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Research Article

Effect of Rock Particle Content on the Mechanical Behavior of a Soil-Rock Mixture (SRM) via Large-Scale Direct Shear Test

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The mechanical strength of the landslide deposits directly affects the safety and operation of the roads in the western mountainous area of China. Therefore, the research is aimed at studying the mechanisms of a landslide deposit sample with different rock particle contents by analyzing its characteristics of the stress-strain behavior, the "jumping" phenomenon, the volumetric strain, and the shear strength parameters via a large-scale direct shear test. Stress-strain results show that stress-strain curves can be divided into 3 different stages: liner elastic stage, yielding stage, and strain-hardening stage. The shear strength of SRM behaves more like "soil" at a lower rock particle content and behaves more like "rock joints" at a higher rock particle content. Characteristics of the "jumping" phenomenon results show that the "intense jumping" stage becomes obvious with the increasing rock particle content and the normal stress. However, the lower the rock particle content is, the more obvious the "jumping" phenomenon under the same normal stress is. Volumetric strain results show that the sample with a lower rock particle content showed a dilatancy behavior under the low normal stress and shrinkage behavior under the high normal stress. The dilatancy value becomes smaller with the increasing normal stress. The maximum shear stress value of the rock particle content corresponds to the maximum value of dilatancy or shrinkage. We also conclude that the intercept of the Mohr failure envelope of the soil-rock mixture should be called the "equivalent cohesion," not simply called the "cohesion." The higher the normal stress and rock particle content are, the bigger the equivalent cohesion and the internal friction angle is.

1. Introduction

The Quaternary loose deposits, some researchers also called them the soil-rock mixture (SRM) [1–3], are widely developed and distributed in the western mountainous area of China [4]. The mechanical strength of the soil-rock mixture directly affects the safety and operation of the road section of the area, which is closely influenced by many factors, such as particle shape, grain size distribution, arrangement, uniformity, sorting, surface roughness, order of structure, clarity of bedding, density, and porosity [5, 6]. Due to the complexity of its components and being easily affected by the external environment, such as rainfall, loads and human activities, the soil-rock mixture is heterogeneous, inhomogeneous, nonlinear, anisotropic, spatially variable [7], and environmental dependent, which means that their physical, mechanical, and engineering properties are quite different and complex.

The soil-rock mixture with different rock particle contents may form different soil skeleton structures, densities, and microporosities, leading to different stress-strain behaviors. Rock particle content can have a significant impact on its shear strength. Researches on the experimental mechanical properties of the soil-rock mixture are mainly based on the large shear test, such as the large direct shear test, large triaxial test, and large in situ test. Zhang et al. [8] studied the mechanical behavior of a soil-rock mixture in the Nuozhadu embankment dam in depth through two large-

scale compaction test fields and a series of in situ direct shear tests. In situ direct shear tests on three samples of rock and soil aggregate in the Three Gorges reservoir area were carried out by Li et al. [9] to examine its mechanical behavior. Gao et al. [10] found that the strength parameters of Zhangmu deposit were correlated with the block proportion by weight (WBP) through in situ SRM and constant-head injection tests on a large scale. Coli et al. [11] studied the bimrock strength parameters via a nonconventional in situ shear test apparatus. The effects of physical properties of Quaternary deposits on the strength parameters of soil in north of Esfahan are studied by Dadkhah et al. [12], and the results showed that all parameters of soil strength and modulus increased with increased dry density. A series of oedometer and triaxial tests were conducted on clay-steel bead and clay-fine gravel mixtures by Kang [13] to understand the effects of coarse particles on the mechanical behavior of clay-aggregate mixtures. Campolunghi et al. [14] identified units characterized by specific geomechanical behaviors within some Holocene alluvial deposits in Rome. Wierzbicki and Stefaniak [15] presented an analysis of geotechnical parameters of the alluvium with a view of using the soil as an earth construction material and as a foundation for buildings constructed on the grounds. Afifipour and Moarefv [16] analyzed Young's modulus, failure strain, and stress-strain behavior of bimrocks with high rock block proportion by a servo control machine through the uniaxial compression tests. A series of large-scale direct shear tests were conducted by Houzhen et al. [17] to study the influence of different water contents and different grain size distributions *s* on the mechanical properties of the SRMs.

At present, a lot of data on stress-strain behavior of SRM have been reported in the published literature. However, there are few articles on the detailed analysis of stress-strain characteristics and volume-strain characteristics in the shear process combined with the shear "jumping" phenomenon considering different rock particle contents. For the soil-rock mixture, the "jumping" phenomenon is a very important feature worth further study, especially for the different rock particle content samples, and there is little analysis of the "jumping" phenomenon in existing studies. Only Deng et al. [18] simply analyzed the shear leap phenomenon of a soil-rock mixture in the Three Gorges Reservoir area in a Chinese paper. Therefore, the shear behavior of landslide deposit samples used for subgrade in the western mountainous area of China is still needed to be studied and further investigated. This paper is aimed at studying the mechanisms of the soil-rock mixture by taking into account the influence of different rock particle contents through the large-scale direct shear tests.

2. Materials

The experimental sample was taken from one of the loose deposits in Lueyang County of Hanzhong City, Shaanxi Province, Western China (Figure 1), in which landslide deposits were widely distributed.

Its components were mainly the mixture of weathered rock and soil (Figure 2). Because of frequent and heavy rainfall there, the stability of the landslide deposit slope was easily affected by the bad conditions. Due to the large size of rock particles, it is difficult to test its density by a general method, so the natural density was measured by the water-filling method. The natural density ranges between 2.15 and 2.37 g/cm^3, and the natural water content ranges between 1.6% and 2.6%. Two groups of deposit samples were taken from the site, and sieve tests were carried out based on Test Methods of Soils for Highway Engineering/JTG E40-2007 of China [19]. The grain size distributions of the natural samples are shown in Figure 3. The coefficient of uniformity C_u and coefficient of curvature C_c were also calculated ($C_u = 133.11$, $C_c = 11.14$). Even though by the uniformity coefficient criterion this sample is well graded, it fails the coefficient of curvature criterion. Therefore, it is indeed poorly graded. The liquid limit and plastic limit of fine-grained soil less than 0.5 mm are 38.05% and 19.14%, respectively.

Figure 1: Landslide deposits in Lueyang County.

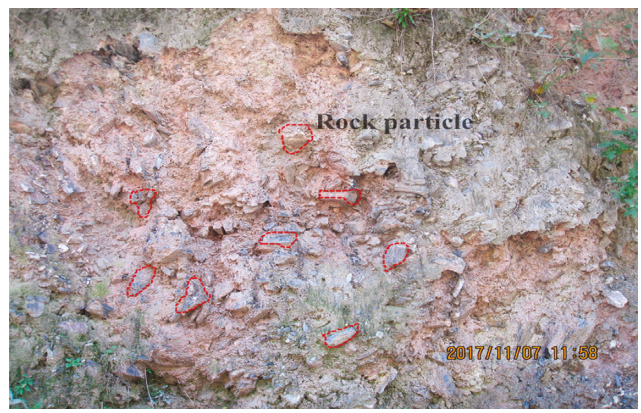


Figure 2: Soil and rock particles mixture of the sampling spot.

Two important factors should be concerned for SRM in the direct shear test. One is how to determine the threshold value of soil and rock, namely, how to divide the "soil" and "rock," which has a great effect on the physical and mechanical properties of the landslide deposits. According to the previous research, there are no good methods to determine the threshold value. In most cases, the value is reasonably determined by theory and practical experience, but the widely accepted

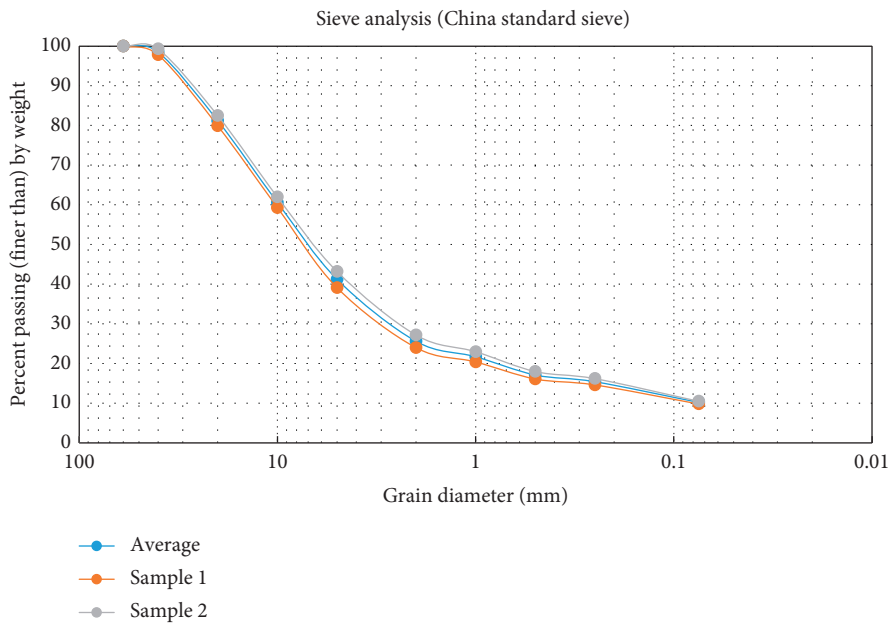


Figure 3: Grain size distributions of the natural samples.

method was put forward by Lindquist et al. [20, 21], which was also adopted by Xu et al. [22]. Therefore, we also use the criteria for "separating" soil and rock of the landslide slope as follows:

$$d_{\text{thr}} = 0.05L_c, \text{ if } d > d_{\text{thr}}, \text{ we regard it as "rock particle"; otherwise, it is "soil,"} \quad (1)$$

where d_{thr} is the threshold value of soil and rock, d is the diameter of the rock particle, and L_c is the characteristic engineering scale of SRM and is an index which varies according to the working scale of the engineering problem under investigation: to a study plane in 2D, L_c is equal to the square root of the study area; to a tunnel, L_c is equal to its diameter; to a triaxial sample, L_c is equal to its diameter; to a direct shear sample, L_c is equal to the height of a single shear box, while on a slope, L_c is equal to its height. Combined with the height of a single shear box and the limitations of the test equipment and reference to other researchers, we finally took 5 mm as the threshold value of SRM (landslide deposits).The other is how to deal with over-size rock particles. As we know, the size of rock particles in the landslide deposits has a wide range of changes, so it is impossible to include all the sizes of rock particles in our test sample. According to the ASTM standards and the limitations of the equipment used in this paper, we determine the maximum size of the rock particle is 60 mm. For those oversized rock particles, there are three commonly used methods at present: the "scalping method," the "replace method," and the "parallel gradation" (JTG E40-2007). Because the "large" rock particles play a very important role in the mechanical behavior of landslide deposits, we finally chose the "replace method," considering the impact of rock particles.

According to the results of sieve analysis, the rock particle content of the natural landslide deposits ranges from 56% to 79%. To study the influence of different rock particle contents on the mechanical behavior of the landslide deposit sample, we designed shear tests with 5 different rock particle proportions (35%, 45%, 55%, 65%, and 75%) of reconstituted samples with same water content (5%). All specimens were tested under four different normal stress conditions (141.5 kPa, 283.1 kPa, 424.6 kPa, and 566.2 kPa). Table 1 shows the basic parameters of group samples.

3. Experimental Apparatus and Test Procedures

3.1. Experimental Apparatus. The equipment used in this test is a large-scale direct shear test machine, developed by Huaxi Institute of Geotechnical Instruments. The equipment is a special device for stress loading and used for measuring the shear strength and deformation of coarse soil with a maximum particle size of 60 mm. The sample size is 300 mm in diameter and 300 mm in height. The apparatus consists of the main body, the measurement operation control system, and the data acquisition system. Figure 4 shows the photo of the large-scale direct shear test apparatus, which is mainly composed of a closed frame host with vertical and horizontal loading cylinders, an upper and lower shear box (a circular shear box is used here because the stress condition and distribution of the circular shear box are better than that of the square or rectangular), a lower shear box displacement rolling mechanism, a permeable plate, a transfer plate, a slit ring, and a slotted roller. The measurement system consists of vertical and horizontal load sensors and two digital display instruments and 3 vertical and 1 horizontal displacement gauges. Operating system mainly includes vertical and horizontal load digital display instrument, hydraulic and pneumatic control components, operation device, hydraulic station and mercury control system, the direction of the hydraulic reversing system, pressure regulation unit, gas-liquid conversion system, and pressure gauge, tube, and shell. The data acquisition system includes computing

Table 1: Basic parameters of group samples.

Rock particle content (%)	35	45	55	65	75
Maximum dry density (g/cm^3)	2.23	2.25	2.26	2.27	2.25
Optimum water content (%)	5.8	4.7	4.5	4.3	3.9
Dry density ρ_d (g/cm^3)	2.214	2.206	2.197	2.209	2.189

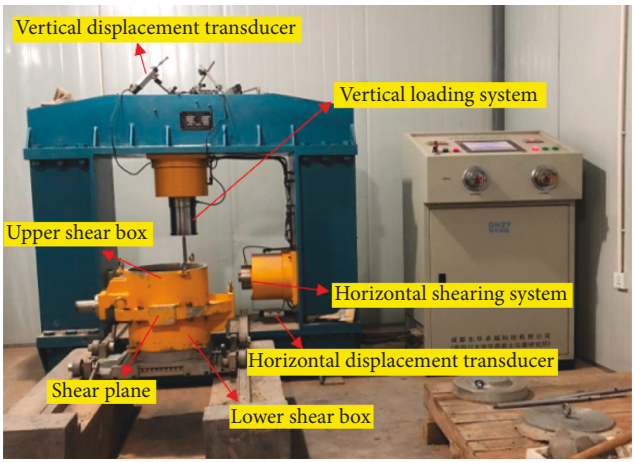


Figure 4: Photo of the large-scale direct shear test apparatus.

sensors, A/D converters, signal amplification, conditioning, and transmission systems. All test parameters are automatically collected and stored by a piece of special software, which also can display timely graphics. After the test is completed, the data collected by the computer are processed accordingly.

3.2. Test Procedures. Before the start of the test, all the samples should be ready. The test process is described as follows:

(1) *Sample Filling.* The control switch of the operating platform should be closed before the sample is completed. The lower shear box plate from the bottom of the frame is pushed out, and the horizontal roller is placed on the plate, and then the lower shear box is placed on the roller with the convex side of it pointing to the horizontal cylinder. Then, the permeable plate, the circular filter cloth (or permeable nonwoven fabric with the same size of shear), the slotting ring, and the slotting roller are placed on the bottom of the lower shear box. The upper shear box is installed on the lower shear box correctly so that the positioning pins can insert the pin holes accurately. Sample filling should be in accordance with the "Test Methods of Soils for Highway Engineering/JTG E40-2007" and ASTM requirements of layered filling and compaction, and then the permeable plate and the force transmission plate are placed on the surface of the specimen. Finally, the shear box is pushed to the bottom of the frame, and the slotting ring and the positioning pins are removed.

(2) *Precontact.* The vertical and horizontal displacement meters are installed after the sample filling is finished, and then the sample precontact is followed. First is the vertical precontact: pull the bar near 950—perform motor adjustment—vertical cylinder loading (when the vertical load on the computer is 0–5 kN)—stop. It should be noted that the interval between the execution of the motor and the loading of the cylinder cannot be too short. If the oil cylinder is loaded without reaction, the bar is pulled to the left end and the motor adjustment is done again. The horizontal precontact operation is similar to vertical precontact.

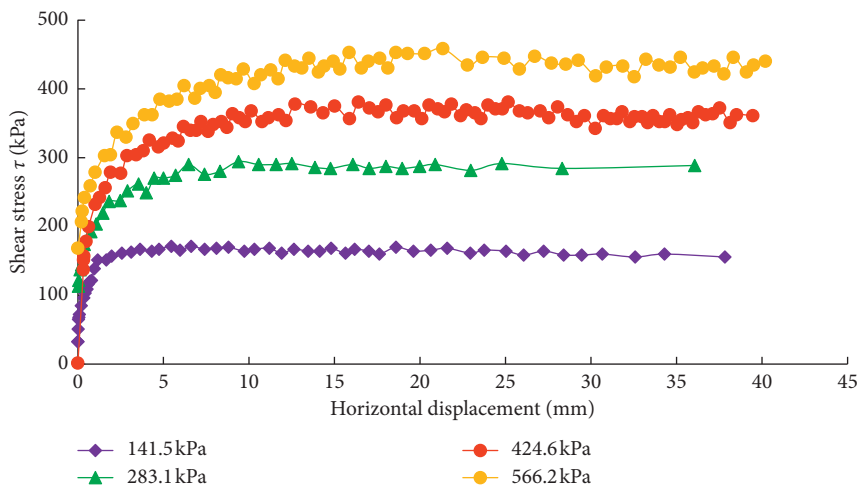
(3) *Load and Shear.* Vertical loading is started after the precontact. At first, we should set up the corresponding parameters according to the design experiment. When the settlement of the specimen is small and stable, the vertical loading can be considered to be completed. Then, the horizontal shear test is started and the tangential load is set up. The first stage tangential load should be 10% vertical (normal load), and the last one (maximum) tangential load is 150% (1.5 times) vertical (normal) load. When the shear deformation reaches 10%–15% of the diameter of the specimen (300 mm), namely, 30–45 mm, it is considered the failure of the specimen. The default failurevalue of the program is 50 mm vertical compression and 60 mm horizontal shear..

(4) *Unload.* The displacement meter is removed first when the shear test is finished. The horizontal unloading is carried out first and then the vertical unloading: pull the bar to the right end—horizontal cylinder unloading—stop. The unloading process of vertical cylinder is similar to that of horizontal cylinder.

4. Analysis of the Test Results

4.1. Characteristics of the Stress-Strain Behavior. Figure 5 shows the shear stress and horizontal displacement curves of the direct shear tests of the landslide deposit samples with different rock particle contents (35%, 45%, 55%, 65%, and 75%) under different normal stresses. RPC represents rock particle content, and WC represents water content.

Curves in Figure 5 present 3 different stages. The first stage is a straight line increase section. We call it the liner elastic stage. The stage is more evident in the sample with a lower rock particle content (Figure 5(a)). At this stage, the stress at any point of the sample is less than the shear strength of the SRM sample and the shear deformation is mainly caused by the decrease of the void in the sample. The second stage is the yielding stage, in which the upward curvature of the curves changed dramatically from steep to smooth. The stress of some part of the specimen has reached its shear strength. The shear deformation is mainly caused by the compression of the void in the SRM sample. At this stage, the shear stress fluctuates with the increasing shear displacement and soils between rock particles yields first. The last stage is the strain-hardening stage, in which the stress



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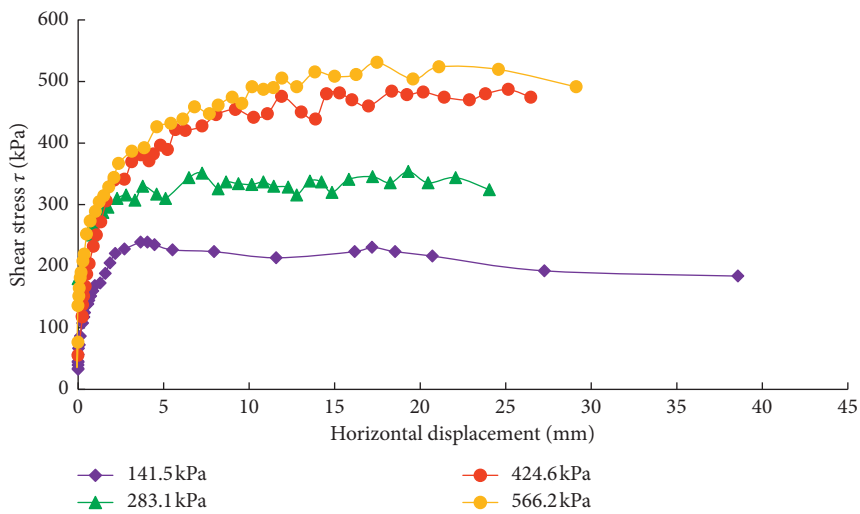

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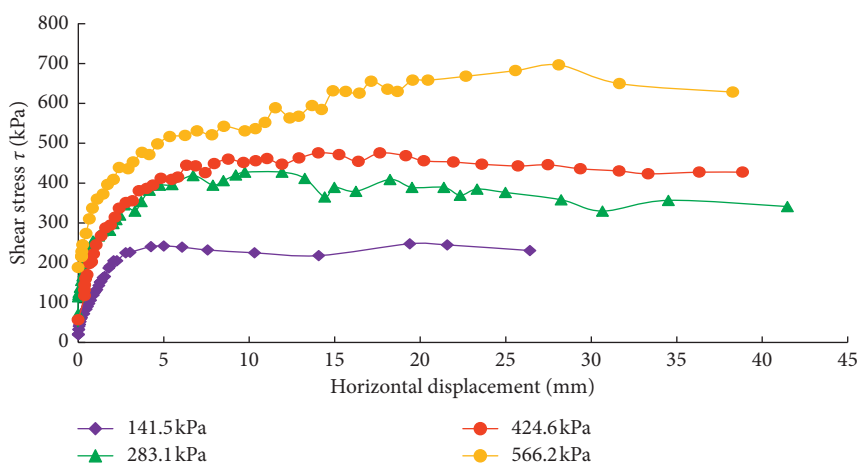

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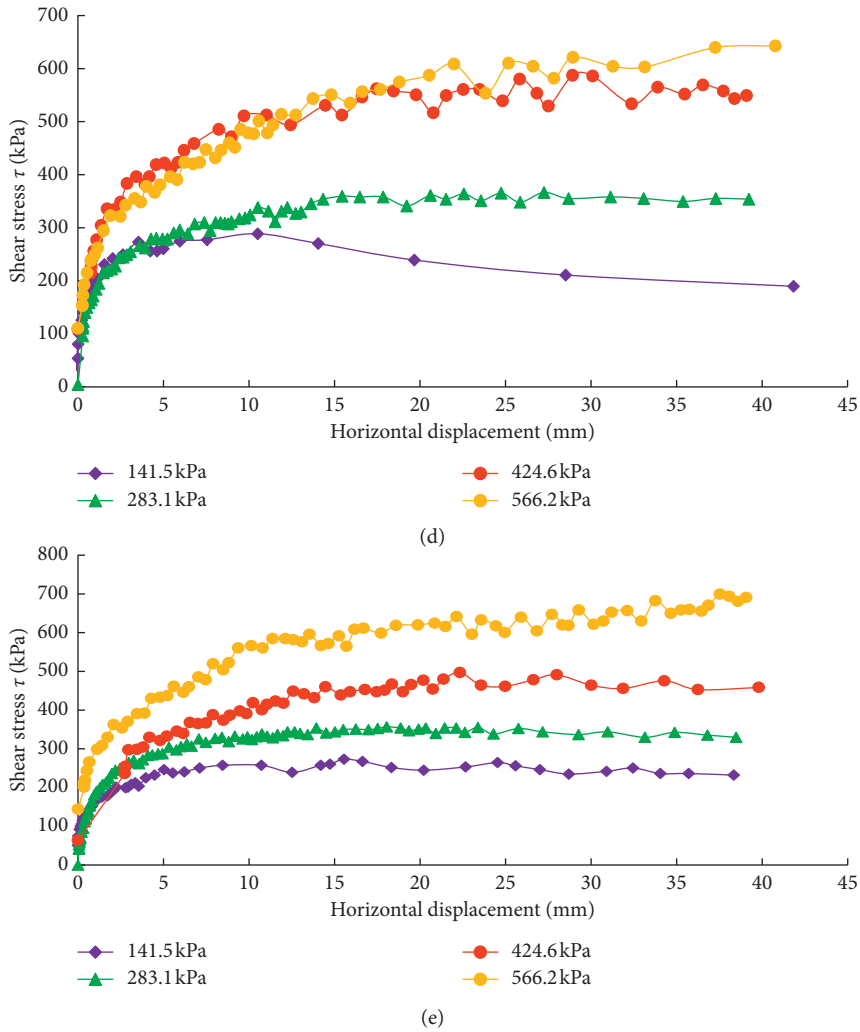
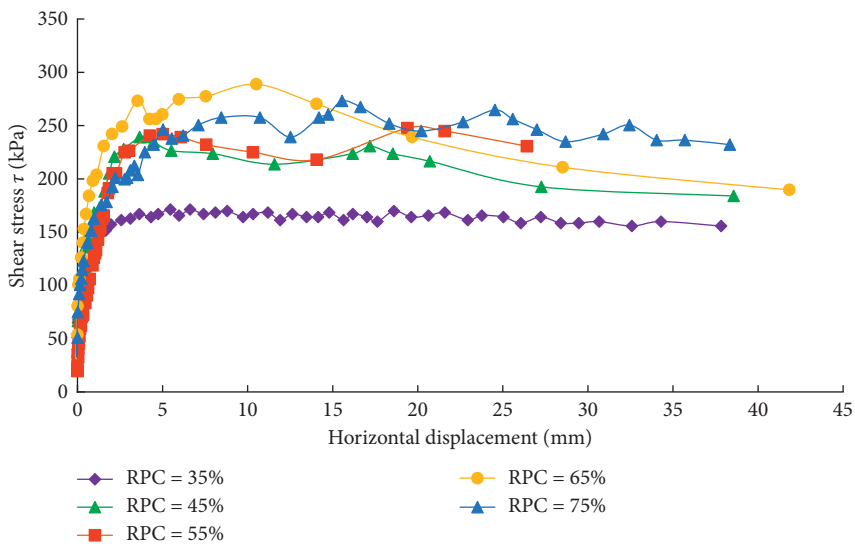


Figure 5: Shear stress and horizontal displacement curves of different rock particle content samples under the different normal stresses: (a) RPC = 35%; (b) RPC = 45%; (c) RPC = 35%; (d) RPC = 65%; (e) RPC = 75%.

value changes very little, but the strain increases significantly. Due to the failure of the filling soil in the sample at the second stage, the original uncontact rock particles are gradually contacted, and the strength of the specimen increases again due to the force of "interlocking" and "rubbing" between the rock particles.

Figure 5 also shows that the shear stress increases with the increasing normal stress under the same rock particle content. At the lower rock particle content conditions, it is difficult for rock particles to contact and the strength of "soil" plays a leading role in the sample, so the strength of the sample is mainly controlled by the "soil." With the increasing rock particle content, the proportion of "soil" decreases and the rock particles can be fully contacted, so the force of interlocking and rubbing enhances a lot and rock particles play the role of the "skeleton." Then, the soil fills the pores between the skeleton, leading to an increase of the density and strength. The strength of the sample is mainly controlled by the "rock particles" this moment. On the contrary, the higher the normal stress, the more limited the "roll-over" phenomenon between rock particles. Therefore, rock particles breakage and rearrangement in the sample leads to an increase of its shear strength to a certain extent.

Figure 6 shows the shear stress and horizontal displacement curves of the different rock particle contents under the same normal stress. As shown in Figures 6(a) and 6(c), the shear stress of the sample increases first and then decreases with the increasing rock particle content and reached its maximum when the rock particle proportion equals 65%. However, the maximum value was reached when the rock particle content is 55% in Figures 6(b) and 6(d). As we all know, the shear strength of most soil and rock mixtures increases with the increasing rock particle content in most cases. Vallejo and Mawby [23] thought when rock particles by weight in the mixtures were lower, the shear strength of the mixtures was basically controlled by the fine particles that surrounded the rock particles. However, if rock particles by weight were higher, the shear strength was controlled by the rock particles alone and if between them, the shear strength of the mixtures was partially controlled by the granular phase. In



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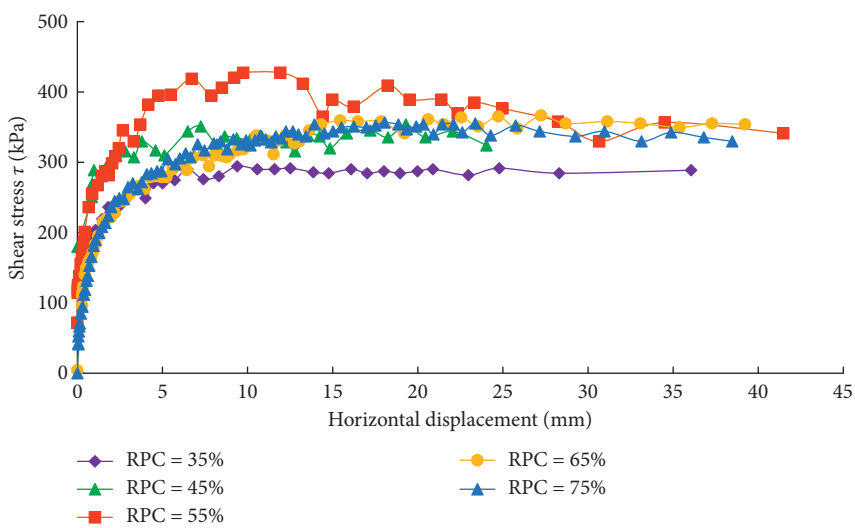

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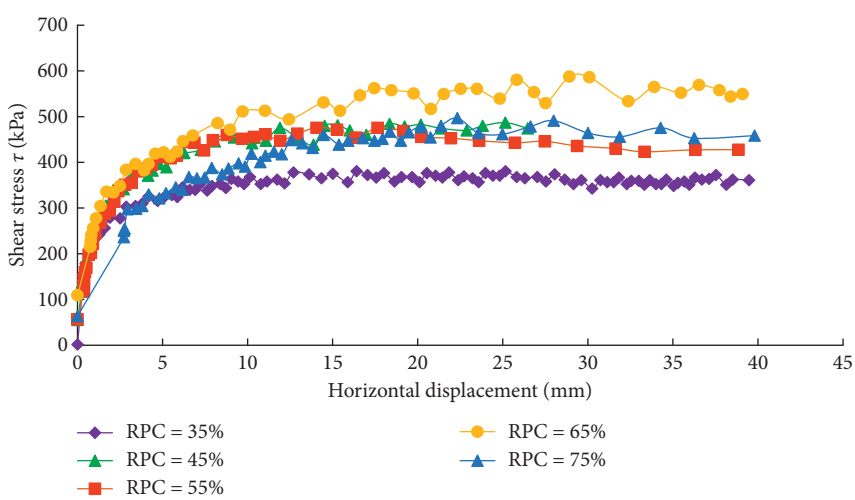

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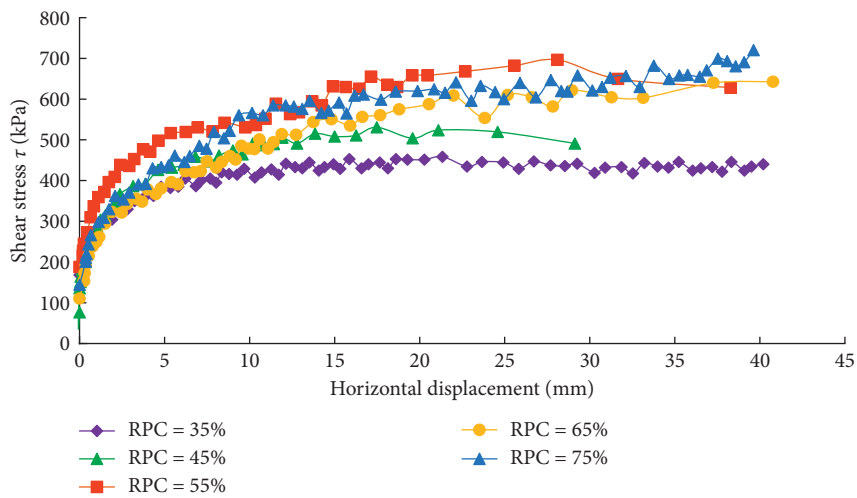


(d)

Figure 6: Shear stress and horizontal displacement curves of different rock particle content samples under the same normal stresses: (a) $\sigma = 141.5$ kPa; (b) $\sigma = 283.1$ kPa; (c) $\sigma = 424.6$ kPa; (d) $\sigma = 566.2$ kPa.

our opinion, both of them worked together in the whole shear process, and it is just who played the leading role in different rock particle content states. The joint force of cohesive soils and the force of "interlocking" and "rubbing" between rock particles reached to the maximum when rock particle content is at 65% in a lower normal stress and 55% in a higher normal stress.

4.2. Characteristics of the "Jumping" Phenomenon. The "jumping" phenomenon actually means the sudden increase or decrease of the shear stress in the shear process caused by overturning, rolling, and crushing of rock particles, which can be characterized by the change rate of shear stress per unit horizontal displacement. Figure 7 shows the curves of $\Delta\tau/(\Delta s \times \tau_{max}) - s$ of the specimens with different rock particle contents under the same normal stress, where $\Delta\tau = \tau_{n+1} - \tau_n$, in which n is the loading step and τ_n (or τ_{n+1}) is the shear stress at the loading step n (or $n+1$), and $\Delta s = s_{n+1} - s_n$, in which $s_n(s_{n+1})$ is the horizontal displacement at the loading step n (or $n+1$), τ_{max} is the maximum value of the shear stress, and s is the horizontal displacement.

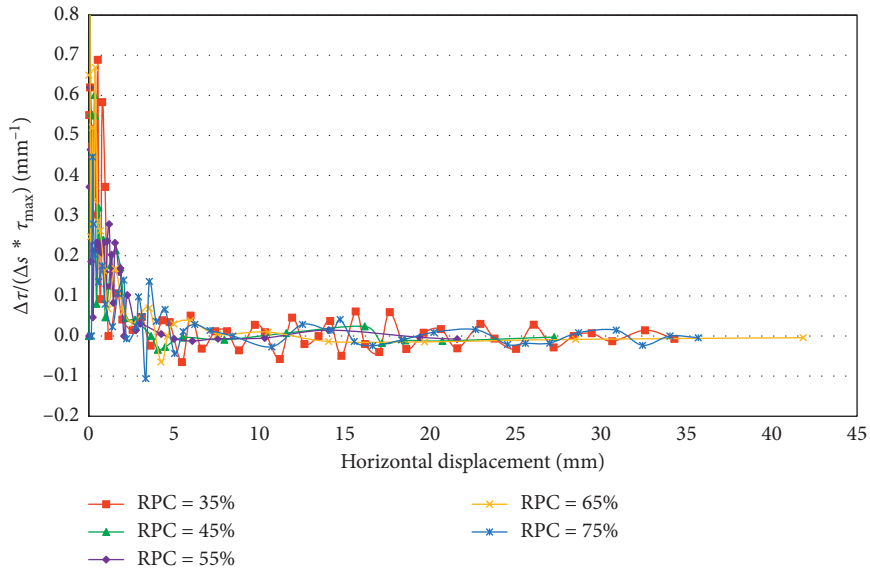
Figure 7 indicates that the "jumping" phenomenon runs through the whole shearing process. When the normal stress is low, the continuous stage of "intense jumping" is shorter, which mainly occurs at the early stage of the shear process. Then, the "intense jumping" stage becomes longer with the increasing normal stress, and the early stage is stronger than the later stage. The "jumping" phenomenon behaves more obvious with the increasing rock particle content and normal stress. Figure 7 (especially the Figures 7(c) and 7(d)) also indicates that, under the same normal stress, the lower the rock particle content is, the more obvious the "jumping" phenomenon is. This is mainly because the lower the rock particle content is, the more the flexible space for rock particles to rotate, rollover, reorient, or meander is, causing the shear stress to fluctuate greatly and leads to a phenomenon of "jumping." This is also in accordance with the description in the previous part.

4.3. Characteristics of Volumetric Strain. The shear dilation and shrinkage behavior of the landslide deposit sample were analyzed through the curves of vertical vs. horizontal displacement, as shown in Figure 8. The vertical and horizontal displacement is zero at the beginning of the shear process. The vertical displacement is positive for dilatancy and negative for contraction.

Figure 8 shows that the sample with a lower rock particle content displayed the dilatancy behavior under low normal stress and shrinkage behavior under high normal stress, which is similar to "soil." With the increase of the normal stress, the specimen transformed from dilatancy to shrinkage at the same rock particle content. Mainly due to the limited test conditions, the specimen cannot be fully compacted. With the increasing shear stress, the particles are moved, filled, and rearranged, leading to the particles interembedded with each other to fill the pores. Then, the specimen is gradually compacted and the shear shrinkage behavior appears.

The sample also displayed dilatancy behavior in the condition of high rock particle content. It is mainly because of the "rotate," "roll-over," "climbing," or "crossover" of the rock particles, leading to the dilatancy deformation in the sample. However, with the increasing normal stress, the dilatancy value becomes smaller, which may be caused by the increase of the fine particles due to the "crushing" or breakage of some rock particles. Then, the fine particles fill some pores of the specimen, resulting in an increase of the density and a reduction of the dilatancy value.

Figure 9 shows the vertical and horizontal displacement curves of the different rock particle contents under the same normal stress. With the increasing rock particle content, the value of the dilatancy or shrinkage has no



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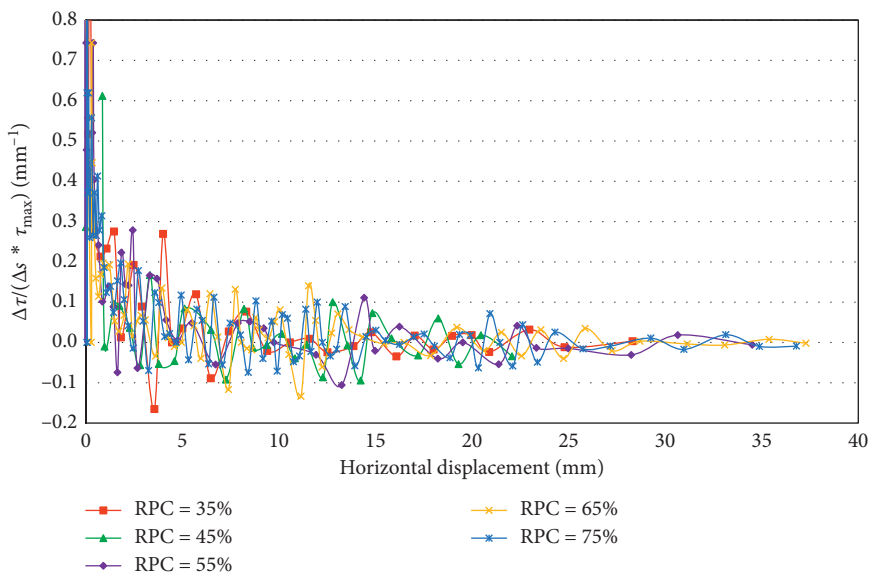

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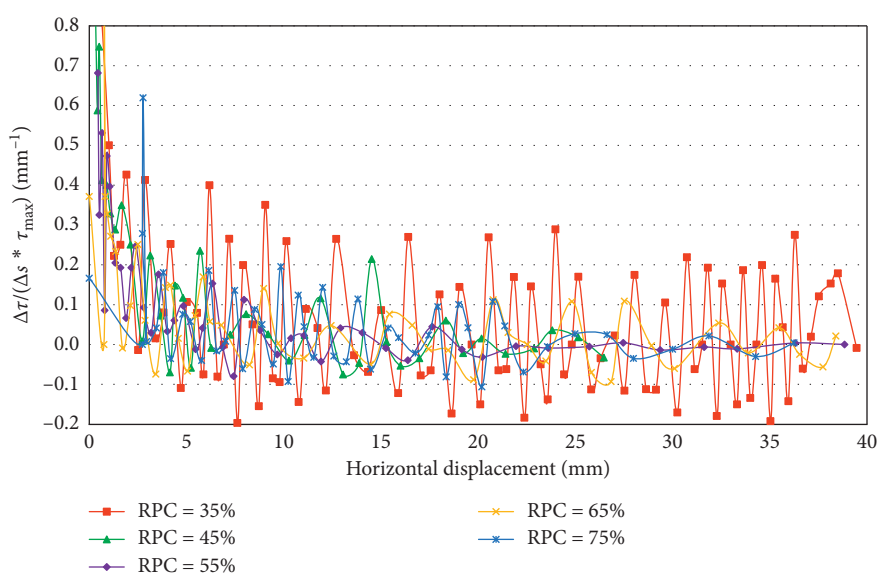

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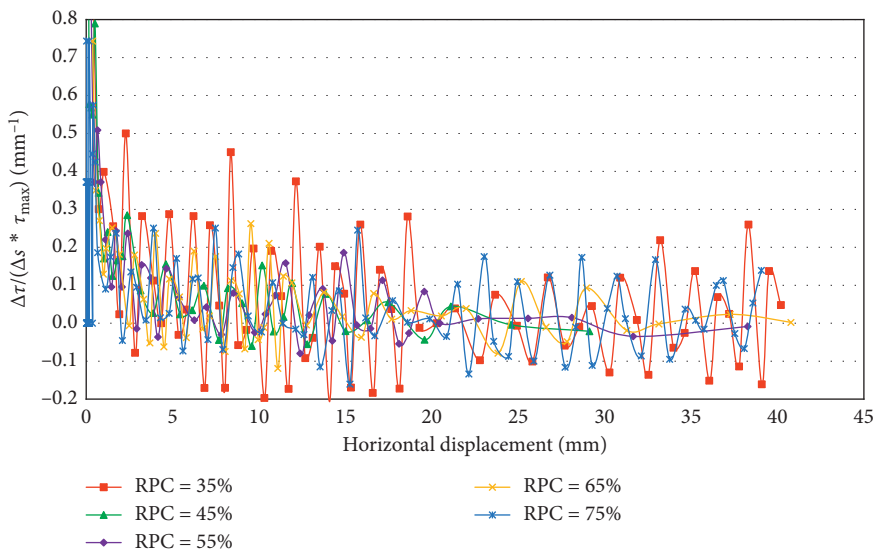


(d)

Figure 7: $\Delta\tau/\Delta s^{*}$ $(\tau_{\max} - s)$ curves of different rock particle contents under the same normal stress: (a) normal stress $\sigma_n = 141.5$ kPa; (b) normal stress $\sigma_n = 283.1$ kPa; (c) normal stress $\sigma_n = 424.6$ kPa; (d) normal stress $\sigma_n = 566.2$ kPa.

obvious change laws in the same normal stress. Comparing with Figure 6, we can find that the maximum shear stress value of the rock particle content corresponds to the maximum value of dilatancy or shrinkage. It means the order of the shear stress of different rock particle contents is the same as that of its corresponding vertical displacement. For example, in Figure 6, the shear stress value with the rock particle mass of 65% is the largest at the normal stress of 141.5 and 424.6 kPa, while it is 55% at the normal stress of 283.1 and 566.2 kPa. However, their corresponding vertical displacement is also the largest in Figure 9. This means under the same normal stress, the shear stress with different rock particle contents and its value of dilatancy or shrinkage are corresponding to each other.

4.4. Characteristics of Shear Strength Parameters. In fact, the cohesion of the soil-rock mixture (SRM) is made up of cohesion of cohesive soils and the force of interlocking and rubbing (or "the bitten force") between the rock particles. Previous studies indicate lots of researchers (e.g., [8]) called it cohesion. It may be correct for the general cohesive soil, but it is not accurate for the loose deposit sample (or SRM). As we all know, cohesion is the component of shear strength of a rock or soil that is independent of interparticle friction. However, the landslide deposit sample is a special geological material, a mixture of cohesive soil and cohesionless rock. It not only shows the mechanical behavior of the soil but also the rock mass; therefore, we cannot simply call the intercept of the Mohr failure envelope "cohesion."

Research ideas of the strength of rock joint used by Patton [24] in the mechanics of rock mass can be improved here. When there are large rock particles on the shear plane, it can be divided into two shear stages of the shear process according to the magnitude of the normal stress (as shown in Figure 10); that is, there will exist a normal stress threshold σ_t. When the normal stress $\sigma \le \sigma_t$, the cutting and biting force between rock particles is small, and the cohesion of cohesive soil is dominant. The specimen at the shear surface will be dilated to crossover the block of the rock particles, which is like the phenomenon of "climbing a slope." The shear strength of the SRM is as follows:

$$\tau = C + \sigma \cdot \tan\varphi, \tag{2}$$

where φ is internal friction angle of SRM when normal stress is less than the normal stress threshold, and C is the cohesion of the SRM sample when $\sigma \le \sigma_t$ (σ is the normal stress).

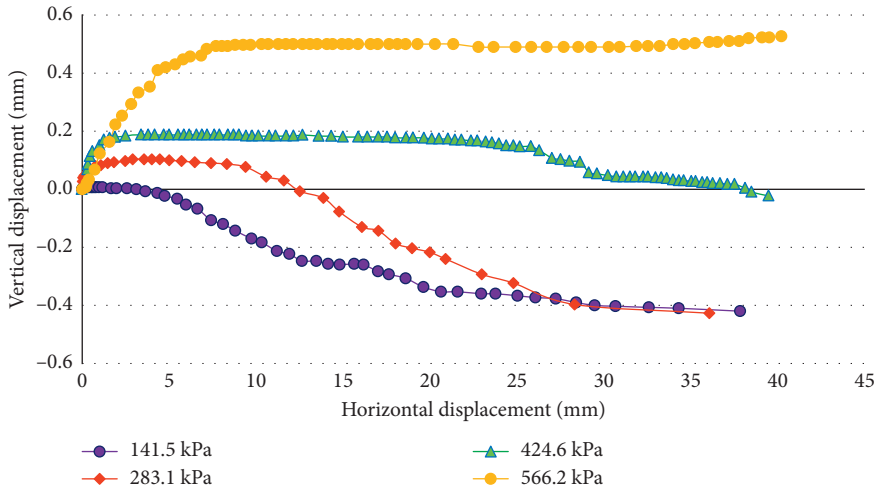
However, when normal stress $\sigma > \sigma_t$, it is difficult for the specimen to "climb" over the rock particles. The most likely way to destroy the specimen is to cut off the rock particles (breakages) directly along the shear movement. At this time, the specimen will show some large "cohesion:"

$$\tau = C_r + \sigma \cdot \tan\varphi_r, \tag{3}$$

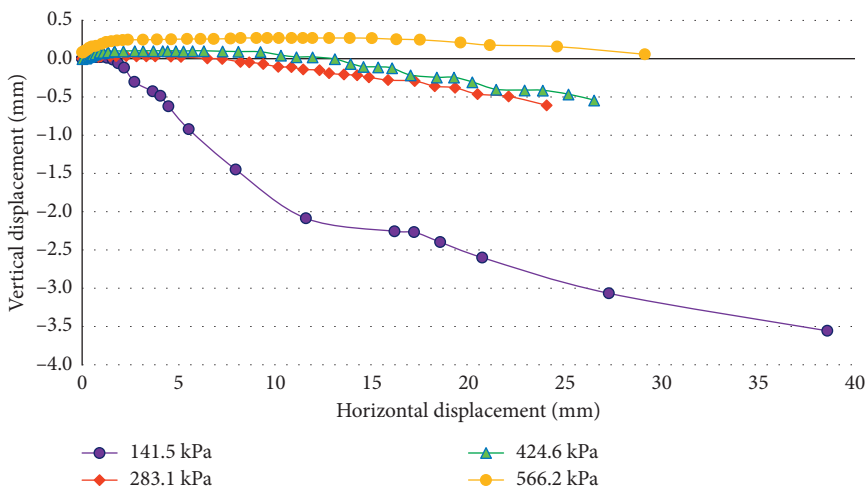
where C_r means the equivalent cohesion as a result of the cutting (breakage) of the rock particles, which will be smaller than the whole rock block and greater than the cohesive soil, and φ_r is the internal friction angle of SRM when normal stress is greater than the normal stress threshold.

Figure 11 shows the linear regression curves of shear strength of different rock particle contents. Different colors represent different rock particle contents, and R^2 represents the correlation coefficient.

Figure 12 shows the curves of equivalent cohesion and internal friction angle. The equivalent cohesion increases first and then decreases and reaches the maximum when the rock particle content is 65% and then again decreases. The equivalent cohesion is also very high when the rock particle content is 45%. It is mainly due to the joint action of cohesion of soils and the force of interlocking and rubbing



(a)



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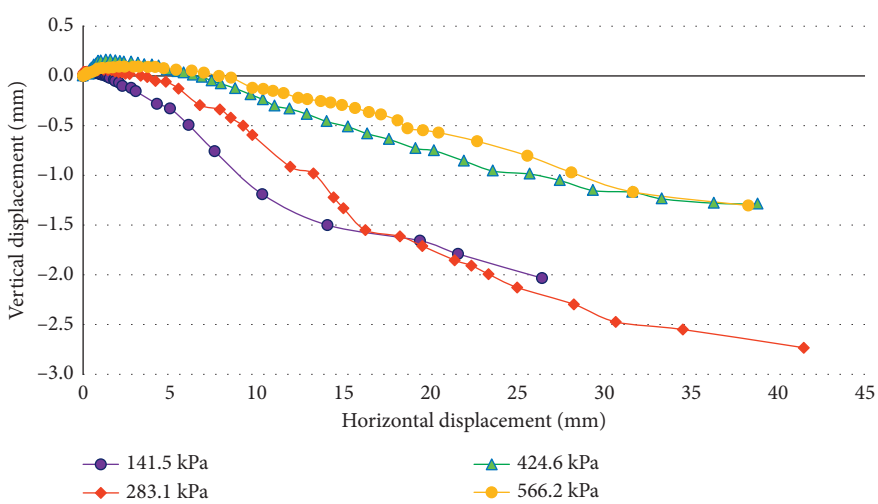
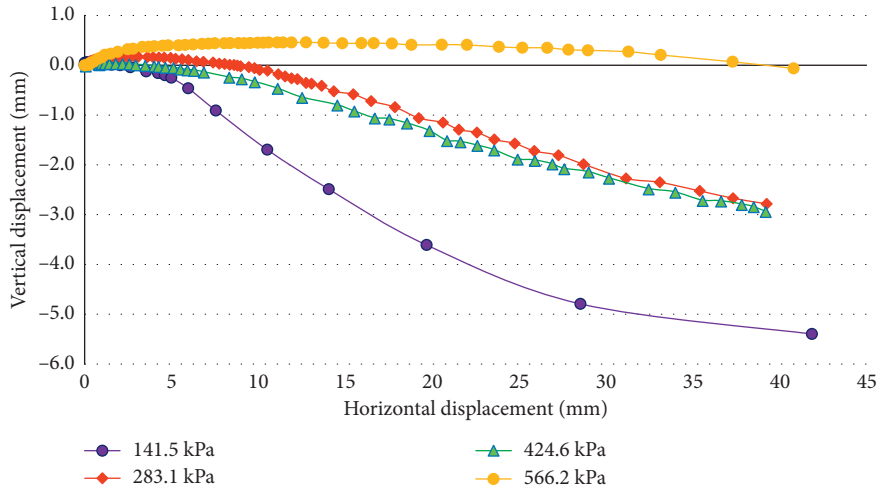
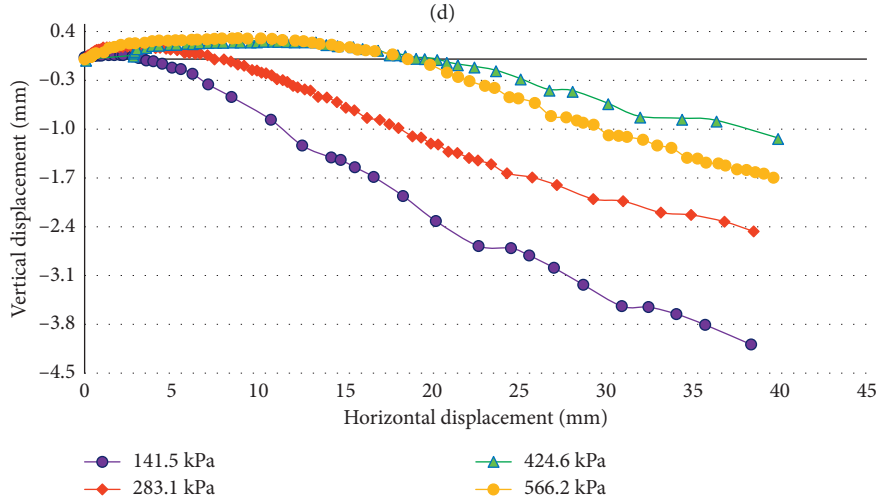


(c)

Figure 8: Continued.

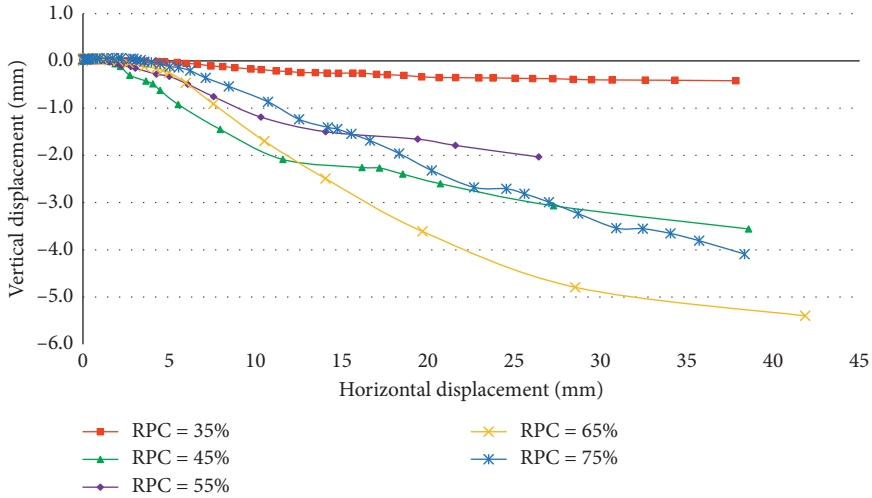


(d)



(e)

Figure 8: Vertical and horizontal displacement curves of different rock particle content: (a) RPC = 35%; (b) RPC = 45%; (c) RPC = 55%; (d) RPC = 65%; (e) RPC = 75%.



(a)

Figure 9: Continued.

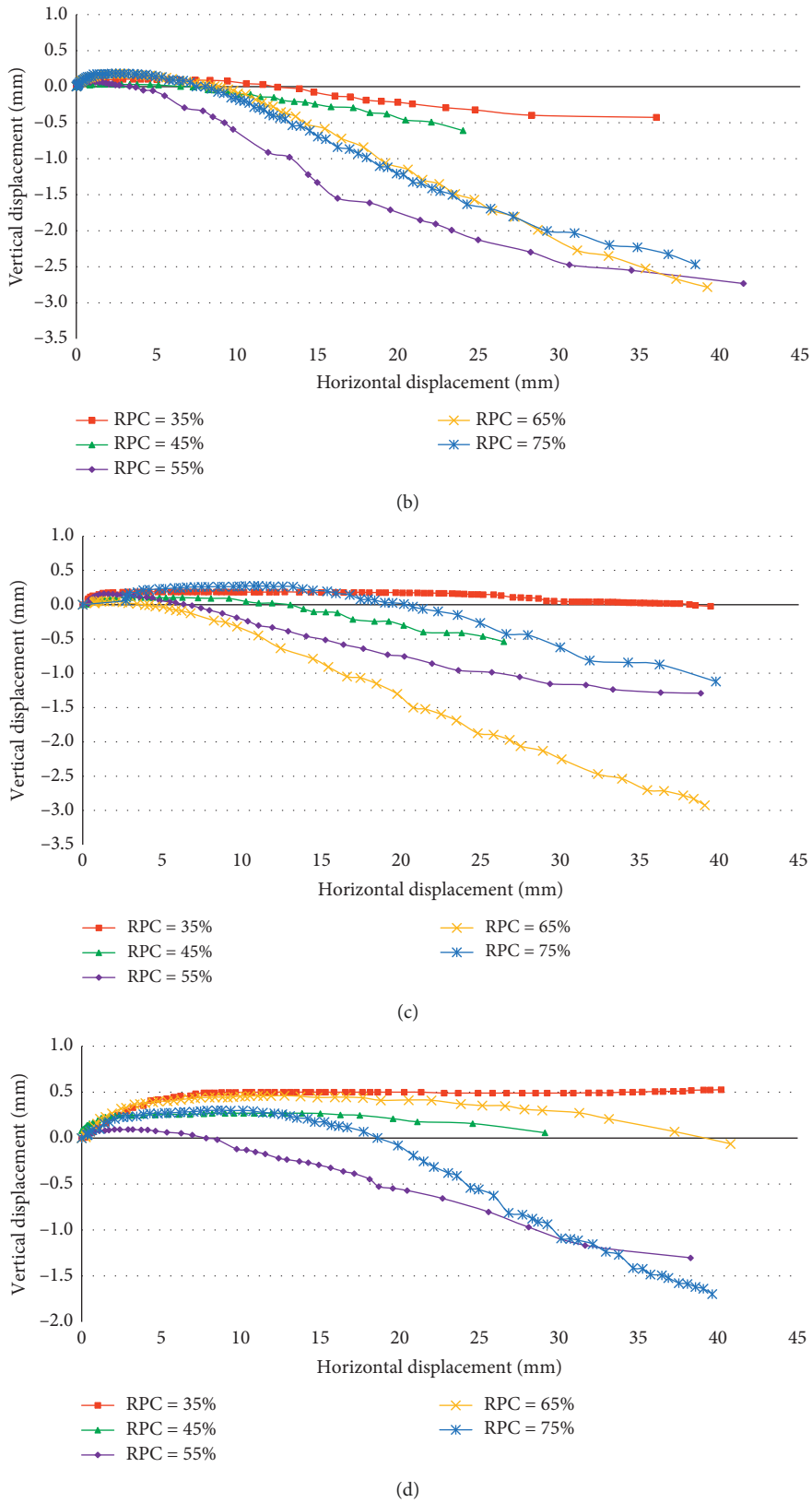


Figure 9: Vertical and horizontal displacement curves of different rock particle content samples under the same normal stresses: (a) normal stress $\sigma_n = 141.5$ kPa; (b) normal stress $\sigma_n = 283.1$ kPa; (c) normal stress $\sigma_n = 424.6$ kPa; (d) normal stress $\sigma_n = 566.2$ kPa.

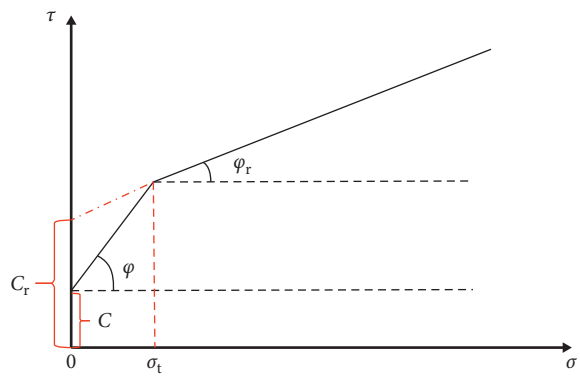


Figure 10: Bilinear strength criterion of SRM.

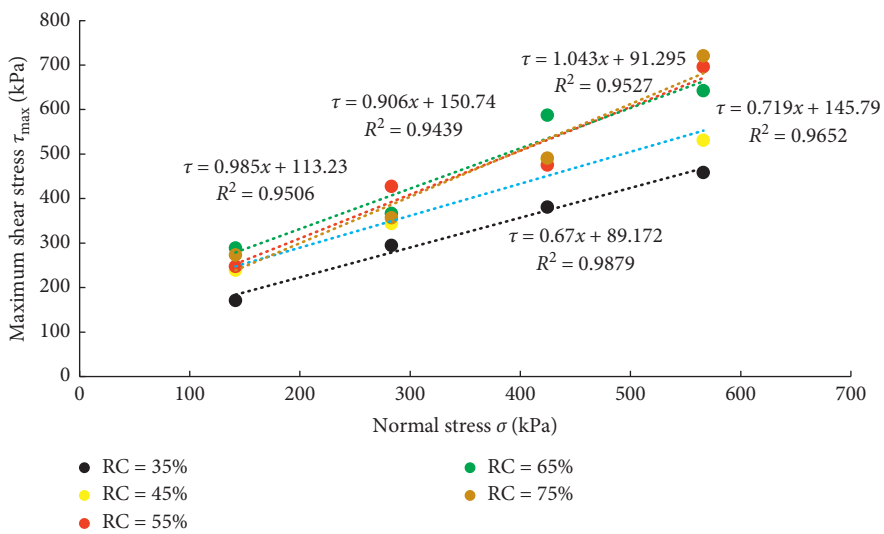


Figure 11: Linear regression curves of shear strength of different rock particle contents.

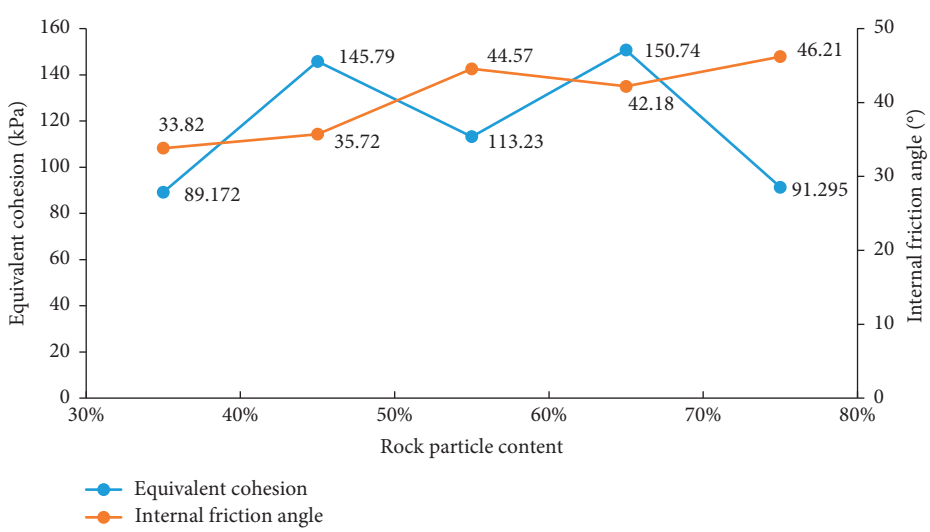


Figure 12: Curves of the equivalent cohesion and the internal friction angle.

between rock particles; maybe, the cohesion of soils played an important role, as described in the previous part, and the sample showed the more mechanical behavior of soil. When the rock particle content is 65%, the sample showed the more mechanical behavior of rock mass and the equivalent cohesion was mainly caused by the cutting and breaking of the

rock particles. The higher the normal stress and rock particle content are, the larger the force of "cutting," "interlocking," and "rubbing" between rock particles is, leading to a higher equivalent cohesion. However, when the content of rock particle is too high, on the contrary, the voids between rock particles cannot be filled with enough soil, leading to a reduction of the sample density. On the contrary, if the normal stress is not large enough to cut off lots of rock particles, it may also cause a decrease of the equivalent cohesion. Therefore, the shear strength of the landslide deposit sample is closely related to its rock particle content and the normal stress.

Figure 12 also shows that the friction angle increases with the increasing rock particle content but decreases a little when rock particle content is 65%. The higher the normal stress and the rock particle content are, the larger the force of "contacting," "interlocking," and "rubbing" between rock particles is, leading to a higher internal friction angle. But more specifically, when the equivalent cohesion is maximum, the angle of friction may decrease a little. The main reason is that when the normal stress approaches its threshold value, there will be no "climbing a slope" process and the process of particle cutting occurs, leading to an increase of the equivalent cohesion and a little decrease of the internal friction angle.

5. Conclusions

(1) Stress-strain results show that stress-strain curves can be divided into 3 different stages: liner elastic stage, yielding stage, and strain-hardening stage. Both soil and rock particles worked together in the whole shear process, and it is just who played the leading role in different rock particle content states. The shear strength of SRM behaves more like "soil" at a lower rock particle content and behaves more like "rock joints" at a higher rock particle content.

(2) Characteristics of the "jumping" phenomenon results show that the "jumping" phenomenon runs through the whole shearing process. The "intense jumping" stage becomes obvious with the increasing rock particle content and the normal stress. However, the lower the rock particle content is, the more obvious the "jumping" phenomenon under the same normal stress.

(3) Volumetric strain results show that the sample with a lower rock particle content showed a dilatancy behavior under the low normal stress and shrinkage behavior under the high normal stress. The dilatancy value becomes smaller with the increasing normal stress. The maximum shear stress value of the rock particle content corresponds to the maximum value of dilatancy or shrinkage.

(4) The intercept of the Mohr failure envelope of the soil-rock mixture should be called the "equivalent cohesion," not simply called the "cohesion." The higher the normal stress and rock particle content are, the bigger the equivalent cohesion and the internal friction angle is.

Data Availability

Because the data in the paper are still a project of the National Natural Science Foundation of China, the data need to be used in the follow-up study of the project. So all the figures and tables data used to support the findings of this study were supplied by the corresponding author under license and so cannot be made freely available. Requests for access to these data should be made to Longqi LIU, School of Highway, Chang'an University, Middle Section of South Second Ring Road, 710064, Xi'an, Shaanxi, China (tel: 086–18710710184; email: longqi@chd.edu.cn).

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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