SEDIMENT MOVEMENT MODEL DEVELOPMENT

Yves Zech, Professor\textsuperscript{1} and Sandra Soares-Frazão, PhD\textsuperscript{1,2}

Benoit Spinewine\textsuperscript{1}, Nicolas le Grelle\textsuperscript{1}, Aronne Armanini\textsuperscript{3}, Luigi Fraccarollo\textsuperscript{3}, Michele Larcher\textsuperscript{3}, Rocco Fabrizi\textsuperscript{3}, Matteo Giuliani\textsuperscript{3}, André Paquier\textsuperscript{4}, Kamal El Kadi\textsuperscript{5}, Rui M. Ferreira\textsuperscript{5}, João G.A.B. Leal\textsuperscript{5}, António H. Cardoso\textsuperscript{5} and António B. Almeida\textsuperscript{5}

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, to give an overview of the experimental work carried out in the frame of the research program, to summarize the new developments in modeling, to outline the validation process and to give some practical conclusions for the future of dam-break wave modeling.

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.

Figure 1. Entrained material from dam or dike failures (Capart, 2000)

\textsuperscript{1} Catholic University of Louvain (Université catholique de Louvain - UCL), Place du Levant 1, B-1348 Louvain-la-Neuve, Belgium, zech@gce.ucl.ac.be

\textsuperscript{2} National Fund for Scientific Research, Rue d’Egmont 5, B-1000 Bruxelles, Belgium, soares@gce.ucl.ac.be

\textsuperscript{3} University of Trento (Università degli Studi di Trento - UdT), Italy

\textsuperscript{4} CEMAGREF, France

\textsuperscript{5} Instituto Superior Técnico (IST), Lisbon, Portugal
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 localized 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).

**Research Issues and Scope**

The main goal of the “Sediment movement” IMPACT work package is, 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 behavior and thus the arrival time and the maximum water level, which are the main characteristics to evaluate for risk assessment and alert organization.

Depending on the distance to the broken dam and on the time elapsed since the dam break, two types of behavior may be described and have to be understood and modeled.

**Near-field behavior**

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 center (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 as a uniform one. A first section is thus devoted to the characterization of the debris flow in uniform conditions.

Figure 2. Near-field geomorphic flow (UCL)
Behind the debris-flow front, the behavior seems completely different: inertial effects and bulking of the sediments may play a significant role. Surprisingly, such a difficult feature appears to be suitably modeled by a two-layer model based on the shallow-water assumptions and methods. The second section relates experiments, modeling and validation of this near-field behavior.

**Far-field behavior**

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, modeling and validation of the far-field behavior.

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

Lake Ha!Ha! 1996 dam break (Brooks and Lawrence, 1999)

**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 kilometers), 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 mobilized, inducing a vertical velocity component able to form a kind of plug at the front of the wave.

**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 materialize steady uniform flow conditions. A re-circulating flume was designed and constructed for this purpose at the Università degli Studi di 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 some 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 Voronoi imaging methods, using the grains themselves as tracers (Capart et al., 1999). The concentration along the wall is deduced from 3-dimensional Voronoi 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.

Figure 4. Trento re-circulating flume – Photograph and plane view

**Modeling 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, utilizing the idea of deriving a set of continuum equations (typically mass, momentum and energy conservation) entirely from microscopic models of individual particle interactions. All of the 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 modeled 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 tur-
bulent 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 is taken to be the quasi-elastic approximation for a dense molecular gas proposed by Chapman & Cowling (1970):

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

where $C_s$ is the grain concentration, 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 is taken:

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

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:

$$\rho_s' = \rho_s \left(1 + \frac{1 + 2 C_s \rho_w}{2(1 - C_s) \rho_s}\right)$$  \hspace{1cm} (3)

resulting in the red line in Fig. 5. More details can be found in Armanini et al. (2003)

Figure 5: Particle pressure and shear stress: points represent experimental results, blue line theoretical kinetic relation, red line accounting for added mass effect
Near-Field Geomorphic Flow

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 considered by means of idealized dam-break experiments. Typically, a horizontal bed composed of cohesionless sediments saturated with water extends on both sides of an idealized "dam" (Fig. 6).

![Figure 6. Scheme of a flat-bed dam-break experiment](image)

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).

![Figure 7. Idealized dam-break experiment (UCL) after 0.25 s (a), 0.50 s (b) and 1.00 s (c)](image)

In the experiments carried out at the Université catholique de 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 (see Fig. 6), and the stepped case where the upstream bed level is higher than the downstream bed level. 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.
Numerical modeling development (UCL)

The near-field modeling generally relies on numerical methods, since analytical solutions (Fraccarollo and Capart, 2002), whilst clever, cannot take into consideration real-case geometry.

Fig. 8 illustrates a simplified but fruitful approach of the problem (Spinewine, 2003; Spinewine and Zech, 2002a). Three zones are defined: the upper layer \((h_w)\) is clear water while the lower layers are composed of a mixture of water and sediments. In the original model (Capart, 2000), the concentration of sediment was assumed to be constant \((C_s = C_b)\) and the upper part of this mixture \((h_s)\) 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.

One of the main improvements brought to the model is to give new degrees of freedom to concentrations \((C_s \neq C_b)\) and velocities \((u_s \neq u_w)\) between the three layers.

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, which implies that no vertical movement is taken into consideration:

**Figure 8. Assumptions for mathematical description of near-field flow**

\[
\frac{\partial h_w}{\partial t} + \frac{\partial}{\partial x} (h_w u_w) = -e_b \frac{C_b - C_s}{C_s} \quad \text{(4a)}
\]

\[
\frac{\partial h_s}{\partial t} + \frac{\partial}{\partial x} (h_s u_s) = e_b \frac{C_b}{C_s} \quad \text{(4b)}
\]

\[
\frac{\partial z_b}{\partial t} = -e_b \quad \text{(4c)}
\]

\[
\frac{\partial (h_s u_s)}{\partial t} + \frac{\partial}{\partial x} \left( h_s (u_s^2 + \frac{g h_s^2}{2}) \right) + g h_s \frac{\partial}{\partial x} (z_b + h_s) = -\frac{\tau_s}{\rho_s} - e_b \frac{C_b - C_s}{C_s} \left\{ \begin{array}{ll} u_s \text{ if } e_b > 0 \\ u_s \text{ if } e_b < 0 \end{array} \right. \quad \text{(5a)}
\]

\[
\frac{\partial (h_i u_i)}{\partial t} + \frac{\partial}{\partial x} \left( h_i (u_i^2 + \frac{g h_i^2}{2}) \right) + g h_i \frac{\partial}{\partial x} \left( \frac{\partial z_b}{\partial x} + \frac{\partial h_s}{\partial x} \right) = \frac{\tau_s}{\rho_s} + e_b \frac{C_b - C_s}{C_s} \left\{ \begin{array}{ll} u_i \text{ if } e_b > 0 \\ u_i \text{ if } e_b < 0 \end{array} \right. \quad \text{(5b)}
\]

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:
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 HLLC Riemann solver (Toro, 1997).

**Validation of the models (UCL, IST, UdT, Cemagref)**

Validation of the models for the near-field behavior was achieved through benchmarking (Spinewine and Zech, 2002b). The test consisted in the situation sketched in Fig. 6 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. 9 presents a comparison between experimental observation at UCL and the model presented above. The first picture (Fig. 9a: 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 modeled front has some advance compared to the actual one.

Looking at the second picture (Fig. 9b: time $t = 0.6$ s), it appears that some characters of the movement are really well modeled, such as the jump at the water surface, the scouring at the dam location, the moving layer thickness. The modeled front is yet ahead but this advance is the same as at the former time, which implies that the front celerity is correctly estimated.

The same test was run concurrently by the Impact teams to compare the characteristics of the various models.

The model of the Technical University of Lisbon (IST) relies also on a three-layer idealization. Localized erosion / deposition processes are represented by vertical fluxes but not their impact on the thickness of the transport layer. The model features total (water and sediment) mass and momentum conservation laws, averaged over the flow depth

$$
\frac{\partial}{\partial t} (z_b + h) + \frac{\partial}{\partial x} (hu) = 0
$$

---

$$
e_b = \frac{1}{p_b |u_s|} (\tau_s - \tau_b)
$$

---

Fig. 9. Comparison between experiments and numerical results (UCL) at times (a) $t = 0.2$ s and (b) $t = 0.6$ s
\[ \frac{\partial}{\partial t} (\rho_m u h) + \frac{\partial}{\partial x} \left( \rho_m u^2 h + \rho_m u h \frac{\partial h}{\partial x} \right) + \frac{1}{2} g \frac{\partial}{\partial x} \left( \rho_m h^2 + 2\rho_m u h + \rho_m h^2 \right) = -g (\rho_m h + \rho_s h) \frac{\partial z_b}{\partial x} - \tau_b \] (8)

and mass conservation equations of the transport layer and of the bed, respectively:

\[ \frac{\partial}{\partial t} (C_z h_z) + \frac{\partial}{\partial x} (C_z h_z u_z) = \Phi_s \] (9)

\[ (1 - \varepsilon_0) \frac{\partial z_b}{\partial t} = -\Phi_s \] (10)

where \( h = h_s + h_w, \quad u = (u_s h_s + u_w h_w)/h \) 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 \( \rho_m h u = \rho_w h_w u_w + \rho_s h_s u_s + \rho_s (1+(s-1)C_s) \) 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: \( h_s \), derived from the equation of conservation of granular kinetic energy; \( u_s \), averaged from a power-law distribution; \( \tau_b \), quadratic dependence on the shear rate; and \( \Phi_s \), depending on the imbalance between capacity and actual transport. Further details can be found in Ferreira et al. (2003) and Leal et al (2003).

The model 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)

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

\[ \frac{\partial}{\partial t} (h + rh_s) u + \frac{\partial}{\partial x} [(h + rh_s) u^2 + \frac{1}{2} g h^2 + \frac{1}{2} r g h^2] + g (h + rh_s) \frac{\partial z_b}{\partial x} = -\frac{\tau_b}{\rho_w} \] (12)

The Cemagref model RubarBE (El Kadi and Paquier, 2003) relies on the classical Saint-Venant equations extended to the whole cross section:

\[ \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \] (13)

\[ \frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( Q^2 \right) + gA \frac{\partial z_w}{\partial x} = -g S_f \] (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 conservation of bed material is expressed by the Exner equation, very similar to Eq. 10:

\[ (1 - \varepsilon_0) \frac{\partial A_s}{\partial t} + \frac{\partial Q_s}{\partial x} = 0 \] (15)

where \( A_s \) is the bed material area and \( Q_s \) the solid discharge. Only the water layer is taken into consideration and the closure of the system is made by the solid discharge.
The comparison of the various models with the experimental data is made in Fig. 10. Regarding the front celerity the results by Trento (UdT) take advantage of the calibration process, which involves these celerity as a calibration parameter. In contrast, their moving sediment layer is 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 Louvain (UCL) and Lisbon (IST) models: in the reality, the concentration of this moving layer has to decrease to allow the movement of the particles. The erosion due to the front mobilization only appears in the Louvain and Cemagref (CEM) models. Even though Cemargref’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. The asymmetric treatment of erosion and deposition in Eq. 6 could explain the success for the UCL model in this regard.

![Figure 10](image)

**Figure 10.** Comparison between experimental and numerical results from the benchmark on dam-break wave over an initially flat erodible bed at $x = 5 \, h_0$. 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.

**Channel alteration in the far field**

The transition between near-field and far-field behavior 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. This corresponds to the so-called far-field behavior.

A spectacular channel widening generally occurs due to bank scouring and collapse (see Fig. 3). This eroded material over-supplies the bed-load transport resulting in bed deposition and eventual generation of natural dams in the downstream reaches, which may rapidly collapse.
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. 11. 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. (2004).

![Figure 11. Experimental set-up](image1.png)

![Figure 12. Bank erosion resulting from intermittent block failure](image2.png)

The experience 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. 12) 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. 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 localized and projected back in 3D space using distinct projective transforms for the immerged and emerged portions. The results were found to be surprisingly reproducible, even though the bank erosion mechanism through intermittent block failures is quite stochastic.
Numerical modeling development (UCL)

The key issue in modeling 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. Two different models were developed by UCL within the frame of the IMPACT project.

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, summarized 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 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 $\phi_c$. An extended failure surface is then defined as a cone centered on the failure location and sloping outwards at residual angle $\phi_r < \phi_c$. 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. 13: angles $\phi_{cs}$ and $\phi_{re}$ apply to the submerged domain, and $\phi_{ce}$ and $\phi_{rc}$ to the emerged domain.

![Stability diagram for the 2D geostatic failure operator](image)

Figure 13. Stability diagram for the 2D geostatic failure operator

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 (paper in preparation). The finite-volume scheme, developed with the aim of coping with complex topographies (Soares-Frazão and Zech, 2002), solves the hydrodynamic shallow-water equations, under the form of Eq. 13-14.

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 (Eq. 15).

In addition to sediment fluxes at the upstream and downstream faces of a cell, lateral sediment inflow resulting from bank failures must be considered. 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. 14 that results in a lateral solid discharge $q_s$. The final shape of the cross section shows a submerged slope of angle $\alpha_{cs}$ (angle of repose under the water level...
after erosion) while the emerged part gets the angle $\alpha_{e,e}$ corresponding to the angle of repose of humid sand above the water level after the erosion process.

![Figure 14. Bank failure triggered by the submergence of the bank](image1)

![Figure 15. Deposition of the material eroded from the banks](image2)

The eroded material deposits into the channel as sketched in Fig. 15. The submerged portion deposits with an angle $\alpha_{d,s}$ corresponding to the angle of repose under water while the emerged portion stabilizes at an angle $\alpha_{d,s}$ (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.

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 (ii) the bank failures and the resulting morphological changes in the cross-section shape.

**Validation of the models (UCL, UT, Cemagref, IST)**

Validation of the models will be achieved through benchmarking at two different levels. A first benchmark concerns the idealized dam-break flow experiment presented in a previous section. The blind test was achieved by the involved partners and the comparison process is underway. 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 modeling by the partners is in progress.

Some preliminary comparisons for the first benchmark are presented in Fig. 16. The experimental measurements are compared to the results obtained by the 1D model developed by UCL. The overall agreement is good: the numerical model appears to follow quite accurately the progressive enlargement of the cross section.

**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 behavior 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 modeled, 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 behavior, the models at this stage can produce valuable results to compare with experimental data from idealized situations. But it is suspected that we are far from a completely integrated model able to accurately simulate a complex real case. A tentative answer to this could probably be given from the results of the second benchmark regarding the Lake HaHa! test case, available after the last IMPACT meeting to be held in Zaragoza, Spain, in November 2004.

Acknowledgements

The authors wish to acknowledge the financial support offered by the European Commission for the IMPACT project under the fifth framework program (1998-2002), Environment and Sustainable Development thematic program, for which Karen Fabbri was the EC Project Officer. In addition, the authors acknowledge the financial support offered by the Fonds pour la Recherche dans l’Industrie et l’Agriculture, Belgium and by the Fonds National de la Recherche Scientifique, Belgium.
References


