Proceedings of the 2006 IASME/WSEAS Int. Conf. on Water Resources, Hydraulics & Hydrology, Chalkida, Greece, May 11-13, 2006 (pp119-127)

Flooding due to Sequential Dam Breaking

C.D.MEMOS, A.I.STAMOU and M.G.POLITIS
School of Civil Engineering
National Technical University of Athens
Heroon Polytechneiou 5, 15780 Zografos
GREECE

Abstract: - Estimation of the hydraulics of the flood wave propagation due to multiple dam breaking is quite complex and realistic answers are difficult to be given by commercially available packages. Some of the complexities of the problem are discussed, including those related to flood routing through a reservoir. Suggestions to overcome the difficulties are given along with a real life application of the procedure proposed.

Key-Words: - Dam break, Flood routing, Reservoir routing, 2D flood propagation, Sequential dam-break.

1 Introduction

The growing concern about possible environmentally adverse impacts due to eventual failure of civil engineering projects, encompasses cases where dam breaches can release in short time enormous amount of water into natural watercourses. This could pose a serious threat to human life and property downstream of the failed dam. To assess the associated risk a detailed description of the hydraulics of the resulting flood wave is required. Many efforts were made during the past century or so to treat the problem of the shock wave formation due to a dam break. Pioneering works such as those of Ritter [33], Dressler [10], Stoker [39] were followed by more sophisticated mathematical approaches, such as those of Rajar [32], Sakkas and Strelkoff [35], Hunt [20], and further by studies involving detailed description in time of the formation of the dam breach, e.g. Broich [4], Paquier et al. [30]. The propagation of the resulting flood wave has also received extensive attention by researchers. The one-dimensional case is almost exclusively treated through solving the St Venant equations of gradually varied flow [34]. Following Stoker's numerical solution [39], a variety of numerical schemes were applied ranging from the classical method of characteristics (Abbott [1]) to finite difference methods, e.g. Cunge et al. [8] and finite element schemes, e.g. Fread [13], Szymkiewicz [42]. A crucial point in these treatments is the capturing of the shock discontinuity present through the early stages of the wave propagation. This can be better achieved by employing an integral approach, such as the modified Godunov technique (Godunov [17], Savic and Holly [36]). A further complication arises when the flood wave encounters during its propagation a reservoir of appreciable depth. Even if the geometry of the reservoir can be assumed as one-dimensional, the formulation of the problem involves now a vertical dimension that plays a significant role in determining the exact flow characteristics, especially at the initial stage of the plunging of the flood wave into the still reservoir water. There, if the flow retains still the characteristics of a shock wave, it cannot be regarded any longer as gradually varied and the St Venant equations may not be applicable in this region. In areas of horizontal expanse, such as in floodplains, two-dimensional treatment of the model equations is needed. A simple approach is based on the one-dimensional diffusive wave equation, e.g. Strelkoff et al. [40], Fread [13], Han and Park [18]. However, there are limitations in using the diffusive wave model related to the width of the flooded area (Moussa and Bocquillon [28]). Another approach is to solve the two-dimensional flow equations by finite difference methods as in Xanthopoulos and Koutitas [43], by the characteristics method (Katopodes and Strelkoff [22]), or by finite element methods, e.g. D'Alpaos et al. [9].

Across many rivers around the world, dams are built for hydropower generation, water supply, irrigation and other uses. Full exploitation of these capabilities leads to building several dams along the same watercourse. Studying, therefore, a dam break event in a situation with multiple "in-line" dams present, involves all above mentioned individual problems leading to a complex situation difficult to analyze and predict. The flow conditions can be either gradually or rapidly varying, in one or two horizontal –and at places two vertical– dimensions. Depending on the location of the failing dam with respect to the other dams of the river, as well as on the amount of water stored in the downstream reservoirs, the breach of the downstream dam(s) has to be estimated. The actual breach of the dam(s) is usually due to either overtopping or piping. It is evident through the above brief exposition that in order to evaluate the flooding, and the associated risk, due to sequential dam breaking, an array of possible scenarios should be developed. These should be provided with the appropriate interfacial modules between the various flow conditions, i.e. wave propagation, reservoir routing, dam breach, floodplain inundation. It is also noted that the input conditions should only be applied to the dam to fail initially, while the fate of the rest downstream dams has to be assessed depending on the conditions prevailing in each scenario considered. In this paper an attempt is made to combine solutions to all previously mentioned individual problems into a coherent approach, easy to apply. The study was initiated by the request of the Power Corporation of Greece to evaluate possible dam break events along a river in northern Greece, where five dams are located.

2 The Breach of a Dam

The dam breach models commonly used determine the following main characteristics of failure: (a) Failure time, t_F, (b) Breach position and dimensions as varied with time, (c) Dam-Break Hydrograph (DBH). The DBH is used as the upstream condition for the flood wave routing downstream of the dam site. The most important characteristics of the DBH are the peak flow, Q_P, and the time to peak flow, t_P. The corresponding characteristics of the associated stage hydrograph are equally important. The breach model to be used in each case depends primarily on the type of dam failure, e.g. overtopping, piping, structural failure.

There are various models in the literature dealing with the modeling of the breaching of earthen dams due to either overtopping or piping. A brief description with application of such models can be found in the proceedings of the Concerted Action on Dambreak Modelling (CADAM) project [6]. The CADAM project has been set in motion by the European Union to investigate current methods and their use for simulation and prediction of the effects of dam failures. The main conclusions of the proceedings of the CADAM project and other similar research works are the following:

(a) The accuracy of existing breach models, which are developed mainly for earthen dams, is very limited. The accuracy of predicting Q_P is estimated at ±50%, while that of t_F is considerably worse. Similar results have been found by Stamou et al. [38].

(b) The uncertainly of the whole dam break modelling process stems mainly from the uncertainly of the breach models.

It is this evident that there are aspects of the problem not yet fully answered and, therefore, research on these is still going on, e.g. Chauhan et al. [7], Wahl [43], Leopardi et al. [25].

Up to now the most well known model for breach of earthen dams is the NWS BREACH [12], a deterministic model in one horizontal dimension, used to predict the main characteristics of failure. BREACH couples the conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity of the unsteady uniform flow along an erosion-formed breached channel. The bottom slope of the breach is assumed to be essentially that of the downstream face of the dam. The downstream face of the dam can have a grass cover or a material of larger grain size than the outer part of the dam. The growth of the breach channel depends on basic properties of the construction materials, i.e. D50 size, unit weight, friction angle, cohesive strength.

3 Wave Propagation on the River Bed

The flood wave formed by the dam breach will propagate downstream along the natural watercourse, considered in most cases of one-dimensional geometry. Under this assumption the unsteady flow describing the flood wave propagation is governed by the St Venant equations, where the pressure is assumed hydrostatic. As mentioned in ch.2 the realistic representation of the dam breaking process smooths out the strong discontinuity the outflow hydrograph would have in case of a sudden dam break. The applicability of the propagation model can, therefore, extend upstream, close to the failed dam site. The governing relations are nonlinear hyperbolic partial differential equations, representing the conservation of mass and momentum along the direction of the flow:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A} + gI_1\right) = gI_2 + gA(S_o - S_f) \qquad (2)$$

where $A(x,t)$ is the wetted cross-sectional area, $Q(x,t)$ is the discharge through any particular cross-section, S_f is the friction slope, S_o is the bed slope, and g the acceleration due to gravity. I_1 represents the hydrostatic pressure force, while I_2 expresses the pressure force due to longitudinal width variations.

Five different finite difference explicit schemes were applied to solving eqs (1), (2), as follows: (a) the first-order Lax-Friedrichs scheme (LF), [15], [24], (b) the first-order central scheme of Kurganov-Tadmor (KT), [23], (c) the second-order central scheme of

Kurganov-Tadmor (KT2), [23], (d) the second-order central scheme of Nessyahu-Tadmor (NT2), [29], (e) the total variation diminishing-MacCormack scheme (TVD-MacCor), [16]. The first four numerical schemes use a Godunov-type approach [17] for the integration of eqs (1),(2). The fifth one (TVD-MacCor) is a predictor-corrector scheme, using a shock-capturing technique with second-order accuracy both in time and space. The above five schemes were tested in two different types of open channel flow, for which analytical solutions exist:

(i) idealized dam break flow in a horizontal, frictionless, rectangular channel, for various tailwater depths, including dry bed.

(ii) steady state flow in channel of variable bed slope as described by MacDonald et al. [27].

Representative results in terms of velocity distribution along the dry channel 15sec after dam failure (problem (i)) are given in Fig.1, where the four numerical schemes (a) to (d) are compared with the available analytical solution.

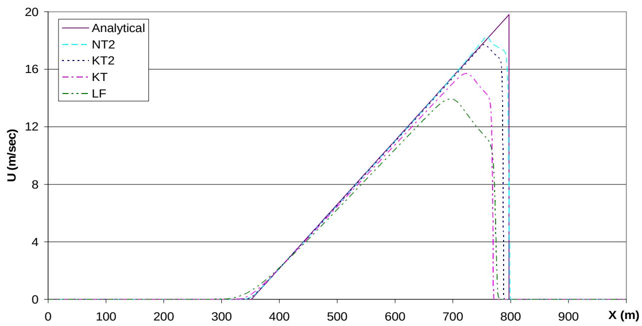

Fig.1. Predicted velocities vs analytical solution

The fifth scheme (TVD-MacCor) did not produce stable results for dry downstream channel. Error analysis for both dry and wet bed conditions showed that for problem (i), NT2 behaved better among the schemes tested. Similar analysis for problem (ii) showed schemes TVD-MacCor and NT2 to approximate more closely the analytical solution than the others, Fig.2. However, the former was rather sensitive to the discretization applied and displayed convergence problems.

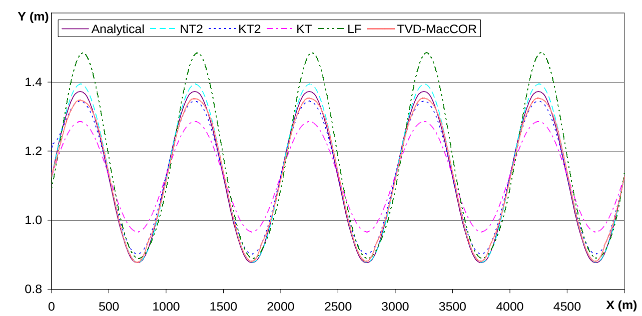

Fig.2. Predicted water depth vs analytical solution

Thus scheme NT2 was selected as the most robust and reliable, and was further compared with the commercial code BOSS DAMBRK [14]. This latter model is extensively used for flood wave routing. Its governing equations are the expanded one-dimensional St Venant equations of unsteady flow, i.e. eqs (1), (2) with additional terms for channel expansion, lateral inflow, etc. The system is solved by a nonlinear weighted 4-point implicit finite-difference method. The two numerical models (NT2 and DAMBRK) were applied to the routing of a triangular input hydrograph in a prismatic channel. Comparison of the results showed that water depths were almost identical between the two models, whereas arrival times of the flood wave were quite close (Fig.3).

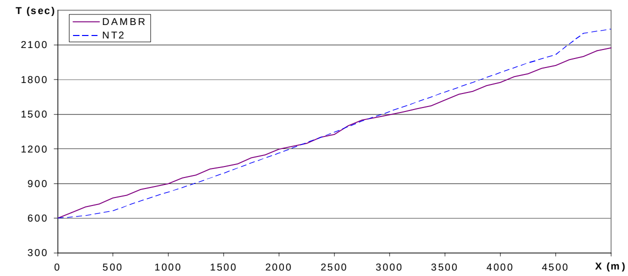

Fig.3. Time-to-peak values along a prismatic channel

4 Propagation through Reservoirs

The general case of fluid motion is described by the Navier-Stokes equations [31]. These are simplified to the St Venant equations (1), (2), under the assumption of gradually varied flow, leading to hydrostatic distribution of the pressure. In a multiple dam-break problem there are, however, reaches of rapidly varied flow that such a simplification does not hold. One such case occurs at the failed dam site, where the vertical velocities of the flow cannot be ignored. This complication is successfully overcome by using a suitable code to simulate the breaching of the dam (ch.2) and to provide the outflow hydrograph just downstream of it. This hydrograph is used as an input to the flood routing code along the watercourse (ch.3). Another case where the vertical velocities should be taken into account refers to the entrance of the flood wave into a deep reservoir. This is a situation met in some multiple dam-break problems. We limit our discussion to the commonly encountered case where the reservoir extends in only one horizontal dimension.

Hydrograph routing through a reservoir can be performed at a lower approximation through hydrologic storage routing techniques, based on the law of mass conservation. It is also assumed that reservoir outflow is only a function of the (horizontal) water level. Application of these methods is appropriate when it can be assumed that the water level changes simultaneously over the whole reservoir, i.e. the latter is not longer than a few miles; the exact characteristics of the flow along the reservoir are not important; and simplicity is required.

A better approximation of the hydraulics along the reservoir can be achieved through dynamic routing, i.e. by solving the St Venant equations, e.g. Stoker [39], Garrison et al. [26]. However, even in this advanced approximation, vertical velocities or shock conditions that violate the hydrostatic assumption of pressure distribution cannot be reasonably described. Under such circumstances it is advisable to revisit the Navier-Stokes equations. Nowadays there are numerical models that approximate the full Navier-Stokes equations, e.g. Horrillo and Kowalik [19]. Application of these models to cases of rapidly varied flow with non-hydrostatic pressure distribution, as e.g. at dam breaking, gives results close enough to those of the approximate nonlinear shallow water (NSW) equations incorporating a shock capturing method, as e.g. the method of characteristics.

Similar situation would apply to the case under consideration, i.e. at the entry of the flood wave into an one-dimensional deep reservoir of variable depth. At the upstream reach of the reservoir a NSW approximation capable of describing an advancing wave front over still water, as in the dam-break analytical solution, is thus acceptable. The flood wave will travel over a short distance in a bore-like fashion described above and then its plunging into the still reservoir water will mobilize the latter by producing a kind of shallow water wave, e.g. a flat solitary wave, that would travel the remaining reservoir length. Apparently this complex phenomenon cannot be described easily in quantitative terms. The initial phase of flow into the water reservoir, prior to surface wave formation, can be addressed roughly by representing the hydrograph as a simple shock wave and by using the analytical expressions of the equivalent dam-break problem as follows. The wave front celerity *c* and the upstream velocity *u* are given by

$$c = \left[gh_2\left(h_1 + h_2\right)/2h_1\right]^{1/2} \tag{3}$$

$$u = c\left(1 - h_1/h_2\right) \tag{4}$$

where h_1, h_2 the downstream, upstream depths respectively.

The above expressions can be used in a formulation, where by increasing in each space-step the still water depth by Δh and assuming over the length of constant depth one of either *c* or *u* to be constant, the water depth is estimated. In reality neither of *c*, *u* remains constant. Simple variational algebra shows that for $h_2>2.41h_1$ c decreases weakly. In deeper waters c increases strongly, thus a finer grid may be justifiable there, within the framework of this approximation. However, in that range ($h_2<2.41h_1$) the variation of c is weaker than the corresponding variation of h_2; similarly u displays there weak variation. That is why variables u, c were selected as the "constants" of the algorithm in an alternating fashion, over consecutive space-steps of the problem. The above-mentioned approximate treatment of the flood wave hydraulics refers to the initial stages of the flow plunging into the reservoir waters at its upstream reach.

The physics downstream of this plunging would be adequately described by shallow water wave propagation, as e.g. by a solitary wave of more or less permanent form. However, it should be pointed out that the determination of such a wave, mainly its wave height, is quite a complex undertaking. Therefore, the usual practice, transplanted in the commercial codes also, is to apply the St Venant equations throughout the propagation of the flood wave along the total length of the reservoir. Such an approach would assume that the initially still water at the upper reach of the reservoir would be rather shallow, thus mobilized as long as the flood wave enter the reservoir and would participate wholly in the flow, down to the reservoir bed. Use of St Venant's equations in the above context is sometimes described as dynamic routing of the flood wave through a reservoir.

5 Flood over Plains

It is quite common that after a travel of the flood wave over a long one-dimensional reach of the natural watercourse, it arrives finally a low-lying plain. There, the two-dimensional nature, in the horizontal sense, of the flow should be taken into account. There exist several models in the recent literature dealing with the 2-D modelling of the flood propagation over plain; see e.g. Alcrudo and Garcia-Navarro [2], Fraccarollo and Toro [11], Brufau and Garcia-Nacarro [5], Bradford and Sanders [3], Jha et al. [21]. A relatively simple model for the calculation of flood propagation in two dimensions is the model FROM-2D (Flood Routing Model in 2 Dimensions), developed in the NTUA (Stamou, [37]). It involves the 2-D continuity and momentum equations of unsteady flow for approximately horizontal flow, written as follows.

$$\frac{\partial h}{\partial t}+\frac{\partial Q_{x_1}}{\partial x_1}+\frac{\partial Q_{x_2}}{\partial x_2}=\frac{\partial y}{\partial t}+\frac{\partial(u_1\cdot h)}{\partial x_1}+\frac{\partial(u_2\cdot h)}{\partial x_2}=0 \qquad (5)$$

$$\frac{\partial u_i}{\partial t}+u_1\frac{\partial u_i}{\partial x_1}+u_2\frac{\partial u_i}{\partial x_2}=g\left(S_{Ox_i}-S_{fx_i}\right) \qquad (6)$$

where x_1, x_2 horizontal co-ordinates; i=1, 2; h flow depth; u_1, u_2 flow velocity component in x_1, x_2 direction respectively; $Q_{x1}=u_1h$, $Q_{x2}=u_2h$ flow rate per unit width in x_1, x_2 respectively; S_{Ox1}, S_{Ox1} bottom slope in x_1, x_2 respectively; S_{fx1}, S_{fx2} friction slope in x_1, x_2 respectively; g acceleration due to gravity.

The friction slope can be approximated by Manning's expression for uniform flow. Ignoring the acceleration terms in the momentum equations (6), the friction slopes in x_1 and x_2 directions are equal to the corresponding surface slopes and equations (5) and (6) can take the following final form

$$\frac{\partial h}{\partial t}+\frac{\partial(u_1 h)}{\partial x_1}+\frac{\partial(u_2 h)}{\partial x_2}=0 \qquad (7)$$

$$-\frac{\partial(h+z)}{\partial x_i}=\frac{u_i^{\,2}}{K^2\cdot h^{4/3}} \qquad (8)$$

where z is the ground elevation, K Manning's roughness coefficient.

In FROM-2D, equations (7) and (8) are solved using the finite differences method in a staggered grid, employing central differences for spatial discretization and explicit temporal discretization, to determine u_i and h.

6 The Study Area

The previously described approaches and considerations were applied in an integrated way to a real-life problem pertaining to a 300km long river in northern Greece with 5 hydroelectric dams along its route (Fig.4). The study area included:

(i) A stretch of 123km extending from the upstream end of the reservoir of the upstream dam (dam #1) to the most downstream dam (dam #5). It is important to note that 93% of this stretch is occupied by the reservoirs of the 5 dams. Nine villages were located close to the watercourse at relatively low elevations. Three road bridges were crossing the river section between the 5 dams. The main dimensions of the dams and their associated reservoirs are given in Table 1.

(ii) A cultivated plain downstream of dam#5 covering an area of about 1100km^2 with maximum difference in elevation of about 20m.

TABLE 1
KEY DATA OF DAMS AND RESERVOIRS

No	Dam Height (m)	Dam length (m)	Max.Water Surface (km^2)	Reservoir length (km)
1	100	500	27	46.6
2	97	295	73	41.8
3	66	260	5	9.8
4	48	200	3	12.6
5	14	1200	2	3.8

The river crosses the plain close to its southern boundary for 50km before discharging into the sea. A large drainage channel combines its discharge with the river flow at an upstream location in the plain, while another one flows independently close to the northeastern border of the plain. The plain is also crossed by four main roads and one railroad built mainly on embankments up to 6m high. Four main bridges were crossing the river along its floodplain route. In total 51 villages and small townships were dispersed in the floodplain sited on average elevations from +2.5m to +29.8m above MSL.

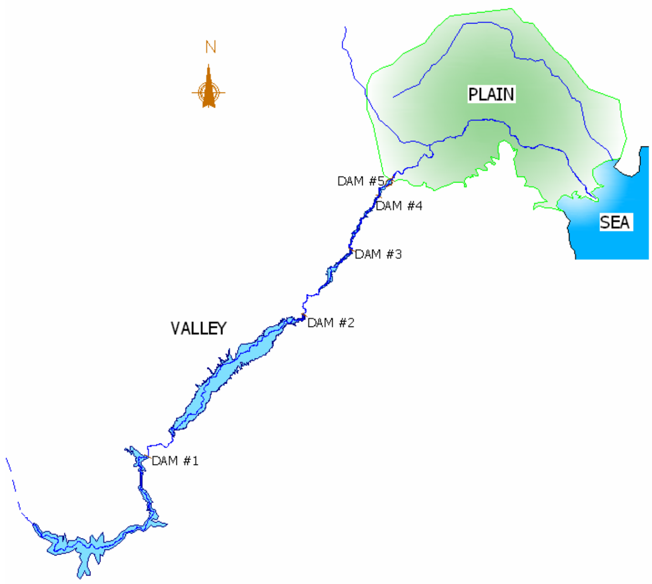

Fig.4. Location of the study area

7 Methodology Applied

7.1 General

In total 13 scenarios of flood wave routing were investigated, based on the following parameters:

(i) Upstream dam to break

(ii) Cause of initial dam break, i.e. either overtopping or piping
(iii) Empty or full reservoir #2

Additionally comparison checks were performed between runs using either BREACH or DAMBRK to simulate the initial dam break.

Six individual problems were addressed, that if linked properly together the complete picture of the sequential dam-break flooding would emerge. These problems refer to the breach of a dam, the one-dimensional routing of the flood wave over a dry bed, the routing through a one-dimensional (horizontally) reservoir, the flood wave run-up on the upstream slope of dam, the eventual modification of the flow due to bridges, and the two-dimensional expansion and propagation of the flood over the low lying plain. In the following a brief description on the applied treatment to each one of those problems will be given.

7.2 Dam breach

As mentioned previously, simulation of the dam breach was performed by both models BREACH and DAMBRK. The latter was found to behave more on the safety side as regards dam failure due to piping, thus it was used in 3 scenarios, out of the 13 in total, involving this type of failure. In the remaining scenarios BREACH was used, since in this model the time to breach and other parameters of the problem are not selected arbitrarily but rather are calculated based on the properties of the dam materials.

The input data into the BREACH model include the following: reservoir characteristics, breach initial characteristics, inner and outer core characteristics, and dam face description. The model is founded on physical grounds dealing with the mathematical description of (i) breach hydraulics (flow over broad crested weir or in a pressure pipe), (ii) soil mechanics (stability of soil slopes) and (iii) sediment transport. It calculates the temporal evolution of the dam breach characteristics (size, shape and time of formation), and then displays the graphs of the computed breach outflow hydrograph. The results can then be used as input to the DAMBRK model so that flood routing calculations be performed.

7.3 Flood routing along the valley

7.3.1 Routing over a dry watercourse

Along the stretch of the river were the 5 dams were present, there were two sections where dry-bed conditions were encountered. The first, downstream of dam#1 down to reservoir #2, of length 5.8-9.8km depending on the downstream reservoir level, and the second, downstream of dam #2 down to reservoir #3, of length 6.1km. Flood routing along these sections was performed by use of the numerical model DAMBRK of BOSS [14]. The output of the model used subsequently was the computed hydrograph at the upstream end of the reservoirs #2 and #3.

Code DAMBRK was finally used in the real sequential dam-break problem due to the following reasons:

(i) It uses a scheme of the implicit type, thus it tends to give results independent of the selection of the time step
(ii) The convergence is quicker and the computational time lower, since there is no restriction by the Courant-Friedrichs-Lewy criterion, as is the case for the NT2 scheme
(iii) Its wide application worldwide gives extra confidence, especially to the authorities that will at the end manage such catastrophic events.

7.3.2 Routing through the reservoirs

As noted earlier, dam breach by overtopping was treated by BREACH model, which manages only storage routing in the associated reservoir (ch.4). In contrast, any closer approximation to reality, as e.g. through dynamic routing, would give a better indication of the inundation area along the reservoir length. Also, at the upstream reach of the reservoir a rapidly varied flow technique might be used. In our case a justifiable approximation was applied, due to the fact that the upstream bed, in all four reservoirs, was mildly sloping downstream, thus no excessive vertical velocities were anticipated there. The approximation consisted of interpolating linearly along the reservoir the known maximum water depths at the upstream end of the reservoir and at the dam site. The former was supplied by the DAMBRK routing over the dry bed section, in case such section existed, or through the dam breach hydrograph provided by BREACH in case of cascading reservoirs with no dry section between them. The water depth at the dam site was produced as output of the dam breach procedure simulated by the BREACH, or DAMBRK, model. It can be easily shown that this technique is conservative in most cases along the upstream part of the reservoir. In our case, the villages at critical low levels were located in that part. Therefore, the conclusions arrived at were thought to represent an upper limit of risk conditions.

7.3.3 Flood wave run-up

Some of the scenarios tested assumed the reservoir #2 at low level. The question thus arose, whether the flood wave produced by the breach of dam #1 would be able to overtop dam #2. Level pool routing of the inflow hydrograph to reservoir #2 showed no overtopping of the dam crest, but it is evident that no dynamic effects, such as wave run-up, can be simulated by this method, or even by the more accurate dynamic routing.

A rough conservative estimate was achieved by assuming that the surface wave formed by the flood wave entering the reservoir carries nearly the same amount of energy, kinetic and potential, as the incoming wave front. By representing the generated water hump by a solitary wave, an upper bound of the resulting wave height can be estimated. It is reminded that the total wave energy per crest width of a solitary wave is

$$E = 1.54 \rho g H^{3/2} d^{3/2} \tag{9}$$

where ρ the water density, H the wave height, d the water depth measured from still water level.

It is also known that the maximum wave height before breaking is $H_{\max} = 0.78d$. Having thus estimated a conservative value for the height of the wave propagating along the reservoir, an equally conservative value of the wave run-up R on the upstream face of the dam can be calculated. This was done by employing the formula used in tsunami applications, namely Synolakis [41]

$$R = 2.831\sqrt{2}\left(H/d\right)^{5/4} d \tag{10}$$

where, d was taken here as the water depth at the upstream toe of the dam structure. Comparison of the obtained upper level of wave run-up with the dam crest leads to whether the latter will be overtopped by the flood wave.

7.3.4 Bridges

The underbridge flow capacities were checked against the required maximum discharge of the flood wave. In order to be able to decide whether an obstacle, such as a bridge, would modify the flow conditions, the integrity of the structure should be assessed. To this end a simplified drag D was calculated by the formula

$$D \approx \rho C_D E u^2 / 2 \tag{11}$$

where, E cross-sectional area of flow impingement, u flow velocity, C_D drag coefficient, dependant on the shape of the bridge member loaded hydrodynamically. The assessment on the possible bridge failure was based on the magnitude of D above and the characteristics of the structure.

7.4 Flood plain propagation

In the present work the model FROM-2D of the NTUA was used to represent flow conditions over the low-lying plain, mainly because of its simplicity. The input data into the FROM-2D model include the geometry of the floodplain and the upstream hydrograph emerging from the downstream section of the valley routing. The model calculates the horizontal flow velocities and water depths. The topography of the computational domain, including abrupt changes such as roads, embankments etc., was taken into account by simply inserting into the model the actual final levels of these man-made constructions. Inferences were then drawn on the structural integrity of these line-obstacles, by comparing the flow depth over their crest elevation with accepted thresholds of these depths denoting the beginning of the erosion process.

8 Results and Discussion

An array of technical data resulted from the application of the above described models and procedures. These included characteristic values of dam breach parameters; flood wave propagation data along the valley; flow rate and stage hydrographs at characteristic cross-sections of the watercourse; inundation zones along the valley as well as over the plain downstream of the dams; arrival times of the flood wave and of the maximum water depth; time history of water depth at the villages; etc.

It was found that for the most critical scenario most of the settlements of the plain will be inundated, whereas in the same area the maximum flow velocities fall around 4m/sec. Drainage times ranged from 5hr to 25hr or more in some villages. A representative diagram of the maximum flow depths over the plain for one of the scenarios tested is given in Fig.5.

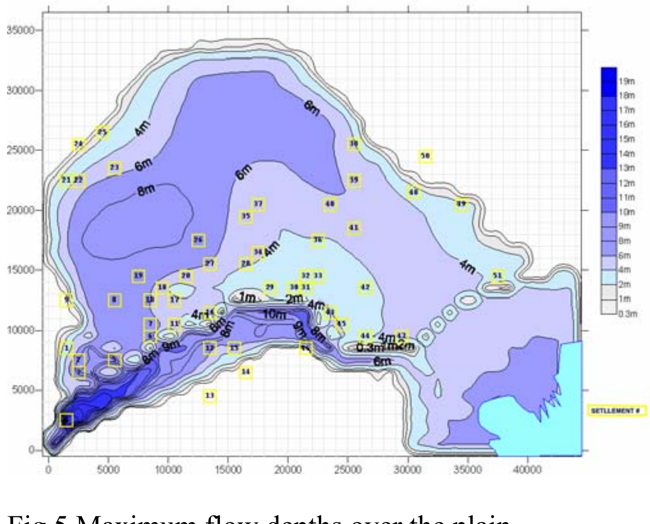

Fig.5 Maximum flow depths over the plain

It was also estimated that appreciable lengths of two of the three highways crossing the plain will be severely damaged, as well as the embankments of the major drainage canal of the area. Severe damages or failures will also suffer most of the main bridges both along the valley and over the plain. Some guidelines were finally given related to the development of a monitoring and warning system in the area under risk of flooding. A crucial point in such a system relates to the clear definition of the start time where all time values refer. The competent authorities should take due consideration of this point.

9 Conclusions

The problem of multiple dam break was briefly described and its components analyzed. Competent models have been developed capable of addressing adequately, despite inherent numerical shortcomings, each individual component of the problem, as e.g. dam breach, flood propagation over dry bed, etc. However, treatment of complex configurations, as the one described in this paper, cannot be easily performed by widely accepted commercial models, as the ones used here. One main disadvantage remains the poor modelling of the flood flow along long valley deep reservoirs. In this respect routing through the St Venant equations seems to be the best available means of those models as yet. This option is incorporated in model DAMBRK. The latter, however, cannot describe the dam break based on physical grounds, in contrast with model BREACH, that can give a better approximation of the phenomenon. One possible improvement would be to analyze first the dam break through BREACH, determine hence the key break parameters and input them into the DAMBRK to perform the dynamic flood routing in the reservoir at the same time with the dam break with calculated rather than arbitrarily defined breach characteristics. This technique is currently used successfully in a similar project on another large Greek river with five in-line dams.

Acknowledgements :
The support provided during the course of this project by the Public Power Corporation of Greece is greatfully acknowledged. The authors express their gratitude to G.Fountoulis and J.Psaras for their valuable contributions; also to P.Demetriadis for supplying us with articles on the dam breach mechanism.

References:

[1] M.B. Abbott, *An introduction to the Method of Characteristics*, Elsevier, New-York, 1966.

[2] F. Alcrudo, P. Garcia-Navarro, A high-resolution Godunov type scheme in finite volumes for the 2d shallow-water equations, *Int. J. Numer. Methods Fluids,*Vol.16, 1993, pp.489–505.

[3] S.F. Bradford, B.F. Sanders, Finite-volume model for shallow-water flooding of arbitrary topography, *J. Hydraul. Eng.,* Vol.128, No. 3, 2002, pp.289–298.

[4] K. Broich, Mathematical modeling of DAM-Break erosion caused by overtopping, *CADAM, 2nd-Project workshop held in Munich*, 8-9 Oct. 1998.

[5] P. Brufau, P. Garcia-Navarro, Two-dimensional dam break flow simulation, *Int. J. Numer. Methods Fluids,* Vol. 33, 2000, pp.35–57.

[6] *CADAM* (Concerted Action on Dambreak Modelling), *2nd-Project workshop held in Munich*, 8-9 Oct.1998.

[7] S.S. Chauhan, D. S. Bowles, L. R. Anderson, Do current breach parameter estimation techniques provide reasonable estimates for use in breach modeling, Utah State University, Utah Water Research Laboratory, 2005.

[8] J. Cunge, F.M. Holly and A. Verwey, *Practical aspects of computational river hydraulics*, Pitman Advanced Publishing Program, London, 1980.

[9] L. D'Alpaos, A. Delfina, B. Matticchio, 2D finite element modeling of flooding due to river bank collapse, in P. Molinaro and L. Natale (eds), *Modelling of Flood Propagation Over Initially Dry Areas*, ASCE, 1994, pp.60-71.

[10] R.F. Dressler, Hydraulic resistance effect upon the dam-break functions, *U.S. NBS. Journal of research*, Vol. 49, No.3, Sept.1952.

[11] L. Fraccarollo, E.F. Toro, Experimental and numerical assessment of the shallow water model for two-dimensional dambreak type problems, *J. Hydraul. Res.,* Vol.33, No.6, 1995, pp. 843–864.

[12] D. Fread, BREACH: An erosion model for earthen dam failures, Hydrologic Research Laboratory, Aug.1991 (www.nws.noaa.gov).

[13] D.L. Fread, Channel routing. In: *Hydrological forecasting*, M.G.Anderson and T.P.Burt (Ed.), Wiley, New-York, 1985.

[14] D.L. Fread, DAMBRK:The NWS dam-break flood forecasting model, Hydrologic Research Laboratory, National Weather Service, Silver Spring, Maryland, 1982, pp.56.

[15] K.O. Friedrichs, Symmetric hyperbolic linear differential equations, *Communications on pure and applied Mathematics*, Vol.VII, 1954, pp.345-392.

[16] P. Garcia-Navarro, F. Alcrudo, J.M. Saviror, 1-D Open-Channel Flow simulation Using TVD-McCormack Scheme, *J. Hydraul.Eng*, Vol.118, No.10, 1992, pp.1359-1372.

[17] S.K. Godunov, A difference scheme for numerical computation of discontinuous solution

of hydrodynamic equations, *Mathematics Sbornik*, Vol.47, 1959, pp.271-306.

[18] K.Y. Han, J.H. Park, Two-dimensional model for the prediction of inundation area in urbanized rivers, *J. of Korean Water Resources Association*, Seoul, Korea, Vol.28, No. 6, 1995, pp. 119-131.

[19] J.J. Horrillo, Z. Kowalik, W. Knight, Full Navier-Stokes approximation in Tsunami investigation, *22nd Int. Tsunami Symposium*, Chania(Greece), 27-29 June, 2005.

[20] B. Hunt, Asymptotic solution of dam-break problem, *Journal of the Hydraulics Division,* ASCE, Jan.1982.

[21] A.K. Jha, J. Akiyama, M. Ura, Flux-difference splitting schemes for 2D flood flows, *J. Hydraul. Eng.,* Vol.126, No.1, 2000, pp.33–42

[22] N.D. Katopodes, T. Strelkoff, Computing two-dimensional dam-break flood waves, *J. of Hydraulics Division*, ASCE, Vol.104, No.HY9, 1978, pp.1269-1288.

[23] A. Kurganov, E. Tadmor, New high-resolution central schemes for non-linear conservation laws and convection-diffusion equations, *Journal of Computational Physics*, Vol.160, 2000, pp.241-282.

[24] P.D. Lax, Weak solutions of non-linear hyperbolic equations and their numerical computation, *Communications on pure and applied Mathematics*, Vol.VII, 1954, pp.159-193.

[25] A. Leopardi, E. Oliveri, M. Greco, Numerical Simulation of Gradual Earth-Dam Failure, *2nd Int. Conf-New Trends in Water and Enviromental Engineering for Safety and Life*, Capri(Italy), 24-28 June, 2002.

[26] J.M. Garrison, J-P. P. Granju, J.T. Price, Unsteady flow simulation in rivers and reservoirs, *Journal of the Hydraulic Division,ASCE*, Vol.95 ,No.HY5, Sept.1969, pp1559-1576.

[27] I.MacDonald, M.J. Baines, N.K. Nichols, P.G. Samuels, Analytical benchmark solutions for open-channel flows, *Journal of Hydraulic Engineering*, ASCE, Vol.123, No.11, 1997, pp.1041-1045.

[28] R. Moussa, C. Bocquillon, Approximation zones of the saint-Venant equations for flood routing with overbank flow, Hydrology and Earth System Sciences, Vol.4 ,No. 2, 2000, pp.251-261.

[29] H. Nessyahu, E. Tadmor, Non oscillatory central differencing for hyperbolic conservation laws, *Journal of Computational Physics*, Vol.87, 1990, pp.408-463.

[30] A. Paquier, P. Nogues, R. Herledan, Model of piping in order to compute dam-break wave, *CADAM, 2nd-Project workshop held in Munich*, 8-9 October, 1998.

[31] A. D. Polyanin, V. F. Zaitsev, Handbook of Nonlinear Partial Differential Equations, Chapman & Hall/CRC, Boca Raton, 2004.

[32] R. Rajar, Modelé mathématique et abaques sans dimensions pour la détermination de l' écoulement qui suit la rupture d' un barrage, *XI Congress Int. Commission on Large Dams, Madrid*, Q.40, R.34, 1973.

[33] A. Ritter, Die Fortpflanzung der Wasserwellen, *Zeitschrift des Vereines Deutscher Ingenieure*, Vol.36, No.33, Aug.1892.

[34] B. Saint-Venant, Théorie du movement non permanent des eaux, avec application aux crues des rivières et à l' introduction des marées dans leurs lits, *C.R. Séan. Acad. Sci*., Vol.73, 1871, pp.147-154 and 237-240.

[35] J. Sakkas, T. Strelkoff, Dimensionless solution of dam-break flood waves, *Journal of the Hydraulics Division*, *ASCE*, Feb.1976.

[36] L.J. Savic, F.M. Holly Jr., Dambreak flood waves computed by modified Godunov method, *Journal of Hydraulic Research*, Vol.31, No.2, 1993.

[37] A.I. Stamou, Flood routing Model in 2 Dimensions, NTUA, 2005.

[38] A. Stamou, M. Politis, I. Xanthopoulou, The importance of dam break analysis in environmental impact studies for dams, *Proceedings of the 9th Int. Conf. on environmental Science and Technology*, Rhodes, Sept.2005.

[39] J.J. Stoker, *Water Waves*, Interscience Publishers Inc., New York, 1957.

[40] T. Strelkoff, D.R. Chamber, N.D. Katopodes, Comperative analysis of routing techniques for the flood wave from a ruptured dam, *Proc. Dam-Break Flood routing Model Workshop, Water Resources Council,* Washington D.C, 1977, pp.229-291.

[41] C.E. Synolakis, The runup of solitary waves, *Journal of Fluid Mechanics*, Vol.185, 1987, pp.523-545.

[42] R. Szymkiewicz, Finite-element method for the solution of the Saint Venant equations in an open channel network, *J.Hydrol*., Vol.122, 1991, pp.257-287.

[43] T.L. Wahl, The Uncertainty of Embankment Dam Breach Parameter Predictions Based on Dam Failure Case Studies, *USDA/FEMA Workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analysis,* Oklahoma City, 26-28 June 2001.

[44] T. Xanthopoulos, C. Koutitas, Numerical simulation of a two-dimensional flood wave propagation due to dam failure, *J. of Hydraulic Research*, Vol.14, No.4, 1976, pp.321-331.