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DAM BREAK ANALYSIS USING MIKE11 FOR LOWER NAGAVALI DAM AND RUKURADAM

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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.

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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 breachwidthhasbeendevelopedontheleftbank.KailaDam,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 Euroofdamage.InSpain1997,failure of adamon theGuadalquivir 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 failureevent.

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.

3. METHODOLOGY

3.1 DamStructure

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 FailureMoment

Four ways of failure are described in Mike 11.
A given number of hours after start of thesimulation.

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- At a specified time(year, month, day, hour,minute)
- OvertoppingFailure
- At a specified reservoirlevel

3.3 FailureMode

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

- InstantaneousFailure
- Linear Failure i.e. the increase in breach dimension is assumed to occur linearly over a given time(the time of breachdevelopment)
- 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 BreachFormulation

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 breachmodeling:

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

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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.

Trapezoidal BreachGeometry:

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 breachflow.

3.4.2 NWS DAMBRK dam-breachmethod:

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 typeequation

Piping failure

The flow through a piping failure is given by

$Q = C_{orifice} A\ p$

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

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3.4.3 Erosion Based Breach Development using the energyequation

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 thesediment

$L_b$is the breach length in the direction of flow t is time

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.

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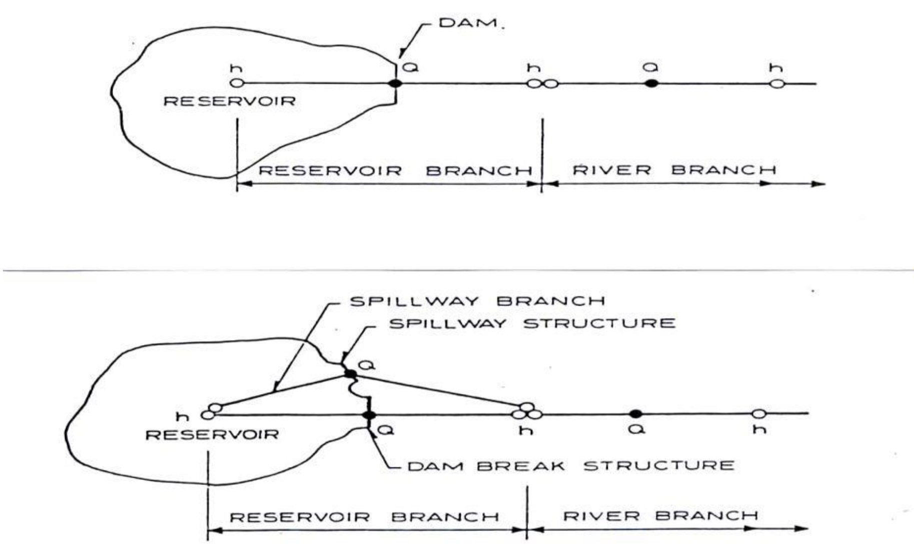

Fig. 4.1: Arrangement of Dam Strucure with Spillway in Mike-11

4.2 Model setup for Lower NagavaliDam

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.

4.1.1 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.

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4.2.3 Upstream BoundaryCondition

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 BoundaryCondition

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 forDownstream

4.3 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 to0.05).

4.4 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 Guidelinesand 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

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and failure time in hours is in between 0.1 to 1.0hours

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) xHD (2.0 to 5.0) xHD	0.5 to4.0 0.1 to1.0 0.1 to1.0	0 to1.0 0 to1.0 0 to1.0	USACE (2007) FERC (1988) NWS(Fread, 2006)
Concrete Gravity	Multiple Monoliths Usually ≤ 0.5 L Usually ≤ 0.5 L	0.1 to0.5 0.1 to0.3 0.1 to0.2	Vertical Vertical Vertical	USACE (2007) FERC NWS (Fread, 2006

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5. 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 waterlevel.

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5.1 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 timeof

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

5.2 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 breacheddamandinabout47min.thepeakfloodisarrivedinthisregion.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 down stream of the dam thenweseethearrivaltimeofdambreakfloodin 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.

5.3 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

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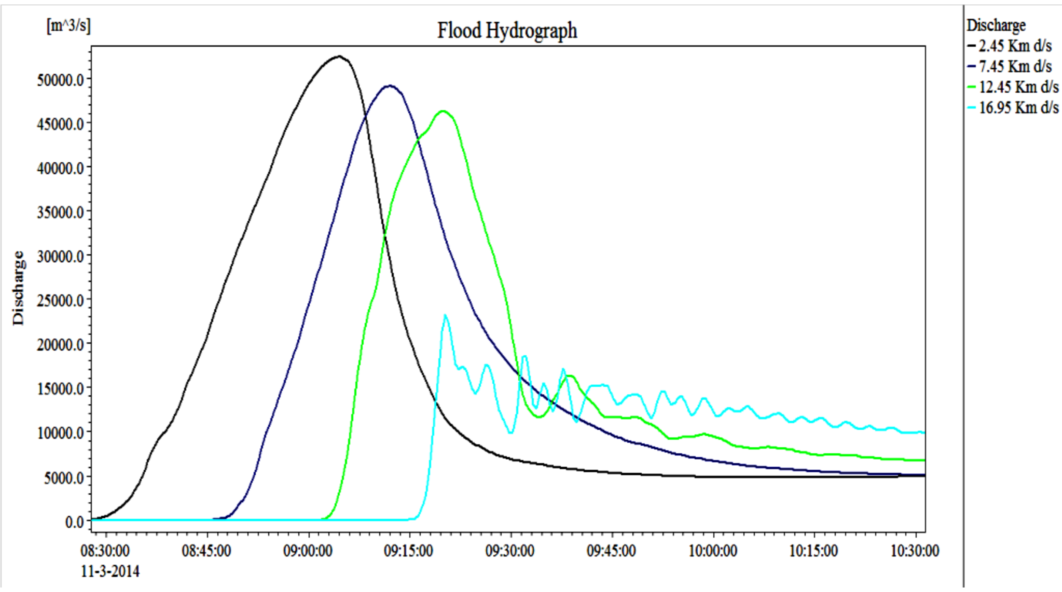

Fig. 5.1: Flood Hydrographs for 2.45 Km, 7.45 Km, 12.45 Km and 16.95 Km d/s of the Lower Nagavali dam

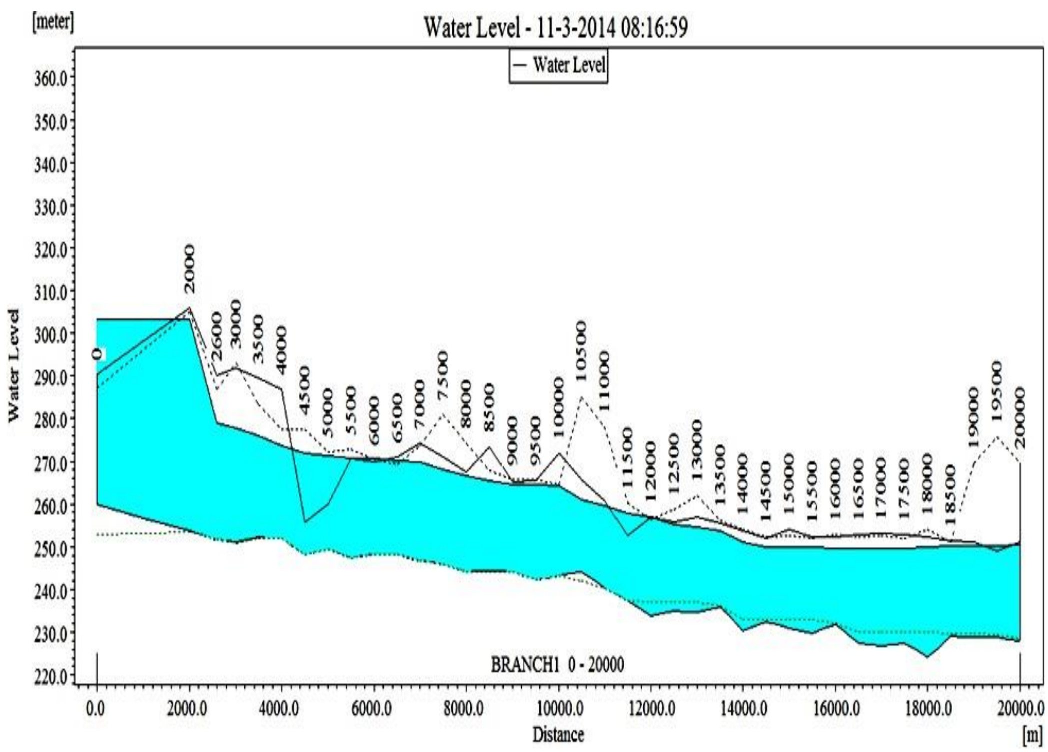

Fig.5.2: Longitudinal bed profile of Nagavali River showing maximum water levels

6. 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 whichare

  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 storagecapacity.

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

- 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 Breaktheincreaseof7.1%isnoticed.There as on behind his 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 lengthdams.

- 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 wasmore.

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

- RukuraDam.

- 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 coefficientused.

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