IMPACT project: Dam-break waves over movable beds

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SUMMARY
This paper presents the results of a numerical study focused on dam-break flood wave over movable beds. The study is performed in the framework of the EC-funded IMPACT project. A one-dimensional sediment transport and riverbed evolution model was used. Data used in this modelling included geometry data, hydraulic data, and sediment data. Water level profiles, bedload and bed variation channel were computed. Results from this study are analysed, discussed and will be compared with laboratory measurements.

1 INTRODUCTION
Evaluating the consequences resulting from a dam-break is an important part of any dam safety study or risk analysis. The propagation of dam-failure flood waves has been the object of intense scientific activity. This problem was initially approached by finding analytical solutions for the shallow-water equations in schematic situations featuring fixed bed and nil flow resistance (Leal et al, 2001). Many sophisticated computer programs have been developed in the last decades and have made it possible to develop numerical solutions for more realistic and complicated situations that include movable beds and sediment transport.

In the framework of the IMPACT project, several benchmarks based on laboratory tests were foreseen under the theme 'Sediments'. The aim of the IMPACT work devoted to sediment transport is to study the propagation of small-scale idealised dam-break waves over a flat movable bed. Dam-break generates intense erosion and solid transport. In particular, this process of sediment transport can alter the flow behavior.

The objective of the experiments is to understand the geomorphic impacts and consequences (erosion and associated solid transport) induced by a sudden migration of floods wave downstream such as those resulting from dam-breaks. It aimed also to serve as a basis for comparison among a range of numerical models relying on various hypotheses and to identify the key aspects that are crucial for modelling this kind of flow (Spinewine and Zech, 2002).

This paper is organised as follows. First, a brief review of the test case is presented. Then, the sediment transport model that had been used is detailed. Finally, the results of the numerical simulations of sediment transport in the test case are discussed and analysed.
2 PRESENTATION OF THE TEST CASE

An idealised dam-break problem is considered (Figure 1). A horizontal flume of rectangular cross-sectional geometry is used. The studied reach has the following dimensions: length = 2.5 m, width = 0.10 m and side-wall height = 0.35 m. The reservoir is assumed to be long and has the following dimensions: length = 10 m, width = 0.10 m and side-wall height = 0.35 m. Hence the reservoir itself was represented by cross-sections. Particles composing the bed are uniform in size. Table 1 shows the characteristics of the bed material.

![Figure 1. Initial conditions for idealised dam-break problem, h₀ and h₁ are the initial depths upstream and downstream of the dam before failure](image)

<table>
<thead>
<tr>
<th>Diameter D (mm)</th>
<th>Standard deviation σ</th>
<th>Porosity p</th>
<th>Density ρ₈ (kg/m³)</th>
<th>Manning Strickler coefficient K (m¹/³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td>1</td>
<td>0.36</td>
<td>1540</td>
<td>54</td>
</tr>
</tbody>
</table>

Table 1. Characteristics of the bed material.

The Strickler coefficient was derived from the particle diameter through the classical Meyer-Peter and Müller formula: \( K = \frac{21.1}{D^{0.7}} \).

Tests consisted in the sudden opening of a vertical gate that separated the initial water and sediments levels upstream and downstream of the gate. Rubarbe, a one-dimensional model was used to estimate flood level downstream of the dam and to predict variation of longitudinal bed profile along the flume and changes in the cross-sectional geometry. This computer model has two components: a component to simulate the flow and a component to characterise the changes in river morphology due to erosion or deposition of sediment.

3 DESCRIPTION OF THE MODEL

One-dimensional sediment transport models tend to be easier to parameterise and require fewer assumptions about sediment transport processes than two-and three-dimensional models (Candfield and al, 2002). However, classical one-dimensional models that represent sediments only by a mean diameter \( D_{50} \) clearly do not fully describe the processes that occur in many channels (armouring, presence of multifraction sediment …). Therefore, Cemagref has developed a 1-D model, RubarBE, using a mean diameter \( D_{50} \) and a complementary parameter, the standard deviation \( S \) (that is defined by the square root of the ratio between \( D_{84} \) and \( D_{16} \)). Data requirements for this model are modest, involving only a few parameters. Thus, the model is relatively easy to calibrate and implement.
3.1 Mathematical basic model

RubarBE is a classical model that relies on these governing equations:

De Saint Venant equations for water:

\begin{align}
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} &= q \\
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + g A \frac{\partial z}{\partial x} &= -g Q^2 - \frac{Q^2}{K^2 A R^{1/3}} + kq \frac{Q}{A}
\end{align}

where \( t \) is time (s), \( x \) is streamwise coordinate (m), \( A \) is cross-sectional flow area (m\(^2\)), \( Q \) is water discharge (m\(^3\)/s), \( q \) is lateral water flow per unit of length (m\(^2\)/s), \( R \) is hydraulic radius (m), \( z \) is water surface elevation (m), \( g \) is acceleration due to gravity (m/s\(^2\)), \( K \) is Manning-Strickler coefficient (m\(^{1/3}\)/s) and \( k \) is coefficient.

Conservation of bed-material (the well-known Exner relationship):

\[(l - p) \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_s\]

where \( A_s \) is bed-material area (m\(^2\)), \( Q_s \) is sediment discharge (m\(^3\)/s), \( q_s \) is lateral sediment flow per unit of length (m\(^2\)/s) and \( p \) porosity.

Sediment transport capacity formula:

(Meyer-Peter Müller, 1948) proposed the following relation for bed load transport capacity:

\[ C_s = \frac{8La \sqrt{g}}{(\rho_s - \rho) \sqrt{\rho}} (\rho JR - \theta_c D_{50}(\rho_s - \rho))^{3/2} \]

where \( C_s \) is sediment transport capacity (m\(^3\)/s), \( D_{50} \) is median diameter of sediment (m), \( J \) is friction slope, \( La \) is active width (m), \( \theta_c \) is dimensionless critical shear stress, \( \rho_s \) is density of sediment (kg/m\(^3\)), \( \rho \) is density of water (kg/m\(^3\)).

Sediment Transport capacity can be calculated from other formulæ such as those from (Bagnold, 1966) or (Engelund and Hansen, 1967).

Bedload transport:

This equation expresses that there is a spatial lag (\( D_{\text{char}} \)) between sediment transport capacity and sediment discharge.

\[ \frac{\partial Q_s}{\partial x} = C_s - Q_s \]

De Saint Venant equations (equations (1) and (2)) are solved by a second-order Godunov-type explicit scheme (Paquier, 1995). If the parameters in the sediment transport function for a cross-section can be assumed to remain constant during a time step, we can suppose that there is a little variation of the cross-sectional geometry (Yang and Simoes, 1998). Thereby, sediment routing (equation 3) can be uncoupled from the water surface profile computations. In practice, this condition can be met by using a small enough time step.
Sediment routing is accomplished by a similar finite difference method. Changes in bottom level are performed at every time step. Solving the equation (3) means estimating at every time step, the input and output of sediments for one cell and spreading the erosion or deposition volume across the cell. Values of A and Q are computed at the middle of a cell between two cross sections. Thus, it is simpler to compute Q, also in this middle and to identify it with respectively input and output of sediments if the sediment cell is shifted by half a space step (Balayn, 2001).

In the case of erosion, the entire movable bed lowers uniformly. In the case of deposition, the volume of deposited sediment is spread across the channel width, starting from the bottom.

3.2 Sediment modelling system

Inside one cell, a sedimentary compartment corresponds to a set of sediments that have a coherent behaviour. We distinguish three compartments:

- A compartment $M_{am}$ of input sediments and a compartment $M_{av}$ of output sediments.

- A compartment A of the active layer: it spans a layer near the bed where sediment particles slide, roll or saltate, and are transported as bed load. Sediment particles are continuously exchanged between flow and the surface layer. Exchange between surface layer (or active layer) and substrate occurs only when the bed scours or fills.

- A compartment B of one or several substrate layers: it reflects historical deposition of sediments on the riverbed or undisturbed subsurface.

Sediment particles of each compartment are characterised by the mass $M$, the mean diameter $D$ and the standard deviation $S$. Thus, in a cell, the sediment transfers are schematised as in Figure 1 (Balayn, 2001).

When sediment particles pass across a cell, they either reach the downstream of the cell, or settle on the active layer. Entrainment of sediment particles from the active layer and its exchanging with flow causes particles travelling from upstream to be mixed with those from the surface layer.

The model assumes that deposit particles are coarser than the eroded material. This process depends on the grain size distribution. Thus, it was decided to include a complementary parameter beside the mean diameter. Standard deviation $S$ defined as the square of the ratio between $d_{84}$ and $d_{16}$ was selected as it appears convenient to describe grain size distribution in a river for which sediments are homogeneous (Shih and Komar, 1990). Thereby, sediment particles of each compartment are characterised by the mass $M$, the mean diameter $D$ and the standard deviation $S$. Thus, in a cell, the sediment transfers are schematised as in Figure 2 (Balayn, 2001).
Figure 2. Representation of the sharing of sediments inside one cell. (Balayn, 2001)

\[ \tau_{fm} \] is the shear stress below which transported sediment particles begin to deposit. \[ \tau_{mm} \] is the shear stress above which sediment particles begin to move.

The mass of the active layer depends on the sediment discharge, the sediment velocities and the dimension of the cell. The model assumes that this mass is defined by \( C_s^* \Delta x / U \), where \( C_s \) is the sediment transport capacity, \( \Delta x \) is the space step and \( U \) the mean water velocity.

The characteristics of sediments resulting from the mixing or the sharing of two compartments are defined in the model by empirical relations (Balayn, 2000).

4 NUMERICAL RESULTS AND DISCUSSION

Due to highly unsteady nature of dam break flood propagation; the frame and the reservoir were described through the use of a dense grid of cross-sections. Three constant grid spacing were used: \( \Delta x = 5 \) cm, \( \Delta x = 10 \) cm and \( \Delta x = 20 \) cm (i.e. 253, 128 and 65 grid points are used to give a domain length of 12.50 m).

A small grid spacing (5 cm for example) allows following better the arrival of the flood wave. A bigger grid allows to model better the transition between super and sub critical flow (presence of hydraulic jump).

The origin of the horizontal axis is located at the gate position, and the vertical reference level is taken at the top of the initial sediment bed.

The initial conditions for the test problem are \( h_0 = 0.10 \) m if \( x < 0 \) and \( h_1 = 0 \) m if \( x \geq 0 \), where \( x \) is the distance along the flume. The total time of the simulation is 2 second. The initial condition for sediment transport is \( Q_s = 0 \) kg/s.
The MPM formula was used in this case; the dimensionless critical shear stress was calculated by the Shields curve.

Because the time of simulation is such that the wave does not interact with the boundary, boundary conditions do not influence the development of the wave. On the upstream side \((x = 10 \text{ m})\), a constant depth of water \(h = 0.10 \text{ m}\) is imposed. On the downstream side \((x = 2.5 \text{ m})\), it is assumed that the flow acted like critical outflow. The sediment boundary condition is \(Q_s = 0 \text{ kg/s}\).

The time step is variable, but it is chosen so that the Courant-Friedrichs-Levy numbers (CFL) of every cell does not exceed a limited value imposed by the model. The simulations were run for two Courant-Friedrichs-Levy numbers (CFL): 0.5 and 0.1.

For all the runs, the figures show that in the near field, rapid and intense erosion accompanies the development of the dam-break wave. In the far field, the solid transport remains intense but the dynamic role of the sediments decreases (Figures 3, 4 and 5). The flow loses its capacity, the transported material is deposited.

Figure 3. Bottom level change: \(\Delta x = 5 \text{ cm}, \ D_{\text{char}} = 1 \text{ m}, t = 2\text{s}\)

Figure 4. Bottom level change: \(\Delta x = 10 \text{ cm}, \ D_{\text{char}} = 1 \text{ m}, t = 2\text{s}\)
Instabilities of calculations were observed during the numerical tests. Results of simulations depend on the grid spacing, the CFL number (Figures 3, 4 and 5) and $D_{\text{char}}$ values (Figure 6). Instabilities are more marked in the case of CFL = 0.1. The difference of accuracy between the numerical results increases in the far field (Figure 7).

Figure 5. Bottom level change: $\Delta x = 20$ cm, $D_{\text{char}} = 1$ m, $t= 2s$

Figure 6. Riverbed evolution with spatial lag $D_{\text{char}}$: $\Delta x = 20$ cm, CFL = 0.5, $t = 2s$

Figure 7. Riverbed evolution: $D_{\text{char}} = 1$ m, CFL = 0.5, $t = 2s$
The numerical simulations show that the migration of the flood wave downstream following the dam break is a sudden event where there are steep fronts (shocks) and transitions between super and sub critical flow (Figures 8 and 9). There is also no discontinuity shown at the critical flow point, which is at original location of the gate (x= 0 m).

![Figure 8: Froude number: \( \Delta x = 5 \text{ cm, } D_{\text{char}} = 1 \text{ m, CFL = 0.5} \)](image1)

![Figure 9: Froude number: \( \Delta x = 20 \text{ cm, } D_{\text{char}} = 1 \text{ m, CFL = 0.5} \)](image2)

The numerical simulations take in account only the bedload transport. This assumption may be restrictive in the modelling of flood or dam break events, where suspended load is important. Added numerical tests were carried out, in which the dimensionless critical shear stress \( \theta_c \) was supposed nil. Similar behaviour is obtained with non-nil critical shear stress (Figure 10). The agreement between results for various space steps is a little improved with regard to those found in the previous simulations. Erosion and deposition are practically located in the same places (Figure 11).
Figure 10. Riverbed evolution with nil or non nil critical shear stress: Space step = 20 cm, $D_{\text{char}} = 1 \text{ m}, \text{CFL} = 0.5, t = 2s$

Figure 11. Riverbed evolution: $D_{\text{char}} = 1 \text{ m}, \text{CFL} = 0.5, \theta_c = 0, t = 2s$

The more relevant results seem to be for the numerical simulation with a space step of 20 cm, $D_{\text{char}} = 1 \text{ m}$ and CFL = 0.5. The variation with space step (5 cm instead of 20 cm) and spatial lag $D_{\text{char}}$ (1 cm instead of 1 m) provides evaluation of uncertainty of the results.

5 COMPARISON WITH EXPERIMENTAL DATA

Experimental data are compared to the numerical results. CEM-1 refers to the simulation with a space step of 20 cm, $D_{\text{char}} = 1 \text{ m}$ and CFL = 0.5; CEM-2 to the simulation with a space step of 20 cm, $D_{\text{char}} = 1 \text{ cm}$ and CFL = 0.5 and CEM-3 to the simulation with a space step of 5 cm, $D_{\text{char}} = 1 \text{ m}$ and CFL = 0.5.

Figure 12 shows the comparison concerning the front characteristics. Further the data shows consistently smaller wave front celerity than for the numerical dam break analysis: the time of
front wave arrival is smaller with the RubarBE model. This behaviour can be explained too by influence of the hypothesis and approaches used by RubarBE (average velocity, hydrostatic pressure...).

![Figure 12. Front characteristics](image)

However, it must be noticed that the closest approximations to the experimental data seem to be for the numerical CEM-3, i.e. space step of 5 cm, $D_{char} = 1$ m and $CFL = 0.5$. At the same cross-section the difference between the arrival times is around 0.15 s. The shape of the experimental wave front is quite similar to the numerical profiles. Small grid spacing allows following better the arrival of the flood wave.

The Channel friction could affect significantly the propagation of the front wave. A Manning friction coefficient of $0.0185$ m$^{-1/3}$, derived from the diameter of the riverbed material, was taken by the model. The influence of the wall friction was neglected. A value of $0.0185$ m$^{-1/3}$ might not be associated to the experimental phenomena. A sensitivity analysis was carried out. Two different Manning coefficients are tested with the simulation Cema-3. The computation results (Figure 13) show that the celerity is quite dependent upon the friction coefficient introduced in the numerical model. The agreement between experimental data and RubarBE simulation (Cema-3) is quite improved with a roughness of 0.02 with regard to those found in the previous simulations with a roughness of $0.0185$ m$^{-1/3}$.

![Figure 13. CEM3: front characteristics with different values of roughness](image)
Figures 14 (a), (b), (c), (d), (e) and (f) show the flow levels evolution at six cross-sections: \(x = -0.50\) m, \(x = -0.25\) m, \(x = 0.00\) m, \(x = 0.25\) m, \(x = -0.50\) m and \(x = 0.75\) m. Computed water levels are in quite agreement with experimental data, with slight differences observed upstream from the gate. In the reservoir, the closest approximations to the experimental data seem to be for the numerical simulation CEM-3. The simulation CEM-1 provides quite good approximations to the experimental data in the upstream from the gate. At gate location, the water levels evolution is underestimated but quite well estimated by the simulations CEM-1 and CEM-2 that provide nearly similar results. There, flow is critical at the gate; a higher space step allows to model better the transition between super and sub critical flow (presence of hydraulic jump). Decoupled model is inherently less stable than the coupled model in the case of Froude numbers vary close to unity, and they may need a larger space step or special treatment of time step (Cui et al., 1996).
water levels at x = 0.00m

(c)

water levels at x = 0.25m

(d)

water levels at x = 0.50m

(e)
Figure 14. Water levels evolution: (a) $x = 0.50$ m, (b) $x = -0.25$ m, (c) $x = 0.00$ m, (d) $x = 0.25$ m, (e) $x = 0.50$ m, (f) $x = 0.75$ m

It must be necessary to emphasise that the experimental data depends upon the distribution of velocities in the flume. The code computes an average velocity while measured data depends upon local velocities. In addition, the dam-break wave involves the formation of shocks and rarefaction fans that are not taken in account in the RubarBE computer model. The difference between results is probably due to the highly 3-D nature of the dam-break wave. The experimental configurations examined violate, almost locally, some of the St. Venant hypothesis (small bottom slopes and curvatures, hydrostatic pressure and uniform velocity distribution in the cross section).

Figures 15 (a), (b), (c), (d), (e) and (f) show the bottom position at six cross-sections: $x = -0.50$ m, $x = -0.25$ m, $x = 0.00$ m, $x = 0.25$ m, $x = -0.50$ m and $x = 0.75$ m. Significant discrepancies between numerical and experimental results are observed through the reservoir and the channel. Numerical scours on the bottom observed in the reservoir are slightly deeper ($x = -0.25$ m); its shape does not resemble the numerical one. At gate location, the bottom changes are underestimated by the model and bottom changes are in $x = 0.5$ m. The differences between the simulations CEM-1, CEM-2 and CEM-3 are negligible.
(b) bed levels at x = -0.25m

(c) bed levels at x = 0.00m

(d) bed levels at x = 0.25m
Figure 15. Bottom levels changes: (a) x = -0.50 m, (b) x = -0.25 m, (c) x = 0.00 m, (d) x = 0.25 m, (e) x = 0.50 m, (f) x = 0.75 m

CONCLUSIONS
The technique and the method used in RubarBE provide oscillating results. The oscillations tend to increase with small spatial steps and small steps of time. This increase is consistent with the representation of the front wave which goes far from the physical process with small space step and similarly a too small time step decreases the numerical stability of the method.

The use of the Exner equation and of a maximum sediment transport capacity does not integrate the presence of a mixture of water and sediments in high concentration. This assumption is valid for long-term simulations of bed aggradation or degradation, but may be restrictive in the modelling of flood or dam break events. The use of another method for calculating the sediment transport seems to be necessary to take into account high concentration transport.
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REFERENCES


