Study of flow landslide impact forces on protection structures with the Material Point Method

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ABSTRACT: to assess the potential damage caused by a landslide to existing structures, an estimate of the impact forces is necessary. Several empirically based relationships have been proposed to calculate the peak force, but significant differences are obtained with various approaches. The earth pressure distribution along depth, which determines the bending moments on the structures, varies with time and flow characteristics. Simplified linear distributions (trapezoidal, triangular or rectangular) are commonly assumed, but the real distribution can be more complex. Advanced numerical analyses, capable of reproducing the key aspects of the phenomenon, can give a valuable insight into this problem. This paper investigates the potentialities of a numerical approach based on the Material Point Method (MPM) to the study of the interaction between flow landslides and retaining structures. The effect of different soil properties and boundary conditions on the impact force is discussed.

1 INTRODUCTION

Flow landslide are very rapid slides that cause severe damages and loss of human lives every year worldwide. Earthquakes, rainfall, and other geological and meteorological events accelerate their occurrence. The assessment of the damage caused by the landslide, as well as the design of protection structures, requires an estimate of the impact forces and bending moments.

There are two traditional simplified models to calculate impact forces (Fig. 1). The hydrostatic approach assumes a triangular load distribution in which the maximum pressure is a function of the bulk density (ρ), the flow high (*h*) and an empirical factor (*k*) (see e.g. Armanini 1997). The hydrodynamic approach assumes a constant load distribution in which the pressure is a function of the square of the impact velocity (*v*), the bulk density, and an empirical factor (*a*). Alternative load models are shown in Hübl et al. (2003), Suda et al. (2009) and Jiang & Towhata (2013).

The empirical factors *a* and *k* are function of the flow characteristics. They vary in a wide range, which renders the practical use of these approaches rather difficult. Values of *k* between 2.5 and 7.5 were measured on small scale tests by Scotton and Deganutti (1997), and between 0.2 and 2 on real scale experiments by Bugnion et al. (2012). Based on fields measurements, Zhang (1993) recommends values of *a* between 3.0 and 5.0, and Bugnion et al. (2012) proposed a range between 0.4 and 0.8.

Numerical methods able to capture the key features of the landslide and its interaction with a structure can contribute to assess the damage to existing

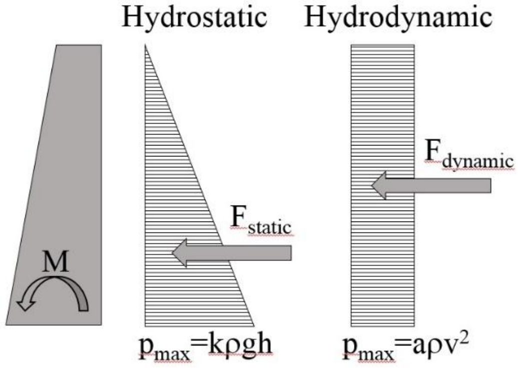

Figure 1 Simplified models to calculate impact forces

structures and guide the design of protection measures. To this end, Eulerian frameworks, such as finite difference techniques (Moriguchi et al. 2009) and control volume methods (Guimarães et al. 2008), Lagrangian meshless methods, such as the Smoothed Particles Hydrodynamics (SPH) (Bui et al. 2008), and the Discrete Element Method (DEM) (Teufelsbauer et al. 2011; Leonardi et al. 2014) can be applied. This paper investigates the potentiality of the Material Point Method (MPM) for these applications.

The MPM has been specifically developed for large deformations of history dependent materials. It simulates large displacements by Lagrangian points moving through an Eulerian grid as shortly described in Section 2. A 3D MPM code featuring a specific algorithm to model soil-structure interaction and frictional sliding is applied in this study.

In addition to the difficulties in simulating large displacements and soil-structure interaction, also the definition of a constitutive model able to capture the soil behavior under a wide range of strain rates is an important issue. It has been observed that the Coulomb friction generates most of the stress in dense granular flows (Iverson 1997), thus in this study, the behavior of soil is simulated by an elastoplastic model with Mohr-Coulomb failure criterion. This model showed to capture very well the propagation of dry granular flows (Bandara 2013; Ceccato & Simonini 2016)

Section 3 shows that the MPM can be applied to estimate the evolution of impact forces and bending moments on a rigid structure. It is shown that the earth pressure distribution is more complex than commonly assumed by simplified models, thus confirming the importance of improving the understanding of the landslide-structure interaction. Moreover, parametric studies can clarify the effect of different parameters on the peak force.

2 THE MATERIAL POINT METHOD

The MPM is a particle-based method developed since the 90's for large deformations of history dependent materials (Sulsky et al. 1994). Recently, the method has been extended to coupled problems in order to simulate the soil-water interaction (Jassim et al. 2013; Abe et al. 2013) and unsaturated conditions (Yerro et al. 2015). The MPM has been successfully applied to the simulation of a number of geotechnical problems such as slope stability (Andersen & Andersen 2010), collapse of dams (Alonso & Zabala 2011) and riverbanks (Bandara & Soga 2015), cone penetration (Ceccato & Simonini 2015; Beuth & Vermeer 2013), and impact of granular avalanches on rigid obstacles (Mast et al. 2014).

The continuum body is discretized by a set of Lagrangian points, called material points (MP). They carry all the information of the continuum such as

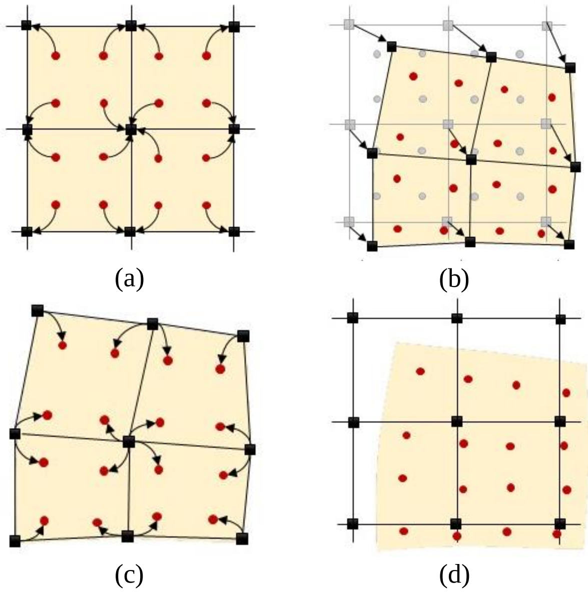

Figure 2 Computational scheme of MPM

density, velocity, acceleration, stress, strain, material parameter as well as external loads. The MP do not represent single soil grains, as in DEM, but a portion of the continuum body. Large deformations are simulated by MP moving through a fix computational finite element mesh which covers the entire region of space into which the solid is expected to move. This grid is used to solve the system of equilibrium equations, but does not deform with the body like in Lagrangian Finite Element Method.

At the beginning of each time increment, the information is mapped from the MP to the computational nodes of the mesh by means of the shape functions (Fig. 2a). The governing equations of motion are solved (Fig. 2b) and the nodal values are used to update the velocity, the position and to compute strains and stresses at the MP (Fig. 2c). At the end of the time step, the mesh is usually reset into its original state. The assignment of MP to finite elements is updated after mesh adjustment (Fig. 2d).

The MPM code used in this study is being developed to solve 3D dynamic large deformation problems in geotechnical and hydromechanical engineering (Vermeer et al. 2013). Soil-structure interaction and frictional sliding are simulated by a contact formulation based on Coulomb's law (Bardenhagen et al. 2001).

3 STUDY OF THE IMPACT OF A DRY GRANULAR FLOW ON A RIGID STRUCTURE

3.1 *Geometry and discretization*

Since most of the experience on the interaction between granular flows and structures has been gained by means of laboratory experiments, a small-scale test is considered here. This simplifies the comparison between numerical and experimental results.

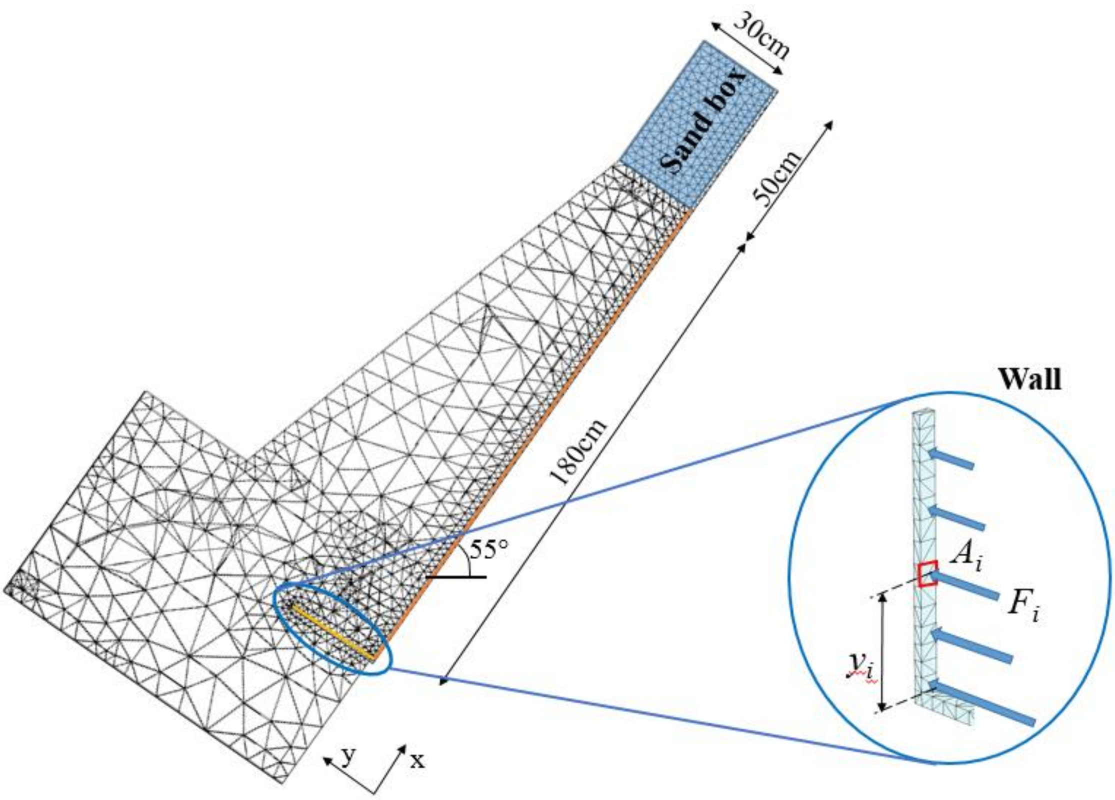

Figure 3 Geometry and discretization of the problem.

The geometry and discretization of the problem is illustrated in Figure 3. A 30cm-high and 50cm-long box of sand is initially placed at the top of a slope inclined 55°. A 30cm-high rigid wall is placed 180cm downslope. The model width is 2cm.

The slope is characterized by a basal friction coefficient of 0.64; the wall is assumed to be smooth.

The computational mesh should be fine where high stress and deformation gradients are expected, like in FEM. In order to improve accuracy, the mesh is refined along the sliding surface and the face of the wall. 20 MP are initially placed inside each sand element inside the box. The optimal discretization has been determined through preliminary analyses as a compromise between accuracy and computational cost.

The constitutive behavior of sand is modelled with and elastic-perfectly plastic model with Mohr-Coulomb failure criterion. The material parameters are summarized in Table 1.

Table 1 Material parameters

Grain density [kg/m^3]	2650	Young mod. [kPa]	50
Porosity	0.48	Poisson ratio	0.2
Friction angle [°]	35°	Cohesion [kPa]	0

The sand is suddenly released at t=0s, then it slides down the slope hitting the rigid wall. The total force, the bending moment and the stress distribution in front of the wall are then calculated.

The pressure distribution is obtained considering the normal force on 15 strips, 2cm-high each, and dividing the force by the strip area (A_i). This average normal stress (p_i) is assumed to act at the level of the barycenter of the considered strip (y_i) (Fig. 3). The bending moment with respect to the base of the wall is calculated as

$$M = \sum_{i=1}^{15} p_i A_i y_i \quad (1)$$

The mean velocity in a section between 10 and 20cm in front of the wall and the flow thickness just before the impact are measured. These measured velocity and flow high are useful to compare the pressures obtained by the model with the one suggested by empirical formulas.

3.2 *The flow-structure interaction*

The sand flow accelerates and elongates while descending the slope; when it reaches the structure, the flow is deviated upwards, parallel to the wall, with the formation of a bulge, and it subsequently decelerates, and compacts (Fig. 4).

The material hits the wall 0.62s after it has been released. The maximum velocity is 4.7m/s and the flow high is 10mm, thus the problem is characterized by a Froude number $F_r = v/\sqrt{gh} = 4.7$.

Figure 5 shows the evolution of force and bending moment with time. The impact force on the wall increases rapidly up to a peak value of 1.78kN/m which occurs at t=0.76s. After the peak, the force decreases to a quasi-static value. The bending moment due to the earth pressure reaches the peak at t=0.9s; this means that the pressure distribution resulting in the highest forces does not coincide with the one giving the maximum moments.

Figure 6 shows the pressure distributions in front of the wall for five instants close to the force peak. It can be observed that the linear distribution commonly assumed in practice is a poor approximation of the real force distribution, which is curved, thus it could lead to a wrong estimate of bending moments. Similar deviations from the linear distributions were observed experimentally in granular flows by Hübl et al. (2003) and Jiang & Towhata (2013).

A maximum pressure p_{max}=19.8kPa is observed, which is in reasonable agreement with the value of 23.5kPa obtained using the modified hydrodynamic formula given by Hübl et al. (2003), i.e.

$$p_{max} = 5\rho v^{0.8}(gh)^{0.6} \quad (2)$$

The empirical coefficients a and k can be evaluated as the ratio between the maximum pressure and the hydrodynamic pressure (ρv^2) or the hydrostatic pressure (ρgh) respectively. Figure 7 shows that the values obtained by the numerical model are in agreement with experimental results for similar Froude numbers collected by Hübl et al. (2009).

3.3 *Factors influencing the impact force*

A parametric study has been carried out to determine the effect on the impact force of key factors such as the soil friction angle (ϕ), the basal friction coefficient (μ), the initial porosity (n), the Young modulus (E), and the Poisson's ratio (ν).

Table 2 summarizes the input parameters of the performed simulations, the maximum flow high (h), the impact velocity (v) and the obtained peak force (F_{max}).

The most significant parameter is the basal friction coefficient. Indeed, when μ decreases the impact velocity increases because less energy is dissipated by friction at the sliding surface; moreover, the soil mass tends to move with a more compact shape, i.e. the flow elongation reduces and the thickness increases as shown in Figure 8.

The soil friction angle has a negligible effect on the peak force, but higher values of the post-peak force are observed increasing ϕ. This is explained by the fact that in static conditions the force on the wall is proportional to the passive earth pressure coefficient, which increases with ϕ.

Decreasing the Young modulus the impact velocity and the flow high remains approximately similar, but the evolution of the force in time changes dramatically and the peak value decreases significantly. Indeed, a very compressible material compacts against the wall, thus reducing the deceleration of the soil mass compared to a stiffer material. This effect can be observed in Figure 9, which shows that the average velocity in the control region decreases suddenly for a higher Young modulus and much slowly in a more compressible material.

Establishing a reasonable value of the Young modulus for the granular flow is a difficult task because

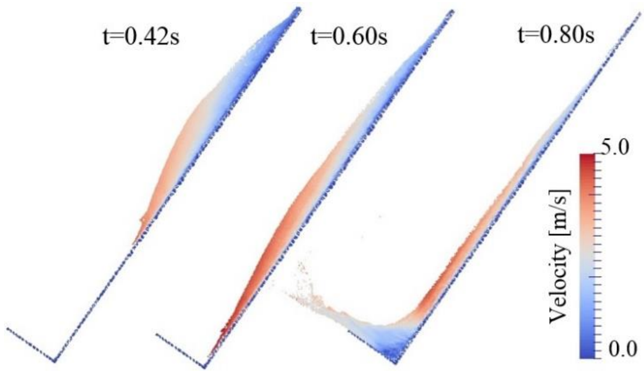

Figure 4 Simulated landslide dynamics

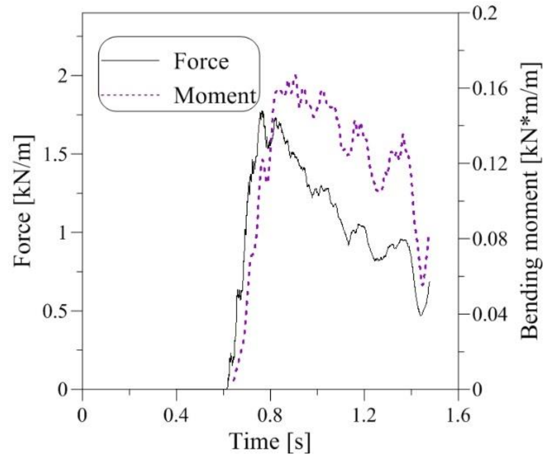

Figure 5 Force and bending moment evolution in time

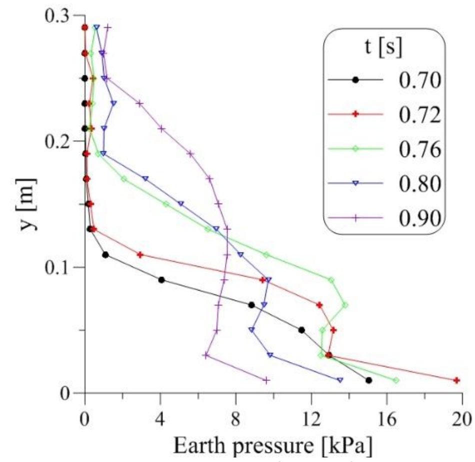

Figure 6 Pressure distribution in front of the wall.

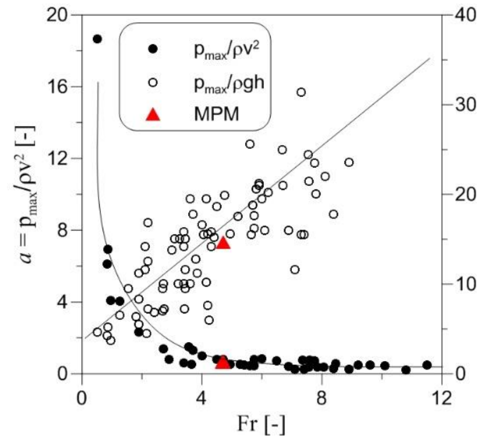

Figure 7 Maximum impact pressure as function of Froude-number; comparison between MPM results and other experimental results collected by Hübl et al. (2009)

its physical meaning in this context differs from what is commonly understood for quasi-static conditions.

Decreasing the initial porosity, i.e. increasing the density, the peak force increases proportionally. The Poisson's ration does not seem to have a significant effect on the peak force.

4 CONCLUSIONS

This paper shows that the MPM is a valuable tool for studying the interaction between flow-like landslides and existing structures. The evolution of force, bending moment and pressure distribution can be studied, which is fundamental for the stability analyses of protecting measures as well as for the assessment of the potential damage to existing structures.

Preliminary analyses showed that the pressure distribution deviates significantly from the linear or constant distributions commonly assumed in practice. The maximum force and the maximum moment do not occur at the same time because the pressure distribution resulting in the peak force does not coincide with the one giving the peak moment; this should be taken into account in structure design.

Additionally, it is shown that one of the most important parameters for the peak force is the basal friction coefficient, while the friction angle and the elastic parameters have a minor effect.

The peak force obtained in this study is overestimated with respect to the measurements of Moriguchi et al. (2009) on similar configurations. This can be attributed to the constitutive model applied in this preliminary study.

The soil behavior was simulated with an elasto-plastic model, which entails the assumption that energy dissipation occurs entirely by frictional contacts between sand grains like in quasi-static conditions. However, a certain amount of energy is also dissipated by particle collisions. Numerous attempts have been mate to incorporate both frictional and collisional contribute in a constitutive model, see e.g. GDR Midi (2004); Kamrin (2010); Forterre & Pouliquen (2008); Redaelli et al. (2015), but a satisfactory solution of the problem has not been found yet. Indeed the problem of movement of granular masses and interaction with structure is very complex, and it is not fully understood up to date. Future development of the research will consider alternative constitutive models, which can better capture the response of the granular material under both quasi-static and collisional state.

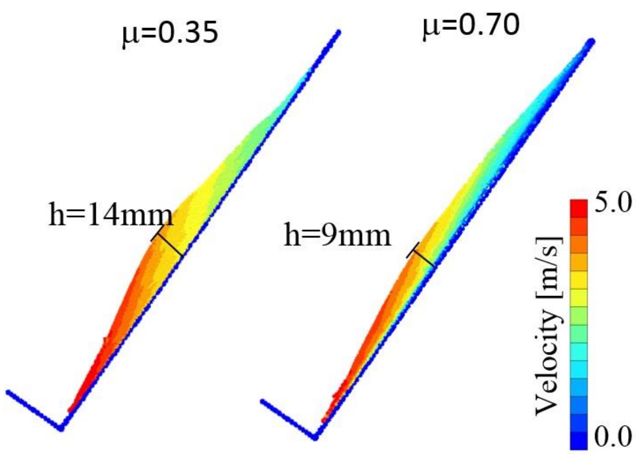

Figure 8 Effect of basal friction coefficient on the flow thickness.

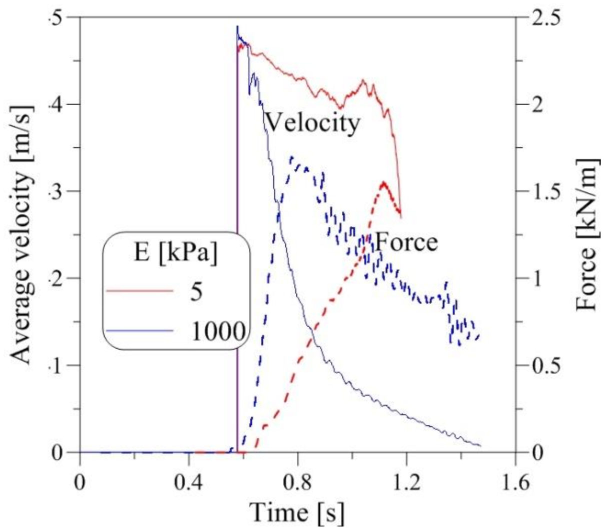

Figure 9 Effect of Young modulus on evolution of impact force and average velocity in the control zone.

Table 2 Summary of parametric analyses

ϕ	μ	n	E	ν	h	v	F_{max}
deg	-	-	kPa	-	m	m/s	kN/m
33	0.64	0.48	50	0.2	0.10	4.76	1.67
35	0.64	0.64	50	0.2	0.10	4.70	1.60
38	0.64	0.64	50	0.2	0.10	4.70	1.64
40	0.64	0.64	50	0.2	0.11	4.78	1.63
35	0.7	0.64	50	0.2	0.09	4.70	1.52
35	0.5	0.64	50	0.2	0.12	5.07	2.19
35	0.35	0.64	50	0.2	0.14	5.12	3.51
35	0.64	0.492	50	0.2	0.11	4.71	1.66
35	0.64	0.38	50	0.2	0.11	4.70	2.17
35	0.64	0.64	1000	0.2	0.11	4.90	1.84
35	0.64	0.64	5	0.2	0.10	4.70	1.40
35	0.64	0.64	50	0	0.10	4.77	1.84
35	0.64	0.48	50	0.35	0.11	4.77	1.62

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