SUMMARY
The present paper aims to present the issues and the scope of the IMPACT research project in the field of dam-break induced geomorphic flows (Work Package 4), to give an overview of the experimental work carried out in the frame of the research program, to summarise the new developments in modelling, to outline the validation process and to give some practical conclusions for the future of dam-break wave modelling.

1 INTRODUCTION
In a number of ancient and recent catastrophes, floods from dam or dike failures have induced severe soil movements in various forms: debris flows, mud flows, floating debris and sediment-laden currents (Costa and Schuster, 1988). Other natural hazards also induce such phenomena: glacial-lake outburst floods and landslides resulting in an impulse wave in the dam reservoir or in the formation of natural dams subject to major failure risk.

Fig. 1 presents some estimates of the volume of sediment material moved by such flows, gathered from published cases studies (Capart, 2000; Capart, Young and Zech, 2001). In some cases, the volume of entrained material can reach the same order of magnitude (up to millions of cubic meters) as the initial volume of water released from the failed dam.

Even when they involve comparatively small volumes of material, geomorphic interactions can lead to severe consequences because of localised changes or adverse secondary effects. In India, for instance, the Chandora river dam-break flow of 1991 stripped a 2 m thick layer of soil from the reaches immediately downstream of the dam (Kale et al., 1994). In the 1980
Pollalie Creek event, Oregon, the material entrained by a debris flow deposited in a downstream reach, forming a temporary dam that ultimately failed and caused severe flooding (Gallino and Pierson, 1985). Another cascade of events was that of the 1996 Biescas flood, Spain, where a series of flood-control dams failed (Benito et al., 1998).

The problem with dam-break induced geomorphic flows is that they combine the difficulties of two types of flow: (1) alluvial flows, where the bed geometry evolves under the flow action, but with a sediment load small enough to play no dynamic role and (2) rapid transients involving such rapid changes and intense rates of transport that the granular component plays an active role in the flow dynamics, and that inertia exchanges between the bed and the flow become important (Capart, 2000).

2 RESEARCH ISSUES AND SCOPE

Four institutions were involved in the “Sediment movement” IMPACT work package: the Université catholique de Louvain (UCL, Belgium: Y. Zech, S. Soares Frazão, B. Spinewine, N. le Grelle), the Università degli Studi di Trento (UdT, Italy: A. Armanini, L. Fraccarollo, M. Larcher, M. Giuliani, G. Rosatti), the Instituto Superior Técnico (IST, Lisbon, Portugal: A.B. Almeida, A.H. Cardoso, R.M. Ferreira, J.G.A.B. Leal) and the Centre National du Machinisme Agricole, du Génie Rural, des Eaux et des Forêts (Cemagref, Lyon, France: A. Paquier, K. El Kadi). The main goal of this work package was, building upon the previously gathered information, to gain a more complete understanding of geomorphic flows and their consequences on the dam-break wave (Zech and Spinewine, 2002).

Dam-break induced geomorphic flows generate intense erosion and solid transport, resulting in dramatic and rapid evolution of the valley geometry. In counterpart, this change in geometry strongly affects the wave behaviour and thus the arrival time and the maximum water level, which are the main characteristics to evaluate for risk assessment and alert organisation.
Depending on the distance to the broken dam and on the time elapsed since the dam break, two types of behaviour may be described and have to be understood and modelled.

2.1. Near-field behaviour

In the near field, rapid and intense erosion accompanies the development of the dam-break wave. The flow exhibits strong free surface features: wave breaking occurs at the centre (near the location of the dam), and a nearly vertical wall of water and debris overruns the sediment bed at the wave forefront (Capart, 2000), resulting in an intense transient debris flow (Fig. 2). However, at the front of the dam-break wave, the debris flow is surprisingly not so different from a uniform one. A first section is thus devoted to the characterisation of debris flow in uniform conditions.

![Figure 2. Near-field geomorphic flow (UCL)](image)

Behind the debris-flow front, the behaviour seems completely different: inertial effects and bulking of the sediments may play a significant role. The second section relates experiments, modelling and validation of this near-field behaviour.

2.2. Far-field behaviour

In the far field, the solid transport remains intense but the dynamic role of the sediments decreases. On the other hand dramatic geomorphic changes occur in the valley due to sediment de-bulking, bank erosion and debris deposition (Fig. 3). The third section is devoted to experiments, modelling and validation of the far-field behaviour.

![Figure 3. Dam-break consequences in the far field](image)

Lake Ha!Ha! 1996 dam break (Brooks and Lawrence, 1999)
3 DEBRIS FLOW IN UNIFORM CONDITIONS

Iverson (1997) reports some interesting information about various debris-flow events in USA, Peru, Colombia and New Zealand. The main characteristics of this type of event are the involved volume, the run-out distance (sometimes hundredths of kilometres), the descent height (till 6000 m in the quoted examples) and the origin of the debris flow (mainly landslides and volcanic events).

A debris-flow also occurs at the front of a dam-break wave, if the latter happens on mobile bed and/or banks. In this case a high amount of sediments is generally mobilised, inducing a vertical velocity component and the formation of a kind of plug at the front of the wave.

3.1. Experimental works (University of Trento - UdT)

To investigate the vertical structure of free-surface liquid-granular flows, it is of particular interest to be able to materialise steady uniform flow conditions. A re-circulating flume was designed and constructed for this purpose at the University of Trento, Italy. It consists in a tilting glass-walled channel linked with a conveyer belt, forming a closed loop for the circulation of both water and sediment (Fig. 4).

From these experiments, it is possible to gather information about the acting forces involved in such debris flow (Armanini et al., 2000). Also the main characteristics of the debris flow may be measured, such as the distribution of the velocities and particle concentration in the normal upward direction. Both can be measured by Voronoï imaging methods, using the grains themselves as tracers (Capart et al., 1999). The concentration along the wall is deduced from 3-dimensional Voronoï cells built by use of stereoscopic imaging (Spinewine et al., 2003).
The velocity of each particle may be decomposed into the sum of a mean velocity and of a random component, taking into account the relative motion of the particle compared to the mean value. It is thus possible to define a granular temperature $T_s$ as the mean square value of the instantaneous deviation from the mean velocity (Ogawa, 1978). In analogy with thermodynamic temperature, the granular temperature plays similar roles in generating pressures and governing the internal transport rates of mass, momentum and energy.

### 3.2. Modelling developments (UdT)

Some physical similarities between rapid granular flows and gases has led to a great deal of work on adapting kinetic theories to granular materials, using the idea of deriving a set of continuum equations (typically mass, momentum and energy conservation) entirely from microscopic models of individual particle interactions. All models are based on the assumption that particles interact by instantaneous collisions, implying that only binary or two-particle collisions need to be considered. Particles are usually modelled in a simple way, ignoring surface friction. Furthermore, molecular chaos is generally assumed, implying that the random velocities of the particles are distributed independently.

Jenkins & Hanes (1998) applied kinetic theories to a sheet flow in which the particles are supported by their collisional interactions rather than by the velocity fluctuations of the turbulent fluid. The purpose of their analysis is the prediction of mean fluid velocity, particle concentration and granular temperature profiles obtained as solutions of the balance equations of fluid and particle momentum and particle fluctuation energy. The flow of the mixture of particles and fluid is assumed to be, on average, steady and fully developed. The grains are taken to be identical spherical particles of diameter $D$ composed of a material of mass density $\rho_s$. The fluid is assumed to have a mass density $\rho_w$. The constitutive relation for the particle pressure $\sigma_s$ is taken to be the quasi-elastic approximation for a dense molecular gas proposed by Chapman & Cowling (1970):

$$
\sigma_s = C_s \rho_s \left(1 + 4C_s \frac{2-C_s}{2(1-C_s)^3}\right)T_s
$$

where $C_s$ is the grain concentration, $T_s$ the granular temperature, the fraction $(2-C_s)/2(1-C_s)^3$ is the radial distribution function at contact, describing the variation with concentration of the rate of collisions among the particles. In the same way, the constitutive relation for the particle shear stress $\tau_s$ is taken:

$$
\tau_s = \frac{8}{5\pi^{1/2}} D \rho_s C_s^2 \frac{2-C_s}{2(1-C_s)^3} T_s^{1/2} \left[1 + \frac{\pi}{12} \left(1 + \frac{5}{8} \frac{2(1-C_s)^3}{C_s(2-C_s)}\right)^2\right] \frac{du}{dz}
$$

From experiments, it is possible to derive $\sigma_s$ and $\tau_s$ by assuming that the buoyant weight of the grains is entirely supported by collisional granular contacts.

In Fig. 5 comparison is made between so-derived experimental results and the theoretical relations in Eq. 1 and Eq. 2 (blue lines). A better fitting is obtained by accounting an added-mass effect by replacing $\rho_s$ in Eq. 1 and Eq. 2 by:

$$
\rho_s' = \rho_s \left[1 + \frac{1+2C_s}{2(1-C_s)} \frac{\rho_w}{\rho_s}\right]
$$

resulting in the red line in Fig. 5. More details can be found in Armanini et al. (2003)
4 Near-Field Geomorphic Flow

4.1. Experimental approaches (Catholic University of Louvain - UCL)
Debris flow is only a part – in time and space – of a dam-break induced geomorphic flow. Other aspects due to the severe transient character of the flow are investigated by means of idealised dam-break experiments. Typically, those consist in a horizontal bed composed of cohesionless sediments saturated with water extending on both sides of an idealised "dam", at the same level (Fig. 6a) or at distinct levels (Fig. 6b).

![Diagram of near-field benchmark experiment](image)

Figure 6. Scheme of near-field benchmark experiment

Upstream lies a motionless layer of pure water, having infinite extent and constant depth $h_0$ above the sediment bed. An intense flow of water and eroded sediments is then released by the instantaneous dam collapse (Fig. 7).

In the experiments carried out at the Catholic University of Louvain within the frame of the IMPACT program, two materials have been used for representing the sediments: PVC pellets and sand, with rather uniform grain-size distribution. Two arrangements were tested: the flat-bed case with the same sediment level on both sides of the dam (Fig. 6a), and the stepped case where the upstream bed level is higher than the downstream bed level (Fig. 6b). Some of those experiments were proposed as benchmarks to the IMPACT partners for comparison with their numerical models.
The measurement techniques were various: gauges, interface imaging by simple cameras, particle tracking using tracers or the sediments themselves.

4.2. Numerical modelling development (UCL)

4.2.1. Vertical two-dimensional level-set model

Considering that the vertical component of the velocity at the first stages of the dam-break flow is not negligible, the initial idea was to develop a 2D-V model able to represent what happens in a vertical plane at the very initial instants after the collapse of the dam.

The best appropriated model appeared to be a level-set method. This approach relies on the assumption that the flow is subdivided in non-miscible layers of approximately homogeneous properties, separated by sharp interfaces. The propagating interfaces between the various media (air, water, sediment) correspond to the zero level sets of higher dimensional functions $\Phi$, defined as the signed distances to the interface (Sethian, 1999).

A Navier-Stokes equation can be developed in terms of vorticity, and thus also in terms of stream functions $\Psi$, from which the velocity field can be derived. The level-set functions can thus be advected according to this velocity field (Fig. 8).

Looking at results of Fig. 8, the method seems indeed promising, as it qualitatively reproduces the features observed in the experiments (Fig. 7a). However, crude assumptions have been done on the behaviour of the sediment layer, namely that the bed layer is considered as a fluid over the whole domain. Further developments would require to account...
for a solid / fluid transition with inter-phase mass exchanges accounting for erosion / deposition.
 Besides it rapidly appeared that use of such sophisticated models in real-life cases is nearly impossible. The constraint of the representation of the whole phenomenon along a unique vertical plane is not realistic since most of the real valleys are rather narrow in the vicinity of the dam with significant variations of water depth along the width. The 2D-V approach may thus rather be considered as an interesting step to a fully 3D approach, the later remaining a far objective for the modellers.

Therefore, most of the efforts to develop numerical models of dam-break wave were oriented to shallow-water approaches, for which realistic developments are possible, as well in terms of available data as in terms of numerical schemes.

4.2.2. Two-layer shallow-water 1D model

The first developments were presented by Capart (2000). The flow is represented by three layers: (1) the upper layer consisting of clear water of depth $h_w$, (2) the moving sediment layer $h_s$ and (3) the fixed-bed layer having the bed level $z_b$ as upper limit.

In the original model (Capart, 2000), the concentration of sediment was assumed to be constant ($C_s = C_b$) and the upper part of the mixture water / sediment ($h_s$) was assumed to be in movement with the same uniform velocity as the clear-water layer ($u_s = u_w$). According to those assumptions the shear stress was supposed as continuous along a vertical line. An analytical solutions was derived for this model (Fraccarollo and Capart, 2002), but this, whilst clever, can of course not be used in real-case geometry.

One of the main improvements (Spinewine, 2003; Spinewine and Zech, 2002a) brought to the model is to give new degrees of freedom to the concentrations ($C_s \neq C_b$) and the velocities ($u_s \neq u_w$) between the three layers (Fig. 9).

![Figure 9. Assumptions for mathematical description of near-field flow](image)

In the frame of shallow-water approach, it is now possible to express the continuity of both the sediments and the mixture and also the momentum conservation with the additional assumption that the pressure is hydrostatically distributed in the moving layers:

$$\frac{\partial h_w}{\partial t} + \frac{\partial}{\partial x} (h_w u_w) = -e_b \frac{C_b - C_s}{C_s}$$

(4a)
\[
\frac{\partial h_s}{\partial t} + \frac{\partial}{\partial x} (h_s u_s) = e_b \frac{C_b}{C_s} \tag{4b}
\]
\[
\frac{\partial e_b}{\partial t} = -e_b \tag{4c}
\]
\[
\frac{\partial (h_s u_s)}{\partial t} + \frac{\partial}{\partial x} \left( h_s u_s^2 + \frac{gh_s^2}{2} \right) + gh_s \frac{\partial}{\partial x} (z_b + h_s) = -\frac{\tau_w}{\rho_s} - e_b \frac{C_b - C_s}{C_s} \left( u_s \text{ if } e_b > 0 \right)
\]
\[= \frac{\tau_w - \tau_b}{\rho_s} + \frac{\rho_w}{\rho_s} \frac{C_b - C_s}{C_s} e_b \left( u_s \text{ if } e_b > 0 \right) \tag{5a}
\]
\[
\frac{\partial (h_s u_s)}{\partial t} + \frac{\partial}{\partial x} \left( h_s u_s^2 + \frac{gh_s^2}{2} \right) + gh_s \frac{\partial}{\partial x} (z_b + \rho_s \frac{\partial h_s}{\partial x}) = \frac{\tau_w}{\rho_s} + \frac{\tau_b}{\rho_s} + \frac{\rho_w}{\rho_s} C_b - C_s e_b \left( u_s \text{ if } e_b < 0 \right) \tag{5b}
\]
where \(e_b\) is the erosion rate (negative is the case of deposition), resulting from the inequality between the shear stresses \(\tau_s\) and \(\tau_b\) on both faces of the bed interface:
\[
e_b = \frac{1}{\rho_b} \left| u_s \right| (\tau_s - \tau_b) \tag{6}
\]
The shear stresses \(\tau_w\) and \(\tau_s\) are evaluated from the turbulent friction, while \(\tau_b\) is related to the grain pressure by the soil cohesion and friction.

The set of Eqs. 4-5 is solved by a second-order Godunov finite-volume scheme, where the fluxes are computed using the LHLL Riemann solver (Fraccarollo et al., 2003).

**4.3. Validation of the 1D models (UCL, IST, UdT, Cemagref)**

Validation of the 1D models for the near-field behaviour was achieved through benchmarking. The first test (Spinewine and Zech, 2002b) consisted in the situation with the same level of sediment at both sides of the dam (flat-bed case: Fig. 6a). The second test (Spinewine and Zech, 2003) features also sediments on both sides of the dam but the level of the upstream sediments is higher than the downstream one (stepped-bed case: Fig. 6b).

Both tests were run concurrently by the IMPACT team members. Each team used a different approach, with the aim of assessing the characteristics, strengths and weaknesses of each of those approaches, relying on different assumptions. Those approaches thus do not represent the current state-of-the-art of each team. The used models will be briefly introduced before presenting the result comparisons.

**4.3.1. Averaged-velocity / distinct concentrations description**

This approach was used by the Technical University of Lisbon (IST) for the benchmarks. It relies also on a three-layer idealisation. Localised erosion / deposition processes are represented by vertical fluxes. However, those fluxes do not induce any change in the thickness of the transport layer. The model features total (water and sediment) mass and momentum conservation laws, averaged over the flow depth
\[
\frac{\partial}{\partial t} (z_b + h) + \frac{\partial}{\partial x} (hu) = 0 \tag{7}
\]
\[
\frac{\partial}{\partial t} (\rho u h) + \frac{\partial}{\partial x} \left( \rho u^2 h + \rho u^2 h \right) + \frac{1}{2} g \frac{\partial}{\partial x} (\rho h^2 h + 2 \rho h h h + \rho h^2 h) = -g (\rho u h + \rho h) \frac{\partial z_b}{\partial x} - \tau_b \tag{8}
\]
and mass conservation equations of the transport layer and of the bed, respectively:
\[
\frac{\partial}{\partial t}(C_s h_s) + \frac{\partial}{\partial x}(C_s h_s u_s) = \Phi_s
\]  

(9)

\[
(1 - \varepsilon_0) \frac{\partial \eta_b}{\partial t} = -\Phi_s
\]  

(10)

where \( h = h_s + h_w \), \( u = (u_s h_s + u_w h_w)/h \) represent the average velocity of the moving layers (whose thickness are \( h_s \) and \( h_w \), respectively), \( \tau_b \) is the bed shear stress, \( \rho_m \) is the mean density of the layers such that \( \rho_m h u = \rho_s h_s u_s + \rho_w h_w u_w \), \( \rho_s = \rho_w(1+(s-1)C_s) \) is the transport layer density, \( \varepsilon_0 \) is the porosity, and \( \Phi_s \) is the flux between the bed and the transport layer.

In the IST model the dependent variables are \( h, u, z_b \) and \( C_s \). Closure equations are required for some parameters: \( h_s \) is derived from a sheet-flow description and depends on the bottom shear stress \( \tau_b \) and on the grain diameter \( d \); \( \tau_b \) is proportional to \( u^2 \); \( u_s \) is linked to \( u \) through a power-law distribution; and \( \Phi_s \), depending on the imbalance between capacity and actual transport, the later being related to \( \tau_b \) and \( u_s \). Further details can be found in Ferreira et al. (2003) and Leal et al (2003).

4.3.2. One-velocity / one concentration description

This approach, used by the University of Trento (Fraccarollo, Capart and Zech, 2003) considers constant concentration of sediment \( (C_s = C_b) \) and the upper part of this mixture \( (h_s) \) is assumed to be in movement with the same uniform velocity as the clear-water layer \( (u_s = u_w = u) \) in such a way that Eq. 4a-c may be combined in the following way:

\[
\frac{\partial}{\partial t} (z_b + h_s + h_w) + \frac{\partial}{\partial x} [(h_s + h_w) u] = 0
\]  

(11a)

\[
\frac{\partial}{\partial t} (z_b + h_s) + \frac{\partial}{\partial x} (h_s u) = 0
\]  

(11b)

and Eq. 5a-b are merged in the following form, where \( h = h_s + h_w \) and \( r = (s-1)C_s \) with \( s = \rho_s/\rho_w \), the latter being the density supplement due to the presence of the sediment load.

\[
\frac{\partial}{\partial t} [(h + rh_s) u] + \frac{\partial}{\partial x} [(h + rh_s) u^2 + \frac{1}{2} g h^2 + \frac{1}{2} r g h_s^2] + g (h + rh_s) \frac{\partial \eta_b}{\partial x} = -\frac{\tau_b}{\rho_w}
\]  

(12)

4.3.3. One-layer description

The Cemagref model RubarBE used in the benchmarks is a simple 1-D mathematical model for routing water and non-equilibrium sediment transport through fixed or movable bed channels (El Kadi and Paquier, 2004d). The water routing component relies on the Saint Venant equations extended to the whole cross-section:

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0
\]  

(13)

\[
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + g A \frac{\partial z_w}{\partial x} = -g A S_f
\]  

(14)

where \( A \) and \( Q \) are the section area and the discharge, \( z_w \) is the water level and \( S_f \) the friction slope.
The sediment routing component has the following major features: (1) computation of sediment transport capacity using a suitable bed load formula, (2) computation of the actual sediment transport rate $Q_s$ by introducing a space lag equation (equation 15a), and (3) numerical solution of the continuity equation for sediment (equation 15b):

$$\frac{\partial Q_s}{\partial x} = \frac{C_s - Q_s}{D_{char}} \quad (15a)$$

$$(1 - \varepsilon_0) \frac{\partial A_s}{\partial t} + \frac{\partial Q_s}{\partial x} = 0 \quad (15b)$$

where $A_s$ is the bed material area, $C_s$ the transport capacity and $Q_s$ the solid discharge, $\varepsilon_0$ is the porosity and $D_{char}$ characterises the spatial lag.

Only the water layer is taken into consideration and the closure of the system is made by the solid discharge. The Saint Venant equations are solved by a second-order Godunov-type explicit scheme (Paquier, 1995). Sediment routing and changes in channel profile are calculated by a finite difference method, uncoupled from the water surface profile computations. Sediments are represented by the mean diameter $d_{50}$ and the standard deviation $\sigma$.

The allocation of scour along the cross-section perimeter is assumed to be a function of $(\tau_j - \tau_{c,j})^m$, $m$ being a fixed exponent. The boundary shear distribution $\tau_j$ across a section is computed either from the uniform equation using water volumetric density $\rho_w$, gravity acceleration $g$, hydraulic radius $R$ and energy slope $S_f$ ($\tau_j = \rho_w g R S_f$), or from the Merged Perpendicular Method described by Khodashenas and Paquier (1999). The boundary critical shear distribution $\tau_{c,j}$ in a cross-section is calculated either from the Ikeda (1982) relation, which introduces the effect of the local side slope, or from the Shields curve established for a horizontal bottom. The model RubarBE computes the deformation of cross-sections with the assumption that scour and deposition are directly related to shear stress. In the case of sedimentation, deposited sediment in a section can be distributed in three different ways: (1) the cross-section is adjusted in horizontal layers, (2) the whole wetted cross-section is moved uniformly up, or (3) the vertical shift $\Delta z_j$ of a cross-section node is assumed to be a function of $(\tau_j)^m$.

**4.3.4. Comparisons with flat-bed benchmark experiments**

The benchmark (Spinewine and Zech, 2002b) is as sketched at Fig. 6a with the following characteristic dimensions: a water layer of depth $h_0 = 0.10$ m in the reservoir, and a fully saturated bed of thickness $h_s = 0.05$ m. The bed material consisted of cylindrical PVC pellets with an equivalent diameter of 3.5 mm and a density of 1.54, deposited with a bulk concentration of about 60%.

Fig. 10 presents a comparison between experimental observation at UCL and the model presented in section 4.2. The first picture (Fig. 10a: time $t = 0.2$ s) clearly evidences the limitation of the model for the earlier stage of the dam-break: some features linked to the vertical movements are missed, like the splash effect on water and sediment. The erosion depth is slightly underestimated, partly due to a kind of piping effect under the rising gate, which is not included in the model. All those phenomena induce energy dissipation that is not accounted for in the model, what explains that the modelled front has some advance compared to the actual one.
Looking at the second picture (Fig. 10b: time $t = 0.6$ s), it appears that some characters of the movement are really well modelled, such as the jump at the water surface, the scouring at the dam location, the moving layer thickness. The modelled front is yet ahead but this advance is the same as at the former time, which implies that the front celerity is correctly estimated.

![Image](image1.png)  ![Image](image2.png)

Figure 10. Comparison between experiments and numerical results (UCL) at times (a) $t = 0.2$ s and (b) $t = 0.6$ s

The comparison of the various models with the experimental data of the flat-bed benchmark is made in Fig. 11, which represents the evolution of the various levels as a function of the time for a given station. Regarding the front celerity the results by Trento (UdT) are close to the observations. In contrast, their moving sediment layer is apparently underestimated, due to the fact that the concentration of this layer is assumed to be the same as the bed material, which is not the case of the models used by Louvain (UCL) and Lisbon (IST): in the reality, the concentration of this moving layer has to decrease to allow the movement of the particles. Multiplying the UdT mobile-layer thickness by an adequate concentration ratio $C_b/C_s$ would result in a better visual adequacy. The erosion due to the front mobilisation only appears in the models used by Louvain and Cemagref (CEM). Even though Cemagref’s simple model cannot provide any results for the moving sediment layer, it still yields a valuable estimate for the water surface after the shock (further details about numerical results can be found in El Kadi and Paquier, 2003). The asymmetric treatment of erosion and deposition in Eq. 6 could explain the success for the UCL model in this regard. Additional graphs with comparisons may be found in Spinewine and Zech (2003b).

![Graph](graph1.png)

Figure 11. Comparison between experimental and numerical results from the benchmark on dam-break wave over an initially flat PVC bed at station $x = 5 h_0$. For each set of results,
the lower line corresponds to the fixed bed level, the middle line to the moving sediment layer and the upper line to the water surface

4.3.5. Comparisons with stepped-bed benchmark experiments

Tests were performed in a new flume designed at the UCL Civil Engineering Department (Spinewine and Zech, 2003a). 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. 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.

Two series of tests were performed with two different bed material: uniform coarse sand on the one hand, and light PVC pellets on the other hand. The initial conditions for the two series of tests were identical. The initial bed profile (Fig. 6b) features horizontal reaches upstream and downstream, with an initial discontinuity at the gate section, so that the upstream level is initially \( h_{s,0} = 10 \text{ cm} \) 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 \( h_0 = 35 \text{ cm} \). The corresponding characteristic time is \( t_0 = (h_0/g)^{1/2} = 0.189 \text{ s} \). 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. So, the downstream water table is initially at the same level as the sediments, which are thus saturated in the initial conditions. Above this downstream weir the outlet is free.

A comparison (Spinewine and Zech, 2004) of the various models with the experimental data of the stepped-bed benchmark over a sand bed is made in Fig. 12, which represents the various levels at a given time in dimensionless form. Two results from Lisbon are represented, one with an equilibrium law (e) for the solid transport and the other one with a non-equilibrium law (ne). Further details about numerical results using Cemagref model for different values of parameters can be found in El Kadi and Paquier (2004a).

Figure 12. Comparison between experimental and numerical results from the benchmark on dam-break wave over an initially stepped sand bed at \( t = 5 \, t_0 \). For each set of results, the lower line corresponds to the fixed bed level, the middle line to the moving sediment layer and the upper line to the water surface.
The global impression is that all the models are rather satisfactory in representing the general flow features. The Lisbon non-equilibrium (ne) model behaves best in the downstream region, but erodes too much at the toe of the step, leading to a water depression with the appearance of a jump, not really observed in the reality. The Louvain model accounts for the development of a slope at the dam location but this model is equipped with a special operator able to liquefy a part of the sediment step, according to the laws of slope stability in soil mechanics.

4.4. Near-field modelling: conclusions

A common weakness of all the compared models is that they advance the front too fast. This is originated by the fact that the first stages of the sediment mobilisation are missed since no vertical velocity components are taken into consideration. The various models display a significant dispersion of water levels and a wide distribution of transport-layer heights, which makes evident the need for further researches in that field. Another clear conclusion in the difficulty to reproduce the erosive behaviour of saturated debris front, above all if this erosion is followed by a partial re-deposition.

However, the progresses of such modelling, compared to the results available some years ago, is spectacular. A part of this development is issued from new measurement techniques based on digital imaging, that have made possible the observation in real time of the velocity field, as well in the liquid phase as in the solid-transport layer.

Finally, to conclude the description of near-field behaviour and modelling, it is interesting to check that taking into account the sediment movement significantly affects the major characteristics of the dam-break wave. To evidence such an effect, a comparison was made between a wave on a fixed frictionless bed and the same wave on mobile sediments at time $t = 1.5$ s after the dam break (Fig. 13).

It clearly appears that the mobilisation of the sediments diverts a part of the available potential energy, in such a way that the wave front is notably delayed, which is an advantage in term of alert and emergency planning for the downstream population. But the water depth is appreciably amplified behind this front, at least in the near field, increasing the endangered area and the associated risk for people living in the vicinity of the collapsed dam.
5 CHANNEL ALTERATION IN THE FAR FIELD

The transition between near-field and far-field behaviour is not absolutely clear. The debris-flow front resulting from the early stage of dam-break forms a kind of obstacle, which is progressively subject to piping and overtopping. That means that a sediment de-bulking occurs and the solid transport evolves to a bed- and suspended-load transport with a particularly high concentration. The flow is highly transient and invades a part of the valley that was probably never inundated in the past. All the bank geotechnical equilibrium characteristics are ruined, in such a way that a dramatic channel metamorphosis may be expected (Fig. 3). This corresponds to the so-called far-field behaviour.

5.1. Experimental approaches (Catholic University of Louvain UCL)

Laboratory scale models of rivers give interesting information about geomorphic evolution but they are generally not used for sudden transients. Bank failure experiments are commonly carried out to study some fluvial mechanisms such as river meandering or braiding. Also channel-width adjustments during floods may be reproduced in laboratory (see e.g. Chang, 1992), but for cases where this evolution is rather progressive.

The experiments carried out within the IMPACT project consist in a dam-break flow in an initially prismatic valley made of erodible material, as sketched in Fig. 14. Such experiments reproduce qualitatively well the features of fast transient geomorphic flows. The upstream part of the channel is fixed, i.e. neither the bed nor the banks can be eroded. The downstream part is made of uniform non cohesive material. A detailed description of the experiment can be found in le Grelle et al. (2003 and 2004)

The experiment is launched by suddenly raising the gate. This releases a dam-break wave which rapidly propagates down the channel and triggers a series of bank failures. The rapid erosive flow attacks the toe of the banks with the consequence that they become steeper near the bed and thus fail. Bank erosion then occurs in fact as a series of intermittent block failures (Fig. 15) that feed the flow with an important quantity of sediments.
The channel enlargement due to bank failures is the most important in the immediate vicinity of the dam. The water depth there is greater and the flow shows a two-dimensional expansion from the reservoir into the channel. After a relatively short time (about 10 s in the scale experiment), most of the geomorphic action has occurred. Only light bedload transport can be observed and the banks are no longer affected.

Flow measurement is achieved using a laser sheet technique (le Grelle et al. 2004) that allows continuous measurement of the geomorphic evolution of a given cross-section during the flow (Fig. 16).

![Figure 16. Laser-sheet technique for cross-section evolution measurement](image)

The overall principle of the method is to use a laser-light sheet to enlighten a given cross section and to film it during the whole duration of the experiment by means of a remote camera through the transparent side-wall of the channel. The trace of the imprinted laser line onto the digital images is then localised and projected back in 3D space using distinct projective transforms for the submerged and emerged portions. The results were found to be surprisingly reproducible, even though the bank erosion mechanism through intermittent block failures is quite stochastic.

In the above-described experiment, the downstream part of the valley is made of uniform non-cohesive material. To check the influence of non-uniformity of the bank material, a test was carried out with a section of coarser material (gravel), as it can be frequently found in the nature (Fig. 17).
It was observed that the coarse material was entrained, above all along the bed and along the toe of the bank, similarly to the sand, but the gravel was deposited at some distance downstream. Nevertheless, the general behaviour of the morphological evolution of the whole channel was not too much affected by the presence of this stronger section. Of course this unique test was only indicative and, for instance, the response of graded material versus uniform one has to be further studied.

5.2. Numerical modelling development (UCL)

The key issue in modelling geomorphic processes is to properly include bank failure mechanisms in the system. Indeed, such important geomorphic changes occur randomly and abruptly, and cannot be considered just as a continuous process such as bedload transport. Three different models were developed by UCL within the frame of the IMPACT project.

5.2.1. Two-dimensional model

First, a 2D extension of the model presented for the near field (Eq. 4-5) was developed, including a bank erosion mechanism. A detailed description of the method, summarised here, can be found in Spinewine et al. (2002) and Capart and Young (2002). The key idea is that by allowing separate water and fluid-like slurry layers to flow independently, the governing equations are fully equipped to deal with flow slides of bank material slumping into the water stream. Once failure is initiated, the post-failure flow can be captured just like any other pattern of water and sediment motion.

A liquefaction criterion is thus needed to determine when and where portions of the banks are to be transformed from a solid-like to a fluid-like medium. Therefore, the following fundamental mechanism is assumed: activation of a block failure event occurs whenever and wherever the local slope exceeds a critical angle $\alpha_c$. An extended failure surface is then defined as a cone centred on the failure location and sloping outwards at residual angle $\alpha_r < \alpha_c$. Finally, sediment material above this cone is assumed to instantaneously liquefy upon failure.

In order to account for the observed contrast between submerged and emerged regions, four distinct angles of repose are defined as indicated in Fig. 18: angles $\alpha_{c,subm}$ and $\phi_{r,subm}$ apply to the submerged domain, and $\alpha_{c,em}$ and $\alpha_{r,em}$ to the emerged domain.
5.2.2. One-dimensional model with global bank failure

The second model selected for coupling with the above bank erosion mechanism is a one-dimensional scheme. It comprises a hydrodynamic finite-volume algorithm and a separate sediment transport routine. The finite-volume scheme, developed with the aim of coping with complex topographies (Soares-Frazão and Zech, 2002), solves the hydrodynamic shallow-water equations:

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_i = 0
\]

\[
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + gA \frac{\partial z}{\partial x} = -g AS_f
\]  

(17)

The changes in cross-sectional geometry due to longitudinal sediment transport (bedload) over one computational time step are derived from the Exner continuity equation of the sediment phase:

\[
(1 - \varepsilon_0) \frac{\partial A_s}{\partial t} + \frac{\partial Q_s}{\partial x} = q_{s,i} = 0
\]  

(18)

In addition to sediment fluxes at the upstream and downstream faces of a cell, lateral sediment inflow resulting from bank failures could be considered but it was preferred to take this eroded volume \( V_s \) in consideration not as a lateral inflow \( q_{s,i} \), but rather as a volume to be redistributed in the cross section at the end of the computation time step. Indeed, deposition mechanism from the bank (deposition mainly at the toe of the bank) is different from bed-load transport mechanism (deposition along the whole width).

A failure is triggered by the submergence of a bank by a rise \( \Delta h \) in water level that destabilises a prismatic portion of material as sketched in Fig. 19. In the experiments, the initial bank slope \( \alpha \) is less than the stability angle \( \alpha_{s,em} \) above the water surface but greater than the stability angle \( \alpha_{s,subm} \) below the water surface. Thus the bank becomes unstable as soon as the water arises, and it fails according to the failure angles \( \alpha_{f,subm} \) and \( \alpha_{f,em} \) corresponding to the submerged and emerged situations, respectively (in practice the failure angles \( \alpha_f \) are slightly less than stability angles \( \alpha_s \)). The so-eroded volume \( V_s \) has now to be redistributed in the cross section.
The eroded material deposits into the channel as sketched in Fig. 20. The submerged portion deposits with an angle $\alpha_{r,\text{subm}}$ corresponding to the angle of repose under water while the emerged portion stabilises at an angle $\alpha_{r,\text{em}}$ (angle of repose above the water level after the deposition process). All those angles of repose are specific to the material used in the experiments and were measured by means of static and dynamic experiments.

The longitudinal sediment transport is computed from the Meyer-Peter–Müller formula. In both cases of erosion or deposition, the erosion / deposition rate is assumed to be uniformly distributed along the bed width.

Finally, the numerical 1D model consists in solving in a de-coupled way the three different key steps of the process: (i) the hydrodynamic routing of the water, (ii) the longitudinal sediment transport and the resulting erosion and deposition, and (iii) the bank failures and the resulting morphological changes in the cross-section shape.

### 5.2.3. One-dimensional model with local bank failure

The above scheme appeared to be well adapted to idealised situations, where the cross sections may be defined for instance as rectangles or trapezes, with a limited number of summits in their polygonal description.

For natural rivers the cross sections become too complicated to be described in such a simplified way, and another approach was preferred, inspired by Schmautz and Aufleger (2002) in another context.

The cross-section profile is discretised in little segments, the stability of each of them being checked starting from the side of the valley (Fig. 21). If the bank section AB is locally steeper than the critical one (stability angle $\alpha_s$), the bank portion rotates until reaching the position A'B', corresponding to the convenient angle of repose $\alpha_r$ (emerged or submerged). In return, this new position may aggravate the stability of the adjacent sections (for instance BC has now moved to B'C position). The whole profile has to be browsed several times till all the sections are stable.
Figure 21. Principle of the local bank-failure model

For the longitudinal sediment transport, the following rules are adopted. In case of erosion, according to the Meyer-Peter–Müller formula, the erosion rate is assumed to be distributed along the submerged width proportionally to the local value of $(\tau_b - \tau_{b,c})^{3/2}$, where $\tau_b$ and $\tau_{b,c}$ are the actual and the critical bed shear stresses, respectively. In case of deposition, the sediment is supposed to deposit uniformly (not horizontally) along the bed, this later being defined as the cross-section elements with a slope less than the submerged angle of repose.

5.3. Validation of the models (UCL, IST, UdT, Cemagref)

Validation of the models is achieved through benchmarking at two different levels. A first benchmark concerns the idealised dam-break flow experiment presented in a previous section. The second level concerns the simulation of a real event, namely the Lake Ha!Ha! flood that occurred in the Saguenay region of Quebec in 1996 (Brooks and Lawrence, 1999). This second benchmark has just started and the blind modelling by the partners is still in progress.

The same tests were run concurrently by the IMPACT teams to compare the characteristics of the various models. The main characteristics of each model are hereafter summarised, at least concerning the bank failure mechanism.

5.3.1. The model used by the Technical University of Lisbon IST

If a bed erosion occurs, for example from time $t_0$ to time $t_1$ in Fig 22, the bank shape simply adapts to this deepening and if the new slope remains less than the stability angle, no failure happens. If this slope becomes too steep (time $t_2$), the section widens until the bank slope becomes stable again.

The volume eroded from the bank is now distributed uniformly (and thus horizontally) along the bottom width, contributing to the source term of Exner equation, inducing in return the evolution $\partial A_s/\partial t$ and the corresponding change in bottom level.
5.3.2. The model used by the University of Trento UdT
The bank erosion mechanism is very similar to the model with global bank failure described in section 5.2. However the deposition is now distributed uniformly on the horizontal bottom width.

5.3.3. The model used by the Cemagref
In the Cemagref approach, no explicit model bank erosion is used. Only the longitudinal transport is taken into consideration. The erosion is assumed to be proportional to the local value of \((\tau_b - \tau_{b,c})^{3/2}\), where \(\tau_b\) and \(\tau_{b,c}\) are the actual and the critical bed shear stresses, respectively. In case of deposition, the option selected corresponds to the sediment supposed to deposit horizontally in every cross-section (further details can be found in El Kadi and Paquier (2004b).

5.3.4. Comparisons with bank erosion experiments
The experiments, described in the section 5.1 consist in a dam-break flow in an initially prismatic valley made of erodible material, as sketched in Fig. 14. Figure 23 shows the characteristic dimensions of the experimental set-up (le Grelle et al., 2003).

Figure 24 compares the experimental results to the UCL 1D model with the global description of bank failure, at a distance of 0.50 m (Fig. 24a and b) and 1.50 m (Fig. 24c and d), respectively, for times \(t = 3\) s (Fig. 24a and c) and \(t = 5\) s (Fig. 24b and d), respectively.
Figure 24. Bank erosion benchmark: comparison between UCL 1D global model and experiments. Distance downstream from the dam: 0.50 m (a-b), 1.50 m (c-d):
- initial situation,
- experiments, and
- numerical modelling

A comparison between the results of various models and the experimental observations is shown on Fig. 25. The represented cross station is 2.25 m downstream from the gate and the time is 10 s after the gate rise.

Figure 25. Bank erosion benchmark. Comparisons at $x = 2.25$ m and $t = 10$ s

Some interesting observations arise from this comparison. The Cemagref model, where only the bed moves, does not account for the typical widening of the section, which evidences the need of a bank stability criterion. In the Trento model, all the bank material is moved to the bottom without reshaping the bank, which clearly leads to overestimate the bed level. The Lisbon model curiously fails in representing the bank failure whilst such a mechanism would be initiated by the deepening of the bed. The model used by Louvain was the global failure
description. It obviously takes advantage of the definition of an angle of repose for the deposition of the material issued from the bank collapse.

It must be noted than in the experiment, the initial bank angle was greater than the critical one, which emphasises the phenomena. With a flatter slope, the morphological effects would be less important.

5.3.5. **Comparison with field data: the Lake Ha!Ha! dam break**

In July 1996, the collapse of a dike along the Lake Ha!Ha! reservoir resulted in a severe dam-break wave with spectacular morphological changes in the 30 km long river, till the confluence with the Saguenay River in the Ha!Ha! Bay (Fig. 26).

![Figure 26. Lake HaHa! (Brooks, 2003)](image)

Practically all the typical features of severe morphological evolution could be observed consequently to the disaster: large deposition areas (Fig. 27a), large-scale widening (Fig. 27b), sometimes blocked by the presence of bed-rock sills and banks (Fig. 27c), changes in bed profile, changes in path, etc.

(a) Deposition  (b) Widening  (c) Bed-rock effect
A huge efforts to interpret the available data was carried out by the Geological Survey of Canada, owner of the data, the University of Quebec, the National Taiwan University and the Catholic University of Louvain (Capart et al., 2003) to produce a usable data set, probably one of the best available for model validation in real-life situations.

Figure 28 shows the evolution of the bed profile in the vicinity of a large-scale avulsion. The initial river profile was controlled by a non-erodible rock area explaining the chute in the green profile of Fig. 28. Due to an overtopping of a depressed point of the bank line, the river diverted its course. The bed-rock area was bypassed, inducing a severe regressive erosion.

Although the 1D Cemagref model does not rely on sophisticated description of the moving sediment (Exner equation with solid transport from common formulae), the resulting bed profile evolves in the right direction, nevertheless with some numerical instabilities. Some differences can be linked with the location of rocks and the fact that the computation was stopped after two days, thus before the completion of the erosion process.

Figure 29 shows the water profile along the same reach of the river as in figure 28. It evidences the role of bed mobility and morphological changes. The water elevation is obtained from two different calculations with the Cemagref model for mobile bed and compared with a fixed-bed approach (El Kadi and Paquier, 2004c). At some locations, the mobility of the bed induces a drop or a rise of the water level up to 5 metres (see, for instance, km 21 and 22 on figure 29).
The diffusion-advection model used by the National Taiwan University is two-dimensional and anisotropic, but it relies on rather simple assumptions, which nevertheless appear as very efficient for the particular application of the Lake Ha!Ha! case. The landscape evolution is described by diffusion along slopes according to local gradients. No explicit hydrodynamic computations is required: the water table is horizontal in depressions and the depth is zero everywhere else.

Such an approach provides impressive results at least in some reaches, for instance in the upper reach of the river (Fig. 30). The 2D model seems able to capture the erosion zone just downstream the failed dam, and also the main deposition zones. The 1D model also predicts some deposition features, and, to a limited extent, some punctual results in the scouring zone. The widening of some sections is also qualitatively represented.

It is interesting to observe that in a complex case as the Lake Ha!Ha! only the simplest approaches succeeded to give some results, while sophisticated models did not cope with the huge amount of required data.
Figure 30. Erosion / deposition in the upstream reach of Ha!Ha! River.
(a) Picture of the river after the dam break
(b) Erosion / deposition surveyed after the catastrophe
(c) Numerical modelling result from the Cemagref model
(d) Numerical modelling result from the NTU model
5.4. Far-field modelling: conclusions
Some preliminary conclusions may be proposed from the above comparisons between experimental or field data and the various models used to reproduce them.

First of all, it seems that the modelling of widening is promising, at least if this enlargement does not exceed a local scale. However the modelling of erosion / deposition is still to be improved. It is clear that the deposition process has to be carefully modelled to represent what happens at the toe of the failed banks.

Other features are really difficult to represent in the present stage of models, above all those whose the initiation is linked to random state or to stochastic phenomena. A weak point in a bank is not always predictable, at least without a huge amount of data. It may originate a local collapse, which is possible to model at least in the average, but it also can offer the favourable environment for the initiation of river diversion or a meander short cut. The prediction of such phenomena needs new modelling approaches, where the physics, the numerical methods and the probabilistic models have to be integrated.

6 CONCLUSIONS
The problem with dam-break induced geomorphic flows is that they combine several difficulties. They involve such rapid changes and intense rates of transport that the granular component plays an active role in the flow dynamics, and that inertia exchanges between the bed and the flow become important. Dam-break induced geomorphic flows generate intense erosion and solid transport, resulting in dramatic and rapid evolution of the valley geometry. In return, this change in geometry strongly affects the wave behaviour and thus the arrival time and the maximum water level.

In the near field, rapid and intense erosion accompanies the development of the dam-break wave, leading to an intense transient debris flow. The numerical models existing at this stage provide encouraging results. The jump at the water surface, the scouring at the dam location and the moving layer thickness are fairly well represented. But the earlier stage of the dam-break flow is not so well modelled, since the vertical movements depart from the shallow-water assumptions. Finally, all those phenomena dissipate some energy, what is not represented in the models, what explains that the computed front is generally too fast at the beginning.

For the far field behaviour, the models at this stage can produce valuable results to compare with experimental data from idealised situations. But it is suspected that we are far from a completely integrated model able to accurately simulate a complex real case. Confrontation to real-life data as the Lake Ha!Ha! case results give an idea of the amount of work remaining to reach the industry-oriented modelling.

An interesting conclusion of the research carried out in the frame of the IMPACT programme is that the initial distinction between near- and far-field behaviour and modelling is less evident than anticipated.

For the near-field modelling, it was supposed that a 2D-V model was required to account for the vertical component of the velocity at the first stage of the dam break. For the far-field behaviour, characterised by a valley widening, it was expected that realistic results could be
obtained only by 2D-H models. Those models are indeed promising, but it was found that in both cases, a simpler 1D model, based on the shallow-water assumptions, was good enough to approach the reality, so far as some improvement could be realised to represent some particular features, like the initial delay and the bank failure mechanism. If the 1D approach may match both near- and far-field behaviour, the good news is that an integrated model could be possible in a closer future than expected.

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