Research Article

Development Characteristics and Sensitivity Analysis of Expansive Soil Slope

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According to laboratory tests of soil samples of Wangyan highway landslide, the mechanism of formation of landslide and the degree of sensitivity of various factors to slope stability are analyzed. It is found that the landslide is destroyed by the properties of expansive soil, the existence of water in the cracks, the loading of artificial filling, and the traction of old landslide. The soil samples were tested for SEM and MIP tests by lyophilization, and the particle morphology, the mode of combination, porosity, and connectivity of the sliding zone soil were observed by a scanning electron microscope. Results show that the sliding zone developed many microfractures and the clay minerals arranged tightly, the pore of size < 0.1 μm was 60% in undisturbed soil, and the pores of different sizes were distributed uniform in reconstituted soil. The undisturbed soil had typical nonlinear rheological property; The deformation rate of reconstituted was bigger; When the load was 50 kPa and 200 kPa, an expansion phenomena occurred in the sliding zone, which indicated that microstructure could influence the deformation. The strength attenuation of expansive soil slope due to the high content of montmorillonite in the sliding body and the high water content in the cracks are two major factors of the Wangyan highway landslide. Based on the results of previous studies, the a, n, and m values of fitting parameters of Fredlund-Xing soil-water characteristic curve are predicted by using the basic physical properties of soil. It is found that the air intake value and residual suction value of filled soil (loose and broken) and completely weathered sandstone are small, and the water holding capacity in the middle suction range is relatively good. Then, the finite difference software Flac3D was used to build a three-dimensional geological model. The influence of water content, highway load, filling weight, and slope angle on slope stability is considered comprehensively. The order of sensitivity of slope stability about various factors is as follows: water content > slope angle > highway load > filling weight. The results of research can provide a scientific basis for the treatment, protection, and reinforcement of the Wangyan highway landslide.

1. Introduction

With the rapid development of slope and tunnel engineering, the damage caused by soft rock has become the focus and difficulty in engineering practice and key problems in science and technology are tackled. For the slope excavated in soft rock, if the weathering process cannot be prevented, the internal shear strength of the slope will be reduced [1]. Therefore, it is necessary to consider comprehensively the action of water, types of soil, weathering erosion, and other factors to study the characteristics of weak interlayer so as to improve the stability of soft rock slope. Otherwise, a miss is as good as a mile will happen [2, 3].

Expansive rock-soil is a special part of soft rock. It came to light that the property of expansive soil expands after water absorption, shrinks after losing water, and deforms repeatedly. In addition, the random distribution characteristics of cracks in the soil mass have serious damage to the slope, subgrade, channel, and other buildings and often have the characteristics of long-term potential danger [4]. Based on this point, it has become a crucial issue in soil mechanics to explore the influence of the internal mesostructure of expansive soil on its evolution.
law [5]. The amount of water content in the crack determines its shear strength. Stoltz et al. [6] studied the engineering characteristics of expansive soil improved by lime, which can inhibit the volume expansion of expansive soil in its initial state effectively. Bao [7] and Zhan and Wu [8] focused on the influence of suction on expansive soil characteristics and analyzed the mechanism of rainfall infiltration and the strength change law of unsaturated fractured expansive soil quantitatively. Huang and Wu [9] and Liu and Yin [10] analyzed the influence of bedding and cracks on the stability of unsaturated expansive soil slope by using simplified shear strength equation combined with hyperbolic fitting method and improved Bishop method. In addition, the effects of swelling-shrinkage deformation, development of cracks, and reduction of strength caused by water and the change of stress cannot be ignored for the instability of expansive soil slope [11]. Kong et al. [12, 13] explored the engineering properties of undisturbed expansive soil with cracks under the action of water content and fracture properties and compared the differences in deformation and strength characteristics of expansive soil improved with lime, remolded expansive soil, and undisturbed expansive soil. The existence of cracks in expansive soil provides infiltration channels for water flow, and transient saturated-unsaturated seepage occurs in cracks, which changes the distribution of volumetric water content and has great damage to slope stability [14, 15]. Jiang et al. [16] and Khan et al. [17] studied the changing process of three-dimensional unsaturated seepage field of slope caused by rainfall infiltration and came to the conclusion that the complete softening condition and rainfall were the main causes of slope failure finally. What is more, the slope angle, the overburden, and the type of filling material will also play an important role in the stability of the slope. For example, Qi and Vanapalli [18] studied the effects of conditions of initial stress, softening rate, and slope angle on the failure time and depth of shallow expansive soil slope. Yin et al. [19] studied the stability of expansive soil slope, considered the large shear stress appearing near the interface between saturated and unsaturated zones, and explain the reason for the failure of expansive soil slope when the slope is relatively flat. Yang and Huang [20] found that the change of earth pressure in expansive soil is the result of the comprehensive action of the change of soil weight and expansive soil pressure caused by the change of water content in soil mass caused by different climatic conditions through the study of expansive soil embankment. Zhang et al. [21] summarized the relationship between the loaded expansion rate of compacted expansive soil and its initial water content, overburden, and compactness.

Wangyan highway landslide is a high filling embankment landslide occurring in Xinyan section of Wangyan highway, which belongs to expansive soil landslide. Its expansibility, dispersibility, extremely low shear strength, the phenomenon of strain softening, and preferential flow are the main reasons for repeated failure of soft rock gentle slope in this area [22]. At present, the quantitative methods used for the analysis of slope stability are mainly limit equilibrium method [23] and numerical analysis method [24]. In this paper, by means of microscopic experiments and numerical analysis, the influence of water content, highway load, filling weight, slope angle, and other factors on the stability of soft rock slope with soft interlayer characteristics is studied, aiming at predicting the stability of expansive soil slope and providing basis for slope treatment and optimization of reinforcement scheme.

2. Analysis of Landslide Mechanism

The paper mainly analyzes the forming mechanism reasons of Wangyan highway landslide from five aspects: expansion rock mass, artificial filling, water content, inheritance relationship between old landslide and recent landslide, and progressive failure. The pictures of Wangyan highway landslide is shown in Figures 1 and 2.

2.1. Expansive Rock-Soil Mass. The material of Wangyan highway landslide is mainly the artificial embankment made of expansive rock-soil. Due to the swelling-shrinkage behavior, strength attenuation, and weathering of expansive soil, as time goes by, the strength of artificial filling subgrade decreases due to environmental factors, leading to the failure of slope [25]. At the same time, it is found that not only the upper part of the embankment is filling with expansive soil, but the silty clay, weathered mudstone, and sandstone in the lower part are also typical expansive soil; that is, the whole slope is composed of expansive rock-soil. The swelling-shrinkage behavior of expansive soil leads to the crack of slope soil, and the slope is seriously damaged by rainwater infiltration. Therefore, the expansive rock-soil mass provided the material basis for the landslide.

2.2. Artificial Filling. On the one hand, the artificial filling changes the original terrain conditions; it makes the slope angle of the original gentle slope increase and provides lateral deformation space for the formation of landslide. On the other hand, artificial filling is made of expansive materials, which is not allowed in engineering. It can make a variety of threats to slope engineering.

2.3. Water Content. The slope is relatively stable without considering the influence of water on it. If the influence of water is considered, the soil mass of expansive rock-soil slope will absorb water and expand, which will lead to the reduction of the structural strength of soil mass, and cause the overall failure of the slope finally. Groundwater and meteoric water are the main sources of water in soil. If not discharged in time, the soil will produce swelling force, resulting in the reduction and destruction of soil strength. It can be seen from Table 1 that the saturated residual shear strength cohesion of silty clay is only 15 kPa, the internal friction angle is only 5°, the saturated residual shear cohesion of completely weathered mudstone and sandstone is only 12.5 kPa, and the internal friction angle is only 4°. It shows that in the saturated state, its strength is very low and easy to destroy. Due to the expansibility and extremely low strength of expansive soil when it meets water, the stress concentration of filling subgrade often occurs at the toe of slope, and the shear stress of the toe of slope increases, resulting in shear failure, and the sliding surface is formed. It can be found in the subsequent analysis.
2.4. Inheritance Relationship between Recent Landslide and Old Landslide. Wangyan highway landslide is located in an area prone to landslides. There have been many landslides that occurred in the past time; the occurrence of new landslide triggered the occurrence of old landslides recently. The sliding bodies of several old landslides in this area basically belong to silty mudstone or argillaceous siltstone strata of Upper Cretaceous Longjing Formation (K 2l) and Tertiary Hunchun Formation (E 2-3h). The strata contain more clay minerals which absorb water and expand, lose water, and shrink and have strong expansivity. There is a certain inheritance relationship that exists between the recent landslide and the previous landslide [26]. According to the field investigation, there are several longitudinal cracks in the original soil layer at the location of the Wangyan highway landslide, and the trend of the cracks is roughly consistent with the direction of the sliding surface of the landslide. There are many bulging cracks with tensile characteristics, and the roots beside the cracks have been pulled apart. According to the field investigation and analysis, because the Wangyan highway landslide and the old landslide were blocked from sliding forward, the front edge of the sliding body was
Table 1: Parameters of soil in the saturated state.

<table>
<thead>
<tr>
<th>Name</th>
<th>Natural water content $\omega$ (%)</th>
<th>Natural density $\rho$ (g/cm$^3$)</th>
<th>Dry density $\rho_d$ (g/cm$^3$)</th>
<th>Saturated volumetric water content $\theta_s$ (%)</th>
<th>Weight $G_s$ (kN/m$^3$)</th>
<th>Void ratio $e$</th>
<th>Liquid limit $\omega_l$ (%)</th>
<th>Plastic limit $\omega_p$ (%)</th>
<th>Friction (°)</th>
<th>Cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial filling loose and broken</td>
<td>22.3</td>
<td>1.86</td>
<td>1.52</td>
<td>47.5</td>
<td>2.70</td>
<td>0.7</td>
<td>42</td>
<td>25.8</td>
<td>4.5</td>
<td>12</td>
</tr>
<tr>
<td>Artificial filling dense</td>
<td>27.4</td>
<td>1.87</td>
<td>1.47</td>
<td>45.9</td>
<td>2.72</td>
<td>0.85</td>
<td>63.1</td>
<td>33.2</td>
<td>6.1</td>
<td>16.2</td>
</tr>
<tr>
<td>Silty clay</td>
<td>28.5</td>
<td>1.89</td>
<td>1.48</td>
<td>45.7</td>
<td>2.72</td>
<td>0.84</td>
<td>54</td>
<td>27</td>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>Completely weathered sandstone</td>
<td>26.3</td>
<td>1.95</td>
<td>1.54</td>
<td>42.5</td>
<td>2.68</td>
<td>0.74</td>
<td>35.7</td>
<td>23.1</td>
<td>4</td>
<td>12.5</td>
</tr>
<tr>
<td>Completely weathered mudstone</td>
<td>23.1</td>
<td>1.96</td>
<td>1.50</td>
<td>46.5</td>
<td>2.79</td>
<td>0.87</td>
<td>51.5</td>
<td>26.3</td>
<td>4</td>
<td>12.5</td>
</tr>
<tr>
<td>Strongly weathered clay rock</td>
<td>29.5</td>
<td>1.96</td>
<td>1.52</td>
<td>45.7</td>
<td>2.79</td>
<td>0.84</td>
<td>56</td>
<td>27.4</td>
<td>8</td>
<td>20</td>
</tr>
</tbody>
</table>
uplifted upward, and the transverse tensile stress increased, leading to the tensile failure of the soil and the formation of cracks, as shown in Figure 3. The existence of these cracks not only destroys the overall structure of the original slope but also strengthens the seepage effect of groundwater and surface water, reduces the strength of the soil, and weakens the antislide force of the soil. It can be seen that there is a certain inheritance relationship between the old landslide and Wangyan highway landslide in terms of the material composition and the sliding direction of landslide. In addition, it can also be proved by the angle of the appearance of cracks and arc-shaped steep ridges [27].

2.5. Progressive Failure. Based on the above analysis, the four factors lead to the reduction of soil strength and the failure of the slope. Under the joint action of these four factors, the strength of the slope soil decreases, the shear stress at the toe of the slope increases and becomes concentrated, resulting in shear failure and the formation of the sliding surface. Then, the stress migrates to the trailing edge of the landslide; the migration and change of the stress form a continuous stair-stepping landslide, which is called pull-type landslide. It is characterized by the occurrence of multiple sliding surfaces and extends backward, the emergence of multistage sliding surfaces, and these bottom sliding surfaces through each other, forming a large range of the entire sliding surface enveloping all levels of the sliding surface. These sliding surfaces are connected with each other to form an integral sliding surface that involves all sliding surfaces in a large range. The failure extending from the toe of the side slope to the top of the slope is called progressive failure.

3. Study on Physical and Mechanical Parameters of Wangyan Highway Landslide

3.1. Sampling of Soil. In this study, samples were taken twice. The first soil sample was taken on May 10, 2012. A total of 9 soil samples were taken, including 3 undisturbed samples, numbered Y1 #–Y3 #, and 6 disturbed samples, numbered R1 #–R6 #. The second soil sample was taken on August 25, 2012, when the landslide had developed to a certain extent compared with the previous one. A total of 4 soil samples were taken this time, including 2 undisturbed samples, numbered Y4 # and Y5 #, and 2 disturbed samples, numbered R7 #–R8 #. The sampling site is shown in Figure 4. Soil samples Y1 and Y2 were taken from the slip zone of Wangyan highway landslide, 70 cm below the ground, which is a silty clay layer. The soil is golden yellow, with large clay content and fine and uniform particles.

3.2. Laboratory Test of Soil Samples

3.2.1. Mineral Composition of Soil Samples. As shown in Figure 5(a), the lattice unit of montmorillonite minerals is composed of an intermediate layer of alumina octahedron which is sandwiched by two outer layers of silica tetrahedron. The interlayers are connected by weak Van der Waals’ forces. Water molecules and exchangeable cations enter the gap between cells easily, increasing the cell distance, which leads to the expansion of clay mineral particles (Figure 5(b)) [28]. Under the action of water, the intergranular connection is weaker, the strength of soil is significantly reduced, and the soil has a larger expansion. Therefore, the soil in the study area has a strong swelling-shrinkage behavior, and the existence of the expansion property and the swelling force has a very negative impact on the stability of the landslide.
4. Experimental Study on Microstructure and Creep Properties of Slip Zone

4.1. Test Materials and Methods. The Wangyan highway landslide has mainly been caused by the newly filled section. The soil samples were taken from the position with a depth of 1.5 m at the leading edge of the landslide tongue. According to the requirements of “Standards for Geotechnical Test Methods” (GB/T50123-1999), the basic physical indexes of the soil sample were obtained: specific gravity $G_s = 2.78$, water content $\omega = 36\%$, natural density $\rho = 1.80\text{g/cm}^3$, porosity ratio $e = 0.48$, liquid limit $\omega_l = 64\%$, and plastic limit $\omega_p = 35\%$. The swelling and shrinkage indexes of the soil samples were as follows: cation exchange capacity CEC = 31.08 cmol/kg, free expansion ratio $F_s = 58\%$, swelling force $P_s = 64.1\text{kPa}$, free expansion rate $\delta_e = 2.25\%$, and volume shrinkage rate $\delta_V = 48.1\%$. The slip zone soil contains a large amount of expansive clay minerals illite and montmorillonite mixed layer, the content of which is 66% by X-ray diffraction test. In addition, the particle distribution curve of soil samples measured by static water settlement test is shown in Figure 6.

4.2. Test Method of Microstructure. First, the undisturbed samples of slip zone soil were cut into small soil blocks of about $1.0\text{cm} \times 1.0\text{cm} \times 1.0\text{cm}$ with a narrow hacksaw. The samples were placed in liquid nitrogen (boiling point -196°C) for rapid freezing for about 15 min, so that the liquid oxygen, aluminum, iron, magnesium, silicon, oxygen-hydrogen, exchangeable cation, weak Van der Waals’ force, water molecules, and silica tetrahedron are shown in Figure 5: Structure and expansion mechanism of montmorillonite crystal layer.

![Figure 5: Structure and expansion mechanism of montmorillonite crystal layer.](image)

![Figure 6: The particle size distribution curve of sliding zone soil.](image)
in the soil turned into nonexpandable amorphous ice. Then, at -50°C, the noncrystalline ice in the soil was sublimated by the lyophilizer for more than 8 h, so as to achieve the purpose of drying the soil sample without destroying the original structure. The prepared samples were used for a SEM test and MIP test, and the SEM test observed the transverse and longitudinal sections of slip zone soil, keeping the size of observation surface at 1.0 cm × 1.0 cm [26]. In this study, the pore distribution characteristics of undisturbed samples and remolded samples were tested.

4.3. One-Dimensional Compression Creep Test. The undisturbed sample was cut directly with a ring knife to remove the undisturbed sample of slip zone soil, and the original structure of the soil sample needed to be destroyed to reshape the soil sample. First, the soil sample was dried and ground then passed through the 2 mm sieve and added to distilled water to form a soil sample with a moisture content of 36% (the same as the undisturbed soil samples). The remolded soil sample with the same density as the original soil sample was prepared by compaction. Then, the soil samples were sealed in a humidator for 24 h to make the water diffuse evenly. Then, the soil samples were cut with a ring knife to obtain the ring knife sample of remolded soil. The one-dimensional compressive creep test was then conducted.

The one-dimensional compressive creep test was carried out by using a lever-type high-pressure consolidation instrument. The soil sample area was 32.2 cm² with a height of 2 cm. Step-by-step loading was adopted until the creep test was completed (when the specimen deformation was less than 0.01 mm within 1 day, the next level of loading was applied). The loading plan of the test was as follows: 25 → 50 → 100 → 200 → 300 (kPa).

5. Analysis of One-Dimensional Compressive Creep Test Results

5.1. Microstructure of Slip Zone Soil. In this study, multiple SEM images were taken from the horizontal and vertical sections of the slip zone soil, and the representative one with a better visual field was selected for qualitative and quantitative analysis. As shown in Figure 7, it can be observed that the cross section of slip zone soil is arranged with an obvious direction in Figure 7(a), and the chain structure can be seen in the SEM figure, magnified by 200 times. In the SEM figure, magnified by 2000 times in Figure 7(b), the edges of the clay mineral particles are warped; a large number of clay mineral aggregates are observed; and the particles are arranged extremely densely, mainly in edge-to-edge contact and surface-to-surface contact. The analysis of the picture shows that the mineral particles of 5 μm have a low agglomeration degree and very small pores.

The fissure of expansive soil is one of the reasons for its poor engineering properties. The macroscopic fissure is well developed in the Wangyan highway landslide area, and the widest fissure is up to 20 cm observed on site. There is no filling material in the fissure, basically, and the fissure provides a channel for water enrichment and flow in the landslide body. From the SEM images magnified by 200 and 800 times in Figure 8, obvious fracture morphology can be observed. The width of the fracture is about 5 μm to 120 μm. Within the statistical field of vision, there are about 22 microfractures with a length greater than 30 μm and about 25 microfractures with a length less than 30 μm. The penetration of microcracks is well. According to Formula (1), the microcrack rate is 6.28 (1/mm).

\[
\delta = \frac{\sum_{i=1}^{n} l_i}{S}.
\]  

In the formula, \(\delta\) is the crack rate (1/mm), \(n\) is the number of cracks, \(l_i\) is the length of crack \(i\) (mm), and \(S\) is the calculated area of the SEM image (mm²).

The pore distribution of undisturbed and remolded slip zone soil samples was measured by the mercury intrusion porosimetry method (Figure 9). The content of small pores in the undisturbed soil samples was the majority, and the pores less than 0.1 μm in diameter accounted for about 64% of the total pores, while the content of pores less than 0.1 μm in the remolded soil samples was only 40%. That indicated that the pore distribution in the undisturbed slip zone soil was mainly small and micropores, while the pore distribution in the remolded soil sample was uniform; and the contents of large pores, medium pores, and small pores were almost the same. It can be observed that the pore connectivity of undisturbed soil sample was poor, while that of remolded soil sample was better. The fractal dimension of the soil skeletons was 2.688 and 2.570, respectively.
indicating that the microstructure complexity of the intact soil sample was higher than that of the remolded soil. The porosity of the undisturbed samples was 21.1%, and that of the remolded soil samples was 24.8%. That is not much difference overall.

5.2. Results of Compressive Creep Test. Figure 10 shows the relationship curves of the strain-time of undisturbed and remolded samples in the slip zone soil. Continuous loading was adopted in the test. It can be seen from the figure that a certain amount of instantaneous deformation would be generated at the moment of loading, especially at the stage of low load. With the passage of time, the compression process was completed gradually, and the instantaneous deformation decreased when the loading of 200 kPa and 300 kPa was applied. At the stage of low load, the creep deformation of undisturbed and remolded samples increases obviously with time, indicating that the soil in the slip zone of the landslide has crept as well. When the vertical load was 50 kPa and 200 kPa, the undisturbed soil samples expanded to different degrees, indicating that the structure of the undisturbed soil samples was strong. Because the slip zone soil was squeezed by the upper soil in the history, the horizontal permeability of soil was good, but the vertical permeability was poor, and the soil anisotropy was obvious. In the process of compression and deformation, the pore water was not discharged smoothly, which led to partial water saturation in the soil, and the illite-montmorillonite mixed layer in the soil samples expanded after absorbing water. However, the remolded samples had good uniformity, the pore distribution was uniform, and no expansion phenomenon occurred in the whole deformation process. All the other properties of the remolded samples were the same as in the undisturbed samples; except for the obvious differences in microstructure, there was little difference in total strain between them.

Next, the deformation characteristics of slip zone soil under each stage of load were analyzed. At the low load, both samples showed decaying creep, and the strain rate approached 0 at 25 kPa, finally. When the load is 50 kPa and 100 kPa, the deformation curve was a nonattenuation creep, and the creep rate did not tend to a constant value but gradually increased with the development of time after a certain stage; the deformation increased greatly until failure. When the later loading was 200 kPa and 300 kPa, the consolidation and compression process of soil samples was basically completed, the strain of soil samples tends to be stable, and the strain does not increase anymore.
According to the compressive creep curves of expansive soil under different vertical stresses, corresponding stress-strain isochronous curves could be obtained. Figure 11 shows the isochronous curves of stress and strain of undisturbed and remolded samples at $t = 60$ min and $t = 1000$ min. The stress-strain isochronous curves at different times were not straight lines, indicating that the rheology showed nonlinear characteristics with obvious inflection points. The greater the increase of the stress level, the more deviated the curve from the straight line, and the nonlinear degree increased with the increase of stress [29].

During the process of compression deformation of unsaturated expansive soil, the friction between solid particles and pore water hinders gas discharge, thus delaying deformation. Conversely, the contact between soil particles depends on the cohesive water film, and the cohesive water is viscous, making the soil deformation dependent on time. In addition, with the development of consolidation deformation of soil, the pore water fills the pores, and the gas discharge decreases gradually. The microstructure and mineral composition of expansive soil indicate that it has low permeability in the vertical direction of soil. The experimental results show that the deformation characteristics of expansive soil are related to the microstructure and the stress-strain relationship, which present significant nonlinear rheological characteristics. When the
axial strain reached 15%, the sample was considered to be damaged, and the remodeled samples were damaged earlier than the undisturbed samples. This indicates that the clay minerals in the remolded soil samples were irregularly arranged. Under the action of load, the mineral particles constantly adjusted their positions, and with the crushing and reggregation of minerals, the large pores in the soil were dominant, providing space for the displacement of mineral particles. The pores of undisturbed soil samples mainly included intergranular pores and isolated pores, among which the isolated pores were large and discontinuous, which is the general reason for the poor vertical permeability of clay. In the undisturbed test of slip zone soil, the plane of the flake minerals was adjusted to the vertical principal stress direction, and the surface-to-surface contact was dominant. The clay minerals are often arranged closely, so the deformation speed of the undisturbed soil samples should be less than that of the remolded soil samples in the compression process. However, they are only different in microstructure, so there is little difference in overall creep deformation between them.


6.1. Soil-Water Characteristic Curve of Rock Mass in Wangyan Highway Landslide. In reference [30], Origin software was used to fit soil-water characteristic curve data of 9 known granular soils and 12 known viscoplastic soils with Fredlund-Xing equation. After fitting, equation fitting parameters $a$, $n$, and $m$ of different soil samples were obtained. In this paper, the results of literature [30] are used to predict soil-water characteristic curve of rock mass in Wangyan highway landslide.

The granular test shows that the rock and soil mass of Wangyan highway landslide belongs to viscoplastic soil. Based on predictions of relevant literatures, the regression equations of $a$, $n$, and $m$ with the maximum correlation coefficient of viscoplastic soil are as follows:

$$ a = 0.5173 \times 10^4 \times e^{(0.7292 \times 10^{-3} - 0.5829 \times 10^{-4} \times \ln (h_{0,r})}) $$
$$ n = 0.1442 \times 10^{-1} \times e^{(-0.8394 - 0.5279 \times 10^3 / h_{0,r})} + 0.3160 \times 10^{-1} \times \ln (h_{0,r}) $$
$$ m = -7.423 + 0.504 \times c + 2.634 \times G_i + 0.027 \times \omega_i - 1.2 \times 10^{-6} \times h_{0,r} $$

where $h_{0,r}$ is equivalent capillary height (cm), $a$ controls the air intake value of soil-water characteristic curve, and the larger the $a$ value is, the greater the air intake value is; $n$ controls the slope of the curve, and the greater the $n$ value, the greater the angle near the intake value; $m$ is related to the residual suction value of the curve, and a smaller $m$ makes the slope of the curve steeper in the high-suction region.

<table>
<thead>
<tr>
<th>Name</th>
<th>$a$</th>
<th>$n$</th>
<th>$m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial filling (loose and broken)</td>
<td>1134.739</td>
<td>0.454</td>
<td>1.019</td>
</tr>
<tr>
<td>Artificial filling (dense)</td>
<td>3797.734</td>
<td>0.411</td>
<td>1.639</td>
</tr>
<tr>
<td>Silty clay</td>
<td>2987.47</td>
<td>0.408</td>
<td>1.433</td>
</tr>
<tr>
<td>Completely weathered sandstone</td>
<td>846.074</td>
<td>0.459</td>
<td>0.856</td>
</tr>
<tr>
<td>Completely weathered mudstone</td>
<td>3130.751</td>
<td>0.403</td>
<td>1.579</td>
</tr>
<tr>
<td>Strongly weathered clay rock</td>
<td>3234.403</td>
<td>0.409</td>
<td>1.656</td>
</tr>
</tbody>
</table>

The rock-soil parameters $a$, $n$, and $m$ values described by Fredlund-Xing soil-water characteristic curve are shown in Table 2.

The five rock-soil parameters in Tables 1 and 2 are input into the Fredlund-Xing soil-water characteristic curve model in SEEP/W, and the curve results are shown in Figure 12.

As can be seen from Table 2 and Figure 12, the $a$ values of artificial filling (loose and broken) and completely weathered sandstone are relatively small, indicating that under the action of small matric suction, air enters into the pores of these two types of soil and drains pore water. When the $n$ value of the two is relatively large, the angle around the intake value is large, and the curve is more stable in the middle suction range than the other three, and the water holding capacity is relatively good. At the same time, the $m$ value of the two is small, indicating that the curve is steeper in the high suction interval and the residual suction value of the two is small. The artificial filling (dense) has the highest $a$ value; that is, the air needs greater matric suction to enter the soil pores. Although the filling material is the fissure interbedded mudstone and sandstone of the Tertiary Hunchun Formation (E 2-3h), the $a$ value of the artificial filling is the largest after compaction. The $a$ value of the artificial filling (loose and broken) layer is much smaller than that of the artificial filling (dense), which is due to the large number of cracks in the expandable soil of the originally compacted subgrade surface under the action of rainfall or evaporation cycle, and the existence of cracks provides channels for rainwater infiltration. Therefore, it is necessary to consider this layer in the analysis of seepage field caused by rainfall infiltration.

The saturated permeability coefficient was measured by an OYI-2 osmometer. The saturated permeability coefficient of five types of rock and soil mass measured in the test is shown in Table 3.

The results show that the saturated permeability coefficient of the soft rock is small, which is more than $10^{-6}$. This is because the clay content of expansive soft rock is high, and the aggregates of hydrophilic minerals and their mixed layers are easy to disperse under the action of water. On the one hand, small microaggregates may block the seepage channel; on the other hand, the bound water film formed by aggregates or particles will also hinder the infiltration of water. In addition, the permeability coefficient of the surface artificial filling (loose and broken) is slightly higher, which is due to the loose particles produced in the soil under the action of wetting and drying cycle. A large amount of water
mainly penetrates along the microcracks, and only a small amount penetrates through the tiny pores. The permeability coefficient of completely weathered sandstone is slightly higher, which is mainly caused by a certain amount of coarse particles in the soil of this layer. The permeability coefficient of highly weathered clay rock is the lowest, which is caused not only by the higher clay content but also by the greater depth and pressure of overlying rock mass.

6.2. Prediction of Unsaturated Permeability Coefficient. In 1980, Van Genuchten [31] proposed the following relation to describe the permeability coefficient of soil in the range of matric suction:

\[ k_w = k_s \left[ 1 - \left( a \psi^{n-1} \right) \left( 1 + a \psi^n \right)^{-m} \right]^2, \]  

(3)

where \( k_s \) is the saturation permeability coefficient; \( a, n, \) and \( m \) are curve fitting parameters, where \( n = 1/(1-m) \); and \( \psi \) is matric suction.

When the saturated permeability coefficient and parameters \( a \) and \( m \) are known, the permeability coefficient function of soil can be obtained by the above equation.

Van Genuchten believed that the value of fitted parameters could be predicted by soil-water characteristic curve. He considered that the optimum point for predicting fitting parameters was the median point between saturation and residual water content on soil-water characteristic curve and proposed the slope formula of soil-water characteristic function.

\[ S_p = \frac{1}{(\theta_s - \theta_r) \left( \log \psi_p \right)}. \]  

(4)
remains stable with the increase of matric suction. The soil permeability coefficient decreases sharply to the saturated permeability coefficient when the suction reaches the air intake value, the air begins to enter the soil, and the soil permeability coefficient changes very little, which is close to the saturated permeability coefficient. When the suction reaches the residual strength, the material parameters of soil in natural state are calculated according to the saturated residual strength. The material parameters of rock-soil mass in the model. The material parameters of rock-soil mass in the model. The material parameters of rock-soil mass in the model. The material parameters of rock-soil mass in the model.

When $S_p$ calculation is known, Van Genuchten proposed the following formula to predict fitting parameter values $a$ and $m$:

$$m = 1 - e^{-0.8S_p},$$  \hspace{1cm} (5)$$

When $0 < S_p < 1$,

$$m = 1 - \frac{0.5755}{S_p} + \frac{0.1}{S_p^2} + \frac{0.025}{S_p^3}.$$  \hspace{1cm} (6)$$

When $S_p > 1$,

$$a = \frac{1}{\psi} \left(2^{1/m} - 1\right)^{(1-m)}.$$  \hspace{1cm} (7)$$

The parameters of Fredlund-Xing soil-water characteristic curve of landslide rock mass in Table 3 and saturated permeability coefficient in Table 4 were input into SEEP/W software. By using the permeability coefficient function method proposed by Van Genuchten above, the relationship curves between unsaturated permeability coefficient and matric suction of different rock masses obtained are shown in Figure 13. The results show that before air intake, the soil is in the saturated state or close to the saturated state, and the soil permeability coefficient changes very little, which is close to the saturated permeability coefficient. When the suction reaches the air intake value, the air begins to enter the soil, and the soil permeability coefficient decreases sharply with the increase of matric suction until the soil reaches the residual water content. The soil permeability coefficient remains stable with the increase of matric suction.

### 7. Slope Stability Analysis by Numerical Simulation

#### 7.1. The Establishment of Slope Model

Then, we analyze the stability of Wangyan highway landslide by numerical simulation. Based on Pettersen’s hypothesis, the shear failure surface is assumed to be an arc surface and the force between the bars is ignored.

In Figure 14, the strata from top to bottom are artificial filling, silty clay, completely weathered mudstone and sandstone, and strongly weathered clay rock, which are numbered from 1 to 4. The displacement monitoring was carried out for the points (1), (2), and (3) of the sliding body during the numerical simulation. The D-D’ section in the profile shown in Figure 1 is used for the model, and the length of the model is 100 m. In Midas, the establishment of geometric model and the mesh generation were carried out, with a total of 27680 units. In Flac3D, constraints are applied to $y$ and $z$ boundaries [29]. Because the geotectonic activity in the landslide area is not strong, the landslide body is mainly composed of the semi-consolidated completely weathered mudstone and sandstone of Tertiary Hunchun Formation, which is not enough for the storage of in situ stress.

Therefore, when considering the initial stress field of the model, only the influence of gravity field is considered and the influence of tectonic stress on slope is ignored. The Mohr-Coulomb model is adopted as the constitutive model of rock-soil mass in the model. The material parameters of soil mass in natural state are shown in Table 4.

#### 7.2. Analysis of Calculation Results

##### 7.2.1. Analysis of Natural State Calculation Results

After calculation, the picture of overall displacement, picture of shear strain increment, plastic zone, and displacement diagram of each monitoring point of the slope in natural state are shown in Figure 15 as follows.

The factor of safety calculated by strength reduction method in Flac3D is 1.81. According to the picture of shear strain increment, it can be seen that the units of upper artificial filling are in shear yield state, and the strip zone formed by these units runs through the entire sliding surface, indicating that the artificial filling of the slope has been damaged. However, the larger factor of safety indicates that the overall slope is stable. According to the diagram of plastic zone, it can be seen that the upper part of the filling is in yield state. At the same time, it can be seen that there is also a soil belt below the completely weathered mudstone and sandstone in yield state; it is in danger of slipping. According to the displacement curves of monitoring points 1, 2, and 3, it can be determined that in the natural state, the displacement of the slope decreases gradually from top to bottom, indicating that the landslide displacement gradually decreases from top to bottom, and the unstable part is mainly artificial filling.

##### 7.2.2. Analysis of Calculation Results of Saturation State

Since the slope is composed of expansive soil, water plays a very important role in the stability of slope. When the soil is in a saturated state, the parameters of soil are calculated according to the saturated residual strength. The material parameters of soil mass in natural state are shown in Table 4.

<table>
<thead>
<tr>
<th>Type of rock-soil</th>
<th>Density (kg/m$^3$)</th>
<th>Friction (°)</th>
<th>Cohesion (kPa)</th>
<th>Bulk modulus (MPa)</th>
<th>Shear modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial filling</td>
<td>1850</td>
<td>10</td>
<td>18</td>
<td>39.58</td>
<td>14.18</td>
</tr>
<tr>
<td>Silty clay</td>
<td>1840</td>
<td>7</td>
<td>22</td>
<td>39.22</td>
<td>15.04</td>
</tr>
<tr>
<td>Completely weathered mudstone, sandstone</td>
<td>1950</td>
<td>17</td>
<td>25</td>
<td>46.3</td>
<td>18.94</td>
</tr>
<tr>
<td>Strongly weathered mudstone, sandstone</td>
<td>2000</td>
<td>25</td>
<td>50</td>
<td>1667</td>
<td>769.23</td>
</tr>
</tbody>
</table>

Table 4: Parameters of soil in natural state.
parameters of the soil are shown in Table 1, and the slope model is still calculated according to Figure 14. The calculated overall displacement diagram, shear strain increment diagram, plastic zone diagram, and displacement diagram of the monitoring point are shown as Figure 16.

The strength reduction method in Flac3D is used to calculate the factor of safety of the slope in saturated state is 0.725. It shows that the slope has been seriously damaged in the saturated state, and the slope needs to be strengthened and protected. According to the picture of shear strain increment, under the saturated state, the slip zone not only penetrated the artificial filling but also extended down to the completely weathered mudstone and sandstone layer, and the slope has slipped, which is extremely dangerous.

![Figure 13](image1.png)

**Figure 13:** Relationship between unsaturated permeability coefficient and matric suction.

![Figure 14](image2.png)

**Figure 14:** The model of Wangyan highway landslide and grid section diagram.
**Zone displacement magnitude**

<table>
<thead>
<tr>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>1.3222E+00</td>
</tr>
<tr>
<td>1.3000E+00</td>
</tr>
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<td>1.2000E+00</td>
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<tr>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.0000E+00</td>
</tr>
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</table>

**Factor of safety**

Value = 1.81

---

**Zone maximum principal strain increment**

Calculated by: constant

<table>
<thead>
<tr>
<th>Value</th>
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<tbody>
<tr>
<td>5.9515E−01</td>
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<td>5.5000E−01</td>
</tr>
<tr>
<td>5.0000E−01</td>
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</tr>
<tr>
<td>5.0000E−02</td>
</tr>
<tr>
<td>4.8775E−21</td>
</tr>
</tbody>
</table>

**Factor of safety**

Value = 1.81

---

(a) Overall displacement diagram of natural state

(b) Shear strain increment diagram in natural state

**Figure 15: Continued.**
According to the diagram of plastic zone, a large area of the slope is in the state of shear yield, which indicates that the stability of the slope is very poor and needs urgent treatment. According to the displacement curves of monitoring points 1, 2, and 3, it can be determined that the displacement of points 1, 2, and 3 gradually increases with the increase of the number of calculation steps in the saturated state. The landslide is an integral slip, and the landslide body not only includes the artificial filling but also extends downward, making the completely weathered mudstone and sandstone become a part of the landslide body.

The study shows that the slope is stable in natural condition, and the dangerous sliding surface is generated in the upper part of the landslide, that is, the artificial filling. However, in the saturated state, the strength of the slope decreases sharply, and the sliding surface penetrates into the lower rock-soil mass. The slope has been damaged and needs to be treated.

8. Analysis of Sensitive Factors

8.1. Influence of Water Content on Slope Stability. According to relevant references and field investigations, both cohesion
Zone displacement magnitude

<table>
<thead>
<tr>
<th>Value</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.725</td>
<td>Value = 0.725</td>
</tr>
</tbody>
</table>

(a) Overall displacement diagram of saturated state

Zone maximum shear strain increment
Calculated by: constant

<table>
<thead>
<tr>
<th>Value</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.725</td>
<td>Value = 0.725</td>
</tr>
</tbody>
</table>

(b) Shear strain increment diagram in saturated state

Figure 16: Continued.
and internal friction angle decrease with the increase of water content [32]. Regression analysis shows that the relationship between the initial water content of undamaged soil and cohesion and internal friction angle is linear, and the regression curve can be expressed by the following formula:

\[
\begin{align*}
    c &= A\omega + B, \\
    \phi &= C\omega + D,
\end{align*}
\]

where \(c\) is cohesion; \(\phi\) is the angle of internal friction; \(\omega\) is water content; and \(A\), \(B\), \(C\), and \(D\) are parameters in the test.

The relationship between the angle of internal friction and cohesive force of each layer of soil and water content is shown in Table 5:

Since the slope is composed of expansive rock-soil mass, considering the influence of rainfall infiltration and ground-water, the strength of slope soil mass will change under different water contents. This section mainly studies the influence of different water contents on soil cohesion and internal friction angle and then analyzes the stability of slope. In the following, taking the water content is 30% as an example to show the influence of water content on slope stability in Figure 17.

Figure 16: The stability of slope in saturated state.
With the increasing of water content, the factor of safety of slope decreases gradually. According to Figure 15, it can be seen that when the water content is less than 40%, the sliding soil is mainly the artificial filling. When the water content increases to 40%, the sliding part not only includes the artificial filling but also extends down to the completely weathered mudstone and sandstone, which further deepens the range of slope failure. When the water content increases to 42%, the factor of safety is 0.977, and the slope is destroyed totally. The reason is that the soil mass of the slope is composed of expansive rock-soil mass, which has the characteristic of swelling after absorbing water. The increase of water content will lead to the swelling of the soil mass, resulting in the swelling force, the reduction of soil strength, and the destruction of the slope. With the increase of water content gradually, the decrease rate of factor of safety increases by degrees.

8.2. Influence of Highway Load. Because the landslide is the subgrade of the expressway, the highway load is bound to have a great impact on the factor of safety of the slope. In the study, the calculation is carried out according to the method of converting the vehicle load into equivalent load. The formula is

$$q = \frac{NQ}{LB},$$

$$B = Nb + (N - 1) \cdot d.$$  (9)

$N$ is the number of laterally distributed vehicles, $Q$ is the total weight of each vehicle (kN), $L$ is the distance between the front and rear axles add the length of the tire contacting with the ground (m), $B$ is the distance between the tire outer edges of laterally distributed vehicles (m), and $d$ is the net distance between two adjacent vehicles (m).

Related parameters of vehicles are valued according to specifications [33], and the distribution of the load is arranged according to the most unfavorable load. Wangyan highway has four lanes in both directions. The number of vehicles in transverse distribution $N$ is 4, and $B$ is 9 meters according to the standard. The total weight of each vehicle and the front and rear wheelbase add the landing length of tires which are evaluated according to the highway-class I standard. The total weight of each vehicle is 550 kN, and the total length of front and rear wheelbase adds the landing length of tires which is 12.8 meters. The data can be obtained by substituting into the formula $q = 19.1$ kPa.

The influence of the highway load on the stability of the embankment slope was studied with the gradual increase of the highway load [34]. In the following, taking the highway load is 5 kN as an example to show the influence of highway load on slope stability. The calculated overall displacement and shear strain increment are shown in Figure 18 as follows.

According to the figure above, the factor of safety of slope stability decreases gradually with the increase of highway load and shows a linear change rule. At the same time, it can be seen that the gradient is small. The analysis shows that the increase of highway load is equivalent to the increase of the sliding force, and the resistance is insufficient to resist the sliding force, leading to the gradual decrease of the stability of the slope. Therefore, the factor of safety of slope decreases gradually with the increase of highway load, but the sensitivity is lower than water content.

### Table 5: The regression equation of the relationship between water content and cohesion and internal friction angle.

<table>
<thead>
<tr>
<th>Type of rock-soil</th>
<th>Regression equation</th>
<th>Regression equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial filling</td>
<td>$c = -23.81\omega + 23.31$</td>
<td>$\varphi = -21.83\omega + 14.91$</td>
</tr>
<tr>
<td>Silty clay</td>
<td>$c = -43.35\omega + 34.81$</td>
<td>$\varphi = -12.5\omega + 10.87$</td>
</tr>
<tr>
<td>Completely weathered mudstone, sandstone</td>
<td>$c = -78.62\omega + 47.49$</td>
<td>$\varphi = -81.76\omega + 40.38$</td>
</tr>
<tr>
<td>Strongly weathered mudstone, sandstone</td>
<td>$c = -185.19\omega + 104.63$</td>
<td>$\varphi = -104.94\omega + 55.96$</td>
</tr>
</tbody>
</table>

8.3. The Influence of Filling Weight. In the construction process of highway engineering, in order to ensure the requirements of bearing capacity, the subgrade is often compacted to ensure that the bearing capacity of the highway subgrade meets the requirements. The influence of different filling weight on slope stability was analyzed by changing the filling weight. In the following, taking the filling weight is $18.5 \text{kN/m}^3$ as an example to show the influence of filling weight on slope stability. The calculated displacement picture, shear strain increment picture, and the relationship between the factor of safety and the filling weight are shown in Figure 19 as follows.

According to the above calculation results, with the increase of the filling weight, the maximum displacement and maximum shear strain of the slope decrease gradually, but the factor of safety of the slope also decreases gradually. After analyzing the reason, with the increase of filling weight, the sliding force of the slope gradually increases, greater than the resistance, resulting in the decrease of the slope safety factor. At the same time, it can be seen that the factor of safety changes linearly with the increase of filling weight.

8.4. The Influence of Slope Angle. Because the artificial filling changes the original terrain, the slope becomes steeper and provides space for lateral deformation for the formation of landslide. When the slope angle changes, it will also have an impact on the slope stability. Next, the influence of slope angle on slope stability is analyzed by changing different slope angles. The slopes with slope angles of $20^\circ$, $25^\circ$, $30^\circ$, $35^\circ$, and $40^\circ$ were selected as the research objects. The model was established again, the strength reduction method was used to calculate the factor of safety. In the following, taking the slope angle is $20^\circ$ as an example to show the influence of slope angle on slope stability in Figure 20.
Figure 17: Effect of water content on slope stability.
It can be seen that as the slope angle increases gradually, the overall displacement of the slope increases gradually, the factor of safety of the slope decreases by degrees, and the reduction rate decreases with the increase of the slope angle. When the slope angle increases to 40°, the maximum displacement of the slope reaches to 4 m. At
Figure 19: Effect of filling weight on slope stability.
Figure 20: Effect of slope angle on slope stability.
9. Conclusion

(1) The destructive process of Wangyan highway landslide can be summarized as follows: since the slope is composed of expansive soil, the soil structure is broken, the cracks are developed, and the strength of the soil decreases under the action of water. Artificial filling changes the original terrain, resulting in steepening of the slope and providing deformation space for the landslide. In addition, under the traction of the old landslide, shear stress concentration occurs at the toe of the embankment of the new filling, resulting in shear failure. The shear stress continues to extend to the interior of the landslide and leads to the progressive failure eventually [36].

(2) Through the scanning electron microscopy and compressive creep test, the main component of expansive soil is montmorillonite, with a maximum content of 66% and a maximum swelling force of 64 kPa. On the one hand, the existence of cracks in expansive soil directly reduces the shear strength of soil. On the other hand, it increases the seepage channels of soil, accelerates and intensifies the seepage in expansive soil, reduces the suction of soil, accelerates the wetting deformation of soil structure, reduces the strength of soil greatly, and reduces the stability of slope eventually [37].

(3) Expansion occurred when the load was 50 kPa and 200 kPa in the compressive creep process of undisturbed soil sample, while no expansion occurred during the deformation process of the remolded soil sample. The soil in the sliding zone of the Wangyan highway landslide had strong rheological properties. The creep curves of the undisturbed soil sample and the remolded soil sample were similar, showing an attenuation creep trend on the whole. The microstructure affected the deformation characteristics of the expansive soil, and the strain rate of the undisturbed soil sample was less than that of the remolded soil sample.

(4) The saturated permeability test results show that the saturated permeability coefficients of filled soil (loose and broken) and fully weathered sandstone are $10^{-6}$ orders of magnitude; those of filled soil (dense), silty clay, and completely weathered mudstone are $10^{-8}$ orders of magnitude; and those of strongly weathered clay rock are $10^{-9}$ orders of magnitude.

(5) The results show that the factor of safety of the slope decreases with the increase of water content, highway load, filling weight, and slope angle, but the amplitude of variation is different: water content is the most sensitive, the next is slope angle and highway load, and the lowest is filling weight. The results can provide a scientific basis for the later treatment and protection of the landslide.

Data Availability

The test data used to support the findings of this study are included within the article. Readers can obtain data supporting the research results from the test data table in the paper.

Conflicts of Interest

The authors declare no conflict of interest.

Authors’ Contributions

Pengfei Jiang completed all the experiments in the paper with Yuanyuan Kong, made a plan of research, and put forward some crucial suggestions. He was the main contributor of the project. Yuanyuan Kong completed all the experiments in the paper with Pengfei Jiang and put forward some crucial suggestions. Zhenyu Song finished the numerical simulation, collect and summarize all experimental data, and drew pictures. Ruixin Zhao assisted to complete numerical simulation and check the article. Jiewei Zhan checked out some language errors in the paper and polished them up. Xuhui Lin assisted me with the numerical simulation.

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