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DEVELOPMENT OF DIMENSIONLESS HYDROGRAPHS FROM MACHHU DAM-II FAILURE USING DAMBRK MODEL

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ABSTRACT

This paper presents a methodology for the quick estimation of dam break flood wave and its characteristics such as peak flows, peak stages and their respective timings at the dam site and at specified locations downstream of Machhu Dam-II using the technique of dimensionless hydrographs of dam break flood wave developed based on different breach area criteria. These dam break flood waves were developed using U.S. National Weather Services DAMBRK model on the data of Machhu Dam -II which failed on 11 August 1979 in Gujarat State. Preliminary investigations made using DAMBRK model showed insignificant impact on the dam break flood hydrograph characteristics relevant to flood warning due to variation in shape of the breach and due to the consideration of trapezoidal breach with area varying from 50% to 250% with 100% area corresponding to the actual area of breach observed at Machhu Dam-II. The dimensionless hydrographs relate the time of hydrograph non-dimensionlised with reference to the time to peak flow and discharge non-dimensionalised with reference to the peak discharge. Relationships have been established for area of breach vs the peak flow and the peak flow vs the time to peak flow at the dam site. Similarly, non-dimensional hydrographs were developed at the specific sites downstream of the dam, besides the relationship between peak flow of upstream site and the next downstream site, time to peak flow of upstream site and next downstream site, and the peak flow and peak stage at the respective sites. Using these dimensionless hydrographs and relationships one can quickly estimate the peak flow and peak stages at specific sites knowing only the breach area at the time of disaster without the need for using the DAMBRK model. The usefulness of this approach has been demonstrated by developing the dam break flood wave hydrographs for a breach area 175% of the actual breach area which was not used for the development of dimensionless hydrographs and other relationships.

1. INTRODUCTION

Problem of dam break analysis is one of the most fascinating hydraulic problems. Although many publications are available on dam break simulation problem since 1892 (Ritter, 1892) but only a very few deal it with practical consideration. There are numbers of models available now for the dam break analysis. Ralph (1985) focussed on the following selected models -

National Weather Services (NWS) Dam Break Flood Forecasting Model (DAMBRK), U.S. Army Corps. of Engineers Hydrologic Engineering Centre (HEC) Flood hydrograph Package (HEC-1), U.S. Army Corps of Engineering South Western Division (SWD), Flow Simulation Models (FLOW SIM 1 and 2), Soil Conservation Service (SCS), Simplified Dam Breach Routine Procedure (TR66), NWS Simplified Dam Breach Flood Forecasting Model (SMPDBK) and HEC Dimensionless Graphs procedure.

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Ralph stated that a dynamic routing should be used whenever obtaining a maximum practical level of accuracy is required and adequate manpower, time and computer resources are available. Ralph also stated that the National Weather Service (NWS) Dam-Break Flood Forecasting Model(DAMBRK) is the optimal choice of model for practical applications.

In the present study an attempt has been made to develop dimensionless hydrographs and the relevant relationships as mentioned earlier, based on the dam break flood waves simulation results, using U.S. National Weather Services DAMBRK model, as this model is considered to be the most practical one available till date.

2. Description of the Study Area

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°46' North and the Longitude of 70°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. An index map showing the flood affected area due to Machhu dam-II failure is shown in figure-1. Earthen dam has its length 7689 ft. and 4588 ft. on left and right side of the spillway. The ogee spillway has its length as 676 ft. and the crest at RL 168 ft. respectively. High flood level and top of dam are at 189 ft. and 197 ft. respectively.

At 1.30 P.M. on 11th August, 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.

3. Availability of Data

3.1 With reference to the data 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.

3.1.1 The reservoir elevation volume relationship of Machhu Dam-II has been taken from Annexure GA-52 of the report (Vol.II).

3.1.2 Annexure: GA-52 of Report (Vol.II) also gives the various gate opening conditions and the corresponding water surface elevation in the reservoir for the purpose of developing spillway rating table. The length of flow over top of the dam due to over-

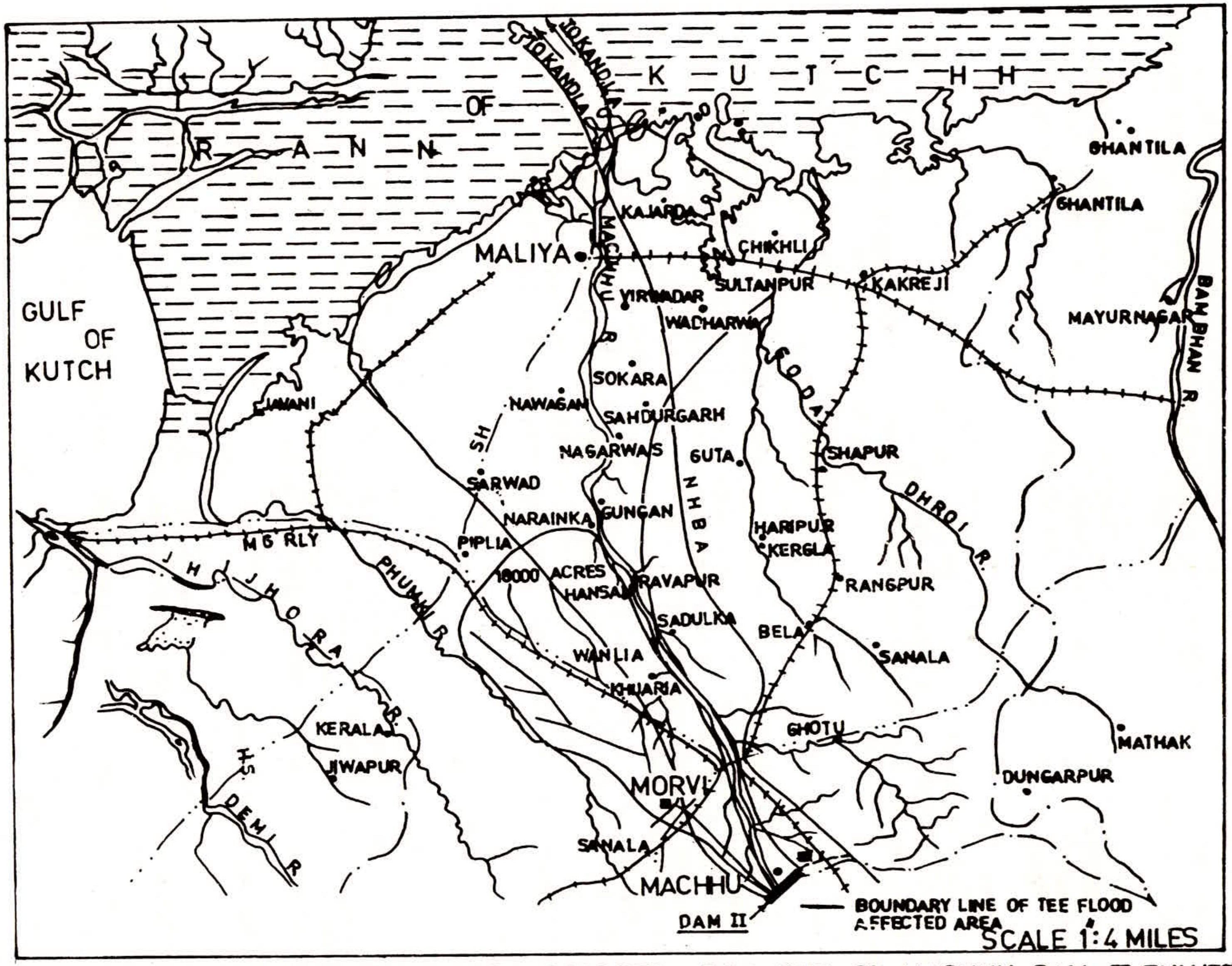


FIG 1 INDEX MAP SHOWING FLOOD AFFECTED AREA DUE TO MACHHU DAM-II FAILURE

topping has been considered as 10133 ft. and this information has been taken from Annexure GA-4 of Report (Vol.II).

3.1.3 Elevation of top of dam = 197 ft.
Elevation of bottom of dam=130 ft.

The elevation of initial water level in the reservoir and the water surface in the reservoir at the time of failure were recorded at 1.30 PM on 11th August 1979 as 198.5 ft.

3.1.4 The profile of the breached earthen 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 this profile.

3.1.5 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.

3.2 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.

4. Methodology

The National Weather Services DAMBRK model developed by Dr. D.L Fread (1984) is used in this study of Machhu Dam-II failure analysis. A brief description of the model capabilities are given herein and for detailed description the reader may refer to the user manual of NWS (Fread, 1984).

4.1 Reservoir Routing

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

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

$$I - Q = \frac{dS}{dt} \qquad ..1.$$

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 dS/dt is the time rate of change of reservoir storage volume. The dynamic routing principle

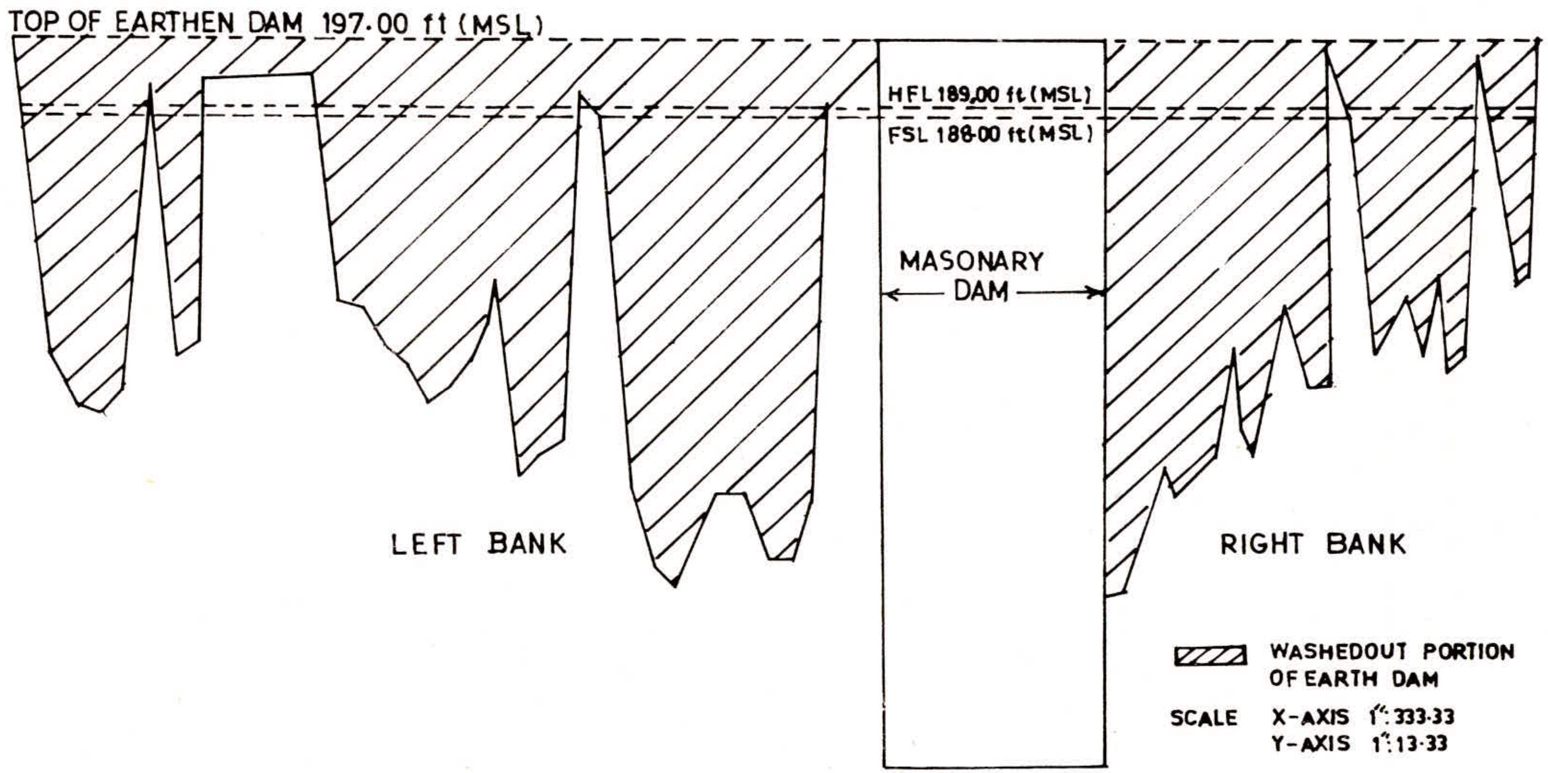


FIG.2. BREACH PROFILE OF MACHHU DAM-II

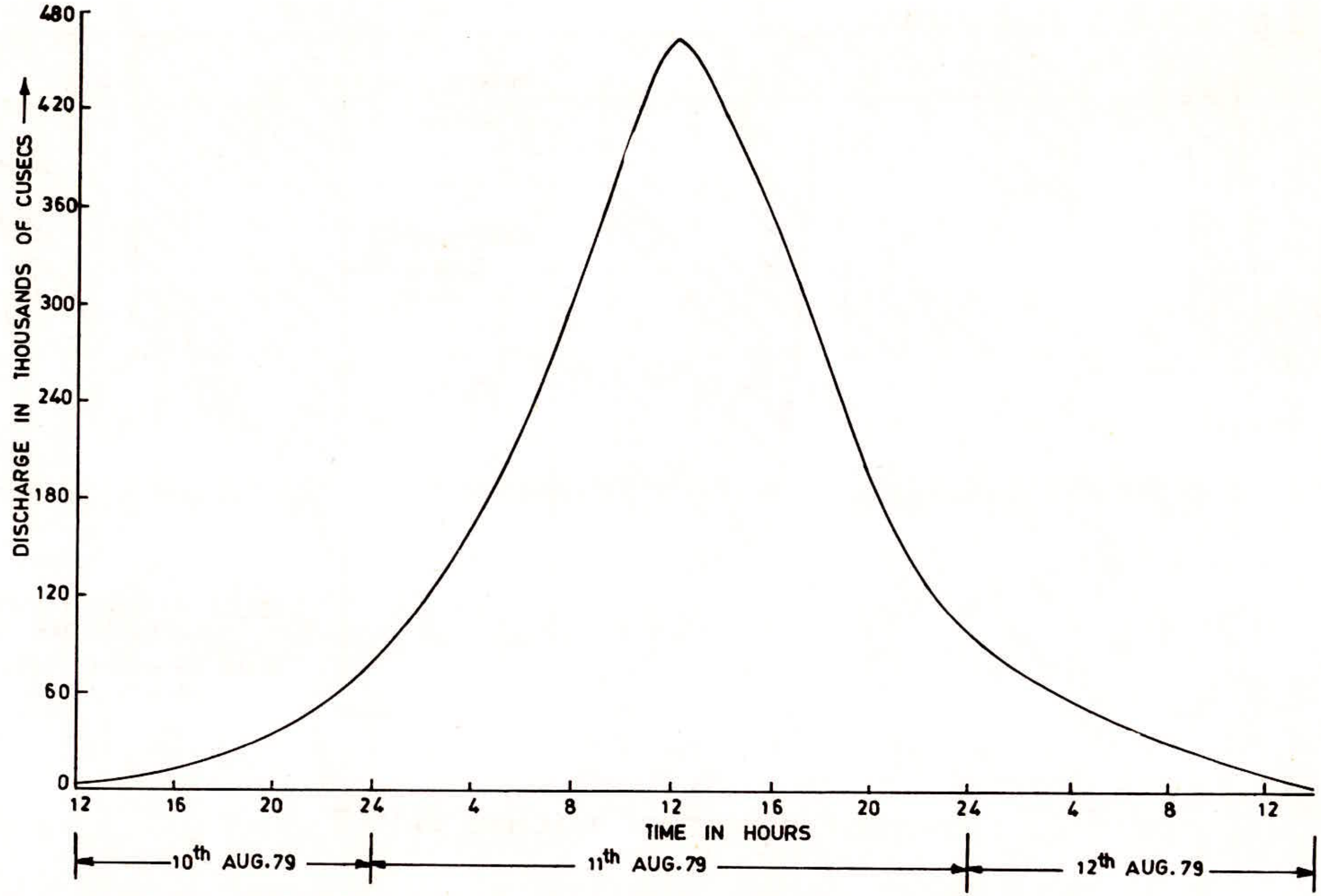


FIG. 3.- ADOPTED RESORVOIR INFLOW HYDROGRAPH FOR DAM BREAK ANALYSIS OF MACHHU DAM - II

for reservoir routing 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.

4.2 Breach Simulation

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.

4.3 River Routing

The movement of the dam break flood wave through the downstream river channel is simulated using St.Venant's equations. These equations consist of the continuity equation.

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

and the conservation of momentum equation:

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

where,

A = 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:

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

and

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

wherein,

n = Manning's roughness coefficient

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

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

$\Delta(Q/A)^2$ = Difference in $(Q/A)^2$ for cross-sections either and of a reach

4.4 Dimensionless hydrographs and the relationships

The dam break flood wave hydrographs were simulated,using DAMBRK model for various breach sizes to obtain the values of peakflow ,time to peak flow and max. stage of specific downstream sites of Machhu Dam-II. To obtain these results the shape of breach occurred due to the over topping of the dam was assumed as trapezoidal and no inflow hydrograph was routed through the reservoir, based on earlier studies made (Santoshi, 1987-88). Using the dam break flood wave simulations results, obtained for various breach sizes, the following relationships were established.

i) Dimensionless hydrographs at the dam site and at all the specific sites downstream of the dam, relating time of hydrograph non-dimensionalised with reference to the time to peakflow, and instantaneous discharge non-dimensionalised with reference to the peak discharge at respective sites.

ii) Area of breach and peak flow at dam site in log domain.

iii) Peak flow of upstream site and next downstream site

iv) Peak flow and time to peak flow at dam site.

v) Time to peak flow at upstream site and next downstream site

vi) Peak flow and peak stage at dam site and at all the specific sites downstream of the dam.

The development of dimensionless hydrographs and establishment of the relationships as described above is known as dimensionless hydrograph procedure for dam break studies of Machhu Dam-II.

5. ANALYSIS

5.1 Spillway Rating Table

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).

5.2 Breach Description

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 shape, 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 37.037 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 embankment 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.

5.3 Channel Routing

According to the DAMBRK model requirement, the first cross section available only at 0.81 miles downstream from the dam was assumed that this cross section was located immediately below the dam. Due to non-availability of the channel roughness information it was assumed that a single roughness coefficient holds good at any section. Accordingly, the first three reaches were assumed to have a manning's roughness coefficient of 0.035 and next two reached as 0.030. Further for the first two reaches the expansion-contraction coefficient was considered as zero and for the remaining three reaches downstream it was considered as -0.50.

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 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. Using available data information as described in section 3 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. The dam break analysis for Machhu Dam-II was carried out and the relevant simulation results for actual breach area obtained from DAMBRK model are given in table 1.

TABLE - 1
SALIENT FEATURES OF THE FLOOD WAVE FROM MACHHU DAM-II FAILURE

S.No.	River Miles 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	30.98	186.66	-	2.30 p.m.
2.	0.81	28.30	180.00	168.40	2.35 p.m.
3.	5.81	12.76	136.76	131.00	3.51 p.m.
4.	10.81	10.18	103.88	101.00	5.03 p.m.
5.	15.81	8.97	80.56	77.00	6.21 p.m.
6.	20.69	8.48	55.16	53.00	7.36 p.m.
7.	24.63	8.16	39.05	-	8.39 p.m.

For further details the reference can be made to the report CS-16 'dam break analysis for Machhu dam-II' (NIH, 1985-86). In the present study, all these variables have been kept same except the breach size. It is presumed that the results of dam break flood wave simulation may not get changed qualitatively even if those variables do not remain constant under different breach conditions.

5.4 ANALYSIS FOR THE DEVELOPMENT OF DIMENSIONLESS HYDROGRAPH PROCEDURE

5.4.1 Dimensionless Hydrographs

The dimensionless hydrographs which relate time t non-dimensionalised with reference to time to peak flow t_p and discharge Q non-dimensionalised with reference to peak flow Q_p and shown in figures 4,5,6,7,8, and 9 at sections immediately below the dam and at section 5.81 miles, 10.81 miles, 15.81 mile, 20.69 miles and 24.63 miles respectively below the dam. By non-dimensionalising the hydrograph the variations in the individual hydrographs due to the variation of breach sizes have been brought down and a hydrograph characteristics of the site considered has been arrived at, for the range of breach sizes studied. The possibility of extending the use of this non-dimensionalised hydrograph for a breach area which falls beyond the range studied has not been investigated.

5.4.2 The relationships between area of breach and peak flow in log domain at dam site is shown in Fig. 10 and it can be expressed in linear relationship form as given below:

$$\text{Log}\,(Q_{po}) = 5.9649 + 0.240\ \text{Log}\,(A_b) \quad ..8$$

wherein

Q_{po} is the peak discharge at dam site in 10^5 cusecs

A_b is the area of breach in % of the actual breach area

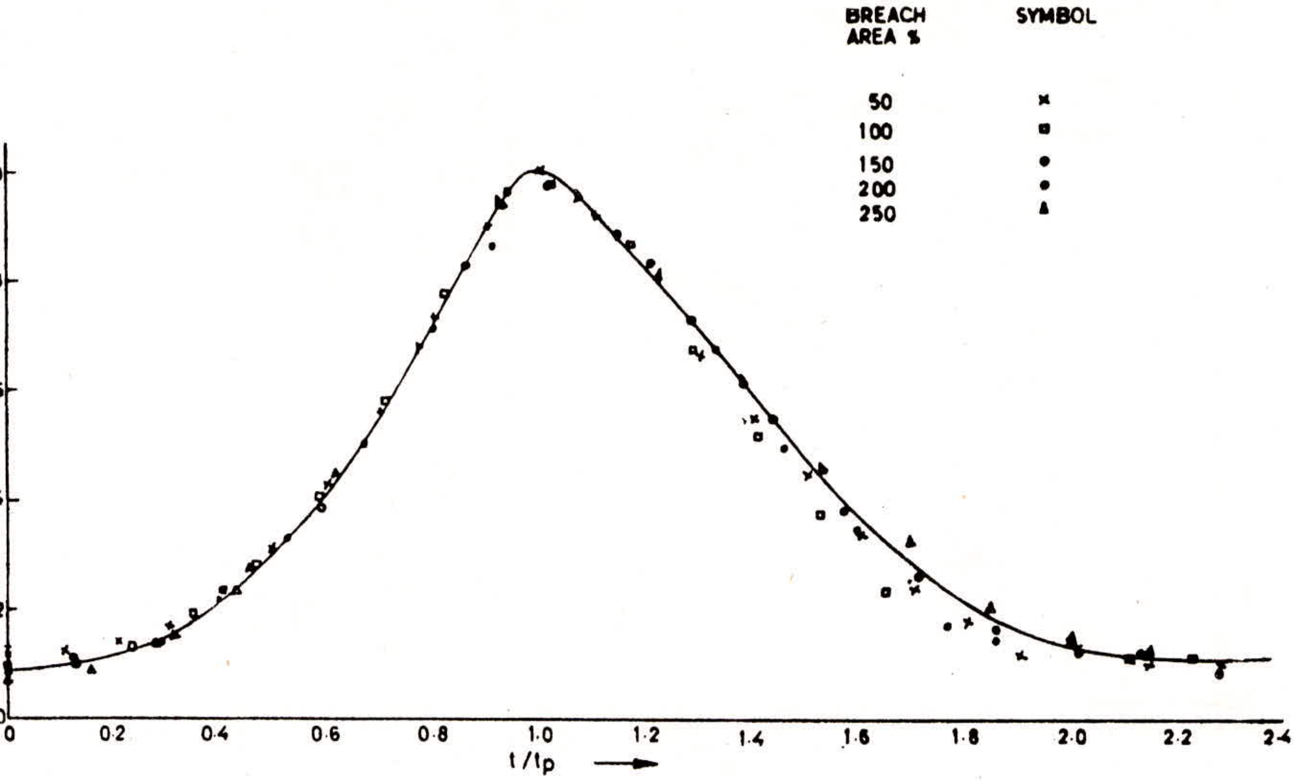


FIG. 4. NON-DIMENSIONAL HYDROGRAPH IMMEDIATELY BELOW THE DAM

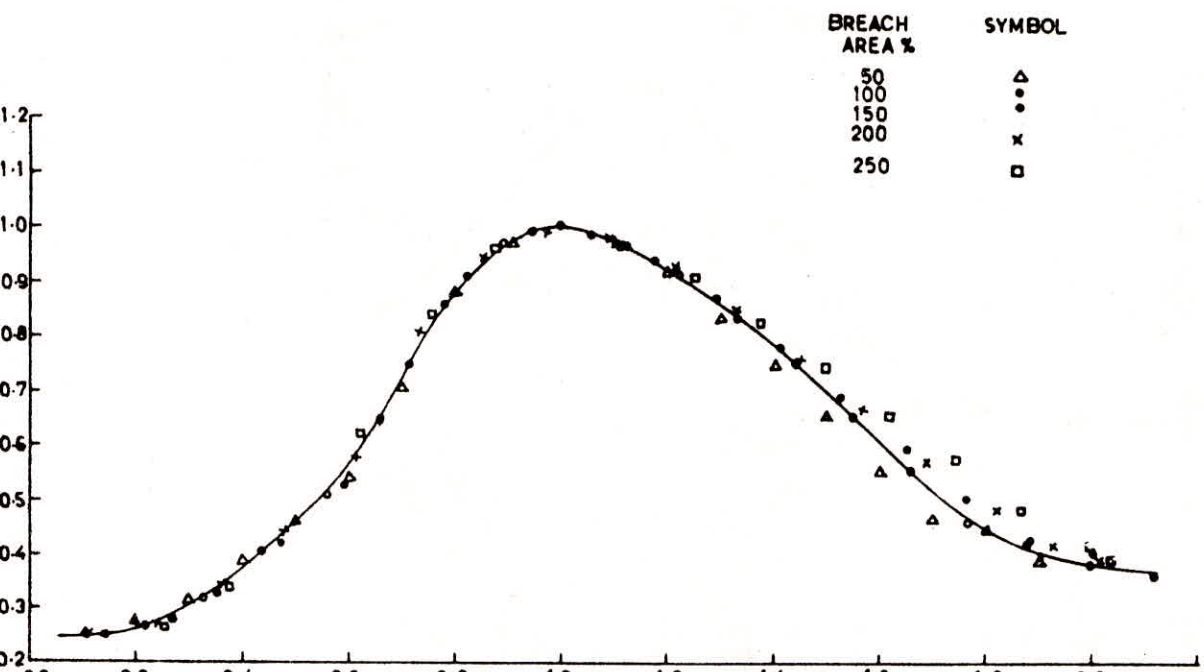


FIG. 5. NON-DIMENSIONAL HYDROGRAPH AT 5·81 MILES BELOW THE DAM

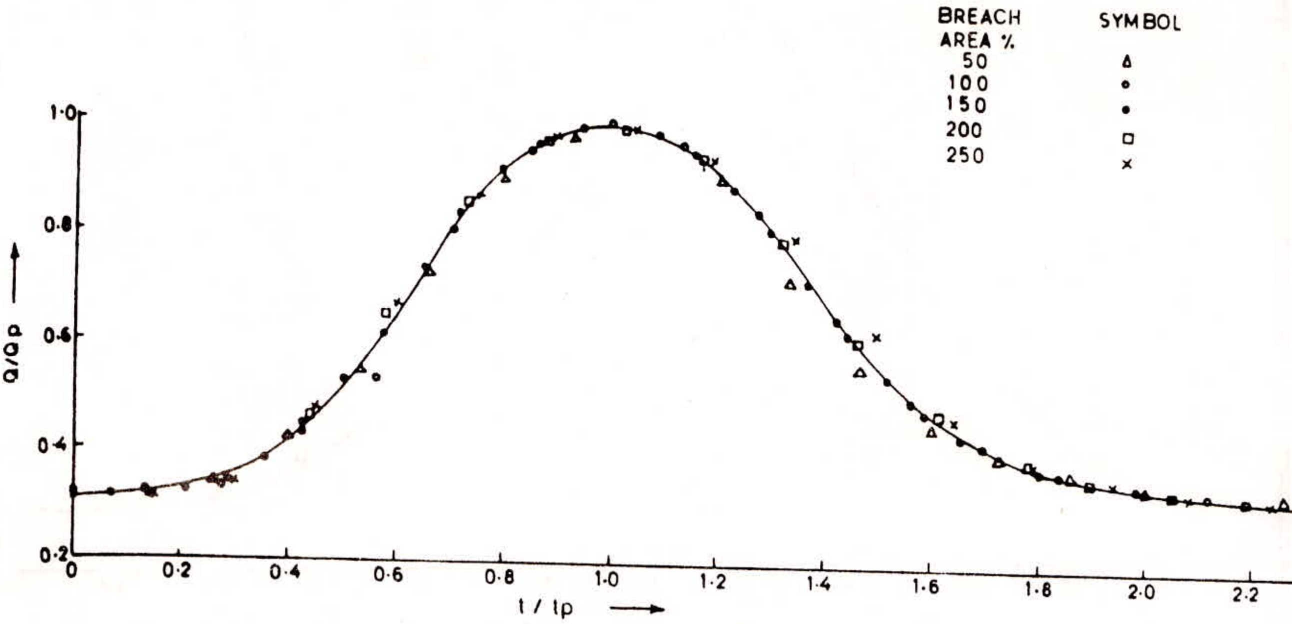


FIG. 6. NON-DIMENSIONAL HYDROGRAPH AT 10·81 MILES BELOW THE DAM

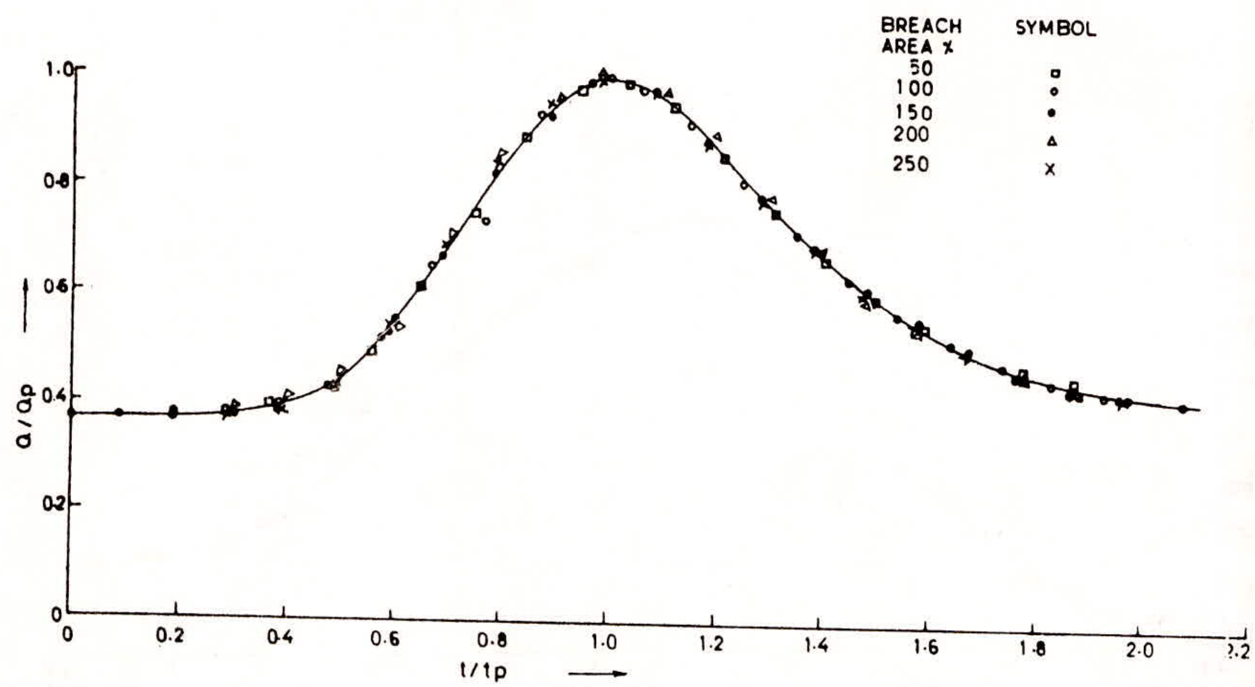


FIG. 7. NON-DIMENSIONAL HYDROGRAPH AT 15.81 MILES BELOW THE DAM

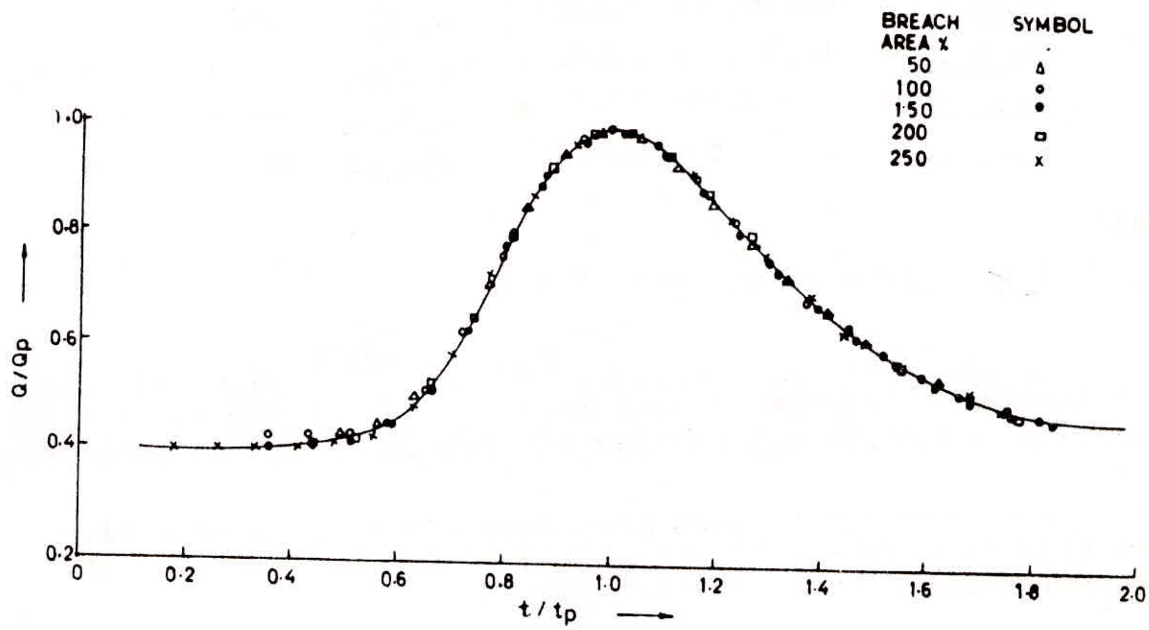


FIG. 8. NON-DIMENSIONAL HYDROGRAPH AT 20.69 MILES BELOW THE DAM

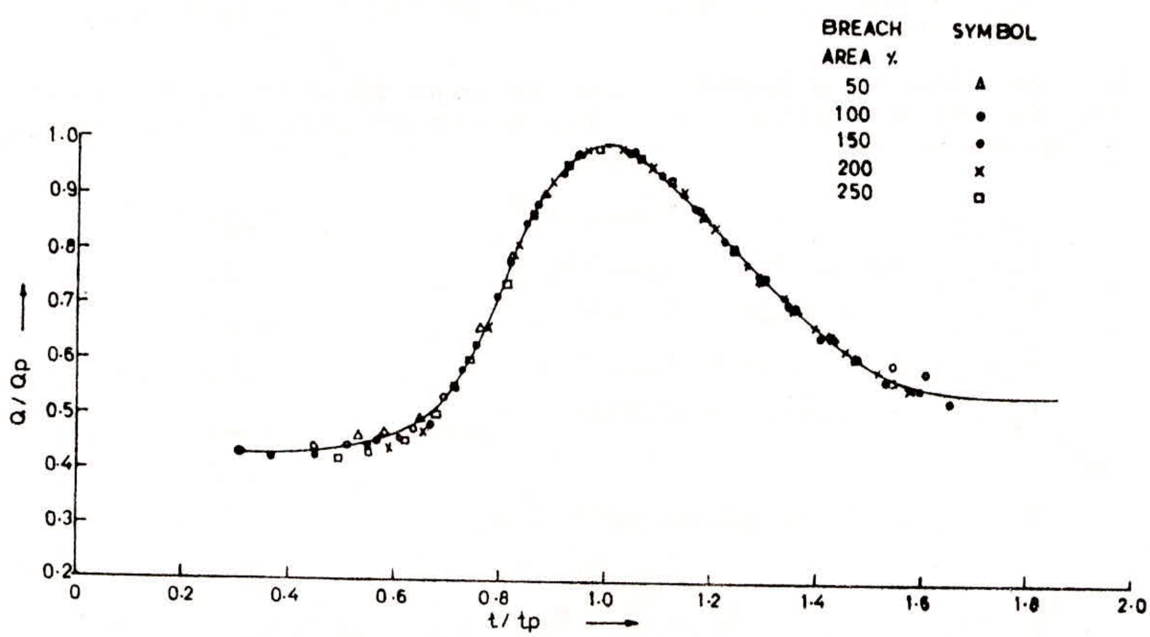


FIG. 9. NON-DIMENSIONAL HYDROGRAPH AT 24.63 MILES BELOW THE DAM

5.4.3 The relationships between peak discharge at upstream site and next downstream site can be expressed in linear relationship form as given below:-

$$Q_{P5.81} = 799778.0 + 0.1001 \times Q_{po} \quad ..9$$

$$Q_{P10.81} = 295156.81 + 0.5321 \times Q_{P5.81} \quad ..10$$

$$Q_{P15.81} = -41830.0 + 0.891 \times Q_{P10.81} \quad ..11$$

$$Q_{P20.69} = -18179.44 + 0.940 \times Q_{P15.81} \quad ..12$$

$$Q_{P24.63} = 11676.75 + 0.930 \times Q_{P20.69} \quad ..13$$

wherein

Q_{po} is same as given in 5.4.2.

$Q_{P5.81}$, $Q_{P10.81}$, $Q_{P15.81}$, $Q_{P20.69}$ and $Q_{P24.63}$ are the peak estimated discharge at 5.81 miles, 10.81 miles, 15.81 miles 20.69 miles and 24.63 miles below the dam in cusecs respectively.

5.4.4 Relationship between peak flow and time to peak flow at dam site is shown in figure 11 and it can be expressed in linear relationship form as given below:

$$t_{po} = 1.7442 - 0.00000033\, Q_{po} \quad ..14$$

wherein

Q_{po} is same as given in 5.4.2.

t_{po} is the time to peak flow at dam site in hours.

5.4.5. Relationship between time to peak flow at upstream site and next downstream site can be expressed in linear relationship form as given below:

$$t_{p5.81} = 0.9477 + 1.044 \times t_{po} \quad ..15$$

$$t_{p10.81} = 1.3781 + 0.8137 \times t_{p5.81} \quad ..16$$

$$t_{p\,15.81} = 1.9618 + 0.7693 \times t_{p10.81} \quad ..17$$

$$t_{p20.69} = 0.8159 + 1.1387 \times t_{p15.81} \quad ..18$$

$$t_{p24.63} = 1.8712 + 0.8538 \times t_{p20.69} \quad ..19$$

wherein

t_{po} is same as given in 5.4.4.

$t_{P5.81}$, $t_{P10.81}$, $t_{P15.81}$, $t_{P20.69}$ and $t_{P24.63}$ are the estimated time to peak flow at 5.81 miles, 10.81 miles, 15.81 miles, 20.69 miles and 24.63 miles below the dam in hours respectively.

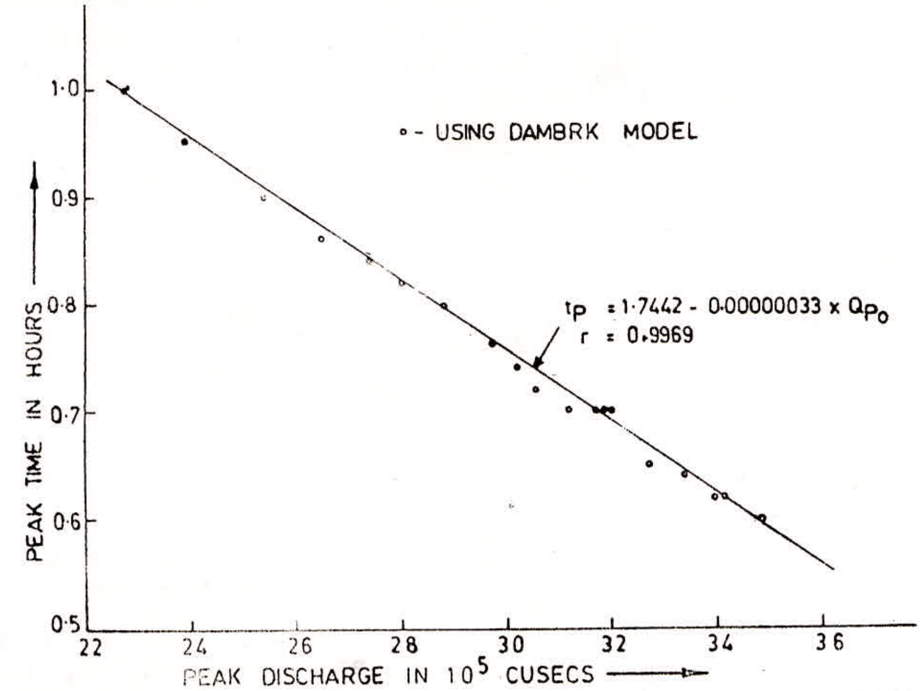


FIG. 11. RELATIONSHIP BETWEEN PEAK DISCHARGE AND PEAK TIME IMMEDIATELY BELOW THE DAM

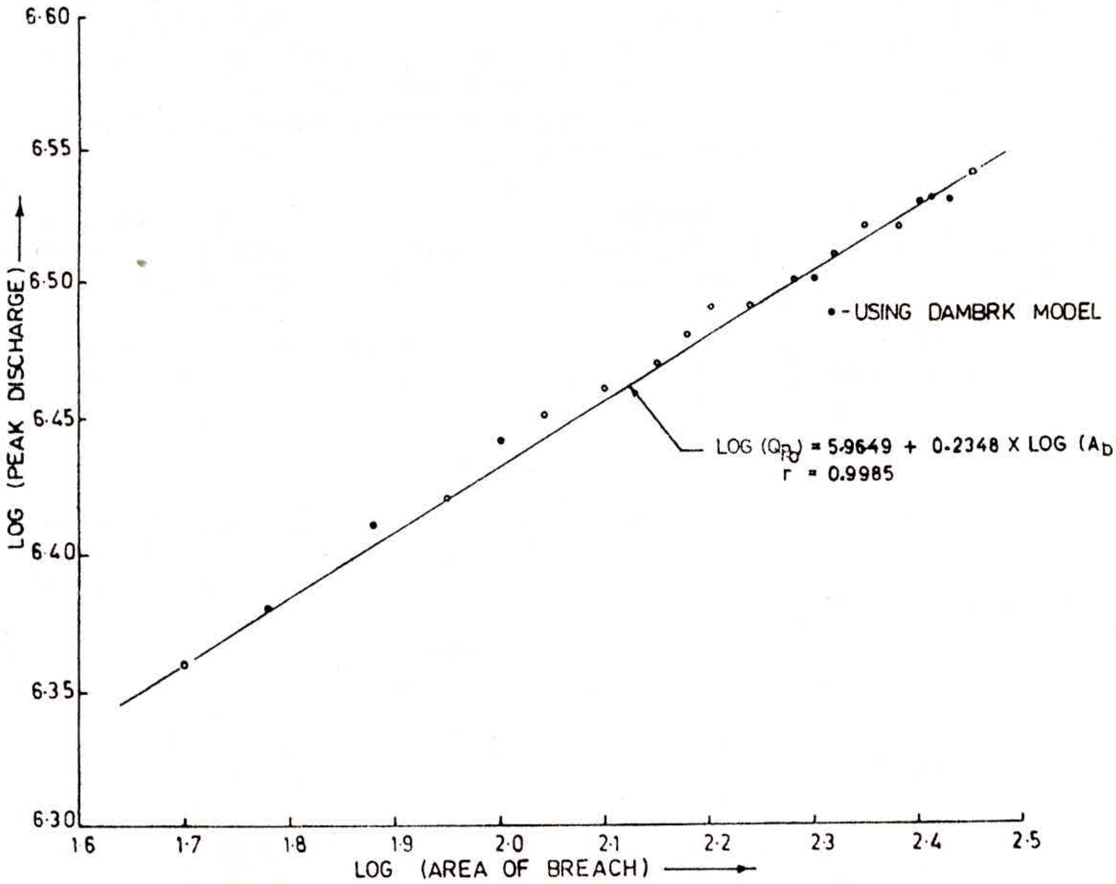


FIG.10. RELATIONSHIP BETWEEN LOG (AREA OF BREACH) AND LOG (PEAK DISCHARGE) IMMEDIATELY BELOW THE DAM

5.4.6 The relationship between peak discharge and peak stage at immediately below the dam and at 5.81 miles, 10.81 miles, 15.81 miles, 20.69 miles and 24.63 miles below the dam can be expressed in linear relationship form as given below:-

$$H_{Po} = 51.3039 + 0.00000426 \times Q_{Po} \quad ..20$$

$$H_{P5.81} = 36.0949 + 0.00001064 \times Q_{P5.81} \quad ..21$$

$$H_{P10.81} = 45.6143 + 0.00001338 \times Q_{P10.81} \quad ..22$$

$$H_{P15.81} = 44.3970 + 0.00000947 \times Q_{P15.81} \quad ..23$$

$$H_{P20.69} = 31.6286 + 0.00000699 \times Q_{P20.69} \quad ..24$$

$$H_{P24.63} = 32.0343 + 0.00000507 \times Q_{P24.63} \quad ..25$$

wherein,

Q_{Po}, $Q_{P5.81}$, $Q_{P10.81}$, $Q_{P20.69}$, and $Q_{P24.63}$ are same as described in section 5.4.3., H_{po} is the peak stage immediately below the dam in feet. $H_{p5.81}$, $H_{P10.81}$, $H_{P15.81}$, $H_{P20.69}$. and $H_{P24.63}$ are the peak stages at 5.81 miles, 10.81 miles, 15.81 miles, 20.69 miles and 24.63 miles below the dam respectively in feet.

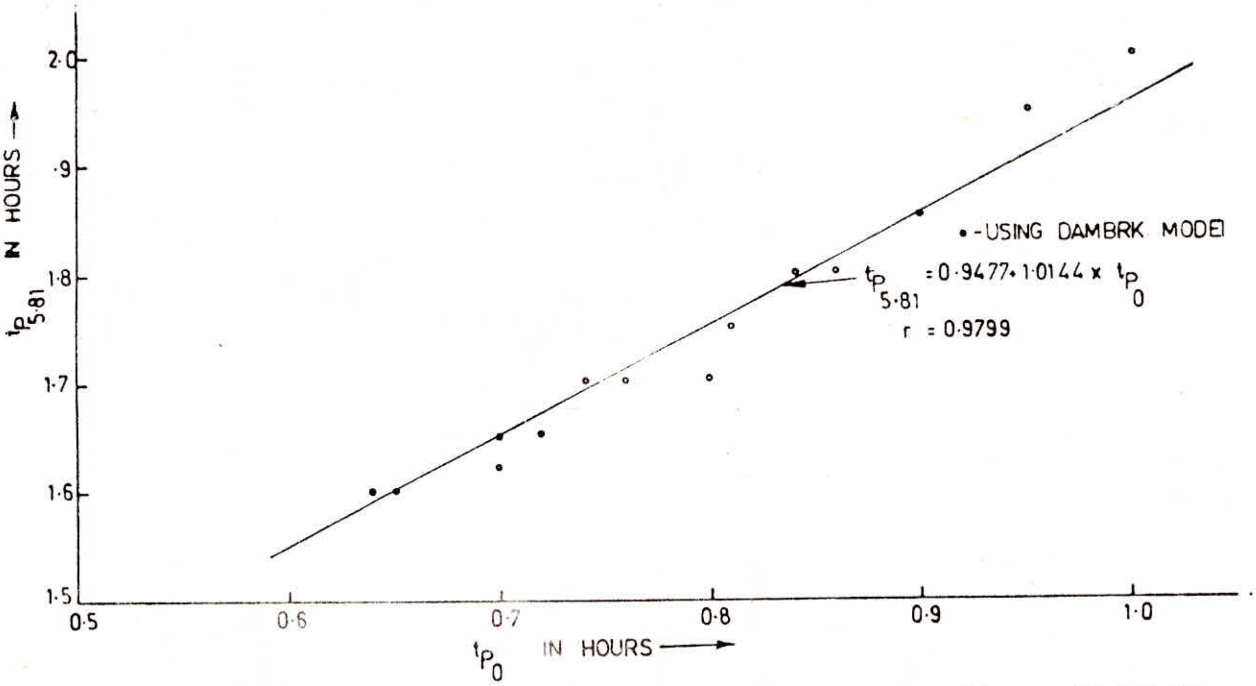
The estimates of slope and intercept for all the linear relationshipes described in section 5.4 have been arrived using least square approach. The linear relationships as established in equations 9, 15 and 20 have also been shown in plotted forms in figures 12,13,14 respectively. The other relationships can also be plotted in a similar manner.

The analysis as described above is useful in finding the dam break flood wave hydrographs and its characteristics at dam site and at specified location downstream of the Machhu dam-II for the range of breach sizes varying between 50% to 250% area with 100% breach area corresponding to the size of the actual breach as observed on 11th August, 1979 at time of disaster.

6. DISCUSSION OF RESULTS

6.1 The dimensionless hydrograph immediately below the dam site as shown in figure 4 is having sharp peak and the other dimensionless hydrographs below the dam site are having flatter peaks. The computed recession limb ordinates of the dimensionless hydrograph at the dam site, corresponding to various breach sizes, shows fluctuations about the mean curve drawn. This may be attributed to the tail water effect at the dam site and these fluctuations are not exhibited in the dimensionless hydrographs downstream of the dam site.

6.2 The relationship between area of breach and peak flow at dam site, peak discharge at upstream site and next downstream site, peak flow and time to peak flow at dam site, and peak flow and peak stage at respective sites as described earlier are the linear form of relationshipswith correlation coefficients greater



IG.13. RELATIONSHIP BETWEEN PEAK TIMES IMMEDIATELY BELOW THE DAM AND AT 5·81 MILES BELOW THE DAM

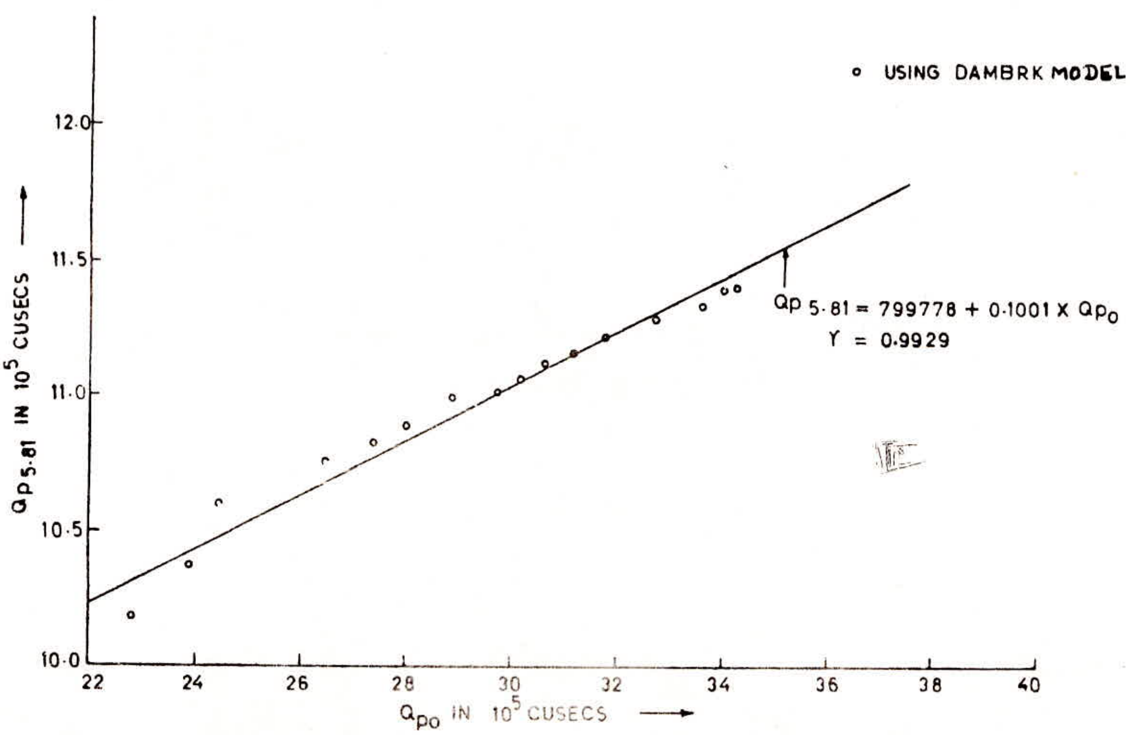


FIG. 12. RELATIONSHIP BETWEEN PEAK DISCHARGES IMMEDIATELY BELOW THE DAM AND AT 5·81 MILES BELOW THE DAM

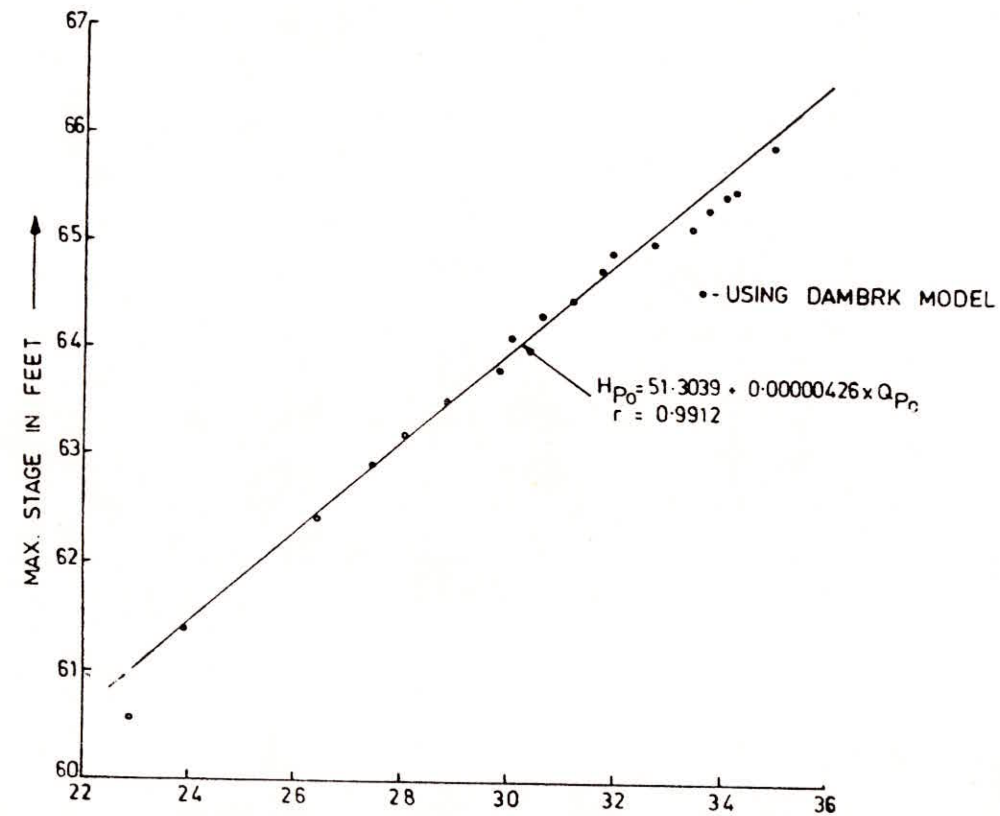


FIG.14. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE IMMEDIATELY BELOW THE DAM

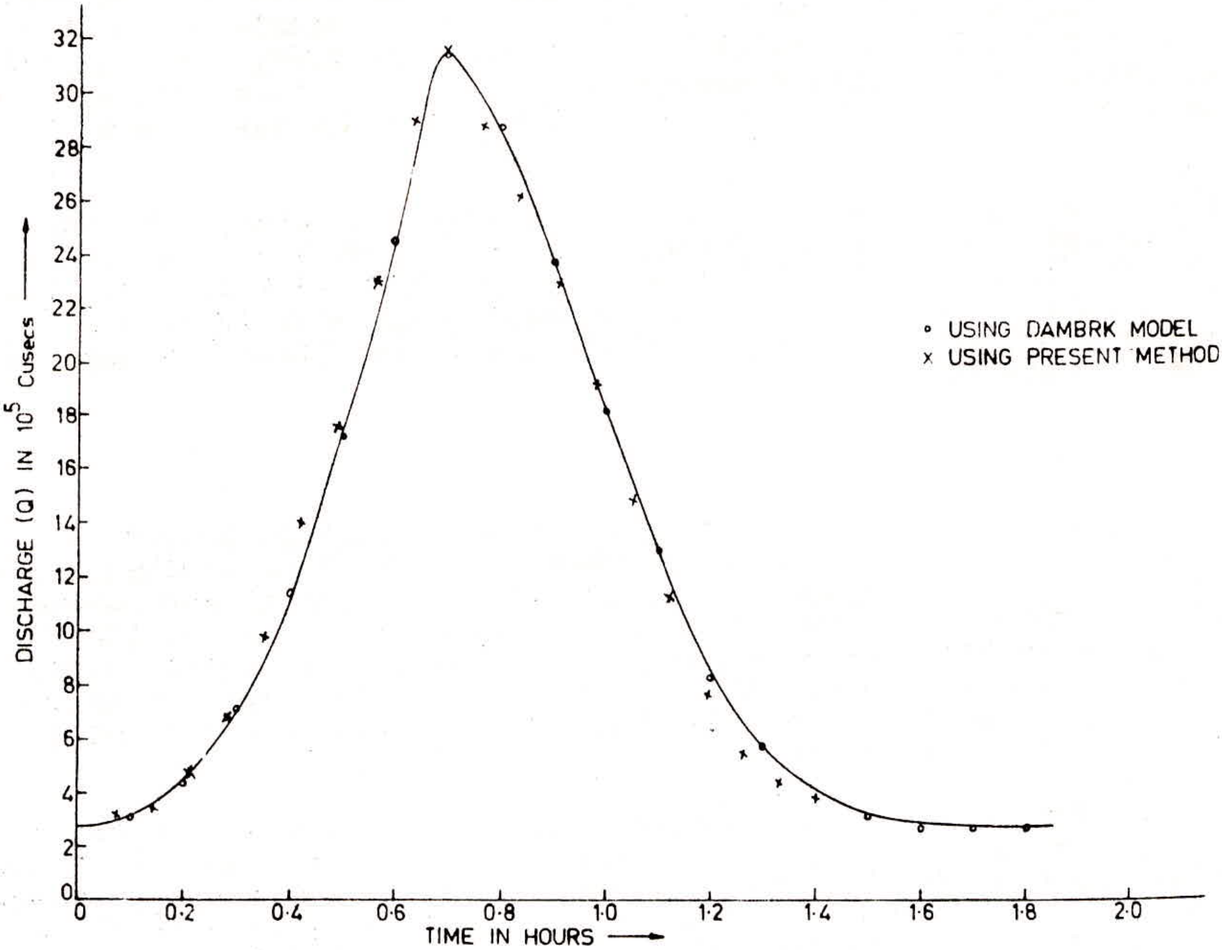


FIG. 15. DISCHARGE HYDROGRAPH IMMEDIATELY BELOW THE DAM

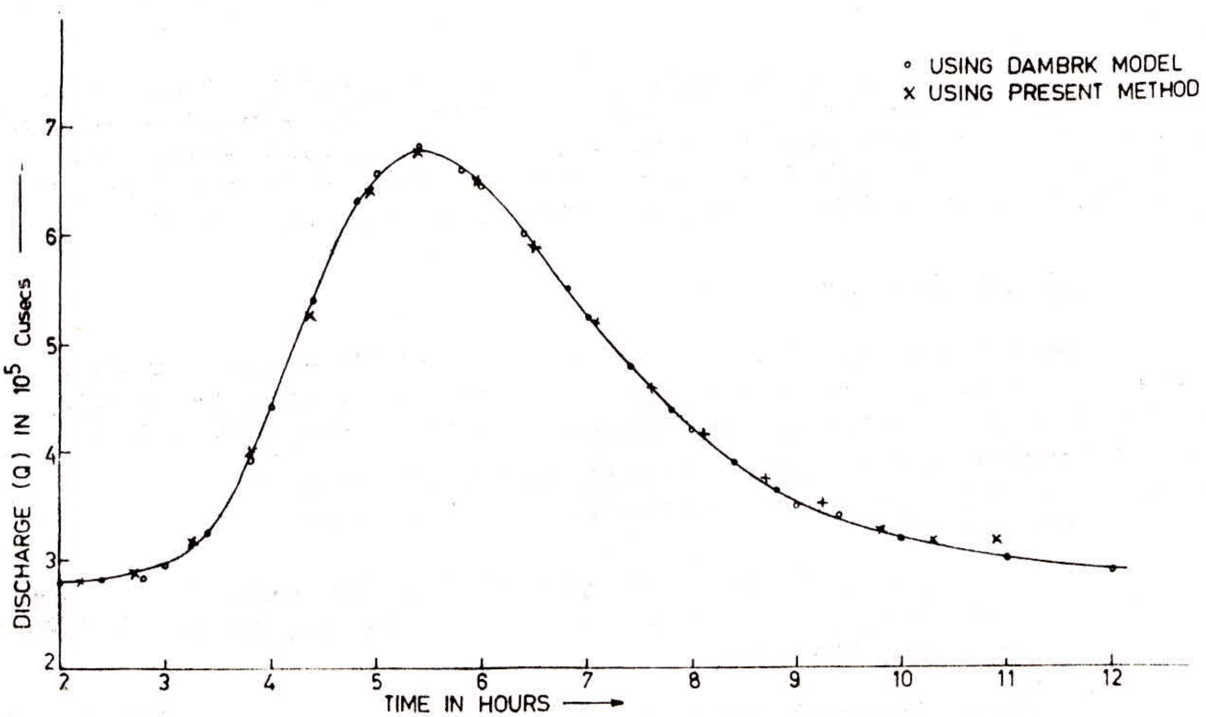


FIG. 16. DISCHARGE HYDROGRAPH AT 20·69 MILES BELOW THE DAM

than 0.99. These high correlation coefficients demonstrate the suitability of the form of the linear relationships adopted. The relationship between time to peak flow at upstream site and next downstream as described in section 5.4.5. is nearly a linear relationship with correlation coefficient as 0.94. For this relationship the deviations exhibited about the plotted mean line seem to be larger but they are very small in obsolute term of time.

6.3 Comparision of the outflow hydrographs obtained by the two different approaches as mentioned in the study are shown in fig. 15 and 16, immediately below the dam and at 24.63 miles downstream of the dam. These hydrographs are well comparable to each other which verifies the developed procedure in the present study and its usefulness.

7. CONCLUSION

The development of dimensionless hydrograph procedure as described in this study for Machhu Dam-II is of limited scope from the point of view of keeping all the variables and parameters same as adopted in the case of simulation of dam break flood wave due to disaster which struck the dam on 11th August 1979, except the breach area. In developing this procedure, no inflow was considered and it was assumed that the dam was overtopping at a water level of 198.5 ft.

The relationships arrived in the study are of immense use for practical purposes such as planning of nuclear installations downstream of the dam, issuing dam disaster flood warning, flood mapping etc. It may be inferred from the study, even if other variables and parameters are varying, besides the breach area size, the results obtained may vary quantitatively but not qualitatively.

The developed procedure has been verified by comparing the hydrograph arrived using this procedure for a breach area of 175% with 100% corresponding to the area of breach which occurred at the time of disaster on 11th August, 1979, with the hydrograph simulated using DAMBRK model for the same area of breach.

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