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MPM hydro-mechanical modelling of flows impacting rigid walls

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Abstract

The study on impact mechanisms of flow-like landslides against structures is still an open issue in the scientific literature. Many researchers have employed so far either experiments or numerical methods, but the evaluation of the impact forces on mitigation obstacles remains difficult especially if the solid-fluid interaction within the flow is considered. This study shows how advanced numerical tools, such as Material Point Method, may be used in simulating those complex processes. The simulations are carried out for two well documented laboratory tests: a dry granular flow impacting a rigid wall under different geometries and testing conditions in a small-scaled flume and a saturated flow with complex propagation pattern in a centrifuge apparatus. The numerical modelling is validated against the observations and then used to explore the response of different flows impacting rigid structures in other conditions than in the experiments. The soil-fluid interaction influences the type of impact mechanism, the kinematics of the flow, and the space-time trend of the impact pressure against the structure.

Keywords: mechanism, numerical, Material Point Method, impact

List of abbreviations

A	(m^2)	impacted surface of the barrier
$\mathbf{a}_L$	(m/s^2)	liquid acceleration
$\mathbf{a}_S$	(m/s^2)	solid acceleration
$\mathbf{b}$	(kPa)	body force vector
c	(m/s)	wave speed
c'	(kPa)	effective cohesion
C_{NB}	$(-)$	Courant number

C_U	(-)	coefficient of uniformity
d	(m)	smallest element length
$\mathbf{D}$	(kPa)	tangent stiffness matrix
D_{max}	(mm)	maximum grain size
D_{min}	(mm)	minimum grain size
dp_L	(kPa)	excess pore pressure
$d\sigma'$	(kPa)	increment of effective stress
E	(kPa)	Young modulus of soil
e_{max}	(-)	maximum void ratio
e_{min}	(-)	minimum void ratio
F	(kPa/m)	impact force
F_E	(-)	Ergun coefficient
f_d	(kPa)	drag force
Fr	(-)	Froude number
$\mathbf{g}$	(m/s ²)	gravity vector
H	(m)	barrier height
h	(m)	flow propagation depth
k	(m ²)	intrinsic permeability
k_0	(m ²)	reference permeability
k_{Darcy}	(m/s)	hydraulic conductivity
$\mathbf{K}_L$	(kPa)	elastic bulk modulus of the liquid
$L_{(+)}$	(m)	length of the influence zone upstream of the wall
$L_{(-)}$	(m)	length of the influence zone downstream of the wall
N	(-)	centrifuge scale factor
n	(-)	porosity
n_0	(-)	initial porosity
n_S	(-)	solid volume fraction
p_L	(kPa)	liquid pressure
t	(s)	time variable
V	(m ³)	volume of the mixture
V_L	(m ³)	liquid phase volume
V_S	(m ³)	solid phase volume
$\mathbf{v}$	(m/s)	velocity vector
$\mathbf{v}_L$	(m/s)	liquid velocity vector
$\mathbf{v}_S$	(m/s)	solid velocity vector
x	(m)	horizontal Cartesian coordinate
y	(m)	vertical Cartesian coordinate
Δt_{cr}	(s)	critical time step
$\dot{\boldsymbol{\varepsilon}}$	(s ⁻¹)	strain rate matrix
ε_{vol}	(-)	volumetric strain
θ	(°)	flume slope angle
μ_L	(Pa · s)	liquid dynamic viscosity
ν	(-)	Poisson's ratio
ρ	(kg/m ³)	material density
ρ_L	(kg/m ³)	liquid density
ρ_m	(kg/m ³)	density of the mixture
ρ_S	(kg/m ³)	solid density
$\boldsymbol{\sigma}$	(kPa)	total stress tensor of the mixture
$\dot{\boldsymbol{\sigma}}$	(kPa/s)	Jaumann stress rate matrix
σ_n	(kPa)	normal stress
ϕ'	(°)	internal friction angle
ϕ'_b	(°)	contact friction angle

ψ (°) dilatancy angle

1 Introduction

Flow-like landslides are dangerous for people and structures around the world (Pastor et al. 2009), due to the long travel distances up to tens of kilometers and the high velocities even in the order of meters per second. According to the updated Varnes' classification (Hungr et al. 2014), such landslides show a complex behavior with a continuum passage from sliding to flowing. This category includes dry granular flows and saturated flows with excess pore water pressure such as flowslides or debris flows.

Determining the impact force is needed to assess the vulnerability of structures, to design the best protection options (Faella and Nigro 2003; Bugnion et al. 2012; Scheidl et al. 2013; Zeng et al. 2015; Feng et al. 2019), to stop or to deviate the flows. Many efforts have been made so far to individuate proper design criteria. The main problem however is related to the difficulties in understanding the impact mechanisms. The measurements in the field are very limited (Hu et al. 2011). Most of the literature focuses on laboratory experiments eventually associated to empirical equations proposed, but each related to specific boundary conditions. Thus, it is difficult to reconcile theoretical concepts with field and experimental observations (Koo et al. 2017). Nevertheless, the experimental results can be used to test the potential of numerical models. Recent simulations of flows interacting with barriers are based on the application of Discrete Element Method (DEM) as reported by Leonardi et al. (2014), Calvetti et al. (2016) and Shen et al. (2018) or continuum mechanics models based on Eulerian methods (Moriguchi et al. 2009), Lagrangian particle-based methods such as Smoothed-Particle Hydrodynamics (SPH), Particle Finite Element Method (PFEM), Finite Element Method with Lagrangian integration points (FEM-LIP), Material Point Method (MPM), (Idelsohn et al. 2006; Bui and Fukagawa 2013; Cuomo et al. 2013; Hamad et al., 2015; Ceccato 2016; Li 2018; Cuomo et al. 2019) or coupled Eulerian-Lagrangian methods (Qiu et al. 2011; Jeong et al. 2019). However, none of those contributions address both the solid-fluid hydro-mechanical coupling inside the flow and the role of fluid in the soil-structure interaction.

The present work investigates these two fundamental issues through the use of an advanced numerical technique, such as MPM. The latter is applied to analyse the behaviour of different flows (dry or saturated) observed in well-documented literature small-scaled laboratory tests.

2 Literature review about impact mechanisms

The impact of flow-like landslides causes the partial or total destruction of the hit structures (Mizuyama 1979; Hungr et al. 1984; Armanini 1997; Zhang 1993; Hu et al. 2011; Shieh et al. 2008;

Hübl et al. 2009; Moriguchi et al. 2009). Many studies focus on real-scale observations through the measurement of the impact forces of real debris flows (Zhang 1993, Wendeler et al. 2007, Hong et al. 2015) or in large-scale experiments using special equipments (Bugnion et al. 2012; Sovilla et al. 2016; Hu et al. 2011; among others). The main problem however is the high cost of the installation and the maintenance of these instrumentations, given the large landslide volumes. In addition, some quantities, such as velocity, depth and other flow parameters are often difficult to know. Therefore, a second group of studies concern reduced-scale laboratory experiments (Moriguchi et al. 2009; Cui et al. 2015; Choi et al. 2016; Zhou et al. 2018, Yifru et al. 2018). Among these, centrifuge tests allow studying the soil behavior under the same stress levels of the in-situ problems, and their applicability to flow analysis has been widely confirmed (Schofield 1980, Bowman et al. 2010, Ng et al. 2016; Song et al. 2017, 2018).

In the relatively simple case of a flow impacting a vertical barrier, the dynamic mechanisms of flow-structure interaction can be grouped in two categories: complete reflection and vertical bulge (Armanini 1993, 1997, 2011; Canelli et al. 2012; Choi et al. 2015; Ng et al. 2017; Zhou et al. 2018). In the first case, a reflected wave forms and propagates away from the wall in a straight direction and with a constant wave speed. Conversely, the vertical bulge occurs when the flow propagates along the vertical wall and then falls creating a hydraulic jump. Experimental evidences showed that for Froude number lower than unity the mechanism of reflected wave predominates, otherwise the vertical bulge takes place and these mechanisms are observed both in homogeneous fluids and two-phase flows (Armanini 2011, Song et al. 2017). When the flow is composed by solid grains and interstitial liquid, the reflection mechanism can be associated to the deposition of granular material just behind the structure in the so-called “dead zone”. Such deposition of debris can develop in turn according two scenarios: pile-up and run-up (Law 2008; Sun and Law 2011; Choi et al. 2015; Ashwood and Hungr 2016; among others). In the former scenario (pile-up), the material behind the barrier is compressed by the approaching material, which transforms into a thin jet rising up the face of the barrier, but without overflowing. This is typical case for dry granular flows and the slope of the final debris deposit is comparable to the internal friction angle of sands (Ashwood and Hungr 2016; Ng et al. 2016; Song et al. 2017). The run-up scenario instead is characterized by the formation of a triangular-shaped deposit behind the wall, which is overridden by the approaching flow. The latter in part goes beyond the barrier developing a vertical jet and in part settles due to the friction with the dead zone forming a new and larger material wedge. This is the typical case of partially or fully saturated granular flows which also generally have an almost-flat debris deposit (Ng et al. 2016; Song et al. 2017, 2018; Zhou et al. 2018). The time for debris filling is higher (hence the impact force is lower) for dry granular flows than two-phase flows, as observed in flume tests (Law 2008; Zhou et al. 2018). In fact, the impact depth of a dry granular flow is always less than barrier height (for moderate inclines obviously), because the flow energy decreases for the contact shearing between grains (Choi et al. 2015; Ng et al. 2016; Wendeler

2016; Song et al. 2017, 2018).

The impact mechanism can be even a combination of the previous scenarios (Law 2008, Ashwood and Hungr 2016). The transition from run-up to pile-up depends from the amount of solid grains in the flow, since high solid fraction produces a well evident dead zone (Takahashi 2014; Song et al. 2017), or likewise from the water content of the flow, as a progressive accumulation of granular material behind the barrier is related to low water content (0-10%) while discrete flow surges occur for high water content (20-30%) during the impact (Zhou et al. 2018). Faug (2015) proposed a so-called phase-diagram for flow-obstacle interaction based on: (i) Froude number and (ii) obstacle height relative to the flow depth (H/h). The diagram comprises four zones: a) Dead zone: the case of relatively slow flows ($Fr \approx 1$) impacting relatively small obstacles ($H/h \approx 1$), with the formation of a quasi-static stagnant zone upstream of the obstacle; b) Airborne jets: rapid flow ($Fr \gg 1$) forming large airborne jet downstream of the obstacle when the obstacle height is low ($H/h \ll 1$); c) Standing jumps: rapid flow ($Fr \gg 1$) propagating downstream of the obstacle with steady-state conditions; d) Bores: rapid flow ($Fr \gg 1$) hitting a wall spanning the entire width of the flow ($H/h \gg 1$) with unsteady conditions. The granular jump (namely bore) hits the obstacle and propagates upstream of it.

While all of these results are extremely useful and will be referred later on, a comprehensive knowledge it is still missing about the quantities from which these complex impact mechanisms of depend on, and even more important would be to assess the role of interstitial fluid on the intensity and the time-dependency of the impact actions. This paper aims to bridge this gap.

3 Material Point Method model

3.1 Fundamental aspects

MPM can be considered a modification of the well-known FEM, and it is particularly suited for large deformations (Sulsky et al. 1994). The continuum body is schematized by a set of Lagrangian points, called Material Points (MPs). Large deformations are modelled by a set of MPs moving through a background mesh, which covers also the domain where the material is expected to move. The MPs carry all the physical properties of the continuum such as stress, strain, density, momentum, material parameters and other state parameters, whereas the background mesh is used to solve the governing equations without storing any permanent information.

A concept scheme of MPM computation is represented in Figure 1. The information is transferred from MPs to the computational mesh. The governing equations are solved at the grid nodes, whereas strains and stresses are calculated at the MPs. The position of the MPs is then updated using nodal displacements. Lastly, the computational mesh is reset back to its initial configuration.

The interaction between phases (solid and liquid in a saturated soil) can be tackled through the *two-phase single-point* formulation (Jassim et al, 2013; Ceccato et al. 2018), where the liquid and the solid acceleration fields ($\mathbf{a}_S - \mathbf{a}_L$ formulation) are the primary unknowns (Fern et al. 2019). Instead, the so-called *one-phase single-point* formulation can be opportunely adopted for dry soils.

3.2 One-phase single-point formulation

The conservation of mass is reported in Eq. 1 and is automatically satisfied as the solid mass remains constant in each MP during deformation.

$$\frac{d\rho}{dt} + \rho \nabla \cdot \mathbf{v} = 0 \quad (1)$$

The conservation of momentum includes the conservation of both linear and angular momentum. The former is represented by the equation of motion, even known as Newton's second law (Eq. 2), while the conservation of angular momentum refers to the symmetry condition of the stress matrix ($\boldsymbol{\sigma} = \boldsymbol{\sigma}^T$).

$$\rho \frac{d\mathbf{v}}{dt} = \nabla \cdot \boldsymbol{\sigma} + \rho \mathbf{g} \quad (2)$$

Finally, the constitutive equation is needed to include the stress-strain dependency (Eq. 3). In order to simulate large deformations and rotations, the Jaumann stress rate can be considered, unlike the Cauchy one which is limited to small strain rate.

$$d\dot{\boldsymbol{\sigma}} = \mathbf{D} \cdot d\dot{\boldsymbol{\varepsilon}} \quad (3)$$

The time integration scheme considered in MPM is explicit, since most of the dynamic problems, including wave or shock propagation, cannot be treated properly by an implicit integration which tends to smooth the solution (Fern et al. 2019).

Let's consider the critical time step Δt_{cr} as the time increment during which a wave with speed c crosses the smallest element length d (Eq. 4).

$$\Delta t_{cr} = \frac{d}{c} = \frac{d}{\sqrt{E/\rho}} \quad (4)$$

The critical time step defines the biggest time increment which can be used for a stable calculation, but often it can't be estimated in case of non-linear problems. For this reason, the critical time step is multiplied by an additional factor C_{NB} (namely Courant number) in order to reach stability. The

Courant number has values between 0 and 1. Generally, the smaller the Courant number and the smaller the time step, improving the accuracy of the numerical results.

3.3 Two-phase single-point formulation

A saturated porous medium is schematized as a solid phase which represents the solid skeleton, whereas the liquid phase fills the voids among the grains. Each MP represents a volume of the mixture V , given by the sum of the solid V_S and liquid V_L phases volumes. The behaviour of a saturated porous medium is here described using only one set of MPs, in which the information about both the solid and liquid constituents is stored.

The velocity field of solid and liquid phases are both used, but the material points move throughout the mesh with the kinematics of the solid skeleton. The equations to be solved concern the balance of dynamic momentum of solid and liquid phases, the mass balances and the constitutive relationships of solid and liquid phases. The accelerations of the two phases are the primary unknowns: the solid acceleration $\mathbf{a}_S$, which is calculated from the dynamic momentum balance of the solid phase (Eq. 5), and the liquid acceleration $\mathbf{a}_L$, which is obtained by solving the dynamic momentum balance of the liquid phase (Eq. 6). Numerically, these equations are solved at grid nodes considering the Galerkin method (Luo et al. 2008) with standard nodal shape functions and their solutions are used to update the MPs velocities and momentum of each phase. The strain rate $\dot{\boldsymbol{\varepsilon}}$ of MPs is computed from the nodal velocities obtained from the nodal momentum.

$$n_S \rho_S \mathbf{a}_S = \nabla \cdot (\boldsymbol{\sigma} - n p_L \mathbf{I}) + (\rho_m - n \rho_L) \mathbf{b} + \mathbf{f}_d \quad (5)$$

$$\rho_L \mathbf{a}_L = \nabla p_L - \mathbf{f}_d \quad (6)$$

The solid and liquid constitutive laws (Eqs. 7-8) are used to compute the increment of effective stress $d\boldsymbol{\sigma}'$ and excess pore pressure dp_L , respectively. The mass balance equation of the solid skeleton is then used to update the porosity of each MP (Eq. 9), while the total mass balance serves to compute the volumetric strain rate of the liquid phase (Eq. 10) since fluxes due to spatial variations of liquid mass are neglected ($\nabla n \rho_L = 0$).

$$d\boldsymbol{\sigma}' = \mathbf{D} \cdot d\boldsymbol{\varepsilon} \quad (7)$$

$$dp_L = \mathbf{K}_L \cdot d\varepsilon_{vol} \quad (8)$$

$$\frac{Dn}{Dt} = n_S \nabla \cdot \mathbf{v}_S = 0 \quad (9)$$

$$\frac{D\varepsilon_{vol}}{Dt} = \frac{n_S}{n} \nabla \cdot \mathbf{v}_S + \nabla \cdot \mathbf{v}_L \quad (10)$$

It is worth noticing that in the *two-phase single-point* formulation, the liquid mass, and consequently the mass of the mixture, is not constant in each material point but can vary

depending on porosity changes. The water is assumed linearly compressible via the bulk modulus of the fluid K_L and shear stresses in the liquid phase are neglected.

The drag force vector $\mathbf{f}_d$ (in Eqs. 5 and 6) is calculated with Eq. 11, where the contribution of the spatial variations of liquid mass is neglected and Darcy's law is used to model solid-liquid interaction forces. For this reason, this formulation is generally adopted in problems with small gradients of porosity, and laminar and stationary flow in slow velocity regime. To investigate the non-linear contribution of the seepage velocity on the solid-liquid interaction forces, which could be relevant in high porosity mixtures where seepage velocity is high, the drag force vector is calculated according to Eq. 12. The coefficient F_E in Eq. 12 is computed through Eq. 13, where A and B are two constants which can be set respectively to 150 and 1.75, according to Ergun (1952).

$$\mathbf{f}_d = \frac{n\mu_L}{k}(\mathbf{v}_L - \mathbf{v}_S) \quad (11)$$

$$\mathbf{f}_d = n \left[\frac{\mu_L}{k} + n\rho_L \frac{F_E}{\sqrt{k}} |\mathbf{v}_L - \mathbf{v}_S| \right] (\mathbf{v}_L - \mathbf{v}_S) \quad (12)$$

$$F_E = \frac{B}{\sqrt{A} n^{3/2}} \quad (13)$$

The intrinsic permeability k is updated with the porosity n . The relationship proposed by Iverson and George (2014) was considered, which links the porosity to a reference permeability k_0 (Eq. 14).

$$k(n) = k_0 e^{(n-0.4)/0.04} \quad (14)$$

The current MPM code uses 3-node elements which suffer kinematic locking, which consists in the build-up of fictitious stiffness due to the inability to reproduce the correct deformation field (Mast et al. 2012). A technique used to mitigate volumetric locking is the strain smoothening technique, which consists of smoothing the volumetric strains over neighbouring cells. The reader can refer to Al-Kafaji (2013) for a detailed description. More details about the numerical MPM integration are instead reported in the Appendix 1.

Regarding the critical time step, the influence of permeability and liquid bulk modulus must be considered as well (Mieremet et al. 2016). In particular, the time step required for numerical stability is smaller in soil with lower permeability (Eq. 15).

$$\Delta t_{cr} = \min \left(\frac{d}{\sqrt{(E + K_L/n)/\rho_m}}, \frac{2(\rho_m + (1/n - 2)\rho_L)k_{Darcy}}{\rho_L g} \right) \quad (15)$$

4 Impact of dry granular flows tested in flume tests

4.1 Experimental tests of Moriguchi et al. (2009)

Small-scale experiments were conducted by Moriguchi et al. (2009) to evaluate the impact force of a dry granular flow on a rigid wall after propagating at different slope angles (θ). The Toyoura sand (Tab. 1) was used, with 50 kg put into a 50x30x30 cm box situated at the top of a plastic transparent flume. The sand was released by the outer opening door of the box, and after sliding 1.80 m, impacted against a wall 0.3 m high. The configuration of the slope model is shown in Figure 2. The upper side of the flume box is not closed hence the sand can overcome the barrier. Some sand was glued on the base of flume to create friction with the mass flowing down, whereas the impact surface of the wall can be considered as smooth.

The inclination of the flume was varied from 45 to 65° at intervals of 5°. The impact force on the wall was recorded using a load cell placed inside the wall itself and for each inclination five trials of experiment were conducted. Among various results, the peak impact force increases rapidly with higher inclinations, except for 45° and 50° where the peak cannot be uniquely identified (Figure 2).

4.2 MPM one-phase modelling

The numerical model schematizes the cross-section of the flume in plane strain conditions. The 2D mesh consists in 23,511 triangular elements with an average size of 0.02 m. The simulations consider: the granular material with elasto-plastic failure behavior, the sliding plane with a contact friction coefficient and the barrier with a rigid behavior (Figure 3). The *one-phase single-point formulation* is adopted to schematize all the materials and the mixed-integration is used to calculate the internal forces of the system (see Appendix 1).

The first stage of the analysis aimed to determine the most likely value of a chosen set of parameters including internal friction angle (φ'), Young modulus of soil (E) and contact friction angle (φ'_b). For the calibration of these parameters, the flume inclination of 45° was considered. The other slope angles (50°, 55°, 60° and 65°) were then used for model validation.

The parameters obtained from calibration are shown in Table 2, with also some other values used later in a sensitivity analysis. The friction coefficient at contact with the sliding surface $\tan(\varphi'_b)$ was assumed as equal to $\tan(\varphi')$, since the flume was coated with sand, whereas the wall was considered smooth and thus the friction coefficient is set to zero. The calibration of model parameters was pursued by comparing the flow configurations at four time lapses (0.4, 0.8, 1.2 and 1.6 seconds), for which the snapshots of the experiments and the numerical results were both available (Figure 4).

The trend of the impact force F over time was also used as observation for the model calibration. The contact formulation was used to ensure that no interpenetration occurs, and the tangential

forces are compatible with the shear strength along the contact. The reaction force at node i was calculated as follows:

$$F_{contact,i} = m_{i,S}\Delta a_{S,contact} + m_{i,L}\Delta a_{L,contact} \quad (16)$$

where $\Delta a_{S,contact}$ and $\Delta a_{L,contact}$ are the change in acceleration induced by the contact formulation, for both solid and liquid phase, and $m_{i,S}$ and $m_{i,L}$ are the corresponding nodal masses.

The total reaction force is the integral of the nodal reaction forces along the barrier.

The calibrated parameters allow a good agreement between simulated and experimental results also in terms of impact force (Figure 5).

The model was then validated with the tests with inclinations of 50° , 55° , 60° and 65° as shown in Figure 6, where it is clear a good match between simulation and experiment for each inclination of the flume. Such correspondence is even emphasized in Figure 7, which shows that both the simulated peak force and time to peak deviate from the experiments by a maximum of 15%. All the output from simulations were post-processed in order to eliminate the outliers and to obtain a mean trend of the impact force curves. In this case, the Gaussian function available in MATLAB's "curve fitting" tool was employed.

The trend in space and time of many physical variables (e.g. stress, strain, velocity, pore pressure and other state parameters) was also investigated. For example, the distribution of deviatoric strain during impact allowed understanding that in the case of dry granular flows if the contact shear stress is high (as for inclination of 45° in Figure 8) the impact mechanism results in a pile-up without evident peak load, as also highlighted by Choi et al. (2015). If the inclination angle is high (like 65° in Figure 9) the impact mechanism of the dry flow resembles the run-up mechanism, which is typical of partially or fully saturated flow (Ng et al. 2016; Song et al. 2017, 2018; Zhou et al. 2018). Also in this case, the impact force trend is consistent with the vertical jet mechanism, previously outlined in the literature, with such behaviour mainly linked to the lower shear stress and easily identifiable from the computed deviatoric strain distribution.

The tempo-spatial distribution of both velocity and depth of the approaching flow is another useful result achieved through the MPM simulations. These quantities are correlated to the Froude number through the well-known equation:

$$Fr(x,y,t) = \frac{v(x,y,t)}{\sqrt{gh(x,y,t)}} \quad (17)$$

It is possible to identify the impact mechanism patterns by comparing the spatial distribution of the Froude number at the impact and the obstacle height relative to the flow depth (H/h). The obvious benefit compared to laboratory experiments or depth-integrated models here is that the flow velocity and depth are not averaged. Let's define h the flow depth at the upstream section of the

granular jump (namely bore), $L_{(+)}$ and $L_{(-)}$ the length of the zones of influence upstream and downstream of the wall, respectively (Faug, 2015).

If the flume is inclined of 45° , a thick granular layer moves upstream of the wall (subcritical flow), and intercepts the grains from the incoming rapid flow (supercritical flow). This is immediately visible from the spatial distribution of Fr (Figure 10). The impact mechanism resembles the bores regime in the phase diagram proposed by Faug (2015), since a granular bore is formed at the transition between the subcritical and the supercritical flow. As H/h decreases, a stationary granular jump is formed leading to a steady value of $L_{(+)}$, while a part of the incoming supercritical flow runs over the wall, forming a jet with very low energy ($L_{(-)}$ much smaller than $L_{(+)}$).

In the case of 65° inclined slope, a granular airborne jet is observed downstream of the wall. As highlighted in Figure 10, the incoming flow is characterized by low H/h and high Fr . In this situation, the flow can easily move over the wall forming a large granular airborne jet. When steady state is reached, the downstream length $L_{(-)}$ is much larger than $L_{(+)}$, which is close to the barrier height.

Based on the satisfactory calibration and validation of the proposed MPM model, it is now possible investigating the role played by some parameters on the flow impact behaviour. It has been seen that a variation of the soil Young modulus E is not significant in terms of flow propagation and it neither influences the impact force trend (Figure 11a), contrarily to soil effective friction angle (φ') and base contact friction (φ'_b) that create substantial differences. Decreasing φ' , the granular flow propagates faster (Figure 11b), the dynamic force increases, while the static component of the force is not much influenced. Other considerations are made with the base contact friction (φ'_b) changed (Figure 11c): i) if φ'_b is reduced to $\varphi'/2$, the impact mechanism changes from pile-up to run-up and the related impact force now reaches an evident peak, ii) if φ'_b is put equal to zero (i.e. completely smooth slope), the peak force increases much.

5 Simulation of saturated flows in centrifuge tests

5.1 Experimental evidences of Song et al. (2017)

The material was put in a container of 0.03 m^3 (model scale) to reproduce a prototype flow-like landslide with a volume of about 170 m^3 and dropped from a height of 500 mm onto the slope by opening a bottom door of 200 mm. A steel barrier, equivalent to a reinforced concrete wall (0.9 m thick and 4.5 m high) was used. A centrifuge scale factor (N) of 22.4 g is here referred to convert the model quantities to the prototype scale, being the value of the gravity acceleration at the mid-height of the barrier during the centrifuge tests. In particular, forces and the times in the prototype scale are amplified of N^2 and N times, respectively, while the conversion factors for other quantities

are reported by Song et al. (2017). The authors examined the flow impact behaviour by varying the solid fraction among the values of 0.0 (fluid case), 0.2, 0.4, 0.5 and 0.58. The Leighton Buzzard (LB) fraction C sand was adopted, characterized by grains of 0.6 mm diameter and a specific gravity of 2.679. The viscous liquid adopted is a solution of water and carboxy methyl cellulose (CMC), with the same density of water and viscosity equal to 0.022 Pa·s in the model scale. The properties of the materials used in the tests are summarized in Table 4. During the tests, 15 sets of semiconductor strain gauges, 5 dynamic load cells fixed inside the barrier and a high speed camera able to measure the velocity of the flow through PIV (Particle Image Velocimetry) analysis were used.

5.2 MPM two-phase modelling

The centrifuge device is numerically schematized by a 3-node triangular unstructured mesh with an average element size of 0.015 m. In order to avoid unrealistic movements of the flow at impact against the barrier, an additional contact node is considered in the top left corner of the barrier allowing the presence of two normals on each adjacent side (Figure 13). The dimensions in the numerical model are the same of the experiment and a gravity multiplier of 22.4 is considered.

It is worth noting that during the tests the flow undergoes a radial increment of N , principally affecting the dropping of the mixture from the box. For instance, the sample is initially triggered at about 17.8 g-level. However, the numerical model cannot consider a linear change of g with the distance from the centre of the centrifuge. To overcome such difficulty, a size re-adjustment of the sizes of the numerical model is performed along the vertical direction, considering that in the centrifuge test the g -level differs from 17.8 (at the storage container) to 22.4 (at middle height of the barrier). First, a uniform 100 mm discretization is done and then each discretized length Δy_i is multiplied by the local N_i in order to obtain the local prototype dimensions, which are finally rescaled to the model size through the N factor of 22.4. In this way, the new dimensions of the box are equal to 238 mm (i.e. $300 \text{ mm} \times 17.8 \text{ g} / 22.4 \text{ g}$) and the distance between the bottom of the storage container and the top of the slope is 454 mm (i.e. $\sum_i \Delta y_i N_i / 22.4$). Without this specific procedure, the numerical model would correspond to a prototype debris flow volume of 340 m³, twice than in the experiments.

The MPM model is calibrated for the mixture with 50% solid fraction (SL50 test), that is representative of real flow-like landslides. The model is then validated in the case of 40% solid fraction (SL40 test) whose impact force diagram is reported in Song et al. (2018). The *two-phase one-point* formulation is used to model the mixture with a fully hydro-mechanical coupled behaviour, while the flume and the rigid barrier are modelled with the *one-phase one-point* formulation. The interactions between the flow and the flume or the barrier are handled through the same frictional contact algorithm used before. An elasto-plastic behavior is assumed for the sand

at failure, while the barrier and the slope are assumed as linear elastic materials. The pore-fluid bulk modulus is set to a low value in order to decrease the computational cost, but still large enough to have negligible influence on the results. The intrinsic permeability of the sand mixture can be estimated by the empirical relationship proposed by Iverson and George (2014), reported in Sect. 3.3 (Eq. 14).

The calibration of the permeability value of the soil in the container $k(n_0)$, Young modulus E and friction angle φ' is done by comparing the simulation results to the impact forces measured over time during the test SL50. The initial values adopted for the calibration are $k = 10^{-9} \text{ m}^2$, $E = 10^3 \text{ kPa}$, and $\varphi' = 31^\circ$ (equal to the critical friction angle of the sand). These values are then adjusted for a best-fitting of the results in these ranges: $k = 10^{-13} \div 10^{-8} \text{ m}^2$ (Iverson and George, 2014); $E = 10^2 \div 10^4 \text{ kPa}$ (Iverson, 1997); $\varphi' = 25 \div 45^\circ$ (Iverson, 1997).

Five zones corresponding to the five load cells used in the experiment are considered to calculate the impact pressure at different barrier heights. Each pressure is then multiplied by the influence zone (in the prototype scale) of the corresponding load cell and then integrated over the height of the barrier to obtain the overall impact force. The initial interaction times between the flow and the barrier are reset to 1.0 s (0.045 s in model scale) as made by Song et al. (2017).

From model calibration a good match is achieved between the experimental evidence (Figure 14) and the simulation outcomes either in terms of flow motion, where both flow configuration and velocity are well captured (Figure 15), and either in terms of impact pressures in almost all the reference points. The impact pressure results have been post-processed using the robust lowess smooth function available in Matlab with a span equal to the 3% of the data.

The impact forces are calculated by modelling the soil-water interaction forces both through the linear Darcy's law (Eq. 11 in Sect. 3.3) and through the non-linear velocity contribution (Eq. 12 in Sect. 3.3). It was interesting to discover that in this case study, the two formulations give negligible differences in the results, even if the non-linear velocity term leads to a slightly better fit of the results (Figure 16).

According to the two-phase single-point formulation, the liquid mass can vary depending on porosity changes. Anyway, this limitation seems to have an irrelevant role in the computation of the impact forces, since porosity change is very large only after the impact against the barrier, not before it. In fact, the values of the porosity in the part of the soil located upstream of the barrier vary between 0.455 and 0.512. Such values of the porosity variation do not produce a relevant increase in terms of mass mixture, with a consequently negligible effect in terms of impact forces. If the non-linear velocity contribution is included, the porosity distribution is more uniform, varying between 0.472 and 0.510 since the soil globally becomes less permeable during the flow. However, for the soil parameters chosen in this paper, its contribution in terms of reaction forces against the barrier is negligible. It can be concluded that, although this formulation should be still improved, it seems sufficiently accurate for the case study described in this paper.

The calibrated parameters are reported in Table 5, and they allow quite good results also for the model validation on the SL40 test (Figure 17). However, some discrepancies between the pressure distribution measured for the SL40 test and the numerical results are also noted. This is probably because flows with low solid fraction are dominated by viscous stresses (Song et al. 2017). Consequently, if viscous stresses are not taken into account higher mobility is developed during the impact mechanism. On the contrary, in the case of larger solid fraction, many studies have demonstrated that viscous shearing is less effective in energy dissipation compared to grain shear stresses (Iverson and George 2014; Choi et al. 2015; Ng et al. 2017; Song et al. 2017). In addition to this, viscosity inside debris flow is not easily measurable. For this reason, the mathematical formulation used in this study voluntarily neglected the viscous stresses inside the flow.

Globally, the impact mechanisms of the two tests can be well analyzed referring to the simulation snapshots of Figure 18, which show the spatial distribution of the Froude number at impact. As observed in the laboratory, the two debris flows are both characterized by a run-up mechanism with the formation of a dead zone behind the barrier. The latter one acts as a springboard along which the remaining flow accelerates, and finally creates a vertical jet. The extent of the dead zone strongly depends on the percent of solid fraction in the flow, since it is larger when the solid fraction increases (Bagnold 1954; Takahashi 2014; Song 2017). The impact mechanisms described through the MPM analysis are in agreement with those observed in the laboratory by Song et al. (2017). Even the Froude number at impact is very close to the one calculated in the experiment: in fact, from laboratory observations Fr is equal to 3.6 and 5 for the SL50 and SL40 tests, respectively.

Even for this case study, the role of some quantities on the impact behaviour is pointed out. Different permeability of the sand has significant effects on porosity distribution within the flow during the impact stage (Fig. 19). For an initial porosity n_0 of 0.5 (SL50 test) and the highest permeability, the dead zone behind the barrier is characterized by a lower porosity (Figure 19a). If permeability is lower than the previous case, then the porosity of the material behind the barrier gradually increases, as shown in Figure 19b and Figure 19c. This is related to the hydro-mechanical coupling, which makes that a change in soil permeability affects the liquid pressure inside the flow, which in turn modifies the stress and strain depending on material constitutive response. As further issue, the volumetric strains change the porosity distribution inside the flow. The higher the volumetric strain, the higher the porosity inside the flow, as observable for the airborne jet where very high porosity are simulated.

For low permeability value, the porosity in the dead zone results higher than the initial value. Since permeability is low, pore-fluid cannot easily escape from the soil at impact moment, so the soil tends to dissipate the excess pore-pressure by increasing porosity. In contrast, for the case with high permeability, the porosity diminishes in the dead zone probably because the flow moves more slowly than the previous case and so the material within the dead zone undergoes a sort of

consolidation.

6 Discussion of numerical results

The MPM numerical simulations provide new insights in understanding the impact mechanisms of both dry and saturated granular flows, related to the many physical properties (e.g. stress, strain, liquid and solid velocity, pore pressure and other state parameters) that can be accurately computed either in space and time.

The phase diagram proposed by Faug (2015) for granular flow-structure interaction is here used to comment more on some of the numerical results. For the sake of brevity, the combination of H/h and Fr are limited to the most relevant cases of this study, i.e. the dry flow simulated along the 45° to 65° inclined slope and the saturated flow observed in the SL50 and SL40 tests (Figure 20). For the dry granular flows, the impact mechanism changes in relation to slope inclination. The impact mechanism resembles the bore regime (slope angle equal to 45°), then becomes a standing jump (50° and 55° inclinations) and finally reaches the airborne jet regime (60° and 65° slopes). This is principally governed by Fr number, since for higher inclinations the flow velocity increases while the flow depth h remains almost the same.

Different is the case of the saturated granular flows, since both the considered cases (SL50 and SL40 tests) can be assimilated to a standing jump regime. The solid-fluid interaction is however highlighted in the phase diagram. Higher solid fractions lead to a decrease in Fr number, confirming the effectiveness of solid fraction in energy dissipation and reduction of debris mobility pointed out by Song et al. (2017).

7 Conclusions

The present paper shows the potential of an advanced numerical method such Material Point Method (MPM) in simulating the impact mechanisms of dry or saturated granular flows against rigid walls, providing novel insights additional to those from laboratory experiments.

The laboratory flume tests reported by Moriguchi et al. (2009) and Song et al. (2017) were simulated in a MPM context. In both the sets of simulations, the calibration and validation of the numerical model lead to a satisfactory correspondence among the experimental and the numerical results in terms of flow kinematics and impact force trend over time. It was possible investigating the influence of some material properties (such as solid Young modulus, internal friction angle, friction at contact surfaces, permeability, Froude number) on the impact behaviour of the flow.

The numerical results provide new insights in understanding the complex impact mechanisms through the computation and time-space tracking of some quantities, such as stress, strain, pore

pressure, solid and liquid velocities, which cannot be easily monitored/obtained from laboratory experiments. Specifically, it is observed that the presence of liquid phase inside the granular flow can lead to different impact regimes in respect to the dry flows, highlighting the importance of considering the solid-fluid interaction in the analyses. The accurate knowledge of the impact mechanism of the flows is crucial for the design of protection structures against avalanches. For example, the accurate estimate of the length of an airborne jet must prevent that the retaining structure is overtopped by a jet, thus being ineffective. Differently from the many empirical equations available in the literature and currently used for the design of these structures, advanced numerical methods like MPM allow to take into account all such important aspects of the granular flows. Further research could focus on the verification of the equations proposed in the literature to evaluate the impact forces against obstacles, even considering the presence of viscous stresses inside saturated granular flows, in order to understand the conditions in which these stresses could change the hydro-mechanical behaviour of the flow. It is also worth stressing that the proposed study is focused on the modelling of flows with a significant volumetric content of solids, this means that the statements could be extended for instance to real debris flows, but hardly to the case of hyperconcentrated flows.

Successful simulation of centrifuge tests is an important achievement. In fact, from centrifuge tests come the most accurate measurable (and repeatable) evidence of such processes at a prototype scale not so far from reality. Further research could try to simulate real-scale case histories, aware that the data from the field are generally few and sometimes uncertain.

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Appendix 1. Mixed integration schemes

The stress integration in MPM is traditionally achieved by calculating the internal force using all MPs inside each element, as follows:

$$f_i^{\text{int}} = \sum_{MP=1}^{N_{MP}^e} B_i^T(\xi_{MP}) \sigma_{MP} \Omega_{MP} \quad (\text{A1})$$

where σ_{MP} is the stress and Ω_{MP} is the integration weight of each material point, N_{MP}^e is the number of material points in the elements and ξ_{MP} is the local coordinate of the material point. The quality of such integration depends on the number and the location of the MPs, and the computational cost can significantly increase with very large number of MPs (Al-Kafaji, 2013).

Contrary to the MP integration, the Gauss integration consists of calculating the internal force in a fixed number of integration points in an optimal location to achieve the highest accuracy.

The internal force in one node is computed replacing the summation over MPs by the summation over Gauss points (GP), as follows:

$$f_i^{\text{int}} = \sum_{GP=1}^{N_{GP}^e} B_i^T(\xi_{GP}) \sigma_{GP} \Omega_{GP} \quad (\text{A2})$$

where σ_{GP} and Ω_{GP} are respectively the stress and the integration weight of the gauss-point in the element. In linear 3-node triangular elements, only one Gauss point is used and the corresponding stress is calculated as follows:

$$\sigma_{GP} = \frac{\sum_{MP=1}^{N_{MP}^e} \sigma_{MP} \Omega_{MP}}{\sum_{MP=1}^{N_{MP}^e} \Omega_{MP}} \quad (\text{A3})$$

In this numerical scheme, the gaussian integration is performed in fully-filled elements, i.e.:

$$\sum_{MP=1}^{N_{MP}^e} \Omega_{MP} > 0.9 \Omega_{el} \quad (\text{A4})$$

where Ω_{el} is the volume of the element. If this condition is not satisfied, the original material point integration is performed.

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Table 1 Characteristics of the Toyoura sand adopted in the experiment

ρ	e_{min}	e_{max}	D_{min}	D_{max}	C_u
1379 kg/m ³	0.61	0.97	0.102 mm	0.425 mm	1.37

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Table 2 Input used for the calibration and validation of the MPM model

Material	Const. model	ρ_s (kg/m ³)	n (-)	E (MPa)	ν (-)	c' (kPa)	ϕ' (°)	ϕ'_b (°)	ψ (°)
Dry sand	MC	2650	0.48	10 20 30	0.2	0	30 35 40	0 20 40	0
Plane	LE	7850	0	infinity	0.3	-	-	-	-
Rigid wall	LE	7850	0	infinity	0.3	-	-	-	-

Draft

Table 3 Calibrated input for 45°-slope case

Material	Const. model	ρ_s (kg/m ³)	n (-)	E (MPa)	ν (-)	c' (kPa)	ϕ' (°)	ϕ'_b (°)	ψ (°)
Dry sand	MC	2650	0.48	20	0.2	0	40	40	0
Plane	LE	7850	0	infinity	0.3	-	-	-	-
Rigid wall	LE	7850	0	infinity	0.3	-	-	-	-

Draft

Table 4 Properties of the materials used in the tests, whose ID are reported in brackets

Material	ρ_s (kg/m³)	μ_L (Pas)	ns (-)	ϕ' (°)
Viscous fluid	1000 (L)	0.022	-	-
LB sand (fraction C)	1825 (SL50) 1660 (SL40)	-	0.5 0.4	31

Draft

Table 5 Materials properties employed in the MPM simulations

Material	Const. model	ρ_m (kg/m ³)	n_0 (-)	E (MPa)	ν (-)	c' (kPa)	ϕ' (°)	ϕ'_b (°)	ψ (°)	$k(n_0)$ (m ²)	k_0 (m ²)	K_L (MPa)	μ_L (Pas)
Saturated sand	MC	1825* 1660**	0.5* 0.6**	1.5	0.2	0	31	31	0	1.2×10^{-8}	9×10^{-10} * 9×10^{-11} **	50	0.022
Plane	LE	7850	0	infinity	0.3	-	-	-	-	-	-	-	
Rigid wall	LE	7850	0	infinity	0.3	-	-	-	-	-	-	-	

*SL50 test; ** SL40 test

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Figure 1. Concept of MPM computation.

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Figure 2. Flume configuration and impact forces recorded in the experiments of Moriguchi et al. (2009)

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Figure 3. Computational mesh and materials

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Figure 4. Comparison between experimental and simulated flow configurations

Figure 5. MPM model calibration for slope inclination $\theta = 45^\circ$

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Figure 6. Validation of MPM model for different inclinations: experimental results in dotted lines and simulation results in continuous lines

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Figure 7. Observations versus simulation results

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Figure 8. Tempo-spatial distribution of soil deviatoric strain for the inclination of 45°

Figure 9. Computed soil deviatoric strains for the inclination of 65°

Figure 10. Impact mechanism regimes for slope inclinations of 45° and 65°

Figure 11. Impact force over time: comparison between experimental and MPM results for different (a) E , (b) ϕ' and (c) ϕ'_b

Figure 12. Centrifuge apparatus used in the experiment (Song et al., 2017)

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Figure 13 Computational mesh and materials

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Figure 14. Experimental observations with PIV analysis on the right

Figure 15. MPM simulation snapshots showing the velocity distribution for SL50 test (calibration)

Figure 16. Experimental and simulated values of impact force over time for test SL50 (calibration)

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Figure 17. Experimental and simulated values of impact force over time for test SL40 (validation)

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Figure 18. Spatial distribution of Froude number during impact ($t=1.9$ s)

Figure 19. Influence of intrinsic permeability on porosity change:

(a) $k_0 = 10^{-10} \text{ m}^2$; (b) $k_0 = 10^{-11} \text{ m}^2$; (c) $k_0 = 10^{-12} \text{ m}^2$

Figure 20. Comparison of the MPM results (black symbols) with the limits (dashed lines) plotted in the phase diagram of Faug (2015).

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