The stepped dam-break test case: results from the IMPACT benchmark

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SUMMARY

Within the IMPACT project, one aim of the Sediment Movement work package is to investigate sediment transport processes under dam break flows. A benchmarking procedure has been set up among the related institutions to face numerical model outputs with experimental data of idealised dam-break experiments under various conditions. In an earlier phase of the project, a first benchmark was dedicated to dam-break wave over a flat bed made of a single uniform loose material. The present benchmark explores a new configuration for two different bed materials and with the presence of an initial bed discontinuity across the dam. Experiments were carried out in a new flume, and instrumentation of the tests performed by means of fast digital imaging through the flume sidewall and bulk measurements at the flume outlet. Five modellers from four different institutions ran blind simulations of the flow. Those are compared with the experimental results in the present paper.

1 INTRODUCTION

The amount of sediment material entrained by catastrophic floods as resulting from dam or dike failures may be huge (Capart, 2000; Costa and Schuster, 1988; Brooks and Lawrence 1999), sometimes the same order of magnitude as the initial volume of water in the reservoir. Through friction, inertial effects and momentum exchanges with the fluid phase, erosion of bed material may in turn significantly affect the flood wave development in terms of arrival time of the wave front and envelope of maximum attained flood levels, two parameters of utmost importance for emergency planning, risk management and damage assessment.

Modelling accurately geomorphic changes associated with dam-break flows is thus important. There is a need for research in the field of sediment movement in severe and transient conditions, for which traditional predictors fail. For the validation of numerical models and the testing of modelling assumptions, constitutive behaviour and erosion criteria, there is a need among the community of modellers for reliable experimental data for idealised configurations with a limited degree of uncertainty and a good reproducibility.

The objectives of the ‘Sediment Movement’ work package of the EU-funded IMPACT project are precisely in phase with that concern: to realise a series of well-documented experimental datasets, and to confront them with simulations that rely on different modelling strategies.
Idealised laboratory dam-break experiments were performed in a straight prismatic flume. The idealisation towards instantaneous dam collapse provides a well-defined initial-value problem for numerical models. Comparisons and interpretation are thus facilitated. Laboratory experiments provide a first step for validation, in view of applying models to real-scale case-studies, the next objective of the IMPACT project (Capart et al. 2003).

In the framework of IMPACT, a first benchmark session consisted in the simulation of dam-break waves over a flat movable bed made of light sediment analogues, namely PVC pellets. The test case was presented in Spinewine and Zech (2002) and comparisons of the modellers results with experimental observations may be found in Spinewine and Zech (2003).

This paper presents the results of a new modelling exercise, associated with a second series of experiments, which investigate the influence of differing initial conditions and bed materials. The bed levels show an initial discontinuity across the dam section, the upstream bed level being higher than its downstream counterpart. Two series of tests were carried out with distinct bed materials differing both in grain size and density, namely the same PVC pellets and coarse sand, the latter being closer to natural material. The bed step provides a rough analogue to reservoirs partially filled with sediments, that are re-mobilised during the dam-break flood.

The description of this second test case is found in Spinewine and Zech (2004). Here we present the experimental results and comparisons with the various model outputs. The experimental observations were obtained using digital imaging through the channel sidewall, and bulk measurements at the flume outlet. Five modellers from four partner institutions ran blind simulations of the flow.

The paper is structured as follows: in section 2 we briefly recall the conditions of the test case. Section 3 is devoted to a general description of the flow. Formatted experimental profiles are presented in section 4. Finally, section 5 lists the benchmark participants, briefly summarises the main characteristics of the models they have used, and finally compares their results with the experimental data.

2 TEST CASE DESCRIPTION

The test conditions for this benchmark are here briefly recalled. They were presented more in details by Spinewine and Zech (2004) in the opening paper launching the benchmark session.

The experiments were carried out in a new flume at the laboratory of the Civil and Environmental Engineering Department, Université catholique de Louvain (UCL), Belgium. The flume is 6 m long, 0.25 m wide and 0.7 m high, and was specifically designed for idealised dam-break experiments on movable beds (Fig. 1a). Breaking of the dam is simulated by the rapid downward movement of a gate at the middle of the flume, entrained by a pneumatic jack. Opening time in the order of 0.1 to 0.2 s is achieved over the full nominal height of 50 cm. The flume provides a series of substantial improvements compared with the previous flume used for the “flat bed” test case experiments, mainly in terms of rapidity of dam collapse (0.1 s.) and direction of movement (downwards), available water depth (50 cm), length (6 m) and measurements (continuous transparent sidewall).
Two series of tests were performed with different bed material. The initial conditions were identical, as summarised in Figure 1b. The initial bed profile feature horizontal reaches upstream and downstream, with an initial discontinuity at the gate section, so that the upstream level is initially $h_{s,0} = 10$ cm higher than the downstream level. In the upstream reservoir, an additional layer of 25 cm of pure water at rest is provided, so that the total head upstream is $h_0 = 35$ cm. The upstream reach is ended by a wall while the downstream reach is closed partially by a weir whose crest level corresponds to the downstream sediment level, in such a way that the sediment bed is fully saturated in the initial conditions.

The two types of particles used as bed material were uniform coarse sand and light PVC pellets. Estimated characteristic properties are summarised in Table 1. Roughness factor was estimated for the coarse sand in uniform flow experiments, leading to a Manning coefficient of $n = 0.0165$ s/m$^{1/3}$.

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Symbol</th>
<th>Dimensions</th>
<th>Sand</th>
<th>PVC</th>
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<tr>
<td>Specific density</td>
<td>$\gamma_s$</td>
<td>[kN/m$^3$]</td>
<td>26.33</td>
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<tr>
<td>Grain size</td>
<td>$d_{50}$</td>
<td>[mm]</td>
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<td>3.6</td>
</tr>
<tr>
<td>Grain shape</td>
<td>–</td>
<td>–</td>
<td>crushed, angular</td>
<td>Cylindrical</td>
</tr>
<tr>
<td>Porosity</td>
<td>%</td>
<td>%</td>
<td>43</td>
<td>42</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\varphi'$</td>
<td>[°]</td>
<td>30</td>
<td>38</td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c'$</td>
<td>[N/m$^2$]</td>
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<td>0</td>
</tr>
<tr>
<td>Permeability</td>
<td>$k_s$</td>
<td>[m/s]</td>
<td>0.0154</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

Table 1. Material properties.
3 FLOW DESCRIPTION

Figure 2 shows images corresponding to the release of the gate, where one may clearly see the nearly vertical wall of water left over when the gate is rapidly removed. Figures 3 and 4 show full flow mosaics at selected instants, for the sand and PVC case respectively.

![Figure 2. Close-ups of initial stages during gate opening. Left: initial condition; middle: nearly unperturbed water body after gate removal; right: wave formation at a later instant.](image)

During the initial stages of wave formation, massive soil movements occur around the bed step. The vertical step exceeds the internal friction angle of the bed material, resulting in an instability and a mass failure. This failure is further amplified by the sudden decrease in pore pressures as the upper water body collapses. Especially for the lighter PVC pellets, a substantial volume of soil around and below the initial step is liquefied, and set in motion (see Figs. 4a and 5b). The plastic behaviour of this soil movement has little to do with traditional sediment transport mechanisms and may be difficult to account for. However, part of this volume is rapidly re-deposited once the vertical step has turned into a smoother profile.

In a second phase, a stable wavefront develops. This front is heavily loaded with sediments, almost completely saturated both in the case of sand and PVC. Bed material continues to be incorporated in the flow as the front progresses. Both the height of the transport layer and the height of the shockwave are notably higher for the PVC tests.

Bedforms tend to develop, again much more pronounced for the PVC bed. Those antidune-like forms are in phase with the water level fluctuations, but they migrate downstream, at a speed which is slightly smaller than the wavefront. They are relatively stable and reproducible, and tend to adopt a regular pattern with stable wavelength. Such bedforms were not observed for previous series of tests performed with a lower initial upstream water level.

In a third phase, the wavefront tends to decelerate and the intensity of transport decreases. Partial re-deposition of sediments occurs in the first reaches.
On the upstream side, the flow is more gentle. We observed the gradual formation of a weak hydraulic jump in the case of the sand tests, but not for the PVC tests. This comes in contrast with the observations by Capart and Young (1998) and Ferreira et al (2003), from which the formation of a hydraulic jump is believed to be induced by the non-equilibrium response of the sediment transport rate. This non-equilibrium behaviour is more pronounced as the sediment density decreases. One would thus expect a hydraulic jump to occur for the PVC tests. However, in the present case of a bed discontinuity, the formation of a hydraulic jump is believed to be affected by the initial mass failure of the step. During the PVC tests, a lot more sediments are set in motion by this slope failure, and thus the sediment transport reaches faster its equilibrium capacity. This interpretation is supported by the observation that on the contrary, on flat beds with no initial mass failure, the PVC tests were more prone to form a hydraulic jump just downstream of the dam than their sand counterparts.
4 EXPERIMENTAL RESULTS

Digital imaging through the transparent sidewall was used to obtain detailed experimental measurements of the flow profiles. The cameras operated at a frequency of 200 frames per second, but only selected instants were chosen for the comparison with model outputs. Flow interfaces were detected using a semi-automated procedure. In addition, the total mass of sediments transported by the flood and reaching the outlet was obtained by collecting outflowing sediments in a sieve: 6.37 kg for the PVC test, and 2.63 kg for the sand test.

Figure 4. Reconstructed flow mosaics for the PVC tests at selected instants $t = 0.25$, $0.5$, $0.75$, $1.0$, $1.5$ and $2.0$ s. Dotted green, yellow and red lines indicated the profiles of water, transport layer and sediment bed levels respectively.
Experimental profiles are plotted in a standard form in Figure 5. The propagation of the downstream shock and upstream rarefaction are easily identified, as well as a relatively stable pivot point around \( x = 0.2 \) m. In order to facilitate the interpretation and comparison with results from the modellers, the profiles are reformatted in a dimensionless form according to Froude similarity (Figs 6 and 7), by plotting \( z/h_0 \) as a function of \( x/(t gh_0)^{0.5} \), \( h_0 \) being the total available head in the upstream reservoir \( h_0 = 35 \) cm. Except in the dam region, self-similarity is relatively well preserved for the sand tests. The progressive deceleration of the wavefront is especially visible for the PVC test.

Figure 5. Experimental profiles at multiples of characteristic time \( t_0 = (h_0/g)^{1/2} = 0.189 \) s. (a) Sand tests; (b) PVC tests.

Figure 6. Dimensionless experimental profiles for the sand test case at multiples of characteristic time \( t_0 = (h_0/g)^{1/2} = 0.189 \) s
Figure 7. Experimental profiles for the PVC test case at multiples of characteristic time $t_0 = (h_0/g)^{1/2} = 0.189$ s. (a) true profiles; (b) dimensionless form.

5 BENCHMARK RESULTS

Participants were requested to provide the following results, formatted in dimensionless form as defined above:

- Snapshots of the flow, with the position of the relevant flow interfaces (water, transport layer, bed), prepared as graphs of $z/h_0$ as a function of $x/h_0$, at selected dimensionless times $t = t_0 \times [1, 2, 3, 4, 5, 6, 8, 10, 12, 14]$, with $t_0 = (h_0/g)^{1/2} = 0.189$ s.
- The same snapshots scaled along an auto-similar reference frame, as graphs of $z/h_0$ as a function of $x/(t_0(g h_0)^{1/2})$, at the same selected times.
- Time evolution profiles of water, transport layer and bed levels $z_w/h_0, z_s/h_0, z_b/h_0$ (see Fig. 5 for definition of the various levels) at selected dimensionless locations $x = h_0 \times [-2, -1, 0, 1, 2, 3, 4, 6, 8]$, prepared as graphs of $z/h_0$ as a function of $t/t_0$.
- Characteristic path of the wave front $x_f$, and the negative wave into the reservoir $x_b$, prepared as graphs of $t/t_0$ as a function of $x/h_0$.
- The envelope of the maximum water levels attained along the flume during the whole duration of the flood wave, prepared as a graph of $z_{w, \text{max}}/h_0$ as a function of $x/h_0$.
- The envelope of the minimum bed levels attained along the flume during the whole duration of the flood wave, prepared as a graph of $z_{b, \text{min}}/h_0$ as a function of $x/h_0$.
- The cumulative mass of solid material collected at the outlet of the flume as a function of time.
5.1 Benchmark participants and summary of the type of model used

The blind test was run by five different modellers from the four institutions involved in the IMPACT ‘Sediment Movement’ Workpackage: El Kadi and Paquier from the Cemagref, France, Spinewine from the Catholic University of Louvain, Belgium, Fraccarollo from the university of Trento, Italy, Ferreira and Leal from the Technical Institute of Lisbon, Portugal. The modellers are listed below along with a short description of the type of model used. For details concerning each particular model we refer to the papers by individual modellers listed in the references. A summary is given in Table 2.

The Cemagref model (CEM) RubarBE (El Kadi and Paquier, 2004) relies on the classical Saint-Venant equations, solved by a second-order Godunov-type explicit scheme. Sediment routing is performed with a Finite Differences scheme, uncoupled from the hydrodynamic computations. The sediment transport capacity computed from a suitable bed load formula is corrected by a space lag equation, and the conservation of bed material is expressed by the Exner equation.

The other four models rely on a multi-layer approach for simulating explicitly the active sediment transport layer. The models mainly differ in the constitutive laws adopted for erosion/deposition and friction, and on the assumptions for flow velocities and sediment concentrations in the two layers.

The model used by Fraccarollo (University of Trento), described in Fraccarollo et al. (2003), relies on a Rankine-Hugoniot-like argument relating the erosion/deposition rate to a difference between the shear stresses experienced on both sides of the bed interface. The latter is viewed as a sharp transition zone between solid and fluid-like behaviour of the sediment grains. Velocity is assumed constant throughout the flow depth, and the solid concentration within the transport layer is equal to that of the immobile bed. The numerical scheme is based on a second-order Finite Volume scheme with the LHLL solver, a lateralised version of the Harten-Lax-Van Leer approximate Riemann solver.

The model from the Catholic University of Louvain (Spinewine, 2003) adopts a similar conceptualisation, but the velocities in the two moving layers are distinct. The solid concentration in the transport layer is homogeneous but different from the solid packing of the immobile bed. As a consequence, in case of erosion/deposition mass and momentum are exchanged between the two moving layers, as water is transferred to relax the deficit/surplus in pore volumes within the transport layer. In addition the model is endowed with a slope failure mechanism (Spinewine et al., 2002) accounting for local mass failure when the bed adopts a profile steeper than the natural angle of repose. The governing equations are solved with a first-order Finite Volume scheme based on the same LHLL Riemann solver. It must be mentioned that, since the experiments were also performed at the UCL, their results may not be thoroughly considered as a blind simulation. The very limited number of empirical factors in the model by Spinewine, however, does not allow for a systematic calibration against the experimental profiles.

The Technical Institute of Lisbon (IST) produced two sets of numerical results, based on equilibrium and non-equilibrium versions of a similar model (Ferreira et al., 2003), respectively by Leal and Ferreira. The models separate the flow in a sheet-flow layer and a pure water layer. The solid concentration in the sheet flow layer is dictated by a governing
The governing equations are solved with a Mac-Cormack TVD Finite Difference scheme at second-order accuracy. Non-equilibrium and equilibrium versions refer or not to an imbalance between actual transport and capacity obtained when the concentration in the sheet flow layer has reached its target value.

The various modellers and institutions are summarised in table 2, along with the abbreviation that is used to identify them in the comparative graphs of next section.

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Author(s)</th>
<th>Code</th>
<th>Multi-layers</th>
<th>Scheme</th>
<th>Order</th>
<th>Mesh size (cm)</th>
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<tr>
<td>CEMAGREF</td>
<td>El Kadi &amp; Paquier</td>
<td>CEM</td>
<td>N</td>
<td>FV</td>
<td>2</td>
<td>6</td>
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<tr>
<td>UDT</td>
<td>Fraccarollo</td>
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<td>Y</td>
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<tr>
<td></td>
<td>Ferreira</td>
<td>ISTne</td>
<td>Y</td>
<td>FD</td>
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<td>1</td>
</tr>
</tbody>
</table>

Y: Yes; N: No; FV: Finite Volumes; FD: Finite Differences;

Table 2: Participants to the benchmark

5.2 Comparison of numerical and experimental results

The five sets of results are compared among each other and with the experimental results. Time-evolution profiles of water and bed levels at fixed sections, as the one used for the comparisons of the flat bed test case (Spinewine and Zech 2003) are omitted for the sake of brevity and because of their delicate interpretation. We preferred instantaneous snapshots of the flow at selected instants, arranged in the same dimensionless form as for the experimental profiles shown in the previous section. Profiles are presented concurrently for the sand and the PVC tests at dimensionless times \( t = 3, 4, 6, 8 \) and 12 \( t_0 \), with \( t_0 = (h_0/g)^{1/2} = 0.189s \) and \( h_0 = 35cm \). Of particular interest is also the comparison with sediment volumes transported throughout the flume and collected at the outlet. This provides a raw quantification for global geomorphic changes in the channel.

5.2.1 Flow snapshots at \( t = 3 t_0 = 0.57s \)

As seen in the complete experimental profiles of previous section, the initial stages of dam-break wave formation are rather delicate, especially for this stepped-bed configuration, involving strong vertical effects during the initial collapse and plastic mass failure of the soil around the bed step. The first comparable instant was thus chosen at \( t = 3 t_0 \), a time at which the wavefront formation has developed, initial mass soil movements have slowed down and the flow is more amenable to a shallow water description.

Figure 8 shows the comparison between experimental and numerical results. The models results differ slightly in terms of wavefront position. The height of the mobilised transport layer is generally well reproduced by the modellers adopting a multi-layer approach, while the Cemagref profiles significantly under-predict the scour in the downstream region. A major observation concerns the geomorphic evolution of the initial bed discontinuity. All but the
UCL model preserve a steep discontinuity in terms of bed and water levels across the dam, whereas the experiments adopt a much milder profile. By incorporating a liquefaction criterion, the model by Spinewine accounts for the initial mass failure of the bed step. Encouragingly however, this phenomenon is seen to have only limited influence on the general shape and speed of the wavefront, as the other models behave satisfactorily in this region.

5.2.2 Flow snapshots at $t = 4 t_0 = 0.75s$

Profiles for $t = 4 t_0$ are plotted in Figure 9. As for the previous instant, the models are quite satisfactory for the sand tests, except around the dam location. There is more scatter among the various results for the PVC tests. Even if behaving better close to the dam, the UCL model significantly overestimates the height of the saturated wavefront shock. The general flow behaviour in the downstream region is best captured by the non-equilibrium version of the IST model by Ferreira.

The PVC profiles further exhibit antidunes-like oscillations of the water and sediment surfaces, that are not reproduced by any of the models. While Capart and Young (2002) have shown that antidunes may be simulated naturally under a two-layer shallow-water framework, it is not clear whether the present bed forms arise from the same mechanisms or are initiated by the vertical and non-hydrostatic effects during the initial stages of wave development.
5.2.3 Flow snapshots at $t = 6 t_0 = 1.13s$

At $t = 6 t_0$ the difference in wavefront celerity between the sand and PVC tests is clear, as seen by comparing the experimental profiles in Figure 10a and b. This difference in behaviour is partly due to the increased frictional resistance of the PVC pellets, but most importantly to the order of magnitude of sediment entrainment. Erosion of bed material is associated with a net loss of flow energy, momentum being transferred to the initially immobile grains to set them in motion. This intrinsic relation between front celerity and bed erosion may explain part of the discrepancies between numerical results. The results by El Kadi and Paquier, for example, significantly underestimate the erosion rate for the PVC tests, and in turn show little difference in wavefront position between sand and PVC tests.

On the experimental PVC profiles, the upstream migrating bedforms are seen to have reached a kind of stable pattern, with a regular wave length over the whole length, and are always in phase with milder fluctuations of the water table.

Figure 10. Experimental vs. numerical results at time $t = 6 t_0$. (a) PVC tests; (b) sand tests.

5.2.4 Flow snapshots at $t = 8 t_0 = 1.51s$

At time $t = 8 t_0$ (Fig. 11), the wavefront for the sand tests has reached the outlet of the flume, which is not yet the case of the PVC tests. Compared with previous instants, the first reaches of the channel ($x/(t \sqrt{gh_0}) < 0.5$) are now enduring partial re-deposition of sediments as the flow intensity decreases. The non-equilibrium version of the IST model by Ferreira, however, continues to develop a deeper scour hole just downstream of the dam, and this in turn affects the water profile.
Figure 11. Experimental vs. numerical results at time $t = 8.0t_0$. (a) PVC tests; (b) sand tests.

5.2.5 Flow snapshots at $t = 12t_0 = 2.26s$

Figure 12 shows the profiles corresponding to $t = 12t_0$. The water table for the PVC tests is best reproduced by the models by Leal (ISTe) and Fraccarollo (UDT), because they do not show the presence of a hydraulic jump. For the sand tests on the contrary, the modelled hydraulic jump, only visible on the results of Ferreira and Spinewine, is too strong compared to the weak jump seen on the experiments. The re-deposition rate in the first reaches has increased, and all of the models fail to account for this phenomenon in the case of the PVC bed.

Figure 12. Experimental vs. numerical results at time $t = 12t_0$. (a) PVC tests; (b) sand tests.

5.2.6 Envelopes of maximum attained water levels and minimum bed levels.

A synthetic way to present simulation results, of particular interest for risk management and damage assessment, is in terms of the envelopes of maximum water levels and minimum bed levels along the channel, attained during the whole duration of the flood wave (Fig. 13). However, these graphs must be interpreted with special care, since no information is provided for the instant at which such extremes are obtained. For example, the deep depression of the experimental bed levels envelope near the dam is only attained for $t < t_0$ during the initial slope failure, and thus may not be interpreted as a developed scour hole. The similar depression in the ISTne profile is obtained for $t > 12t_0$, at much later instants.
Except for the UCL, the results are good for the envelope of water levels in case of the sand bed. The same success holds for the PVC test if we exclude the location around the dam.

![Figure 13. Envelopes of maximum water levels and minimum bed levels, experimental vs. numerical results. (a) PVC tests; (b) sand tests.](image)

5.2.7 Mass of sediments collected at the outlet

In order to quantify globally the amount of geomorphic changes and sediment transport induced by the flood wave along the channel, the modellers were requested to provide the cumulative mass of transported sediments that reach the flume outlet as a function of time. The information is incomplete, since no results were obtained from UDT, and results from IST and CEM, available until \( t = 15 t_0 \), have not reached the ultimate state. The experiments furnished only the final value, indicated by a horizontal line in Figures 14a and b.

The scatter here is much larger than for the previously compared snapshots and envelopes. In fact, a reasonably good correspondence of bed profiles and heights of the transport layer is not sufficient to insure that the correct amount of sediments is eroded and transported throughout the channel to the downstream reaches. A correct estimation of this quantity should however be of prime importance, since those are the sediments that will deposit in the flat areas that are commonly encountered in the lower reaches of natural valleys.

By inspecting Figures 14a and b, it may be observed that the tendencies are similar for the sand and PVC test cases. The results of Leal largely overestimate the transported masses. On the other hand, the results of El Kadi and Paquier, as was already observed from the stable bed profiles in the snapshots of previous sections, seem to under-predict the amount of eroded sediments. Results of Ferreira and Spinewine are in the intermediate range, and approach the ultimate measured value in a more realistic way.
Figure 14. Mass of sediments reaching the flume outlet as a function of time. The experimental line is the total mass gathered during the whole duration of the test.
(a) PVC tests; (b) sand tests.

6 CONCLUSION

Results from a benchmark session devoted to geomorphic dam-break waves over movable beds were presented. The test case consisted in an idealised dam-break flow in a prismatic channel presenting a discontinuity in the bed levels across the dam. Two types of bed material were investigated, differing in both particle size and density.

Experimental results were compared with five numerical models, which differ notably in numerical strategy and constitutive assumptions for flow conceptualisation and closure relations. The results are encouraging, and show limited variability for the case of a sand bed. This variability was greater in the case of a PVC bed. Among the difficulties were the ability to account for the initial reshaping of the bed step due to mass failure and liquefaction, the reproduction of antidune-like bedforms, and the correct evaluation of wavefront arrival time and slowing down. More generally, all of the exposed models are seen to endure difficulties to reproduce in a unified description the very intense transport regimes (first instants and scour at the wavefront) and the much milder transport regimes (tail of the wave and later instants).

All models were numerically stable and able to simulate the flow. If the magnitude of error is appreciable for the evolution of bed levels, a good agreement is generally observed for the water levels, except around the location of the dam. This is where a weak hydraulic jump is gradually formed over the sand bed. No evident hydraulic jump is visible over the PVC bed, its formation may be hindered by the adjusting slope of the gradual bed step profile.

More scatter was observed for the estimation of the total mass of sediments eroded along the channel and transported to the outlet, a valuable quantity that furnishes an estimation of global geomorphic changes in the valley. More detailed comparisons and validation should be performed in this regard, since this is an important factor for risk management and damage assessment.
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- Université Catholique de Louvain (Belgium)
- CEMAGREF (France)
- Università di Trento (Italy)
- University of Zaragoza (Spain)
- CESI (Italy)
- Stakraft Grøner AS (Norway)
- Instituto Superior Technico (Portugal)

REFERENCES


