CS/AR-15/1999-2000

DAM BREAK ANALYSIS OF GHODAHODA PROJECT, ORISSA

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1999-2000

PREFACE

Dam Break flood analysis occupies very important place in water resources engineering practices. Such an analysis would help the administrators for developing efficient flood management practices. A large number of mathematical models have been reported to estimate the amount of dam break flood discharge propagating at different section along the downstream of the channel. One of these models called DAMBRK developed by Dr D. L. Fread of National Weather Service, Office of Hydrology, USA is the most widely accepted with various levels of data availability. This model is useful to estimate the flood elevation, discharge and travel time. This model has been employed for conducting a hypothetical dam failure study of Ghodahoda project in Orissa and is presented in this report.

As a part of work program of Deltaic Regional Center, Kakinada for the year 1999-2000, the report entitled '*Dam break analysis of Ghodahoda project, Orissa*' has been prepared by Mr. P. C. Nayak, Scientist 'B', K. P. Sudheer, Scientist 'B' and S. M. Saheb, PRA under the guidance of Dr K. S. Ramasastri, Scientist 'F', Coordinator and Head. The study has been revised by Dr. M. Perumal, University of Roorkee and Dr. S. K. Mishra, scientist 'E' & Head, Flood Studies Division, NIH, Roorkee.

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ABSTRACT

The past failure study is a needed exercise to be made for assessing the flood magnitude and their behavior downstream of the river valley so as to make a preparatory plan to safe guard the lives and properties on the flood plains of the river downstream of the dam structure. To estimate the amount of flood discharge reaching at different sections along downstream channel, a large number of mathematical models are reported in the literature. One of these models, 'DAMBRK' developed at the National Weather Service, USA is the most widely accepted model for it's various capabilities and advantages. The model could be employed in different levels of data availability and is reported to be efficient in computing the flood hydrograph after the breach.

The present study reports a hypothetical dam-failure study using DAMBRK model, which was carried out for simulation of flood wave in downstream of the Ghodahoda project, Orissa. Ghodahoda project is rock-filled earthen dam located in one of the tributaries of the Rishikulya River basin of Orissa State. The data used for the analysis included elevation details of the spillway and dam, spillway-rating table, elevation capacity table for the reservoir, design hydrograph for dam and cross sectional details of various downstream reaches. From the simulation, it is observed that the outflow peak over the spillway is 1776.4 cumec. The dam break flood reaches a peak discharge of 4402.9 cumec at the dam site in half an hour and the propagated flood downstream of the dam travels 28.43 km in 4 hours with a peak discharge of 3359.1 cumec. A sensitivity analysis was also carried out to study the effect of most influencing parameters such as breach length, time of breach and bed roughness, on the flood wave propagation. The analysis resulted that effect of the parameters has mere effect on the dam break flood characteristics.

The dam is one of the important structures constructed across a stream or a river to store the water in the reservoir. Basic purposes of a dam are to release water for water supply for domestic and industrial use, irrigation, hydropower generation, navigation, etc, and whenever there is a demand. And in times of flood, the dam can serve as protection for towns and cities farther down the river. Apart from the various advantages and uses of the dam, the devastation due to a flash flood resulting from sudden failure often results in loss of human life and caused extensive property damages in downstream area. The main causes for dam failure are overtopping failure, foundation failure and piping and seepage from the embankment dam.

One of the preventive measures in avoiding dam disaster is by issuing flood warning to the public of the downstream when there is a failure of the dam. However, it is quite difficult to conduct analysis and determine the warning time of the disaster. Therefore, pre-determination of the warning time assuming a hypothetical dam break situation is a needed exercise in dam safety analysis. The method used for such analysis gains more credibility if one can simulate the past dam break failure scenario using that method with reference to failure mode and flood wave movement downstream of the dam.

In the recent years, significant effect made of safety of dam against the dam failure. Many mathematical models have been developed for the simulation of dam break problems. Dam Break flood analysis occupies very important place in water resources engineering practices. Many researchers have developed models and method to evaluate flash floods due to dam failure and for routing them through areas. Using these methods inundated areas, flow depths and velocities can be estimated for different hypothetical dam failure situations. With the help of such studies, it could be possible to prepare strategies for disaster management. The main difficulty in using these mathematical models is to represent the failure description adopted in models. Under this circumstances, suitable assumptions with regard to the adjustment of actual failure mode to suit the model failure mode is necessary. So studies to evaluate the sensitivity of various types of failure modes on the resulting flood wave characteristics become important.

Analysis of a dam break flood can be performed using analytical (Stoker 1957, Hunt 1982), experimental (WES 1960, Dressler 1954), and /or numerical models (Fread 1980, Fenema and Chaudhury 1987, Hervouet 1996). However, a numerical model is the most convinient tool for a fast and systematic study. Detailed review articles on the available models for the dam break flood analysis have been presented by Singh (1996) and Almeida and Franco (1994). Among various numerical models available for dam break flood analysis, DAMBRK is one of the most widely used one. Unlike many research articles on the dam break floods, it has the considerations for practical applicability. Geometrical features of the channel and reservoir, time and shape of dam-breach, flow in a compound cross-section, presence of other hydraulic structures are taken into account.

The present study reports a hypothetical dam-failure study using DAMBRK model, which was carried out for simulation of flood wave in downstream of the Ghodahoda project, Orissa. Ghodahoda project is rock-filled earthen dam located in one of the tributaries of the Rishikulya River basin of Orissa State. A sensitivity analysis was also carried out to study the effect of most influencing parameters such as breach length, time of breach length, time of breach and bed roughness, on the flood wave propagation. From the present analysis, it was observed that overall effect on breach length, time of breach and bed roughness on the maximum water level was insignificant.

DAMBRK model is developed by Dr D. L. Fread of Office of Hydrology, National Weather Services, USA. The model consists of two conceptual parts, 1) description of the dam failure ie. the temporal and geometrical description of the breach and 2) a hydraulic computational algorithm for determining the hydrograph of the outflow through the breach as affected by the breach description, reservoir inflow, reservoir storage characteristics, spillway outflow, and downstream tail water elevation and for routing of the outflow hydrograph through the downstream valley in order to account for the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dam.

The input data required for the National Weather Services DAMBRK model can be categorised into two groups. The first data group pertains to the dam, the breach, spillways, and physical characteristics of the reservoir. The breach data consists of the following parameters as shown in Fig. 1:

- Time of breach formation, t (hr)
- Final bottom breach width, b (ft)
- Side slope of breach, z (ft/ft)
- Final elevation of the breach bottom, h_{bm} (ft)
- Initial elevation of water level in the reservoir, h_o (ft)
- Elevation of water when breach begins to form, h_f (ft)
- Elevation of top of dam, h_d (ft)

The spillway data consists of the following:

- Elevation of uncontrolled spillway crest, h_s (ft)
- Coefficient of discharge of uncontrolled spillway, C_s
- Elevation of centre of submerged gated spillway, h_g (ft)
- Coefficient of discharge of gated spillway, C_g
- Coefficient of discharge of crest of the dam, C_d
- Constant head independent discharge from dam, Q_t (cusec)

The Q_t is the discharge that takes place from the dam in the form of seepage. The reservoir data consists of a table describing the storage features of the reservoir, surface area (acre) or volume (acre-ft) versus elevation (ft).

The second group of data pertains to the cross-sectional features of the downstream river. This consists of a description of the cross-sections, hydraulic resistance coefficients, and expansion coefficient. The cross-sections are specified by location in mile, and tables of the top of width (active and inactive) and corresponding elevations. The active top widths may be total widths as for composite section, or they may be left flood plain, right flood plain, and channel widths. The active width of flow is determined by visual inspection. It is the width of flow that allows discharge to take place in the river. On the other hand, the inactive width does not allow water to flow across a section of a river. The water so stored is known as off-channel storage. It can be described by three cross-sections shown in Fig. 2.

One on the upstream end of the off-channel storage portion, one the downstream end of the off-channel portion, and the other in the middle of it. The inactive width (BSS) for the middle cross-section is computed as below:

BSS=2SA/L Where,

BSS- is the off-channel storage width in ft. for middle cross-section at an elevation (ft)
SA - surface area of off-channel storage in square feet at that elevation (ft).
L-Distance in ft. between first and third cross-sections

Implicit in this treatment of is the assumption that the time required by the water to occupy and evacuate this off-channel portion is negligible. This assumption would not be valid for a long tributary for which filling and draining times are significant relative to the time required for the flood wave to pass the mouth of the tributary. A different treatment would be necessary to model it accurately.

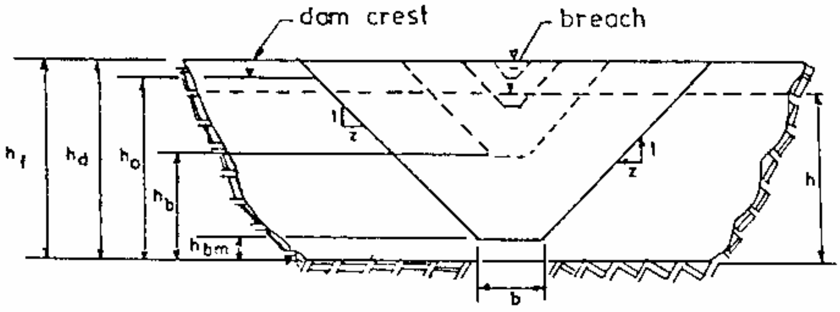

FIG.1-FRONT VIEW OF DAM SHOWING FORMATION OF BREACH

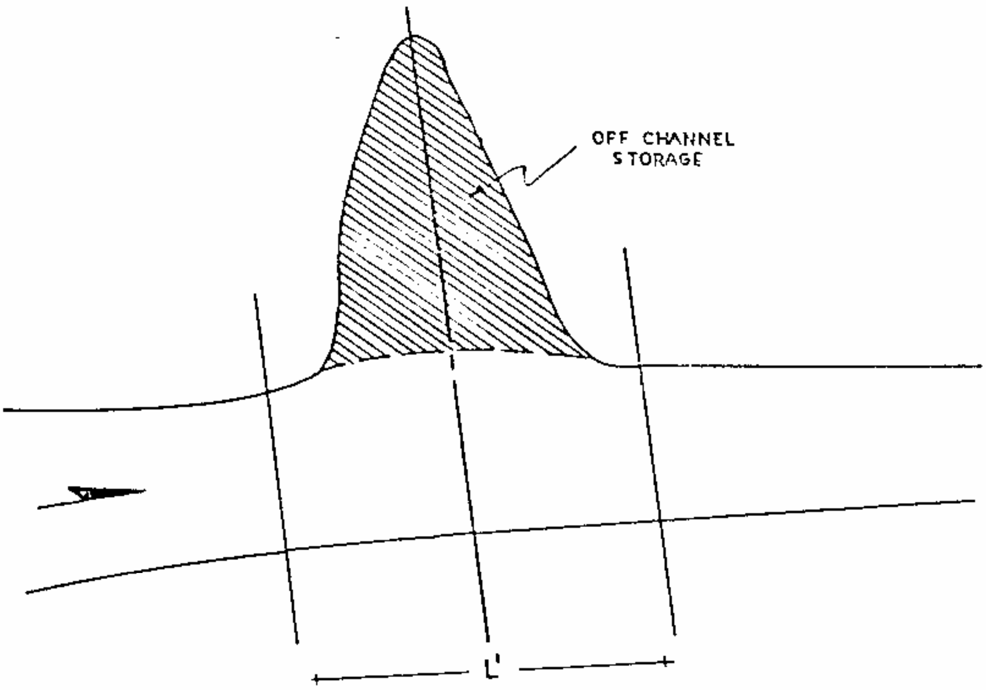

FIG.2-OFF CHANNEL STORAGE (PLAN VIEW)

The other properties of channel include hydraulic resistance and expansion/contraction coefficients are as below:

Roughness Representation

Boundary Representation is reflected in the equation of motion through the friction slope, which is defined, with Manning's equation. Friction slope is determined for a reach in terms of an arithmetic average of the hydraulic radii, for the effective flow portions cross-section at each end of the reach. The Manning's roughness coefficient (n-values) for varying elevations of flow are supplied to the DAMBRK model as input.

Expansion/contraction coefficients

The expansion and contraction coefficients are determined from the geometry of the river. The contraction values vary from 0.1 to 0.3 and the expansion values vary from −0.5 to −1.0. If the contraction and /or expansion effects between two consecutive cross-section is negligible, zero values are assumed.

Methodology

Overall details of the 'DAMBRK' model, its data requirement and capabilities have already been described briefly in the previous chapter. A brief description of methodology used for and the basic program capabilities are given below.

2.1 Reservoir Routing

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

2.1.1: Storage Routing

Under the assumptions that the reservoir surface is horizontal at all times, the hydrologic storage routing technique based on the law of conservation of mass is used.

$$I - Q = ds/dt \qquad (1)$$

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;

$(I + I')/2 - (Q/Q')/2 = \delta s/\delta t$ (2)

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

$\delta s = (As + A's)\,(h-h')/2$ (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.

2.1.2: 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, which will be described, later in the section on downstream routing.

2.2 : Reservoir Outflow Computation

The total reservoir outflow Q at any instance is the sum of flow through the breach, flow through dam outlets, spillway and over the dam crest. As already mentioned, two types of reaching may be simulated. Flow through an overtopping breach at any instant is calculated using a broad crested weir equation. In the case of 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 h_f and grows linearly in time until $h_f = h_{bm}$, where h_b is the elevation of the breach bottom at any time and h_{bm} is the final elevation of the breach bottom. h_{bm} 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 $h_f < h_d$ where h_d 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 estimate tail water depth does not include any dynamic effects or back water effects due to downstream constructions. When such effects are there, the 'simultaneous method' of computations should be used.

2.3 : Downstream Routing

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

$$\frac{\partial Q}{\partial X} + \frac{\partial (A + A_0)}{\partial t} = q \qquad (4)$$

and, conservation of momentum:

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial X} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e\right) + L = 0 \qquad (5)$$

where,

A and A_o 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;

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

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

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 .1 to .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 and 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.

The 'DAMBRK' model uses M. K. S. system of units for all parameters. i.e., elevations in meter, distance in kilometer, discharge in cumec (m^3/sec), storage capacity in m^3 etc.

2.4 : Limitation of NWSBRK model

Limitations of the of NWSBRK model developed by Dr D L Fread, National Weather Service, USA are listed below:

1. The model assumes one-dimensional flow.
2. The model assumes a rigid bed.
3. Bed roughness is a complex function of various flow and bed characteristics. The model however, assumes a constant value of Manning's roughness coefficient at all time levels.
4. Breach parameters, such as breach width and breach time used in the model do not represent the exact breach phenomenon.
5. The model excludes uncertainties associated with the volume losses due to infiltration and detention storage.

Ghodahoda project is an irrigation project constructed across the river Ghodahoda, a tributary of Rushikulya. It is located at 19^0 17' 00'' N latitude and 84^0 20' 30'' E longitude of Ganjam district of Orissa. The dam intercepts a catchment of 138 square km. The project was completed in 1974. The Ghodahoda dam is zoned rolled earth fill dam of 1724m long with maximum height of 25.98m. The surplus floodwater is passed through an ungated ogee spillway of 71.63m length with flood capacity of 906cumec at flood lift of 2.89m. The live storage of the reservoir is 2511.47 hac-meter. The water for irrigation is let out through a head regulator located on right bank.

SALIENT FEATURES OF GHODAHODA PROJECT

LOCATIHON

1. State	: Orissa
2. District	:Ganjam
3. Dam site location	: 19^0 17' 00'' N latitude and 84^0 20' 30'' E longitude
4. River	: Ghodahoda- a tributary of Rishikulya

HYDROLOGY

1. Catchment area	: 138 km^2
2. Mean annual rainfall	: 1200mm
3. Maximum rainfall	: 2094mm
4. Minimum rainfall	: 968mm
5. Design flood	:1494 cumecs

DAM

1.	Type of dam	: Rolled fill earth dam
2.	Total length of dam	: 1724m
3.	Deepest foundation level(RL)	: 69.60m
4.	Top width of dam	: 3m

RESERVOIR

1	Maximum water level	: 118.3m
2.	Full reservoir level	: 117.8m
3.	Minimum drawdown level	: 106.68m
4.	Lowest river bed level	:93.05m
5.	Capacity at FRL	: 3489.47Ham
6.	Capacity at DSL	: 388.53Ham
7.	Live storage	: 3100.47Ham
8.	Water spread area at MWL	: 400Ha
9.	Water spread area at FRL	: 390Ha

SPILLWAY

1.	Location	: Right flank
2.	Type of spillway	: Ungated ogee
3.	Total length	: 71.63n
4.	Crest level	: 113.1m
5.	Spillway discharge capacity	:1494cumec
6.	Height from deepest foundation upto crest	:5.33m

4.1 : Input data

In the present hypothetical dam break study the worst possible scenario is considered. It considered maximum size of breach opening, time of failure and Manning's roughness coefficient in the analysis for deriving maximum possible amount of dam break flood peak discharge and corresponding stages in the downstream reach of the valley. Based on the experience of Machhu11 dam failure (Chandra & Perumal, 1985), the breach parameter values for the worst possible flow phenomenon have been choosen. The final breach elevation is taken as the river bed levels at the dam site and breach length is assumed to be of 136.25mt. that corresponds approximately to the river width at the bed level. The time of failure generally taken as 30 minutes (Palaniappan, Mishra & Mohapatra, 1999). Manning's roughness coefficient of 0.035 for the riverbed is considered for the dam break analysis. It is worth mentioning that since the dam break analysis primarily pertains to the over bank flood, the bed roughness does not play a significant role in the description of depth-wave characteristics whereas flood plains do. Therefore, a higher value of 0.05 for Manning's roughness is assumed for flood plains for producing higher depths of flow at various locations in the river valley.

The dam break is assumed to take place by overtopping failure and the initial water elevation in the reservoir behind the dam is taken as 120.7 mt that corresponds to the top of the dam. Based on the hypothesis, the rising parts of the design flood impinged the reservoir to such a level that this volume was sufficient to take reservoir level to the top of the dam and cause over topping. The receding part of inflow hydrograph entering into the reservoir to augment dam break flood is presented in Fig. 3 (DSO,1986). Under the circumstances of dam failure, the spillway is assumed to be functioning at it's maximum capacity, i.e. spillway design discharge. Area elevation curve of the reservoir used in hypothetical dam break analysis is shown in Fig. 4.

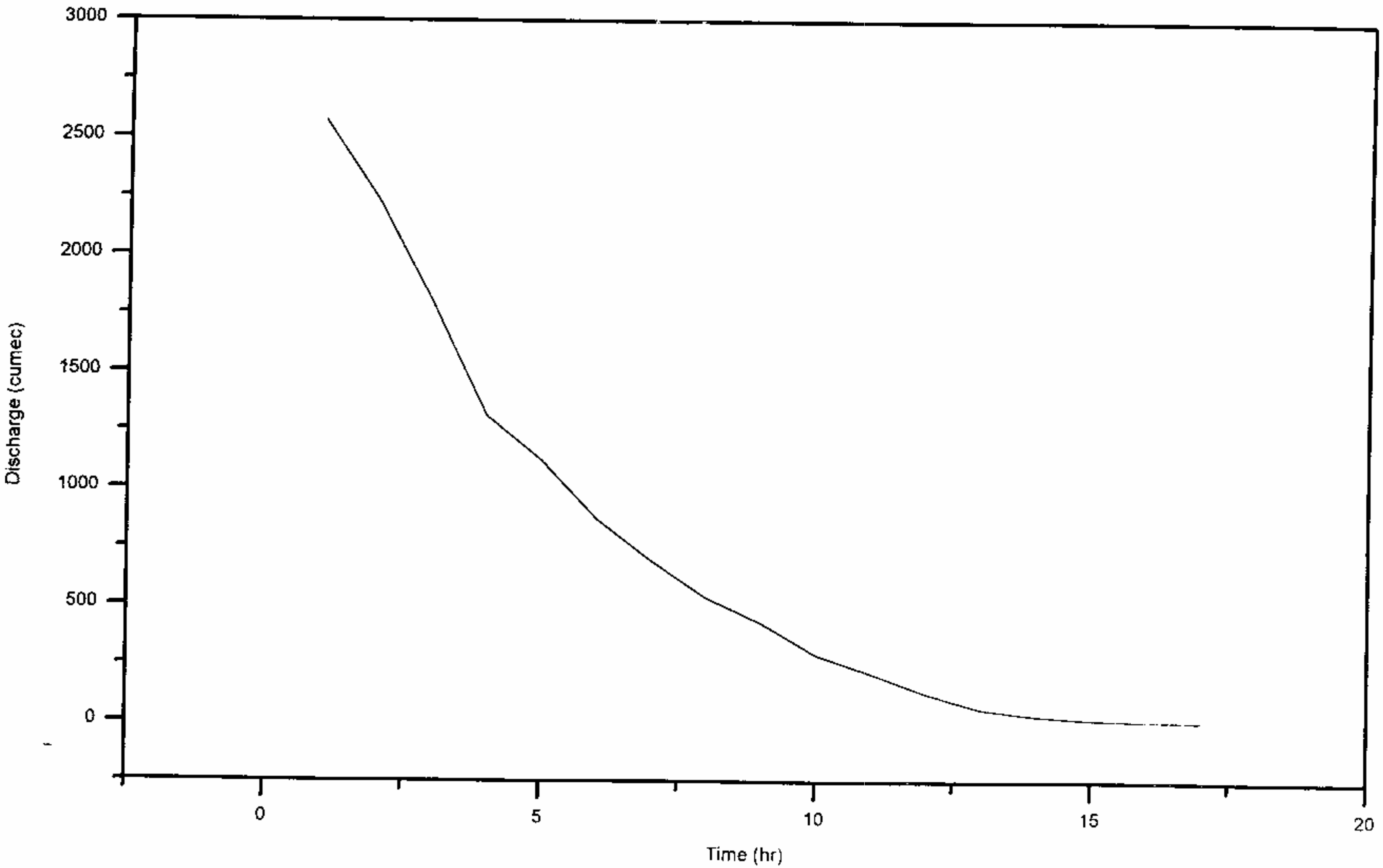

Fig 3 Inflow hydrograph used for dam break flood

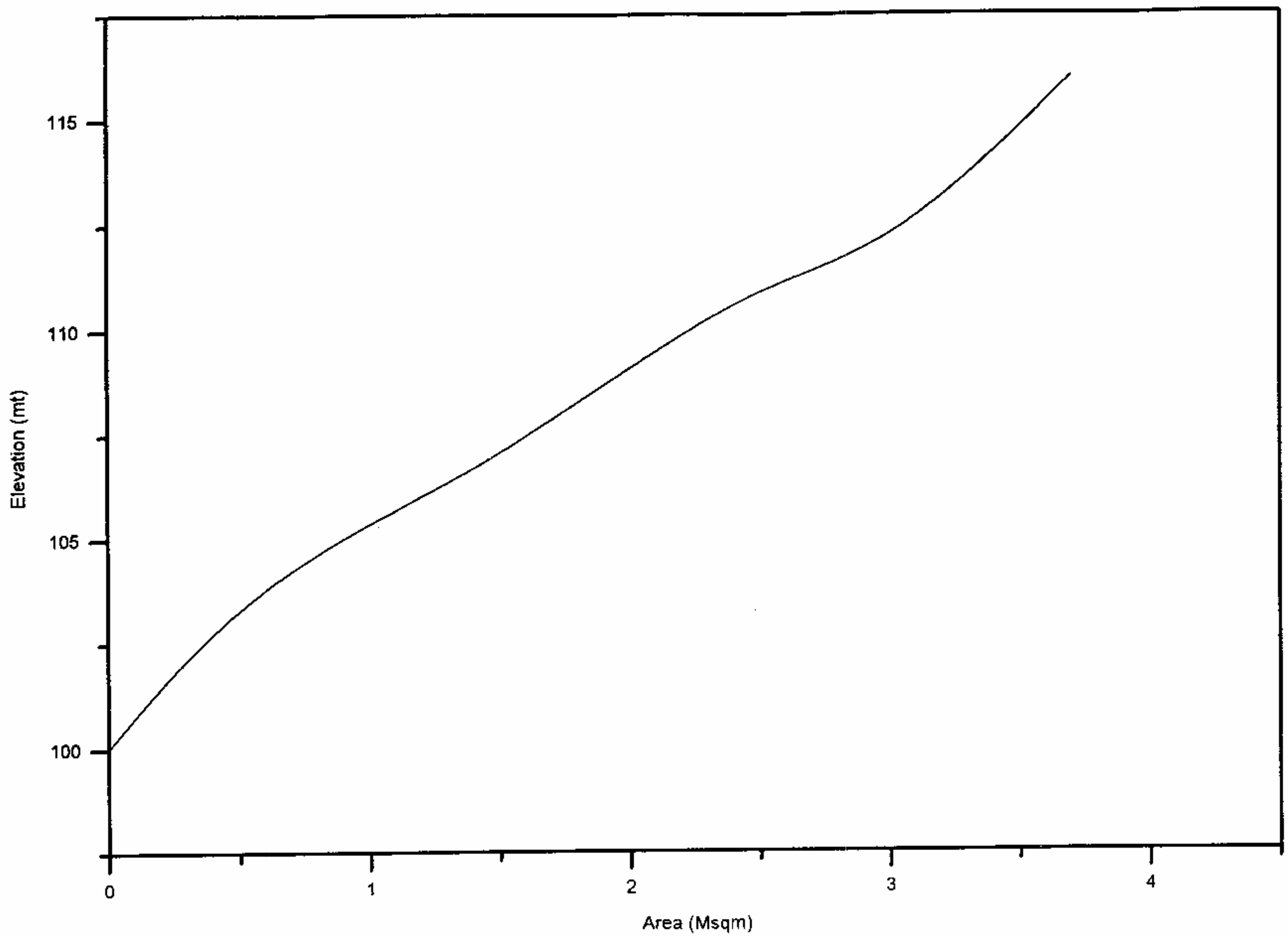

Fig 4 Area Vs. Elevation curve for the Reservoir

Cross-sectional details, channel width at different elevations at various downstream locations are available and used in DAMBRK model. Total six cross-sections at 0, 4.51, 9.09, 20.11, 23.17, 28.43 Km downstream from the dam are used for the analysis. The input data used in the computer program are presented in Appendix-1.

4.2 : Dam break flood & routing computation

In the NWS DAMBRK program, above described data was used as input file and the outflow hydrograph at different location are computed and are presented in Fig. 5. The computed peak discharge and time to peak discharge for dam site and 4.51, 9.09, 20.11, 23.17, 28.43 Km downstream of the dam are presented in the Table-1. From the table it is observed that as the flood wave propagates downstream, the peak value of discharge decreases and travel time increases. The shape of the hydrograph goes on becoming smooth and smoother as the flood wave propagates farther downstream.

From the Table-1& Fig. 5, it is observed that outflow peak over the spillway is 1776.4 cumec, the dam break flood reaches a peak discharge 4402.9 cumec at the dam site and 3359.1 cumec at 28.43 km. from the dam at 0.5hr and 4hr after the dam break. respectively

Table 1: Peak discharge and time to peak of outflow hydrographs

Location (km) d/s of dam	Peak discharge(cumec)	Time to peak discharge(hr)
0.0	4402.9	0.5
4.51	4305.9	0.7
9.09	4066.8	1.2
20.11	3681.2	2.6
23.17	3509.9	3.2
28.43	3359.1	4.0

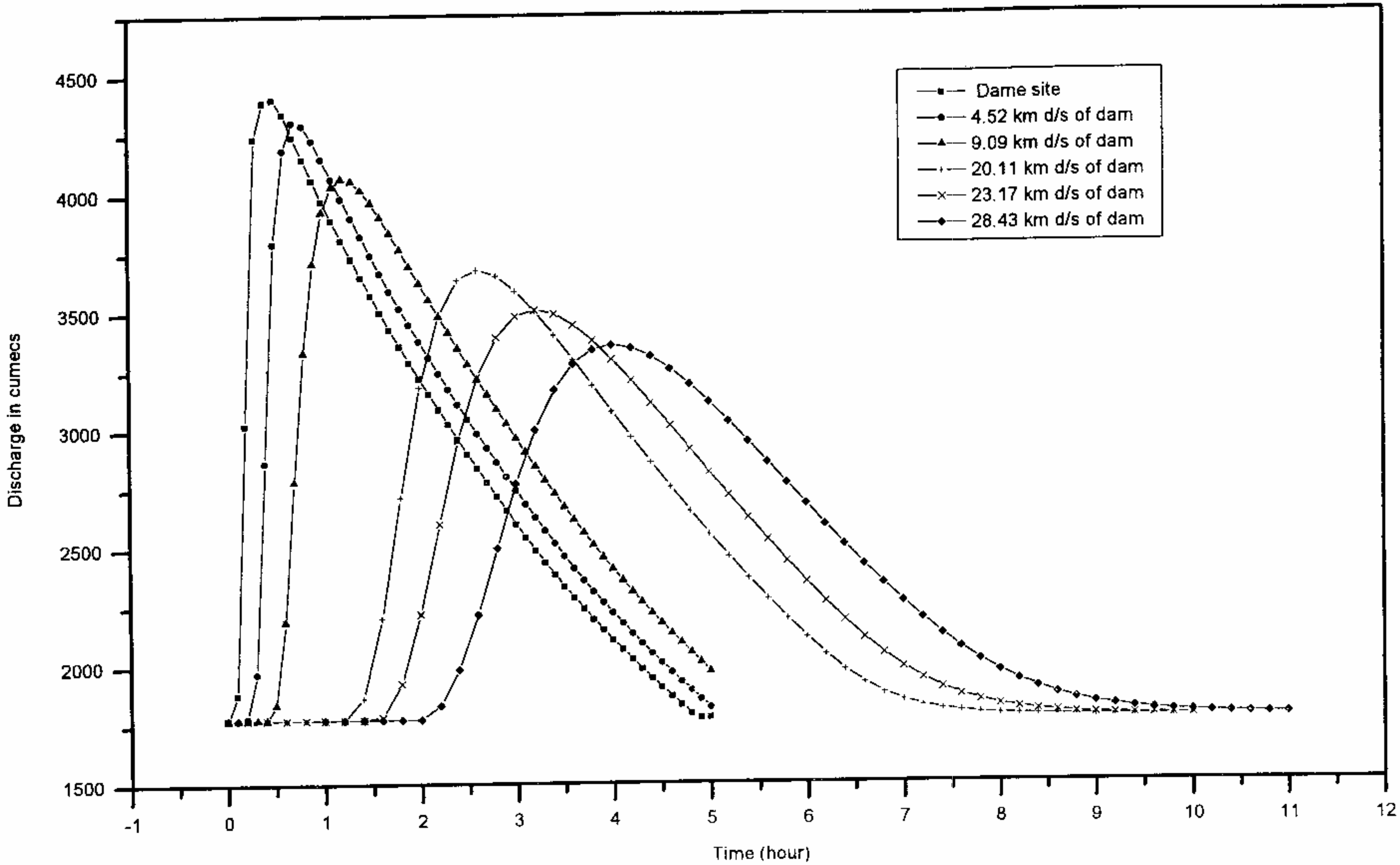

Fig 5 Computed outflow hydrographs at different locations

4.3 : Sensitivity analysis

A sensitivity analysis is carried out to study the effect of the most influencing parameter such as breach length, time of breach and bed roughness on the flood wave propagation. The input parameters and their variation for the above cases are presented in Table -2.

Table 2: Variation of input parameters for Sensitivity analysis

Sensityvity analysis-I Effect on breach length	Case- 1(a)	Breach Length- 136.25mt
	Case- 1(b)	Breach Length- 45.72mt
Sensitivity analysis –II Effect on time of breach	Case- 2(a)	Time of breach- 30 min
	Case- 2(b)	Time of breach- 60 min
	Case- 2(c)	Time of breach- 12 min
Sensitivity analysis –III Effect on roughness	Case- 3(a)	River bed roughness - 0.05 Flood plain roughness- 0.07
	Csae- 3(b)	River bed roughness - 0.06 Flood plain roughness- 0.08

4.3.1 : Effect on breach length

Three values of breach length 136.25, 45.72 mt. were taken for sensitivity analysis, in order to study the effect of breach length on dam break flow. The comparative result for maximum water surface elevation for different breach length is presented in Fig. 6. From the analysis it is observed that the variation in breach length from 45.72 mt. to 136.25 mt. result in increase of peak discharge and maximum water surface elevation.

4.3.2 : Effect on time of failure

The sensitivity analysis for the time of failure is presented in Fig. 7. Failure timing are considered as 0.2hr.0.5hr and 1hr for constant values of other parameters. From the fig it is

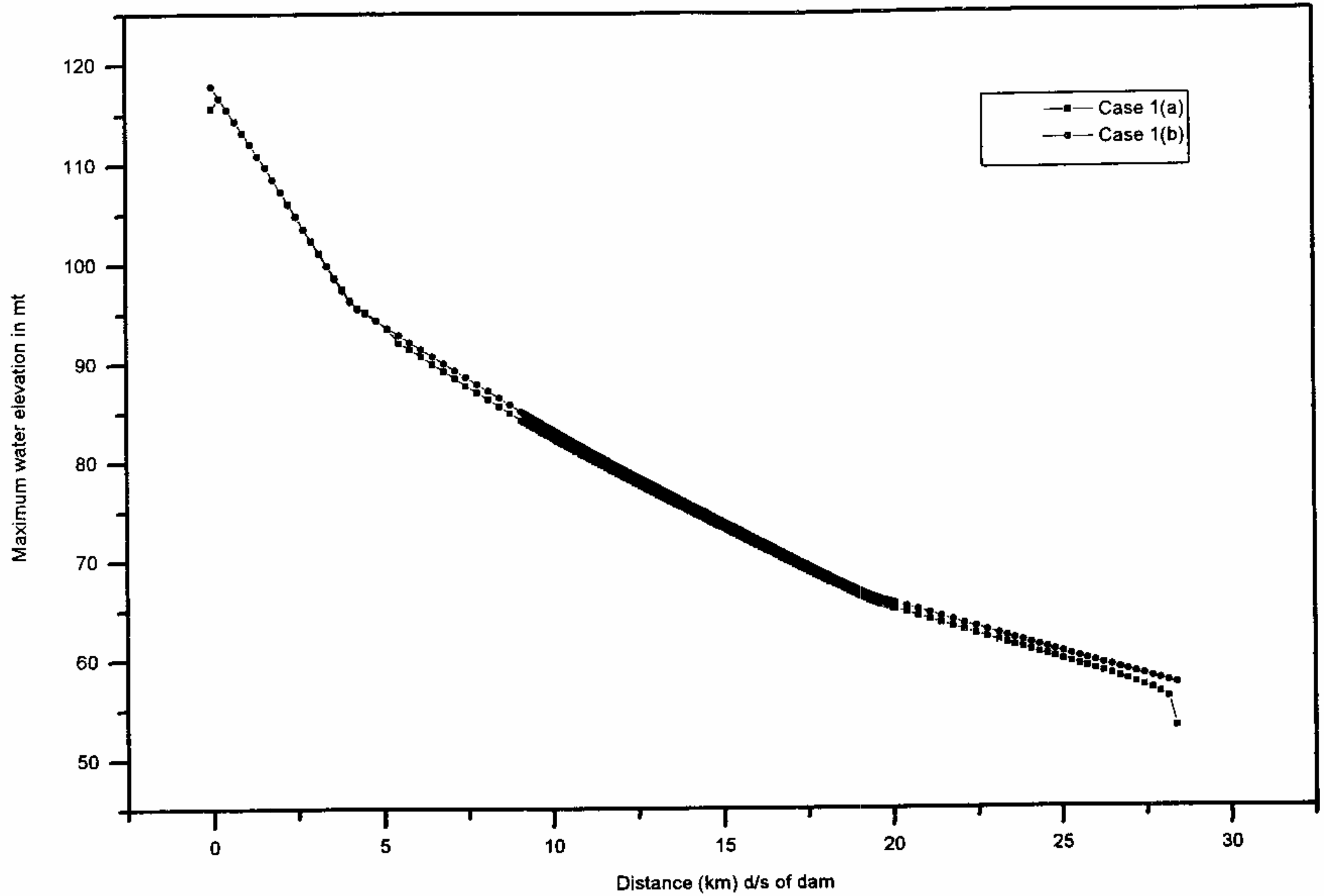

Fig 6 Sensitivity analysis-I: Effect of breach length on maximum water surface elevation

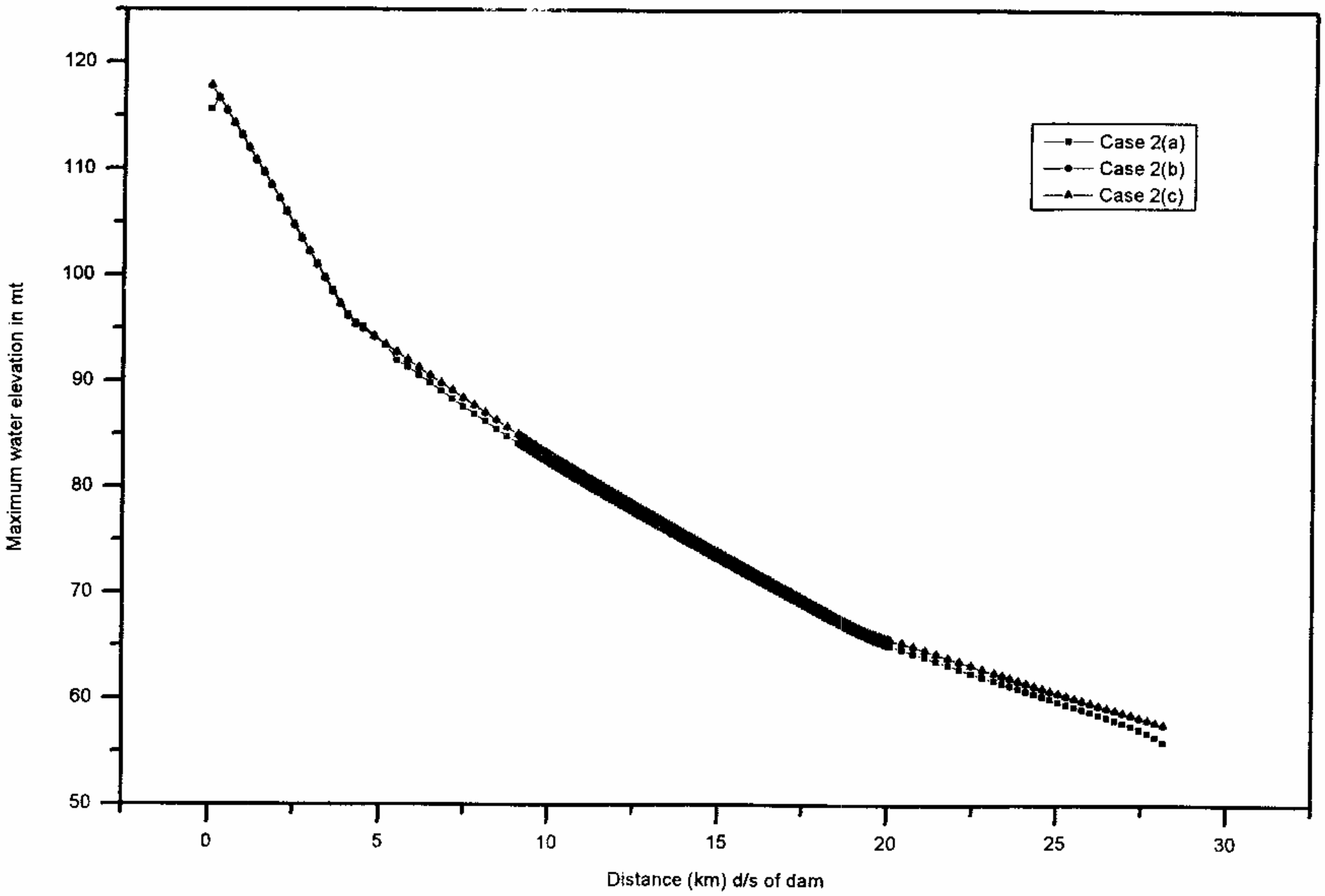

Fig 7 Sensitivity analysis-II: Effect of time of breach on maximum water surface elevation

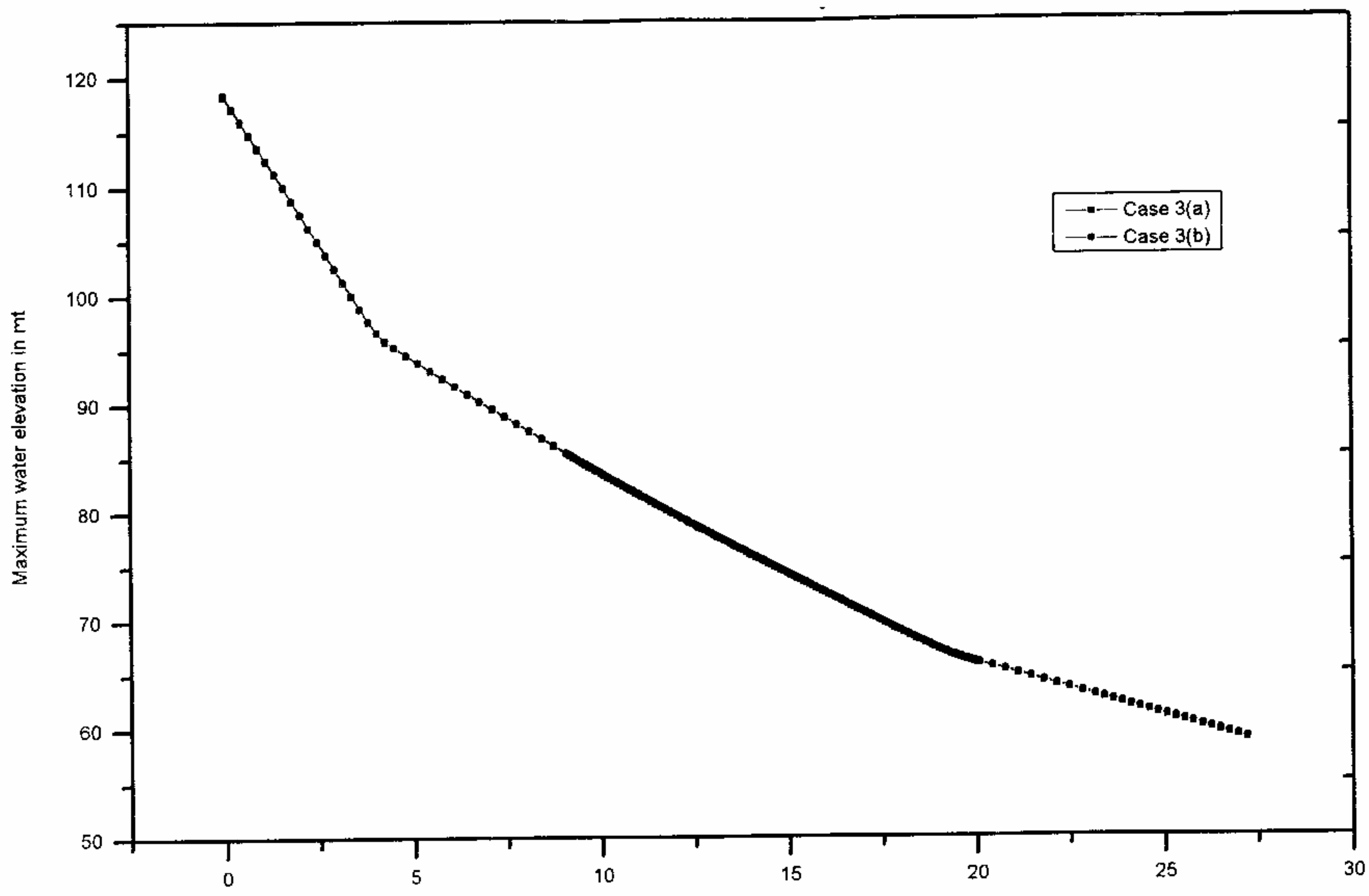

Fig 8 Sensitivity analysis-III: Effect of bed roughness on maximum water surface elevation

understood that variation in maximum water level upto 5.8 km from the dam is negligible, after that as the time of failure increases discharge and maximum water elevation decreases.

4.3.3 : Effect on bed roughness

Sensitivity analysis was carried out considering two sets of roughness data as 0.05, 0.07 and 0.06, 0.08. The maximum water elevation at different location downstream of dam is shown by Fig. 8. From the figure it is observed that variation in bed roughness has no effect in maximum water elevation and discharge.

In the present study an analysis of dam break flood due to hypothetical dam failure of Ghodahoda project was carried out using a widely used DAMBRK model of National Weather Service, USA. Most of the data required for the model were collected from Dam Safety Organisation, Secha sadan, Bhubaneswar and some of the parameters were reasonably assumed based on reported research work. The following conclusions were made based on present hypothetical dam break study.

- The outflow hydrograph at dam site, and 4.51, 9.09, 20.11, 23.17, 28.43 km downstream of dam were computed due to hypothetical dam failure. The dam break flood peak due to hypothetical dam failure study was about 1.5 times the corresponding peak for no failure case. It was observed that outflow peak of 1776.4 cumec over the spillway propagates a peak discharge of 4402.9 cumec at dam site and 3359.1 cumec at 28.43 km downstream of dam at 0.5 hr and 4 hr after the dam break, respectively.

- In the present study breach length of 136.25 mt, time of breach of 30 minutes and channel bed roughness of 0.035 was assumed. A sensitivity analysis considering variation in different input parameters was carried out and presented in Table-3. It was observed that overall effect on breach length, time of breach and bed roughness on the maximum water level was insignificant.

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APPENDIX-1
INPUT FILE FOR DAMBRK

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GHODAHODA PROJECT
NATIONAL INSTITUTE OF HYDROLOGY,           ROORKEE-247 667 U.P.
       1         0         0         3        17         0         0         1
   370.0      350.       250       200       150       120        50         0
   116.0       112       111     109.1       107       106     103.7       100
    2.74     120.3      2.00     101.0    136.25       0.5      93.1       0.0
  120.31     120.3     113.1        0.    164.88        0.     172.0     228.6
     1.0      16.0
  2575.1    2230.2      1796    1314.5    1122.1     870.7    692.30     534.7
  426.63     292.4     214.5     128.8      63.5      34.4      18.3        11
    9.27
       6         4         6         0         0         0         0         0
       1         2         3         4         5         6
     0.0
   108.9     109.0      114.    120.88
      0.        60        80     140.0
      0.        0.        0.        0.
    4.51
     85.        87        89        90
       0      60.6     120.4    148.60
      0.        0.        0.       0.0
    9.09
    74.2        75        77        78
      0.        40      70.4      90.4
      0.        0.        0.        0.
   20.11
      57      60.5      61.2      61.5
       0      100.     150.6     219.8
       0         0         0         0
   23.17
    53.9        55      56.5      57.5
      0.      80.8     113.4     180.4
       0         0         0         0
   28.43
   50.50     51.00     52.50     53.03
      0.     150.2     200.2     240.4
      0.        0.         0         0
   0.035     0.035     0.035     0.050
   0.035     0.035     0.035     0.050
   0.035     0.035     0.035     0.050
   0.035     0.035     0.035     0.050
   0.035     0.035     0.035     0.050
   0.138       0.2     0.052       0.2      .145
   -0.05       0.1      -0.1       0.1      -0.2
     0.2       0.2       0.2       0.2
     0.0       0.0      -0.0      -0.0
     0.0       0.0       0.0       0.0
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Acknowledgement

We express our deep sense of gratitude to Dr. S. M. Seth, Director, National Institute of Hydrology and Dr K. S. Ramasastri, Scientist 'F' Co- ordinator to conduct this study as a part of the work program for the year 1999-2000 of Deltaic Regional Centre, Kakinada. We are grateful to Dr. S. K. Mishra, Scientist 'E', and Dr P. K. Mohapatra, Scientist 'B' National Institute of Hydrology, Roorkee for valuable suggestion and execution of the study.

We express our sincere gratitude to the following personal for providing the data required for completion of the study.

- Sh B. Nayak, Chief Engineer, Lower Mohanadi Basin, Sech sadan, Bhubaneswar.
- Sh B. B. Singh Samant, Director, OWPA, Sech sadan, Bhubaneswar.
- Sh Ajit K. Rath, Director, Dam safety Organisation, Sech sadan, Bhubaneswar.

We do express our sincere gratitude to the scientists and staff members of Deltaic Regional Centre, Kakinada for providing support in various means throughout the course of the study.

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