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Modelling earthquake-triggered landslide runout with the Material Point Method

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Landslides triggered by earthquakes cause devastating consequences to downstream infrastructure. The simulation and prediction of these large-strain events remain challenging. The objectives of this paper are i) to validate the Material Point Method (MPM) framework for the study of coseismic landslides, and ii) to compare the capabilities of MPM with mesh-based methods and simplified Newmark-type methods to simulate post-failure runouts. To achieve these objectives, the MPM framework is presented whereby nodal kinematic boundary condition is employed with a moving mesh. Secondly, the framework is validated with a shaking-table laboratory test of a saturated clay slope. Thirdly, a parametric analysis is conducted using 25 real ground motions on a simple theoretical slope. The MPM results are compared to those obtained with mesh-based methods and three state-of-the-art Newmark-type approaches. It is concluded that mesh-based methods are consistent with MPM predictions for small-strain instabilities associated with low energy ground motions (i.e. Arias intensity lower than 4 m/s). When using ground motions with energy above this threshold, mesh-based methods accumulate significant errors associated with bad geometry. MPM results consistently matched permanent displacements predicted with the Newmark-type methods employed in this analysis.

1. Introduction

Coseismic landslides are considered as one of the most catastrophic large-strain manifestations of earthquakes. A recent example of such problem was in January 2020, whereby an earthquake in Puerto Rico (M_w 6.4) triggered more than 120 landslides, and mobilised soil mass up to 40 km from the initial epicenter (NASA, 2020). Consequently, the lives of 7,500 residents were disrupted due to these events (USGS, 2020). Despite the socio-economic impacts associated with coseismic landslides, it is still challenging to predict the relationship

between the triggering factors (e.g., intensity measures of the earthquake) and the final runout of these events. The mechanisms controlling the kinematics of these failures involve soil non-linearity, large deformation, and multiphase interaction, all in one dynamic framework.

Traditional seismic slope stability analyses include several methods. In the pseudostatic limit equilibrium analysis (Terzaghi, 1950), the slope stability is crudely represented as a factor of safety, which is only valid when there is low crest accelerations and less than 15% soil strength degradation due to pore pressure build up (Seed, 1979). The rigid sliding block analytical solution was proposed by Newmark (1965) to predict permanent displacements resulting from seismic loading. Newmark (1965) assumes that soil behaves in a rigid-perfectly plastic manner across a well-defined slip surface, with no strength loss due to shaking. The block exhibits relative sliding episodes when the factor of safety goes below unity, and accumulates permanent displacement as it moves downstream. This model provides a simple framework with only two input parameters, namely yield acceleration, a_y as proxy for its threshold dynamic resistance, and the acceleration-time history of the basal ground. Practitioners often employ the pseudostatic limit equilibrium analysis to quantify the value of a_y. However, this ultimately restricts the scope of Newmark (1965) method to simple constitutive behaviours and geometries. Many researchers modified the Newmark (1965) approach to include internal volumetric straining as a mechanisms of slope deformation. Makdisi and Seed (1978) developed charts to semiempirically estimate Newmark-type permanent displacements considering one-dimensional acceleration amplification from the slope base to crest using the Finite Element Method (FEM), producing the so-called "decoupled approaches". Further subsequent advancements such as the "fully coupled approaches" were developed to accomplish a simultaneous analysis of deformations caused by the dynamic response and the sliding response (Bray, 2007). Newmark-type sliding block methods can be simplified to data-driven procedures where the results are correlated to the slope's dynamic characteristics and ground motions parameters to develop generalised semiempirical equations known as the "simplified Newmark-type methods" to estimate permanent displacements.

Advancements in understanding the large-strain mechanics of coseismic landslides often resort to controlled shaking-table experiments or centrifuge models (e.g., Wartman et al. (2005)). However, these laboratory facilities are extremely expensive and challenging. For this reason, the validation of numerical methods capable of modelling large strains, multiphase problems, advanced constitutive behavior, and time-dependent boundary conditions play an important role for evaluating shaking effects on the triggering and post-failure deformations of coseismic landslides. Mesh-based formulations, such as FEM or Finite Difference Method (FDM), are among the most commonly used tools for conducting dynamic numerical analyses (e.g., Griffiths and Lane (1999)). They use a Lagrangian formulation whereby the continuum is discretised into a computational mesh that deforms to represent the material behaviour. At large strains, element distortion leads to extreme error accumulation leading to mesh tangling. Efforts to extend these techniques to large-strain problems include rezoning, remeshing, and remapping algorithms. However, these algorithms are computationally expensive and interpolate the history variables at the nodes, which ultimately reduces the accuracy of the stress calculation (Zhang et al., 2016). To overcome the distortion of the computational mesh, particle-based numerical tools are used for the modelling of landslides, such as the Material Point Method (MPM) and the Smoothed Particle Hydrodynamics (SPH). In the MPM computational cycle, the state variables are mapped back and forth between the material points and the nodes which significantly increases the computational cost compared to traditional mesh-based approaches (e.g., FEM, FDM). Numerical oscillations may arise in the MPM computation due to cell-crossing error (Bardenhagen and Kober, 2004). Expensive computational cost is also seen with SPH as a result of the use of particle search functions. SPH suffers from difficulties when defining boundary conditions and numerical instabilities when insufficient neighboring particles are found (Zhang et al., 2016). This research focuses on MPM; validation is necessary to build confidence in the MPM as an emerging tool for the analysis of coseismic landslides.

The objectives of this study are i) to present and validate the MPM framework for the modelling of earthquake-induced shaking using a time-dependent boundary condition in a moving mesh approach; and ii) to evaluate the performance of MPM in comparison with other existing approaches for the prediction of runout such as mesh-based methods (i.e. FEM and FDM) and simplified Newmark-type methods. To achieve these goals, the paper is organized as follows. First, the MPM numerical framework is introduced with particular emphasis on

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advancements to model earthquake-induced shaking. Secondly, the capabilities of the MPM in modelling large-strain coseismic landslides are validated against a laboratory test (Wartman et al., 2005) whereby a saturated clay slope was tested on a shaking table apparatus. The results and sensitivity of the MPM to mesh refinement is discussed. Thirdly, parametric analysis is performed on a theoretical slope geometry using 25 real strong-motions records from the Pacific Earthquake Engineering Research (PEER) center database. The MPM results are compared with predictions from the rigid sliding block method, simplified Newmark method, and the mesh-based methods (FEM and FDM). The limitations of the mesh-based numerical methods are demonstrated in the context of error accumulation due to large-strain runout.

2. The Material Point Method

MPM is a hybrid Lagrangian-Eulerian numerical procedure capable of modelling large-strain deformations in history-dependant materials. The spatial discretisation consists of an Eulerian discretisation of the entire domain using a background mesh and a Lagrangian discretisation of the material domain (i.e., continuum body) using a set of integration points. Each point, so-called a Material Point (MP), moves attached to the material carrying state variables including the mass of the represented subdomain. Limitations of the MPM include noise due to cell-crossing instabilities, particularly when using lower-order shape functions (Bardenhagen and Kober, 2004). Higher order and B-spline shape functions provide continuity of gradients of the shape functions which eliminate cell-crossing error. However, the construction of the mass matrix and force vectors become more challenging. Negative values of the shape functions in the element can lead to a singular mass matrix (Tielen, 2016). In addition, higher order shape functions are computationally expensive when considering high number of degrees of freedom. Trade off between the overall stability and the computational cost limit the application of higher order shape functions in MPM. Noise reduction techniques include the use of mixed MPM-Gauss-Point integration, which allows the computation of an averaged incremental stress using optimally positioned Gauss points (Al-Kafaji, 2013). In this paper, the mixed MPM-Gauss-Point integration scheme is adopted to compute the nodal internal forces, but the constitutive equation is evaluated at each individual MP to retain the history of the MP state variables. Least squares volumetric strain-smoothing algorithm is also employed, as implemented in previous works (e.g.,Yerro (2015b)), to overcome element locking and prevent over-stiffening effects in undrained incompressible material modelling. Further details of the MPM basis, formulation and implementation is presented in Fern et al. (2019) as employed in the Anura3D software. Recently, MPM has demonstrated promising capabilities for the simulation of landslides, including advanced hydro-mechanical coupling (Al-Kafaji, 2013; Yerro et al., 2015; Bandara et al., 2016). Considering an explicit time integration scheme, the MPM computational cycle, can be summarized as follows (Sulsky et al., 1995; Fern et al., 2019):

- 1: Interpolate state variables from MPs to nodes using shape functions to calculate nodal mass, nodal external forces, and nodal internal forces.
- 2: Solve discretised linear momentum governing equation to compute nodal acceleration.
- 3: Compute MP velocity using the explicit time integration scheme.
- 4: Compute MP and nodal linear momentum. Subsequently, evaluate the nodal velocity.
- 5: Compute incremental MP strain rate using the nodal velocity.
- 6: Solve constitutive equation at the MP, and update stresses and state variables.
- 7: Update MP displacement and discard nodal variables to reinitiate a non-distorted mesh for the next computational cycle.

2.1. Developments of MPM for the modelling of coseismic landslides

Earthquake loading in MPM can be prescribed as velocity, displacement, or stress-time history. Although the specification of input ground motion using stress-time history could be attractive for use with viscous boundary conditions, Green et al. (2008) found that the use of stress-time input is least accurate when comparing the nonlinear numerical dynamic numerical response of a viscoelastic column with its equivalent linear solution. Since displacement-time histories are less commonly available for real ground motions, this study specifies the input ground motion in the MPM as a velocity-time history.

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Figure 1. MPM computational cycle using the time-dependent kinematic nodal prescribed velocity boundary condition in a moving mesh approach: (a) Initial configuration whereby the prescribed velocity is applied on the nodes; (b) information is mapped from MPs to nodes; (c) velocity is overwritten on the nodal solution of the momentum balance equation; (d) velocity of the MPs is corrected to be consistent with the nodal solution; (e) final deformed configuration.

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Few attempts exist in the literature that model earthquake shaking with MPM (Bhandari et al., 2016; M. He at al., 2019; Ering and Babu, 2020). All of them apply a time-dependent kinematic velocity boundary condition on the MPs. This approach requires the definition of a 'basal' material (e.g., shaking table box or bedrock frame) that moves within the mesh according to a prescribed velocity-time history. As such, the basal MPs move back-and-forth within the computational domain. In these boundary conditions, when the conventional MPM approach (i.e., fixed background mesh) is employed, the nature of the shaking increases the cell-crossing (i.e., MPs crossing element boundaries) as most of the MPs move back and forth with respect to the mesh. This may lead to numerical instabilities cell-crossing instability is not prevented. Very recently, the authors of this paper, proposed a more computationally efficient non-zero kinematic velocity boundary condition to overcome some of these drawbacks in Alsardi and Yerro (2021). This boundary condition applies the prescribed velocity on the boundary nodes of the background mesh of the MPM model. A moving mesh technique (Al-Kafaji, 2013) is also incorporated to reduce the cell-crossing to only those MPs that move with respect to the seismic motion (i.e., MPs in the sliding mass). In this manner, the computational mesh moves following the input velocity-time history of the ground motion. Therefore, the noise possibly generated from cell-crossing in the coseismic landslide simulation is considered comparable to the one generated in the accepted MPM models for non-seismically triggered landslides (e.g., Soga et al. (2016)). Additionally, this approach provides more flexibility in the geometry of the problem, as the definition of the basal MPs is not mandatory. The work presented here is implemented in an internal version of the Anura3D software (http://www.anura3d.com) within the framework of the one-phase formulation.

Figure 1 summarizes the implementation of the nodal kinematic boundary condition within the MPM computational cycle Alsardi and Yerro (2021). The prescribed nodal velocity is applied along the boundary domain Ω_u of the mesh (Figure 1a). Consistent with the presented computational cycle, step 1 to 4 are identical whereby the information is mapped to the nodes and thereof solved to obtain the nodal velocity, v_i, as shown in Figure 1b. To specify the shaking criteria in the MPM system along the boundaries of the mesh, the nodal velocity solution is overwritten with the velocity-time history, v^{input}, at the beginning of each computational cycle, as shown in Figure 1c. The MP velocity is updated using shape functions particularly for those MPs in the elements which contain a prescribed velocity node to ensure solution consistency in both Lagrangian and Eulerian domains (Figure 1d). Finally, as opposed to discarding the nodal displacement in step 7, the mesh is incrementally rigidly moved by Δu^{mesh}, following the ground motion input and ensuring that the MPs remain within the computational domain (Figure 1e).

Figure 2. Plane-strain MPM model of the shaking table experiment by Wartman et al. (2005)

Figure 3. Results from processing the acceleration-time history used in Wartman et al. (2005): (a) velocity-time history. (b) displacement-time history.

Table 1. Parameters of the synthetic Kaolinite-Bentonite clay material used in the MPM model

Parameter	Soft Clay	Stiff Clay
Saturated unit weight, γ [kN/m^3]	13.6	13.6
Peak undrained strength, $S_{u,peak}$ [kPa]	2.68	5.90
Residual undrained strength, $S_{u,resd}$ [kPa]	1.77	4.03
Undrained poisson ratio, ν_u [-]	0.485	0.485
Undrained small-strain stiffness, E_u [kPa]	615	4190

3. Numerical validation using shaking table test on model-scale clay slope

The objective of this section is to validate the MPM framework for earthquake-induced motion using experimental data from a shaking table test on a clay slope conducted by Wartman et al. (2005) at the University of California, Berkeley. This section focuses on modelling the translational/rotational deformation mechanism observed in the largest slope model built in the testing program by Wartman et al. (2005).

3.1. Physical testing

The physical testing program by Wartman et al. (2005) involved building, 30 cm high, saturated slopes made from synthetic clay mixture, placing each slope in a rigid box container, and shaking them by means of a shaking table apparatus. The geometry and strength parameters used to construct the slope were not based on a real prototype, albeit Wartman et al. (2005) used approximate model scaling for mimicking a tall natural embankment. The cross-section of the considered slope used is depicted in Figure 2, which was comprised of a soft clay deposited over a stiff clay base. Both materials were made from a synthetic mixture of identical mineralogy, consisting of 75% kaolinite and 25% bentonite by dry weight, with different initial water content to control their strength and stiffness. Potentiometers, labelled sensor 11 and 12 in Figure 2, were used to measure the displacements across the surface of the slope during the shaking table test. The shaking table box (highlighted in red in Figure 2), made from rigid fully-reflective Plexiglass, plays no effect on the experimental results as it is not in direct contact with the soft clay. The undrained strength of the soft clay was characterized using a miniature vane shear test to an average confining depth of 18 cm (Wartman et al., 2005). The mobilised undrained shear strength for a reference sample was presented against peripheral displacement along the circumference, as shown in Figure 4. The in-situ small-strain stiffness was calculated by measuring the shear wave velocity from striking a hammer on the constructed clay slope. The acceleration response of the slope was also analysed using the shaking table prior to the actual test, whereby gentle sweep tests were applied using low-amplitude ground motion without causing permanent deformation. The clay surface was marked and surveyed before and after the shaking table test to measure the final displacement caused by the failure mechanism of the slope.

The behaviour of the synthetic clay was seen to be complex, including strain-softening behaviour, moderate strain-rate effects, and shear modulus degradation (Biscontin and Pestana, 2001; Wartman et al., 2005). Since the shear strain rates were more than 100 times larger than that of the hammer blow tests, the measured shear wave velocities were required to be corrected by calibrating the simulated and recorded acceleration response spectra from the sweep test (Travasarou, 2003). Consistently, Chen and Qiu (2012) performed nonlinear dynamic analysis of the same slope with the SPH method. They showed that the shear wave velocity required to reproduce the slope response during the sweep test needs to be increased with respect to those measured from the hammer strike by approximately 65% and 80%, for soft clay and stiff clay, respectively.

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3.2. Ground motion correction scheme

The strong ground motion input used to shake this slope in the experiment was created synthetically to cover a frequency content ranging from 1 Hz to 20 Hz, with PGA of 3.46g. In the present work and as recommended by Wartman et al. (2005), a linear baseline trend correction is applied to remove the drift in the acceleration-time record. Additionally, high-pass filtering at 0.5 Hz and low-pass filtering at 60 Hz, is considered to eliminate significant long period noise contamination. The acceleration-time history filtering is conducted using Obspy, an open-source Python package by Beyreuther et al. (2010). The filter is applied in the forward and backward direction to prevent any phase-shifts. The acceleration-time history is numerically integrated, using the trapezium rule, to obtain the velocity-time history (Figure 3a) for use as an input in the kinematic boundary condition in the MPM. The velocity-time history is integrated in a similar way to obtain the displacement-time history (Figure 3b), which confirms the elimination of long period noise and baseline drift, while ensuring negligible amplitude difference of less than 10% when comparing with the displacement-time history without the correction scheme.

3.3. Constitutive model

In this paper, the strength of the synthetic clay is modelled by means of an associated Tresca constitutive model that incorporates an additional strain-softening law based on (Yerro, 2015b) to account for the material brittleness. Strain-rate effects and stiffness degradation are not modelled here for simplification. The employed softening law (Equation 1) reduces the undrained shear strength, S_u, exponentially as a function of the accumulated equivalent plastic strain, ϵ^p_{eq}. The peak undrained shear strength, $S_{u,peak}$, and the residual shear strength, $S_{u,resd}$ are input properties. Also, the shape factor, η, is required to calibrate the exponential functional form of this softening law against experimental data. Large values of η correspond to greater strength decrease with plastic strain accumulation (i.e., more brittle nature).

Table 1 presents a summary of the total stress material parameters based on Wartman et al. (2005), with exception to the undrained small-strain stiffness, E_u, which is based on the aforementioned calibration performed by Chen and Qiu (2012).

$$S_u = S_{u,resd} + (S_{u,peak} - S_{u,resd})e^{-\eta\epsilon^p_{eq}} \quad (1)$$

Besides the basic mesh h-type refinement associated with reducing the element size to ensure better failure localisation, it is known that the use of a softening law in continuum-based constitutive models adds extra complexity (Oliver and Huespe, 2004; Soga et al., 2016). The energy dissipation across each element in the MPM is dependant on the mesh size and a mesh convergence analysis is required to calibrate η with laboratory testing. The goal is to ensure an equivalent dissipated energy between the numerical model and the physical experiment. Yerro (2015b) proposed to use the 'smeared-crack' fracture approach defined by (Oliver and Huespe, 2004) to calibrate the factor η. Following Yerro (2015b), the strength mobilised in strain-controlled single-element tests needs to be calibrated (i.e. using different values of η) to be consistent with the one observed in the vane shear laboratory test (Wartman et al., 2005). Wroth (1984) showed that the stress path of the soil in a vane shear test is partly mechanically equivalent to a reoriented direct simple shear test (DSS) such that the vertical and horizontal confining stresses of a soil element near the shear blade are reversed. Inspired by Morris and Williams (1993); Wroth (1984), two-dimensional (2D) plane-strain DSS are modelled using a single row of elements. Assuming K_0=0.5 at an average blade depth of 18 cm, the in-situ total vertical stress is applied to confine the elements horizontally, and the in-situ lateral total stress is applied on the simulation vertically. Prescribed velocity of 12 cm/min, consistent with the circumferential speed of vane shear test, is applied at the top of the row of elements to generate the DSS total stress path. The results from the vane shear test presented in Wartman et al. (2005) from a representative clay sample are used here for reference. The value of η is varied by increments of 0.05 until the area under the stress-displacement graph is approximately the same for the MPM simulation and the vane shear laboratory test, to ensure an equivalent energy dissipation in all the simulations. The calibration was conducted for element length, h, of values associated with a coarse-medium (3 cm), medium (2.3 cm) and fine mesh (1.6 cm) as shown in Figure 4.

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3.4. Numerical model and calculation stages

The MPM model consists of a plane-strain analysis. Triangular elements with first order (linear) shape functions are considered in the MPM model. The initial configuration is depicted in 2. Each element of the soft clay material is initiated with the minimum number of MPs to prevent sporadic empty elements at large strains. Six MPs per element is determined to be the minimum for the soft clay material. One MP per element is sufficient for the elements of the stiff clay as its MPs are close to the boundary and do not exhibit cell-crossing. First, the stresses due to gravity are initiated in quasi-static conditions, assuming a fully fixed base and rollers in the vertical boundaries. Then, horizontal fixities are removed and the shaking is applied on the boundary nodes (highlighted in red in Figure 2) with the velocity-time history shown in Figure 3a to represent the rigid, fully-reflective shaking table box. Artificial local damping of 5% is applied to the computational system during the shaking stage. The background mesh is moved in a rigid manner with the box to minimize cell-crossing noise. The MPs at locations corresponding to sensors 11 and 12 are tracked to plot the displacement-time history for quantitative validation with the physical experiment.

3.5. Results and discussion

The relationship of the strain-softening calibration with the employed mesh refinement is highlighted in Figure 5, that shows the effect of mesh size, h, in the results. It is observed that the use of a single value of η produces inconsistent displacement values at sensor 11, whereby finer meshes result in larger displacements. Contrarily, after calibrating η for each value of h, the accumulated total displacement is consistent for all mesh sizes. This highlights the importance of calibrating the constitutive model at a single-element level, which is of paramount importance in ensuring the consistency of the results. The final geometry of the slope is shown for different mesh

Figure 4. Calibration of the strain-softening exponential fitting factor, η

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refinements in Figure 6. Although the shear band thickness was observed to decrease with smaller element size, this qualitative comparison of the displacement profile shows that the mesh dependency is reduced when the value of η of the constitutive model is calibrated. In this conditions, the computational cost increases with mesh refinement (as shown in CPU times in Figure 5). Similar results were also reported by Soga et al. (2016) using the MPM for modelling strain-softening progressive failure in natural scale slopes. To optimise the computational cost, the coarse-medium mesh size (h=3.0 cm) with its calibrated shape factor (η=4.35) is sufficient for practical purposes. However, in this paper, the model with fine mesh is presented for better visualisation of the slope profile.

Figure 7a and 7b depicts the displacement (magnitude) and final deviatoric strain profiles of the slope, respectively, and compares them with the outline of the final experimental profile. It is clear that the interface between the soft and stiff clay plays a major role in the development of the shear band, whereby the soft clay develops a translational/rotational mechanism at this interface. The deformed MPM profile is in acceptable match with the experimental profile, particularly at the toe and the upper surface of the embankment, whereby a maximum displacement magnitude of 12 cm is observed (Figure 7a). The experimental runout value was approximated by Wartman et al. (2005) to be 12-13 cm which is consistent with the MPM results. It is noted that the numerical profile at the crest scarp does not match the physical experiment. This is due to the continuum-scale nature of the MPM model which prevents free-separation between the MPs and requires advanced constitutive contact algorithms to reproduce the scarp transition. As such, the vertical settlement observed in MPM near the slope scarp (i.e., 1-1.3 m from the front of model container) is underestimated as shown in Figure 7c. In Figure 7, the MPM results are also compared with those presented by Chen and Qiu (2012), who simulated the same slope using SPH employing an advanced Drucker-Prager constitutive model. Despite that the SPH model incorporates cyclic shear modulus degradation and strain rate-dependent strength, the SPH results are almost identical to the MPM model. Similar to MPM, the SPH model shows inconsistency at the scarp due to its continuum approach.

Finally, Figure 8 presents the horizontal displacement of sensors 11 and 12, located near the toe and the crest, respectively (Figure 2). An acceptable match between the MPM results and the experimental data is observed. This includes movement initiation at 11 seconds, which corresponds to the beginning of the strong motion shaking, and the subsequent general displacement trend. Note that the experimental runout results were cut short at 25 seconds due to the failure of the potentiometer sensor at large-strain. From these results, it is arguable that the strain-softening behaviour dominates in capturing the failure mechanism and the runout of the slope. Further investigation of the advanced constitutive model can provide insights into the compensating nature of the strain-rate strength increase and the shear modulus degradation on cumulative runout.

Figure 5. Final runout (t = 30 s) mesh sensitivity analysis of the MPM model to the strain-softening constitutive model calibration.

Figure 6. MPM displacement profile at t = 30 s for different mesh sizes using the calibrated constitutive factor, η.

4. Comparative overview of the MPM against mesh-based and Newmark-type methods

The objective of this section is to investigate the capabilities of the MPM to model coseismic slope failure runouts compared to other available methods. In particular, two mesh-based methods (i.e., FEM and FDM) and three Newmark-based sliding block analyses are considered. A parametric analysis on a model-scale theoretical slope, in a shaking table set-up, is conducted using 25 ground motions. The ground motions correspond to 14 different earthquakes from the PEER strong motion database (Bozorgnia et al., 2014) as summarised in Table 2, covering a range of moment magnitudes, M_w from 6.0 to 8.0. Arias intensity, I_a, is computed for each ground motion as defined in Equation (2). The value of I_a provides an intensity measure of the energy delivered by the input ground motion as a function of its acceleration, a_t, across the overall duration, d, whereby g is the gravitational acceleration. This I_a intensity measure was selected as it holistically captures the amplitude, duration, and frequency characteristics of the input motion employed in the analysis (Arias, 1970). For each station, the horizontal ground motion component that generated higher I_a was considered in the analysis. The relationship between the slope toe runout and I_a is investigated.

$$I_a = \frac{\pi}{2g}\int_0^d {a_t}^2 dt \tag{2}$$

Figure 7. Experimental and numerical deformed profile of the slope; (a) displacement magnitude, (b) deviatoric strain, (c) final vertical displacement of the ground surface (t = 30 s).

Figure 8. Horizontal displacement recorded at the toe (sensor 11) and top (sensor 12) of the slope. Comparison between MPM and experimental data recordings.

The geometry of the homogeneous theoretical slope used in the parametric analysis is schematically depicted in Figure 9. The shaking boundary conditions (highlighted in red) correspond to a rigid box in a similar shaking table setup to the one discussed in the previous section. The unit weight, γ, of the soil was selected as 17 kN/m^3. A non-associated Mohr-Coulomb constitutive model is employed to model the strength of a clayey sand soil with the following parameters: friction angle, ϕ'=25°; dilatancy angle, ψ=0°; cohesion, c'=1 kPa; Young's modulus, E'=3000 kPa; Poisson's ratio, ν'=0.33. Another important dynamic parameter is the fundamental period of the slope, T_n, which is a measure of the slope's rigidity as a function of the geometry and shear wave velocity. Equation (3) is employed to obtain the fundamental period of the adopted model-scale theoretical slope, consistent with Bray (2007). This is considered as an acceptable approximation of the slope period when the potential sliding mass of crustal homogeneous slope is expected to be approximately shaped as a segment of a circle and the acceleration response throughout the depth of the model is approximated to be one-dimensional (Bray, 2007; Rathje and Bray, 2001). The adopted model-scale theoretical slope has T_n=0.13 s, as calculated using Equation (3), whereby H is the vertical height of the slope and V_s is the shear wave velocity as computed from (4). The calculated value of T_n is used to guide whether the slope is characterised as 'rigid' or 'flexible'. For this study, a threshold value of T_n=0.2 s is adopted as the limit of rigid behaviour, consistent with Fotopoulou and Pitilakis (2015). Thus, the slope analysed here can be characterised as rigid.

$$T_n = \frac{4H}{V_s} \tag{3}$$

$$V_s = \sqrt{\frac{E'}{\frac{\gamma}{g}2(1+\nu')}} \tag{4}$$

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4.1. Parametric analysis

The numerical analysis was conducted using FEM, FDM, and MPM. Details regarding the numerical implementation are summarized in Table 3. Note that the numerical damping schemes for each method were default, as recommended by the respective software developers to ensure stability and smoothness of the dynamic solution according to the employed time integration scheme. The initial configurations of the MPM, FEM, and FDM numerical models are presented in Figures 10a, 10b and 10c, respectively. Mesh size (h-type) convergence analysis was conducted for each model, and the largest mesh size, whereby convergence was achieved, was subsequently selected for further investigation in the parametric analysis. The calculation stages in FEM, FDM, and MPM are identical to those considered in the previous example. The MP at the toe was tracked for investigating the displacement-time history kinematics using MPM. Similarly, the node at the toe is tracked for the FEM and FDM analyses.

The state-of-the-art engineering practice relies on empirical simplified regressions based on the Newmark (1965) framework. Taking into account that the slope is characterized as rigid, the Newmark (1965) rigid sliding block method and two other simplified rigid Newmark-type methods (i.e., Jibson (2007); Hsieh and Lee (2011)) are selected for comparison with MPM. The Jibson (2007) model in Equation (5) was developed from an exhaustive regression of 875 sliding rigid-block displacements for yield pseudostatic coefficient, k_y, values between 0.2 and 0.8, and recommended for slopes that exhibit shallow landslides and rigid behavior. Similarly, the Hsieh and Lee (2011) model in Equation (6) corresponds to a global ground motion data set (form 2) correlation. Both semiempirical models consider the displacements D in cm and I_a in m/s. Note that other well-known decoupled and coupled simplified Newmark-type methods (e.g., Makdisi and Seed (1978); Bray and Travasarou (2007)) were developed for large-scale geometries and high V_s values to account for the one-dimensional ground response, rendering their model assumptions inconsistent for comparison with the theoretical slope adopted here. The Newmark (1965) rigid sliding block method was implemented considering only the positive side of the accelerations for integration, so up-slope

displacements are neglected. Using this approach, earthquake-induced deviatoric permanent displacements are computed at a predefined slip surface. To determine k_y, pseudostatic analyses using the Spencer (1967) limit equilibrium framework were performed to identify the failure surface with yield pseudostatic coefficient k_y that provides a factor of safety equal to one, resulting in k_y=0.20.

$$\log(D) = 2.401\log(I_a) - 3.481\log(k_y) - 3.230 \tag{5}$$

$$\log(D) = 0.847\log(I_a) - 10.62k_y + 6.587k_y\log(I_a) + 1.84 \tag{6}$$

4.2. Parametric analysis results and discussion

The effects of strong ground motions from earthquakes on slopes vary from minimal displacement to failure. For comparison, two representative ground motion cases with I_a values equal to 2.5x10^{-}3 and 11.8 m/s are analysed to assess the numerical models' performance at small-strain and large-strain, respectively. Figure 11a shows that shaking with low I_a induces small strains and the displacement-time history obtained with FEM, FDM, and MPM match perfectly. All methods, including the Newmark (1965) rigid sliding block method, consistently predict zero permanent displacement. For large-strain runouts (Figure 11b), the FDM simulation crashes at 6 second due to bad element geometry at the toe. Similarly, the mesh of the FEM simulation accumulates error, entangles, and the mobilised mass interpenetrates the base platform downstream at 11 seconds. The erroneous behavior of the mesh-based models at large-strain are depicted for the FEM and FDM models in Figure 10e and Figure 10f, respectively. By contrast, the MPM successfully models the entire slope instability process until stabilization at 33 seconds with final deformed profile shown in Figure 10d. The toe displacement predicted with MPM is consistent with the permanent displacement calculated using Newmark (1965). Note that while the displacement-time history trends obtained with FEM, FDM and MPM are sensitive to the direction of the dynamic load, the Newmark approach predicts a monotonic increase of the permanent displacement because only the downward slide motion is accounted. Figure 12 presents the final runouts (i.e., horizontal displacement recorded at the toe) obtained with the different methodologies correlated with the I_a from the 25 ground motions. For reference, a quadratic trend of the MPM results is also included. Figure 12a compares the results obtained with the numerical simulations (FEM, FDM, and MPM). Note that only successful simulations are plotted in this figure (i.e., no entanglement and no crashing). The capabilities of the mesh-based and particle-based methods are similar only for small-strain slope instabilities (0< I_a <4 m/s). For large-strain instabilities associated with high energy ground motions (I_a >4 m/s), the particle-based MPM is necessary to model the entire slope instability mechanism. Figure 12b shows the comparison between the MPM runout results and the permanent displacement predicted by the sliding block analyses (Newmark, 1965; Jibson, 2007; Hsieh and Lee, 2011). In general, the Jibson (2007) and Hsieh and Lee (2011) simplified methods are in good match, exhibiting less scatter than the Newmark (1965) rigid sliding block solution. A threshold value of I_a =4 m/s is observed. Below that threshold, the Newmark-type methods slightly underpredict the permanent displacement with respect to the MPM. Conversely, they estimate higher displacements than the MPM above that threshold. Since the permanent displacement outputs of the simplified Newmark-type methods are merely seen as an order of magnitude indices, the general variation of the runout with the I_a value of the input groundmotion in Figure 12b is deemed consistent with the MPM. This parametric analysis successfully confirms that the advancements in the large-strain numerical framework of the MPM are consistent with state-of-the-art simplified Newmark-type methods presented in this paper.

5. Conclusions

The MPM formulation was outlined and discussed for earthquake-induced shaking, employing a nodal time-dependent boundary condition in a moving mesh approach. Subsequently, a slope failure on a shaking table based on Wartman et al. (2005) was successfully modelled

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Figure 9. Slope geometry with shaking boundary conditions (highlighted in red)

Figure 10. Initial configurations of the numerical models as part of the parametric analysis, and the deformed profiles associated with the Tabas earthquake, Iran (I_a=11.8 m/s). (a) MPM, (b) FEM, (c) FDM.

using MPM, and a Tresca with strain-softening constitutive model. The numerical results in terms of the slope displacement and runout match well the physical experiment. Mesh-dependency effects were discussed, highlighting the importance of constitutive model calibration on the runout result of the slope. To understand the capabilities and limitations of the mesh-based, particle-based, and simplified Newmark approaches, a parametric analysis was conducted for a theoretical 'rigid' slope in a shaking table setup, using 25 real ground motions from the PEER database. The parametric analysis highlighted the limited capabilities of the mesh-based approaches at large-strain, whereby a maximum Arias intentsity of 4 m/s was identified as the energy threshold limit of the input ground motion in mesh-based models of the slope. Good match was seen between MPM results and selected state-of-the-art simplified Newmark-type methods. It is concluded that the validated MPM framework offers a promising numerical tool for modelling coseismic landslides and has good potential to simulate more complex large-strain scenarios.

Figure 11. Toe runout at (a) low Arias intensity (i.e., I_a=2.5x10^{-3} m/s); (b) high Arias intensity (i.e., I_a=11.8 m/s)

Figure 12. Permanent displacements obtained from 25 ground motions using (a) MPM, FEM, FDM and (b) MPM and selected Newmark-type sliding block methods.

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Table 2. Summary of the ground motions.

Event	Date	Region	M_w	Number of records
Morgan Hill	April 24, 1984	California	6.19	1
Whittier Narrows	October 1, 1987	California	5.99	1
Palm Springs	July 8, 1986	California	6.06	1
Northridge	January 17, 1994	California	6.69	1
Chalfant Valley	July 21, 1986	California	6.19	1
Landers	June 28, 1992	California	7.28	2
San Fernando	February 02, 1971	Califronia	6.61	3
Cape Mendocino	April 25, 1992	California	7.01	4
Mammoth lakes	May 25, 1980	California	6.06	1
Parkfield	September 28, 2004	California	6.19	2
Helena	October 18, 1935	California	6.00	1
Kobe	January 17, 1995	Japan	6.90	1
Izmit	August 17, 1999	Turkey	7.51	1
Tabas	September 06, 1978	Iran	7.35	5

Table 3. Summary of the numerical analyses conducted in the parametric investigation.

Numerical method	Material Point Method (MPM)	Finite Element Method (FEM)	Finite Difference Method (FDM)
Software	Anura3D (plane-strain)	Plaxis2D (plane-strain)	Flac2D (plane-strain)
Elements	3-noded triangular element with 3 material points	15-noded triangular element with 12 gauss points	4-noded quadilateral element
Time integration scheme	Explicit Forward-Euler	Implicit Newmark Beta	Explicit Forward-Euler
Numerical damping scheme	5% artificial local damping	10% Newmark Beta damping ($\alpha = 0.3025$, $\beta = 0.6$)	5% artificial local damping
Dynamic boundary conditions	Rigid nodal prescribed velocity	Rigid nodal prescribed velocity	Rigid nodal prescribed velocity

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