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

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ABSTRACT

This report 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 11th 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 inflow hydrograph. Therefore, the dimensionless hydrographs were developed based on 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-dimensionalised with reference to the time to peakflow 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 at 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 stage 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 which was not used for the development of dimensionless hydrographs and other relationships.

1.0 INTRODUCTION

Dam failures are often caused by over topping of the dam due to inadequate spillway capacity during large inflow to the reservoir from heavy precipitation runoff. Dam failures may also be caused by Seepage or piping through the dam or along internal conduits, slope embankment slides, earth quake damage and liquefaction of earthen dams from earth quakes and land slide generated waves in the reservoir. Usually the response time available for warning is much shorter than for precipitation-runoff-floods. The protection of the public from the consequences of dam failures has taken on increasing importance as population have concentrated in areas vulnerable to dam break disasters. This has created general interest in the dam safety analysis in recent years. The organisations which are responsible for the safety of dams should plan in such away so that in the eventuality of dam failures the disaster will not struck the lives of the public living downstream of the dam. Although many publications are available on dam break simulation problem since 1892 (Ritter, 1892) only a very few deal it with practical consideration. One of the unrealistic assumptions made in many of these publication is that the dam fails completely and instantaneously. These assumptions are somewhat appropriate for concrete arch type dams, but they are not appropriate for earthen dams. Earthen dams which exceedingly out number all other types of dams do not tend to completely fail nor do they fail instantaneously. The breach require a finite interval of time for its formation through erosion of the dam materials by escaping water.

One of the preventive measure in avoiding dam disaster is by issuing flood warning to the public of downstream where there is a dam failure. However it is quite difficult to conduct analysis and determine the warning time regarding dam break flood at the time of disaster. Therefore, predetermination of the warning time assuming a hypothetical dam break situation is a needed excercise in dam safety analysis.

For the range of breach parameters studied, the outflow peaks and flood stages downstream of the dam can be determined for regulatory and disaster prevention measures.

The purpose of this report is to develop dimensionless hydrographs which relate the time of hydrograph non-dimensionalised with reference to time to peak flow and instantaneous discharge non-dimensionalised with reference to the peak discharge at all the specific sites downstream of dam and at dam site, relationships between area of breach vs. peak flow, and peak flow vs. time to peak flow at dam site, relationships between peak flows of upstream site and the next downstream site, relationship between time to peak flow at upstream site and next downstream site, and the peak flow and peak stage at respective sites using the U.S. National Weather Services DAMBRK model for quick estimation of peak flow, time to peak flow, maximum flood stage and outflow hydrographs at dam site and at all the specific sites downstream of the dam for any breach area.

The procedure is verified by comparing the outflow hydrographs developed by DAMBRK model and dimensionless hydrographs method with breach area of 175%, which has not been used for the development of dimensionless hydrographs and other relationships stated above.

2.0 REVIEW

Problem of dam break analysis is one of the most fascinating hydraulic problems and the literature concerned with the study is extensive. There are number of models available now for the dam break analysis.

Ritter (1892) carried out dam break study using the method of characteristics to obtain a closed form solution for a dam of semi-infinite height extent upon a horizontal bed with zero bed resistance. However this study is not relevant from practical considerations as it has been shown that the neglect of bed resistance invalidates the Ritter Solution in a region that starts near the leading edge of flood wave. Dressler (1952) used a perturbation procedure to obtain a first order correction for resistance effect.

U.S. army corps of Engineers (1960) have recognised the need to assume partial rather than complete breaches, however they assumed the breach occurred instantaneously. Recognising this practical aspect Cristofano (1965), Harris and Wanger (1967) incorporated the partial time dependent breach formation in their dam break models. Sakkas and Strelkoff (1973), Chem and Armbruster (1980) have used the method of characteristics to obtain numerical solutions for dam break problems on slopping beds. These solutions were for reservoirs of finite length and included the effect of bed resistance. Investigations of dam break flood wave such as Ritter (1892), Re(1946), Dressler (1954), Stoker (1957), Su and Barnes (1969) and Sakkas and Strelkoff (1973) assumed that the breach encompasses the entire dam and that it occurs instantaneously. Ralph (1985) presented a comparative evaluation of alternative state of the art methods for predicting the flow characteristics of a flood wave resulting from a breached dam. Wurbs (1987) 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 Engineers 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. DAMBRK, FLOW SIM 1 and FLOW SIM 2 are dynamic routine models. FLOW SIM 1 and 2 are identical, except that FLOW SIM 1 uses and explicit and FLOW SIM 2 an implicit finite difference scheme.

Wurbs (1987) stated that a dynamic routing should be used whenever 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 and so attention herein would be focussed on DAMBRK model and the dam break model as adopted in HEC-1 Flood Hydrograph Package.

For reasons of simplicity, generality, wide applicability and uncertainity in the actual failure mechanism, the DAMBRK model allows to input the failure time interval and terminal size and shape of breach. The possible shapes of breaches which can be accomplished by the DAMBRK model are rectangular, triangular and trapezoidal. DAMBRK model uses St. Venant's equations for routing dam break floods in channels. This model has the capacity of adopting either storage routing or dynamic routing methods for routing floods through reservoirs depending on nature of the flood wave movement in reservoir at the time of failure. After computing the outflow hydrograph the extent of and the time of occurance 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 (St. Venant's equations) is the appropriate technique to route the dam break flood hydrograph through downstream valley. The applicability of the St. Venant's equation to simulate abrupt waves such as the dam break wave has been demonstrated by Terzides and Strelkoff (1970) and by Martine and Zorne (1971)

The U.S. Army Corps of Engineers HEC-1 dam break model (HEC, 1981) adopts storage routing technique for routing of flood through reservoirs as well as through channels. Singh and Snorrason (1983) carried out dam break flood studies for Teton dam with DAMBRK and HEc-1 model and found that the ability of DAMBRK model to simulate the maximum water level profile is better than the HEC dam break model, attributing the reason for channel routing technique adopted in the respective models.

Gundalach and Thomas (1977) analysed the dam break flood from Tetan dam using a generalised unsteady flow computer program to determine water surface elevations resulting from various breach sizes and n-values. The analysis used data generally available to field personnel engaged in the study of the impact of a dambreak flood, such as topographic maps, aerial photography, dam description, gaged streams flows and reservoir elevation capacity curves. The channel configuration upstream and downstream of the dam was modelled independently according to the generalised computer program (GEDA) (HEC, 1976). Data were coded in the standard HEC-2 (1973) format and input in GEDA programme. The unsteady flow data models of reservoir and channel were developed according to the generalised computer program, (HEC-1976). Using the above generalised program and selecting appropriate n-values, Gundlach and Thomas found that neither the size of breaches tested (30 to 40% of the dam size) 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 break 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 break flood wave formation. Sakkas (1980)developed dimensionless graphs using the non-dimensional form of St. Venant's equations, for computing the time of arrival of the wave front, maximum flood level and time of occurrence of the maximum flood level downstream of the dam site assuming instantaneous breach of dam section, initially dry channel and with rectangular, triangular and parabolic shapes of channel cross sections. The assumption used in developing dimensionless graphs, that the dam fails instantaneously is conservative in the sense of simulating the worst possible downstream flooding condition, but in most cases it is unrealistic. Observations of the past dam failures have indicated that earthen dams do not fails instantaneously. Failure of a concrete arch dam is the case in which the assumption of complete instantaneous removal of the dam is most likely to approximate the reality. Further the assumptions of initially dry channel and prismatic cross section are also unrealistic. To alleviate the impact of unrealistic assumptions on dimensionless graphs developed by Sakkas and make it more realistic from the practical consideration an attempt has been made in the present study, to develop dimensionless hydrographs 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.

The development of dimensionless hydrographs consists of relating t/t_p and Q/Q_p, relationships based on area of breach vs. peak flow at dam site, peak flow of upstream site and the next downstream site, peak flow and time to peakflow, and peak flow and peak stages at respective sites. Such an estimation of dam break flood wave characteristics would be useful for flood warning purposes and for planning nuclear installations downstream of a dam, flood mapping etc.

3.0 STATEMENT OF THE PROBLEM

The objective of the study is to develop dimensionless hydrographs based on t/t_p and Q/Q_p relations at dam site and at specific downstream sections of the breached dam, wherein t and Q are time and intantaneous discharge respectively, Q_p and t_p are peak flow and time to peak flow of the dam of the dam break flood wave hydrograph arrived from DAMBRK model for the sites where dimensionless hydrographs are desired and relate area of breach and peakflow at dam site, peak flow of upstream site and the next downstream site, peak flow and time to peak flow at dam site, time to peakflow of upstream site and next downstream site and peak flow and peak stage at dam site and at respective sites downstream of dam corresponding to various breach sizes of Machhu Dam-II, assuming the structural details of the dam remaining same as it was before the disaster of 11th August, 1979.

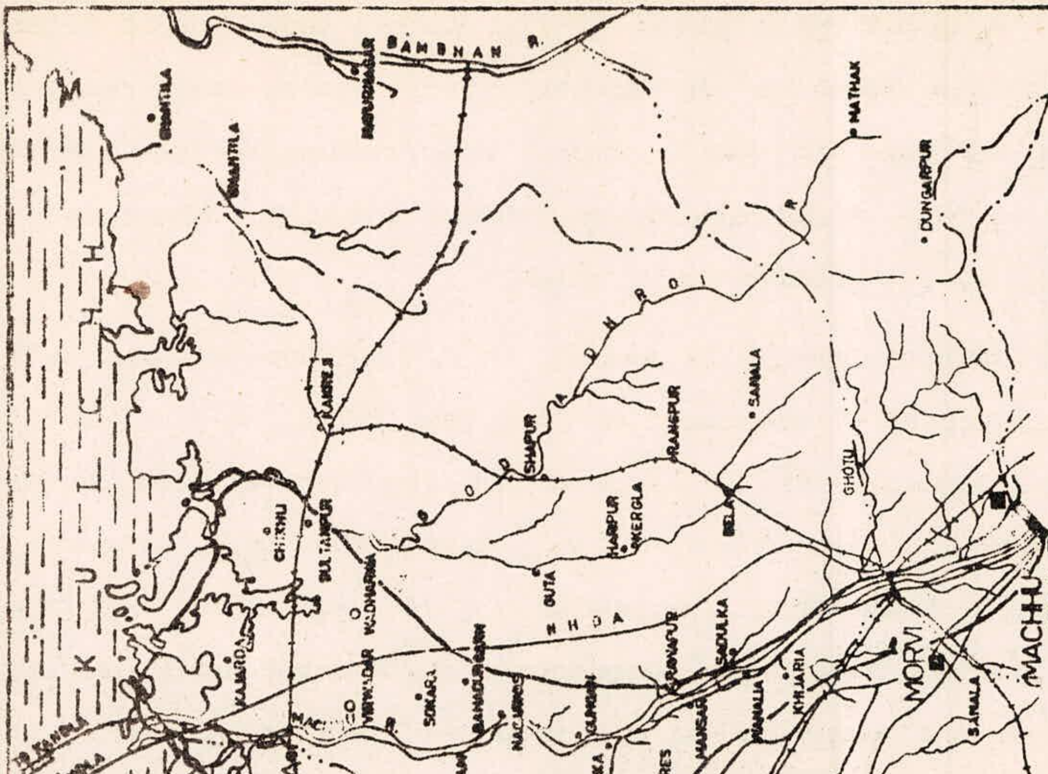
4.0 DESCRIPTION OF THE STUDY AREA

A brief description of the Machhu Dam-II with reference to its location on Machhu river basin, the relevant 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 46' North and the longitude 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. 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 Ranna of Kutch. An index map showing the flood affected area due to Machhu dam-II failure is shown in Fig-1.

The relevant design aspects of the dam are given below:

Type of dam	Masonary spillway with dam falnks 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.



- BOUNDARY LINE OF THE FLOOD AFFECTED AREA

SCALE:-1"=4 MILES

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

Shape of spillway	Ogee
Crest of spillway	RL 168 ft.
Spillway design flood	2,18,330 cusecs
Details of the radial gates of the spillway	10 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 Aug. 1979, leading to sustained overtopping of the dam by the flood water for nearly two hours. At 1.30 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 disaster. 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. Transportation network was damaged in these areas as the railway tracks and National Highways were breached due to overtopping by flood water.

5.0 DATA AVAILABILITY FOR THE STUDY

The input data required in FPS system for the National Weather Services 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 (A c.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 availability 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 1.30PM 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 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.

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 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.

5.2 Second Data Group

The second group of data pertaining to the routing of the outflow hydrograph through the downstream valey 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 table 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.

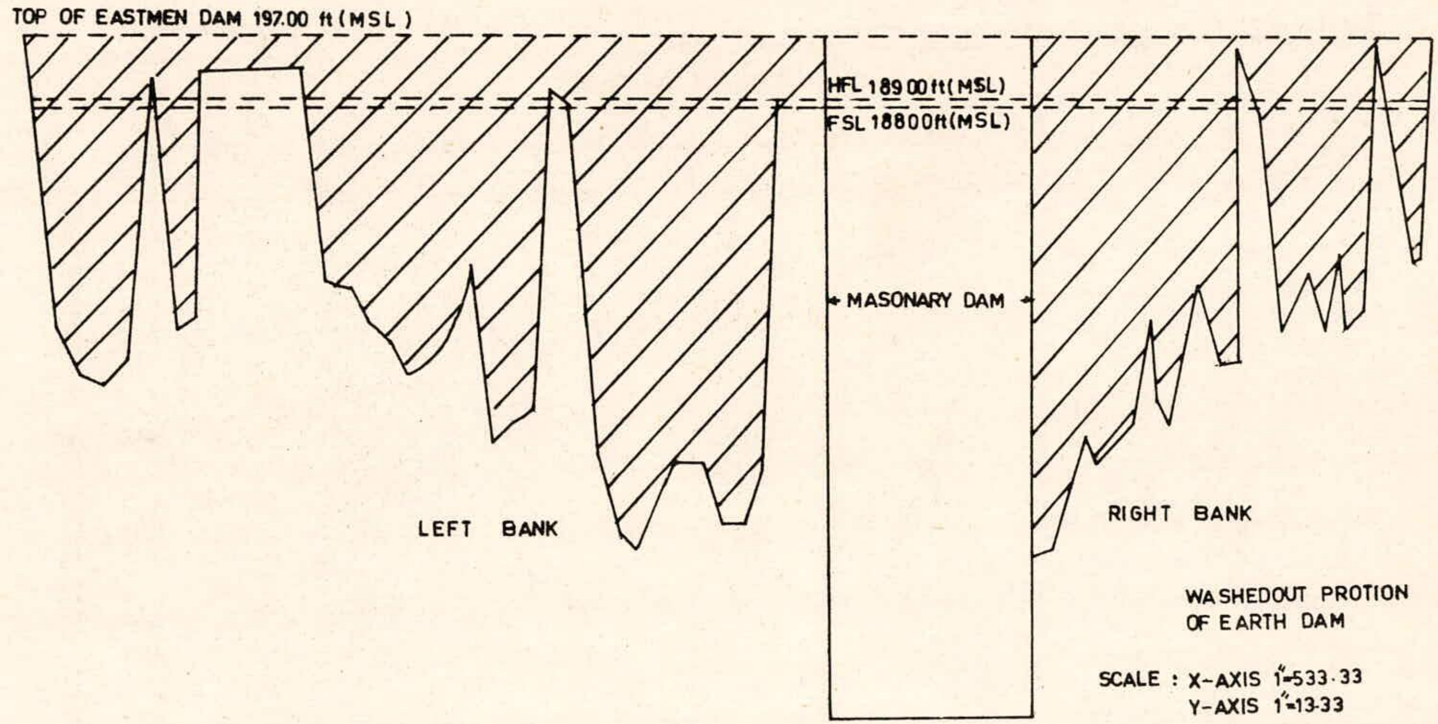


FIG. 2. BREACH PROFILE OF MACHHU DAM – II

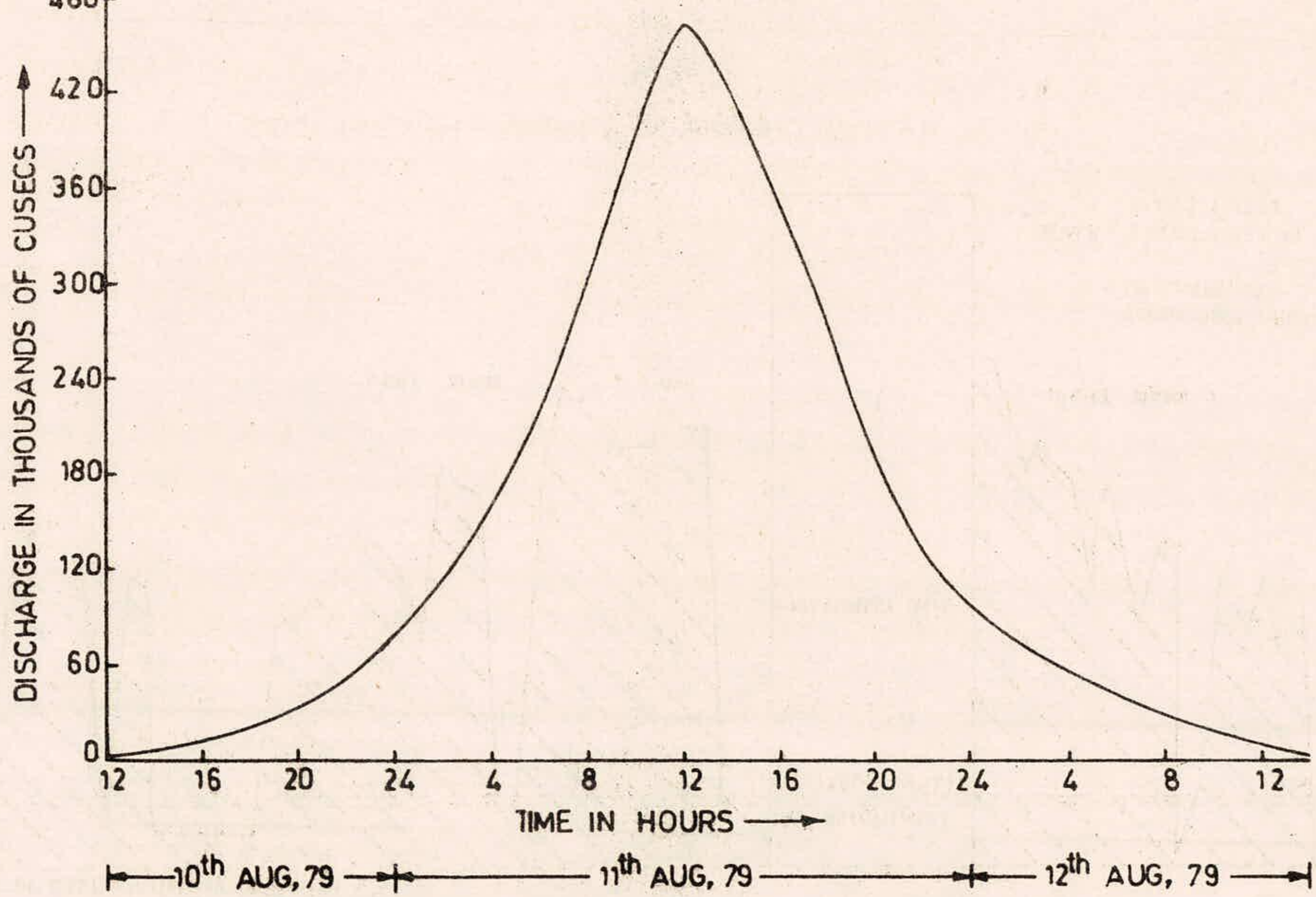


FIG.3 - ADOPTED RESERVOIR INFLOW HYDROGRAPH FOR DAM BREAK ANALYSIS OF MACHHU DAM - II

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- Δt and the notation Δ approximates the differential. The term S may be expressed as:

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

in which A_s is the reservoir surface area confidential with 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.

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:

i) An overtopping failure in which the breach is simulated as a rectangular, triangular, or trapezoidal 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.

ii) 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 parameter, 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 the 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 tall 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 the dimensional open channel flow, alternatively known as St. Venant's equations. These equations consists 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:

$$S_f = \frac{n^2 |Q| Q}{2.21 \; A^2 R^{4/3}} \quad \text{...(6)}$$

and

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

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 equaion. 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 sub-critical flow in the reach or the upstream reach having supercritical flow and downstream reach having subcritical 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.

6.4 Breach Shape

In case of an over topping failure, DAMBRK model can simulate the breach shape as rectangular, trapezoidal or triangular as described in section 6.2. In order to assess the impact of different shapes of breaches in the outflow hydrograph characteristics, DAMBRK model was analysed for three different shapes of breach. Bottom level of the breach, depth of flow over the dam, breach area and other conditions were kept same for the three different shapes of breach. Thus the results obtained, using the DAMBRK model, for peak flow, time to peak flow, and maxm. stage at downstream sites of the dam were compared as given below in table.2. It can be inferred from table 2 that there is not considerable different in the values of peak flows, time to peak flows and max. stage, for the rectangular, triangular and trapezoidal shapes of breaches. It is also seen from the study of the case histories of dam failure (Mac-Donald) and Monopolis, 1984) that the breach shape occured during the dam failures were mostly trapezoidal. Therefore, a trapezoidal shape of breach has been adopted for development of dimensionless hydrographs and other relevant relationships for estimation of dam break flood wave characteristics for Machhu Dam-II.

6.5 Assessment of Dam Break Flood Wave Characteristics With and without Inflows

In case of hypothetical dam break studies for estimation of flood wave characteristics the real inflow hydrograph is not known and the consideration of design flood hydrograph is not appropriate.

In the present study dam break analysis is done with and without inflow, to assess the effect of inflow on flood wave characteristics. The inflow hydrograph used for the analysis is described in section 5.1.5, and the breach shape adopted is trapezoidal as discussed in section 6.4. The results obtained for the flood wave characteristics with & without inflow are given in table 3. It can be inferred from the results shown in table 3 that there is no significant difference in the values of max. stage computed with & without inflow consideration & similarly for peak flow and time to peak flow at dam site and at specific sites downstream of the dam. Though the inflow volume routed through the reservoir is comparable with that volume of water stored behind the dam, it has not contributed much for the increase of maximum stage significantly when compared with the maximum flow stage formed due to the passing of volume of water stored behind the dam at the time of failure of the dam. The possible reason which can be attributed for this small difference may be that the released water behind the dam due to the failure has occupied a wider area of flood plain downstream of the dam, and the routed inflow

Table : 2

Peak Discharge, Peak Stage and Time to Peak Flow Computed by DAMBRK Model for Actual Breach Area with Inflow Hydrograph

Distance below the Dam (miles)	Trapezoidal Breach			Rectangular Breach			Triangular Breach		
	Peak Discharge (Cusecs)	Max Stage (ft.)	Time to peak discharge (hours)	Peak discharge (Cusecs)	Max. Stage (ft.)	Time to peak discharge (hours)	Peak discha-rge (Cusecs)	Max. stage (ft.)	Time to peak discha-rge (hrs.)
0	3097537	65.51	0.85	3043820	65.26	0.85	3183232	65.82	0.875
5.81	1275920	49.95	1.90	1270054	49.90	1.90	1279687	49.97	1.925
10.81	1017895	59.29	3.15	1015102	59.25	3.15	1020092	59.34	3.175
15.81	896510	52.83	4.65	891525	52.79	4.60	901280	52.87	4.675
20.69	847614	37.47	5.95	840729	37.42	5.95	851860	37.49	5.975
24.63	816288	36.14	7.0	809223	36.10	6.95	820434	36.16	6.975

Table : 3

Salient Features of Flood Wave Using DAMBRK Model for Actual Breach Area of Machhu Dam-II with Trapezoidal Breach Shape

Location of site down-stream of the dam (miles)	With inflow to the Reservoir				Without inflow to the reservoir		Peak discharge ratio Q_1/Q_2	Max. stage ratio S_1/S_2	Time to peakflow ratio T_1/T_2
	Peak discharge (Cusecs) Q_1	Max stage (ft) S_1	Time to peakflow (Hrs) T_1	Peak discharge (Cusecs) Q_2	Max. state (ft) S_2	Time to peakflow (Hrs) T_2			
0.0	3097537	65.51	1.0	2731498	63.20	0.85	1.134	1.036	1.176
5.81	1275920	49.95	1.90	1082108	47.62	1.80	1.179	1.048	1.055
10.81	1017895	59.29	3.15	871099	57.26	2.82	1.168	1.035	1.117
15.81	896510	52.83	4.65	730177	51.31	4.145	1.227	1.029	1.121
20.69	847614	37.47	5.95	668576	36.30	5.525	1.267	1.032	1.076
24.63	816288	36.14	7.0	636315	35.26	6.623	1.282	1.025	1.056

hydrograph volume through the breached down has spread over and above the vest flood plain already submerged, thus causing a small increase in the maximum stage realised over and above the maximum stage realised due to only the release of stored water behind the dam.

6.6 Dimensionless Hydrograph and the Relationships

The dam break flood wave hydrographs were simulated using DAMBRK model for various breach sizes to obtain the values of peak flow time to peak flow and max. stage at 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 as discussed in section 6.4 and no inflow hydrograph was routed through the reservoir, as discussed in section 6.5. 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 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.

iii) Peakflow of upstream site & next downstream site

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

(v) Time to peakflow 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.

7.0 ANALYSIS

The section describes the Machhu Dam-II failure analysis carried out using the DAMBRK model programme for the actual breach area as observed during the failure on 11th Aug. 1979. 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 analysis 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., 6ft., and 4 ft.) since 1.30 a.m of 11th August 1979 until 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 USSR 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 : 4

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 required 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 occured 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 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.

The variation of model breach flow from the actual breach flow due to the variation of shape, breaches occured 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 occuring 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 thre 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 sections that 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 & for 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 occurence 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.

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 Aug. 1979 during which time the flow in the channel may be considered to be approximately steady. But in such circumstances, the uncertainity 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 reaches 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 such 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 anlysis. It was considered that the effect of so assumed initial condition on the peakflow of routed hydrograph would be small when compared with the effect of uncertainities of the inflow information of the reservoir, had the analysis started prior to 1.30 p.m. of 11th Aug. 1979.

The outflow hydrograph from the breached dam formed the upstream boundary condition and the downstream boundary condition was given by the channel control. Under 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 falure 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, and variables describing channel characteristics as briefed in section 7.1, 7.2, and 7.3, 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 5.

Table : 5

Salient Features of the Flood Wave from Machhu Dam-II Failure

Sl.No. No.	River Mile from Machhu Dam-II	Peak discha-arge(in 10^5 cusecs)	Peak flood eleva-tion in ft.		Time to peak flood eleva-tion as on 11.8.79 (Computed)
			Computed	Observed	
1.	0.0	30.98	186.66	-	2.30 PM
2.	0.81	28.30	180.00	168.40	2.35 PM
3.	4.81	12.76	136.76	131.00	3.51 PM
4.	10.81	10.18	103.88	101.00	5.03 PM
5.	15.81	8.97	80.56	77.00	6.21 PM
6.	20.69	8.48	55.16	53.00	7.36 PM
7.	24.60	8.16	39.05	-	8.39 PM

For further details the reader may refer 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.

7.4 Development of Dimensionless Hydrograph Procedure

This section describes the development of dimensionless hydrograph procedure for the dam break analysis of Machhu Dam-II corresponding to a given range of breach sizes varying from 50 to 250% area with 100% area corresponds to the actual breach size as observed on 11th August, 1979. Corresponding to different breach sizes at the time of disaster only the side slope of breach was varying and the remaining breach parameters such as bottom level of the breach and bottom width of breach were assumed same as that occurred on 11th August, 1979. Besides, the over topping depth was assumed as 1.5 ft. above the top of the dam, same as it was observed at the time of disaster. The breach shape was considered as trapezoidal according to the earlier stated reason in section 6.5 and also no inflow was considered entering the reservoir at the time of failure.

7.4.1 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 are shown in figures 4,5,6,7,8 and 9 corresponding to the sections immediately below the dam and at section 5.81 miles, 10.81 miles, 15.81 miles, 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 characteristic of the site considered has been arrived at for the range of breach sizes studied as mentioned in section 7.2. 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.

7.4.2 Relationship between area of breach and peakflow at dam site.

The relationship between area of breach and peak

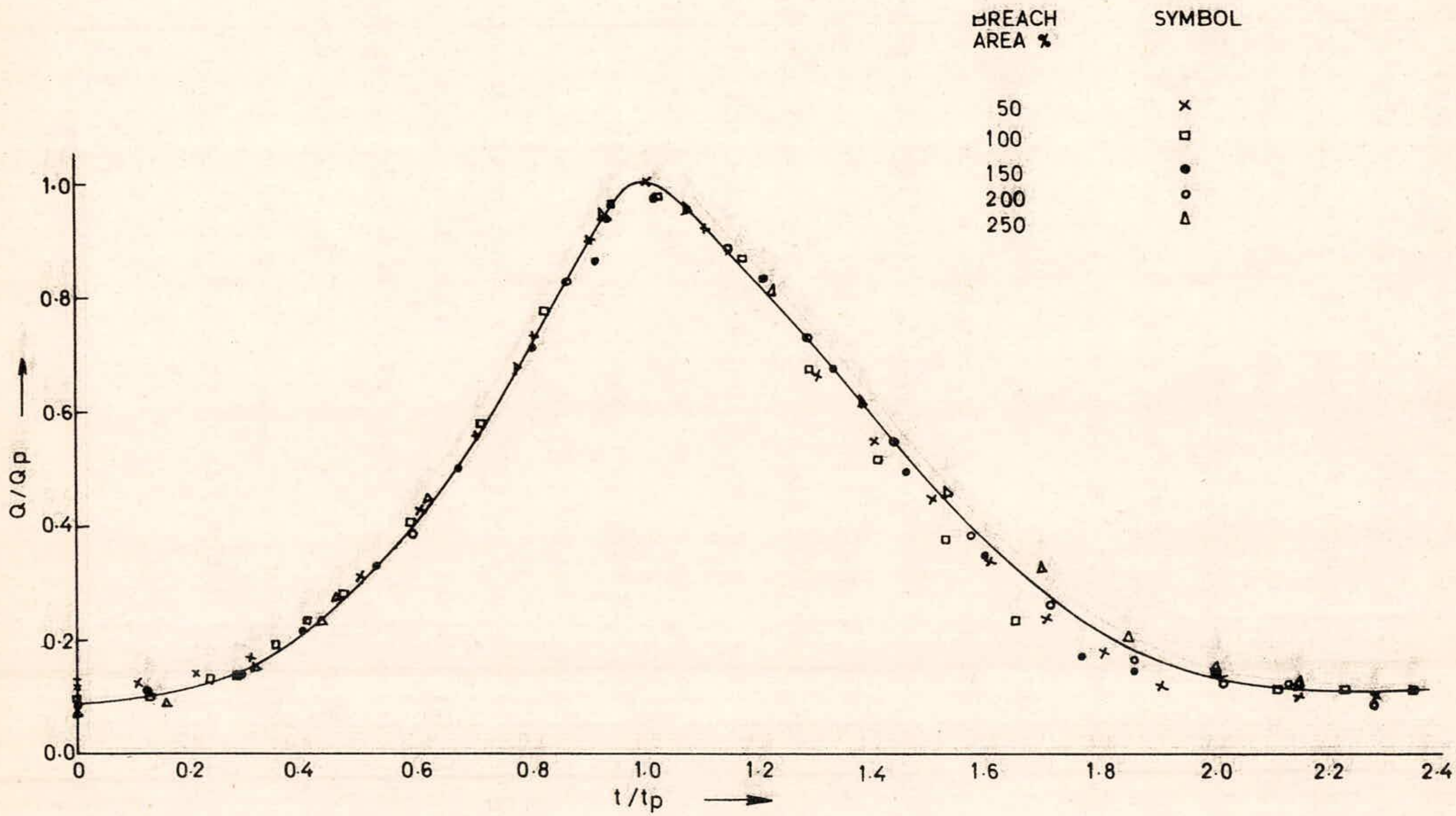


FIG. 4. NON-DIMENSIONAL HYDROGRAPH IMMEDIATELY BELOW THE DAM

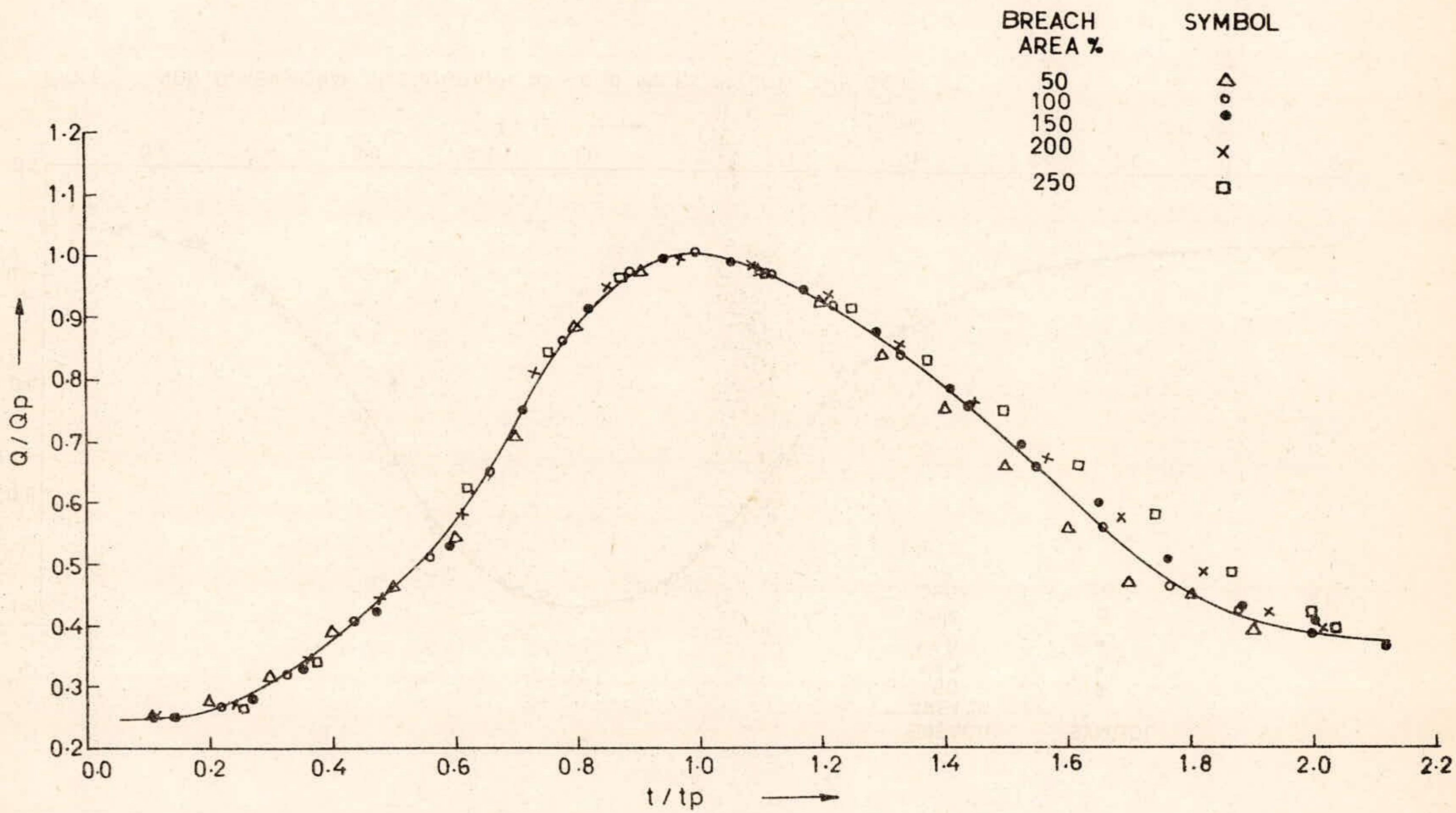


FIG. 5. NON-DIMENSIONAL HYDROGRAPH AT 5·81 MILES BELOW THE DAM

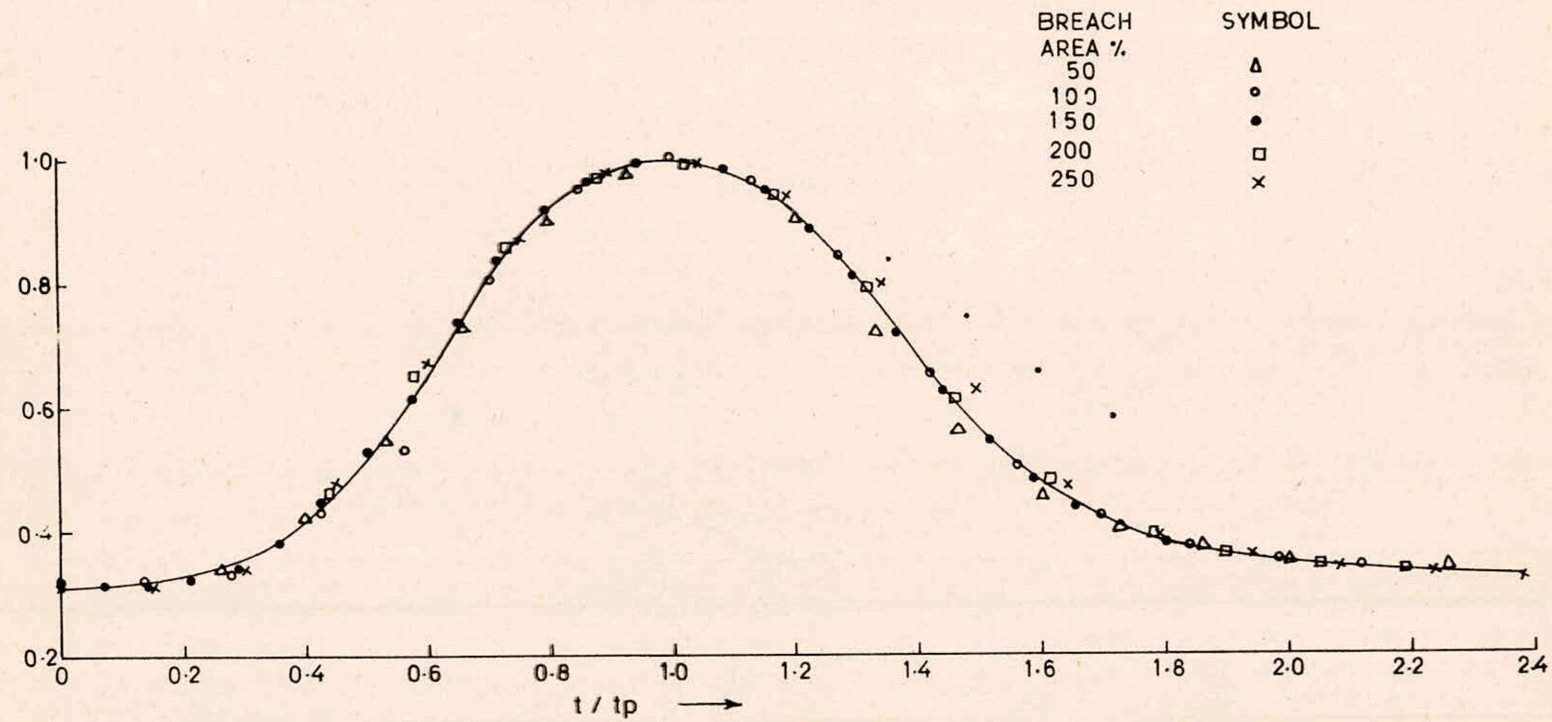


FIG. 6. NON-DIMENSIONAL HYDROGRAPH AT 10·81 MILES BELOW THE DAM

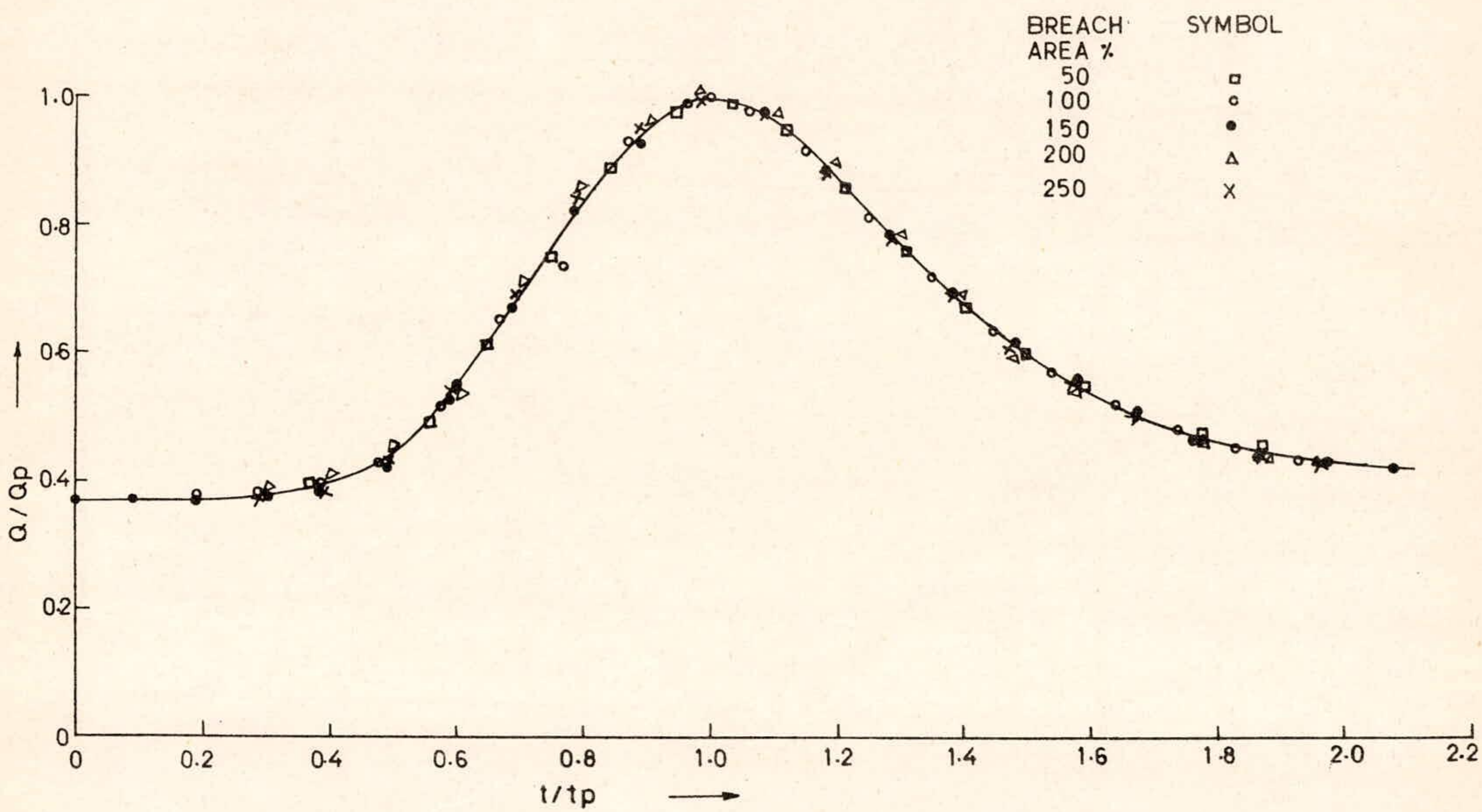


FIG. 7. NON-DIMENSIONAL HYDROGRAPH AT 15 81 MILES BELOW THE DAM

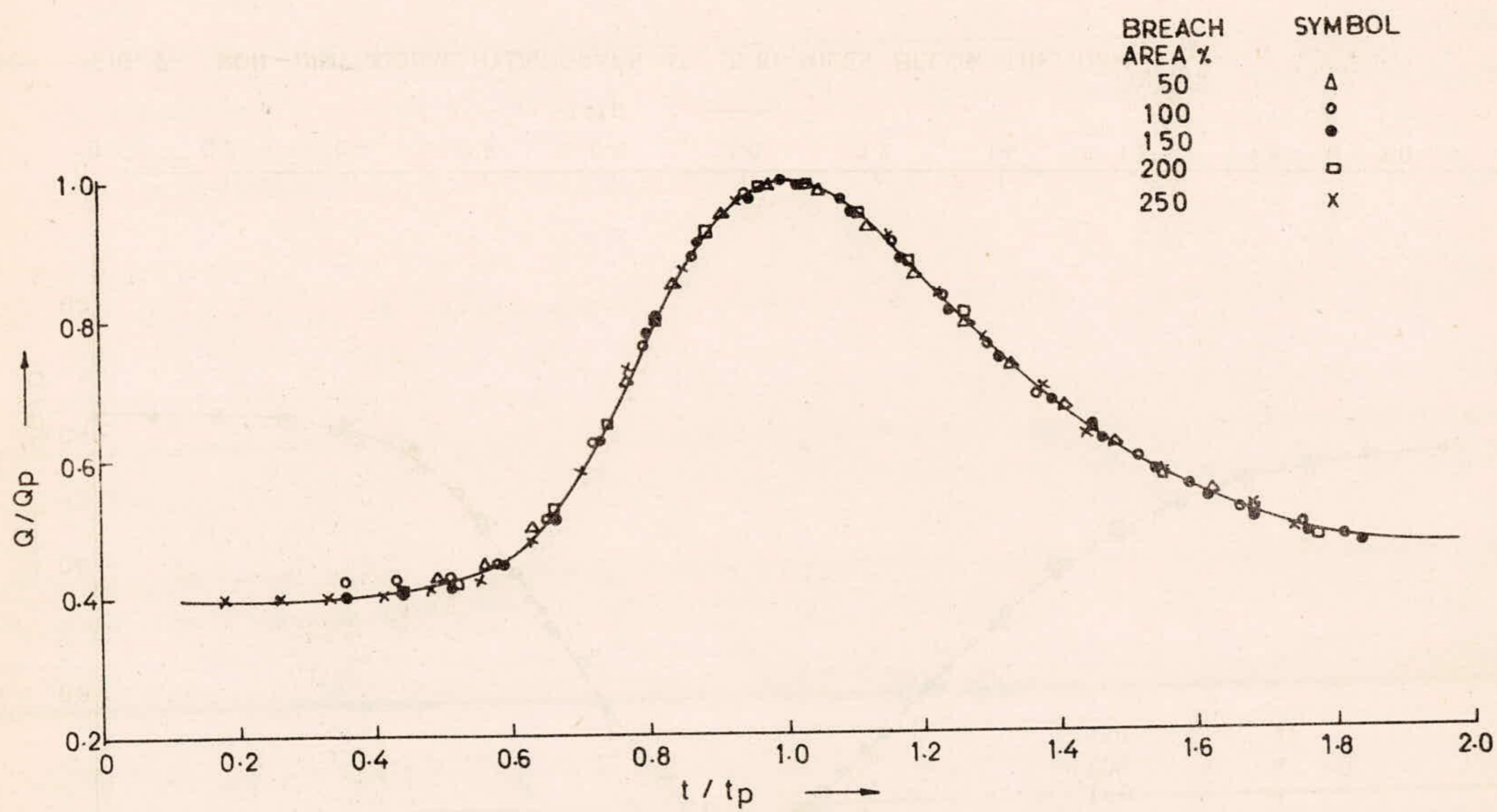


FIG. 8. NON - DIMENSIONAL HYDROGRAPH AT 20 69 MILES BELOW THE DAM

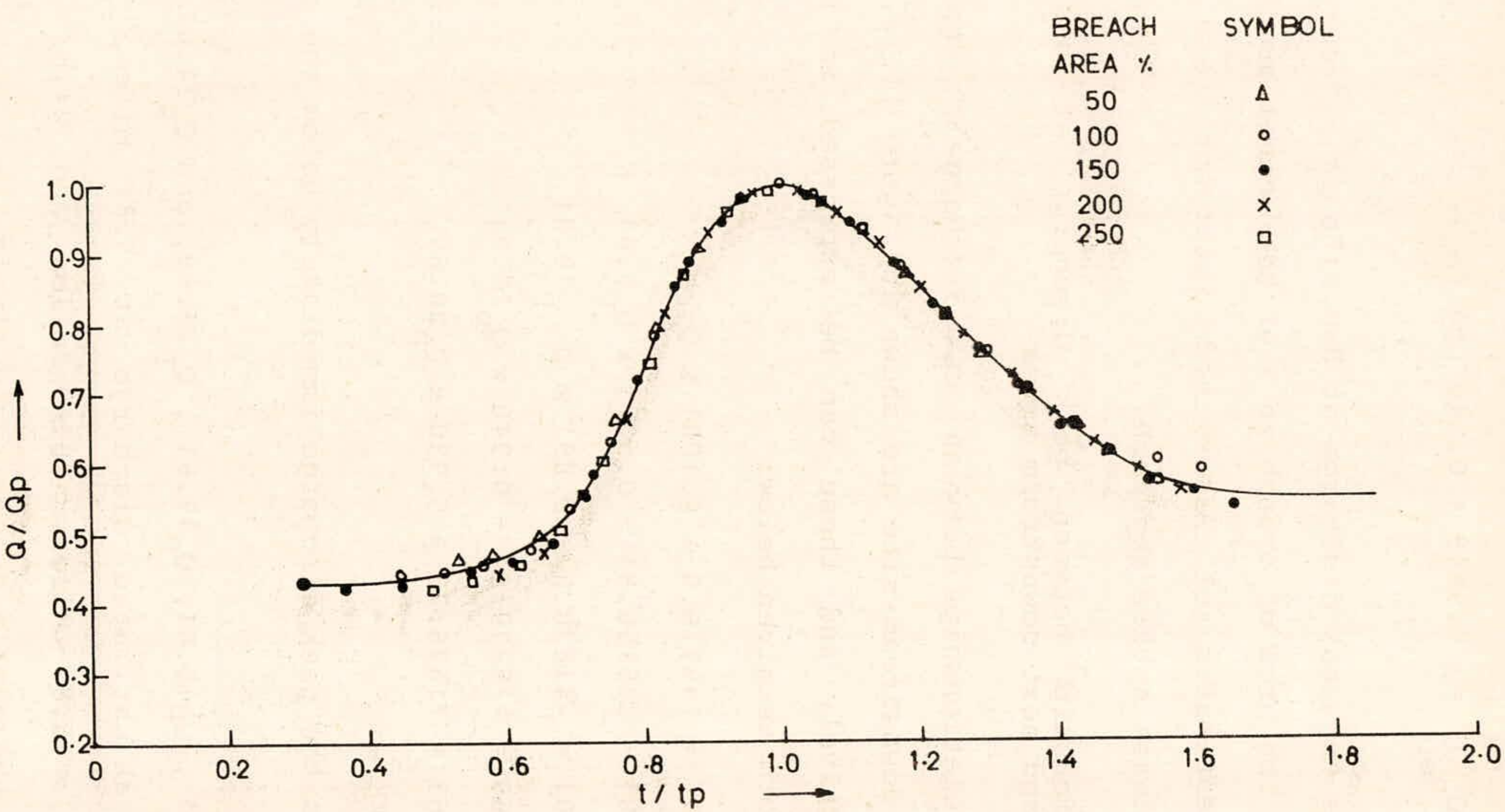


FIG. 9. NON-DIMENSIONAL HYDROGRAPH AT 24.63 MILES BELOW THE DAM

flow in log domain at dam site is shown in fig. 10 and it can be expressed in linear relationship form as given below:

$$\text{Log}\ (Q_{p_o}) = 5.9649 + 0.240\ \text{Log}\ (A_b) \qquad \ldots(8)$$

wherein

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

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

The estimates of slope and intercept have been arrived using least square approach.

7.4.3 Relationship between peak discharge at upstream site and next downstream sites

The relationships between peak discharge at upstream site and next downstream site are shown in figure 11,12,13,14 and 15 respectively and these can be expressed in linear relationship form as given below:

$$Q_p5.81 = 799778.0 + 0.1001 \times Q_{po} \qquad \ldots(9)$$

$$Q_p10.81 = 295156.81 + 0.5321 \times Q_p5.81 \qquad \ldots(10)$$

$$Q_p15.81 = -41830.0 + 0.891 \times Q_p10.81 \qquad \ldots(11)$$

$$Q_p20.69 = -18179.44 + 0.940 \times Q_p15.81 \qquad \ldots(12)$$

$$Q_p24.63 = 11676.75 + 0.930 \times Q_p20.69 \qquad \ldots(13)$$

where,

Q_{p_o} is the peak discharge immediately below the dam in cusecs

$Q_p5.81$, $Q_p10.81$, $Q_p15.81$, $Q_p20.69$, 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.

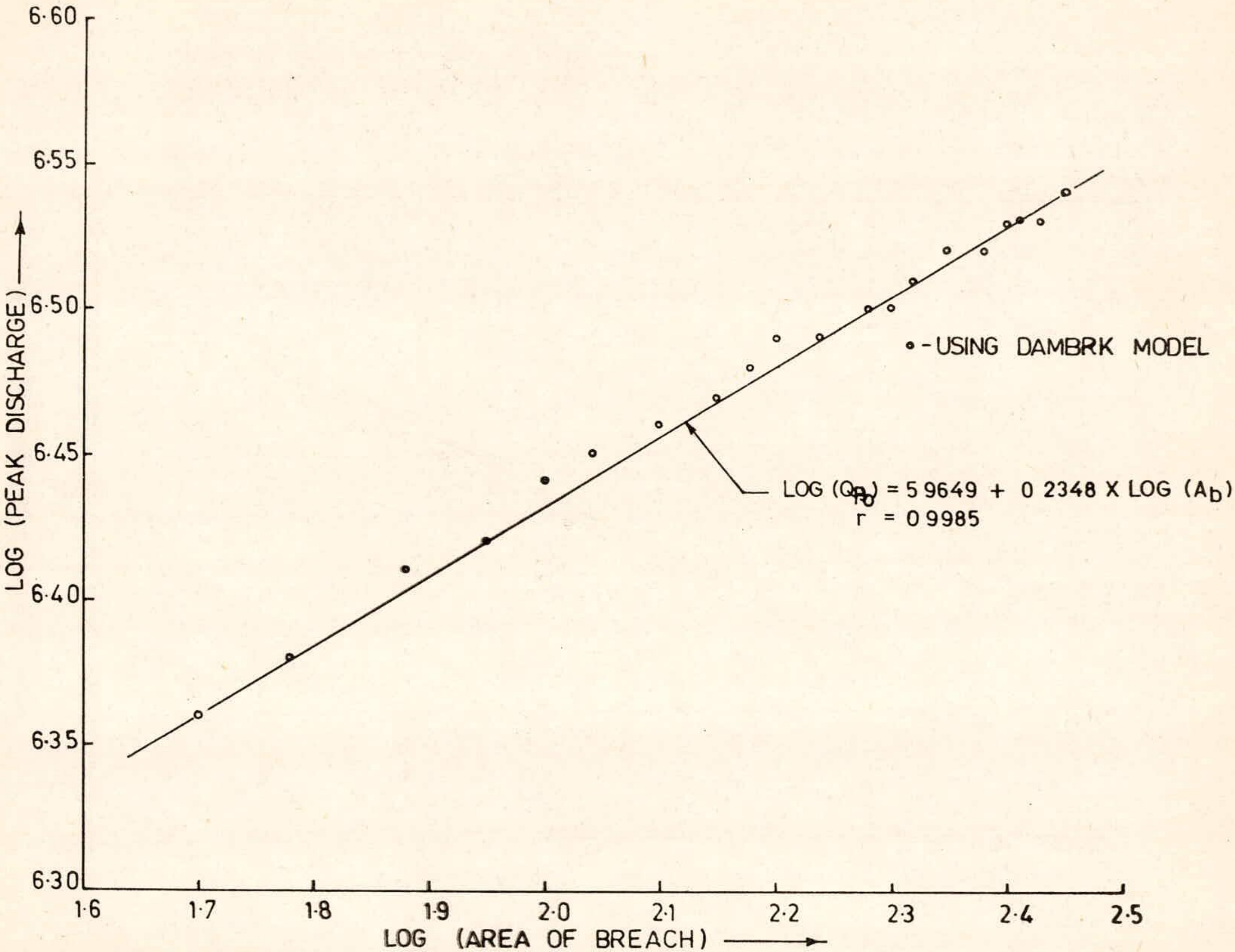


FIG.10. RELATIONSHIP BETWEEN LOG (AREA OF BREACH) AND LOG (PEAK DISCHARGE) IMMEDIATELY BELOW THE DAM

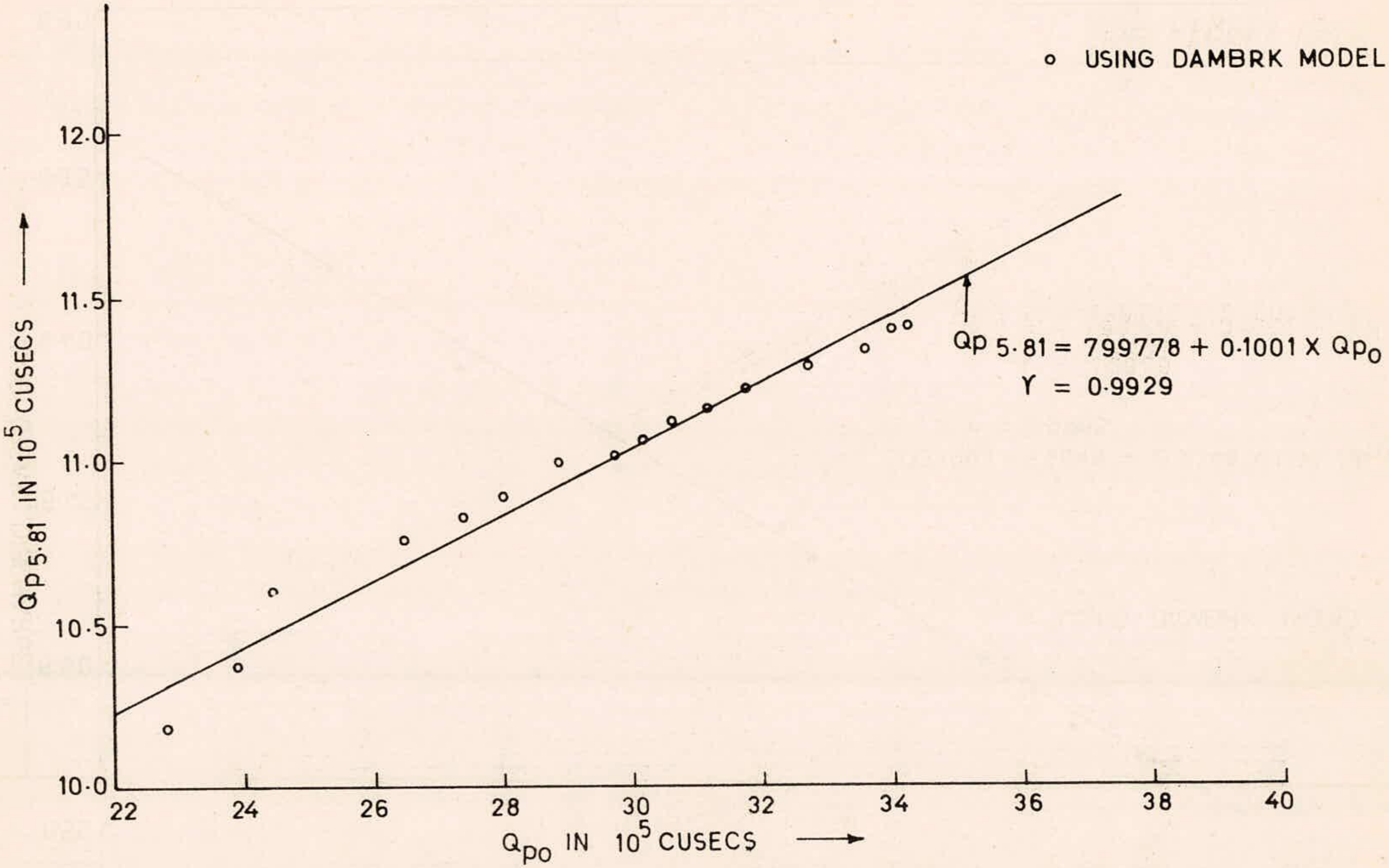


FIG. 11. RELATIONSHIP BETWEEN PEAK DISCHARGES IMMEDIATELY BELOW THE DAM AND AT 5.81 MILES BELOW THE DAM

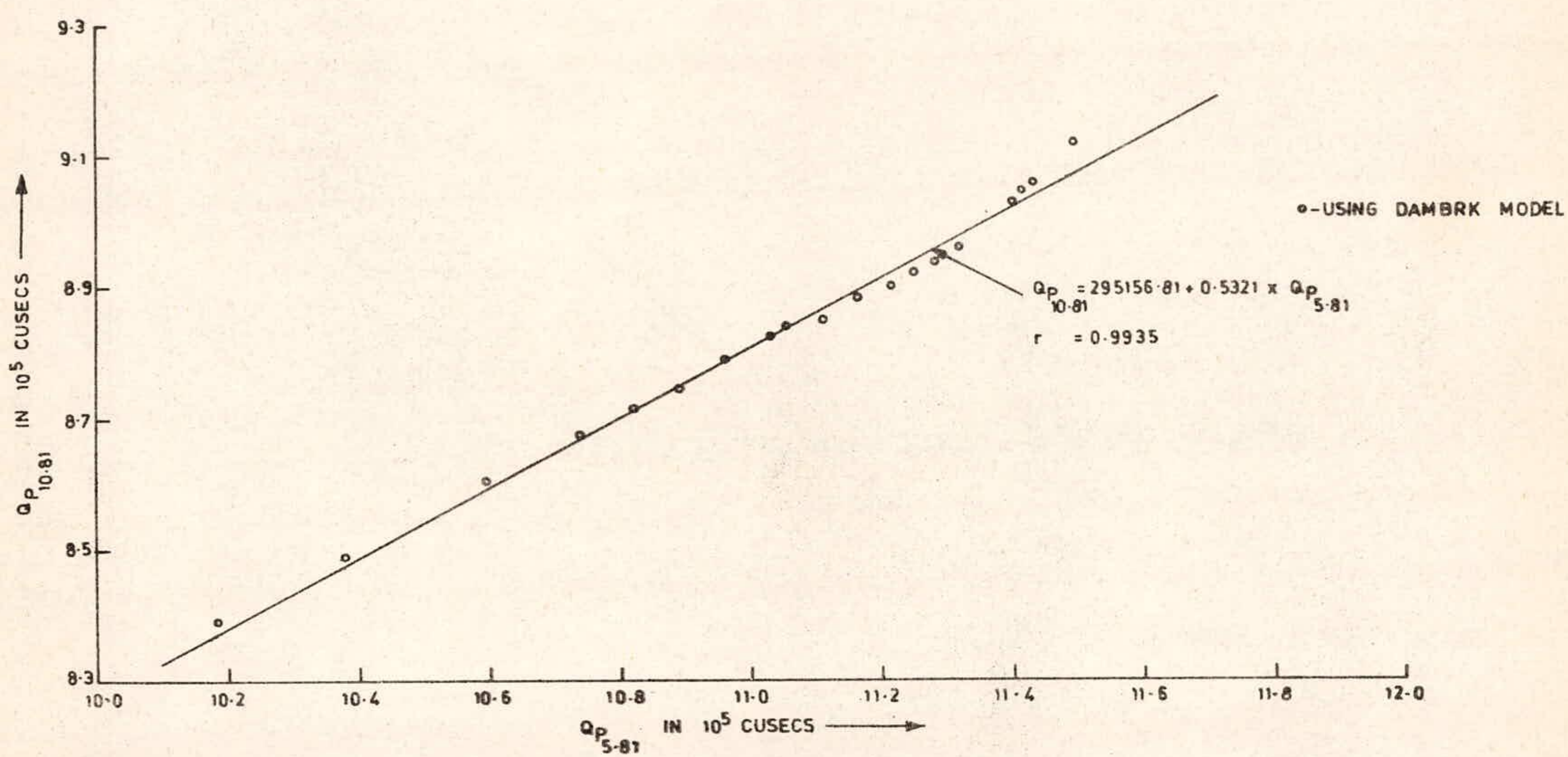


FIG.12. RELATIONSHIP BETWEEN PEAK DISCHARGES AT 5·81 MILES AT 10·81 MILES BELOW THE DAM

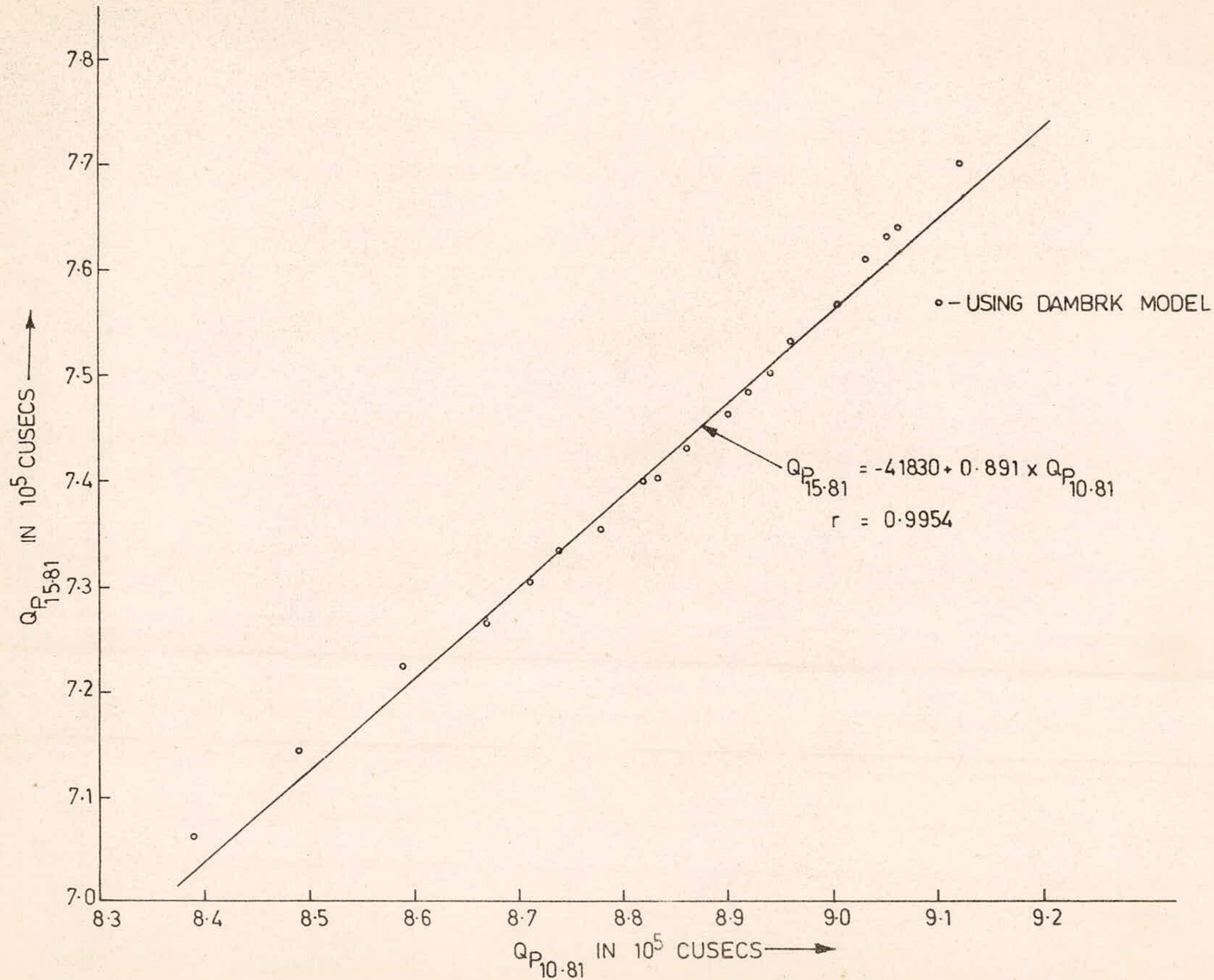


FIG.13. RELATIONSHIP BETWEEN PEAK DISCHARGES AT 10·81 MILES AND AT 15·81 MILES BELOW THE DAM

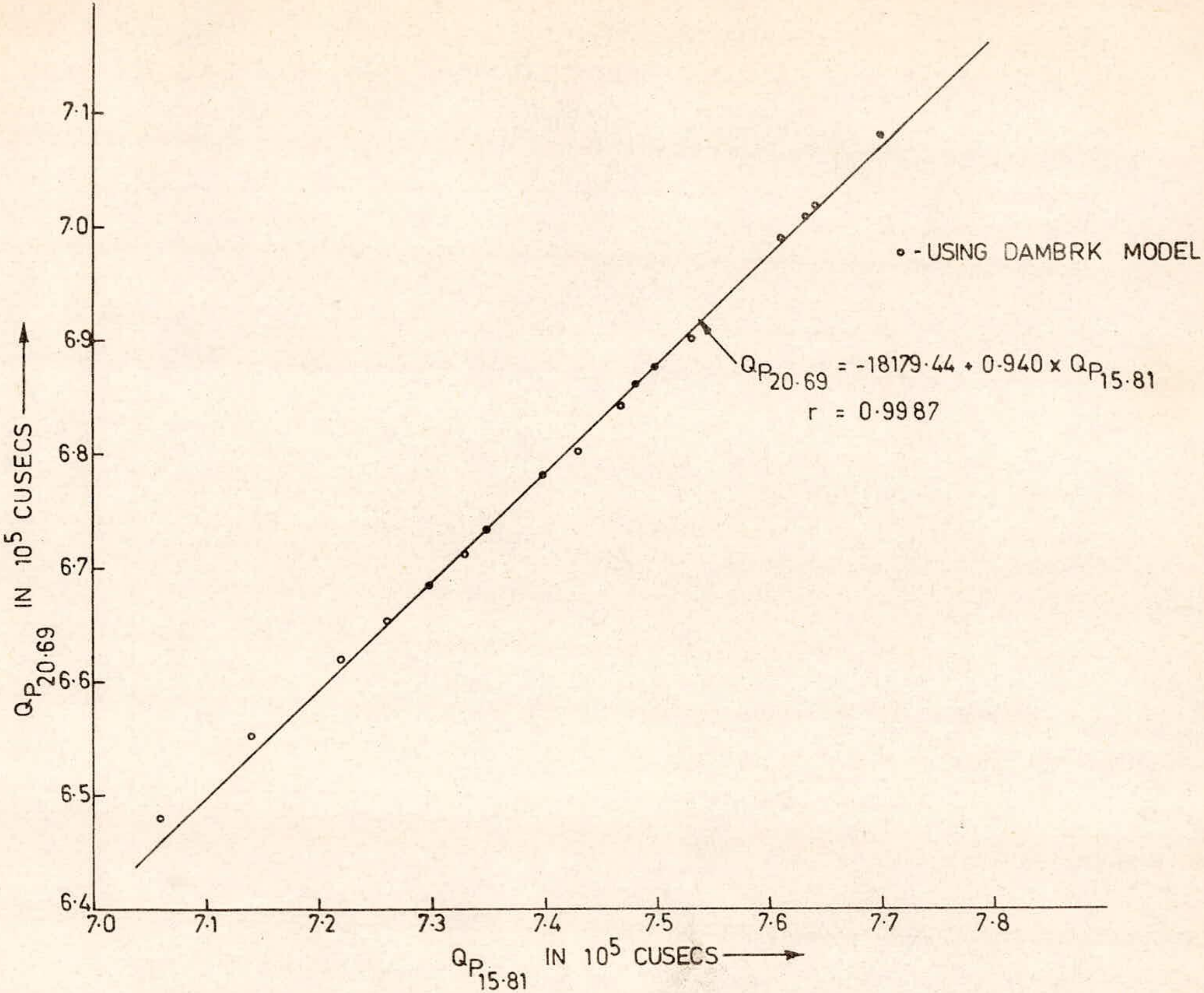


FIG.14. RELATIONSHIP BETWEEN PEAK DISCHARGES AT 15·81 MILES AND AT 20·69 MILES BELOW DAM

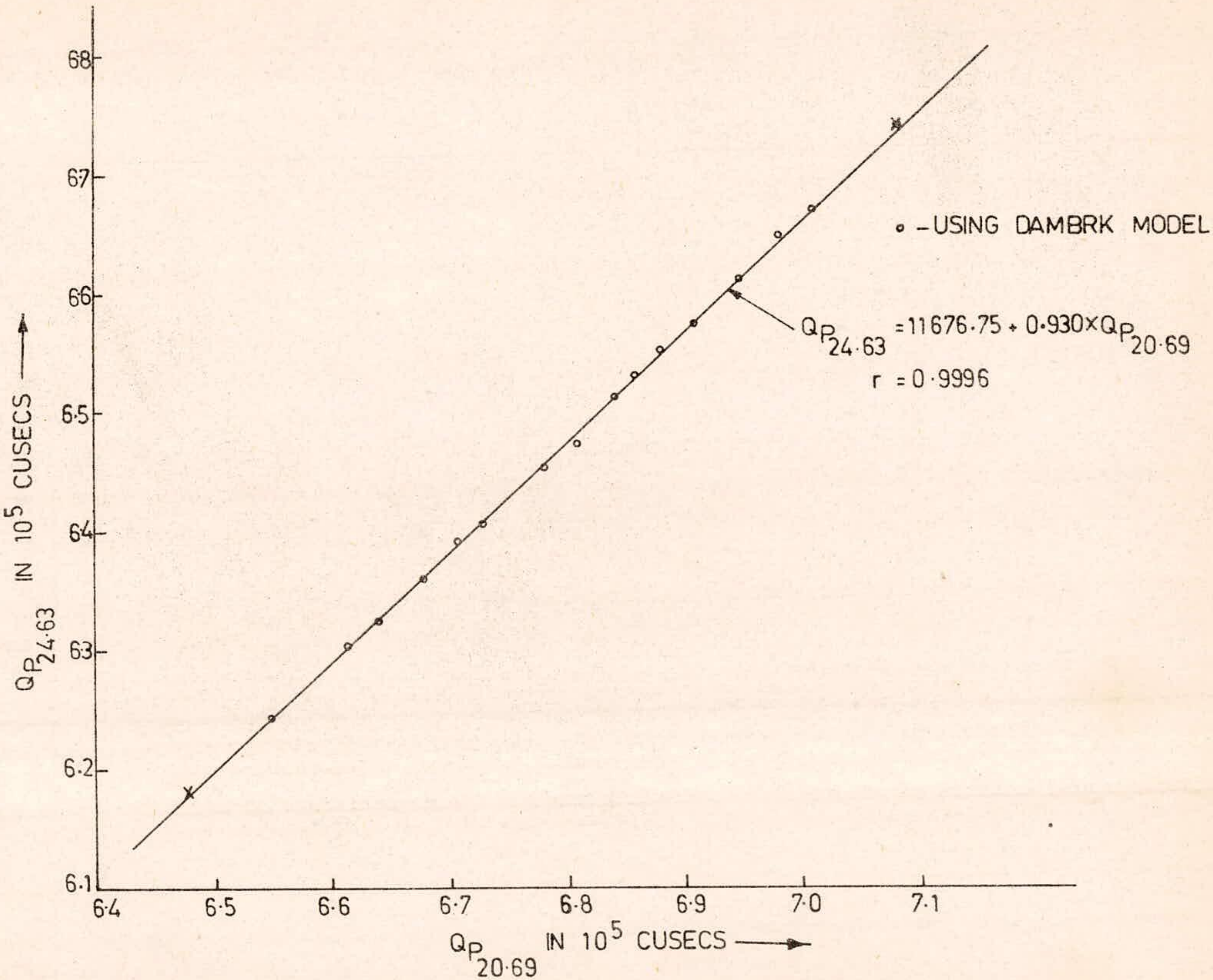


FIG. 15. RELATIONSHIP BETWEEN PEAK DISCHARGES AT 20·69 MILES AND AT 24·63 MILES BELOW THE DAM

7.4.4 Relationship between peak flow and time to peak flow at dam site

Relationship between peakflow and time to peakflow is shown in figure 16 and it can be expressed in linear relationship form as given below:

$$t_{po} = 1.7442 - 0.00000033\ Q_{po} \qquad \ldots(14)$$

where

Q_{po} is the peakflow at dam site in cusecs

t_{po} is the time to peakflow at dam site in hours

The estimates of slope and intercept have been arrived using least square approach.

7.4.5 Relationship between time to peak flow at upstream site and next downstream site

Relationship between time to peak flow at upstream site and next downstream site are shown in figure 17,18,19,20 and 21 and these can be expressed in linear relationship form as given below:

$$t_{p\ 5.81} = 0.9477 + 1.044 \times t_{po} \qquad \ldots(15)$$

$$t_{p\ 10.81} = 1.3781 + 0.8137 \times t_{p5.81} \qquad \ldots(16)$$

$$t_{p\ 15.81} = 1.9618 + 0.7693 \times t_{p10.81} \qquad \ldots(17)$$

$$t_{p\ 20.69} = 0.8159 + 1.1387 \times t_{p15.81} \qquad \ldots(18)$$

$$t_{p\ 24.63} = 1.8712 + 0.8538 \times t_{p20.69} \qquad \ldots(19)$$

Where,

t_{po} is the time to peakflow immediately below the dam in hours

$t_{p5.81}$, $t_{p\ 10.81}$, $t_{p\ 15.81}$, $t_{p\ 20.69}$ 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

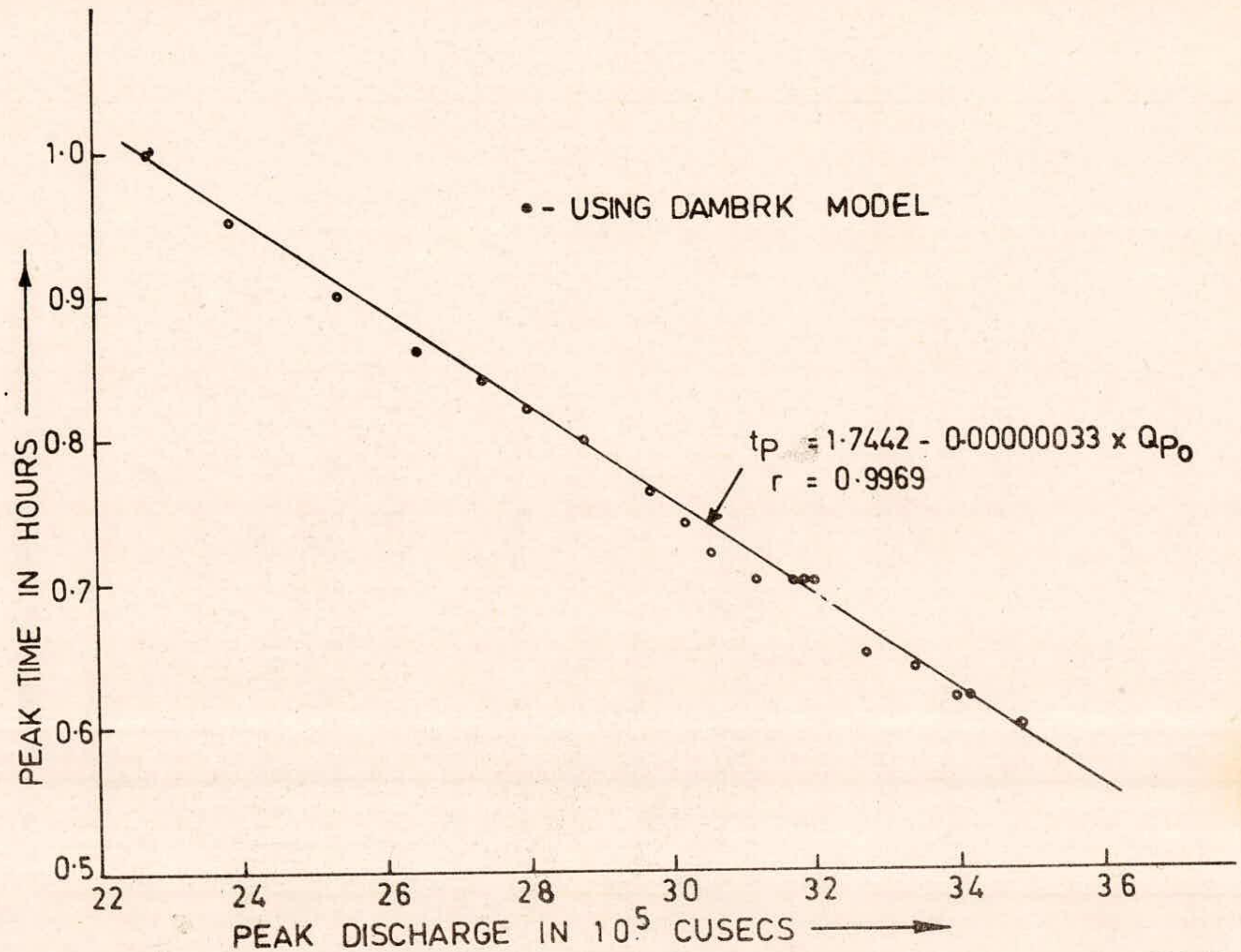


FIG. 16. RELATIONSHIP BETWEEN PEAK DISCHARGE AND PEAK TIME IMMEDIATELY BELOW THE DAM

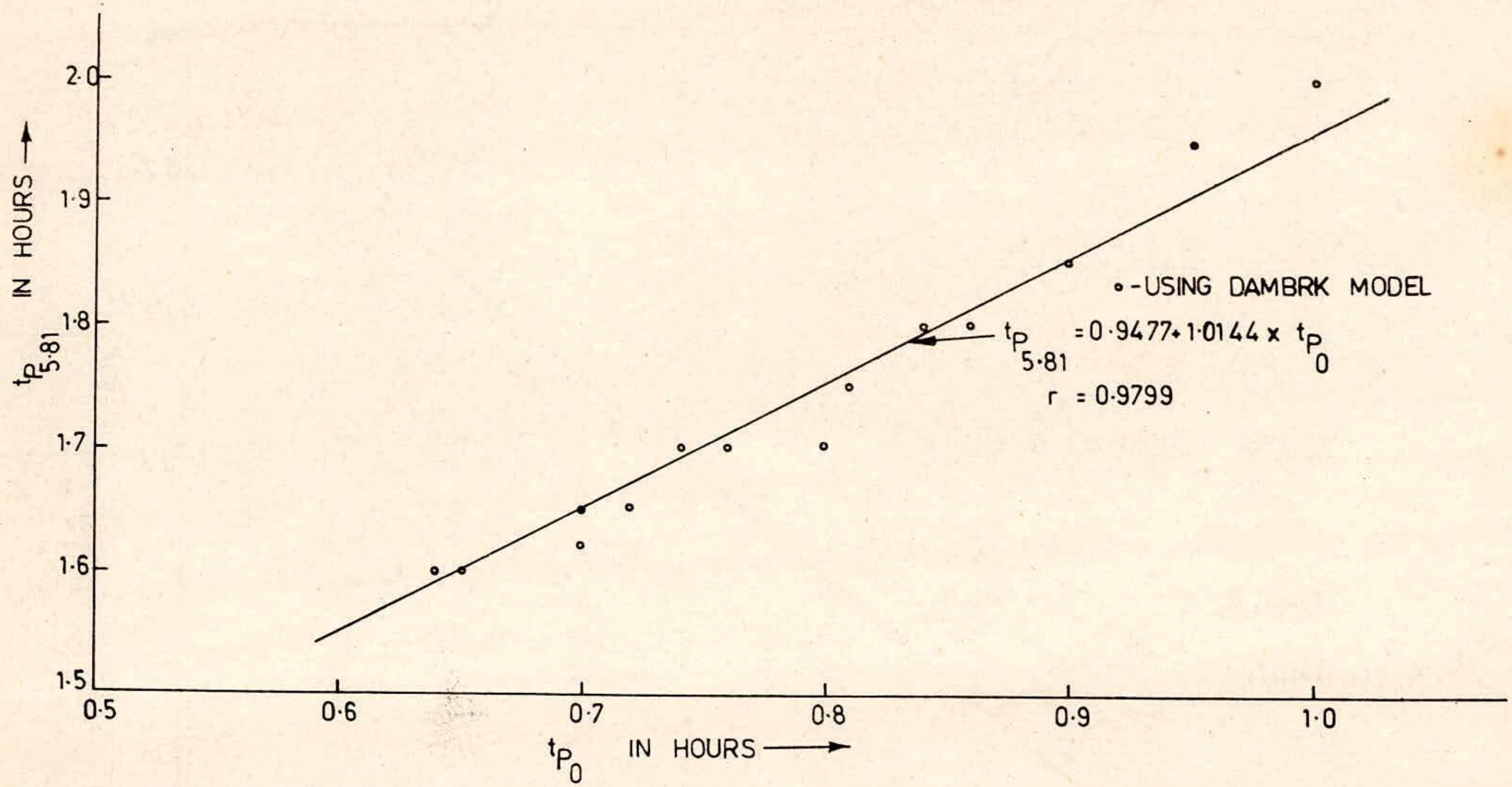


FIG.17. RELATIONSHIP BETWEEN PEAK TIMES IMMEDIATELY BELOW THE DAM AND AT 5·81 MILES BELOW THE DAM

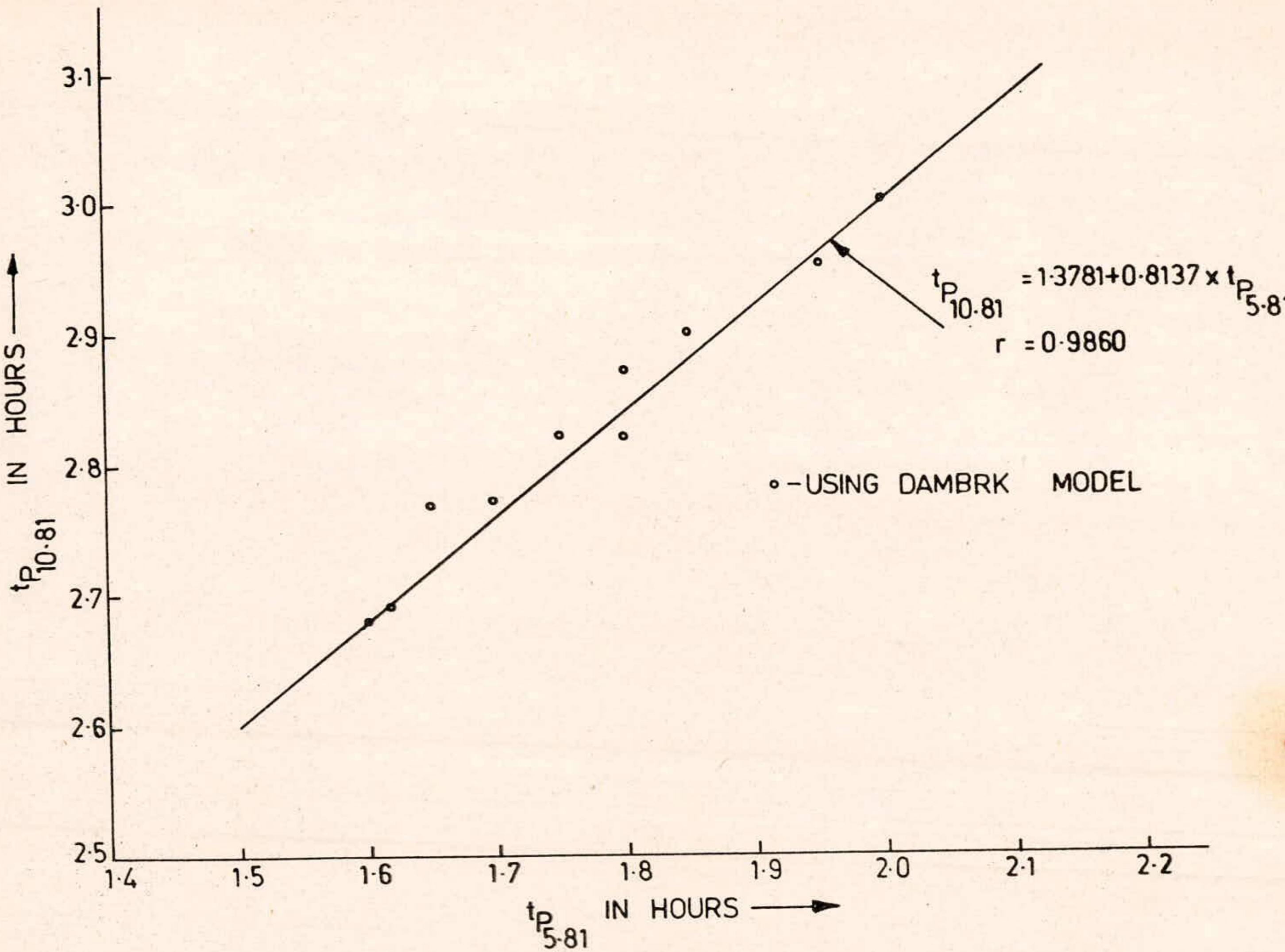


FIG.18. RELATIONSHIP BETWEEN PEAK TIMES AT 5·81 MILES AND AT 10·81 MILES BELOW THE DAM

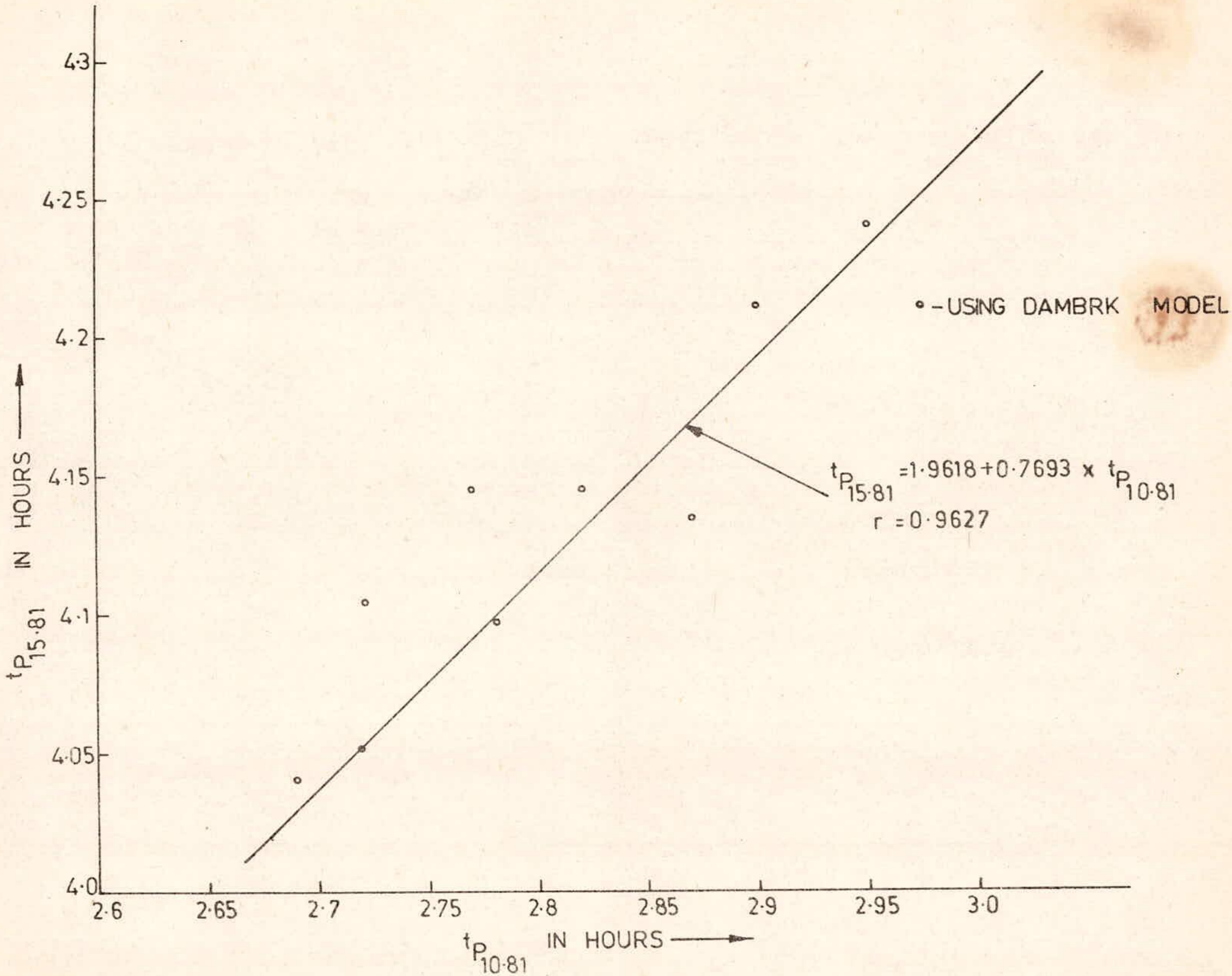


FIG.19. RELATIONSHIP BETWEEN PEAK TIMES AT 10·81 MILES AND AT 15·81 MILES BELOW THE DAM

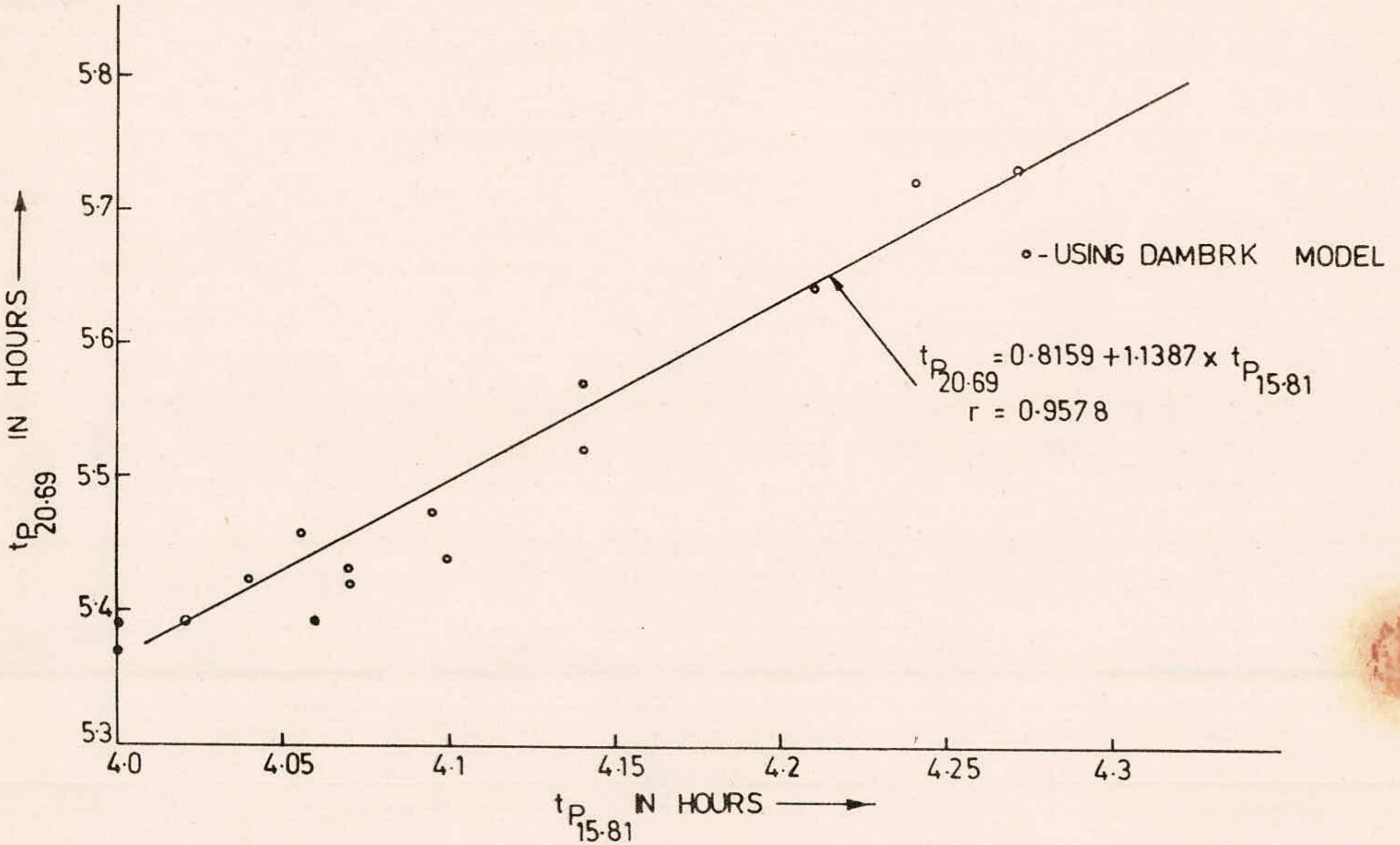


FIG. 20. RELATIONSHIP BETWEEN PEAK TIMES AT 15·81 MILES AND AT 20·69 MILES BELOW THE DAM

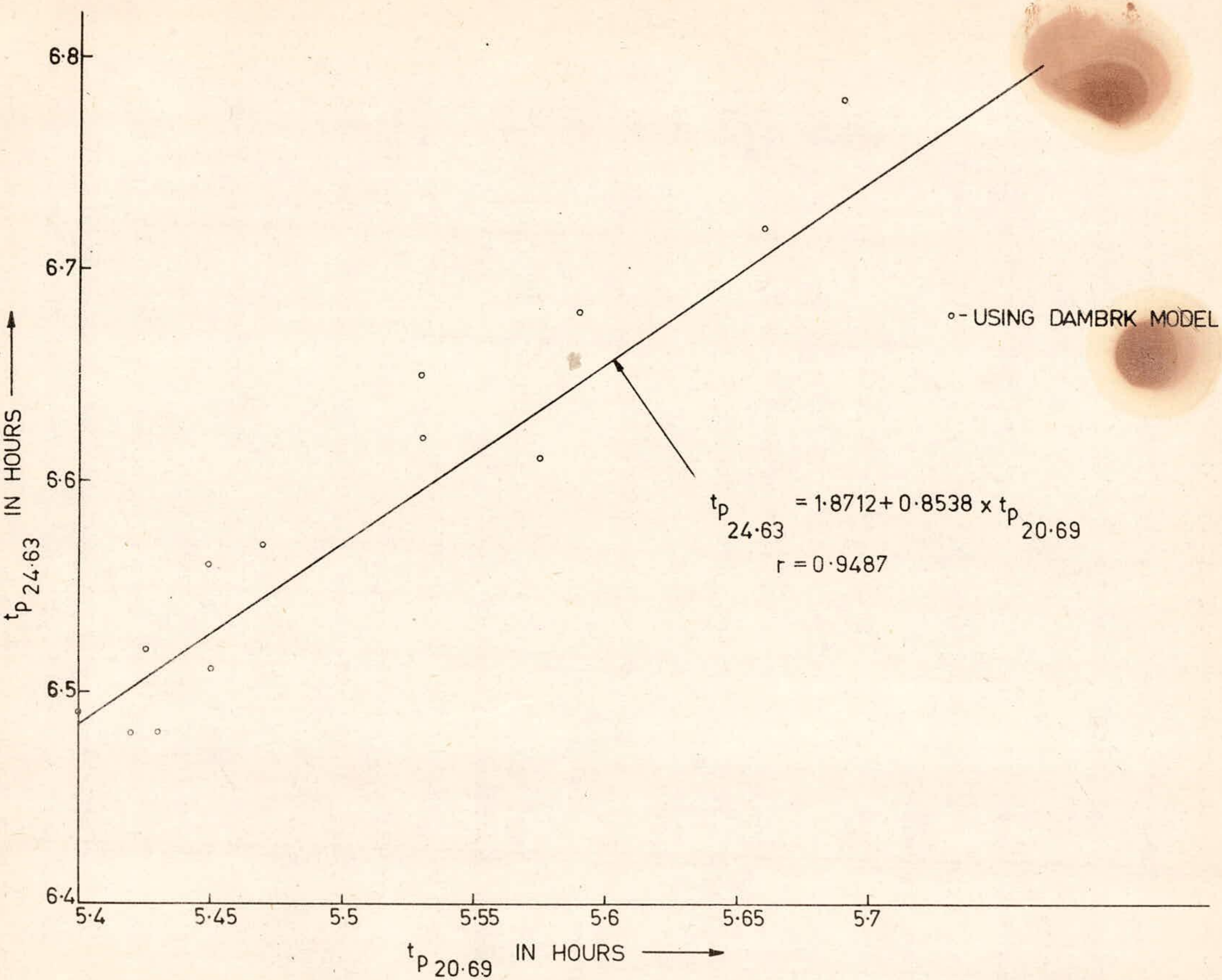


FIG. 21. RELATIONSHIP BETWEEN PEAK TIMES AT 20·69 MILES AND AT 24·63 MILES BELOW THE DAM

dam respectively in hours.

7.4.6 Relationship between peak discharge and peak stage

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 are shown in figure 22,23,24,25,26 and 27 respectively and these can be expressed in linear relationship form as given below:

$$H_{po} = 51.3039 + 0.00000426 \times Q_{po} \quad \ldots(20)$$

$$H_{p5.81} = 36.0949 + 0.00001064 \times Q_{p5.81} \quad \ldots(21)$$

$$H_{p10.81} = 45.6143 + 0.00001338 \times Q_{p10.81} \quad \ldots(22)$$

$$H_{p15.81} = 44.3970 + 0.00000947 \times Q_{p15.81} \quad \ldots(23)$$

$$H_{p20.69} = 31.6286 + 0.00000699 \times Q_{p20.69} \quad \ldots(24)$$

$$H_{p24.63} = 32.0343 + 0.00000507 \times Q_{p24.63} \quad \ldots(25)$$

where,

Q_{po}, $Q_{p5.81}$, $Q_{p10.81}$, $Q_{p15.81}$, $Q_{p20.69}$ and $Q_{p24.63}$ are same as described in section 7.4.3. H_{po} is the peak stage immmediately below the dam in feet. $H_{p5.81}$, $H_{p10.81}$, $H_{p15.81}$, $H_{p20.69}$ and $H_{p\ 24.63}$ are the peak stages at 5.81 mile, 10.81 15.81 miles, 20.69 miles and 24.63 miles below the dam respectively in feet.

The estimates of slope and intercept have been arrived using least square approach.

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 Aug 1979 at time of disaster.

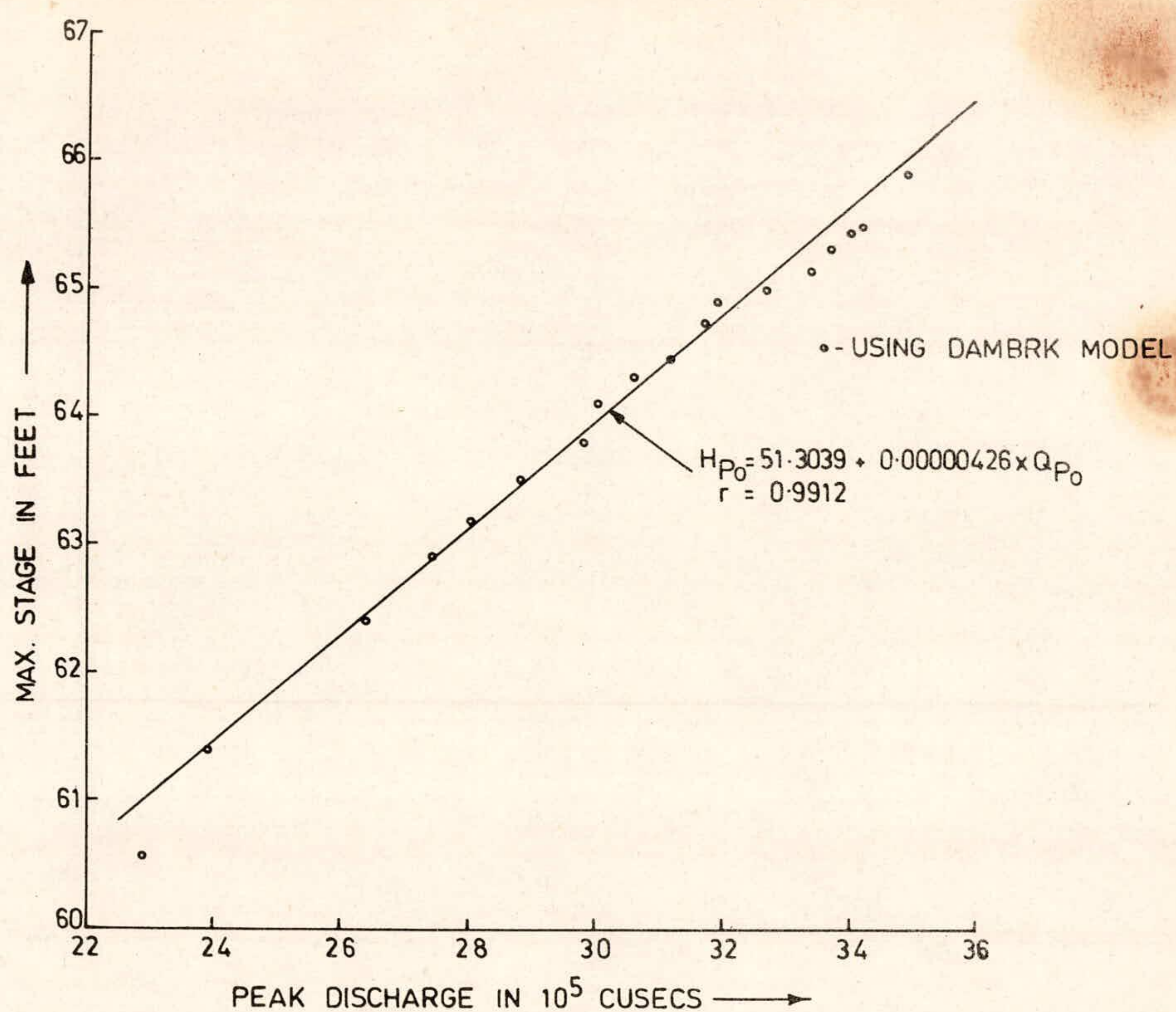


FIG.22. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE IMMEDIATELY BELOW THE DAM

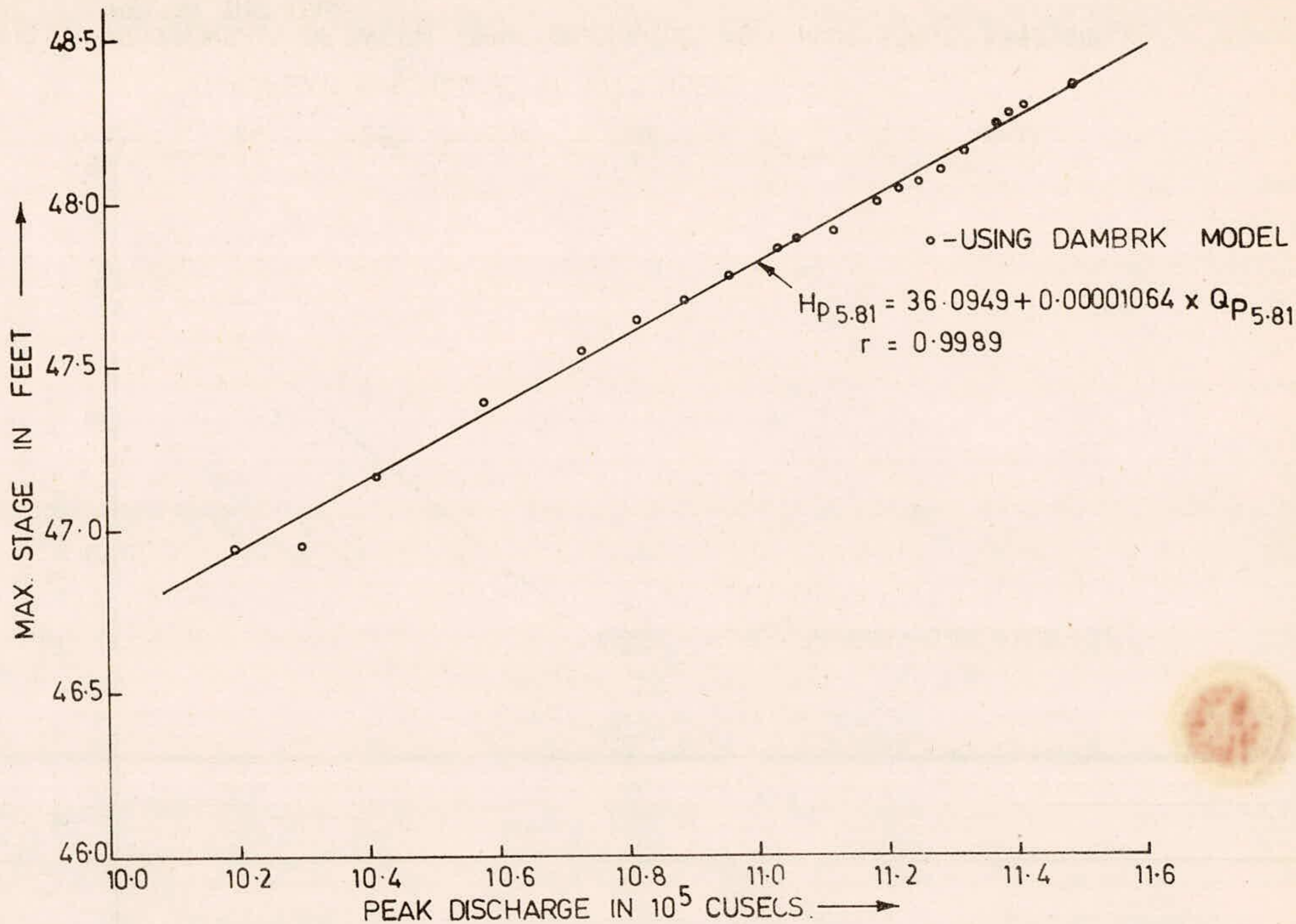


FIG. 23. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE AT 5·81 MILES BELOW THE DAM

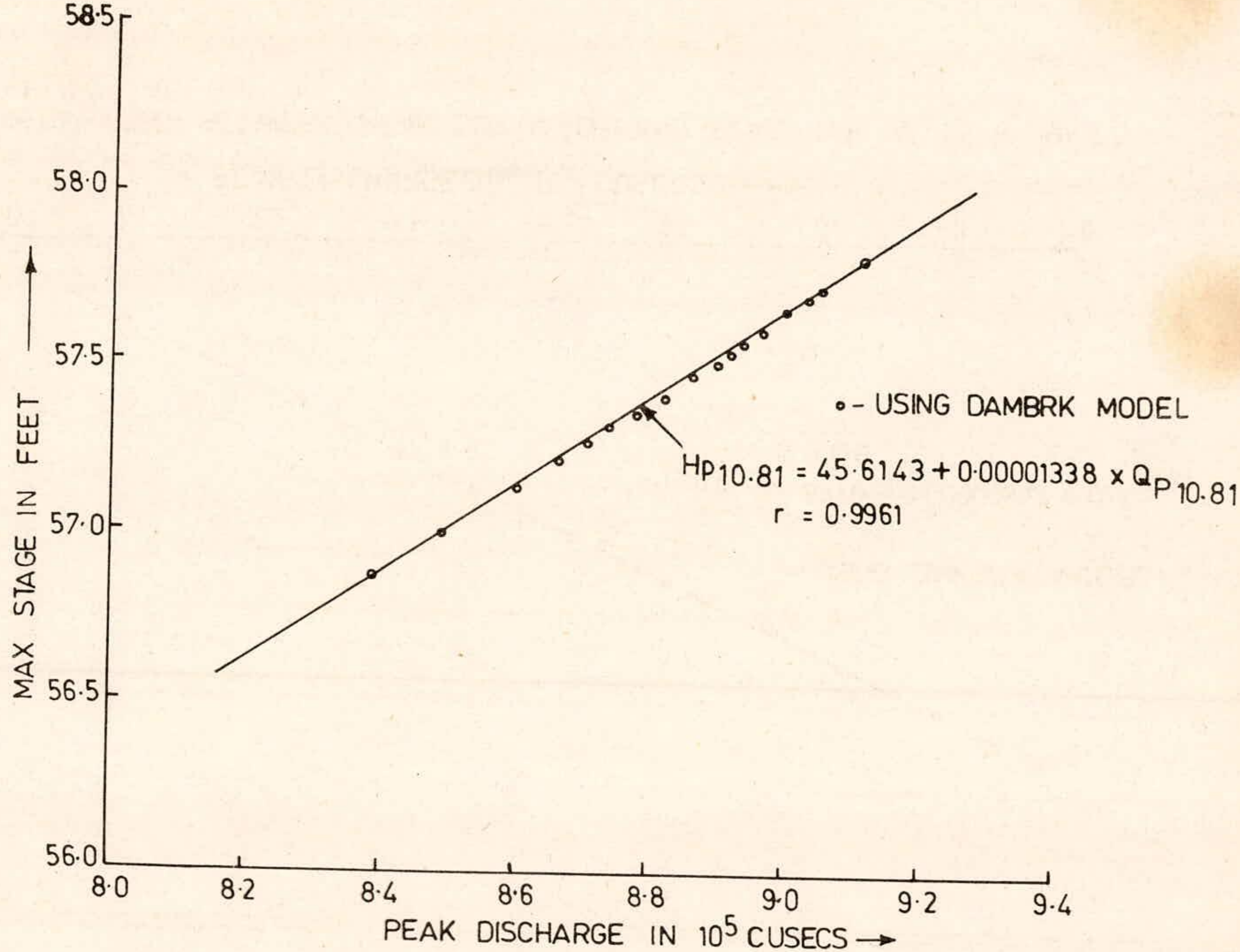


FIG. 24. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE AT 10·81 MILES BELOW THE DAM

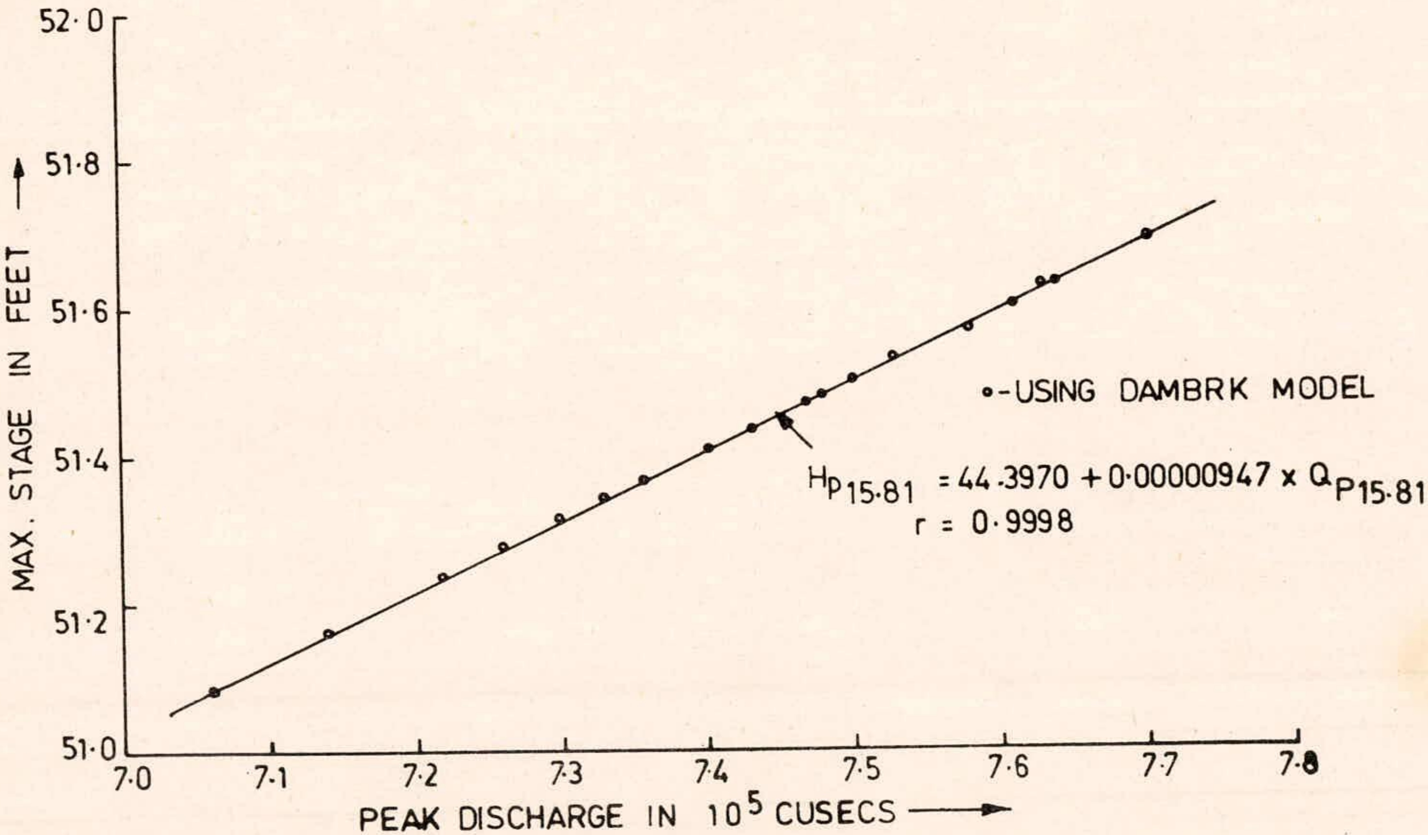


FIG.25. RELATIONSHIP BETWEEN PEAK DISCHARGE AND PEAK TIME AT 15·81 MILES BELOW THE DAM

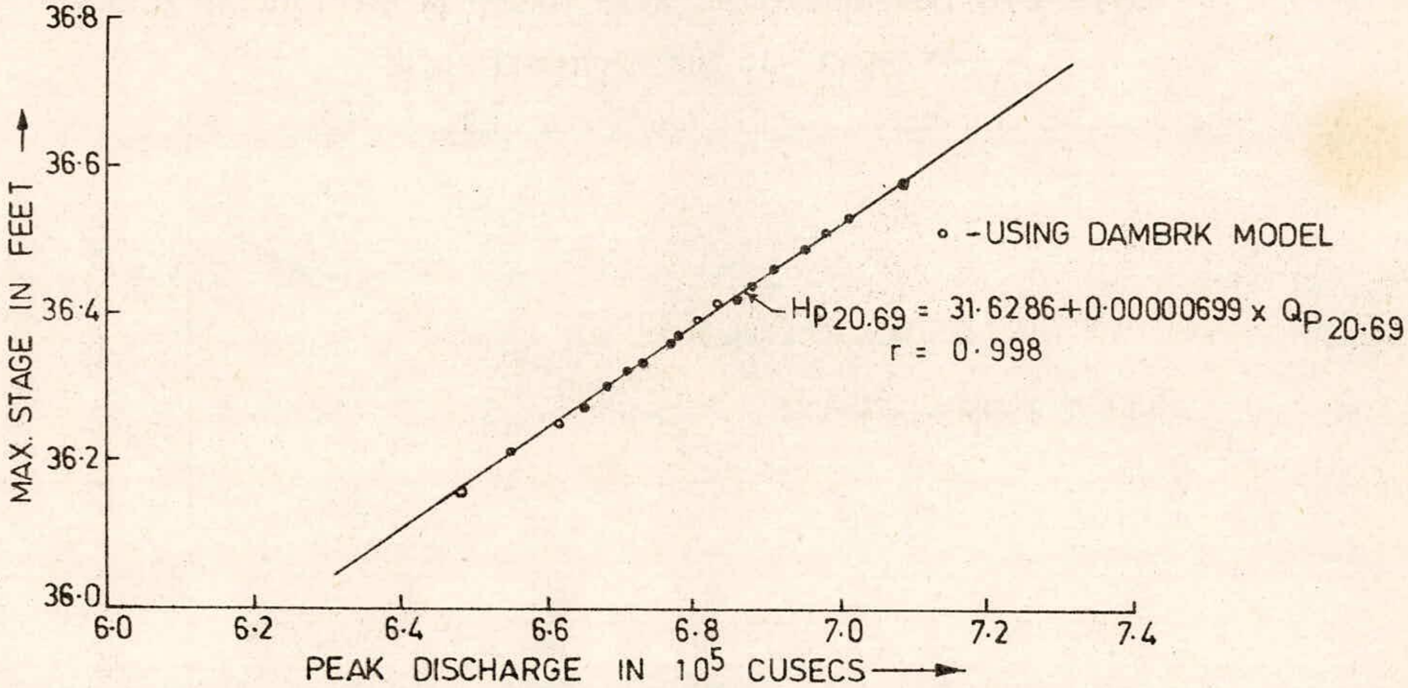


FIG. 26. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE AT 20·69 MILES BELOW THE DAM

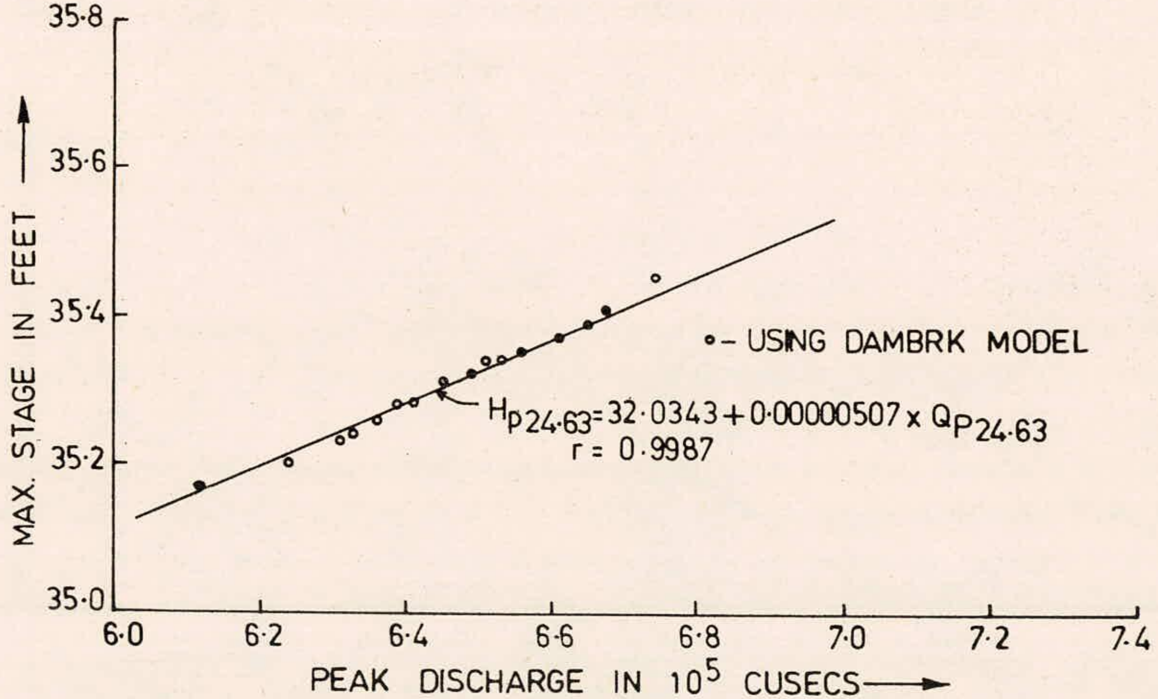


FIG. 27. RELATIONSHIP BETWEEN PEAK DISCHARGE AND MAX. STAGE AT 24·63 MILES BELOW THE DAM

8.0 DISCUSSION OF RESULTS

The section presents the discussion of results given in section 7.

8.1 Dimensionless Hydrograph

The dimensionless hydrograph immediately below the dam site as shown in figure 4 is having sharp peak and the other dimensionless hydrograph below the dam site is having flatter peak. The computed recession limb ordinates of the dimensionless hydrograph at the dam site corresponding to various breach sizes show 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, indicating well behaved dam break flood wave as it moves further downstream. However no scattering of computed ordinates have been exhibited in the rising limb of all the developed dimensionless hydrographs. The aspect is useful in determining the peakflow uniquely without any ambiguity.

8.2 Relationship between Area of Breach and Peak Flow at Dam Site.

The relationship between area of breach and peakflow described by linear relationship in log domain is shown in figure 10, with correlation coefficient as 0.9985. These high correlation coefficients demonstrate the suitability of the form of the relationship adopted.

8.3 Relationship between Peak Discharge at Upstream Site and Next Downstream Site.

Relationship between peak discharge at upstream site and next downstream site are shown in figure 11,12,13,14 and 15. These relationships are also linear in form with increasing correlation coefficients of 0.992 at 5.81 miles downstream of the dam to 0.999 at 24.63 miles downstream of the dam. This shows the suitability of the linear relationship and the well behaved dam break wave movement as it moves further away from the dam site.

8.4 Relationship between Peakflow and Time to Peak Flow at Dam Site

The relationship between peak flow and time to peak flow at dam site is shown in figure 16. This relationship is also linear in form with correlation coefficient as 0.9969 and with negative slope. This relationship demonstrates that the time to peakflow reduces as the peak discharge at the dam site increases.

8.5 Relationship between Time to Peak Flow at Upstream Site and Next Downstream Site

The relationship between time to peak flow at upstream site and next downstream sites are shown in figures 17,18,19,20 and 21. These relationships are not as perfect linear relationships as demonstrated in the other relationships, but nevertheless they may be considered as appropriate as the correlation coefficients of these relationships are greated than 0.94. Although, the deviations exhibited about the plotted mean line seem to be larger, they are very small in absolute terms of time.

8.6 Relationship between Peak Flow and Peak Stage

The relationship between peakflow and peak stage are shown in figures, 22,23,24,25,26 and 27. These relationships are also linear in form with correlation coefficients greater than 0.99 at all the sites. This demonstrates the perfect linear form of relationship.

8.7 Verification of the Dimensionless Hydrographs Procedure

The methodology described in this study is capable of producing outflow hydrographs and its characteristics at dam site and downstream sites of the dam due to dam failures caused by over topping. To verify this developed dimensionless hydrograph procedure, outflow hydrographs are obtained at immediately below and other downstream sites of the dam for a breach area equal to 175% of the actual breach area which was observed at the time of disaster on 11th August 1979.

A separate analysis using DAMBRK model was carried out for the same breach size as mentioned above and the outflow hydrographs were estimated. The comparision of the outflow hydrographs obtained by these two different approaches are shown in Figure 28,29,30.31,32 and 33. These hydrographs are well comparable to each other which verifies the developed dimensionless hydrograph procedure and its usefulness.

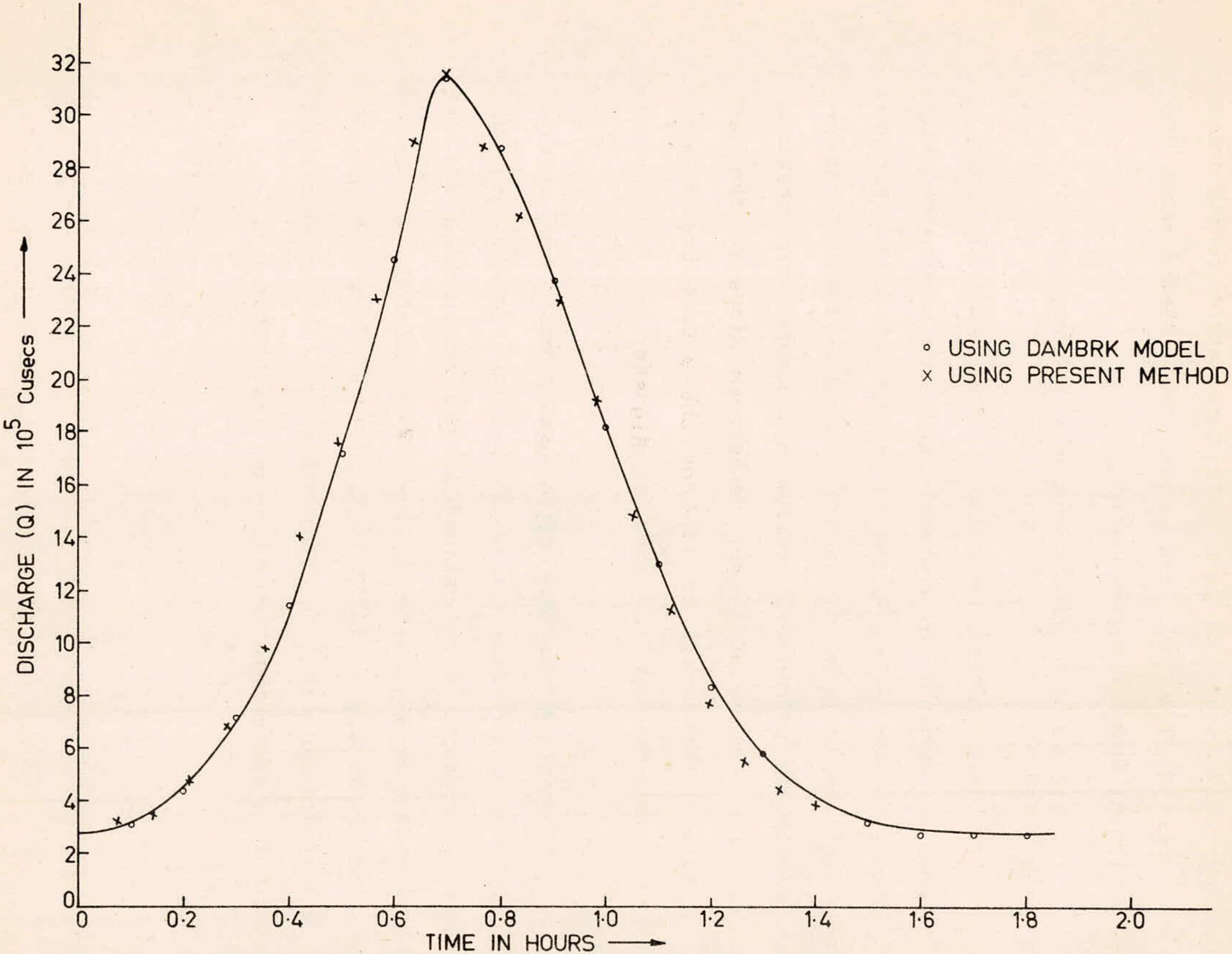


FIG.28. DISCHARGE HYDROGRAPH IMMEDIATELY BELOW THE DAM

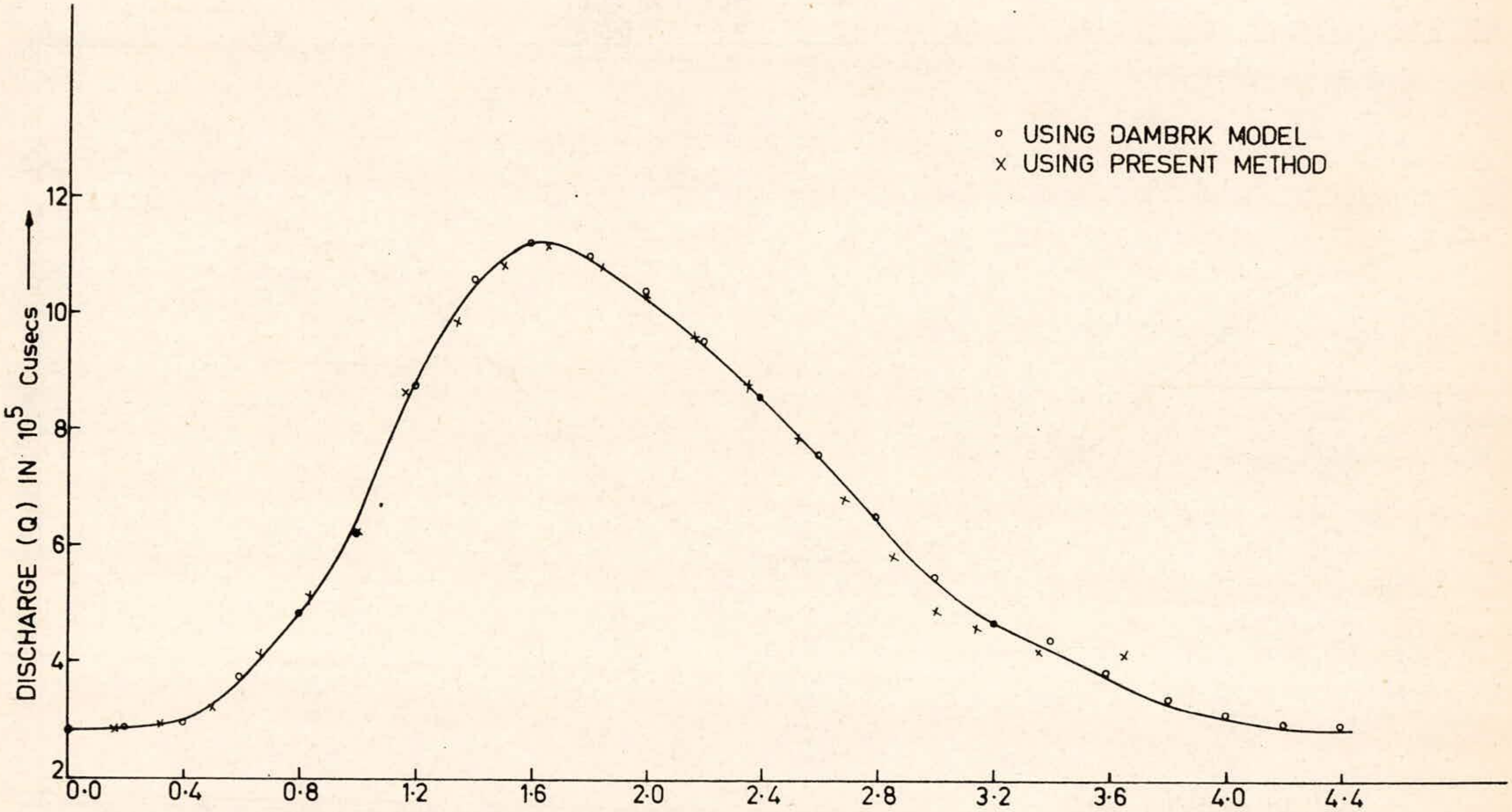


FIG.29. DISCHARGE HYDROGRAPH AT 5·81 MILES BELOW THE DAM

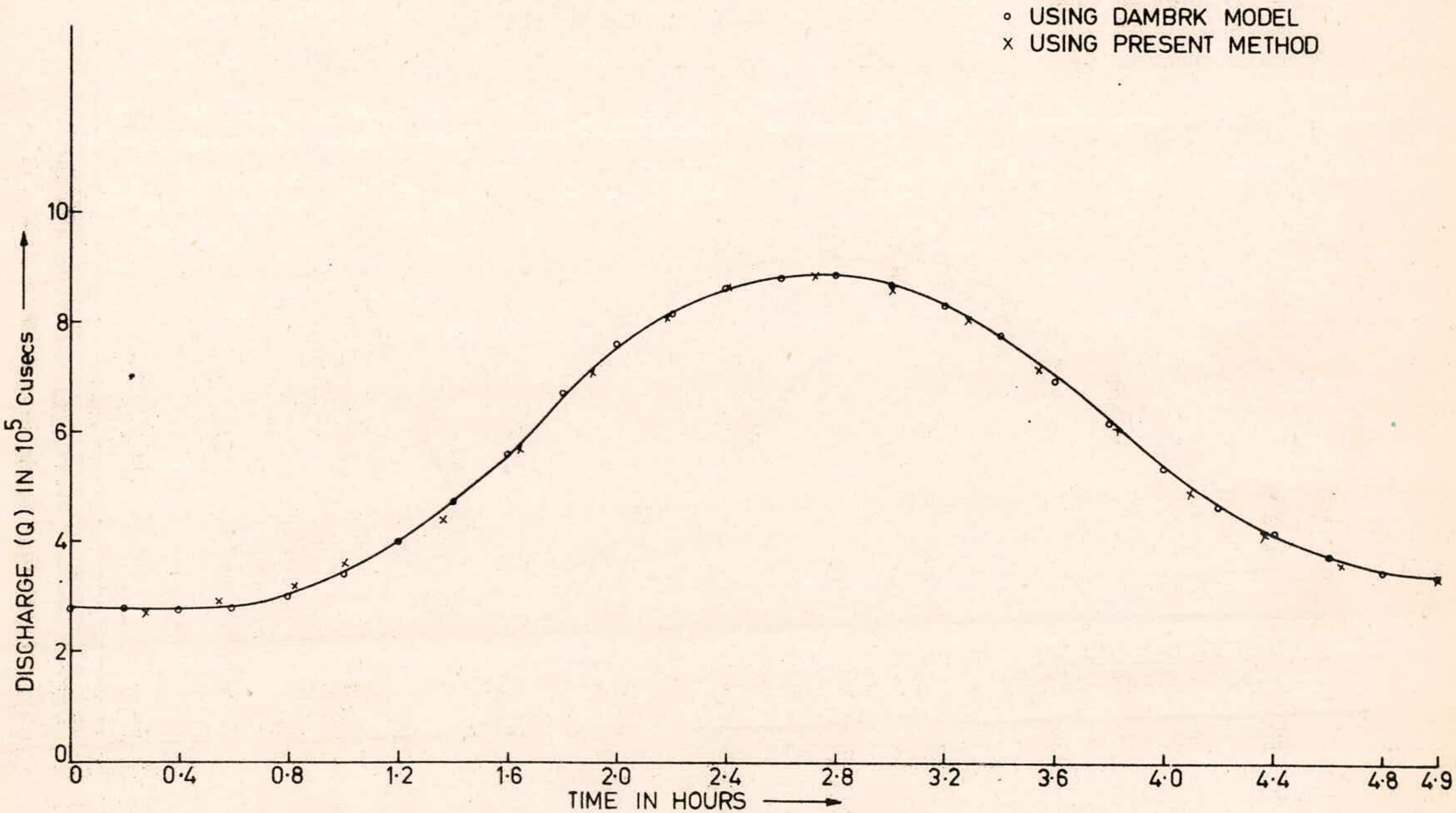


FIG. 30. DISCHARGE HYDROGRAPH AT 10·81 MILES BELOW THE DAM

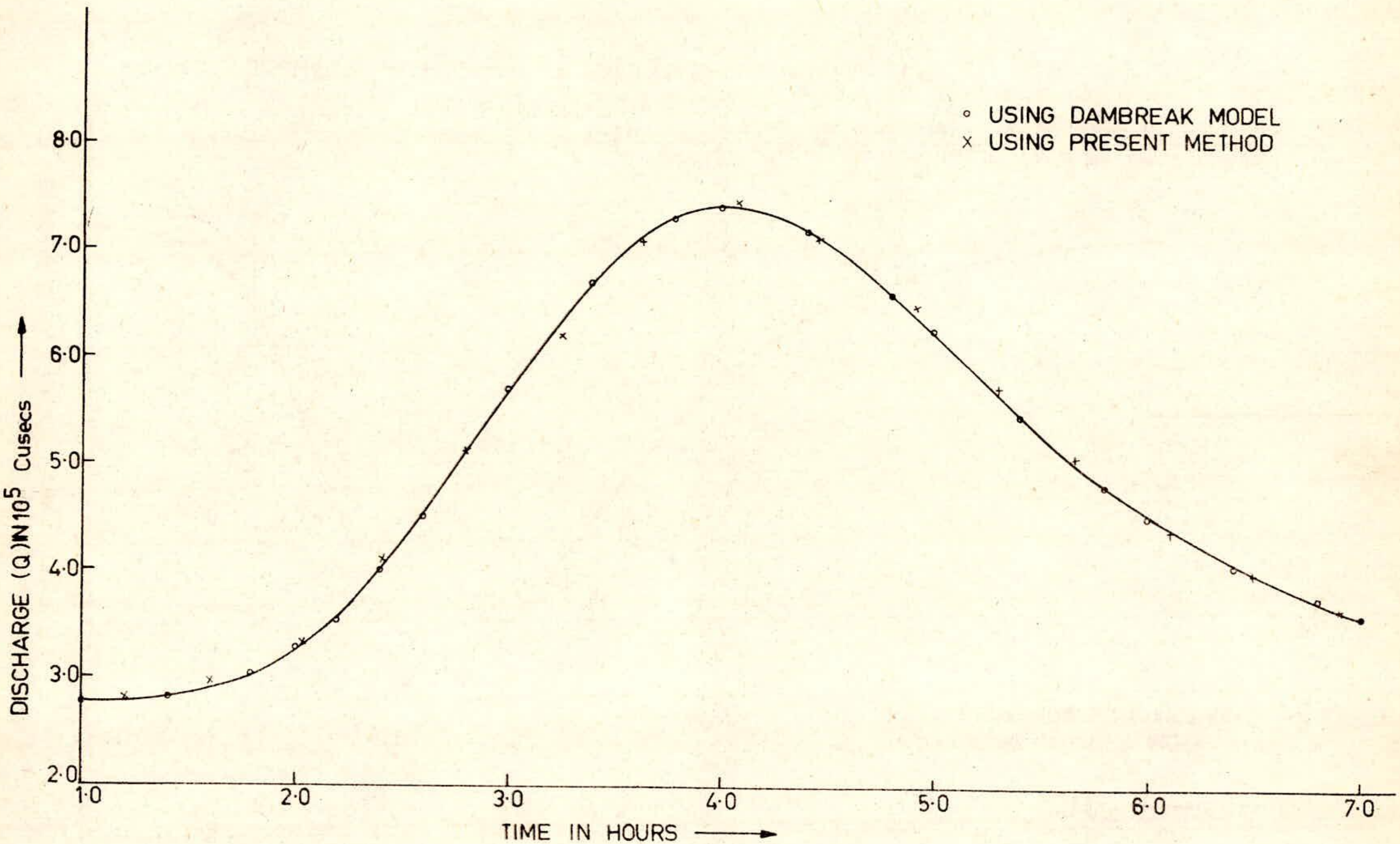


FIG.31. DISCHARGE HYDROGRAPH AT 15·81 MILES BELOW THE DAM

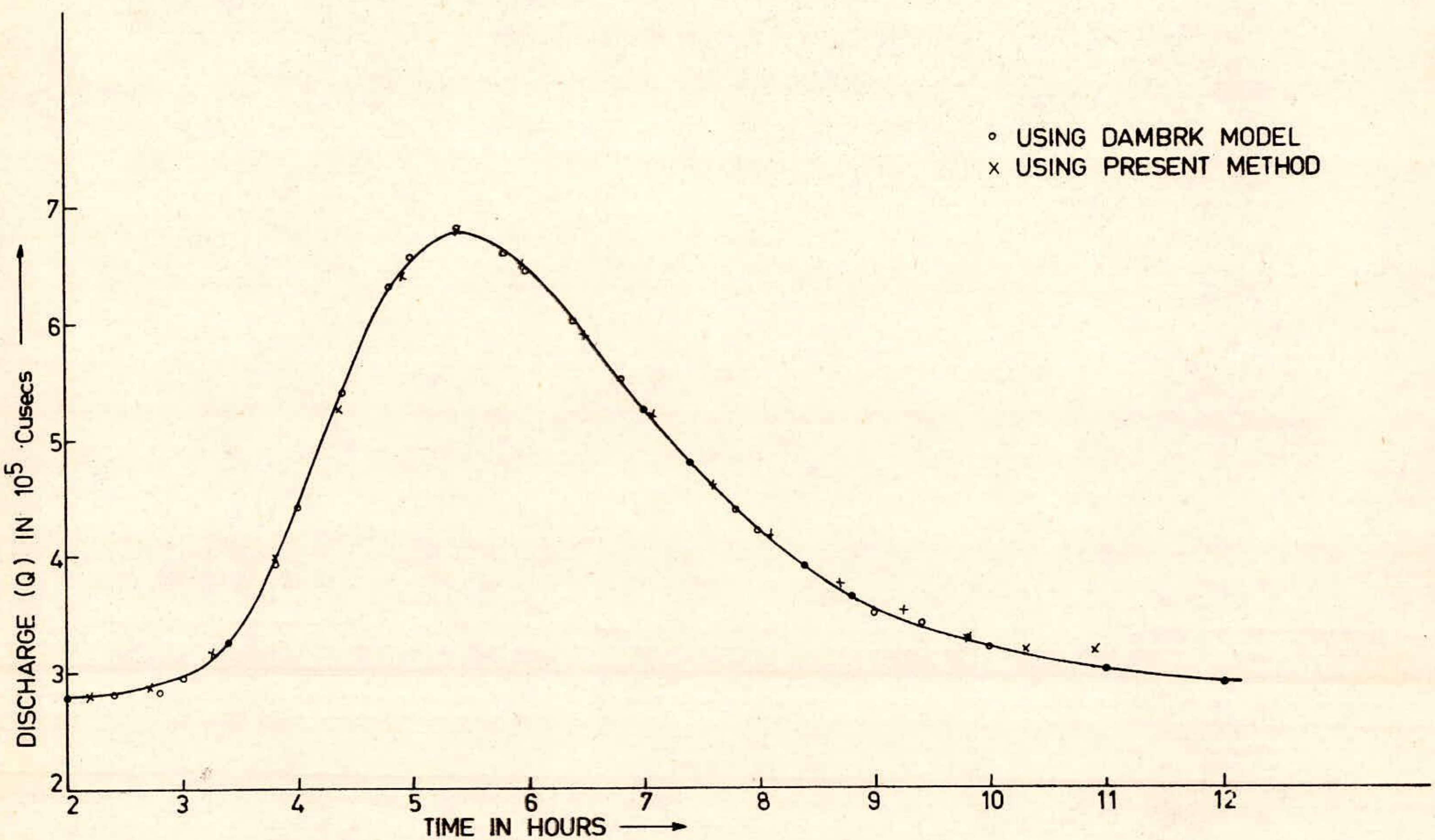


FIG.32. DISCHARGE HYDROGRAPH AT 20·69 MILES BELOW THE DAM

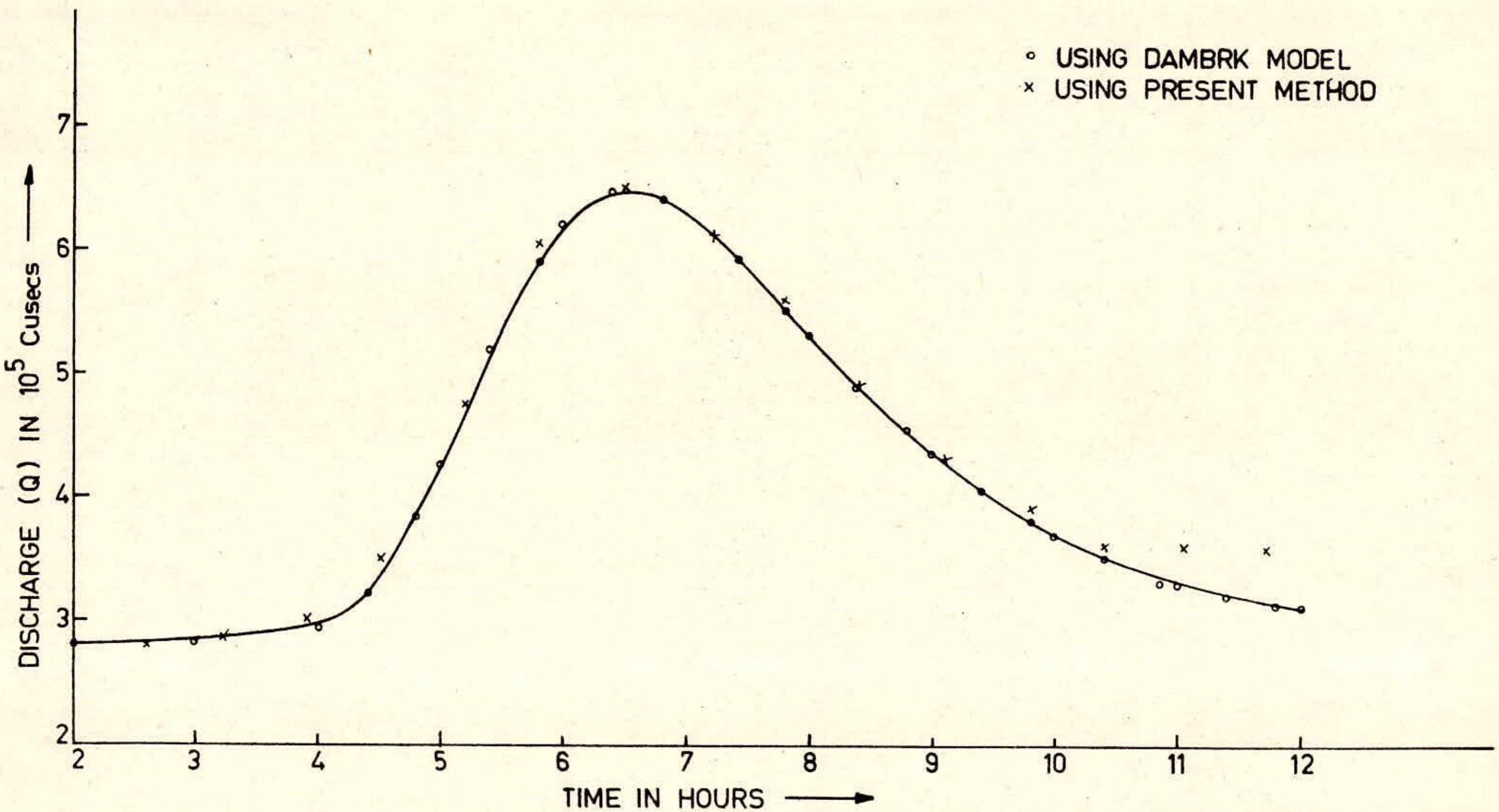


FIG.33. DISCHARGE HYDROGRAPH AT 24·63 MILES BELOW THE DAM

9.0 CONCLUSIONS

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. Although these assumptions may be contradictory to each other from practical point of view, it is not going to affect the usefulness of the arrived results significantly as it had been demonstrated in section 7.4. It is possible, for the considered initial water level of 198.5 ft. and for other considered variables, the range of breach areas studied may be too wider to be of any practical use. Nevertheless 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. Using the results obtained by DAMBRK model, the dimensionless hydrographs at dam site and other downstream sites were developed, besides the relationships established between area of breach and peak flow at dam site, peak flow at upstream site and next downstream site, peak flow and time to peak flow at dam site, time to peak flow at upstream site and next downstream site, peak flow and peak stage at dam site and other downstream sites of the dam.

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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