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Research Article

Experimental Study on Variation Law and Mechanism of Soil Shear Strength Parameters along the Slope

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Under the action of rainwater seepage, geological origin, and human activities, the soil shear strength parameters will have spatial variability along the slope direction. After collecting samples of silty clay at a slope in the Three Gorges Reservoir area as the research object, not only the large-scale direct shear test was carried out on the site but also the direct shear test, water content test, density test, and particle grading analysis test were carried out in the laboratory with the undisturbed soil. The variation law and mechanism of soil shear strength parameters along slope were studied. The results indicate the following: (1) The coefficient of variation of shear strength parameters along the slope is relatively large. With the decrease of the elevation of the test location, the cohesion value tends to be strengthened, while the friction angle tends to degrade. (2) The mechanism of the variation law of soil shear strength parameters along the slope, which is mainly due to the decrease of the elevation, the decrease of the edges and angles between the particles, and the increase of the clay content is determined. (3) The variation model of shear strength parameters along the slope is proposed, which can provide a reference for relevant projects.

1. Introduction

The shear strength parameters of soil are very important in the calculation of soil slope stability. The accuracy of shear strength parameters directly affects the reliability of slope stability calculation results [1–5]. However, due to the effects of rainfall, human activities, and geological genesis on the slope soil, the shear strength parameters have spatial variability not only in the depth direction but also in the slope direction [6–8]. Therefore, studying the spatial variability of soil shear strength parameters and assigning more realistic soil shear strength parameters at different spatial locations of slopes are of great significance for improving the reliability of slope stability calculation.

In recent years, scholars carried out series of studies on the spatial variability and probability distribution of soil shear strength parameters of the slope. Luo et al. [9] used the homogeneous embankment slope as an example. It was found that the different variation levels of soil shear strength parameters had a significant impact on the safety factor of slope. Degroot and Baecher [10], Chiasson et al. [11], Cafaro and Cherubini [12], Chenari and Farahbakhsh [13], and Lin et al. [14] conducted field tests and investigations on clay in various areas and found that the shear strength parameters showed obvious linear changes along the buried depth. Wang et al. [15] studied the average particle-size distribution of rock and soil in the slope of the dumping site based on field investigation and laboratory test and established the relationship between the particle size composition and the friction angle of the rock and soil mass. Huang et al. [16] studied the variability of soil shear strength parameters and suggested that when the cohesion and friction angles variability were large, they follow lognormal distribution and *t*-distribution, respectively. Li et al. [17] analyzed the geological data of nearly 400 landslides in the Three Gorges Reservoir area and concluded that the cohesion of the slip

zone follows a lognormal distribution, and the friction angle obeys normal distribution. Luo et al.'s study was [18] based on the statistical analysis of the cohesive and friction angle of 1074 slip zone soils in Wanzhou District of the Three Gorges Reservoir area and concluded that the cohesion and friction angle of the slip zone soil in this region obey the normal distribution. Su et al. [19] based on the shear strength of Shanghai soft soil, using K-S statistical analysis, concluded that the logarithmic normal distribution is more reasonable and convenient for the shear strength parameters. Based on the nonstationary random field model, Jiang and Huang [20] found that the mean and standard deviation of the undrained shear strength parameters of the soil increased with the increase of the buried depth. Zhang et al. [21] established a joint probability density function of cohesion and friction angle of soil, based on the Bayesian model. Gong et al. [22] proposed a normal information diffusion method for estimating the probability distribution of shear strength parameters of rock and soil, which was almost independent of the change of sample capacity.

The above studies mainly focus on the spatial variability and probability distribution models of the shear strength parameters of regional slopes, and most of them focus on the difference of the shear strength parameters of different soil types in the buried depth direction, while few studies were conducted on the variation of the shear strength parameters along the slope of the same geological layer in a single slope. In this paper, the silty clay of a sediment slope in Shazhenxi Town of the Three Gorges Reservoir area was taken as the research object, and the large-scale direct shear test, laboratory direct shear test, water content test, density test, and particle sieving analysis test were adopted, respectively. The variation law and mechanism of shear strength parameters of soil along the slope direction (from high to low) were studied.

2. Test Scheme Design

2.1. In Situ Test Scheme Design

2.1.1. In Situ Test Location Selection. The test locations were selected based on the following principles: ① The test locations should be relatively flat, and it is convenient to arrange the test equipment and carry out the in situ direct shear test. ② In order to study the variation law, there should be enough height difference between adjacent test locations and be in a line along the slope direction.

Based on the above principles, 4 test locations were selected for the in situ large-scale direct shear test. The numbers were ranged from high to low termed XCZJ1, XCZJ2, XCZJ3, and XCZJ4, and the corresponding elevations were 285 m, 271 m, 251 m, and 224 m. The elevation of slope foot was 183 m, as shown in Figure 1.

2.1.2. In Situ Sample Preparation. When preparing the in situ direct shear test samples, first remove the cover layer about 30 cm thick on the surface, then draw a square of 510 mm × 510 mm on the ground, excavate the grooves along the circumference of the square, and process it into a sample of 500 mm × 500 mm × 400 mm, as shown in Figure 2.

2.1.3. In Situ Test Equipment. The in situ direct shear tests were carried out using a self-made test equipment system, as shown in Figure 3. The test equipment system includes the following: ① Upper and lower shear boxes connected by sliding tracks. ② Jacks that apply the normal force and the horizontal thrust, the three samples of XCZJ1, XCZJ2, XCZJ3, and XCZJ4 at each test location, respectively, subjected to three normal stresses of 20.00 kPa, 33.33 kPa, and 46.67 kPa. ③ Dial indicators for measuring horizontal and normal displacements and some magnetic supports for fixing the dial indicators. ④ Reaction frames that provide counterforces for the normal and tangential jacks. In order to provide a stable normal force, 4 steel columns are arranged for the test equipment, the bottom end of each steel column is fixed on the lower shear box, and the top end is anchored to the ground by a wire rope.

2.2. Laboratory Test Scheme Design

2.2.1. Laboratory Sample Preparation. Four plastic storage boxes were used to seal the undisturbed soil in four test locations on site and then transported back to the laboratory. According to the standards [23], the ring knives with a sampling area of 3000 mm^2, an inner diameter of 61.8 mm, and a height of 20 mm were used in the laboratory to sample the undisturbed soil in the plastic storage boxes and perform the direct shear test. The water content tests, the natural density tests, and the particle sieving tests were also carried out on the undisturbed soil in the laboratory.

2.2.2. Laboratory Test Equipment. The ZJ strain-controlled quadruple direct shear instrument was adopted in the laboratory direct shear test, which can simultaneously perform shear tests of four samples under different normal stresses. The normal stress was, respectively, applied at 100 kPa, 200 kPa, 300 kPa, and 400 kPa. The direct shear instrument is shown in Figure 4.

3. Result Analysis of the Tests

3.1. Result Analysis of the In Situ Tests. The samples in the in situ direct tests after shear failure are shown in Figure 5. The curves of shear stress versus shear displacement of four different elevation test locations under three different normal stresses are shown in Figure 6.

It can be seen from Figure 6 that the relationship curves of shear stress and shear displacement show the same plastic deformation characteristics; that is, the shear stress increases rapidly in the early stage and slows down in the later stage, and the slope of the curves changes continuously from large to small. Under the same normal stress, the lower the elevation of the test location, the earlier the peak strength appears.

The peak strength curves of the four different elevation test locations are shown in Figure 7 under the three normal

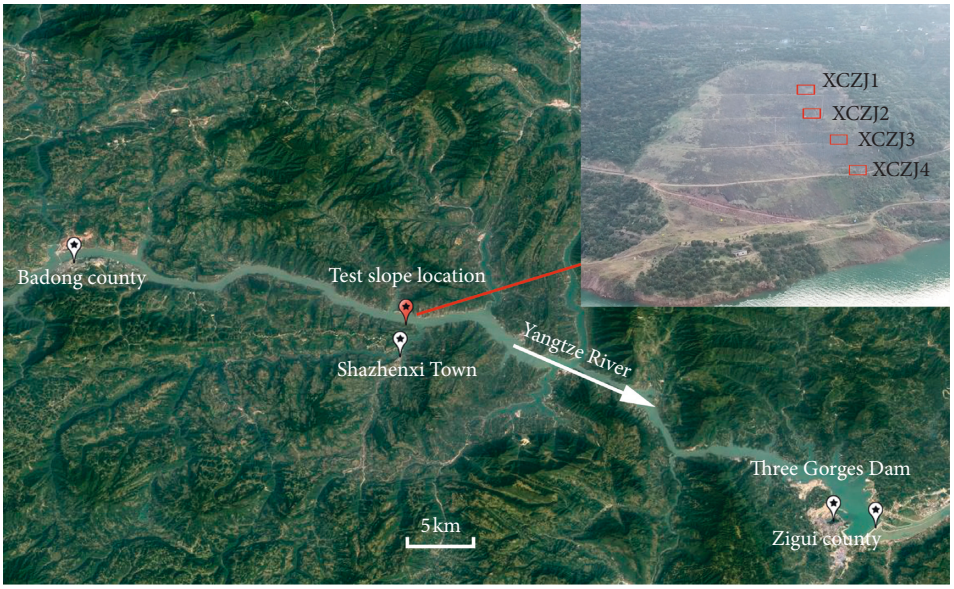

Figure 1: Map of in situ direct shear test locations.

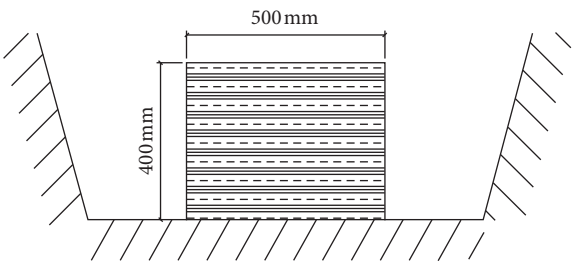

Figure 2: In situ direct shear samples preparation.

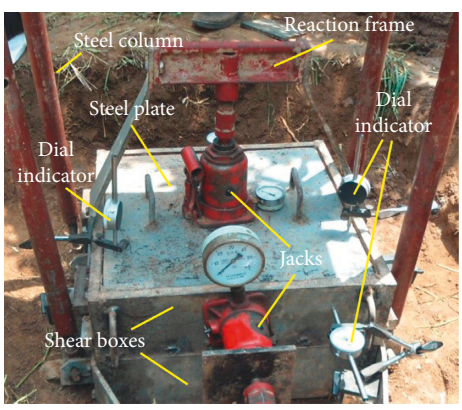

Figure 3: In situ direct shear test equipment.

Figure 4: ZJ strain-controlled quadrupole direct shear instrument.

stresses. It can be seen from Figure 7 that, under the same normal stress condition, the peak strength first decreases and then increases with the decrease of the test location elevation; the peak strength increases with the increase of normal stress under different normal stress conditions.

Based on the Mohr–Coulomb criterion, the in situ direct shear test results at four different elevation test locations under different normal stresses were analyzed, and the cohesion and friction angles of the four test locations were obtained, as shown in Table 1.

In order to analyze the variation of cohesion and friction angle along the slope direction, the degree of deterioration was used to indicate the degree of decrease of the parameters along the slope, and the degree of strengthening was used to indicate the degree of enhancement of the parameters along the slope, as shown in the following equations:

$$L_i = \frac{l_1 - l_i}{H_1 - H_i}, \quad i = 2, 3, 4, \tag{1}$$

$$\Delta L_i = \frac{l_{i-1} - l_i}{H_{i-1} - H_i}, \quad i = 2, 3, 4, \tag{2}$$

$$Q_i = \frac{q_i - q_1}{H_1 - H_i}, \quad i = 2, 3, 4, \tag{3}$$

(a) (b)

Figure 5: Damaged samples of in situ direct shear tests.

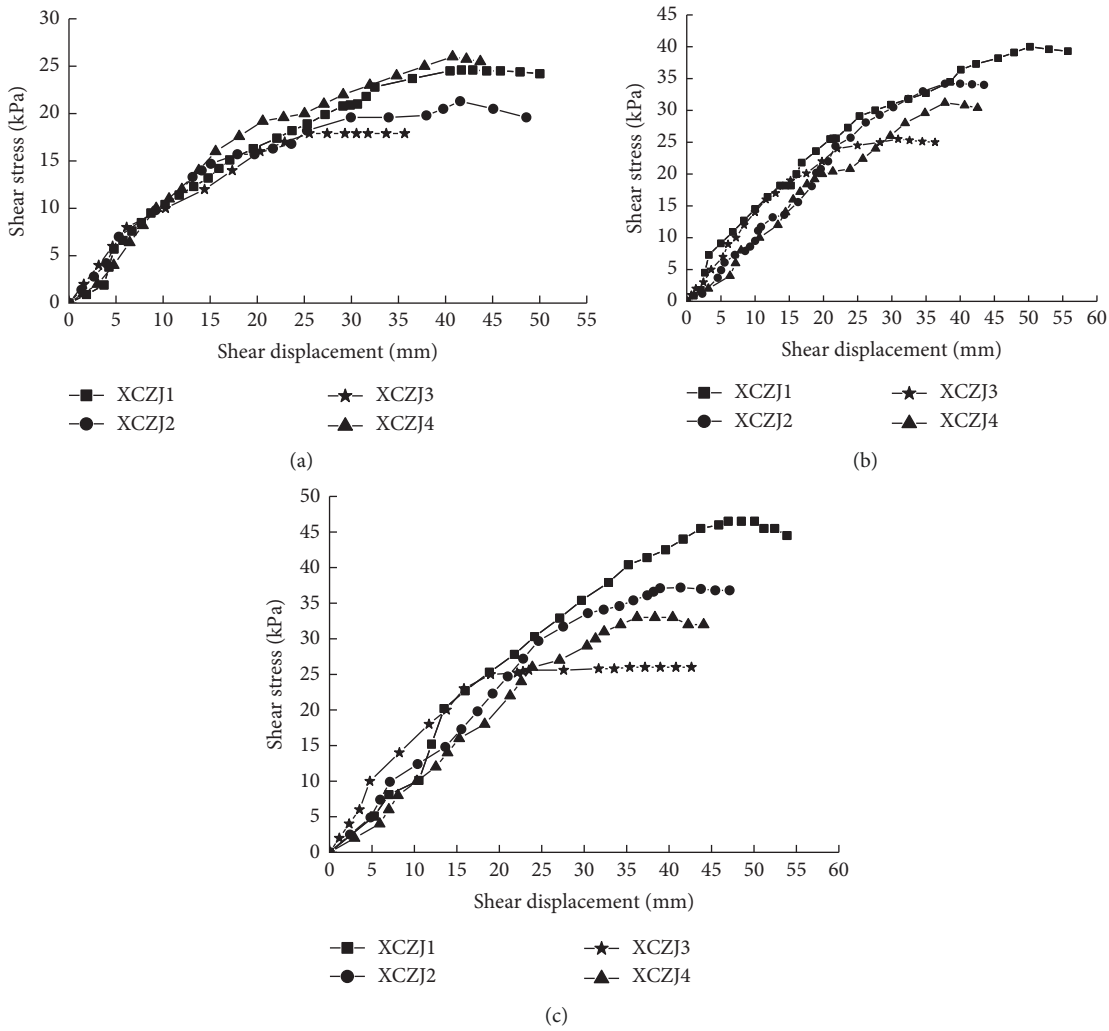

Figure 6: Shear stress and shear displacement curves of in situ direct shear tests: (a) 20.00 kPa; (b) 33.33 kPa; (c) 46.67 kPa.

$$\Delta Q_i = \frac{q_i - q_{i-1}}{H_{i-1} - H_i}, \quad i = 2, 3, 4, \tag{4}$$

where L_i is the total deterioration degree of the parameters, ΔL_i is the stage deterioration degree of the parameters, l_1 is the shear strength parameters obtained at the reference test location, l_i and l_{i-1} are the shear strength parameters obtained at other test locations, i is the test location number, H_1 is the elevation of the reference test location, H_i and H_{i-1} are elevations of other test locations, Q_i is the total strengthening degree of the parameters, ΔQ_i is the stage strengthening degree of the parameters, q_1 is the shear strength parameters

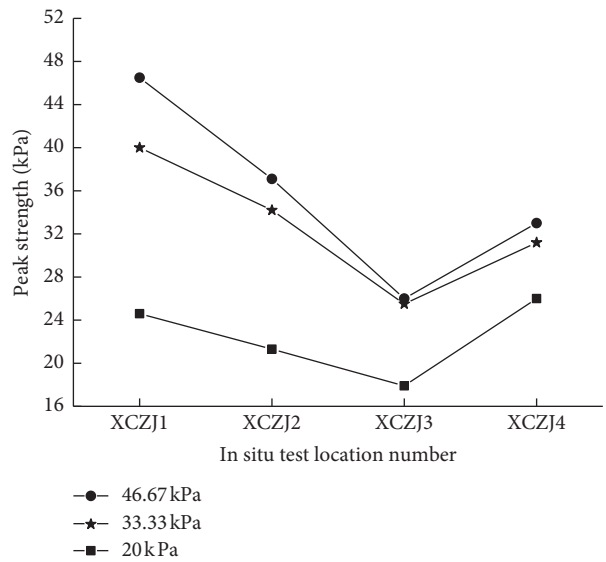

Figure 7: Variation of in situ test peak strength with different test locations.

Table 1: Cohesion and friction angle of the in situ test.

Test location number	Elevation of test location (m)	c (kPa)	Total strengthening degree (kPa/m)	Stage strengthening degree (kPa/m)	φ (°)	Total deterioration degree (°/m)	Stage deterioration degree (°/m)
XCZJ1	285	9.66			39.35		
XCZJ2	271	11.12	0.10	0.10	30.54	0.63	0.63
XCZJ3	251	13.01	0.10	0.09	16.70	0.67	0.69
XCZJ4	224	21.32	0.19	0.31	14.57	0.41	0.08

obtained at the reference test location, and q_i and q_{i-1} are the shear strength parameters obtained at other test locations.

Taking the test data obtained from the test location with an elevation of 285 m as a reference, the total strengthening degree and the stage strengthening degree of the cohesion and the total deterioration degree and the stage deterioration degree of the friction angle are calculated. The results are summarized in Table 1 and Figure 8.

It can be seen from Table 1 and Figure 8 that the cohesion tends to increase with the decrease of the test location elevation. The lower the elevation, the greater the total and stage strengthening degree. The friction angle decreases with the decrease of the test location elevation. The lower the elevation, the total and stage deterioration degree first increase and then decrease, and the overall trend is decreasing.

In order to further study the variation law of soil shear strength parameters with the slope direction and analyze the mechanism of this variation law, the laboratory direct shear tests, water content tests, natural density tests, and particle sieving tests were carried out by using the undisturbed samples taken from the site.

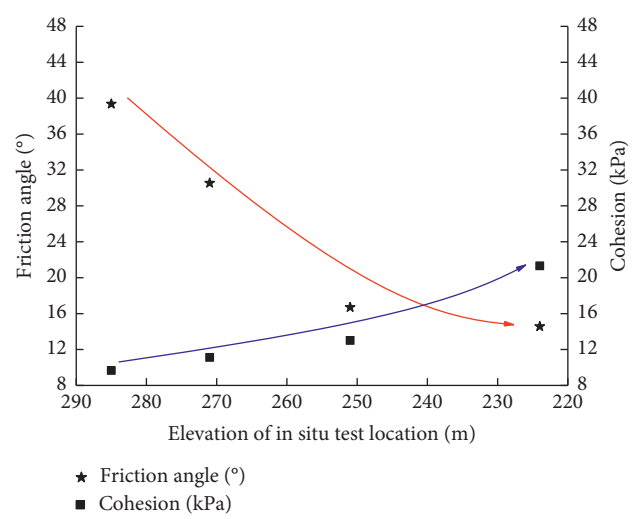

Figure 8: The evolution curve of cohesion and friction angle in in situ direct shear tests.

3.2. Result Analysis of the Laboratory Tests

3.2.1. Laboratory Direct Shear Tests. The laboratory direct shear tests were carried out on the undisturbed soil samples taken from the site. The numbers were ranged from high to low termed SNZJ1, SNZJ2, SNZJ3, and SNZJ4, and the corresponding elevations were 285 m, 271 m, 251 m, and 224 m, respectively. The shear stress-displacement curves of four different elevation samples under four different normal stresses are shown in Figure 9.

As can be seen from Figure 9, the characteristics of the shear stress and shear displacement curves of the laboratory direct shear tests are generally consistent with that in the in situ direct shear tests.

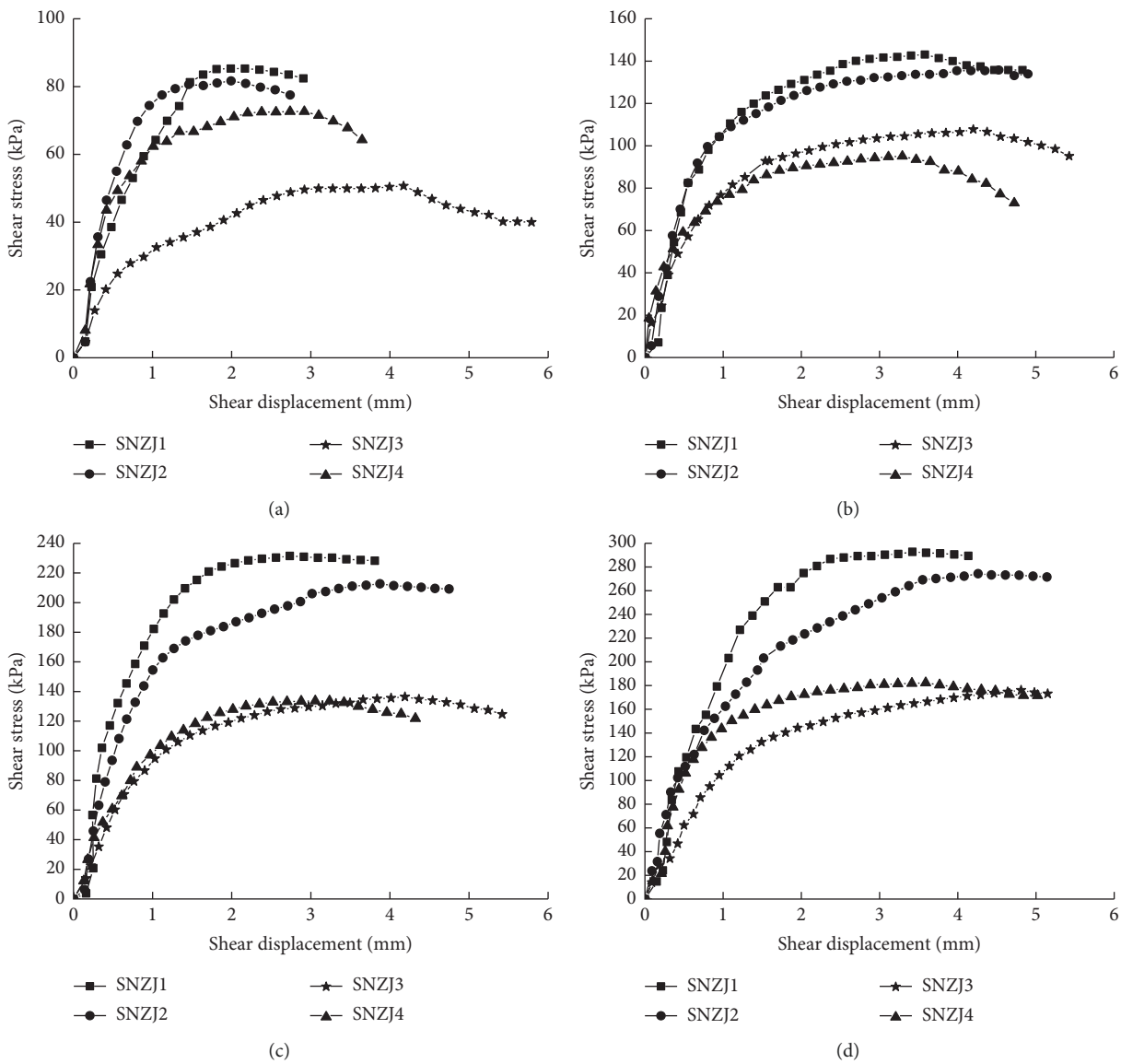

Figure 9: Shear stress-shear displacement curves of laboratory direct shear tests: (a) 100 kPa; (b) 200 kPa; (c) 300 kPa; (d) 400 kPa.

The peak strength curves of four different elevation test locations under four normal stresses were calculated separately, as shown in Figure 10. It can be seen from Figure 10 that, under the same normal stress condition, the peak strength generally decreases with the decrease of the test location elevation; under different normal stress conditions, the peak strength increases with the increase of the normal stress.

Based on the Mohr–Coulomb criterion, the results of laboratory direct shear tests under different normal stresses at four different test locations were analyzed, and the cohesion and friction angles of the four test locations were obtained. The total strengthening degree, the stage strengthening degree, the total deterioration degree, and the stage deterioration degree were calculated, as shown in Table 2 and Figure 11.

It can be seen from Table 2 and Figure 11 that the cohesion tends to increase with the decrease of the test location elevation. The lower the elevation, the greater the total strengthening degree and the stage strengthening degree. The friction angle decreases with the decrease of the test location elevation. When the elevation is lower, the total deterioration degree and the stage deterioration degree first increase and then decrease, and the overall trend is decreasing. The variation of the cohesion and friction angles obtained by the laboratory direct shear test along the slope direction is the same as that the in situ direct shear test.

3.2.2. Evolution Mechanism Analysis. The main factors affecting the shear parameters of soil include mineral composition of the soil, particle gradation, water content, and compactness [24–26]. Because the mineral composition of the soil in the same stratum of a slope is basically the same, this paper studies the variation mechanism of the shear strength parameters from the soil particle gradation, water content, and compactness.

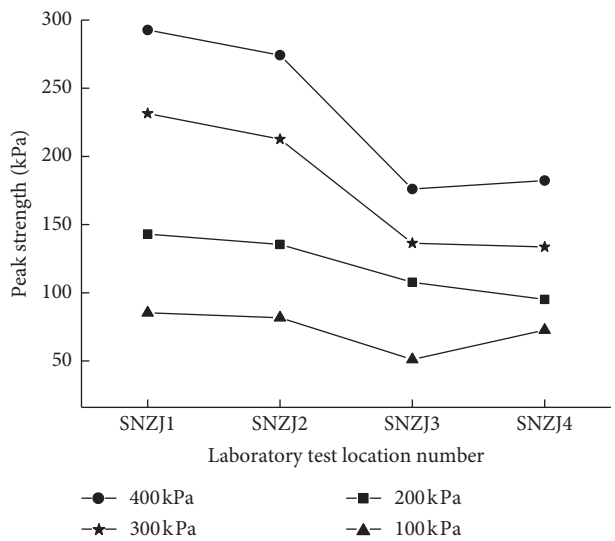

Figure 10: Variation of peak strength versus laboratory test location.

Table 2: Cohesion and friction angle of the laboratory test.

Test location number	Elevation of test location (m)	c (kPa)	Total strengthening degree (kPa/m)	Stage strengthening degree (kPa/m)	φ (°)	Total deterioration degree (°/m)	Stage deterioration degree (°/m)
SNZJ1	285	10.48			35.40		
SNZJ2	271	12.27	0.13	0.13	33.22	0.16	0.16
SNZJ3	251	16.94	0.19	0.23	21.80	0.40	0.57
SNZJ4	224	29.05	0.30	0.45	20.17	0.25	0.06

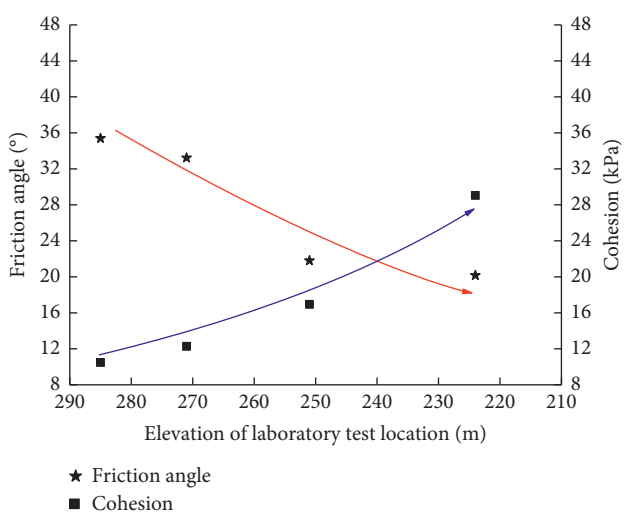

Figure 11: Evolution of shear strength parameters in laboratory direct shear tests.

The same numbering principle as the laboratory direct shear tests is adopted. The water content test numbers are SNHS1, SNHS2, SNHS3, and SNHS4; the natural density test numbers are SNMD1, SNMD2, SNMD3, and SNMD4; and the particle sieving analysis test numbers are SNKF1, SNKF2, SNKF3, and SNKF4. The water content test results are shown in Table 3, and the natural density test results are shown in Table 4.

The water content test results in Table 3 show that the water content of the samples at different test locations is between 18.14% and 18.42%, and the standard deviation is 0.111, with almost no difference. The natural density test results in Table 4 show that the natural density of the samples at different test locations is between 2.005 g·cm^{-3} and 2.023 g·cm^{-3}, and the standard deviation is 0.007, which is basically the same. It is obvious that the water content and natural density of different location samples are very close, which is not the fundamental cause of the change of the shear strength parameters.

Grading curves of different test location soils are shown in Figure 12.

Particles with a particle diameter larger than 2 mm are called the gravel group, and the particle with 2 mm diameter is the threshold with or without capillary force. Therefore, it can be considered that the percentage of particles larger than 2 mm is the main factor affecting the friction angle of soil [27, 28]. The particle diameter of 0.5 mm is the limit value with or without adhesion. It can be considered that the percentage of particles smaller than 0.5 mm is the main factor affecting the cohesion of soil [27].

The percentages of particles larger than 2 mm and smaller than 0.5 mm at different test locations are shown in Table 5. Figure 13 plots the percentage of particles larger than 2 mm and the friction angle versus the elevation of the test location. Figure 14 plots the percentage of particles smaller than 0.5 mm and the cohesion versus the elevation of test location.

Table 3: Water content test results.

Number	SNHS1	SNHS2	SNHS3	SNHS4
Water content (%)	18.42	18.14	18.40	18.30
Standard deviation		0.111		

Table 4: Natural density test results.

Number	SNHS1	SNHS2	SNHS3	SNHS4
Natural density (g·cm^{-3})	2.023	2.020	2.005	2.023
Standard deviation		0.007		

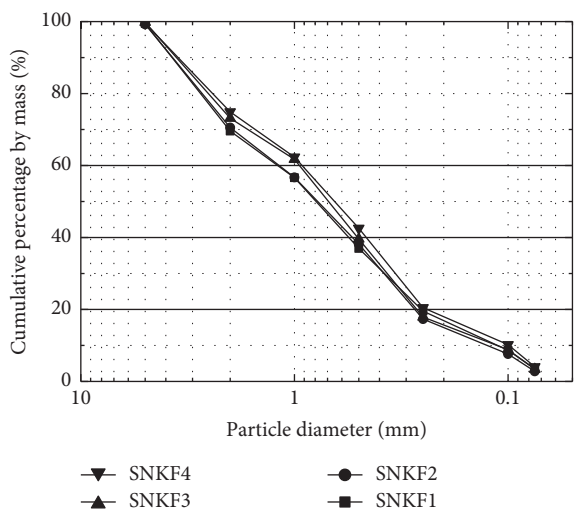

Figure 12: Grading curves of undisturbed soil samples.

The percentage of particles larger than 2 mm decreases with the decrease of elevation, while the percentage of particles smaller than 0.5 mm increases with the decrease of elevation. This may be caused by the seepage of rainfall, which brings the fine particles in the soil from high to lower. The friction angle of the soil decreases with the decrease of the percentage of particles larger than 2 mm because the angular angle between the contact of soil particles decreases and the bite force decreases. The cohesion increases with the increase of the percentage of particles smaller than 0.5 mm mainly because the voids between the soil particles are reduced, the clay content is increased, and the adhesion is increased. The test results are consistent with the opinions of Li et al. [29].

4. Variation of Shear Strength Parameters

The range of cohesion and friction angle variation coefficients involved in the existing literature is statistically summarized in Table 6 [30, 31], which together with the cohesion and friction angle variation coefficients is obtained in this paper.

Comparing and analyzing the data in Table 6, it can be seen that the variation coefficients of the cohesion and friction angles obtained in this paper are within a reasonable range, but the variation coefficients are large, indicating that

Table 5: Percentage of particles larger than 2 mm and smaller than 0.5 mm.

Number	Larger than 2 mm (%)	Smaller than 0.5 mm (%)
SNKF1	30.44	36.96
SNKF2	29.47	38.16
SNKF3	26.85	39.59
SNKF4	25.10	42.56

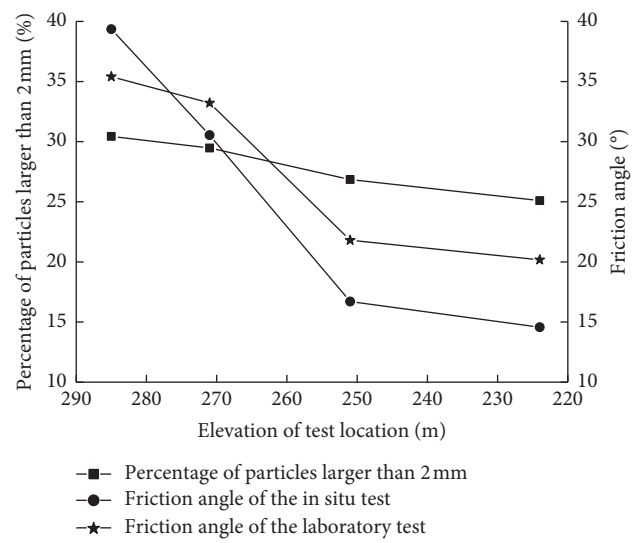

Figure 13: Percentage of particles larger than 2 mm and the friction angle versus the elevation of test location.

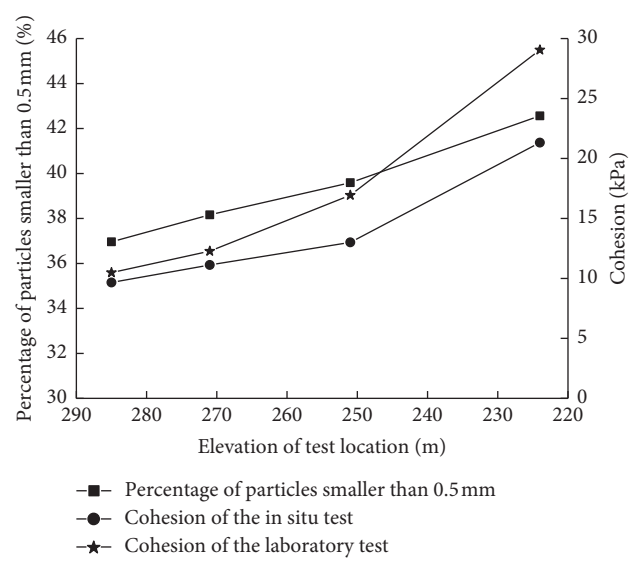

Figure 14: Percentage of particles smaller than 0.5 mm and the cohesion versus the elevation of test location.

Table 6: Cohesion and friction angle variation coefficients of soil.

Category	Cohesion (kPa)	Friction angle (°)
Literature	0.19–0.55	0.05–0.40
In situ test	0.33	0.40
Laboratory test	0.42	0.24

Table 7: Synthetic shear strength parameters.

Elevation of test location H (m)	Elevation difference h (m)	c (kPa)	Total strengthening degree of soil cohesion c' (kPa/m)	φ (°)	Total deterioration degree of soil friction angle φ' (°/m)
285	0	10.07	—	37.38	—
271	14	11.70	0.115	31.88	0.393
251	34	14.98	0.145	19.25	0.533
224	61	25.19	0.245	17.37	0.328

the variability of shear strength parameters of the soil along the slope should not be ignored.

Assuming that the change of shear strength parameters along the slope is a continuous process, the evolution models of shear strength parameters along the slope can be established. Due to the integrity of the fracture network and high fracture rate of large-size samples, the shear strength parameters obtained by the in situ direct shear test are generally smaller than those obtained by the laboratory direct shear test. The weight of in situ test results and laboratory test results was taken as 50%, respectively, and the friction angle and cohesion were calculated to analyze the variation law of shear strength parameters. Taking 285 m test location as the reference location, the height difference, the total strengthening degree of cohesive, and the total deterioration degree of friction angle are calculated, and the results are summarized in Table 7. Figure 15 plots the evolution curves of the total strengthening degree of cohesion and the total deterioration degree of friction angle with the height difference. The evolution models are shown in equations (5)–(8).

The evolution model of total strengthening degree of cohesion along the slope:

$$c' = 5 \times 10^{-5} h^2 - 0.0011h + 0.1212. \quad (5)$$

Soil cohesion along the slope to a certain elevation:

$$c = c_0 + h \times c' \leq [c]. \quad (6)$$

The evolution model of total deterioration degree of friction angle along the slope:

$$\varphi' = 3 \times 10^{-4} h^2 - 0.0219h + 0.1472. \quad (7)$$

Soil friction angle along the slope to a certain elevation:

$$\varphi = \varphi_0 - h \times \varphi' \geq [\varphi], \quad (8)$$

where c' is the total strengthening degree of cohesion (kPa/m), h is the elevation difference from the desired location of the slope to the reference location (m), c is the cohesion of the soil in the desired location (kPa), c_0 is the cohesion of soil at reference location (kPa), $[c]$ is the limit value of soil cohesion (kPa), φ' is the total deterioration degree of soil friction angle (°/m), φ is the friction angle of the soil in the desired location (°), φ_0 is the friction angle of soil in the reference location (°), and $[\varphi]$ is the limit value of soil friction angle (°).

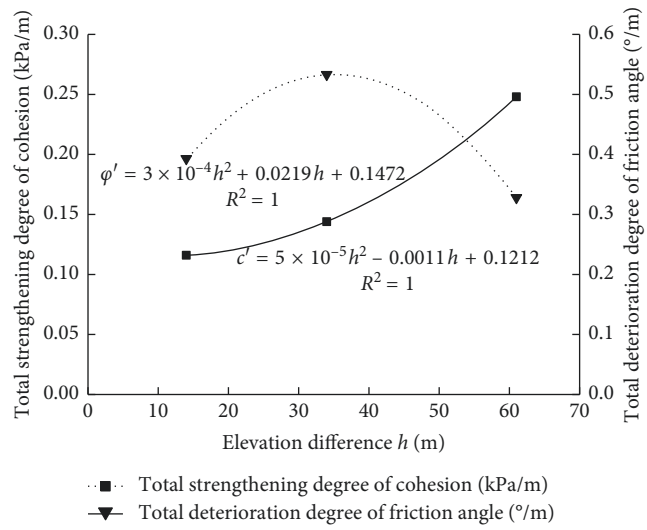

Figure 15: Evolution curve of shear strength parameters with elevation difference.

5. Conclusions

This paper is based on the in situ direct shear test, laboratory direct shear test, water content test, density test, and particle sieving analysis test, and the main conclusions are as follows:

(1) The curve of shear stress change with shear displacement shows the plastic deformation characteristics. Under the same normal stress condition, the lower the test location elevation, the earlier the peak strength of the soil appears. The peak strength of the soil generally decreases with the decrease of the elevation of the test location.

(2) The cohesion of the soil along slope shows a trend of continuous strengthening with the decrease of the test location elevation. The lower the elevation, the greater the total and the stage strengthening degree. The friction angle shows a continuous deterioration trend with the decrease of the test location elevation. When the elevation is lower, the total and the stage deterioration degree first increase and then decrease, and the overall trend is decreasing.

(3) The mechanism of the abovementioned variation law of the shear strength parameters of the soil along slope is mainly due to the dragging force of rainfall seepage. The fine particles in the soil are brought by the dragging force from the high level location to the low level location, resulting in the reduction in the edge angles of the soil particles at the lower level

location and the increase in the clay content and adhesion. But the superposition effect of other occurrence conditions of the soil is not excluded. The mechanism is independent of the water content and natural density of the soil.

(4) In view of the large variation coefficient of the shear strength parameters of the soil along the slope, it is suggested to consider the variability of the shear strength parameters along the slope and the buried depth in the stability analysis of the slope. The evolution model of the shear strength parameters of soil along the slope is presented in this paper, which can serve as a reference for similar projects.

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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