CS- 49

APPLICATION OF NWS DAM BREAK PROGRAM USING DATA OF GANDHI SAGAR DAM

NATIONAL INSTITUTE OF HYDROLOGY
JALVIGYAN BHAWAN
ROORKEE - 247 667 (U.P.)

1990-91

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 by Dr. D.L. Fread of National Weather Service is the most accurate with various levels of data availability and less time consuming on high-tech computers.

This study presents the simulation of flood wave formation due to hypothetical failure of Gandhi Sagar Dam in Madhya Pradesh State using dam break model, with more emphasis on the sensitivity of dam failure parameters.

This report entitled: '*Application of NWS Dam Break Program Using Data of Gandhi Sagar Dam*', is part of the work programme of Flood Studies Division of the Institute. The study has been carried out by Shri T. Chandra Mohan, Sc.'B' under the guidance of Dr. S M Seth, Sc.'F'.

Satish Chandra
(Satish Chandra)
Director

CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Gandhi Sagar dam is a straight gravity structure and is located on Chambal River in Madhya Pradesh State. The simulation of dam break flood wave due to a hypothetical failure of Gandhi Sagar Dam using the N.W.S. DAMBRK Model developed by Dr. D L Fread is presented here. The data available for the analysis include the elevation details of spillway and dam, spillway rating table, elevation capacity table for reservoir, P.M.F. hydrograph for the dam, cross-sectional details of various downstream reaches etc. More emphasis is given to analyse the sensitivity of dam break flood wave characteristics (stage and discharge) due to the changes in dam breach parameters (time taken for breach formation, reservoir water level, inflow hydrograph to the reservoir, and size and shape of breach). The option of reservoir storage routing to compute outflow hydrograph from reservoir for different combinations of breach parameters and option of sub-critical dynamic routing for routing this flood hydrograph through the downstream channel is used.

From the various studies, it is seen that changes in time taken for failure and inflow hydrograph to the reservoir have insignificant/negligible effects on the flood wave characteristics. It may be because of very large surface area and storage capacity of the reservoir when compared to the total inflow to reservoir. But it can be seen that, the stage and discharge values of dam break flood wave at different reaches are significantly increasing with the increase in the values of reservoir water level and breach width.

Eventhough the data were rather inadequate for simulating two of the special options of 'DAMBRK' Model, namely; simulation of lateral flow, and downstream dam, an attempt has been made here for the same using some assumed data.

1.0 INTRODUCTION

The dam is one of the important hydraulic structures whose purpose is to store excess water in the reservoir and to release it for irrigation, whenever there is a demand for water by the crops in the surrounding area, due to uneven distribution of rainfall in space and time. It may also be constructed to moderate flood flows through a river and thus to protect the downstream areas from loss of life and property. Other purposes include water supply, hydropower generation, navigation, etc. Usually areas downstream of a dam and adjoining areas are highly cultivated because of high fertility of the flood plains and/or availability of water through canal network, and are consequently thickly populated because of various developmental activities.

In recent years, significant effort has been directed at determining the safety of dams in different countries. One aspect of dam safety is the potential for loss of human life and extensive property loss in downstream flood plain that would result in the event of dam failure. The organizations which are responsible for the safety of dams have to plan for preventive measures so that in the eventuality of dam failures the disaster will not lead to loss of lives of the population living in downstream areas.

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 such 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 public of downstream and prepare strategies for disaster management when there is a failure of dam. The main difficulty in using these mathematical models is that the failure description adopted in the model. Under these circumstances, suitable assumptions with regard to the adjustment of actual failure mode to suit the model failure mode is necessary. Also, for this study the inflow hydrograph has to be simulated using suitable rainfall-runoff models because it is possible that the gauging sites may be washed away due to severe storm associated with the failure.

The DAMBRK Model, which is used in this study is a dynamic routing model, prepared by the U.S. National Weather Service (NWS), after taking into consideration the practical aspects of dam failure condition and subsequent flood wave movement, for the purpose of forecasting downstream flooding with reference to flood inundation information and warning times resulting from dam failure.

This report presents a study of hypothetical failure of Gandhi Sagar Dam in Madhya Pradesh and resulting flood wave movement. The dam break flood wave characteristics are presented at three different sections downstream of the dam for different failure characteristics. The results of such a study also provide an idea of the sensitivity of the dam break flood wave to the failure characteristics.

2.0 REVIEW

The dam break problem 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, and 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.

More recently, 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 characteristics 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 (1983) 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 which 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 (1981, 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 were 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 uncertainity 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. The applicability of the St.Venant's equations to simulate abrupt waves such as the dam break wave has been demonstrated by Terzidis and Strelkoff (1970).

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.

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; it 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 , ie. 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 for 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- mileages of the cross sections from the dam, a table of top widths (active and inactive), and corresponding elevations at each sections, hydraulic resistance coefficients (Mannings 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, ie. reservoir outflow hydrograph and a looped rating curve based on the Mannings 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 a 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 seperate 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 components.

5. **Land Slide Modelling:** DAMBRK program is capable of simulating the generation of a wave due to land slide into a reservoir. The program assumes that the land slide 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 in cfs/mile.

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. Data preparation for all these cases have been described in the report 'Data Requirement and Data Preparation for DAMBRK Program' (TN-22), of National Institute of Hydrology. Similar studies have already been completed in National Institute of Hydrology;

1. For the actual failure of Machhu dam in Gujarat, and
2. For a hypothetical failure of Dharoi dam in Gujarat

4.0 METHODOLOGY

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

$$I - Q = \frac{ds}{dt} \qquad \text{........(1)}$$

where, I = Reservoir Inflow
Q = Reservoir Outflow
$\frac{ds}{dt}$ = Rate of change of storage volume

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

$$(I+I')/2 - (Q+Q')/2 = \delta s/\delta t \qquad \text{....(2)}$$

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,

$$\delta s = (As + A's)(h - h')/2 \qquad \text{.......(3)}$$

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 which 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 estimate 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:

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

and, conservation of momentum:

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

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;

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:

$$S_f = \frac{n^2 |Q\ |\ Q}{2.21\ A^2\ R^{4/3}} \qquad \ldots\ldots \qquad (6)$$

and

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

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 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 F.P.S system of units for all parameters. ie., elevations in feet, discharge in miles,discharge in cusecs, storage capacity in acre-ft. etc.

5.0 PRESENT STUDY

5.1 Study Area

The 'dambrk' model has been used in this study to simulate dam break flood flow due to hypothetical failure of Gandhi Sagar Dam. This dam is the first of a series of structures constructed on Chambal River which is flowing through Madhya Pradesh and Rajasthan States. The other structures downstream of Gandhi Sagar are Ranaprathap Sagar Dam, Jawahar Sagar Dam and Kota Barrage. Gandhi Sagar Dam is located at latitude of 24 44'N and longitude of 75 33'E, in Mandsaur District of Madhya Pradesh.

This dam is a straight gravity structure 1685 ft. long and 212 ft.(64.62 m) high above the deepest foundations. This is mainly used for irrigation and generation of hydropower. The relevant design aspects and other details are as given below:

Length of spillway	834 ft
Length of power dam section	311 ft
Length of non-overflow section	540 ft
Catchment area upto the site	8700 sq. miles
Design peak flood discharge	1197000 cusecs
Gross reservoir storage capacity	6.28 M. acre ft
Length of reservoir	69 miles
Maximum spillway capacity	0.48 M. cusecs
Surplusing arrangements	10 crest gates 60 ft 28 ft, 9 sluice openings of 10 ft * 25 ft
Power plant capacity	115 MW
Project irrigation potential	273300 hect.
Deepest foundation level	+1115.00 ft
Average bed level of river	+1120.00 ft
Centre line of penstokes	+1205.00 ft
Centre line of sluice opening	+1206.00 ft
Dead storage level	+1250.00 ft
Spillway crest level	+1285.00 ft
FRL	+1312.00 ft
MWL	+1312.00 ft
Level of road on top	+1324.00 ft
Level of top of parapet	+1328.00 ft

Note: 1 ft = 0.3048 m, 1 mile = 1.609 km, 1 sq.mile = 2.59 km^2

1 cusec = 0.0283 cumecs, 1 acrefeet = 1232.75 m^3

5.2 Availability of Data

Almost all data for Gandhi Sagar Dam, reservoir and downstream channel are available. Since this a hypothetical study, the data related to dam failure are to be given and they are treated as variables. The other constant values are as follows:

1. First Data Group - The PMF at Gandhi Sagar Dam site as calculated by the Central Water Commission in their study (8) is taken here as the inflow hydrograph required for reservoir routing at the time of failure and is shown in fig. (1). The reservoir elevation-volume relationship and spillway rating curve have been obtained from the CBIP report on Chambal Project (4).

They are shown in tables 1 and 2 respectively.

Table 1 : Reservoir Elevation - Volume Relationship

Elevation ft .(metres)	Volume 10^6 Acre ft. (10**8 cubic m)
1120 (341.38 m)	0.125 (1.54)
1230 (374.90 m)	0.253 (3.12)
1250 (381.00 m)	0.675 (8.32)
1260 (384.05 m)	1.105 (13.63)
1270 (387.10 m)	1.675 (20.66)
1285 (391.67 m)	2.835 (34.96)
1300 (396.24 m)	4.505 (55.57)
1328 (404.77 m)	9.047 (111.59)

Table 2 : Spillway Rating Table

Head ft. (meters)	Discharge cusecs (cumecs)
0 (0.00)	0 (0.00)
6 (1.83)	27793 (786.55)
12 (3.66)	78621 (2224.97)
18 (5.49)	151197 (4278.88)
24 (7.32)	230199 (6514.63)
30 (9.14)	330829 (9362.46)
36 (10.07)	406054 (11491.33)
42 (12.80)	495269 (14016.12)

Since the spillway rating curve is available, the

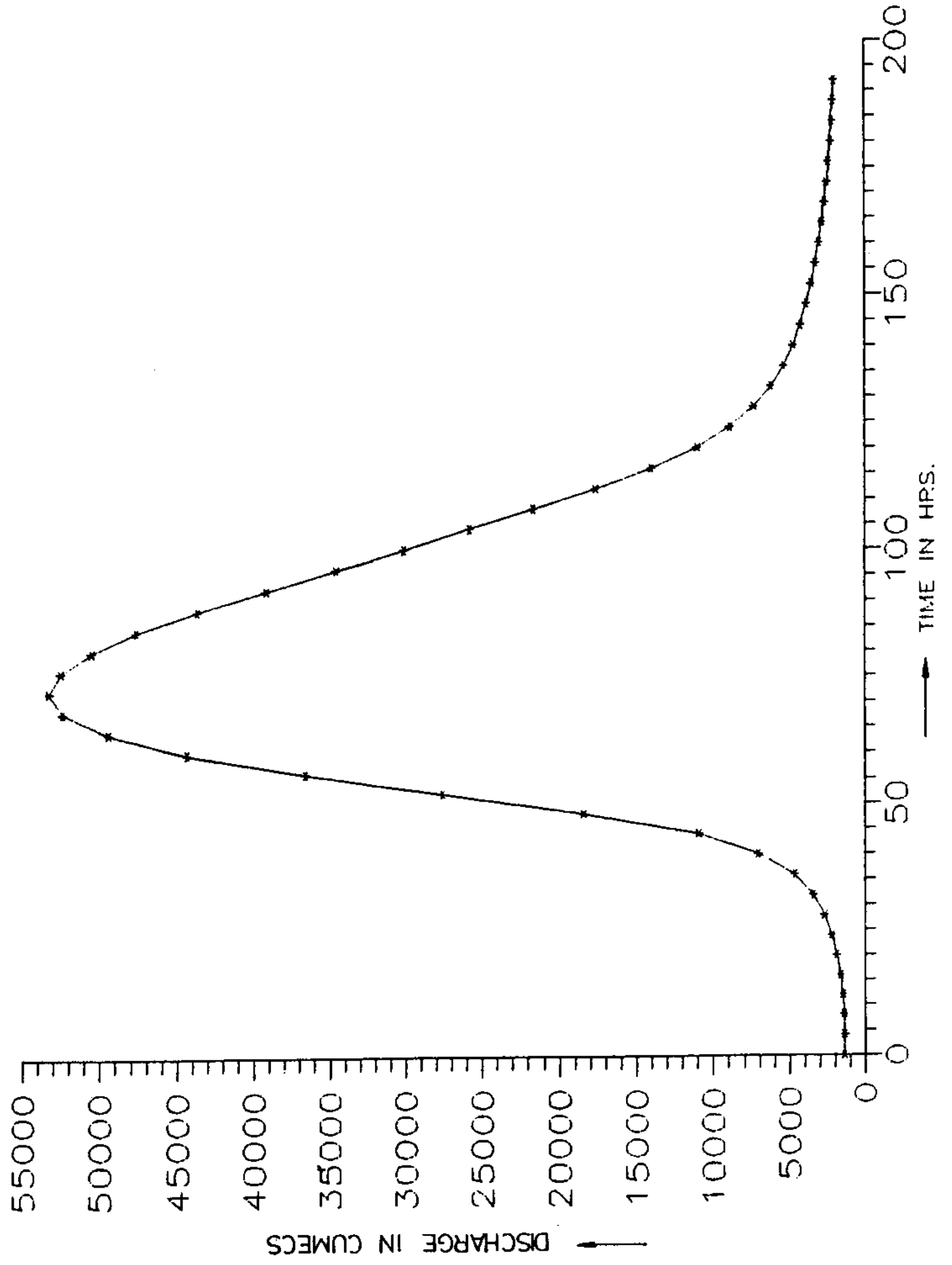

FIG:(1) INFLOW HYDROGRAPH (PMF)

discharge coefficients for uncontrolled spillway and gates could be neglected. Discharge coefficients for the weir flow over the top of dam has been calculated using the formula, CDO = (2.6 to 3.2) * (length of the dam crest less the length of the uncontrolled spillway and gates).

2. Second Data Group - In this study 9 cross section details including roughness coefficients at each sections are available from 0.0 mile to 22.60 mile downstream of the dam.

For cross sections at 3 miles and 5 miles, inactive (off channel storage) widths are also available. They are intended to account for an area where water ponds and/or does not have a significant velocity component in the direction of flow. The expansion and contraction coefficients have been assumed suitably (the contraction coefficient varies between 0.1 and 0.3 and the expansion coefficient varies from -0.5 to -1.0).

3. Variables - In the hypothetical dam break study using 'DAMBRK' model, the user has to supply the expected breach parameters. The typical ranges of breach parameters for different types of structures, as suggested by Dr.D.L.Fread, are given in the table 3.

Table 3 : Typical Ranges of Breach Parameters

	Earth Dam	Concrete Gravity	Concrete Arch
Breach width	(0.5-4) dam height	Some multiple of monolith widths	Total dam width
Side slope of breach	0-1	0	Valley wall slope
Failure Time	0.5-4 hrs	0.1-0.5 hrs	Instantaneous
Reservoir level	1-5 ft.above dam crest	10-50 ft.above dam crest	10-50 ft.above dam crest

Since Gandhi Sagar Dam is a straight gravity structure, a rectangular shaped breach has been assumed for this study. The final level of base of breach has been fixed at +1125 (+342.90 m.) Width of the breach is ranging from 602 ft (183.49 m.) to 946 ft.(288.34 m.) and failure time between 0.1 and 0.5 hrs. The water level in the reservoir at the time of failure has been taken between +1328 ft.(+404.77 m.) and +1348 ft.(+410.87 m.)

The inflow hydrograph to the reservoir has also been taken as ranging from 90% to 110% of PMF, for the purpose of a sensitivity study.

6.0 APPLICATION

The channel geometry below Gandhi Sagar Dam is very irregular. The river is passing through a gorge upto about 5 miles. After this it meets a tributary and the section expands suddenly and the flow takes a clockwise turn of about 120. Again it takes an anticlockwise turn after some distance. Rana Pratap Sagar Dam is situated about 22.6 miles from Gandhi Sagar Dam. Hence the present study is a typical case in which two of the special options of 'dambrk' model, i.e. modelling of lateral inflow and multiple dam, could be used. But the study of these options was beyond the scope of the present study because of non-availability of data for lateral inflow and for the downstream dam.

The 'DAMBRK' model has been applied here mainly by focusing on the sensitivity of the characteristics of flood wave resulting from a hypothetical dam failure, to the assumptions made regarding dam breach variables. Since the dam break flood flow joins with a lateral flow at about 4 to 5 miles from the dam, this analysis has been restricted for sections upto 5 miles downstream of the dam.

Initial values for dam breach variables have been fixed as follows and the dam break flood wave has been simulated and routed through downstream channel. The magnitudes of flood discharge and flood stage and their time of occurrence have been noted at cross section at 0.0 mile, 3.0 mile (4.83 km), and 5.0 mile (8.05 km).

Initial values are:

Inflow Hydrograph	= PMF
Final breach width	= 774 ft.(235.92m)
Time taken for full breach formation	= 0.3 hr.
Reservoir level when breach occurs	= 1338 ft.(407.82m)

Analysis has been repeated by keeping values of three dam breach variables constant and varying the fourth as described below (Table 4) so that the effect of that particular variable on the resulting flood wave could be evaluated and its sensitivity studied:

Table 4 : Breach Parameters for Various Runs

Inflow Hydrograph	Breach width ft (meters)	Reservoir Level ft (meters)	Time for failure (hr)
0.9 PMF	602 (183.49)	1328 (404.77)	0.5
0.95 PMF	688 (209.70)	1333 (406.30)	0.4
1.05 PMF	860 (262.13)	1343 (409.35)	0.2
1.10 PMF	946 (288.34)	1348 (410.870	0.1

Even though the data were rather inadequate for simulating the lateral flow and the downstream dam, an attempt has been made for the same using some assumed data. The catchment area of tributary is about 10% of that of Gandhi Sagar Reservoir. Hence 10% of PMF for the Gandhi Sagar Reservoir has been taken as the lateral inflow which is joining the downstream channel between sections at 3 miles (4.83Km.) and 5 miles.(8.05km.) Level of water at Ranaprathap Sagar (RPS) dam site has been taken as 1157.5 ft (352.81 m.) (FRL of Reservoir) and a very small breach has been assumed for the RPS Dam so as to run the program.

7.0 RESULTS

For studying the sensitivity of each dam break variable on the resulting dam break flood values on the downstream side of the dam, various runs have been made using 'dambrk' model as explained in the previous chapter. The results are given in the Tables 5 to 8 giving the values of flood discharge and flood stage for the hypothetical G.S. dam failure for each of the four dam break variables. The outputs for two special cases, i.e. introduction of lateral inflow and downstream dam, are given in tables 9 and 10.

1. Effect of time taken for the failure - In this study, the dam failure timings are considered as 0.1 hr, 0.2 hr, 0.3 hr, 0.4 hr, and 0.5 hr for constant inflow hydrograph (PMF), breach size 774'(235.92m) and reservoir water level (1338' (407.82m).The results are given in table 5. From these it can be seen that the effect of time for failure is not so significant.

Table 5 : Effect of Time Taken for Failure

I = PMF, Breach Width =235.92m.,Reservoir Level = 407.82m.

Time (hrs)	Stage at (in meters)		
	0 Km.	4.83 Km.	8.05 km
0.1	39.79	23.21	20.74
0.2	39.78	23.21	20.74
0.3	39.77	23.21	20.73
0.4	39.76	23.20	20.73
0.5	39.76	23.20	20.73

Time (hrs)	Discharge at (in 10 ** 5 cumecs) 0 Km	4.83km	8.05kms
0.1	1.31	1.19	1.14
0.2	1.31	1.18	1.14
0.3	1.31	1.18	1.14
0.4	1.30	1.18	1.14
0.5	1.30	1.18	1.14

2. Effect of inflow hydrograph to the reservoir - For the purpose of sensitivity study, five hydrographs were selected (90%, 95%, 100%, 105%, 110% of PMF) with constant time for failure (0.3 hr), reservoir water level 1338 ft.(407.82m) and breach size 774 ft.(235.92m).The outputs of various runs are as shown in Table 6. It is found that the effect of change in inflow hydrograph on dam break flood wave is negligible.

Table 6 : Effect of Inflow Hydrograph

Time = 0.3 hrs, Breach width = 235.92 m., Reservoir level = 407.82 m.

Inflow Hydrograph	Stage at (in meters) 0 Km.	4.83 kms	8.05 kms
0.9 PMF	39.78	23.21	20.73
0.95 PMF	39.78	23.21	20.73
1.0 PMF	39.78	23.21	20.73
1.05 PMF	39.78	23.21	20.73
1.10 PMF	39.79	23.21	20.74

Inflow Hydrograph	Discharge at (in 10 ** 5 cumecs) 0 Km.	4.83 Km.	8.05 Km.
0.9 PMF	1.31	1.19	1.14
0.95 PMF	1.31	1.19	1.14
1.00 PMF	1.31	1.19	1.14
1.05 PMF	1.31	1.19	1.14
1.10 PMF	1.31	1.19	1.14

3. Effect of reservoir water level at the time of failure For this study inflow hydrograph is taken as PMF, size of breach as 774 ft.(235.92 m.) and time for breach formation as 0.3 hr. The reservoir water level is kept at a height of 0 to 6.10m above the top of the dam is 1328 ft., 1333 ft., 1338 ft., 1343 ft. and 1348 ft.(404.77 m), 406.30 m), 407.82 m), 409.35 m) and 410.87 m). The results of various runs are shown in Tables 7, and graphically represented in fig. 2a and 2b. It can be seen that as the water level increases in the reservoir, the dam breach flood wave characteristics are also increasing.

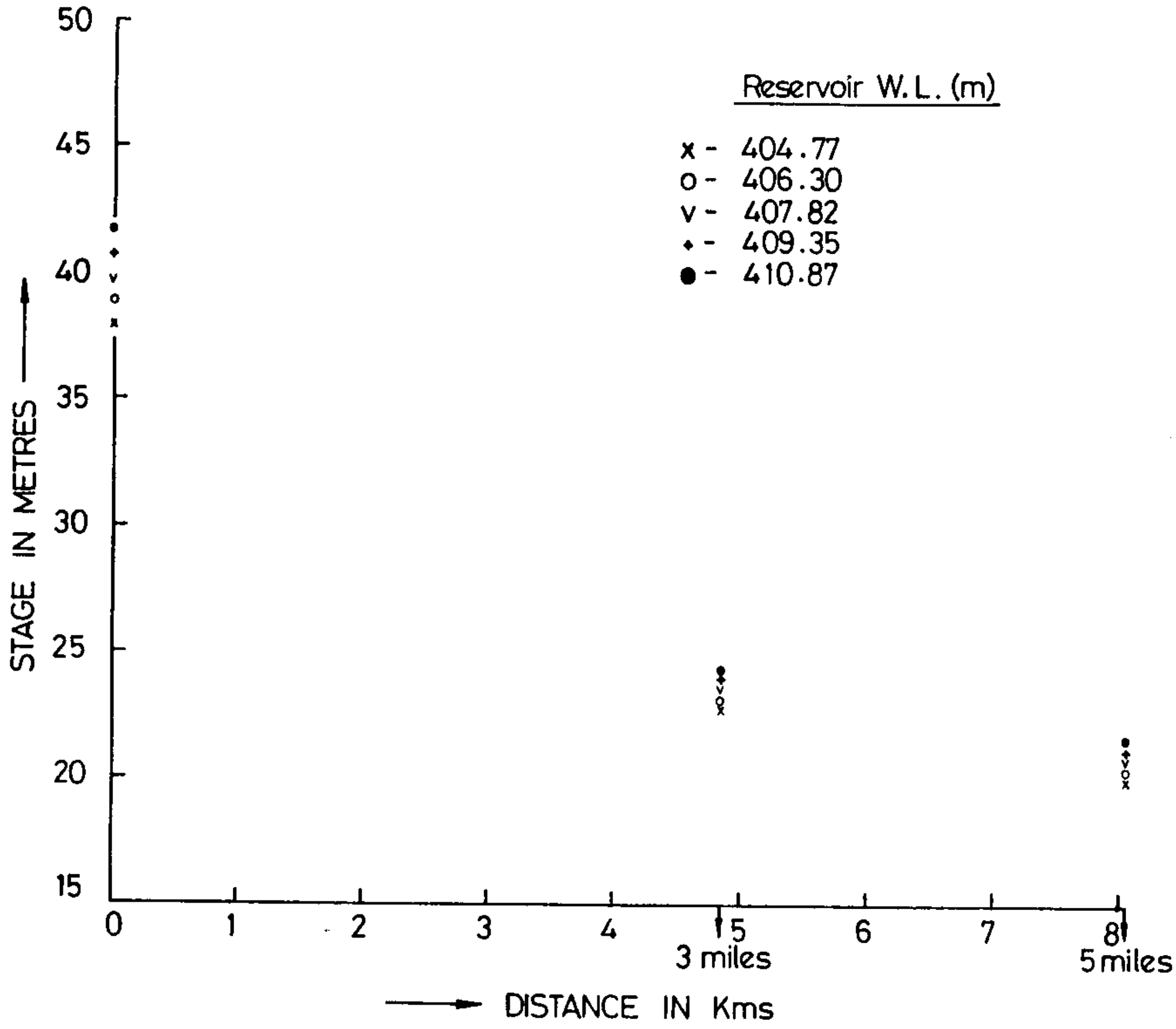

FIG. 2a. EFFECT OF RESERVOIR WATER LEVEL ON FLOOD STAGE

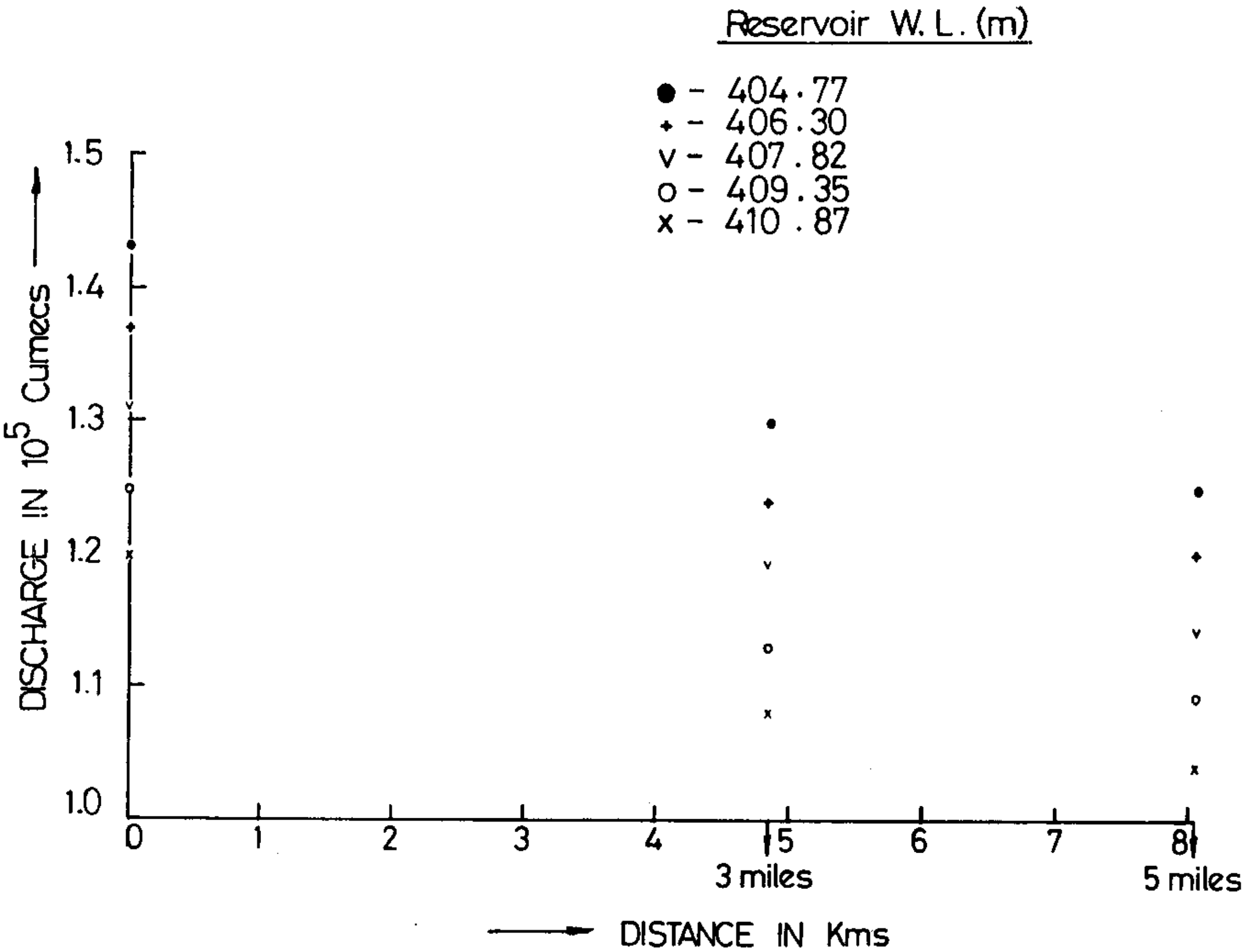

FIG. 2b. EFFECT OF RESERVOIR WATER LEVEL ON FLOOD DISCHARGE

Table 7 : Effect of Reservoir Water Level

I=PMF, Breach Width=235.92 m., Time=0.3 hrs.

Reservoir Water Level (meters)	Stage at (in meters.) 0 Km.	4.83Kms.	8.05 kms.
404.77	38.04	22.74	19.95
406.30	39.02	23.13	20.35
407.82	39.78	23.21	20.73
409.35	40.74	23.46	21.12
410.87	41.75	23.97	21.51

Reservoir Water Level (meters.)	Discharge at(in 10 ** 5 cumecs) 0 Km.	4.83 km.	8.05 Km.
404.77	1.20	1.08	1.04
406.30	1.25	1.13	1.09
407.82	1.31	1.19	1.14
409.35	1.37	1.24	1.20
410.87	1.43	1.30	1.25

4) Effect of size of breach - Five values of final breach widths are taken for sensitivity study for constant inflow hydrograph (PMF), reservoir water level 1338 ft.(407.82 m) and time taken for failure (0.3 hr). These values are 602ft., 688 ft., 774 ft., 860 ft. and 946 ft.(183.49 m, 209.70 m, 235.92 m, 262.13 m, and 288.34 m.) The results are given in Tables 8 and graphically represented in Fig. 3a and 3b. It can be seen that when the breach size changes from 602 ft., (183.49 m.) 946 ft.(28.34 m), the flood peak increases by 129.54 m. and flood discharge increases by 8.50 x 10 ** 3 cumecs. just at the downstream of dam.

Table 8 : Effect of Breach Width

I = PMF, Time = 0.3 hrs, Reservoir Level = 407.82meters.

Breach Width (meters)	Stage at (in meters) 0 Km	4.83 Kms.	8.05Kms.
183.49	38.97	22.96	20.41
209.70	39.43	23.11	20.60
235.92	39.78	23.21	20.73
262.13	40.05	23.29	20.84
288.34	40.26	23.34	20.92

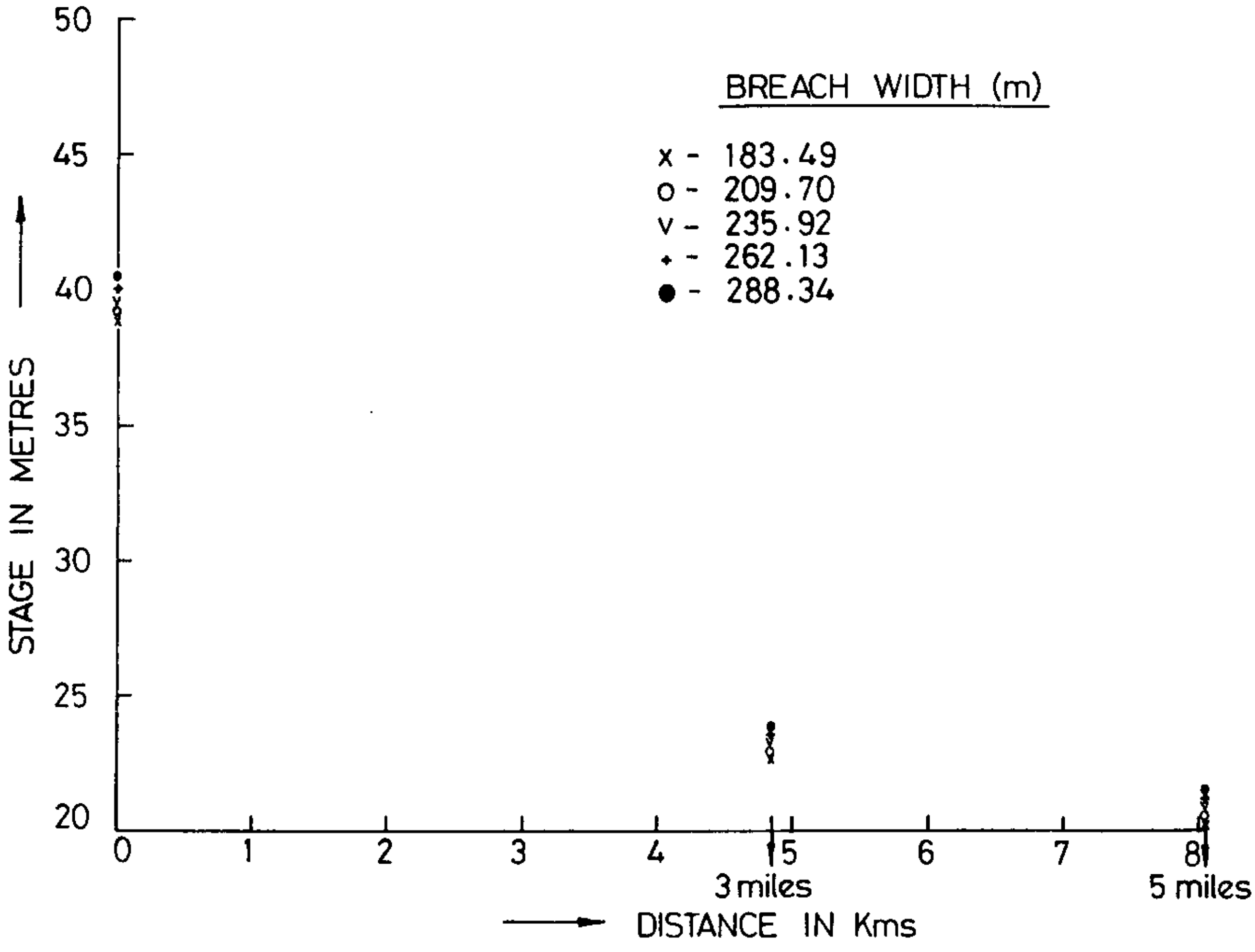

FIG. 3a EFFECT OF BREACH WIDTH ON FLOOD STAGE

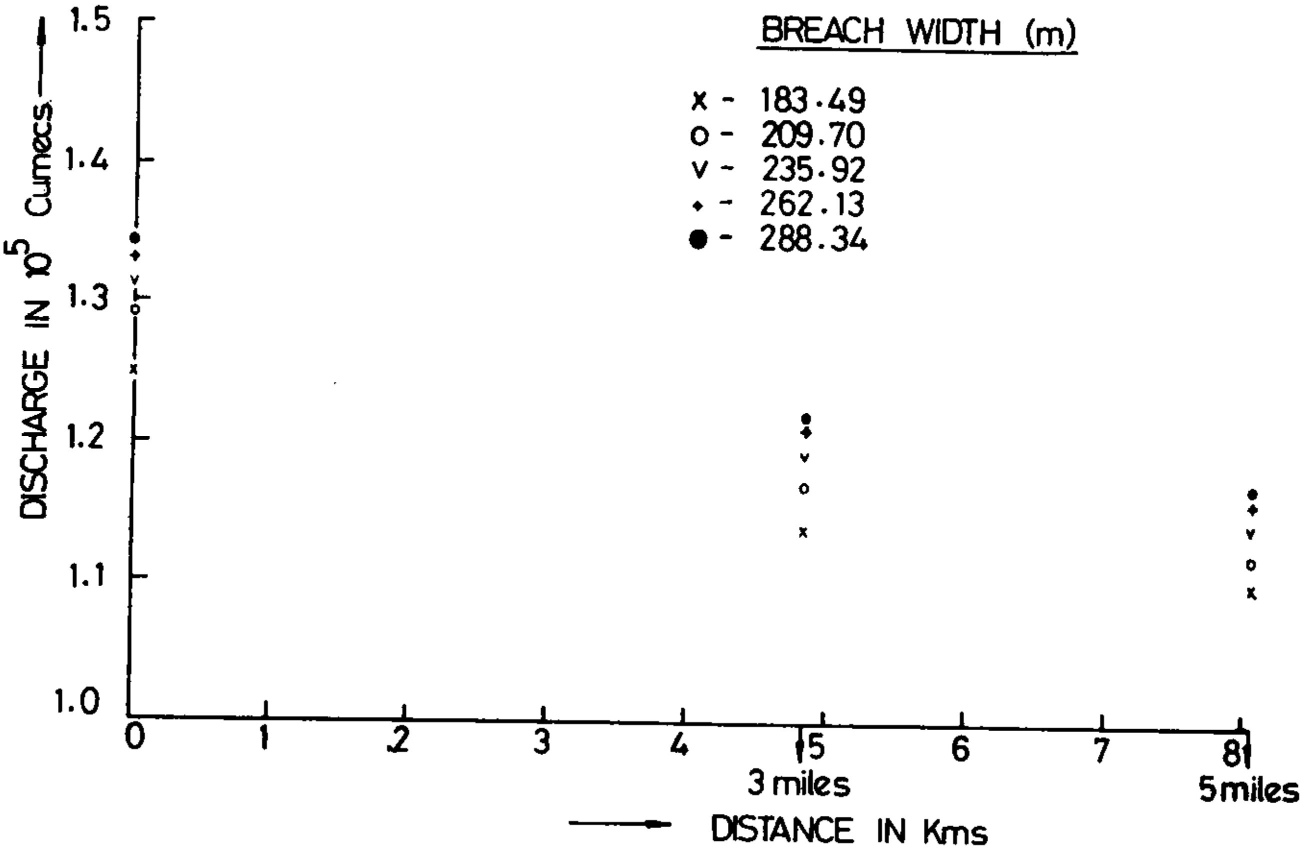

FIG. 3b. EFFECT OF BREACH WIDTH ON FLOOD DISCHARGE

Breach Width (meters.)	Discharge at (in 10 ** 5 Cumecs.) 0 Km	4.83 Km	8.05 Km
183.49	1.25	1.14	1.10
209.70	1.29	1.17	1.12
235.92	1.31	1.19	1.14
262.13	1.33	1.21	1.16
288.34	1.34	1.22	1.17

Assuming a lateral flow (10% of PMF) is joining the downstream channel after the second cross section, the characteristics of resulting flood wave have been obtained using one of the special option of 'dambrk' model. The result is given in the Table 9. A comparison of this result is done with the standard dam break flood characteristics and is shown in Fig. 4a and 4b.

The results of the run made by using the special option of stimulating a downstream dam the channel below Gandhisagar Dam is shown in Table 10. A comparison of characteristics of flood wave resulting from this case is done with the standard dam break flood characteristics and is shown in Fig. 5a and 5b.

Table 9 : Effect of Lateral Flow

I = PMF, Breach width = 235.92 m., Reservoir Level 407.82 m.
Time = 0.3 hrs.

Cross Section at (Km)	Stage (ft) (meters.)	Discharge (10 ** 5 Cumecs.)
0	39.79	1.31
4.83	23.91	1.18
8.05	26.30	1.34

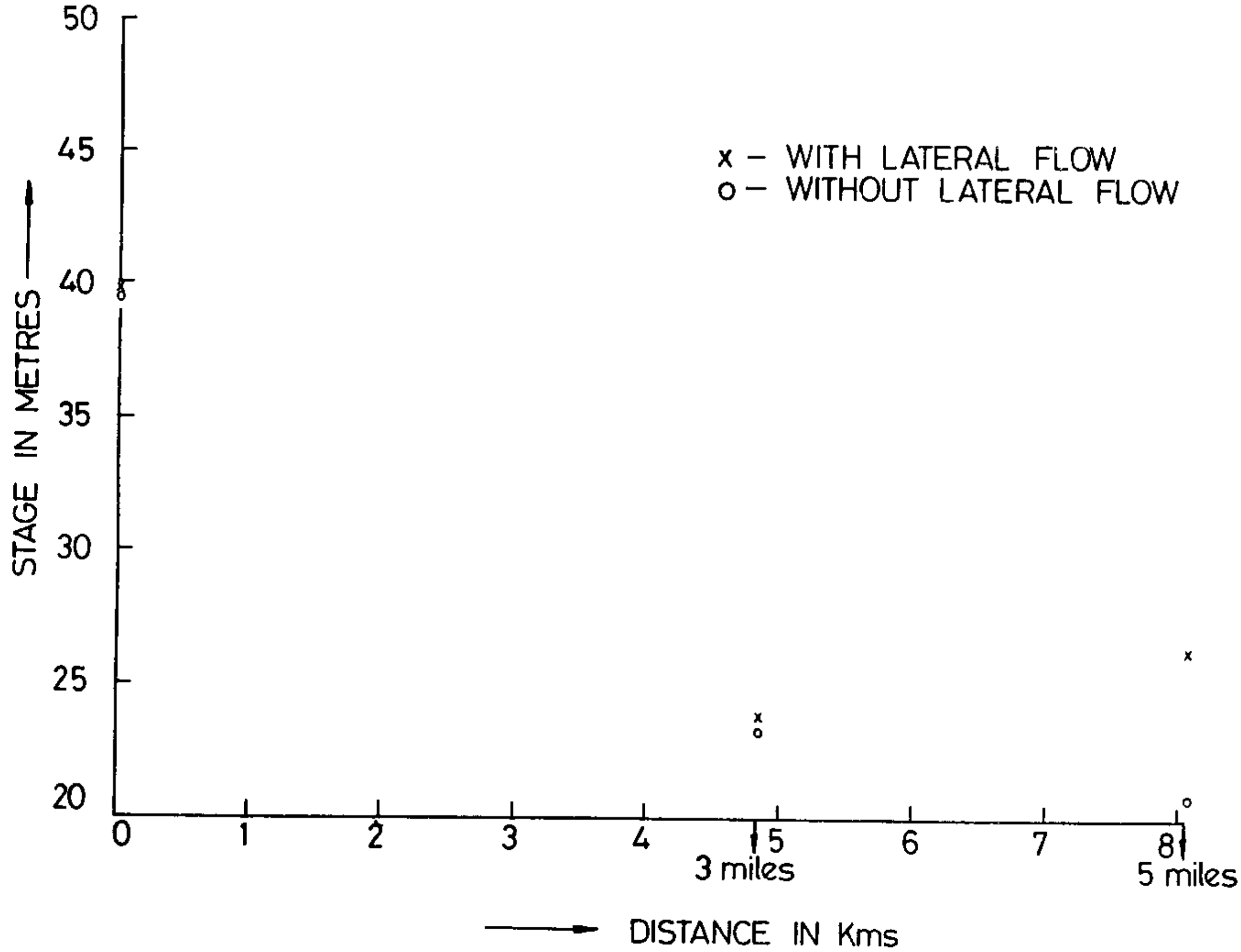

FIG. 4a. EFFECT OF LATERAL FLOW ON FLOOD STAGE

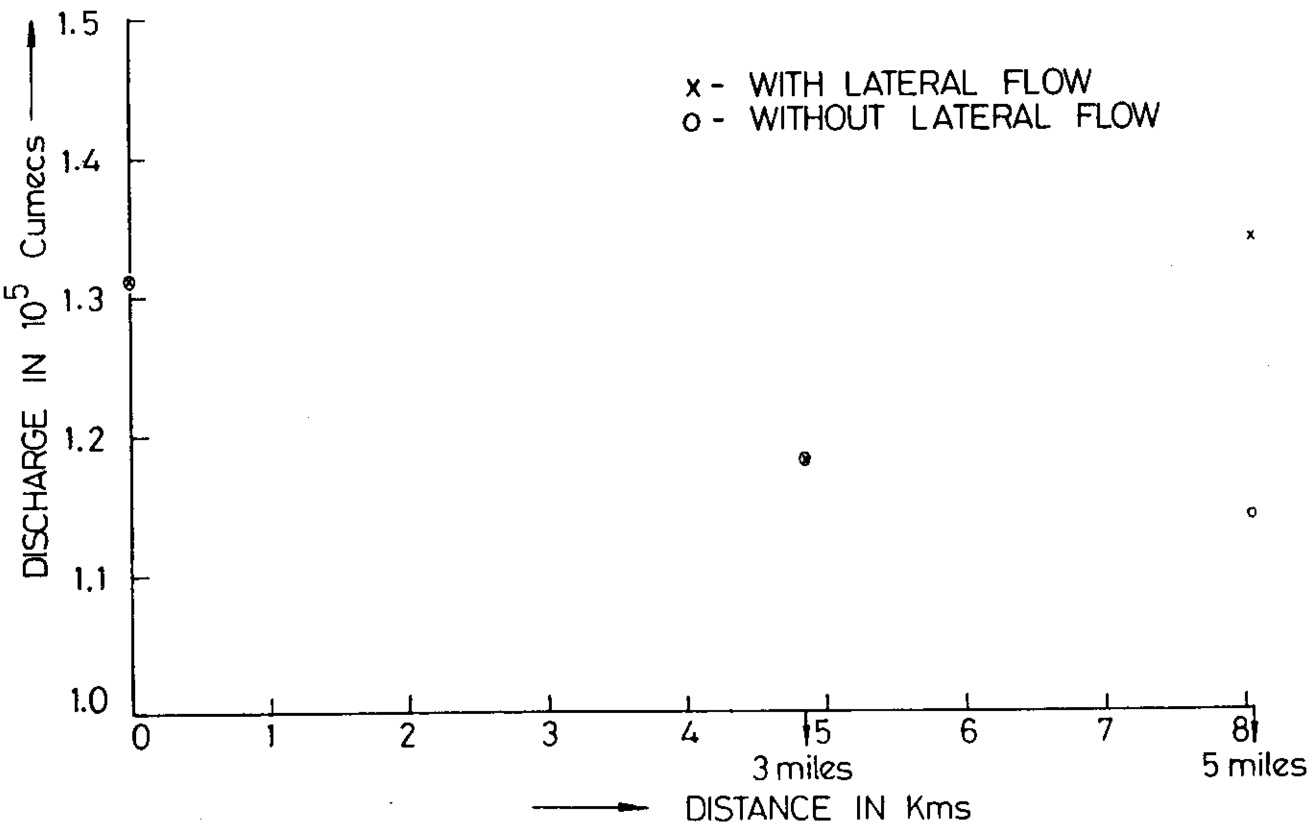

FIG. 4b. EFFECT OF LATERAL FLOW ON FLOOD DISCHARGE

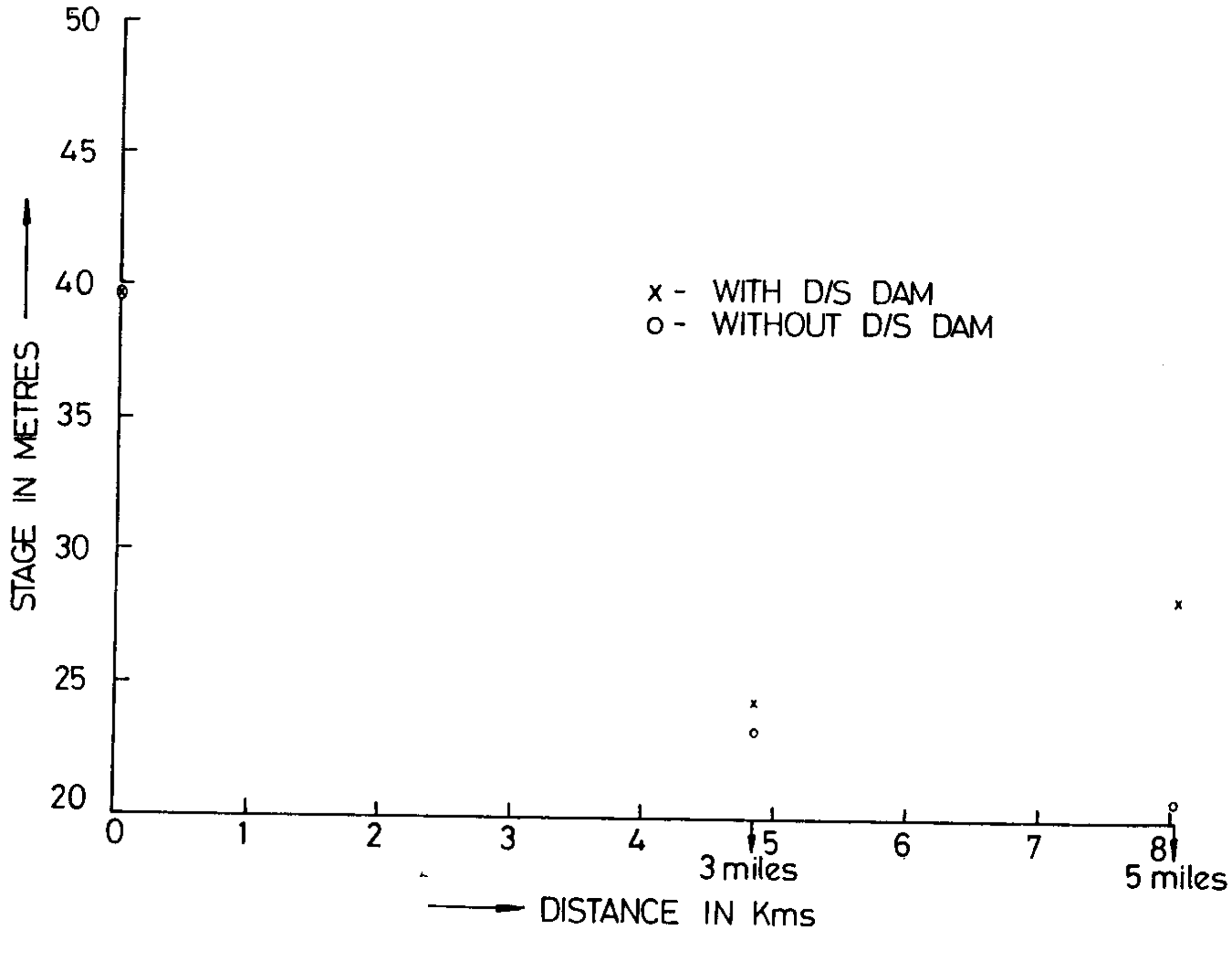

FIG. 5a. EFFECT OF A D/S DAM ON FLOOD STAGE

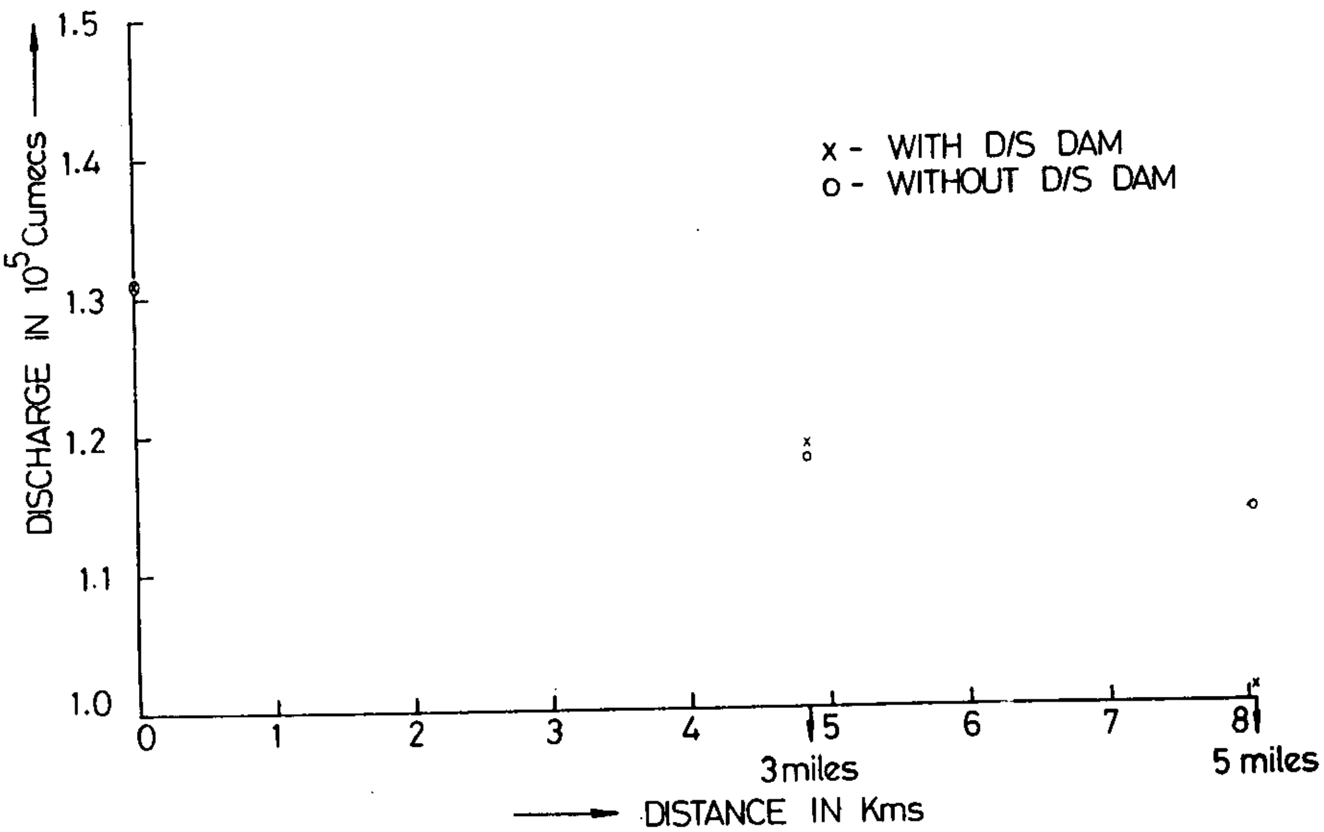

FIG. 5b. EFFECT OF A D/S DAM ON FLOOD DISCHARGE

Table 10 : Effect of Down Stream Dam

I = PMF, Breach width = 235.92 m., Reservoir Level 407.82 m.
Time = 0.3 hrs.

Cross Section at (Km)	Stage (meters.)	Discharge (10 ** 5 cumecs.)
0	39.79	1.31
4.83	24.53	1.19
8.05	28.41	1.01

8.0 CONCLUSIONS

This study simulates the dam break flood wave formation due to a hypothetical failure of Gandhi Sagar Dam in Madhya Pradesh, using NWS 'DAMBRK' model. Most of the assumptions regarding the failure phenomenon are on the same lines as were adopted in the case study of Machhu Dam II failure and hypothetical failure study of Dharoi Dam. Here a sensitivity study has been performed to study the effect of various dam failure parameters on the outflow flood wave due to failure. An effort has also been made to simulate a lateral inflow to the downstream channel and a dam in the downstream channel. The following points can be noted form this study:

1. Due to the large drainage area above the dam and a large reservoir storage, the effect of changes in time taken for breach formations on the dam break flood characteristics is insignificant.

2. Since the reservoir volume (storage capacity) is almost equal to the total volume of inflow hydrograph (PMF), there is no effect of slight changes in inflow hydrograph on the outflowing flood characteristics.

3. The flood wave characteristics are affected by the changes in reservoir water level at the time of failure and the size of breach. Flood stage and discharge values are increasing with the increase in magnitudes of reservoir water level and breach size.

4. The special options of 'dambrk' model can be utilized accurately if proper date in available.

The results obtained by this study may be used for the purpose of flood warning and flood inundation mapping. The variation of model breach flow from actual breach flow due to the variation of breach shape and size and effect of resistance and contraction expansion coefficients could be assumed to be somewhat insignificant in comparison to the magnitudes of resulting flow from breached dam.

REFERENCES

1. Akan, A.O. and Yen, B.C., 'Diffusion-Wave Flood Routing in Channel Networks', ASCE Journal of the Hydraulic Division, HY 6, 1981.

2. Bruce Hunt, 'Asymptotic Solution for Dam Break Problems', ASCE Journal of Hydraulic Division, Vol.108 (1/4), Page 115, 1982.

3. Bruce Hunt, 'Dam Break Solution', ASCE Journal of Hydraulic Division, Vol.110 (HY5), Page 675, 1984.

4. Central Board of Irrigation & Power Publication, 'History of Chambal', Vol. 1 & 2, 1987.

5. Central Board of Irrigation & Power Publication, 'Major Dams in India', 1979.

6. Chang-Lung Chen and Ambruster, J.T., 'Dam Break Wave Model Formulation and Verification', ASCE Journal of Hydraulic Division, Vol. 106 (HY5), 1980.

7. Chang-Lung Chen, 'Laboratory Verification of a Dam Break Flood Model', ASCE Journal of Hydraulic Division, Vol.106(4), 1980.

8. Central Water Commission Report, 'Design Review and Dam Break Studies (Phase 2) for Gandhi Sagar and Ranapratap Sagar Dam', 1990.

9. Dressler, R.F., 'Comparison of Theories and Experiments for the Hydraulic Dam Break Wave', IASH No 38, 1984.

10. Fread, D.L., 'DAMBRK': The NWS Dam Break Flood Forecasting Model', Office of Hydrology, National Weather Services, Maryland, 1989.

11. Katapodes, N.D., and Schamber, D.R., 'Application of Dam Break Flood Wave Models', ASCE Journal of Hydraulic Division, Vol.109 (5/8), 1982.

12. Land, L.F., 'Evaluation of Selected Dam Break Flood Wave Models by Using Field Data', USGS, Water Resources Investigations, 1980.

13. MacDonald, T.C. and Monopolis, J.L., 'Breaking Characteristics of Dam Failures', ASCE Journal of Hydraulic Division, Vol.110 (HY5), 1984.

14. National Institute of Hydrology Report (CS-16), 'Dam Break Analysis for Machhu Dam II', 1985-86.

15. National Institute of Hydrology Report (TN-22), 'Data Requirements and Data Preparation for DAMBRK Program', 1985-86.

16. Sakkas, J.G., and Strelkoff, T., 'Dam Break Flood in a Prismatic Dry Channel', ASCE Journal of Hydraulic Division, Vol.99 (HY12), 1973.

17. Schamber, D.R. and Katopodes, N.D., 'One Dimensional Model for Partially Breached Dam', ASCE Journal of Hydraulic Division, Vol.110 (HY5), 1984.

18. Singh, K.P. and Snorrason, A., 'Sensitivity of Outflow Peaks and Flood Stages to the Selection of Dam Break Parameter and Simulation Models', Journal of Hydrology, 68 (1/4), 1984.

19. Users Manual, 'DAMBRK', The N.W.S. Dam Break Flood Forecasting Model', 1984.

20. Wurbs, R.A., 'Dam Breach Flood Wave Models', ASCE Journal of Hydraulic Division, Vol.113, 1987.

21. Xanthopouloss and Koutitas, 'Numerical Simulation of a Two Dimensional Flood Wave Propagation Due to Dam Failure', Journal of Hydraulic Research, Vol.14 (4), 1976.

DIRECTOR	....	SATISH CHANDRA
SCIENTISTS	....	T. CHANDRA MOHAN S.M. SETH
OFFICE STAFF	....	PRABHAKARA PANICKER, T.