Effect of reservoir side slopes on dam-break flood waves

Farhad Hooshyaripor and Ahmad Tahershamsi*

Faculty of Civil and Environment Engineering, Amirkabir University of Technology, Tehran, Iran

(Received 18 November 2014; final version received 8 April 2015)

1. Introduction

Dam failure is a topic of great importance for hydraulic researchers because in reality this catastrophic phenomenon can result in immense economic damage and loss of lives. A variety of hydraulic, hydrological, geomorphological, geotechnical, and geometric factors affect the outflow characteristics and the consequent damage downstream; however, there are considerable uncertainties involved in the problem (Hooshyaripor, Tahershamsi, & Golian, 2014). It has been shown that the reservoir geometry can influence the dam-break outflow hydrograph. Tahershamsi, Shetty, and Ponce (2003) developed an empirical relationship to relate the peak outflow discharge to reservoir geometry. Feizi Khankandi, Tahershamsi, and Soares-Frazão (2012) investigated the effect of reservoir shape on outflow hydrographs at different points of the downstream channel and showed the considerable impact of this factor on the bell-shaped hydrograph, the value of peak-discharge, and time to peak. Pilotti, Tomirotti, Vale-río, and Bacchi (2010) showed that peak outflow depends on the reservoir bathymetry and the breach area. They defined the valley’s geometry where the dam is located as prismatic. This definition meets a wide range of valley cross sections, from rectangular and parabolic (‘U’ shape) to triangular (‘V’ shape). The results emphasized the effect of reservoir geometry on the outflow hydrograph. Despite the importance of this topic, to the authors’ knowledge, very few works have considered the role of reservoir geometry. A literature review indicates that most of the former investigations have focused on the features of the channel downstream of the dam and the effect of the reservoir geometry has rarely been studied. Therefore, because of the possible influence of the reservoir geometry on the dam-break flow, it seems worthwhile to study the effect of the reservoir’s features on the outflow hydrograph. Accordingly, this paper aims to carry out an in-depth investigation into the differences in outflow hydrographs caused by altering the reservoir side slope, and rout the hydrographs at the downstream channel. To this end, several experiments were conducted and the results were analyzed. The experimental set-up included a reservoir upstream, a rectangular channel downstream, and a gate between them which could be pulled up rapidly. One of the main challenges of this study was to determine the cross section of the reservoir. Feizi Khankandi et al. (2012) considered reservoirs with a rectangular cross section of different dimensions. Pilotti et al. (2010) collected the data for several real valleys of alpine mountain reservoirs and characterized the cross sections with parabolic functions. In this paper, the complicated cross section of real reservoirs was simplified as trapezoidal, in which the lateral diagonal faces can have different slopes. This simplified cross section can easily be constructed in the laboratory on the one hand, and on the other hand it is an acceptable representative for the geometry presented by Pilotti et al. (2010).

The other challenge was the limitation of the apparatus in data acquisition. It was impossible to measure the velocity variations at the gate point and far downstream locations where the low water level influences the tool’s performance. Furthermore, recording the time evolution of

*Corresponding author. Email: tshamsi@aut.ac.ir

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water level and velocities at every point along the reservoir and channel in the laboratory required a considerable amount of time, so in this study the key parameters were measured in a few chosen positions. To overcome these limitations, three-dimensional numerical modeling, which has been used in various water engineering problems (Chau & Jiang, 2001, 2004; Dargahi, 2006; Liu & Yang, 2014), was applied in order to study the temporal and spatial variations of the water level and velocities at any arbitrary location. Recently, two-dimensional shallow water equations (SWEs) have been utilized to provide weak solutions to the dam-break flow (Gallegos, Schubert, & Sanders, 2009; LaRocque, Imran, & Chaudhry, 2013; Liang, Lin, & Falconer, 2007; Pu, Shao, Huang, & Hussain, 2013). Liang (2012) showed that in the location of the dam, the vertical momentum is considerable where the flow changes abruptly, so the flow is no longer horizontal and the pressure distribution is no longer hydrostatic. Ozmen-Cagatay, Kocaman, and Guzel (2014) evaluated the Reynolds-averaged Navier-Stokes (RANS) equations and the SWEs in the modeling of a dam-break flood wave in a flume with a hump downstream and showed that the RANS equations have a slightly better performance. Consequently, the present study employs the full Navier-Stokes equations in studying dam-break phenomena near dam locations to better link the mathematics to the physical features of the studied flow. The rest of the paper is organized as follows. Section 2 describes the experimental apparatus.
and the modalities of the tests. Section 3 presents the calibration and verification steps of the numerical model. Section 4 discusses the experimental and numerical results, and Section 5 reports some conclusions.

2. Experiments

2.1. Experimental set-up

Figure 1 illustrates the experimental set-up, which was located in the Hydraulics Laboratory of the Civil Engineering Department, Amirkabir University of Technology, Iran. The upstream reservoir with a length of 4.5 m and a width of 2.25 m was made of galvanized sheet and fixed by nine steel columns to the laboratory floor. The dimensions of the rectangular flat and horizontal flume downstream are 0.51 m width, 0.7 m height, and 9.5 m length, and its Manning roughness was estimated under steady flow condition as 0.011 m/s^{1/3}. In the set-up there is a gate between the reservoir and the flume which has the ability to be opened instantaneously. The quick gate removal is accomplished with the aid of a pneumatic jack simulating a sudden dam break (Feizi Khankandi et al., 2012). The end of the downstream reach is open (Figure 1). To minimize the interference of the gate and water, the gate is made of Plexiglas plate of 10 mm thickness. The scale effect in a smooth prismatic and rectangular channel is insignificant, provided that the initial water depth against effect in a smooth prismatic and rectangular channel is made of Plexiglas plate of 10 mm thickness. The scale effect in a smooth prismatic and rectangular channel is insignificant, provided that the initial water depth against effect is 0.35 m above the flume bottom and the flume bed is initially dry in all tests. The gate opening time \( t_{op} \) is 0.14 s. Dam-break simulation can be considered as instantaneous if \( t_{op} \) is less than \( t_{cr} = 1.25 \sqrt{h_u/g} \) (Vischer & Hager, 1998), where \( g \) is the gravity acceleration (m/s^2). In this experiment, \( t_{cr} = 0.24 \) s.

2.2. Flow measurement

The \( x \)-coordinate of the measurement points are presented in Table 1. As illustrated in Figure 1(b), all the points are positioned on the centerline. The table also indicates the flow features which were measured at each point. The water-level evolution was measured by means of six ultrasonic sensors and the velocity components were acquired by an Acoustic Doppler Velocimeter (SonTec MicroADV). The ultrasonic sensors with response frequencies up to 100 Hz were placed at points G4 to G14 (see Figure 1). All sensors were installed above the flume on a movable device mounted on a rail at the top of the model. To reduce the side wall boundary-layer effects, the measurement points were positioned at the flume centerline. Water level measurement has an accuracy of ±0.12 mm (Feizi Khankandi et al., 2012). The ADV has response frequencies up to 50 Hz and has been successfully used in dam-break studies (Feizi Khankandi et al., 2012; Soares-Frazão & Zech, 2007). The small diameter of the ADV probe (6 mm) causes local (Soares-Frazão & Zech, 2007) and negligible (Feizi Khankandi et al., 2012) perturbations in the flow.

2.3. Program of the experiments

In the laboratory, the cross section of the reservoirs is considered to be trapezoidal, in which the lateral diagonal faces are representative of the reservoir sides with slopes of \( ZH:IV \). To assess the effect of this feature on dam-break outflow, reservoirs with different side slopes (\( \alpha/Z \)) were examined, where \( Z = 1.73, 1.0, 0.58, \) and 0.0 (\( \alpha = 30^\circ, 45^\circ, 60^\circ, \) and 90°, respectively; see Figure 2). The geometric characteristics of the reservoir are top reservoir width (\( w_t \)), bottom reservoir width (\( w_r \)), initial water depth (\( h_u \)), length (\( l \)), and capacity (\( V_c \)).

In the experiments, the reservoir capacity is considered to be the same in all the tests. In order to have a constant capacity after changing the side slopes in which the cross-sectional area of the reservoir \( A \) increases or decreases, the upstream end of the reservoir needed to be shifted downstream or upstream. Therefore, the reservoir length is a function of the side slope. According to this assumption, the reservoir of 90° side slope is the longest with a length of 4.5 m. Considering the 0.35 m initial water depth in this reservoir with \( w_r = w_t = 0.51 \) m, the reservoir capacity is 0.803 m³ (Figure 3). Since the breach cross section (\( \alpha \)) is unchanged in all the experiments while the reservoir cross section changes as the side slope varies, the value of breach ratio \( a/A \) defined by Pilotti et al. (2010) will depend on the reservoir side slopes.

Table 2 summarizes the reservoir dimensions in the experiments. Based on the number of the reservoirs, instrument limitations (just one ADV and six ultrasonic sensors) and retesting runs to ensure that the results from different

| No. | Point x (m) | \( X = x/h_u \) | Experimental model Water level | Velocity | Numerical model Water level | Velocity |
|-----|-------------|-----------------|-------------------------------|----------|-----------------------------|----------|
| 1   | G1          | 4.38 – 12.51    | –                             | –        | √                           | –        |
| 2   | G2          | 1.88 – 5.371    | –                             | –        | √                           | –        |
| 3   | G3          | 1.00 – 2.857    | –                             | –        | √                           | –        |
| 4   | G4          | 0.12 – 0.343    | √                             | –        | √                           | –        |
| 5   | Gate        | 0.00            | 0.000                         | –        | –                           | –        |
| 6   | G5          | 0.50            | 1.429                         | √        | √                           | –        |
| 7   | G6          | 0.80            | 2.286                         | √        | √                           | –        |
| 8   | G7          | 1.00            | 2.857                         | √        | √                           | –        |
| 9   | G8          | 1.20            | 3.429                         | √        | √                           | –        |
| 10  | G9          | 1.50            | 4.286                         | √        | √                           | –        |
| 11  | G10         | 2.00            | 5.714                         | √        | √                           | –        |
| 12  | G11         | 2.50            | 7.143                         | √        | √                           | –        |
| 13  | G12         | 3.50            | 10.000                        | –        | –                           | –        |
| 14  | G13         | 5.50            | 15.71                         | –        | –                           | –        |
| 15  | G14         | 8.00            | 22.86                         | √        | √                           | –        |
runs could be combined, a total of 312 runs needed to be carried out.

2.4. Observation from the experiments
At time $t = 0$, the gate is removed instantaneously and the water that was at rest in the reservoir abruptly starts to flow to the downstream with a high turbulence. More perturbation in the outflow is seen next to the gate downstream when the side slope decreases. Figure 4(a) shows the generated cross waves downstream from the gate with a reservoir of 30° side slope. These cross waves cause variations of the water level in the downstream channel, both in the transverse and longitudinal directions (Figure 4(b)).

On the other hand, the flow pattern in the reservoir is influenced by the reservoir’s side slopes. A one-dimensional flow pattern is seen if the side walls are vertical. In this case (90° side reservoir), as soon as the gate is opened, a negative wave propagates upstream, while in the other cases two other negative waves are generated which propagate to the side walls (Figure 5). The negative side waves meet the walls and then return to incident in the reservoir centerline. The wave interference continues sequentially until the reservoir depletion, so the resultant periodic perturbation propagates downstream the channel.

3. Numerical model
To determine the flow characteristics at the gate and improve the design of the process equipment, the 3D-module of the Computational Fluid Dynamics (CFD) package FLUENT (ver. 6.3.26) was used to simulate the experiments. FLUENT has previously been applied to and evaluated for dam-break simulation (LaRocque et al., 2013; Maghsoodi, Roozgar, & Sarkardeh, 2012) and modeling a flow below a spillway gate when the gate is opening (Liu & Yang, 2014). The FLUENT software uses the finite volume method to solve the Reynolds-averaged conservation equations for mass and momentum for an incompressible fluid as the governing equations

$$\frac{\partial \tau_{ij}}{\partial x_j} = 0$$

$$\frac{\partial \rho \tau_{ij}}{\partial t} + \frac{\partial \rho \tau_{ij} \tau_{kl}}{\partial x_l} = -\frac{\partial P}{\partial x_j} + \frac{\partial}{\partial x_j} \left[ \mu \left( \frac{\partial \tau_{ij}}{\partial x_k} + \frac{\partial \tau_{kj}}{\partial x_i} \right) \right]$$

$$+ \frac{\partial}{\partial x_j} (-\rho u_i u_j)$$

where $\tau$ and $P = \text{Reynolds-averaged velocities and pressure}$, respectively, $x = \text{Cartesian coordinate axes}$,
Figure 4. (a) Cross waves downstream from the gate at $t = 1.0$ s (30° reservoir) and (b) effect of the cross waves on the flow depth near the channel walls.

Figure 5. Negative waves in the 30° reservoir at (a) $t = 1.0$ s and (b) $t = 2$ s after gate removal.

Figure 6. Boundary condition in the numerical model.

$\rho =$ density of fluid, $t =$ time, $\mu =$ viscosity of fluid, and $-\rho u' u'_i$ is known as Reynolds stress.

In this study the so-called segregated solver was used, so the governing equations were solved sequentially, i.e., segregated from each other. As the governing equations are non-linear and coupled, a number of iterations of the solution loop must be performed to reach a converged solution. In this iterative scheme, all the equations are solved iteratively, for a given time step, until the convergence criteria are met. With this iterative scheme, non-linearity of the individual equations and inter-equation couplings are fully accounted for, eliminating the splitting error. In order to capture the free surface, the Volume of Fluid (VOF) scheme was applied. The back, lateral, and bottom faces were specified as the wall, top and end downstream boundaries respectively as pressure inlet and outlets, and a plane of symmetry was defined along the model (Figure 6). To simulate the initial conditions of dam failure, the area of water in the upstream reservoir was designed to a specified level with air filling the rest of the grid.

3.1. Calibration and verification

The unsteady free-surface calculations required fine-enough grid sizes and small initial time steps. Therefore, various grid sizes and time steps were examined to check
their effectiveness in the simulation. Four sets of grids were created to the dimensions of the experiments using Gambit (ver. 2.3.16), a mesh-generation package: uniform mesh in both the $x$ and $y$ directions with spacing of 15, 30, and 50 mm, and varied mesh sizes of 1–30 mm. Figure 7(a) illustrates the differences between the numerical and experimental water levels for different mesh sizes. Considering a 90° reservoir, two indices of (1) average error (AE) and (2) root mean square error (RMSE) are calculated and presented. In addition, the figure shows the computational time for each of the simulations. Regarding both the accuracy and the computational cost, the grid size of 30 mm was found to be appropriate for the purpose of this study. Again a sensitivity analysis was conducted on the value of the time step by considering $\Delta t = 0.001, 0.003, 0.005, 0.0075, \text{and } 0.01 \text{ s (Figure 7(b))}$. Analysis of the results showed that $\Delta t = 0.003 \text{ s}$ is an acceptable value because it satisfies the accuracy and convergence criteria as well as the Courant-Friedrichs-Lewy condition. Considering $\Delta x = 30 \text{ mm}$ and $\Delta t = 0.003 \text{ s}$, the sensitivity analyses on the different turbulence models, bed roughness values, and pressure-velocity coupling algorithms showed a negligible difference between the results.

Thus, the $k$-$\varepsilon$ turbulence model was used to account for turbulence, the velocity pressure coupling was resolved via a PISO-type, and a nominal roughness height of 0.01 mm was assigned to the wall boundaries. In addition, the PRESTO! pressure discretization scheme and the first-order upwind momentum and turbulent kinetic energy discretization scheme were applied (Karki & Patankar, 1989). Convergence is reached when the scaled residual of each variable is in the order of 0.001. Moreover, the free surface is defined by a value of 0.5 for VOF (Fluent Manual, 2006). In FLUENT, a time step different from the one used for the rest of the transport equations is defined for the volume fraction calculation. This time step was refined based on the input for the maximum Courant number allowed near the free surface (Fluent Manual, 2006). In this study the Courant number of 0.25 was predefined. In order to verify the calibrated numerical model, the 90° reservoir was simulated in FLUENT. Figure 8 compares the numerical and experimental results at point G6. Table 3 shows the average difference between the numerical and experimental results at the points G5 and G6.

4. Analysis of the results

In Figure 9 the experimental results of the water level variations for the 90° reservoir and channel (points G4 and G6, respectively) are compared to the analytical solution of Ritter (1892). In the figure, two vertical dashed lines indicate the time limits in which the limited upstream
Table 3. Comparison between numerical and experimental results at points G5 and G6.

| Non-dimensional parameter          | G5 ($X = 1.429$) | G6 ($X = 2.286$) | Average at points G5 and G6 |
|------------------------------------|------------------|------------------|-----------------------------|
|                                    | RMSE AE (%)      | RMSE AE (%)      | RMSE AE (%)                 |
| Water level ($H$)                  | 0.07 32.3        | 0.06 23.2        | 0.07 27.7                   |
| Velocity ($U$)                     | 0.05 $-10.6$     | 0.04 5.9         | 0.05 $-2.3$                 |
| Flow discharge ($Q$)               | 0.01 $-8.1$      | 0.03 18.5        | 0.02 5.2                    |

Figure 9. Experimental and analytic (Ritter, 1892) results of temporal water level variation at $X = -0.343$ (point G4) and $X = 2.286$ (point G6).

reservoir length starts to influence the wave levels at the measuring points. The value of time limit is estimated as $T_{\text{limit}} = (2L + X)/CNF$, where $L = l/h_u =$ dimensionless reservoir length and $CNF =$ dimensionless negative wavefront celerity (Leal, Ferreira, & Cardoso, 2009). The figure indicates that the water level curve of the Ritter solution separates from the experimental curve from $T = T_{\text{limit}}$ such that for $T > T_{\text{limit}}$, the Ritter results get close to the constant value of $H = 4/9$, while the experimental data decrease gradually due to reservoir depletion. Figure 9 shows that the water level in the reservoir decreases more rapidly when the Ritter solution is used. Moreover, the wave-front in the Ritter solution reaches G6 sooner than the experimental results.

Figure 10 illustrates the attained experimental and numerical results at point G5. The vertical dashed lines show the $T_{\text{limit}}$ values of the reservoirs and from left to right correspond to the $30^\circ$, $45^\circ$, $60^\circ$, and $90^\circ$ reservoirs, respectively. According to Figure 10(a), the value of $H_{\text{max}}$ (= $h_{\text{max}}/h_u$) and its corresponding time depend on the side slope of the reservoirs such that the corresponding time of the maximum water level reduces as the side slope decreases. Thus, the rising limb of the curve corresponding to a $30^\circ$ reservoir is sharper than the other ones. Note also that the $30^\circ$ reservoir has the highest maximum water level but a faster attenuation of the peak points. Figure 10(b) compares the temporal velocities at G5. The figure shows that the side slope value highly affects the velocity of the dam-break flow downstream. Also, it can be seen that the experimental curves of the $45^\circ$ and $60^\circ$ reservoirs have some fluctuations, while the curve of the $90^\circ$ reservoir is smooth. The fluctuation could relate to the side negative waves in the reservoir which are transported downstream. Note that Figure 10(b) does not include the experimental $30^\circ$ side slope reservoir because of the high turbulent flow which affects the ADV performance. The results show that $U_{\text{max}}$ (= $u_{\text{max}}/(gh_u)^{0.5}$) of the $45^\circ$ and $60^\circ$ reservoirs are approximately 30% and 21% greater than that of the $90^\circ$ reservoir, respectively; thus, the lower the side slope, the higher the maximum average velocity at downstream locations. Moreover, it can be observed that numerical modeling could not simulate these perturbations very well. Figure 10(c) shows the dam-break flow rates at point G5. According to this figure, the outflow hydrograph of the $30^\circ$ reservoir is sharper than the others, i.e., the higher $Q_{\text{max}}$ (= $q_{\text{max}}/(gh_u)^{3/2}$) values occur at the earlier times, while the area under the curves is the same (due to the same capacity of the reservoirs). According to Figure 10(c) the peak outflow values of the $30^\circ$, $45^\circ$, and $60^\circ$ reservoirs are almost 23%, 16%, and 11% more than that of the $90^\circ$ reservoir, respectively. Note that the rapidly rising limb of the hydrographs is missed in the experiments. Also the $T_{Q_{\text{max}}}$ value occurs faster when the reservoir has lower side slopes, such that this amount for $30^\circ$, $45^\circ$, $60^\circ$, and $90^\circ$
reservoirs is calculated as 6.35, 11.12, 15.09, and 15.88, respectively (from the numerical results).

Figure 11 shows the numerical results in comparison with the data achieved from the experiments on the 45° reservoir. The figure also illustrates the analytical results based on the Ritter (1892) solution, which are time-independent at the gate location ($X = 0$). As it is shown, the Ritter solution underestimates the water level and overestimates the velocity; however, the results are somehow satisfactory for the flow rates. At G5, G10, and G13 it is observed that the numerical and experimental results are in good agreement for water level, although the fluctuations have not been simulated in the numerical model. Moreover, Figure 11(a) shows that in each location the numerical
wave is earlier than the experimental wave. According to Figure 11(b), in the case of the 45° reservoir the velocity measurement by ADV has not been possible at points G10 and G13, so the experimental hydrographs have not been calculated for these points.

In Table 4 the values of the maximum water level and the outflow discharge from the numerical analysis are calculated and summarized. In this table the values for another reservoir with 75° side slopes are also presented. These values are presented graphically in Figure 12. It can be seen that the influence of side slopes is much more considerable at the points close to the gate but negligible for $X \geq 5.714$. Moreover, the figure indicates that the water level is affected considerably by the reservoir side slope such that the maximum water level at the gate position for the case of the 30° reservoir is almost 58% greater than that
Table 4. Maximum value of dam-break outflow parameters at downstream locations.

| X   | Hydraulic parameter | α (deg) | 90   | 75   | 60   | 45   | 30   |
|-----|---------------------|---------|------|------|------|------|------|
| 0.0 | $H_{max}$           | 0.733   | 0.666| 0.596| 0.534| 0.465|      |
|     | $Q_{max}$           | 0.371   | 0.339| 0.312| 0.291| 0.257|      |
|     | $T_{H_{max}}$       | 7.145   | 11.910| 16.670| 19.850| 19.850|      |
| 0.571| $Q_{max}$           | 0.389   | 0.339| 0.306| 0.279| 0.255|      |
|     | $T_{Q_{max}}$       | 5.550   | 11.910| 10.320| 14.290| 17.460|      |
|     | $H_{max}$           | 0.458   | 0.436| 0.423| 0.420| 0.428|      |
|     | $T_{H_{max}}$       | 13.490  | 15.830| 18.260| 21.440| 20.900|      |
| 1.429| $Q_{max}$           | 0.321   | 0.292| 0.274| 0.261| 0.253|      |
|     | $T_{Q_{max}}$       | 6.350   | 11.110| 14.290| 17.460| 15.880|      |
|     | $H_{max}$           | 0.397   | 0.396| 0.390| 0.385| 0.418|      |
|     | $T_{H_{max}}$       | 15.850  | 18.260| 20.640| 23.030| 22.230|      |
| 2.286| $Q_{max}$           | 0.286   | 0.273| 0.253| 0.241| 0.252|      |
|     | $T_{Q_{max}}$       | 8.730   | 16.680| 16.670| 18.260| 20.640|      |
|     | $H_{max}$           | 0.366   | 0.355| 0.330| 0.343| 0.340|      |
|     | $T_{H_{max}}$       | 18.710  | 22.230| 18.430| 27.790| 27.600|      |
| 5.714| $Q_{max}$           | 0.271   | 0.262| 0.247| 0.234| 0.235|      |
|     | $T_{Q_{max}}$       | 16.670  | 17.470| 19.850| 23.810| 23.810|      |
|     | $H_{max}$           | 0.239   | 0.250| 0.252| 0.255| 0.263|      |
|     | $T_{H_{max}}$       | 27.790  | 30.170| 32.550| 35.730| 35.730|      |
| 15.714| $Q_{max}$           | 0.191   | 0.198| 0.195| 0.188| 0.187|      |
|      | $T_{Q_{max}}$       | 25.400  | 29.380| 30.960| 32.560| 30.170|      |

Figure 12. Effect of the side slope on the maximum water level and discharge.

of the 90° reservoir, while this percentile is about 44% for discharge values at the same position.

5. Conclusion

Several investigations have shown that reservoir geometry can affect dam-break outflow and consequently the flood damage that occurs downstream. The geometry of a natural reservoir is extremely complicated, so it is difficult to model such geometry in the lab – but it is possible to simplify its cross section as parabolic or trapezoidal. Assuming trapezoidal geometry for the reservoirs, in this paper experimental and numerical attempts were conducted on dam-break outflows from those reservoirs with different side slopes of 30°, 45°, 60°, and 90° to analyze the water level, velocities, and discharge values at different points. Such side slope values could represent the reservoirs located in the mountain valleys. To attain a robust comparison over the outflow hydrographs, the reservoir capacity was taken unchanged in the four models; thus, the reservoir length decreases as the degree of the side slope reduces. Measurement of the key parameters was not possible at every arbitrary point, so FLUENT CFD software was applied to attain the temporal and spatial variations of the parameters at any position. Comparison of the results showed that the reservoir side slope is a parameter that affects the emptying process, such that the outflow hydrograph from the 30° side slope reservoir had the highest peak-discharge
and the earliest time to peak. According to the results, a shorter emptying time could be expected from reservoirs with lower side slopes. Results indicated that the side slope has a greater effect on the water level than the peak-discharges. Overall, in terms of the risk assessment, it can be concluded that reservoir geometry has a considerable impact on the dam-break flood wave. Therefore, because of the variety of shapes and sections of actual reservoirs, their complex behavior in the case of dam failure, and to the authors’ knowledge the lack of studies in this area, it would be worthwhile to perform further investigations and study more deeply the effect of reservoir features on dam-break flow.

Disclosure statement
No potential conflict of interest was reported by the authors.

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