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FLOOD PLAIN ZONING FOR DOWNSTREAM AREA OF MACHHU DAM - II

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PREFACE

In recent years, there have been somewhat indiscriminate development of urban settlement and industries in the flood plains. This has led to considerably more damage due to floods besides the loss to crops and other property in the flood plains. To reduce or eliminate such damage there is need for taking up both short term and long term structural and non-structural measures of flood control and flood plain management.

. This report presents a typical study for flood plain zoning of downstream area of Machhu Dam-II in Gujarat State. The maximum flood elevations due to 100, 200, 500 and 1000 years return period flood, 1979 flood, design flood and dam break flood from Machhu Dam-II, etc. have been computed using the National Weather Services DAMBRK model developed by Fread (1984) and the same have been marked on contour map of scale 1:15000. The area inundated downstream of Machhu Dam-II based on maximum flood elevation criteria, due to these floods has also been assessed to the extent possible.

This report entitled: 'Flood Plain Zoning for Downstream Area of Machhu Dam-II', is part of the work programme of Flood Studies Division of the Institute. The study has been carried out by Shri M.K.Santoshi, Sc.'B' under the guidance of Dr. S.M. Seth , Sc. 'F'.

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ABSTRACT

This report presents a typical study for flood plain zoning of downstream area of Machhu Dam-II in Gujarat State. This study uses the National Weather Services DAMBRK Model developed by Fread (1984)to compute the maximum flood elevation due to dam break flood of Machhu Dam-II downstream of the dam. The routing option of DAMBRK model using dynamic wave theory is used to compute the maximum flood elevations due to floods with different peaks, which includes 100, 200, 500 and 1000 years return period floods, 1979 flood, design flood and dam break flood from Machhu Dam-II and floods with peak 8765.3, 9734.9, 10956.3 and 11879.7 cumecs, representing earlier estimates of 100, 200, 500 and 1000 year floods. The maximum flood elevations due to floods as mentioned above are computed at about half km interval along the reach of 39.65 kms downstream of the dam and the same have been marked on contour map on both the sides of river reach at corresponding locations. These maximum flood elevation marks at different downstream location have been joined by straight lines indicating the limits of the inundated areas due to the corresponding flood. The inundated areas within these limits have also been measured to assess the total inundated area due to the corresponding flood downstream of the dam. It is seen that dam break flood affects the maximum area (160.40 sq.km.) along the reach and 100 years unrevised flood affects the minimum area (37.287 sq.km.) along the reach. The quantitative results are however dependent upon accuracy of available information regarding river cross sections and contours in the flood plain, and assumptions made for routing of flood wave.

1.0 INTRODUCTION

Every year floods play havoc in our country. The magnitude of the flood depends upon the intensity of the rainfall, its duration and also the ground conditions where the heavy spell of rainfall occurs. Erosion and silting which are increasing due to deforestation lead to reduction in conveyance capacity of river channels, and thus accentuate the flooding. Flood plains are generally heavily populated, since they are very fertile and are easily accessible. In recent years, there have been somewhat indiscriminate development of urban settlement and industries in the flood plains. This has led to considerably more damage due to floods besides the loss to crops and other property in flood plains. There is need for taking up both short term and long term structural and non-structural measures of flood control and flood plain management.

Flood management includes all planning and actions needed to determine, implement and devise plans for the best use of flood plains and their water resources for the welfare of the country. With increasing population and overall development in economic sphere coupled with exploitation of water and land resources in the flood plains, comprehensive flood plains management programme will have to be formulated for their optimal use. For reduction of flood damage both structural and non-structural methods are employed. Structural methods such as dams, embankments and levees, diversion works etc. tend to modify floods and their peaks. The non-structural methods such as flood plain regulations, flood proofing, flood forecasting and warnings etc. modify susceptibility to flood damage and disruption and measures such as flood insurance, flood fighting, post flood recovery etc. tend to modify the impact of flooding.

One of the important aspect associated with the non-structural measures for a flood plain is to assess the flood hazard at each specific location for existing conditions and future condition involving modifications in runoff characteristics. This is characterised by the elevation and spatial area delineation of specific exceedance interval flood events. Keeping in view all such measures, flood plain zoning provides for management, control and regulation of developmental activities and encroachments in the flood plains. Flood plain zoning means restricting/regulating any human activity in the flood plains of a river where the plains are affected by over flow of water from the channels and streams. Generally the terms flood plain includes water,channel, flood channel and area of nearby low land susceptible to flooding by inundation. The flood plain zoning has the short terms objective and preventing more damage from flooding and in the long term to reduce and event eliminate such damage. Surveys have to be carried out for determining the nature and extent of flood plains of rivers. Such surveys form the basis of establishing flood plain zones. This includes delineation of the areas which are subjected to flooding including classification of land with reference to relative risk of flood plain use intended to safeguard the health, safety and property of the general public.

The hydrological computations include:

(a) Consideration of a specific reach of river having gauge and discharge measurement sites at upstream and downstream ends;

(b) Establishment of rating curves for both sites,

(c) Division of the reach into sub-reaches with consistent parallel water surface profiles for the range of discharges,

(d) Establishment of rating curves for sub-reaches (in cases where stage computations are not made as a part of flood routing computations).

(e) Selecting and calibrating appropriate flood routing technique,

(f) Carrying out flood frequency analysis to estimate flood peaks for different return periods,

(g) Determining corresponding flood hydrographs for different return period floods,

(h) Routing of flood hydrograph through the river reach and computing flood levels for different return periods for each subreach.

(i) Marking the corresponding flood limits for each flood on the contour map of the basin area covered by the river reach,

(j) Taking into consideration existing land use within these limits, compute stage v/s damage function,

(k) Decide about limits of flood plains for restricting different types of land uses for future activities.

Flood frequency analysis is a tool used to estimate the frequencies of future floods. In this approach the sample data are used to fit frequency or probability distributions which in turn are used to extrapolate from recorded events to design events either graphically or analytically. The sample data can be either annual flood series or partial duration series. In annual flood series highest floods for each year are considered while in partial duration series all the floods above a particular threshold are considered. The only restriction about partial duration series is that the floods considered should be independent. Generally in field practice annual flood series is considered. Commonly used probability distribution for flood frequency analysis include log normal - two parameter distribution, log normal -3 parameter distribution, extreme value type 1 distribution, Pearson type III distribution and log Pearson type III distribution. Various methods for estimation of the parameters of these distributions are available in literature. Among these the method of moments and method of maximum likelihood are more popular. In frequency analysis goodness of fit of various distribution is examined based on some statistical criteria. The two most commonly used tests of goodness of fit are the Chi-square and Kolmogrov-Smirnov tests. An additional check on goodness of fit may be made by computing the sum of squares of differences between observed and computed event magnitudes. Once the distribution of sample data is decided the floods of various return periods can be estimated from the sample data. The standard errors of flood estimates increase as the return period increases.

Another method of carrying out flood frequency analysis is to first transform the data to normal distribution using power transformation, carry out the frequency analysis and again transform the results to original domain. Thus, adopting such measures with appropriate legislation the disaster due to floods occuring in the area can be avoided and damage caused by floods could be minimised upto the extent possible.

The rating curve i.e. the relationship between stage and discharge can be developed and used to find out stage or flow as the case may be when measurements are not available at shorter interval and for complete duration of flood. When the observed values of discharges are plotted against the corresponding stages, it gives a relationship that represents the integrated effect of a wide range of channel and flow parameter. The combined effect of these parameters is termed as control. If the stage discharge relationship for a gauging station is constant and does not change with time the control is said to be permanent. It is called shifting control if it changes with time. This problem generally arises in case of alluvial rivers and for large and instantaneous floods such as dam break flood. One of the most important factor in this case is unsteady flow effects of a rapidly changing stages. Thus the rating curve is established for such effects and then used to obtain corrected flows. Routing is used to predict the temporal and spatial variations of a flood wave as it traverses a river reach or can be employed to predict the outflow hydrograph from a watershed subjected to a known amount of precipitation. Routing techniques can be classified into two categories - hydrologic and hydraulic routing. Hydrologic routing employs the equation of continuity with either an analytic or an assumed relationship between storage and discharge within the system. Hydraulic routing, on the other hand uses, both the equation of continuity and momentum equation. This particular form utilises the partial differential equations for unsteady flow in open channels. It more adequately describes the dynamics of flow than does the hydrologic-routing technique. Hydraulic Routing techniques are helpful in solving river routing problem, overland flow or sheet flow and in systems simulation of a composite watershed.

The purpose of the report is to assess the area affected by the floods of different years return period and different peaks as well as year 1979 flood, Design Flood and dam break flood for Machhu Dam-II. The information may be used by the flood control authority to warn the public living in flood plains which are likely to be affected by such floods and can take appropriate safety measures.

2.0 REVIEW

It is one of the fascinating hydrological problem to study, how to reduce damages (direct and indirect) caused by flood using short term and long term structural and non-structural measures of flood control and flood plain management. Some of important studies carried out or methods used to solve such problems, by different researchers or organisations, either in published or unpublished form have been reviewed as follows:

A Minister's committee on Flood control (1964) recommended that more attention should be paid to the methods of flood control other than embankment like minor, medium and major storages, canalisation, inter-basin diversion and to such non-physical measures like flood warning and forecasting, flood plain zoning and flood insurance. A minister's committee on flood and flood relief (1972) made same recommendations on the strategy for flood protection including that the action should be taken by the various state Governments for preparation of maps of the different river basins, demarketing therein the areas where human habitation will have to restricted by administrative measures.

Johnson (1970) presented the 'State flood-plain management activities including necessity for flood damage reduction, early flood plain management, 1960 flood control act, regulation of flood plain uses, administration and legal problems and concern with social aspects. He stated that primary emphasis was placed on corrective measures which includes dams and reservoirs, levees, channel improvements, watershed treatment and other structural measures The 1960 flood control act say that flood plains cannot be wisely managed, of course, without first being adequately identified. Johnson also stated that state government is confronted with a number of administrative and legal problems relating to implementation of flood plain regulations and management measures.

Ministry of Agriculture and Irrigation (Deptt. of Irrigation) Govt. of India, circulated a model bill for flood plain zoning to the State Governments in July 1975, including the following aspects:

a) Flood zoning authority and its power

b) Surveys and delineation of flood plain area

c) Notification of limits of flood plains

d) Prohibition of restriction on the use of the flood plains.

e) Compensation

f) Power to remove construction after prohibition

The final report of the working group on Flood control, as brought in 1978, by the Ministry of Agriculture and Irrigation recommended some important points with regards, to the strategy for flood protection including that in connection with flood plain regulation and management the basic work of detailed surveys and preparation of contour maps should be carried out in the central sector and the State Government should demarcate the areas liable to flood of different frequencies and intensities both on maps as well as on ground and enforce necessary land use regulation.

In the fourth conference of State Ministers of Irrigation (1979) several resolutions were passed on floods and flood protection. Those relating to important flood control strategy are summarised below:

i) Completion of the work of preparation of master plans for flood control and drainage within the frame work of overall development of water resources.

ii) Preparation of detailed contour maps in respect of flood prone area be expideted by taking up the work in the central sector and to take steps to demarcate flood zones corresponding to various flood frequencies and intensities.

iii) Necessity of administrative and legal steps to be taken immediately by the concerned authorities to regulate development activity in the flood plain. The other points were regarding the new embankment Schemes and anti erosion works to be taken for protection of town, industrial areas and a group of thickly populated village abadis, railway lines and roads where relocation is not possible on techno-economic grounds.

Davis (1979) developed and tested analytical methodologies for application in comprehensive flood plain information studies. The methodology permits and encourages comprehensive, systematic, practical assessments of present and alternative future basin-wide development as reflected by alternative land use patterns and physical works in terms of flood hazard, economic damage potential and selected environmental consequences. The analysis methodologies were centred about integrated use of computerised spatial, gridded geographic and resources data files. The methodology was applied to Trail Creek in Clarke County, Georgia.

Ford (1980) stated that the goals of non structural flood control planning are formulation, evaluation, selection and implementation of a practicable management plan that provides optimal protection from adverse effects of flooding. Many alternative flood control measures can be dismissed by the planner on the basis of judgement, but a substantial number will require detailed analysis before a suitable plan can be selected. Software developed at HEC allows efficient data storage in a structure oriented data bank and provides for selective retrieval and manipulation of data from an interactive terminal. Thus the planner is able to propose nonstructural measures and to evaluate rapidly the economic and technical feasibility of these measures in an interactive scheme that allows the required input from the planner.

R.B.A. (1980) under' Lines of Remedial Action' states that it is possible to have recourse to the following measures of protection and flood control -

1. Feasibility of Embankments to keep the flood out of areas which are otherwise subject to inundation.
2. Storage reservoirs, preferably on the tributaries
3. Detention basins where the excess of flood water may be stored for a short time
4. Diversion of water of one river into the other
5. Increasing the slope of the river by cutting down loops.
6. Dredging and channelling of the river where water way has been reduced due to silting.

Besides these, some other measures like revetments and spurs will be necessary to safeguard any particular town exposed to the danger of being eroded. After collection of requisite data, the choice of a appropriate method or combination of methods can be made.

R.B.A. (1980) describes the different methods of flood management and states that each flood situation is different and we should have different alternative adjustment from which to choose. These adjustments were classified in four groups, each of which represents an attempt either to deal with the flood waters or the activities that would be affected by them. These groups are:

i) Attempts to modify the flood

ii) Attempts to modify the susceptibility to flood damage.

iii) Attempts to modify the loss burden

iv) Bearing the loss.

Methods of modifying the floods can be grouped under the following three heads.

i) Construction of flood control or protection works.

ii) Adoption of measures which will cause reduction of floods.

iii) Alteration of precipitation patterns through weather modification.

Flood control or protection works are further classified as under:

a. Embankments and flood walls or dowl walls.

b. Reservoirs

c. Natural detention basins

d. Channel improvements

e. Emergency flood ways

f. River diversions

g. Inter basin transfer

h. Bank stabilisation and anti erosion measures

i. Ring bunds and

j. Under ground storage reservoirs

Flood modification, acting alone,. leaves a residual flood loss potential. It can also encourage a false sense of security landing to in appropriate use of lands in area that are directly protected and often in adjacent area as well. For this reason measures to modify the possible flood should usually be accompanied by measures to modify the susceptibility to flood damages by land use regulations. Modification of the susceptibility to damages of property and activities in flood plain is usually carried out by flood plain management, development and redevelopment policies, structural changes, flood proofing, disaster preparedness response plans and flood forecasting and warnings. This strategy is expressed as actions to avoid dangerous, un-economic, undesirable or unwise use of flood plains.

Flood plain management covers land use regulations, statutes, zoning ordinances, sub-division regulations, Govt. purchase of property and relocation. The purpose of land use regulation within a Flood hazard area are two-fold (i) to ensure that the existing hazard and flood damage potential are not increased and (ii) to ensure that new developments are not themselves subjected to serious damage. The flood zone concept assumes that the river channel and flood plain are one unit and flood plain is really a part of the river channel which is used to transient water only in frequently.

The outer limits are taken to be the estimated boundaries of the largest flood to be expected. The degree of hazard decreases away from river channel and the zones are defined, each with its own group of regulation and its own special set of planning devices to take into account of this variability of hazard.

The modification of the loss burden is actually a strategy for mitigating the losses by means of actions designed to assist the individual and the community in the preparatory, survival and recovery phase of floods. This category comprises - i) emergency measures and ii) redistribution of losses. The emergency action consists mainly of i) evacuation ii) flood fighting and iii) public health measures. The approach of redistribution of losses to modify the burden by spreading is over a larger segment of community than that immediately affected it more evenly over the time. The common methods of this approach are disaster relief, tax remission and flood insurance.

The report of High Level Committee on flood 1957, indicated the necessity of a new thinking on the ultimate nature of the problem regarding providing flood protection. The Central Flood Control Board, in its 7th meeting held in May 1958, under the Chairmanship of the Union Minister in charge of Planning, Irrigation and power who was also incharge of flood control, noted the vital recommendation of High Level Committee that absolute or permanent immunity from flood damage was not physically attainable by known methods of flood control and hence flood plain zoning, flood forecasting and warning, and like measures should, therefore, be given due importance.

El-Jabi and Rousselle (1983) presented a stochastic model based on the recent development in the theory of extremes value to describe the flood occurrence. The model was applied to the 'Rivers des Prairies' assuming that the exceedences (Flood peak and/or stage above a given base flow and/or stage base stage) are independent and identically distributed in the time interval for a given year and season.

Veissman and Welty (1985) stated that the nonstructural approaches to flood damages reduction generally refers to all other adjustment than structural approaches employed to modify the exposure of people to flood. This includes both measure to modify the susceptibility to flood damage and disruption and measures to modify or reduce adverse impacts of floods on the individual or community. They described the following methods under nonstructural approaches to flood damage reduction such as, Flood plain regulation, Development and Redevelopment Policies, Flood Insurance, Flood proofing, flood forecasting and response systems, Disaster Preparedness, and Relief and Rehabilitation. It was specifically mentioned that, in 1973 congress strengthened the National Flood Insurance act of 1968, by passing the flood disaster protection act, which required communities to participate in the flood insurance programme in order to obtain any form of federal financial assistance for land acquisition or construction in any flood designated area or federally funded relief for flood victims.

Mistry (1986) discussed the flood warning dissemination and flood forecasting system that has been set up in the state to warn the people in the flood plain zones.

Bisaria (1986) discussed the trends in flood damages during the last 33 years. The total flood damages evaluated in monetary terms comprise the damages to crops, houses, and public utilities. An analysis was made to indicate the varying extend to which these losses (at constant prices) have contributed to the total damage during the years. He also stated that a programme of flood control was launched at the national level in the year 1954. This has provided reasonable protection to an area of 13 million ha by March 1985 out of total flood prone area of 40 million ha as assessed by Rashtriya Barh Ayog (1980). The analysis of the damage data could be a pointer for areas needing attention. The trend of flood damages in the light of investiments in flood control sector and their effect on economic welfare, need to be studied and discussed.

Chandra and Perumal (1986) explained a generalised methodology for estimating annual flood damage to house and property component of a given river reach and its use in our country was suggested in the planning stages of flood control studies. They also mentioned that in order to understand the severity of the flood problem and to compare benefits and costs of a flood management measures, there is need for development of procedures to estimate flood damage.

Patel and Purohit (1986) made at attempt to identify the causes of the flood damages, historic events of the floods and flood damages in the country as well as in Gujarat State. The measures to mitigate the flood damages through structural and non-structural flood plain management were also been discussed.

Ganga Flood Control Commission, Patna (1986) gave some suggestions indicating the classifications that could be made for collection of floo d damage.

Rangachari (1986) summarised the picture of flood damages in India as reflected by the data reported by the States. The existing system of data collection and compilation agencies were discussed. The methodology and procedures followed for the data collection were discussed under broad headings like crop damages, damage to private houses and properties, loss of lives, damage to public property and indirect damages. The many recommendations of the Rashtriya Barh Ayog in respect of these issues were discussed and it was concluded that, by and large, these recommendations had not been satis factorily taken into account.

Chandra and Perumal (1987) brought out in brief the problems of flood and its extent in our country. The measures adopted in practice for the management of floods in various parts of our country have been brought out. The flood problem was discussed from the hydrological point of view and the required hydrological analysis for various flood management measures was indicated Although the strategies of modifying the flood and reducing the impact of flooding are widely practiced in our country, the strategy of reducing the susceptibility to damage has not been given much attention. Therefore, they brought out in detail the measures available under this strategy paying due emphasis to the improvements required in the adopted measures under this strategy and other measures to be practiced in managing floods.

Hegde (1987) made an attempt to indicate the types of measures that have been taken and also proposed to be taken direct and indirect to minimise the losses due to flood disasters. Floods cannot be controlled totally at any cost, and have to be lived with. However, with improved science and technology flood disaster areas can be better managed and the possible losses minimised.

El-Jabi and Rousselle (1987) described an integrated approach which includes consideration of the hydrologic, hydrodynamic, physical and economic components of the total system, and stated that hydrologic and economic information must be integrated in flood plain management. On the basis of these components, they proposed a theoritical model which provides a rational procedure for estimating flood damages from projection of economic development within area. The utility of the model was also demonstrated by applying it to a flood-prone region in Southern Que bee, Canada.

Das and Lee (1988) presented a new methodology to calculate economic losses from hypothetical, extreme flood events, such as PMF, using economic data compiled from already available secondary sources such as U. S. census data on magnetic tape, utilizing microcomputer and other electronic media. Estimates of land elevations were obtained from topographic maps and flood elevation were estimated using a dam breach and flood routing (DAMBRK) model (Fread, 1984). The calculations were performed at a disaggregate spatial scale by various land used and industrial classification categories. Depth-damage functions, which provides, an estimate of damages as a proportion of the existing value of the structure were estimated statistically. Computer software (DAMAGE) was used to combine economic, flood elevation, and depth damage information to compute economic losses for different possible flood stages and for different inflow events.

Sukegawa (1988) traced the history of flood control in Japan and stated that in planning, flood control measures, the degree of protection to be achieved is calculated in terms of return period of the project storm, which may be 50,100 or 200 years. A hydrograph of the project storm is produced by extrapolation from historical data and proportions of the project flood are then assigned to various control measures. The most common of these measures are levees, channel improvements, flood ways, reservoirs and retarding basins. In addition, sophisticated prediction and warning systems are being developed to allow effective evacuation of threatened areas.

Refsgaard et. al. (1988) presented a general mathematical modelling system for real time forecasting and flood control planning. The system comprises a lumped conceptual rainfall runoff model, a hydrodynamic model for river routing, reservoir and flood plain simulation, an updating procedure for real time operation and a comprehensive data management system. The system was applied for real time forecasting of the two 20,000 Km^2 (Yamuna and Damodar) catchments in India as well as for flood control modelling at the same two catchments in India. In another project the system was established for the entire Bangladesh with a coarse discretization and for the south East Region of Bangladesh with a fine model discretization.

3.0 PROBLEM DEFINITION

The objective of this study is to assess the area affected by the Floods with peak 8765.3, 9734.9, 10956.3, 11879.7 and 17149.8 cumecs, 1979 Flood, design flood and dam break flood for Machhu Dam-II and also to mark the limit of areas affected by these floods on contour map of scale 1:15,000, downstream of Machhu Dam-II simulating the flood wave movement by dynamic wave routing technique as adopted in DAMBRK model, using the information available for the study.

The extent of flooding on both sides of river reach has also been studied for hydrographs for revised estimates of 100, 200, 500 and 1000 year return periods.

4.0 METHODOLOGY

This study uses the National Weather Service's DAMBRK model (Fread, 1984) to compute the maximum stage reached d/s of Machhu Dam-II due to Dam break flood, [Refer user manual of NWS (Fread, 1984)] and for computation of peak stages reached d/s of Machhu Dam-II due to 100,200, 500 and 1000 years return period floods, year 1979 flood and design flood hydrograph for Machhu Dam-II, the routing option of DAMBRK model using dynamic wave theory is used. This option simulates the movement of flood wave through downstream channel using the complete unsteady flow equations, for the one dimensional open channel flow, alternatively known as Saint Venant's equations.

The Saint-Venant unsteady flow equations consist of a conservation of mass equation, i.e.,

$$\frac{\partial A}{\partial X} + \frac{\partial (A+A_o)}{\partial t} - q = 0 \qquad \ldots(1)$$

and a conservation of momentum equation, i.e.,

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

Where A is the active cross-sectional area of flow, A_o is the inactive (off-channel storage) cross-sectional area, x is the longitudinal distance along the channel (valley), t is the time, q is the lateral inflow or outflow per linear distance along the channel (inflow is positive and outflow is negative in sign), g is the acceleration due to gravity, S_f is the friction slope, and S_e is the expansion-contraction slope. The friction slope is evaluated from Manning's equation for uniform steady flow, i.e.,

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

in which n is the Manning coefficient of frictional resistance and R is the hydraulic radius defined as A/B where B is the top width of the active cross-sectional area. The term (S_e) is defined as follows:

$$S_e = \frac{k\ \Delta\ (Q/A)^2}{2\ g\ \Delta\ x} \qquad \ldots(4)$$

in which k is the expansion-contraction coefficient varying from 0.0 to ± 1.0 (+ if contraction, - if expansion), and $\Delta\,(Q/A)^2$ is the difference in the term $(Q/A)^2$ at two adjacent cross-sections separated by a distance Δx. L is the momentum effect of lateral flow assumed herein to enter or exit perpendicular to the direction of the main flow. This term has the following form : 1) lateral inflow, L=0; 2) seepage lateral outflow, L = -0.5 q Q/A; and 3) bulk lateral outflow, L = -qQ/A.

Equations 1-2 were modified by Fread (1975, 1976) to better account for the differences in flood wave properties for flow occurring simultaneously in the river channel and the over bank flood plain of the downstream valley. As modified equation 1-2 become:

$$\frac{\partial(K_c Q)}{\partial X_c} + \frac{\partial(K_1 Q)}{\partial x_1} + \frac{\partial(K_r Q)}{\partial x_r} + \frac{\partial A}{\partial t} - q = 0 \qquad \ldots(5)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(K_c^2\, Q^2/A_c)}{\partial\, x_c} + \frac{\partial(K_1^2 Q^2/A_1)}{\partial\, x_1} + \frac{\partial(K_r^2\, Q^2/A_r)}{\partial\, x_r}$$

$$+ gA_c[\frac{\partial h}{\partial x_c} + S_{fc} + S_e] + gA_1\, [\frac{\partial h}{\partial x_1} + S_{f1}]$$

$$+ gA_r\, [\frac{\partial h}{\partial x_r} + S_{fr}] = 0 \qquad \ldots(6)$$

in which the subscripts (c), (1), and (r) represent the channel, left flood-plain, and right flood-plain sections, respectively. The parameters (K_c; K_1; K_r) proportion the total flow (Q) into channel flow, left flood-plain flow, and right flood-plain flow, respectively. These are defined as follows:

$$K_c = \frac{1}{1 + K_1 + k_r} \qquad \ldots(7)$$

$$K_1 = \frac{k_1}{1 + k_1 + k_r} \qquad \ldots(8)$$

$$K_r = \frac{kr}{1 + k_1 + k_r} \qquad \ldots(9)$$

in which

$$k_1 = \frac{Q_1}{Q_c} = \frac{n_c}{n_1}\,\frac{A_1}{A_c}\,[\frac{R_1}{R_c}]^{2/3}\,[\frac{\Delta x_c}{\Delta x_1}]^{1/2} \qquad \ldots(10)$$

$$K_r = \frac{Q_r}{Q_c} = \frac{n_c}{n_r} \frac{A_r}{A_c} \left[\frac{R_r}{R_c} \right]^{1/2} \left[-\frac{\Delta x_c}{\Delta x_r} \right]^{1/2} \qquad \text{...(11)}$$

Equations 10-11 represent the ratio of flow in the channel section to flow in the left and right flood-plain (overbank) sections, where the flows are expressed in terms of the Manning equation in which the energy slope is approximated by the water surface slope ($\Delta h / \Delta x$).

The friction slope terms in equation (6) are given by the following:

$$S_{fc} = \frac{n_c^2 | K_c Q | K_c Q}{2.21 A_c^2 R_c^{4/3}} \qquad \text{...(12)}$$

$$S_{f1} = \frac{n_1^2 | K_1 Q | K_1 Q}{2.21 A_1^2 R_1^{4/3}} \qquad \text{...(13)}$$

$$S_{fr} = \frac{n_r^2 | K_r Q | K_r Q}{2.21 A_r^2 R_r^{4/3}} \qquad \text{...(14)}$$

In equation 5 the term A is the total cross-sectional area, i.e.,

$$A = A_c + A_1 + A_r + A_o \qquad \text{...(15)}$$

Where A_o is the off-channel storage (inactive) area.

Equations (1) - (2) and (5) - (6) constitute a system of partial differential equations of the hyperbolic type. They contain two independent variables, x and t, and two dependent variables, h and Q;

the remaining terms are either functions of x, t, h and/or Q, or they are constants. These equations are not amenable to analytical solutions except in cases where the channel geometry and boundary conditions are uncomplicated and the non-linear properties of the equations are either neglected or made linear. The equations may be solved numerically by performing two basic steps. First, the partial differential equations are represented by a corresponding set of finite difference algebraic equations and second, the system of algebraic equations is solved in conformance with prescribed initial and boundary conditions.

Eqs. (1) - (2) and (5) - (6) can be solved by either explicit or implicit finite difference techniques (Liggett and Cunge, 1975). Explicit methods, although simpler in application, are restricted by mathematical stability considerations to very small computational time steps (on the order of a few minutes or even seconds) Such small time steps cause the explicit methods to be very inefficient in the use of computer time. Implicit finite difference techniques Amein and Fang, 1970: Strelkoff, 1970), however, have no restrictions on the size of the time step due to mathematical stability; however, convergence considerations may require its size to be limited (Fread, 1974 a).

Of the various implicit schemes that have been developed, the "weighted four-point" scheme appears most advantageous since it can readily be used with unequal distance steps and its stability convergence properties can be easily controlled. In the weighted four-point implicit finite difference scheme, the continuous x-t region in which solutions of h and Q are sought, is represented by a rectangular net of discrete points. The net points are determined by the intersection of lines drawn parallel to the x and t axes. Those parallel to the x axis represent time lines; they have a spacing of Δt, which also need not be constant. Each point in the rectangular network can be identified by a subscript (i) which designates the x position and a superscript (j) which designates the time line.

The time derivatives are approximated by a forward difference quotient centered between the i^{th} and $i+1$ points along the x axis, i.e.,

$$\frac{\Delta K}{\Delta t} = \frac{K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j}{2\Delta t_j} \quad \ldots(16)$$

where K represents any variable.

The spatial derivatives are approximated by a forward difference quotient positioned between two adjacent time lines according to weighting factors of θ and $1-\theta$ i.e.,

$$\text{(i)} \quad \frac{\partial K}{\partial x} = \theta\left[\frac{K_{i+1}^{j+1} - K_i^{j+1}}{\Delta x_i}\right] + (1-\theta)\left[\frac{K_{i+1}^j - K_i^j}{\Delta x_i}\right] \quad \ldots(17)$$

(ii) Variables other than derivatives are approximated at the time level where the spatial derivatives are evaluated by using the same weighting factors, i,e.,

$$\text{(iii)} \quad K = \theta \left[\frac{K_i^{j+1} + K_{i+1}^{j+1}}{2} \right] + (1-\theta) \frac{K_i^j + K_{i+1}^j}{2} \qquad \ldots(18)$$

A θ weighting factor of 1.0 yields the fully implicit or backward difference scheme used by Baltzer and Lai (1968). A weighting factor of 0.5 yields the box scheme used by Amein and Fang (1970). The influence of the θ weighting factor on the accuracy of the computations was examined by Fread (1974a), who concluded that the accuracy decreases as θ departs from 0.5 and approaches 1.0. This effect becomes more pronounced as the magnitude of the computational time step increases. Usually, a weighting factor of 0.60 is used so as to minimize the loss of accuracy associated with greater values while avoiding the possibility of a weak or pseudo instability noticed by Baltzer and Lai (1968), and Chaudhry and Contractor (1973); however, θ may be specified other than 0.60 in the data input to the DAMBRK model.

When the finite difference operators defined by equation (16)-(18) are used to replace the derivatives and other variables in equation (1) - (2) the following weighted four-point implicit difference equations are obtained:

$$\theta \left[\frac{Q_{i+1}^{j+1} - Q_i^{j+1}}{\Delta x_i} \right] - \theta\, q_i^{j+1} + (1-\theta) \left[\frac{Q_{i+1}^{j} - Q_i^{j}}{\Delta x_i} \right]$$

$$-(1-\theta)\, q_i^j + \frac{(A+A_o)_i^{j+1} + (A+A_o)_{i+1}^{j+1} - (A+A_o)_i^j - (A+A_o)_{i+1}^j}{2\,\Delta t_j}$$

$$= 0 \qquad \ldots(19)$$

$$\left(\frac{Q_i^{j+1} + Q_{i+1}^{j+1} - Q_i^j - Q_{i+1}^j}{2\ \Delta t_j} \right) + \theta \left[\frac{(Q^2/A)_{i+1}^{j+1} - (Q^2/A)_i^{j+1}}{\Delta x_i} \right.$$

$$+ g\, \bar{A}^{j+1} \left(\frac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + \bar{S}_f^{j+1} + S_{ce}^{j+1} \right) \Big] + (1-\theta)$$

$$\left[\frac{(Q^2/A)_{i+1}^j - (Q^2/A)_i^j}{\Delta x_i} + g\, \bar{A}^j \left(\frac{h_{i+j}^j - h_i^j}{\Delta x_i} \right. \right.$$

$$+ \bar{S}_f^j + S_{ce}^j \Big) \Big] = 0 \qquad \ldots(20)$$

where:

$$\bar{A} = (A_i + A_{i+1}) / 2 \qquad \ldots(21)$$

$$\bar{S}_f = n^2\, \bar{Q} |\bar{Q}| / (2.2\ \bar{A}^2\ \bar{R}^{4/3}) \qquad \ldots(22)$$

$$\bar{Q} = (Q_i + Q_{i+1}) / 2 \qquad \ldots(23)$$

$$\bar{R} = \bar{A}/\bar{B} \qquad \ldots(24)$$

$$\bar{B} = (B_i + B_{i+1}) / 2 \qquad \ldots(25)$$

The terms associated with the j^{th} time line are known from either the initial conditions or previous computations. The initial conditions refer to values of h and Q at each node along the x axis for the first time line (j = 1).

Equations (19) - (20) cannot be solved in an explicit or direct manner for the unknowns since there are four unknowns and only two equations. However, if equation (19) - (20) are applied to each of the (N-1) rectangular grids between the upstream and downstream boundaries, a total of (2N-2) equations with 2N unknowns can be formulated (N denotes the total number of nodes). Then, prescribed boundary conditions, one at the upstream boundary and one at the downstream boundary, provide the necessary two additional equations required for the system to be determinate. The resulting system of 2N non-linear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (Amein and Fang, 1970)

Computations for the iterative solution of the non-linear system are begun by assigning trial values to the 2N unknowns. Substitution of the trial values into the system of non-linear equations yields a set of 2N residuals. The Newton-Raphson method provides a means for correcting the trial values until the residuals are reduced to a suitable tolerance level. This is usually accomplished in one or two iterations through use of linear extrapolation for the first trial values. If the Newton-Raphson corrections are applied only once, i.e., there is no iteration, the non-linear system of difference equations degenerates to the equivalent of a quasi-linear formulation which may require smaller time steps than the non-linear formulation for the same degree of numerical accuracy.

A system of 2N x 2N linear equations relates the corrections to the residuals and the Jacabian coefficient matrix composed of a partial derivatives of each equation with respect to each unknown variable in that equation. The coefficient matrix of the linear system has a banded structure which allows the system to be solved by a compact quad-diagonal Gaussian elimination algorithm (Fread, 1971), which is very efficient with respect to computing time and storage. The required storage is 2N x 4 and the required number of computational steps is approximately 38N.

The DAMBRK model has the option to use either equation (1) - (2) or equations (5) - (6). The former is a somewhat simpler treatment in which a total or composite cross-section is used, whereas the latter set utilizes a more detailed representation of the flow cross-section. Eqs. (5) - (6) are recommended when the channel is sufficiently large to carry a significant portion of the total flow and the channel has a rather meandrous path through the downstream valley.

5.0 APPLICATION

The methodology as described in Section 4.0 is applied to the reach about 40 km long downstream of Machhu Dam-II. The important towns on the reach to be affected by different floods are Morvi and Malia, located respectively at 8.246 and 37.007 kms downstream of Machhu Dam-II. The unrevised floods of 100,200, 500, and 1000 years return periods with peaks 8765.3, 9734.9, 10956.3, and 11879.7 cumecs respectively were readily available for this study. These floods were estimated considering the unit hydrograph and storm depth of desired period in absence of long term data of observed peak floods of Machhu River. Recently while the study was in progress, the revised peaks of 100,200, 500 and 1000 years return period floods as 12707, 14037, 15905, and 17150 cumecs respectively were provided by the Gujarat Irrigation Department. The flood hydrographs with these peaks were computed considering the ratio of flow at different times, to peak flow as in case of unrevised return period floods. The flood hydrographs of 1979 flood (peak = 13098.8 cumecs), Design flood (peak = 26319 cumec) and dam break flood (peak =86885.6 cumecs, as computed by DAMBRK Model) were also available. The hydrographs with revised and unrevised peaks of 100, 200, 500 and 1000 year return period flood, design flood, 1979 flood and dam break flood are given in Table 1-3 and also shown in Fig. 1-5.

A contour map covering the details of about 40Km long reach downstream of the Machhu Dam-II was made available by Gujarat Irrigation Department with 0.5 m interval contours at the scale 1:15,000, however, the details are missing on right side of river reach from 0.0 to 3.0 km. downstream of dam site.

In this study, six cross-section details are available at locations 1.303, 9.348, 17.393, 25.438, 33.290, and 39.65 km. downstream of Machhu Dam-II. The cross-sections are specified by location and table of top width and corresponding elevations. In the case of first three cross-sections, measurements on the top widths have been made upto HFL marks noted on both sides of banks and in case of last three sections, the top widths were not measured upto the HFL marks noted on both sides of the banks. The Manning's roughness coefficient and expansion-contraction coefficients for different reaches downstream of the Machhu Dam-II have been take as mentioned in NIH report, Case Study No. 16.

Using the data available as mentioned above, maximum flood elevations were computed at various sections downstream of Machhu Dam-II, for above mentioned different floods and accordingly the areas affected by different floods were marked as well as assessed respectively.

Table 1 : FLOOD HYDROGRAPH OF DIFFERENT RETURN PERIOD AT MACHHU DAM-II SITE

TIME	ORDINATES OF FLOOD OF RETURN PERIOD			
(Hrs)	100 years (cumecs)	200 years (cumecs)	500 years (cumecs)	1000 years (cumecs)
0.00	175.00	178.00	181.00	184.00
2.00	453.00	488.00	535.00	570.00
4.00	1084.00	1190.00	1339.00	1444.00
6.00	1904.00	2103.00	2381.00	2574.00
8.00	2923.00	3236.00	3717.00	4034.00
10.00	4003.00	4456.00	5087.00	5537.00
12.00	4891.00	5452.00	6235.00	6779.00
14.00	5775.00	6433.00	7343.00	7959.00
16.00	7603.00	8443.00	9600.00	10375.00
18.00	9769.00	10842.00	12300.00	13281.00
20.00	11467.00	12685.00	14372.00	15502.00
22.00	12707.00	14037.00	15905.00	17150.00
24.00	12439.00	13745.00	15557.00	16773.00
26.00	10695.00	11835.00	13362.00	14405.00
28.00	8540.00	9457.00	10661.00	11489.00
30.00	6435.00	7132.00	8004.00	8620.00
32.00	4501.00	4969.00	5549.00	5967.00
34.00	3011.00	3304.00	3680.00	3944.00
36.00	2049.00	2227.00	2463.00	2626.00
38.00	1470.00	1582.00	1737.00	1841.00
40.00	1091.00	1164.00	1267.00	1335.00
42.00	833.00	881.00	952.00	996.00
44.00	656.00	687.00	735.00	766.00
46.00	130.00	130.00	130.00	130.00
48.00	130.00	130.00	130.00	130.00

TABLE 2 : Flood Hydrograph at Machhu-II Dam Site

Time (Hrs)	Design flood ord'nate (m^3/Sec)	1979 Flood ordinate (m^3/Sec)	Dam-break Time (hrs)	flood Discharge (m^3/s)
0	141.5	187.12	0.0	7893.136
2	141.5	374.24	0.1	8641.348
4	424.5	623.76	0.2	11193.584
6	707.5	997.99	0.3	16159.13
8	990.5	1497.01	0.4	23919.839
10	1415.0	2245.52	0.5	34701.969
12	1981.0	3243.46	0.6	48563.253
14	2405.5	4615.79	0.7	65472.672
16	3113.0	6362.23	0.8	82485.925
18	3962.0	8607.81	0.9	86885.612
20	4695.5	11227.60	1.0	80868.608
22	5518.5	13098.85	1.1	68574.409
24	5801.5	11857.70	1.2	56562.135
26	6226.0	9979.99	1.3	46270.018
28	6792.0	5364.26	1.4	37577.363
30	7560	3742.53	1.5	30305.196
32	9339.0	2745.10	1.6	24055.821
34	9905.0	2122.50	1.7	18876.269
36	11037.0	1621.59	1.8	16061.807
38	12735.0	1248.03	1.9	14393.04
40	16272.5	810.79	2.0	13300.293
42	20376.0	561.47	2.2	11992.833
44	25753.0	311.87	2.4	11278.625
46	26319.00	124.52	2.6	10818.722
48	19810.0	113.20	2.8	11465.717
50	14150.0	113.20	3.0	13498.477
52	8914.5		3.2	12279.993
54	5377.0		3.4	10742.708
56	3254.5		3.6	9355.414
58	1981.0		3.8	8263.571
60	1132.0		4.0	8021.097
62	566.0			
64	141.5			

TABLE 3 : FLOOD HYDROGRAPHS CORRESPONDING TO DIFFERENT PEAK FLOODS AT MACHHU- II DAM SITE

Time (hours)	Ordinates of flood hydrographs in cumecs with peaks 8765.3 Cumecs	9374.9 Cumecs	10956.3 Cumecs	11879.7 Cumecs
1	2	3	4	5
0	160.7	162.7	165.2	167.2
2	352.3	376.9	409.7	434.4
4	785.0	862.3	959.9	1036.9
6	1348.4	1493.3	1674.7	1817.1
8	2048.0	2288.9	2591.7	2825.4
10	2789.2	3117.8	3532.0	3862.6
12	3399.1	3806.3	4320.2	4720.4
14	4006.0	4483.0	5079.5	5534.6
16	5261.0	5873.3	6629.2	7202.9
18	6762.5	7527.8	8481.5	9209.3
20	7913.5	8801.0	9904.1	10742.3
22	8765.3	9734.9	10956.3	11879.7
24	8580.8	9532.8	10717.2	11620.0
26	7383.7	8213.7	9211.3	9985.0
28	5904.2	6572.3	7357.1	7972.1
30	4458.9	4965.5	5534.0	5990.5
32	3130.8	3472.4	3848.5	4160.1
34	2107.7	2321.7	2565.6	2762.6
36	1447.5	1577.7	1731.1	1852.5
38	1049.9	1132.8	1233.0	1311.4
40	789.5	843.9	910.4	962.2
42	612.9	649.4	693.9	728.4
44	491.0	515.3	545.3	568.5
48	130.4	130.4	130.4	130.4

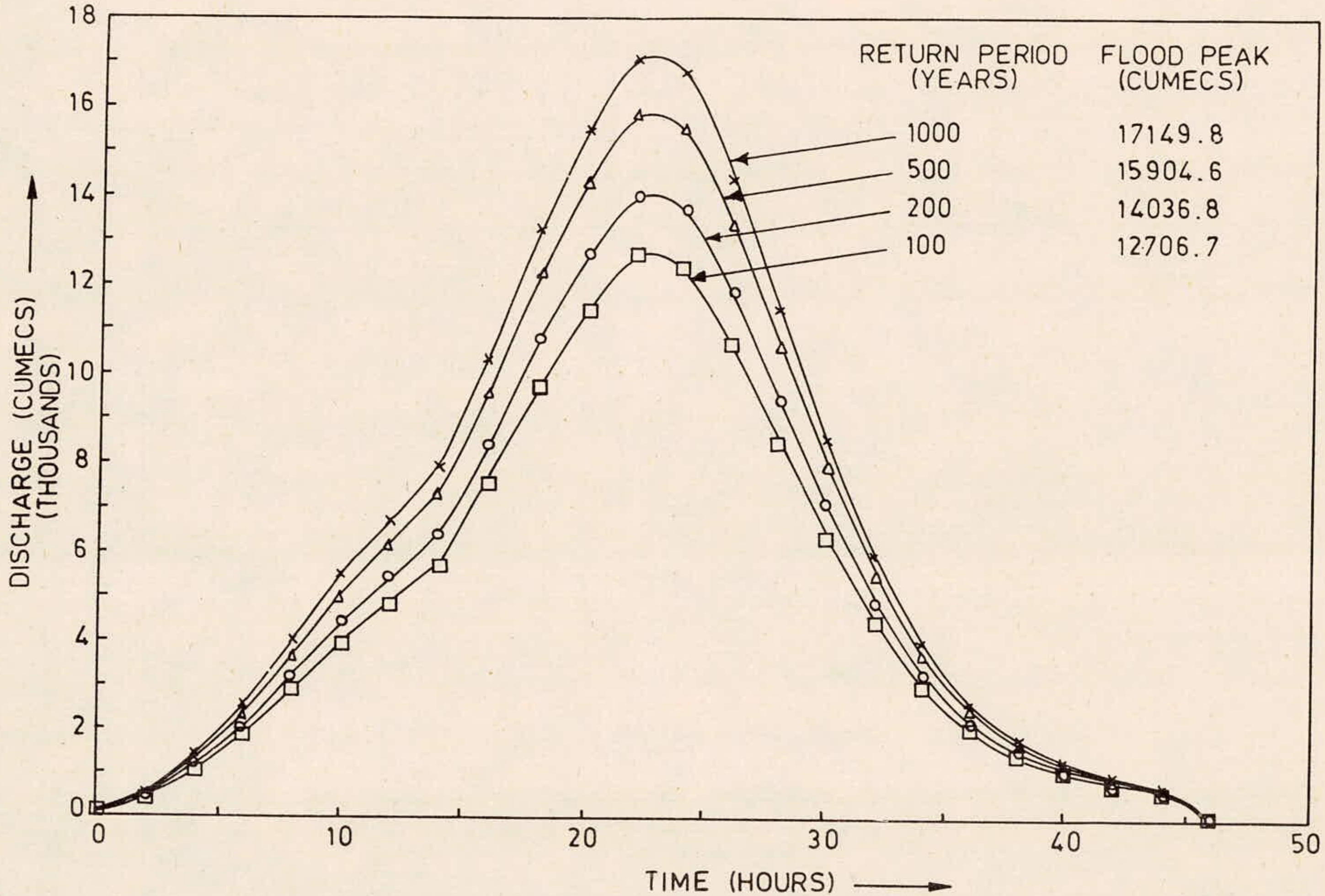

FIG.1 FLOOD HYDROGRAPHS OF DIFFERENT RETURN PERIOD AT MACHHU DAM-II SITE.

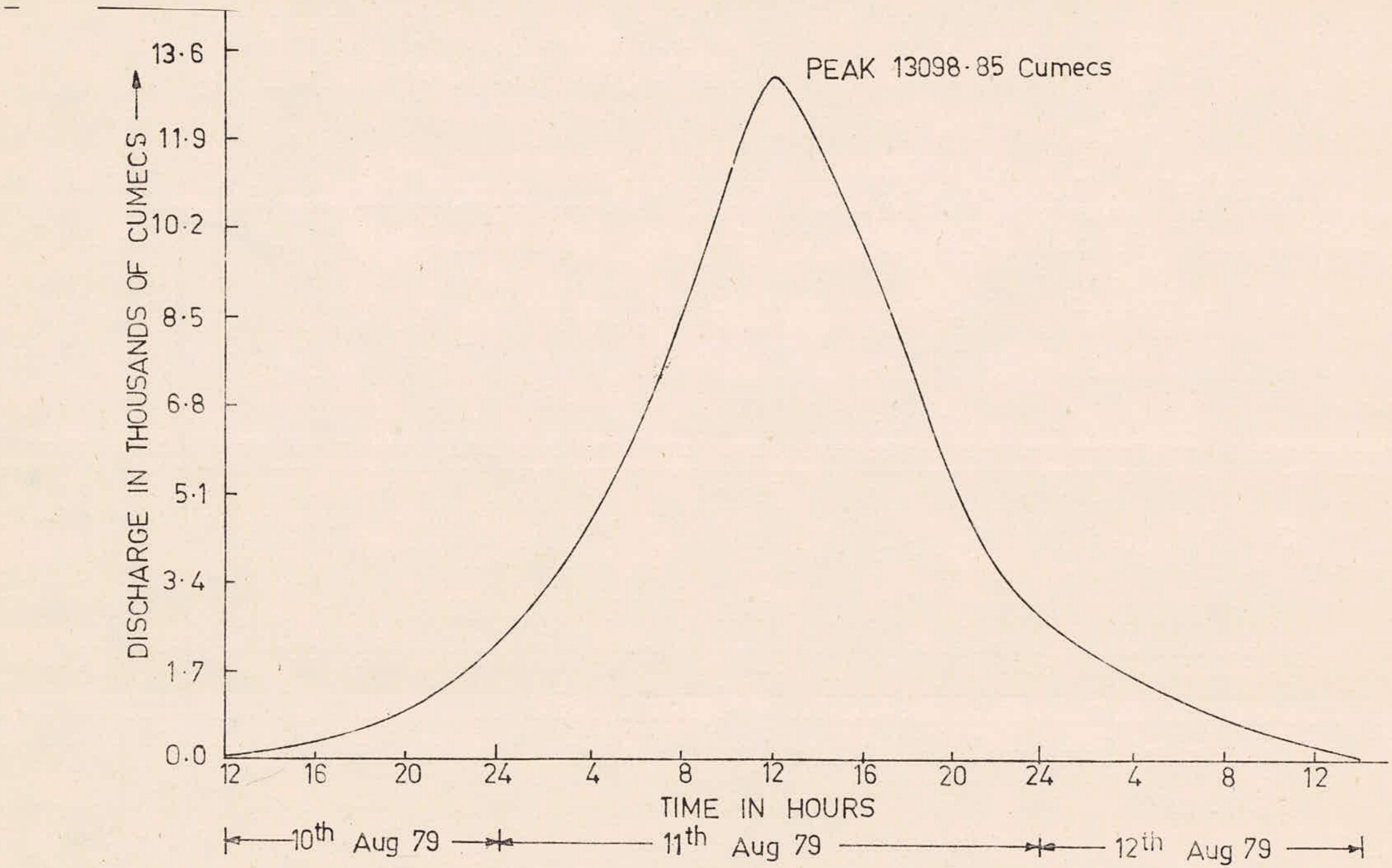

FIG. 2. 1979 FLOOD AT MACHHU II DAM SITE

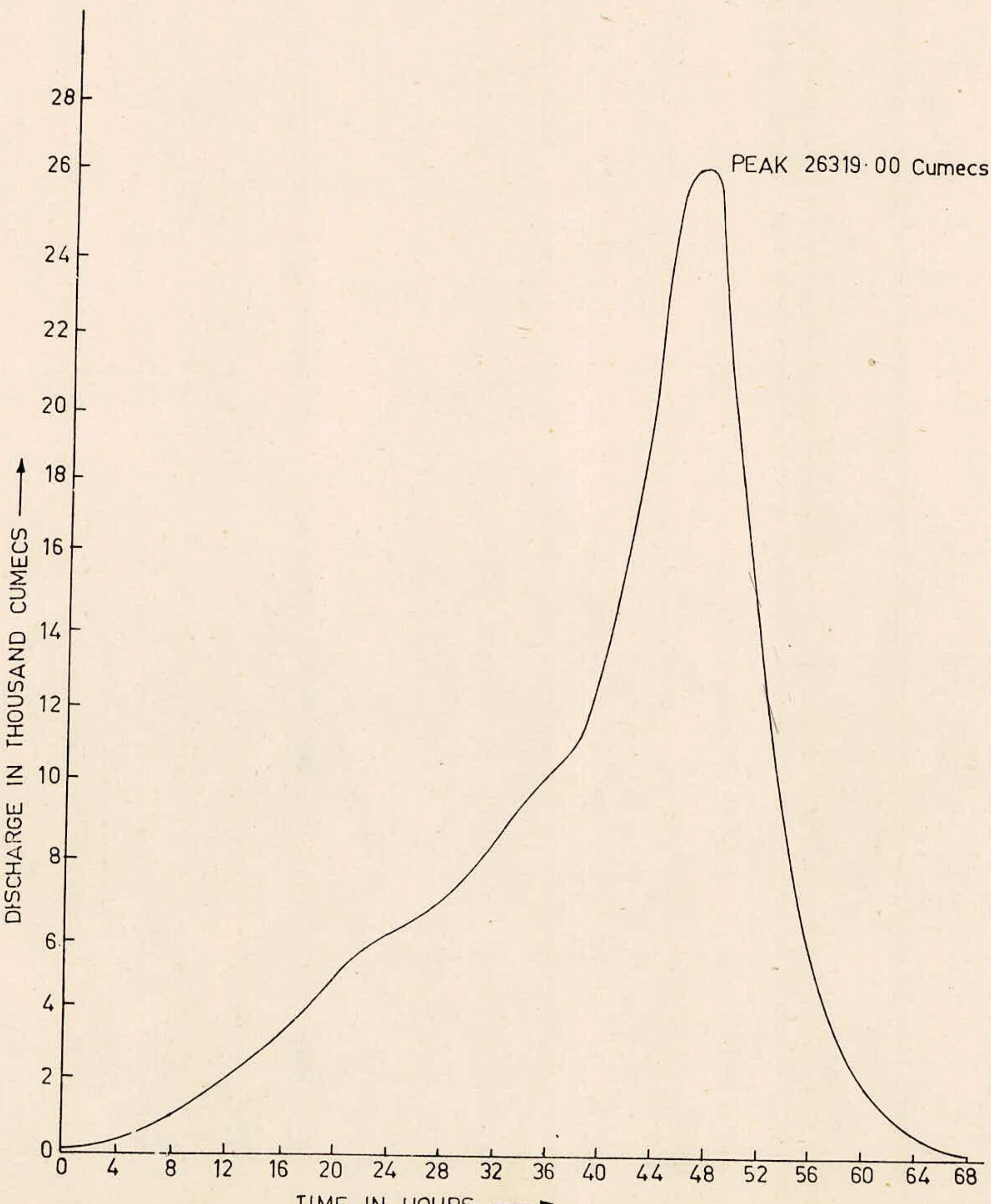

FIG. 3. DESIGN FLOOD HYDROGRAPH (M.P.F.) FOR MACHHU II PROJECT

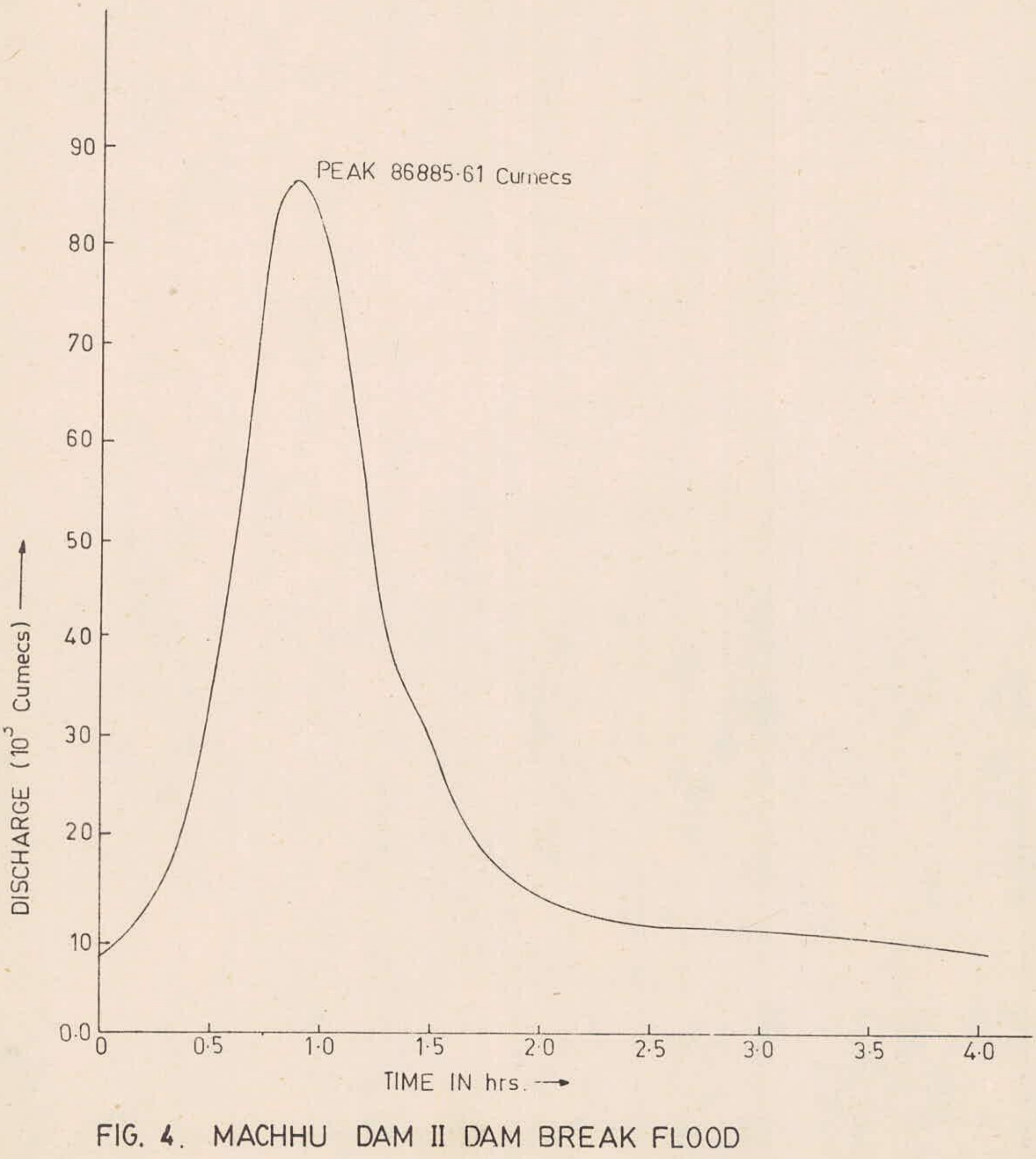

FIG. 4. MACHHU DAM II DAM BREAK FLOOD

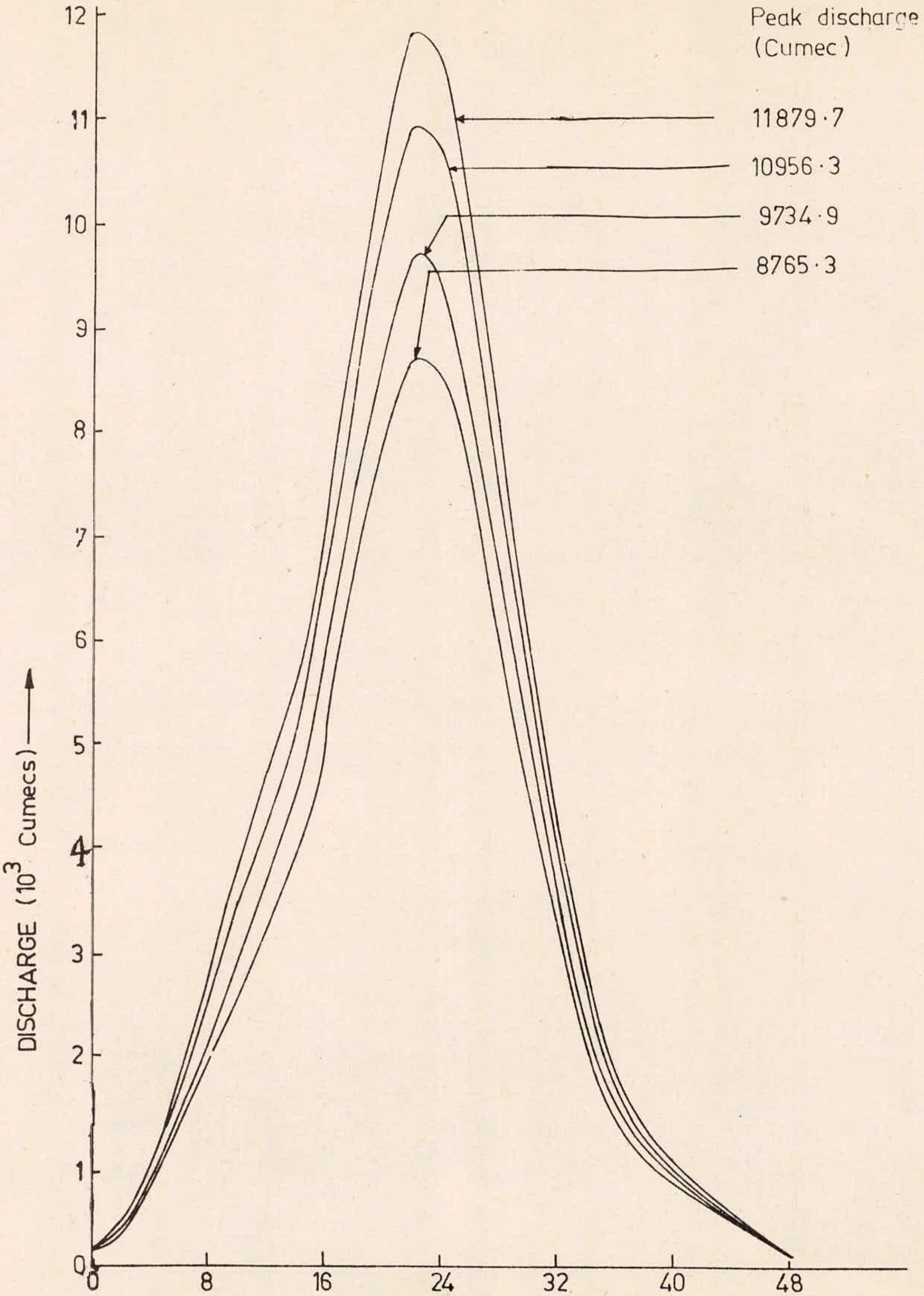

FIG. 5. FLOODS OF DIFFERENT PEAKS AT MACHHU -II DAM SITE

6.0 RESULTS

The results obtained such as peak stage, area inundated and width of water spread along the reach due to various floods as mentioned above are tabulated in Table 4-7 and discussed as follows:

6.1 Dam Break Flood

It is the severe most flood among all the flood events with peak 86886.6 cumecs, used for the analysis. It affects the area downstream of the dam throughout the reach. The total area affected by dam break flood on left and right side of 39.65 km long river reach are 95.326sq. km. and 65.075 sq. km. The maximum extent of water spread by the flood is 6.12 km. at location 8.50 km. and minimum extent of water spread is 3.21 km. at location 12.57 km. The peak stages computed at 0.0 km. and at downstream end of the reach (at 39.65 km.) are 56.97 m and 11.97 m respectively.

6.2 Design Flood

This is the second severe most flood among floods used in this study. This flood also affects the area throughout the 39.65 km. long river reach downstream of the dam. The total area affected by the flood at left and right side of the reach are 76.716 sq. km. and 57.22 sq. km. The maximum water spread by this flood is 5.52 km. at location 34.2 km. and minimum water spread is 2.04 km. at 12.57 km. The peak stages computed at 0.0 km. and at 39.65 km. are 51.83 m. and 11.76 m respectively.

6.3 1000 Year Flood (Revised)

This flood also affects the downstream area on both the sides of the reach. The area inundated on left and right side of the reach are 46.171 sq. km. and 26.571 sq. km. respectively. The maximum water spread by the flood along the reach is 5.925 km. at 35.12 km. location and minimum water spread is 1.71 km. at location 8.5 km. The peak stages computed at 0.0 km. and at 39.65 km. are 51.24 m and 11.503 km. respectively.

6.4 1979 Flood

This represents observed flood in 1979 at Machhu Dam-II site and due to which failure of Machhu Dam-II had taken place. This flood remains within the banks of river upto first 19 km. of river reach downstream of the dam. The total area affected by the flood for remaining river reach on left and right sides is 37.49 sq. km. and 31.05 sq. km. respectively. The maximum water spread due to the flood is 5.52 km. at 37.84 km. location and minimum water spread is 2.16 km. at 19.8 km. location. The peak stage computed for the flood at 0.0 km. and at 39.65 km. are 47.70 m and 11.06 m respectively.

6.5 100, 200, 500 and 1000 Year Return Period Floods with Unrevised Peaks

100 and 200 years flood remains within the bank upto river reach 21.6 km. while 500 and 1000 year floods remain within the banks upto 18.0 km. of the river reach. The total area affected by the 100 200, 500 and 1000 year floods are 37.287 sq. km., 43.921 sq.km., 57.231 sq.km. and 54.588 sq. km. respectively. The maximum water spread by these floods is 4.26 km. 4.335 km, 4.83 km. and 5.115 km. respectively while the minimum water spread is 1.815 km. 1.95 km., 0.87 km. and 2.07 km. respectively. The peak stages computed at 0.0 km. for these floods are 45.63 m., 46.15 m, 46.74m and 47.17 m respectively and the peak stages at 39.65 km. are 10.74 m, 1083 m, 10.94 m and 11.02 m respectively.

Comparison

It can be inferred from results given in Table 6 that dam break flood affects the maximum area 160.40 sq. km. throughout the reach and 100 years unrevised flood affects the minimum area (37.287 sq. km.) along the reach. The areas affected by other floods used in this study lie in between these two figures. It can also be observed from results given in Table 4 and 5 that peak stage decreases as the distance increases downstream for all the flood events studied.

TABLE 4: MAXIMUM FLOOD ELEVATIONS DOWNSTREAM OF MACHHU DAM-II DUE TO VARIOUS FLOODS

	MAXIMUM FLOOD ELEVATION REACHED DUE TO						
	UNREVISED FLOODS PEAK OF						
DIST-ANCE (km)	100 Yr (m)	200 Yr (m)	500 Yr (m)	1000 Yr (m)	1979 FLOOD (m)	DESIGN FLOOD (m)	DAM BREAK FLOOD (m)
0.00	45.63	46.15	46.74	47.17	47.70	51.83	56.97
0.85	44.60	45.12	45.70	46.13	46.50	50.83	55.78
1.70	43.60	44.12	44.67	45.10	45.47	49.84	54.57
2.55	42.62	43.12	43.66	44.08	44.45	48.85	53.35
3.40	41.60	42.14	42.66	43.08	43.44	47.86	52.12
4.25	40.08	41.17	41.67	42.09	42.44	46.85	50.89
5.10	39.73	40.20	40.70	41.10	41.45	45.84	49.65
5.90	38.77	39.24	39.72	40.12	40.46	44.80	48.39
6.80	37.81	38.26	38.75	39.13	39.46	43.72	47.11
7.65	36.83	37.27	37.75	38.13	38.45	42.57	45.77
8.50	35.80	36.23	36.71	37.08	37.40	41.34	44.29
9.36	34.63	35.00	35.57	35.95	36.26	40.00	42.59
10.16	33.50	33.94	34.50	34.89	35.22	38.97	41.39
10.96	32.37	32.86	33.48	33.89	34.23	37.90	40.20
11.77	31.27	31.84	32.52	32.96	33.28	36.86	39.04
12.37	30.23	30.90	31.64	32.11	32.44	35.83	37.90
13.40	29.29	30.10	30.86	31.34	31.67	34.81	36.78
14.18	28.50	29.46	30.19	30.64	30.98	33.83	35.69
14.99	27.85	28.93	29.60	30.04	30.37	32.89	34.64
15.80	27.38	28.48	29.10	29.50	29.77	32.00	33.69
16.60	27.04	28.09	28.63	28.93	29.12	31.33	32.85
17.40	26.71	27.71	28.16	28.40	28.57	30.75	32.15
18.20	26.33	27.27	27.65	27.89	28.05	30.18	31.46
19.00	25.99	26.78	27.13	27.37	27.51	29.59	30.78
19.80	25.65	26.25	26.60	26.84	26.97	28.99	30.08
20.62	25.28	25.73	26.00	26.31	26.44	28.38	29.38
21.40	24.83	25.20	25.54	25.77	25.90	27.75	28.68
22.23	24.52	24.68	25.00	25.22	25.34	27.11	27.97
23.00	23.76	24.12	24.43	24.64	24.76	26.46	27.23
23.84	23.16	23.54	23.84	24.04	24.15	25.75	26.50
24.65	22.49	22.89	23.18	23.38	23.47	25.00	25.69
25.45	21.75	22.11	22.39	22.58	22.68	24.10	24.73
26.32	21.12	21.41	21.68	21.87	21.96	23.50	23.89
27.20	20.43	20.70	20.96	21.14	21.22	22.47	23.04
28.00	19.72	19.98	20.22	20.39	20.46	21.64	22.19
28.94	19.00	19.24	19.47	19.62	19.70	20.80	21.33
29.80	18.26	18.48	18.70	18.85	18.91	19.97	20.47
30.60	17.51	17.70	17.90	18.07	18.13	19.13	19.61
31.55	16.74	16.94	17.14	17.28	17.34	18.30	18.75
32.44	15.93	16.13	16.30	16.45	16.50	17.45	17.87
33.30	14.89	15.11	15.33	15.47	15.53	16.52	16.91
34.20	13.88	14.00	14.30	14.45	14.51	15.56	15.92
35.12	13.18	13.34	13.53	13.66	13.72	14.73	15.06
36.03	12.61	12.75	12.90	13.03	13.08	14.00	14.31
36.93	12.10	12.23	12.37	12.47	12.52	13.38	13.65
37.84	11.63	11.75	11.87	11.97	12.00	12.80	13.06
38.75	11.20	11.29	11.40	11.49	11.53	12.27	12.51
39.65	10.74	10.83	10.94	11,02	11.06	11.76	11.97

TABLE 5: MAXIMUM FLOOD ELEVATIONS DOWNSTREAM OF MACHHU DAM-II DUE TO VARIOUS FLOODS

DISTANCE (km)	MAXIMUM FLOOD ELEVATION REACHED DUE TO						
	UNREVISED FLOODS PEAK OF						
	100 Yr (m)	200 Yr (m)	500 Yr (m)	1000 Yr (m)	1979 FLOOD (m)	DESIGN FLOOD (m)	DAM BREAK FLOOD (m)
0.00	45.03	46.15	46.74	47.17	47.70	51.83	56.97
0.85	44.00	45.12	45.70	46.13	46.50	50.83	55.78
1.70	43.00	44.12	44.57	45.10	45.47	49.84	54.57
2.55	42.02	43.12	43.66	44.08	44.45	48.85	53.35
3.40	41.00	42.14	42.66	43.08	43.44	47.86	52.12
4.25	40.08	41.17	41.67	42.09	42.44	46.85	50.89
5.10	39.73	40.20	40.70	41.10	41.45	45.84	49.65
5.90	38.77	39.24	39.72	40.12	40.46	44.80	48.39
6.80	37.81	38.26	38.75	39.13	39.46	43.72	47.11
7.65	36.83	37.27	37.75	38.13	38.43	42.57	45.77
8.50	35.80	36.23	36.71	37.08	37.40	41.34	44.29
9.36	34.63	35.00	35.57	35.95	36.26	40.00	42.59
10.16	33.50	33.94	34.50	34.89	35.22	38.97	41.39
10.96	32.30	32.86	33.48	33.89	34.23	37.90	40.20
11.77	31.20	31.84	32.52	32.96	33.28	36.86	39.04
12.57	30.23	30.90	31.64	32.11	32.44	35.83	37.90
13.40	29.29	30.10	30.86	31.34	31.67	34.81	36.78
14.18	28.30	29.46	30.19	30.64	30.98	33.83	35.69
14.99	27.85	28.93	29.60	30.04	30.37	32.89	34.64
15.80	27.38	28.48	29.10	29.50	29.77	32.00	33.69
16.60	27.04	28.09	28.63	28.93	29.12	31.33	32.85
17.40	26.71	27.71	28.16	28.40	28.57	30.75	32.15
18.20	26.33	27.27	27.65	27.89	28.05	30.18	31.46
19.00	25.99	26.78	27.13	27.37	27.51	29.39	30.78
19.80	25.65	26.25	26.60	26.84	26.97	28.99	30.08
20.62	25.28	25.73	26.00	26.31	26.44	28.38	29.38
21.40	24.83	25.20	25.54	25.77	25.90	27.75	28.68
22.23	24.34	24.68	25.00	25.22	25.34	27.11	27.97
23.00	23.76	24.12	24.43	24.64	24.76	26.46	27.25
23.84	23.16	23.54	23.84	24.04	24.15	25.75	26.50
24.65	22.49	22.89	23.18	23.38	23.47	25.00	25.69
25.45	21.75	22.11	22.39	22.58	22.68	24.10	24.73
26.32	21.12	21.41	21.68	21.87	21.96	23.30	23.89
27.20	20.43	20.70	20.96	21.14	21.22	22.47	23.04
28.00	19.72	19.98	20.22	20.39	20.46	21.64	22.19
28.94	19.00	19.24	19.47	19.62	19.70	20.30	21.33
29.80	18.26	18.48	18.70	18.85	18.91	19.97	20.47
30.60	17.51	17.70	17.90	18.07	18.13	19.13	19.61
31.55	16.74	16.94	17.14	17.28	17.34	18.30	18.75
32.44	15.93	16.13	16.30	16.45	16.50	17.45	17.87
33.30	14.89	15.11	15.33	15.47	15.53	16.52	16.91
34.20	13.88	14.00	14.30	14.45	14.51	15.56	15.92
35.12	13.18	13.34	13.55	13.66	13.72	14.73	15.06
36.03	12.61	12.75	12.90	13.03	13.08	14.00	14.31
36.93	12.10	12.23	12.37	12.47	12.52	13.38	13.65
37.84	11.63	11.73	11.87	11.97	12.00	12.80	13.06
38.75	11.20	11.29	11.40	11.49	11.53	12.27	12.51
39.65	10.74	10.83	10.94	11.02	11.06	11.76	11.97

TABLE 6 : AREA INUNDATED BY VARIOUS FLOODS D/S OF MACHHU DAM-II(Sq.Km)
(excluding the area of river)

RIVER REACH (Km)	DAM BREAK FLOOD LSIDE	DAM BREAK FLOOD RSIDE	DESIGN FLOOD LSIDE	DESIGN FLOOD RSIDE	REVISED 1000 YRS FLOOD LSIDE	REVISED 1000 YRS FLOOD RSIDE	1979 FLOOD LSIDE	1979 FLOOD RSIDE	UNREVISED FLOOD OF RETURN PERIOD T = 1000 YRS. LSIDE	1000 YRS. RSIDE	500 YRS. LSIDE	500 YRS. RSIDE	200 YRS. LSIDE	200 YRS. RSIDE	100 YRS. LSIDE	100 YRS. RSIDE
	AREA INUNDATED DUE TO VARIOUS FLOODS (sq. km) AS															
0-2.20	5.085	DNA	3.51	DNA	2.769	----	----	----	DNA	----	----	----	----	----	----	----
2.2-5.90	9.154	DNA	6.400	DNA	3.825	----	----	----	DNA	----	----	----	----	----	----	----
5.9-9.36	14.40	6.275	12.138	4.768	9.09	1.755	----	----	WB	----	----	----	----	----	----	----
9.36-14.0	9.338	5.967	7.879	3.774	5.625	0.855	----	----	WB	----	----	----	----	----	----	----
14.0-18.0	8.185	7.525	6.936	6.426	1.777	----	----	----	WB	----	----	----	----	----	----	----
18.0-21.6	11.551	5.94	5.176	5.76	7.245	3.352	3.55	3.08	3.15	3.015	3.127	----	WB	----	----	----
21.6-25.3	9.996	9.435	8.925	9.027	7.425	4.725	8.568	5.890	8.032	5.635	6.043	4.995	4.815	4.86	WB	4.52
25.3-28.9	DNA	9.69	DNA	9.154	DNA	7.087	6.324	DNA	6.12	5.049	5.916	WB	5.712	WB	5.559	WB
28.9-32.1	15.121	4.692	15.718	4.437	5.940	0.515	10.51	3.366	DNA	3.264	10.12	WB	7.446	WB	7.293	WB
32.1-36.0	12.469	9.588	11.934	8.058	9.900	6.435	6.732	7.497	5.916	7.318	5.406	5.304	5.253	4.998	5.074	4.386
36.05-39.65 39.65	DNA	5.967	DNA	5.814	DNA	2.047	8.134	4.896	7.599	4.539	6.936	4.335	6.63	4.207	6.324	4.131
sum .	95.326	65.075	76.716	57.22	46.17	26.57	37.49	31.05	24.697	29.89	36.68	20.55	24.14	19.78	18.691	18.596
Total Area(sq.km.	160.40		133.334		72.742		68.543		54.588		57.231		43.921		37.287	

NOTE: LSIDE=LEFT SIDE OF THE RIVER REACH, RSIDE=RIGHT SIDE OF THE RIVER REACH
DNA=DETAILS NOT AVAILABLE , WB=FLOW WITHIN THE BANKS OF RIVER

TABLE 7 : WIDTH OF WATER SPREAD AT VARIOUS LOCATION D/S OF MACHHU DAM-II DUE TO VARIOUS FLOODS

	WIDTH OF WATER SPREAD DUE TO							
DIST-ANCE	DAM BREAK FLOOD	DESIGN FLOOD	1000 yrs FLOOD (REVISED)	1979 FLOOD	UNREVISED FLOODS OF T = 1000 YR	500 YR	200 YR	100 YR
(Km)	(Km)	(Km)	(Km)	(Km)	(km)	(Km)	(Km)	(Km)
0.85	2.19(LS)	1.44(LS)	1.08 LS		DNA			
1.70	2.59(LS)	1.365(LS)	1.02 LS		DNA			
3.40	4.44	2.865	2.55		WB			
4.20	5.48	2.16	1.95		WB			
5.10	5.3	2.22	1.905		WB			
6.80	5.115	4.515	1.71		WB			
8.50	6.12	5.025	4.32		WB			
9.36	5.49	4.935	1.575LS		WB			
10.96	3.72	3.18	0.525LS		WB			
12.57	3.21	2.04	1.5 LS		WB			
14.18	3.855	3.645			WB			
16.60	4.455	4.26			WB			
17.40	4.095	3.645			WB			
19.00	4.815	4.25	3.15		WB			
19.80	4.545	4.335	3.255	2.16	2.06	0.87	WB	
24.65	3.265	3.175	5.025	4.215	4.065	3.78	2.76	2.64
26.00	3.46(RS)	3.57(RS)	2.84RS	2.31RS	2.265RS	2.255RS	2.205RS	3.435
29.80	0.85(RS)	0.64(RS)	0.845RS	0.39RS				3.96
34.20	5.61	5.52	4.905	3.045	2.95	2.085	1.95	1.815
35.12	3.51(RS)	3.40(RS)	5.925	3.405	3.259	2.88	2.805	2.745
37.84	2.38(RS)	2.35(RS)	5.76	5.52	5.115	4.83	4.335	4.26

Note: Where one side width is available, river width is excluded
LS= left side of the reach, RS= right side of the reach
DNA=details not available, WB= flow is within the banks

FIG. 6 AREA INUNDATED FOR RIVER REACH 0.0 - 2.30 Km.

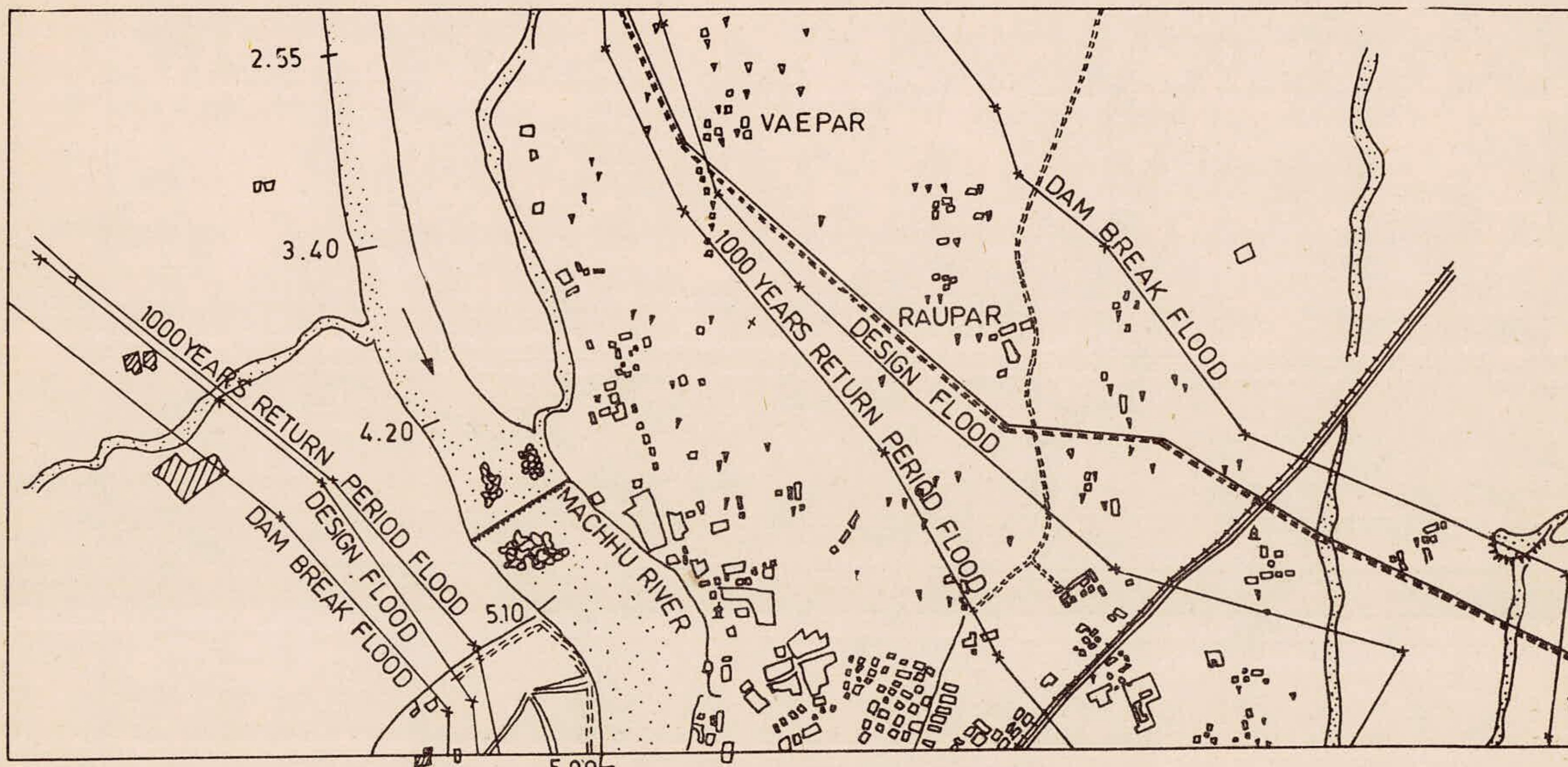

FIG. 7 AREA INUNDATED FOR RIVER REACH 2.30 - 5.90 Km.

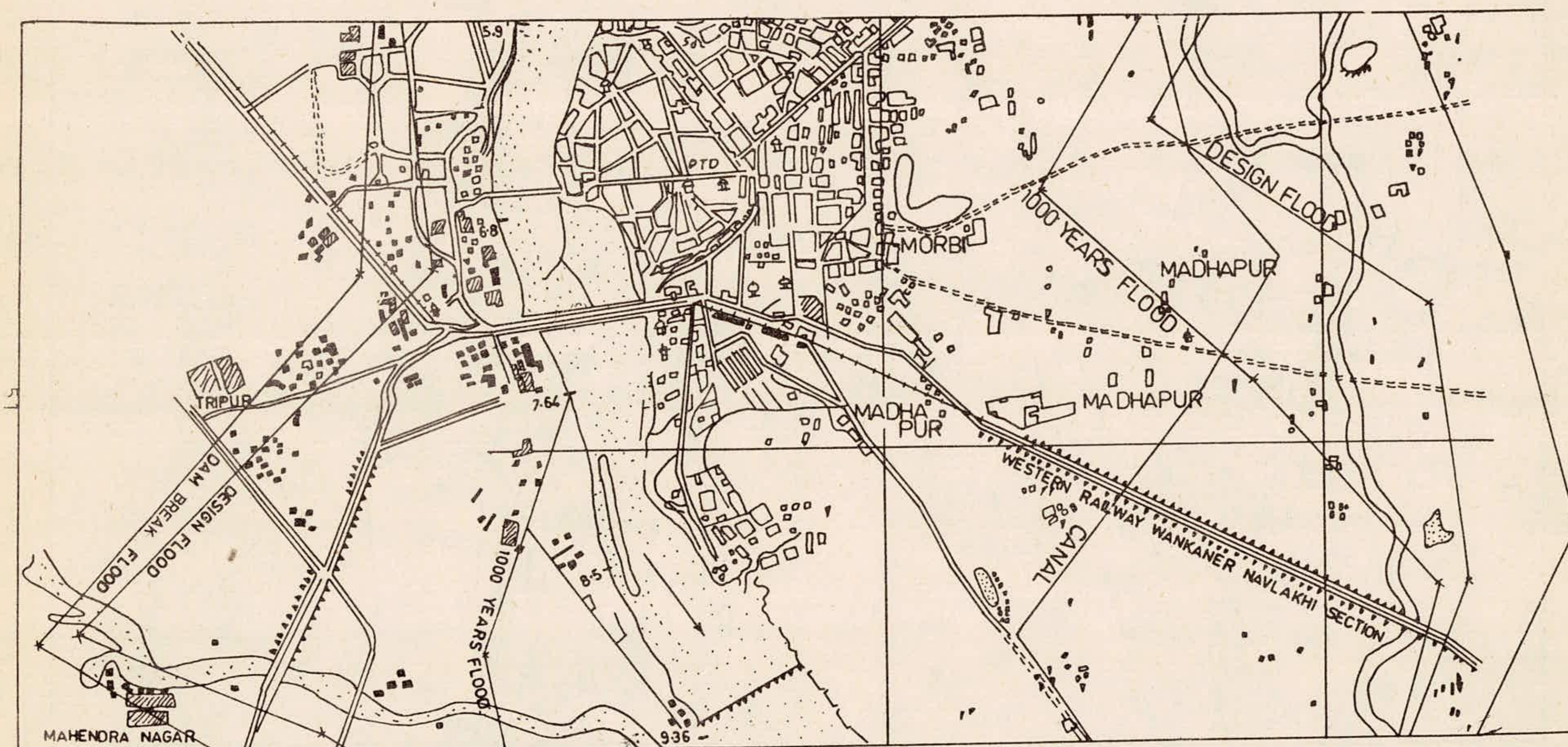

FIG. 8 - AREA INUNDATED FOR RIVER REACH 5.90 - 9.36 Km

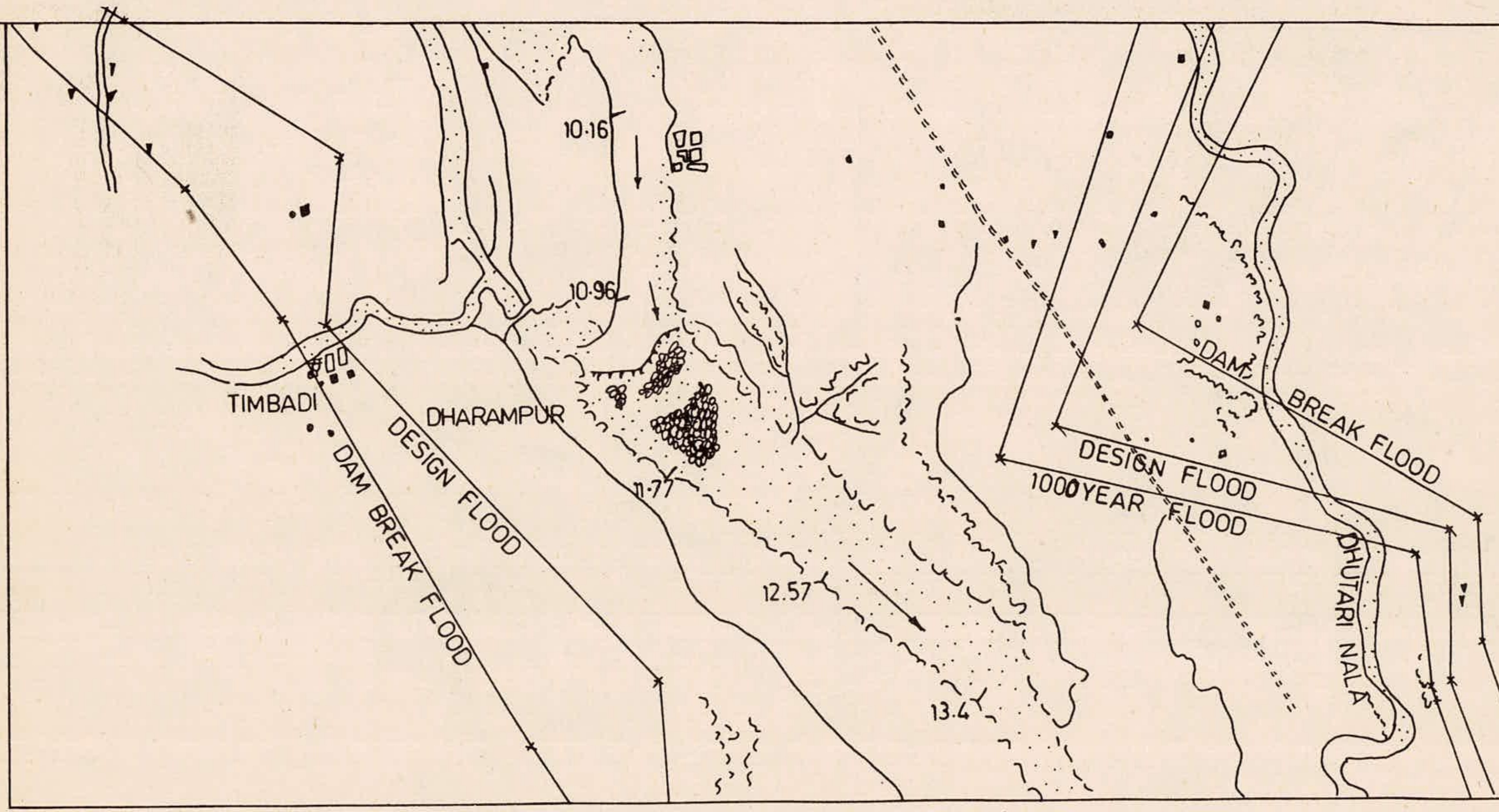

FIG. 9 - AREA INUNDATED FOR RIVER REACH 9·36 - 14·0 Km.

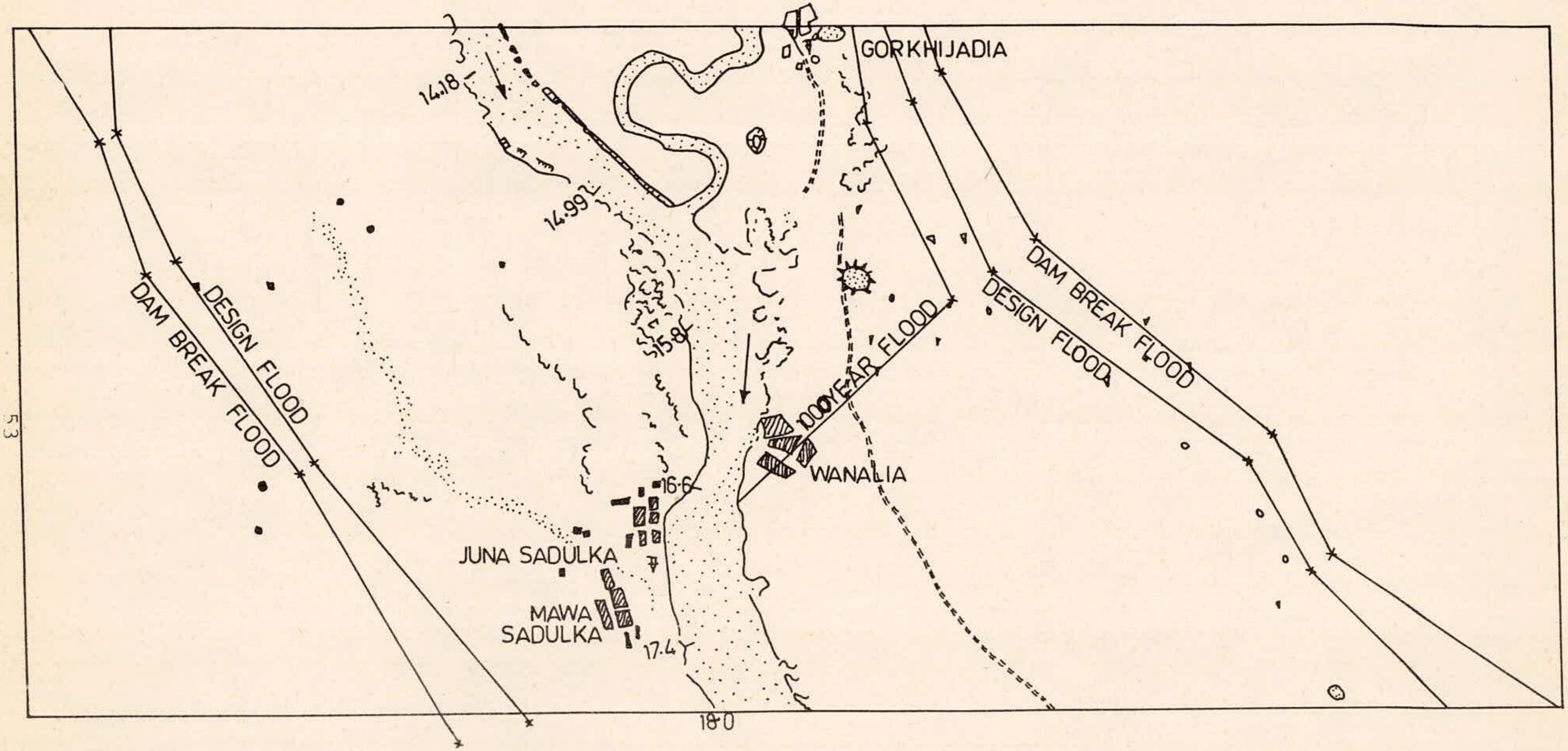

FIG. 10 - AREA INUNDATED FOR RIVER REACH 14.0 - 18.0

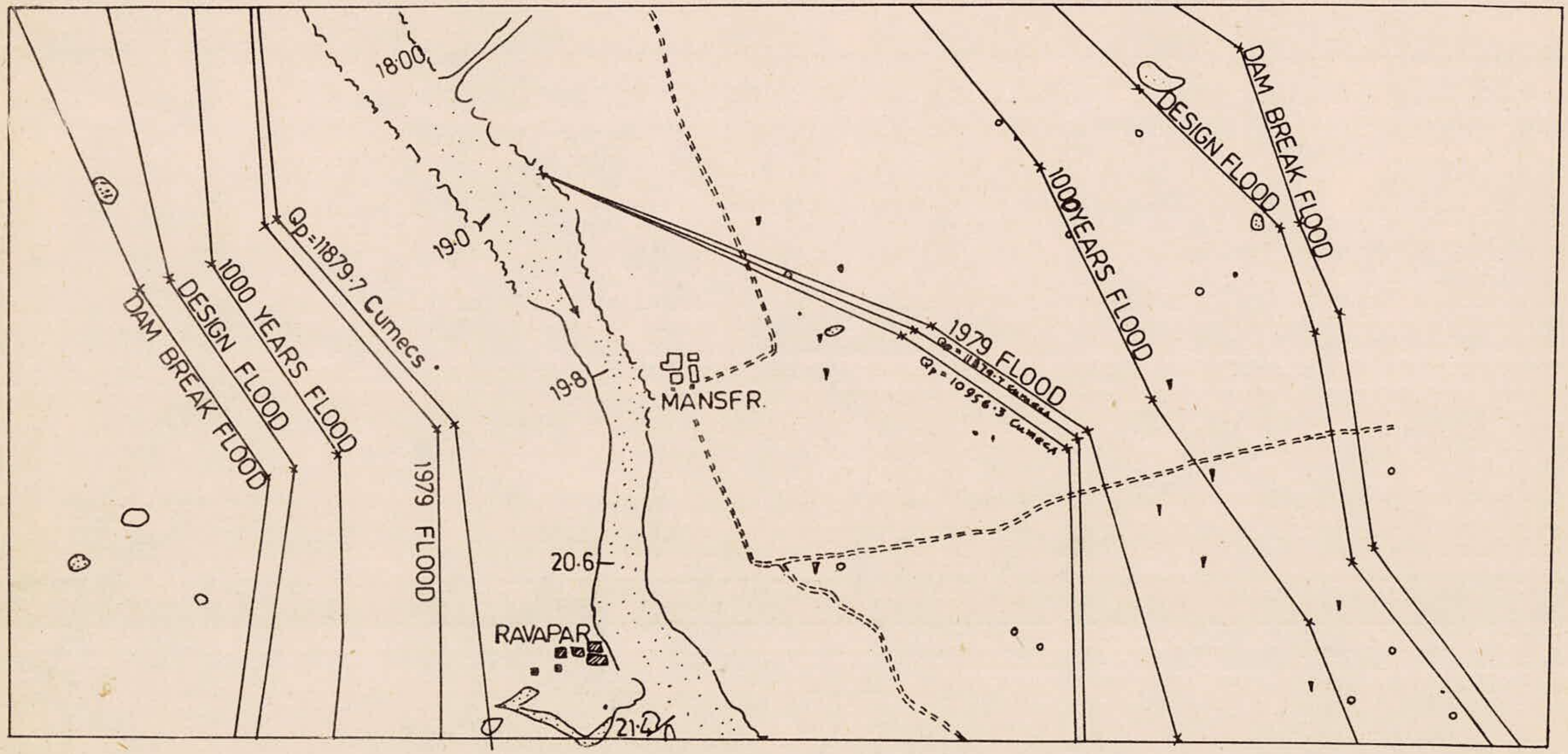

FIG.11- AREA INUNDATED FOR RIVER REACH 18·0 - 21·6 Km.

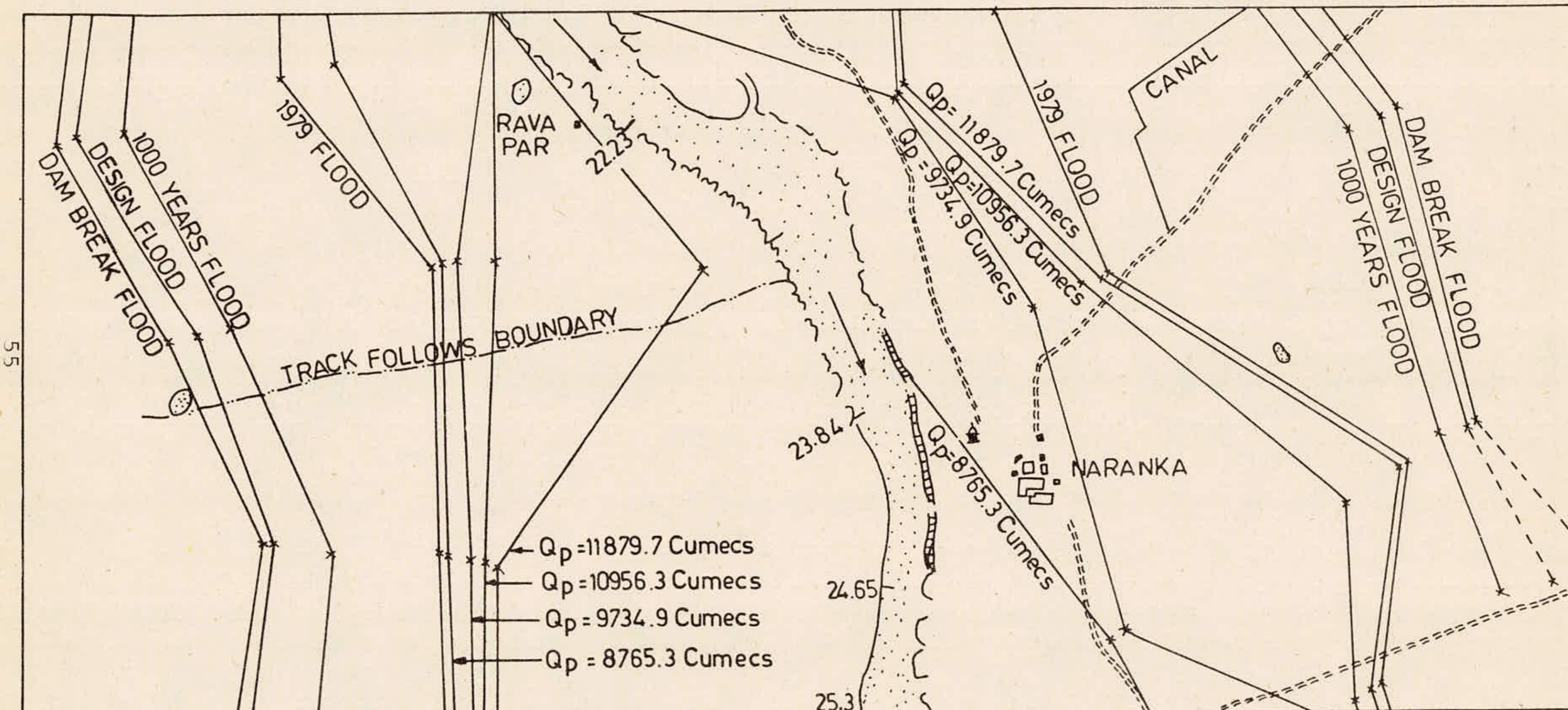

FIG. 12 - AREA INUNDATED FOR RIVER REACH 21.6 - 25.3 Km

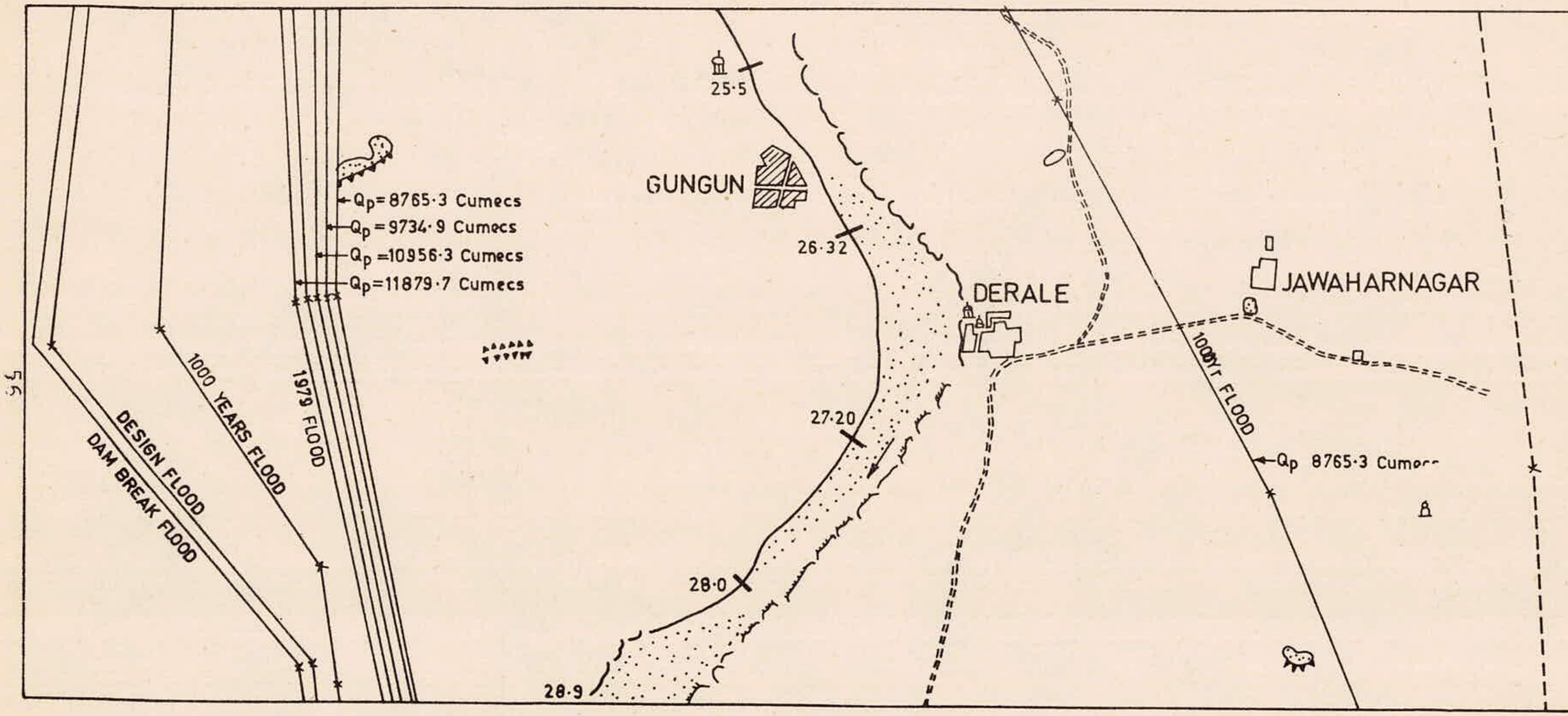

FIG.13 - AREA INUNDATED FOR RIVER REACH 25·3–28·9 Km.

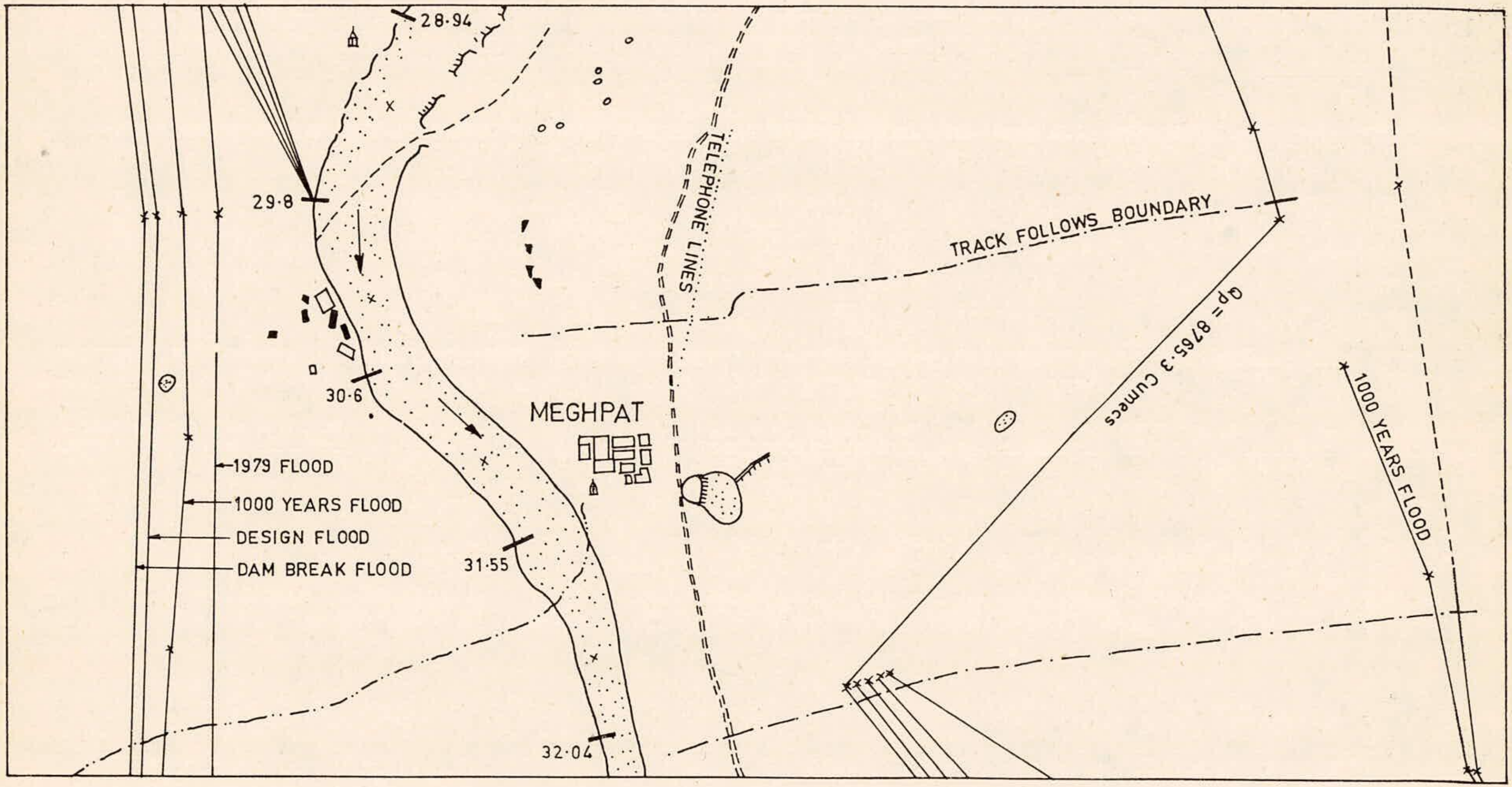

FIG. 14—AREA INUNDATED FOR RIVER REACH 28·9—32·10 Km.

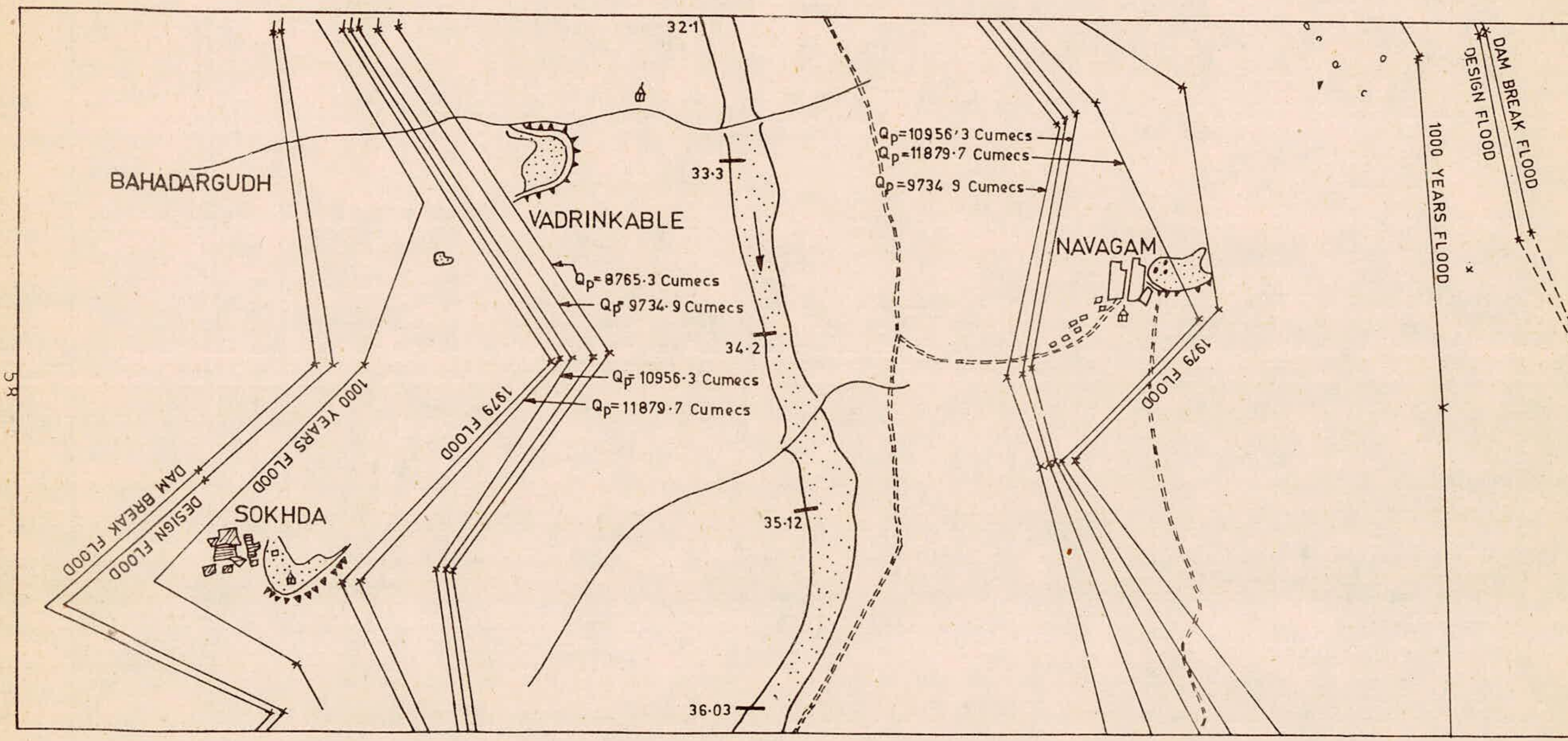

FIG. 15 - AREA INUNDATED FOR RIVER REACH 32·10 - 36·03 Km.

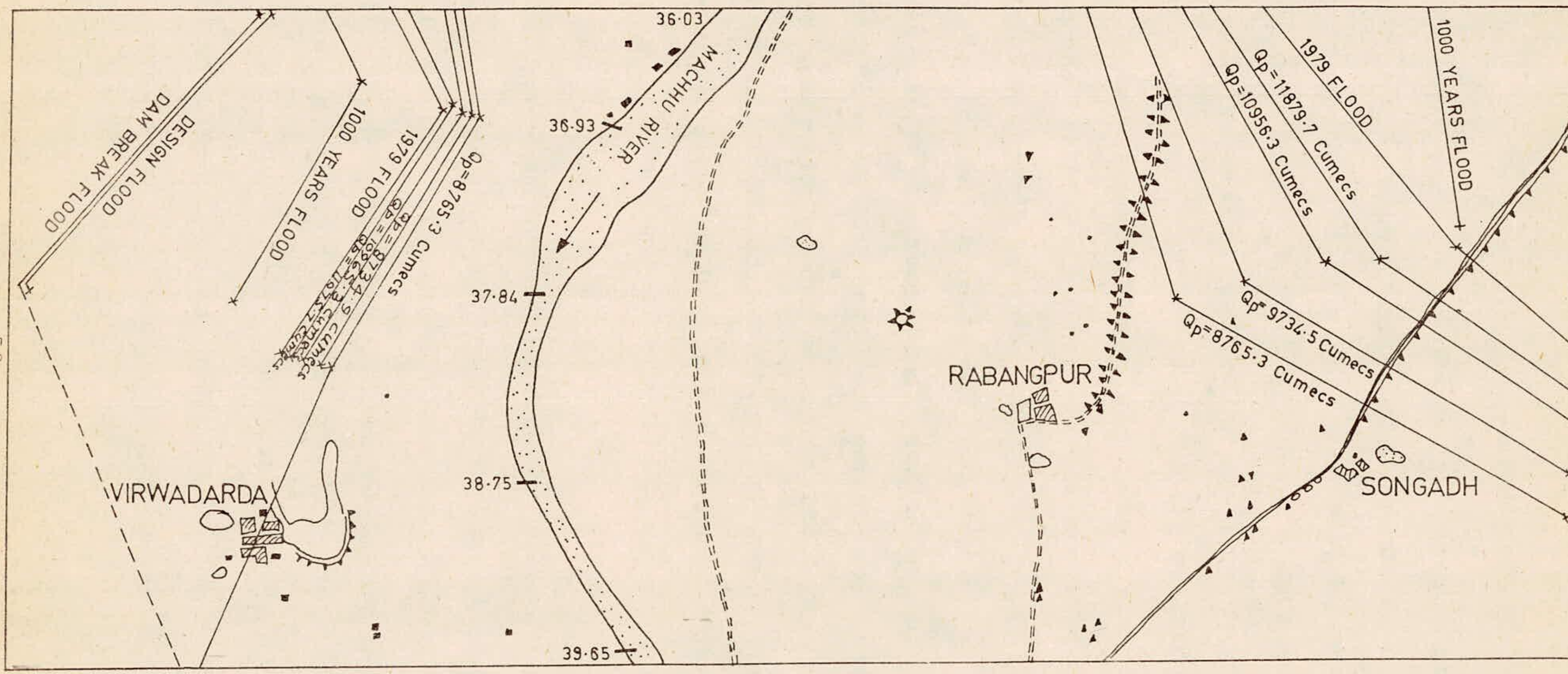

FIG. 16 – AREA INUNDATED FOR RIVER REACH 36·03 – 39·65 Km.

7.0 CONCLUDING REMARKS

This typical study emphasis on hydrological aspects of flood plain zoning. The flood events used for assessing the area affected by the different peaks of floods ranges from 8765.3 cumecs to 86886.6 cumecs also includes the dam break flood, which is the severe most flood in all possible flood events. The area affected by the different floods has been assessed assuming the boundaries of flood area as straight line and the pattern of water spreading in transverse direction of river has been assumed as at linear rate, which may not be really occur in actual flood events. Before carrying out the study or making the limit/boundary of the area affected by a flood event, a survey of the area, for which flood plain zoning is to be done, is required. Moreover, the accuracy of the results could be increased if the contour map was available containing the contour at lesser interval.

Hydrological study on these lines enable demarcation of flood inundation zones corresponding to different flood peaks. This information can be utilized for damage assessment after detailed survey of land use and property in respective zones, and also for regulation of future development.

The flood plain zoning has the short term objective of preventing more damage from flooding and in long term to reduce and even eliminate such damage. This may be considered as alternative to structural measures and more attention should be paid for use of such measures in future practices. For balanced flood control program a combination of structural measures wherever necessary to contain flood water, with non-structural measure such as flood plain zoning for limiting flood plain development and flood control is necessary.

The quantitative results are however dependent upon accuracy of available information regarding river cross sections and contours in the flood plain, and assumptions made for routing of flood wave.

REFERENCES

1. Bisaria, M.S. (1986) 'Review of flood damages in the country'. proc. workshop on flood damage assessment, Delhi.

2. Das S. and R.Lee (1988), ' A non traditional methodology for flood stage-damage calculation'. Water Resources Bulletin, Journal of AWRA, Vol. 24.

3. Davis D.W. (1979), 'Comprehensive flood plain studies using spatial data management techniques; Technical Paper no. 47. HEC, California.

4. El-Jabi N. and J. Rousselle (1983), 'Flood plain Management: Stage or discharge'. Journal of Hydrology, ASCE, Vol. 64, No. 1/3.

5. El-Jabi N. and J. Rouselle (1987), 'A flood damge model for flood plain studies'. Water resources bulletin, Journal of AWRA, Vol. 23.

6. Ford D.T. (1980), 'Interactive non-structural flood control planning'. Technical paper no. 68, HEC, California.

7. Ganga Flood Control Commission (1986) 'Classfication of Flood damages'. Proc. Workshop on flood damage assessment, New Delhi.

8. Hedge, M. (1987), 'Management of natural disasters area-flood disasters', Proc. first National Water Convention, New Delhi.

9. Johnson J.G. (1970), State Flood-Plain management activities'. Journal of hydraulics Division, Proc. of ASCE, Vol. 96, No. HY 10.

10. Mistry, J.F. (1986), 'Flood Warning disserination and forecast in Major and Flashy rivers of Gujarat State, Journal of IAH. Vol. IX, No. 4.

11. Patel, V.B. and M.U. Purohit (1986), 'Concept of flood damage', Proc. workshop on flood damage assessment,. N.Delhi.

12. Perumal, M. and Satish Chandra, (1986) 'Probability concept of flood damage assessment', Proc. Workshop on flood damage assessment, N.Delhi.

13. Perumal, M. and Satish Chandra (1987) 'Present practices and future strategies of flood plain management'. Proc. first National Water Convention, New Delhi.

14. Rangachari. R. (1986) 'Data collection and estimation of flood damages', Proc. workshop on flood damage assessment, New Delhi.

15. Rashtriya Barh Ayog Report (1980), Vol. I and II, Publication of Ministry of Energy and Irrigation, Deptt. of Irrigation.

16. Refsgaard J.C., K. Havno H.C. Ammentorp and A.Verwey (1988), 'Application of Hydrological models for flood forecasting and flood control in India and Bangladesh'. Advances in Water Resources, Vol. II, No.2.

17. Veissman W. Jr. and C. Welty (1985), 'Water Management Technology and Institutions'. A text book, Harper and Row Publication, New York.

18. Yevjevich V. (1974), 'Systemization of flood control measures. Journal of Hydraulics Division, Proc. of ASCE, Vol. 100, No. HY 11.

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