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An experimental study of creep characteristics of sliding zone soil of Huangtupo landslide under different consolidation stresses

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Abstract: In order to study the creep characteristics of the sliding zone soil of the Huangtupo landslide under different consolidation stresses, the unidirectional load, load–unload, load–unload–reload tests were used to consolidate the sliding zone soil, and then the shear creep tests of the sliding zone soils in different consolidation states were carried out. The experimental results showed that the initial void ratio of the sliding zone soil was 0.49 and the compressibility coefficient $a_{1\text{-}2}$ was between 0.37 and 0.45 MPa^{-1}, belonging to the moderate compressibility soil. After unidirectional loading to the predetermined pressure, the void ratio of sliding zone soil reached to the maximum and the compression amount the minimum. After loading-unloading to the predetermined pressure, the void ratio of sliding zone soil was the minimum and the compression amount the maximum. The void ratio and compression amount after loading-unloading-reloading to the predetermined pressure was somewhere in the middle. For the same initial state sliding zone soil, after different loading-unloading consolidation state, under the same normal stress and shear stress level, the creep shear strain of unidirectional loading was the largest, but the creep shear strain of loading-unloading was the minimum. The creep shear characteristics of sliding zone soil are closely related with the loading state and the void ratio after loading. The Burgers model was used to fit the creep test data and the creep parameters of Maxwell model and Kelvin model were obtained. The fitting and test curves were in good agreement, which indicates that Burgers model can reflect the creep characteristics of sliding zone soil under different consolidation stresses.

Keywords: sliding zone soil; consolidation stress; void ratio; creep characteristics; Burgers model

1 Introduction

The creep behavior is one of the important mechanical properties of sliding zone soil of landslides. In the literature, there are a large number of theoretical and experimental studies focusing on the creep characteristics and creep models of sliding zone soil, and those studies have made remarkable achievements in creep theory and engineering practice[1–7]. Among hundreds of creep models as reported, the Mesri model[8–9], Burgers model[6, 10], SinghMitchell[11] model, etc. are more suitable for describing the creep characteristics of sliding zone soil pertinent to landslides. Those various models have their own characteristics and are suitable for different working conditions and stress states. In particular, the Burgers model is the most suitable for describing the shear creep curve before the third stage of the soil, and it has been widely used [12–13].

The deformation evolution and stability of landslides are affected by the combined effects of internal and external factors. The internal factors include the mineral composition, particle gradation, microstructure and other factors of the sliding zone soil, and the external factors include ground loads such as rainfall, the rise and fall of the reservoir water level, and human relocation. The latter is the main factor inducing landslide. At present, the study of creep characteristics of sliding zone soil usually adopts a single loading path for creep test. For example, Yan et al.[14] studied the creep properties of sliding zone soil and found that the normal stress is related to the shear modulus and long-term deformation. Jiang et al.[15] studied the creep properties of the sliding zone soil of the slowly resurrected landslide, and concluded that the critical stress and shear rate of the slip zone soil entering the accelerated creep are linearly positively correlated with the normal stress. Sun et al.[10] studied the creep characteristics of the sliding zone soil of Majiagou landslide, and deemed that the absolute creep of the sliding zone soil and the deformation rate in the stable creep stage are positively correlated with the axial stress value. The current research results seldom involve the creep characteristics of the sliding

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zone soil under different consolidation states. However, the landslides represented by the Huangtupo landslide and Outang landslide in the Three Gorges Reservoir area involve human activities such as continuous migration and immigration that changes the ground loading conditions. This phenomenon makes it meaningful to study the creep characteristics of sliding zone soil under the states of load–unload or load–unload–reload.

Based on previous studies, this article combines the actual situation of the Huangtupo landslide and considers the load, load–unload and load–unload–reload conditions of the sliding zone soil to carry out the shear creep tests of the soil. Based on test, the creep curve and isochronous stress-strain curve of the sliding zone soil are obtained, and the characteristics of creep stage and creep rate under different consolidation states are analyzed. The Burgers model was used to fit the creep test data and the creep parameters of Maxwell model and Kelvin model were obtained. This work is of great significance for both theoretical research on the rheological mechanics of sliding zone soils and practical engineering applications.

2 Landslide overview

Huangtupo is located in Xinling town, Badong county, Hubei province. Due to the construction of the Three Gorges Project, the Hubei provincial government approved Huangtupo as the county's new urban area in 1982. After the expert investigation and demonstration, Huangtupo was identified as a large-scale ancient landslide, which is of complex causes and has great safety risk. In April 2008, the government clarified the plan of the overall risk avoidance relocation of human and properties in Huangtupo area. The relocated population was 15,700. The overall view of the Huangtupo landslide after the relocation is shown in Fig.1. All the houses above were demolished, leaving only interception and drainage facilities and highways. Huangtupo landslide is composed of Linjiang landslide accumulation body #1, Linjiang landslide accumulation body #2, substation landslide and garden landslide. The total volume is about 6.934×10^7 m^3. It is an ultra-deep and super large landslide. At the end of 2012, China University of Geosciences (Wuhan) built a large-scale comprehensive field test site in Badong county on Linjiang #1 collapsed landslide body, including a 908 m long main test tunnel and five test branch tunnels of different lengths. The test adit of branch tunnel #3 revealed a layer of slip zone. Studying the creep characteristics of this layer of the sliding zone before and after the risk-avoidance relocation is of great significance to the evolution and stability analysis of landslide deformation.

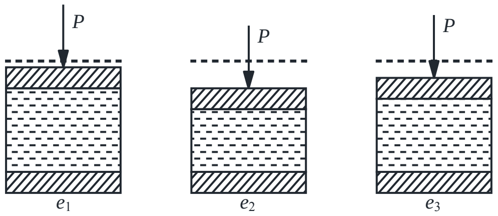
Fig.1 The whole picture of Huangtupo landslide after relocation

3 Test plan and the devices

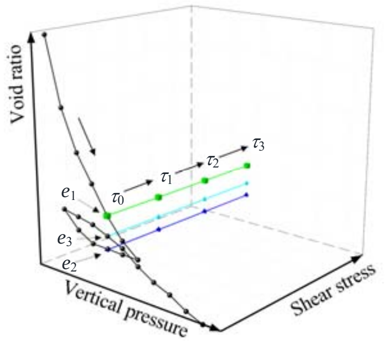
The soil used in this test was taken from the sliding zone exposed in test adit of branch tunnel #3 in Linjiang landslide body #1 of Huangtupo landslide in Badong county, Three Gorges Reservoir area, with a buried depth of 20–50 m, as shown in Fig. 3(a), and its basic physical and mechanical properties are listed in Table 1. Results obtained from X-ray diffraction test indicate that the main mineral components of sliding zone soil include calcite (30%), quartz (20%), illite (29%), montmorillonite (11%), chlorite (5%) and feldspar (5%). The sliding zone exposed by the boreholes on the sliding body and by the test tunnel in the landslide shows that the soil in the sliding zone is saturated, and the upper and lower surfaces of the sliding zone are drained. According to the buried depth, density and moisture content of the in-situ sliding zone soil, remodeled sliding zone soil samples were prepared indoors. Put the prepared sliding zone soil sample in a vacuum saturated tank for 5 hours, and then inject deionized water to saturate it for 48 hours, and lastly carry out the conventional shear tests and shear creep tests under the consolidation states of load, load–unload and load–unload–reload, respectively. The load process is shown in Fig. 2. Firstly, consolidate the sample to achieve the void ratios e_1, e_2 and e_3 respectively,，and then start the slow shear test（shear rate of 0.02 mm/min）or apply different levels of shear force（τ_1, τ_2, τ_3……）for creep tests.

Table 1 Physical and mechanical properties of the sliding zone soil

Natural moisture content/ %	Natural density / (g·cm^{-3})	Relative density of soil	void ratio	Slow shear test	
				Cohesion /kPa	friction angle / (°)
13.87	2.05	2.68	0.49	27.50	17.80



(a) Void ratio after load–unload–reload



(b) Creep test after load (e_1)—unload(e_2)—reload(e_3)

Fig.2 The load–unload–reload state and creep curve

In view of the complicated evolution process of the Huangtupo landslide on the Three Gorges Reservoir area and the actual situation of the immigrants' hazard avoidance and relocation, this paper studies the shear creep characteristics of the soil in the sliding zone in different pre-consolidated stresses. In general, the consolidation state of the sliding zone soil was discussed from four aspects: (a) According to the buried depth of the sliding zone soil, the vertical pressure on the sliding zone soil is set to 600 kPa, i.e. σ_3 = 600 kPa, which is also the consolidation method adopted by most researchers[6]. (b) In 1982, Hubei Provincial government approved Huangtupo as a new urban area of Badong county, so large-scale landfill, leveling, building and other engineering activities began to be implemented. Considering that the height of the local filling is 8–10 m and the building load, the vertical stress is taken as σ_3 = 800 kPa. (c) In April 2008, 26 years after the completion of the new urban construction of Badong county, the government clarified the plan of the overall risk-avoidance relocation of Huangtupo, with a relocated population of 15,700. Additionally, some factors such as the construction of the Huangtupo test tunnel and the fluctuation of reservoir water level should also be considered, thus, the minimum load is taken as σ_3 = 400 kPa. (d) With the demolition of the old city of Badong and the construction of the new city of Badong, a large amount of construction waste was piled up on the slope of Huangtupo. Therefore, according to the buried depth of the slip zone soil and the actual situation of the hazard relocation, the vertical pressure on the sliding zone soil is set as σ_3 = 600 kPa. This stress can also be compared with the stress of the Huangtupo sliding zone soil before load–unload. The final vertical consolidation pressures of the three tests are the same, but the consolidation processes are different. The specific test plan is listed in Table 2.

Table 2 Creep testing schemes of the sliding zone soil

Sample number	Consolidation path (number)	Void ratio after consolidation	Test type
S-C-01	0→600 kPa (I)	0.271–0.277	Creep
S-C-04	0→600 kPa (I)	0.271–0.277	Slow shear
S-C-02	0→800 kPa→600 kPa (II)	0.257–0.261	Creep
S-C-05	0→800 kPa→600 kPa (II)	0.257–0.261	Slow shear
S-C-03	0→800 kPa→400 kPa→600 kPa (III)	0.260–0.263	Creep
S-C-06	0→800 kPa→400 kPa→600 kPa (III)	0.260–0.263	Slow shear
S-C-07	0→600→0→600 kPa (IV)	0.272–0.273	Creep
S-C-08	0→600→0→600 kPa (IV)	0.272–0.273	Slow shear

During the test, the chamber temperature was controlled to 25℃, the saturated sliding zone soil samples, vertical and horizontal displacement sensors, and pressure sensors were installed. For different load, unload, and reload paths, when the normal stress reached 600 kPa and the vertical displacement rate was not greater than 0.005 mm/h, we can start slow shear test or apply shear stress in stages for creep test. The shear stress of each specimen was loaded at 5 to 6 levels, and the next level of shear stress shall be loaded after the displacement of the previous level was stable, until the specimen had creep failure. The criterion for determining the stability of creep displacement was that the observation time after each level of shear stress loading was not less than 5 d, and the average rate of horizontal displacement change was not more than 0.002 mm/d. When conducting consolidation and creep tests on the sample, wet cotton yarn was used to surround the pressure cap to reduce the influence of the environment on the moisture content of the sample. Two or three sets of tests were performed under each stress state.

The shear creep test adopts the DZR-8 type creep direct shear device from the Soil Mechanics Laboratory of China University of Geosciences (Wuhan), as shown in Fig.3(b). The device consists of a data acquisition system, a vertical load system and a horizontal load system. Vertical load–unload is controlled by adding and subtracting weights, and horizontal loading is controlled by a horizontal piston driven by hydraulic pressure. The vertical and horizontal displacements are collected by the displacement sensor, and the horizontal shear force is collected by the pressure sensor. The data collection time interval is 10 s. The DZR-8 creep direct shear device has a good long-term voltage stabilization effect and its minimum data collection time interval and accuracy meet the specification requirements, hence, it can capture the stress–strain correlation of the sliding zone soil during the load–unload–reload process more realistically. After the test, the samples were photographed and sketched, as shown in Fig. 3(c).

(a) The sliding zone soil sampling

(b) The DZR-8 creep direct shear device

(c) Soil samples after creep shear

Fig.3 The sliding zone soil and test equipment

4 Experimental results and analysis

4.1 Analysis of consolidation characteristics of the sliding zone soil

The consolidation test of sliding zone soil was carried out in three ways. The first was to simulate the actual self-weight stress of the soil. In the laboratory test, the consolidation stress *P* was applied in the order of 50, 100, 200, 300, 400, 500, 600 kPa, and the vertical rate of change of the sample was not more than 0.005 mm/h, which was considered stable. The second was to simulate the stress state of the sliding zone soil before and after the hazard relocation. The maximum load was 800 kPa during loading, and then the pressure was relieved to 600 kPa. During the test, the order of applying the consolidation stress was 50, 100, 200, 300, 400, 500, 600, 700, 800, 700, 600 kPa. The third was to simulate the stress state before and after the excavation of test tunnel, road construction and accumulation of construction waste on Huangtupo landslide. During the test, the order is 50, 100, 200, 300, 400, 500, 600, 700, 800, 700, 600, 500, 400, 500, 600 kPa. In order to compare with the first two working conditions, the third working condition rebounded first and then loaded to 600 kPa. In this way, the final consolidation stress under the three working conditions was 600 kPa, but the consolidation path was different. As shown in Fig. 2, the void ratios after consolidation is stabilized are e_1, e_2 and e_3, respectively. Since there is a unique relationship between the void ratio, effective stress and shear strength of clay soils, the change characteristics of void ratio should be analyzed before creep analysis.

Taking the *e*-*p* curve of a certain set of test data as an example, it can be seen from Fig.4 that when a unidirectional load reaches 600 kPa, sliding zone soil has the smallest compression and the largest void ratio, which is about 0.277. When the soil is loaded to 800 kPa and then unloaded to 600 kPa, the sliding zone soil had the largest compression and the smallest void ratio, which is about 0.261. When the soil is loaded to 800 kPa and unloaded to 400 kPa, and then loaded to 600 kPa, the compression of the sliding zone soil is between the abovementioned two, and the void ratio is about 0.263. This shows that the load, unload, and reload conditions seriously affect the final void ratio of the sliding zone soil. According to the *e*-*p* relationship curve of sliding zone soil, it can be calculated that the compressibility coefficient $a_{1\text{-}2}$ of the sliding zone soil is between 0.37 and 0.45 MPa^{-1}, which belongs to the moderate compressibility soil. The modulus of com- pressibility $E_{s(1\text{-}2)}$ of the sliding zone soil is between 3.0 and 3.6 MPa.

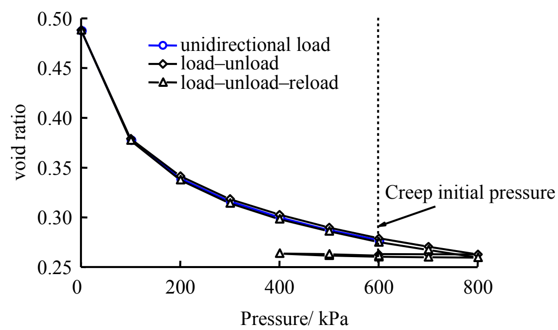


Fig.4 *e*-*p* curves of load–unload–reload of the sliding zone soil

4.2 Relationship between creep displacement and time

Before the creep test, the slow shear tests on the saturated sliding zone soil under the vertical pressures of 400, 600, 800 kPa were carried out to obtain the effective shear strength index. The shear strength under the vertical pressure of 600 kPa was taken as the maximum value, the creep shear load was applied in stages, and the duration of each stage of shear load was 5 d. The shear creep design is shown in Fig.5.

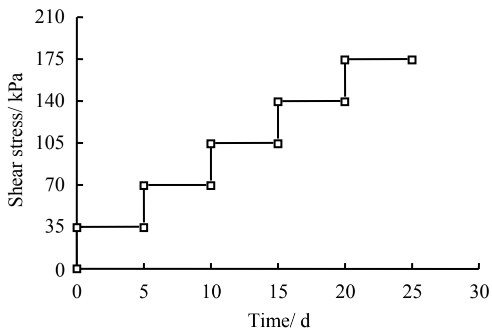


Fig.5 Creep shear stress applied to sliding zone soils

The shear creep curves under different consolidation stresses are shown in Fig.6. It can be seen that under all levels of horizontal shear stress, the creep curves show attenuation and steady-state creeps. As the shear stress increases, the horizontal displacement has an increasing trend, the attenuation creep time gradually increases, and the steady-state creep rate gradually increases. Figure 7 shows the curve of attenuation creep time under different load, unload, and reload paths. It can be seen that as the shear stress increases, the time required for attenuation creep is greater. Under the unidirectional load condition, the void ratio of the sliding zone soil is the largest, so the attenuation creep time under each shear stress is the longest. Under load–unload conditions, the void ratio of the sliding zone soil is the smallest, and the consolidation is the best, so the attenuation creep time is the shortest under all levels of shear stress.

4.3 Characteristics of creep rate changing

As shown in Fig. 6，the creep curves of the sliding zone soil at all levels of shear stress include the decay creep process and the steady-state creep process. In the process of decay creep, the strain rate has been decreasing. Under lower shear stress ($\tau = 35$, 70, 105 kPa), the strain rate will decay to a constant close to 0. At higher shear stress ($\tau = 140$, 175 kPa), the strain rate will decay to a larger constant, which is the constant strain rate of the second stage of creep, namely the steady-state creep stage, its value can reflect

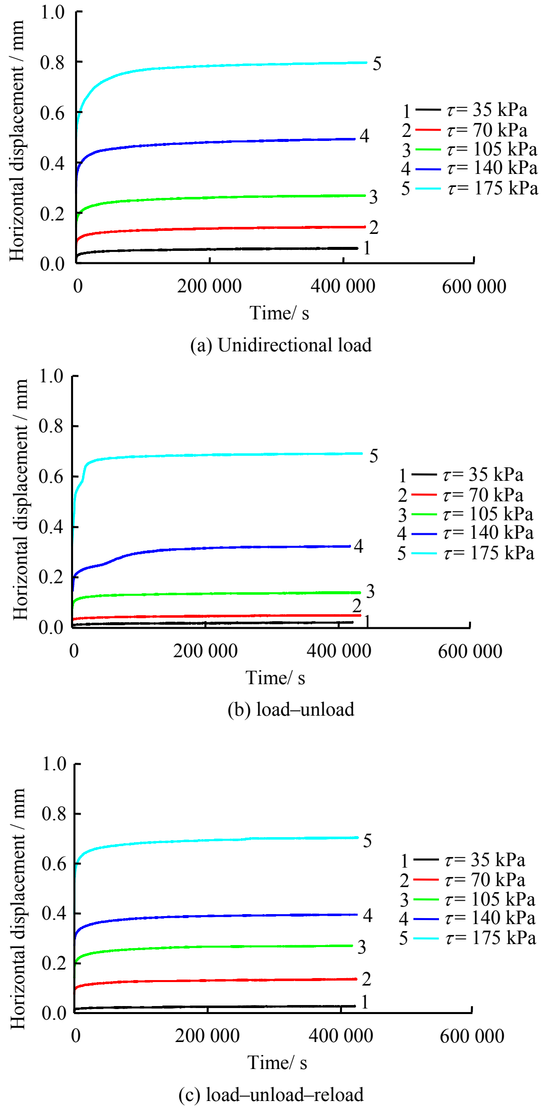


Fig.6 Creep curves of different consolidation states

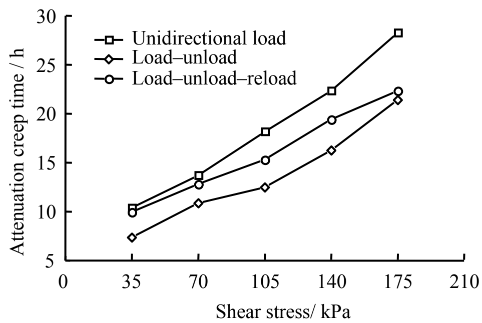


Fig.7 Influence of consolidation state on decay creep time

the overall rate of movement of the landslide. Figure 8 presents the steady-state creep rate under different shear stresses. It can be seen that the steady-state creep rate under the 35 kPa horizontal shear stress state is close to zero. When the horizontal shear stress is the same, the steady-state creep strain rate is the largest under unidirectional load condition, and the steady-state creep strain rate is the smallest under the load–unload condition. This is because the sliding zone soil is equivalent to over-consolidated soil under the load–unload condition.

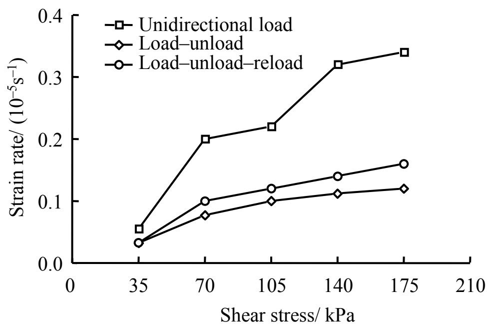


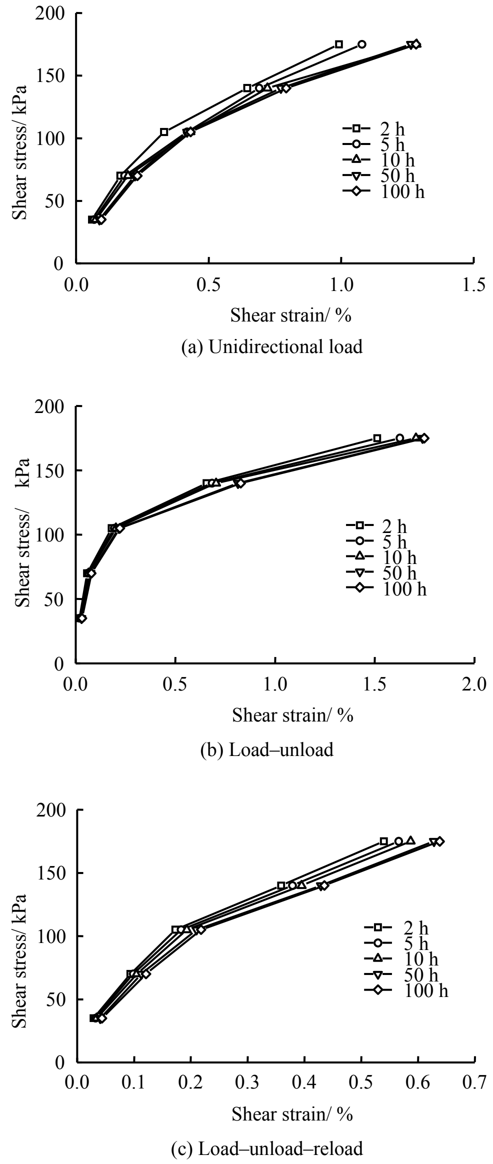
Fig.8 Influence of consolidation state on creep strain rate

4.4 Isochronous creep curve

Figure 9 illustrates the isochronous creep curves of the sliding zone soil subjected to unidirectional load, load–unload, load–unload–reload. It shows that the stress–strain isochronous curve is nonlinear, implying the shear stress level of sliding zone soil affects degree of nonlinear creep of the soil. There is a point with the smallest radius of curvature ($\tau = 105$ kPa) on the stress–strain isochronous curve, which is its yield stress for reshaped sliding zone soil. When the horizontal shear stress is less than 105 kPa, the isochronous curve is close to linear increase, indicating that the creep of the sliding zone soil before is dominated by elastic strain; when the horizontal shear stress is greater than 105 kPa, the isochronous curve has an obvious inflection point, indicating that the subsequent sliding creep of the sliding zone soil is dominated by viscoplastic strain.

4.5 Influence of creep time

The above sections show the creep characteristics of sliding zone soils when the creep time is 5 d. However, sometimes the shear force rises rapidly in a short time due to excessive loading rate or short-term heavy rain. At this time, is the creep behavior of sliding zone soil similar to the results of aforementioned studies? In response to this question, further studies about the creep characteristics of the sliding zone soil after loading-unloading-reloading with a creep time of 1 d have been carried out. The creep test curves are plotted in Fig.10. The case of $\tau = 175$ kPa (τ_5 and τ_1 represent the shear stresses with creep time of 5 d and 1 d, respectively) is analyzed as an example. It is found that the horizontal displacement (0.558 mm) and steady-state creep rate (0.004 mm/d) of sliding zone soil after 1 d shear creep are less than the horizontal displacement (0.679 mm) and steady-state creep rate (0.026 mm) after 5 d shear creep. It can be seen that a longer steady-state creep time of the sliding zone soil can result in a greater shear creep of the sliding zone soil.



(a) Unidirectional load

(b) Load–unload

(c) Load–unload–reload

Fig.9 Influence of consolidation state on isochronous creep curves

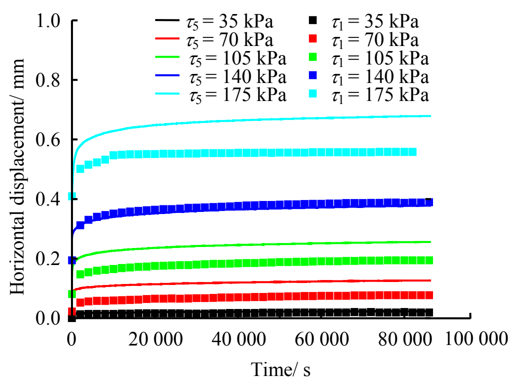
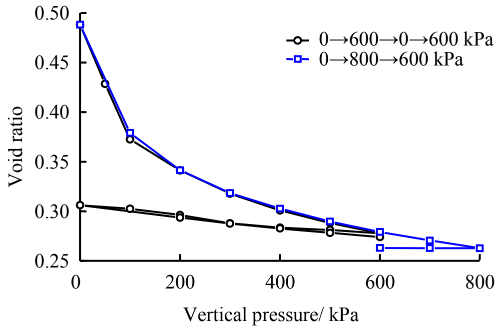


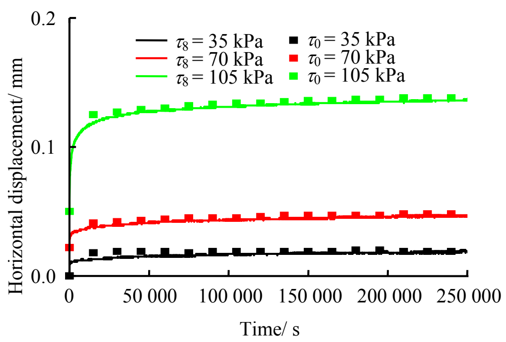
Fig.10 Influence of creep time on creep curves

4.6 Loading–unloading influence on the creep curve

The process of loading–unloading changes the skeleton structure, void ratio and shear modulus of the soil. In order to reveal the influence of different loading-unloading conditions on the creep curve of the sliding zone soil, the loading–unloading tests of 0→800→600 kPa and 0→600→0→600 kPa were carried out, the results are plotted in Fig11(a). It can be seen that the void ratio after loading–unloading–reloading to 600 kPa is greater than that after loading–unloading to 600 kPa, which leads to the attenuation creep and steady-state creep of sliding zone soil greater than that of overconsolidated soil, as shown in Fig11 (b)（τ_8 and τ_0 represent the shear stresses in the consolidated states II and IV, respectively）.



(a) *e*-*p* curves under different consolidation pressures

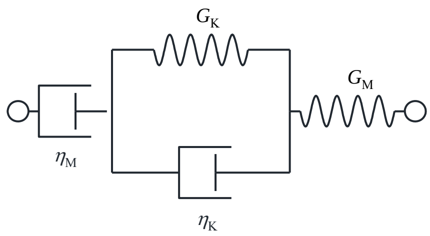


(b) Creep curves under different consolidation pressures

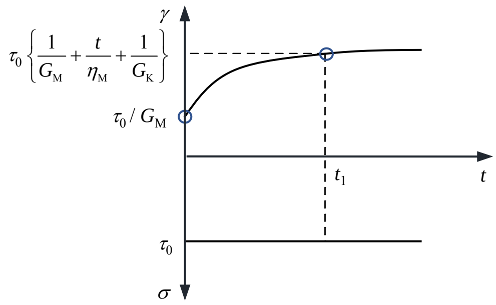
Fig.11 Influence of load–unload on creep curves

4.7 Analysis of the loading–unloading-reloading effect by Burgers creep model

The creep test data of sliding zone soil shows there is a certain instantaneous deformation at the beginning of the creep curve, then the shear strain increases at a decreasing rate, and then the strain rate gradually tends to be stable. The viscoelastic creep behavior of sliding zone soil is consistent with the typical Burgers model curve. Burgers model is a composite creep model composed of Maxwell model and Kelvin model in series, as shown in Fig. 12.



(a) Burgers element model



(b) Burgers creep characteristic curve

Fig.12 Schematic diagram of the Burgers model

The constitutive equation of Burgers model is as follows：

$$\tau+\left(\frac{\eta_M}{G_M}+\frac{\eta_K}{G_M}+\frac{\eta_K}{G_K}\right)\dot{\tau}+\frac{\eta_M\eta_K}{G_M G_K}\ddot{\tau}=\eta_K\dot{\gamma}+\frac{\eta_M\eta_K}{G_M}\ddot{\gamma} \quad (1)$$

where G_M is the shear modulus of Maxwell body; η_M is the viscosity coefficient of Maxwell body; G_K is the shear modulus of Kelvin body; η_K is the viscosity coefficient of Kelvin body; τ is the shear stress; and γ is the shear strain.

The creep stress condition of Burgers model is $\tau=\tau_0=\text{Const}$.

The initial condition is

$$t=0,\ \gamma_{t0}=\tau_0/G_M \text{ and } \gamma_{t0}=\tau_0\left(\frac{1}{G_M}+\frac{1}{G_K}\right).$$

Deducing creep equation by using Laplace transform：

$$\gamma_t=\tau_0\left\{\frac{1}{G_M}+\frac{t}{\eta_M}+\frac{1}{G_K}\left[1-\exp\left(-\frac{G_K}{\eta_K}t\right)\right]\right\} \quad (2)$$

where γ_t is the shear strain at time *t*; τ_0 is the initial shear stress.

The creep equation of Burgers model is composed by Maxwell model and Kelvin model, when $t=0$, the instantaneous strain $\gamma_0=\tau_0/G_M$. This model manifests the creep characteristics of the model under the action of constant stress, and it can describe the instantaneous elastic strain, attenuation creep and stable creep stages of the material[16].

Table 3 lists the parameters of Burgers creep model. The comparison between the data fitting and the measured results indicates that the Burgers model is reasonably acceptable for this study.

Table 3 Creep parameters for the Burges model

Consolidation path	Shear stress / kPa	G_K / kPa	η_K / (kPa·s)	G_M / kPa	η_M / (kPa·s)	R^2
I	35	1.00×10^5	2.00×10^9	7.75×10^4	1.00×10^{11}	0.96
	70	1.10×10^5	2.50×10^9	4.90×10^4	1.10×10^{11}	0.97
	105	9.80×10^4	2.50×10^9	3.59×10^4	1.10×10^{11}	0.98
	140	1.22×10^5	3.50×10^9	2.21×10^4	1.15×10^{11}	0.97
	175	6.90×10^4	3.00×10^9	1.76×10^4	1.50×10^{11}	0.96
II	35	8.00×10^5	7.00×10^9	1.75×10^5	1.65×10^{11}	0.87
	70	8.00×10^5	9.00×10^9	1.20×10^5	2.15×10^{11}	0.90
	105	5.00×10^5	1.00×10^{10}	5.50×10^4	3.50×10^{11}	0.81
	140	3.56×10^4	2.90×10^8	3.62×10^4	9.50×10^{10}	0.80
	175	3.88×10^4	2.90×10^8	1.38×10^4	2.10×10^{11}	0.88
III	35	8.00×10^5	1.00×10^9	1.90×10^5	1.40×10^{11}	0.83
	70	7.80×10^5	9.00×10^9	7.50×10^4	1.40×10^{11}	0.90
	105	9.00×10^4	1.00×10^9	3.50×10^4	2.00×10^{11}	0.87
	140	1.20×10^5	3.50×10^9	2.77×10^4	3.50×10^{11}	0.97
	175	6.34×10^4	9.10×10^8	2.11×10^4	2.20×10^{11}	0.85

5 Conclusions

According to the stress state of the sliding zone before and after the relocation of Huangtupo landslide, the conventional shear test and creep test on the sliding zone soil under the condition of loading-unloading-reloading were carried out. It was concluded that under the same overburden pressure condition, if the consolidation history is different, the long-term strength will be different. Burgers model was used to analyze the creep characteristics of the sliding zone soil under the same overburden pressure, and the following conclusions were drawn:

(1) In order to better reflect the creep characteristics of the sliding zone before and after the relocation, the consolidation paths of single loading, loading-unloading and loading-unloading-reloading should be adopted to study the shear creep characteristics of the sliding zone soil under the same overburden stress with different consolidation paths.

(2) Compared with the single load consolidation path, the load–unload and load–unload–reload consolidation paths obtained a smaller pore ratio under the same overburden pressure, the sliding zone soil was further compacted, and the long-term strength was higher.

(3) Burgers model could reflect the creep characteristics of the sliding zone soil before accelerated creep, but the influence of different consolidation states and different consolidation paths must be considered.

(4) The creep test curve was in good agreement with the theoretical fitting curve which means that the Burgers creep model considering consolidation path could describe the creep characteristics of sliding zone soil under different consolidation states, which may provide basic data for landslide stabilization analysis.

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