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DAM BREAK STUDY OF MYNTDU LESKA DAM USING DAMBRK MODEL

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PREFACE

The safety of downstream area of dam against its possible failure is one of the most important aspects to be considered during the planning, design, construction and operation of the dam. Flood wave due to a failure always assumes large magnitudes and inundates large area in the downstream portion. To estimate the amount of flood discharge reaching at different sections along downstream channel, its elevation and travel time, a large number of mathematical models have been developed in the past. The model named 'DAMBRK' developed U. S. National Weather Service is the most accurate with various levels of data availability and less time consuming on high-tech computers,

This report presents a study of hypothetical failure of Myntdu Leska dam in southern part of the Meghalaya near the international Indo-Bangladesh border. The maximum water level attained by the dam break flood in the river downstream of dam has been calculated and ascertained for extent of submergence of the area. Also the dam break flood wave characteristics at four different sections downstream of dam is presented for different failure parameters, namely; breach width, breach time and inflow. The sensitivity studies of the various parameters have also been done to know their effect over the movement of dam break flood wave.

This report titled **"Dam Break Study of Myntdu Leska Dam Using DAMBRK Model"** is part of the work program of North Eastern Regional Centre of National Institute of Hydrology. The study has been carried out by Sh. Pankaj Mani, Scientist – B.

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ABSTRACT

The safety of downstream area of dam against its possible failure is one of the most important aspects to be considered during the planning, design, construction and operation of the dam. Flood wave due to a failure always assumes large magnitudes and inundates large area in the downstream portion.

The DAMBRK model developed by U. S. National Weather Services (NWS) attempts to represent the current state-of-art in understanding of dam failures and the utilization of hydrodynamic theory to predict the dam break wave formation and downstream progression. The model has wide applicability; it can function with various levels of input data ranging from rough estimates to complete data specification, the required data is readily accessible and it is economically feasible to use, i.e. it requires a minimal computation effort on large computing facilities.

This report presents a study of hypothetical failure of Myntdu Leska dam in southern part of the Meghalaya near the international Indo-Bangladesh border. The maximum water level attained by the dam break flood in the river downstream of dam has been calculated and ascertained for extent of submergence of the area. Also the dam break flood wave characteristics at four different sections downstream of dam is presented for different failure characteristics. The sensitivity studies of the various parameters have also been done to know their effect over the movement of dam break flood wave.

It has been concluded after the study that the spillway capacity is sufficient enough to pass the PMF even when the reservoir is at FRL and therefore the dam will not fail by overtopping. The dam may fail by breaching for which the study has been done. The reservoir storage is very low while the carrying capacity of the river is very high in general

1.0 INTRODUCTION

Dam failures are often caused by over topping of the dam due to inadequate spillway capacity during large inflow to the reservoir from heavy precipitation runoff. Dam failures may also be caused by seepage or piping through the dam or along internal conducts, slope embankment slides, earthquake damage and liquefaction of earthen dams from earthquakes and land slide generated waves in the reservoir. Usually the response time available for warning is much shorter than for precipitation-runoff-floods. The protection of the public from the consequences of dam failures has taken an increasing importance as population has concentrated in areas vulnerable to dam break disasters.

Occurrence of a series of dam failures has increasingly focussed attention of scientific workers on the need for developing generally applicable models and methods to evaluate flash floods due to dam failure and for routing them through downstream areas, susceptible to heavy losses, so that potential hazards might be evaluated. Using these methods inundated areas, flow depths and flow velocities can be estimated for different hypothetical dam failure situations. With the help of such studies it could be possible to issue warnings to the downstream public and prepare strategies for disaster management when there is a failure of dam. The main difficulty in using such mathematical models is the failure description adopted in the model. Under these circumstances, a suitable assumption with regard to the adjustment of actual failure mode to suit the model failure mode is necessary.

The DAMBRK model developed by U. S. National Weather Services (NWS) attempts to represent the current state-of-art in understanding of dam failures and the utilization of hydrodynamic theory to predict the dam break wave formation and its downstream progression. The model has wide applicability; it can function with various levels of input data ranging from rough estimates to complete data specification, the required data is readily accessible and it is economically feasible to use, i.e. it requires a minimal computation effort on large computing facilities.

The model consists of three functional parts. , *Viz.* (i) description of the dam failure mode, (ii) computation of the time history (hydrograph) of the outflow through the breach, and (iii) routing of the outflow hydrograph through the downstream valley. This determines the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations (stages) and flood wave travel time.

This report presents a study of hypothetical failure of Myntdu Leska dam in southern part of the Meghalaya near the international Indo-Bangladesh border. The dam break flood wave characteristics at four different sections downstream of dam is presented for different failure characteristics. The sensitivity studies of the various parameters have also been done to know their effect over the dam break flood wave movement.

2.0 REVIEW

The dam break modelling is an old problem in mathematical hydraulics and the concerned literature is extensive. The first solution was given in 1892 by Ritter, who used the method of characteristics to obtain a closed form solution for a dam of semi-infinite extent upon a horizontal bed with zero bed resistance. However, experimental and theoretical considerations showed that the solution is invalid in a region that starts near the leading edge of the flood wave and extends rapidly upstream with time, because of zero bed resistance assumption. In 1952, Dressler used a perturbation procedure to obtain a first order correction for resistance effect. Whitham obtained a second solution three years later by using a technique that was similar to the Pohlhansen method of boundary layer theory. Whitham's solution agreed with Dressler's results and he noted that his solution would not apply for large values of time since the width of the boundary layer grew very rapidly with time.

Afterwards, Sakkas and Strelkoff (1973), Chen and Armbruster (1980) have used the method of characteristics to obtain numerical solution for dam break problems on sloping beds. These solutions were for reservoirs of finite length and included the effects of bed resistance. But in almost all of these methods, it was assumed that the breach covers the entire dam and it occurs instantaneously. U.S. Army Corps of Engineers (1960) recognized the need to assume partial breaches, however, they assumed an instantaneous failure.

In 1965, Cristofano and in 1967, Harris and Wagner incorporated the partial time dependent breach formation in their models. Cheng Lung Chen (1980) developed a numerical model on the basis of an explicit scheme of the characteristic methods with specified time intervals. He also carried out some laboratory experiments for the verification of his model. Bruce Hunt (1982) used the kinematic approximation to obtain a simple, closed form solution for the failure of a dam on a dry, sloping channel. It was found that this solution becomes asymptotically valid after the flood wave has advanced about four reservoir lengths downstream. N. D. Katopodes and D. R. Schambar (1982) formulated five mathematical models based on equations ranging from the complete dynamic system to a simple normal depth kinematic wave equation. In 1984, they have presented a theory for flow through a partial dam failure. In this, the breach section is treated as an internal boundary condition that interrupts the continuous long wave occurring upstream and downstream of the dam.

The U.S. Army Corps of Engineers, HEC-1 dam break model (HEC-1, 1981) adopts storage routing techniques for routing of flood through reservoirs as well as through channels. National Weather Service (NWS) DAMBRK Model (1984) adopts dynamic routing techniques for routing of flood through channel and a choice of dynamic routing and storage routing for the reservoir, depending on the nature of flood wave movement in reservoir at the time of failure.

Singh and Snorrason (1984) carried out dam break flood studies using the above two models. They found that the flood stage profiles predicted by the NWS DAMBRK Model are smoother and more reasonable than those predicted by the HEC-1. For channels with relatively steep slopes, the methods compared fairly well, whereas for channels with mild slope, the HEC Model often predicted oscillatory, erratic flood stages, mainly due to its inability to route flood waves satisfactorily in non- prismatic channel.

Ralph A. Wurbs (1987) made a comparative evaluation of several dam break models. The models selected for comparison were : National Weather Service (NWS) Dam Break Flood Forecasting Model (DAMBRK); U.S. Army Corps of Engineers South-Western Division (SWD) Flow Simulation Models (FLOW SIM 1&2), U.S. Army Corps of Engineers Hydrologic Engineering Centre (HEC) Flood Hydrograph Package (HEC-1), Soil Conservation Service (SCS) Simplified Dam Breach Routing Procedure (TR66), NWS Simplified Dam break Flood Forecasting Models (SMPDBK), HEC dimensionless graphs procedure and the Military Hydrology Model (MILHY) developed by WES specially for military use. He concluded that a dynamic routing model should be used whenever a maximum practical level of accuracy is required and adequate man power, time and computer resources are available. According to him the NWS DAMBRK is the optimal choice of model for most practical applications. Some applications require the capability to perform an analysis as expeditiously as possible. The NWS SMPDBK is the optimal choice of model for these types of application.

DAMBRK model uses St.Venent's equations for routing dam break floods in channels. For reasons of simplicity, generality, wide applicability and uncertainty in the actual failure mechanism, this model allows the failure timing interval and terminal size and shape of breach as input. It gives the extent of and the time of occurrence of flooding in the downstream valley by routing the outflow hydrograph through the valley. The dynamic wave method based on the complete equations of unsteady flow is the appropriate technique to route the dam break flood hydrograph. Terzidis and Strelkoff (1970) have demonstrated the applicability of the St.Venant's equations to simulate abrupt waves such as the dam break wave.

Gundalach & Thomas (1977) analyzed the dam break flood from Teton dam using a generalized unsteady flow computer program to determine the water surface elevations resulting from various breach sizes and roughness values (n). They found that neither the size of breaches tested (30 to 40% of the size of dam) nor the rates of failures assumed were very significant in predicting peak elevation at dam axis but the calculated peak flood elevations near the dam were very sensitive to n-values. Sakkas (1980) envisaged the development of dimensionless graphs for quick estimation of dam breach flood wave characteristics. These graphs would be useful in case when either the communication system or computation facilities are not available at the time of dam breach flood wave formation. Singh and Snorrason (1984) studied the sensitivity of outflow peaks and flood stages to the dam breach parameters. They have taken an earthen dam for the study and found that the breach outflow peaks are affected significantly by the base width of breach but less so by the water level in the reservoir at the time of breach formation. They also found that the ratio of outflow peak to inflow peak and the effect of time of failure on outflow decreases as the drainage area above the dam and impounded storage increases.

National Institute of Hydrology has also carried out many works related to dam break study since 1985-86. These includes preparation of data requirements for dam break models to many case study with actual and hypothetical dam failure data. The dam break studies conducted by National Institute of Hydrology so far are tabulated below:

Dam break analysis for Machu dam-II	CS	16
Application of NWS Dam break programme using data of Gandhi Sagar Dam	CS	49
Application of dam break programme MIKE 11 Machhu II dam and its comparison with NWS DAMBRK application results	CS	89
Dam break study of MITTI dam	CS/AR	126
Dam break analysis of Machhu dam -II failure using DAMBRK and SMPDBK models of NWS	CS/AR	133
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Dam break study of Barna dam	CS/AR	20/96-97
Development of dimensionless flood hydrographs from Machhu dam - II failure using dambrk model;	TR	34
Effect of downstream boundary conditions on the propagation characteristics of the Dam Break flood	TR/BR	117
Development of an empirical formula for approximate dam break flood estimation	TR/BR	1147

3.0 'DAMBRK' MODEL

3.1 Introduction

The DAMBRK model attempts to represent the current state-of- the art in understanding of dam failure and the utilization of hydrodynamic theory to predict the dam break wave formation and its downstream progression. The basic computer program was developed over a period of several years by Dr. Danny.L.Fread of the National Weather Services (NWS). The model has wide applicability; and can function with various levels of input data specifications, and requires a minimal computation effort on large computing facilities.

The model consists of three functional parts:

1. Description of the dam failure mode;

2. Computation of outflow hydrograph through the breach as affected by the breach description, reservoir storage characteristics, spillway outflows and downstream tailwater elevations; and

3. Routing of the outflow hydrograph through the downstream valley in order to determine the change in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations and flood wave travel time.

3.2 Assumptions

The following assumptions were used in developing the model:

1. Cross sections in the downstream channel are oriented perpendicular to the flow so that the water surface is horizontal across the section.

2. The channel boundaries are rigid, i.e. cross sections do not change shape due to scour or deposition.

3. The pool elevation at which breaching begins, rate of breach development, and shape and size of the breach must be supplied by the user.

3.3 Data Requirements

The input data requirements far the 'DAMBRK' program are flexible. When a detailed analysis is not feasible due to lack of data or insufficient data preparation time, the unknown or unavailable data can be ignored (left blank in the input file or omitted altogether). Nonetheless the resulting approximate analysis is more accurate and convenient to obtain than that could be computed by other techniques.

The input data can be basically classified into two groups:

1. Data group pertains to the dam:

Reservoir data- inflow hydrograph, length of reservoir, initial elevation of water in reservoir, elevation of water in reservoir when breach occurs, elevation of top of dam, elevation of bottom of dam, and reservoir volumes or surface areas and their corresponding elevations,

Breach data - time taken for the full breach formation, final bottom width of breach, side slope of breach, and final elevation of breach bottom.

Spillway data- spillway rating curve, elevation of uncontrolled spillway crest, coefficient of discharge of uncontrolled spillway, elevation of centre of submerged gate opening, coefficient of discharge of crest of dam, and constant discharge from dam like discharge through turbines.

2. Data group pertains to the downstream routing reach:

Cross section details - mileage of the cross sections from the dam, a table of top widths (active and inactive), and corresponding elevations at each sections, hydraulic resistance coefficients (Manning's roughness coefficients), expansion/contraction coefficients, slope of the downstream channel for the first mile below the dam, and initial conditions in the downstream channel.

3.4 Basic Program Capabilities

1. Reservoir Routing: An inflow hydrograph can be routed through a reservoir using either storage or dynamic routing. Outflow at the dam at any instant is computed by summing the discharge over the spillway, over the top of the dam, through the breach, through a gated outlet and through turbines,

2. Breach Simulation: Two types of breaching may be simulated: -

a) An overtopping failure in which the breach shape can be triangular, rectangular or trapezoidal which grows progressively downward from the dam crest with time.

b) A piping failure in which the breach can be simulated as a rectangular orifice that grows with time and is centered at any specified elevation within the dam. If the elevation of water surface in the reservoir, when breach occurs, is below the top of the dam, the model will automatically take the failure as a piping failure.

3, River Routing: The breach outflow hydrograph is routed through the downstream river valley using the complete flow equations.

3.5 Other Capabilities

1. Lateral Inflow and Outflow: The program treats the flow as being uniformly distributed in a reach between two adjacent downstream cross sections. The user must specify the sequence number of the cross section immediately upstream of where the lateral flow occurs. Lateral inflow carries a positive sign and outflow a negative sign. Backwater effects of the dam break flood flow on the tributary flow are neglected and the lateral flow is assumed to enter perpendicular to dam break flow.

2. Super Critical Flow: The 'DAMBRK' program can simulate flow that is either sub critical or super critical. However only one type of flow can be accommodated in a given routing reach throughout the duration of the flow. Super critical flow usually occurs when the slope of the down stream valley exceeds about, 50ft/mile. In that case two upstream boundary conditions, i.e. reservoir outflow hydrograph and a looped rating curve based on the Manning's equation in which the slope is defined as the water surface slope at the end of the previous time period, are required. It is also possible to simulate a situation in which a super critical flow occurs in an upper reach and sub critical flow occurs in an adjacent downstream reach.

3. Multiple-Dam Modeling: DAMBRK has the capability to model a situation in which two or more dams occur in series. There is choice of two methods for simulating dam break flows in a valley having multiple dams

In the 'Sequential Method', stream segments bounded by dams are treated as separate entities, and all flow routing is completed for a segment prior to progressing to the segment down stream. Breaches may be simulated at any of the dams. Storage routing through the upstream reservoir may also be employed.

In the 'Simultaneous Method', the entire length of the river being studied is treated as one segment, with the exception that storage routing through the upstream reservoir may be employed. Flow conditions at dams are treated as internal boundary conditions. Breaches may be simulated at any of the dams. This method is preferred where tail water effects are significant.

The simultaneous method can also be employed to simulate the flow through bridges and their associated earthen embankments, by treating them as internal boundary conditions.

4. Flood Plain Modelling: For situations in which the main channel and overbanks each carry substantial portions of the flow, and the mean velocity in the main channel differs largely from that in the overbanks, the flood plain modelling capability of 'DAMBRK' can be used. It enables representation of a cross section with three separate components - left overbank, main channel and right overbank. The program determines conveyance for each cross sectional components separately and sums it to obtain the total conveyance of the cross section. Separate tables of elevation vs. width and sets of 'n' values and reach lengths should be specified for each component.

5. Land Slide Modelling: DAMBRK program is capable of simulating the generation of a wave due to landslide into a reservoir. The program assumes that the landslide material is deposited within the reservoir in layers and reduces the original dimensions of the cross section. Reservoir dynamic routing must be used with this option.

6. Routing Losses: Capability exists in 'DAMBRK' to simulate losses of water that vary with time in accordance with flow magnitude. The user is required to specify the maximum rate of lateral outflow.

The 'DAMBRK' program has the capability of simulating 12 different cases corresponding to combinations of various reservoir routing techniques and channel flood routing techniques with the above special options.

4.0 METHODOLOGY

An overall study of the 'DAMBRK' model, its data requirement and capabilities has already been done in the previous chapter. Here a brief description of methodology used for the basic program capabilities is given.

4.1 Reservoir Routing

In this model, the reservoir routing may be performed either using storage routing or dynamic routing.

a) Storage Routing: Under the assumption that the reservoir surface is horizontal at all times, the hydrologic storage routing technique based on the law of conservation of mass is used.

$$\mathbf{I - Q = \frac{ds}{dt}} \tag{1}$$

Where, I = Reservoir Inflow
Q =Reservoir Outflow
ds/dt = Rate of change of storage volume

Equation (1) can be expressed in finite difference form as

$$\mathbf{(I + I')/2 - (Q + Q')/2 = \delta s / \delta t} \tag{2}$$

in which I' and Q' denotes values at time t and (t+δt) and the notation approximates the differential. The term δs may be expressed as,

$$\mathbf{\delta s = (As + A's)(h - h')/2} \tag{3}$$

in which, As is the reservoir surface area corresponding to the elevation h and it is a function of h. The discharge Q which is to be evaluated from equation (2) is a function of h and this unknown h is evaluated using Newton-Raphson iteration technique and thus the estimation of discharge corresponding to h.

b) Dynamic Routing: When the breach is specified to form almost instantaneously so as to produce a negative wave within the reservoir, and/or the reservoir inflow hydrograph is significant enough to produce a positive wave progressing through the reservoir, a routing option which simulates the negative and/or positive wave occurring within the reservoir may be used in 'DAMBRK' model. Such a technique is referred to as dynamic routing. The routing principle is same as dynamic routing in river reaches and it is performed using St. Venent's equation that will be described later in the section on downstream routing.

4.2 Reservoir Outflow Computation

The total reservoir outflow Q at any instant is the sum of flow through the breach, flow through dam outlets, spillway and over the dam crest. As already mentioned, two types of breaching may be simulated. Flow through an overtopping breach at any instant is calculated using a broad crested weir equation. In the case of a piping failure, instantaneous flow through the breach is calculated with either orifice or weir equations depending on the relation between pool elevation and the top of the orifice. The breach begins when the reservoir water surface elevation exceeds a user specified elevation Hf and grows linearly in time until Hb=Hbm, where Hb is the elevation of the breach bottom at any time and Hbm is the final elevation of the breach bottom. Hbm is usually taken to be the channel bottom or the dominant ground elevation of the dam, except when this is not physically justifiable due to backwater effect. Therefore, cross sectional information immediately downstream of the dam in order to calculate tail water elevation for any needed correction for partial submergence is required. An overtopping failure is simulated if Hf = Hd where Hd is the elevation of top of the dam.

The peak shape of the outflow hydrograph due to dam breach is governed largely by the geometry of the breach and its development with time.

The tail water is estimated from Manning's equation. The geometric properties for this are obtained from the input cross section immediately downstream of the dam. This estimated tail water depth does not include any dynamic effects or back water effects due to downstream constrictions. When such effects are there, the 'simultaneous method' of computation should be used.

4.3 Downstream Routing

The movement of the reservoir outflow hydrograph (dam break flood wave) through the downstream river channel is simulated using the complete unsteady flow equations for one- dimensional open channel flow, known as St. Venent's equation.

These equations are conservation of mass:

$$\frac{\partial \mathbf{Q}}{\partial \mathbf{x}}+\frac{\partial(\mathbf{A}+\mathbf{A}_0)}{\partial \mathbf{t}}=\mathbf{q} \tag{4}$$

And, conservation of momentum,

$$\frac{\partial \mathbf{Q}}{\partial \mathbf{t}}+\frac{\partial(\mathbf{Q}^2/\mathbf{A})}{\partial \mathbf{x}}+\mathbf{gA}\left(\frac{\partial \mathbf{h}}{\partial \mathbf{x}}+\mathbf{S}_f+\mathbf{S}_e\right)+\mathbf{L}=0 \tag{5}$$

Where,

A and A_0 are active and inactive flow area;

x is the distance along the channel

t is the time;

q is the lateral inflow or outflow per unit distance along the channel;
g is the gravitational acceleration;

Q is the discharge;

h is the water surface elevation;

S_f is the friction slope;

S_e is the expansion-contraction loss slope, and

L is the lateral inflow or outflow momentum effect due to assumed flow path of inflow being perpendicular to the main flow.

The friction slope and expansion-contraction loss slope are evaluated by the following equations:

$$S_f = \frac{n^2|Q|Q}{2.21 A^2 R^{4/3}} \tag{6}$$

And

$$S_e = \frac{K \Delta\left(Q/A\right)^2}{2g\Delta x} \tag{7}$$

Where,

n is the Manning's roughness coefficient;

R is (A/B) where B is the top width of active portion of the channel;

K is an expansion-contraction coefficient varying from 0.1 to 0.3 for contraction and -0.5 to -1.0 for expansion, and $\Delta(Q/A)^2$ is the difference in $(Q/A)^2$ for cross-sections at either end of a reach.

The non-linear partial differential equations (4) and (5) are represented by a corresponding set of non-linear finite difference algebraic equations. They are solved by Newton- Raphson method using weighted four point implicit scheme to evaluate Q and h. The initial conditions are given by known steady discharge at the dam, for which water surface elevations at each cross sections are calculated by solving the steady state non-uniform flow equation. The outflow hydrograph from the reservoir is the upstream boundary condition for the channel routing. There is a choice of downstream boundary conditions such as internally calculated loop rating curve, user provided single valued rating curve, user provided time dependent water surface elevation, critical depth and a dam which may pass the flow via spillways, overtopping and/or breaching.

5.0 STUDY AREA

Myndtu (Leska) hydroelectric project stage – I is a run of the river scheme on the Myntdu river in the Jaintia Hills district of Meghalaya, conceived by Meghalya State Electricity Board (MeSEB). This project will comprise of a diversion concrete gravity type dam of 59 m height for utilising the inflow from a catchment area of 350 km^2. The installed power generation capacity of the project is 2x 42 MW. The salient features of the projects pertaining to dam and spillway are given below:

Type of dam	Concrete gravity dam
Height of dam	620 m
Full reservoir level	618 m.
Gross storage at full reservoir level	14 M cubic m
Minimum draw down level	606.15 m
Bed level	563 m
Length of spillway	103.75 m
Crest level	597 m
Spillway capacity	1200 cumecs
Gates	5 nos. (16.75 m x 17 m)
Length of non overflow section	187 m
Length of reservoir	4 km

Availability of Data – Almost all the data for the dam, reservoir and downstream channel are collected from MeSEB, Barapani. The following data have been used in the present study:

1. Maximum probable flood, as calculated by Central Water Commission (CWC) is taken as inflow hydrograph required for reservoir routing at the time of dam failure. The 24 hour hydrograph is tabulated in **Table.1** and shown in **Fig.1** (MeSEB, 1999).

The reservoir area elevation curve is obtained from the same detailed project report and represent in **Table.2** and shown in **Fig.2**. Because the rating curve for the spillway could not be found out, the discharge coefficient for spillway gate flow CG, and discharge coefficient for uncontrolled weir flow over top of dam CDO, has been used in the model.

2. Second data group – In this study 12.1 km of river stretch, represented by 25 sections at an interval of 0.5 km has been used. The cross sectional details are taken from survey report of MeSEB done specially for dam break study, (MeSEB, 2000).

Each section is divided into six zones and Manning's constant for each zone has been estimated in accordance with the available literature (Chow, 1988) and ground condition from topographical map of the area at 1:15,000 scale.

3. Variables – The breach parameters have been assumed as suggested in DAMBRK User's manual (NWS DAMBRK, 1988). The breach parameters are as follows:

1. Breach Width – 80 m
I. Side slope of Breach – 0
II. Failure time – 0.1 hour (6 min.)

III. Initial water level in reservoir – 616m
IV. Water level in reservoir when breach starts – 616.1m

Table.1 Inflow hydrograph (PMF)

Time (hour)	Runoff (cumec)	Time (hour)	Runoff (cumec)	Time (hour)	Runoff (cumec)	Time (hour)	Runoff (cumec)
0	100	10	9075	20	1956	30	512
1	141	11	11229	21	2085	31	354
2	268	12	12056	22	2472	32	256
3	218	13	11085	23	2525	33	192
4	1330	14	9056	24	3080	34	150
5	2340	15	6897	25	2791	35	126
6	3268	16	4956	26	2238	36	110
7	4201	17	3507	27	1660	37	104
8	5328	18	2587	28	1149	38	101
9	6904	19	2107	29	765	39	100

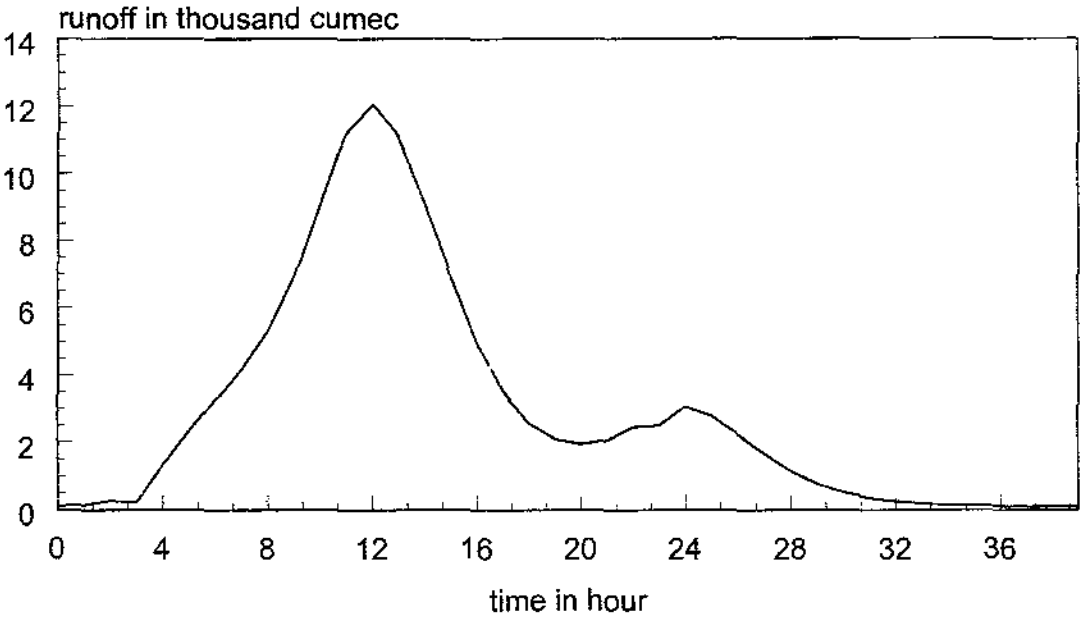

Fig.1 Inflow hydrograph

For the study of sensitivity of breach width on reservoir outflow the model is run for breach width of 60 m, 100 m, and 120 m. Model has also been run for failure time of 4 min., 8 min. and 10 min. The inflow hydrograph of the reservoir has also been changed to 90%, 110% and 120% of PMF to study the sensitivity of the inflow over the dam fail.

Table.2 Area elevation data of the reservoir.

Level (m)	Area (ha)
618	71
615	65
610	52.8
605	42
601.4	35.8
601	35.1
595	25.5
590	18.8
585	14.2
580	10.4
575	6
570	0

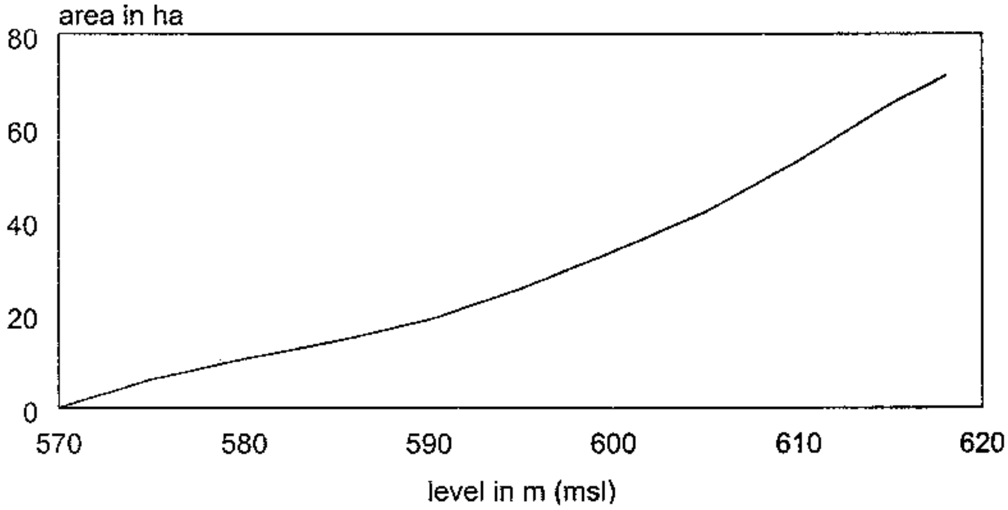

Fig.2 Area elevation curve for the reservoir.

6.0.RESULTS AND ANALYSIS

The topographical details of the Myntdu River down stream of Myntdu Leska dam site is available up to a distance of about 12 km. Therefore all the analysis in this study has been done in this portion of the river. The detailed survey of cross sections of the river has been done by MeSEB at an interval of 0.5 km down stream of dam site and this information is used in the study (MeSEB, 2000). This stretch of the river is represented by 25 sections, 0 to 24, and zero starts at the dam site. Slopewise the river can be divided into 3 zones. Zone-1 is having very steep slope (5%) up to Ch (chainage) 7.43 km and after that slope is very mild (0.27%) up to 9.93 km, zone-2. Again from Ch-9.93 km onwards the slope increases (2.13%), zone-3. Further the river bed level at Ch-7.93 km is higher in comparison to its down stream and up stream sections. This feature of river profile affects the flow characteristics in these three zones.

When the flow travels from steeper to mild slope, zone-1 to zone-2, flow changes from super critical to sub critical and water depth increases. Further when flow moves from mild to steep slope, zone-2 to zone-3, the flow characteristic changes from sub critical to super critical and water depth decreases for the same flow. Moreover, the bed slope is not the only variable to decide the water level at any section. The cross sectional area of river at different chainage also directly effect the water level. Cross sectional area at Ch-6.93 km is about 15% less than that of Ch-6.43 km. All these physical properties of the river has been taken into account in the study. It has been observed that whenever the flow increases the maximum water depth at different section of stream increases in zone–1 and zone-3 while it decreases very rapidly in zone-2.

The Manning's constant has been estimated based on information available in topographical map at 1:15,000 scale according to standard values available in literature (Chow, 1988).

6.1 Dam Break Analysis

The most critical situation for dam break is the condition when the reservoir is at FRL and then peak of the most severe flood (PMF) impinges over the reservoir. In this condition it is further assumed that all the gates of the spillway remains open. In this case when the model (NSW DAMBRK) is run at initial water level of 618 m (FRL), it is observed that the maximum water level attained by impingement of PMF is less than 618 m. This is because when reservoir is at 618 m and all spillway gates are open the outflow is larger than the inflow and water level decreases rapidly. The model is run for different initial water levels from 608 m to 616 m. It is found that maximum water level reached in reservoir is 616.19 m for initial water level of 616 m. This means that the dam will not fail by overtopping in any condition for this magnitude of PMF. For the dam break analysis it is assumed that the dam may fail by breaching. Since the dam is of concrete gravity type the time of breach is assumed as 0.1 hour (6 minutes). The breach width of 80 m is assumed. The initial water level in the reservoir is 616 m and it reaches 616.1 m when PMF inflows into reservoir, and then breach starts.

The model is also run for routing of PMF without dam. The PMF hydrograph is applied at dam section and maximum water levels at different sections downstream of dam have been calculated. **Fig. 3** shows the bed level and minimum bank level of river and the maximum water level reached due to PMF flow with dam break flood and without the dam. This figure shows that the flow without the dam is well confined within the banks of

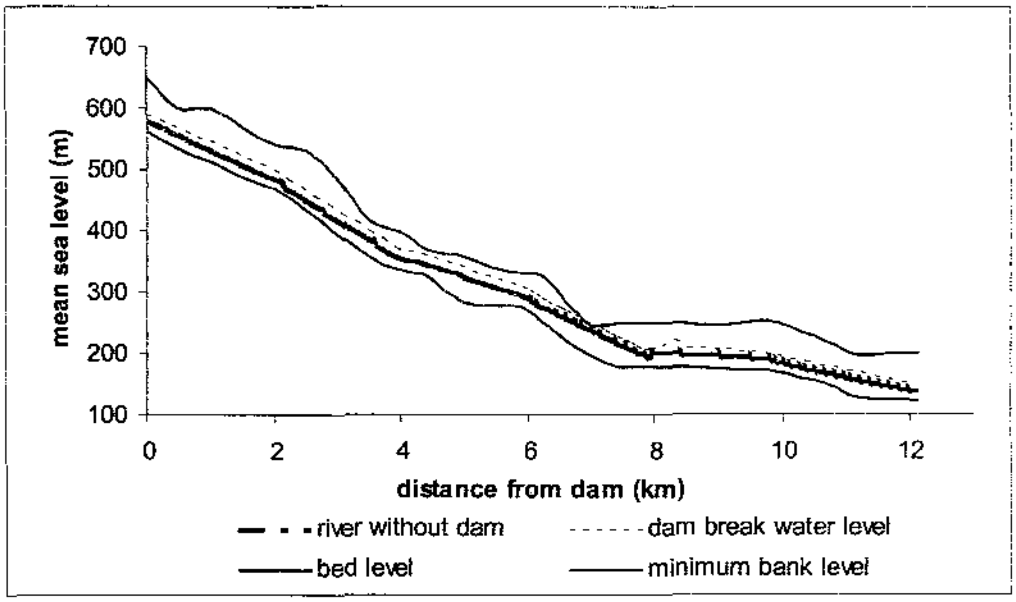

Fig. 3 Longitudinal profile of river, minimum bank level, normal river flow and dam break flood flow

the river. Even the dam break flow is below minimum of bank level everywhere except at Ch-6.93 km. At this section the maximum water level reached due to dam break flood is 252.87 m while bank level is 249.9 m` only. It means the dam break flood have 2.97 m of water column to spill over the bank at this section. To know the water spread due to this spilling the section at Ch-6.93 km is studied in detail. The river cross section at this chainage is shown in **Fig. 4**.

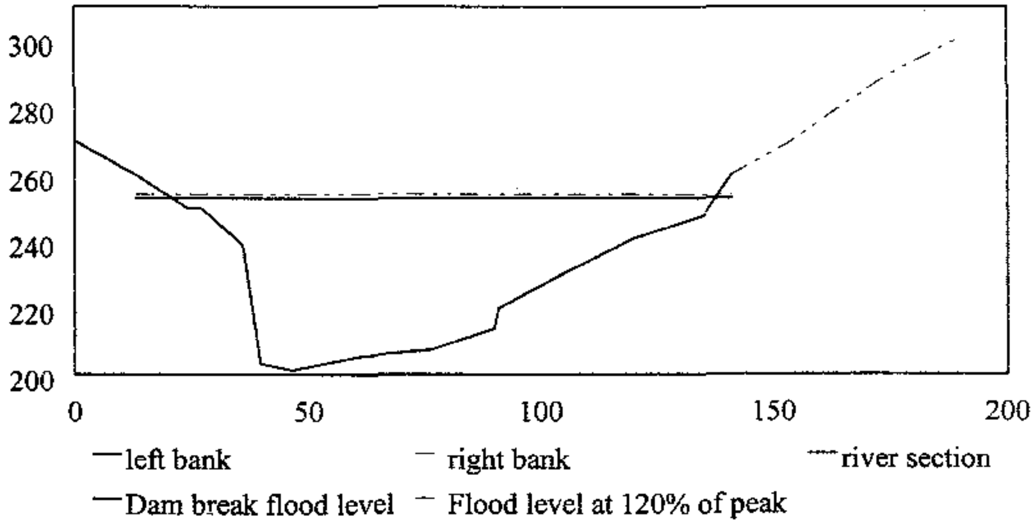

Fig. 4 River cross-section at Ch-6.93 km.

The river section is shown in dotted lines as obtained from river survey done by MeSEB at this section. The bank section is drawn further on either side of the river from the contour survey at Ch-6.93 km which is shown by continuous line in the figure. Also the water level reached by dam break flood and for 120% of PMF is shown continuous and dotted lines respectively. It can be seen from this figure that additional stage of 2.97 m will pass through the section to this section just by spreading over water by 7 m on left bank. The land use of this 7 m is dense forest as observed from topographical map. On the right bank side this water level remains well within the marked bank level.

6.2 Sensitivity Analysis

As the dam failure condition is totally hypothetical the sensitivity analysis has been done for its variable parameters. Keeping other parameters constant the value of one parameter is changed and its effect over the maximum flow and maximum water level is studied. The model has been tested for breach widths of 60 m, 100 m and 120 m. **Fig. 5a** to **5c** represent the effect of breach width over the dam failure. In these figures the maximum water level at different sections corresponding to dam fail (breach width of 80 m) and with changed breach widths are compared. Also, as the difference in maximum water level is very less it is plotted on different scale. Form these figures it is clear that when breach size increases more water outflow from reservoir and water level increases in super critical zones (zone-1 and 3) and decreases in sub critical zone (zone-2).

The sensitivity of peak flow is done by varying the reservoir inflow by 90%, 110% and 120% of PMF. In case of 90% of PMF the water level in the reservoir does not rise to 616.1 m and therefore the dam does not breach. In this case maximum water level at every section remains below the dam break flood level and its difference is negative at each section as shown in **Fig 6a**. **Figs. 6b** and **6c** show that water level increases when 110% and 120% of PMF are applied.

The sensitivity analysis for breaching time has been done by changing the breach time to 4 min, 8 min and 10 min. When breach time is decreased outflow through spillway is comparatively less and more water is available in reservoir when dam fails. Therefore the maximum water level at different sections increases as shown in Fig **7a**. With increase in breach time, significant amount of reservoir water outflow through spillway and at time of dam failure less water is available. This causes the decrease in maximum water level at every section as shown in **Figs. 7b** and **7c**.

6.3 Movement of Flood Hydrograph

When the flood wave moves downstream of the dam the magnitude of the hydrograph peak decreases while its time of occurrence increases. It means when dam fails higher peak at early stage occurs just near the dam site and its effect diminishes as the flood wave move down stream. For different cases of failure the flood hydrograph at 4 sections, *viz.* at Ch-2, 5.93, 9.93 and 12.1 km, down stream of the dam have been calculated.

Peak of maximum flow hydrograph due to dam break reaches the remotest section (Ch-12.1 km) in 0.75 hours as shown in **Fig.** 8. With decrease in breach width the breach outflow from the reservoir decreases and therefore at every section peak of hydrograph reduces and also it takes more time to attain it as shown in **Fig.** 9a. With increase in

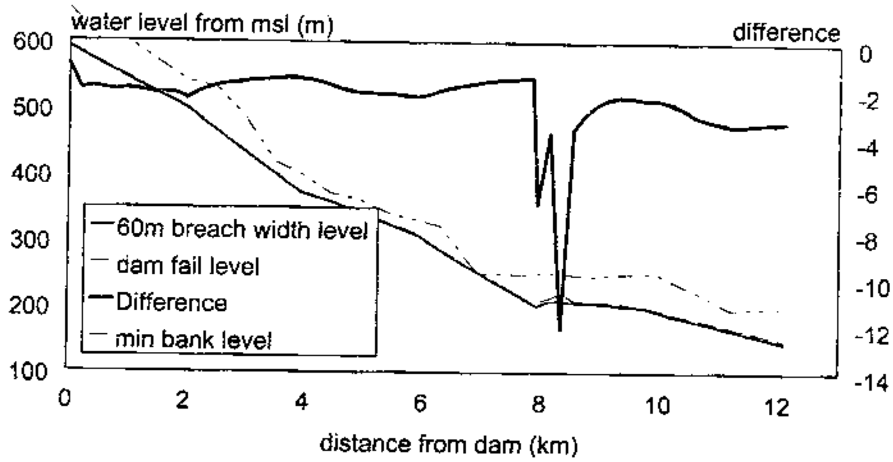

Fig. 5a Water level of dam fail flood when breach width is 60m

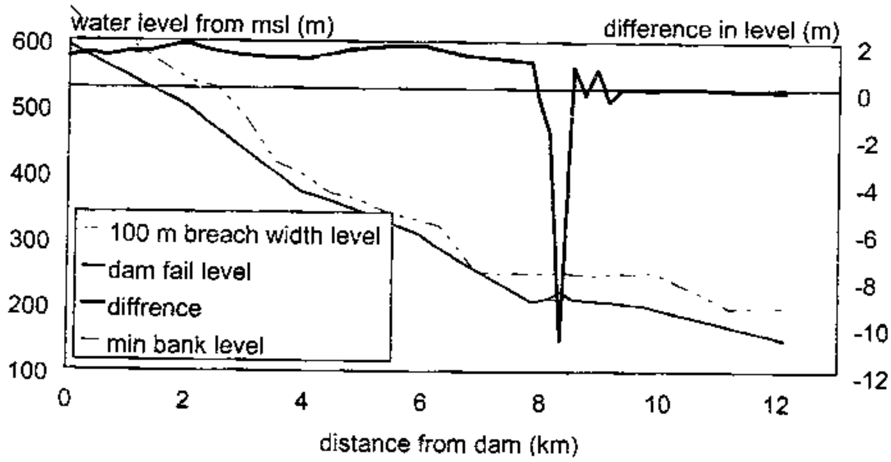

Fig. 5b Water level of dam fail flood when breach width is 100m

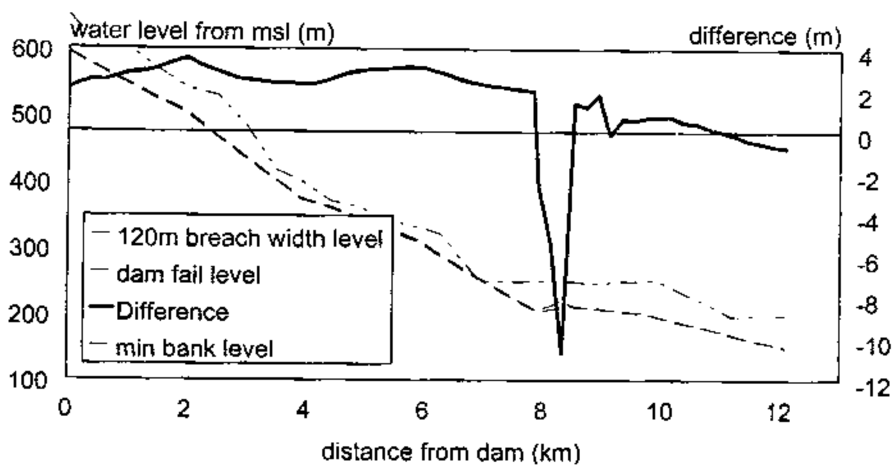

Fig. 5c Water level of dam fail flood when breach width is 120m

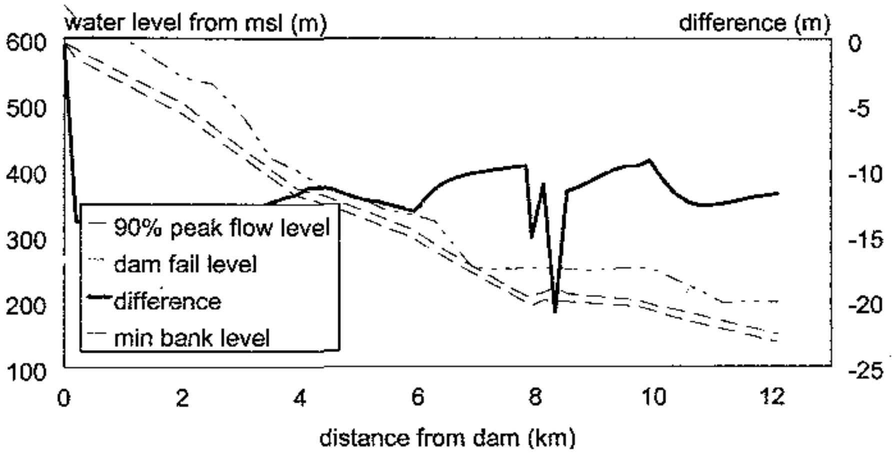

Fig. 6a Water level of dam fail flood when 90% peak flow passes through the reservoir

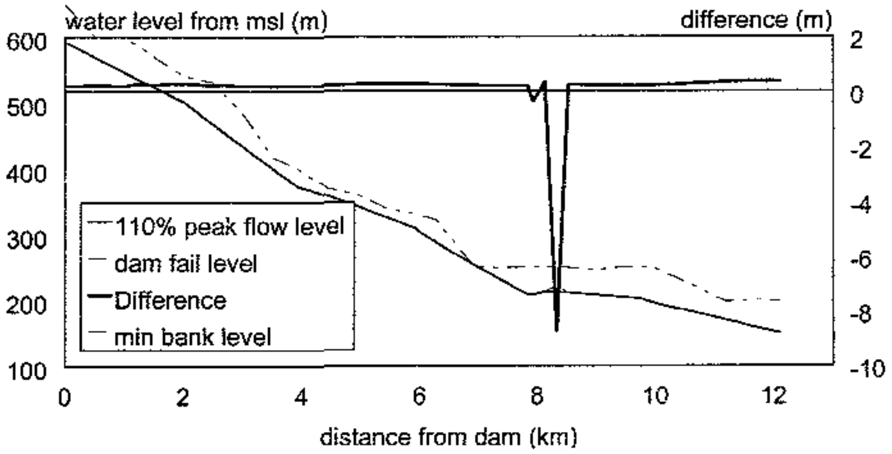

Fig. 6b Water level of dam fail when 110% peak flow passes through the reservoir

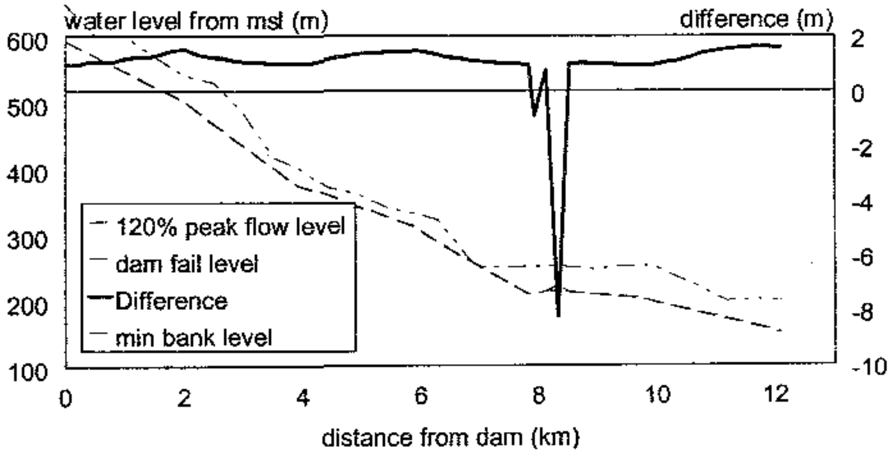

Fig.6c Water level of dam fail flood when 120% peak flow passes through the reservoir

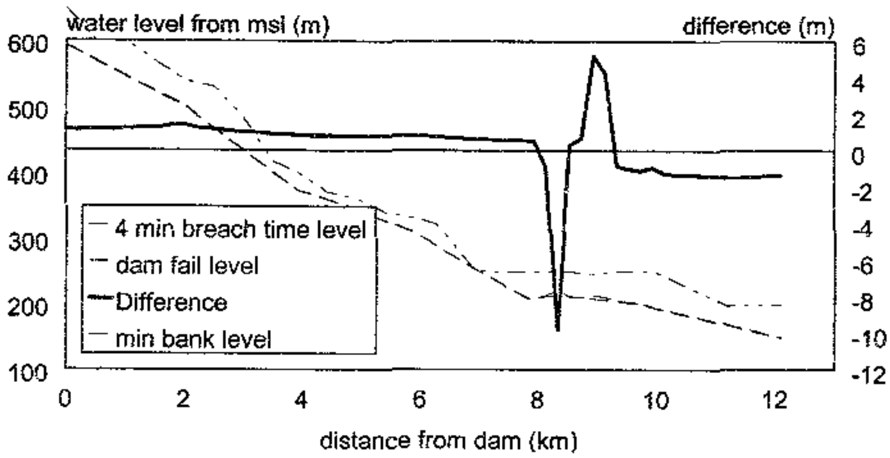

Fig. 7a Water level of dam fail flood for breach time of 4 min

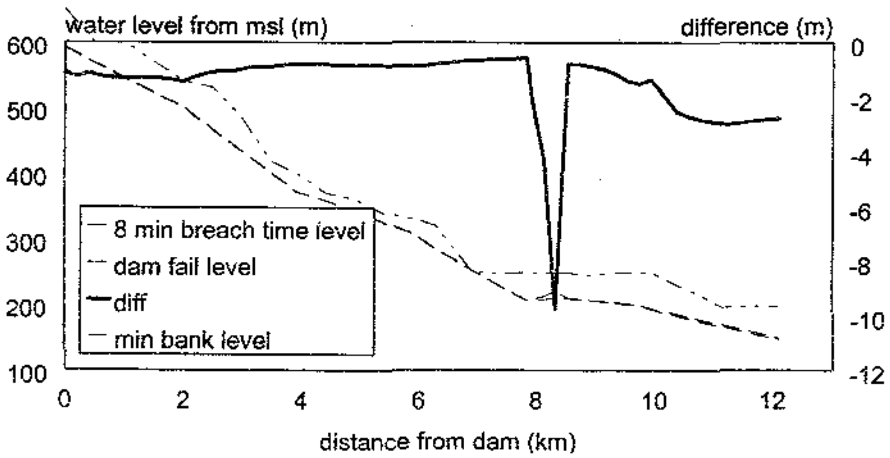

Fig. 7b Water level of dam fail for breach time of 8 minute

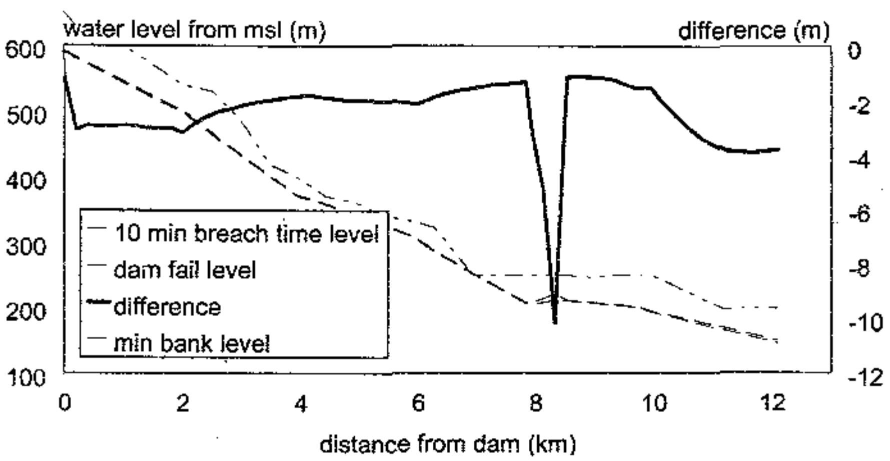

Fig. 7c Water level of dam fail for breach time of 10 minute

breach width the peak of hydrograph increases and also it occurs earlier at every section as shown in **Figs. 9b** and **9c**.

With reservoir inflow at 90% of PMF, dam does not break and the flow is entirely the spillway flow as shown in **Fig. 10a**. When reservoir inflow increases to 110% and 120% of the PMF, the peak increases and its time of occurrence also increases as shown in **Figs. 10b** and **10c**.

Effect of breach time on hydrograph movement is also very significant. With decrease in breach time the peak increases and its occurrence time decreases as shown in **Fig. 11a**. When breach time is increased as the significant amount of reservoir storage is out flowed through spillway before failure, the peak of hydrograph reduces and its time of occurrence increases as shown in **Figs. 11b** and **11c**.

Fig. 12a shows the sensitivity of breach width over the maximum flow at any section downstream of dam. In the fig the out flow is shown as the multiple of inflow (PMF). Similarly, **Figs. 12b** & **12c** shows the sensitivity of inflow and breach time over the maximum out flow at various sections.

6.4 Comparison with dam break study using MIKE 11

Central Water Commission, New Delhi, has done dam break analysis of the same project. using MIKE 11 model (CWC,2000). But there is a wide difference in the results obtained from two model studies. In the present study, (using NWS DAMBRK model) maximum flow of about 50,000 cumecs occurs at section 2 km down stream of dam while in MIKE 11 model study maximum flow of 18,434 cumec has been observed at 6.68 km down stream of dam site. This difference is due to different assumptions of initial water level in the reservoir. In MIKE 11 study initial water level is assumed to be at FRL (618 m) and it is further assumed that breach in dam starts when water level reaches 617.1 m. This assumption does not look reasonable. The reason is that breach must start at water level higher than the initial water level. It means with the impingement of PMF the water level of reservoir should increase and the extra hydrostatic pressure may be the cause for breaching. In MIKE 11 model study why the breach should wait for PMF to come when water level in the reservoir is 618 m and it is supposed to occur at 617.1 m level? In MIKE 11 study the assumption of initial water level at 618 m provides large head at spillway to discharge significant amount of reservoir water even before the breach occurs. It distributes the flow over time and the outflow hydrograph gets flattened and magnitude of peak flow decreases.

In the present study, the reservoir is assumed to be at 616 m and with the impingement of PMF water level increases to 616.1 m when breach occurs. This way the outflow hydrograph peak occurs quickly and also its magnitude is higher. This is a more critical condition for the flooding in down stream section of river.

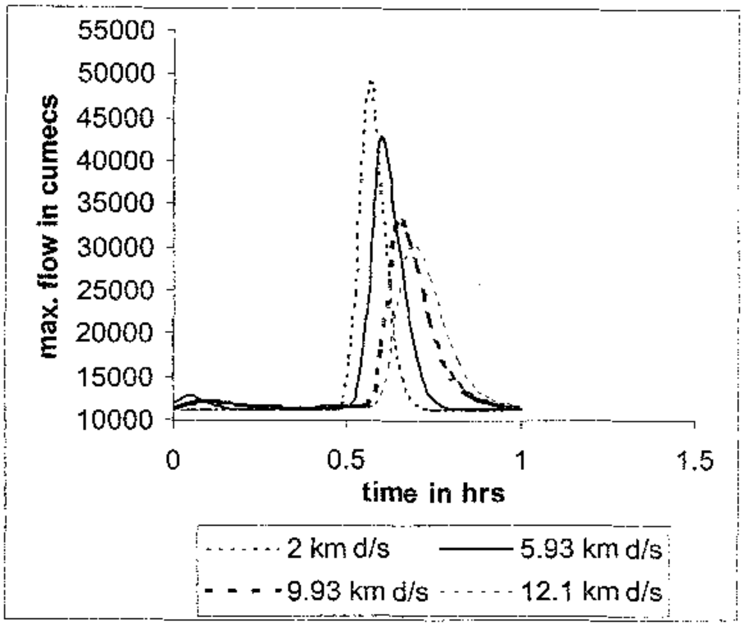

Fig. 8 Dam break flood hydrograph

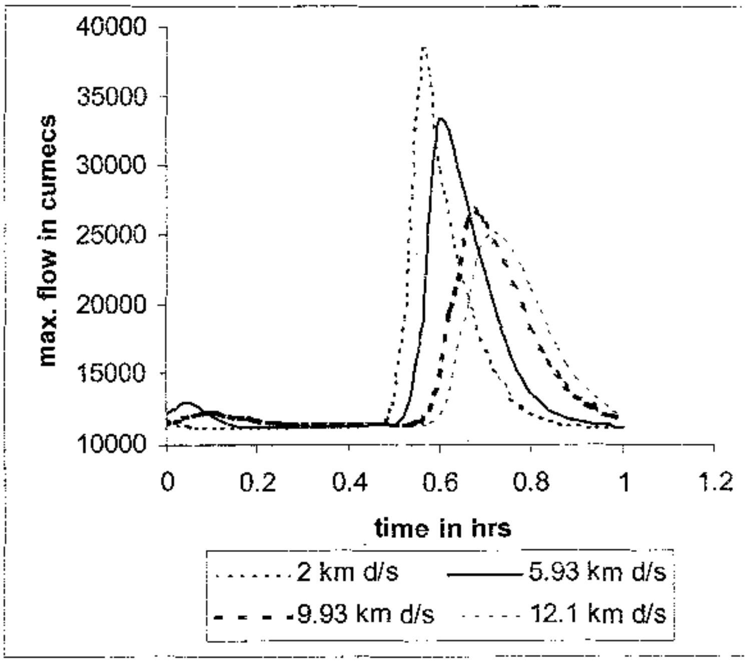

Fig. 9a Dam break flood hydrograph for breach width 60m

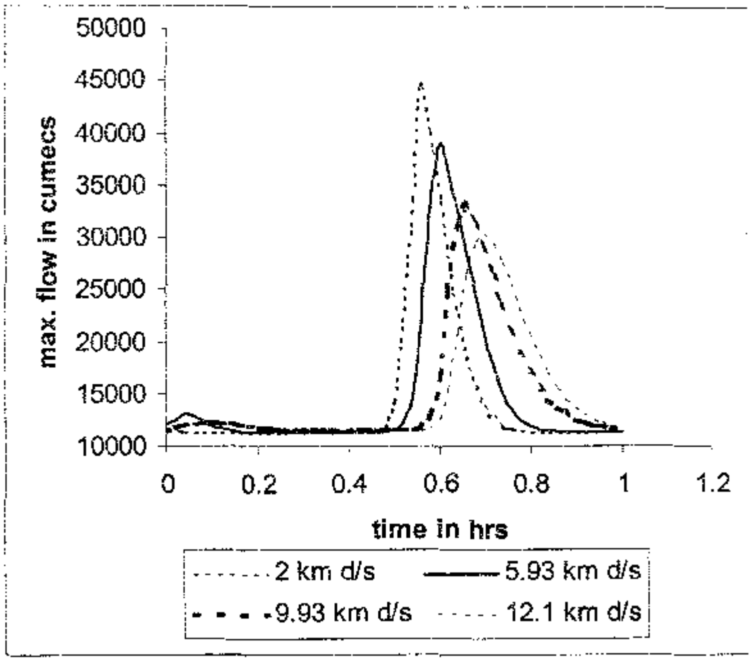

Fig. 9b Dam break flood hydrograph for breach width 100m

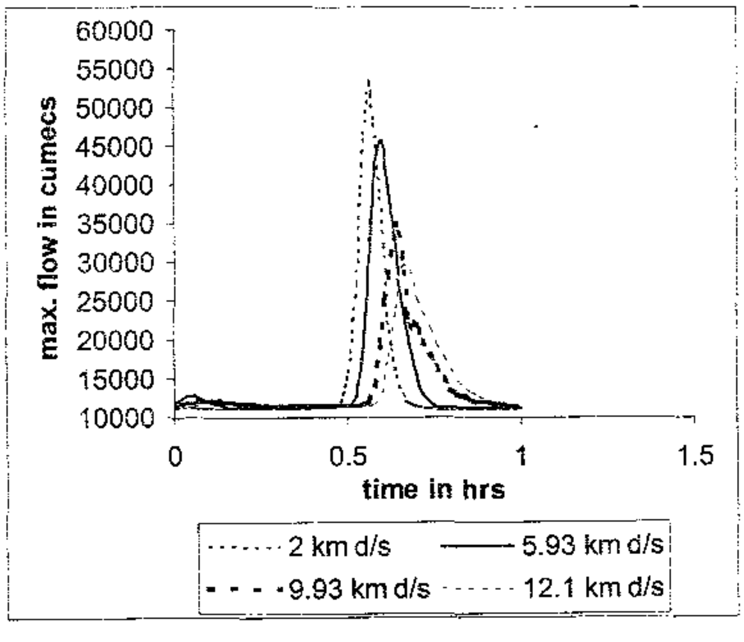

Fig. 9c Dam break flood hydrograph for breach width 120m

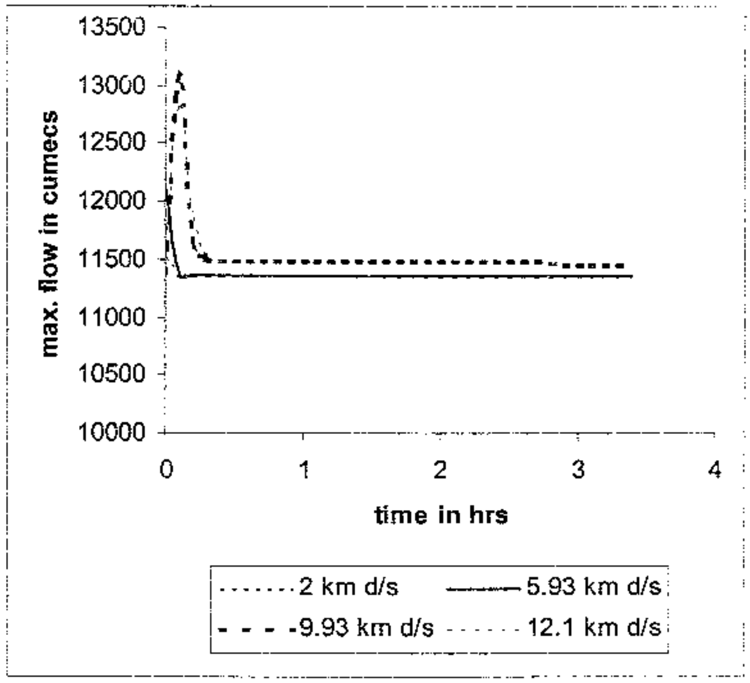

Fig. 10a Dam break flood hydrograph when 90 % of peak flow passes

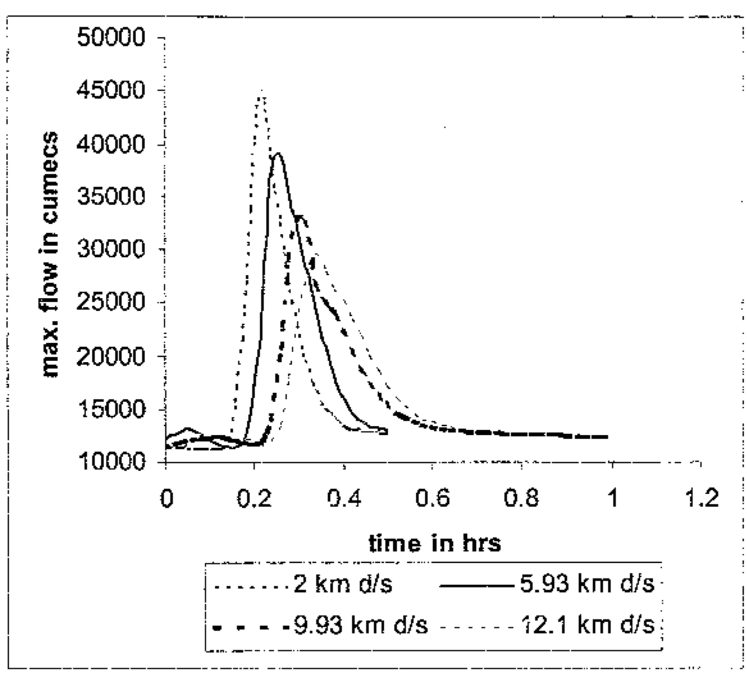

Fig. 10b Dam break flood hydrograph when 110 % of peak flow passes through the reservoir

A

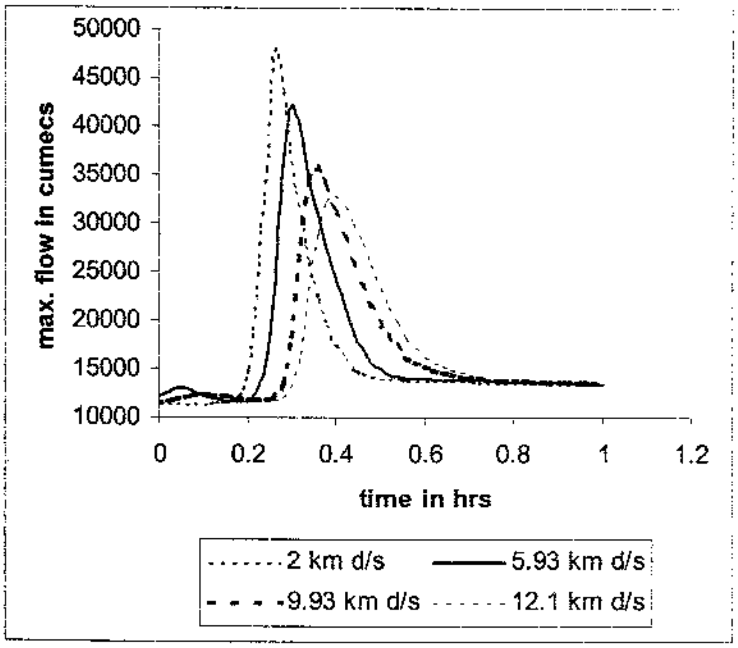

Fig. 10c Dam break flood hydrograph when 120 % of peak flow passes through the reservoir

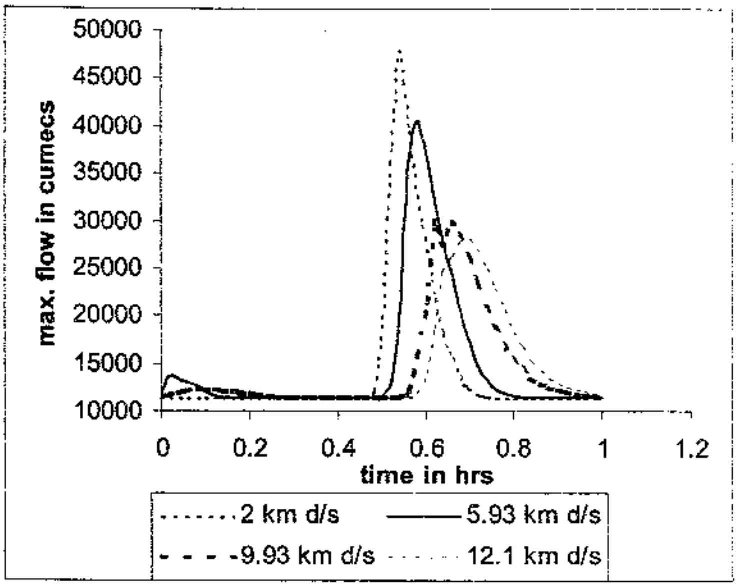

Fig. 11a Dam break flood hydrograph for breach time 4 min.

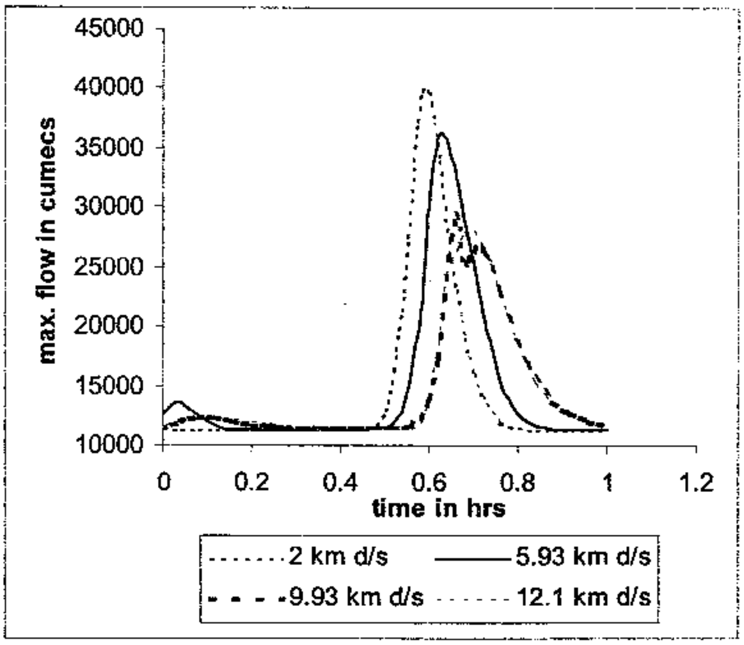

Fig. 11b Dam break flood hydrograph for breach time 8 min.

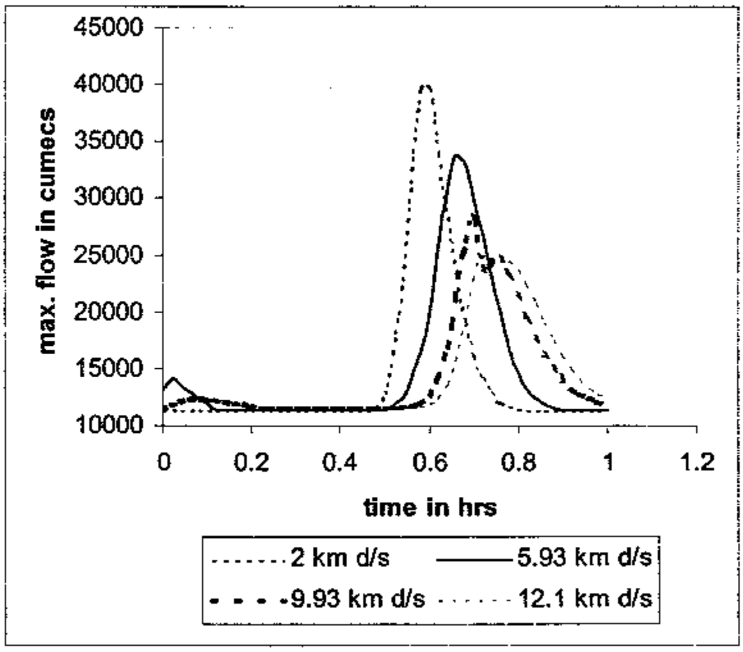

Fig. 11c Dam break flood hydrograph for breach time 10 min.

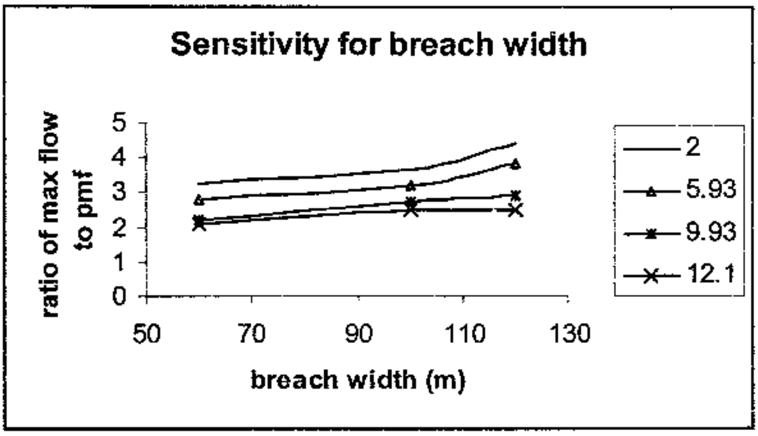

Fig. 12a Sensitivity of breach width

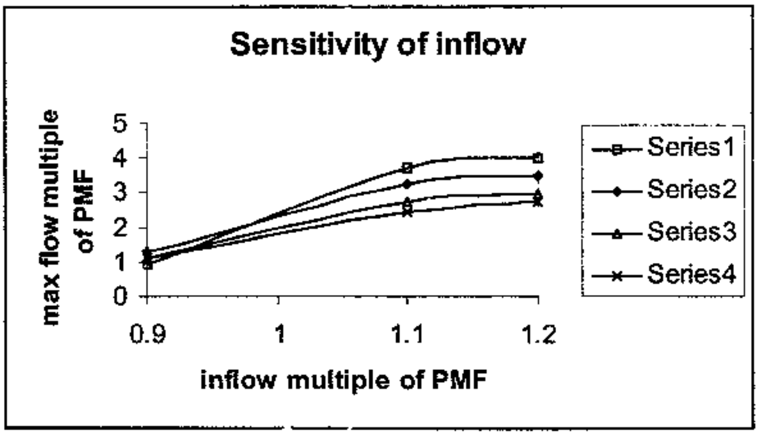

Fig. 12b Sensitivity of inflow

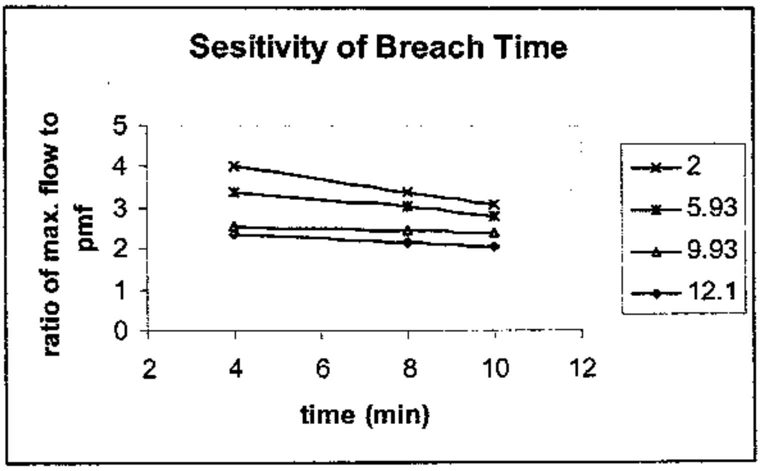

Fig. 12c Sensitivity of breach time

7.0 CONCLUSION

Based on the present study following conclusions have been drawn.

1. The spillway capacity is sufficient enough to pass the PMF even when the reservoir is at FRL and therefore the dam will not fail by overtopping. The dam may fail by breaching for which the study has been done.
2. The reservoir storage is very low 14 million cubic meters while the carrying capacity of the river is very high in general. Therefore the dam break flood does not spill at any location except at Ch-6.93 km. Even at this section with the water spread of about 7 m on left bank of the river the peak of flood hydrograph passes safely.

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