6TH INTERNATIONAL CONFERENCE ON DAM ENGINEERING
C.Pina, E.Portela, J.Gomes (ed.)
Lisbon, Portugal, February 15-17, 2011

COMPARISON BETWEEN 1D AND 2D DAM-BREAK FLOOD NUMERICAL MODELS

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Keywords: Dam-break, flood wave propagation, downstream valley, 2D numerical model

Abstract. *Numerical simulation models are powerful tools for dam and downstream valley risk assessment and management, including the evaluation of the impact of floods due to dam failure events and the valley emergency planning. Several codes are available for numerical simulations of floods caused by dam failures: the most well known being 1D models. This last type of models can lead to possible errors namely in the cases where there are important cross-section changes (like enlargements or narrowings) in the propagation river stretch or whenever the flow occupies a flood-plain. In these cases a 2D model is required. The 2D BIPLAN model was developed for the simulation of dam-break flood waves in valleys with irregular topography whenever the 1D approach loose validity. This paper concerns the use of the 2D BIPLAN model and of the well-known 1D DAMBRK model for the simulation of the dam-break flood propagation in a real valley: the Arade River localized in the South of Portugal. The accuracy of both models is presented through a comparative analysis of their results with experimental data obtained in the physical model of Arade River valley. Firstly, the main characteristics of the numerical models are presented and the case study is described. Next the paper briefly describes the physical model of the case study, constructed in order to validate dam-break flood propagation models along river valleys of irregular topography. Finally, the paper also presents the results of 2D BIPLAN and 1D DAMBRK models and the comparative analysis of their results, also involving experimental data.*

1 INTRODUCTION

The main dangerous effect in the downstream valley following a dam failure is the sudden release of a flood wave and its consequent propagation. The risk assessment in dam downstream valleys is basically a definition of the potential flood (hazard) consequences (damages and losses) based on hydraulic analysis and computational simulations.

There are several codes available for numerical simulation of dam-break floods: the most well known being 1D models. Those can lead to possible errors namely in the situations where important cross-section changes (enlargements or narrowings) exist in the propagation river stretch or whenever the flow occupies a flood-plain. In these cases a 2D model is required.

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This paper concerns the use of the 2D BIPLAN model and of the well-known 1D DAMBRK model for the simulation of the dam-break flood propagation in a real valley. The case study presented in this paper concerns Funcho and Arade dams and their shared downstream valley, localized in the South of Portugal. The paper presents both dam break flood simulation results using 2D BIPLAN and 1D DAMBRK models. The accuracy of both models is presented through a comparative analysis of their results with experimental data obtained in the Arade River physical model.

Following this introduction, Section 2 is a presentation of the main characteristics of the numerical models. Section 3 describes the case study and Section 4, presents the comparison of the numerical results and experimental data. Finally, the paper ends with the main conclusions which arise from the study, namely those regarding the accuracy of 2D BIPLAN model results.

2 NUMERICAL MODELS

2.1 1D DAMBRK model

The 1D DAMBRK model simulates the failure of a dam, computes the resultant outflow hydrograph and simulates the routing of the dam-break flood wave through the downstream river valley. The model is fully described by Boss DAMBRK 1991[1].

The outflow hydrograph in the dam cross-section is computed, in the general case, by the addition of the flow through the breach, overtopping the dam and through the spillways. The flow overtopping and through the breach is calculated using a broad-crested weir equation.

The model governing equations in which the routing process is based are the complete one-dimensional Saint-Venant equations that are coupled with internal boundary equations representing the flow through structures such as dams and also external boundary equations for the upstream and downstream end of the routing reach. The system of equations is solved by a nonlinear weighted 4-points implicit finite-difference method.

The upstream boundary condition is an hydrograph (Q,t), defined by the program user. Generally the occurrence of a natural flood hydrograph river is envisaged. It is also the user that must define the downstream condition. DAMBRK model allows the following conditions: a) rating curve Q= Q(h) (single and dynamic loop), b) critical flow rating and c) water level time series h = h(t).

2.2 2D BIPLAN model

The 2D BIPLAN model was developed for the simulation of dam-break flood waves in valleys with irregular topography where the 1D approach looses validity, such as: flood plains and zones where strong changes of the cross-section occur. The physical equations, the numerical method and the computational description of this model are presented on Table 1. The model is fully described by Franco[2].

The model is based on the full 2D shallow water equations – the two-dimensional Saint-Venant equations – solved by the MacCormack-TVD method. The model uses a Cartesian computational grid. The topography, i.e. ground level at each node, can be defined by a digital elevation model (DEM). The output of the model consists on hydrographs at any point of the grid (flow depths and velocities) and tables prepared to produce, by means of Geographic Information System (GIS) software, digitalized inundation maps and velocities field maps.

Two external boundary conditions must be imposed: one at the upstream river zone of the simulated area and another at the downstream river zone. 2D BIPLAN model only computes the routing of hydrographs, i.e., it does not calculate the failure of the dam and the resultant outflow hydrograph. Therefore, the model user must first define, as data input, the hydrograph to impose in the upstream boundary. In this case, the boundary hydrograph was obtained with the 1D DAMBRK numerical model and was next linked, according to BIPLAN upstream boundary algorithm, to a characteristic equation C^-, allowing the calculation of flow depths and velocities at the boundary, along the axis of the channel. At this section the uni-dimensionality of the flow is admitted and the velocities are considered zero on the normal direction of the channel axis. At the downstream boundary a radiation condition is used linked to the characteristic equation C^+. At this section the uni-dimensionality of the flow is also admitted.

	Description
Physical equations	The 2D full Saint-Venant equations in the conservation form
	The friction term is calculated with the Manning-Strickler formulation
Time discretization	Second order explicit, mixed forward and backward finite differences, fractional two steps scheme
Space discretization	Finite difference two steps scheme (predictor and corrector steps)
	Regular and rectangular computational grid
Numerical flux	TVD limited
	Local linearization with Roe method: Roe[3]
	Entropy condition: Harten and Hyman[4]
	Flux limiter formula: Van Leer[5]
Source terms	Explicitly integrated
Time step	Variable in each time step calculated by the Courant-Friedrichs-Lewi (CFL) number
CFL condition	Equal to 0.9
Numerical treatment of boundaries	Walls - fictitious points considering symmetric reflection
	Downstream channel - characteristic C^+

Table 1: General characteristics of 2D BIPLAN model.

To ensure the numerical model capacity to simulate flows in domains with general irregular topography and to recognize possible paths of the flow in each time step, a special treatment imposing internal boundaries conditions was developed. These boundaries, which are temporary and mobile, are generated depending on the water levels values and ground surface elevations in the adjacent computational nodes. Three situations were envisaged where the direct application of the Saint-Venant equations is not allowed and, for each one, additional conditions were defined, in order to determine the intercell fluxes, namely the followings: i) dry cell, ii) high point and iii) low point.

In the first situation – dry cell – the model sees if there are wet cells in the neighbouring of the grid point (i,j) under calculation, verifying if the water depths in its adjacent grid points are lower than ε , value almost equal to zero. If this condition occurs, the model imposes flow depth and velocities equal to zero in grid point (i,j) and the Saint-Venant equations are not applied. The algorithm is, for example, in the case of the predictor step with progressive difference in the x direction:

$$\begin{cases} h_{i,j} < \varepsilon \\ h_{i+1,j} < \varepsilon \quad then \;\; h^P_{i,j} = u^P_{i,j} = v^P_{i,j} = 0 \\ h_{i,j-1} < \varepsilon \end{cases} \tag{1}$$

where h = water depth; u and v = velocity components along the x and y directions and the superscripts P denotes the predictor steps.

The second situation – high point – is guaranteed if the water depth in the grid point (*i*,*j*) is zero and the ground surface elevation Z(*i*,*j*) is greater than the water level in the adjacent grid point (Figure 1). For example, in the case of the predictor step with regressive differences for the x direction, the model verifies if $h_{i,j} = 0$ and $Z_{i,j} > Z_{i-1,j} + h_{i-1,j}$. If those conditions are guaranteed, the Saint-Venant equations are also not applied and the variables predictions are:

$$h^P_{i,j} = u^P_{i,j} = v^P_{i,j} = 0 \tag{2}$$

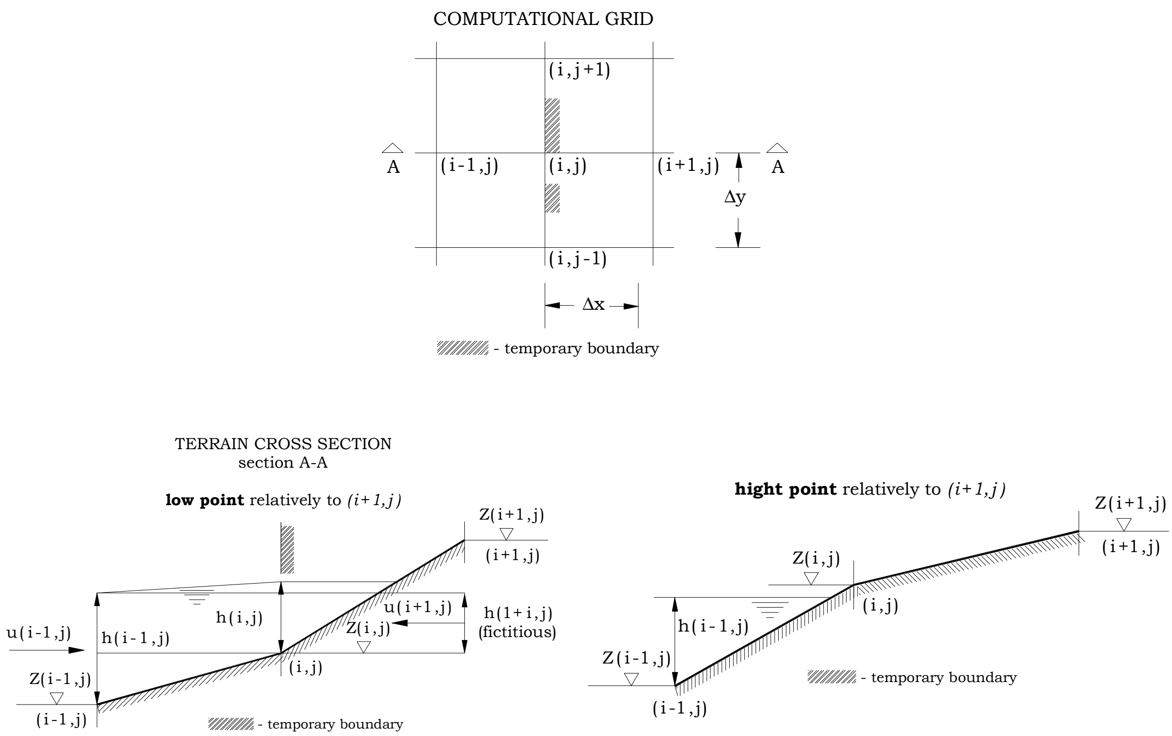

Figure 1: Definition sketch of the internal temporary boundaries.

The third situation – low point – is defined when the water level at the grid point (*i*,*j*) is lower than the ground surface elevation at the neighbouring grid point, i.e., in the case of $Z_{i,j} + h_{i,j} < Z_{i+1,j}$, the model generates a reflection boundary following the methodology presented by Fennema and Chaudhry[6] and a fictitious point is created – (*i+1*,*j*). In the case of using progressive difference in the x direction (Figure 1) the following conditions are imposed:

$$\begin{aligned} Z_{i+1,j} &= Z_{i,j} \\ h_{i+1,j} &= Z_{i-1,j} + h_{i-1,j} - Z_{i,j} \\ u_{i+1,j} &= -u_{i-1,j} \\ v_{i+1,j} &= v_{i-1,j} \end{aligned} \tag{3}$$

With these values in the fictitious grid point (*i+1,j*) it is possible to apply the MacCormack-TVD method and solve the Saint-Venant equations, obtaining the prediction for the water depth value at grid point (*i,j*), $h^P_{i,j}$, and imposing, in the x direction, a null velocity ($u^P_{i,j}$ = 0,0). At the end of this computational step, the variables values at grid point (*i+1,j*), that have been altered during the previous step, are returned to their normal and initial values.

3 CASE STUDY

3.1 General characteristics

The Arade River is situated in the South-western part of Portugal in the Algarve region and has a total length of 45 km, until the river mouth, at the Atlantic Ocean. Its basin has an ENE-WSW orientation and a surface area of 800 km^2. Arade River is one of the most important Algarve surface water resource, and has, split apart by 5.7 km, two dams in cascade: first, the Funcho dam (km 4.7) and, then, the Arade dam (km 10.4) - Figure 2. The river stretch downstream of Arade dam has a total length of 23.6 km.

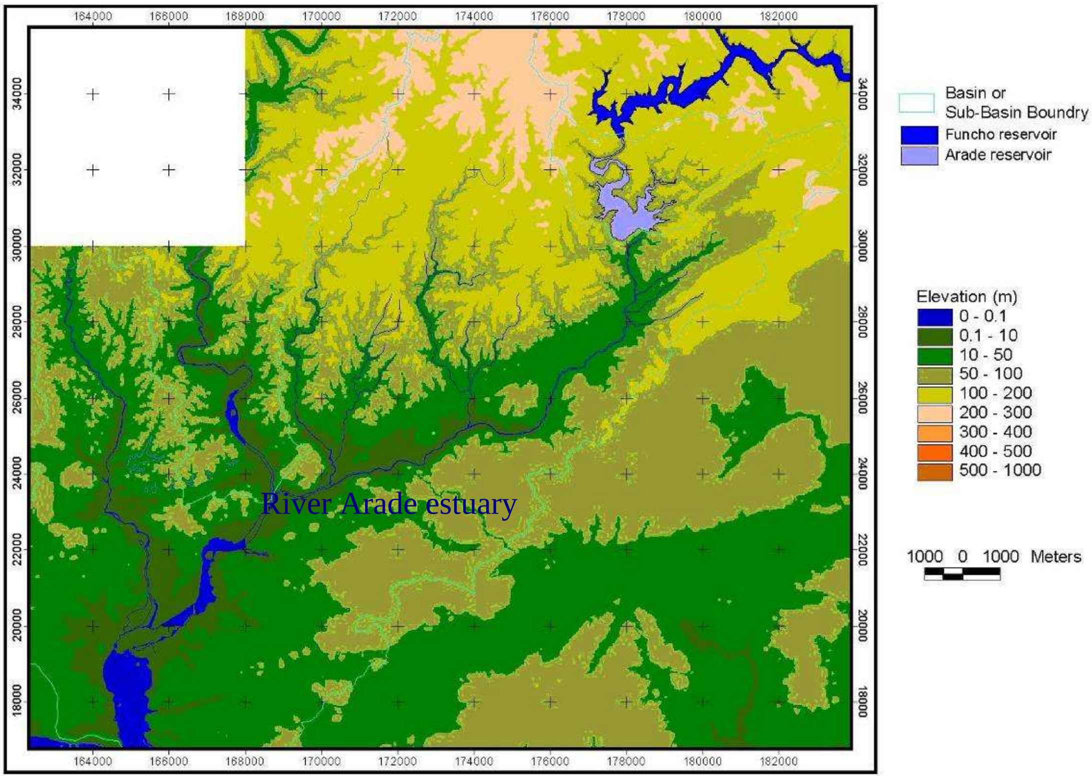

Figure 2: Funcho and Arade dams downstream valley.

Funcho dam is a concrete arch dam, built in 1991, with a height of 49 m and a crest length of 165 m. The reservoir has a gross capacity of 43.4 hm^3. Arade dam is an earthfill dam, built in 1955, with a height of 50 m and a crest length of 246 m. The reservoir has a gross capacity of 28.0 hm^3.

Arade River spreads throughout six districts, Silves, Portimão, Loulé, Monchique Lagoa and Almodôvar. The total population downstream the dams is concentrated in two main poles – Silves and Portimão cities. Silves city, which is localized 9.6 km downstream of Arade dam, has 10 674 inhabitants, according to 2001 year population census. Further downstream, in the river estuary, 21.6 km downstream of Arade dam, lies the city of Portimão, with 33 468 inhabitants[7].

The Arade River margins are sloppy and narrow in the Funcho dam section. Maximum elevation there is 486 m. Overall medium elevation of the downstream subsection is 73 m. The Arade reservoir is a transition zone to a more plain terrain. Downstream of Silves city, the river gives place to the estuary, where the riverbed becomes much wider.

3.2 Arade river physical model and tests

A reach of the Arade River valley, immediately downstream of the Arade dam (km 10.4) was chosen for the construction of the physical model. It is a non-distorted model at a scale of 1:150 and reproduces a stretch of 6000 m of the river valley.

Over the initial 1.7 kilometres, the modelled river is a narrow and straight valley, with 1D flow behaviour. At the end of this stretch, a tributary - the Baralha Creek - enters from the left margin. The following second stretch of the modelled river is 3.7 km long. It is an area where 2D effects prevail, mainly due to the existence of a wide plain with several singularities such as two consecutive river bends and a narrowing performed by a bridge. Over the final modelled river stretch, the cross-sections narrow again and the flow practically occurs in the x direction; due to this topography, critical flow will occur at the downstream boundary.

The physical model is 40 m long and 20 m wide (Figure 3) and is fully described in Viseu[8]. The discharges are modelled by a closed loop control system for reproducing selected flood hydrographs in the upstream boundary of the physical model.

Figure 3: General view of the Arade River valley physical model.

A set of 20 rectangular metallic plates are situated over the model, each one equipped with submersible transducers (pressure sensors) to measure the time evolution of water depths. This paper presents the record data for three of them – transducers T_{m1}, T_{m4} and T_{m6}. The placement of these sensors, which signal is acquired and registered in a second computer, is shown on Figure 4 and was chosen to illustrate the results of the most important singularities of the model. Starting at the upstream boundary, at (km 0.0), and proceeding downstream, the transducers are the following:

- T_{m1}, located in the first narrow river stretch, half way between the upstream boundary and the confluence with the Baralha Creek tributary (km 12.0);
- T_{m4}, located just upstream the existent bridge (km 14.1);
- T_{m5}, located in the floodplain, just upstream the second river narrowing (km 15.4).

Both tests in steady and unsteady flow regimes were undertaken. As it was important to characterize the flow behaviour for very high floods (corresponding to dam failures), the minimum value tested was set equal to 500 m^3/s ($Q_{physical}$ = 1.8 l/s, being $Q_{physical}$ the value of discharge in the physical model). This value corresponds to the spillway design discharge of Arade dam and yet leads to important flood levels in the downstream valley. The maximum discharge tested was the one allowed by the laboratory installation: 27557 m^3/s ($Q_{physical}$ = 100 l/s).

In what concerns unsteady flow regime, four dam-break scenarios (Scenarios A through D) have been defined for the failure of Funcho and Arade dams, for the following conditions:

- Scenario A: single and total failure of Funcho dam;
- Scenario B: “domino” failure of both the dams; total failure of Funcho dam and partial failure of Arade dam;
- Scenario C: single and total failure of Arade dam;
- Scenario D: “domino” and total failures of both the dams.

This paper presents the results for Scenario D.

3.3 1D DAMBRK simulation conditions and domain

In what concerns the domain for the simulations, the DAMBRK model was applied along the 34 km river reach, between a first section in the Funcho reservoir, localized 4.7 km upstream this dam (km 0.0), and the estuary near the city of Portimão (km 34.0). This river reach was characterized by 32 basic cross-sections, obtained from topographical maps at the scale 1:25 000. Only the hydrograph correspondent to a “domino” failure of both dams is used in this paper: Funcho dam was considered to have a total and instantaneous failure and Arade dam to have a total and gradual failure. The dam break flood propagation ended in the sea and the flood hydrograph in the second dam cross section was used as boundary condition for the 2D BIPLAN simulations.

The 1D DAMBRK simulations were carried out considering a Manning-Strickler coefficient for Arade River roughness equal to $20 m^{1/3}/s$. This coefficient translates not only the roughness of the river bottom and banks but also the roughness due to the irregularity of the cross sections along the river valley as well as the local losses, both very important for the dissipation process.

The breach formation process for both dams has been simulated and the correspondent discharge outflow hydrographs calculated, for all four scenarios, using 1D DAMBRK model. Figure 4 shows, for the four dam-break scenarios considered, the calculated outflow hydrographs in the upstream boundary of the physical model (km 10.4).

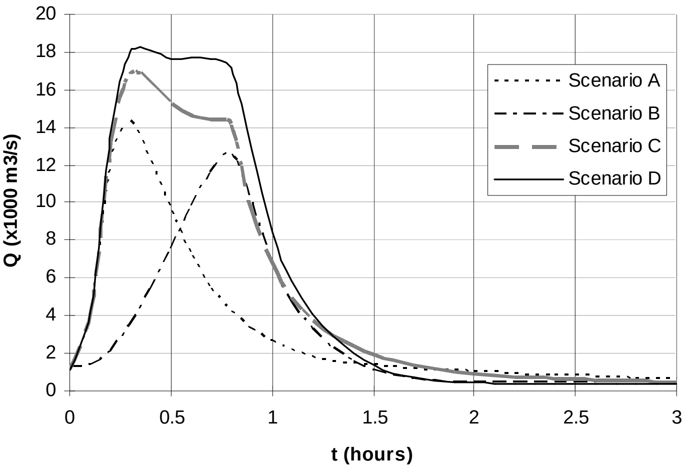

Figure 4: Dalculated dam-breakflow hydrographs used as upstream boundary conditions for unsteady flow regime tests and 2D numerical simulations.

3.4 2D BIPLAN simulation conditions and domain

The domain of the 2D BIPLAN simulations is equal to the physical model domain. The upstream boundary (km 10.4) is localized just downstream the second dam (the Arade Dam). The downstream boundary is localized in the second river narrowing (km 16.4).

The topographic characterization in the 2D simulations is no more undertaken using cross-section of the river valley but is based in a regular and rectangular computational fully and dense grid extended to the entire domain. A digital terrain model was developed including not only the river valley but also the surrounding system, defining an external boundary with the general dimension of 5025 x 4500 m^2. This area was discretized into uniform square grid elements (202x181 nodes); each square being $\Delta x = \Delta y = 25$m. Figure 5 shows the simulation grid and the points where the water levels were calculated (T_{m1} until T_{m6}), which correspond to the same localization of some of the used submersible transmitters in the physical model.

To each element was assigned a location (x,y) characterized by a terrain elevation (Z) and by a Manning-Strickler coefficient. This last varied between 20 and 30 $m^{1/3}/s$. Most important values of roughness are expected to be attributed to 1D models (note that Ks = 20 $m^{1/3}/s$ was assumed to the DAMBRK model) because in 1D models the resistance must translate not only the bottom and river bank energy losses but also the geometry losses that are automatically represented in 2D models.

Simulations were performed for steady flow regimes corresponding to hypothetical floods where the maximum discharge value, varied between 500 m^3/s and 27 557 m^3/s. Four simulations were performed for unsteady flow regimes corresponding to different hypothesis of dam failure. For all four scenarios the upstream dam-break flood hydrograph in a river section just downstream Arade dam was obtained by 1D DAMBRK computational model, which has simulated the breach formation processes and has calculated the correspondent discharge outflow hydrographs. Only the hydrograph correspondent to a “domino” failure of both dams is used in this paper to present results.

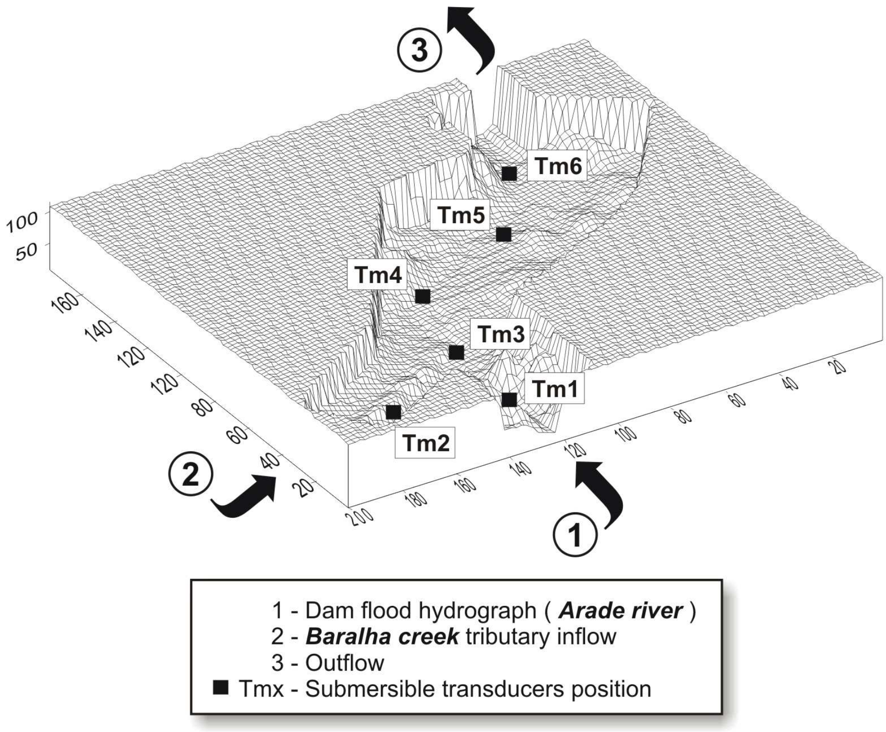

Figure 5: Computational grid for the River Arade valley numerical simulations with submersible transducer positions.

4 RESULT ANALYSIS

A comparative analysis of the results associated to both numerical models and of the experimental data is presented in Figure 6 and concerns flow depths hydrographs in three points of the computational domain.

The following conclusions are achieved from the analysis of Figure 6:

- in cross section (km 12.0), a general good agreement is observed between the three hydrographs; in this zone, the modelled river is narrow and straight, having a preferential 1D flow behaviour and the 1D model reproduces accurately the flood characteristics;
- in cross section (km 14.1), the front wave is slightly more abrupt in the measured and 1D DAMBRK hydrographs and the 2D BIPLAN model smoothes slightly the peak flow;
- in cross section (km 15.4), level measurements are better predicted with 2D BIPLAN model; in this zone the modelled river is basically a flood plain, having a preferential 2D flow behaviour and the 2D model shows a better agreement with the experimental data.
- the results concerning time of flood arrivals shows that the velocity of the 2D BIPLAN numerical flood wave is smaller than the correspondent to the experimental flood.

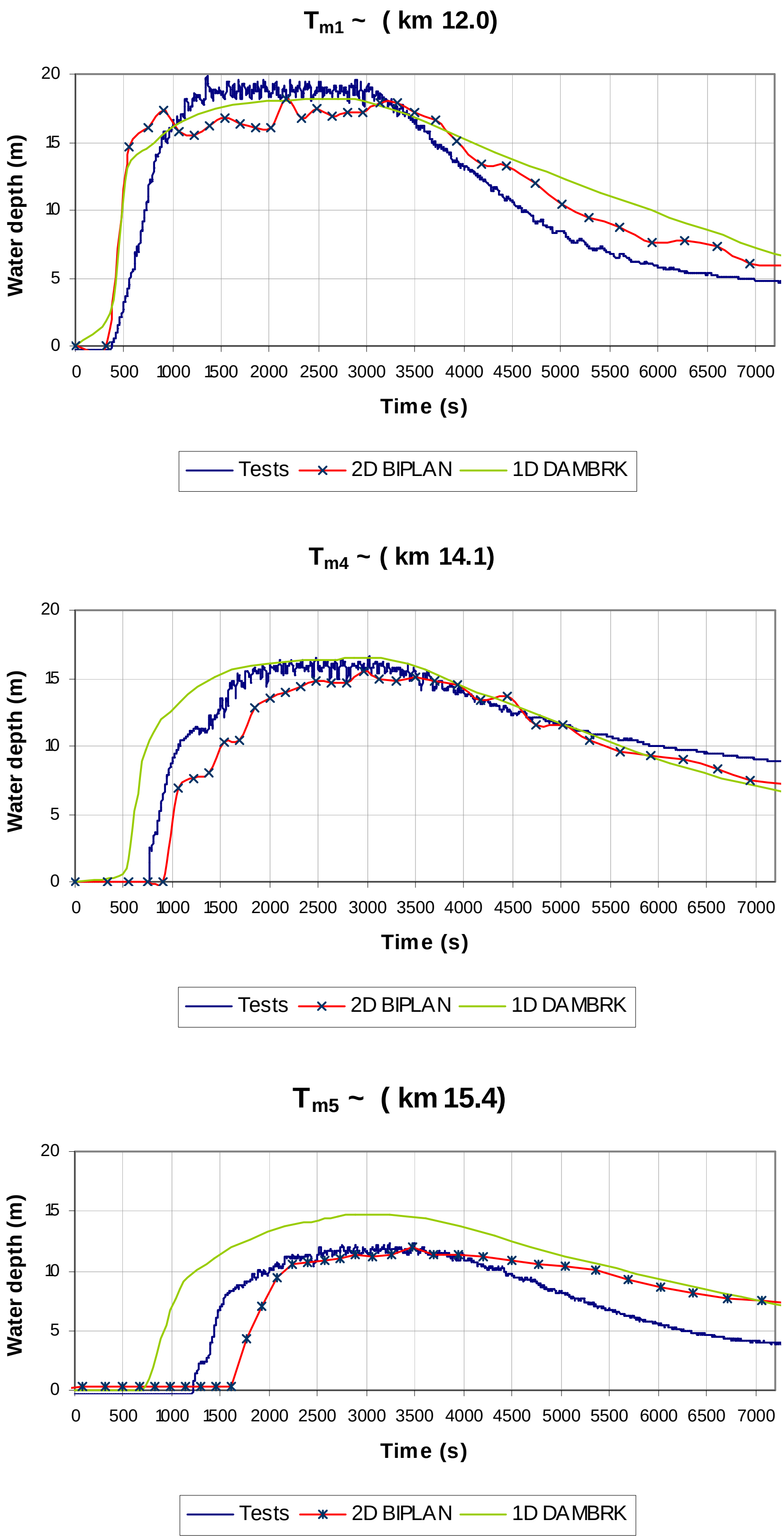

Figure 4: Comparison of the results obtained by 1-D and 2-D computational models and experimental data: a) T_{m1} at (km12.0); b) T_{m4} at (km 14.1); c) T_{m5} at (km 15.4).

5 CONCLUSIONS

In this paper, numerical simulations performed jointly with both 1D and 2D numerical models, showed that the dam-break flood propagation along a downstream valley with irregular topography can present significant differences when a comparison between 1D and 2D numerical results and experimental data is undertaken.

The systematic comparison performed between computed results with 2D BIPLAN and 1D DAMBRK models and measured data in the physical model of Arade River valley showed that the first numerical model presents similar water levels to those obtained with experimental data. The second model seems to overestimate this variable.

Despite the good results, in what concerns water levels, the times of flood arrival computed with 2D BIPLAN model are generally higher than those measured in the physical model. Surely those differences are important in the process of risk mitigation: a smaller time of flood arrival has consequences on, for example, warning population remaining in the downstream valley, compelling to define additional practices for emergency planning.

Finally, the physical model of Arade valley proved that physical models are important tools in the validation process of numerical models, namely in what concerns dam-break floods propagation in valleys with irregular topography, whenever the 1D approach is not valid.

6 ACKNOWLEDGEMENTS

The authors thankfully acknowledge support given by Prof. António Bento Franco in the BIPLAN model development and validation.

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