

2D Dam-break Flows over Erodible Floodplain. Comparison of Two Shallow-water Models against Laboratory Experiments: a Saint-Venant – Exner Classical Approach and a Two-layer Description

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ABSTRACT: Dam or dyke collapse constitutes a major danger for the surrounding inhabitants and infrastructures. Indeed, it induces a fast transient flow that may entrain large amounts of sediments and modify the valley morphology. The understanding of the interaction between such a rapid flow and the sediment transport remains a challenge for fluvial hydraulics. A combined approach of laboratory, theoretical and numerical modelling is still needed to predict the geomorphic consequences of a dam failure, mainly in a complex topography. A laboratory study of a dam-break wave expanding over a flat mobile bed in a floodplain was conducted in the Hydraulics Laboratory, Institute of Mechanics, Materials and Civil Engineering, Université catholique de Louvain, Belgium. The flume was 3.6 m wide and about 36 m long. The rising gate featuring the dam break was 1 m wide, inducing a wave expanding longitudinally as well as transversally. An initial 85 mm layer of coarse sand was put down upon the fixed bed from 1 m upstream to 9 m downstream of the gate representing the dam. Two cases were simulated: (1) an initial water level of 47 cm above the fixed bed in the upstream reservoir and no water downstream; (2) an initial water level of 51 cm in the upstream reservoir and a water level of 15 cm downstream. Time evolution of the water level was measured at 8 points by means of ultrasonic gauges and the final topography was measured by a bed profiler every 5 cm. Only bed-load transport was observed in these experiments. This laboratory test was carried out as a benchmark experiment in NSF-Pire project (<http://pire.ce.sc.edu/>) and allowed two-dimensional (2D) numerical models from several authors to be analysed. Among these models, two shallow-water models are here compared in details. In the first model, a clear-water layer approach is used: the Saint-Venant–Exner (SVE) equations are solved, using a Manning formula for the friction source term and a Meyer-Peter–Müller closure equation based on an equilibrium solid transport assumption. The second model is a two-layer model (2L), where the flow is described along a vertical by three regions of homogeneous properties separated by sharp interfaces. The upper layer is made of clear water while the lower layer, standing for the bed-load transport layer, is a mixture of water and moving sediments. These layers flow over a motionless bed. Both models are solved on unstructured triangular meshes by a finite-volume scheme with a lateralized Harten-Lax-Van Leer (HLL) flux computation. Interesting conclusions may arise from comparisons between both models and comparisons with laboratory observations. The main features experimentally observed are captured by the numerical models. The water level evolution is better reproduced by the two-layer model than by the SVE model. The final deposition crests near the walls are still difficult to predict by both models.

KEY WORDS: Sediment transport, Mobile bed, Dam-break flow, Finite volume, 2D models.

1 INTRODUCTION

Dam-break waves over mobile beds can convey great amounts of sediments, potentially inducing a considerable morphological evolution and a major risk for the surrounding lives and infrastructures. The

total volume of entrained material may be comparable to the volume of released water in some extreme cases (Capart and Young 2001). To be able to predict and conveniently mitigate the consequences of a dam failure, the interactions between the water flow dynamics, the bed erosion / deposition and the sediment transport have to be understood, for instance thanks to a combined approach of laboratory experiments, theoretical considerations and numerical models.

Among some previous dam-break experiments over mobile beds in a two-dimensional (2D) geometry, a dam-breach flow was investigated in a basin made of fixed bed, uniform sediment, and non-uniform sediment, respectively (Xia et al. 2010), an abrupt widening was analysed by Goutière et al. (2011) and a wave expansion on a floodplain made of coarse sand was proposed as another benchmark test for two-dimensional numerical models by Soares-Frazão et al. (2012). In this later case, water levels and the final topography were measured.

Theoretical and numerical models could then be validated against these laboratory measurements. The first attempts to simulate dam-break flows over mobile beds used a shallow-water formulation of the mass balance of each phase, i.e. liquid and solid phase, and momentum balance for the whole mixture. The solid phase description often reduces to the bed time-evolution following erosion and deposition, neglecting both the suspended load and the inertia effects of the moving sediment phase to be mobilised. In that frame, assuming hydrostatic pressure, incompressible fluid and small slope, the Saint-Venant–Exner set of equations is obtained (Lyn and Altinakar 2002, Soares-Frazão and Zech 2010). These models are closed with formulas for the energy slope and for the bed-load transport (see El Kadi Abderrezzak and Paquier 2011 for a comparison of these formulas). In addition, a failure operator could be included into these shallow-water models in order to predict bank morphological evolution (Swartenbroekx et al. 2010).

Another way, now accounting for the solid transport inertia, is to develop two-layer approaches. Fraccarollo and Capart (2002) separated the vertical structure of a dam-break flow in three areas: (1) an upper layer made of clear water, (2) a lower layer that is a mixture of water and sediments, representing a bed-load transport layer, (3) the motionless bed. In the model adopted by Spinewine and Zech (2005) and Zech et al. (2009), the depth-averaged velocities are decoupled in the two flowing layers and the granular concentration are assumed distinct between the transport layer and the motionless bed in order to account for the required granular phase dilatation when sediments from the motionless bed are mobilised into the bed-load layer. Mass and momentum balances are imposed separately in each layer while closure equations express the shear-stresses at interfaces between layers and along the bed. A 2D extension of this two-layer model is presented by Swartenbroekx (2012) and Swartenbroekx et al. (2013).

In this paper, a comparison is made between the results of two 2D shallow-water models against the laboratory test of a wave expansion on a floodplain: a Saint-Venant-Exner (SVE) and a two-layer model (2L). The paper is organised as follows. The laboratory set-up and main results are first summarised. The governing equations and the numerical solution of both numerical models are then recalled. Comparisons of the results are discussed before drawing some conclusions.

2 LABORATORY DAM-BREAK WAVE EXPANSION OVER AN ERODIBLE FLOODPLAIN

A 2D dam-break wave expanding over a flat mobile bed was carried out as a benchmark experiment in NSF-Pire project (Soares-Frazão et al. 2012). The test was conducted in the Hydraulics Laboratory, Institute of Mechanics, Materials and Civil Engineering, Université catholique de Louvain, Belgium. The flume was 3.6 m wide and about 36 m long (Figure 1). The gate featuring the dam was 1 m wide and its rise induces a wave expanding longitudinally as well as transversally. An initial 85 mm layer of coarse sand was put down upon the fixed bed, from 1 m upstream to 9 m downstream of the gate representing the dam. Two cases were simulated: (1) an initial water level of 47 cm in the upstream reservoir and no water downstream; (2) an initial water level of 51 cm in the upstream reservoir and a water level of 15 cm above the fixed bed downstream. Time evolution of the water level were measured at 8 points by means of ultrasonic gauges (locations in Figure 2) and the final topography was measured by a bed profiler every 5 cm. From the measured bed profiles, a contour map of the final bed topography was reconstructed.

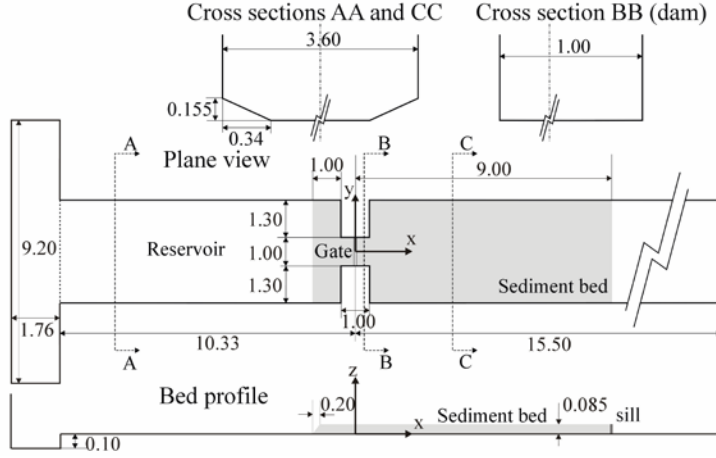


Figure 1 Experimental set-up, with dimensions in metres.

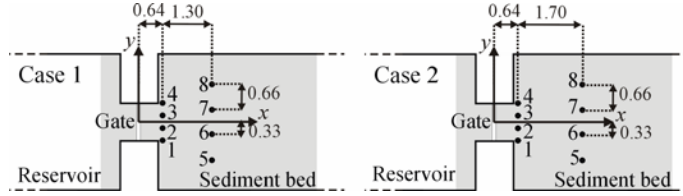


Figure 2 Ultrasonic gauge location.

3 SHALLOW-WATER MODELS

3.1 Saint-Venant-Exner model

The 2DH Saint-Venant equations describe mass and momentum conservation for water while the Exner equation describes the bed evolution following erosion or deposition. These equations form a coupled system that can be written in vector form as

$$\frac{\partial \mathbf{U}}{\partial t} + \frac{\partial \mathbf{F}(\mathbf{U})}{\partial x} + \frac{\partial \mathbf{G}(\mathbf{U})}{\partial y} = \mathbf{S} \quad (1)$$

with

$$\mathbf{U} = \begin{pmatrix} h_w \\ u_{wx} h_w \\ u_{wy} h_w \\ z_b \end{pmatrix} = \begin{pmatrix} h_w \\ q_{wx} \\ q_{wy} \\ z_b \end{pmatrix}, \quad \mathbf{S} = \begin{pmatrix} 0 \\ gh_w (S_{0,x} - S_{f,x}) \\ gh_w (S_{0,y} - S_{f,y}) \\ 0 \end{pmatrix}, \quad (2a-b)$$

$$\mathbf{F} = \begin{pmatrix} q_{wx} \\ q_{wx}^2/h_w + gh_w^2/2 \\ q_{wx} q_{wy}/h_w \\ q_{sx}/(1-\varepsilon_0) \end{pmatrix}, \quad \mathbf{G} = \begin{pmatrix} q_{wy} \\ q_{wx} q_{wy}/h_w \\ q_{wy}^2/h_w + gh_w^2/2 \\ q_{sy}/(1-\varepsilon_0) \end{pmatrix} \quad (2c-d)$$

where $\mathbf{U}$ is the vector of conserved variables that are the unknowns of the problem, $\mathbf{F}$ and $\mathbf{G}$ are the streamwise and transverse flux vectors, $\mathbf{S}$ is the source term vector, x and y are co-ordinates in a given Cartesian system, for instance, the main-streamwise and transverse co-ordinates, respectively, h_w is the

water depth, u_{wx} and u_{wy} are the depth-averaged velocity components in the x and y directions, respectively, q_{wx} and q_{wy} are the unit water discharge components, z_b is the erodible bed elevation, q_{sx} and q_{sy} are the components of the sediment transport rate per unit width, ε_0 is the bed porosity, g is the gravitational acceleration, $S_{0,x}$ and $S_{0,y}$ are the bed slope components and $S_{f,x}$ and $S_{f,y}$ are the friction slope components, respectively.

The friction slope S_f is provided by the Manning formula. Only bed load is considered as sediment transport. The sediment transport rate q_s is obtained for instance from Meyer-Peter and Müller (1948) as

$$q_s = 8 \sqrt{g(s-1)d_{50}^3} (\tau_* - \tau_{*c})^{3/2} \quad (3)$$

where s is the ratio of sediment to water densities, d_{50} is the median grain diameter, τ_* is the dimensionless bed shear stress (Shields number) and τ_{*c} is its critical value. It is considered that sediments on the bed start to move if $\tau_* > \tau_{*c} = 0.047$.

3.2 Two-layer model

The proposed two-dimensional (2D) two-layer shallow-water description considers an upper layer made of clear water (denoted by the subscript w) and a lower layer, called the bed-load transport layer, made of a dense mixture of water and moving grains (denoted by the subscript s). These layers flow over a motionless bed (denoted by the subscript b) and present distinct depth-averaged velocity components u_{wx} , u_{wy} , u_{sx} and u_{sy} (Figure 3).

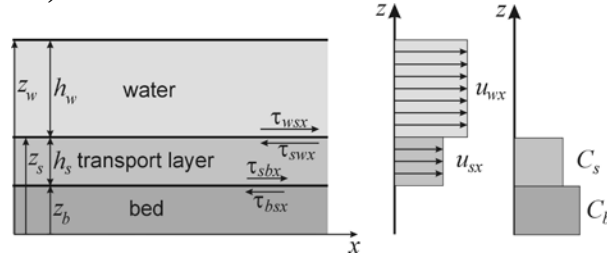


Figure 3 Two-layer shallow-water description.

Neither wash-load nor suspension is considered in the clear-water layer and only bed-load transport occurs in the lower layer. The volumetric sediment concentrations C_s and C_b in the transport layer and in the bed, respectively are assumed constant in time and space, but distinct. They are specified a priori thanks to experimental observations. For the motionless bed, the granular concentration $C_b = 0.53$ was measured by analysing bed samples taken in laboratory conditions (Spinewine and Zech 2007). For the bed-load transport layer, the granular concentration $C_s = 0.22$ was estimated from images of a 1D dam-break test, comparing areas of the motionless bed and areas of the transport region at several times (Spinewine 2005).

Similarly to most dam-break wave models, shallow-water assumptions are adopted, implying hydrostatic pressure distribution in both layers. This assumption is invalid in the so-called near-field, i.e. the early stage of wave formation where vertical accelerations are present; but applies when the wave moves further downstream in the so-called far-field (Zech et al. 2008).

The bed erosion rate e_b is defined as the evolution of the bed upper level z_b along time, due to the vertical sediment transfer between the bed and the transport layer, and is positive in case of erosion:

$$\frac{\partial z_b}{\partial t} = -e_b \quad (4)$$

This relation is considered as one of the governing equation. In a similar way, the rate e_s is related to the displacement $-\partial z_s / \partial t$ of the interface between the two flowing layers. Both erosion rates can be linked. Indeed, the assumption of distinct depth-averaged granular concentration in the transport layer and the bed allows the required dilatation of the transport layer during the bed erosion, affecting not only the bed level z_b but also the top level z_s of the bed-load transport layer. The vertical conservation of the mass in the whole system in a static configuration, i.e. assuming no change in water level z_w (Zech et al. 2009),

yields

$$e_s = -\frac{C_b - C_s}{C_s} e_b \quad (5)$$

According to these definitions of the erosion rates, the continuity equation is written for the clear-water and for the bed-load transport layers, respectively:

$$\frac{\partial h_w}{\partial t} + \frac{\partial q_{wx}}{\partial x} + \frac{\partial q_{wy}}{\partial y} = e_s \quad (6)$$

$$\frac{\partial h_s}{\partial t} + \frac{\partial q_{sx}}{\partial x} + \frac{\partial q_{sy}}{\partial y} = e_b - e_s \quad (7)$$

The 2D momentum conservation is written for each layer:

$$\frac{\partial q_{wx}}{\partial t} + \frac{\partial}{\partial x} \left(\frac{q_{wx}^2}{h_w} + \frac{1}{2} g h_w^2 \right) + \frac{\partial}{\partial y} \left(\frac{q_{wx} q_{wy}}{h_w} \right) = -g h_w \frac{\partial z_s}{\partial x} + u_{wx} e_s - \frac{\tau_{wsx}}{\rho_w} \quad (8)$$

$$\frac{\partial q_{wy}}{\partial t} + \frac{\partial}{\partial x} \left(\frac{q_{wx} q_{wy}}{h_w} \right) + \frac{\partial}{\partial y} \left(\frac{q_{wy}^2}{h_w} + \frac{1}{2} g h_w^2 \right) = -g h_w \frac{\partial z_s}{\partial y} + u_{wy} e_s - \frac{\tau_{wsy}}{\rho_w} \quad (9)$$

$$\begin{aligned} & \frac{\partial q_{sx}}{\partial t} + \frac{\partial}{\partial x} \left(\frac{q_{sx}^2}{h_s} + \frac{1}{2} g h_s^2 + g \chi h_w h_s \right) + \frac{\partial}{\partial y} \left(\frac{q_{sx} q_{sy}}{h_s} \right) \\ & = g \chi h_w \frac{\partial z_s}{\partial x} - g (\chi h_w + h_s) \frac{\partial z_b}{\partial x} - \chi u_{wx} e_s + \frac{\tau_{wsx}}{\rho_s'} - \frac{\tau_{bsx}}{\rho_s'} \end{aligned} \quad (10)$$

$$\begin{aligned} & \frac{\partial q_{sy}}{\partial t} + \frac{\partial}{\partial x} \left(\frac{q_{sx} q_{sy}}{h_s} \right) + \frac{\partial}{\partial y} \left(\frac{q_{sy}^2}{h_s} + \frac{1}{2} g h_s^2 + g \chi h_w h_s \right) \\ & = g \chi h_w \frac{\partial z_s}{\partial y} - g (\chi h_w + h_s) \frac{\partial z_b}{\partial y} - \chi u_{wy} e_s + \frac{\tau_{wsy}}{\rho_s'} - \frac{\tau_{bsy}}{\rho_s'} \end{aligned} \quad (11)$$

where x and y are co-ordinates in a given Cartesian system; h_w is the depth of the clear-water layer; h_s is the thickness of the bed-load transport layer such that $z_s = z_b + h_s$; $q_{wx} = u_{wx} h_w$, $q_{wy} = u_{wy} h_w$, $q_{sx} = u_{sx} h_s$, $q_{sy} = u_{sy} h_s$ are the unit discharges of the flow in the clear-water layer and the bed-load transport layer, in the x direction and the y direction, respectively; u_{wx} and u_{wy} are the depth-averaged velocity components in the clear-water layer; u_{sx} and u_{sy} are the thickness-averaged velocity components in the bed-load transport layer; g is the gravitational acceleration; ρ_w , ρ_s' and ρ_b' are the water, transport layer and bed mass densities, respectively; $\rho_s' = (1 - C_s) \rho_w + C_s \rho_s$, $\rho_b' = (1 - C_b) \rho_w + C_b \rho_s$, in which ρ_s is the grain density; the ratio $\chi = \rho_w / \rho_s'$; (τ_{wsx}, τ_{wsy}) , (τ_{sbx}, τ_{sby}) and (τ_{bsx}, τ_{bsy}) are the shear stresses exerted along the interfaces by the layer corresponding to the first index onto the layer corresponding to the second index (Figure 3). A proof and a discussion of these equations are detailed by Swartenbroekx (2012).

The total shear stresses τ_{ws} , and τ_{sb} are driven by Chezy-like closure equations and τ_{bs} is driven by Mohr–Coulomb relation:

$$\tau_{ws} = \rho_w C_{f,w} \|\mathbf{u}_r\| \mathbf{u}_r \quad (12)$$

$$\tau_{sb} = \rho_s' C_{f,s} \|\mathbf{u}_s\| \mathbf{u}_s \quad (13)$$

$$\tau_{bs} = (\tau_{crit} + \tan \phi (\rho_s' - \rho_w) g h_s) \frac{\mathbf{u}_s}{\|\mathbf{u}_s\|} \quad (14)$$

where $C_{f,w}$ and $C_{f,s}$ are water and sediment friction coefficients, $\mathbf{u}_r = \mathbf{u}_w - \mathbf{u}_s$ is the relative velocity, τ_{crit} the threshold value of τ_{bs} allowing for erosion and φ the angle of repose. The threshold value τ_{crit} corresponds to the soil cohesion. Sands are cohesionless, so that this threshold value is zero. The shear stress τ_{bs} is assumed to act in the same direction as τ_{sb} because it is a resistive stress.

As a closure relation, following a Rankine-Hugoniot shock approach, the bed erosion rate is considered as depending on the shear stresses applied on both faces of the lower interface (see Zech et al. 2009 for the details):

$$e_b = \frac{\|\tau_{sb}\| - \|\tau_{bs}\|}{\rho_b \|\mathbf{u}_s\|} \quad (15)$$

3.3 Numerical solution

For both models, the coupled system of governing equations is solved on a 2D unstructured mesh. A finite-volume discretisation is applied (Toro 2001):

$$\mathbf{U}_i^{n+1} = \mathbf{U}_i^n - \frac{\Delta t}{\Omega_i} \sum_{j=1}^{nb} \mathbf{T}_j^{-1} \mathbf{F}_j^* (\bar{\mathbf{U}}_j) L_j + \mathbf{S}_i \Delta t \quad (16)$$

where n is the time index, Δt is the time step, Ω_i is the i -cell-base area, nb is the number of cell interfaces (3 for triangular cells) and L_j is the j -interface length (Figure 4). The vectors $\bar{\mathbf{U}}$ and $\mathbf{F}(\bar{\mathbf{U}})$ express $\mathbf{U}$ and $\mathbf{F}$ in terms of normal and tangential velocity components u_n and v_t , attached to the considered interface, using a transformation matrix $\mathbf{T}$, where n_x and n_y are the components of the outward unit vector normal to the interface (Toro 2001).

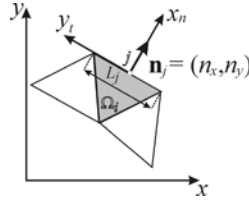


Figure 4 Two-layer shallow-water description.

The fluxes $\mathbf{F}_j^*$ are calculated thanks to a modified first-order HLLC solver for the Saint-Venant-Exner approach (Soares-Frazão and Zech 2010). As regards the two-layer model, a modified HLL scheme is used (Spinewine et al. 2011, Swartenbroekx et al. 2013). In both cases, the bed slope source terms are treated in a lateralised way as suggested by Fraccarollo *et al.* (2003), i.e. by up-winding of the bed slope as for the momentum flux.

Two unstructured triangular computation meshes are used: (1) a coarser mesh made of 34389 cells of several sizes, with an average edge length of about 5 cm along the nine meters downstream of the gate and a coarser grid elsewhere, (2) a finer mesh made of 90342 cells of several sizes, with an average edge length of about 3 cm along the nine meters downstream of the gate. The boundary conditions are solid wall conditions except at the downstream part where the outflow spills freely.

The following values, corresponding to a coarse sand (median diameter $d_{50} \approx 1.82$ mm), are adopted as proposed by Spinewine and Zech (2007): $\rho_w = 1000$ kg/m³, $\rho_s' = 1369.6$ kg/m³, $C_s = 0.22$, $C_b = 0.53$, $\tau_{crit} = 0$ and $\varphi = 30^\circ$. Due to the explicit nature of the numerical scheme, computational stability is dictated by the Courant-Friedrichs-Lewy (CFL) condition on the time step Δt and a CFL number of 0.9 is adopted. For the 2L model, the friction coefficients are calibrated upon the water level evolution: $C_{f,w} = 0.02$, $C_{f,s} = 0.03$.

4 COMPARISONS AND DISCUSSION

4.1 Water level

Figures 5 and 6 compare the measured (grey) and simulated (black) time evolution of the water level for Case 1 (no water downstream) and Case 2 (15 cm of water downstream), respectively. The

measurements, systematically taken at symmetric locations, are superimposed to check the accuracy of the experimental results. The numerical results are obtained with the coarser mesh. Except at the inner corners of the channel (G1 and G4), the predictions are satisfactory for both cases. At the corners, the models spread the flow without resistance, while, in the reality, a detachment of the main flow is observed with the development of two recirculation areas in the shadow zones behind the corners. This cannot be accurately reproduced by the models, as no turbulent shear stresses are accounted for in the model along vertical interfaces. So, the lateral spreading of the dam-break wave is faster in the numerical results than in the reality, in spite of a slight (and classical) numerical diffusion in the models that partly compensates for this approximation. For Case 1, the SVE model gives better results at G2 and G3 (Figure 5b), while, for both cases, the 2L model is more accurate at G5, G6, G7 and G8 (Figures 5c-d, 6c-d).

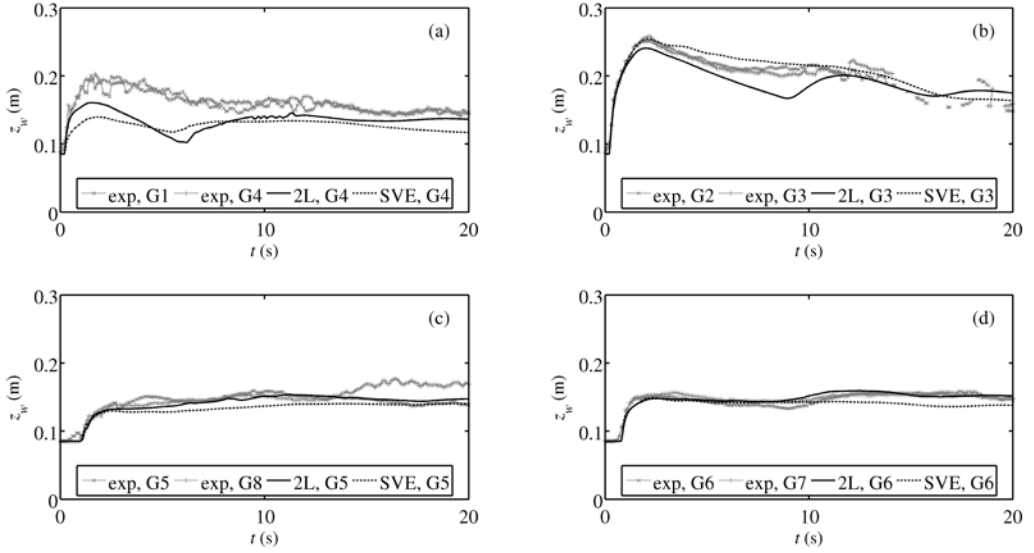


Figure 5 Water level time-evolution in Case 1.

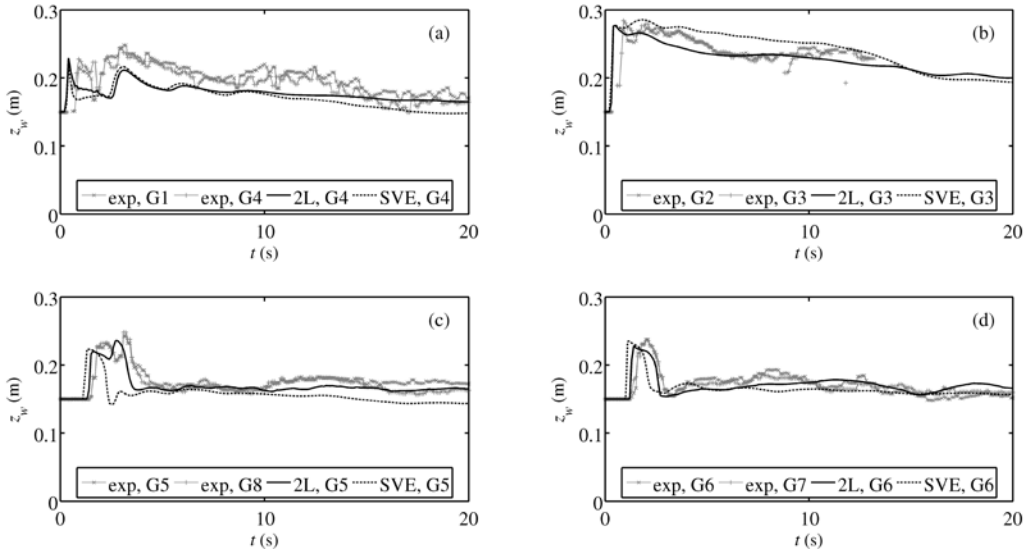


Figure 6 Water level time-evolution in Case 2.

4.2 Topography

Figures 7 to 10 compare the measured and the simulated ‘final’ topography (actually, at $t = 20$ s) for (a) Case 1 and (b) Case 2, respectively. The numerical topography is computed either by the SVE model on a coarse mesh (Figure 8), or by the 2L model with the same coarse mesh (Figure 9) or by the 2L model

with a fine mesh (Figure 10). The thick black lines correspond to the initial bed level $z_b = 8.5$ mm and thus separates the eroded area from the deposited area. The following main features are captured both experimentally (Figure 7) and numerically (Figures 8-10): intense erosion occurs immediately downstream and in the continuation of the dam channel, while a deposition area propagates downstream with deposition crests near the walls. For Case 2 with an initial wet downstream, the propagation of the deposition is lower than for Case 1, both numerically and experimentally. However, the deposition propagation is more underestimated by the 2L model (Figure 9) than by the SVE model (Figure 8). The magnitude of the deposition crests is better simulated by the 2L model. The numerical results are almost symmetric, especially when using a fine unstructured triangular mesh of about 3 cm average edge cells in the part of interest (Figure 10). Kinds of bedforms are observed experimentally. The 2L model applied on a fine mesh is able to reproduce some small bed forms.

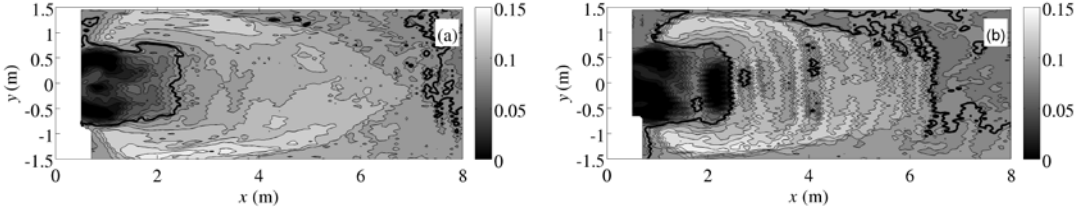


Figure 7 Measured topography: (a) Case 1, (b) Case 2.

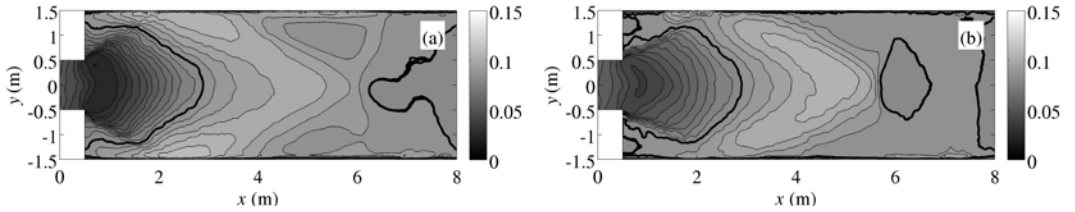


Figure 8 Simulated final topography with SVE model on coarse mesh: (a) Case 1, (b) Case 2.

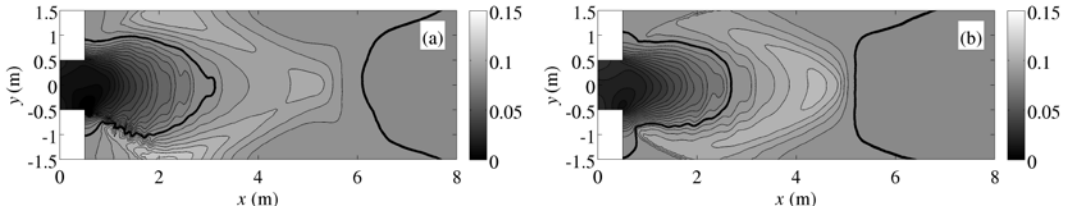


Figure 9 Simulated final topography with 2L model on coarse mesh: (a) Case 1, (b) Case 2.

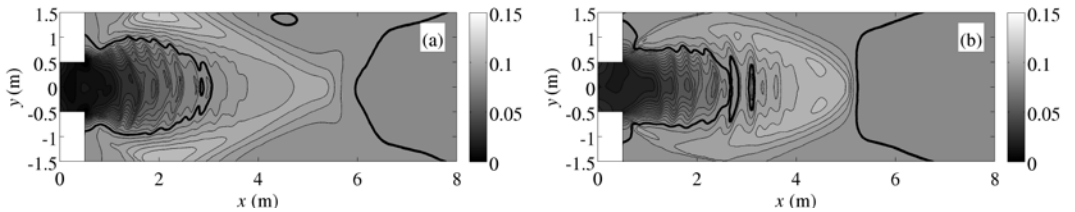


Figure 10 Simulated final topography with 2L model on fine mesh: (a) Case 1, (b) Case 2.

The bed profiles simulated by the 2L model on the coarse mesh are compared with two runs of measurements and with SVE results for Case 1 in Figure 11 along three longitudinal profiles: $y =$ (a) 0.2 m, (b) 0.7 m and (c) 1.45 m. The deposition crest is underestimated by both models (Figure 11c). The scour hole is well predicted by the 2L model at $y = 0.2$ m near the axis of symmetry (Figure 11a) but is missed, at least by the 2L model, just after the corner, at $y = 0.7$ m (Figure 11b). This is in line with previous comments about the limitation of the model in areas where flow detachment occurs. Moreover, the scour is probably due to 3D effects induced by flow path curvature and resulting secondary

currents that are not accounted for in a 2D approach. Paradoxically, the Exner approach, which neglects the momentum required to mobilise the sediments, is better in predicting this scouring.

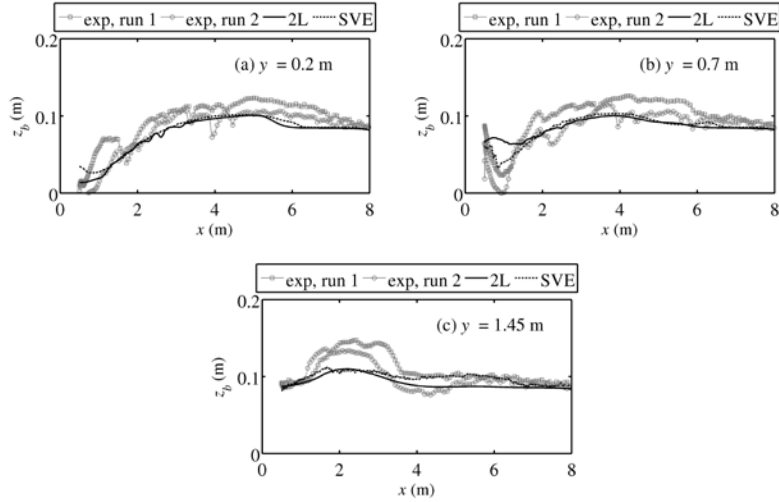


Figure 11 Final longitudinal bed elevation at $y =$ (a) 0.2 m, (b) 0.7 m, (c) 1.45 m.

4.2 Velocities

A two-layer description distinguishes a clear-water velocity from a bed-load velocity, both in magnitude and direction. So, it allows the interaction between the hydrodynamics and the morphological evolution to be better understood. The two-layer velocity vectors are mapped for Case 1 (Figure 12) and for Case 2 (Figure 13) at $t =$ (a) 2 s, (b) 5 s and (c) 20 s. The wave is first expanded (Figures 12a, 13a) and, then, reflected against the walls (Figures 12b, 13b) before to be aligned mainly with the longitudinal x -axis of the channel. The velocities have distinct directions in each layer at the beginning of the wave propagation and the bed-load velocity is smaller in magnitude than the clear-water vectors especially at later times. For Case 2, the propagation is slower due to the initial water level downstream.

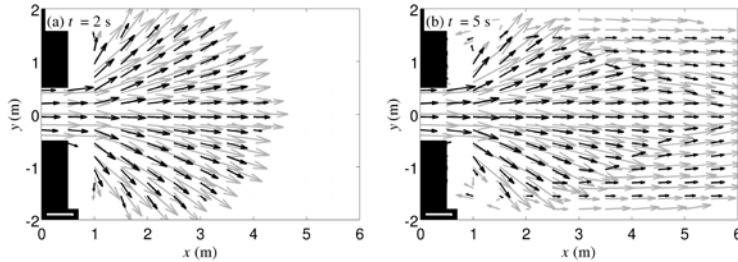


Figure 12 Simulated depth-averaged velocity vectors in Case 1 at $t =$ (a) 2 s, (b) 5 s. (Grey) clear water, (black) bed-load transport. The white arrow represents a scaled velocity of 1 m/s.

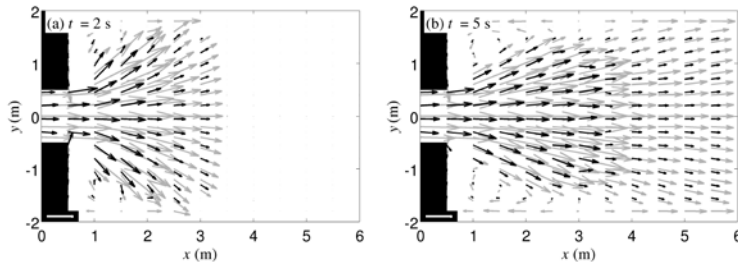


Figure 13 Simulated depth-averaged velocity vectors in Case 2 at $t =$ (a) 2 s, (b) 5 s. (Grey) clear water, (black) bed-load transport. The white arrow represents a scaled velocity of 1 m/s.

5 CONCLUSIONS

An experiment of a dam-break wave was conducted on a laboratory flume with floodplain expansion in order to measure (1) the intense erosion occurring immediately downstream and in the continuation of the dam channel, (2) the deposition area featuring deposition crests near the side walls. The dam-break flow was simulated by two 2D shallow-water models: (1) a classical Saint-Venant–Exner model and (2) a two-layer model. The main features experimentally observed are captured by the numerical models thanks to a mesh refined in the area of interest. The water level evolution is better reproduced by the 2L model than by the SVE model, especially at some distance downstream of the gate. The deposition crests near the walls are still difficult to predict, probably due to turbulent shear stresses and 3D effects that are not accounted for by both models. The two-layer model has the advantage to distinguish the magnitude and the direction of the velocities between water and sediment transport layers.

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