Dam-break floods downstream of Papadia dam

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Abstract

The propagation of a flood resulting from a dam failure has been investigated by several engineers over the last decades. Many algorithms have been developed for the solution of the Saint-Venant equations, governing the flow propagating downstairs in natural channels, after a dam failure. The algorithms are based on characteristic equations, finite difference equations, and finite element equations and describe the flood propagation through the channels. As the breach formation’s mechanics are not well understood in previous attempts to predict flood waves downstream, it was assumed that the dam failure was complete and instantaneous Only over the last years has the breach formation of a dam been the subject of engineers’ research and the breach was assumed to develop over a finite interval of time.

In this paper the dam breach formation of Papadia dam (North Greece) has been investigated, and the propagation of the flood through the stream and the valley downstream is described. A rather empirical model due to Fread predicting the time of the dam failure and the geometric breach evolution of the dam is considered. For routing dam-break flood waves a particular hydraulic algorithm is chosen. The geometric and physical characteristics of the stream and valley downstream for a distance of 30 km have been chosen from maps and by measurement in situ. Different scenarios concerning the Papadia dam failure have been considered and for each of them flood hydrographs of water depth and discharge have been calculated.

1 Introduction

Dam breach is one of the most important problems in a dam’s history, because a productive and creative construction is destroyed without any possibility of
direct restoration and also destructive consequences are projected in human lives and huge material losses are created downstream. Actually, when a dam fails or is deliberately demolished, large quantities of water are suddenly released, creating major flood waves capable of causing disastrous damage to downstream civil and military installations. These damages and losses could constitute a national disaster and adversely affect a nation’s economic, social or war effort. Singh [6] has noted 1000 dam failures since the 12th century, and about 200 have occurred in the 20th century so far causing a loss of more than 8000 lives and damage worth millions of dollars. The annual probability of dam failure is estimated at $10^{-4}$ and the probability of failure during the lifetime of a dam (100 years) is $10^{-3}$.

The actual failure mechanics of a dam are not well understood for either earthfill or concrete dams. It should be mentioned that until 1973 engineers’ research assumed that the breach occurs instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch dams, but they are not appropriate for earthfill and concrete gravity dams. In our days earthfill dams are exceedingly outnumber all other types of dams and for this reason we study this kind of dams.

Earthfill dams do not tend to completely fail, nor do their fail instantaneously. The fully formed breach in earthfill dams tends to have an average width ($\bar{b}$) in the range: $h_d < \bar{b} < 3h_d$, where $h_d$ is the height of the dam (Freda [3]). Total time of failure may be in the range of a few minutes to usually less than an hour, depending on: a) the height of the dam, b) the type of materials used in construction, c) the extent of compaction of the materials, d) the magnitude and duration of the overtopping flow of the escaping water.

The time of failure as used in the model is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasses has progressed back across the width of the dam crest to reach the upstream face.

Froelich [4] used the properties of 43 breaches of dams ranging in height from 15 to 285 ft (5 to 87 m approximately) and he obtained the following predictive equations:

$$\bar{b} = 9.5k_0(V_t h_d)^{0.25},$$  \hspace{1cm} (1)

$$\tau = 0.8 \left( \frac{V_t}{h_d^2} \right)^{0.5},$$  \hspace{1cm} (2)

where: $\bar{b}$ = the average breach width (ft), $\tau$ = the time of failure (hrs), $k_0$= 1.0 for piping and 1.4 for overtopping, $V_t$ = the reservoir volume (acre-ft), $h_d$ = the height of water over the breach bottom (usually about the height of the dam) (ft).
Another way of checking the reasonableness of the breach parameters ($\bar{b}$ and $\tau$) is to use the following equations:

$$Q_p^* = 370(\sqrt{V_r h_d})^{0.5},$$  \hspace{1cm} (3)

$$Q_p = 3.1\bar{b}\left[\frac{C}{\tau + C/\sqrt{h_d}}\right]^3,$$ \hspace{1cm} (4)

$$C = 23.4A_s/\bar{b},$$ \hspace{1cm} (5)

where: $Q_p^*$ = the expected peak discharge (cfs) through the breach, $Q_p$ = the expected peak discharge (cfs) through the breach, $V_r$ = the reservoir volume (acre-ft), $A_s$ = the reservoir surface area (acres) at the top of the dam.

Eqn (3) was developed by Hagen [5], based on historical data from 14 dam failures and provides a maximum envelope of all 14 of the observed discharges. Eqns (4) and (5) were developed by Fread [2] and are used in the NWS Simplified Dam Break Model, SMPDBK [7].

After selected $\bar{b}$ and $\tau$, eqn (4) can be used to compute $Q_p$, which then can be compared with $Q_p^*$ computed from eqn (3). Thus, if $Q_p > Q_p^*$ then either $\bar{b}$ is too large or $\tau$ is too small. However if $Q_p < Q_p^*$ then either $\bar{b}$ is too small or $\tau$ is too large. Fread [2] has found that eqn (3) over-estimated the peak discharges by an average of 130 percent.

![Figure 1: Research area.](image)

In this paper it was considered: a) the breach of Papadia's dam which will be constructed by National Electric Company and b) the propagation of flood waves downstream at a distance of about 30 km from the dam. The research program took place at the Laboratory of Agricultural Hydraulic in A.U.Th. Papadia's dam
is located on the Papadia river in the West Macedonia region (figure 1) and is classified as an earthfill dam. The height of the dam from foundation is 72 m, the length of the dam crest is 538 m and its width 12 m. The formed reservoir will cover upstream of the dam an area of approximately 600,000 m² (overflow level) and will store 13,964,000 m³ of water with usable stored quantity of 13,100,000 m³.

2 Mathematical model – boundary conditions

Equations describing the propagation of flood waves downstream of a dam due to its failure have been classified as a) kinematic wave routing, b) diffusion wave routing and c) dynamic wave routing. Both the kinematic wave model and the diffusion wave model are helpful in describing downstream wave propagation when the channel slope is greater than about 0.01 percent and there are no waves propagation upstream due to disturbances such as tides, tributary inflows or reservoir operation. When both inertial and pressure forces are important, such as in mild-sloped rivers and backwater effects from downstream disturbances are not negligible, then both the inertial force and pressure force terms in the momentum equation are needed. Under these circumstances the dynamic wave routing method is required, which involves numerical solution of the full Saint-Venant equations. Equations describing the propagation of flood waves at the last case, are the equations of Saint-Venant, expressed in conservation form:

I. Continuity equation:

\[
\frac{\partial Q}{\partial x} + \frac{\partial}{\partial t} \left[ s \left( C_A + A_0 \right) \right] = 0
\]  \hspace{1cm} (6)

II. Momentum equation:

\[
\frac{\partial \left( s_m Q \right)}{\partial t} + \frac{\partial \left( \beta Q^2 / A \right)}{\partial x} + gA \left( \frac{\partial h}{\partial x} + S_f + S_e + S_i + L' = 0 \right)
\]  \hspace{1cm} (7)

where: Q = the river discharge, A = the active cross-sectional area of flow, A₀ = the inactive cross-sectional area of flow, q = the lateral inflow or outflow per linear distance along the channel, s_c, s_m = sinuosity factors, β = the momentum coefficient for velocity distribution, g = the acceleration due to gravity, h = the water surface elevation, S_f = the boundary friction slope, S_e = the expansion – contraction slope, S_i = the additional friction slope associated with internal viscous dissipation of non-Newtonian fluids, L' = the momentum effect of lateral flow assumed herein to enter or exit perpendicular to the direction of the main flow, x = the longitudinal distance along the channel, t = the time.
The above continuity and momentum equation can be expressed in finite difference form, as follows:

I. Continuity equation:

\[
\theta \left( \frac{Q_i^{j+1} - Q_i^{j+1}}{\Delta x_i} \right) + (1 - \theta) \left( \frac{Q_i^j - Q_i^j}{\Delta x_i} \right) + \frac{s_{c_i}^{j+1} (A + A_0)_i^{j+1} + s_{c_i}^{j+1} (A + A_0)_i^{j+1} - s_{c_i}^j (A + A_0)_i^j - s_{c_i}^j (A + A_0)_i^j}{2\Delta t} = 0
\]  

II. Momentum equation:

\[
\left( \frac{\varepsilon_{m_i} \dot{Q}_i}{\Delta t} \right)^{j+1} + \left( \varepsilon_{m_i} \dot{Q}_i \right)^{j+1} - \left( \varepsilon_{m_i} \dot{Q}_i \right)^{j} + 2\Delta t \left[ \frac{\beta Q^2 / A}{\Delta x_i} \right] + g \bar{A} \left\{ h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_i^{j+1} + \bar{S}_i^{j+1} - \bar{S}_i^{j+1} - \bar{S}_i^{j+1} \right\} - L^{j+1} = 0
\]

where \(0 \leq \theta \leq 1\).

For the above case the boundary conditions are as follows:

**Upstream boundary.**

The upstream boundary is required to obtain a solution of the Saint – Venant equations. In most applications is simply a specified discharged hydrograph, i.e. \(Q_1 = Ql(i)\), which is used at first upstream cross section combined with dam breach.

**Downstream boundary.**

When the flow near the downstream extremity of the routing reach is subcritical, i.e. \(Fr_N = V_N / \sqrt{gA_N / B_N} < 1\), (where \(N\) designates the number of the most downstream cross section), a known relationship between flow and depth or depth and time must be specified. Depending on the physical characteristics of the downstream section, the model allows the appropriate specification of one of the following downstream boundary equations:

- **Single value rating:**

  \(Q_i = Q(h)\)

  in which \(Q(h)\) represents a specified tabular relation of \(Q\) and \(h\), and \(i = N\).

- **Generated dynamic loop-rating:**

  \(Q_i = 1.49 \frac{A_i R_i^{2/3}}{n_i} S_i^{1/2} = K_i S_i^{1/2},\)
where:

\[ S = \frac{(h_{i-1} - h_i)}{\Delta x_{i-1}} + \frac{(Q_i' - Q_i)}{0.5g(A_i + A_{i+1})\Delta t} + \frac{(Q_i'^2/A_{i-1} - Q_i^2/A_i)}{0.5g(A_i + A_{i-1})\Delta x_{i-1}} \]

- Critical flow rating:
  \[ Q_i = \sqrt{gA_i^{3/2}/B_i^{1/2}} \]
- Water level time series:
  \[ h_i = h(t) \] in which \( h(t) \) represents a specified time series of water elevation versus time.

3 Finite difference numerical model

The above system of nonlinear equations with boundary and initial flow conditions can be expressed in functional form in terms of the unknown \( h \) (water surface elevation) and \( Q \) (discharge) at time level \( j+1 \), as follows:

\( C_i(h_i, Q_i, h_{i+1}, Q_{i+1}) = 0 \)  continuity eq. for grid \( i \).

\( M_i(h_i, Q_i, h_{i+1}, Q_{i+1}) = 0 \)  momentum eq. for grid \( i \).

This system \( (2N \times 2N) \) is solved for each time step by the Newton-Raphson iterative method by assigning trial values to the \( 2N \) unknowns and with a special technique developed by Fried [1].

4 Application

Three scenarios dealing with different ways of dam-break were investigated, as follows:

\[ \text{I} : \text{"Dam failure due to extreme floods."} \]

\[ \text{II} : \text{"Dam failure due to flood hydrograph and concurrent extreme inflow in Tripotamos area."} \]

\[ \text{III} : \text{"Dam break due to man-made destructive actions."} \]

This paper describes only the first scenario due to space limitations. At scenario I dam failure and flood wave routing is studied for the case when the cause of failure is the overtopping of water over the dam crest. Dam failure is considered to start when the level of water in the reservoir is at the same level with the dam crest and the maximum probable flood routing has occurred at the same time. The maximum flow of hydrograph is 669 m³/s and takes place at the dam for time \( t = 0 \). The spillway of the dam has the ability to divert the above peak discharge, so dam breach cannot occur due to this effect. A small increase of this quantity, since the dam is in borderline condition, can start dam breach.

The time evolution of the dam breach from the beginning of the destruction till the final condition is computed as 0.5 hr with breach foundation width \( b = 126.5 \text{ m} \). The lower point of break has absolute altitude +884.8 m when the absolute altitude of dam foundation is +870.0 m. The breach stops at that level,
because the water volume remaining upstream of the dam, is too low for the phenomenon to be continued.

The routing of flood wave, which will be created after dam-break, is studied from the location of dam at Papadia village up to the borders. The length of the routing is computed as 31 km and the study time of routing is 3.2 hr. This time is sufficient for all the necessary data to be computed, such as the peak of flow elevation, the peak of discharges, and the peak of velocities (figures 2, 3, 4).

Along the routing of the flood wave, cross-sections in characteristic locations to the topography of the area are selected and studied. The order of magnitude of the flood wave was such that utilization of more detailed information (as bridges, buildings, etc.) was impractical.

**SCENARIO 1**

Figure 2: Peak hydrographs of each cross-section in the corresponding time of appearance after dam failure.

Figure 3: Peak discharges.

Figure 4: Peak velocities.
5 Conclusions

The maximum discharge, 15000 m³ s⁻¹, is observed in section Δ1 downstream of the dam, while the peak discharge which is transferred downstream is continuously decreasing, compared to the initial maximum discharge, obeying a rather exponential decreasing rate. The exponential decrease of the discharge as a function of distance travelled by the flood wave downstream, is shown in figure 3. It should be pointed out that the downstream region has smaller river slope and a higher potential for water leak, consequently flooding larger areas.

The occurring velocities of the flood wave vary in relation to the ground slope and the width of the riverbed. The velocity of the flood wave starts from zero at the point of the dam. Afterwards the velocity increases suddenly due to the river slope and the narrow width of the riverbed and finally decreases because of the smaller river slope and the width of the flood area. All these variations are shown in figure 4.

References