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Research Article

Study of Material Composition Effects on the Mechanical Properties of Soil-Rock Mixtures

Yanxi Zhao [1] **and Zhongxian Liu** [2]

[1] *School of Architectural Engineering, Nanjing Institute of Technology, Nanjing 211167, China*
[2] *School of Civil Engineering, Tianjin Chengjian University, Tianjin 300384, China*

Correspondence should be addressed to Yanxi Zhao; 249543863@qq.com

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Soil-rock mixtures are often seen in geological deposits. Mechanical properties of these mixtures are controlled by microstructural characteristics such as rock size distribution, rock shape, locations, and content. The effects of material composition on soil-rock mechanical properties were studied in the laboratory. The soil-rock material was screened into different size categories. Medium-scale shearing and triaxial experiments were used to study the relationships among macrodeformation, strength, content, size, and random location of rocks. The medium-scale triaxial shearing instrument included the computer control system, EDC control system, and sensor response. The stress-strain curve of soil-rock mixtures was found as a hardening curve which is approximately hyperbolic, and there was no obvious peak intensity value. When the Mohr–Coulomb criterion was used to depict the curve under a shear strain of 0.15, cohesion first increased and then decreased, a finding opposite to the internal friction angle with a decrease in particle size. Elastic modulus increased with an increase in rock size, but Poisson's ratio remained constant. In similar conditions, the random location of rocks can lead to a variation range of 4 degree of the internal friction angle, and cohesion values can change in a large range than the mean value.

1. Introduction

Soil-rock mixture is a geological body composed of gravel or stone as an aggregate and clay and sand as a filling material, which is often found in geological deposits. A landslide, composed of soil and rock, is illustrated in Figure 1, and a geological survey shows that its internal structure is composed of matrix soil grains with different cementation degrees and rubble with greater strength and stiffness. Soil-rock mixtures are usually not homogeneous; discontinuity and spatial variability are determined by rock content, cementation degree, and microscopic characteristics such as particle size, roughness, and texture. The mechanical deformation properties and strength parameters are closely related to structure composition such as rock content, size, shape, and spatial distribution. The successful development of China hydropower dam structures will require greater understanding of the mechanical properties of soil-rock mixtures.

A number of research studies have been conducted to investigate the effects of soil-rock mixtures on mechanical properties. Medley [1] proposed that the mechanical properties of soil-rock mixtures are related to rock content and physical arrangement. Xu et al. [2–6] studied the correlation between the microscopic structure and macromechanical properties and the effects of different interface types of soil-rock on mechanical properties and failure mechanisms. Tu [7] found that, under low-stress conditions, the Mohr–Coulomb strength envelopes (for the same sample, under different principal stress, envelopes of the stress circle when the sample reaches the limit state) of soil-rock mixtures were almost linear. Under higher stress, strength envelopes were nonlinear and concave. A relationship between stress and shear strength parameters was therefore proposed. Xiuli [8, 9] developed a numerical simulation method for unsaturated soil-rock mixtures and analyzed the influence of contact characteristics, rock content, saturation, etc., on mechanical properties and failure mechanisms.

Figure 1: Soil-rock mixtures seen in engineering.

Figure 2: Medium-scale triaxial shearing instrument system: ① medium-scale triaxial chamber; ② system of axis force applied; ③ system of confining pressure applied; ④ system of volume change measured; ⑤ system of electric control; ⑥ system of data collection with a computer.

It is critical to determine physical and mechanical parameters in the design and construction of engineering structures using soil-rock mixtures [10–12]. However, because of poor traffic, the field test is difficult to execute, and when the rock size varies over several orders of magnitude, it is more difficult to simulate the actual rock size in the laboratory [13–17]. The mechanical properties of soil-rock mixtures are complex because these mixtures are multiphase, multicomponent, and discontinuous materials.

In this paper, based on the Mohr–Coulomb criterion, medium-scale shearing instruments are used and small-scale tests are designed to study the influence of rock content, rock size, and the random distribution of rocks on the mechanical parameters of soil-rock mixtures.

2. Materials and Methods

2.1. Experimental Instrument. A medium-scale triaxial shearing instrument was used for all tests. When the test material is loaded, the computer can control the stress or strain (Figure 2). Through the EDC control system and sensor response, beam displacement can be controlled. In the system, Figure 2 (③) controls the change of confining pressure. To control the experiment process, serial programming was used. A computer program was used to control and record confining pressure.

2.2. Experimental Materials. The soil material used was low liquid-limit (the water content at which soil behaves practically like a liquid) clay. The liquid limit, plastic limit, and plasticity index were, respectively, 29.2%, 17.2%, and 12%. The gradation of grains is shown in Table 1, while grain size is the particle diameter of soil and is classified by the sieving method.

Soil in the soil-rock mixtures is a relative concept and different from the traditional notion of silt, clay, and other fine-grained soil. The range of grain size in these mixtures will vary depending on the research scale. Particle size varies from several millimeters to tens of centimeter. Defining the soil and rock is important in determining the rock content of this medium. Medley found a size independence characteristic in soil-rock mixtures and defined the threshold value of soil and rock as follows:

Table 1: Gradation of the soil material.

Grain size (mm)	1–0.5	0.5–0.25	0.25–0.075	>0.075
Content (%)	1.1	2.6	28.1	68.2

$$d_{thr} = 0.05L_c, \tag{1}$$

where d_{thr} is the threshold of soil and rock size and L_c is the size of research scope.

For the triaxial shearing experiment test,

$$L_c = D, \tag{2}$$

where D is the sample diameter. The criterion for judging soil and rock is therefore

$$d_{thr} = 0.05L_c = 0.05D = 0.05 \times 100\,\text{mm} = 5\,\text{mm}. \tag{3}$$

In the process of test configuration, the minimum size of particles was 5 mm. To meet specifications of the medium-scale triaxial shearing instrument, the rock size in soil-rock mixtures should not exceed 20 mm. So rocks in site between 5 mm and 20 mm were used to simulate the action of rock particles, which is equivalent to removing larger rocks from field samples. The physical and mechanical parameters of soil and rocks are shown in Table 2 according to the Mohr–Coulomb criterion. In Table 2, ρ is the density, E is the elastic modulus, μ is Poisson's ratio, C is the cohesion, and ϕ is the internal friction angle.

2.3. Experimental Program. Tests were focused on the influence of rock content, size, and random distribution on macromechanical parameters and deformation characteristics. In each test, the total mass of soil and rock was 3 kg, and the used rock is shown in Figure 3. The experimental design is shown in Table 3. The mixed size consists of one-third of the sizes: 16–20 mm, 10–16 mm, and 5–10 mm. For each group, 3 samples were used, and the confining pressure was set to 200 kPa, 400 kPa, and 800 kPa, respectively. The shear rate of pure soil was set at 0.05 mm/min, the mixture medium was 0.1 mm/min, and the shear process terminated when the displacement reached 30 mm.

Table 2: Mechanical parameters of soil and rocks.

Name	ρ (kg/m^3)	E (MPa)	μ	C (MPa)	ϕ (°)
Soil	2100	50.00	0.34	0.06	22
Rocks	2800	1.0E4	0.29	3	45

(a) (b) (c)

Figure 3: Artificial rock sizes (mm): (a) 5~10; (b) 10~16; (c) 16~20.

Table 3: Test design of soil-rock mixtures.

Category of test	Rock content (%)	Size of particle (mm)	Mass of dried soil material (g)	Mass of total particle (g)
Influence of rock content	0	—	3000	0
	20	16~20	2400	600
	30	16~20	2100	900
	50	16~20	1500	1500
	70	16~20	900	2100
Influence of rock size	50	16~20	1500	1500
	50	10~16	1500	1500
	50	5~10	1500	1500
	50	Mixed size	1500	1500
Random distribution 1	50	16~20	1500	1500
Random distribution 2	50	16~20	1500	1500
Random distribution 3	50	16~20	1500	1500
Random distribution 4	50	16~20	1500	1500

2.4. Experimental Methods. The diameter and height of the 2 cylindrical samples were 101 mm and 200 mm. The 2 end surfaces of each sample were parallel to each other and perpendicular to the axis. To meet these requirements, a special sample kit was used to make samples (Figure 4). When the sample was installed, inlet-water stone, samples, filter, permeable stone, and sample cap were installed in turn on the basis of the pressure chamber, and then, a rubber membrane was used to entangle the sample. The sample was a saturated consolidation. Because the permeability of soil-rock mixtures in engineering is usually greater, the head saturated method was adopted to saturate samples.

Figure 4: Special sample kit for medium-scale triaxial experiments.

3. Results

3.1. Influence of Rock Content. Rock content is one of the most important factors [18–20] determining mechanical properties of soil-rock mixtures. Treatments are listed in Table 3. Five groups of medium-scale triaxial consolidation draining shear tests using different rock contents with confining pressures of 200 kPa, 400 kPa, and 800 kPa were completed, and the curves of $(\sigma_1 - \sigma_3)\sim\varepsilon$ and $(\sigma_1 - \sigma_3)\sim\sigma_3$ are shown in Figure 5.

The curves of $(\sigma_1 - \sigma_3)\sim\varepsilon$ under different confining pressures are hyperbolic and have no obvious peak intensity, which belong to hardening deformation curves (Figure 5). Treatments with the same rock content illustrate that confining pressure has a great effect on the distribution of peak stress appearance. When rock content is lower, stress-strain curves are smoother, but an increase of rock content (especially when the rock content is 70%) causes greater

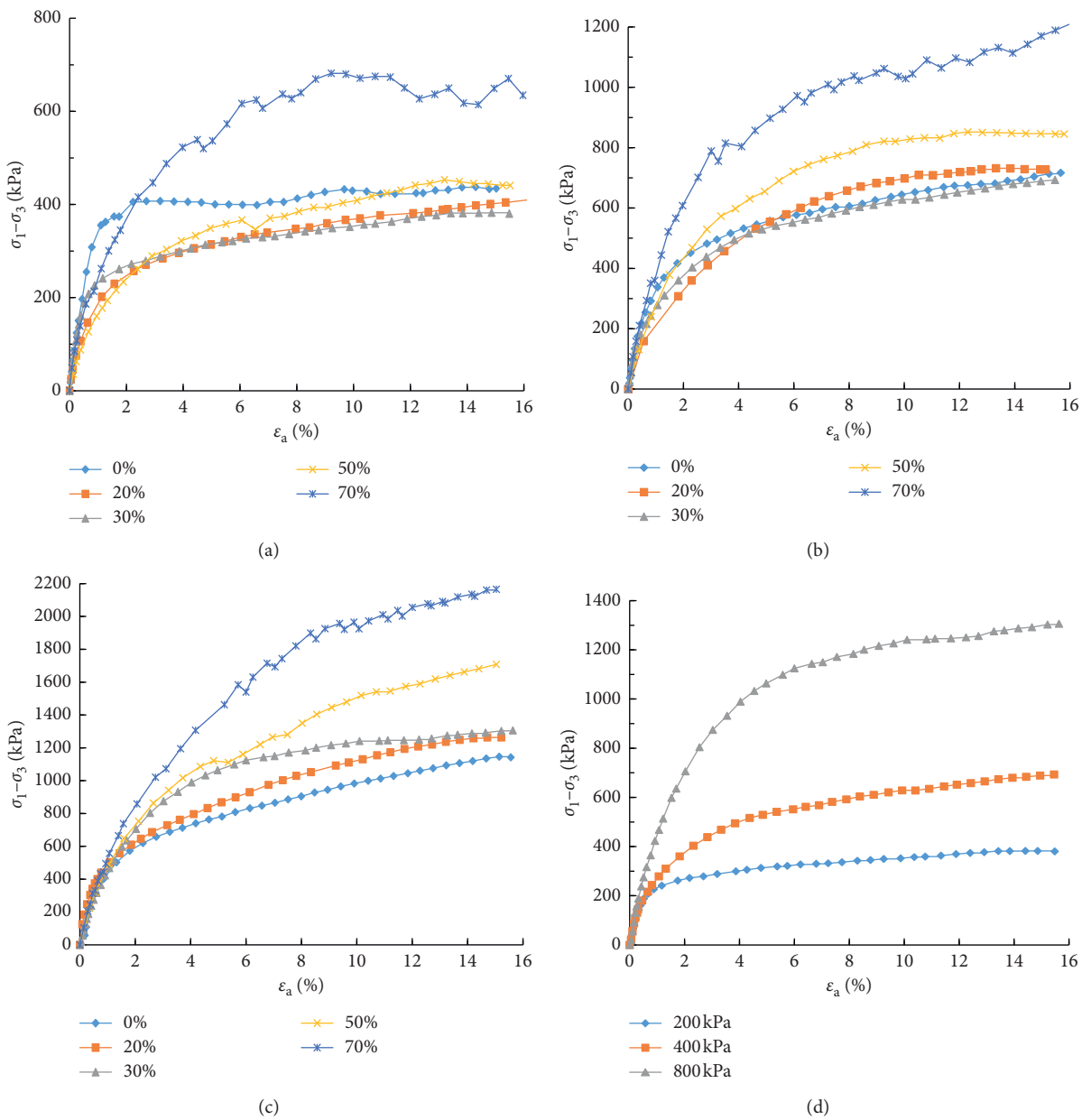

Figure 5: Curves of $(\sigma_1 - \sigma_3)\sim\varepsilon$ under different confining pressures: (a) 200 kPa confining pressure; (b) 400 kPa confining pressure; (c) 800 kPa confining pressure; (d) 30% rock percentage.

fluctuation in the stress-strain curves. This fluctuation is caused by interaction forces between larger particles, leading to an increase in shear displacement. Under the same confining pressure, peak strength increases with an increase in rock concentration. At the same time, the curve of $(\sigma_1 - \sigma_3)\sim\varepsilon$ fluctuations increases and the hardening phenomenon is enhanced, especially under high confining pressure. Under 30% rock block content, peak intensity increases with the increase in confining pressure, and the peak intensity of 800 kPa confining pressure is about 4 times that of 200 kPa confining pressure (Figure 5).

Because the curve of $(\sigma_1 - \sigma_3)\sim\varepsilon$ is hyperbolic, there is no obvious peak intensity. In order to compare the shear characteristic, failure stress when at 15% is used to calculate shear strength. Through analysis of the strength envelope and Mohr circle under different confining pressures, it was found that the tangent of the Mohr circle is linear and the Mohr–Coulomb criterion can be used to reflect macro-deformation and strength.

The results of the triaxial shear test, the strength parameter index according to the Mohr–Coulomb criterion, are shown in Table 4.

Cohesion decreases at first and then increases with an increase in rock content. However, the friction angle increases continuously. As rock content increased from 0 to 70%, the internal friction angle varied from 22.1° to 34° with

Table 4: Strength index of different rock contents according to the Mohr–Coulomb criterion.

Rock block content (%)	ϕ (°)	C (kPa)
0	22.1	72.6
20	25.0	41.1
30	27.7	24.2
50	30.7	6.9
70	34.0	41.0

Note: C is the cohesion; ϕ is the internal friction angle.

a maximum increase of 53.8%. The interaction forces between large particles contribute to this phenomenon. As rock content increases, the contact area of the rock particles will increase. The roughness and increased stiffness will enhance the mechanical properties of the mixture to increase the internal friction angle in the shearing process. As the rock content increased from 0 to 50%, the cohesion varied from 72.6 kPa to 6.9 kPa. The decreasing rate is 90.5%. As the rock content increased from 50% to 70%, the cohesion increased from 6.9 kPa to 41.0 kPa. This increasing cohesion was also the result of mutual force between particles.

To illustrate the effect of rock content on deformation characteristics, secant modulus with an axial strain of 1% is used as the elastic modulus and Poisson's ratio is used when the axial strain was 15%. The elastic modulus and Poisson's ratio of different treatments and confining pressures are shown in Table 5. The elastic modulus increases with increasing rock content under the equal confining pressure; the relationship is more evident under greater confining pressure. The elastic modulus also increases with confining pressure in mixtures with the same rock content. Table 5 and Figure 6 show that Poisson's ratio remains the same over a range of rock content when exposed to low confining pressure (200 kPa); under conditions of high confining pressure (800 kPa), Poisson's ratio increased slightly at first and then decreased slightly with increasing rock content; Poisson's ratio continued to decrease with increases in confining pressure within mixtures of equal rock content.

3.2. Influence of Rock Size. Rock size can significantly affect the strength and deformation characteristics of soil-rock mixtures. However, there is little information about the contribution of rock size to mechanical properties. We therefore conducted medium-scale triaxial experiments to study the influence of rock size. According to the experimental design listed in Table 3, the rock content was constant at 50%; then, 4 groups of medium-scale triaxial consolidation draining shear tests were carried out and curves of $(\sigma_1 - \sigma_3)\sim\varepsilon$ were determined.

Figure 7 illustrates that, with the same rock size and different confining pressures, the relationship curve of $(\sigma_1 - \sigma_3)\sim\varepsilon$ is approximately hyperbolic with no obvious peak intensity, and the curve belongs to hardening curves. When the rock (or rock concentration) size is greater, stress-strain curves are more variable. This is caused by interaction forces between larger particles due to irregular movement, rotation, and position changes, which increase with the shear

Table 5: Elastic modulus and Poisson's ratio (in brackets) of rock content mixtures and variable confining pressures.

Rock block content (%)	Confining pressure 200 kPa	Confining pressure 400 kPa	Confining pressure 800 kPa
0	18 (0.46)	21 (0.41)	44 (0.38)
20	19 (0.46)	21 (0.41)	48 (0.43)
30	23 (0.46)	27 (0.45)	45 (0.45)
50	22 (0.46)	28 (0.44)	46 (0.43)
70	24 (0.45)	36 (0.42)	53 (0.40)

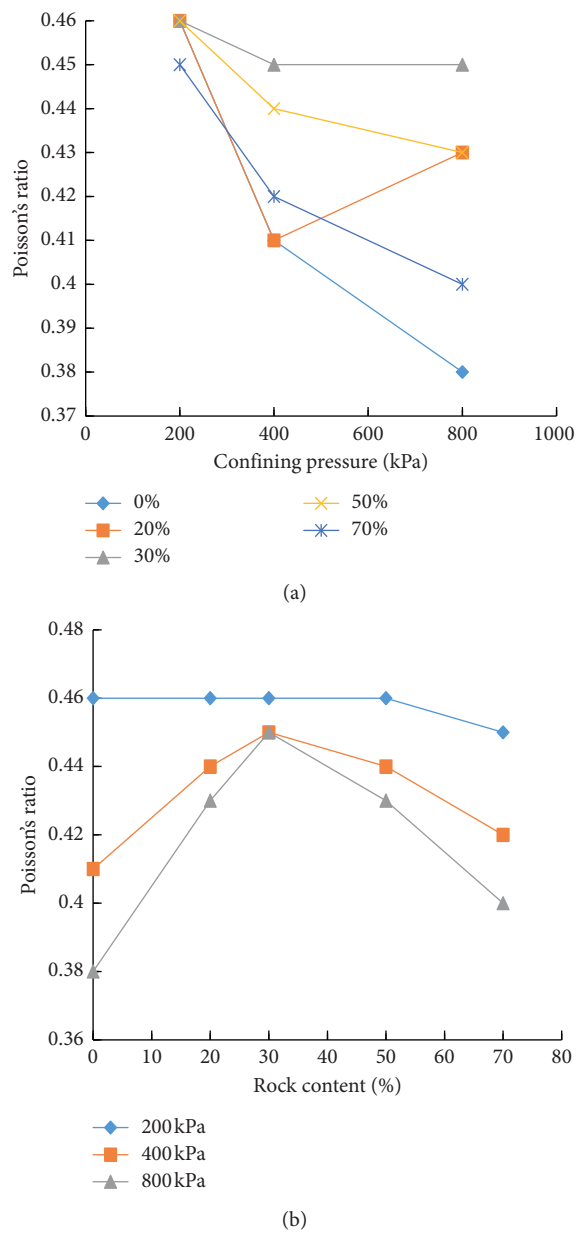

Figure 6: Poisson's ratio variation at different confining pressures (a) and rock percentages (b).

displacement. Under conditions of equal confining pressure, peak strength increases with the increase in rock size. The curve of $(\sigma_1 - \sigma_3)\sim\varepsilon$ fluctuation and hardening increased

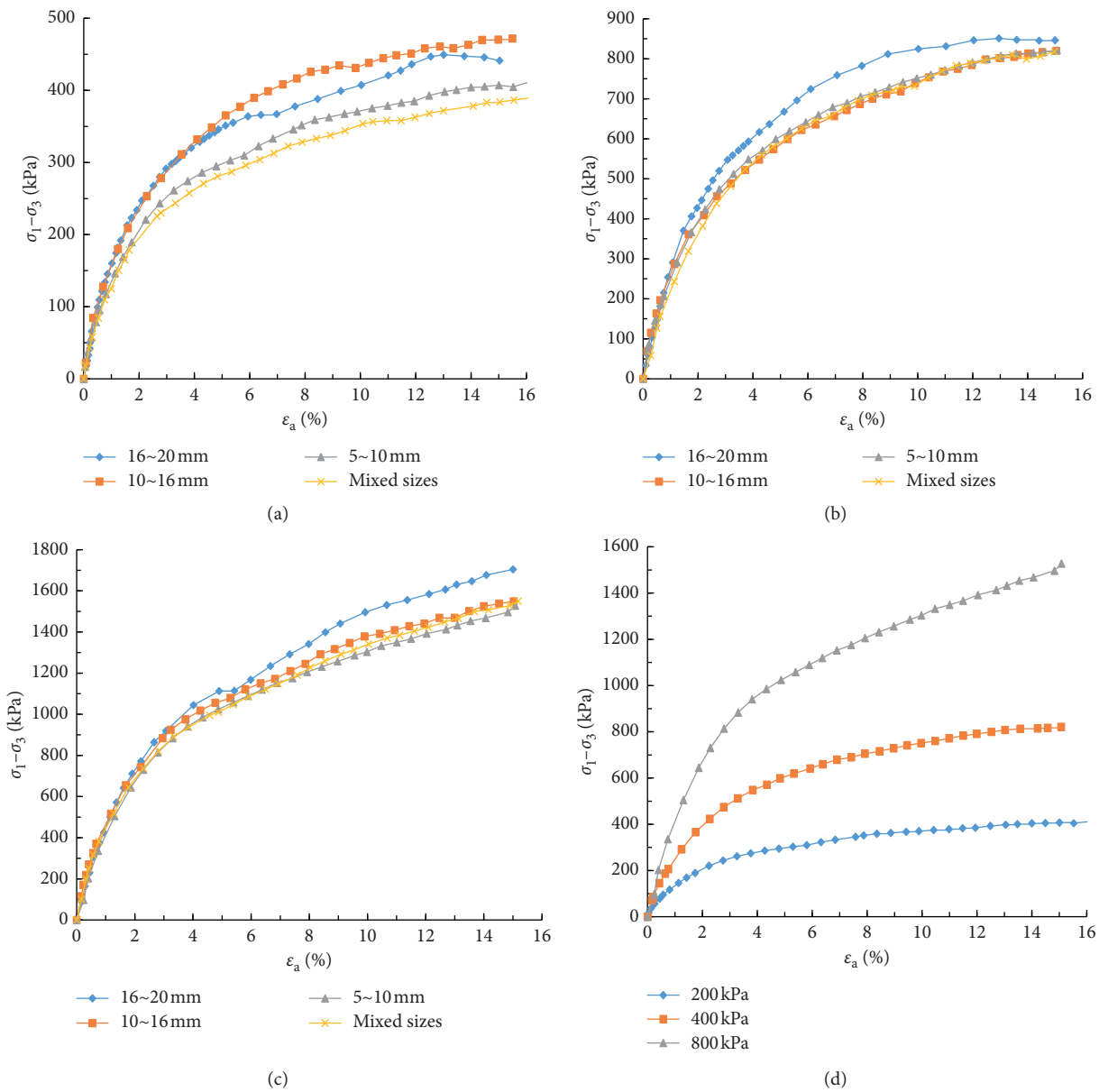

Figure 7: Curves of $(\sigma_1 - \sigma_3)$~ε under variable rock percentage or 5~10 mm size: (a) 200 kPa confining pressure; (b) 400 kPa confining pressure; (c) 800 kPa confining pressure; (d) 5~10 mm block size.

gradually under conditions of high confining pressure. In 5 mm~10 mm rock size, the peak intensity increased with the increased confining pressure, especially at 800 kPa (Figure 7(d)). The strength parameter index under different block sizes was obtained using the triaxial shear test (Table 6). Cohesion initially increased and then decreased with a decrease in rock size. In contrast, friction angle decreased initially and then increased. When rock size increased from 16 mm to 20 mm, the internal friction angle decreased 8.1% from 30.7° to 28.2°. When rock size decreased from the range of 10 mm~16 mm to the range of 5 mm~10 mm, the internal friction angle increased 2.8% from 28.2° to 29.0°. When the rock sizes are mixed, the internal friction angle is 29.6°, which is intermediate between angles of the 2 rock sizes tested independently. When rock size decreases from 16 mm~20 mm to 10 mm~16 mm, cohesion increases from 6.9 kPa to 32.2 kPa. When rock size decreases from 10 mm~16 mm to 5 mm~10 mm, cohesion decreases form 32.2 kPa to 15.5 kPa; when the rock particle size is mixed, the internal friction angle is only 5.3 kPa, the lowest value recorded among the 4 different rock size tests.

Table 6: Strength index of different rock sizes according to the Mohr–Coulomb criterion.

Rock size	ϕ (°)	C (kPa)
16~20 mm	30.7	6.9
10~16 mm	28.2	32.2
5~10 mm	29.0	15.5
Mixed size	29.6	5.3

Secant modulus with an axial strain of 1% is used as the elastic modulus (Table 7). The elastic modulus decreased with a decrease in rock size under constant confining pressure but with the same particle size, it increased with an increase in confining pressure. The curves of confining pressure and Poisson's ratio, calculated for equal rock size, are shown in Figure 8. The exact value of Poisson's ratio is shown in Table 7. Figure 8 and Table 7 show that Poisson's ratio does not change with the decrease of particle size under constant confining pressure. However, when rock size was constant, Poisson's ratio decreased with increasing confining pressure.

3.3. Influence of Random Distribution. Random distribution of particles can result in a variation of grain movement during the shearing process. As such, deformation and strength characteristics of soil-rock mixtures show variation with shearing fluctuations. Four sets of medium-scale triaxial consolidation draining shear tests, using 50% rock content and 16 mm~20 mm rock size, were completed to compare macroproperties (Table 3).

Figure 9 shows that peak stress at 15% strain varies under constant rock content and confining stress. The curve of $(\sigma_1 - \sigma_3)\sim\varepsilon$ shows a similar variation. When the strength parameter index is determined (Table 8), the random distribution of rocks shows a significant effect on cohesion and friction angle. When the rocks have different distributions, the friction angle varies between 30.7° and 26.1°. The changing magnitude of friction angle exceeds 4.6°. Cohesion fluctuates greatly between 4.7 kPa and 53.7 kPa, suggesting that the rock position and space arrangement have a more significant effect on cohesion than on friction angle. The peak intensity of different confining pressures in random location 1 is shown in Figure 9(d).

The elastic modulus of different groups of secant modulus, with axial strain at 1%, is shown in Table 9. Random distributions have a greater effect on the elastic modulus under low confining pressure (200 kPa), while the distributions have a less effect on the elastic modulus under high confining pressure (800 kPa). The elastic modulus will increase with an increase in confining pressure under the same conditions. Poisson's ratio curve with confining pressure under different rock distributions is shown in Figure 10. Poisson's ratio values, under different particle distributions, are shown in Table 9. Poisson's ratio was not significantly affected by changes in the random distribution of particles under constant confining pressure, but it decreased slightly with an increase in confining pressure under constant random distribution of particles.

4. Discussion

The strength characteristics of soil-rock mixtures vary greatly, which is one of the main characteristics different from soil and rock mass [21–23]. The rock strength is greater than soil strength, and the strength of soil-rock mixtures mainly composed of rocks greatly exceeds that of natural deposits which are mainly composed of soil [24–26]. The failure strength is influenced by material composition of soil-rock mixtures, and the failure strength of the mixtures typically increases with increasing rock content, especially under high confining pressure (Figure 5). Jafari and Shafiee [27] demonstrated that the presence of aggregates within a cohesive matrix can produce a heterogeneous matrix and the failure strength increased with the increase of aggregate content.

Table 7: Elastic modulus and Poisson's ratio (in brackets) of different confining pressures and rock sizes.

Rock size	Confining pressure 200 kPa	Confining pressure 400 kPa	Confining pressure 800 kPa
16~20 mm	22 (0.46)	28 (0.44)	46 (0.43)
10~16 mm	16 (0.45)	27 (0.44)	46 (0.43)
5~10 mm	14 (0.45)	25 (0.44)	42 (0.43)
Mixed size	13 (0.45)	23 (0.44)	44 (0.43)

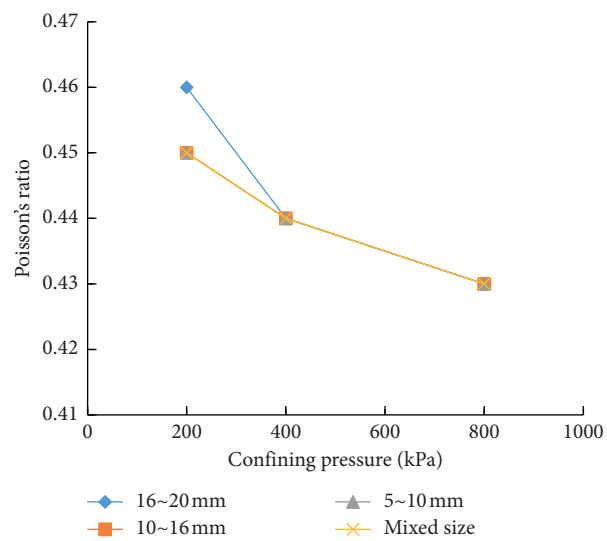

Figure 8: Poisson's ratio variation related to rock size.

The shearing tests of this study demonstrated that the mechanical properties of soil-rock mixtures are influenced by the particle sizes and proportions of soil and rock in the mixture. We conclude that soil content and compressive strength of the mixture are positively correlated. Rock content is a decisive factor affecting soil mechanical properties. But variation in the structural composition of soil can lead to significant variation in its physical and mechanical properties. Particle size, content in soil-rock mixtures, and the geometric distribution of particles greatly influence the dominant contact type. When contacts between soil-soil grains are dominant, particle content's influence on mechanical properties is minimal. When soil-rock contacts are dominant, particle content has a positive influence on compression and shear strength. When rock-rock contacts are dominant, the roles of grained soil and particles are reversed, and the properties of soil-rock mixtures are similar to those of rock-filling materials [28–31].

The variation of the internal friction angle is about 0°–4.6° under the constant rock content, which is consistent

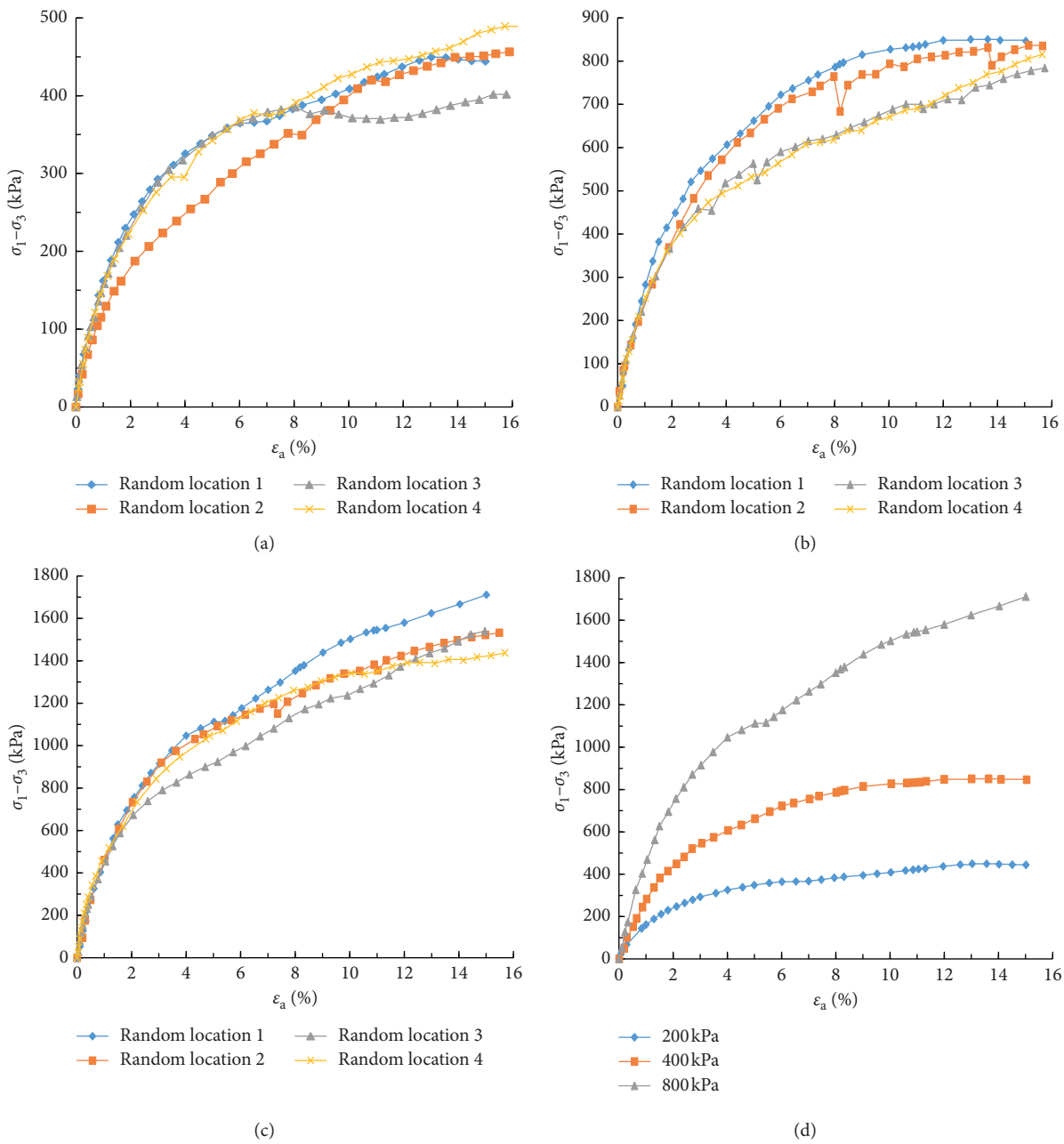

Figure 9: Curves of $(\sigma_1 - \sigma_3)\sim\varepsilon$ under different rock contents or random location 1: (a) 200 kPa confining pressure; (b) 400 kPa confining pressure; (c) 800 kPa confining pressure; (d) random location 1.

Table 8: Strength index of different random distributions.

Random distributions	φ (°)	C (kPa)
Distribution 1	30.7	6.9
Distribution 2	28.3	30.7
Distribution 3	29.2	4.7
Distribution 4	26.1	53.7

with the fluctuation observed in the stress-strain curves. Regarding soil-rock mixtures, where the geometry and particle distribution are not homogeneous, shear strength seems to be mainly associated with the percentage of rock material. The randomness of rock mass distribution contributes to significant variation of the mechanic parameters. The internal friction angle may vary significantly in mixtures with the same rock content. This contributes to the variation found in the geotechnical experiments.

Though it is difficult to characterize the mechanical parameters of soil-rock mixtures in laboratory tests, mixture properties can still be evaluated using indirect approaches. The mechanical parameters of grained soil and rocks can be studied in the laboratory. The biaxial compression numerical experiment such as the granular discrete element method can be used to analyze the influence of large-scale particles and also be used in indoor tests on the properties of soil-rock mixtures.

Table 9: Elastic modulus and Poisson's ratio (in brackets) of different confining pressures and random distributions.

Random distributions	Confining pressure 200 kPa	Confining pressure 400 kPa	Confining pressure 800 kPa
Distribution 1	22 (0.46)	28 (0.44)	46 (0.43)
Distribution 2	12 (0.45)	23 (0.44)	46 (0.43)
Distribution 3	16 (0.46)	24 (0.44)	45 (0.43)
Distribution 4	15 (0.46)	25 (0.45)	48 (0.43)

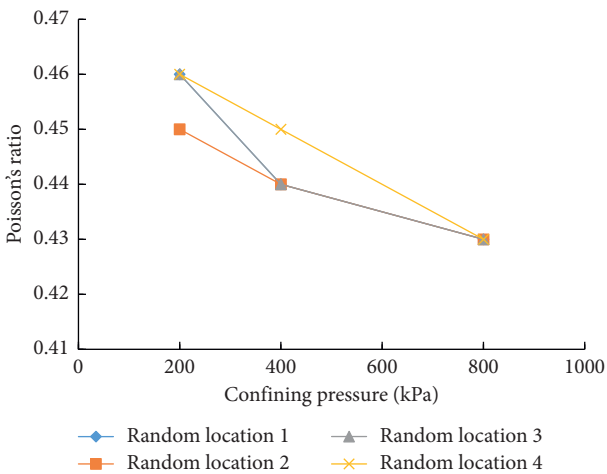

Figure 10: Rule of Poisson's ratio under different distributions.

5. Conclusions

Soil-rock mixtures have complex mechanical properties that are influenced by material composition, rock size distribution, rock content, and other factors. We studied the effects of materials and composition of mixtures on the mechanical properties of simulated deposits.

Medium-scale shearing triaxial studies were used to determine the relation between macrodeformation, strength and rock content, particle size, and random distribution of particles. Mechanical parameters and deformation characteristics of soil-rock mixtures were nonlinear, and the distribution of particles can lead to variation in strength parameters.

The strength characteristics of soil-rock mixtures vary greatly, which is prone to cause all kinds of geological hazards and engineering construction problems. In engineering design and analysis of soil-rock mixture slopes and tunnels, it is usually regarded as homogeneous and isotropic materials, which is not consistent with the actual situation. Therefore, an anisotropic analysis method should be developed to consider the shape and distribution of rock masses.

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 that there are no conflicts of interest regarding the publication of this paper.

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