The mobilization of debris flows from shallow landslides

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Abstract

According to critical state theory, a soil will approach a critical void ratio during shear such that loose soils contract and dense soils dilate. Theory indicates that failing soils must be loose to generate the pore pressures needed for the mobilization of debris flows. Previously published results from large-scale experiments have also suggested that soils must be initially loose to fail as debris flows. In this contribution, this mechanism for soil liquefaction is tested in the field through observations and geotechnical analysis of soils that failed during a large storm in central California. Surprisingly, we find that the debris flows mobilized from soils that were initially dense. In addition, we find that the potential for debris flow mobilization was strongly linked to the fines/sand ratio. We present results from a numerical model that indicate that, as dilatational soils approach the critical void ratio, the arresting effect of negative pore pressures generated by dilation is greatly reduced, leading to a rapid increase in basal pore pressure and rapid downslope acceleration. In addition, the model results suggest that the downslope displacement required to reach the critical state porosity in a dilative soil will be on the order of 0.1 to 1 m. Because the rate of the approach to critical state is fundamentally a function of the hydraulic conductivity of the soil, sandy soils will approach critical state much more rapidly than clay-rich soils.

Keywords: Debris flow; Landslide; Liquefaction; Critical-state porosity; Natural hazard

1. Introduction

By virtue of their velocities and their sizes, debris flows are important geomorphological agents. Debris flows are the most destructive type of landslide and have caused the most deaths (Alexander, 1989) and as the urban fringe around the world continues to encroach into the surrounding foothills and mountains, the potential loss of life and property from debris flows will escalate (Gares et al., 1994). In steep terrain, debris flows are also important sources of sediment to channel networks (Benda and Dunne, 1997; Gabet and Dunne, 2003), altering stream morphology and affecting aquatic habitats (Benda et al., 2003). Over longer time scales, debris flows may control the morphological evolution of steep catchments (Stock and Dietrich, 2003).

Most debris flows begin as rigid translational slides that liquefy (Iverson et al., 1997), but this process is poorly understood. Early attempts to explain this transformation proposed that a failing soil behaves like a Bingham viscoplastic (Johnson, 1996). The Bingham model is attractive because it is characterized by a yield strength, which has been interpreted as “a material property denoting the transition between ‘solid-like’
Several studies have invoked critical-state theory to explain the mobilization of debris flows. Sassa (1984) concluded that this mechanism was responsible for the liquefaction of loose sand in laboratory experiments. In relatively undrained conditions (i.e., when pore water is unable to enter or leave the soil mass rapidly enough), dilative and contractive tendencies of the soil result in changes in pore pressure during shear because water is incompressible. Iverson et al. (2000) demonstrated, through large-scale flume experiments, that the shearing of a loose loamy sand triggered rapid pore pressure increases because, as the soil began to collapse, the weight of the soil was shifted onto the pore fluid. This abrupt increase in pore pressure resulted in immediate liquefaction of the experimental soil. In addition, Iverson et al. (2000) found that the same loamy sand, but densely packed, exhibited dilative tendencies during failure, thus reducing pore pressures and inhibiting mobilization.

During the winter of 1997–1998, Sedgwick Reserve, in the central coast region of California (Fig. 2), lay in the path of a series of El Niño-generated rainstorms. In February, 21 cm of rain fell in a 48-h period (Fig. 3), triggering over 150 shallow soil failures in a 10 km² area vegetated by both coastal sage scrub and grasslands (Gabet and Dunne, 2002). On the grass-covered slopes, about half (56%) of the failures mobilized as debris flows, whereas many of the other failures moved only a few meters before stopping as slumps, sensu Kesseli (1943) (Fig. 4). This presented an ideal opportunity to test the critical-state porosity theory for debris flow mobilization.

We hypothesized that the soils that produced debris flows were contractive and those that produced the slumps were dilative. To test this hypothesis, we analyzed soil samples taken from the sites of a subset of these shallow failures.

2. Methods

2.1. Field site and surveys

The field site, Sedgwick Reserve, is located on the margin of the Santa Ynez valley, ~60 km north of Santa Barbara, CA. The climate is subhumid, Mediterranean. The bedrock is a Pliocene fanglomerate (Dibblee, 1993).

The failures were inspected several weeks after they occurred. In both the slumps and the debris flows, the soil–bedrock boundary defined the failure plane. The failures that mobilized as debris flows completely evacuated the scars (Figs. 4 and 5). In the slumps, the displacement of the center of mass ranged from 1 to 2 m, and the failing mass never overrode the soil surface downslope of the failure (Figs. 4 and 5).

During the spring of 1998, the recent landslides were mapped from aerial photos. Thirty-two of these landslides were surveyed to measure hillslope angle and to estimate volumes of sediment evacuated (Gabet and Dunne, 2002). From this initial set, we selected failures in the grassland that met two criteria. First, to control for the effect of hillslope angle on debris flow mobilization, only failures that occurred on slopes within a narrow range of angles were chosen and the range of slope angles with the highest density of failures was 28° to 32°. Second, to ensure a relatively uniform spatial rainfall distribution, only failures in...
close proximity to each other were chosen. Four debris flows and four slumps within a 1.2-km² area met these conditions. Because the degree of saturation of the soils at failure could have had an important effect on liquefaction potential, the topographic index (Beven and Kirkby, 1979), a measure of potential saturation, was calculated for each failure according to

\[
\text{Top. index} = \ln\left(\frac{a/w}{\tan \theta}\right)
\]

where \( w \) is the failure width, \( a \) is the contributing area upslope of the failure (measured from the margins of the failure), and \( \theta \) is the hillslope angle.

### 2.2. Texture and porosity

A total of two soil samples, taken on either lateral margin of the failure scar (Fig. 4), were collected from each site. For consistency, the sampling depth was approximately halfway between the soil surface and the slip plane (~0.25 m). The sand fraction was measured by...
sieved, drying, and weighing. The silt and clay fractions were determined with a Mastersizer™ automatic particle size analyzer. Particle size data from the two samples taken at each landslide scar were averaged.

Dry bulk densities \((q_b)\) of the soil samples were measured using the resin-coated clod technique (Brasher et al., 1966). Soil sample volumes were measured with saturated samples to account for volume expansion from the presence of shrink–swell clays. The porosity \((n)\) of each sample was calculated with:

\[
n = 1 - \frac{\rho_b}{\rho_m}
\]

where \(\rho_m\), the density of the mineral grains, was assumed to be 2.65 g/cm³.

### 2.3. Constant-shear-drained tests

Soil samples were collected within 1 m of the failure scars of two debris flow failures and two slumps. Samples were obtained by carefully pushing 71-mm diameter, 200-mm-long tubes into the soil at approximately 50 cm depth; soil was then carved away from the outside and bottom of each tube. The samples were collected shortly after a rainstorm to minimize the disruption that might have occurred by pushing the core-tubes into a dry, brittle soil. Although the soil immediately adjacent to the core-tubes may have been compressed while retrieving the sample, the bulk of the sample did not appear to be disturbed by the procedure. Care was taken to ensure that the samples were from areas that had not undergone any strain. The samples were extruded vertically at the UC Berkeley Geotechnical Laboratory and tested using the constant-shear-drained (CSD) method described in Reimer (1992).

The testing of soils to understand their behavior during shallow failures requires a method that mimics the stress field under natural conditions. Shallow landslides are triggered by elevated pore pressures that...
decrease the effective normal stress (i.e., the normal load minus the pore pressure) rather than by an increase in the shear stress (Anderson and Reimer, 1995).

Whereas typical triaxial shear testing is done by increasing the shear stress, the CSD test approximates the conditions during rainfall-induced failure by holding the shear stress constant while reducing the effective stress (Reimer, 1992) (Fig. 6). Stress conditions for the laboratory testing were selected to approximate field conditions. The rate of strain was limited in the samples such that a large number of volumetric strain data were collected after the sample had reached the failure state.

3. Results

The mean topographic index of the debris flows and the slumps (Table 1) are not statistically different (t-test: \( \alpha = 0.05 \)), suggesting that differences in soil moisture from subsurface flow might not explain the different types of failure behavior. Nonetheless, the spatial heterogeneity of the lithology could have caused pooling of water behind patches of soil that had relatively lower hydraulic conductivities.

The mean porosities of the two sets of failures (Table 1) did not show a statistical difference (t-test: \( \alpha = 0.05 \)), suggesting that the two failure types cannot be distinguished on the basis of porosity. This result, however, is not conclusive because soils with different particle size distributions may have different critical porosities (Fear and Robertson, 1995). Nevertheless, this strongly suggests that the initial porosity of a soil may not always be a reliable indicator of its liquefaction potential, a conclusion also reached by Yamamuro and Lade (1998).

The results from the texture analysis indicate that the soils are sandy clay loams. The soils that mobilized as debris flows were significantly sandier than the slumps (t-test: \( \alpha = 0.05 \)) with a sand content of ~45% sand distinguishing the two types of failures (Fig. 7; Table 1). This supports the common observation that differences in the fines/sand ratio may result in fundamental differences in shearing behavior (Xenaki and Athanassopolous, 2003).

In all CSD tests, the effective normal stress was reduced until failure. Initial and failure stress states are shown in Table 2. No attempt at defining a failure envelope was made because the samples, from different

Fig. 5. Surveyed longitudinal profiles of a slump (SF100; top) and debris flow scar (DF9; bottom). Inset: Sequence leading to slumps and debris flows. (A) Incipient landslide (shaded) and failure planes (dashed line). (B) Failure begins at the upper scarp. In some failures, movement was arrested and these formed slumps. (C) Others mobilized as debris flows, completely evacuating the scar.
sites, were considered to be composed of different material with different material properties (e.g., varying particle size distributions, Fig. 7). Unexpectedly, the CSD tests revealed that the soils in both the slumps and the debris flows were dilational under ambient field conditions (Fig. 8). The volumetric increase of the soils during shear is evidence that they were dilating as the soil particles rode up and over each other. But if the increase in pore pressure that leads to liquefaction can only be generated in a soil that contracts, then how does a dilative soil reach a contractive state?

4. Discussion

4.1. Liquefaction of dilative soils

Because CSD testing is complex, requires specialized equipment, and is labor-intensive, only a few studies have applied the CSD test to colluvial soils. Anderson and Reimer (1995) tested a series of colluvial soil samples from the San Francisco Bay Area and observed that they all dilated. In an investigation of debris flow initiation, Anderson and Sitar (1995) performed CSD tests on undisturbed soil samples from hillslopes that had numerous debris flows and also found that the soils were dilative. Field experiments by Harp et al. (1990) have also shown soils dilating during failure. In fact, Anderson and Sitar (1995) con-

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**Table 1**

<table>
<thead>
<tr>
<th>Failure type</th>
<th>Sample</th>
<th>Slope (°)</th>
<th>Topographic index</th>
<th>Porosity (%)</th>
<th>% Sand (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slumps</td>
<td>SF1</td>
<td>31</td>
<td>3.6</td>
<td>0.55</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>SF2</td>
<td>29</td>
<td>4.5</td>
<td>0.48</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>SF100</td>
<td>29</td>
<td>5.3</td>
<td>0.50</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>SF101</td>
<td>28</td>
<td>5.1</td>
<td>0.48</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>Ave. + 1 S.D.</td>
<td>29.3 ± 1.3</td>
<td>4.6 ± 0.8</td>
<td>0.50 ± 0.04</td>
<td>37 ± 6</td>
</tr>
<tr>
<td>Debris flows</td>
<td>DF1</td>
<td>29</td>
<td>5.1</td>
<td>0.53</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>DF6</td>
<td>29</td>
<td>4.6</td>
<td>0.48</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td>DF9</td>
<td>30</td>
<td>4.3</td>
<td>0.56</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>DF100</td>
<td>32</td>
<td>5.1</td>
<td>0.51</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>Ave. + 1 S.D.</td>
<td>30.0 ± 1.4</td>
<td>4.8 ± 0.4</td>
<td>0.52 ± 0.03</td>
<td>51 ± 5</td>
</tr>
</tbody>
</table>

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include that most, if not all, natural colluvial soils will dilate under the stress conditions found in shallow landslides. It is generally accepted that liquefaction only occurs in soils that have a porosity greater than the critical-state porosity (Casagrande, 1936) and that liquefaction of hillslope soils during storm events leads to debris flows (Iverson et al., 2000). In soils well above the critical-state porosity, liquefaction will occur immediately upon slope failure (Iverson et al., 2000). Less well understood is the mechanism by which a soil in an initially dense and dilative state transforms into a contractive soil that may then collapse catastrophically.

Several studies have proposed a mechanism to explain the mobilization of debris flows from dilative soils. Fleming et al. (1989) and Dai et al. (1999a,b) hypothesized that the landslide slides forward a limited distance and stops, its motion inhibited by the dilatation of the soil and the concomitant decrease in pore pressure. The soil, now looser and in a dilative state, absorbs water either from continued rainfall or from water ponding behind the slump (Harp et al., 1990, 2004). As the slumped mass resaturates, pore pressures climb once again, initiating a second failure. Because the soil is now in a dilative state, this second failure leads to the contraction of the soil that produces the rapid spike in pore pressure necessary for liquefaction. This process may include a pause that has been noted in eyewitness accounts. For example, Ellen et al. (1989) provided descriptions of failures that mobilized as debris flows only after a period of time had elapsed after the initial failure. In an analysis of landslides triggered by a typhoon, Harp et al. (2004) reported several instances where the landslide initially failed as a slump, impounded water draining from upslope, and then catastrophically failed as a debris flow. A field experiment by Harp et al. (1990) also produced this type of behavior. Below,

<table>
<thead>
<tr>
<th>Sample</th>
<th>Initial $\sigma'$</th>
<th>Initial $\tau$</th>
<th>$\sigma'$ at failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>SF1</td>
<td>28.1 $\pm$ 0.2</td>
<td>30.4 $\pm$ 0.1</td>
<td>3.4 $\pm$ 0.1</td>
</tr>
<tr>
<td>SF2</td>
<td>19.4 $\pm$ 0.6</td>
<td>21.0 $\pm$ 0.2</td>
<td>3.8 $\pm$ 0.3</td>
</tr>
<tr>
<td>DF1</td>
<td>21.3 $\pm$ 0.4</td>
<td>20.8 $\pm$ 0.2</td>
<td>1.7 $\pm$ 0.3</td>
</tr>
</tbody>
</table>

The testing apparatus experienced some variability in stress readings, values are averages over 5 min, errors are 1 S.D. Time and state of failure is approximate due to variability in stresses through the testing. Data file for SF6 was corrupted and we were unable to recover some data.

Fig. 8. Results from the CSD tests. Arrows indicate the stress paths. The soil samples representing the slumps, SF1 and SF2, exhibited an increase in volumetric strain with a reduction in effective stress, indicating that the soils dilated during shear. Surprisingly, samples from the debris flow sites, DF1 and DF6, were also dilative.
we investigate this mechanism with a numerical model.

4.2. Numerical model

Iverson (2005) presented a model which couples the motion of a sliding block with a parsimonious description of pore water pressures within the block that respond to dilation or collapse of the soil matrix. We combine this model with critical state theory to investigate the mobilization process described above. The motion of the fault block can be described by a balance of driving and resisting forces (Iverson, 2005):

\[
\frac{1}{g} \frac{d^2}{dx^2} = \cos^2 \Psi \left[ \sin(\theta - \Psi) \right] - \left[ \cos(\theta - \Psi) - \frac{p(0,t)}{\rho g H} \cos \Psi \right] \tan \phi
\]

where \( g \) is gravitational acceleration, \( u_x \) is the downslope displacement of the failure block, \( \Psi \) is the dilation angle, \( \theta \) is the slope angle, \( \phi \) is the friction angle, \( p \) is the pore pressure at the base of the slide block, \( \rho \) is the density of the slide block, \( H \) is the thickness of the slide block, and \( \Psi \) is defined as \( \Psi = \frac{d u_z}{d t} \) where \( u_z \) is the vertical displacement due to dilation or contraction of the shear zone.

Following Iverson (2005), the pore pressure at the base of the slide block, \( p(0,t) \), is the sum of an imposed pressure, \( p_i \), which is due to rain infiltration, and an excess pore pressure, \( p_e \), which is due to the dilation or contraction of the soil, such that \( p = p_i + p_e \). Both the excess and imposed pore pressures obey the diffusion equation: \( \frac{\partial p}{\partial t} = D \frac{\partial^2 p}{\partial z^2} \), where \( D \) is the hydraulic diffusivity. The excess pore pressure obeys two boundary conditions: 1) it is zero at the top of the water table (a distance \( z \) above the top of the shear zone, measured normal to the failure plane) and, 2)

\[
\frac{\partial p_e}{\partial z} (0, t) = \frac{\rho w g}{K} \Psi v
\]

where \( \rho_w \) is the density of water, \( K \) is the saturated hydraulic conductivity, and \( v = \frac{du_x}{dt} \) is the velocity of the slide block (Iverson, 2005). The location \( z = 0 \) corresponds to the top of the shearing surface. The imposed pore pressure is increased linearly in the simulations following Iverson (2005) according to \( p_i(t) = p_{crit} + W K Z D \), where \( p_{crit} \) is the critical pressure required for downslope motion to commence (e.g., the pressure when \( \frac{d^2 u}{d x^2} = 0 \)) and \( W \) is a dimensionless rate of imposed pore pressure increase. Eq. (3) is solved using the numerical method described by Iverson (2005), and the equation governing the diffusion of excess pore pressure is solved with a fully implicit finite difference method (Smith, 1986). Our model

![Fig. 9. (A) Period shortly before and after critical void ratio \( (e_c) \) is reached in the shear zone of a sliding block. Model parameters listed in Table 3. As \( e \) approaches \( e_c \), the dilation angle \( \Psi \) approaches zero. (B) As \( \Psi \) approaches zero, the basal pore pressure \( p \) grows beyond the critical pore pressure \( p_{crit} \) necessary to cause downslope motion.](image)
differ from that of Iverson (2005) in our treatment of the dilation angle, $\Psi$, as described below. In addition, we track the evolution of the void ratio in order to extend the analysis to include critical state theory.

It has been found that $\Psi$ changes as the shear zone void ratio approaches the critical state (Li, 1997). As the void ratio in the shear zone approaches the critical void ratio, $\Psi$ will approach zero. In our simulations, $\Psi$ is determined by

$$\psi = \text{sgn}(e_c - e)\psi_0 \left( 1 - e^{-\frac{\|e_c - e\|}{c}} \right)$$

where $\text{sgn}(e_c - e)$ returns the value 1 if $e_c - e > 0$ and $-1$ if $e_c - e < 0$, $\psi_0$ is the initial dilation angle, $e$ is the void ratio, $e_c$ is the critical void ratio, and $\gamma$ is a dimensionless material constant that determines how quickly $\Psi$ approaches zero as the shear zone approaches the critical void ratio. The critical void ratio, $e_c$, is a function of the effective mean stress (Verdugo and Ishihara, 1996), which can be modeled as

$$e_c = e_0 - \zeta \left[ \frac{\rho g H \cos \theta - p(0, t)}{p_A} \right]^{\frac{1}{\lambda}}$$

where $p_A$ (M T$^{-2}$ L$^{-1}$) is atmospheric pressure and $e_0$, $\zeta$, and $\lambda$ are dimensionless material constants. The change in void ratio of the shear zone will depend on its thickness, $l_{sz}$ (L). We assume that there is no volumetric expansion or contraction of the shear zone in the slope parallel direction, such that the volumetric strain is accommodated only by vertical expansion or contraction.
traction. In such a case the void ratio, $e$, is related to the initial void ratio $e_i$ by

$$ e = e_i + \frac{H_y}{L_{sz}} (1 + e_i). $$

(7)

Iverson (2005) found that if the dilation angle $\Psi$ is allowed to decay with increasing downslope motion, a sliding block whose shear zone begins in a dilative state may exhibit behavior similar to the liquefaction behavior that accompanies a sliding block whose shear zone begins in a contractive state. Our model reproduces these results, but we explicitly model the decay of the dilation angle based on the relation of the void ratio in the shearing layer to the critical void ratio. As the shear zone dilates during failure, negative pore pressures are maintained which arrest the downslope movement of the sliding block. When the void ratio in the shear zone approaches the critical void ratio, however, the reduced dilation can no longer dissipate the pressure being generated in the shear zone (Fig. 9), leading to the rapid acceleration of the slide block.

Using the numerical model, we examined the downslope displacement necessary for the void ratio to approach to within a threshold of the critical void ratio $(e_c - e > 0.0001)$, and found that relatively little displacement (~10$^{-1}$ to 1.00 m) is needed for the shear zone to approach the critical state porosity (Fig. 10). The values of $\Psi/\Psi_0$ at the threshold void ratio $(e_c - e > 0.0001)$ ranged from 1.4 × 10$^{-3}$ to 3.5 × 10$^{-3}$, therefore the equivalent $u_{xref}$ distances used by Iverson (2005) are much smaller than the displacement distances to the threshold void ratio. We chose the threshold void ratio because this was the approximate difference between the void ratio and critical void ratio at which rapid acceleration began in our simulations. The parameter values for these simulations are shown in Table 3.

If relatively little downslope displacement is needed to achieve the critical void ratio for even very densely packed soils, why did some of the failures at Sedgwick Ranch mobilize and others did not? We suggest that the difference in behavior is due to the particle size distribution of the soils (Fig. 7). The rate at which the shear zone of a failing block can dilate depends fundamentally on the rate at which water can be forced into the dilating area through negative pore pressures generated in the shear zone. This rate of dilation will determine how quickly a failing soil can approach the critical state porosity. Because sandier soils have greater hydraulic conductivities (Fetter, 1997), pore water in these soils will be able to flow into the dilating shear zone at a faster rate than in clay rich soils, thus a sandy slide block will be able to dilate and approach the critical void ratio more rapidly. In the case of the clay rich slumps, the elevated pore pressures necessary to cause failures induced by rainstorms presumably did not last for a great enough period of time for the slump block to reach the critical void ratio. To illustrate the effect of

Fig. 11. Simulations of two soils with different saturated hydraulic conductivities. (A, B) $K=2 \times 10^{-5}$ m s$^{-1}$. (C, D) $K=2 \times 10^{-6}$ m s$^{-1}$. An order of magnitude reduction in $K$ leads to an order of magnitude increase in the time elapsed between commencement of failure and rapid downslope acceleration. Note different scales on x-axes. Parameter values listed in Table 3.
hydraulic conductivity on the rate of change in the void ratio, we simulated two failing blocks with different hydraulic conductivities. The simulations indicate that a tenfold decrease in hydraulic conductivity results in an equivalent increase in the time elapsed between the initiation of failure and rapid downslope acceleration (Fig. 11). The observation that sandier soils are more apt to liquefy has often been made by others; and it is generally agreed (with some exceptions) that liquefaction potential increases with sand content (Xenaki and Athanasopolous, 2003).

5. Conclusion

In this study, we examine the processes that transform rigid translational landslides into debris flows. Soils adjacent to shallow landslides were subjected to triaxial shear tests designed to simulate realistic changes in the stress field during failure. The results indicate that debris flows mobilized in soils that were dilative under ambient field conditions, an unexpected finding given current theory on debris flow mobilization. With a numerical model, we explore the conditions under which a dilative soil may approach the critical void ratio. We conclude that a particular site’s potential for debris flow mobilization was independent of porosity but sensitive to sand content.

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