EPiC Series in Engineering
Volume 3, 2018, Pages 1113–1122
HIC 2018. 13th International Conference on Hydroinformatics

The Computation of hydraulic characteristics of flood flow downstream from the reservoir with dam safety scenarios in North Vietnam

Hien Le Thi Thu[1], Hung Viet Ho[1]

[1] Thuy loi University, 175 Tay Son, Hanoi, Vietnam
Corresponding author: lethuhien.thuyluc@tlu.edu.vn

Abstract

Using a reservoir is an effective solution to prevent lowland flooding and mitigate socio-economic damages. However, due to the high density of river network and the presence of reservoirs, dam safety assurance is becoming one of the most important mission in water resource management in Vietnam. Hydraulic characteristics of dam-break wave are necessary information to generate early warning plans for downstream area of reservoir. To aim this purpose, the Finite Volume Method with Godunov-type is considered to solve two-dimensional shallow water equations and develop a numerical model. In this study, the numerical model for dam-break simulation is suggested and verified through a comparison between calculated results and observed data of two reference tests. Very good agreement shows the effectiveness and accuracy of the proposed model. The Nam Chien reservoir in Vietnam has been chosen and the numerical model is applied to simulate flooding wave for the scenario of arch dam collapse. Alternative solutions are produced, such as: water depth, discharge hydrographs, arrival time, time to reach maximum water level; flooding map. The simulated result implies that this model is an indispensable tool for simulating dam-break scenarios.

1 Introduction

Using a reservoir for regulating flow and downstream flood control is an effective solution. Reservoirs play an important role in preventing lowland flooding and mitigation of socioeconomic damages caused by floods. However, in recent decades, due to the high density of river network and the presence of about 7000 reservoirs, dam safety is becoming one of the most important missions in water resource management in Vietnam. There is an increasing concern in terms of potential risk for the people, properties and facilities located downstream of the dams. Hydraulic characteristics of dam-break wave are necessary information to generate early warning plans for area downstream from the reservoir. Pilotti et al. (2010), Aureli et al. (2014) present a simplify method to predict discharge hydrograph at dam site. However, this method is only suitable for simple reservoir's geometry. Two-dimensional shallow water equation (2D SWE) based on hydrostatic pressure assumption have been widely used to simulate a wide range of surface environmental flow including dam-break flow; urban flooding; etc. Whereas, Finite Volume Method (FVM) with Godunov-type, nowadays, is considered as the most applied numerical strategy to solve 2D SWE because it is a robust numerical scheme to produce accurate and stable numerical solutions for these applications (Toro and Garcia 2007).

The dam of the Nam Chien reservoir is the first arch dam constructed in Vietnam. The maximum dam height is 135m, and the storage volume is 154.106 m3, also contain the potential hazard if the arch dam failures. Therefore, in this paper, the FVM with Godunov type is selected to compute hydraulic characteristics of flood flow such as: peak discharge at dam site; water hydrograph, discharge hydrograph or inundation maps of Nam Chien dam-break scenario.

2 Numerical model

Bases on conservation form of the two-dimensional shallow water equations:

$$\frac{\partial \mathbf{U}}{\partial t}+\frac{\partial \mathbf{K}(\mathbf{U})}{\partial x}+\frac{\partial \mathbf{H}(\mathbf{U})}{\partial y}=\mathbf{S}_1(\mathbf{U})+\mathbf{S}_2(\mathbf{U})$$

(1)

$$\mathbf{U}=\begin{bmatrix} h \\ hu \\ hv \end{bmatrix};\ \mathbf{K}(\mathbf{U})=\begin{bmatrix} hu \\ hu^2+0.5gh^2 \\ huv \end{bmatrix};\ \mathbf{H}(\mathbf{U})=\begin{bmatrix} hv \\ huv \\ hv^2+0.5gh^2 \end{bmatrix};\ \mathbf{S}_1(\mathbf{U})=\begin{bmatrix} 0 \\ -gh\partial z_b/\partial x \\ -gh\partial z_b/\partial y \end{bmatrix};$$

$$\mathbf{S}_2(\mathbf{U})=\begin{bmatrix} 0 \\ -\tau_x/\rho \\ -\tau_y/\rho \end{bmatrix} \quad (2)$$

In (1), U is the vector of conserved variables; K and H are flux vectors;

Here, S1 and S2 are bed slope term and friction term. τx and τy are bottom shear stresses: $\tau_x=\rho C_f u\sqrt{u^2+v^2};\ \tau_y=\rho C_f v\sqrt{u^2+v^2};\ C_f=g.n^2/h^{1/3}$; h is flow depth, u and v are the velocity components in x and y directions; zb is bottom elevation; n is Manning roughness coefficient.

The flow variables are updated to a new time step by the Eq. 3, based on Godunov type,

$$\mathbf{U}_{i,j}^{n+1}=\mathbf{U}_{i,j}^{n}-\frac{\Delta t}{\Delta x}\left[\mathbf{K}_{i+1/2,j}-\mathbf{K}_{i-1/2,j}\right]-\frac{\Delta t}{\Delta y}\left[\mathbf{H}_{i,j+1/2}-\mathbf{H}_{i,j-1/2}\right]+\Delta t\mathbf{S}_{1i,j}+\Delta t\mathbf{S}_{2i,j}$$

(3)

where superscripts denote time levels; subscripts i and j are space indices along x and y directions; Δt, Δx, Δy are time step and space sizes of the computational cell.

The proposed numerical model is written by Fortran 90 language. Several test cases are simulated to validate it to ensure that it is robust, effective and good application (Le 2014). Therefore, it can be applied to a real case study.

3 Validation

3.1. Dam break flow over L-shaped channel

This test was implemented by the CADAM to verify the capacity of the numerical methods, to simulate dam-break flows in the presence of geometrical singularities. The experiment facilities consisted of a square reservoir, which has a bed elevation of -0.33m, and a L-shaped channel forming a vertical step at the entrance to the channel, (Fig.1).

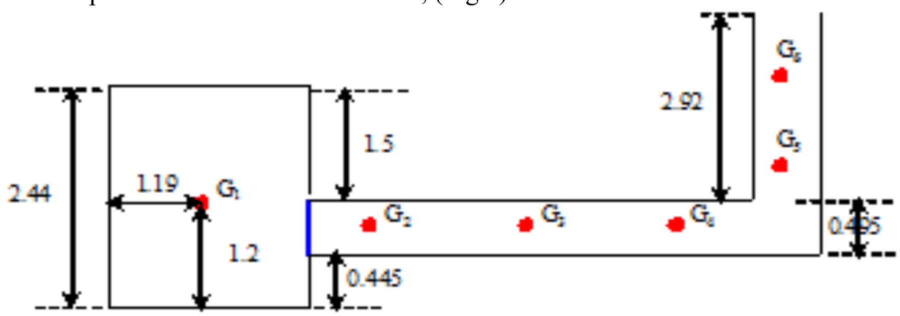

Figure 1: Plan view of reservoir's geometry (dimension in meter).

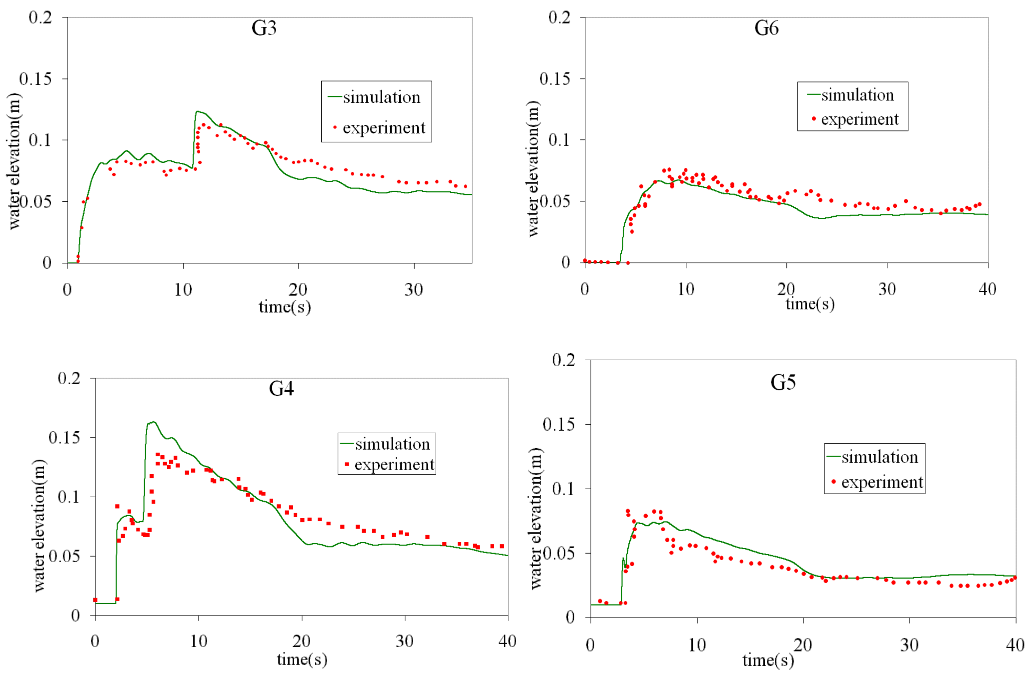

Figure 2: Water hydrographs at stations G_3, G_6 (dry downstream) and G_4, G_5 (wet downstream condition)

The initial water elevation in reservoir is equal to 0.2m. Both dry bed and wet bed conditions are considered. The Manning coefficient n is 0.013. The computational grid size is discretized as 0.04m × 0.02m.

In general, the capacity and the accuracy of the presented model in simulating dam-break flow over vertical step bed and complex domain are well-validated. The arrival time to study points of predicted solutions in both cases are quite the same and quite close to observed data. However, there are some differences against observed data due to the regardless of head loss at vertical step of entrance in the numerical scheme (Guan and Wright 2013).

3.2. Dam-break flow over horizontal floodable area

Aureli et al. (2010) carried out a new methodology based on image technique to gain experimental data of dam-break flow in laboratory of the Parma University. These data are very useful to verify numerical schemes and were also used to this purpose by the same authors. The configuration of the laboratory tests is shown in Fig.3. A rapidly varying flow is induced by the sudden removal of a gate. The following different cases are considered:

Case 1: Partial dam-break flow over horizontal, wet area without obstacle at downstream. The initial water depth in reservoir is 15cm and 1.0cm downstream.

Case 2: Partial dam-break flow over horizontal, dry area with obstacle. The initial water depth at upstream of the gate is the same as in the Case 1.

According to Aureli et al. (2010), computational domain is discretized using a grid size of 5mm × 5mm and Manning coefficient n is set equal to 0.007sm-1/3, threshold water depth is imposed by hε = 4·10-4m and Courant number is 0.9 (Cr = 0.9). Reflective boundary condition is imposed to all sides of the domain. The removal of the gate is considered as instantaneous.

The calculated water depths downstream of the gate at different times for case 1 are shown in Fig. 4 together with experimental data. The presence of an initially wet bottom caused the formation of a shock that moved downstream, and after few seconds, was reflected by the walls of the experimental facility. Clearly, at t = 1.64s, the water wave has not still reached the downstream boundary in both experiment and numerical simulation; anyway, due to the effect of reflected wave, the water depth along side walls are much higher than that in the middle of the floodable area. And then, back wave occurs at t = 2.35s and t = 3.05s, in the figures, because a solid wall is at downstream end.

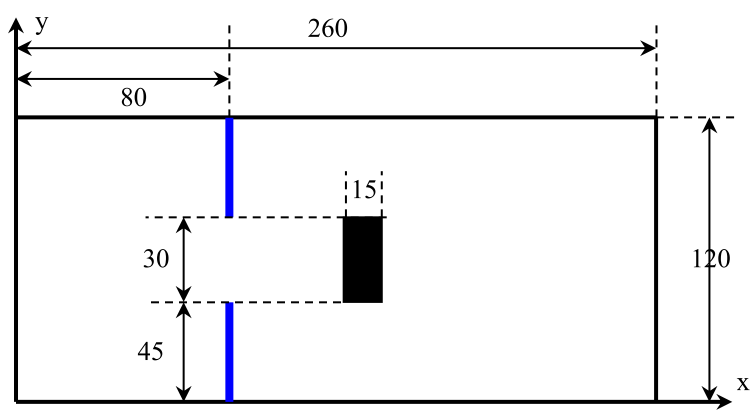

Figure 3: Sketch of computational domain (dimension in cm)

Similarly, Fig. 5 indicates both numerical and experimental results for case 2. Unlikely case 1, an unsubmersible column of 30cm × 15cm is placed downstream of the gate. The presence of the obstacles induced the formation of hydraulic jumps and multiple wave reflections in the flow field. Water released by the gate removal crashes into this obstacle and separated into two parts (see flood maps at t = 0.75s and t = 1.45s). The back wave resulting by three closed boundaries can be also observed in flood maps at t = 2.16s and t = 2.86s. This behavior can be seen in both numerical and experimental solutions.

As consequently, for both case 1 and case 2, the numerical results of flooding map at given time are quite good agreement with those obtained by Aureli et al (2010) as well as the experimental data corresponding with each case in the same paper.

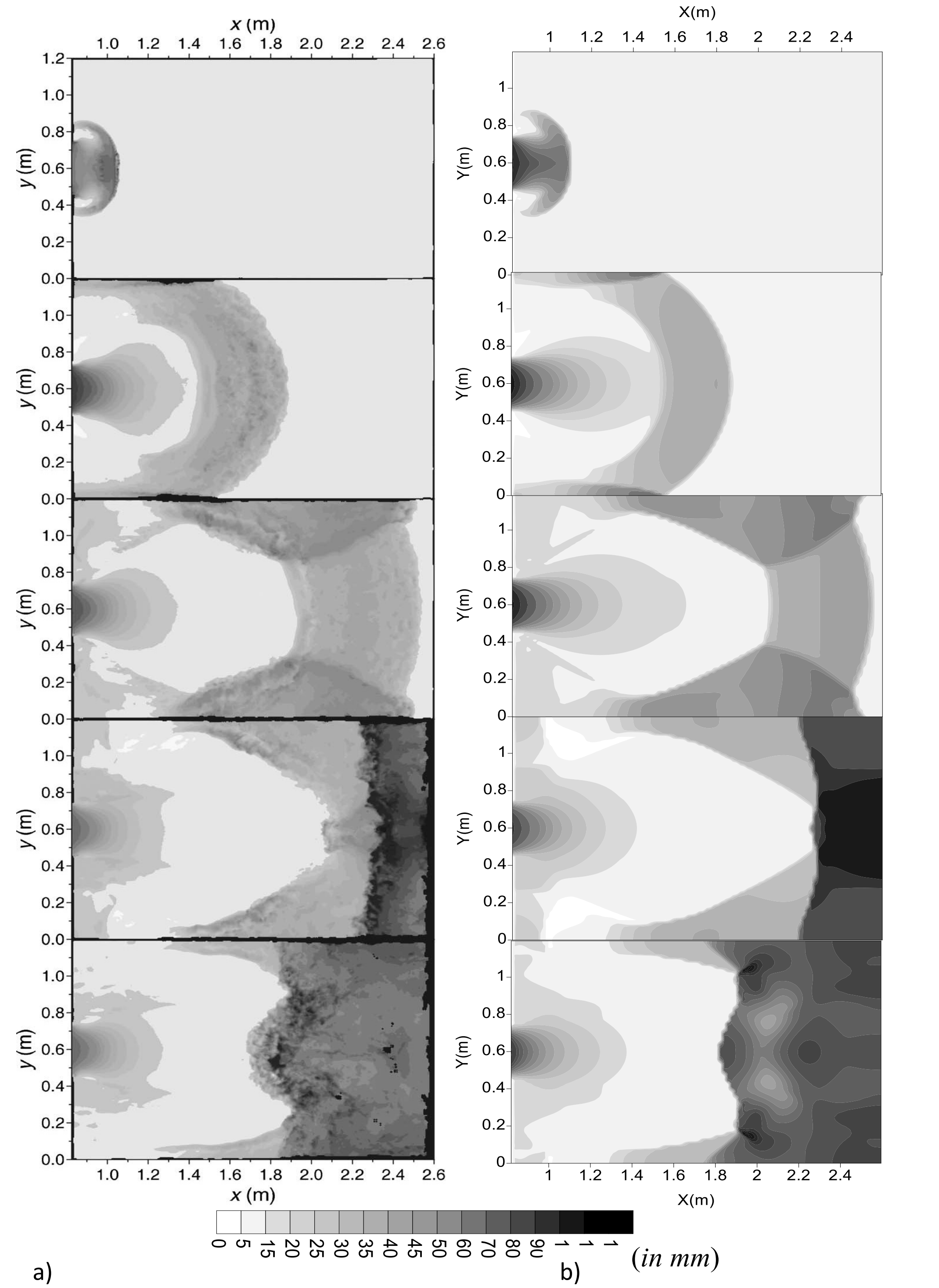

Figure 4: Case 1: Water depth maps in flooded area at: t = 0.24s; t = 0.94s; t = 1.64s; t = 2.35s and t = 3.05s. a) Experimental and b) numerical results.

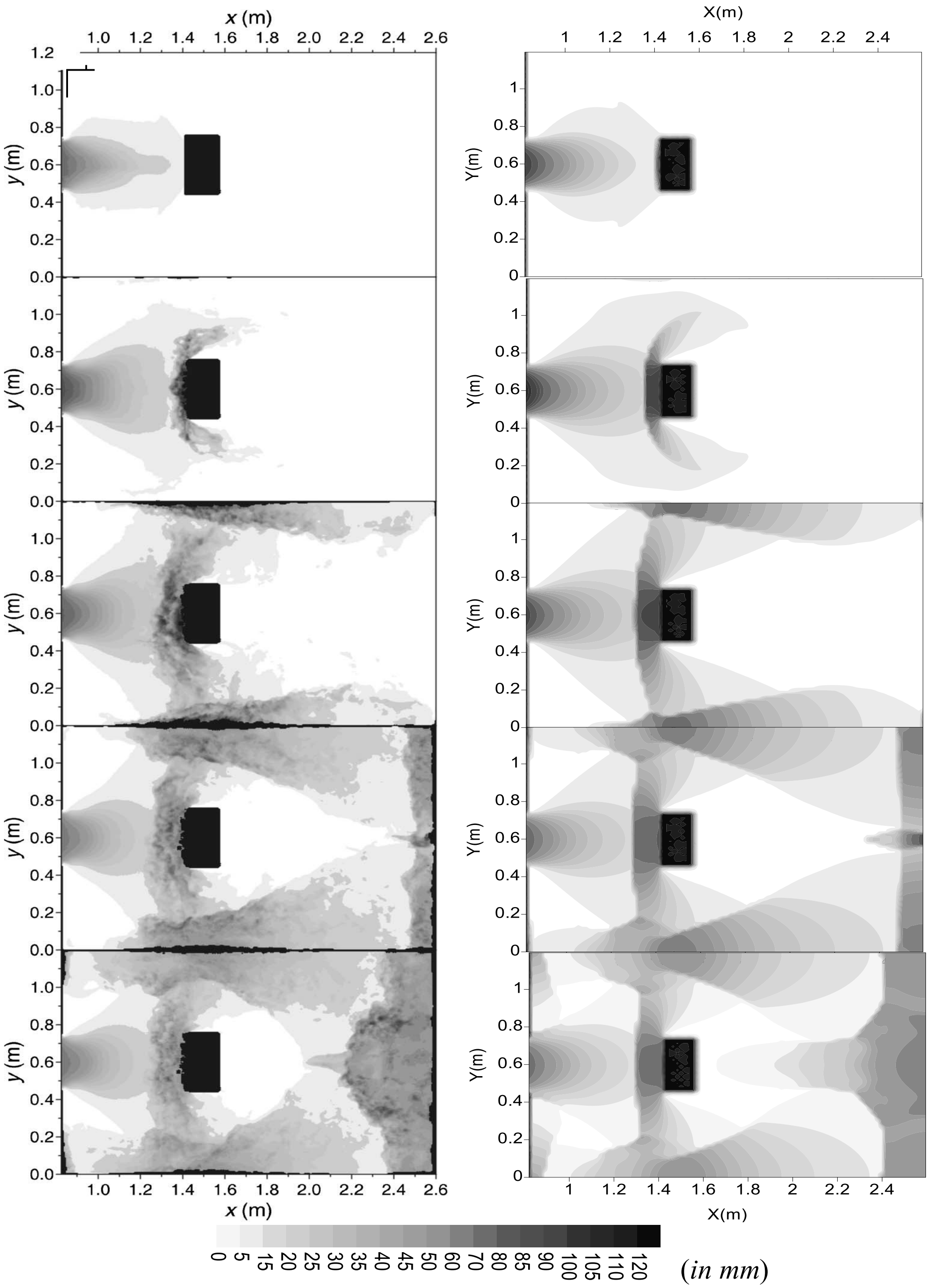

Figure 5: Case 2: Water depth maps in flooded area at: t = 0.40s; t = 0.75s; t = 1.45s; t = 2.16s; t = 2.86s. a) Observed data; b) Numerical result.

4 Application

The numerical model is applied to simulate dam-break wave propagation downstream from the Nam Chien reservoir. The most dangerous scenario of arch dam failure is instantaneous and total dam collapse. Consider initial conditions of water elevation in the reservoir as normal water level of 945m, meanwhile downstream one is dry and Manning coefficient is equal to 0.04. Boundary end is open and others are close.

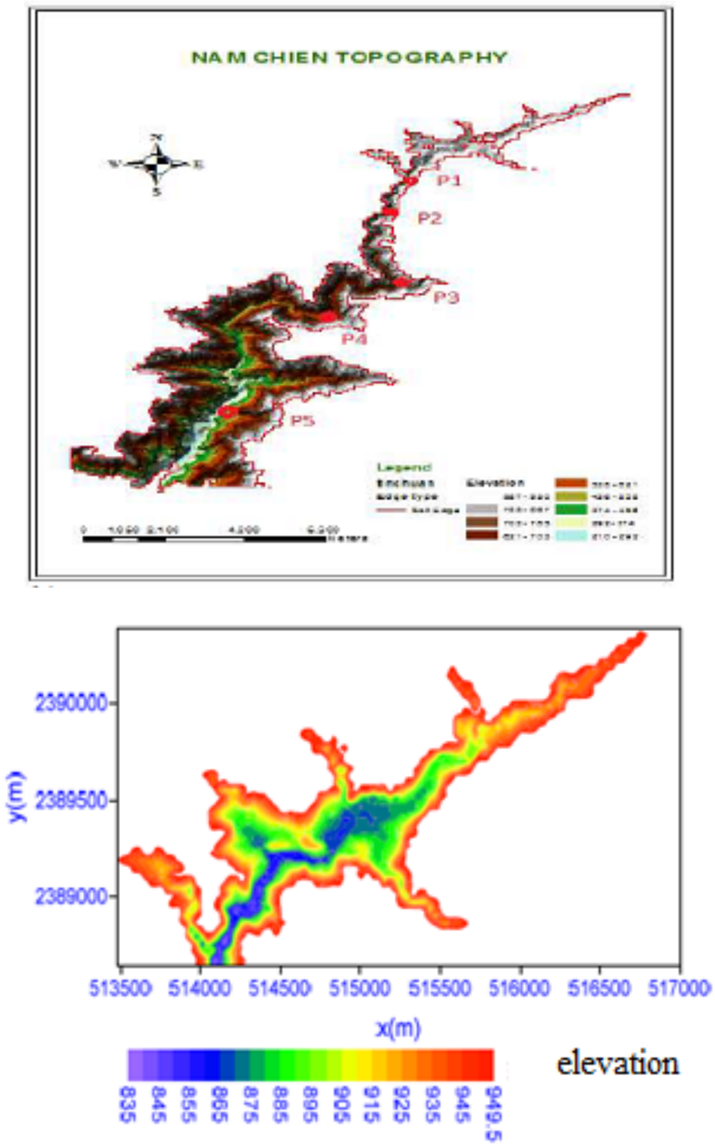

Figure 3: Nam Chien watershed and Reservoir geometry

A computation domain extracted from DEM map of 90m × 90m with its dimensions of 13000m × 14000m is discrete by different resolutions. In general, the finer resolution gives better numerical prediction, however, time consuming is also increased. So, the suitable cell size ensures having acceptable numerical result as well as the computation time is not so expensive should be found out. Firstly, the reservoir domain is divided with different grid sizes to calculate peak discharge at dam site. Then, select the reasonable grid size to generate a computational domain for whole basin. Alternative numerical solutions are indicated: arrive time; time to reach maximum water level and water hydrograph; discharge hydrographs obtained at 5 study points, including: P1 (at dam site), P2, P3, P4 and P5 at hydropower plant (Fig. 3 and Table 2); inundation maps at different times.

Table 1 shows maximum discharge of flood flow at the dam cross section corresponding with different cell sizes. Percentage error is calculated by equation (4).

$$\%_{err} = (Q_{\Delta xi} - Q_{\Delta x \min}) / Q_{\Delta x \min} \times 100\% \tag{4}$$

Clearly, simulation results obtained corresponding to grids of 20m and 30m give error of +7.6% and +12.44%. However, when grid size is coarser 40m, this error is only +3.8%. The grid sizes of 50m and 90m indicate the results quite far from with the others in comparison with solution of grid size of 10m. From that, grid cell of 40m is chosen to apply for all domains of the Nam Chien basin.

Δx (m)	***Q**$_{max}$.**10**^{3} (m^3/s)*	***% error***	**Δx *(m)***	***Q**$_{max}$.**10**^{3} (m^3/s)*	***% error***
10	114.41				
20	123.11	+7.6	40	119.60	+4.5
25	121.78	+6.4	50	132.26	+15.6
30	128.65	+12.44	90	106.52	-6.9

Table 1: Peak discharge at dam site

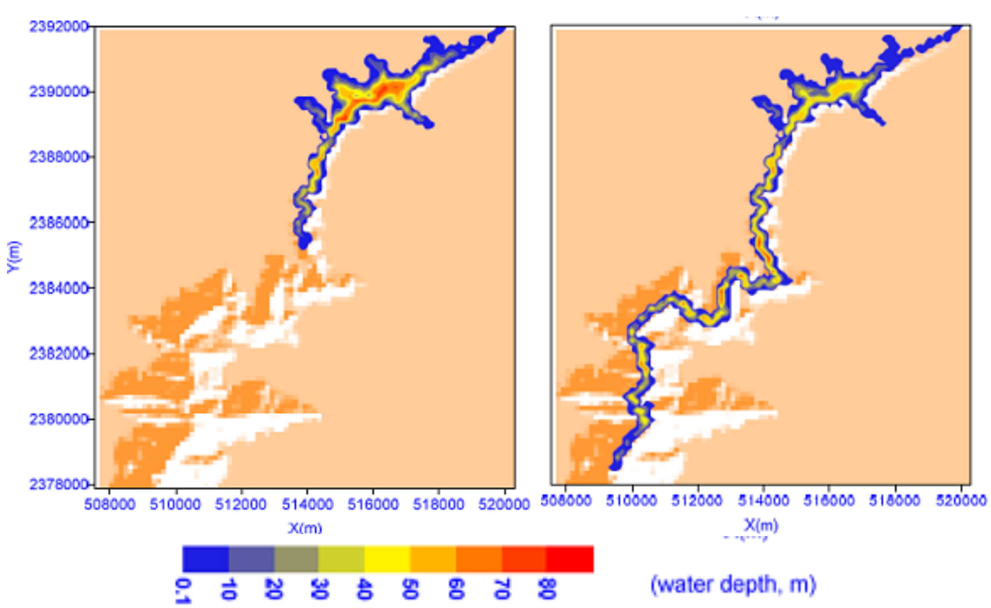

Figure 4: Inundation maps at t = 300s and t = 1700s.

Hydraulic characteristics of arrival time and time to reach maximum water level at these points are presented in table 2. The necessary time for dam-break wave propagating to the power house (P5) is 2000s, and after 2200s, the water level reaches maximum value at P5.

Gauges	Arrival time (s)	Time to reach maximum water level (s)	Maximum water depth (m)
P2	36.6	406	71.5
P3	455.3	1183	57.1
P4	790	1467	48.9
P5	1526	2200	38.3

Table 2: Hydraulic characteristics at study points

5 Conclusion

In this work, the FVM is selected to solve 2D-SWEs on the Cartesian mesh. By two tests presented in this paper, the scheme demonstrated to behave satisfactorily with respect to their effectiveness and robustness in simulating total and partial dam-break flow over complex topographies, which can be able to work with real case study. The dam-break flood flow from the Nam Chien reservoir is simulated by using the presented model to obtain outflow hydrograph and flooding map. This study indicated that the proposed numerical model is an indispensable tool for calculating and simulating dam-break scenarios.

Reference

M. Pilotti, M. Tomirotti, G. Valerio and B. Bacchi. *Simplified Method for the Characterization of the Hydrograph following a Sudden Partial Dam Break.* J. Hydraulic Engineering (ASCE). 136 (2010), No. 10, 693-704.

F. Aureli, A. Maranzoni, P. Migosa, *A semi-analytical method for predicting the outflow hydrograph due to dam-break in natural valleys.* Advances in Water Resources. 63(2014), 38-44.

F. Aureli, A. Maranzoni, P. Mignosa, C. Ziveri. *Dam-break flows: Acquisition o Experimental Data through an Imaging technique and 2D numerical modeling.* J. Hydraulic Engineering. 134(2010), No.8 1089-1101.

E.F. Toro and P. Garcia-Navarro, *Godunov type methods for free surface shallow flows: A review. Journal of Hydraulic Research.* 45(2007) 736- 751.

Le T.T.H, *2D Numerical modeling of dam break flows with application to case studies in Vietnam*, Ph.D thesis (2014), University of Brescia, Italia.