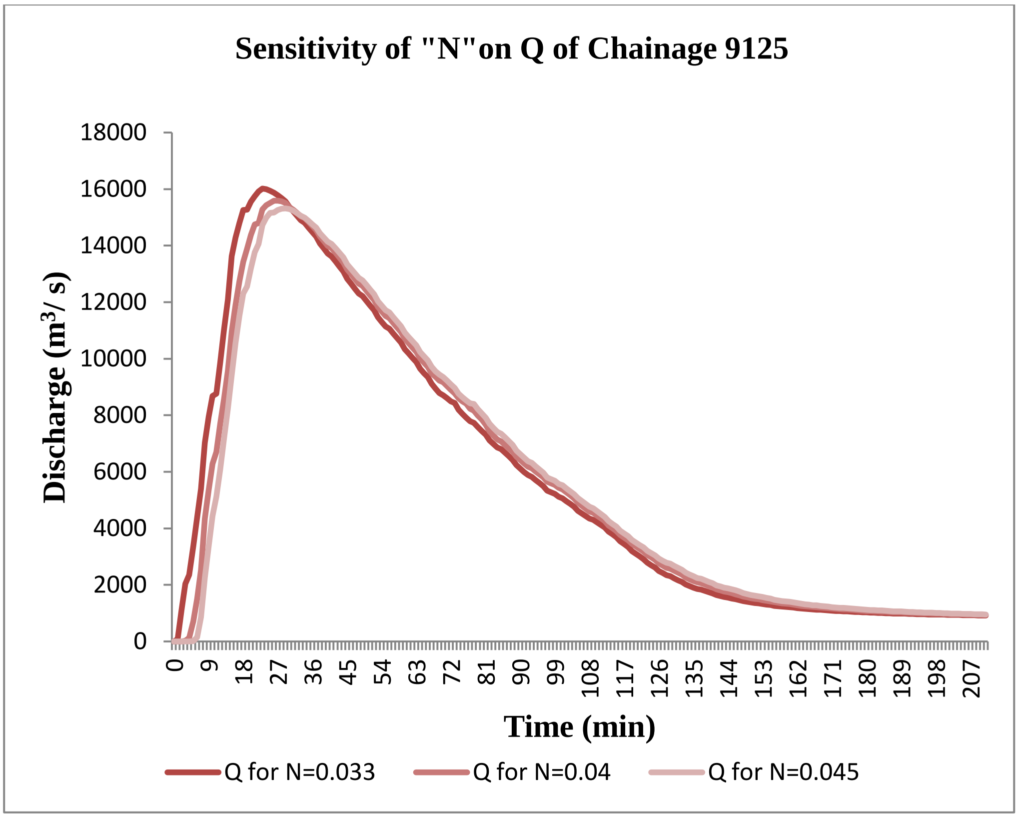

Fig. 6.49 Sensitivity of "N" on Flood Hydrograph for Ch. 9125 m of Rukura River

CONCLUSIONS AND RECOMMENDATIONS

In this thesis the simulation of hypothetical failure of "Lower Nagavali dam and Rukura Dam" is carried out, both the dams are earthfill dam having height of 51 m and 26 m respectively. The impact of Dam Break in the downstream area is observed in terms of flood hydrograph, flood duration, water level, velocity and flood map. Further the sensitivity analysis of Breach Time, Breach Width, Manning's Roughness and Inflow to the reservoir is carried out. As dam geometry, reservoir capacity and environmental conditions are different therefore the results obtained for both Dam Break Models is different. So, conclusions are drawn by comparing their results as written bellow.

- In case of Lower Nagavali the Peak discharge is 53334 m^3/ s which are 5.8 times greater than the probable maximum flood and for Rukura Dam Break the peak discharge is 17740 m^3/ s which is 11.5 times greater than the Standard Probable Flood.

- We observe huge difference in the peak discharge values as both the dams have almost same storage capacity. The reason behind that is there dam geometry is different, mostly height and length of dam plays the crucial role in the development of peak outflow. With few assumptions we can conclude that dam having more height will develop high peak outflow compare to low height dams of almost same reservoir storage capacity.

- As from the sensitivity analysis of both the dams we conclude that effect of breach time on discharge is much more pronounced than the water level.

- Effect of breach width on Lower Nagavali dam result is less pronounced than the results of Rukura Dam. For Rukura Dam Break with the increase in Breach Width from 3*HD to 5*HD than about 39% of increase in discharge and in Case of Lower Nagavali Dam Break the increase of 7.1% is noticed. The reason behind this is there dam crest length and longitudinal span of water storage in the reservoir. So, we conclude that effect of breach width on discharge is more in case of long dams compare to short length dams.

- Sensitivity of Manning's Roughness is less pronounced in case of Rukura Dam Break model as because the length of Rukura River examined here is less but in case of Lower Nagavali River the effect of Manning's Roughness on discharge and water level of downstream locations was more.

- The peak discharge of 15983 m^3/ s was added in the Brahmini River after the failure of Rukura Dam.

- Our Dam Break modelling results can be used as flood hazard maps and can assist communities in planning future developments in areas that are prone to flooding.

- For obtaining best results the accuracy of data is of very much important. So, with the data obtained from tool available in remote sensing (DEM) few surveyed data are required to get the real time condition for dam break analysis.

- Further the flood propagation scenarios depend on the roughness coefficient used.

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