TR(BR) 14/97-98

EFFECT OF NON-HYDROSTATIC PRESSURE DISTRIBUTION ON DAM-BREAK FLOOD WAVE MOVEMENT

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1997-98

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ABSTRACT

Dam-break flows are well known to be with non-hydrostatic pressure distribution in a vertical plane. However, all most all the mathematical models for dam-break flows use Saint-Venant equations which assume hydrostatic pressure in the vertical; direction. In this report, an attempt has been made to study dam-break flows taking the effect of non-hydrostatic pressure distribution into account. For this purpose, the governing equations used in the present mathematical model are Boussinesq equations. Comparing with the Saint-Venant equations, these equations have three extra terms in the momentum equation. Due to the presence of a third order term in the equations, the numerical procedure adopted is third order accurate. The mathematical model is validated against previous experimental results. The model is applied to study the effect of non-hydrostatic pressure distribution on the free surface profile and the height and propagation of dam-break flood wave for various depth ratios (ratio of initial depths, downstream and upstream of the dam). The effect due to bed roughness and bed slope is also studied. The results show that inclusion of extra terms introduce oscillations that are actually taking place in the free surface. These oscillations are significant only for depth ratios greater than 0.4. The maximum deviation of the water level is of the order of 7%. The effect of non-hydrostatic pressure distribution exists only for a short time after the breaking of the dam. This time period depends on the depth ratio. However this may prevail for a considerable distance down stream of the dam. Out of the three extra terms used in the governing equations, first term (consisting of time derivative) is unimportant.

1.0 INTRODUCTION

Understanding of the nature of flow and the details needed for planning increases with the demand on harnessing for multipurpose use. More and more complicated problems are encountered in modern time. For example flood flow computation may be required to compute only volume at a reservoir or a flood hydrograph or water levels with reference to river bank-line as per improved planning methods requiring details of inundation.

Analysis of unsteady open channel flows is generally carried out by three methods, viz. analytical method, experimental method, and numerical method. Each method has its own advantages and disadvantages (Anderson et al. 1984). However, most of the studies are performed numerically solving the governing flow equations (Saint-Venant Equations). These equations are based on the assumption that the velocity distribution in a vertical section is uniform. This assumption results in zero vertical acceleration and ultimately vertical pressure distribution becomes hydrostatic. This assumption is valid for open channel flows with small gradients in the free surface. However, many unsteady flow situations in streams occur with relatively large gradients in water surface. Some examples of flows with large gradients in free surface are; spiked river hydrographs, rapid reservoir releases, dam-break flows, flow over sharp crested weir and flow over a bottom slot. The hydrostatic pressure assumption also fails for flow on a steep bed slope. Runoff in a steep gorge, flow over a spillway are some other examples of such flow situations. Flow in channel transitions (e.g. expanded channel, contracted channel and curved channel) is also susceptible to non-hydrostatic pressure distribution in a vertical plane. The effect of non-hydrostatic pressure may cause roll waves, hydraulic jumps, bores and breaking waves.

Flows can also be analyzed without using the hydrostatic pressure assumption. Navier-Stokes equations (Harlow and Welch 1965, Amsden and Harlow 1970), Boussinesq Equations (Palaniappan 1981, Carmo et al. 1993) and Vertically Averaged Moment (VAM) Equations (Khan and Steffler 1996) can be successfully used to analyze the open channel flows with non-hydrostatic pressure distribution. Methods to solve these equations can be seen in literature (Anderson et al. 1984, Roache 1978). However, use of these equations for open channel flows is not straight forward and easy. For example, the presence of the free surface makes the problem ill-conditioned in case of application of Navier-Stokes equations. While using VAM equations, six non-linear partial differential equations have to be solved which require enormous amount of computation. Similarly, solution of Boussinesq equations requires the method to be at least third order accurate as these equations contain third order terms. In this technical report an attempt has been made to use a set of equations, which do not assume the hydrostatic pressure distribution in vertical plane, for the study of dam-break flow which is well known to be with non-hydrostatic pressure distribution.

Dam is a potential source of hazard to life and property. The analysis of Dam-Break Flood (DBF) wave movement is important in many engineering applications (Almeida and Franco 1994). It is useful in (a)establishing the required dam spillway capacity, (b)environmental and safety impact evaluation of dams or other special structures built in a river valley, (c)valley planning and zoning, (d)implementation of operational emergency and safety procedures as warning systems and evacuation plans downstream dams, and, (e)solving special and unexpected problems due to accidents with a very high risk of a dam or other river obstruction failure.

Although, analytical models (Ritter 1892, Stoker 1957, Hunt 1982) and physical models (WES 1960, Dressler 1954) are available for the analysis of dam-break flood wave movement, a computer model is the most convenient tool for a fast and systematic study. The hydraulic effects are the water depth and flow velocity variations with time and the changes in the valley topography due to aggradation/degradation caused by DBF. At a dam design stage or at a dam rehabilitation analysis, DBF studies can be a primary method of a hydrologic safety evaluation. This can be done using, among others, the inflow design flood (IDF) methodology. The IDF is the flood inflow at which failure of the dam would not represent a significant additional hazard to lives and property downstream (Pansic and Borg 1992). According to the ASCE task committee on spillway design flood selection, quantitative risk assessment is the appropriate methodology to support selections of dam safety design floods (Newton 1989). The recommended procedures include probability-based assessments and the probable maximum flood (PMF) determination as the upper limit to natural flooding at a site considering both climatic and watershed variables. The analysis to select the safety design flood (SDF) include the identification and quantification of adverse consequences of dam failure. This provides information for a series of decision which ultimately lead to the selection of the SDF. The first step in the analysis is to define the extent of downstream flooding that would be caused by dam failure by defining the expected flood profiles and flooded areas with and without dam failure. Selection of the SDF for a dam is based on anticipated consequences of failure due to hydrologic events.

The area in a DBF analysis may be divided into a number of parts. In each part the importance of various parameters are to be considered. The computation model should be able to handle all situations. Inertia effects, two- and three- dimensional effects and, breach characteristics are dominant in the area close to dam. In the computational model, breach equations, submergence effects and flow dynamic compatibility equations are important. In the first valley reach downstream of the dam, bore propagation and friction effects are very important. Sediment erosion/deposition may also be important. The computational model requires the capability to model the sub- and super- critical flow regimes and to capture the shock. Two-dimensional effects and storage effects are dominant in flood plains. Also important are the local and convective inertia effects. The model should be able to consider the two-dimensional flow on an initial dry bed. Distant valley reaches from the dam site is subjected to one-dimensional flow where it may be done by simplified flood routing techniques.

DBF is well known to be with large gradients in the water surface. At the time of the dam-break, the velocity distribution in any vertical plane downstream but nearer to the dam location is not uniform. In a dam-break flow, very large quantity of water tries to rush into the channel from the reservoir within a short duration. This gives rise to three-dimensional flow with turbulence characteristics. Just at the time of dam-break, the water surface resting on the dam will be exposed to atmospheric pressure. Thus, pressure distribution in a vertical plane near to the dam body, which was hydrostatic prior to the dam-break, would become non-hydrostatic with a low value of pressure. Also, after the dam-break, a wave front will be formed. The pressure distribution near to the wave front will not be hydrostatic. Therefore, strictly speaking, Saint-

Venant Equations must not be used for the analysis of Dam-Break Floods. However, in the past, many studies for DBF have been performed using the Saint-Venant equations, which assume that the pressure distribution in a vertical plane is hydrostatic. These studies give satisfactory results when compared with flows occurring after sufficiently long period of movement or at a considerably long distance from the dam where the flow has already been attenuated.

NWS-DAMBRK is a commercially available computer program developed by Daniel Fread (1979), for the study of dam-break flows. It has been widely used by field engineers for the analysis of dam-break flood wave movement. Unlike many research articles published on DBF studies in various journals where field conditions are generally neglected, it has the considerations for practical applicability. The geometrical features of the channel and reservoir, the time and shape of dam-breach, flow in a compound cross-section, presence of other hydraulic structures are taken into account. However, it, like many other models, uses the Saint-Venant equations as the governing equations. Therefore, it is investigated here that if the use of equations which consider non-hydrostatic pressure has any bearing on the results of a dam-break flood wave movement.

The objectives of the present work are:

(1) to compare the performance of two numerical methods (Finite-Element-Method & Finite-Difference-Method) for the analysis of dam-break flood wave movement,

(2) to compare the results of dam-break flood wave movement using Boussinesq equations (assumes non-hydrostatic pressure distribution) and Saint-Venant equations (assumes hydrostatic pressure distribution) and verify the validity of the hydrostatic pressure assumption, and,

(3) to analyze the mechanics of DBF for both wet and dry bed conditions with an emphasis on
   (i) the time evolution of the flow depth at the dam-site, and
   (ii) the effect of the initially non-hydrostatic state on the long-term surface profile and the wave velocity.

2.0 REVIEW OF LITERATURE

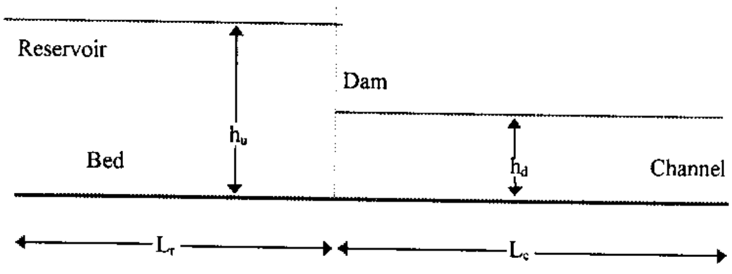
Review of literature presented here is based on two different aspects, i.e. (i) Dam-break flow, and, (ii) Non-hydrostatic pressure distribution. Different aspects related to the dam-break flows, giving an emphasis to the effect of non-hydrostatic pressure distribution, are presented in 'Dam-break flow'. Under 'Non-hydrostatic pressure', previous works using equations which consider the non-hydrostatic pressure are presented.

2.1 Dam-break flow

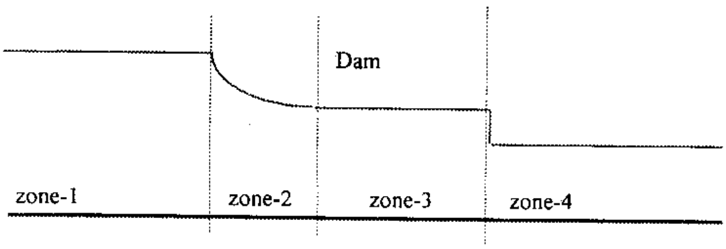
Due to importance of DBF in planning of water resources, the basic mechanics of flow have been subjected to considerable attention of researchers since many years. In literature one comes across a number of analytical, experimental and numerical studies concerning DBF. There are several aspects of DBF studies. Some of the important ones are; (a) breach mechanism (including size and time of the breach), (b) breach hydraulics, (c) changes in alluvial channel, (d) two- and three- dimensional effects, (e) flood routing (capturing of shock) and (f) risk analysis. For a detailed review on different aspects, Almeida et al. (1994) and Singh (1996) may be referred. However, in this technical report, the routing aspect is only considered and effect of non-hydrostatic pressure distribution on dam-break flood wave movement is emphasized and therefore, only the relevant references are cited in the following paragraphs.

The first attempt in the study of DBF was by Ritter (1892). He derived an analytical solution for the hydrodynamic problem of DBF. In his solution, Ritter assumed; (i) flow was one-dimensional after the dam-break, (ii) dam-break was instantaneous and complete, (iii) channel bed was horizontal, frictionless and was of wide rectangular cross-section, (iv) both the reservoir and the channel were infinitely long, (v) the reservoir contained static water prior to dam-break, and, (vi) the channel was dry prior to dam-break. He found that the flow depth and the discharge attained a constant value at the dam-site. These values were attained instantaneously. The shape of the free surface was a parabola and the tip speed was twice that of the disturbance propagated upstream. Later, Thirriot (1973) obtained a generalized Ritter solution, taking the canal cross-section into account. Dressler(1952) and Whitham(1955) included the effect of the bed roughness in the analysis of DBF and derived analytical expressions for the velocity and height of the wave-front. Stoker(1957) extended the Ritter solution to the case of a wet-bed condition in the downstream. He derived analytical expressions for the surface profile in terms of the initial depths upstream and downstream of the dam. In his solution there are four distinct zones, viz. two undisturbed zones, one each in the upstream and downstream side of the flow domain, one draw-down zone and a zone with a constant bore height (Fig. 1). In Stoker's solution, a constant velocity of bore propagation and the constant bore height are assumed to have attained instantaneously. The analytical equations derived by Hunt (1982, 1987) considered a finite length of the reservoir. However, Hunt's solution was based on the assumption of movement of a kinematic wave.

The analytical solutions mentioned above are based on one-dimensional Saint Venant equations. They are with the following limitations: (a) mathematical singularities at the initial phase of dam removal, and, on the wave front moving on dry bed, (b) uniform velocity on each



(a) Water surface profile (prior to dam-break)



(b) Water surface profile (at time, t, after dam-break)

FIG. 1 Stoker solution for DBF

vertical section of the flow, hydrostatic pressure distribution with no vertical acceleration. In order to obtain a more accurate analysis of the flow immediately after dam removal, Pohle (1952) considered a two-dimensional domain in a vertical plane where any liquid particle can have horizontal and vertical velocity and acceleration components. By using a Lagrangian approach, the basic equations are solved by assuming that the solution exist in the form of power series developments in the time. The conclusions drawn by this remarkable work are; (a) in the initial regime vertical acceleration is the predominant parameter. While the vertical acceleration is decreasing, the effects of channel cross-section, bed friction and bed slope become more important and the wave profile will then converge to one-dimensional analytical solutions. (b) in Ritter solution the water depth at dam site is attained instantaneously. In Pohle solution, it takes time, $t_D = C\sqrt{(h_u/g)}$ for the surface at the dam site to drop from the original value of h_u to $4/9\ h_u$, where C is a constant and h_u is the initial depth of water upstream of the dam. (c) the pressure distribution along verticals will depart from the hydrostatic one near the dam site during a short time interval after the dam break.

Physical modeling and experimental studies of DBF are also available in literature. Prominent among such studies are; (i) laboratory data on DBF by WES(1960), (ii) Dressler's

(1954) study for DBF on dry bed, and, (iii) two-dimensional DBF measurements by Bellos et al.(1991). Depending on the purpose of study, the measurements and methodologies differ. Numerical methods using the shallow water theory both in one- and two-dimensions are abundantly available in literature (Fenema and Chaudhry 1987, Cunge et al. 1983, Chaudhry 1993). Two numerical codes, DAMBRK (Fread 1979) and MIKE11(Abbott 1988) are very popular among practicing engineers as these codes take into account various practical issues like flow in the flood plain, sinuosity of the channel, time and size of dam-break. These codes use classical implicit schemes for the solution of one-dimensional shallow water equations (Saint-Venant Equations). In recent years, Total Variation Diminishing (TVD) and Essentially Non-Oscillating (ENO) schemes have been introduced to capture the front in the dam-break flow (Alcrudo and Navaro 1993).

It is well established that the pressure distribution is non-hydrostatic immediately after the dam failure (Pohle 1952, Strelkoff 1986). Basco (1989) has presented the limitations to the use of Saint Venant equations for dam-break flood wave calculations. Dressler(1954) experimentally showed that the depth at the dam site does not attain a constant value instantaneously after the dam failure, as predicted by Ritter using the hydrostatic assumption. It takes approximately nine non-dimensional time units (non-dimensional time = $t.\sqrt{[g/h_u]}$) to reach the constant Ritter value. Kosorin(1983) presented experimental results obtained in an horizontal flume with maximum water depth 0.5 m. The range of determined values of the vertical velocity and vertical acceleration were -1.4 m/s to 0, and, -g to 2g, respectively. He concluded that the maximum values of velocity and acceleration in vertical direction increases with dam height. Due to the two-dimensional effects, the wave front velocity is approximately equal to $\sqrt{(2gh_u)}$ and is not $2\sqrt{(gh_u)}$ as obtained by Ritter. Thus, the non-hydrostatic pressure distribution reduces the wave speed by about 30 %. All the above studies are for dry bed downstream conditions. However, there have been no comparable studies for wet bed conditions. Even in the works cited above there are considerable gaps, as no study of the complete evolution of the pressure field after the dam-break, and the short term and the long term consequences thereof, has been done.

Numerical simulations of DBF using two-dimensional equations in a vertical plane have been reported earlier by various researchers (Harlow and Welch 1965, Amsden and Harlow 1970, Hirt and Nichols 1981), but only for dry bed downstream conditions. Furthermore, DBF was used only as a test case in all these studies and no detailed analysis was presented. Strelkoff(1986) reports an exploratory numerical study on DBF, also for dry bed conditions, performed by Nichols using the Hirt and Nichols (1981) method.

2.2 Non-hydrostatic pressure

There are various research articles, which analyze free surface flows without assuming the usual hydrostatic pressure distribution in a vertical plane. For simplicity, we present them here under two categories, viz., (i) Depth-averaging and, (ii) Without depth-averaging.

2.2.1 Depth Averaging

BOUSSINESQ APPROACH: The theory that incorporates vertical accelerations, to a limited extent, in approximations to the horizontal motion equation is called Boussinesq Theory

(Boussinesq 1872). Various forms of the Boussinesq equations are found in literature. Variations are due to the order of accuracy of terms retained and methods of derivation. Boussinesq theory can be applied to finite amplitude quasi-long waves propagating in shallow water. In these cases the non-linearity, dispersiveness and bottom curvature are interrelated. Boussinesq equations can be derived by three different principles viz. (1) Asymptotic expansion method, (2) Variation method, and, (3) Conservation method.

Perigrine(1967) was the first to use asymptotic expansion method to derive Boussinesq equations for water of variable depth. In literature one may find application of Boussinesq equations known by various other names, such as *Serre equations, Perigrine equations* etc. However, in this work, the term *Boussinesq equation* is used. Though application of Saint-Venant equations for the study of open channel flows can be seen in a good number of research articles, a few using Boussinesq equations are also available. Palaniappan(1981) used the Boussinesq equations to study the flow in the tidal region of rivers with a curved boundary and slopping bottom. He used Finite-Element Method (FEM) to solve the flow equations. Rehman and Chaudhry (1996) studied the hydraulic jump using an adaptive grid technique. Gharanzik and Chaudhry (1991) also attempted the above problem by a different numerical method. Carmo et al. (1993a) applied the MacCormack Finite Difference scheme to solve the Boussinesq Equation. They studied the propagation of solitary wave, dam-break flow and a solitary wave overpassing an island. In another work, the above authors (Carmo et al. 1993b) solved the Boussinesq equations by FEM. In the finite-element code TELEMAC (Hervouet 1996), there is a module using Boussinesq equations, which were not found suitable for simulations with wetting and drying zones. In all the works cited above, Boussinesq equations considering non-hydrostatic pressure distribution were used to analyze the flow.

VERTICALLY AVERAGED AND MOMENT (VAM) EQUATIONS: VAM equations are depth averaged equations and can be derived by taking the moment of the momentum equations (Khan and Steffler 1996). Unlike the Saint-Venant Equations, which do not give correct results for flow over a sharp crested weir, these equations give satisfactory results.

OTHER EQUATIONS: As suggested by Paterson and Apelt (1988) depth averaged equations can be derived by assuming various forms of distribution for the velocity in a vertical plane. For example, they have derived equations for four different types of assumptions.

2.2.2 Without Depth Averaging

Navier-Stokes equations can be used to solve open channel flows. These equations are not depth averaged equations. These equations are valid for any value of Reynolds number and describe the flow field completely. Numerical solutions, for Navier-Stokes equations, based on various techniques (Lagrangian approach, Eulerian approach, and, Mixed Lagrangian-Eulerian approach) are available in the literature (Hyman 1989). A large *number* of turbulence models are also available in literature (Rodi, 1980). These models solve the Reynolds equations, which use average turbulence parameters.

3.0 PROBLEM FORMULATION

To simulate the unsteady flow due to the Dam-break flood wave movement taking non-hydrostatic pressure distribution into account, the partial differential equations governing the flow are presented in this section. In order to consider the effect of non-hydrostatic pressure distribution, Boussinesq equations are used in this work. Boussinesq equations for one-dimensional flow may be written as (Chaudhry, 1994):

Continuity Equation:

$$\frac{\partial h}{\partial t} + \frac{\partial u h}{\partial x} = 0 \qquad (1)$$

Momentum Equation:

$$\frac{\partial uh}{\partial t} + \frac{\partial (u^2 h + 0.5 g h^2 + B)}{\partial x} = gh(S_b - S_f) \qquad (2)$$

In the above equations, x = longitudinal direction, u = depth averaged velocity in x-direction, t = time, h = flow depth measured vertically, g = acceleration due to gravity, S_b = bed slope in x-direction, S_f = friction slope in x-direction and is calculated from the Manning equation. B in Eq. 2 is an expression containing differential terms that are known as the Boussinesq (Perigrine) terms. They account the non-hydrostatic pressure distribution. The expression for B is:

$$B = -\frac{h}{3}\left\{ \frac{\partial^2 u}{\partial x \partial t} + u\frac{\partial^2 u}{\partial x^2} - \left(\frac{\partial u}{\partial x}\right)^2 \right\} \qquad (3)$$

It may be noted here that *Saint-Venant Equations are a special case of Boussinesq equations* when $B = 0$.,

$$S_f = u^2 n^2 / h^{4/3} \qquad (4)$$

The assumptions used in the above equations are;

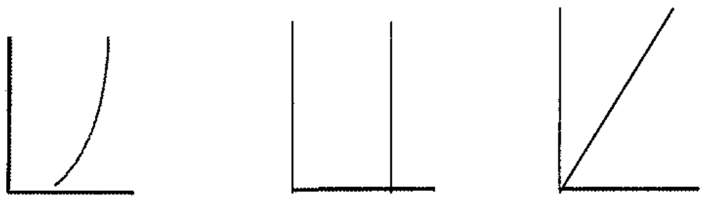
(1)Flow is one dimensional,

(2)Channel section is wide and rectangular,

(3)Velocity in a vertical direction varies linearly from zero at the bed to the maximum value at the surface,

(4)Bottom shear stress is dominant and all other shear stresses are negligible, (5)The friction losses computed using steady state formula are valid even for unsteady flow conditions.

The above assumptions are valid in most of the flow situations. However, the governing equations do not account for the effective stresses due to laminar viscous stresses, turbulence stresses



(a)True flow field **(b)Saint Venant Equation** **(c)Boussinesq Equation**

Fig. 2 Velocity distribution in vertical direction

and other stresses due to depth averaging. The assumption in the velocity distribution in a vertical plane is shown in Fig.2. Although, Boussinesq equations apply the procedure of depth averaging, the assumption in the velocity distribution considers a vertical acceleration and the pressure distribution in a vertical plane is non-hydrostatic.

The equations described above along with the boundary conditions appropriate to the problem considered will be solved for the flood flow caused due to abrupt removal of a dam in a wide rectangular channel. Further, the problem will be solved in a non-dimensional form as explained in the next chapter.

4.0 NUMERICAL SOLUTION

The governing equations, Boussinesq equations (Eqs. 1 and 2), are non-linear partial differential equations and are hyperbolic in nature. Generalized analytical solutions for the above equations are not possible. Therefore, these equations have to be solved numerically. First- and second- order numerical schemes perform satisfactorily for the solution of Saint-Venant equations. But, Boussinesq equations have third-order terms (Eq. 2). Therefore, considerable effort must be expended to reduce the truncation errors while approximating these terms by Finite-Difference Methods (Abbot 1979). It is necessary to use third or higher- order accurate numerical methods to solve these equations. In this work, the method developed by Gotlieb and Turkel (1976) (described by Chaudhry 1993) is used to study DBF waves.

4.1 NUMERICAL MODEL

PROBLEM FORMULATION: Initially, i.e. prior to dam-break, a flow of Q_i with flow depths, h_u and h_d on the upstream and downstream of the dam respectively (see Fig.1a) exists. The bed is rigid, wide and rectangular. Instantaneously, the dam is broken completely. The purpose is to find out the velocity and flow depth after a time, t, at desired locations downstream. The reservoir and the channel are so long that the disturbance due to the dam-break does not reach the upstream end of the reservoir and the downstream end of the channel. The time considered for the dam-break flow is also consistent with the above assumption.

The governing equations as discussed in the previous section are solved numerically for the above purpose. The flow reach is divided into a number of computational nodes. The partial differential equations are first discretized and the corresponding algebraic equations are solved using the principles of finite-difference method. A computer code for carrying out the computations described in the above model is developed. The solution strategy is presented in Fig.3. Various components of the model are explained below.

INPUT:
The following parameters are input to the model.

Geometrical: reservoir length, L_r, channel length, L_c, location of dam, x_{dam}

Computational: Number of divisions of the channel reach, N, Courant Number, C_n, time elapsed since dam break, t

Bed: bed slope, S_b, bed roughness, n

Flow: initial flow depth in the reservoir, h_u, initial flow depth in the channel, h_d, initial velocity in the channel, (u_d) and reservoir, (u_u), acceleration due to gravity, g.

GRID:

The grid size, Δx is calculated as
$\Delta x = (L_c+L_r)/N$ and
the i index of the computational reach is denoted by 1 (upstream end of the reservoir) to i_{last} (downstream end of the channel), where, $i_{last} = N+1$. The i index

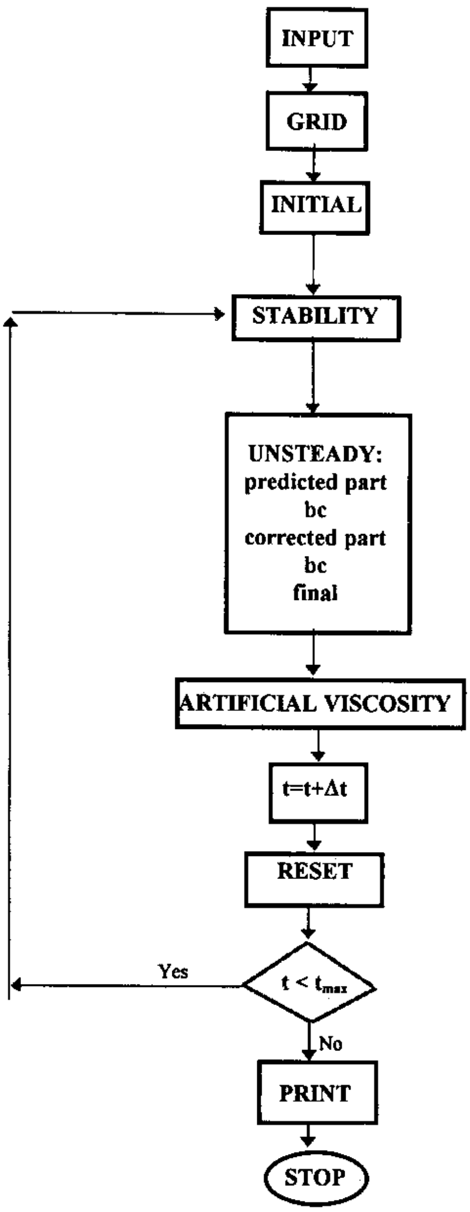


Fig.3 Flow chart showing the numerical method

of the dam, i_{dam} is calculated by $i_{dam} = 1 + L/\Delta x$. The number of divisions, N, should be such that further increase in N value should not change the results significantly

INITIAL CONDITION:

The values of the flow depth and velocity at all the nodes are given. The initial condition is set for the present problem as:

$$t = 0 \tag{5}$$
$$h_i = h_u \text{ for } 1 \le i \le i_{dam} \tag{6}$$
$$h_i = h_d \text{ for } i_{max} \ge i > i_{dam} \tag{7}$$
$$u_i = u_u \text{ for } 1 \le i \le i_{dam} \tag{8}$$
$$u_i = u_d \text{ for } i_{max} \ge i > i_{dam} \tag{9}$$

STABILITY:

The present model is using an explicit finite-difference method. Therefore, it has to satisfy the stability condition. The calculation of Δt is done using the Courant-Friedrich-Lewis (CFL Condition) stability criteria,

$$\Delta t = C_n \frac{\Delta x}{|u| + \sqrt{gh}} \tag{10}$$

where, C_n is the Courant number and its value is less than 1.

UNSTEADY:

The model now calculates the unsteady flow parameters by solving the continuity and the momentum equations as given in section 3. It has three components and these are presented below. First, the values are predicted for the new time level and then these values are corrected. The final values of the flow parameters are obtained by averaging the values already got for the corrected step and known time level. The following finite-difference approximations known as the two-four scheme is used.;

Predicted part:

$$\{h_i\}^P = \{h_i + (1/6)(\Delta t/\Delta x)[(uh)_{i+2} - 8(uh)_{i+1} + 7(uh)_i]\}^t \tag{11}$$

$$\{u_i\}^P = (1/h_i)^P \{(uh)_i + (1/6)(\Delta t/\Delta x)[(u^2h + gh^2/2 + B)_{i+2} - 8(u^2h + gh^2/2 + B)_{i+1} + 7(u^2h + gh^2/2 + B)_i] + \Delta t g h_i (S_b - S_f)\}^t \tag{12}$$

Corrected part:

$$\{h_i\}^C = \{h_i + (1/6)(\Delta t/\Delta x)[-(uh)_{i-2} + 8(uh)_{i-1} - 7(uh)_i]\}^P \tag{13}$$

$$\{u_i\}^c = (1/h_i)^c \{(uh)_i + (1/6)(\Delta t/\Delta x)[-(u^2h+gh^2/2+B)_{i-2} + 8(u^2h+gh^2/2+B)_{i-1} - 7(u^2h+gh^2/2+B)_i] + \Delta t g h_i (S_b - S_f)\}^p \quad (14)$$

Final:

$$(h)^{t+\Delta t} = 0.5(h^t + h^c) \quad (15)$$

$$(u)^{t+\Delta t} = 0.5(u^t + u^c) \quad (16)$$

Boundary Conditions:

By looking at the discretized equations presented in the predicted and corrected steps (Eqs. 11 to 14), it is clear that the variables can only be obtained for the internal nodes by solving the continuity and the momentum equations. Therefore, the flow variables at the boundaries are obtained by the application of boundary conditions. It is assumed that the disturbance due to the dam-break has not reached the extreme ends.

$$h_1 = h_u \quad \text{for all } t \quad (17)$$

$$h_2 = h_u \quad \text{for all } t \quad (18)$$

$$h_{ilast-1} = h_d \quad \text{for all } t \quad (19)$$

$$h_{ilast} = h_d \quad \text{for all } t \quad (20)$$

$$u_1 = u_u \quad \text{for all } t \quad (21)$$

$$u_2 = u_u \quad \text{for all } t \quad (22)$$

$$u_{ilast-1} = u_d \quad \text{for all } t \quad (23)$$

$$u_{ilast} = u_d \quad \text{for all } t \quad (24)$$

The purpose of the study here is to verify that if the use of Boussinesq equations give any significantly different results compared to the Saint-Venant equations. Therefore, this simple boundary condition is used to verify the results for a simplified case. However, in reality, the actual time history of the flow both at the down-stream and the up-stream ends should be prescribed as the boundary conditions. The usual practice is to prescribe the inflow hydrograph at the upstream end and the rating curve at the downstream end.

ARTIFICIAL VISCOSITY:

Dam-break flows are associated with wave fronts. Therefore, a sharp gradient in the water surface profile takes place. Due to the numerical approximation, oscillations *in the computed parameters* may occur near the sharp gradients. To dampen these high frequency oscillations, the concept of artificial viscosity (Jameson et al. 1981) is used here. A parameter ν_i is first computed using the computed flow depths at time level $t+\Delta t$;

$$\nu_i = \frac{|h_{i+1} - 2h_i + h_{i-1}|}{|h_{i+1}| + 2|h_i| + |h_{i-1}|} \quad \textbf{(25)}$$

$$\varepsilon_{i+1/2} = \kappa \, max(\nu_{i+1}, \nu_i) \quad \textbf{(26)}$$

κ is the dissipation coefficient and regulates the amount of dissipation. The computed values of the flow variables (h_i and u_i) already obtained from the final of the unsteady flow computation are modified as ;

$$hm^{t+\Delta t} = h^{t+\Delta t} + \varepsilon_{i+1/2}(h^{t+\Delta t} - h^{t+\Delta t}) - \varepsilon_{i-1/2}(h^{t+\Delta t} - h^{t+\Delta t}) \quad \textbf{(27)}$$
$$um^{t+\Delta t} = u^{t+\Delta t} + \varepsilon_{i+1/2}(u^{t+\Delta t} - u^{t+\Delta t}) - \varepsilon_{i-1/2}(u^{t+\Delta t} - u^{t+\Delta t}) \quad \textbf{(28)}$$

The modified values are then reset as old values for the next time step and the procedure is repeated till the desired time level.

4.2 LIMITATIONS

The numerical method described above has a number of limitations. Before using such model for any specific case the following points should be borne in mind.

(1)*Dam-break is assumed to be instantaneous.* In reality, a dam breaks gradually. The breaking of a dam in a short duration causes heavy flooding in the downstream side. Therefore, this assumption results in overestimating the flooding. However, it is in the safe side.

(2)*Channel bed is rigid.* All natural channel beds are mobile and sediment transport must be considered in the flow analysis. Moreover, the sudden release of heavy discharge in a dam break flow situation may erode the dam material ,bed ,or bank just down stream of the dam and the eroded material may get deposited at a downstream location. Thus the constant bed slope is not maintained.

(3)*Channel cross-section is wide rectangular and flow is one-dimensional.* Any river cross section will be unsymmetrical and irregular. Considering the width, the dam site will be the narrowest. Therefore, a correct approach would be to consider a converging channel connected to a diverging channel. The flow in the flood plain area is important in DBF. The DBF is always three-dimensional and is associated with turbulence.

(4) *Channel bed roughness is constant.* Bed roughness value is different for different locations. For the same location, it may also change depending on the flow depth and discharge.

All these limitations will not affect the object of need for considering non hydrostatic pressure distribution. in dam break flows.

5.0 RESULTS AND DISCUSSION

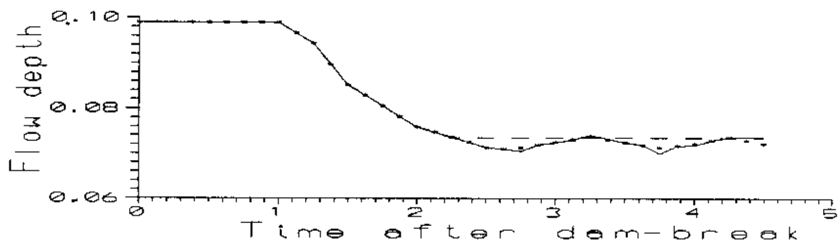
The numerical algorithm presented in the previous section is now used for simulating and analysing the DBF caused by the instantaneous rupture of the dam shown schematically in Fig.1(a). Validation of the present model is presented in the following subsection.

5.1 VALIDATION

Validation of the mathematical model is an essential part of a model study. In the present computational model, the non-linear partial differential equations of flow, Boussinesq equations, are solved by a finite-difference approach. The results are compared with previous experimental results presented by Carmo et al. (1993b). Fourier analysis of the above results is also presented. The performance of the present model is compared with that of a FEM model originally developed by Palaniappan (1981) for estuarine condition and modified for the present problem.

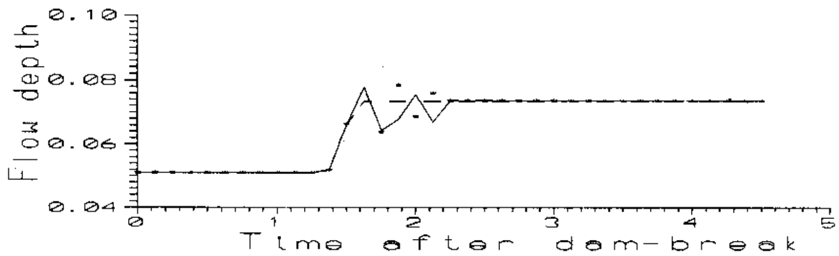
5.1.1 Comparison with experimental data

A set of measured data is available in the Hydraulics Laboratory of the Dept. of Civil Engineering, University of Coimbra (Carmo et al. 1993b). A 7.5m long by 0.30m wide horizontal rectangular channel was used. The dam was located at a distance of 3.85 m from the inlet of the channel. The instantaneous dam-break was performed by the opening of the glass sluice gate within a short duration (about 0.5 s). Three depth gauges were used, located at 2.65m, 5.25m and 6.25m, from the inlet section of the channel, respectively. Results are presented for the depth ratio (a ratio of channel water level to reservoir water level = h_d/h_u) of 0.515 with h_u=0.099 *m* and h_d=0.051 *m*. Results from the present model and the experimental studies are shown in Fig. 4. In this figure, flow depth at the gauging stations, as a function of time, is presented. Also the results obtained by Saint-Venant equations in this figure are presented. For gauging station 1, which is at an upstream section of the dam, the initial flow depth, h_u, is gradually decreased to the constatnt value, h_b, flow depth of the bore. In case of the downstream gauging stations (Stations 2 and 3), the initial value h_d is gradually increased to the constant value, h_b. It is clear from figure 4 that the oscillations in the water surface profile are dissipated in case of the Saint-Venant equations. The flow depths at the stations do not show oscillations. This is due to the assumptions used in the Saint-Venant equations. However, Boussinesq equations give results comparable with the experimental values. The amplitude of the first wave is always the maximum in case of the present model, where as the amplitude of the second wave is the maximum for the third station, only in case of the experimental values. There is a slight phase difference between the observed and the simulated results. However, the maximum value of the wave height is predicted satisfactorily by the present model. Also the speed of the wave is simulated correctly. The above characteristics are not simulated by the Saint-Venant equations. This clearly shows the limitations of the hydrostatic pressure distribution.



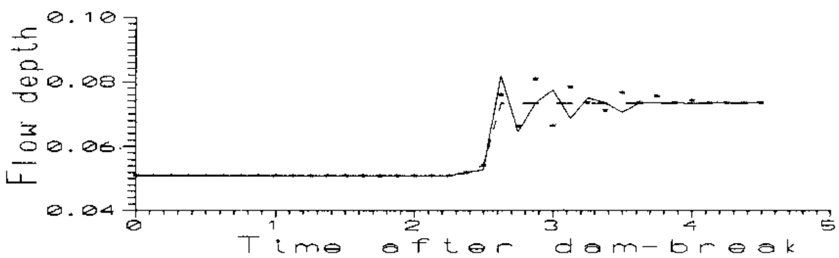
Non-hydrostatic * Experimental --- Hydrostatic

(a)Flow depth vs. time (Gauging station 1)



Non-hydrostatic * Experimental --- Hydrostatic

(b)Flow depth vs. time (Gauging station 2)



Non-hydrostatic * Experimental --- Hydrostatic

(c)Flow depth vs. time (Gauging station 3)

Fig. 4 Comparison of present model with experimental data

5.1.2 Comparison with FEM

A Finite Element Model was developed for the solution of Boussinesq equations by Palaniappan (1980). This model was run for the following input values. L_r=100 *m*, L_c=100 *m*, h_u=1.0 *m*, h_d=0.2 *m*, *t*=10 *s*. The results for these input parameters using the FDM model and the FEM model (Fig. 5) shows the free surface profile after the end of the computation time. There is not much difference between the results obtained from the two models. Although the water surface obtained by the FEM model seems more smooth, the development of the bore region (see zone 3 in Fig. 1b) in this model is not proper. Between the FDM and the FEM, there is a choice of smooth solution can be made. The FEM model, where Semi Discrete Method (SDM) is used, largely depends on the time stepping up scheme adopted. However, the FDM model runs smoothly for a higher value of time and a well-developed zone 3 can be obtained (Result not presented in this figure, it can be seen from subsequent figures).

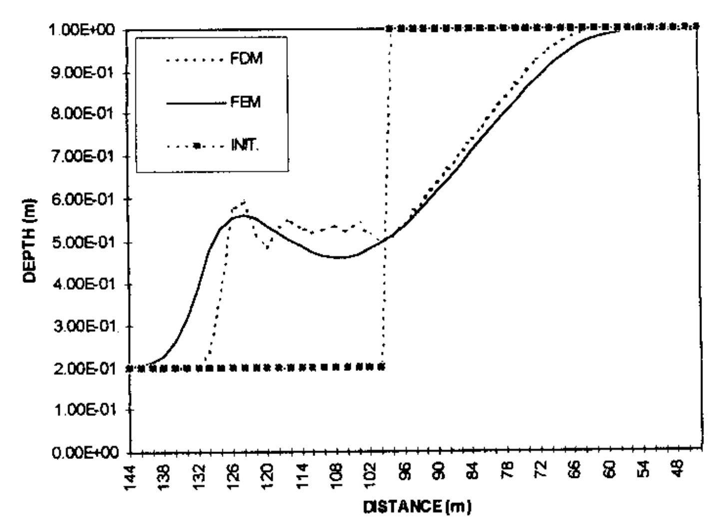


FIG. 5 Comparison of FDM &FEM for the case of dam break wave (r=.0.2), at the end of 10sec.

5.2 EFFECT OF NON-HYDROSTATIC PRESSURE DISTRIBUTION

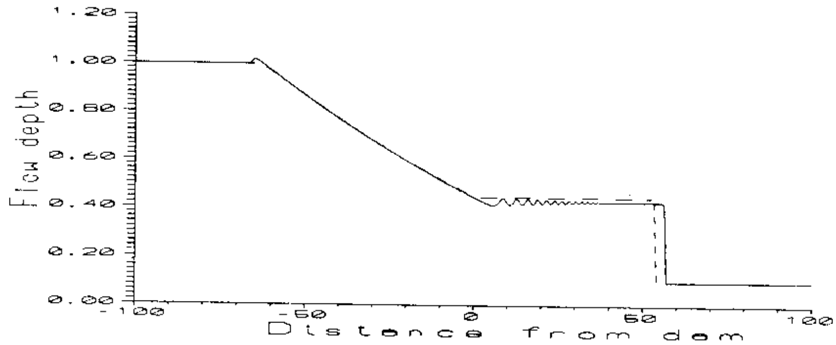
Mechanics of DBF for both wet and dry bed conditions is analyzed based on numerical simulations of the Boussinesq equations. Emphasis is laid on simulating the free surface profile, the wave front velocity and capturing the time evolution of the flow depth at the dam-site. The numerical results using the Saint Venant equations are also presented. The effect of bed roughness and bed slope is also studied.

The input parameters for the present computational model are; L_r=100 *m*, L_c=100 *m*, x_{dam}=0.0 *m*, *t*=20 *s*, *g*=9.81 m/s^2, C_n=0.6, h_u=1.0, h_d=0 to 0.5. Other input values are prescribed in the respective subsections.

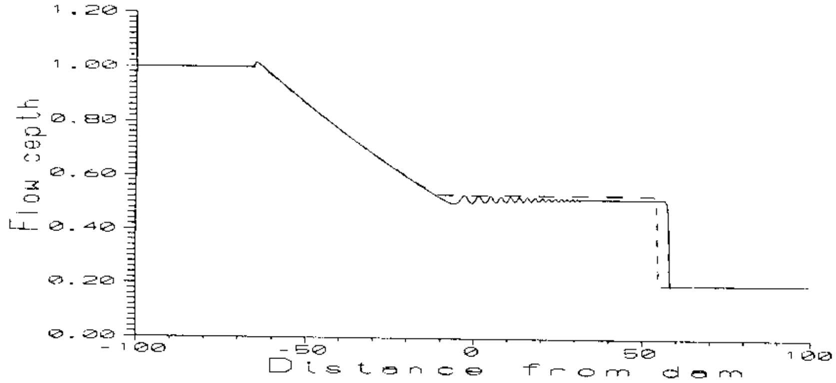
It is well known that the pressure distribution is non-hydrostatic immediately after the dam failure. However, shallow water wave models are commonly used for simulating the dam-break flows, thereby assuming that the initial non-hydrostatic state does not affect the long-term results. This underlying assumption on the applicability of shallow water wave models has not been rigorously tested earlier. On the other hand, some researchers (e.g., Kosorin 1983) contend that the initial two-dimensional effects have a bearing on the long-term results. In this section, we compare the numerical results for the long-term free surface profiles and the wave front velocity with the solutions obtained using the assumption on hydrostatic pressure distribution. In the following figures all quantities are shown in terms of non-dimensional parameters. The scales used for non-dimensionalisation are; Length scale = L = h_u, Velocity scale = V = $\sqrt{(g\ h_u)}$, Time scale = T = $\sqrt{(h_u/g)}$. Any quantity can be simplified to the respective non-dimensional quantity by dividing with the proper scale. For example, non-dimensional flow depth (h*) = actual flow depth (h)/Length scale (L).

5.2.1 Free Surface Profiles

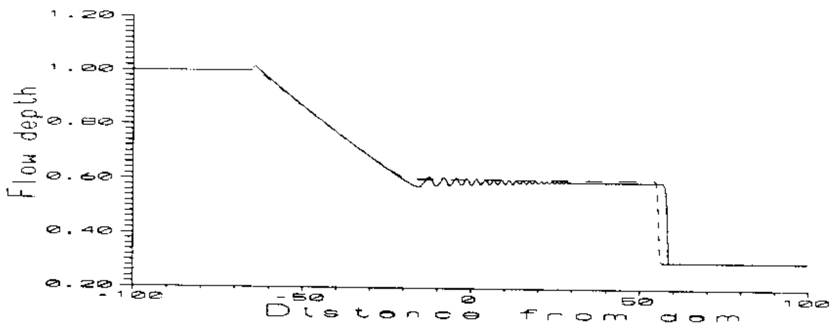
The free surface profiles for wet bed conditions at a non-dimensional time of 62.67 (refers to t=20s) are shown in Fig.6 for five different depth ratios, r = 0.1, 0.2, 0.3, 0.4, and 0.5, respectively. In all the figures, the corresponding result of the model using Saint-Venant equations (i.e. B = 0 in equation 2) are also shown. In these figures, *x -axis* represents the non-dimensional distance from the dam, and *y-axis* the non-dimensional flow depth. All the four zones (corresponding to Stoker solution) are clearly seen. It is clear from these figures, that a constant bore height is obtained by Saint Venant equations. In case of Boussinesq equations, there are oscillations in the transition of zone-2 and zone-3 (as also evident from experimental studies). These oscillations increase as the depth ratio increases. Also, the average bore height obtained by Boussinesq equations is marginally (1% to 2%) lower than that obtained by Saint-Venant equations for depth ratios, r<0.3. The waves may cause more area of inundation. For example, in case of r = 0.5, the highest value is 2% more than the average value. This could be dangerous if initially there is a very high water level in the reservoir. Thus, the highest value of the wave height, and not the average value, should be used for safety analysis. It is clear from the figures that Saint Venant equations cannot simulate the wavy nature and therefore, the effect of non-hydrostatic pressure should be accounted. However, considering the practical situation of dam-break flood, the depth ratio is less than 2, many of the times and a high value of depth ratio is very uncommon. Therefore, the use of Saint-Venant equations to analyse the dam-break flood is



(a) r=0.1, t=62.67



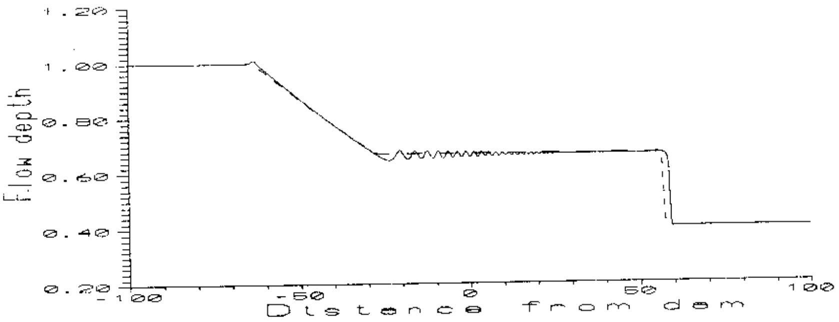
(b)r=0.2, t=62.67



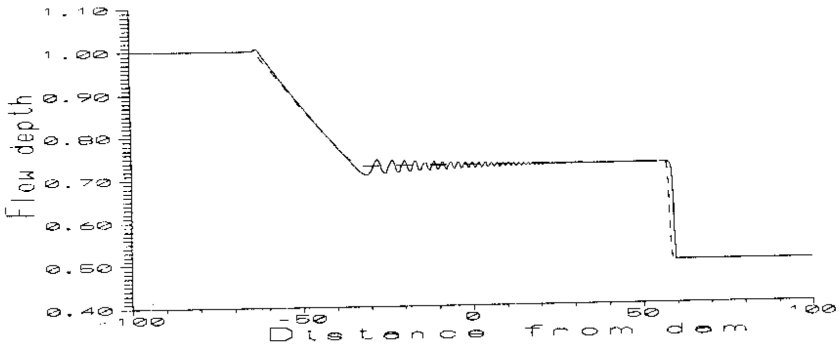
(c)r=0.3, t=62.67

Non-hydrostatic .... Hydrostatic

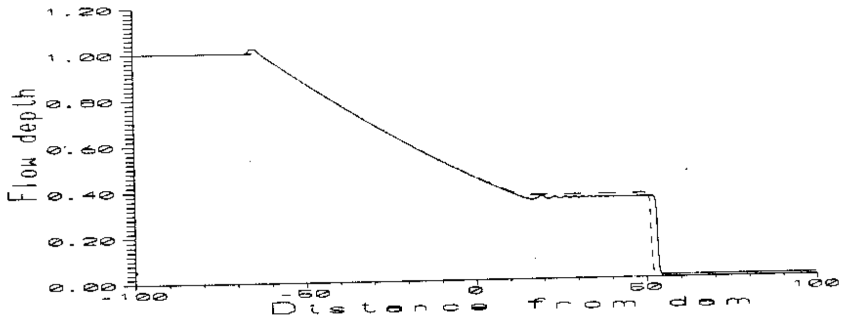
Fig 6 Effect of non-hydrostatic pressure on long-term surface profile



(d)r=0.4, t=62.67



(e)r=0.5, t=62.67



(f)r=0.01, t=62.6

Non-hydrostatic .... Hydrostatic

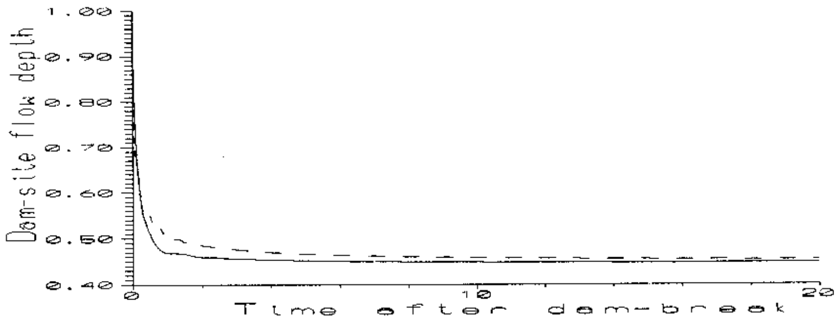
Fig. 6 Effect of non-hydrostatic pressure on long-term surface profiles (contd.)

justified. The distance traveled by the wave is an important parameter. In all the figures (Fig. 6), the distance traveled by the wave is marginally underestimated when predicted using Saint-Venant equations.

FLOW ON A DRY BED: In case of dam-break flood wave moving over a dry bed, the flood wave moves in the form of a tip and there will be no bore. The presence of the tip, which is mathematically singular, poses difficulties for numerical models. The correct initial conditions for this case would be $h_x = 0$ *for* $x > x_{dam}$. However, due to numerical difficulties this condition is not used in computer models and a small value is generally used. Although numerical models using Saint-Venant equations with second order accuracy can run for a very small value of the depth ratio, $r = 0.00001$, (Mohapatra and Bhallamudi 1996), the present model which use a third order scheme runs for $r = 0.001$ with Saint-Venant equations and $r = 0.01$ for Boussinesq equations. In case of Boussinesq equations, formation of undulations in the water surface makes the scheme unstable. In Fig. 6(f), the surface profile is shown for a depth ratio, r = 0.01. It shows a well developed bore region which is contrary to the general observation of a tip. Therefore, these results should be treated as results for r=0.01 and not for a dry bed condition. The present model cannot simulate the flood wave movement, due to dam-break, over a dry bed. This is one of the limitations of the present model.

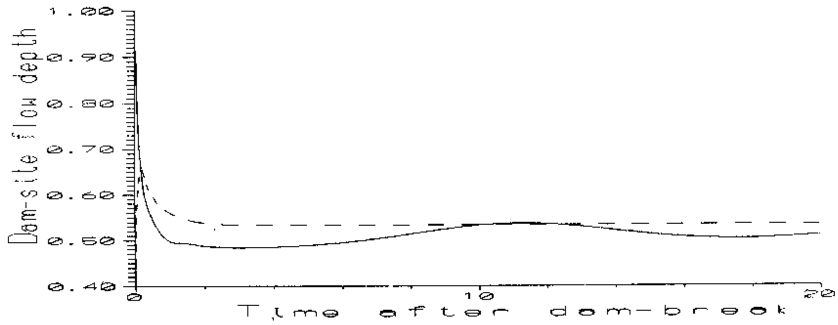
5.2.2 Evolution of Flow depth at dam-site

Ritter solution, which use the hydrostatic assumption, predicts that the flow depth at the dam-site attains a constant value (equal to 4/9 of the reservoir water level prior to dam-break, h_u) instantaneously after the dam-break. However, Dressler (1954) has shown experimentally that in case of dam-break flood wave on a dry bed, it takes 9 non-dimensional time units for the dam-site depth to attain this constant value. The variation with non-dimensional time after the dam-break, of the non-dimensional flow depth at the dam-site, obtained using the present numerical model is shown in Fig. 8. Results for all the five depth ratios are presented. The results obtained from the Boussinesq equations show oscillations. These oscillations die away after some period in case of small values of depth ratios. However, the oscillations are predominant for longer time in case of larger depth ratios. This result is consistent with the experimental results observed by Nakagawa et al. (Yevzevich 1975). (They observed the formation of undular bore for depth ratio, r > 0.4). This is due to the effect of non-hydrostatic pressure distribution. The average value for the flow depth at dam-site obtained using Boussinesq equations is less than the corresponding value obtained from Saint Venant equations. The lowering of the value is due to the effect of non-hydrostatic pressure distribution. Due to the non-hydrostatic pressure distribution, there will be development of vertical acceleration (vertical acceleration in the flow field obtained using Saint-Venant equation is zero). Some energy is used for this development and thereby resulting in a lower value of flow depth at the dam-site.



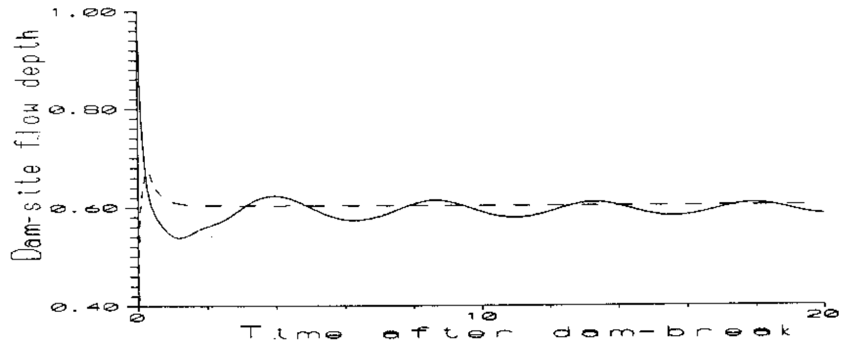
Non-hydrostatic --- Hydrostatic

(a)r=0.1



Non-hydrostatic --- Hydrostatic

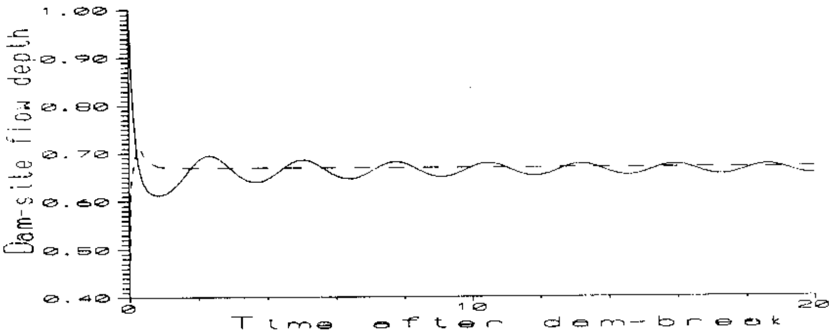
(b)r=0.2



Non-hydrostatic --- Hydrostatic

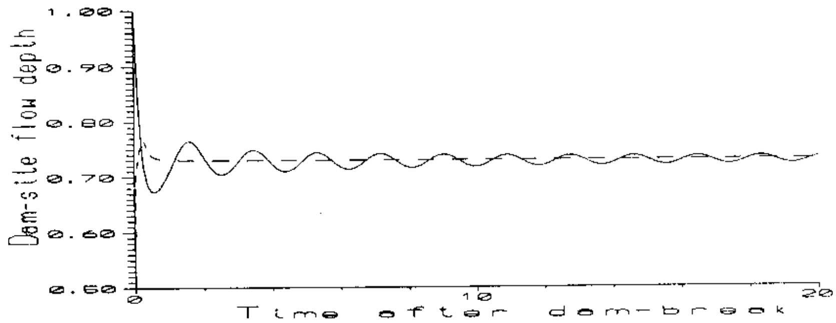
(c)r=0.3

Fig. 7 Evolution of flow depth at dam-site



Non-hydrostatic .... Hydrostatic

(d)r=0.4



(e) r= 0.5

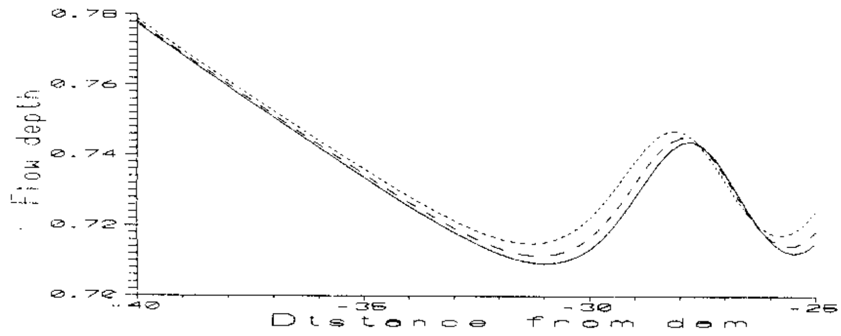
Non-hydrostatic .... Hydrostatic

Fig. 7 Evolution of flow depth at dam-site

5.3 EFFECT OF BED ROUGHNESS

Effect of bed roughness comes into the model through the computation of friction slope, S_f, using Manning equation (wide rectangular channel, R=depth of flow). All the figures presented above (Figs. 6 and 7) are with a zero value of bed roughness and a horizontal bed. The effect of roughness is studied for a depth ratio, $r=0.5$ (as the effect of non-hydrostatic pressure distribution is maximum for this case) and bed slope, S_b = 0.0001, by varying the Manning's coefficient for three different values (n = *0.01, 0.02, 0.03*). Result for the oscillations in the surface is presented in Fig. 8. No significant difference is found out either for the bore height or for the distance traveled by the bore. Only, a very marginal decrease in the bore height (0.6 %) and distance traveled by the bore (2 %) is observed. This is due to the fact that in case of dam-break flood wave movement, wave velocity is predominant and therefore, bed roughness has no significant effect on the flood wave movement at the initial stage. As the wave progresses, the effect of the non-hydrostatic pressure will be reduced, and, perhaps bed roughness will effect the flow.

Therefore, at the initial reaches of the dam-break flow, the non-hydrostatic pressure distribution is important but not the bed roughness.

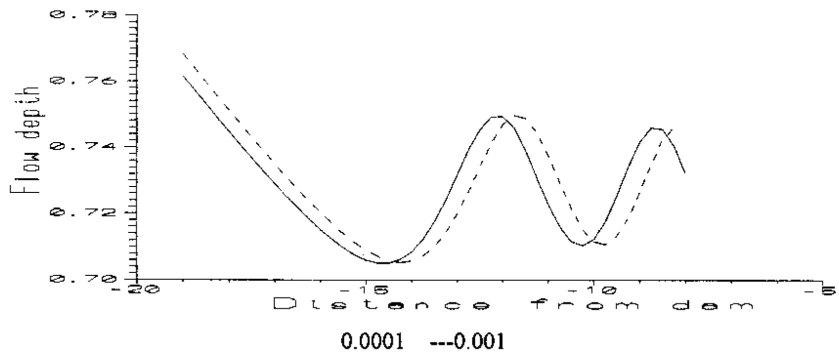


0.01 ---0.02 ...0.03

Fig. 8 Effect of bed roughness on surface profile (*r = 0.5, S_b = 0.0001, t = 60.67*)

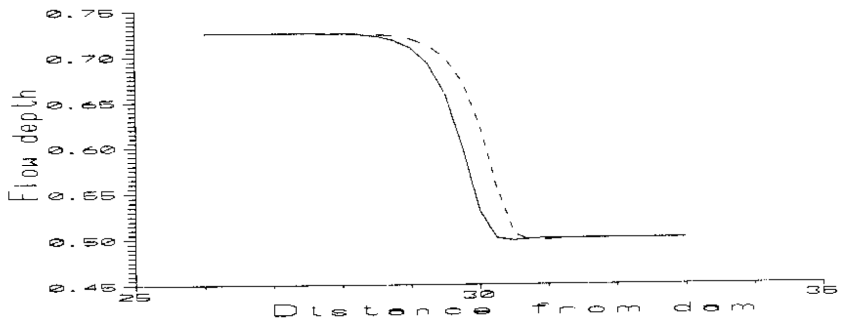
5.4 EFFECT OF BED SLOPE

The effect of the bed slope is studied by varying S_b for two different values (S_b = 0.0001 and S_b = 0.001). Other input values are r = 0.5, n = 0.03, t = 10. Results are presented in Figs. 9 and 10. In Fig.9, effect of bed slope on the oscillations is shown. The amplitude of the oscillations remains same but, there is a phase difference. The bore moves slightly faster when the bed slope is more. This is clearer in Fig. 10 where the distance traveled by the wave is shown. The bore height remains same and the bore velocity is only marginally more (3 %) when traveled on a bed with more slope. This may lead to a larger value of difference in the distance traveled by the bore when considered for a longer time.



0.0001 ---0.001

Fig.9 Effect of bed slope on oscillations



0.0001 ---0.001

Fig. 10 Effect of bed slope on bore speed

5.5 EFFECT OF INDIVIDUAL TERMS OF THE BOUSSINESQ EXPRESSION

Boussinesq equations are presented in section 3. Considering the momentum equation (Eq. 2) B is the expression introduced by non-hydrostatic pressure distribution. This contains three different terms. The individual effects of these terms are studied. The effect may be a complex function of distance, time, initial and the boundary conditions. Thus it is difficult to estimate the effects and generalise this. However, here the effects are presented only after the end of the computation time in Appendix -1. The input values used for the purpose are; $r = 0.5$, $n = 0$, $S_b = 0$, $t^* = 62.67$. In Appendix -1, the absolute values and the corresponding % value with respect to the values obtained using $B = 0$ are presented. The individual effects and the total effect are also presented. First column in the Appendix represents location, second, third, fourth and fifth columns represent the difference in flow depth due to individual terms and all the three terms, respectively. Other columns represent the % difference. The effect of the local acceleration is less compared to the total effects. Depending on the location, the second or the third term is important. Therefore, it is suggested that the first term may be neglected in the Boussinesq expression. It will reduce the cost of computation, though only to a marginal level.

6.0 CONCLUSION

In the present work, the effect of the non-hydrostatic pressure distribution on the dam-break flood wave movement was studied. For the purpose, the governing equations (Boussinesq equations) were solved numerically.

The main conclusions of the present study are:

1.Dam-break flood moves with non-hydrostatic pressure distribution in the vertical direction. Effect of non-hydrostatic pressure on dam-break flood wave movement cannot be estimated with the help of Saint-Venant equations.

2. The effect of non-hydrostatic pressure distribution on the dam-break flood wave movement is important only at the initial time period after the dam-break.

3. The effect of non-hydrostatic pressure as the wave progresses is negligible. The long-term bore height and bore velocity can also be computed to the degree of accuracy imoprtant to engineering applications with the help of less accurate Saint-Venant equations.

4. A non-hydrostatic pressure distribution results in oscillations in the free surface. These oscillations are imortant for dam-break floods with depth ratios greater than 0.4.

5. The evolution towards the constant value of flow depth at the dam-site shows the effect of non-hydrostatic pressure distribution only for a small time after the dam-break. However, the value is decreased due to this effect.

6. Initially, effect of bed roughness is insignificant for the movement of dam-break flood wave.

7. The bore velocity increases slightly due to the effect of bed slope.

8. Out of the three terms used in the Boussinesq expression, first term (consisting of time derivative) is unimportant.

The following works are recommended for future studies.

1.Inclusion of curvilinear coordinate system/ two-dimensional rectangular coordinate system in the governing equations to consider the effect of two-dimensinal flow.

2.Application of a fully coupled model using sediment transport equations.

3.Effect of partial dam-breaching.

4.Dam-break flood wave movement on a dry bed.

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Appendix - 1

st.	B_1	B_2	B_3	total	B_1 (%)	B_2 (%)	B_3 (%)	total (%)
0.00	0.00	-0.01	-0.00	-0.01	0.00	-1.33	-0.67	-2.00
9.75	0.00	-0.01	-0.00	-0.01	0.00	-1.14	-0.67	-1.80
-9.50	0.00	-0.00	-0.00	-0.01	0.00	-0.63	-0.66	-1.29
-9.25	0.00	0.00	-0.00	-0.00	0.00	0.02	-0.66	-0.63
-9.00	0.00	0.00	-0.00	-0.00	0.00	0.56	-0.66	-0.10
-8.75	0.00	0.01	-0.00	0.00	0.00	0.75	-0.66	0.10
-8.50	0.00	0.00	-0.00	-0.00	0.00	0.51	-0.66	-0.14
-8.25	0.00	- 0.00	-0.00	-0.01	0.00	-0.05	-0.66	-0.70
-8.00	0.00	-0.01	-0.00	-0.01	0.00	-0.69	-0.66	-1.34
-7.75	0.00	-0.01	-0.00	-0.01	0.00	-1.14	-0.66	-1.80
-7.50	0.00	-0.01	-0.00	-0.01	0.00	-1.24	-0.66	-1.89
-7.25	0.00	-0.01	-0.00	-0.01	0.00	-0.93	-0.66	-1.59
-7.00	0.00	-0.00	-0.00	-0.01	0.00	-0.35	-0.66	-1.00
-6.75	0.00	0.00	-0.00	-0.00	0.00	0.27	-0.66	-0.38
-6.50	0.00	0.00	-0.00	-0.00	0.00	0.64	-0.66	-0.01
-6.25	0.00	0.00	-0.00	-0.00	0.00	0.59	-0.66	-0.06
-6.00	0.00	0.00	-0.00	-0.00	0.00	0.14	-0.66	-0.51
-5.75	0.00	-0.00	-0.00	-0.01	0.00	-0.48	-0.66	-1.14
-5.50	0.00	-0.01	-0.00	-0.01	0.00	-1.00	-0.66	-1.66
-5.25	0.00	-0.01	-0.00	-0.01	0.00	-1.18	-0.66	-1.84
-5.00	0.00	-0.01	-0.00	-0.01	0.00	-0.94	-0.66	-1.60
-4.75	0.00	-0.00	-0.00	-0.01	0.00	-0.39	-0.67	-1.05
-4.50	0.00	0.00	-0.00	-0.00	0.00	0.22	-0.67	-0.44
-4.25	0.00	0.00	-0.00	-0.00	0.00	0.58	-0.67	-0.08

-4.00	0.00	0.00	-0.00	-0.00	0.00	0.51	-0.67	-0.15
-3.75	0.00	0.00	-0.00	-0.00	0.00	0.05	-0.67	-0.62
-3.50	0.00	- 0.00	-0.00	-0.01	0.00	-0.56	-0.67	-1.23
-3.25	0.00	- 0.01	-0.00	-0.01	0.00	-1.02	-0.67	-1.68
-3.00	0.00	- 0.01	-0.00	-0.01	0.00	-1.09	-0.67	-1.75
-2.75	0.00	- 0.01	-0.00	-0.01	0.00	-0.74	-0.67	-1.41
-2.50	0.00	- 0.00	-0.00	-0.01	0.00	-0.15	-0.67	-0.82
-2.25	0.00	0.00	-0.00	-0.00	0.00	0.37	-0.67	-0.29
-2.00	0.00	0.00	-0.00	-0.00	0.00	0.55	-0.67	-0.12
-1.75	0.00	0.00	-0.00	-0.00	0.00	0.27	-0.67	-0.40
-1.50	0.00	- 0.00	-0.00	-0.01	0.00	-0.30	-0.67	-0.96
-1.25	0.00	-0.01	-0.00	-0.01	0.00	-0.83	-0.67	-1.50
-1.00	0.00	-0.01	-0.00	-0.01	0.00	-1.05	-0.67	-1.71
-0.75	0.00	-0.01	-0.00	-0.01	0.00	-0.83	-0.67	-1.50
-0.50	0.00	-0.00	-0.00	-0.01	0.00	-0.30	-0.67	-0.96
-0.25	0.00	0.00	-0.00	-0.00	0.00	0.25	-0.67	-0.41
0.00	0.00	0.00	-0.00	-0.00	0.00	0.49	-0.67	-0.17
0.25	0.00	0.00	-0.00	-0.00	0.00	0.28	-0.67	-0.38
0.50	0.00	-0.00	-0.00	-0.01	0.00	-0.26	-0.67	-0.92
0.75	0.00	-0.01	-0.00	-0.01	0.00	-0.78	-0.66	-1.44
1.00	0.00	-0.01	-0.00	-0.01	0.00	-0.99	-0.66	-1.65
1.25	0.00	-0.01	-0.00	-0.01	0.00	-0.76	-0.66	-1.43
1.50	0.00	-0.00	-0.00	-0.01	0.00	-0.23	-0.66	-0.89
1.75	0.00	0.00	-0.00	-0.00	0.00	0.27	-0.66	-0.39
2.00	0.00	0.00	-0.00	-0.00	0.00	0.43	-0.66	-0.23
2.25	0.00	0.00	-0.00	-0.00	0.00	0.14	-0.66	-0.52
2.50	0.00	-0.00	-0.00	-0.01	0.00	-0.41	-0.66	-1.06
2.75	0.00	-0.01	-0.00	-0.01	0.00	-0.85	-0.66	-1.50

3.00	0.00	-0.01	-0.00	-0.01	0.00	-0.91	-0.66	-1.56
3.25	0.00	-0.00	-0.00	-0.01	0.00	-0.55	-0.66	-1.21
3.50	0.00	-0.00	-0.00	-0.00	0.00	-0.00	-0.66	-0.66
3.75	0.00	0.00	-0.00	-0.00	0.00	0.36	-0.66	-0.30
4.00	0.00	0.00	-0.00	-0.00	0.00	0.29	-0.66	-0.37
4.25	0.00	-0.00	-0.00	-0.01	0.00	-0.16	-0.66	-0.82
4.50	0.00	-0.00	-0.00	-0.01	0.00	-0.67	-0.66	-1.33
4.75	0.00	-0.01	-0.00	-0.01	0.00	-0.89	-0.66	-1.54
5.00	0.00	-0.00	-0.00	-0.01	0.00	-0.66	-0.66	-1.32
5.25	0.00	-0.00	-0.00	-0.01	0.00	-0.16	-0.66	-0.82
5.50	0.00	0.00	-0.00	-0.00	0.00	0.27	-0.66	-0.39
5.75	0.00	0.00	-0.00	-0.00	0.00	0.30	-0.66	-0.36
6.00	0.00	-0.00	-0.00	-0.01	0.00	-0.09	-0.66	-0.75
6.25	0.00	-0.00	-0.00	-0.01	0.00	-0.59	-0.66	-1.25
6.50	0.00	-0.01	-0.00	-0.01	0.00	-0.84	-0.66	-1.50
6.75	0.00	-0.00	-0.00	-0.01	0.00	-0.65	-0.66	-1.31
7.00	0.00	-0.00	-0.00	-0.01	0.00	-0.17	-0.66	-0.82
7.25	0.00	0.00	-0.00	-0.00	0.00	0.23	-0.66	-0.42
7.50	0.00	0.00	-0.00	-0.00	0.00	0.24	-0.66	-0.42
7.75	0.00	-0.00	-0.00	-0.01	0.00	-0.14	-0.66	-0.80
8.00	0.00	-0.00	-0.00	-0.01	0.00	-0.63	-0.66	-1.28
8.25	0.00	-0.01	-0.00	-0.01	0.00	-0.80	-0.66	-1.45
8.50	0.00	-0.00	-0.00	-0.01	0.00	-0.53	-0.66	-1.19
8.75	0.00	-0.00	-0.00	-0.01	0.00	-0.06	-0.66	-0.72
9.00	0.00	0.00	-0.00	-0.00	0.00	0.25	-0.66	-0.41
9.25	0.00	0.00	-0.00	-0.00	0.00	0.12	-0.66	-0.54
9.50	0.00	-0.00	-0.00	-0.01	0.00	-0.34	-0.66	-0.99
9.75	0.00	-0.01	-0.00	-0.01	0.00	-0.71	-0.66	-1.37

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