Research Article

An Experimental Study on the Hydromechanical Behaviours of the Evolution of Postearthquake Landslide Deposits

Huan Cai, Zong-Ji Yang, Li-Yong Wang, Xiao-Qin Lei, Xiao-Long Fu, Shi-Hao Liu, and Jian-Ping Qiao

1Institute of Mountain Hazards and Environment, CAS, Chengdu 610041, China
2University of Chinese Academy of Sciences, Beijing 100049, China

Correspondence should be addressed to Zong-Ji Yang; yzj@imde.ac.cn

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In this study, a series of experiments of the full-scale physical model was employed to investigate the hydromechanical behaviours of the postearthquake landslide evolution, in forms of rill erosion and shallow headward failure on the rill bank slopes under unsaturated conditions. Soil-water characteristic curves (SWCC) were established using the Brooks-Corey (BC) and van Genuchten (VG) models. The stability of the shallow failure was then analyzed via a one-dimensional and unsaturated stability analysis model of the infinite slope. This measurement revealed that the preferential flow and the matrix flow coexisted when infiltration occurred and the VG model performs better in fitting the SWCC than the BC model. Consistent feedback between stability calculations and experimental observations enables the analysis of mechanisms of rill erosion and slope failure of postseismic landslide under the impact of preferential flow. Furthermore, the seepage-induced internal erosion phenomenon was observed in the experiment. This work thus provides a new perspective on the triggering mechanisms of debris flow during the postseismic period.

1. Introduction

The Wenchuan earthquake in 2008 undermined the geologic stability and thoroughly changed the microgeomorphic conditions in the Longmenshan region of Sichuan province, southwestern China [1–3]. Numerous earthquake-induced landslide deposits have undergone remarkable mass remobilization under heavy precipitation during the postseismic period [4–7]. These deposits are unstable, and the surface runoff caused by heavy precipitation strongly influences the accumulation of landslide deposits in the debris flow source region, where rills gradually cut deep trenches that provide effective free surfaces for both banks of the channel [7, 8]. In such a condition, shallow headward failures have occurred on both sides of the extensional rills and gullies in these landslide deposits. Mass remobilization provided a considerable amount of source materials for the initiation of debris flows, thereby directly causing postseismic debris flow outbreaks in the earthquake-stricken area [9–11] and seriously threatening local reconstruction and safety. Investigations of abrupt topographic transformations, rapid erosional processes, and changes in material source conditions have become new priorities for prevention and management of postseismic debris flow. However, the mechanisms underlying these phenomena remain untested.

Understanding the evolution mechanism of shallow headward failure and rill erosion of earthquake-induced landslide deposits in the headwater valleys caused by heavy rainfall is key to developing an early warning system of postseismic debris flow disasters [12, 13]. The study of rainfall-induced landslides is a fairly complex topic involving a multidisciplinary technology [14]. So far, a laboratory model test is still the most reliable method for studying the rainfall-triggered landslide since the soil properties and boundary conditions can be controlled and the pore water pressures and stresses inside the slopes can be monitored [15]. Many scholars have conducted model tests to study the processes and mechanisms of landslide evolution.
[11, 14, 16–22], and they have made important contributions to our understanding of the mechanisms behind rainfall-triggered landslides. Besides, the earthquake-induced landslide deposit is composed of coarse-textured gravelly soil whose structure is characterized by fracture development, high permeability, and an extremely low fraction of fine particle [23]. Thus, the rapid preferential flow infiltration becomes the most important hydraulic characteristic of earthquake-induced landslide deposits under unsaturated conditions [24, 25]. Consequently, the soil-water characteristic curve (SWCC) and coupled hydromechanical processes of preferential flow are distinctive. However, due to rainfall-induced shallow failure as well as sediment delivery on postearthquake landslide deposits, the hydromechanical behaviour of shallow headward failures triggered by preferential flow in response to rainfall under partially saturated conditions is poorly understood. To date, tests on infiltration flows and hydromechanical behaviours of unsaturated soils have mainly focused on fine-textured soils (e.g., clays and silts) with relatively small particle sizes and pore diameters [26, 27]. There have been relatively less studies on the coupled hydromechanical behaviours of coarse-textured gravelly soils. Moreover, no experimental study of the shallow headward failure of earthquake-induced landslide deposits under rainfall conditions has been reported.

With the development of soil mechanics theories, greater attention has been paid to unsaturated soil materials and their failure mechanisms are inconsistent with the principles and concepts of classical, saturated soil mechanics. Unlike the simpler saturated sands, silts, and clays, these materials puzzled scientists for decades [28]. Because the shear strength of saturated soils could be estimated more conservatively than that of unsaturated soils, the traditional soil mechanics theories have been used for engineering assessments as an orthodox method. Yet, the discrepancy between theoretical assumptions and the actual physical states of soils, together with such factors as rising construction cost, has motivated us to rethink these problems [29]. For example, to assess the stability of slopes with seepage flow more accurately, slope stability analyses have been expanded to include coupled hydromechanical processes under variably saturated conditions [30–35]. These analyses incorporate the variation of saturation, leading to more accurate assessments of the stability of slopes under infiltration conditions and demonstrating that a better physical representation of water flow and stress can be attained in unsaturated soils [35]. The main objective of this work is to integrate the hydromechanical behaviours and failure mechanism of unsaturated landslide material during artificial rainfall model tests.

Based on the patterns of postearthquake mass transportation attributable to the shallow failure mode of the landslide deposits in the region affected by the Wenchuan earthquake, a series of full-scale physical model tests were conducted using the natural, coarse-textured soils from earthquake-induced landslide deposits. With the aid of an unsaturated infinite slope stability analysis model, changes in the one-dimensional unsaturated factor of safety (FS) of the experimental slope with artificial rainfall were computed at first. The experimental and calculation results were then compared and analyzed. By providing experimental data for predicting the infiltration characteristics and stability of landslide deposits composed of coarse-textured gravelly soils with rill erosion and shallow headward failure under unsaturated conditions, this study is aimed at providing new insights into the failure modes and mechanisms of rainfall-induced landslides and debris flows during the postseismic period.

2. Methods

2.1. Model Test

2.1.1. Case Study. The study focuses on the shallow failure mechanisms of the Yindongzi landslide in the Baisha River Basin. This landslide occurred in the Yindongzi Trench, Lianhe Village, Hongkou Township, Dujiangyan county, Chengdu City, Sichuan province, China, which is the meizoseismal area of the Wenchuan earthquake, and it is filled with typical earthquake-induced landslide deposits [13]. The Yindongzi landslide deposit is on the right side of the junction between the initiation zone and the transportation zone of the main gully (Figure 1(a)). The crown elevation of the landslide is 1520 m, the toe elevation is 1352 m, and the relief between the crown and the toe is 168 m. The horizontal projection area is 7.6 × 10^4 m^2, and the slope surface area is 10 × 10^4 m^2. The principal sliding direction is 182°, and the landslide is generally fan shaped. The landslide deposit accumulated in the gully along a slope of 35–42° and with a volume of approximately 31 × 10^4 m^3. The postseismic Yindongzi landslide exhibits a shallow failure mode along the banks of the gully under heavy rainfall. The rill erosion banks of the landslide deposit were chosen for model testing (Figure 1(b)). Soils on the bank surfaces have undergone shallow failure over a long period of rainfall and have continuously expanded on both sides, providing source materials for debris flow initiation in the Yindongzi gully. These materials have become the main source of subsequent debris flows, threatening the safety and property of 260 people in 56 households at the settlement site at the trench mouth (Figure 1(d)).

2.1.2. Test Design. In order to reproduce the rainfall-induced failure process of natural rill banks as realistically as possible, a full-scale (1:1) model was built according to the observed rill banks (Figure 1(b)). The height and length of natural rill banks in the Yindongzi landslide deposits were almost 1–3 m, and the slope gradient was dominantly 60°. The physical model was built in strict accordance with the actual shape of the slope, with a height of 1.2 m, a length of 3.0 m, and a slope angle of 60° (Figure 2). Thus, this model meets the requirements of geometric similarity. There were two measurement lines in the physical model. Two sets of pore water pressure and soil volumetric water content sensors were buried along each measurement line at the heights of 25 cm, 50 cm, 75 cm, and 100 cm from the ground within the 120 cm thick longitudinal section. Land surface tilt meters were deployed on the bank surface at the same height as sensors. A schematic diagram of the layout of the sensors is...
shown in Figure 3. The volumetric water content sensors were numbered from VWC-1 to VWC-8, the pore water pressure sensors were numbered from PWP-1 to PWP-8, and the surface tilt meters were numbered from TS1 to TS8. The experiments were conducted under three rainfall intensity conditions with 170 mm/h, 140 mm/h, and 110 mm/h of...
rainfall. Three sets of tests were conducted as shown in Table 1.

The model tests were conducted on an indoor landslide simulation platform (Figure 4), which is mainly composed of a rainfall system, a landslide model tank, and a measurement system. The artificial rainfall simulator has 36 sprinklers, by which rainfall intensity could be adjusted from 30 mm/h to 180 mm/h. To evaluate the sprinkle uniformity of the artificial rainfall, the coefficient of rainfall homogeneity was introduced. It was originally proposed by Christiansen [36], and the calculation formula is

\[ C_u = 100 \left(1 - \frac{\sum X}{mn}\right), \]  

where \( X \) is the difference between the observed spray intensity and the average intensity value \( m \) at each point; \( n \) is the total number of observation points; \( C_u \) is the sprinkle homogeneity (%). To ensure that water can be sprinkled evenly, \( C_u \) is required to be over 80% [37] and this value is larger than 85% in this test. The length, width, and height of the model tank were 3.0 m, 2.0 m, and 1.2 m, respectively. The framework was made of structural steel, and the base was a flat steel plate. Tank walls were made of transparent, toughened glass, except for the front wall, which was left open for allowing large displacement of landslides and debris flow.

The shallow failure occurred mainly at the depth of 0.5 m, where the gravelly soil exhibited high permeability.

<table>
<thead>
<tr>
<th>Series</th>
<th>Rainfall intensity (mm/h)</th>
<th>Slope angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>170</td>
<td>60</td>
</tr>
<tr>
<td>Test 2</td>
<td>140</td>
<td>60</td>
</tr>
<tr>
<td>Test 3</td>
<td>110</td>
<td>60</td>
</tr>
</tbody>
</table>
Therefore, the seepage observed in the natural rill banks was vertical infiltration with no groundwater rising and failure occurred in an unsaturated condition instead of failure with lateral saturated seepage. Permeable boundary conditions are typical for shallow failure with unsaturated infiltration. To simulate the permeable boundary, 13 rows \( \times 20 \) columns totalling 260 round holes with a diameter of 10 mm were evenly distributed at the bottom of the model tank so that the infiltrating rainwater could freely flow out. Before the experiment, a nylon gauze with a mesh diameter of 1 mm was laid to prevent large particles from leaking. Finally, two shallow troughs of equal size were placed beneath the model tank to collect the mud generated by seepage in the soil. The front view and side view of the physical model are shown in Figure 2.

The measurement system included a data acquisition system (Figure 5(a)) and the following three sensors: pressure sensor (Figure 5(b)), ground surface tiltmeter (Figure 5(c)), and soil moisture sensor (Figure 5(d)). The data acquisition system was a Jiangsu Donghua DH3820 high-speed static strain test analysis system, which was used for collecting and analyzing the voltage signal output of each sensor. The type of pore water pressure sensors was PGM-1KG low-pressure transducer by KYOWA, Japan, which can measure the pore water pressure and soil suction with the installation of a customized porcelain component. The wireless surface tiltmeters were developed by the Chuo Kaihatsu Corporation, Japan, and were compatible with the SCA100T-D01 series 2-axis MEMS tiltmeter module made by Murata [12, 38], Japan, with an accuracy of 0.0025°. The type of soil moisture sensors was EC-5 by Decagon, USA; the measurement accuracy of this sensor for the gravelly soil used in this test is \( \pm 3\% \), and its resolution is 0.25%.

2.1.3. Test Materials. After the Wenchuan earthquake in 2008, years of heavy rainfall caused landslide deposits to become unconsolidated, develop high water permeability, and lose fine particles. To reproduce the preearthquake gradation characteristics of the soil in this region by inferring the grading curves of the analogous landslide deposits from the earthquake-stricken area [7, 11], the material for tests was gravelly soil obtained from the overlying deposit layer of the Yindongzi landslide with a particle size of less than 6 cm, mixed with 5% fine material with a particle size less than 1 mm. A series of geotechnical tests of model materials have been done, including a particle analysis test (for particle size distribution), a triaxial test (for soil cohesion \( c \) and friction angle \( \phi \)), a direct shear test (for soil cohesion \( c \) and friction angle \( \phi \)), infiltration test (for saturated hydraulic conductivity \( k_s \)), and a routine soil test (specific gravity \( G_r \) and dry density \( \rho_d \)). The coefficient of uniformity \( (C_u) \) of the model gravelly soil is 25.79, and the coefficient of curvature \( (C_c) \) is 1.63, indicating a well and continuous particle grading. Although some boulders were removed to improve particle grading, the soil still maintained the same characteristic well grading as the original state, which allowed a...
realistic simulation of the soil structure as well as physical and mechanical properties of unconsolidated landslide deposits. The basic soil parameters are listed in Table 2, and particle gradation of the natural landslide deposit soil, as well as the experimental deposit soil, is shown in Figure 6.

2.1.4. Soil Porosity and Evidence for Preferential Flow. The coarse-textured gravelly soil gathered from the landslide site has a loose porous structure, which enables preferential flow to run through certain pathways [39]. In order to prove the occurrence of preferential flow in the soil, the relationship between soil density and porosity was verified via a special device (Figure 7(a)). By controlling the weight of soil in a limited volume, the soil density ($\rho$) could be set as the goal. When water flows through the pipe into the gravelly soil, the mass would sink and the porosity could be calculated by measuring the added weight of water. The calculation equation and details of this device can be found in an article written by Zhao et al. [40]. The result is shown in Figure 7(b) that the soil porosity is nearly linear with the density under the same water content. The density of model slopes was controlled at a range of 1.6-1.8 g/cm$^3$, and porosity ranged from 36.9% to 26.6% correspondingly. Such a high porosity in gravelly soil indicates the possibility for the generation of preferential flow. Furthermore, dye infiltration experiences have been done to show the flow path in this soil column with densities of 1.6 g/cm$^3$, 1.7 g/cm$^3$, and 1.8 g/cm$^3$ (Figure 8). The height of the soil column was 35 cm, and methylene blue solution was used as a dye tracer so that the flow path could be observed clearly. It can be seen in Figure 8 that preferential flow occurred in all conditions. With the increasing soil density, the depth of initiation for preferential flow increases significantly from 3 cm to 13 cm, indicating more resistance for water flow in denser soil.

2.2. Theoretical Analysis

2.2.1. Soil-Water Characteristic Curve. In this study, the two most commonly used models, the Brooks-Corey (BC) and van Genuchten (VG) models, were utilized to describe the soil-water characteristic curve (SWCC) of the experimental soil. The SWCC can be fitted by the measured matric suction and the corresponding volumetric water content. A relatively simple equation of the soil-water characteristic curve was proposed by Brooks [41] as follows:

$$S_e(Z,t) = \frac{\theta(Z,t) - \theta_r}{\theta_i - \theta_r} = \begin{cases} [\alpha\psi(Z,t)]^{-n}, & \alpha\psi < -1, \\ 1, & \alpha\psi > -1. \end{cases}$$  \hspace{1cm} (2)
Genuchten [42] proposed a smooth, closed, three-parameter soil-water characteristic curve model with the following expression:

\[
S_e(Z, t) = \left\{ \begin{array}{ll}
1 + |\psi(Z, t)|^{-m}, & \psi < 0, \\
1, & \psi \geq 0,
\end{array} \right.
\]

where \( \theta_r \) is the residual water content, \( \theta_s \) is the saturated water content, \( \alpha \), \( m \), and \( n \) are the fitting parameters, where \( m = 1 - 1/n \), \( \alpha \) approximately is equal to the reciprocal of the air entry pressure value and its unit is kPa \(^{-1} \), and \( S_e \) is the effective saturation, \( \psi \) is the matric suction, and \( Z \) is the soil depth.

2.2.2. Infinite Slope Stability Analysis Model. The stability of rainfall-induced shallow landslides is usually evaluated with the one-dimensional limit equilibrium model named as “infinite slope stability model” [16, 27, 43]. The classical slope stability model used to assume saturated conditions in practice; later, Baum et al. [43] and Lu et al. [34] extended this model to unsaturated conditions through coupling suction stress.

The suction stress of the VG model is

\[
\psi(Z, t) = \frac{\left(S_e(Z, t)^{-(1/m)} - 1\right)}{\alpha},
\]

\[
\sigma'(Z, t) = S_e \psi(Z, t)
\]

\[
\frac{\theta(Z, t) - \theta_r}{\theta_s - \theta_r} \frac{\left((\theta(Z, t) - \theta_r)/(\theta_s - \theta_r)\right)^{-(1/m)} - 1}{\alpha}.
\]

The unsaturated infinite slope stability analysis model is

\[
Fs(Z, t) = \tan \phi' \tan \beta + \frac{c' - \sigma'(Z, t) \tan \phi'}{(\gamma_d + \gamma_w \cdot \theta(Z, t))Z \sin \beta \cos \beta}.
\]

Equation (5) is then substituted into equation (6) to obtain a coupled analysis model for unsaturated infinite slope stability by the VG model. In equation (6), \( \sigma' \) is the suction stress defined by Lu and Likos [29] and \( \psi \) is the matric suction; \( \gamma_d \) is the dry weight of soil, \( \gamma_w \) is the volumetric weight of water, \( c' \) and \( \phi' \) are the effective cohesion and effective internal friction angle of soil, respectively, \( \beta \) is the angle of the slope, and \( Z \) is the soil depth.

3. Test Results

3.1. Slope Shallow Failure Mode and Process Analysis. Experimental observations reveal the shallow failure mode of the slope with a gradient of 60°. The typical failure process of the slope is test series 3 shown in Figure 9, which can be generally described as the following steps. Before rainfall started, the slope is stable (Figure 9(a)). With rainwater infiltrating into the slope, part of the slope changes from an unsaturated state to a saturated state at first. After 7 min 58 s of rainfall, the first local failure occurred at the upper right part of the slope (Figure 9(b)). The soil structure began to change accompanied by the water content and pore water pressure variation. Cracks occurred on the upper left part of the slope and fine particles migrated a lot at the surface (Figure 9(c)). Then, cracks extended and the first general failure occurred, leaving a scrape in the back edge of the failure (Figures 9(d) and 9(e)). After 18 min 57 s of rainfall, the second general failure occurred in the middle of the slope and the total area of failure enlarged (Figure 9(f)). Rainwater accumulated in gullies on the slope surface, and the runoff brought particles away (Figure 9(g)). The failure area continued to enlarge.

Figure 7: (a) Diagram of the soil porosity measurement device and (b) relationship between soil porosity and density.
until fine particles were washed away. Finally, the slope tended to be stable and the failure area no longer enlarged (Figure 9(h)). The typical failure model of the slope with a gradient of 60° can be summarized as local failure expansion causes large-scale overall collapse.

3.2. Soil Moisture and Matric Suction Changes. The variations of matric suction and volumetric water content in test series 1-3 under rainfall intensities of 170 mm/h, 140 mm/h, and 110 mm/h are shown in Figure 10. The responses of volumetric water content and matric suction are used in this study to
Figure 10: Continued.
represent the roles of preferential flows and matrix flows. Due to the appearance of preferential flows, the volumetric water content rapidly increases and the soil moisture redistributes while the matric suction has not responded yet and therefore exhibits delay. The role of matrix flows is reflected in the reduction of matric suction caused by water infiltration in the soil matrix (capillary) and is observed as a delayed response of the matric suction compared to that of soil-water content. As shown in Figures 9(a)–9(c), the volumetric water content of the soil kept nearly constant at the beginning of rainfall in all tests. After 4-34 minutes of rainfall, the volumetric water content at each measuring point in the slope increased sharply by the sequence of soil layers. During this period, the rainwater infiltrated through large pores and fissures in longitudinal sections and it finally remained at 19.7%-48.4% and the matric suction decreased almost simultaneously after the volumetric water content increased. It is worth noting that the delay of matrix flow did not occur at soil depths of 25 cm and 50 cm, which indicated that both preferential flow and matrix flow formed almost simultaneously in shallow soil layers. Therefore, tests revealed that the seepage characteristics and processes of headward failure of the landslide deposits resulted from the interaction of a double-seepage field of coexisted preferential flow and matrix flow.

3.3. Soil-Water Characteristic Curve. Under heavy rainfall conditions, the preferential flow can rapidly flow through fractures and macropores causing rapid increases of water content and pore water pressure, thus affecting the infiltration process and the stability of the bank slope. To understand the influence of preferential flow on the stability of landslide deposits under different rainfall intensities, it is necessary to first establish the soil-water characteristic curve representing the impact of preferential flow. The soil-water characteristic curves described with the BC model and the VG model can be fitted with the measured matric suction and volumetric water content at different depths during the wetting process. Curve fitting is done in Origin software with its analysis and fitting module, in which user-defined functions can be created (e.g., BC model and VG model) and the fitting curve can be accomplished automatically according to the fitting data and the input expression. The fitting algorithm of Origin is based on a nonlinear least squares method, and fitting results and graphs can be exported when the fitting process is over. As shown in Figure 11, the VG model has higher goodness of fitting than the BC model and its correlation coefficient $R^2$ is larger than 0.9. In contrast, the BC model is less accurate in fitting SWCC ($R^2 < 0.75$). Therefore, the VG model can better fit the soil-water characteristic curve of coarse-textured gravelly soils.

(c)

Figure 10: Matric suction and volumetric water content changes under different rainfall conditions: (a) test 1, $I = 170$ mm/h; (b) test 2, $I = 140$ mm/h; (c) test 3, $I = 110$ mm/h.
Table 3: Fitting parameters for wetting process at different depths in test series 3.

<table>
<thead>
<tr>
<th>Fitting model</th>
<th>Depth (cm)</th>
<th>$\alpha$ (kPa$^{-1}$)</th>
<th>$n_{vg}$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Van Genuchten</td>
<td>25</td>
<td>0.6067</td>
<td>68.6838</td>
<td>0.9947</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.4609</td>
<td>175.0497</td>
<td>0.9639</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>0.5369</td>
<td>9.2780</td>
<td>0.9261</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.4749</td>
<td>77.3908</td>
<td>0.9765</td>
</tr>
<tr>
<td></td>
<td>Depth (cm)</td>
<td>$\alpha$ (kPa$^{-1}$)</td>
<td>$n_{bc}$</td>
<td>$R^2$</td>
</tr>
<tr>
<td>Brooks-Corey</td>
<td>25</td>
<td>0.0081</td>
<td>0.0706</td>
<td>0.1766</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.1326</td>
<td>0.3233</td>
<td>0.5256</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>0.3656</td>
<td>0.5348</td>
<td>0.5291</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.1314</td>
<td>0.5786</td>
<td>0.7244</td>
</tr>
</tbody>
</table>

Figure 11: SWCC at different depths in test series 3: (a) $d = 25$ cm; (b) $d = 50$ cm; (c) $d = 75$ cm; (d) $d = 100$ cm.
under the impact of preferential flow. Fitting parameters are shown in Table 3. Because of model limitations, the fitting goodness of the BC model is low when moisture reaches a relatively high content and the advantage of the BC model is that it requires fewer parameters with a clear physical meaning. However, because the fitting curve of the BC model does not have inflection points, the result is a discontinuous fitting. As a consequence, the curve is less representative in fitting the high-water-content state (near saturation) or the quick response of preferential flow. Moreover, compared with silt and clay, it can be seen that coarse-textured gravelly soil has a lower matric suction [29, 44]. The air entry pressure under the VG model ranges from 1.4 to 2.2 kPa, which is relatively low. Due to the scarcity of fine particles and the large percentage of coarse particles (e.g., sand and gravel), a low air entry value was observed, which is similar to the results of a recent study by Yang et al. [11] on the soil-water characteristic curve of coarse-textured, gravelly soil of landslides in the Wenchuan earthquake area. In this study, the volumetric water content at which matric suction is near zero in the VG model was defined as the “relative saturated water content.”

3.4. Stability Analysis. Stability of bank slopes is related to the gravity and the matric suction of the unsaturated soil. Under unsaturated conditions, the changes in water content caused by rainfall infiltration result in changes in the moisture and matric suction of the soil and thus greatly influence slope stability. Because rainfall-induced landslides occur under the influence of water infiltration, the wetting process is usually more important in describing the physical processes of slope failure. Meanwhile, the camera recorded the behaviour and timing of deformation during experiments and the surface tiltmeter was able to accurately capture the tilt angle changes of the soil at measuring points. By comparing the slope failure processes under different rainfall intensities with the rainfall duration, it was found that the surface inclination variations match with the failure stages observed.

The relationship between the surface inclination and the factor of safety (FS) calculated by the VG model during the infiltration process in all tests is shown in Figure 12. The FS calculated in tests is between 1.04 and 0.97, and a value below 1.0 indicates bank instability. In the case of rainfall intensities of 170 mm/h, 140 mm/h, and 110 mm/h, after 5 min 30 s, 11 min 51 s, and 13 min 51 s of rainfall, the first general failures occurred in test series 1, 2, and 3, respectively. The moment when the FS begins to drop coincides with that of the failure initiation, and they completely fail when FS drops to less than 1.0 at different depths except for TS1-FS-1, TS2-FS2, and TS4-FS4 in the test. The reason for the discrepancies between FS and inclination of TS1, TS2, and TS4 is that the embedded sensors at the same level may not detect the water content and matric suction changes because the wetting front in the slope has not yet reached the depth while shallow failure occurred on the surface. Meanwhile, the upper shallow slide on the slope also pushes the lower tilt sensors and causes its fall. The negative tilt angle means that the tipping direction of the sensor was the opposite of the defined positive direction. Therefore, a change of the tilt angle from a negative value to a positive value shows that tilt meters tumbled during rapid sliding. After the deformation was complete, the tilt angle stabilized and remained unchanged thereafter. For example, under a rainfall intensity of 140 mm/h (Figure 12(b)), the tilt angles of the x-axis of TS1 and TS5 tilt meters began to change suddenly after 10 min 53 s of rainfall, corresponding to the first local failure on the right side of the model. Immediately following this, the x-axis tilt angles of TS4 and TS8 tilt meters change abruptly. The second general failure occurred on the left side of the slope at 11 min 15 s, and the tilt angles of the x-axis and y-axis stabilized after 13 minutes of rainfall. At this time, the tiltmeter was completely overturned and the tilt sensors were buried in the soil and no longer moving. Inclination changes recorded by the surface tilt meter indicate bank failure at the corresponding positions, which can be utilized as an early warning index and needs further study.

Combining Figures 10 and 12, it can be seen that the variation of the FS at different depths is related to the performance of matric suction and soil moisture. FS tends to be stable below 1.0 when matric suction bottoms out and the soil moisture reaches the highest point. Although the influence of matric suction on the factor of safety is very limited (maximum decrease of 0.07), this also implies that the matric suction, as a meaningful component of the effective stress of unsaturated soils, contributes to the stability of shallow rill banks to a certain extent. The soil mechanics parameters for the FS calculation are shown in Table 4.

3.5. Change in Particle Size Distribution. The seepage erosion causes changes in the soil pore structure, seepage field, and stress field and is one of the failure mechanisms in rainfall-induced landslides [45]. To study particle migration caused by rainfall infiltration, soil samples were taken at equal intervals and equal penetration depths (30 cm) at different locations along the same cross-section at 0 cm, 30 cm, and 60 cm (upper, middle, and lower sections, respectively) from the top surface of the bank after the experiment shown in Figure 13. Three samples were collected per section, and there were nine soil samples in total. In order to assess the average changes of particle migration in depth, samples from the same soil layer were mixed for particle analysis. The characteristic particle size changes of each soil layer under the rainfall intensity of 140 mm/h are shown in Figure 14. The effective diameter $d_{10}$ of the upper, middle, and lower soil samples increased from 0.17 mm to 0.23 mm, 0.18 mm, and 0.19 mm, respectively. The continuous particle size $d_{50}$ increased from 1.10 mm to 1.80 mm, 1.65 mm, and 1.45 mm, respectively. The limiting particle diameter $d_{60}$ increased from 4.40 mm to 6.10 mm, 6.40 mm, and 6.00 mm, respectively. The increase in particle size of the upper, middle, and lower sections verifies the occurrence of fine-grained soil particle migration in the landslide deposits during rainfall infiltration (i.e., the occurrence of seepage erosion). Changes in the characteristic particle size can reflect the vertical migration of fine particles from top to bottom during rainfall infiltration. The $d_{30}$ and $d_{60}$ of the lower section are smaller than those of the middle and upper sections, indicating that the loss of fine particles in the lower section is less pronounced.
Figure 12: Continued.
than that in the upper and middle sections. The particle gradation curves of the original soil sample and the soil sample from each layer are given in Figure 15. The cumulative percentage of particles by mass with a diameter of less than 1.0 mm in the upper, middle, and lower sections of the deposits decreased from 29.1% in the original soil sample to 25.7%, 26.6%, and 27.6%, respectively. The cumulative fraction of particles by mass with a diameter of less than 1.0 mm in the lower soil sample is 1.9% and 1.0% greater than those in the upper and middle sections, respectively. This directly reflects the loss of fine particles in each layer of soil, where the proportion of coarse particles increases as the fine particles migrate.

Figure 16 demonstrates real-time rainfall intensity and cumulative rainfall in all tests. It can be seen that the amount of the cumulative rainfall in test series 1-3 are 101 mm, 118.2 mm, and 76 mm with the rainfall duration of 38

<table>
<thead>
<tr>
<th>Depth Z (cm)</th>
<th>Effective cohesion $c'$ (kPa)</th>
<th>Effective friction angle $\phi'$ (°)</th>
<th>Natural unit weight $\gamma_s$ (kN·m$^{-3}$)</th>
<th>Slope angle $\beta$ (°)</th>
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<tr>
<td>25</td>
<td>1.5</td>
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<td>100</td>
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minutes, 50 minutes, and 53 minutes, respectively. After test series 2, a total dry weight of particles with a diameter being less than 1.0 mm of 4.579 kg was gained from the collected slurry. Under rainfall intensities of 170 mm/h and 110 mm/h, the final particles collected with a diameter being less than 1.0 mm had dry weights of 4.659 kg and 3.997 kg, respectively. Therefore, although the rainfall intensity decreases in test series 2 compared with test series 1, there is a considerable increase in the rainfall duration and this greater abundance of accumulated rainwater causes the fine particle to migrate more efficiently, suggesting that fine particle migration is positively correlated with both rainfall duration and cumulative rainfall.

4. Discussion

The variations of rainfall intensities in the experiment were not significant for all the tests except for the first 10 minutes of test 3; consequently, the cumulative rainfall showed approximately linear increases with time (Figure 16). With the rainfall intensities of 170 mm/h, 140 mm/h, and 110 mm/h, the critical cumulative rainfall amounts at which the tilt meter detected an abrupt change were 15.82 mm, 25.63 mm, and 14.31 mm, respectively. The rainfall thresholds in test 3 did not rise with smaller rainfall intensity or longer rainfall duration compared with those in test 1, which can be explained by the slope infiltration theory proposed by Horton [46]. For an initially dry and well-drained soil layer, the infiltration capacity is relatively high at the beginning of rainfall, with vertical infiltration at a steady rate. Over time, the infiltration rate decreases by orders of magnitude and eventually converges upon a constant. On a sloped surface such as a rill bank, if the rainfall rate is greater than the water uptake capacity of the soil, the remaining water will flow along the ground surface (i.e., “excessive rainfall” results in surface runoff) [46]. The rainfall infiltration rate is usually much slower than that of the surface runoff. When rainfall exceeds soil absorption capacity, the runoff will form simultaneously along all banks of the basin or drainage basin. In test 1, the actual rainfall intensity is 162.6 mm/h \((4.52 \times 10^{-3} \text{ cm/s})\), which is about twice as large as that in test 3 \((2.44 \times 10^{-3} \text{ cm/s})\). However, the hydraulic conductivity of the soil in model tests is only \(1.67 \times 10^{-3} \text{ cm/s}\) (shown in Table 2). Under the high-intensity rainfall of test 1, rainwater cannot infiltrate completely in time because rainfall intensity far exceeds the soil infiltration capacity, resulting in the runoff, which causes erosion on the slope surface and reduces the strength of the slope. Conversely, under the low-intensity rainfall of test 3, it approaches the infiltration capacity of the soil and the soil weight rapidly increases after the rainwater infiltration, resulting in the shallow failure.

The phenomenon of grain coarsening due to vertical migration of the fine particles was observed in the tests, which also has been mentioned in other pieces of research [8, 47–49]. In these researches, the displacement or mobility of soil with different gradings was modeled and observed. Results showed that the mobility of soil tended to be lowered with the decreasing of fine particles, suggesting that grain
coarsening improved the resistance of mass remobilization. Lei et al. [45, 50] pointed out that the relatively impermeable layer caused by fines blockage leads to the generation of positive excess pore pressure and initiates the slope failure. However, the pure erosion (i.e., absolute fines migration) generally increases the soil porosity and facilitates the rainfall infiltration, which advances the grain coarsening degree in soil. To some extent, the grain coarsening linked with pure erosion in soil is beneficial for slope stability and this phenomenon may be one of the reasons for explaining the geohazard mitigation and posteffect in earthquake regions.

5. Conclusions

This paper focuses on the phenomena in which earthquake-induced landslide deposits in the debris flow source region undergo rill erosion and failure under heavy rainfall conditions and are transformed into material sources for future debris flows after major earthquakes. The Yindongzi landslide in the Baisha River Basin, a typical meizoseismal area in the mountainous area of Sichuan province, southwestern China, was used as an example. A model test of shallow failures of rill banks was conducted indoors on a full-scale slope with a steep gradient under heavy rainfall conditions, using natural soils from the earthquake-induced landslide deposits. Rainfall infiltration characteristics were integrated with the hydromechanical coupling mechanism to investigate the evolution, outbreak, and propagation of rill bank failure under heavy rainfall. The conclusions are as follows:

(1) Through experimental observations, the typical failure model of the unsaturated gravelly soil slope with a gradient of 60° can be summarized as local failure expansion causes large-scale overall collapse. With rainwater infiltrating into the slope, part of the slope changes from an unsaturated state to a saturated state at first. Secondly, the soil structure begins to change. The soil moisture and pore water pressure vary

Figure 16: Real-time rainfall intensities and cumulative rainfall in (a) test series 1, (b) test series 2, and (c) test series 3.
continuously and fine particles migrate both in the slope and at the surface. Then, soil strength reduces and failure initiates. The localized failure begins to occur and expands to large-scale, general slope failure later on, forming scarp at the back edge of the failure. Finally, the slope tends to be stable and the failure area no longer enlarges.

(2) The seepage characteristics of rainfall-induced slope failure in tests are the result of the interaction between preferential flow and matrix flow. The matrix flow is affected by matrix suction, which shows a significant delay compared with the occurrence of preferential flow in soil layers of 75 cm and 100 cm in depth. The VG model provides a better fit to the soil-water characteristic curve, which reflects the dominance of preferential flow in coarse-textured gravelly soils \((R^2 > 0.9)\). Tests reveal that coarse-textured gravelly soils have lower matrix suction and air entry pressure values. Such soils exhibit unique properties because of their structural characteristics, namely, lower fine particle content, larger pore size, and wider particle size gradation when compared with clay and silt.

(3) Calculation results in FS show that the matrix suction can contribute to the stability of shallow rill banks under unsaturated conditions. Sudden changes of the inclination captured by tiltmeters are well correlated with a reduction in the FS. Therefore, the inclination can be utilized as an index for early warning.

(4) The experiments verified the vertical migration of fine particles caused by rainfall infiltration. Statistics reveal that a significant increase in either rainfall duration or cumulative rainfall amount enables fine particle to migrate fully.

This study presents a new perspective on the formation mechanisms and the source transformation conditions of debris flows induced by rainfall after earthquakes. Further, it provides a framework for the development of early warnings for rainfall-induced landslides and debris flows during the postseismic period.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare no conflict of interest.

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