Research Paper

Investigation of retrogressive and progressive slope failure mechanisms using the material point method

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Abstract

Retrogressive and progressive slope failures are a dynamic process, in the sense that they involve a progressively changing scenario. This paper uses the contemporary material point method (MPM), to provide a view of how such failures develop. Two main scenarios are presented: (a) a relatively small slope, which, when subjected to an initial failure, is steepened, leading to the initiation of further failures retrogressing backwards; and (b) a long slope, where an initial perturbation (e.g. an excavation) triggers a series of failures that can retrogressively move up-slope.

1. Introduction

Informally, "a movement of a mass of rock, earth or debris down a slope" is defined as a landslide [1]. Based on the type of material (e.g. rock, soil) and the mode of the movement (e.g. falls, slides) involved, various types of landslides have been identified [2–4]. The corresponding failure mechanisms, identified mostly through the back analysis of case histories, are also diverse and complicated, due to the interactions of adjacent sliding bodies [4]. A number of related descriptions have been summarised [3], such as advancing, enlarging, progressive or retrogressive.

If an initial slide occurs and the material in the failure flows away, which is usually caused by a high degree of strength loss, a steep main scarp will usually be formed and therefore support for the remaining soil will be removed. This can result in another failure, termed a retrogressive failure. This process can repeat itself in a multiple-retrogressive fashion, and can result in a bigger landslide. In some slopes, such strength loss does not occur almost instantaneously, but is associated with the magnitude of shear strain, so that the rupture surface propagates through the soil profile over time. In this case, the term “progressive” is used. Reported cases include the retrogressive failures of cemented sensitive clays in the Ottawa–St. Lawrence Lowlands [5]; retrogressive landslide complexes in the Boone valley in the French Alps [6]; and the progressive failures of observed landslides in Scandinavia and eastern Canada [7]. The recent Oso landslide in Washington was observed to have multi-rotational retrogressive failures in parts and large translational slides in the longer slopes [8].

Investigations into the conditions triggering landslides have also been initiated [9–13], in order to find efficient ways to mitigate landslides along with their significant impacts. Common destabilising factors include rainfall infiltration, water level rise, and earthquakes. Site investigations on real slope failures provide very valuable information; however, the intervals between individual failures in a larger slide can sometimes amount to some tens of years [14]. In most cases of real slope failures, instrumentation of the failure and material characterisation are not undertaken. Hence, for helping to investigate slope failure conditions in a time efficient way, numerical modelling shows certain advantages. A recent publication [15] compares material point method (MPM) simulation with two real cases, although the only comparison is the final failed slope configuration; hence there is no comparison with any failure or propagation mechanisms, e.g. rotational or progressive failures.

This paper presents a numerical framework which is able to simulate the whole slope failure process, from initiation, through failure propagation, to the final equilibrium configuration. For convenience in benchmarking this technique, some idealised assumptions are made, but these can easily be changed for more site specific analyses. The simplifying assumptions are: (a) the flow material is a clay idealised by a linear elastic, cohesion strain softening Von Mises model; (b) the slope is assumed to be initially...
unstable under the in-situ stress condition, so that self-weight is the trigger for the slope failure rather than any of the factors mentioned above; (c) no pore pressure changes are simulated. Hence, a simple total stress approach is adopted in this paper, with the aim of giving a clear (albeit simplified) picture of some of the main geometric features of slope failure mechanisms in cohesive soils; that is, as a prequel to future investigations involving more realistic material and triggering scenarios. The emphasis here is to reproduce commonly seen clay-type slope failures [5–7] (e.g. rotational and translational slides), and to interpret the failure mechanisms within the proposed framework; that is, to explain the observed translational and rotational slides through the concepts of retrogressive and progressive failure. Comparison of simulations to field cases is beyond the scope of this investigation.

For modelling slope instability, traditional numerical tools such as the finite element method are often limited in their applicability to problems involving large deformations, due to potential excessive mesh distortions that can occur in such cases. This can give an incomplete description of failure, in that the initial slip is considered and the ongoing sliding failure is ignored. That is, continual changes in geometry cannot easily be simulated without extensive re-meshing. However, by using the implicit material point method (IMPM) [16] coupled with a cohesive softening (Von Mises type) constitutive model, the process of retrogressive failure in an undrained soft clay under self-weight loading is possible, as will be demonstrated herein. For this purpose, two types of slope are analysed, which, for convenience, are called “short slope” (slope height = 5.0 m) and “long slope” (down-slope length = 25.0 m). The factors influencing the post-failure and retrogressive failure behaviours of the two slopes have been investigated. For the long slope, different slope angles are considered, to investigate the link between slope geometry and the various failure mechanism categories.

2. Implicit material point method

MPM has proven to be a useful finite element method (FEM) variant for simulating large-strain problems in geotechnical applications [17–20], with the material points representing the continuum being capable of moving through a background mesh, thereby removing the limitation of excessive mesh distortions that can occur in FEM. The implicit material point method (IMPM) here refers to an MPM framework where the governing equation is solved implicitly, which can be used for both quasi-static and dynamic analyses. It addresses time step size limitations, which are an inherent problem in explicit dynamics, and can thus reduce the computational cost in many cases. The full details of IMPM can be found in Wang et al. [16] and Guikely and Weiss [23]. The final equilibrium equation can then be expressed in matrix form as,

\[
\mathbf{K}' \mathbf{u} = \mathbf{R}^\text{int} - \mathbf{F}_\text{int}^t
\]

where \( \mathbf{K}' = \mathbf{K}' + \mathbf{K}_\text{int} \) taking into account the large strain deformation, \( \mathbf{\Delta u} \) is the vector of incremental nodal displacements, \( \mathbf{R}^\text{int} \) is the external loading accounting for both traction and body loads on the continuum and \( \mathbf{F}_\text{int}^t \) is the internal force.

As an example, the linear elastic stiffness at time \( t \) is expressed as,

\[
\mathbf{K}'_p = \sum_p \left( \mathbf{B}_p^t (\mathbf{x}_p) \mathbf{C}_p \mathbf{B}_p^t (\mathbf{x}_p) \right) V_p
\]

where \( \mathbf{B}_p \) is the matrix of shape function spatial differentials, \( \mathbf{C}_p \) is the stress–strain relationship which is traced on each individual material point, \( \mathbf{x}_p \) are the coordinates of a material point, \( V_p \) is the volume associated with a material point, and subscript \( p \) refers to a material point. The non-linear part of the stiffness term is expressed in a similar manner [16].

A dynamic solution can be readily obtained by adding an inertial term in Eq. (2), to give

\[
\mathbf{K}' \mathbf{u} + \mathbf{M} \dot{\mathbf{a}}^\text{int} = \mathbf{R}^\text{int} - \mathbf{F}_\text{int}^t
\]

where \( \mathbf{M} \) is the mass matrix and \( \dot{\mathbf{a}} \) is the acceleration. The following relationship between the kinetic variables is also assumed [24]:

\[
\mathbf{v}^{\text{int}} = \dot{\mathbf{v}}^t + [(1 - \delta) \mathbf{a}^t + \delta \mathbf{a}^\text{int}^t] \Delta t
\]

\[
\mathbf{u}^{\text{int}} = \dot{\mathbf{u}}^t + \mathbf{v}^t \Delta t + \left[ \left( \frac{1}{2} - \alpha \right) \mathbf{a}^t + \alpha \mathbf{a}^\text{int}^t \right] \Delta t^2
\]
where $\Delta t$ is the time step, $\mathbf{v}$ and $\mathbf{u}$ are the velocities and displacements, respectively, and $\alpha$ and $\delta$ are time stepping parameters which determine the integration accuracy and stability.

By substitution of Eqs. (5) and (6) into Eq. (4) the equation can again be formed in terms of displacement increments. A modified stiffness matrix $\mathbf{K}'$ is utilised which incorporates the mass matrix terms related to the displacement [16].

2.2. Further features in this implementation

In this section, the adopted techniques for calculating the damping force and frictional boundary conditions within the IMPM framework are introduced. Based on the acquired damping force, as well as the reaction forces from the underlying ground against the bottom of the soil layer, a final governing equation for the slope analysis is given thereafter. Further details can be found in [16].

2.2.1. Damping force

Damping is included in the formulation to avoid non-physical vibration in dynamic analyses. Rayleigh damping [25] is often used, although it is frequency dependent, and hence prior knowledge about the frequency of the system is needed. The approach of Cundall [26], which involves a local non-viscous damping to overcome the issue of frequency dependence, has been adopted here. Hence, the damping force on a node $i$ is proportional to the magnitude of the out-of-balance forces, with a directional function that ensures that vibrational modes are damped, i.e.

$$
\mathbf{F}_{\text{damp},i} = -c_i [\mathbf{f}_i \cdot \text{sign}(\mathbf{v}_i)]
$$

where $c_i$ is a dimensionless damping factor, $\mathbf{f}_i$ are the nodal resultant forces and $\mathbf{v}_i$ is the velocity direction.

The choice of damping factor in the analyses is not necessarily easy to define. The factor should be large enough to dissipate any unbalanced energy causing unrealistic oscillations, but not so large as to cause an overdamped system, which would unrealistically reduce the speed of the slide. To achieve a quasi-static equilibrium efficiently, the factor was chosen mostly to be 0.75 in Al-Kafaji [27]; however, for the unsaturated one-point three-phase MPM in Yerro et al. [28], the factor was chosen to be only 0.05, as it was stated that the drag force between the solid and the fluid introduces an implicit damping force; and in the river levee collapse analysis in Bandara and Soga [29], the damping force was not considered during the evolution of the failure mechanism, as the plasticity of the material and the viscous effects of the fluid were reported to be sufficient to dissipate the unbalanced energy. Hence, the damping factor is problem/material dependent. In this work, a factor of 0.15 was found to produce sufficient damping to reduce the oscillations, while not slowing the soil movement unduly. Without using the damping factor, displacements became irregular and stresses oscillated, causing spurious plastic yielding. This factor, while reducing spurious oscillations, was observed not to give different failure mechanisms, although it did predict slightly smaller run-out distances.

2.2.2. Frictional boundary conditions

The interaction between the soil and a non-deforming material outside the domain has been modelled using a Coulomb frictional algorithm. This algorithm is a simplified version of the contact algorithm introduced in Bardenhagen et al. [30].

Initially, a fully fixed boundary condition is applied and the reaction forces on the boundary, $f_{\text{react},i}$ are calculated and termed trial forces. The tangential component of this force is then compared to the maximum allowable tangential force considering frictional behaviour, i.e. $\mu \max\left(0, f_{\text{react},i}^t\right)$, where $\mu$ is the friction coefficient between the soil and material outside the domain, the superscript $tr$ indicates that this refers to the trial force and the subscript $n$ refers to the normal component. If the trial force exceeds the allowable force, then a slip condition occurs and the allowable force is used as the boundary condition and the increment is re-calculated. This can be expressed as

$$
f_{\text{react},i}^t = \left\{ \begin{array}{ll}
\mu \max\left(0, f_{\text{react},i}^t\right) & \text{if } f_{\text{react},i}^t > \mu \max\left(0, f_{\text{react},i}^n\right), \ i.e. \ slip \\
0 & \text{otherwise, i.e. stick}
\end{array} \right.
$$

where the subscript $t$ refers to the tangential component of the reaction force.

The reaction force can be applied to all the background element nodes attached to the boundary and included in the vector $\mathbf{F}_{\text{react}}$. The trial reaction force at each node, $i$, is then given by

$$
\mathbf{F}_{\text{react},i} = -\mathbf{F}_{\text{ext},i} + \mathbf{F}_{\text{int},i} + \mathbf{M}_i \mathbf{a}_i
$$

where $\mathbf{F}_{\text{ext}}$ is the external force.

Further details of the implementation of the frictional algorithm, in particular for slope problems, can be found in Shin [31] and Bandara [32].

2.2.3. Final governing equation

By including the appropriate damping and the boundary reaction forces in Eq. (4), and utilising the modified stiffness matrix $\mathbf{K}'$, the final governing equation for the analysis can be formulated as

$$
\mathbf{K}' \Delta \mathbf{u} = \mathbf{F}_{\text{ext}} - \mathbf{F}_{\text{int}} + \mathbf{F}_{\text{react}} + \mathbf{F}_{\text{damp}}
$$

2.3. Constitutive model

Since a soft clayey slope is being investigated, under undrained total stress conditions, a simple model which can capture the basic soil behaviour, i.e. post-failure softening, has been used. This is based on cohesion softening and the von Mises yield criterion (i.e. for plane strain conditions),

$$
F_{\text{int}} = \mathbf{\sigma} - \sqrt{3c\mathbf{\tau}_p} = \sqrt{3J_2} - \sqrt{3c\mathbf{\tau}_p}
$$

where $J_2$ is the second deviatoric stress invariant, and $c$ is the undrained shear strength which is a function of the accumulated plastic deviatoric strain invariant $\mathbf{\tau}_p$, which, in incremental form, is defined as $d\mathbf{\tau}_p = \sqrt{2d\mathbf{\epsilon}_p} : d\mathbf{\epsilon}_p = \sqrt{2/3} d\mathbf{\varepsilon}_p$, where $d\mathbf{\varepsilon}_p$ is the plastic strain increment, which is normal to the plastic potential surface. Other constitutive models, such as Mohr–Coulomb, could be utilised; however, in this case the response would be constrained by the undrained conditions and equivalent pore water pressure response, and thereby give a similar outcome to Von Mises.

![Fig. 2. Sketch of cohesion softening model [16].](image-url)
A relatively short cohesive soil slope has first been analysed using IMPM, combined with the cohesion softening constitutive model. Parametric studies based on the softening modulus, residual strength and frictional boundary are provided towards the end of the section.

3. Retrogressive failure of a short slope under self-weight loading

Table 1
Material properties for the short slope analysis.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (kPa)</td>
<td>1000</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.33</td>
</tr>
<tr>
<td>Self-weight (kN/m²)</td>
<td>20</td>
</tr>
<tr>
<td>Peak cohesion (kPa)</td>
<td>20</td>
</tr>
<tr>
<td>Residual cohesion (kPa)</td>
<td>4</td>
</tr>
<tr>
<td>Softening modulus (kPa)</td>
<td>–50</td>
</tr>
</tbody>
</table>

The stress–strain relationship in the softening stage is assumed to be linear and given by,

\[
\begin{align*}
\sigma(p) &= c_0 + H\cdot\tau_p, \quad \tau_p < \tau_{pr} \\
\sigma(p) &= c_r, \quad \tau_p > \tau_{pr}
\end{align*}
\]  

(12a)

(12b)

where \(c_0\) is the initial cohesion, \(c_r\) is the residual cohesion, \(\tau_{pr}\) is the plastic deviatoric strain invariant at the onset of the residual strength and \(H\) is the softening modulus (which is taken to be negative). The proposed softening model is illustrated in Fig. 2.

An elasto-plastic algorithm has been used, following the approach used by Smith et al. [25], in which the constitutive equation, Eq. (2), is first solved with an initial elastic trial. The stress states at the material points are then checked to determine whether the stress state is allowable, or yield has occurred. A sub-stepping algorithm [33] has been used to re-distribute stresses and ensure that all stress states are within, or on, the yield surface.

Fig. 3 shows the initial geometry of the slope. It has a height of 5 m and a slope angle of 45°, and the distance from the crest to the left-hand boundary of the mesh is taken to be 10 m. The material properties are shown in Table 1. These values are hypothetical and do not represent a real case, although they are representative of a soft clay material. The chosen elastic parameters do not cause significant deformation, so that most of the deformation is due to plastic yielding. The problem is assumed to be plane strain. The boundary conditions are rollers on the left-hand edge allowing only vertical movement, and the interaction between the slope base and the underlying ground is modelled using the Coulomb frictional algorithm from Section 2.2.2, with the friction coefficient set to 0.3. The computational grid is made up of 4-node quadrilateral elements, with each element initially having 4 material points located at the Gauss point positions. The background computational mesh is shown as light grey squares in Fig. 3 and consists of 20 elements vertically and 80 horizontally. There are 4040 material points. The simulation was run until a final quasi-static equilibrium state was reached. Using an Intel Xeon E5-1620 processor, the analyses take approximately 3–4 h to execute on a single core.

Fig. 4 illustrates the collapse process and development of retrogressive failures within the slope, with the locations of the material points indicating the displaced soil and the coloured contours representing the accumulated plastic shear strain invariant \(\tau_{pr}\). The static factor of safety (FOS) of the slope is 0.96; this has been determined using the strength reduction method, which involves gradually reducing the shear strength until plastic strains are indeterminate and the algorithm fails to converge, at which point the strength reduction factor is interpreted as the FOS [25]. The slope has been analysed by applying gravity (body) loads to the material points in a single increment, in order to generate the soil’s in situ (self-weight) stresses. Hence, as an initially unstable slope is being considered, it is the self-weight loading that triggers the slope failure. An initial band of plastic shear strains can be seen initiating from the slope toe in Fig. 4(a). The band then propagates backwards and upwards, so that a complete slip surface is formed soon afterwards, as shown in Fig. 4(b). It is observed that the body of soil above the critical slip surface starts to slide along the surface, and that a second slip plane, also originating from the slope base, starts to form a second failure block. The remaining part of the slope remains largely intact, i.e. elastic. Fig. 4(c) clearly shows that plastic strains (and therefore strain softening) are concentrated along the slip surfaces, and that the sliding soil moves as discrete blocks. Formations of soil wedges with shapes of graben and horst are shown in Fig. 4(d), where, apart from the two distinct failure planes, the failure propagates in the horizontal direction as well, forming the wedge base. This is in accordance with the previous conceptual model of Odenstad [34] in explaining the retrogressive failure mechanism, where a horizontal weak plane/layer was assumed. However, in this case it is the strain softening that causes the “weak layer”. As time progresses, the retrogression continues to move backwards and upwards and, after approximately 40 s, the exposed back scarp behind the second slide has become large enough so that, due to the removal of downslope support, it becomes unstable and a third slide is triggered, as shown in Fig. 4(e). Unlike the second slide, the block of almost intact clay displaces laterally on the horizontal plane. The final stable configuration is displayed in Fig. 4(f), once the three failure blocks have formed. By observing and comparing the successive slides, it can be noticed that, as the failure propagates backwards, the time interval between successive slides becomes larger.

Fig. 5(a)–(c) depicts the stages of the constitutive model governing the behaviour of the material points within the slope. Hence, blue," light blue, grey and red colours indicate elastic, elastic unloading, softening and residual stages of the model, respectively. By identifying the stage that each material point is associated with...
at different times, these plots provide a clear view of the retrogressive failure phenomenon. It can be seen that the strength loss does not occur simultaneously throughout the slope, but according to the stress state and the accumulated plastic strain. When forming a distinct slip surface, material points within the shear band are seen to lose their strength very quickly, after which their response is governed by the residual cohesion. Material points in the immediate vicinity of the shear band are mostly softening, which decreases the soil strength and thereby perpetuates the retrogression of the failure mechanism back into the slope. Large changes in the plastic strain invariant are also seen at the slope base, which experiences the formation of a horizontal weak plane governed by the residual soil strength. In contrast, the soil that is remote from the shear band (mostly near the left-hand boundary or above the critical surface) remains intact.

During the failure evolution process, there are two observations worthy of note: firstly, when a block of soil moves out of the sliding area, the exposed soil at the back scarp experiences elastic unloading; secondly, as a new shear band is formed, the soils within the previous shear band(s) remain at the residual state or elastically unload. No special technique is used here to avoid mesh dependency problems due to the inclusion of strain softening, although these could be included via similar treatments as for FEM. The mesh was shown, via a sensitivity study, to give almost the same results as using a finer mesh in terms of the mechanism and propagation behaviour, although some limited mesh dependency was still observed.

Fig. 5(d) shows the shear stresses at the material points at time $t = 45.0$ s. It is seen that the stress states coincide with the deformations in a reasonable way, i.e. shear stresses are largest either side of shear bands of failing or recently failed segments. Although stress oscillations can be seen to exist spatially (as observed most clearly on either side of the shear band furthest to the left in Fig. 5 (d)), these can be reduced by using Gaussian integration techniques, GIMP, or other appropriate measures [20,35].

To conclude, this type of slope failure is attributed to the slope geometry change during the retrogression. As the previous slide moves far, rendering the exposure of a steepened scarp behind it, the force imbalance will be transferred backwards and trigger other slides. When a stable slope geometry is eventually formed, i.e. the incremental strain energy and kinetic energy become zero due to plastic deformation, friction, etc., the retrogression stops.

3.2. Investigation of slope retrogression behaviour

3.2.1. Influence of the softening modulus

To further investigate the possible failure modes of the clayey slope, the softening modulus is discussed in this sub-section. Three cases have been analysed, corresponding to softening moduli of $C_0/25.0$ kPa, $C_0/50.0$ kPa (as in the previous section) and $C_0/75.0$ kPa. Other material properties are the same as for the base case. The simulations have again been run until quasi-static equilibrium states have been reached.

Fig. 6 shows the failure process of the slope for a softening modulus of $C_0/25.0$ kPa, in which three things should be noted. Firstly, the time interval between the first and second slides is much longer than for the slope with a softening modulus of $C_0/50.0$ kPa. Secondly, the failure is seen to originate from the middle part of the previous slip circle, not from the slope base, which makes the failed soil volume smaller. Thirdly, the final deposition of the slope is apparently different, with a smaller sliding distance; or, in other words, it poses a smaller risk to the surrounding area if the failure consequence is considered. Fig. 7 shows the final displacement configuration and contours of accumulated shear strain for the slope with a softening modulus of $C_0/75.0$ kPa. The failure process
was found to be similar to that of the slope with a $-50.0$ kPa softening modulus, and is therefore not presented. As seen in the figure, the formation of the last "strange" vertical failure block is due to the boundary condition, which implies a larger sliding distance than the prescribed computational domain.

To conclude, the softening modulus has a strong impact on the slope post-failure behaviour, e.g. the extent of retrogression, sliding distance and number of failure blocks, as illustrated in this case. As the retrogression moves backwards, the released system energy will be gradually counteracted by the soil plasticity (where energy is lost) and friction on the bottom boundary.

### 3.2.2. Influence of residual shear strength

In this sub-section, different residual shear strengths were considered, but with the same softening modulus, $-50$ kPa, to test how the post-failure slope behaviour is governed by the weakest soil strength. The coefficient $\eta$ is the ratio of the residual strength over the peak strength. Apart from the original ratio, i.e. 0.2, used
in the analysis presented in Section 3.1, two other ratios have been chosen, i.e. 0.4 and 0.6, corresponding to residual strengths of 8.0 kPa and 12.0 kPa, respectively. All other material properties and boundary conditions are the same as in Section 3.1, and the simulations were run until quasi-static equilibrium was reached.

Fig. 6. Collapse process of a short soft clayey slope with a softening modulus of 25.0 kPa.

Fig. 7. Final slope configuration corresponding to a softening modulus of 75.0 kPa at $t = 82.5$.

Fig. 8. Final quasi-static slope configurations corresponding to different residual shear strengths; $\eta$ is the ratio of the residual to peak shear strengths.
The final slope geometries are shown in Fig. 8. Fig. 8(a) shows that, when the residual strength is 8.0 kPa (i.e. \( g = 0.4 \)), two sliding soil blocks are formed, leaving a shallow headscarp exposed when the whole system is in equilibrium. Compared to the initial slide, the second slide doesn’t propagate to the slope base and it is smaller in size. The maximum plastic shear strain invariant is observed at the slope base boundary. In Fig. 8(b), where the residual strength is 12.0 kPa (i.e. \( g = 0.6 \)), only one slide occurs, and the accumulated plastic strains are significantly lower than in the other two cases.

3.2.3. Influence of friction at the bottom boundary

The friction angle imposed at the bottom boundary has an influential effect on the propagation of the slope failures, which can alter the failure behaviour. Here two friction coefficients (\( \mu \)) have been considered, equal to 0.1 and 1.0. The softening modulus was set to −25.0 kPa, and the other parameters and geometry are the same as above. The final slope configurations are shown in Fig. 9. The results can also be compared with the results presented in Fig. 6(c), where the friction coefficient is 0.3. It can be observed that, as the friction becomes larger, the number of failure blocks reduces, giving a shorter run-out distance. By comparing Fig. 6(c) with Fig. 9(a), although two blocks are formed in each case, the second failure is deeper into the soil where the friction at the boundary is small due to the soil running out more quickly. The time taken for the slope to reach equilibrium is also seen to increase when the friction is reduced.

4. Retrogressive/progressive failure of a long inclined slope

This section analyses a long inclined slope, comprising a 5 m thick layer of soft clay deposited on top of a sloping bedrock. A cutting has been made at the toe of the long slope which makes the slope unstable. The development of the failure mechanisms is investigated and discussed.

4.1. Collapse process of a long inclined slope

The main slope is inclined at 10° and, to avoid boundary effects, a horizontal section is included at the top of the slope, as shown in Fig. 10. Towards the bottom of the slope, a smaller second slope has been cut to a depth of 3.75 m; this slope is inclined at an angle of 45° to the main slope surface (i.e. 55° to the horizontal). The horizontal section is 15 m long, whereas the inclined slope is over 40 m long (along the line of the slope), with the main (5 m deep) section being over 26 m long. Parameters for the analysis are shown in Table 2. The boundary conditions include rollers at both ends of the domain, preventing horizontal displacement, and a fully fixed bottom boundary which simulates the rough interaction with the bedrock below. As in the previous numerical example, the slope fails under its own self weight. A total of 14,980 material points and 4600 4-node background mesh elements are generated initially. Since the focus of the analysis is the failure modes of the
long inclined part of the slope, to reduce the computational cost
the simulation is only run for 6.0 s, by which time the failure is
fully developed in the inclined part, but not fully developed in
the horizontal part. The small time for the failure to develop along
the whole slope length is due to a number of features of the anal-
ysis. In the main, these are the low residual cohesion and the low
damping coefficient. Once again, an Intel Xeon E5-1620 processor
has been used, with the analysis taking approximately 1 h to exe-
cute on a single core.

Fig. 11 shows the accumulated shear strains at four times dur-
ing the slope failure evolution, in order to provide a comprehensive
explanation of the failure mechanism development. Fig. 11(a)
shows that an initial rotational failure develops at the bottom of
the slope. This slide increases the shear stresses at the bottom of
the soil layer, leading to the propagation of a shear band back up
along the base of the layer, to form the basal failure plane of a large
translational slide, as shown in Fig. 11(b). At the top of the main
slope, a curved slip plane develops, which is linked tangentially
to the basal plane to form one large failure block. Around the same
time, a secondary curved failure plane initiates from the middle
part of the basal plane. Fig. 11(c) illustrates the step-like character
of the slope due to the soil movement along the secondary planes.
The development of the basal failure plane continues, as seen by
the increasing plastic strain invariant along the bottom line. Also,
more secondary failure planes are shown in Fig. 11(d), for the pre-
scribed final time of \( t = 6.0 \) s. It is also seen that the soil mass dis-
locates due to active failures in the downslope area behind the
initial circular slide; that is, soil wedges with shapes of sharp horst
and graben are exhibited.

4.2. Influence of slope angle

For the above slope, a parametric study involving the slope
angle is now presented. All properties and domain details are the
same, except that slope angles of \( 5^\circ, 15^\circ \) and \( 20^\circ \) are now consid-
ered. The corresponding failure configurations are illustrated in
Fig. 12. Note that, in each case, the analysis has been terminated
before the onset of retrogressive failure at the headscarp.

With reference to Figs. 11(d) and 12, all slopes exhibit a trans-
lational failure, as well as a rotational failure at the slope toe. For a
gentle slope angle (i.e. \( 5^\circ, 10^\circ \)), the rotational failure at the toe
occurs prior to the occurrence of the translation; hence the rota-
tional failure can be regarded as the trigger for the whole slope col-
lapse, due to the removal of lateral restraint on, and subsequent
increase of shear stress in, the soil behind. Specifically, for the case
of \( 5^\circ \), after the initial rotational failure, there are a series of retro-
gressive slides moving back up the slope from the toe, which is
not seen in the case of \( 10^\circ \), before the formation of the entire trans-
lational failure of the main slope. There is also secondary move-
ment within the moving mass, resulting in the appearance of
horst and graben.

(a) first critical (rotational) slip surface formed at \( t = 1.75 \) s

(b) translational failure surface formed at \( t = 2.75 \) s

(c) secondary movement occurs at \( t = 4.0 \) s

(d) configuration of the collapsed slope at \( t = 6.0 \) s, revealing a step-like character

Fig. 11. Collapse process of a long inclined slope.
For a steeper slope angle (i.e. 15° as illustrated in Fig. 12(b)), the initial rotational and translational failure planes occur almost simultaneously. Moreover, the moving block in the translational failure is bigger than those in the cases with shallower angles (in terms of failing volumes). For the steepest slope considered (i.e. 20°, Fig. 12(c)), there are three main differences in the failure mechanism development: firstly, slope failure is triggered by the translational slide; secondly, the toe rotational slide does not trigger retrogressive failures, although further slides occur within the moving soil mass due to the global translational failure; thirdly, there are a greater number of secondary slip planes, making individual failure blocks smaller, and all secondary movements are in the form of a series of rotational failures.

5. Conclusion

Retrogressive failures within a clayey soil slope have been analysed using the material point method (MPM) in conjunction with a simple elasto-plastic (Von Mises) softening constitutive model. The results show that MPM is a promising method to simulate slope failures, especially for capturing the post-failure behaviour of slopes. The evolution of the collapse process in a short slope, triggered by the soil self-weight, is provided first. Geometry changes during slope failure are thought to be a main reason for the retrogressive failure mechanism, with the removal of downslope support accounting for the successive slope failures. The failing area/volume gets smaller as the failure moves backwards up the slope, and the time interval between successive failures becomes larger. The results show that multiple retrogressive failures in slope collapse processes are associated closely with the softening modulus and residual shear strength. Friction at the slope base is also shown to have a significant influence on the slope failure development.

A long inclined slope has also been investigated. It has been seen that, as the slope angle increases, the failure mode changes. For a shallow slope, an initial rotational slide seems to trigger the whole slope collapse; then, as the regression moves back up the slope, the appearance of horst and graben (wedge) structures occurs. As the slope gets steeper, successive secondary rotational slides are usually observed, although the sliding distances of individual failure blocks with respect to each other are small compared to the shallower slopes in which the rear scarps of the slides are usually completely free. The failure mode for steeper slopes can be generalised as a primary large translational failure, which propagates down to the base of the soil layer, giving a common basal sliding plane, while, at the same time, secondary failure surfaces give the failed slope a step-like character.

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References
