CS-16

DAM BREAK ANALYSIS FOR MACHHU DAM-II

SATISH CHANDRA

DIRECTOR

STUDY GROUP

M PERUMAL

SATISH CHANDRA

NATIONAL INSTITUTE OF HYDROLOGY

JAL VIGYAN BHAWAN

ROORKEE-247 667 (U.P.)

INDIA

1985 - 86

CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Machhu Dam-II which is located on Machhu river in the state of Gujarat failed on 11th August 1979 due to overtopping and subsequent failure of earthern dam portion leading to the dam break flood formation. This report presents the simulation of dam break flood wave formation due to Machhu dam-II failure using the U.S. National Weather Services DAMBRK model. The data available for the analysis were related to the details of spillway and earthern embankments, breach profile, gate opening conditions and the water levels in the reservoir at the time of failure, the cross-section details of the channel reaches and the highest water level marks reached by the flood wave at these sections. The inflow hydrograph was made available for this study, and the analysis was carried out using the option of reservoir storage routing to compute outflow hydrograph from reservoir and the option of sub-critical dynamic routing for routing this hydrograph through the down-stream reaches. The assumptions made regarding spillway rating table computation, dam breach formation, down stream water level conditions at the time of failure etc. have been explained. The report presents the discussions on the results dealing with the highest peak flood elevations and highest peak discharges along the reach and their comparison with the observed values. It is concluded based on this study that the DAMBRK model is able to simulate the flood wave formation due to Machhu dam-II failure to the extent needed for practical purposes.

1.0 INTRODUCTION

More and more dams have come up or being constructed with the aim of using the available water resources optimally for developmental purposes or for protecting lives and properties from the fury of floods. With the assured water resources facility and flood protection provided by the dam, the encouragement for improving the overall economy of the country has led to various developmental activities in the downstream of the dam resulting in the settlement of large population and properties in the flood plain and adjoint areas. However, in the eventuality of any dam failure, the disaster would be catastrophic had the dams not being existing with flow occupying not only the erstwhile flood plain area but also the adjoining area. Therefore, it is the responsibility of the organisations involved with the safety of the dams, to plan preventive measures so that in the eventuality of dam failure the disaster will be minimum to the extent possible.

One of the preventive measures in avoiding dam failure disaster is by issuing flood warning to the public of downstream when there is a failure of a dam. However, it is quite difficult to conduct analysis and determine the warning time of the dam break flood at the time of disaster. Therefore, pre-determination of the warning time assuming a various hypothetical dam break situations is a needed exercise in dam safety measures. Before attempting a hypothetical analysis of dam failures for various existing dams, it would be appropriate to establish the credibility of the method used for such analysis by simulating the past dam failure scenarios with reference to the flood wave movement downstream of the failed dam. Further a knowledge of the case studies of the dam failures would give an insight in evaluating and reviewing the existing conditions of the dams. With this view, the failure of Machhu dam-II which occurred on 11th August 1979 in western part of Gujarat state, resulting in the loss of 2000 lives has been analysed and presented in this report.

The dam failure study involves the following component steps:

i) Development or identification of the inflow hydrograph to the reservoir at the time of failure,

ii) Routing that hydrograph through the reservoir

iii) Development of the failure condition of the structure

iv) Calculating the outflow hydrograph from the failed structure, and

v) Modelling the movement of the flood wave downstream to determine travel time, maximum water level reached, inundated areas etc.

Generally the case study of dam failures using the mathematical models pose various problems with regard to matching the model assumptions. The difficult problem is concerned with regard to the failure description of the structure as the failure occurred in nature would be different from the failure description adopted in the mathematical model. Under these circumstances, suitable assumptions with regard to the adjustment of actual failure mode to suit the model failure mode is necessary. Besides, the dam failures of overtopping generally occur due to severe storm with high inflow into the reservoir and due to this either the flow measurements were not made or the gauging site was washed away resulting in no information on the inflow hydrograph to the reservoir. Therefore, for the dam failure study the inflow hydrograph is usually simulated using suitable rainfall-runoff models. Also due to failure of the dam, the downstream gauging stations are generally submerged resulting in no information on the downstream hydrographs. Therefore, in many cases, the only available information is the maximum water level marks reached at the time of passing of the flood wave.

Machhu dam-II failure analysis posed all such problems. The purpose of this report is to present some findings in the verification of the reconstituted flood wave resulting from the failure of Machhu dam-II using the U.S. National Weather Services DAMBRK model developed by Dr D L Fread (1984). The methodology adopted in the DAMBRK model for simulating the analysis is briefly described. The report also describes the assumptions made with regard to the description of the failure of the dam to suit the actual failure mode with that of the mode required by the model. Also reported are the field data used either as input in the model or as bench marks in the evaluation of the computed results. The study attempts to match the maximum water level marks recorded during the passage of the flood wave in the downstream of the dam.

2.0 REVIEW

The dam break analysis problem is one of the most fascinating hydraulic problem and the concerned literature is extensive. The first study was carried out by Ritter (1892) 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. Both experimental and theoretical consideration, however, have shown that the neglect of bed resistance invalidates the Ritter solution in a region that starts near the leading edge of the flood wave. Dressler (1952) used a perturbation procedure to obtain a first order correction for resistance effects. Sakkas and Strelkoff (1973), Chen and Armbruster (1980) have used the method of characteristics to obtain numerical solutions for dam break problems on sloping beds. These solutions were for reservoirs of finite length and included the effects of bed resistance. Investigaters of dam break flood waves such as Ritter (1892), Re(1946), Dressler (1954), Stoker (1957), Su and Barnes (1969), and Sakkas and Streelkoff (1973) assumed the breach encompasses the entire dam and that it occurs instantaneously. U.S. Army Corps of Engineers (1960) have recognised the need to assume partial rather than complete breaches; however, they assumed the breach occurred intantaneously. The assumption of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analysing dam break flood waves.

Recognizing this practical aspect Cristofano (1965), Harris and Wagner (1967) incorporated the partial time-dependent breach formation in their dam break models. Cristofano (1965) attempted to model the partial, time-dependent breach formation in earthern dam; however, this procedure requires critical assumptions and specification of unknown critical parameter values. Also Harris and Wagner (1967) used a sediment transport relation to determine the time for breach formation, but this procedure requires specification of breach size and shape in addition to two critical parameters for the sediment transport relation. For reasons of simplicity, generality, wide applicability and the uncertainty in the actual failure mechanism, the DAMBRK model allows to input the failure time interval and the terminal size and shape of the breach. The possible shapes of the breach which can be accompolished by the DAMBRK model are rectangular, triangular and trapezoidle.

As the failure in earthern dams occur due to overtopping, the breach formation is not instantaneous and therefore, the formation of negative wave and its travelling upstream of reservoir does not generally arise. Under such circumstances the reservoir routing may be carried out using hydrologic routing technique. The U.S. Army Crops of Engineers HEC-1 programmes dam break model adopts only storage routing technique for routing of floods through reservoirs, while that of DAMBRK model is capable of adopting either storage routing or dynamic routing methods for routing floods through reservoirs depending on the nature of flood wave movement in reservoirs at the time of failure.

After computing the hydrograph of the reservoir outflow, the extent of and time of occurrence of flooding in the downstream valley is determined 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 through the downstream valley. This method is derived from the original equations developed by St. Venant. The applicability of St. Venant's equations to simulate abrupt waves such as the dam break wave has been demonstrated by Terzidis and Strelkoff (1970) and by Martin and Zovne (1971).

DAMBRK model uses St. Venant's equations for routing dam break floods in channels. But the HEC-1 programme's dam break model (HEC, 1981) uses the storage routing technique for routing dam break floods in the channel reaches. Singh and Snorrason (1983) have found that the ability of DAMBRK model to simulate the maximum water level profiles is better than the HEC dam break model, attributing the reason for the channel routing techniques adopted in the respective models. It may be considered that the DAMBRK model is a versatile model than any of the models developed so far for simulating the dam failure scenario.

3.0 STATEMENT OF THE PROBLEM

The computation of flood wave resulting from a dam breach basically involves two problems which can be considered jointly or separately: (1) the outflow hydrograph from the reservoir; and (2) the routing of the flood wave downstream from the breached dam along the river channel and the flood plain. If breach outflow is independent of downstream conditions, or if their effect can be neglected, the reservoir outflow hydrograph is referred to as the free outflow hydrograph. In this case, the computation of the flood characteristics is divided into two distinct phases: (a) the determination of outflow hydrograph with or without the routing of the negative wave along the reservoir, and (b) the routing of flood wave downstram from the dam breach.

In this study the problem of simulating the failure of Machhu dam-II by computing the free outflow hydrograph from the breached dam using storage routing technique and routing this hydrograph along the downstream channel using dynamic routing technique with the aim of reproducing the maximum water level marks reached during the passage of flood wave is considered. The information regarding inflow hydrograph into the reservoir due to the storm at the time of failure, the structural and the hydraulic characteristics details of the dam, the time of failure, the channel cross sectional details, the maximum water level marks reached in the reservoir at the time of failure and those observed in the downstream reach of the dam due to the passage of flood wave etc. are available for the study.

4.0 DESCRIPTION OF THE STUDY AREA

A brief description of the Machhu dam-II with reference to its location on Machhu river basin, the relevent details of the dam and a brief description of the Machhu dam-II failure event are given herein for the better understanding of the problem under study.

Machhu dam-II is mainly an irrigation project built by the Gujarat Government in the year 1972 in the western part of Gujarat. It is located at the latitude of $22^{o}46'$ North and the longitude of $70^{o}52'$ East. The total catchment area at Machhu dam - II reservoir is 745 sq. miles of which 284 sq.miles have been intercepted by Machhu dam - I project. The important towns below Machhu dam-II are Morvi and Malia, and they are located respectively at 5.125 miles and 23 miles downstream of the Machhu dam-II site. The Machhu river traverses a distance of 36 miles before ending in little Rann of Kutch. An index map showing the flood affected area due to Machhu dam-II failure is shown in figure 1.

The relevent design aspects of the dam are given below:

Type of dam	Masonary spillway with earth dam flanks on either side
Length of the earthern dam on left	7689 ft
Length of the earthern dam on right	4588 ft
Length of non-overflow masonary portion	272 ft
Spillway length	676 ft
Shape of spillway	Ogee
Crest of spillway	RL 168 ft.
Spillway design flood	2,18,330 cusecs

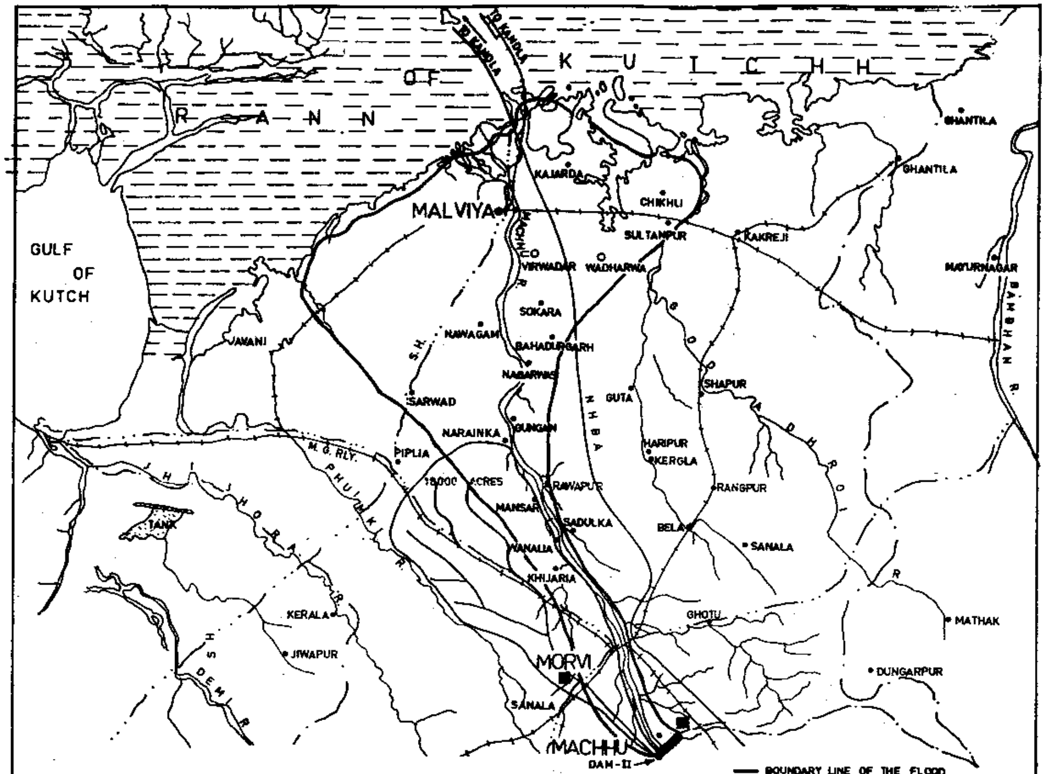

FIG. 1 - INDEX MAP SHOWING FLOOD AFFECTED AREA DUE TO MACHHU DAM-II FAILURE

Details of the radial gates of the spillway	18 gates of 30 ft. long and 20 ft. high
Low Water Level	RL 155 ft.
Dead Storage	7926 ac.ft.
Full Reservoir Level (FRL)	188 ft.
Gross storage	81520 ac.ft.
Flood cushion	1 ft.
High Flood Level	189 ft.
Free Board	8 ft.
Top of the dam	RL 197 ft.

On 10th and 11th August 1979, there were progressively heavy to very heavy rains in the Machhu catchment causing excessive floods in Machhu river which far exceeded the spillway design flood of Machhu dam-II. The water levels in the reservoir rose very fast on 11th August 1979, leading to sustained overtopping of the dam by the flood water for nearly two hours. At 1.30 PM on 11th Aug.1979, the water level had risen to 198.5 ft. i.e. 1.5 ft. above the top of the dam. Due to sustained overtopping, the dam breached on both sides of the spillway over a stretch of about 3600 ft on the left bank and about 1850 ft. on the right bank. However, the masonary dam survived the disastor. As a result of the breach at Machhu dam-II, the flood wave travelled downstream and the towns of Morvi, Malia and a number of villages on the two banks were flooded causing extensive damage to life and property. Transporation network was damaged in these areas as the railway tracks and National Highways were breached due to overtopping by flood waters.

5.0 AVAILABILITY OF DATA

The input data required in FPS system for the National Weather Service's DAMBRK model can be categorised into two groups. The first data group pertains to the dam and inflow hydrograph into the reservoir, and the second group pertains to the routing of the outflow hydrograph through the downstream valley.

5.1 First Data Group

With reference to the data group pertaining to the dam, the information on reservoir elevation-volume relationship, spillway details elevation of bottom and top of dam, elevation of water surface in the reservoir at the beginning of analysis and at the time of failure, breach description data are required. The particulars of the data availability under each of the above mentioned categories are given herein. Most of these information have been taken from the reports in two volumes on the statement of facts and opinions of the Machhu dam-II failure submitted to the Machhu dam-II enquiry commission by the Government of Gujarat in March 1980. These reports will be hereafterwards referred to as report (Vol.I) and report (Vol.II) for the purpose of brevity.

5.1.1 Reservoir elevation-volume relationship

The reservoir elevation-volume relationship of Machhu dam-II has been taken from Annexure: GA-52 of the report (Vol. II) and the information supplied to the model as input is reproduced below:

TABLE 1

RESERVOIR ELEVATION-VOLUME RELATIONSHIP

Sl.No.	Elevation (ft.)	Volume (Ac-ft.)
1	198.5	177915
2	197.0	158402
3	194.0	128318
4	184.0	60026
5	178.0	38092
6	170.0	21359
7	155.0	7926
8	130.0	0

5.1.2 Spillway details

The spillway related information are required for the development of spillway rating table. Also under this category of data, information on the coefficient of uncontrolled weir flow is needed for computing the discharge due to overtopping of dam.

Annexure GA-52 of Report (Vol.II) gives the various gate opening conditions and the corresponding water surface elevations in the reservoir for the purpose of developing spillway rating table. The length of flow over top of the dam due to overtopping has been considered as 10133 ft. and this information has been taken from Annexure GA-4 of Report (Vol. II).

5.1.3 Elevation details

Elevation of top of dam = 197 ft.

Elevation of bottom of dam = 130 ft.

Since the dam failure analysis using DAMBRK model has been considered in this study to begin at the same time when the failure of dam begins, the elevation of initial water surface in the reservoir and the water surface in the reservoir at the time of failure are both one and the same. This value was recorded at 1.30 PM on 11th August 1979 as 198.5 ft.

5.1.4 Breach description

It can be inferred, from Annexure: GA-52 of Report (Vol.II), that the water level was at the elevation of 198.5 ft at 1.30 PM on 11th August 1979 and according to the available statements in Report (Vol.I), that water was seen rushing from left embankment at 2.15 PM and within another 20 minutes from right embankment. Therefore, it may be assumed that the breach started forming around 1.30 PM and fully developed by 2.30 PM, i.e. time for the maximum breach size may be considered as 1.00 hr.

The profile of the breached earthern embankment as traced from Annexure GA:4 of Report (Vol.II) is shown in figure 2 and the required breach description details for the model can be derived from the profile.

5.1.5 Inflow hydrograph

The inflow hydrograph needed for reservoir routing at the time of failure was not recorded, but was simulated using rainfall-runoff model and it is available in a report entitled 'Report on Investigations for Machhu Dam-II (Part II)' submitted by University of Roorkee to the Government of Gujarat in May 1981. The inflow hydrograph used in this study has been reproduced from the said report and it is shown in figure 3.

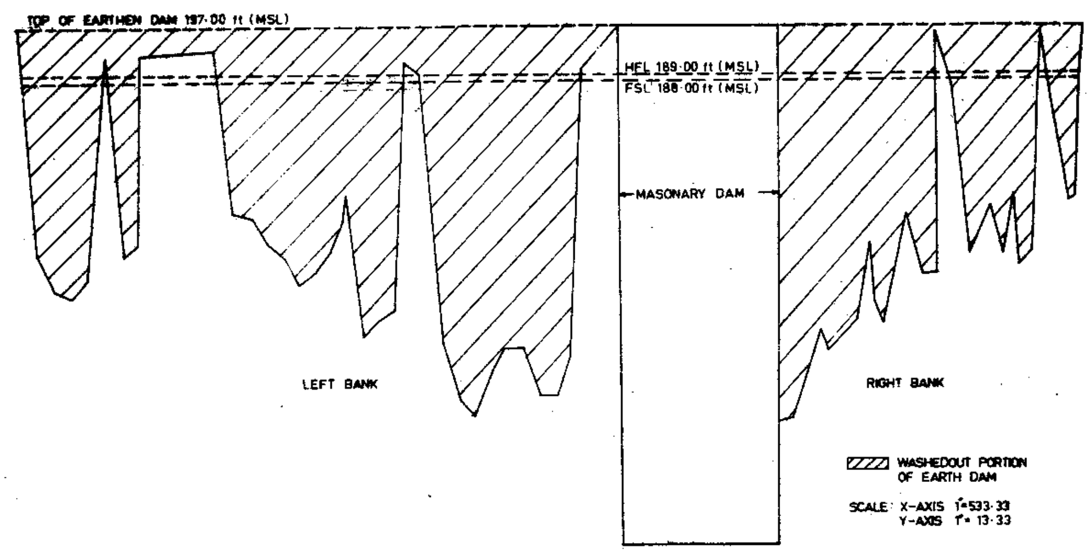

FIG. 2 : BREACH PROFILE OF MACHHU DAM-II

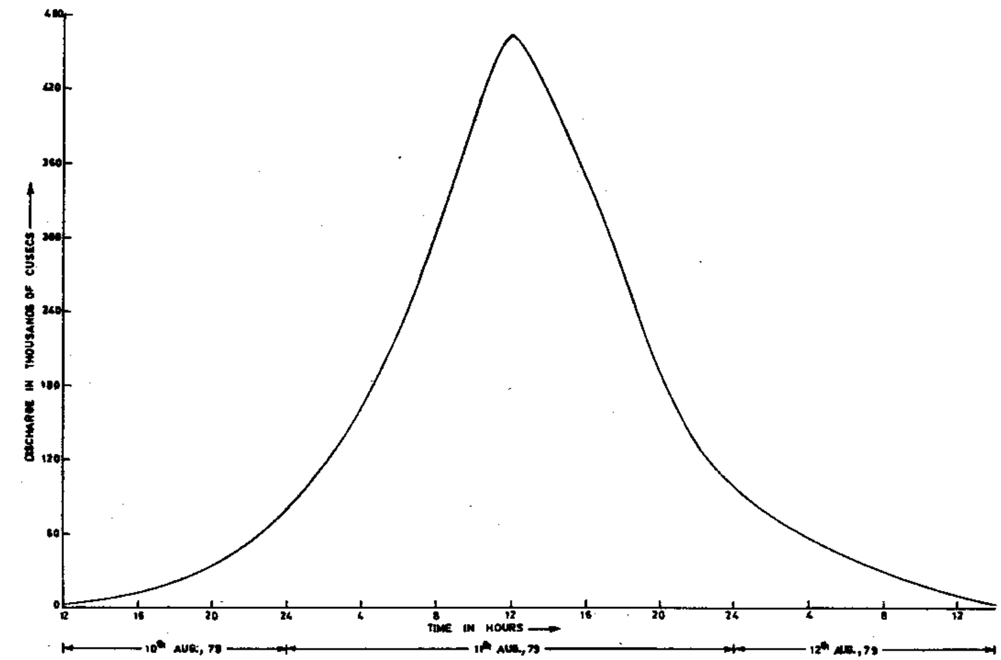

FIG. 3-ADOPTED RESERVOIR INFLOW HYDROGRAPH FOR DAM BREAK ANALYSIS OF MACHHU DAM-II
(TAKEN FROM REFERENCE (7))

5.2 Second Data Group

The second group of data pertaining to the routing of the outflow hydrograph through the downstream valley consists of a description of cross-sections, hydraulic resistance coefficients and contraction-expansion coefficients of the reach, steady state flow in the river at the beginning of the simulation and the downstream boundary condition. The cross sections are specified by location milage, and tables of top width and corresponding elevations.

In this study, six cross-section details are available at locations 0.8125 mile,5. 8125 mile, 10.8125 mile, 15.8125 mile, 20.69 mile and 24.625 mile. In the case of first three cross-sections measurements on the top widths have been made upto the highest water level (HFL) marks noted on both sides of the banks and in the case of last three sections, the top widths were not measured upto the HFL marks noted on both sides of the banks. There is no information available on the resistance or roughness coefficients and on the contraction-expansion coefficients of the reach.

6.0 METHODOLOGY

The National Weather Service's DAMBRK model developed by Dr D L Fread (1984) is used in this study of Machhu dam-II failure analysis. This model simulates the failure of a dam, computes the resultant outflow hydrograph and simulates movement of the dam break flood wave through the downstream river valley. The model is built around three major capabilities which are reservoir routing, breach simulation and river routing. However, it does no rainfall-runoff analysis and storm inflow hydrographs to the upstream of reservoir must be developed external to the model. A brief description of these model capabilities are given herein and for detailed description the reader may refer to the user manual of NWS (Fread, 1984).

6.1 Reservoir Routing

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

6.1.1 Storage routing

The storage routing is based on the law of conservation given as:

$$I - Q = \frac{dS}{dt} \qquad \ldots (1)$$

in which, I is the reservoir inflow, Q is the total reservoir outflow which includes the flow from spillway, breach, overtopping flow and head independent discharge, and $\frac{dS}{dt}$ is the time rate of change of reservoir storage volume. Equation (1) can be expressed in finite difference form as:

$$(I + I')/2 - (Q + Q')/2 = \Delta S/\Delta t \qquad \ldots (2)$$

in which the prime (') superscript denotes values at the time $t - \Delta t$ and the notation Δ approximates the differential. The term ΔS may be expressed as:

expressed as:

$$\Delta S = (A_s + A'_s)(h - h')/2 \qquad \ldots (3)$$

in which, A_s is the reservoir surface area coincidental with the elevation (h) and it is a functrion 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.

6.1.2 Dynamic routing

The hydrologic storage routing technique, expressed by equation (2) implies that the water surface elevation within the reservoir is horizontal. This assumption is quite adequate for gradually occuring breaches with no substantial reservoir inflow hydrographs. However, when (1) the breach is specified to form almost instantaneously so as to produce a negative wave within the reservoir, and/or (2) 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. Venant's equation which will be described later in the section on river routing.

6.2 Breach Simulation

Two types of breaching may be simulated using this model.

(1) An overtopping failure in which the breach is simulated as a rectangular, triangular, or trapezodial shaped opening that grows

progressively downward from the dam crest with time. Flow through the breach at any instant is calculated using a broad crested weir equation.

(2) A piping failure in which the breach is simulated as a rectangular orifice that grows with time and is centred at any specified elevation within the dam. 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 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 actual formation of a breach in earth dams is a complex process, depending on various hydraulic, hydrological and structural factors, and parameters. This process can be expected to be highly non-linear with time, and partial collapse may occur when the downstream face of the dam has suffered considerable erosion.

DAMBRK model defines the breach due to overtopping in five parameters, viz. side slope of the breach section, Z; the final bottom width of the breach, YBMIN; the time from inception to completion of breach, TF; and, the failure elevation, HF. The model assumes that the breach starts at a point and both the breach width and depth increase at a linear rate over the failure time. The elevation of the breach bottom, YBMIN, is usually taken to be the channel bottom or the dominant ground elevation of the dam, except when this was not physically justifiable due to backwater effects. 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.

6.3 River Routing

The movement of the dam break flood wave through the downstream river channel is simulated using the complete unsteady flow equations for one dimensional open channel flow, alternatively known as St. Venant's equations. These equations consist of the continuity equation:

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

and the conservation of momentum equation:

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

where,

A = active cross-sectional flow area,

A_o = inactive (off-channel storage) cross-sectional area

x = distance along the channel

q = lateral inflow or outflow per unit distance along the channel

g = acceleration due to gravity,

Q = discharge,

h = water surface elevation,

S_f = friction slope,

S_e = expansion-contraction loss slope,

L = lateral inflow/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 equation:

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

and,

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

wherein,

n = Manning's roughness coefficient,

R = A/B where B is the top width of the active portion of the channel

K = An expansion-contraction coefficient varying from 0.1 to 0.3 for contraction and -0.5 to -1.0 for expansion

$\Delta(Q/A)^2$ = 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 the 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 elevation at each cross section 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 and the model is capable of dealing with fully supercritical flow or fully subcritical flow in the reach or the upstream reach having supercritical flow and downstream reach having subscritical flow. 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 dam which may pass flow via spillways, overtopping and/or breaching.

7.0 ANALYSIS

This section describes the Machhu dam-II failure analysis carried out using the DAMBRK model programme. Before analysing the data using DAMBRK model programme, some preliminary analyses for the formulation of input data as required by the programme were made. These analysis dealt with the establishment of spillway rating table information and breach description of the dam. The details of these preliminary analyses have been explained herein alongwith the assumptions involved. Also the assumptions involved in the channel routing analysis of this dam break flood wave have been explained.

7.1 Spillway Rating Table Establishment

Due to the fact that the analysis of Machhu dam-II failure using DAMBRK model has been considered to begin, in this study, at the time of failure of the dam and due to the reason that the gate openings remained constant (15 gates fully opened, three gates opened at the level of 16 ft., 6 ft. and 4 ft.) since 1.30 a.m. of 11th August 1979 untill the failure of the dam took place, it was considered appropriate to input the spillway rating table corresponding to that existing gate opening conditions. The spillway discharge computations were made considering gated condition as well as free overfall condition of flow over the ogee spillway according to the procedure given in the USBR publication on 'Studies of Crests for Overfall Dams' Boulder Canyon Project, and the extract of the relevant pages of this reference is available in Report (Vol. II).

The spillway rating table so established is given below:

TABLE 2

SPILLWAY RATING TABLE

Head (ft.)	Flow over spillway (cfs)
0.0	0
4.0	17190
10.0	63751
14.0	103149
21.0	160690
24.0	184780
27.0	205069
30.5	229217

7.2 Breach Description

It was stated in section 5.1.4 that the required breach description details for the model can be derived from the actual profile shown in figure 2. The DAMBRK model requires only the rectangular or triangular, or trapezoidal shape for describing the dam breach. Therefore, suitable assumption has to be made to approximate the actual breach profile to correspond to any one of these breach profile.

This requirement of the model has led to the approximation of right embankment breach profile to a triangular shape and the left embankment breach profile to a combination of rectangular and triangular shape with the area of the triangular shape being the same as that of the right embankment breach. The programme also requires the breach to be located in the dam at one place. But the breaches in Machhu dam-II had occurred in two different places; one on the left embankment and the other on the right embankment. Therefore, it was necessary to combine both the right embankment breach, which has been approximated to triangular shape, and the left embankment breach which has been approximated to a combination of rectangular and triangular shape, as described earlier, to form a trapezoidal shape of breach located at one place. Accordingly the side slope and bottom width of this trapezoidal breach section has been computed as 0.0272 and 1036 ft. respectively. Although the bottom of the actual breach profile was located around 135 ft., considering the irregularities in the left and right mebankment profile shapes, the bottom of the assumed trapezoidal breach was considered to be located at an elevation of 130 ft. which corresponds to the elevation of bottom of dam. It has to be mentioned herein that the area of actual breach described by the profile given in figure 2 and the so assumed trapezoidal breach, for the analysis purpose, remain same.

The variation of model breach flow from the actual breach flow due to the variation of shape, breaches occurred at different locations, effect of roughness on the flow through breaches, effect of contraction-expansion coefficients of the breaches on the flow may be assumed to be small when compared with the magnitude of flow occurring from the breached dam.

7.3 Channel Routing

The DAMBRK model requires the first channel section to be located immediately downstream of the dam. However, the first cross-section is available only at 0.81 miles downstream from the dam. In order to satisfy the model requirement, it was assumed that this cross-section was located immediately below the dam.

As there was no information available regarding the variation of channel roughness coefficient with respect to elevation, a single roughness coefficient was assumed to hold good at any section. Accordingly, the first three reaches enclosed by the first four cross-sections were assumed to have a Manning's roughness coefficient of 0.035 and the next two reaches, where the flow was wide spread when compared with the flow over the earlier reaches and flowing over the delta area, the roughness coefficient was assumed to be 0.030.

It was seen from the flood affected area, upto the channel length of 10.81 miles downstream of dam which has been enclosed by the first three cross section, there was not significant expansion or contraction of the flow and afterwards the flow was expanding. As such for the first two reaches the contraction-expansion coefficient was considered as zero and for the remaining three reaches downstream it was considered as -0.05.

The programme demands for the channel routing analysis a steady flow situation throughout the channel reach before routing the inflow hydrograph. Therefore, the initial flow in the channel at the beginning of analysis was assumed to be the flow just prior to the occurrence of breach and this assumption leads to a flow of 278920 cusecs which was the sum of spillway discharge for the then existing gate opening condition and overtopping flow, corresponding to the water surface elevation of 198.5 ft. in the reservoir. However, at the time of failure at which the dam break analysis starts in this study, the flow in the channel would not have been steady and it is very unlikely that its magnitude would have been 278920 cusecs throughout the reach.

In order to satisfy the assumption of steady flow initial condition in the channel reach, for routing purpose, the analysis should begin two or three days earlier prior to 1.30 p.m. of 11th August 1979 during which time the flow in the channel may be considered to be approximately steady. But in such circumstances, the uncertainty regarding the information of inflow data of the reservoir increases. Therefore, it was considered appropriate to start the analysis to begin at 1.30 p.m. on 11th August 1979 as there was definite information at this time regarding the water level reached in the reservoir i.e. 198.5 ft. and also the dam was not breached. This analysis starting time also allows to input the inflow hydrograph ordinates beginning from 1.30 p.m. onwards at 2 hours interval without losing much of vital information of that inflow hydrograph which was entering into the reservoir at the time of dam failure and thus completely avoiding the effect of uncertainty of inflow information prior to 1.30 p.m. on the analysis. It was considered that the effect of so assumed initial condition on the peak flow of routed hydrograph would be small when compared with the effect of uncertainties of the inflow information of the reservoir , had the analysis started prior to 1.30 p.m. of 11th August 1979.

The outflow hydrograph from the breached dam formed the upstream boundary condition and the downstream boundary condition was given by the channel control. Using the available data information as described in section 5 and the information derived in this section, the input data file required for Machhu dam-II failure analysis using DAMBRK programme was prepared. The programme was run in the VAX-11/780 system available at the National Institute of Hydrology. The run time was 5.25 minutes.

Figure 4 shows the outflow hydrograph from the breached dam beginning from 1.30 p.m. on 11th August 1979 and the routed hydrographs at cross sections 5.81, 10.81, 15.81, 20.69 and 24.63 miles for a period around time to peak. Table 3 shows the salient features of these hydrographs.

TABLE 3

SALIENT FEATURES OF THE FLOOD WAVE FROM MACHHU DAM-II FAILURE

Sl. No.	River Mile from Machhu Dam-II	Peak discharge (in 10^5) (cusecs)	Peak flood elevation (in ft.)		Time to peak flood elevation as on 11.8.1979 (computed)
			Computed	Observed	
1.	0.0	19.36	179.92	-	2.45 p.m.
2.	0.81	17.50	174.00	168.40	2.57 p.m.
3.	5.81	11.29	135.59	131.00	4.27 p.m.
4.	10.81	9.69	103.38	101.00	5.33 p.m.
5.	15.81	8.67	80.33	77.00	6.48 p.m.
6.	20.69	8.23	55.01	53.00	8.03 p.m.
7.	24.60	7.95	38.95	-	9.06 p.m.

Figure 5 shows the peak flood elevation profile calculated using the model at various cross-sections obtained by interpolating the available measured cross sections. The observed peak flood elevations noted at cross sections located at 0.81, 5.81, 10.81, 15.81 and 20.69 miles downstream of dam have also been shown in figure 5. Similarly the profile of the peak flood discharges computed at these interpolated as well as available cross sections have been shown in figure 6.

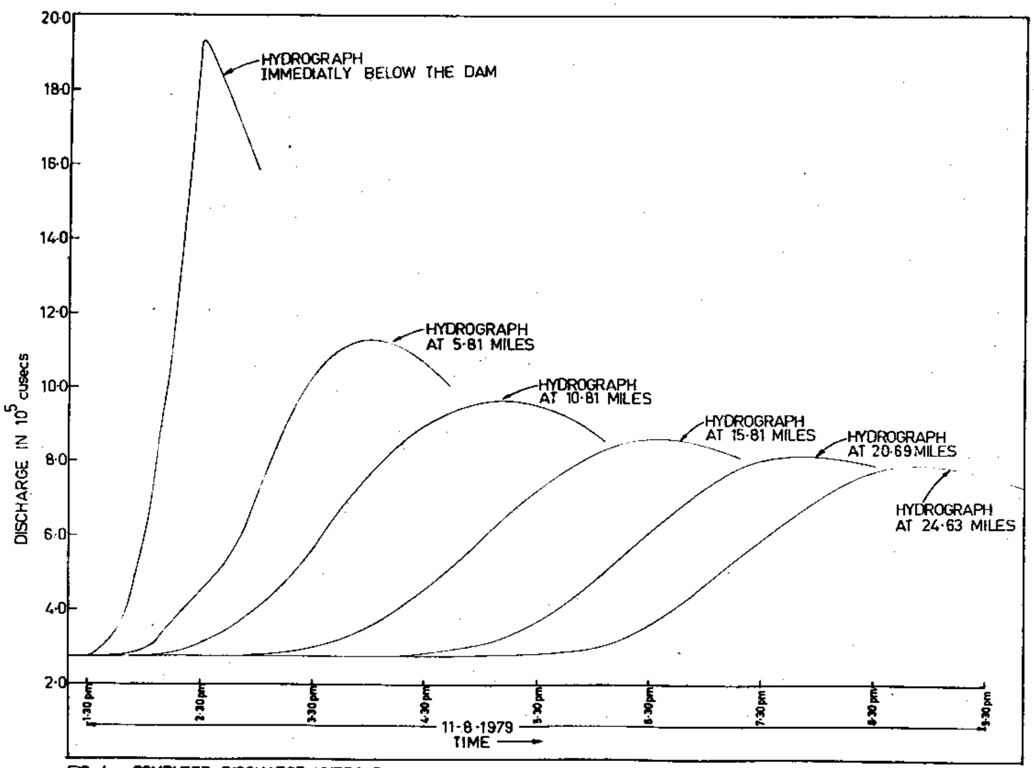

FIG. 4 – COMPUTED DISCHARGE HYDROGRAPHS DOWNSTREAM OF MACHHU DAM-II AFTER DAM BREAK

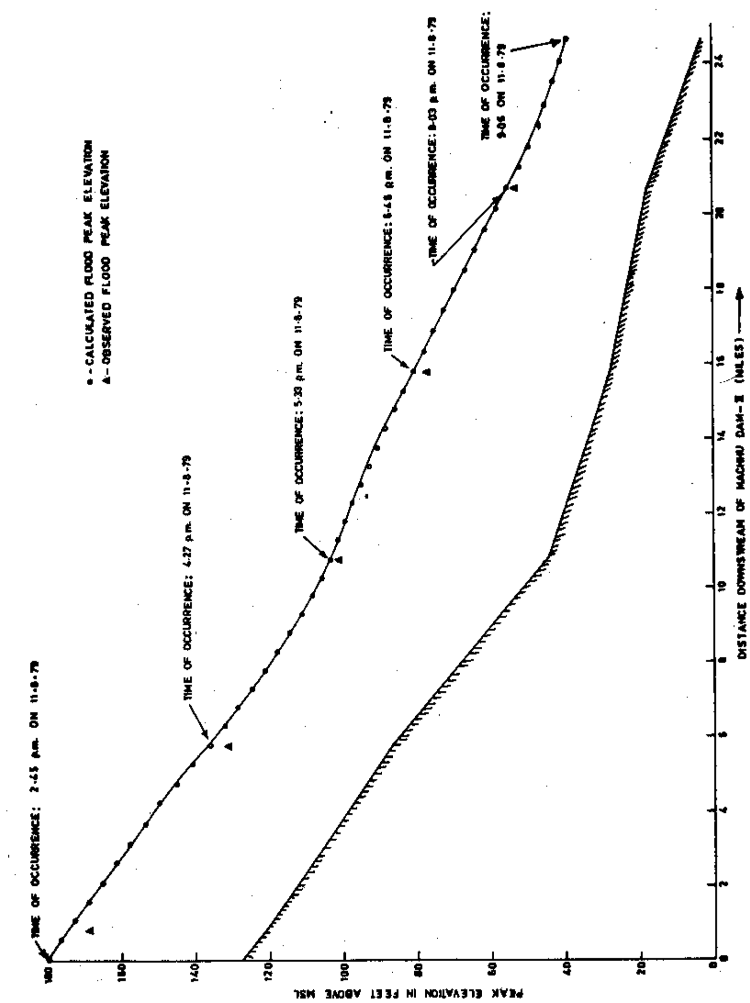

FIG. 5 - PEAK FLOOD ELEVATION PROFILE FROM MACHHU DAM-II FAILURE

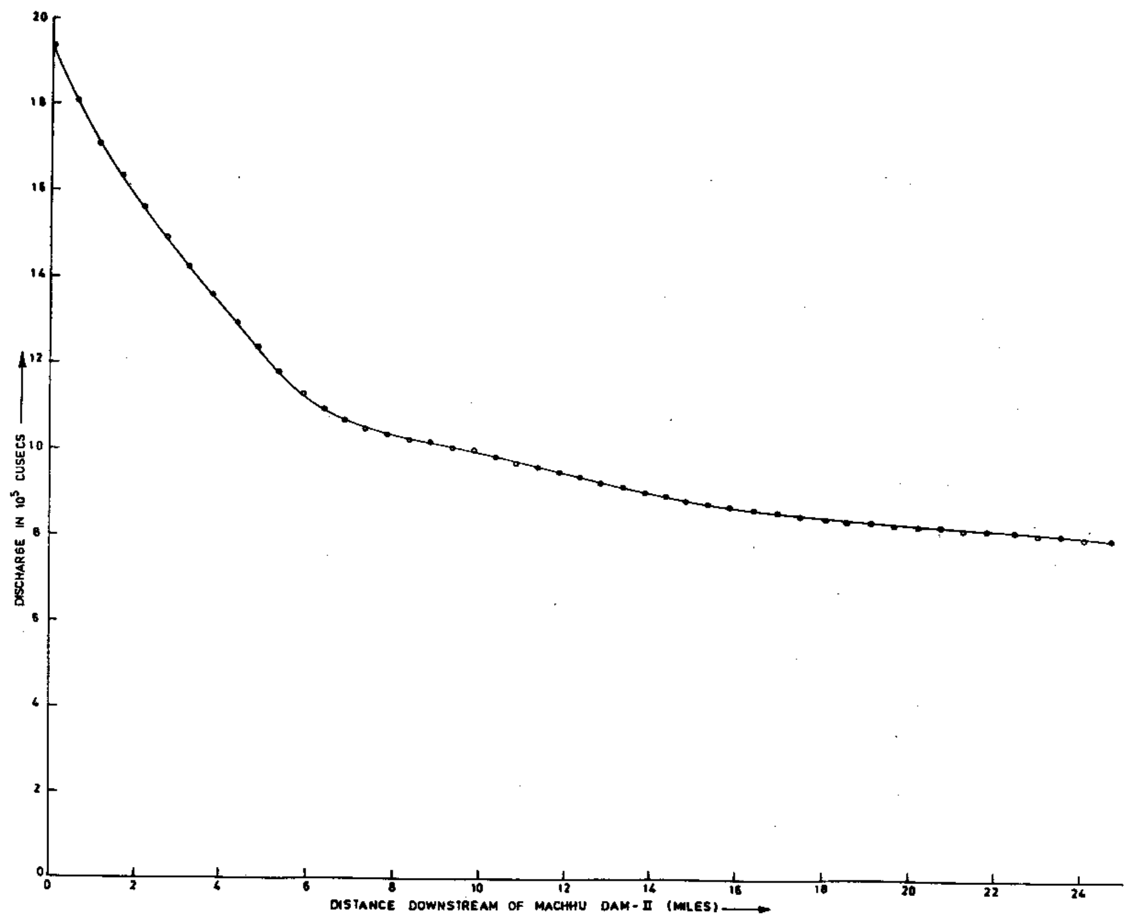

FIG. 6 - COMPUTED PEAK DISCHARGE PROFILE FROM MACHHU DAM-II FAILURE

8.0 DISCUSSION OF RESULTS

It can be inferred from figure 4 that the attenuation of the outflow hydrograph from the breached dam is very significant in the first reach and the rate of attenuation of the hydrograph decreases when the distance increases from the breached dam. This implies that the effect of breach parameters on the hydrograph characteristics may be dominant immediately downstream of the dam and for reaches farther downstream, the flood wave characteristics may be predominantly influenced by the channel geometry. This inference is also verified by the peak discharge profile shown in figure 6, wherein the rate of change of computed peak discharge is steep upto a distance of 6-8 miles from the downstream of the dam and it is very small or negligible beyond this distance. This behaviour of the dam break flood wave is understandable since, after some distance from the dam, a given volume of flood water is distributed over a certain area according to hydraulic principles which do not allow for too much variation. This strenghthens the concept that the assumptions made regarding the breach description of Machhu dam-II may not invalidate the results and the analysis would be indeed useful for flood inundation mapping farther away from the breached dam. However, for flood warning purposes the breach description may be important as it affects the flood wave movement downstram of the dam.

But considering the uncertainty regarding inflow information into the reservoir, breach time, initial flow in the channel at the time of routing, roughness coefficient of the channel, one may consider that the analysis performed herein yields results which are not very much different from the observed flood characteristics. Table 3 shows that the maximum difference in the observed and computed peak flood elevation is 5.6 ft. just at 0.81 miles and then onwards this difference decreases except at the location of 15.81 miles. One may infer from figure 5 that the computed peak flood elevations are comparable with the available observed peak flood elevations.

The available statements regarding flooding of Morvi town which is located at 5.125 miles downstream of the dam, indicate that the peak elevation of flood reached around 4.30 p.m. on 11th August 1979. The analysis shows that at section 5.81 miles downstream of the dam, which is nearer to Morvi town, the peak flood elevation reached at 4.27 p.m. which shows the credibility of the DAMBRK model for analysing dam break problems.

9.0 CONCLUSIONS

The flood wave formation due to Machhu Dam-II failure which occurred on 11th August 1979 in the western part of Gujarat has been simulated using U.S. National Weather Services' DAMBRK model. The computed peak flood elevations and the corresponding observed peak flood elevations compare reasonably well. This demonstrates that the assumptions made in modifying the actual mode of breach to suit the mode of breach required by the model are valid and acceptable for the present study.

It may be concluded based on this study that the National Weather Services' DAMBRK model may be suitable for analysing hypothetical dam failure cases for the purpose of flood warning and flood inundation mapping even though the breach formation in reality may not occur in the mode demanded by the model.

REFERENCES

1. Chen, C.L. and J.T. Armbruster (1980), 'Dam Break Wave Model: Formulation and Verification'. Journal of the Hydraulics Division, Proc. ASCE Vol. 106, No.HY5, May pp. 747-767

2. Cristofano, E.A. (1965), 'Method of Computing Rate for Failure of Earth Fill Dams'. Bureeau of Reclamation, Denver, Colo., April.

3. Dressler, R.F., (1954), 'Comparison of Theories and Experiments for the Hydraulic Dam Break Wave'. IASH, No. 38, pp. 319-328.

4. Fread, D.L., (1984), 'DAMBRK: The NWS Dam-Break Flood Forecasting Model'. Office of Hydrology, National Weather Service, Maryland.

5. Government of Gujarat, (1980), 'Before the Machhu Dam-II Inquiry Commission: The Statement of Facts and Opinions'. Vol. I and II , Sachivalaya, Gandhi Nagar, March.

6. Harris, G.W. , and D.W. Wagner (1967), 'Outflow from breached Dams'. Univ. of Utah, USA.

7. Jagdish Narain, Satish Chandra, B.S. Mathur and D.K.Srivastava, (1981),. 'Report on Investigations for Machhu Dam-II (Part II)', University of Roorkee, Roorkee, May.

8. Martin, C.S., and J.J. Zovne, (1971), 'Finite-Difference Simulation of Bore Propagation'. Journal of the Hydraulics Division, ASCE, Vol. 97, HY 7, July, pp. 993-1010.

9. Petrascheck, A.W., and P.A. Sydler (1984), 'Routing of Dam Break Floods'. Water, Power and Dam Construction , July

10. Re. R., (1946), 'A Study of Sudden Water Release from a Body of water to Canal by the Graphical Method'. Houille Blanche (France), No.3, pp. 181-187.

11. Ritter, A., (1892), 'The Propagation of Water Waves'. Ver. Deutsch Ingenieure Zeitschr (Berlin), 36, pt. 2, No. 33, pp. 947-954.

12. Sakkas, J.G., and T Strelkoff, (1973), 'Dam Break Flood in a Prismatic Dry Channel'. Journal of the Hydraulics Division, ASCE, Vol. 99, HY 12, Dec., pp. 2195-2216.

13. Singh, K.P., and A Snorrason, (1984), 'Sensitivity of outflow Peaks and Flood Stages to the Selection of Dam Break Prameters and Simulation Models'. Journal of Hydrology, Vol. 68 No.1/4, February.

14. Stoker, J.J., (1957), 'Water Waves'. Inter-Science Pub., New York.

15. Se, S.T., and A.H. Barnes (1970), 'Geometric and Factional Effects on Sudden Releases'. Journal of the Hydraulics Division, ASCE, Vol. 96, HY 11, Nov., pp. 2185-2200.

16. Terzidis, G., and T Strelkoff (1970),' Computation of Open Channel Surges and Shocks'. Journal of the Hydraulics Division, ASCE, Vol. 96, HY 12, Dec., pp. 2581-2610.

17. U.S. Army Corps. of Engineers (1960), 'Floods Resulting from suddenly Breached Dams - Conditions of Minimum Resistance, Hydraulic Model Investigations'. Misc. paper 2-374, Report 1, WES, Feb. 176 pb.

18. U.S. Army Corps of Engineers (1981), 'Flood Hydrograph Package (HEC-1), User's Manual for Dam Safety Investigations, Hydrologic Engineering Center, Davis, Calisfornia.