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# DAM BREAK STUDY OF MITTI DAM

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# PREFACE

The floods and droughts are the two sides of a coin. These are the phenomena which can not be checked fully but can be managed to a certain extent that the magnitude of possible loss of human lives and properties due to either of these can be brought down to the minimum. The dams fall under the category of structural measures to fight with these problems. In the eventuality of failure of a dam, the floods generated are of much higher magnitude than those, which might have occurred, had the dam not been there. Hence, the dam break floods are more man-made than the natural. However, the inflows to the reservoir, which cause the dam to fail are natural. The management of the dam break floods needs the studies of the dams which have already failed and the experience so gained can be of utilized in determination of the floods due to possible failure of dams.

The present study deals with the study of Mitti dam which failed on July 17, 1988. The dam is located in Kachcha district of Gujarat State. The dam break study of the dam deals keeping in view the above aspects.

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( S M Seth )
**Director**

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## ABSTRACT

The post failure study of dams is a needed exercise to be made to assess the flood magnitudes and their behaviour downstream in the river valley so as to make a preparatory plan to safeguard the lives and properties on the flood plains of the river downstream of the dam structure. Fread's National Weather Service Model and the lately developed MIKE 11 at the Danish Hydraulic Institute are the examples of the models which deal with the dam break modelling. The basic difference between the two lies in the solution techniques of the St. Venant's equations used for dynamic routing of the dam break flood wave. The former uses well established four point finite difference scheme and the later the 6-point Abbott Scheme.

Present study deals with the dam break study of Mitti dam, located in Gujarat State, using MIKE 11. The dam failed on 17th July 1988 and inundated the flood plains in the downstream valley of the dam. The report describes in brief the methodology adopted for the solution of the routing equations, data availability and discussion on the analysis, data and the possible cause of failure of the dam.

## 1.0 INTRODUCTION:

Tremendous increase in population is one of the major causes behind the thinking of (i)optimal use of water resources on one hand and (ii) safeguarding the lives and properties, which have encroached the flood plains, from the fury of floods on the other. The dams are the best possible remedy to the problems as known today. For the reason, more and more dams have come up with the age. With the assured water resources facility and flood protection provided by the dam, the encouragement for improving the overall economy of the country has led to various developmental activities in the downstream of the dam resulting in the settlement of the large population and properties in the flood plain and adjoining areas. However, in the eventuality of any dam failure, the disaster would be catastrophic had the dams not being existing with flow occupying not only the erstwhile flood plain area but also the adjoining area. Therefore, it is the responsibility of the organisations involved with the safety of dams, to plan preventive measures so that in the eventuality of dam failure the disaster would be minimum to the extent possible.

One of the preventive measures in avoiding dam failure disaster is by issuing flood warning to the public of downstream when there is a failure of a dam. However, it is quite difficult to conduct analysis and determine the warning time of the dam break flood at the time of disaster. Therefore, pre-determination of the warning time assuming a various hypothetical dam break situations is a needed exercise in dam safety measures. Before attempting a hypothetical analysis dam failures for various existing dams, it would be appropriate to establish the credibility of the method used for such analysis by simulating the past dam failure scenarios with reference to the flood wave movement downstream of the failed dam. Further a knowledge of the case studies of the dam failures would give an insight in evaluating and reviewing the existing conditions of the dams. With this view, the failure of Mitti dam, in Kachcha district of Gujarat State, which occurred on 17th July 1988, resulting a number of loss of lives has been analyzed and presented in this report.

Basically, a dam failure study consists the following component steps:

1. Development or identification of the inflow hydrograph to the reservoir at the time of failure
2. routing this hydrograph through the reservoir
3. development of the failure conditions of the structure
4. calculating the outflow hydrograph through the breach section of the dam
5. modelling the movement of the flood wave downstream to determine the travel time, maximum water level reached, inundated areas etc.

Generally the case study of dam failures using the mathematical models pose various problems with regard to matching the model assumptions. The difficult problem is concerned with regard to the failure description adopted in the mathematical model. Under these circumstances, suitable assumptions with regard to the adjustment of actual failure mode to suit the model failure mode is necessary. Besides, the dam failures of overtopping generally occur due to severe storm with high inflow into the reservoir and due to this either the flow measurements were not made or the gauging sites were washed away resulting in no information on the inflow hydrograph to the reservoir. Therefore, for the dam failure study the inflow hydrograph is usually simulated using suitable rainfall-runoff models. Also due to failure of the dam, the downstream gauging stations are generally submerged resulting in no information on the downstream hydrographs. Therefore, in many cases, the only available information is the maximum water level marks at the time of passing of the flood wave. Time of breach development is also a major component in deciding the dam break flood hydrograph. Generally, an approximate value is available.

Miiti dam failure is not an exception of these problems. the purpose of the report is to present some findings in the verification of the reconstituted flood wave resulting from the failure of Mitti dam using MIKE 11 developed by Danish Hydraulic Institute. The methodology adopted in the model for simulating the analysis is briefly described. The report also describes the assumptions made with regard to the description of the failure of the dam to suit the actual failure mode with that of the mode required by the model. Also reported are the field data used in the model The study attempts to estimate the dam break flood hydrograph and its behaviour downstream of the dam.

## 2.0 REVIEW:

The few documented earth dam failures have shown one striking similarity and that is that the failure is any thing but sudden. For instance, in the overtopping case a breach will form and grow gradually under the erosive action of the waters. The gradual failure of an earth dam is of particular interest to disaster relief planners because the rate of growth of the breach strongly influences the peak and shape of the ensuing flood wave.

Considerable research is available for the case of instantaneous failure (Ritter,1892;Dressler,1952;and Whitham,1955). Instantaneous failure causes a positive wave in the down stream direction and a negative wave in the upstream direction. As evidenced by recent work of Brown and Rogers (1977), such an assumption is likely to be far from reality in the case of gradual breach. The duration of the Teton dam breach was approx. 3 hrs.

Cristfano's work is perhaps the first attempt to simulate the growth of a breach in an earth dam. Using geotechnical principles, he equated force of water flowing through the breach to resistive shear strength acting on the bottom surface of the overflow channel. Thus, he was able to relate rate of change of erosion to rate of change of water flowing through the overflow channel. The analysis led to an algebraic equation relating amount of eroded material to flow of water through the breach. Cristfano assumed that the breach top width would remain constant over time and that the breach would maintain a trapezoidal shape throughout the failure process. In addition, he fixed side slopes of the breach equal to angle of repose of the bank material and bottom slope of overflow channel equal to angle of friction of bed material. However, the use of an arbitrary constant renders, in effect, empirical.

In the late 1950's, the United States of Army Corps of Engineers Waterways Experiment Station (WES) used physical models to conduct an extensive investigation into floods resulting from suddenly breached dams (WES,1960 and 1961). A correlation was found between peak outflow from a suddenly breached dam and a shape factor describing the geometry of the breach. The WES findings support the conclusion that the Froude no. based on peak flow would reach a value of 0.29 which verifies the Schoklitsch equation (Harris and Wanger, 1967) for peak outflow from a sudden breach dam failure.

The WES findings do not apply to the case of a gradual breach of an earth dam. The experiments were carried out in a laboratory flume of specified bed slope, cross-section and roughness characteristics and the failure was simulated by an almost instantaneous removal of part or all of the dam. Therefore, while the tests were representative of sudden failure case, the results can not be associated with gradual failures because the hydraulics of the two cases are in fact quite different.

Prince at al. (1974) of the TVA have reported on models that use the relations for sudden dam breaches developed by WES. They do not apply their models to earth dams but rather to sudden failure of large gravity dams. Su and Barnes (1970) studied geometric and frictional effects of sudden releases and concluded that both resistance and cross-sectional shape were significant in determining behaviour of waves caused by sudden releases.

Brown and Rogers (1977) developed a computational model based on earlier work by Harris and Wanger(1967) in which the Schoklitsch formula was used to compute suspended sediment. They considered the failure of an earth dam immediately upon overtopping, degradation of the breach and erosion to datum level.

Brown and Rogers make several rather lucid statements regarding the mechanics of the breach developments. They point out the need for incorporating lateral erosion into the model simulation. They also address some of the differences in modes of failure for exceptionally high earth dams as opposed to long, low embankments. In addition, they point out that the bulk of the material eroded from the breach is deposited almost immediately downstream of the dam thus affecting tailwater depths and outflow hydrograph.

Fread(1980) has substantially contributed to modelling of dam breach phenomena in recent years. His doctoral dissertation dealt with a dam breach model which used the method of characteristics as its numerical solution procedure. The current version of Fread's model uses the four point finite difference scheme(Fread, 1980).

Fread assumes the rate of growth of the breach to be time dependant with either rectangular, triangular or trapezoidal shape. He accomplishes this by considering vertical erosion to take place at a constant, predetermined rate. This assumption is convenient because it allows the time scale of the phenomena to be fixed as a priori. However, it renders the model incapable of predicting the breach induced flood wave properties. It can produce a range of flood events for a given range of vertical erosion rates but which flood event is likely to occur is not discernible. Fread does not indicate that the outflow hydrograph is extremely sensitive to the chosen rate of vertical erosion but assumes any errors in prediction to be damped as the sharp wave moves down stream.

Fread's main concern is the downstream valley routing for the flood wave from a breached earth dam which is the ultimate goal of any investigation of potential dam breaches. However, it should be emphasized that rate of erosion and mode of failure of dam determine to large extent the shape and duration of the flood wave. Until this mechanism is better understood and properly described by mathematical modelling, Fread's approach can be considered only as an approximation giving a range of probable events.

## 3.0 METHODOLOGY:

The MIKE 11 developed by the Danish Hydraulic Institute, Denmark is used for the the analysis of Mitti dam failure. The MIKE 11 has got wide range capabilities on the estimation of rainfall-runoff relationship, sediment transport, flood forecasting, morphological aspects and dam break through the modules named as hydrodynamic, NAM, Forecasting, Solute Transport. The hydrodynamic module has been used for the dam break simulation study. A brief description of the capabilities i.e. hydrodynamic module, utilized in the study are described herein.

### Hydrodynamic Module:

MIKE 11 is a one dimensional, unsteady flow modelling package. It is capable of modelling unsteady flows in open channel systems through a numerical solution of the one dimensional de St. Venant Equations. Dam breaks are one particular case of the general flow conditions and some points to note in modelling dam breaks on MIKE 11 are given below.

The model provides a choice between three different flow descriptions viz.

1. Kinematic wave approach- The flow is calculated from the assumption of balance between the friction and gravity forces. This simplification implies that the kinematic wave approach can not simulate backwater effects.

2. Diffusive wave approach - In addition to the friction and gravity forces, the hydrostatic gradient is included in this description. This allows the user to take downstream boundaries

into account and thus simulate backwater effects.

2. Dynamic wave approach - Using the full momentum equation, including acceleration force, the user is able to simulate fast transients, tidal flows etc. in the system.

MIKE 11 uses the basic flow equations associated with the hydraulic resistance and the lateral inflow into these equations. These are:

Continuity equation

$$\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = q \qquad (1)$$

Momentum equation

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\left(\alpha \frac{Q^2}{A}\right) + gA\frac{\delta h}{\delta x} + \frac{gQ\,|Q|}{C^2\,AR^*} = 0 \qquad (2)$$

where,

| | | |
|---|---|---|
| $A$ | = | flow area ($m^2$) |
| $C$ | = | Chezy resistance coefficient ($m^{1/2}/s$) |
| $g$ | = | acceleration of gravity ($m/s^2$) |
| $h$ | = | stage above horizontal reference level (m) |
| $Q$ | = | discharge (cumecs) |
| $R^*$ | = | hydraulic radius (m) |
| $\alpha$ | = | momentum distribution coefficient |
| $q$ | = | lateral inflow ($m^2/s$) |

The spilling over the embankments crest along the river can be taken into account as a negative lateral inflow. In that case broad crested weir equation is applied, yielding :

$$Q = 1.705\ b\ H_c^{3/2} \tag{3}$$

where, b is the relevant embankment width and $H_c$ the depth (above embankment) at a critical section.

In order to obtain a stable solution to the finite difference schemes the following conditions have to be satisfied

Velocity Condition

$$\frac{V.\ \Delta t}{\Delta x} \leq 1\text{-}2 \tag{4}$$

where,

V = velocity (m/s)

$\Delta t$ = time step (sec.)

$\Delta x$ = distance in the computational nodes (m)

Courant Condition

$$C_r = \frac{(V+\sqrt{gd})\ \Delta t}{\Delta x} \leq 10\text{-}15 \tag{5}$$

The maximum $\Delta x$ values should be chosen with regard to the above requirements.

Solution Technique :

The transformation of Eqs. 1 and 2 to a set of implicit finite difference equations is performed in a computational grid consisting of alternating Q- and h-points i.e. points where the discharge Q and water level h, respectively, are computed at each time step (Fig. 1). Q-points are always placed midway between neighbouring h-points , while the distance between h-points may differ. The discharge, as a rule, is defined as positive in the positive x-direction (increasing chainage).

The solution technique adopted in MIKE 11 is a 6-point Abbott-Scheme as shown in Fig. 2.

Continuity Equation:

In the continuity equation (Eq. 1), the storage width is introduced first

$$\frac{\delta A}{\delta t} = b_s \frac{\delta h}{\delta t} \qquad (6)$$

which brings the equation of the form

$$\frac{\delta Q}{\delta x} + \frac{\delta h}{\delta t} = q \qquad (7)$$

As only Q has a derivative with respect to x, the equation can be centered at an h-point (Fig. 2). The derivatives in Eq. 7 are expressed at the time level, n+1/2 as follows:

$$\frac{\delta Q}{\delta x} \sim \frac{1}{2\Delta x_j} \left(\frac{1}{2} \left(Q_{j+1}^{n+1} + Q_{j+1}^{n}\right) - \frac{1}{2} \left(Q_{j-1}^{n+1} + Q_{j-1}^{n}\right)\right) \qquad (8)$$

$$\frac{\delta h}{\delta t} \sim \frac{1}{\Delta t} \left(h_j^{n+1} - h_j^{n}\right) \qquad (9)$$

$b_s$ in Eq. is approximated :

$$b_s = \frac{A_{0,j} + A_{0,j+1}}{\Delta 2x_j} \qquad (10)$$

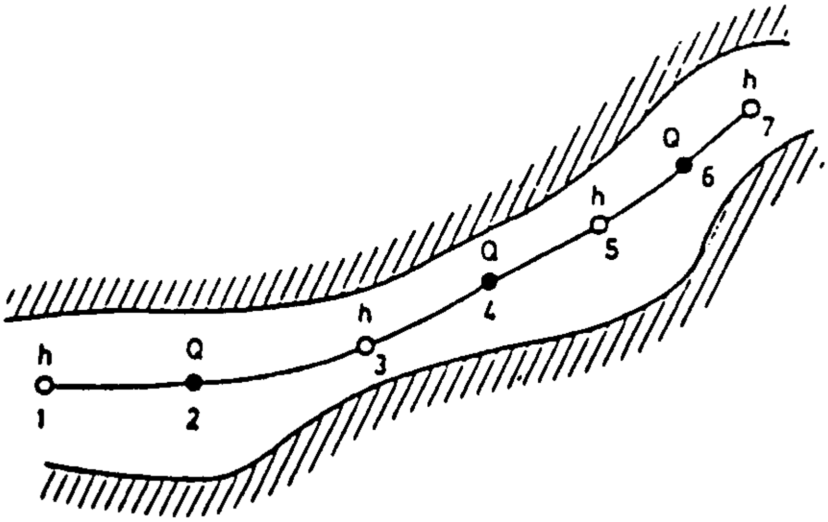


Fig. 1 : Channel Section with Computational Net

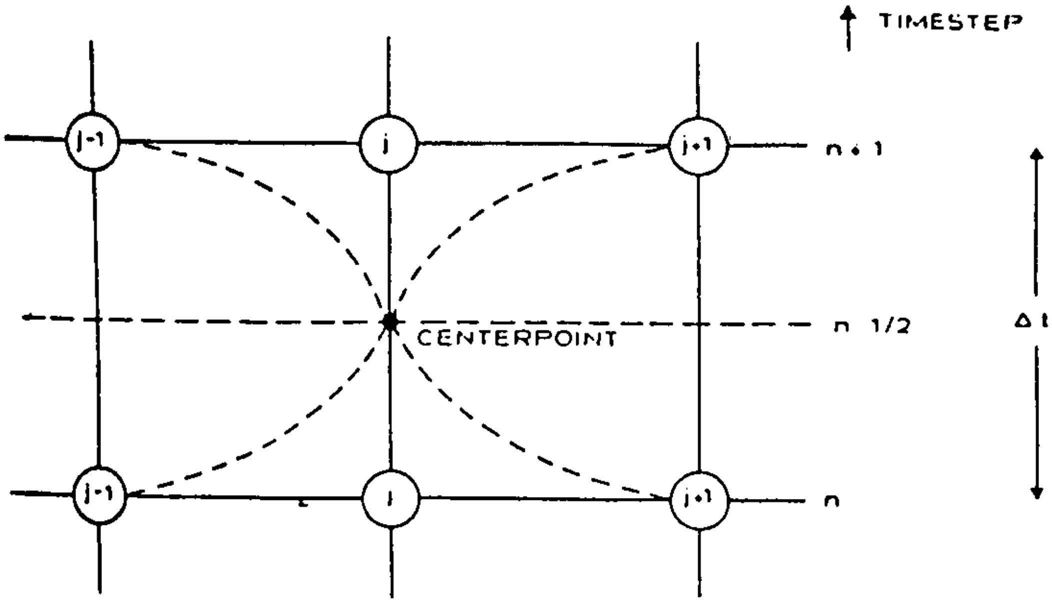


Fig. 2 : Centered 6-point Abbott Scheme

where,

$A_{0,j}$ = surface area between grid point j-1 and j

$A_{0,j+1}$ = surface area between grid point j and j+1

$\Delta 2x_j$ = distance between grid point j-1 and j+1

Substituting the derivatives in Eq. 7 gives a formulation of the following form

$$\alpha_j Q_{j-1}^{n+1} + \beta_j h_j^{n+1} + \gamma_j Q_{j+1}^{n+1} = \delta_j \qquad (11)$$

where, $\alpha$, $\beta$ and $\gamma$ are functions of b and $\delta$, moreover, depends on Q and h at time level n and q on time level n+1/2.

Momentum equation :

The momentum equation (Eq. 2) is centered at Q-points as illustrated in Fig. 3.

The derivatives of Eq. 2 are expressed in the following way:

$$\frac{\delta Q}{\delta t} \sim \frac{1}{\Delta t} ( Q_j^{n+1} - Q_j^n ) \qquad (12)$$

$$\frac{\delta}{\delta x} ( \alpha \frac{Q^2}{A}) = \frac{1}{2\Delta x_j} (( \alpha \frac{Q^2}{A})_{j+1}^{n+1/2} - (\alpha \frac{Q^2}{A})_{j-1}^{n+1/2} ) \qquad (13)$$

$$\frac{\delta h}{\delta x} \sim \frac{1}{2\Delta x_j} (\frac{1}{2} (h_{j+1}^n + h_{j+1}^{n+1}) - \frac{1}{2} (h_{j-1}^n + h_{j-1}^{n+1} )) \qquad (14)$$

For the quadratic term in Eq. 13, a special formulation is used to ensure the correct sign for this term when the flow direction is changing during a time step:

$$Q^2 \sim fQ_j^{n+1}Q_j^n - (f-1)\, Q_j^n Q_j^n \qquad (15)$$

where, f can take the user specified value but by default it is equal to 1.0.

With all the values substituted, the momentum equation (Eq.2), can be written in the following form :

$$\alpha_j\, h_{j-1}^{n+1} + \beta_j Q_j^{n+1} + \gamma_j\, h_{j+1}^{n+1} = \delta_j \qquad (16)$$

where,

$$\alpha_j = f(A)$$
$$\beta_j = f(Q_j^n, \Delta t, \Delta x, C, A, R)$$
$$\gamma_j = f(A)$$
$$\delta_j = f(A, \Delta x, \Delta t, \alpha, \Delta x, \phi, h_{j-1}^n, Q_{j-1}^{n+1/2}, Q_j^n, h_{j+1}^n, Q_{j+1}^{n+1/2})$$

To obtain a fully centered description of $A_{j+1}$, these terms should be valid at time level n+1/2 which can only be fulfilled by using an iteration. For this reason, the equations are solved by default two times at every time step. The first iteration starting from the results of the previous time step and the second iteration using the centered values from this calculation.

All other flow types can also be described, however, these have not been as they are not used in dam break modelling in the present study.

Double Sweep Algorithm:

For the solution of the equations, these are written in a general form as below:

$$\alpha_j Z_{j-1}^{n+1} + \beta_j Z_j^{n+1} + \gamma_j Z_{j+1}^{n+1} = \delta_j \tag{17}$$

Applying a local elimination, the coefficient matrix can be transformed as shown in Fig.4. It is thus possible to write any water level or discharge variable within the branch as a function of the water levels in the upstream and down stream nodal points $H_1$ and $H_2$, i.e.

$$h,Q = h,Q(H_1,H_2) \tag{18}$$

The continuity equation around a nodal point can be expressed as

$$ah_{node} + bh_{branch\ 1} + cQ_{branch\ 1} + dh_{branch\ 2} + eQ_{branch\ 2} + \ldots = z \tag{19}$$

where, a...z are quasi-constants. If Eq.. are substituted herein, a global relation can be obtained:

$$AH_1 + BH_2 + \ldots\ldots\ldots = z \tag{20}$$

A,B,...... to z are quasi constants. Eq.(20) expresses that the water level in a nodal point can be described as a function of the water level in the neighbouring nodal points. It is therefore possible to set up a nodal point matrix at each time step using the coefficients from Eq.(20). and the solution to the matrix yields by backward substitution of the water levels in all nodal points at the next time step.

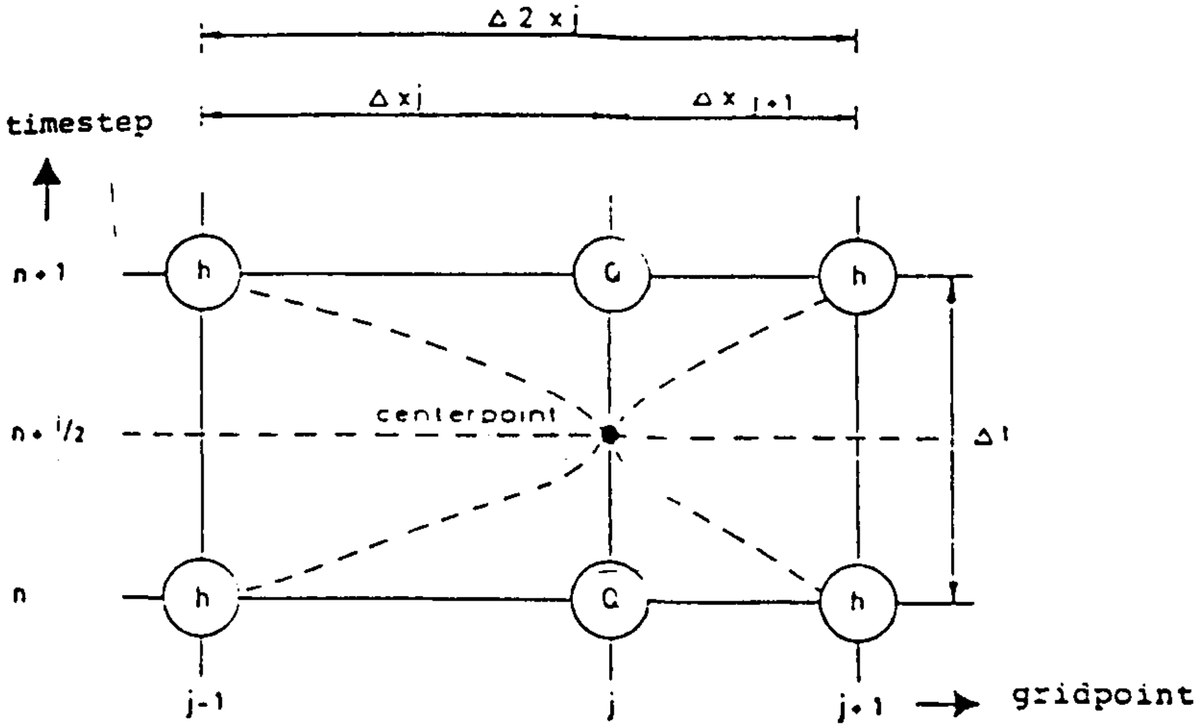


Fig.3 :Centering of Momentum Equation in 6-point Abbott-scheme.

$$
\left[\begin{array}{c} \alpha_0^* \\ \alpha_1^* \\ \alpha_2^* \\ \alpha_3^* \\ \alpha_4^* \\ \alpha_5^* \\ \alpha_6^* \end{array}\right]
\begin{array}{ccccccc}
\beta_0^* & & & & & & \\
 & \beta_1^* & & & & & \\
 & & \beta_2^* & & & & \\
 & & & \beta_3^* & & & \\
 & & & & \beta_4^* & & \\
 & & & & & \beta_5^* & \\
 & & & & & & \beta_6^*
\end{array}
\begin{array}{c} \gamma_0^* \\ \gamma_1^* \\ \gamma_2^* \\ \gamma_3^* \\ \gamma_4^* \\ \gamma_5^* \\ \gamma_6^* \end{array}
\left[\begin{array}{c} \delta_0^* \\ \delta_1^* \\ \delta_2^* \\ \delta_3^* \\ \delta_4^* \\ \delta_5^* \\ \delta_6^* \end{array}\right]
$$

Fig. 4 :Branch matrix after local elimination

Boundary Conditions:

Internal boundary conditions are :

1. Links at nodal points
2. Structures
3. Internal inflows
4. Wind friction

External boundary conditions may consist of :

1. constant values for h or Q
2. time varying values for h or Q
3. relation between h and Q (i.e. rating curve)

The structure description combines a wide range of elements, covering weirs, narrowing cross-sections, flood plains, reservoir operations etc. and can be regarded as an internal boundary condition. The description is obtained by replacing the the momentum equation with either an h-Q-h relation, an h-Q relation or a Q assignment. The grid used to describe a structure will consist of h-points at either side and a Q point at the structure.

## 4.0 DATA PREPARATION FOR MIKE 11

Most dam break setups consist of a single or several channels, a reservoir, the dam structure and perhaps auxiliary dam structures such as dam spillways, bottom outlets etc. Further down stream the river may be crossed by bridges, culverts etc.

### River Channel Setup:

Setting up of the river channel description in the cross-section data base is the same for dam breaks as it is for other types of modules. However, due to the highly unsteady nature of the dam break flood propagation, it is necessary 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 varies 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 highest level, the MIKE 11 has the capability to extrapolate the cross-section.

### Reservoir and Other Structures:

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 hydrograph may be specified.

The description of the reservoir storage is carried out by describing the elevation and additional flooded area which is the surface area corresponding to that elevation. The lowest water level should be somewhere below the final breach elevation of the dam and should be associated with some finite flooded area. This first value, however, describes a type of slot in the reservoir.

The cross-sectional area is set to a large finite value. It is only used in calculating the inflow headloss into the breach as

$$\Delta H = \frac{V_s^2}{2g} C_i \left(1 - \frac{A_s}{A_{res}}\right) \tag{21}$$

where,

$V_s$ = velocity through breach

$C_i$ = inflow headloss coefficient

$A_s$ = flow area through the breach

$A_{res}$ = cross-section area of the breach.

Therefore, in order to obtain reasonable headloss description, it is only necessary that $A_{res} >> A_s$ such that $(1 - A_s/A_{res}) \cong 1$. The hydraulic radius is not used in these calculations therefore a non zero value is supplied and the top width is set to zero so as to make the total surface area equal to add. flooded area as below:

$$A_{TOTAL} = (b * 2\Delta x) + \text{Add. flooded area} \tag{22}$$

The dam break structure itself is located on a separate branch which should contain three calculation points only, as shown in Fig. 5.

Dam

As the momentum equation is not used at the Q-point, the $\Delta x$ step used between the adjoining h-points is of no consequence. Therefore, the maximum $\Delta x$ to be specified should be greater than the

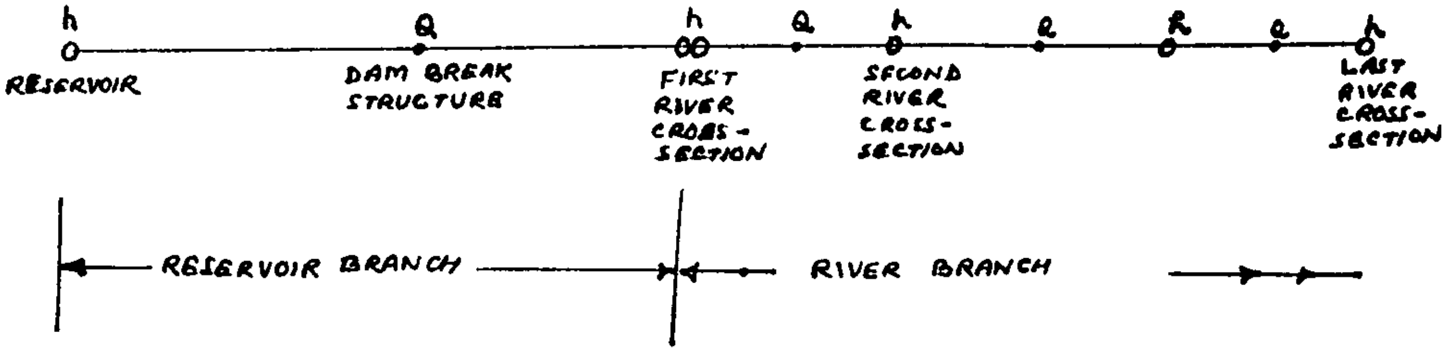


Fig. 5 : Set up for the Model

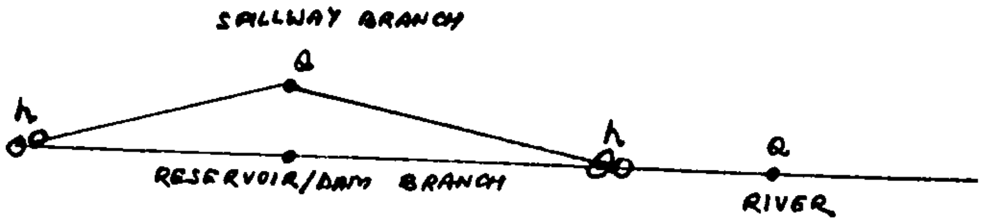


Fig. 6 : Set up for the Model with Spillway

difference between given chainages to prevent the insertion of the interpolated cross sections.

## Spillway

If a spillway is added to the dam itself, it is described as a separate branch, as shown in Fig. 6. At the node where the two branches meet, the surface flooded area is taken as the sum of the individual flooded areas specified at each point. Hence if the reservoir storage has already been described in the reservoir h-point should contain no additional flooded area. In this case both top width (b) and the additional flooded area are set to zero to describe the geometry of the first h-point.

## Boundary Conditions:

For the dam break simulation MIKE 11 needs boundary conditions to be specified at the first h-point and at the most down stream cross-section of the river valley. The upstream boundary will generally be an inflow hydrograph (i.e. the inflow hydrograph that caused the dam to fail) to the reservoir. The down stream boundary condition can either be a rating curve, or stage hydrograph or a outflow hydrograph. In the present study the upstream boundary condition used is the inflow hydrograph (Fig. 8) and the stage hydrograph (Fig. 10; stage hydrograph at chanaige 2 Km.) as the downstream boundary.

## 5.0 STUDY AREA:

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

Mitti dam is basically an irrigation scheme located on River Mitti near village Trambau in Abdasa taluka of Kachchh district in Gujarat. The latitude and longitude of the site are $23^{0}20'0''$ and $68^{0}49'30''$, respectively. The scheme drains a catchment area of 468.79 sq. Km and serves a gross command area of 5567 ha. The failure of the dam resulted in inundation of urban and sub-urban land in the flood plains of river Mitti downtream of the dam. Some of the inundated area falls in the Sukhapar district. An index map showing the flood affected area due to Mitti dam failure is shown in Fig. 7. The relevant design aspects of the dam are given below:

| | |
|---|---|
| Type of dam | Rock filled type earth dam with Masonry spillway |
| Length of earthen dam | 3180 m. |
| Crest level of dam | 24.60 m. |
| Maximum height from stripped level | 16.60 m. |
| Length of spillway | 120 m. |
| Type of spillway | Ogee type Chute spillway |
| Crest of spillway | 18.60 m. |
| Spillway design flood | 6808 cumecs |
| Gross storage capacity | 17.40 Million Cubic Meter |
| Dead storage | 2.68 Million Cubic Meter |
| Live storage | 14.72 Million cubic meter |
| Gross area of submergence | 578 ha. |

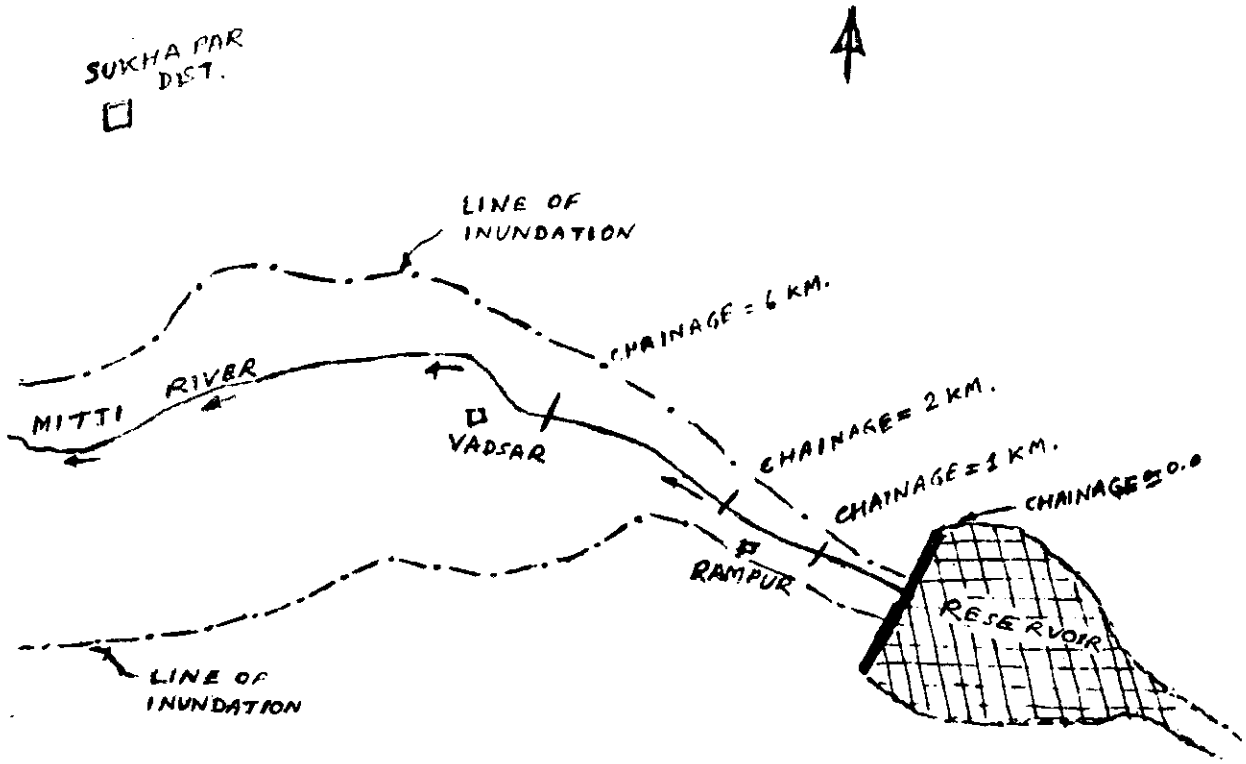


Fig. 7 : Index Map of River Mitti d/s of Mitti Dam

| | |
|---|---|
| Full supply level | 18.5 m. |
| High flood level | 22.10 m. |
| Free board | 2.50 m. |

A comparative study of the two hydrographs namely inflow hydrograph which caused the dam to fail and the design flood hydrograph for which the dam was designed indicates, from Fig.8, that the peak of the design flood hydrograph, being 5328 cumecs (spillway design capacity=6808 cumecs) is more than the peak of the inflow hydrograph i.e. 4734 cumecs and even then the dam reached to a level which led to failure. The inflow hydrograph, which caused the dam to fail, started rising on 16.07.1988 at 12.00 Hrs. and went on rising till 17.07.1988 (9.00 Hrs.). The peak of the inflow hydrograph (4734 cumecs) was reached in 21 hrs.

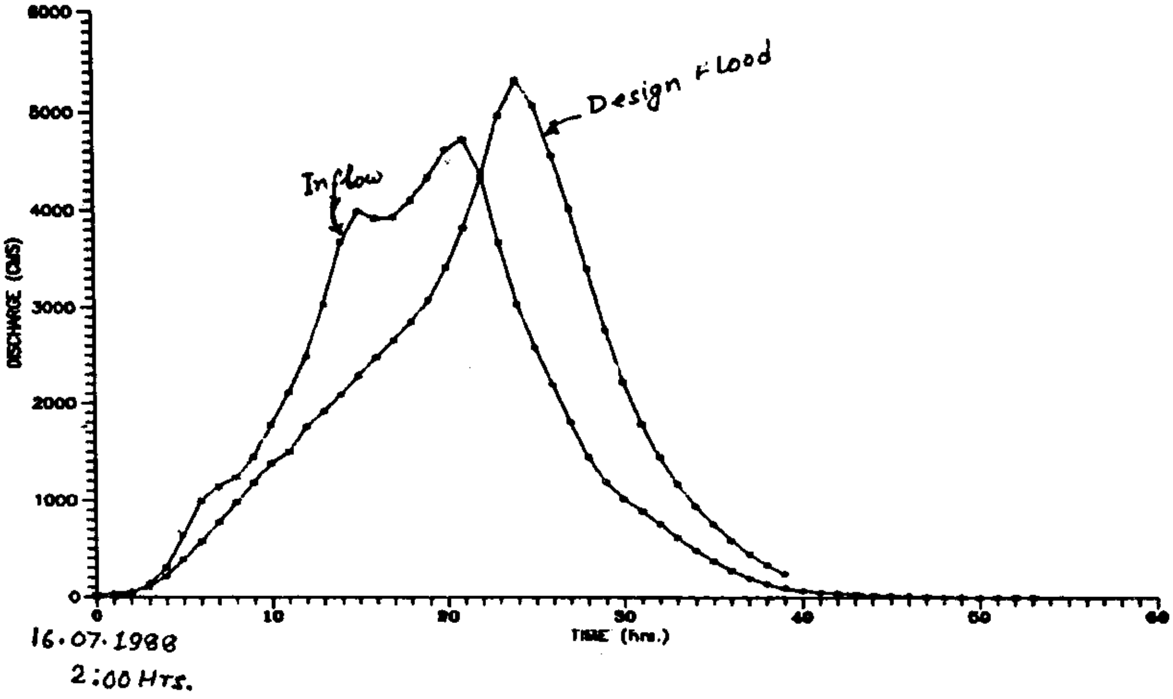


Fig. 8 : Flood Hydrographs

## 6.0 DATA AVAILABILITY :

The input data required for the Mike 11 can be categorized into three groups. The first group pertains to the dam, the second group to discharge/stage data and the third to the routing through the down stream river valley.

**First Group :**

With reference to the data group pertaining to the dam, the information on reservoir details like elevation-volume/surface area relationship, spillway details i.e. rating table, crest level, width are required. In addition to these, the information on top of dam, breach size, shape, time of breach development. This information was supplied by the Central Design Organisation of Gujarat Irrigation Dept.

**Reservoir Elevation-Volume-Surface area Relationship :**

The reservoir Elevation-Volume-Surface area relationship of Mitti dam supplied by the CDO is reproduced below:

TABLE-1 ELEVATION-AREA-CAPACITY RELATION

| Sl. No. | Elevation (m) | Area ($Mm^2$) | Capacity (MCM) |
|---|---|---|---|
| 1. | 8.0 | 0.0 | 0.0 |
| 2. | 9.0 | 0.133 | 0.060 |
| 3. | 10.0 | 0.259 | 0.262 |
| 4. | 11.0 | 0.449 | 0.614 |
| 5. | 12.0 | 0.518 | 1.245 |

| | | | |
|---|---|---|---|
| 6. | 13.0 | 0.840 | 2.024 |
| 7. | 14.0 | 1.338 | 3.188 |
| 8. | 15.0 | 2.060 | 4.912 |
| 9. | 16.0 | 3.133 | 7.508 |
| 10. | 17.0 | 3.711 | 10.930 |
| 11. | 18.0 | 5.375 | 16.273 |
| 12. | 19.0 | 6.825 | 22.373 |
| 13. | 20.0 | 8.052 | 29.811 |
| 14. | 21.0 | 9.564 | 38.613 |
| 15. | 22.0 | 10.785 | 48.793 |
| 16. | 23.0 | 12.026 | 60.198 |

**Spillway Details :**

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

Length of spillway = 120 m.

Crest level = 18.60 m

Coefficient of Discharge = 3.86

TABLE : 2 : SPILLWAY RATING TABLE

| Sl. No. | Head over crest level (m) | Discharge (cumecs) |
|---|---|---|
| 1. | 0.0 | 0.0 |
| 2. | 0.098 | 6.26 |
| 3. | 0.198 | 18.29 |

| | | |
|---|---|---|
| 4. | 0.299 | 34.18 |
| 5. | 0.399 | 53.18 |
| 6. | 0.500 | 75.38 |
| 7. | 1.000 | 221.94 |
| 8. | 1.500 | 421.46 |
| 9. | 2.000 | 668.51 |
| 10. | 2.500 | 959.66 |
| 11. | 3.000 | 1292.17 |
| 12. | 3.500 | 1657.08 |
| 13. | 4.000 | 2061.35 |
| 14. | 4.500 | 2503.36 |
| 15. | 5.500 | 3460.70 |
| 16. | 6.500 | 4532.09 |

The dam failure analysis using MIKE 11 has been considered in this study to begin at the time the level in the reservoir reaches to 24.55 m (just below the top of the the dam).

Breach Description:

The profile of the breached dam is shown in Fig. 9. The total area of the breach section comes out to be 3913 sq. m. The breached segments as shown in the figure are rectangular in shape, therefore, the shape of the breach has been considered to be rectangular. It is indicative here that the final breach level of has gone down to 7.43 m, even below the dam structure. There is a condition to be satisfied in the DAMBRK modelling by MIKE 11 and that is the final breach level must be well above the lowest elevation specified in the elevation-surface area relationship. This is due to the reasons that the surface area can not be taken

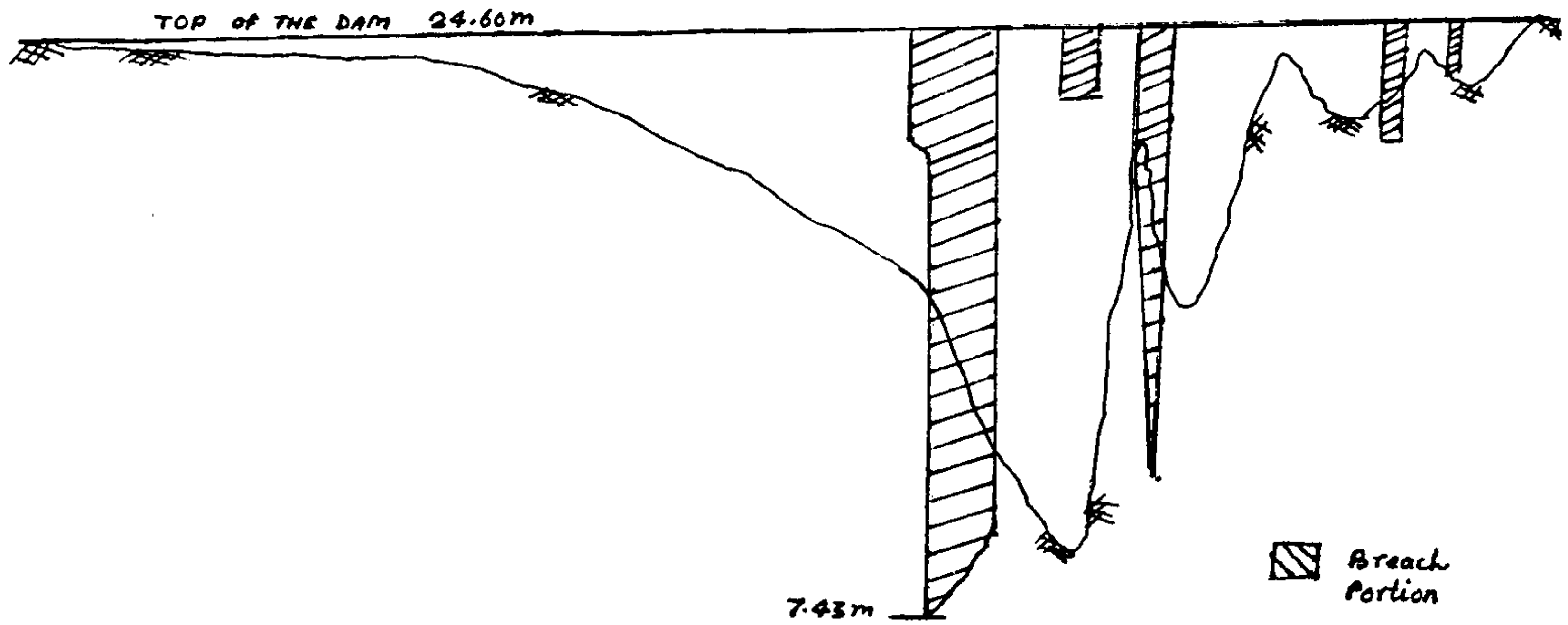


Fig. 8 : Breach Profile of the Dam

as negative. Therefore, the final breach level has been kept at 8.00 m. One more run has been taken for the case hen the breach shape is trapezoidal in shape.

The time of breach development, as reported by the CDO is 4 Hrs. since the start of the breach. The mode of erosion failure is considered to be linear.

Second Group :

The second group of data pertains to the flow and/or level data. The Inflow hydrograph,reported by the CDO is shown in Fig.8. Also reported is the level data of the dam break flood observed 2 Km. downstream of the dam site. This is shown in Fig.10. These two are used as the boundary conditions in the MIKE 11.

There is a constraint with the use of MIKE 11 that the movement of the flood in the downstream valley of the dam can not be simulated on dry bed. Therefore, an initial discharge of 1000 cumecs along with the water surface elevation data available at 2 Km. down stream of the dam are used for the establishment a flow continuum. However, this condition does not affect the final output.

Third Group :

The third group of data pertaining to the routing of the outflow hydrograph through the down stream valley consists of the description of the channel geometry i.e. the cross-sections, hydraulic resistance ( Chezy or Manning's roughness coefficients).

In this study, three cross-sections are available at locations 1 Km., 2 Km and 5 Km downstream of the dam structure. The model requires the description of ground elevations for describing the channel geometry and the geometrical parameters like hydraulic radius, perimeter, cross-sectional area etc. are calculated by the model itself. There is little information available on the roughness characteristics of the channel.

## 7.0 RESULTS AND DISCUSSION :

MIKE 11, developed by Danish Hydraulic Institute, has been used in the dam break analysis of Mitti dame failed on 17.07.1988. The analysis consists of (i) estimation of the dam break flood, which occurred after the dam failed, (ii) study of the behaviour of the flood wave during its travel downstream. In addition to these, the study of possible cause of the dam failure has been identified.

### 1. Estimation of Dam Break Flood Hydrograph :

The dam break flood hydrograph has been computed using the supplied data on the inflow hydrograph which caused the dam to fail, geometry of the breach, the time of breach development, cross-sectional data of the downstream reach and the observed fluctuations of the water level during the passage of the flood wave at Rampur village cause way located at 2 km downstream of the dam structure. The inflow hydrograph (Fig. 8) is used as the upstream boundary condition, while the water level fluctuations (Fig. 10) is used as the downstream boundary condition.

The inflow hydrograph which failed the dam is not an observed one rather a derived hydrograph after the dam failed. The accuracy of the inflow hydrograph may be a question mark but this is out of the scope of this report. The supplied hydrograph is assumed as to be the observed hydrograph.

The fluctuations of the water level which have been used as a downstream boundary condition are available at four hourly interval. The time interval has varied even more during the course of observations. Had the water fluctuations been at a closer interval more accurate the estimates were. A linear interpolation has been made so as to make them at hourly interval.

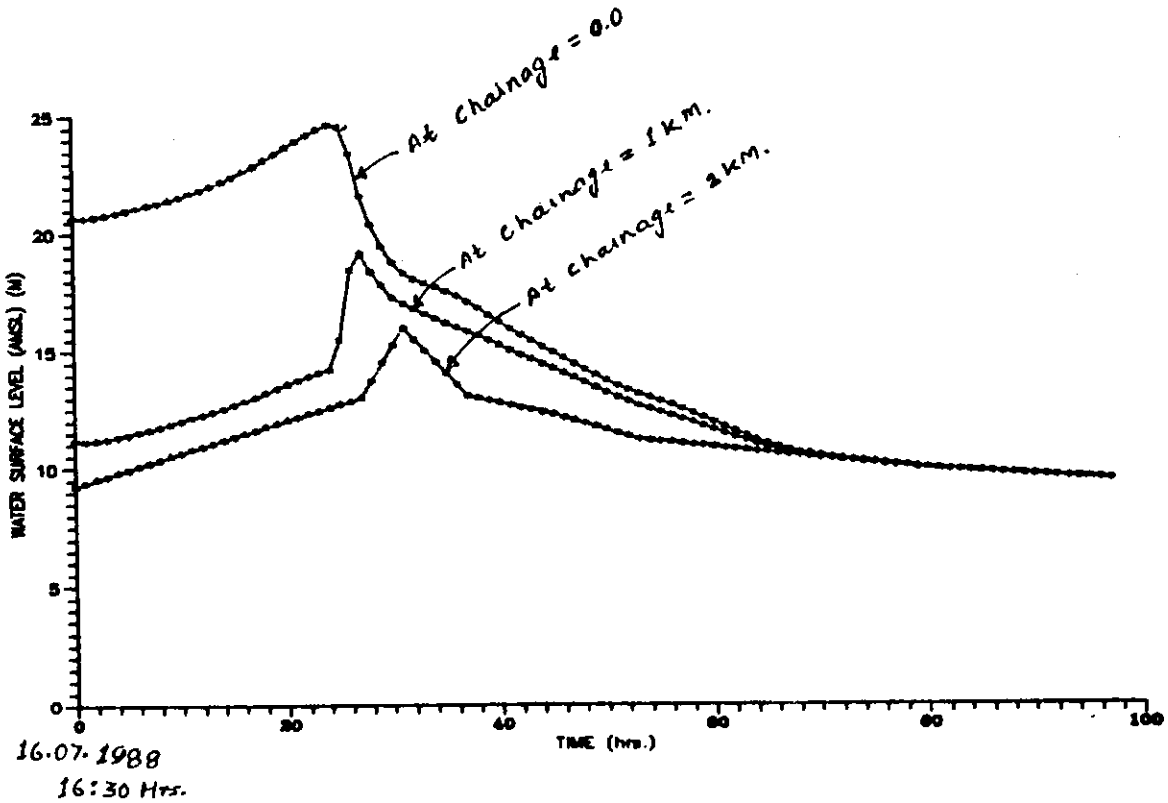


Fig. 10 : Water level fluctuations (Stage Hydrographs)

In MIKE 11, there is a facility provided that the dam break floods can be estimated even without the spillway. But in most cases, the spillway ia a necessary component of the structure so as to safeguard the dam structure from the eventuality of its failure or to safely pass the design flood. The length of the chute spillway is 120 m and the coefficient of discharge is taken equal to 1.705 for the development of spillway rating curve which acts as an internal boundary condition for the model.

It is indicative that the MIKE 11 model is not structured to run on dry beds. Therefore, an initial flow condition to be supplied in the model is a necessity of the model. It is, of course, a lack of computational ability of the model. The initial condition, however, does not affect much the overall shape or the magnitude of the dam break flood hydrograph. The initial condition assumes that a discharge of 1000 cumecs had been flowing throughout the channel reach to maintain the continuum.

As indicated in the literature review, the time of breach development has a great bearing on the shape and the peak flood magnitudes of the dam break flood hydrograph. Generally, it varies between 1-3 hrs. in the case of earth dams. The supplied time of breach development is 4 hrs. The time is out of the above range, This may be because of more strength of the dam structure to erosion. Mitti dam is a rock filled dam. Its core is made of rocks which are more resistant to erosion. Hence, more time has been taken in the development of the breach fully. The supplied time of breach development may be approximated as reasonably accurate.

The breach pattern of the dam (Fig. 9) indicates that the shape of the breach is rectangular. The dam has been eroded even below the ground level of the dam structure. The force of the flood wave, passing through the breach, has been sufficient to erode even below the ground level of the dam. Here, it is worth pointing out that the total volume of the outflow hydrograph is approximately equal to the total storage behind the dam. The time of the rise of the dam break flood hydrograph is almost equal to the time of breach development. Less is the time of breach development higher would be the peak and sharper the rise of flood hydrograph. Higher is the peak of flood magnitudes greater would be the velocities of the flow through the breach and hence faster would be the erosion activities. As the breach has gone down the ground level of the dam structure, indicating faster rate of erosion, it can be inferred that the time of breach development would be less than reported 4 hrs. But in the estimation of the inflow hydrograph in this report, the breach development time has been taken as 4 hrs.

The failure mode is also an important aspect to be looked into carefully while dealing with the dam break flood modelling. Most of the dams, as indicated in the literature review are failed due to overtopping and hence the failure mode is taken as overtopping. The mode of erosion, because of non-availability of the insight into the way it actually failed, it is taken as linear. However, at the start of the breach, its development is at a lower rate and there afterwards the erosion rate is faster.

Taking care of the above aspects in the analysis the estimated dam break flood hydrograph is shown in Fig. 11. The figure consists two hydrographs; one through the spillway and the other through the breach. However, the summation of the two is the total dam break flood hydrograph. The Flood hydrograph at 1.5 Km. downstream of the dam is presented in Fig. 12.

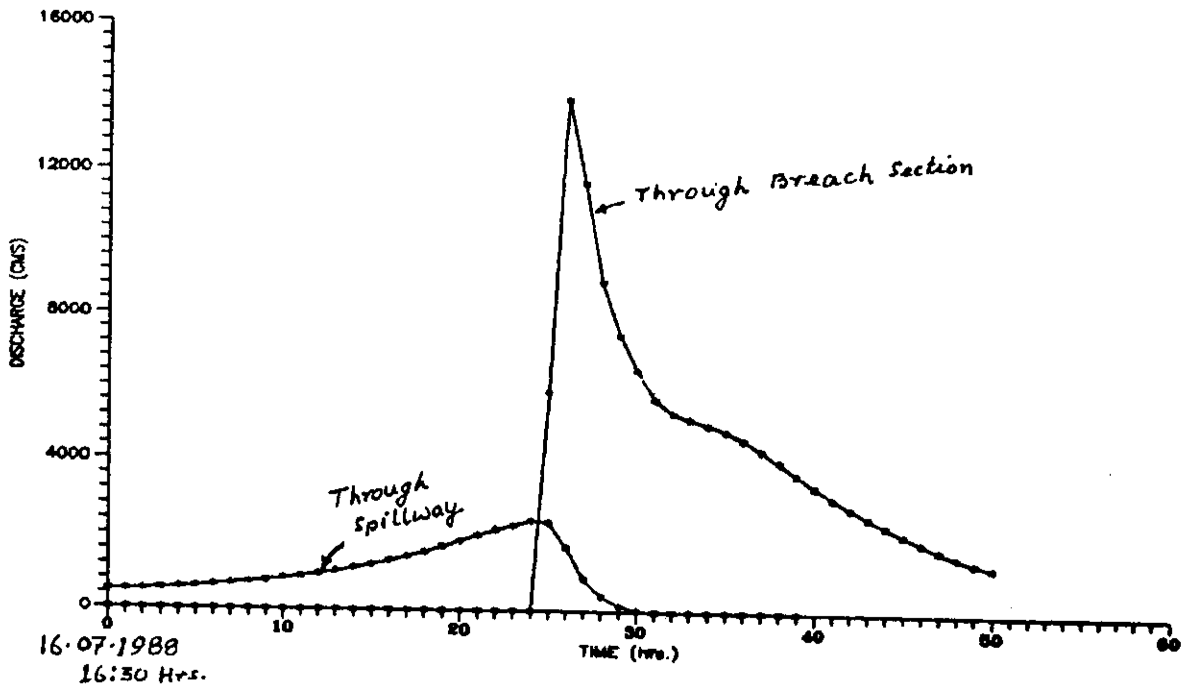


Fig. 11 : Dam break flood hydrograph (at Chainage = 0.0)

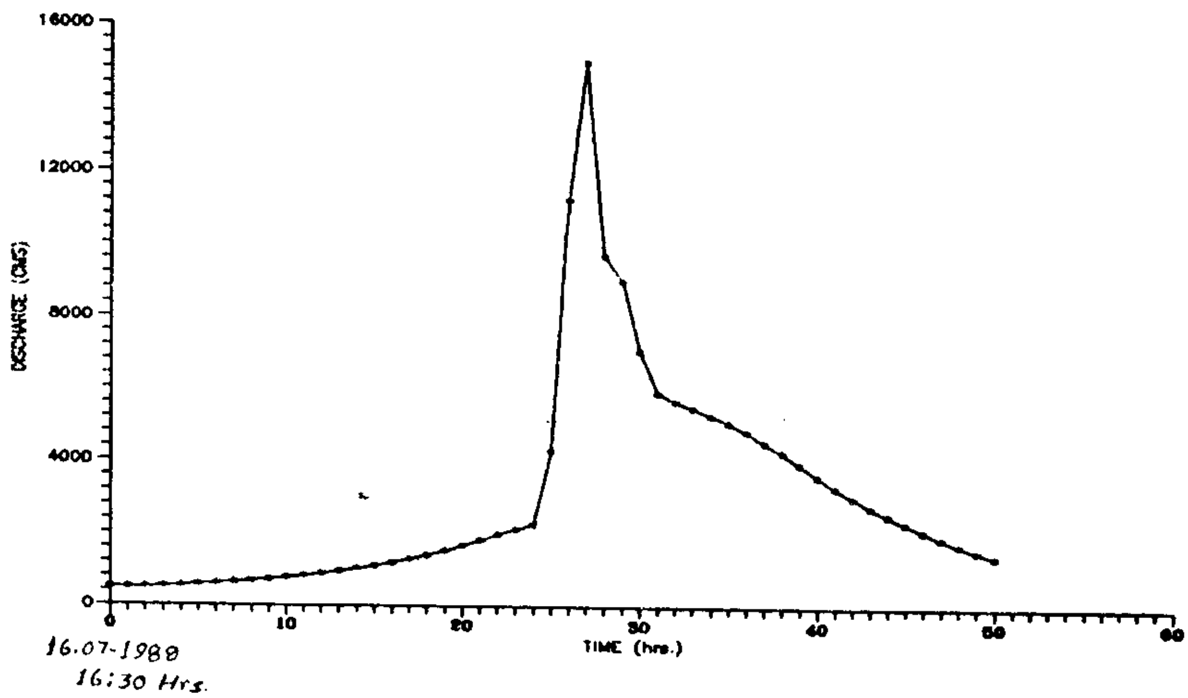


Fig. 12 : Flood hydrograph at Chainage 1.5 km.

2. Flood Wave Movement in the River valley:

The water level fluctuations in the reservoir and at 1 and 2 km downstream of the dam structure are shown in Fig. 10. There is a sharp decrease in the max. water elevations attained at different locations of the reach. The water surface profile of max. elevations attained at different locations of the reach is shown in Fig. 13. As the cross-sections of the downstream reach and the rating curves were not available, the study has been made using the available two cross-sections (d/s) of the river. Although a third cross-section was available at 6 km downstream of the dam but due to no information availability on roughness of the section, thereby non-availability of the rating curve, it was not used.

3. Possible Cause of failure :

As indicated earlier (Fig. 8), the inflow hydrograph peak is less than the design flood hydrograph. Even then the reservoir level behind the dam structure reached to a level which caused the dam to fail. The reported spillway discharge coefficient of the spillway was 3.86 (metric system) giving rise to a discharge capacity of 6808 cumecs when the reservoir level at the crest level of the dam. But generally, in metric system is of the order of 1.70 whereas the reported is more than double of the standard value. It means that the spillway has been designed for a higher discharge capacity than it could actually pass through it. The insufficient spillway capacity is the main reason behind the failure of the dam. the designed spillway length of 120 m is now planned to be extended to 235 m in future. This length, with the discharge coefficient of 1.70, is able to pass this inflow

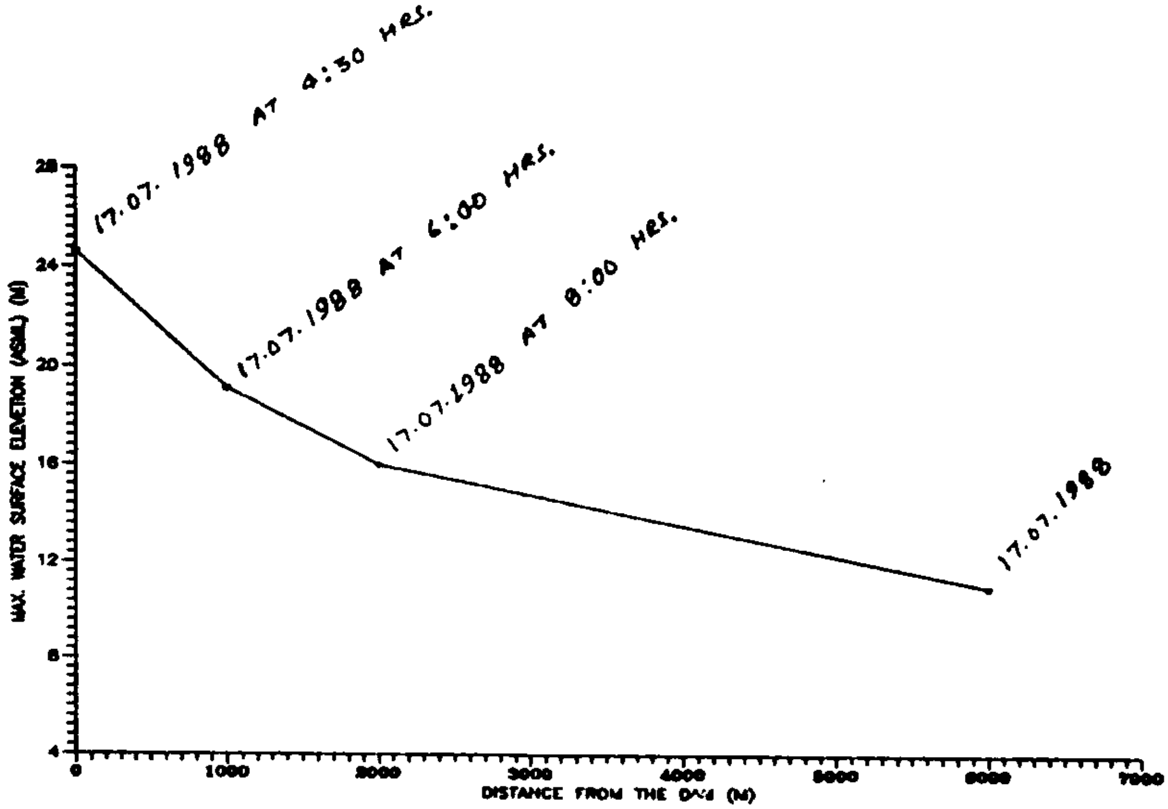


Fig. 13 : Water Surface Profile

hydrograph, which failed the dam, leaving a free board of 1.15 m. The free board would further be lessened if the design flood hydrograph is routed through the reservoir. This is simply because of the design flood peak being greater than the inflow hydrograph peak, which failed the dam.

## 8.0 CONCLUSION :

The dam break modelling study of Mitti dam is carried out using the MIKE 11 developed by the Danish Hydraulic Institute. Besided the other capabilities of the model, it can be used for dam break modelling using its hydrodynamic module which utilises the six-point Abbott Scheme for the solution of de St. Venant's equations used for the dynamic routing of the flood wave. Mitti dam failed on July 17, 1988. The post failure study of the dam revealed the following:

1. The dam break flood is of the order of 14000 cumecs.

2. The downstream water fluctuations are used in the model as a boundary condition. The observed fluctuations are faithfully simulated by the dam break flood estimated above.

3. The final breach level has gone even below the ground level of the dam structure indicating that the dam has failed completely. Literature review also support the concept of complete failure.The forecasting of other dam break floods may take advantage of this concept.

The time of breach development, in the present case, is 4 hrs. Normally, earth dam failures occur within 1-3 hrs. The experience may be of use in forecasting other dam break floods.

4. The bye-product of the study indicates that the possible cause of dam failure is the insufficient capacity of the spillway. The inflow hydrograph peak, which caused the dam to fail, is less than the peak of design flood.

The literature review on the subject area indicate that the dam break floods are very much dynamic in the vicinity of the structure and sharply tends to be diffusive with their travel downstream. If the downstream boundary condition, in present case water level fluctuations, is used as a unique rating curve which is a representation of the kinematic wave would be inappropriate due to the dynamic nature of the flood wave instead loop rating curve should be used for obtaining reliable results. However, the water level fluctuations is a better option for boundary condition in respect of accuracy of the results.

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