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APPLICATION OF DAM BREAK PROGRAMME MIKE 11 FOR MACHHU II DAM AND ITS COMPARISON WITH NWS DAMBRK APPLICATION RESULTS

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ABSTRACT

MIKE 11 is a software package developed by Danish Hydraulic Institute,Denmark.It is an integrated package for studies of river hydraulics having provision also for Dam break modelling.This report presents the simulation of dam break flood wave formation for Machhu II dam failure in Gujarat state using MIKE 11 and the comparision of results with earlier study of failure of this dam carried out at N.I.H using N.W.S Dam break programme by taking common data base in to consideration.The comparision of results of both cases reveals that there is a significant difference in the peak flood computation by the two models.For the same values of breach parameters,the MIKE 11 gives peak discharge of 36921 Cumecs at the dam site while the N.W.S model gave 54800 Cumecs.(the variation is nearly 32%).In the sensitivity analysis using MIKE 11 programme the final breach width and the the breach slope are found to be more sensitive than the other parameters like Breach development time,Manning's n etc.The non availability of the documentation of MIKE 11 programme is a major handicap in arriving at any definite conclusions with regard to the reasons for differences in results of two models.

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1.0 INTRODUCTION

The devastation due to a flash flood resulting from a sudden dam failure often results in loss of human life and causes extensive property damage since most of the population have concentrated in areas vulnerable to dam break disasters.Occurrence of a series of dam failures has increasingly focussed the attention of scientists on the need for developing generally applicable models so that potential hazards can be evaluated.

One of the preventive measures in avoiding dam disaster is by issuing flood warning to the people living in downstream areas when there is a failure of the dam.However it is quite difficult to conduct the analysis and determine the warning time regarding dam break flood at the time of disaster.Therefore pre determination of the warning time assuming a hypothetical dam break situation is a needed exercise in dam safety analysis.The method used for such analysis gains more credibility if one can simulate the past dam break failure scenario using that method with reference to failure mode and flood wave movement down stream of the dam.

Many mathematical models have been developed for the simulation of dam break problems out of which Danny Fried's U.S National Weather Service model and the recently developed MIKE 11 programme package of Danish hydraulic Institute have been applied for many studies.Generally the case study of dam failures using mathematical models pose various problems with regard to matching the model assumptions.The difficult problem

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is concerned with regard to the failure description of the structure as the failure which occurred in nature would be different from the failure description adopted in the mathematical model.Under these circumstances,suitable assumptions with regard to the adjustment of actual failure mode to suit the model failure mode is necessary.

The MIKE 11 program model which is used in this study is a software package developed by Danish hydraulic Institute,Denmark,which offers a unique and user friendly tool for design,management, operation of river basins and channel net works.It is capable of modelling unsteady flows in open channel systems through a numerical solution of the one dimensional Saint Venant equations.

This study focusses the application of MIKE 11 soft ware for the Machhu II dam failure in Gujarat State.It also includes the comparision of results obtained by MIKE 11 with earlier study at N.I.H using N.W.S dam break programme by taking common data base in to consideration. The report also includes sensitivity study of the dam breach parameters in order to highlight the effect of the parameters on the water levels as well as discharges resulting from dam failure at various locations in the down stream reach.

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2.0 REVIEW OF LITERATURE

The Dam break analysis problem is one of the most fascinating hydraulic problem and the concerned literature is extensive.The first solution was given in 1892 by Ritter,who used the method of characteristics to obtain a closed form solution for dam of semi-infinite extent upon a horizontal bed with zero resistance.However experimental and theoretical considerations showed that the solution is invalid in a region that starts near the leading edge of the flood wave and extends rapidly upstream with time because of zero bed resistance assumption.Dressler(1954)used a perturbation procedure to obtain a first order correction for resistance effect and Whitham obtained a second solution three years later by using a technique that was similar to the Pohlhansen method of boundary layer theory.Whitham's solution would not apply for large values of time since the width of the boundary layer grew very rapidly with time.(Perumal and Chandra,1985-86).

Investigators of dam break flood waves 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. U. S. Army Corps of Engineers (1960) have recognized the need to assume partial rather than complete branches however they assumed the breach occured instantaneously. The assumption of instantaneous and complete branches were used for reasons of convenience when applying certain mathematical techniques for analyzing the dam break

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flood waves.

DAM BREAK FLOOD ROUTING TECHNIQUES

MacDonald and Monopolis (1984) have highlighted the dependence of available computer programs for dam break analysis on certain inputs regarding the geometric and temporal characteristics of the dam breach. It is pointed out that the state of art in estimating these breach characteristics is not as advanced as the computer techniques for hydrograph development and routing of flood wave. The authors have conducted studies using data and information about case histories of dams that have failed and used the same to develop a methodology for estimating breach characteristics for certain types of dams. The breaching characteristics used as input to existing computer programs are: (i) the ultimate size of the dam breach; (ii) the shape of the dam breach; (iii) the time that is required for the breach to develop, and (iv) the reservoir water surface elevation at which breaching begins. These characteristics are dependent to a large extent on the breach formation mechanism.

The authors have classified the breach formation mechanism into two general categories:
(i) Breaches formed by a sudden removal of a portion or all of the embankment structure as a result of over stressing forces on the structure. Examination of the literature on historical failures has been mentioned regarding failure of concrete arch and gravity dams by the sudden collapse,

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overturning or sliding away of the structure due to over stresses caused by inadequate design or excessive forces that may result from overtopping of flood flows, earthquakes, and deterioration of the abutment or foundation material. In safety analysis of these types of dams the breach is assumed to develop rapidly (on the order of ten minutes) and the size and shape of the breach is taken as equal to the entire dam in the case of an arch dam or a reasonable maximum number of dam sections in the case of a gravity dam.

(ii) Breaches formed by erosion of the embankment material. The earth fill dams is by erosion of the embankment material by the flow of water either over or through the dam. Causes

that can initiate erosion type breaches include: (a) over-topping of the embankment by flood flows, and (b) seepage or piping through the embankment, foundation or abutments of the dam. In this type of dam failure, the breach size continuously grows as material is removed by outflows from storage and storm water runoff. The size, shape and time required for development of the breach is dependent on the erodibility of the embankment material and the characteristics of the flow forming the breach. Breaches of this type can occur fairly rapidly or can take several hours to develop. Moreover, the size of the breach is often significantly less than the entire dam.

All dam breaches are formed not solely by one of the above mechanisms. Some breaches are formed by a combination of the two mechanisms. MacDonald and Monoplis (1984) analyzed

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data for failure of earth fill dams in which breaches were formed by erosion of the embankment material and non-earth fill dams that may have failed partly due to erosion and partly due to sudden collapses caused by instabilities. They defined a variable called Breach Formation Factor (BFF) as the product of the outflow volume of water (V) and peak reservoir water surface and breach base (h). For both earth fill and non-earth fill embankments, it was concluded that the breach shape can be assumed to be triangular with 2V:1H side slope if the breach does not extend to the base of the embankment and trapezoidal with 2V:1H side slopes if additional material is washed away after the breach reaches the base of the embankment, and the breach shape should only be assumed if the breach size is less than embankment size. The authors also presented graphical relationships for estimating the volume of embankment material removed during a dam failure and the time for breach development. The authors also presented a graphical relationship between the BFF and the peak rate of outflow from historic dam failures.

Wurbs (1987) carried out a comparative evaluation of several leading dam breach flood wave models representative of the current state of the art after a comprehensive literature survey and application of the selected models to several case study data sets. The author discussed in detail various aspects of dam, breach flood wave simulation including : (i) breach simulation, (ii) outflow hydrograph computations, (iii) flood routing, (iv) dynamic routing, (v) generalized dynamic routing relationships, (vi) hydrologic storage

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routing, etc. While discussing the topic of flood routing, the author has explained the three dimensional nature of the flow characteristics of dam breach flood wave. "Flood plain irregularities such as abrupt contractions and expansions in valley topography, tributaries, bridges, control structures and overtopped levees cause accelerations with horizontal and vertical components perpendicular to the flow axis. Water may flow laterally outward from the river channel to fill over bank flood plain storage as the stage rises and then laterally back toward the channel as the stage falls. Three dimensional accelerations can be expected to be particularly significant near the dam breach in the case of a dam breach flood wave. Multidimensional models of unsteady flow are much more complex and difficult to apply than one dimensional models. For practical applications the current state of the art of dam breach flood wave analysis is one dimensional modelling. One dimensional flood routing models can be categorized as: (i) dynamic routing methods, (ii) generalized dynamic routing relationships, (iii) simplified hydraulic routing methods, (iv) hydrologic storage routing, and (v) purely empirical methods.

The comparative evaluation by Wurbs (1987) focussed on the following selected models, which were compared by application to four case studies including assessment of accuracy, computer requirements, documentation and maintenance, user experience, versatility, robustness, theoretical and observed accuracy, etc.

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NATIONAL WEATHER SERVICE (NWS) DAM BREACH FLOOD FORECASTING MODEL (DAMBRK)

It is a dynamic routing model. It contains breach simulation routines in which the breach begins at the top of dam and grows uniformly downward and outward. A breach of a fixed shape is initiated when the reservoir water surface reaches a given elevation and then breach dimensions grow linearly with time. The water surface elevation at which failure begins and the breach formation time are to be provided as input data. A trapezoidal, rectangular or triangular shaped breach is specified by inputting the breach side slopes and terminal breach bottom width and elevation. It also has an additional option for simulating a piping failure. A rectangular breach grows outward from a point linearly over the time to failure. A breach erosion model for use in conjunction with DAMBRK has also been developed.

Discharges through spillway and outlet works structures and dam breaches are computed as a function of reservoir water surface elevation using empirical weir and orifice equations. This model also have provision to consider tail water submergence effects in the outflow hydrograph computations. Reservoir routing is accomplished by either hydrologic or dynamic routing methods. Hydrologic routing is based on an assumed level water surface, the reservoir geometry is described by a storage versus elevation relationship and it is applicable for wide, flat reservoirs with gradual change in water surface levels. Dynamic routing methods can handle the negative waves than may be caused by sudden reservoir drawn

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downs and positive waves produced by large reservoir inflows. Water surface profiles through an upstream of a reservoir can be developed as well as the outflow hydrograph. In dynamic routing, the reservoir geometry is described by cross sections and Manning roughness coefficient, as is the downstream valley. Dynamic routing methods are most advantageous for long, narrow reservoirs with rapid water level changes at the breached dam, and are more accurate when the slope of the reservoir water surface is significant. In many typical situations for which the basic assumptions are valid, hydrologic reservoir routing is generally easier to use than dynamic routing and is essentially as accurate.

DAMBRK Model uses one dimensional dynamic method of flood routing, based on four point implicit solution of St. Venant equations for conservation of mass and conservation of momentum.

U.S. ARMY CORPS OF ENGINEERS HYDROLOGIC ENGINEERING CENTRE (HEC) FLOOD HYDROGRAPH PACKAGE (HEC1)

This model simulates the precipitation-runoff process and routes flood hydrographs using hydrologic methods. For dam breach simulation it has a routine similar to DAMBRK model, except that the breach does not expand laterally. Discharges through spillway, outlet works and dam breaches are computed as a function of reservoir water surface elevation. The model, however, does not reflect tail water submergence effects on in the outflow computations. Flood routing is accomplished by hydrologic storage routing methods.

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U.S. ARMY CORPS OF ENGINEERS SOUTH-WESTERN DIVISION (SWD) FLOW SIMULATION MODELS (FLOW SIM1 AND FLOW SIM 2)

Both these models are dynamic routing models and are identical except FLOW SIM1 uses an explicit and FLOW SIM2 an implicit finite difference solution scheme. There breach simulation routine is similar to DAMBRK model. They also contain an optional routine in which the rate of growth of a trapezoidal breach is computed using the Schoklitsch bed load formula Discharges through spillway, outlet works and dam breaches are computed as a function of reservoir water surface elevation. Both these models also include tail water submergence effects in the outflow computations. Both these models are dynamic routing models.

SOIL CONSERVATION SERVICE (SCS) SIMPLIFIED DAM BREACH ROUTING PROCEDURE (TR66)

This provides a simplified relationship based on data from past dam failures, in which peak discharge from a breach is dependent solely on the depth of the reservoir. The procedure assumes an instantaneous breach with the maximum discharge occurring at time zero. Only peak discharge, not the entire hydrograph is computed. It uses simplified attenuation kinematic model for flood routing.

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NWS SIMPLIFIED DAM BREACH FLOOD FORECASTING MODEL (SMPDBK)

This model assumes a time dependent rectangular breach. Discharges through spillway, outlet works and dam breaches are computed on same lines as DAMBRK model. It uses generalized dynamic routing relationship for flood routing, which involves use of a family of dimensionless curves that have been developed using a dynamic routing model. The assumption of prismatic channel of specified shape is one of the major simplifications made to develop set of dimensionless curves.

HEC DIMENSIONLESS GRAPHS PROCEDURE

This model also uses generalized dynamic routing relationship and involves use of a family of dimensionless curves for flood routing.

Wurbs (1987), while concluding observes that the various dam breach flood wave models provide a wide range of trade offs between accuracy and ease of use, and in general the modelling is still not highly accurate. The author recommends that a dynamic routing model should be used whenever a maximum practical level of accuracy is required and adequate manpower, time and computer resources are available. The case study analyses confirmed that the dynamic routing models are the most versatile and accurate of the models tested. The National Weather Service (NWS) Dam Break Flood Forecasting Model (DAMBRK) is recommended by the author as the optimal choice of model for most practical applications and the NWS Simplified Dam Break Flood Forecasting Model (SMPDBK) for performing analysis as expeditiously as possible.

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3.0 MIKE 11 PROGRAMME

3.1 INTRODUCTION

MIKE 11 is a software package developed at the Danish Hydraulic Institute (DHI) for the simulation of flows, sediment transport and water quality in estuaries, rivers, irrigation systems and similar water bodies. MIKE 11 is developed especially for application of micro-computers and is based on DHI's well known SYSTEM 11.MIKE 11 offers a unique and user friendly tool for design, management and operation of river basins and channel networks.It is an integrated software package which enables the user to simulate catchment runoff, river flow, sediment and pollutants transport. MIKE 11 consists of a number of modules which in principle operate independently. This gives a rational and user-friendly execution and enhances the flexibility of the package.

The core of the MIKE 11 system consists of the hydrodynamic (HD) module, which is capable of simulating unsteady flows in a net work of open channel system.The results of the simulation consist of time series of water levels and discharges. Associated with the HD module, is the rainfall - runoff model NAM, which may be used to generate inflows to the HD module. Transport dispersion (TD) and sediment transport (ST) calculations may be carried out from special modules which utilize the results of a HD computation.

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The interface between the HD and NAM modules and the rest of MIKE 11 is shown in Fig. A.1. This figure gives an indication of a typical way through MIKE 11.

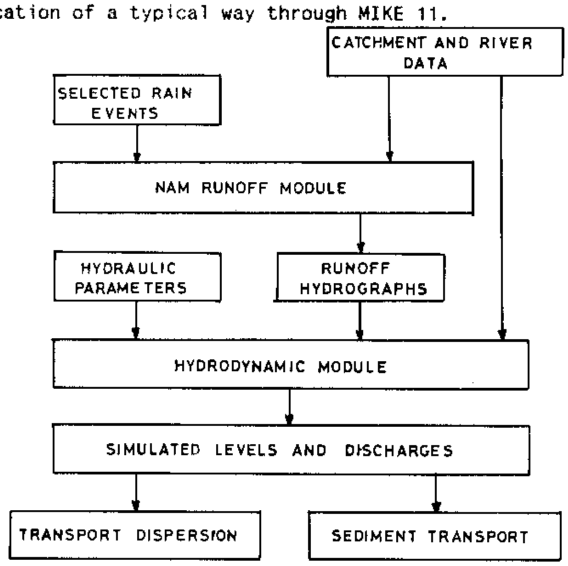

FIG. 1– A TYPICAL MIKE 11 SIMULATION

The model provides a choice between 3 different flow descriptions, viz:

o Kinematic wave approach: The flow is calculated from the assumptions of balance between the friction and gravity forces. This simplification implies that the kinematic wave approach cannot simulate backwater effects.

o Diffusive wave approach: In addition to the friction and gravity forces, the hydrostatic gradient is included in this description. This allows consideration of the downstream boundaries and thus simulate backwater effects.

o Dynamic wave approach: Using the full momentum equation, including acceleration forces, the user is able to simulate fast transients, tidal flows,etc.in the system,depending on

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the requirement.

3.2 Data requirements

The input data requirements for the MIKE 11 program are flexible.When a detailed analysis is not feasible due to lack of data or insufficient data preparation time,the unknown or unavailable data can be ignored.The input data pertains to MIKE 11 includes the data related to the dam,breach characteristics,reservoir and the boundary conditions at the upstream and down stream conditions.In addition to the above initial conditions are to be specified in MIKE 11 model.

The input data can be basically classified into two groups.

1.Data group pertaining to the dam:

a) Reservoir Data : Inflow hydrograph, length of the reservoir, The surface area/elevation curve of the reservoir, Top of the dam, Length of the earthen dam, top width of the dam, Grain diameter, relative density, porosity, critical shear stress.

b) Breach data : Start breach level, Final breach level, Start breach width, Final breach width, Breach slope and the Breach development time.

c) Spillway data : Spillway rating curve, elevation of the uncontrolled spillway crest, Length of the spillway.

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2.Data group pertains to the downstream routing reach:

a)Cross section details:chainages of the cross sect s in kms.from the dam,a table of top widths(active and inactive)and the corresponding elevations at each sections,hydraulic resistance coefficients(Manning's n,expansion / contraction coefficients, slope coefficients,slope of the down stream channel for the first km below the dam,and the initial conditions in the downstream channel.The typical way of giving data in the MIKE 11 is that it should in the form of raw data(X-Z co-ordinates).The MIKE 11will automatically computes the processed data and utilizes that data in the computation phase.(The data files are enclosed in the Appendix I)

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3.3 METHODOLOGY

This chapter presents a brief description of the methodology for running Dam break modelling using MIKE 11 soft ware.The points to be noted in modelling dam breaks on Mike 11 are given below.

3.31 General:

Most dam break set ups consist of a single or several channels, a reservoir, the dam structure and perhaps auxiliary dam structures such as spillways, bottom outlets, etc. Further downstream, the river may be crossed by bridges, culverts, etc. It is important to describe the river setup accurately in order to obtain reasonable results.

3.32 River Channel Setup

Setting up the river channel description in the cross-section database is the same for dam break models as it is for other types of models. However, due to the highly unsteady nature of dam break flood propagation, it is advisable that the river course be described as accurately as possible through the use of as many cross sections as necessary, particularly where the cross section is changing rapidly. Another consideration is that the cross-sections themselves should extend as far as the highest modelled water level, which will normally be in excess of the highest recorded flood level. If the modelled water level exceeds the

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highest level in the cross-section data base for a particular location, MIKE 11 will extrapolate the processed data, i.e. A(h), R(h), b(h), etc.

3.4 Reservoir description and the Appurtenant structures.

3.4.1 Reservoir

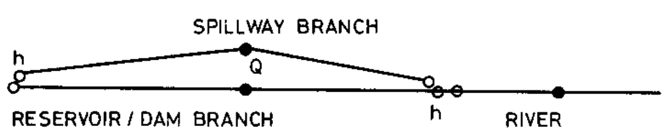
In order to obtain an accurate description of the reservoir storage characteristics,the reservoir is normally modelled as a single h-point in the model.This point will usually correspond also to the upstream boundary of the model where inflow hydrographs may be specified.

3.4.2 Dam

The dam break structure itself can be located as a seperate branch.At this point the momentum equation is replaced by an equation which describes the flow through the structure which may be either critical or sub critical.

3.4.3 Spillways and other structures

If the spill way is added to the dam itself it should be described as a seperate branch as shown below:



3.4.4 Boundary conditions.

Boundary conditions must be specified at both upstream and down stream limits of the model.The upstream conditions

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will generally be an inflow into the reservoir.At the down stream conditions the river rating curve at the end chainage should be given.

3.4.5 Initial conditions.

In many cases dam failures may occur on a dry river bed down stream.However such initial conditions are not possible in MIKE 11 which require a finite depth of water to be present through out the model in order to ensure the connectivity of the finite difference algorithm.

The nodel set up adopted in the Machhu II dam failure is shown in the figure.

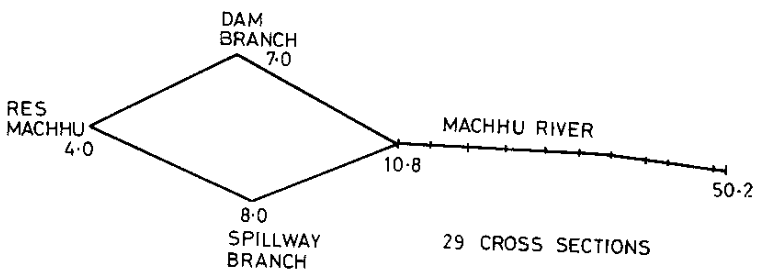

FIG. 2-ADOPTED NODEL SET UP IN THE MACHHU II DAM FAILURE

The dam break modelling of Machhu II dam using MIKE 11 package consists of two parts.The first part includes the creation of hot start file.The second part includes the subsequent simulations carried out using hot start file as initial condition.

This study also includes the sensitivity analysis of dam breach parameters to investigate the effect of the parameters on the water levels and discharges in the down stream end.

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4.0 RESULTS AND DISCUSSIONS

4.0 GENERAL

In this chapter, the results of two Dam break simulations carried out by the application of MIKE 11 Model to the Machhu II dam failure have been discussed.The two cases of studies are:CASE(1) Using the data/parameters supplied in C.W.C training,CASE(2) Using data/parameters on same lines as those used in earlier study at N.I.H with N.W.S Dam break programme.The adopted Dam breach parameters in MIKE 11 supplied by C.W.C are Start breach level(60m),Final breach level(37.6m),Final breach width(150m), breach development time (2hours),breach slope (0.075m) and Manning's n is 0.033. In contrast to this the adopted breach parameters in N.W.S study were Start breach level- (60m),Final breach level(39.6m),Final breach width(360m),breach- development time(1hour),breach slope(0.027m) and Manning's n is 0.033.It also includes the results of sensitivity analysis carried out by using MIKE 11 to investigate the effect of dam breach parameters on water levels and discharge.

4.1 DATA REQUIREMENTS FOR N.W.S MODEL

The DAMBRK model was developed so as to require data that was accessible to the forecaster. The input data requirements are flexible in ,so far as much of the data may be ignored.The input data can be categorized into two groups.

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(i) The first data group pertains to the dam(The breach,spillways and reservoir storage volume).The breach data consists of the following parameters:(failure time of breach in hours) b(final bottom width of breach),Z(side slope of the breach).hbm (final elevation of breach bottom),ho(initial elevation of water in the reservoir),hd(elevation of dam).The spillway data consists of the following:hs(elevation of uncontrolled spillway crest),Cs(Coefficient of discharge of uncontrolled spillway),hg (elevation of centre of submerged gated spillway),Cd(coefficient of discharge of crest of the Dam), Qt(constant head independent discharge from the Dam).The storage parameters consists of the following: a table of surface area(As) in acres or volume in Acre ft, and the corresponding elevation with in the reservoir.

(ii)The second group pertains to the routing of the outflow hydrograph through the downstream valley.This consists of description of cross sections,hydraulic resistance coefficients and expansion coefficients.The cross sections are specified by location maps and tables of top width(active and inactive) and corresponding elevations.

4.11 DATA REQUIREMENTS FOR MIKE 11 DAMBREAK MODEL.

The input data pertains to the MIKE 11 includes the data related to the dam,breach,reservoir and the boundary conditions at the upstream and down stream conditions. In addition to the above initial conditions are to be specified in MIKE 11 programme.

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The dam data which includes the breach are top of the dam(m),length of the dam(m),top width of the dam(m),start breach level(m),final breach level(m), Start breach width(m),Final breach width(m) and breach slope.the spillway data consists of the length of the spillway(m),and the spillway rating curve of the Machhu dam. the storage parameters consists of a table of surface area in m^2 and the corresponding elevation in the reservoir.boundary conditions at the upstream and downstream end of the rivers in the form of river rating curve(Q vs H)and the cross sectional data along the river chainage.The typical way of giving data in MIKE 11 is that it should be in the form of raw data(X-Z co-ordinates).MIKE 11 automatically computes the processed data and utilizes that data in the computation phase.(The data files are enclosed in the Appendix I).

4.12 COMPARISION

A comparision of data requirements for both the models reveals that there is no significant difference between the two except that the representation of input data in MIKE 11 differs from N.W.S model.A nodel set up representing the reservoir,spillway and dam branches along with many cross sections to be made in MIKE 11 set up and the data is to be fed accordingly.Boundary conditions must be specified at both up stream and down stream limits of the MIKE 11 model set up.

In many cases dam failures may occur on a dry river bed down stream.However such initial conditions are not possible in MIKE 11which requires a finite depth of water to be present

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through out the entire model in order to ensure the connectivity of the finite difference algorithm.

4.2 DATA PREPARATION FOR MIKE 11

Case 1:Using Machhu II dam failure parameters adopted for C.W.C Training.

The accuracy with which the Dam break simulations can be made depends on the fidelity with which breach parameters, flow,geometry and roughness are represented.Two types of breaching may be simulated using this programme.

1.An over topping failure in which the breach is simulated as a rectangular,triangular or trapezoidal shapped opening that grows progressively down word from the dam crest with time.

2. A Piping failure in which the breach is simulated as a rectangular orifice that grows with time and is centered at any specified elevation with in the Dam.Representation of data of spillways and reservoir storage volume are straight forward and are readily available in practice. Flow geometry and storage are represented by means of user specified in terms of river Kms.from the Dam.(The data files are enclosed in the Appendix I)

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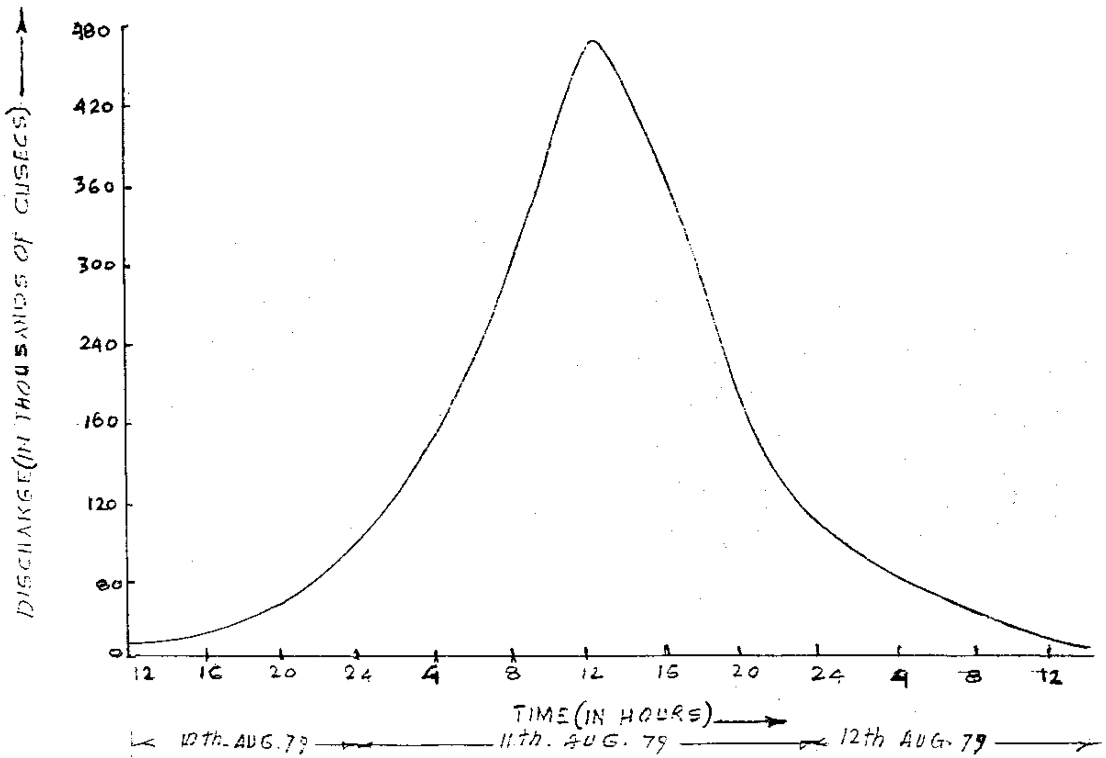

FIG 2 – ADOPTED RESERVOIR INFLOW HYDROGRAPH FOR DAM BREAK ANALYSIS OF MACHHU DAM-II.

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CASE 2 USING MACHHU II DAM FAILURE PARAMETERS OF N.W.S STUDY AT N.I.H

All the data related to case1 are included in this case except a slight difference in the Dam breach data viz .breach development time,final breach width and breach slope.(1Hour,300m and 0.027m)respectively.

4.3 MODEL APPLICATION RESULTS.

4.31 Case 1:

For the adopted breach parameters the dam breach started at 9 Hours 15 minutes of 8.07.1979.The peak discharge through the breach was 22153 m^3/sec at 11 Hours 18 minutes.Table 1 shows the maximum water levels in the river channel for some typical cross sections.The cross sections were adopted are Reservoir Machhu at4.0km chainage and along the river at chainages 10.8,14.8,17.040,24.1,32.1,43.9 and 50.2.It has been observed that along the river cross sections the water levels goes on decreasing.At reservoir Machhu 4.0,the water level was maximum i.e 60.54m At the downstream end of the river i.e Machhu 50.2 km the water level was found to be decreasing.(10.88m).However the the time taken to reach the maximum water level goes on increasing.At reservoir Machhu 4.0 the time taken to reach the peak level was 9 hours 15 minutes as the time taken to reach the peak level at the downstream end of the river was found to be 16 hours 18 minutes.

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Case 2:

For the adopted breach parameters,the dam breac. started at 7hours 50 minutes of 8.07.1979.The peak discharge through the breach was found to be 36921m^3/sec at 13 hours 54minutes. Table 2 shows the maximum water levels in the river channel for some typical cross sections.The cross sections adopted were same as in Case 1.It has been observed that along the river cross sections the water level goes on decreasing.At the reservoir Machhu 4.0m the water level is found to be maximum.i.e 60.26m.Similarly at the downstream end of the river,Machhu 50.2 river,Machhu 50.2 km the maximum water level was found to be 11.08m.However,the time taken to reach the desired peaks goes on decreasing along the river chainages.At the upstream and downstream end of the set up it was found to be 8hours 36minutes and 14hours 14minutes respectively.

4.4 SENSITIVITY ANALYSIS OF BREACH PARAMETERS.

4.41 CASE 1-SENSITIVITY STUDY

Sensitivity analysis was carried out for the breach parameters such as breach development time,breach width,Manning's n and breach slope to investigate the effect of change in values of each parameters on water levels,peak discharge and travel time of peak flow through the river.

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(A) BREACH DEVELOPMENT TIME.

The maximum water level at different chainages on river Machhu and also in the reservoir along with their time of occurrence are given in table 1. The maximum water levels at different chainages and the travel time to peak flow along the longitudinal profile of the river for different breach development times are plotted in in fig 4.1.

It is seen from the Table 4.1 that the the maximum discharge through the breach for different breach development times is nearly same As the breach development time increases there is corresponding increase in the value of the peak discharge through the breach. Also the maximum water levels at different chainages along the river increases slightly with increase in the breach development time from 1to 3Hours.The water levels at chainage 24.1 is more than the height of the embankment.Similarly the river beyond chainage 40Km is more than the height of the embankment for all the breach development times.This indicates that at these locations,the river will over flow the banks.It is clear from the Fig.4 that the travel time of peak flow since the start of the breach at any chainage is almost proportional to the breach development time.The time taken by the peak flow to cross the chainage 50.2 are 6hours 21 minutes,7 hours 36 minutes and 7 hours 42 minutes for breach development times of 1hour,2hours and 3 hours respectivaly.

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(B)FINAL BREACH WIDTH.

It is observed from the table 4.1 that the breach width has a significant effect on maximum water levels as well as peak discharge through through the breach.fig 5 shows the maximum water levels at different chainages for different final breach widths.It is clear from the fig 5 that with the increase in the value of final breach width from 50 t0 300m,there is an increase in the water levels along the river chainages and the peak discharge is found to increase from 12623 Cumecs to 35402 Cumecs.The travel times of peak flow are found to decrease i.e the peak comes earlier.

(C) MANNING'S ROUGHNESS CO-EFFICIENT(n)

The increase in the value of Manning's n has no significant effect on the peak discharge through the breach but it has some effect on the on water levels.With the increase in the roughness co efficient the water levels was found to increase and also the time of peak increased.

(D)BREACH SLOPE

With the increase in the breach slope,there is no significant effect on water levels,but the peak discharge is found to increase slightly(Table 4.1).It is clear from the table 4 that by doubling the breach slope from 0.075to0.150,the peak discharge is found to increase from 21153 Cumecs to 2250C Cumecs.Fig shows the trend of water

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levels at different chainages for different breach slopes.and the travel time of peak flow .

It is seen from the fig that the travel time of peak flow at different river cross sections are almost same with increase in breach slope except at the chainage 50.2 it is found to be slightly less.

4.46 CASE 2:SENSITIVITY STUDY

The results of sensitivity analysis of breach parameters show almost same trends and characteristics as in Case 1.Plots in figs. indicate the main findings of the analysis.

It is thus clear from the sensitivity study of both the cases that,out of all breach parameters,Final breach width and breach slope are sensitive with regard to peak discharge as well as travel time for the peak.By doubling the breach width from 150 to 300m in case 1 the peak discharge is found to increase from 22153Cumecs to 35402 Cumecs.At the same time the travel time is found to decrease from 7 hours 3minutes to 5 hours 45 minutes.Similarly by doubling the breach slope from 0.075 to 0.150 the peak discharge is found to increase from 22153 Cumecs to 22500 Cumecs.

4.5 MODEL APPLICATION RESULTS OF N.W.S STUDY (Taken from N.I.H Report CS-16)

The hydrograph from the breached dam beginning from 1.30Pm on 11.08.1979 and the routed hydrographs at cross sections 1.3,9.4,17.4,25.4,and 33.3 Kms

TABLE 1 : SENSIVITY ANALYSIS OF BREACH PARAMETERS (CASE I)

PARAMETERS	RIVER CROSS SECTIONS																Peak discharge through breach (Cumecs)	
	RES.MACHHU 4.00		10.8		14.8		17.040		24.1		32.1		43.9		50.2			
	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	Dis-charge	Time (h/m)
I. Breach development Time																		
i) 1 hour	60.33	8.45	52.22	10.42	45.52	10.59	41.32	11.06	31.23	11.54	25.41	12.48	15.78	14.86	10.85	15.36	21781	10.33
ii) 2 hours	60.54	9.15	52.27	11.21	45.57	11.42	41.41	11.51	31.32	12.39	25.47	13.47	15.80	15.15	10.88	16.18	22153	11.18
iii) 3 hours	60.72	9.48	52.29	12.06	45.60	12.36	41.46	12.42	31.36	13.21	25.50	14.12	15.80	15.51	10.89	16.57	22328	12.06
II. Breach Width																		
i) 50 m	60.24	9.48	50.70	12.36	44.14	12.45	38.85	13.15	29.02	13.54	23.75	14.39	15.11	16.12	10.21	17.21	12623	12.39
ii) 150 m	60.54	9.15	52.27	11.21	45.57	11.42	41.41	11.51	31.32	12.39	25.47	13.57	15.80	15.15	10.89	16.18	22153	11.18
iii) 300 m	60.74	9.00	53.63	10.48	47.05	11.09	43.41	11.12	22.85	11.48	26.27	12.33	16.06	14.12	11.10	15.00	35402	10.09
III. Mannings (n)																		
i) 0.017	60.54	9.15	52.26	11.20	45.57	11.40	41.41	11.50	31.30	12.35	25.42	13.56	15.80	15.13	10.85	16.12	22150	11.10
ii) 0.033	60.54	9.15	52.27	11.21	45.57	11.42	41.41	11.51	31.32	12.39	25.47	13.57	15.80	15.13	10.88	16.18	22153	11.19
iii) 0.050	60.54	9.15	52.26	11.21	45.57	11.48	41.40	11.51	31.36	12.30	25.47	13.36	15.80	15.21	10.87	16.09	22343	11.39
IV. Breach Slope																		
i) 0.037	60.54	9.15	52.27	11.20	45.52	11.40	41.40	11.50	31.30	12.30	25.47	13.50	15.75	15.12	16.71	16.12	22112	11.09
ii) 0.075	60.54	9.15	52.27	11.21	45.57	11.42	41.41	11.51	31.32	12.39	25.47	13.57	15.80	15.15	10.88	16.18	22153	11.18
iii) 0.150	60.54	9.15	52.28	11.21	45.58	11.42	41.43	11.51	31.34	12.42	25.48	13.30	15.80	15.09	10.88	16.12	22500	11.39

Dam breach started at 9.15 min of 8.07.1979

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TABLE 2 : SENSIVITY ANALYSIS OF BREACH PARAMETERS (CASE II)

PARAMETERS	RIVER CROSS SECTIONS																Peak discharge through breach (Cumecs)	
	RES.MACHHU 4.00		10.8		14.8		17.040		24.1		32.1		43.9		50.2			
	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	W.L. (m)	Time (h/m)	Dis-charge	Time (h/m)
I.Breach development Time																		
i) 0.5hour	60.16	8.21	53.85	9.27	47.30	9.48	43.66	9.51	32.90	10.33	26.23	11.21	16.04	13.30	11.06	13.57	38010	10.33
ii) 1 hour	60.42	8.36	53.91	9.51	47.39	10.12	43.78	10.15	32.99	10.51	26.29	11.39	16.07	13.24	11.08	14.14	36921	13.54
iii) 2 hour	60.42	8.57	53.94	10.36	47.45	10.57	43.84	11.00	33.08	11.36	26.37	12.27	16.11	14.06	11.12	14.57	34368	16.06
II. Breach Width																		
i) 150 m	60.20	8.27	52.22	10.09	45.80	10.27	41.90	10.36	31.61	11.21	25.64	12.12	15.96	14.03	10.92	14.51	24217	12.39
ii) 300 m	60.26	8.36	53.91	9.51	47.39	10.12	43.78	10.15	32.99	10.51	26.21	11.39	16.07	13.24	11.02	14.14	36921	13.54
iii) 600 m	60.21	8.27	54.91	9.248	48.37	9.42	44.31	9.42	33.36	10.24	26.40	11.15	16.09	13.00	11.09	13.54	73440	16.09
III. Breach slope																		
i) 0.013	60.22	9.15	52.26	11.20	45.57	11.40	41.41	11.50	31.30	12.35	25.42	13.56	15.80	15.13	10.65	16.12	30145	16.10
ii) 0.027	60.26	8.36	53.91	9.51	47.39	10.12	43.78	10.15	32.99	10.51	26.29	11.39	16.07	13.24	11.08	14.14	36921	13.54
iii) 0.054	60.54	7.15	52.29	11.21	45.57	11.48	41.40	11.51	31.36	12.30	25.47	13.36	15.80	15.21	10.87	16.09	42343	11.39
IV. Manning's (n)																		
i) 0.037	60.26	8.36	53.91	9.51	47.39	10.12	43.79	11.50	32.09	10.59	26.29	11.39	16.07	13.24	11.06	14.12	35340	11.09
ii) 0.075	60.26	8.36	53.91	9.51	47.39	10.12	43.78	10.15	32.99	10.51	26.29	11.39	16.07	13.24	11.09	14.14	36921	13.54
iii) 0.150	60.26	8.36	53.318	9.51	47.39	10.12	43.78	10.15	32.99	10.51	26.29	11.39	16.07	13.24	11.03	14.14	36921	15.48

Dam breach started at 7.50 min of 3.07.1979

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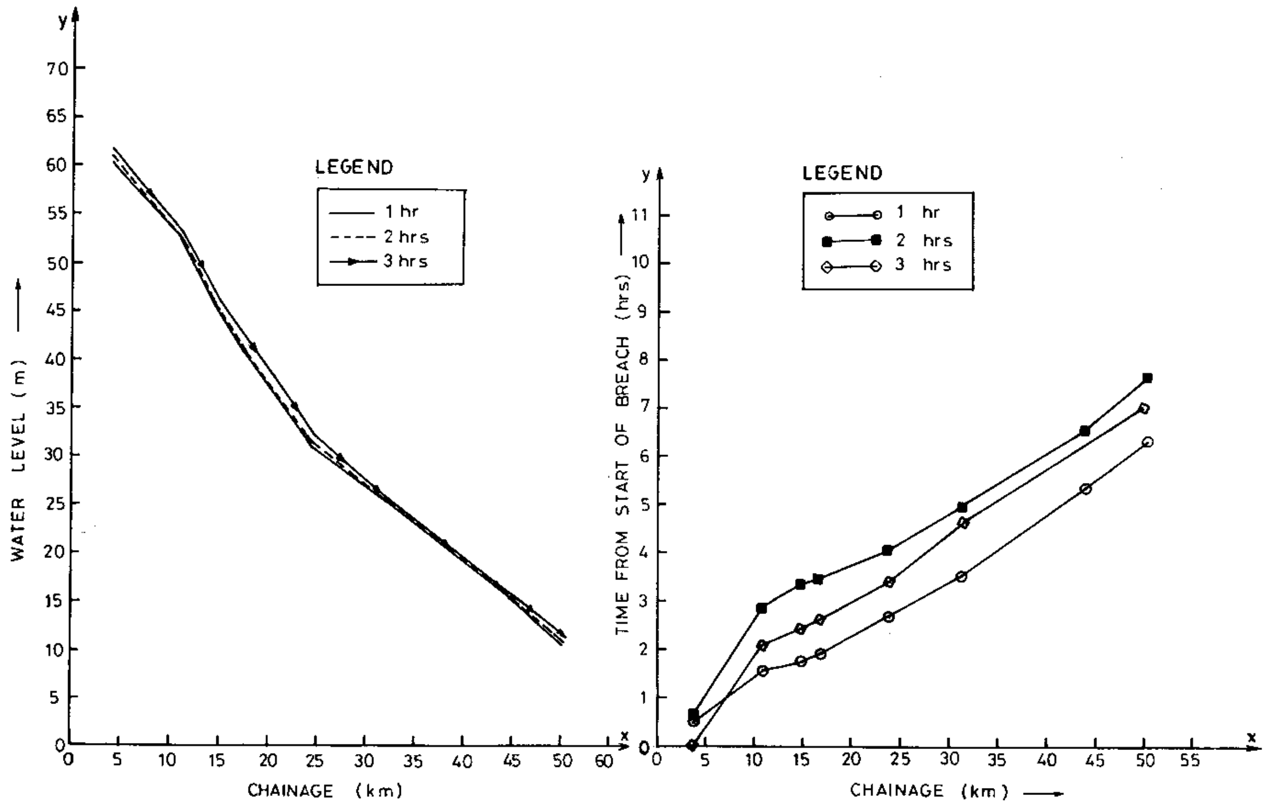

FIG4 — WATER LEVEL PROFILE AND TRAVEL TIME OF PEAK FLOW FOR DIFFERENT BREACH DEVELOPMENT TIMES

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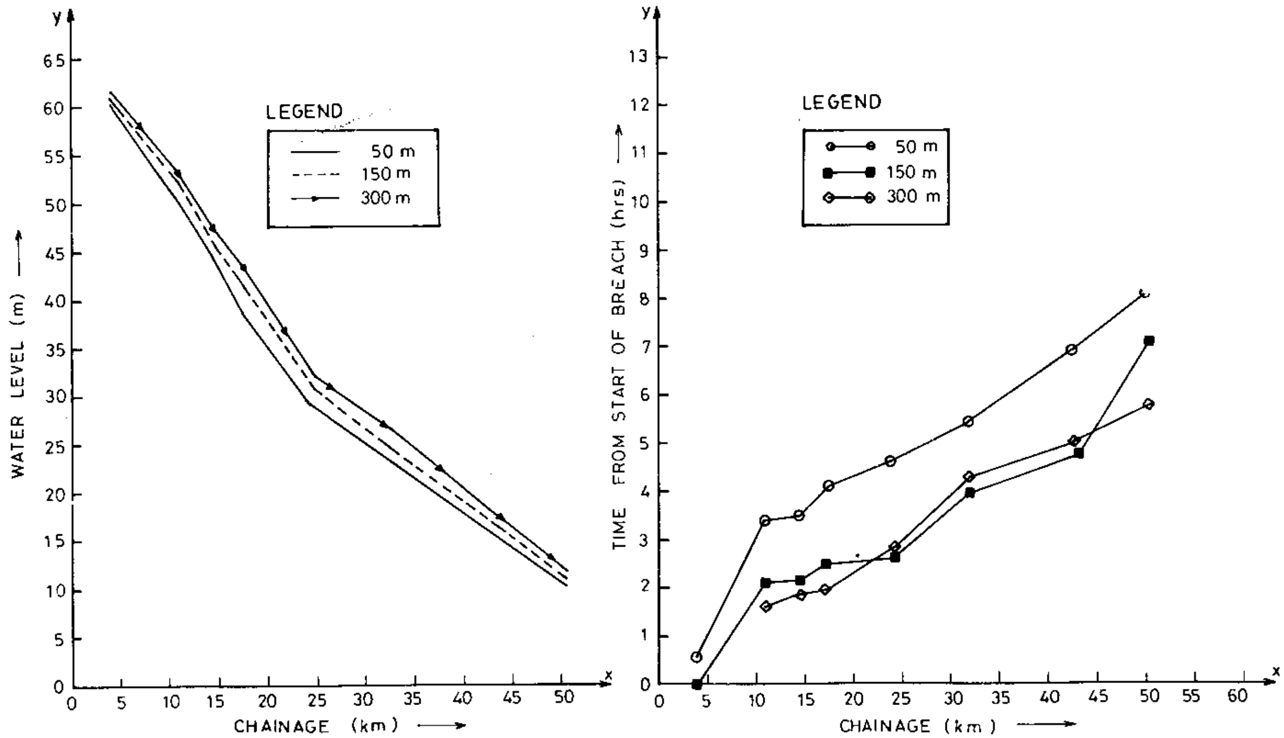

FIG. 5 WATER LEVEL PROFILE AND TRAVEL TIME OF PEAK FLOW FOR DIFFERENT FINAL BREACH WIDTHS

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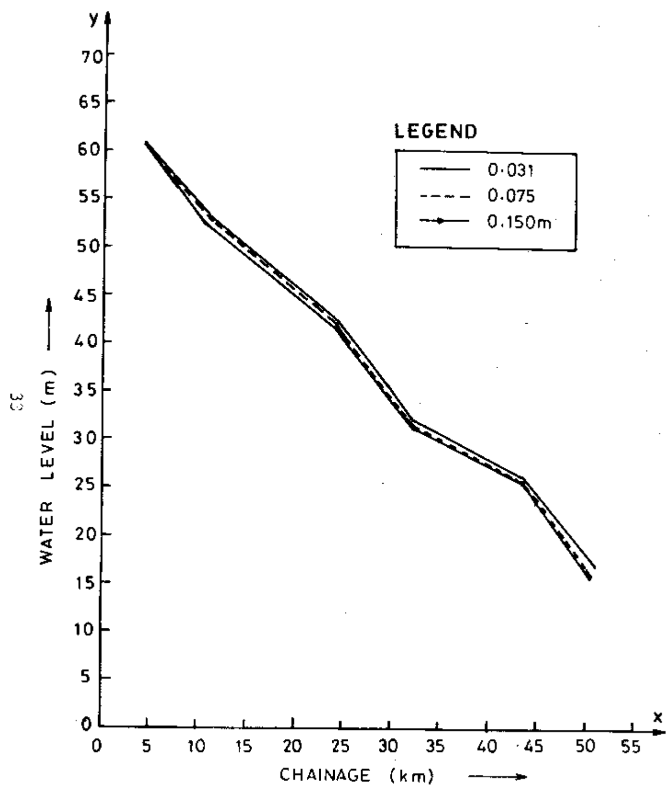

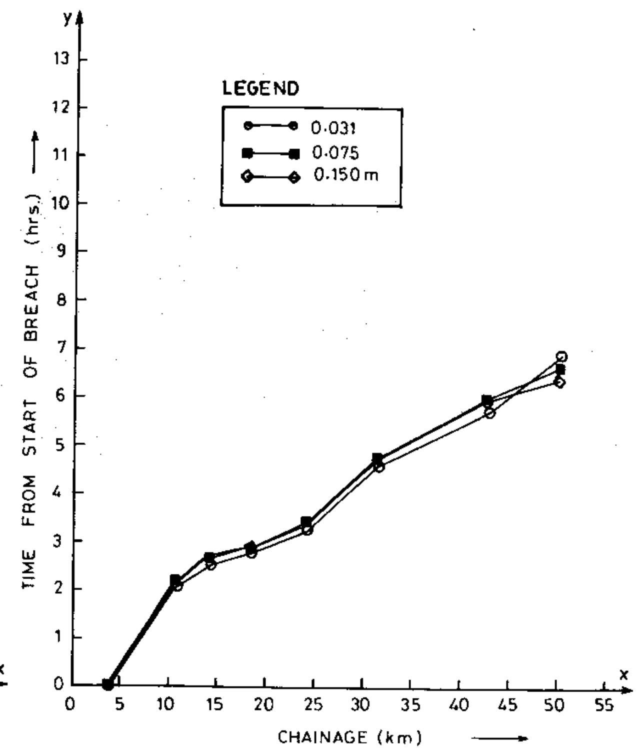

FIG.6 WATER LEVEL PROFILE AND TRAVEL TIME OF PEAK FLOW FOR DIFFERENT BREACH SLOPES

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for a period around time to peak. Table shows the salient features of these hydrographs.

Table 3

Salient features of the flood wave from Machhu Dam II failure

Sl no	River Km. from Machhu damII.	Peak discharge (m^3/sec)	Peak flood elevation (m) Computed	Peak flood elevation (m) observed	Time to peak flood elevation
1	0	54828	54.85	----	2.45Pm
2	1.3	49561	53.04	51.34	2.57Pm
3	9.4	31973	41.33	39.93	4.27Pm
4	17.4	27442	31.51	30.79	5.33Pm
5	25.4	24553	24.49	23.47	6.48Pm
6	33.3	23307	16.77	16.15	8.03Pm
7	50.2	22514	11.87	-----	9.06Pm

Fig 7 shows the peak flood elevation profile calculated using the model at various cross sections obtained by interpolating the available measured cross sections.The observed peak flood elevation noted at cross sections located at 0,1.3,9.4,17.4,25.4,33.3 and 50.2 down stream of the dam have also been shown in fig 7

Table 3 shows that the peak flood just below the dam comes to be 54828 Cumecs and the elevation comes to be 54.85m

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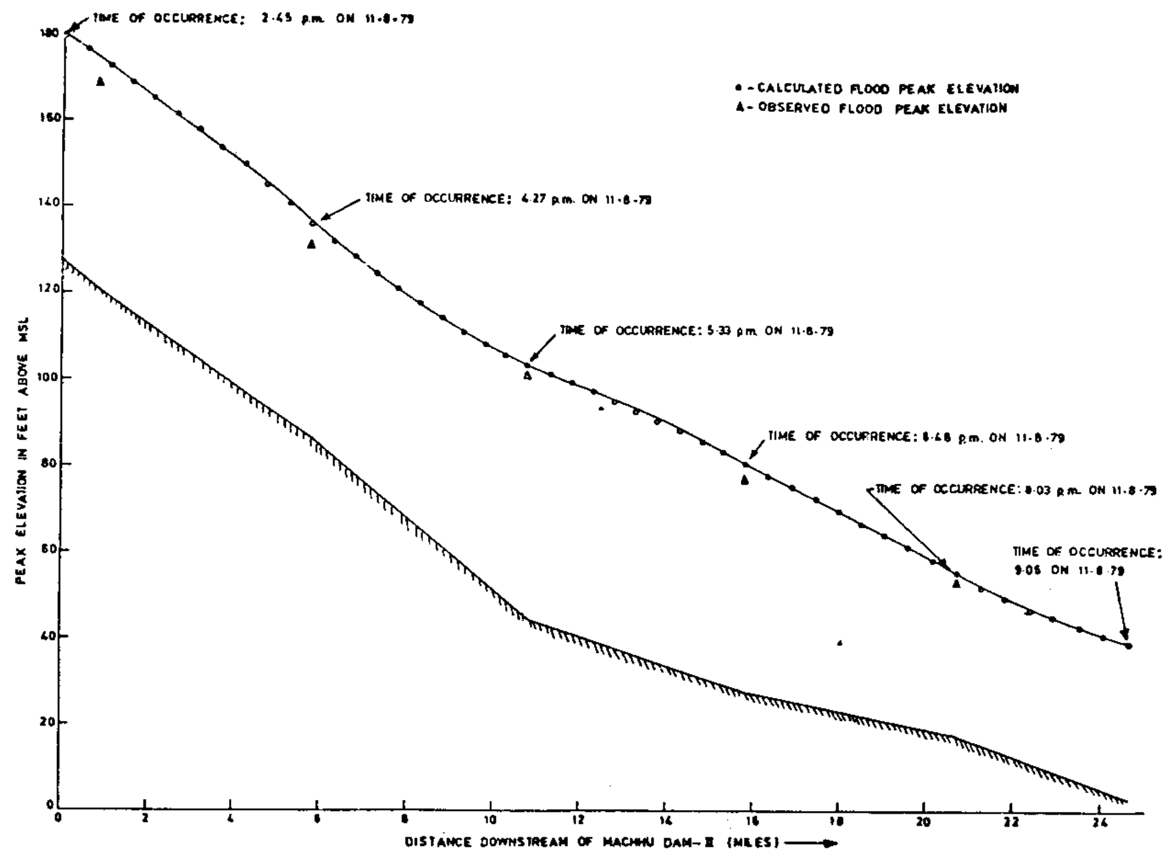

FIG. - PEAK FLOOD ELEVATION PROFILE FROM MACHHU DAM-II FAILURE

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.The time to peak flood elevation computed was 2.45 Pm.

4.51 COMPARISION OF RESULTS OF MIKE 11 USING PARAMETERS OF N.W.S STUDY TO THE RESULTS OBTAINED BY DAMBREAK STUDY USING N.W.S DAMBREAK PROGRAMME.

The comparision of results of both cases reveals that there is significant difference in the peak flood computation.With the same values of parameters,viz start breach level(0.1m),Final breach level(39.6m),Breach width(300m),breach development time(1 hour),and breach slope of 0.027m the MIKE 11 study gives the peak discharge of 36921 Cumecs at the Dam site,when compared to the N.W.s study of 54800 Cumecs.The travel time to attain the peak flow in both cases were obtained same i.e 1 hour 19 minutes and 1 hour 15 minutes respectively.

Fig 8 shows the hydrograph of MIKE 11 study at the dam site.It is evident from the figure that the peak discharge and the time taken to reach the peak is 21153 and 15 hours 48 minutes in Case 1 when compared to the Case 2 of 36921 and 13 hours 31 minutes respectively.There is a difference of around 5700 cumecs has been noted.

In order to further investigate in to the difference caused by the dam break flood at the dam site in both the cases, the theoretical description of the Reservoir outflow hydrograph of N.W.S Dam break programme has been taken and the difference is checked by substituting the required parameters.

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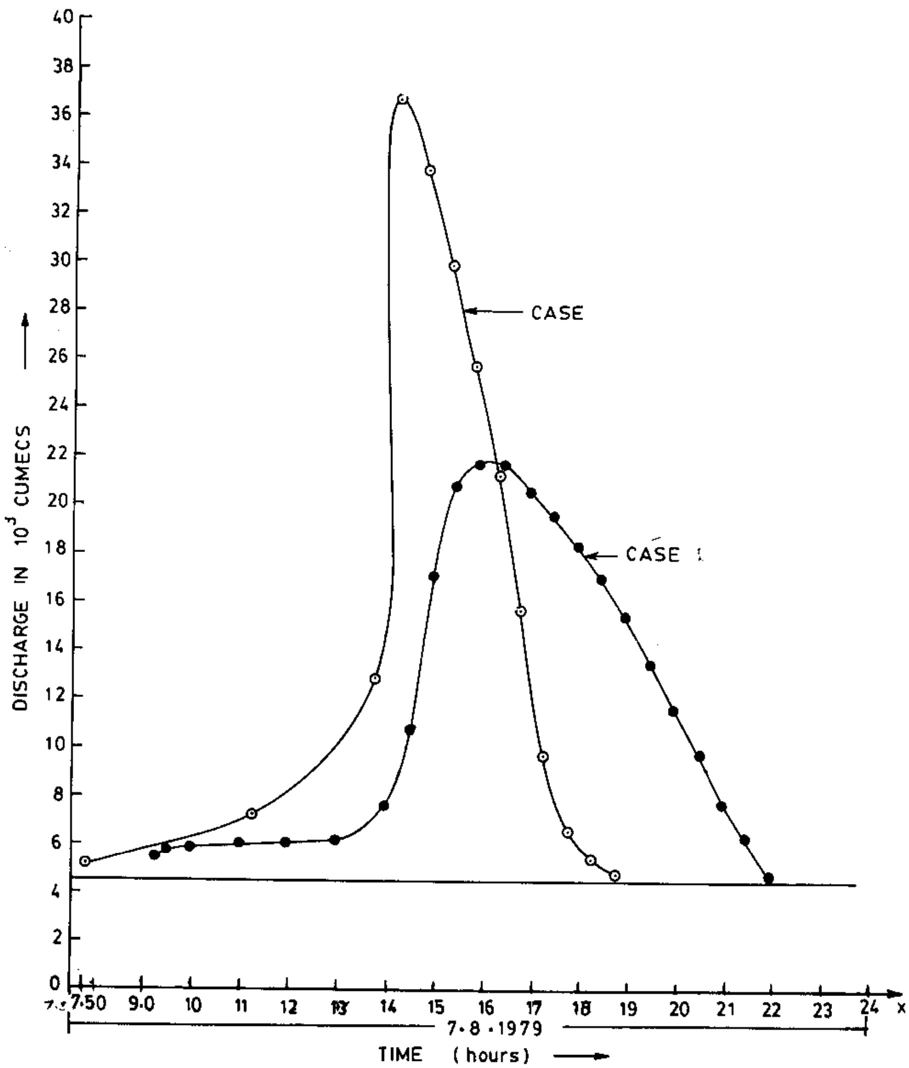

FIG. 8 COMPUTED DISCHARGE HYDROGRAPHS FOR DAM BREAK FLOOD AT THE DAM SITE

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The total reservoir outflow consists of broad crested weir flow through the breach and flow through any spillway outlets.i.e.,

$$Q = Q_b + Q_s \tag{1}$$

The breach out flow (Q_b) is computed as:

$$Q_b = C_1(h-h_b)^{1.5} + C_2(h-h_b)^{2.5} \tag{2}$$

Where:

$$C_1 = 3.1\ b_i\ c_v\ k_s \tag{3}$$

$$C_2 = 2.45\ z\ c_v\ k_s \tag{4}$$

$$h_b = h_d - (h_d - h_{bm})^{t_b/\tau} \qquad \text{if } t_b < \tau \tag{5}$$

$$h_b = h_{bm} \qquad \text{if } t_b > \tau \tag{6}$$

$$b_i = b.t_b/\tau \qquad \text{if } t_b < \tau \tag{7}$$

$$c_v = 1.0 + 0.023\ Q^2/[B_d^{\ 2}\ (h - h_{bm})^2(h - h_b)] \tag{8}$$

$$k_s = 1.0 \qquad \text{if } \frac{h_t-h_b}{h - h_b} < 0.67 \tag{9}$$

Otherwise:

$$k_s = 1.0 - 27.8\ [\frac{h_t-h_b}{h-h_b} - 0.67]^3 \tag{10}$$

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in which h_b is the elevation of the breach bottom, h is the reservoir water surface elevation, b_i is the instantaneous breach bottom width. t_b is the time interval since breach started forming, C_v is the correction for velocity of approach (Barter,1959), Q is the total out flow from the reservoir at the dam, k_s is the submergence correction for tail water effects on weir flow (Benard,1954), and h_t is the tail water elevation(water surface elevation immediately downstream of the dam).

The spill way out flow is computed as :

$$Q_s = c_s L_s (h - h_b)^{1.5} + c_g A_g (h - h_g)^{0.5} + c_d L_d (h - h_d)^{1.5} + Q_t$$

in which c_s is the un controlled spillway discharge coefficient, h_s is the uncontrolled spillway crest elevation, c_g is the gated spill way, discharge coefficient, h_g is the center line elevation of the gated spill way. c_d is the discharge coefficient for flow over the crest of the dam. L_s is the spill way length, A_g is the gate flow area, L_d is the length of the dam crest less, L_s and Q_t is the constant out flow term which is head dependent. The uncontrolled spillway flow or the gated spill way flow can also be represented as a table of head-discharge values. The gate flow may also be specified as a function of time.

A simple fortran programme has been written covering the above equations and the discharge at the dam site was compared in the Case 1 as well as Case 2.(The program and the data files are enclosed in the Appendix II).

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CASE 1: Adopted parameters

t_b= 1.999, τ=2.0,h_d = 60.05, b_d=3825, hb_m = 39.6,

b =150, h =57.6, z = 0.075, Qs = 4552, ht = 0.0, Cv = 0.5

CASE 2: t_b = 0.999, τ =1.0,h_d = 60.05, b_d = 6840,hb_m = 39.6,

b = 300,h = 57.6,z= 0.03,Q_s = 4552,h_t =0.0,C_v = 0.5

With these values of the parameters, Case 1 gives the discharge of 40228.33 Cumecs and the correction for velocity as 0.006,as compared to the total discharge of 75409.20 Cumecs and the correction for velocity as 0.006.in Case 2.

The results of both cases indicate that some different formulae has been used in MIKE 11 for computation of flood magnitude at the dam site.Since the exact documentation covering the above details is not available, it is not possible to arrive at any definite conclusion in this regard.

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5.0 CONCLUDING REMARKS

The present study involved use of Machhu II dam failure data with MIKE 11 programme.When the data set provided by the C.W.C for training purposes was used,it gave a peak value of dam break flood which is much less than that estimated for historical failure.The earlier study at N.I.H for this dam's failure using N.W.S dambrk.programme gives a peak flood value of 54800 Cumecs which compared reasonably well with historical value.In this study,dam breach parameters used in N.W.S study were assumed for application of MIKE 11 programme.The resulting peak flood discharge is 32% less than that for N.W.S application.Further study of sensitivity of dam breach parameters were also carried out.The final breach width and the breach slope are found to be more sensitive than the other breach parameters viz breach development time and Manning's n. In order to make any conclusive remarks regarding relative merits of N.W.S and MIKE 11 programmes,it would be necessary to examine the documentation in detail for clear understanding of factors leading to different peak floods for same set of dam breach parameters.How ever this could not be done in the present study due to non-availability of documentation of MIKE 11 programme.

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APPENDIX 1

DATA FILES OF MIKE 11 STUDY

a) RESERVOIR DATA(RESERVOIR ELEVATION/SURFACE AREA IN M^2)

ELEVATION (M)	SURFACE AREA(M^2)
42.50	1
45.80	987800
48.80	2518827
51.80	4544356
54.27	11065140
56.00	17625560
57.80	24167200
57.60	26026200
60.50	41827800

b) SPILLWAY DATA

WATER LEVEL(m)	Discharge (m^3/sec)
51.20	0
52.53	487
54.53	1806
55.47	2922
57.6	4552
58.52	6234
59.43	5908
60.42	6493

c) DAM DATA

Top of the Dam	60.05m
Length of earthen Dam	3825m
Top Width of dam	6.0m
Start breach level60m	
Final breach level	39.6m
Final breach width	150m
Breach slope	0.75m
Manning's n	0.033
Breach development time	2 Hours.

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d) BOUNDARY CONDITIONS (DOWN STREAM)

Level(m)	Discharge(M^3/sec)
0.79	0
7.61	1062
7.74	1122
7.98	1179
8.16	1240
9.0	2982
9.11	3999
9.40	5097
9.62	6630
9.90	9151
10.0	10000
11.0	20000
12.0	40000

e) RESERVOIR INFLOW HYDROGRAPH

Time (7/8/79)	Discharge (Cumecs)
0	97
4	364
8	970
12	2304
16	4609
20	8490
24	13099
28	9885
32	5579
36	2790
40	1695
44	970
48	364

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APPENDIX - II

```
C       PROGRAM FOR DMBRK USING NWS THEORY
        REAL KS
        OPEN(UNIT=1,FILE='DMB.DAT',STATUS='old')
        OPEN(UNIT=2,FILE='DMB.OUT',STATUS='new')
        READ(1,*)TB,TAU,HD,BD,HBM,B,H,Z,QS,HT,CV
        IF(TB.LT.TAU) THEN
        HB=HD-(HD-HBM)*(TB/TAU)
        ELSE
        HB=HBM
        ENDIF
        IF (TB.LT.TAU) B1=B*TB/TAU
        CHECK1=(HT-HB)/(H-HB)
        IF(CHECK1.LT.0.67) THEN
        KS=1.0
        ELSE
        KS=1.0-27.8*(((HT-HB)/(H-HB)-0.67)**3.0)
        ENDIF
1       C2=2.45*Z*CV*KS
        C1=3.1*B1*CV*KS
        QB=C1*(H-HB)**1.5+C2*(H-HB)**2.5
        TOTQ=QB+QS
        CV1=1.0-0.023*TOTQ*TOTQ/((BD*BD*(H-HBM)**2)*(H-HB))
        CCV=ABS(CV-CV1)
c       write(*,*)'cv,ccv',cv,ccv
        IF(CCV.LT.0.001) THEN
        WRITE(2,2)TOTQ,CCV
2       FORMAT(1X,'TOTAL DAM BREAK DISCHARGE =',F12.2/,'CORRECTION FOR
     1 VELOCITY =',F12.4)
        stop
        ELSE
        CV=CCV
        GO TO 1
        ENDIF
        END
```

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