Integrated numerical study of rainfall-induced landslides, from initiation to propagation using saturation-based rheological model

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Abstract

This paper presents an integrated two-dimensional numerical framework for simulating rainfall-induced landslides from instability initiation to post-failure flow. To describe the entire
process, three steps are considered in this study: 1) a coupled hydro-stability analysis which
detects the failure plane using the Finite Element Method (FEM) (pre-failure stage), 2)
computing the local rheology of the failed mass (wet sandy soil) based on the water saturation at
the onset of failure, using the saturation-based rheological model (transition stage), and 3) a
continuum-based propagation analysis which solves the flow of the wet material by employing
the Smoothed Particle Hydrodynamics (SPH) method (post-failure stage). Finally, to investigate
the influence of rheological model on the post-failure behavior, the computed final deposition
profile and flow kinetic energy are compared with those of a viscoplastic model.

**Keywords:** Rainfall-induced landslide, Slope stability, Saturation-based rheological model,
Propagation, Initiation.

1 Introduction
Slope failures and debris flows triggered by intense rainfall, are devastating natural hazards that
occur frequently all around the world. The integrated modeling of such landslides, from initiation
of instability to post-failure flow-like motion remains a challenge. At present, only a few studies
have addressed the entire process of rainfall-induced landslide in a single numerical framework [1-3]. The seepage-stress coupling and stress-strain modeling of the wet soil are the main difficulties in the integrated modeling [2].

Literatures pertinent to rainfall-induced landslides are classified into two main categories [4]: (1) the pre-failure stage [5,6] and (2) the post-failure stage studies [7,8]. The pre-failure stage includes seepage and slope stability analysis which determines the time and the location of landslide, while the post-failure stage deals with debris flow and its propagation.

In the pre-failure stage, at each step of infiltration, the seepage analysis determines pore water pressure distribution, while the stability analysis assesses the stability of the slope considering the effect of pore water pressure. The possible failure plane is detected by defining the factor of safety ($F_s$) as the ratio of the available local shear strength to the minimum local shear stress required to maintain equilibrium [9]. When the $F_s$ for a surface falls below one, its location is considered as the failure surface. The Limit Equilibrium Methods (LEM) are the most commonly methods used for this purpose. They divide the soil mass into vertical slices and calculate the factor of safety by applying the principle of moments and/or force equilibrium [10,11]. The LEM is computationally convenient to use but it only satisfies the equations of statics and doesn’t consider strain and displacement compatibility. Alternatively, one can use a FEM analysis to obtain the stress field and thereby evaluate the stability of the soil. Although this approach is computationally more expensive, it can provide detailed information about the status of the slope [11,12]. The effects of soil parameters (angle of internal friction and cohesion [13], soil suction and ground water table [14] on the stability of soil slope have been previously investigated. Some studies analyzed the influence of spatial variability, such as heterogeneity and discontinuity within the granular assembly, on the failure behavior [15,16]. To examine the influence of local failure on global failure, the effect of microstructural instability on the macroscopic failure has been also studied using a multiscale approach [17].

Due to the large deformation, the traditional Lagrangian FEM are not optimal for numerical modeling of the post-failure motion [18]. Instead, meshless or particle-based methods, such as Smoothed Particle Hydrodynamic (SPH) method [19], Discrete Element Method (DEM) [20], Material Point Method (MPM) [21] and hybrid Finite/Discrete Element Method (FDEM) [22] have been successfully used to simulate the propagation. When particle-based methods are used,
the particle-water interaction (e.g., drag and buoyancy forces) is modelled using a flow solver coupled with the particle phase solver. These methods solve a single-phase flow system first and then apply the hydrodynamic forces on the moving particles. A commonly used approach blends the DEM with a flow solver [23,24]. Despite their capabilities, the computational efficiency remains a challenge in these methods [25]. On the other hand, the continuum-based methods need a rheological model to determine the behavior of the material. This is a major challenge for continuum-based methods and choosing the most appropriate rheological model for wet granular flow is still a matter of debate. For example, Mast et al. [26] compared the hardening–softening Matsuoka-Nakai model and the Drucker-Prager model and showed that the final deposition profile is affected by the choice of the model. Fern and Soga [27] used Mohr–Coulomb and Nor-Sand models for simulating the collapse of a granular mass and showed that the energy dissipation is dependent on the rheological model. Ceccato and Simonini [28] applied Mohr–Coulomb failure criterion with an elastic perfectly plastic model, and Drucker-Prager yielding condition with a viscoplastic model, and showed that using different models leads to different results in the impact forces and destruction potential of a granular flow.

In almost all of the few continuum-based studies performed on the integrated modeling of rainfall-induced landslides [3,29], the same failure criterion (Mohr-Coulomb) is used for both pre-failure and post-failure stages and the soil behavior is often described by a Mohr–Coulomb strain-softening model based on Bishop’s effective stress [3]. However, the variation in soil rheological behavior induced by rainfall infiltration and its variation with water saturation was not considered in the propagation stage. This exhibits a missing link between the two stages and can lead to noticeable errors in the computed results (e.g., landslide velocity and runout). Using a saturation-based rheological model can lead to a better connection between pre-failure and post-failure stages.

The rheology of wet granular materials has been studied experimentally [30], numerically using discrete simulation [30] and multiscale modeling based on thermodynamic principles [31]. In these studies, the liquid distribution and inter-particle forces are described in terms of liquid bridges and capillary forces between grains, respectively. The pendular liquid bridge model [32], is commonly used in weakly wetted conditions. However, for simulating the flow of landslides, determining the rheological behavior as a function of saturation is essential. In their recent study [33], the authors proposed a power law rheological model for simulating the flow of wet granular
materials over a wide range of saturations. The rheological model was a function of Inertial number \( I \) and saturation. The inertia number is defined as \( \dot{y}d_p/\sqrt{\sigma/\rho_p} \), where \( \dot{y} \) is the shear rate, \( d_p \) and \( \rho_p \) are the diameter and density of soil particles, and \( \sigma \) is the normal stress. This saturation-based rheological model can be used as a transition stage to determine the local rheology of wet soil at the onset of failure based on its saturation distribution.

To describe the entire process of rainfall-induced landslides, this paper presents an integrated model based on a three-step (pre-failure, transition and post-failure) strategy. We start with the water saturation at the onset of failure, compute the local rheology of the wet soil based on the saturation, and then solve the flow of wet material using a continuum-based method. The pre-failure results (seepage analysis and slope failure detection) are benchmarked against experimental and numerical studies from the literature. The rheological model (transition stage) was previously verified in [33] for the dry soil and the results were reported for soils with different degrees of saturation. In the present paper, the rheological model is verified for the saturated soil and then used for the propagation modeling.

2 Computational Procedure

In this study, the coupled hydro-stability FE analysis, the saturation-based rheological model and the propagation SPH solver are integrated to simulate the entire process of rainfall-induced landslides. The flowchart of the simulation procedure is shown in Fig. 1, and the mathematical formulations are described as follows.

2.1 Step 1: Pre-failure stage

Equations governing rainfall infiltration and slope stability analysis are described in this section.

2.1.1 Hydrological model
Considering the soil as a rigid isotropic porous medium, the unsteady Darcian flow of water in a variably saturated soil is mathematically described by Richards’ equation [34] (in 2D form):

\[
\frac{\partial \theta(\psi)}{\partial t} = \frac{\partial}{\partial x} \left[ K(\psi) \frac{\partial \psi}{\partial x} \right] + \frac{\partial}{\partial y} \left[ K(\psi) \left( \frac{\partial \psi}{\partial y} - 1 \right) \right],
\]

where the flow of air is neglected due to large mobility contrast between the air and water [35].

In Equation 1, \( \theta \) is the non-dimensional volumetric water content \( [L^3] \), \( \psi \) is the pressure head \( [L] \), \( x \) is the horizontal spatial coordinate \( [L] \), \( y \) is the vertical spatial coordinate taken as positive downward \( [L] \) and \( K \) is the hydraulic conductivity \( [LT^{-1}] \).

\( \theta(\psi) \) and \( K(\psi) \) are in general highly nonlinear functions [35]. A number of empirical correlations have been proposed for these functions [36,37]. In this study, \( \theta(\psi) \) and \( K(\psi) \) proposed by van Genuchten [38] are used. These models use the statistical pore-size distribution model of Mualem [39]. The van Genuchten models are given by [38]:

\[
\theta(\psi) = \begin{cases} 
\theta_s + \frac{\theta_r - \theta_s}{(1+|\alpha \psi|^n)^\theta} & \psi < 0 \\
\theta_s & \psi \geq 0
\end{cases}, \tag{2}
\]

\[
K(\psi) = K_s \cdot S^l \cdot \left[ 1 - (1 - S^l)^l \right]^2, \tag{3}
\]

where:

\[
S = \frac{\theta(\psi) - \theta_r}{\theta_s - \theta_r},\tag{4}
\]

\[
J = 1 - \frac{1}{n}, \quad n > 1, \tag{5}
\]

in which \( \theta_r \) and \( \theta_s \) denote the residual and saturated water contents, respectively; \( K_s \) is the saturated hydraulic conductivity, \( \alpha \) is the inverse of the air-entry value (or bubbling pressure), \( n \) is a pore-size distribution index, and \( l \) is a pore-connectivity parameter. Equations 2 to 5 contain six independent parameters: \( \theta_r, \theta_s, \alpha, n, K_s \) and \( l \). [39]. \( l \) is set to 0.5 in the current simulation [31].
The initial pressure head and water content in the domain \( \Omega \) are:

\[
\psi(x, y, t) = \psi_0(x, y),
\]

(6)

and

\[
\theta(x, y, t) = \theta_0(x, y),
\]

(7)

where \( \psi_0 \) and \( \theta_0 \) are known functions.

The model can deal with pressure head and flux boundary conditions. Head boundary conditions are given by:

\[
\psi(x, y, t) = B_1(x, y, t) \quad \text{for} \quad (x, y) \in \Omega_{B_1},
\]

(8)

and flux boundary conditions are of the form:

\[
K \left( \frac{\partial \psi}{\partial x} \right) = B_2(x, y, t) \quad \text{for} \quad (x, y) \in \Omega_{B_2},
\]

(9)

\[
K \left( \frac{\partial \psi}{\partial y} \right) = B_3(x, y, t) \quad \text{for} \quad (x, y) \in \Omega_{B_3},
\]

(10)

where \( \Omega_{B_1}, \Omega_{B_2} \) and \( \Omega_{B_3} \) indicate head and flux type boundary segments and \( B_1[L], \ B_2[LT^{-1}] \) and \( B_3[LT^{-1}] \) are prescribed functions. In this study, the flux condition is used for boundaries with no fluxes ( \( B_2 \) and \( B_3 \) are set to be zero) and for boundaries on which precipitation occurs ( \( B_3 \) is set to be the rainfall intensity function).

Flux boundaries are controlled by atmospheric condition for soil-air interfaces. For such boundaries, If the applied flux is larger than the soil saturated hydraulic conductivity ( \( K_s \) ), the flux boundary condition switches to the head boundary condition. In fact, this atmospheric boundary condition removes the excess water resulting from oversaturation as surface runoff from the surface [40]. The current model ignores the evaporation and transpiration, since the temporal scale of the flow processes in this study [41].

The governing differential equation (Equation 1) is solved using the standard Galerkin finite elements along with the backward Euler time integration scheme as outlined in [35] and [42].
\( \psi(x,y,t) \) and \( S(x,y,t) \) are the outputs of the hydrological analysis. According to the Bishop’s equation, the total stress (\( \sigma \)) must be replaced by the effective stress (\( \sigma' \)) in partially saturated soils. So, the calculated pore water pressure distribution can affect the stability of the soil mass. The next step is to examine the stability of the soil slope at each time step of infiltration.

### 2.1.2 Slope stability model

In this study, the finite element shear strength reduction (SSR) method [12] based on the Mohr-Coulomb failure criterion, is used for detecting the failure plane. The SSR method defines the factor of safety (\( F_s \)) as the ratio of the real shear strength to the minimum shear strength required for failure prevention. In this technique, the shear strength parameters are changed until the failure conditions are met. The reduced shear strength parameters \( \bar{c} \) and \( \bar{\phi} \) are given by [12]:

\[
\bar{c} = \frac{c}{SRF},
\]

(11)

\[
\bar{\phi} = \arctan\left(\frac{\tan \phi}{SRF}\right),
\]

(12)

where \( c \) and \( \phi \) are the cohesion and the friction angle of the granular material and SRF is the strength reduction factor. The value of SRF that corresponds to the slope failure is equal to the factor of safety (\( F_s \)) [12]. In this study, the non-convergence criteria [12] is applied to detect the failure plane. By increasing the values of the SRF, the algorithm gradually weakens the soil parameters, until the iteration limit was reached without convergence. This actually means that no stress distribution can be achieved to satisfy both the global equilibrium and the failure criterion. This condition goes with a sharp increase in the nodal displacements inside the mesh, and means that the failure has occurred.

### 2.2 Step 2: Transition stage: The saturation-based rheological model
In their recent study [33], the authors proposed a saturation-based rheological model for simulating the flow of wet granular materials. In [33] the discrete element method (DEM) has been employed to establish a rheological model that correlates the apparent viscosity of a granular material to shear rate, normal stress, and water saturation. A power law rheological model was devised as follows:

$$\mu = A(S) \times I^{-B(S)},$$

(13)

Where $A$ and $B$ values are listed in Table 1. In Equation 13, $\mu$ is the apparent viscosity [Pa.s] and $I$ is the inertia number defined as $I = \frac{\dot{\gamma} d_p}{\sqrt{\sigma / \rho_p}}$, where $d_p$ and $\rho_p$ are the diameter and density of soil particles, $\sigma$ is the normal stress [Pa] and $\dot{\gamma}$ is the shear rate.

For a soil subjected to the rainfall infiltration, it has been shown that up to a critical degree of saturation, the apparent viscosity increases with saturation [33]. However, when the saturation exceeds this critical value, the apparent viscosity drops below the viscosity of dry soil. For a homogenous soil composed of mono-sized spherical particles with a cubic lattice initial particle arrangement, the critical saturation was calculated to be 34.2% [33]. The details of derivation of the rheological model (Equation 13) can be found in [33]. In this study, the transition stage is modelled by applying Equation 13 along with Table 1.

### 2.3 Step 3: Post-failure stage: The propagation model

In this section the Smoothed Particle Hydrodynamics (SPH) method [43] is used for studying the flow of wet granular material. This study considers the flowing material as an incompressible fluid and utilizes the saturation-based rheological model to determine the behavior of the wet material. To capture the realistic behavior of granular materials, a slip boundary condition is also imposed on the solid boundaries [19].

In the present study, it is assumed that change in the bulk density of the material is not significant during flow. Accordingly, the continuity and momentum for the equivalent fluid are:

$$\nabla \cdot \vec{V} = 0,$$

(14)

$$\frac{D\vec{V}}{Dt} = -\frac{1}{\rho_b} \nabla P + \frac{1}{\rho_b} \nabla \cdot \tau + \vec{g},$$

(15)
where \( \rho_b \) is the bulk density, \( P \) is the pressure of fluid, \( \vec{V} \) is the velocity vector, \( \vec{g} \) is the gravitational acceleration vector and \( \tau \) is the stress tensor, which is determined using the saturation-based rheological model. The bulk density of the wet material is obtained as:

\[
\rho_b = (1 - \kappa) \rho_p + \kappa S \rho_{\text{Water}},
\]

(16)

where \( \kappa \) is the porosity of material and \( \rho_{\text{Water}} \) is the density of water. A detailed description of solving Equations 14 and 15 using the SPH method, can be found in [19].

In this paper, an in-house flow solver named SePeHr [19] is used. This code has been recently used for modeling the collapse of two-dimensional dry granular material using a viscoplastic rheology on horizontal rigid and entrainable beds [19].

3 Results

The test case presented by Park in [44] was chosen as a benchmark for this study. Park presented both experimental data and numerical results in his paper. In his experiments, Park measured the water content and soil suction (negative pore water pressure) changes due to artificial rainfall at four locations within a slope composed of frictional non-cohesive initially dry sand. The location of the failure plane was also reported and then the experimental observations were compared with numerical results. Fig. 2 schematically shows Park’s experimental setup and Table 2 shows the material properties used in his experiment. The parameters of van Genuchten model considered in the current numerical analysis are presented in Table 3. The current parameters, obtained based on the measured soil suctions and volumetric water contents [44], are more consistent with experimental data in comparison to the values considered by Park. The soil sits on top of a 5 cm drainage layer made of crushed stones and the rainfall intensity was 30 mm/h in the experiments.

The current results are divided into four parts: validation of the hydrological (Section 3.1), the slope stability (Section 3.2), and the rheological model (Section 3.3) and then simulating the propagation of the wet collapsed material (Section 3.4).
3.1 Seepage Results

The computational grid consists of 3777 triangular elements and 2005 nodes as shown in Fig. 3, and the boundary conditions are imposed as shown in Fig. 4. The initial condition is set based on the measured porewater pressures before the onset of rainfall \( \theta(x, y) = 0.05 \) at \( t = 0 \) [44].

Fig. 5 shows the contours of volumetric water content for different time steps after the onset of rainfall. In accordance with [44], the rainfall starts 180 min after measuring the initial water content. As can be observed when the water reaches the crushed stone layer, due to the higher hydraulic conductivity, it starts moving faster and eventually leaves the soil from the free drainage boundary. The crushed stone layer was formed to ensure that slope failure occurred only when there was a change in the shear strength of the slope due to rainfall seepage and so the influence of the water table rising has been eliminated (the maximum saturation in the domain is about 99.5%).

Figs. 6 and 7 compare the results of the current numerical simulation with experimental and numerical results of Park [44] at point C in Fig. 2. As it can be seen, the current numerical results are closer to the experimental data. A possible reason for the differences in the numerical results is the definition of hydraulic functions in van Genuchten model.

3.2 Slope Stability Results

Fig. 8 compares the experimental and numerical results of Park [44] and the result of the current numerical analysis in the prediction of the failure plane. As it can be seen, the current result is closer to the experimental observation in comparison with Park’s numerical analysis.

In the experimental results, the failure time, i.e., the time when the factor of safety falls below 1 has been reported. Park [44] analyzed the failure time experimentally and the changes in \( F_s \) numerically. Fig. 9 compares the results obtained in present study with those presented in
As the figure shows, ten minutes after the onset of rainfall ($t = 190 \text{min}$), the factor of safety has started decreasing. In the current study, $F_s$ has fallen below one at $t = 325 \text{min}$, which means the failure occurred after 145 minutes from the onset of rainfall. In the Park’s experiment, and numerical analysis, the failure happened 140 and 130 minutes after the rainfall onset respectively. So, the current simulation predicts the time of failure more accurately. More accurate simulation of the water infiltration and using the FE stability analysis, instead of the LEM method, can be mentioned as the reasons for this improvement.

3.3 Verification of The Saturated Rheological Model
According to Fig. 8, the slope failure occurred in the saturated zone. So, only the saturated state of the rheological model (Section 2.3) must be included in the propagation model. In [33], the rheological model was verified for the sand at dry state and the results were reported for the wet sands at different saturations. Before using the rheological model in the post-failure stage, this section, verifies the model in predicting the behavior of the saturated sand. To achieve this goal, the saturated rheological equation (section 2.2) was implemented into the SePeHr [19] code and the validity of the rheological model examined by comparing the simulation with experimental results. To conduct this verification, the collapse of a fully-saturated granular material on a horizontal rigid bed is compared with the experimental results obtained by Cessato et al. [45]. Fig. 10 schematically shows the case studied in this section and the properties of the sand are listed in Table 4. The base and lateral walls are made of glass and the gate is made of plexiglass. The details of sand preparation and test are outlined in [45].

One of the most important indicators of collapses is the distance traveled by the moving material, called “runout”, shown here by $\bar{x}_z$. In the current numerical simulation, the slip length was changed to obtain runout values close to the experiments. A value $l_s = 0.009$ for the slip length gave the best match with the experimental data.

The shape of the collapsed material is compared with the experimental results at different times in Fig. 11. As can be seen from the results, current numerical simulations are in close agreement with the experimental data.
The assumption that the material bulk density remains constant during the collapse, can be a reason why the results do not completely match. The top part desaturation during collapse is another phenomenon that the current simulation has not captured.

3.4 Propagation Results

The final step of study is the propagation simulation. By assigning the saturated rheology to the flowing material, it is possible to simulate the propagation of the saturated soil. Fig. 12 shows the simulation of the post-failure flow in which the SPH particles are colored based on their velocity magnitude. As illustrated in Fig. 12, the propagation stops at $t = 1.65 \text{s}$.

In order to investigate the influence of the rheological model on the post-failure motion, the propagation simulation is repeated with a modified Bingham viscoplastic rheology (b-viscous model) that does not consider the effect of saturation. This model reads [19]:

$$
\tau = \begin{cases} 
M \mu_B \dot{\gamma} & \text{if } |\dot{\gamma}| < \frac{\tau_y}{M \mu_B}, \\
\tau_y + \mu_B (\dot{\gamma} - \frac{\tau_y}{M \mu_B}) & \text{if } |\dot{\gamma}| \geq \frac{\tau_y}{M \mu_B} 
\end{cases}
$$

(22)

where $\mu_B$ is the Bingham viscosity and $M$ is the viscosity factor of order $10^4$ [19].

Fig. 13 shows the final deposition profile for both simulations. As it can be seen, the computed runout distance is longer when using the saturation-based rheology.

To compare the flow destructive potential, the kinetic energy of the flow is considered as a criterion. The kinetic energy ($KE$) at each time step is computed as [19]:

$$
KE = \sum_{i=1}^{N} m_i \frac{V_i^2}{2},
$$

(23)

where $m_i, V_i$ and $N$ are the mass, the velocity and the total number of the SPH particles.
Fig. 14 shows the variations of the kinetic energy of the flow with time for the two different rheologies. As the figure shows, the destructive potential is higher when a saturation-based rheological model is used.

4 Conclusion

In this paper, an integrated numerical framework based on a three-step strategy was presented for simulating rainfall-induced landslides from the initiation of instability to the debris flow. The main results of the present study are summarized as follows:

- In the pre-failure stage, the progressive changes in the pore pressure, volumetric water content and $F_s$ during water infiltration can be evaluated. To make a better connection between pre-failure and post-failure stages, a transition stage is proposed which uses the saturation-based rheological model to compute the local rheology of the failed mass based on its saturation. The main advantages of the current integrated framework are: nonexistence of pre-assumed failed mass and considering the variation in the soil rheological behavior induced by rainfall infiltration in the propagation stage.

- The entire landslide process was simulated for a soil slope exposed to rainfall and the results were presented. The results showed that considering the effect of the water saturation on the soil rheological behavior can affect the propagation results. The current simulation which used the saturation-based rheological model predicts different deposition profiles with higher runout and higher flow kinetic energy in comparison with simulations using a visco-plastic rheology.
This study proposed a numerical procedure for integrated simulation of rainfall-induced landslides. This research has some limitations; 1) the computational domain was two-dimensional, 2) the bed was rigid and non-entrainable, 3) In the rheological model, the granular soil was assumed to be a homogenous assembly of mono-sized spherical particles which initially form a cubic lattice packing. Despite this, no other physical restriction is foreseen to limit the application of the present procedure.

Nomenclature

\( F_s \)  \hspace{1cm} \text{Factor of safety}  \hspace{1cm} l  \hspace{1cm} \text{Pore connectivity parameter}

\( \tau \)  \hspace{1cm} \text{Shear stress}  \hspace{1cm} K  \hspace{1cm} \text{Porosity}

\( \dot{\gamma} \)  \hspace{1cm} \text{Shear rate}  \hspace{1cm} l_s  \hspace{1cm} \text{Slip length}

\( \mu \)  \hspace{1cm} \text{Apparent viscosity}  \hspace{1cm} \mu_B  \hspace{1cm} \text{Bingham viscosity}

\( I \)  \hspace{1cm} \text{Inertia number}  \hspace{1cm} M  \hspace{1cm} \text{Viscosity factor}

\( d_p \)  \hspace{1cm} \text{Particle diameter}  \hspace{1cm} KE  \hspace{1cm} \text{Kinetic energy}

\( \rho_p \)  \hspace{1cm} \text{Particle density}  \hspace{1cm} N  \hspace{1cm} \text{Total number of SPH particles}

\( \rho_b \)  \hspace{1cm} \text{Bulk density}

\( \rho_{\text{water}} \)  \hspace{1cm} \text{Water density}

\( \sigma \)  \hspace{1cm} \text{Normal stress}

\( \sigma' \)  \hspace{1cm} \text{Effective stress}

\( \psi \)  \hspace{1cm} \text{Pressure head}

\( \phi \)  \hspace{1cm} \text{Angle of internal friction}

\( \bar{\phi} \)  \hspace{1cm} \text{Reduced angle of internal friction}

\( c \)  \hspace{1cm} \text{Cohesion}
\( \bar{c} \) Reduced cohesion
SRF Shear reduction factor
\( P \) Pressure of fluid
\( S \) Saturation degree
\( \theta \) Volumetric water content
\( \theta_r \) Residual volumetric water content
\( \theta_s \) Saturated volumetric water content
\( g \) Gravitational acceleration
\( m \) Mass
\( V \) Velocity
\( t \) Time
\( x \) Horizontal spatial coordinate
\( y \) Vertical spatial coordinate
\( K \) Hydraulic conductivity
\( K_s \) Saturated hydraulic conductivity
\( \alpha \) Inverse of air-entry value
\( n \) Pore size distribution index

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Figure 1. The flowchart of the current integrated numerical simulation.
Figure 2. Schematic of the case study considered for validation of the infiltration model. A, B, C and D are the locations of water content and soil suction sensors.
Figure 3. The geometry and mesh of the current computational domain.
Figure 4. Boundary conditions for Park’s benchmark problem.
Figure 5. Contours of volumetric water content at different times after the onset of rainfall. a) $t = 180\,\text{min}$, b) $t = 220\,\text{min}$, c) $t = 260\,\text{min}$ and d) $t = 300\,\text{min}$. Rainfall starts at $t = 180\,\text{min}$. 
Figure 6. Comparison of the water content results in the current simulation with the experimental and numerical results of Park at point C in the slope. Rainfall starts at $t = 180 \text{ min}$.

Figure 7. Comparison of the negative pore water pressure results in the current simulation with the experimental and numerical results of Park at point C in the slope. Rainfall starts at $t = 180 \text{ min}$.

Figure 8. Comparison of the observed failure plane in the Park’s experiment and the predicted failure plane in Park’s and current numerical simulations.

Figure 9. Factor of safety vs time obtained in this work and those presented in [44] by Park.

Figure 10. Schematic of the case study considered for verification of the rheological model.

Figure 11. Comparison of free surface obtained by numerical simulation with experimental results [45] at different simulation times, a) $t = 0.044 \text{ s}$, b) $t = 0.109 \text{ s}$, c) $t = 0.219 \text{ s}$ and d) $t = 0.497 \text{ s}$.

Figure 12. Post failure simulation of the debris flow based on the saturation-based rheology.

Figure 13. Comparison of the deposition profiles obtained by the saturation-based and modified Bingham rheology.

Figure 14. Variations of the kinetic energy of the flow with time for both simulations.
Table 1. Coefficients of the rheological equation for different saturations [33].
Table 2. Physical properties of the materials in Park’s experiments [44].
Table 3. Parameters of van Genuchten model.
Table 4. Specimen specifications in Cessato et al. experiments [45].
The slope geometry, soil and rainfall specifications, initial and boundary conditions.

Water infiltration FE solver
Pressure head $\psi(x,y,t)$ and saturation $S(x,y,t)$ are obtained

Next time step

Slope stability FE solver
Safety factor for a critical surface, $F_s > 1$

Pre-failure Stage

Saturation-based rheological model
Rheology of the failed wet granular mass is set

Transition Stage

The Propagation SPH solver

Post-failure Stage

Figure 1
Figure 4
Figure 5

Figure 6
Figure 9

Figure 10
Figure 11

t=0 s

t=0.15 s

t=0.3 s

t=0.45 s
Figure 12
Figure 13

Figure 14
### Table 1

| Coefficient | Saturation degree |
|-------------|-------------------|
|             | 10 %   | 20 %   | 30 %   | S ≥ 34.2% |
| A           | 9 e-4  | 4.2e-4 | 1.3e-4 | 0.17      |
| B           | 2.39   | 2.58   | 2.84   | 0.85      |

### Table 2

| Material      | Parameter          | Value       |
|---------------|--------------------|-------------|
| Soil          | $K_s$ (m/s)        | $1.3 \times 10^{-4}$ |
|               | $\theta_s$        | 0.38        |
|               | $\theta_r$        | 0.05        |
|               | Density (kg/m³)    | 1605        |
|               | Cohesion (kPa)     | 0           |
|               | Friction angle     | 33.6        |

### Table 3

| Parameter                  | Park’s numerical simulation | Current numerical |
|----------------------------|-----------------------------|-------------------|
| Crushed stone              | $K_s$ (m/s)                 | 0.13              |
|                            | Density (kg/m³)             | 1900              |
|                            | Cohesion (kPa)              | 0                 |
|                            | Friction angle ($^\circ$)   | 45                |
Table 4

| Specification                        | Value           |
|--------------------------------------|-----------------|
| Grains mean diameter ($d_p$)         | $2.5 \times 10^{-3}$ m |
| Grain density ($\rho_p$)             | $2625$ kg/m$^3$  |
| Initial porosity ($\phi$)            | $0.4$           |
| Cohesion ($C$)                       | $0$ Pa          |
| Angle of internal friction ($\zeta$) | $35^\circ$      |

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