João G. A. B. Leal
Assistant Professor, Department of
Civil Engineering, Faculty of Sciences
and Technology, Universidade Nova
de Lisboa, Portugal

Rui M. L. Ferreira
Assistant Professor, Department of
Civil Engineering and Architecture,
Instituto Superior Técnico, UTL,
Portugal

António H. Cardoso
Full Professor, Department of Civil
Engineering and Architecture,
Instituto Superior Técnico, UTL,
Portugal

Geomorphic dam-break flows. Part II: numerical simulation

J. G. A. B. Leal PhD, R. M. L. Ferreira PhD and A. H. Cardoso PhD

This paper presents the comparison between numerical results obtained with a conceptual model proposed in a companion paper and experimental results collected from previous studies. The numerical model is based on a finite difference approximation using the MacCormack scheme with a total variation diminishing high-resolution correction term for the mixture mass and momentum conservation equations and an artificial viscosity term for the sediment mass conservation equation. From the comparison between numerical and experimental results, it can be verified that the model generally gives results in good agreement with the experimental data and the wave structure is in accordance with what has been reported in previous studies. The estimation of model parameters based on existing sheet-flow data seems a promising way of developing models without arbitrary choice of values for tuning parameters. Some limitations can be identified, namely the equilibrium sediment transport approach implies that the model cannot simulate the hydraulic jump formed in the higher mobility bed for dry bed conditions. The use of a constant friction factor does not allow simulating the exact wave-front location for all instants in time.

NOTATION

$\tilde{A}_{i+1/2}$	Jacobian matrix that linearises the nonlinear system (1)
a_i^k	exact eigenvalue k of the shallow water equations (m/s)
$\tilde{a}_{i+1/2}^k$	eigenvalue k of the Jacobian matrix $\tilde{A}_{i+1/2}$ (m/s)
C	sediment volumetric concentration
c_i	celerity of small perturbations (m/s)
$\tilde{c}_{i+1/2}$	Roe's approximation for small perturbation celerity (m/s)
D	TVD correction term
$\tilde{e}_{i+1/2}^k$	eigenvector k of the Jacobian matrix $\tilde{A}_{i+1/2}$
Fr	Froude number
$F(U)$	vector of flux
g	gravity acceleration (m/s ²)
h	flow depth (m)
m	parameter defined by Equation 12
m_d	mesh coordinate corresponding to the longitudinal coordinate of the dam location
p	bed porosity
$r_{i+1/2}^k$	parameter defined by Equation 11
S	vector of source terms
t	time (s)
U	vector of dependent variables

u	mean mixture or flow velocity (m/s)
$\tilde{u}_{i+1/2}$	Roe's approximation for mean flow velocity (m/s)
x	longitudinal spatial coordinate (m)
z	elevation (m)
z_e	equivalent bed elevation including the sediment storage in the water column (m)
z_{ws}	water surface elevation (m)
$\alpha^{(2)}$	parameter equal to one-quarter
$\alpha^{(4)}$	parameter equal to 1/256
$\tilde{\alpha}_{i+1/2}^k$	coefficients obtained from the linearisation process (m)
Δt	time step (s)
Δx	spatial step (m)
$\varepsilon_{i+1/2}^{1,2}$	parameters in Equation 8, given by Equation 9 (m/s)
$\varepsilon_{i+1/2}^{(2)}$	second-order term of Jameson's artificial viscosity (m/s)
$\varepsilon_{i+1/2}^{(4)}$	fourth-order term of Jameson's artificial viscosity
ρ	mixture mass density (kg/m ³)
τ_{bc}	bottom shear stress (N/m ²)
$\phi_{i+1/2}^k$	flux limiter for second-order oscillations
φ	parameter in Equation 2 (kg/s ²)
$\psi_{i+1/2}^k$	entropy condition (m/s)

Subscripts or superscripts

b	immobile bed
c	sheet-flow layer
cor	corrector step
d	initial value downstream of the gate
f	fluid or liquid constituent
i	mesh coordinate corresponding to the longitudinal coordinate $x = i\Delta x$
n	mesh coordinate corresponding to time $t = n\Delta t$
pre	predictor step
s	solid or sediment constituent
u	initial value upstream of the gate
w	clear water layer

1. INTRODUCTION

Geomorphic dam-break flows have been numerically simulated by several authors (Abderrezzak *et al.*, 2008; Cao *et al.*, 2004; Capart and Young, 1998; Ferreira and Leal, 1998; Fraccarollo and Capart, 2002; Leal, 2005). Most of the solutions were obtained by using explicit second-order total variation diminishing (TVD) methods. Some authors (Abderrezzak *et al.*, 2008; Cao *et al.*, 2004; Capart and Young, 1998) preferred an upwind approximation solving the Riemann problem at cell interfaces (Godunov-type methods);

others adopted space-centred schemes (Lax–Wendroff type) adding a TVD correction term (Ferreira and Leal, 1998; Leal, 2005). Space-centred schemes with TVD correction give similar results as upwind schemes (Hirsch, 1990). Both schemes, upwind and centred, imply the determination of the Jacobian matrix and its eigenvalues and eigenvectors. For the shallow water equations, the Riemann problem is easy to solve and several Riemann solvers can be found in the literature (Toro, 2001) (HLLC and Roe's solver are the most widely used). For centred schemes the TVD correction term can also be easily computed by using Roe's approximations (Hirsch, 1990). If the sediment mass conservation is added to the shallow water equations, the eigenstructure of the equations becomes more complex and the determination of an algebraic expression for the eigenvalues and the eigenvectors is not possible. Nevertheless, several authors (Abderrezzak *et al.*, 2008; Cao *et al.*, 2004; Capart and Young, 1998; Ferreira and Leal, 1998; Leal, 2005) have successfully used approximations and solvers derived for the shallow water equations in the resolution of shallow water equations plus sediment mass conservation equation.

Experimental data concerning geomorphic dam-break flow are still rare (Capart and Young, 1998; Leal, 2005; Leal *et al.*, 2002; Spinewine and Zech, 2007). For horizontal dry bed, the bed mobility plays an important role. The excavation of a scour hole in the bed provokes the formation of a hydraulic jump (Capart and Young, 1998; Leal *et al.*, 2002). Besides the impact on the water surface and bed elevation, the bed mobility influences the wave-front location; that is, it determines the magnitude of the energy losses (Leal *et al.*, 2006; Spinewine and Zech, 2007). Several conceptual models have been developed; most rely on a vertical subdivision of the flow into several transport layers. They can be grouped into two families: those assuming equilibrium sediment transport (Fraccarollo and Capart, 2002; Ferreira and Leal, 1998; Leal, 2005; Leal *et al.*, 2006) and those assuming non-equilibrium sediment transport (Abderrezzak *et al.*, 2008; Cao *et al.*, 2004; Capart and Young, 1998). The advantages of each assumption are discussed here. Most authors adopted standard sediment transport formulas, derived at low shear rates and under uniform flow conditions, and standard resistance laws, obtained in uniform flows with fixed bed conditions. Such formulas include parameters whose values can be imposed arbitrarily by modellers to fit numerical solutions to experimental data. This parameter estimation lacks physical basis and cannot be reproduced in different contexts. In a companion paper (Leal *et al.*, 2010) a conceptual model was presented. The sediment transport is obtained by equalising the collisional shear stress above the bottom interface to the frictional shear stress below that interface. The model parameters depend on sediment properties and are estimated based on sheet-flow results obtained by other authors (Pugh and Wilson, 1999; Sumer *et al.*, 1996). Therefore, a physically sound model is proposed, without arbitrary choice of values for tuning parameters. The results of the model are compared with experimental data and the main differences are discussed based on the model limitations. The model is expected to be useful for engineers working in inundation and mapping morphological changes, especially when sediment transport is relevant (i.e. when the bed is mobile).

2. NUMERICAL MODEL

The system of conservation laws presented in the companion paper (Leal *et al.*, 2010) can be written in conservative form (Hirsch, 1990; Toro, 2001)

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

where t is time, x is the longitudinal spatial coordinate, $\mathbf{U}$ is the vector of dependent variables, $\mathbf{F}(\mathbf{U})$ is the vector of flux and $\mathbf{S}$ is the vector of source terms. According to the conceptual model (Leal *et al.*, 2010) the three vectors can be written as follows

$$2 \quad \begin{aligned} \mathbf{U} &= \begin{bmatrix} z_{ws} \\ \rho u h \\ z_c \end{bmatrix} \\ \mathbf{F}(\mathbf{U}) &= \begin{bmatrix} hu \\ \rho_w u^2 h_w + \rho_c u^2 h_c + \varphi \\ Chu \end{bmatrix} \\ \mathbf{S}(\mathbf{U}) &= \begin{bmatrix} 0 \\ -g(\rho_w h_w + \rho_c h_c) \frac{\partial z_b}{\partial x} - \tau_{bc} \\ 0 \end{bmatrix} \end{aligned}$$

where $\varphi = \frac{1}{2}g(\rho_w h_w^2 + 2\rho_w h_w h_c + \rho_c h_c^2)$, $\rho = \rho_f(1 - C) + \rho_s C$ is the mixture mass density; subscripts f and s stand for fluid and solid constituents, respectively; $C = (C_c u_c h_c)/(uh)$ is the sediment volumetric concentration; $\mathbf{u} = (u_c h_c + u_w h_w)/h$ is the mean mixture velocity; $z_{ws} = h + z_b$ is the water surface elevation; $z_c = (1 - p)z_b + C_c h_c$ is an equivalent bed elevation including the sediment storage in the water column; subscripts w , c and b stand for clear water layer, sheet-flow layer and immobile bed, respectively; h is the flow depth; z is the elevation; p is the bed porosity; g is the gravity acceleration; and τ_{bc} is the bottom shear stress. From Equation 2 it can be concluded that the system of equations is not strictly written in conservative form, since the source term contains a convective term $\partial z_b/\partial x$.

The eigenstructure of system (1) is hyperbolic; one characteristic speed is associated with the fluid phase and must originate a shock wave, while the other two are influenced by the solid phase, one leading to an expansion wave and the other one originating an expansion wave or a shock wave depending on Froude number (Leal, 2005).

The numerical solution of system (1) is obtained by a finite differences discretisation using MacCormack's explicit scheme (Hirsch, 1990). This scheme is very popular in the numerical simulation of dam-break flows (Alcrudo, 1992; Aureli *et al.*, 2000; Garcia and Kahawita, 1986; Leal, 2005) since it has second-order accuracy, both in time and space, and is able to capture shocks (discontinuities). The scheme is composed of two steps (predictor and corrector), applying alternately regressive and progressive differences. It can be written as

$$\begin{aligned}
 U_i^{n+1} &= \frac{1}{2} (U_i^{\text{pre}} + U_i^{\text{cor}}) \\
 U_i^{\text{pre}} &= \begin{cases} U_i^n - \frac{\Delta t}{\Delta x} (F_{i+1}^n - F_i^n) & \text{if } n = 2, 4, \dots \\ U_i^n - \frac{\Delta t}{\Delta x} (F_i^n - F_{i-1}^n) & \text{if } n = 3, 5, \dots \end{cases} \\
 U_i^{\text{cor}} &= \begin{cases} U_i^n - \frac{\Delta t}{\Delta x} (F_i^{\text{pre}} - F_{i-1}^{\text{pre}}) & \text{if } n = 2, 4, \dots \\ U_i^n - \frac{\Delta t}{\Delta x} (F_{i+1}^{\text{pre}} - F_i^{\text{pre}}) & \text{if } n = 3, 5, \dots \end{cases}
 \end{aligned}$$

where superscripts 'pre' and 'cor' stand for predictor and corrector steps, respectively; i is the mesh coordinate corresponding to the longitudinal coordinate $x = i\Delta x$; Δx is the spatial step; n is the mesh coordinate corresponding to time $t = n\Delta t$; and Δt is the time step (Figure 1). In Figure 1, m_d stands for the mesh coordinate corresponding to the longitudinal coordinate of the dam location.

The solution of system (1) contains discontinuities and therefore, according to Godunov's theorem (Toro, 2001), the use of a scheme with accuracy higher than the first order will originate spurious oscillations. To avoid these oscillations, a high-resolution TVD correction is needed (Hirsch, 1990), rendering

$$U_i^{n+1} = \frac{1}{2} (U_i^{\text{pre}} + U_i^{\text{cor}}) + \frac{\Delta t}{\Delta x} (D_{i+1/2}^n - D_{i-1/2}^n) + \Delta t S(U_i^n)$$

Figure 1. Computational mesh domain

where D is the TVD correction term. The source term computed as $S(U_i^n)$ leads to good results (Leal, 2005).

The TVD correction can be written for mesh points i and $i+1$ as (Alerudo, 1992)

$$\begin{aligned}
 D_{i+1/2}^n &= \frac{1}{2} \sum_{k=1}^2 \tilde{a}_{i+1/2}^k \psi_{i+1/2}^k \\
 &\times \left[1 - \frac{\Delta t}{\Delta x} |\tilde{a}_{i+1/2}^k| \right] \left[1 - \phi_{i+1/2}^k \right] \tilde{e}_{i+1/2}^k
 \end{aligned}$$

where $\tilde{a}_{i+1/2}^k$ and $\tilde{e}_{i+1/2}^k$ are, respectively, the k eigenvalues and k eigenvectors of the Jacobian matrix $\tilde{A}_{i+1/2}$ that linearises the non-linear system (1); $\tilde{a}_{i+1/2}^k$ are coefficients obtained from the linearisation process; $\psi_{i+1/2}^k$ is the entropy condition; and $\phi_{i+1/2}^k$ is the flux limiter for second-order oscillations.

For the shallow water equations in fixed bed, the Jacobian matrix $\tilde{A}_{i+1/2}$ is easily computed rendering the well-known Roe's approximations (Alerudo, 1992; Toro, 2001)

$$\begin{aligned}
 \tilde{c}_{i+1/2} &= \sqrt{g \frac{(h_{i+1} + h_i)}{2}} \\
 \tilde{u}_{i+1/2} &= \frac{\sqrt{h_{i+1}} u_{i+1} + \sqrt{h_i} u_i}{\sqrt{h_{i+1}} + \sqrt{h_i}}
 \end{aligned}$$

The eigenvalues and eigenvectors of Jacobian matrix $\tilde{A}_{i+1/2}$ and the linearisation coefficients are given by

$$\begin{aligned}
 \tilde{a}_{i+1/2}^{1,2} &= \tilde{u}_{i+1/2} \pm \tilde{c}_{i+1/2} \\
 \tilde{c}_{i+1/2}^{1,2} &= \begin{bmatrix} 1 \\ \tilde{a}_{i+1/2}^{1,2} \end{bmatrix} \\
 \tilde{a}_{i+1/2}^{1,2} &= \frac{h_{i+1} - h_i}{2} \\
 &\pm \frac{1}{2\tilde{c}_{i+1/2}^{1,2}} [h_{i+1} u_{i+1} - h_i u_i - \tilde{u}_{i+1/2} (h_{i+1} - h_i)]
 \end{aligned}$$

The entropy condition $\psi_{i+1/2}^k$ developed by Harten and Hyman (1983) can be written as (Hirsch, 1990)

$$\psi_{i+1/2}^{1,2} = \begin{cases} \tilde{a}_{i+1/2}^{1,2} & \text{if } |\tilde{a}_{i+1/2}^{1,2}| \geq \varepsilon_{i+1/2}^{1,2} \\ \varepsilon_{i+1/2}^{1,2} & \text{if } |\tilde{a}_{i+1/2}^{1,2}| < \varepsilon_{i+1/2}^{1,2} \end{cases}$$

where $\varepsilon_{i+1/2}^{1,2}$ is a value near zero that should be estimated by

$$\varepsilon_{i+1/2}^{1,2} = \max \left[0, \left(\tilde{a}_{i+1/2}^{1,2} - a_i^{1,2} \right), \left(a_{i+1}^{1,2} - \tilde{a}_{i+1/2}^{1,2} \right) \right]$$

In the equation, $a_i^{1,2} = u_i \pm c_i$ represents the exact eigenvalues of the shallow water equations at point i and $c_i = \sqrt{gh_i}$ is the celerity of small perturbations. The flux limiter of Van Leer will be adopted since it has been verified to give good results in

dam-break flow simulations (Alcrudo, 1992; Wang *et al.*, 2000). This flux limiter can be written (Hirsch, 1990)

$$\phi(r_{i+1/2}^k) = \frac{r_{i+1/2}^k + |r_{i+1/2}^k|}{1 + r_{i+1/2}^k}$$

where $r_{i+1/2}^k$ is given by

$$r_{i+1/2}^k = \frac{\tilde{a}_{i+1/2-m}^k \left(1 - \left|\frac{\Delta t}{\Delta x} \tilde{a}_{i+1/2-m}^k\right|\right) \tilde{a}_{i+1/2-m}^k}{\tilde{a}_{i+1/2}^k \left(1 - \left|\frac{\Delta t}{\Delta x} \tilde{a}_{i+1/2}^k\right|\right) \tilde{a}_{i+1/2}^k}$$

with

$$m = \begin{cases} -1 & \text{if } \tilde{a}_{i+1/2}^k < 0 \\ +1 & \text{if } \tilde{a}_{i+1/2}^k > 0 \end{cases}$$

For a mobile bed, due to the complexity of the Jacobian matrix of system (1), it is not possible to determine the algebraic expressions of the TVD correction. To overcome this difficulty, it is proposed to apply the TVD correction computed for fixed bed to the mixture mass and momentum conservation equations. To avoid oscillations in the bed profile an artificial viscosity term was used in the sediment mass conservation equation. The artificial viscosity proposed by Jameson in 1981 is adopted rendering a correction term (Hirsch, 1990)

$$D_{i+1/2}^n = \varepsilon_{i+1/2}^{(2)}(U_{i+1} - U_i) - \varepsilon_{i+1/2}^{(4)}(U_{i+2} - 3U_{i+1} + 3U_i - U_{i-1})$$

where the second-order term is written

$$\varepsilon_{i+1/2}^{(2)} = \max(\varepsilon_i^{(2)}, \varepsilon_{i+1}^{(2)})$$

with

$$\varepsilon_i^{(2)} = \alpha^{(2)}(|u_i| + c_i) \frac{|z_{ws,i+1} - 2z_{ws,i} + z_{ws,i-1}|}{z_{ws,i+1} + 2z_{ws,i} + z_{ws,i-1}}$$

The fourth-order term is given by

$$\varepsilon_{i+1/2}^{(4)} = \max\left[0, \left(\alpha^{(4)} - \frac{\varepsilon_{i+1/2}^{(2)}}{u_{i+1/2} + c_{i+1/2}}\right)\right]$$

where $\alpha^{(2)}$ and $\alpha^{(4)}$ are adjustable coefficients that are typically equal to 1/4 and 1/256, respectively (Hirsch, 1990).

The discretisation of the partial derivative included in the source term is obtained by applying centred differences $\partial z_b / \partial x \approx (z_{b,i+1} - z_{b,i-1}) / (2\Delta x)$, which gives good results (Leal,

2005). Finally, since the scheme is explicit, the Courant–Fredrich–Lewy stability condition is used (Hirsch, 1990; Toro, 2001).

3. COMPARISON BETWEEN NUMERICAL AND EXPERIMENTAL RESULTS

The experimental data are taken from the geomorphic dam-break flow experiments carried out by the authors (Leal, 2005; Leal *et al.*, 2002, 2006). The tests consisted in a 20.05 m long and 0.5 m wide horizontal channel with a 10.08 m long reservoir closed by a gate that simulates the dam ($x = 0$ m being fixed at the gate cross-section). The gate was instantaneously removed using a counterweight mechanism (removal times below 0.12 s). The tests comprised dry and wet conditions downstream of the gate and two types of mobile bed: sand and pumice. The dry bed condition corresponds to an initially saturated bed with water depth below 0.01 m. The properties of the sediments and the corresponding parameter estimation are presented in a companion paper (Leal *et al.*, 2010). The initial conditions of the experiments are summarised in Table 1. In this table, superscripts u and d stand for initial values upstream and downstream of the gate, respectively, and the elevation z is measured starting from the concrete bottom of the flume. The numerical model was run with Courant number of 0.9 and with a spatial step $\Delta x = 1$ cm.

3.1. Wave and bed profiles

Figure 2 presents the comparison of wave and bed profiles at time $t = 1, 2$ and 4 s obtained numerically against experimental data for sand bed and pumice bed tests and initial dry bed downstream of the gate.

For sand bed (Figures 2(a), (c) and (e)) it can be concluded that the numerical results are generally in good agreement with the experimental data. However, for small times ($t = 1$ s), some discrepancies can be mentioned, namely in the height and position of the wave-front for test S1 (dry bed condition) and in the negative wave propagation for both tests S1 and S2 (dry and wet bed conditions, (Figure 3)). The discrepancies in the wave-front are discussed in the next section. The overestimation of the negative wave celerity by the model can be attributed to the negligence of vertical acceleration, that is, neglecting curvature of streamlines (Lauber and Hager, 1998; Stansby *et al.*, 1998).

Comparing the sand and pumice bed results (Figure 2), it can be concluded that the procedure for estimating the friction coefficient, based only in sediment properties, is generally adequate. For dry pumice bed (Figures 2(b), (d) and (f)), the model adjustment to the experimental data is similar to the sand bed case (Figures 2(a), (c) and (e)) enhancing the adequacy of the

Test	Type of bed	$z_b^u = z_b^d$, m	z_{sw}^u , m	z_{sw}^d , m
S1	Sand	0.071	0.400	0.003
S2				0.042
P1	Pumice			0.013
P2				0.042

Table 1. Initial conditions of the experimental tests by Leal (2005)

model to simulate mobile bed regardless of its mobility. In the higher mobility bed (pumice) the bed morphological changes are neater and a scour hole is carved near the gate. The numerical model is able to simulate the scour hole, though wider and shallower than observed in the experiments (Figures 2(d) and (f)). The discrepancies, which are confined to a narrow region near the gate, are probably to be ascribed to the sediment transport equilibrium assumption. The underestimation of the scour hole depth reflects itself in the wave profile, where, instead of the observed hydraulic jump (Capart and Young, 1998; Leal *et al.*, 2002), a constant state appears (Figures 2(d) and (f)).

For wet bed conditions (Figure 3), the sediment transport intensity is drastically reduced and the wave-front location depends mostly on the inertia associated with the initial downstream water level and therefore the numerical results, for both types of bed, show better agreement with the experimental data.

3.2. Time evolution of the water surface elevation

Figure 4 presents the comparison of the numerical time evolution of the wave surface elevation against experimental

data for sand bed and pumice bed tests with dry and wet bed conditions. The comparison between numerical results and experimental data shows good agreement, especially for wet bed conditions, indicating that the model is suitable for predicting water levels.

For both sand and pumice dry bed (Figures 4(a) and (b)) the model reveals some inability in capturing the exact location of the wave-front, being the numerical propagation too fast in the sections near the gate and too slow in the sections located far from the gate. This can be caused by two shortcomings of the model, the exclusion of vertical accelerations and the use of a constant friction coefficient along the wave profile, or by experimental data bias due to the gate lifting process that can artificially disturb the bed (Spinewine and Zech, 2007). The vertical accelerations influence the initial stages of the wave propagation (Fraccarollo and Capart, 2002; Stansby *et al.*, 1998), namely for times smaller than $5\sqrt{h_u/g} = 5\sqrt{0.4/9.8} = 1$ s. Therefore, this effect can only explain discrepancies found in the gate nearest sections. The same argument can be used for the influence of the gate lifting process in experimental data bias. Far apart from the gate it is clear that the use of a

constant friction factor is not rigorously adequate. Since near the wave-front the flow has high sediment concentration, friction should be determined by collisional and frictional grain-scale interactions (Leal *et al.*, 2010).

Small differences are also found in the shape of the wave-front, which is sharper in the numerical solution than in the experimental observations. The air entrance observed in the wave-front, and not accounted for in the model, can be responsible for those differences.

4. WAVE STRUCTURE

Observing the Froude number and mean velocity obtained numerically for the conditions of tests S1 and P1 (see Figure 5) it can be concluded that the wave structure is composed of an initial upstream constant state followed by a rarefaction wave where velocity increases (Figure 5) and water depth decreases (Figure 2). This rarefaction wave, in subcritical regime ($Fr < 1$), ends on a constant state, that is formed near the gate section, with a Froude number initially near 1, corresponding to critical regime. As time progresses the length of this constant state

increases and its Froude number decreases (Figure 5(a) and (c) or 5(b) and (d)), since the water depth increases due to the increasing scour-hole depth (Figure 2). This time evolution of the constant state is neater for the higher mobility bed (pumice). After the constant state a second rarefaction wave develops, but this time in supercritical regime ($Fr > 1$), ending in a new constant state that extends until the wave-front. This wave structure is similar to the one presented in a previous analytical study (Fraccarollo and Capart, 2002).

If the bed is initially wetted, as in the case of tests S2 and P2, the wave structure is similar (Figure 6) but all the wave profile is in subcritical regime ($Fr < 1$) and the second rarefaction wave is much smaller.

5. CONCLUSIONS

A numerical solution of the geomorphic dam-break flow conceptual model presented in a companion paper (Leal *et al.*, 2010) was developed. The wave structure of the numerical solution is in accordance with previous studies (Fraccarollo and Capart, 2002) being composed of two undisturbed regions

Figure 4. Water surface elevation time evolution at sections $x = 0.2, 1, 2, 3, 5, 7$ and 9 m. Comparison between experimental results (grey circle) and numerical results (black line): (a) test S1 (sand dry bed); (b) test P1 (pumice dry bed); (c) test S2 (sand wet bed); (d) test P2 (pumice wet bed)

Figure 5. Froude number, mean flow velocity and mean velocity in the sheet-flow layer obtained numerically for the conditions of: (a) test S1 at $t = 1$ s; (b) test P1 at $t = 1$ s; (c) test S1 at $t = 2$ s; (d) test P1 at $t = 2$ s

connected by two rarefaction waves and one shock/rarefaction wave intercalated with two almost constant states.

The comparison between numerical results and experimental data reveals that generally a good agreement is obtained. The

procedure proposed for estimating the friction coefficient, based only in sediment properties, is confirmed to be adequate for simulating both sand and pumice bed. Moreover, the estimation of model parameters based on existing sheet-flow data (Pugh and Wilson, 1999; Sumer *et al.*, 1996) revealed to

be a promising way to improve conceptual models and reduce uncertainty in parameters estimation.

However, some localised discrepancies were found, which can be explained based on the conceptual model limitations. Namely, the equilibrium approach seems unable to reproduce the hydraulic jump observed in the bed composed of higher mobility material (pumice). The use of a constant friction coefficient throughout the profile overestimates the wave-front location in the initial stages and underestimates that location for higher times.

Some improvements of the conceptual model can be made, namely the friction coefficient estimation might be improved by a more in-depth analysis of existing sheet-flow data or by performing more sheet-flow experiments under uniform and non-uniform conditions.

As for incorporating non-equilibrium sediment transport through the introduction of an adaptation length or a relaxation process (Tsuji *et al.*, 1990), the subject is more controversial. This is not only because it introduces an additional tuning parameter but mostly because of the difficulty in justifying the use of non-equilibrium sediment transport to simulate short-term bed morphological changes.

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