

COMPARISON BETWEEN MODELS USED TO SIMULATE DAM BREAK FLOWS OVER A MOBILE BED MADE OF DIFFERENT MATERIALS

Ilaria Fent¹

Place du Levant 1 L5.05.01, 1348 Louvain-la-Neuve, Belgium
ilaria.fent@uclouvain.be

Sylvie Van Emelen, Sandra Soares-Frazão

Place du Levant 1 L5.05.01, 1348 Louvain-la-Neuve, Belgium
s.vanemelen@ecam.be, sandra.soares-frazao@uclouvain.be

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ABSTRACT

The hydrodynamic and morphodynamic processes are very important aspects in the study of dam break flow over mobile bed. It is possible to simulate and thus to forecast the water and sediment movements thanks to appropriate models. The hydrodynamics is well represented by the Saint-Venant shallow water equations that constitute a simple hyperbolic system and for which different numerical models exist. The inclusion of the morphodynamics introduces additional difficulties, so different solutions have been developed in order to better represent the movement of sediments and the associated changes to the morphology. Several models with different approaches have been developed and are available in the literature: (i) the clear water layer model, representing one layer of clear water flowing over a mobile sediment bed; (ii) the mixture layer model, considering one layer of a variable density mixture of sediment and water; (iii) the two-layer model, which distinguishes the upper clear water layer from the lower moving sediment layer, where the bed load transport takes place and (iv) the two-phase model, which considers the water and the sediments as two immiscible phases characterized by different velocities. This paper aims at assessing the performances of these models using two different test cases of dam-break flow over mobile bed, for which experimental data exist. The results from the different models are compared to data measured in dam-break flow experiments over mobile bed composed by (i) coarse sand for the first case and (ii) light PVC pellets for the second case.

1. INTRODUCTION

The study of dam-break waves is important to forecast catastrophic flood events, responsible for damages to properties and of loss of human lives. All information about the event are determinant: the initial conditions, the wave-front celerity, the water and bed levels, the velocity of the flow and the type and evolution of the bed made of movable sediments. Therefore an in-depth comprehension of this phenomenon is needed to better assess the risk posed by a dam, to predict what could happen in case of failure and to mitigate the consequences. This implies a thorough analysis of experimental data reproducing the phenomenon, in order to allow for an appropriate choice of a numerical model able to reproduce the event.

Even if this phenomenon is three-dimensional in nature immediately after the dam failure, the flow rapidly evolves to a mainly horizontal flow [1] and from the engineering point of view it is widely accepted that shallow-water assumptions provide a reasonable compromise between the computational effort and the adequacy of the physical representation [25]. The hydrodynamics is well represented by the Saint-Venant shallow water equations that constitute a simple hyperbolic system and this has already been solved by different numerical models [23]. The inclusion of the morphodynamic aspect within a model introduces additional difficulties, so different solutions have been developed in order to better represent the movement of sediments and the associated changes to the river morphology. In morphodynamical models, the sediment

¹ Corresponding author: Ilaria Fent, ilaria.fent@uclouvain.be

transport is usually estimated using the local hydrodynamic conditions. In the so-called ‘equilibrium’ or ‘capacity’ models, the sediment transport is evaluated through an algebraic closure equation, valid for uniform flow conditions ([11], [14], [18]), and it is assumed that sediment transport is always in equilibrium. But in most cases the flow is far from being uniform and sediments are not always transported at flow capacity ([3], [4], [7], [25]), as proved by Armanini & Di Silvio [2], who showed the importance of non-equilibrium equations to calculate the fluxes through the bed surface at short time scales. To take into account the inertia of the moving sediments, several authors proposed to express the full momentum balance of the sediments, leading either to a two-layer model ([8], [21]) or to a two-phase model ([13], [12], [16]).

The most basic model consists of the Saint-Venant-Exner set of equations, and is called the Clear Water Layer (CWL) model [5]. This model represent one layer of clear water flowing over a mobile sediment bed, where all the interaction mechanism is confined at the bed surface acting as source or sink of mass and source of head losses. No inertial effects of the moving sediment bed are considered.

The two-layer (2L) model combines two flow layers: the upper layer consists of clear water and the second one of a water-sediment mixture, each one with its own velocity ([9], [26]). Each flow zone is characterized by specific concentration and velocity profiles, but sediments in the lower layer are assumed to be transported at the same velocity as water, forming a unique phase consisting of a mixture of water and sediments. In their recent study, Spinewine and Capart [20] show that the velocity and concentration profiles are linear in the bed-load transport layer, especially in case of intense transport. So Spinewine and Capart [20] developed a model including a linear profile of velocity and concentration in the bed-load layer, neglecting the concentration in the upper layer.

The two-phase (2P) model represents the flow as one layer of a water-sediment mixture but with different velocities for water and sediment. Such an approach is better appropriate when modelling intense sediment transport, but it has been also applied to simulate bed load transport during fast geomorphic flows, as done by Greco et al. [12].

Another studied model is the Mixture Layer (ML) model, which involves only one layer constituted by a mixture of sediment and water, with variable density, over an erodible bed ([4], [10], [24] and [25]). This is a simplification of the 2-Phase (2P) model as the two phases are assumed to have the same velocity and it is adequate in particular to simulate suspension-dominant sediment transport processes, or intense total sediment transport, including both suspension and bed-load.

All these models are used to reproduce a fast transient flow over mobile bed, as it is the case of the dam break flow, but it is still not well established which one better reproduces the event. The scope of this work is to compare the results from these models and to confront them to experimental data. Not only the water and bed surfaces profiles are analysed, but also the velocities in the liquid and solid phases. Several parameters for the different closure equations must be calibrated, depending on the model. In order to verify and calibrate the models, two different dam-break flow experiments have been considered: (i) in the first one the sediment on the bed consist of PVC pellets and are lighter than sand, so they go easily in suspension when the dam break wave arrives, (ii) in the second one the bed is made by sand, so the transport of sediment is principally confined near the bed.

The results of the comparisons contribute to assess the scope of the shallow-water models applied to fast transient flow over mobile bed and how they could be improved.

2. DESCRIPTION OF SHALLOW WATER MODELS

The shallow water models assume hydrostatic pressure, incompressible fluid and small bed slope. The models presented hereafter are the clear water layer (CWL) model, the mixture layer (ML) model, the two-layer (2L) model and the two-phase (2P) model.

2.1 Clear water layer model

The CWL model is described by equations (1). The two first equations of (1) are the classical Saint–Venant equations for clear water, coupled to the Exner equation for sediment conservation

$$\begin{cases} \frac{\partial h}{\partial t} + \frac{\partial(hu)}{\partial x} = 0 \\ \frac{\partial(hu)}{\partial t} + \frac{\partial}{\partial x} \left(hu^2 + \frac{gh^2}{2} \right) = gh \left(-\frac{\partial z_b}{\partial x} - S_f \right) \\ \frac{\partial(z_b)}{\partial t} + \frac{\partial}{\partial x} \left(\frac{q_s}{(1-\varepsilon_0)} \right) = 0 \end{cases} \quad (1)$$

where t is the time, x is the longitudinal direction, h is the water depth, u is the water velocity, $q=hu$ is the water discharge, g is the gravitational acceleration, z_b is the bottom level, S_f is the friction slope that can be expressed using the Manning equation, q_s is the sediment discharge and ε_0 is the bed porosity.

2.2 Mixture layer model

The ML model is described by Cao et al. [3] and Wu et al. [24] and it involves only one layer of mixture constituted by sediments and water moving at the same velocity, with variable density, over an erodible bed.

$$\begin{cases} \frac{\partial h}{\partial t} + \frac{\partial z_b}{\partial t} + \frac{\partial(hu)}{\partial x} = 0 \\ \frac{\partial(\rho hu)}{\partial t} + \frac{\partial}{\partial x} \left(\rho hu^2 + \frac{\rho gh^2}{2} \right) = \rho gh \left(-\frac{\partial z_b}{\partial x} - S_f \right) \\ \frac{\partial(hC)}{\partial t} + \frac{\partial(huC)}{\partial x} + (1-\varepsilon_0) \frac{\partial(z_b)}{\partial t} = 0 \end{cases} \quad (2)$$

In system (2), $\rho = \rho_w(1-C) + \rho_s C$ is the mixture density, depending on the water density ρ_w , on the sediment density ρ_s and on the concentration $C = uh/q_s$. Assuming equilibrium sediment transport, a direct expression of the concentration C in the flow layer can be used [10].

When non-equilibrium sediment transport is assumed, it is computed through a direct expression of erosion E^{cap} and deposition D^{cap} capacities ([3] and [25])

$$\frac{\partial z_b}{\partial t} = \frac{D^{cap} - E^{cap}}{1-\varepsilon_0} = -e_b \quad (3)$$

$$E^{cap} - D^{cap} = -\frac{huC - q_{ss}^{cap}}{L_s}, \quad L_s = -\frac{hu}{\alpha_s w_s} \quad (4)$$

where L_s is the adaptation length for suspension, α_s is an adaptation parameter, q_{ss} is the suspension transport and w_s is the vertical sedimentation velocity. The evaluation of both adaptation parameters α_s and L_s is still subject to various uncertainties [24], and is therefore the main drawback of using non-equilibrium sediment transport formulations [2]. Replacing the temporal derivative of the bed level z_b by the closure equation (3), system (2) is transformed as follows

$$\begin{cases} \frac{\partial(h+z_b)}{\partial t} + \frac{\partial(hu)}{\partial x} = 0 \\ \frac{\partial(hu)}{\partial t} + \frac{\partial}{\partial x} \left(hu^2 + \frac{gh^2}{2} \right) = \rho gh \left(-\frac{\partial z_b}{\partial x} - S_f \right) - \frac{(\rho_s - \rho_w)gh^2}{2\rho} \frac{\partial C}{\partial x} - \frac{(\rho_0 - \rho)(E^{cap} - D^{cap})u}{\rho(1-\varepsilon_0)} \\ \frac{\partial(hC)}{\partial t} + \frac{\partial(huC)}{\partial x} + (1-\varepsilon_0) \frac{\partial z_b}{\partial t} = 0 \end{cases} \quad (5)$$

where $\rho_0 = \rho_w \varepsilon_0 + \rho_s(1-\varepsilon_0)$ is the density of the bed.

2.3 Two-layer model

In the two-layer model, each flow zone is characterized by a specific concentration and velocity profile, but sediments in each layer are assumed to be transported at the same velocity as the water in the layer, forming a unique phase [19]. The two layers here consist of a clear water upper layer (indicated with index 1, in (6)) and a lower mixture layer (indicated with index 2, in (6)). The forces acting at the boundary between the layers are τ_{12} , i.e. the shear stress between the layers, and τ_b , which is the shear stress between the sediment layer and the immobile bottom. Exchanges of sediment and water may occur between each zone, leading to the vertical evolution of the boundaries between each layer. This approach presents the advantage to be rather physical, but requires the evaluation of the exchanges between both flow layers. Yet these terms are difficult to evaluate, and subject to uncertainties [17]. The corresponding system of equations reads

$$\left\{ \begin{array}{l} \frac{\partial h_1}{\partial t} + \frac{\partial(h_1 u_1)}{\partial x} = e_s \\ \frac{\partial h_2}{\partial t} + \frac{\partial(h_2 u_2)}{\partial x} = e_b - e_s \\ \frac{\partial(h_1 u_1)}{\partial t} + \frac{\partial}{\partial x} \left(h_1 u_1^2 + \frac{g h_1^2}{2} \right) + g h_1 \frac{\partial(h_2 + z_b)}{\partial x} = u_1 e_s - \frac{\tau_{12}}{\rho_1} \\ \frac{\partial(h_2 u_2)}{\partial t} + \frac{\partial}{\partial x} \left(h_2 u_2^2 + \frac{g h_2^2}{2} \right) + g h_2 \frac{\rho_1}{\rho_2} \frac{\partial}{\partial x} \left(z_b + \frac{\rho_1}{\rho_2} h_1 \right) = -\frac{\rho_1}{\rho_2} u_1 e_s + \frac{\tau_{12} - \tau_b}{\rho_2} \\ \frac{\partial z_b}{\partial x} = -e_b \end{array} \right. \quad (6)$$

where e_s is the entrainment-deposition rate between the clear water and moving sediment layers and the e_b is the entrainment-deposition rate and depends on the erosion E^{cap} and deposition D^{cap} of material between the mixture layer (moving sediments) and the immobile sediment bed.

2.4 Two-phase model

The two-phase model used here is based on Pelanti's model [15]. This approach can be thought as an evolution of the ML model, where the velocities of liquid and solid phases are different. The model expresses mass conservation and momentum balance for each constituent by writing equations for the mass and momentum variables as functions of the volume fraction φ : $h_s = \varphi h$ and $h_w = (1-\varphi)h$; $q_s = \varphi h u_s$ and $q_w = (1-\varphi)h u_w$, where $h = h_s + h_w$ is the total mixture depth. The concentration expressed in the 2P model as the volume fraction $\varphi = h_s/(h_s+h_w)$ differs from the concentration C of the ML model. The final system of equations for the two-phase model is

$$\left\{ \begin{array}{l} \frac{\partial h_s}{\partial t} + \frac{\partial(h_s u_s)}{\partial x} = p e_b \\ \frac{\partial(h_s u_s)}{\partial t} + \frac{\partial}{\partial x} \left(h_s u_s^2 + \frac{1}{2} g \left(h_s^2 + \frac{\rho_s - \rho_w}{\rho_s} h_s h_w \right) \right) + \frac{\rho_w}{\rho_s} g h_s \frac{\partial h_w}{\partial x} = \\ - g h_s \frac{\partial z_b}{\partial x} - \frac{\tau_{b,s}}{\rho_s} + \frac{\rho_w}{\rho_s} F^D \\ \frac{\partial h_w}{\partial t} + \frac{\partial(h_w u_w)}{\partial x} = (1-p) e_b \\ \frac{\partial(h_w u_w)}{\partial t} + \frac{\partial}{\partial x} \left(h_w u_w^2 + \frac{g h_w^2}{2} \right) + g h_w \left(\frac{\partial h_s}{\partial x} \right) = - g h_w \frac{\partial z_b}{\partial x} - \frac{\tau_{b,w} - \tau_{b,s}}{\rho_w} - F^D \\ \frac{\partial z_b}{\partial x} = -e_b = -\frac{E-D}{p} \end{array} \right. \quad (7)$$

where $\tau_{b,s}$ and $\tau_{b,w}$ are the shear stresses acting on the solid and the liquid phases respectively and F^D is the drag force. The bottom shear stress for the solid phase is expressed through Mohr-Coulomb law (8) and the

bottom shear stress for the liquid phase is evaluated by means of Chezy uniform flow formula (10), as proposed in [6]. For the drag force expression, linear dependence on the velocity difference between the solid and the liquid phases is assumed, yielding expression (10).

$$\tau_{b,s} = \rho_s \mu_D g h_s \left(1 - \frac{\rho_w}{\rho_s} \right) \frac{u_s}{|u_s|} \quad (8)$$

$$\tau_{b,w} = \rho_w \frac{u_w |u_w|}{X^2} \quad (9)$$

$$F_D = c_D (h_s + h_w)(u_s + u_w) \quad (10)$$

3. DAM-BREAK FLOW EXPERIMENTS OVER MOBILE BED

Experiments were performed in a dedicated flume at the Université catholique de Louvain, Civil Engineering Department. The dam-break channel is 6 m long with a rectangular cross section of 0.25 m width. The channel is divided in two equal reaches by a downward moving gate used to simulate the instantaneous removal of a dam [22]. The walls of the channel are made of glass, allowing optical access to the flow in order to record images with a high speed camera, allowing for the application of Particle Image Velocity technique (PIV) technique.

In the present study, two distinct bed materials are considered: (i) extruded PVC pellets and (ii) uniform coarse sand. The first experiments, performed by Spinewine and Capart [20], use PVC pellets having a cylindrical shape, with an equivalent spherical diameter $d_{50} = 3.9$ mm, a specific density $\rho = 1580$ kg m⁻³, a friction angle $\theta = 38^\circ$ and set in place at the random close packing $c_b = 58\%$. The second data set consists of new experiments involving non-cohesive sand, with the following characteristics: the grain-size distribution is quasi-uniform with a mean diameter $d_{50} = 1.72$ mm, the density is $\rho = 2682$ kg m⁻³, the friction angle is $\theta = 38^\circ$ and the bed porosity is $\varepsilon_0 = 0.42$. In both cases, the upstream half of the channel (i.e. the first 3 meters) consisting of the reservoir is filled with water, of which the depth is $h_0 = 0.350$ m over the PVC bed and $h_0 = 0.325$ m over the sand bed.

4. RESULTS

All the models are solved using a finite volume scheme that applies the HLL fluxes as Riemann solver and a mesh size of $\Delta x = 0.006$ m is used, with a CFL number of 0.5. The boundary conditions include a wall upstream and a free outflow downstream of the flume. The parameters used in each model for the specific closure equations are listed in Table 1. For the sand-bed experiments, measurements are only available around the initial gate location.

4.1 Bottom and water surface profile comparison

Results of the CWL for both cases are illustrated in Figure 1. No transport layer is represented, as this simplified model only considers sediment fluxes without moving sediment layer. Intense sediment transport implies a rapid temporal variation of the bed dz_b/dt that is not negligible in comparison with the variation of the flow depth dh/dt . In the fast dam-break flows considered here, the assumption of slow variation of z_b is rather questionable. Indeed, the results in Figure 1 show the limitation of the simplified CWL model as the head of the dam-break wave presents an unstable water surface profile. This is particularly visible in the PVC-pellet case, but is also present in the sand-bed case.

Figure 2 shows the results obtained with the ML for the non-equilibrium case, using the system of equations (5). The illustrated bottom line from the ML model corresponds to the fixed bed. The upper boundary z_s of the fictitious sediment transport layer could be calculated as $z_s = z_b + h_s = hC/(1 - \varepsilon_0)$. This artificial sediment transport layer is equivalent to the transport layer obtained for the 2L model, but it is not represented in the Figure 2 as it is not the real sediment boundary. The non-equilibrium ML model was calibrated with $L_s = 10\Delta x$ for the sand case and $L_s = 100\Delta x$ for the PVC case in order to avoid non-physical oscillations. So the bottom evolution is not well reproduced, as the erosion is underestimated, but the water surface profile matches well the experimental data for both experiments.

Model	sediment	$n_{Manning}$	L_s	X	μ_D	c_D
CWL	PVC	0.022				
	sand	0.027				
ML	PVC	0.025	$100 \Delta x$			
	sand	0.016	$10 \Delta x$	11.2	0.51	
2L	PVC	0.0165	$100 \Delta x$	4.1	0.51	
	sand	0.035	$10 \Delta x$			
2P	PVC	0.019	30	6.5	0.01	0.1
	sand	0.030	50	50	0.01	0.1

Table 1: List of parameters used in the simulation, for the different models and sediments.

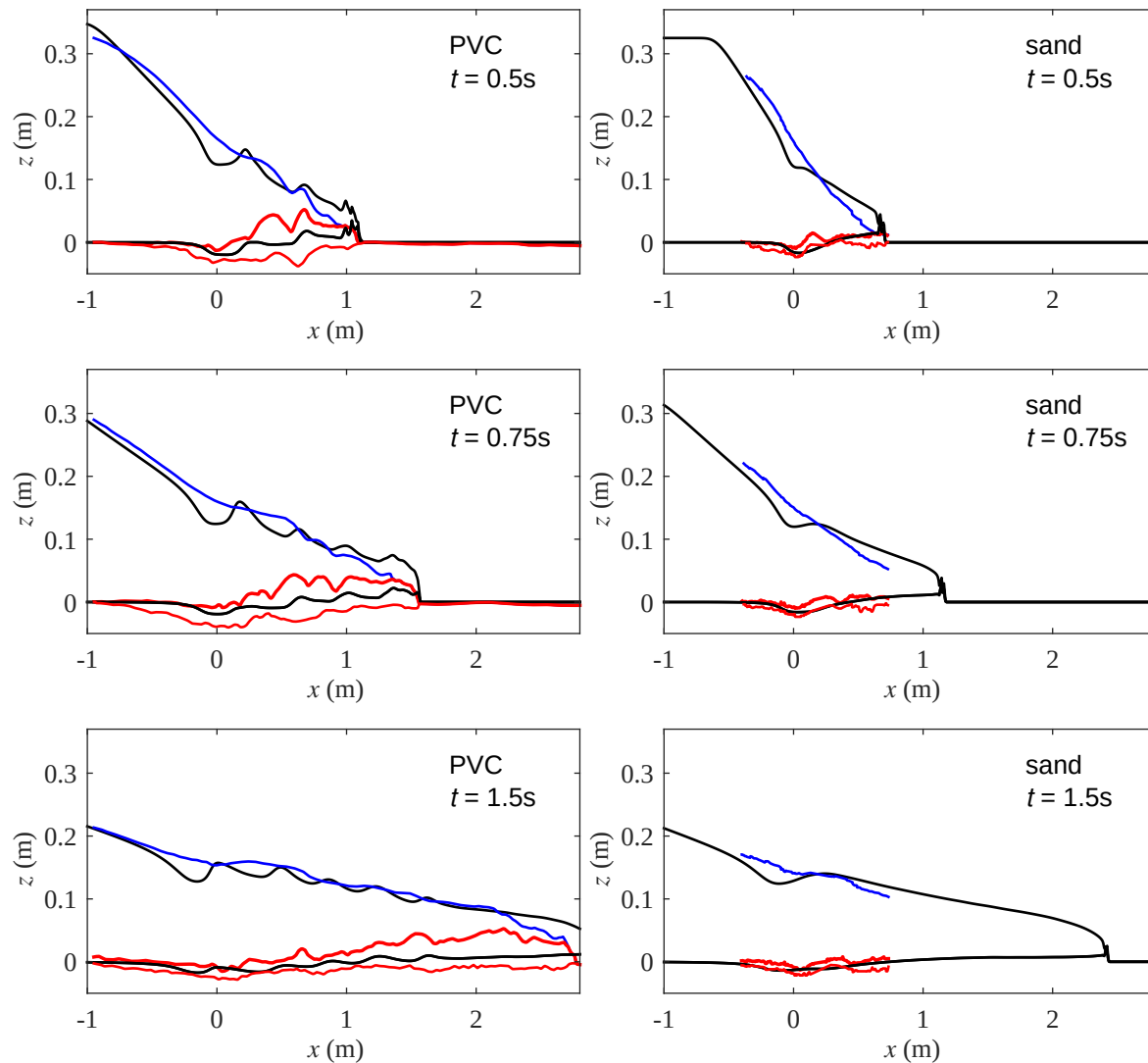


Figure 1: Comparison between experimental data and model resulting profiles using the Clear Water Layer model, for the sand and PVC cases at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, — CWL model results.

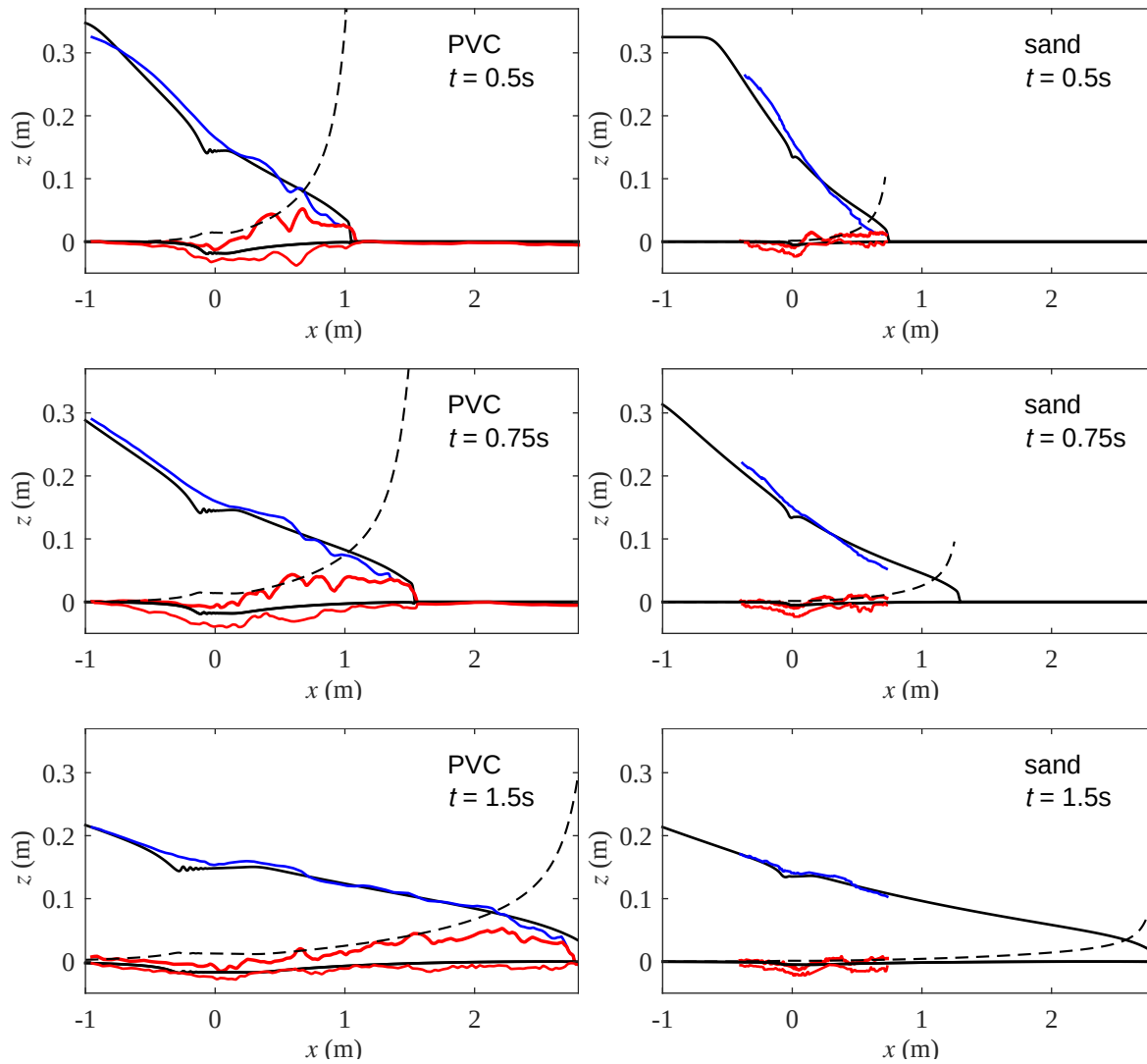


Figure 2: Comparison between experimental data and model resulting profiles using the Mixture Layer model, for the sand and PVC cases at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, — ML model results, ---- concentration.

The 2L model (Figure 3) allows for the representation of the immobile bed level, the moving sediment layer level, the water elevation and the concentration evolution. The same erosion law (3) used for the ML model is here applied and the choice of large L_s is responsible of a reduced amount of eroded and movable sediments. Directly from the first instants, the water profile is well reproduced and it follows well the movement of the dam break wave head. The model reproduces well a different thickness of the sediment layer for the PVC and sand cases, in particular the PVC transport layer being thicker due to the lighter moving particles.

The results of the 2P model are illustrated in Figure 4. Here, the concentration is illustrated as $\varphi = (h_s + h_w)/h_s$, through the equivalent depth h_s . The results for the water level and the boundary between the mobile and immobile bed are very similar to the ones given by the 2L model, as these two use the same bottom evolution formulation (3). However, with a different formulation for the erosion/depositions processes, Spinewine and Capart [20] obtained a better agreement with the measured data. This illustrates the importance of the closure models in such simulations.

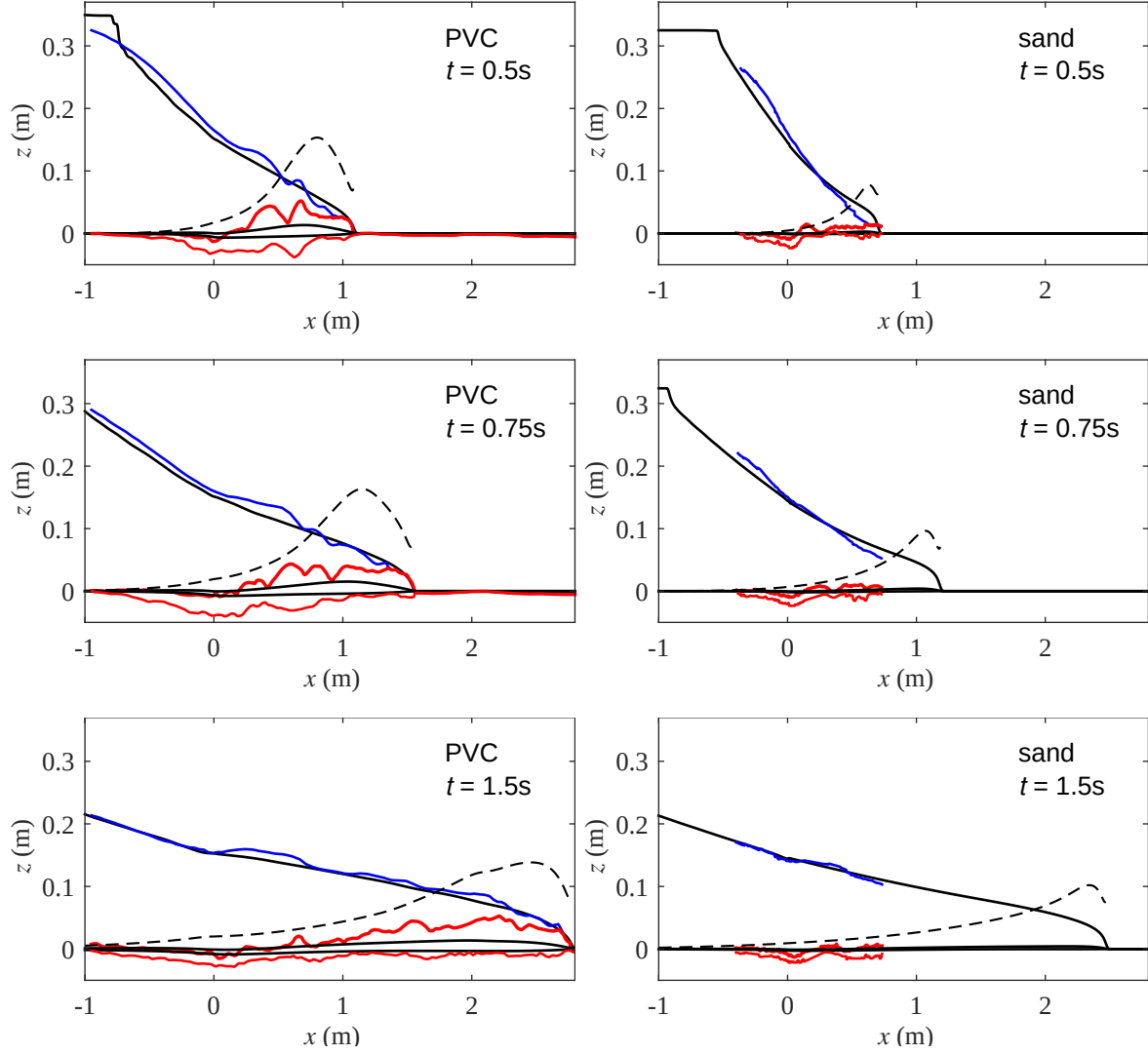


Figure 3: Comparison between experimental data and model resulting profiles using the Two Layer model, for the PVC and sand cases at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, — 2L model results, ---- concentration.

4.1 Longitudinal velocity profile comparison

The use of the PIV technique for the sand-bed experiment allows to determine the evolution of the flow velocity throughout the water column in time and space. These measurements are compared to the numerical model predictions.

The CWL and ML models assume only one velocity for the entire water depth. The computed velocity is superposed to the experimental velocity u_x vertical profile in Figures 5 and 6. It can be observed that the computed velocity is overestimated for the first instants ($t = 0.5$ s and $t = 0.75$ s) but it corresponds exactly to the experimental results at $t = 1.5$ s. Actually, at the beginning of the event, the flow has not reached yet its shallow water condition, but after 1.5 s we can assume that the flow is one-dimensional and that the pressure distribution is hydrostatic.

The 2L and 2P models assume two different velocities: the 2L presents one velocity for the upper clear water layer and one for the lower mixture layer (Figure 7) while the 2P model presents one velocity for the liquid part and one for the solid part (Figure 8). The representation of velocities in Figure 7 follows closely the measurements, especially for the higher velocity in the upper clear water layer. In Figure 8, corresponding to the 2P model, the two different velocities of the solid and liquid phases are represented.

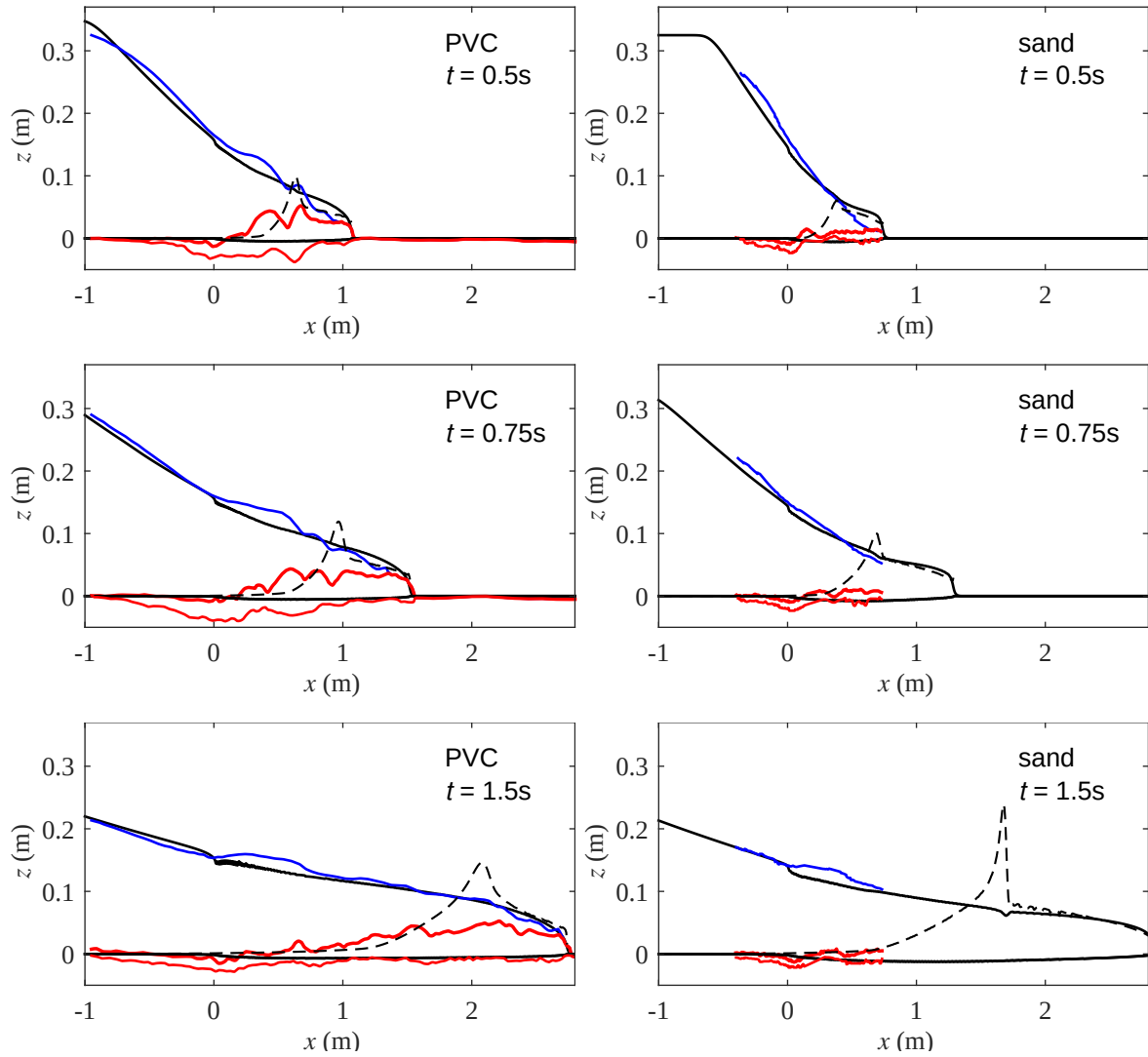


Figure 4: Comparison between experimental data and model resulting profiles using the Two Phase model, for the PVC and sand cases at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, — CWL model results, ---- concentration.

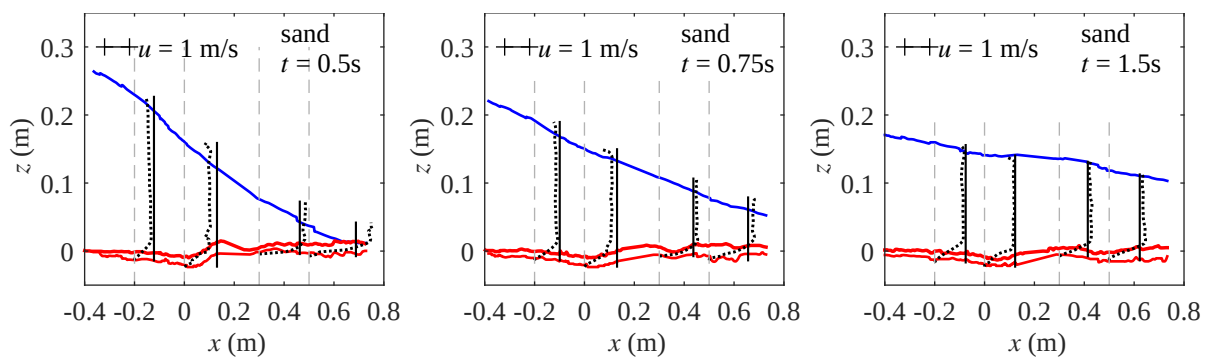


Figure 5: Comparison between experimental data to model resulting velocities using the Clear Water Layer model, at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, ... experimental velocity profile, — CWL model velocity.

The smaller velocity, corresponding to the sediment phase, is illustrated in the lower part of the profile where sediments are moving, while the velocity of the water phase is plotted only in the upper flow layer. Both the

2L and the 2P models represent well the velocities and, as in the CWL and ML models, the results better match the experimental velocity profiles after $t = 1.5$ s of the simulation.

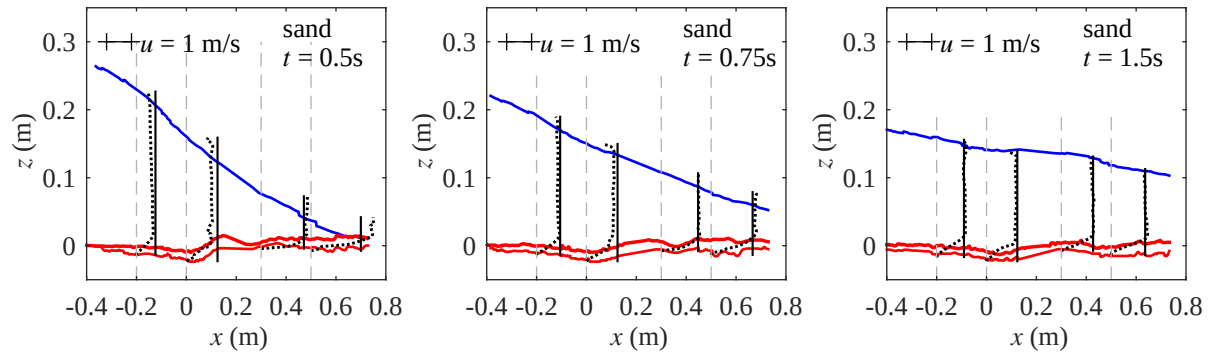


Figure 6: Comparison between experimental data and model resulting velocities using the Mixture Layer model, at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, ... experimental velocity profile, — ML model velocity.

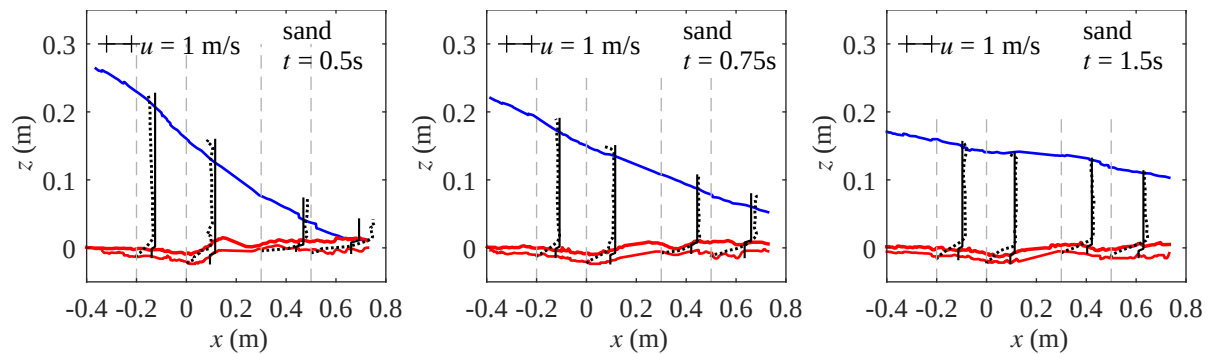


Figure 7: Comparison between experimental data and model resulting velocities using the Two Layer model, at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, ... experimental velocity profile, — 2L model velocity.

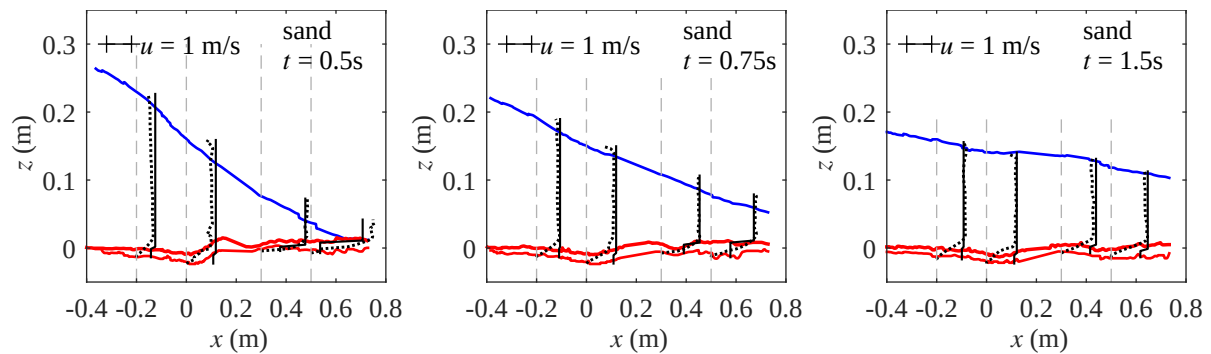


Figure 8: Comparison between experimental data and model resulting velocities using the Two Phase model, at times 0.50 s, 0.75 s and 1.50 s.

— experimental bed surface, — experimental water surface, ... experimental velocity profile, — 2P model velocity.

5. CONCLUSIONS

In this paper, a comparison between different shallow water models applied to fast geomorphic transients occurring in dam-break flows is presented. The models analysed are the clear water layer model (CWL), the mixture layer model (ML) for the equilibrium and non-equilibrium sediment transport cases, the two-layer

model (2L) and the two-phase model (2P). All the closure equations chosen for each model are presented and the parameters used are listed in order to understand their influence on the final numerical results. The parameters were all calibrated in order to obtain stable results that compare well with the experimental data, so differences in the values of similar parameters between the models indicate a fundamental difference in the model behaviour that will need to be further investigated.

The comparisons between numerical and experimental results show the capability of almost all models to reproduce the key features of the flow, regarding water surface profiles and velocity distribution. All the models however fail in representing the correct bed evolution induced by the dam-break flow. For example the ML, the 2L and the 2P models include numerous source terms that have an important impact on the system structure. In particular, in the calculation of the bottom evolution, the choice of large adaptation length L_s , which intervenes in the computation of non-equilibrium sediment transport, reduces and delays the erosion, leading to important discrepancies with experimental data.

The comparisons between the PVC and the sand-bed cases show that for intense sediment entrainment as in the PVC case, models considering inertial effects of the moving sediment layer provide a better prediction of the scoured bed and of the front propagation speed. For the sand-bed case, the differences are less marked in terms of erosion, but the front propagation speed presents a significant variability.

Future work will focus on investigating the fundamental difference in the model behaviours to select the best appropriate closure equations for sediment transport and to provide guidance about the calibration of the parameters, especially in what concerns the adaptation length that appears as a key parameter for the stability of the results

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