Research Article

Measurement and Investigation on 1-D Consolidation Permeability of Saturated Clay considering Consolidation Stress Ratio and Stress History

Le Zhang¹, Faning Dang^{1,2,3}, Jun Gao¹ and Jiulong Ding¹

¹School of Civil Engineering and Architecture, Xi'an University of Technology, Xi'an 710048, China

²State Key Laboratory of Eco-Hydraulics in Northwest Arid Region, Xi'an University of Technology, Xi'an 710048, China

³Shaanxi Key Laboratory of Loess Mechanics and Engineering, Xi'an University of Technology, Xi'an 710048, China

Correspondence should be addressed to Faning Dang; dangfn@163.com

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To study the influence of consolidation stress ratio and stress history on 1-D consolidation permeability of saturated clay, one-dimensional consolidation permeability tests were carried out with GDS triaxial device. The results indicated that the permeability coefficient and void ratio of normally and overconsolidated saturated clay decreased with the increase of consolidation stress ratio under different consolidation stress ratios but the same stress history. And the amount of final sample's compression increased with the increase of the consolidation stress ratio. Under the condition of the same consolidation stress ratio but different stress history, the amount of final compression of the overconsolidated saturated clay was smaller than that of the normally consolidated saturated clay. Besides, the stress difference σ_{dv} between consolidation pressure σ and gravity stress σ_{cz} was fitted to the amount of the final sample's compression, and a good linear relationship between the stress difference σ_{dv} and the amount of the final sample's compression under each consolidation pressure was obtained. The results showed that it is necessary to consider the influence of consolidation stress ratio and stress history simultaneously on 1-D consolidation permeability of saturated clay. Meanwhile, it can accurately predict the amount of the final sample's compression after knowing the gravity stress. Moreover, a model prediction analysis was conducted on the saturated clay and recommended to use the modified Kozeny-Carman's equation to predict the permeability coefficient of Luochuan saturated clay during one-dimensional consolidation.

1. Introduction

Saturated clay (especially soft soil) has a considerable areal distribution range in China's coastal and inland areas. Due to the fact that soft clay foundation has the characteristics of high compressibility and low permeability, the duration of consolidation deformation tends to be longer. Therefore, the prediction of foundation settlement and settlement rate has become a major problem in engineering design [1]. In recent years, some new foundation improvement methods have been proposed, such as biogrouting and drainage powder sprayed piles [2, 3]. And the relationship between the foundation settlement and the time is always based on the one-dimensional Terzaghi's consolidation theory. The theory assumed that the permeability coefficient k , initial void ratio e

, and compressibility coefficient a are all constant during the consolidation process, and the permeability of water in the soil obeys Darcy's law. However, some simplified assumptions are often not met in actual projects and can be extremely complicated. Davis and Raymond [4] obtained the analytical solution of the one-dimensional nonlinear consolidation equation. And Xie et al. [5] proposed an analytical solution for one-dimensional nonlinear consolidation of double-layered soils based on Davis. A method for simulating nonlinear characteristics of soil during compression by the hyperbolic curve was developed according to the compression behavior of soil [6]. Dumais and Konrad [7] established a one-dimensional model for the consolidation of thawed soil by using the relationship among effective stress, void ratio, and permeability coefficient. Yin and Feng [8, 9] stated that the

variable compressibility must be considered in settlement calculation of soft soil stratum and proposed a new simplified method to calculate the settlement of a thick soil layer by considering the nonlinear compressibility. Besides, a one-dimensional nonlinear consolidation theory considering the thermal effect was established, which showed that the pore water dissipation rate or consolidation rate considering the secondary consolidation and temperature was higher than that when only considering secondary consolidation [10]. Some researchers have also conducted one-dimensional consolidation nonlinear studies on layered foundations. Xia et al. [11] assumed that the change in compressibility and permeability of the soil during the consolidation process was proportional, and the analytical solution of one-dimensional nonlinear consolidation for layered saturated soft soil was derived based on e -log σ' and e -log k_v relations. An analytical solution for one-dimension consolidation of a clay layer with variable compressibility and permeability under a ramp loading was derived based on the assumption that the initial effective stress of the clay layer remains constant with depth [12]. To consider the limitation of Terzaghi's theory, Lekha et al. [13] presented a generalized theory for the vertical consolidation of a compressible medium with finite thickness, and an analytical closed-form solution was obtained for vertical consolidation considering the variation in the compressibility and permeability. Moreover, Abbasi et al. [14] derived a one-dimensional nonlinear partial differential equation for the prediction of consolidation characteristics of soft clays considering variable values for C_v , and took into consideration the changes in C_v during consolidation.

However, the aforementioned one-dimensional nonlinear consolidation models were based on the assumption of Darcy's flow. The characteristics of the non-Darcy's flow of water in the soil during one-dimensional consolidation have long been recognized. Mishra and Patra [15], Liu et al. [16], and Zhao and Gong [17] have studied the consolidation characteristic of saturated clay by considering the non-Darcy's flow. Li et al. [18] proposed a nonlinear model of consolidation by considering the non-Darcy's flow and stress history and found that the consolidation rate of overconsolidated soil with the non-Darcy's flow was slower than that with Darcy's flow. Moreover, Terzaghi's one-dimensional consolidation equation was modified and generalized based on the non-Darcy's flow in the soil [19]. However, the consolidation theory which can consider both geometrical nonlinearity and non-Darcy's flow is rarely reported. Li et al. [20] proposed a model for soft clay, which can allow for these two factors simultaneously, and the results showed that the difference between the consolidation rates caused by the non-Darcy's and Darcy's flows is significant. Besides, some researchers have also considered the effect of stress history on the one-dimensional consolidation [21–24] and have proposed one-dimensional nonlinear consolidation equations considering stress history.

Besides, the Terzaghi's one-dimensional consolidation theory assumed that the external load is applied instantaneously and remains unchanged during the consolidation process. However, the load acting on the foundation generally increases gradually with the construction process, and it is impossible to have "instantaneous" loading as assumed

by Terzaghi. Schiffman [25] gave a theoretical solution to this situation. Next, Olson [26] proposed the mathematical solution of the average degree of consolidation in the first-order linear loading process. In recent years, some researchers have also conducted one-dimensional consolidation analysis under ramp loading mode according to different working conditions. Liu and Ma [27] studied the one-dimensional consolidation of soft ground with impeded boundaries under a depth-dependent ramp load. Similarly, Sun et al. [28] also introduced the exponentially time-growing drainage boundary and investigated the one-dimensional consolidation problem of soil under a ramp load. This one-dimensional consolidation theory can be utilized to describe the asymmetric drainage characteristics of the top and bottom drainage surfaces of the actual soil layer. However, the present solution is just for homogeneous soil. Tian et al. [29] obtained an analytical solution of excess pore-water pressure under a multi-stage load and the continuous drainage boundary. A different conclusion from Terzaghi's one-dimensional consolidation theory was obtained; that is, the degree of consolidation is also related to the applied time-dependent load. In the same year, a new simplified method was developed to calculate the settlement of multilayer soft soils exhibiting creep subjected to the multistage loading under a one-dimensional straining condition [30]. The advantage of this new simplified method is that the soft soils at both the normally consolidated state and overconsolidated state are considered in this approach by the "equivalent time" concept.

For further studying the nature of saturated clay permeability and consolidation, many researchers conducted laboratory tests on saturated clay. Experiments on the change characteristics of soil permeability coefficient under different head pressures were carried out and obtained that the permeability coefficient of soft soil decreased with the decreased of void ratio [31]. The consolidation and permeability combined tests of Xiaoshan soft clay were carried out by Qi [32] and showed that the compressibility and permeability of overconsolidated soils vary nonlinearly with the consolidation pressure during consolidation. Besides, Wen [33] had studied the one-dimensional consolidation theory of double-layered soil considering effects of stress history and revealed that the settlement was smaller and the rate of consolidation was greater than that of the one without considering the effects of the stress history. Moreover, the initial static stress state of the natural foundation soil; i.e., consolidation stress σ_1 and σ_3 or consolidation stress ratio K_c , where $K_c = \sigma_1/\sigma_3$, is also different. The effect of consolidation stress ratio on the pore pressure characteristics of saturated undisturbed loess under different stress paths was studied and showed that the effect of stress path on pore pressure characteristics during equal pressure consolidation was significantly different from that during K_0 consolidation [34].

Through reviewing the relevant studies, it was found that the influence of the initial stress state (consolidation stress ratio K_c) and stress history of natural foundation soil on the one-dimensional consolidation and permeability characteristics of saturated clay was rarely considered and only involved the properties of normally consolidated soil or overconsolidated soil on the one hand. In this study, the GDS-advanced consolidation device was utilized to conduct a series of one-

dimensional consolidation-permeability combined tests under ramp loading on the loess silty clay in Luochuan, Shaanxi. Then, the effects of consolidation stress ratio and stress history on one-dimensional consolidation permeability of saturated clay were explored based on test results. Finally, a model prediction analysis was performed which is aimed at selecting a model suitable for predicting the permeability coefficient of Luochuan saturated clay.

2. Test and Method

2.1. Sample Preparation and Method. The clay soil samples used in this study were loess-like silty clay taken from Luochuan, Shaanxi. The sampling depth is 7–8 m, and the main physical properties of the silty clay are illustrated in Table 1.

Due to the heterogeneity of undisturbed clay samples, the samples used in this study were all saturated homogeneous remolded soil samples, and the preparation processes were as follows:

- (1) Firstly, the soil was air-dried, ground, and sieved through a 2 mm sieve. Then, the air-dried moisture content was measured after fully mixing. The soil was prepared to a moisture content (11.6%) that can be used for sample preparation. Finally, the prepared soil was placed in a plastic bag and sealed for 24 hours to allow moisture equalization
- (2) The moisture content of the prepared soil sample was measured, and the mass of soil required for sample preparation was calculated. In this study, the pressure sampling method was used for sample preparation, and the samples with 3.91 cm in diameter and 8 cm in height were used. The sample was divided evenly into five layers during sample preparation. For making better contact between the sample layers, the contact surface should be shaved before each compaction. According to the e -log p curve, the pre-consolidation pressure of Luochuan silty clay was 115 kPa, and the soil was considered to be in a normally consolidated state. Therefore, the remolded soil samples were consolidated under a pressure of 115 kPa. After the consolidation was completed, the soil samples were considered to be in a normally consolidated state
- (3) To reflect the influence of consolidation stress ratio on one-dimensional consolidation permeability of saturated clay, a consolidation test was conducted under the condition of $K_c = 1.5$ firstly, and it was obtained that the amount of soil compression reached 10 mm when the consolidation stress was 600 kPa. According to the Chinese standard for the geotechnical testing method (GB/T 50123; National Standards of People's Republic of China 2019), the sample was considered to be damaged when the strain reaches 15%. Therefore, the consolidation stress ratio was taken as 1, 1.3, and 1.5, respectively

2.2. Experimental Device. A GDS triaxial device imported from the UK was used for the one-dimensional consolidation and permeability tests, as shown in Figure 1. The test device can not only perform traditional consolidation tests but also perform permeability tests under the condition of the constant consolidation pressure, which can directly measure the permeability coefficient under various consolidation stress levels. For conducting the consolidation permeability test for saturated remolded soil samples, a set of saturated soil base and dual-channel sample cap was redesigned and processed to achieve the connection of back pressure controller with sample's base and cap, which ensured that the water drained from the upper and lower ends of the sample was drained into the back pressure controller. Hence, the sample's volume change can be accurately measured. The schematic diagram of the GDS device was shown in Figure 2.

2.3. Test Procedure. For studying the one-dimensional consolidation permeability characteristics of saturated clays considering the initial consolidation state and consolidation stress ratio, one-dimensional consolidation permeability tests were performed. The specific test steps were as follows:

- (1) Before loading the sample into the pressure chamber, the base and sample cap were vented to reduce the volume of gas entering the sample as much as possible when the sample was saturated with back pressure
- (2) After completion of the sample preparation, the soil sample was saturated by the vacuum saturation method firstly and then by the back pressure. When the pore pressure coefficient B was close or equal to 1, the sample was considered to be fully saturated
- (3) Then, the sample was consolidated by applying consolidation stress with a constant duration of 24 h at each stress level. When the pore pressure dissipated completely, the axial deformation was basically unchanged, and the volume change became stable; the consolidation was considered complete. The specific test scheme is shown in Table 2, and the schematic diagram of multistage ramp loading is shown in Figure 3. In Figure 3, $t_{j1}-t_{j4}$ represented the time required to reach each level of load, and $t_{j1}-t_{j4} = 30$ mins. t_1-t_4 represented the total duration of consolidation at each stage, and $t_1 = t_2 = \dots = t_4 = 24$ h
- (4) Subsequently, the permeability test was performed by the water head difference formed by the back pressure and the bottom pressure. To ensure that the effective stress acting on the soil sample remains unchanged in the process of the permeability test, the following equation must be satisfied between the pressure

$$p_1 - p_2 \geq p_2 - p_3, \quad (1)$$

where p_1 is confining pressure, p_2 is back pressure, and p_3 is bottom pressure. The bottom pressure is always 0 kPa in this

TABLE 1: Physical properties of intact Q₃ loess.

Specific gravity G_s	Water content	Dry density	Liquid limit	Plastic limit	Particle size fraction (%)			Soil classification
	w (%)	ρ_d (g/cm ³)	w_L (%)	w_p (%)	>0.075 mm	0.075~0.005 mm	<0.005 mm	
2.7	13.3	1.34	28.4	19.2	4	73	23	Silty clay

FIGURE 1: GDS triaxial device.

study, and the duration of permeability induced from water head difference is 70 min. The test plan is listed in Table 3.

To study the influence of stress history on the one-dimensional consolidation permeability of saturated clay, the consolidation stress with the same increment was applied. In this study, the double-drainage condition was adopted, and the one-dimensional consolidation diagram of the saturated clay layer is shown in Figure 4.

3. Result and Discussion

3.1. Effect of Consolidation Stress Ratio on One-Dimensional Consolidation Permeability of Normally Consolidated Saturated Clay

3.1.1. Permeability Test Results and Analysis. When the consolidation stress ratio $K_c = 1, 1.3, \text{ and } 1.5$, the relationship curves of permeability coefficient k and void ratio e with consolidation stress σ in one-dimensional consolidation permeability of normally consolidated soil are shown in Figures 5 and 6, respectively.

It can be seen from Figure 5 that the initial permeability coefficients at three consolidation stress ratios were the same before applying consolidation stress. The permeability coefficient k decreased with the increase of consolidation stress, which was independent from the consolidation stress ratio. And the permeability coefficient decreased as the consolidation stress ratio increased under the same consolidation stress. On the whole, the permeability coefficient showed a

nonlinear decreasing trend with the increase of consolidation stress. As the consolidation stress continued to increase, the rate at which the permeability coefficient k decreased as the consolidation stress increased gradually slows down.

The consolidation stress ratio K_c had a certain effect on the permeability coefficient k . As the consolidation stress increased, the smaller the consolidation stress, the greater the effect of the consolidation stress ratio on the permeability coefficient. Besides, the permeability coefficient is an important parameter for the consolidation analysis of saturated clay. Therefore, it is necessary to study the consolidation stress ratio on one-dimensional consolidation permeability of saturated clay.

In the one-dimensional consolidation process of saturated clay, the determination of the void ratio e , compression coefficient a_v , and compression modulus E_s is the same as the traditional consolidation test method.

The void ratio e_i of the sample after consolidation stability under various consolidation pressures can be obtained from

$$e_i = e_0 - (1 + e_0) \cdot \frac{\sum \Delta h_i}{h_0}, \quad (2)$$

where e_0 and h_0 are the void ratio and initial height of the sample after back pressure saturation, e_i is the void ratio of the sample after consolidation stability, and $\sum \Delta h_i$ is the cumulative compression of the sample at the level of

FIGURE 2: Schematic diagram of pressure application for consolidation and permeability tests.

TABLE 2: Scheme and procedure of staged consolidation test.

Consolidation state	Loading step
Normally consolidated	115→140→165→190→215 kPa
Overconsolidated	115→25→50→75→100 kPa

consolidation stress σ_i . Then, the height of the sample after compression is expressed by

$$h = h_0 - \sum \Delta h_i. \quad (3)$$

The compression coefficient a_v in a certain stress range can be yield by

$$a_v = \frac{e_i - e_{i+1}}{\sigma_{i+1} - \sigma_i}, \quad (4)$$

where e_i and e_{i+1} are the void ratios corresponding to σ_i and σ_{i+1} , respectively, σ_i and σ_{i+1} are the consolidation stress values of a certain level and the next level, respectively.

The compression modulus E_s in a certain stress range can be obtained by

$$E_s = \frac{1 + e_0}{a_v}. \quad (5)$$

According to Equations (2), (3), (4), and (5), the deformation parameters of the sample during the consolidation process are illustrated in Table 4. The definition of each parameter in Table 4 was the same as above. For example, the amount of compression, void ratio, compressibility, and compression modulus of the corresponding sample under the consolidation stress of 165 kPa were the calculated results of various parameters within the range of consolidation stress of 140–165 kPa.

In this study, the void ratio after back pressure saturation was taken as the initial void ratio e_0 of the sample. It can be seen from Figure 6 that the corresponding initial void ratio e_0 when $K_c = 1, 1.3$, and 1.5 was 1.015. It also can be seen from Equation (2) that the void ratio e during the consolidation stage is related to the initial void ratio e_0 , initial height h_0 , and the cumulative compression of the sample $\sum \Delta h_i$ under certain consolidation stress.

From Figure 6, the void ratio decreased with the increase of consolidation stress. When $\sigma = 215$ kPa, the corresponding void ratios of $K_c = 1.0, 1.3$, and 1.5 were 0.904, 0.876, and 0.834, respectively. After obtaining the amount of sample's compression, the variation curve of void ratio and time can be drawn, which is shown in Figure 7.

Both Figures 5 and 6 show the nonlinear characteristics of the soil sample during the consolidation process. As increased with the consolidation stress, the pore volume of the soil decreased continuously due to the drainage of pore water, which resulted in the permeability coefficient of the soil was continuously decreasing. The reduction of the void ratio was also related to the consolidation stress ratio K_c to a certain extent. When $K_c = 1, 1.3$, and 1.5 , the corresponding reductions for the void ratio were 10.97%, 13.70%, and 17.79%, which illustrated the importance of considering the consolidation stress ratio on one-dimensional consolidation permeability of saturated clay in this study.

It can be seen from Figure 7 that the void ratio e showed a stepwise downward trend with time. The void ratio of the sample hardly changes during the permeability stage, and the change of the sample's void ratio mainly occurred during the consolidation stage.

3.1.2. Consolidation Test Results and Analysis. The consolidation coefficient is an important parameter in both geotechnical and geoenvironmental engineering [35]. According to the Terzaghi's one-dimensional consolidation theory, the consolidation coefficient of the sample can be obtained according to

$$C_v = \frac{k(1 + e_1)}{a_v \cdot \gamma_w}, \quad (6)$$

where C_v represents the consolidation coefficient of the soil (cm^2/s); k is the permeability coefficient (cm/s); e_1 is the initial void ratio; a_v is the compression coefficient in a certain consolidation stress range (MPa^{-1}); and γ_w is the unit weight of water (chosen as $10 \text{ kN}/\text{m}^3$). In this study, the initial void ratio and the initial height were determined by assuming that the completed state of the previous level of stress consolidation is the initial state of this level.

FIGURE 3: Schematic diagram of multistage ramp loading.

TABLE 3: Permeability test scheme.

Consolidation state	Consolidation stress (kPa)	Water head difference (kPa)
Overconsolidated	25	5, 10
	50	10, 20
	75	10, 20
	100	20, 40
Normally consolidated	140	20, 40, 60
	165	20, 40, 60, 80
	190	20, 40, 60, 80
	215	20, 40, 60, 80, 100

According to Equation (6), the consolidation coefficients of the sample with $K_c = 1, 1.3$, and 1.5 are shown in Table 5. Following Table 5, the C_v - σ relationship curve was plotted with the consolidation stress σ as the abscissa and the consolidation coefficient C_v as the ordinate, which is shown in Figure 8.

It can be seen from Figure 8 that when $K_c = 1, 1.3$, and 1.5 , the consolidation coefficient C_v showed a trend of increasing first and then decreasing with the increase of consolidation stress. Some researchers have also studied the changing law of consolidation coefficient with consolidation stress and time during the process of consolidation [14, 36]. It was found that with the increase of consolidation stress, the change of consolidation coefficient was not only monotonously increasing or decreasing. Li et al. [36] established the consolidation coefficient expression related to pore development and revealed that the development of the sample pore greatly affected the changing law of the consolidation coefficient. In the consolidation process, the consolidation

FIGURE 4: One-dimensional consolidation sketch.

coefficient of the soil was a variable, which was related to the effective stress acting on the soil sample. The consolidation coefficient was an increasing function relative to the void ratio under a relatively lower effective stress level. With the soil sample being further compressed, the void ratio continued to decrease, which resulted in a decrease in the consolidation coefficient. As shown in Figure 8, the consolidation stress ratio had an obvious influence on the consolidation coefficient, but it did not affect the distribution shape of the consolidation coefficient. The larger the consolidation stress ratio, the smaller the consolidation coefficient.

The relationship curve between the amounts of sample's compression and time is shown in Figure 9.

FIGURE 5: The curve of permeability coefficient of normally consolidation clay with consolidation pressure at different consolidation stress ratios.

FIGURE 6: The curve of void ratio of normally consolidation clay with consolidation pressure at different consolidation stress ratios.

TABLE 4: Calculation results of mechanical parameters of normally consolidation clay during consolidation.

Consolidation stress ratio K_c	Consolidation stress (kPa)	Sample's compression (mm)	Sample's height after compression (mm)	Void ratio e	Compression coefficient (MPa^{-1})	Compression modulus E_s
1	140	3.96	76.03	0.915	0.71	2.83
	165	4.08	75.91	0.912	0.12	16.46
	190	4.27	75.72	0.907	0.19	10.72
	215	4.43	75.56	0.904	0.17	12.16
1.3	140	4.73	75.26	0.896	0.85	2.37
	165	4.98	75.01	0.890	0.25	8.13
	190	5.24	74.75	0.883	0.27	7.57
	215	5.52	74.47	0.876	0.28	7.20
1.5	140	6.24	73.75	0.858	1.12	1.79
	165	6.60	73.39	0.849	0.36	5.63
	190	6.86	73.13	0.842	0.27	7.57
	215	7.17	72.82	0.834	0.31	6.59

FIGURE 7: The curve of void ratio of normally consolidation clay with time.

It can be seen from Figure 9 that the amount of sample's compression increased stepwise with time under three consolidation stress ratios. The amount of sample's compression at the permeability stage was almost 0 mm, and the sample's compression mainly occurred at the consolidation stage. It can also be seen from the figure that the amount of sample's final compression corresponding to $K_c = 1$, 1.3, and 1.5 were 4.43, 5.52, and 7.17 mm, respectively. Therefore, considering the initial stress state of the soil in the one-dimensional consolidation of saturated clay plays a good role in accurately predicting the final settlement of the soil in the future.

Figure 10 displays the variation curve of the permeability coefficient of normally saturated clay with the void ratio in the logarithm coordinate. Taking the slope of the linear fitting line as permeability index C_k and C_k corresponding to $K_c = 1$, 1.3, and 1.5 was expressed by C_{k1} , $C_{k1.3}$, and $C_{k1.5}$, respectively. Then, $C_{k1} = 0.047$, $C_{k1.3} = 0.067$, and $C_{k1.5} = 0.093$.

It can be seen from Figure 10 that the permeability coefficient k of the sample decreased as the void ratio decreased. When the consolidation stress $\sigma = 215$ kPa, the corresponding permeability coefficients of $K_c = 1$, 1.3, and 1.5 were as follows: 0.493×10^{-5} cm/s, 0.335×10^{-5} cm/s, and 0.213×10^{-5} cm/s.

TABLE 5: Calculated value of consolidation coefficient.

Consolidation stress ratio K_c	Consolidation stress (kPa)	Consolidation coefficient ($10^{-5} \text{ cm}^2/\text{s}$)
1	140	0.230
	165	1.152
	190	0.571
	215	0.567
1.3	140	0.148
	165	0.401
	190	0.302
	215	0.225
1.5	140	0.061
	165	0.167
	190	0.184
	215	0.128

$^{-5} \text{ cm/s}$, which was about the initial permeability coefficient 0.173, 0.118, and 0.075 times. The greater the K_c , the greater the amplitude of variation.

Figure 11 represents the relation curve of the void ratio of normally consolidated saturated clay with the consolidation stress. The compression index corresponding to $K_c = 1, 1.3$, and 1.5 was represented by C_{c1} , $C_{c1.3}$, and $C_{c1.5}$. Therefore, C_{c1} , $C_{c1.3}$, and $C_{c1.5}$ were equal to 0.063, 0.106, and 0.123, respectively, where the compression index was a maximum when $K_c = 1.5$.

It can be seen from the e -log σ curve in Figure 11 that the void ratio decreased as the consolidation pressure increased, and the greater the consolidation stress ratio K_c , the smaller the void ratio e . The test results showed the nonlinearity of the one-dimensional consolidation of the soil sample.

3.1.3. Analysis of Compression Curve. For a homogeneous soil layer with a natural gravity of γ , at any depth z under the natural ground, the vertical self-weight stress σ_{cz} of the soil is equal to the gravity of the soil column per unit area. That is,

$$\sigma_{cz} = \sum_{i=1}^n \gamma_i h_i, \quad (7)$$

where σ_{cz} is the vertical effective self-weight stress of the soil at any depth z under the natural ground (kPa); n is the total number of soil layers within the depth z ; h_i is the thickness of the i th soil layer (m); γ_i is the natural unit weight of the i th soil layer; and the effective unit weight is taken for the soil layer below the groundwater level (kN/m^3).

The sampling depth is 7–8 m, and its self-weight stress σ_{cz} is in the range of 106.4–121.6 kN. The preconsolidation pressure is 115 kPa, and the soil is considered to be in the normally consolidated state. Therefore, it was determined that the self-weight stress of the sample used in the study was 115 kPa. Then, the difference σ_{dv} between the consolida-

tion pressure σ applied in the test and the self-weight stress σ_{cz} of the soil layer was expressed by

$$\sigma_{dv} = \sigma - \sigma_{cz}. \quad (8)$$

The calculated values of the difference between the consolidation pressure and the self-weight stress of the soil layer when the consolidation stress ratio $K_c = 1$ are listed in Table 6. The fitted relation curve between the amount of sample's final compression s_f and σ_{dv} is shown in Figure 12.

It can be seen from Figure 12 that the stress difference σ_{dv} had a good linear relationship with the amount of sample's final compression at each consolidation pressure. After knowing the self-weight stress of the soil, the amount of final compression of the sample can be predicted more accurately, which had an important application value to the engineering.

3.2. Effect of Consolidation Stress Ratio on One-Dimensional Consolidation Permeability of Overconsolidated Saturated Clay. As mentioned above, the preconsolidation pressure of Luochuan loess-like silty clay σ_c was 115 kPa. Therefore, the consolidation stress applied to the soil sample was 25, 50, 75, and 100 kPa, and the corresponding overconsolidation ratios (OCR) are 4.6, 2.3, 1.5, and 1.2, respectively.

3.2.1. Permeability Test Results and Analysis. Table 7 shows the measured values of the permeability coefficient at various consolidation pressures for overconsolidated soil.

According to Table 7 and Equation (2), the relationship curves of permeability coefficient, void ratio, and consolidation stress at different consolidation stress ratios K_c were drawn, which are shown in Figures 13 and 14.

As can be seen from Figures 13 and 14, for overconsolidated soils, both the permeability coefficient and the void ratio decreased with increasing consolidation pressure at different consolidation stress ratios, and the larger K_c , the smaller e and k . Taking the consolidation stress ratio $K_c = 1.0$ as an example, the permeability coefficients when the consolidation stress was 50, 75, and 100 kPa were reduced by 24.20%, 26.66%, and 43.48%, respectively, compared with that at 25 kPa. Besides, the permeability coefficient and void ratio of overconsolidated soil decreased with the increase of the consolidation stress ratio at the same consolidation pressure. This phenomenon was the same as that of normally consolidated soil; that is, the permeability coefficient was related to the sample's void ratio, and the larger the consolidation stress ratio, the smaller the sample's void ratio.

3.2.2. Consolidation Test Results and Analysis. According to Equations (2), (3), (4), and (5), the deformation parameters of consolidation and the amount of compression of overconsolidated soils were calculated, as shown in Table 8.

According to Table 8, the relationships between the sample's void ratio and compression over time during the consolidation and compression process are shown in Figures 15 and 16.

It can be seen from Figure 15 that the amount of sample's compression increased with time during the one-dimensional consolidation of the overconsolidated soil. And the sample's compression mainly occurred during the consolidation stage.

FIGURE 8: The curve of consolidation coefficients of normally consolidation clay with consolidation pressure at different consolidation stress ratios.

FIGURE 9: The curve of normally consolidation clay compression with time at different consolidation stress ratios.

FIGURE 10: e - $\log k$ curve.

Taking the consolidation stress ratio $K_c = 1$ as an example, when the consolidation stress was 25, 50, 75, and 100 kPa, the corresponding amount of sample's compression was 3.23, 3.31, 3.41, and 3.53 mm, respectively. However, the amount of sample's compression during the permeability stage under the corresponding consolidation stress hardly changed. The amount of final sample's compression varied with different consolidation stress ratios. When $K_c = 1$, 1.3, and 1.5, the amount of final sample's compression was 3.53, 3.92, and 4.57 mm, respectively, which revealed that the greater the consolidation stress ratio, the greater the final sample's compression.

Accordingly, from Figure 16, it can be seen that the void ratio of the sample gradually decreased with time, and the variation of the void ratio mainly occurred in the consolidation stage, while hardly changing in the permeability stage. Different consolidation stress ratios resulted in a different final void ratio of the sample. That is, when $K_c = 1$, 1.3, and 1.5, the corresponding void ratio was 0.926, 0.916, and 0.900, respectively. In other words, the greater the consolidation stress ratio, the smaller the final void ratio of the sample. The final void ratio was reduced by 8.76%, 9.73%, and 11.35% compared with the initial void ratio.

3.2.3. Compression Curve Analysis. According to Equations (7) and (8), the difference between the consolidation pressure of saturated clay and the self-weight stress of the soil layer can be obtained. Only the calculated values when $K_c = 1$ are listed in Table 9.

The relationship curve between the stress difference σ_{dv} and the amount of final sample's compression s_f was linearly fitted, as shown in Figure 17.

It can be seen from Figure 17 that there was a good linear relationship between the stress difference of overconsolidated soil and the amount of final sample's compression under each consolidation pressure, and the amount of final compression of overconsolidated soil can be accurately predicted after knowing the self-weight stress of soil.

3.3. Effect of Stress History on One-Dimensional Consolidation and Permeability of Saturated Clay. The compressibility and permeability of soil play an important role in consolidation, which are closely related to stress history [33]. The research results showed that the settlement of the foundation was smaller and the consolidation development was faster when the stress history effect was considered. Besides, the settlement development rate increased with the increase of preconsolidation pressure.

According to the obtained test results, a series of consolidation and settlement curves of normally and overconsolidated soil under the condition of a certain consolidation stress ratio K_c (and $K_c = 1$) was acquired.

Figure 18 is the variation curve of the amount of sample's compression with the time of normally and overconsolidated saturated clay during the consolidation process when the consolidation stress ratio $K_c = 1$. It can be seen from the figure that the amount of compression of

FIGURE 11: e - $\log \sigma$ curve.

TABLE 6: Stress calculations of normally consolidation clay.

Stress state	σ (kPa)	σ_{cz} (kPa)	σ_{dv} (kPa)
Normally consolidated	140	115	25
	165		50
	190		75
	215		100

overconsolidated soil was less than that of normally consolidated soil during the same consolidation time. Hu [37] used the semianalytical method to study the one-dimensional linear and nonlinear consolidation behavior of single-layer overconsolidated saturated soil foundation. The results revealed that the settlement of the foundation was smaller and the consolidation developed faster when the influence of stress history was considered, which was consistent with the conclusion of this study. That is, the amount of final compression of the overconsolidated soil under the 5th continuous load was 3.53 mm, while the normally consolidated soil was 4.43 mm.

Figures 19 and 20 are the variation curve of the void ratio e and the permeability coefficient k with the consolidation pressure during the process of one-dimensional consolidation permeability of normally and overconsolidated saturated clay.

It can be seen from Figure 19 that the void ratio of normally and overconsolidated clays decreased monotonously with the increase of consolidation stress during the process of one-dimensional consolidation permeability. Accordingly, the permeability coefficient decreased nonlinearly with the increase of consolidation stress. The compressibil-

ity and permeability of soil in the overconsolidation stage were much less than that in the normally consolidation stage.

As can be seen from Figures 18, 19, and 20, considering the stress history was of great significance for one-dimensional consolidation of saturated clay when the consolidation stress ratio K_c was constant. However, stress history did not affect the relationship between the permeability coefficient and void ratio.

4. Model Prediction of Saturated Clay Permeability Coefficient

All fields of geotechnical engineering are closely related to the permeability of rock and soil [38, 39]. In engineering practice, it is particularly important to accurately and quickly predict the permeability coefficient of saturated clay. The previous section proposed that stress history did not affect the relationship between permeability coefficient and void ratio; thus, only the permeability coefficient of normally consolidated saturated clay was predicted in this section. The study selected the prediction equations of permeability coefficient considering the initial consolidation state proposed by Gao et al. [40], Stokes flow permeability coefficient equation, and consolidation degree permeability equation to predict the permeability coefficient of saturated clay in Luochuan, Shaanxi.

- (1) When expressed by the permeability coefficient prediction equations proposed by Gao et al. [40], there were

FIGURE 12: The relation curve between soil ultimate compression and stress difference.

$$k = k_0 \frac{1 + e_0}{e_0^2} \frac{(e_0 - C_c \lg((\sigma_0 + \Delta\sigma)/\sigma_0))^2}{1 + e_0 - C_c \lg((\sigma_0 + \Delta\sigma)/\sigma_0)}, \quad (9)$$

$$k = k_0 \frac{1 + e_0}{e_0^3} \frac{(e_0 - C_c \lg((\sigma_0 + \Delta\sigma)/\sigma_0))^3}{1 + e_0 - C_c \lg((\sigma_0 + \Delta\sigma)/\sigma_0)}. \quad (10)$$

Equations (9) and (10) were the permeability coefficient prediction equations for normally consolidated soil based on the Darcy's permeability coefficient equation and the Kozeny-Carman's permeability coefficient equation, respectively, where k_0 and e_0 are the initial permeability coefficient and void ratio of the soil, respectively

(2) When expressed by the Stokes flow permeability coefficient equation, there was

$$k = \frac{\gamma_{wz} R^2 e}{8\eta(1 + e)}, \quad (11)$$

where R is the radius of the capillary (cm); η is the dynamic viscosity coefficient of free water ($\text{g}\cdot\text{s}\cdot\text{cm}^{-2}$); γ_{wz} is the weight of free water (kN/m^3); and e is the void ratio of soil. According to the method used by Gao et al. [40], during the soil compression process, the basic physical parameters such as the dynamic viscosity coefficient of water, and the weight of free water are constant; only the permeability coefficient and void ratio are changing. Therefore, these constant

TABLE 7: Permeability coefficient at various consolidation pressures.

Consolidation stress ratio K_c	Consolidation stress (kPa)	Average permeability coefficient k (10^{-5} cm/s)
1	25	1.504
	50	1.140
	75	1.103
	100	0.850
1.3	25	0.802
	50	0.650
	75	0.615
	100	0.513
1.5	25	0.776
	50	0.604
	75	0.563
	100	0.476

physical parameters can be expressed by the initial permeability coefficient k_0 and the initial porosity ratio e_0 . Equation (11) yields

$$\frac{\gamma_{wz} R^2}{8\eta} = k_0 \frac{1 + e_0}{e_0}. \quad (12)$$

Substituting Equation (12) into Equation (11), the permeability coefficient of soil was obtained as Equation (13).

FIGURE 13: The curve of permeability coefficient of overconsolidation clay with consolidation pressure at different consolidation stress ratios.

FIGURE 14: The curve of the void ratio of overconsolidation clay with consolidation pressure at different consolidation stress ratios.

TABLE 8: Calculation results of deformation parameters of overconsolidation clay during consolidation.

Consolidation stress ratio K_c	Consolidation stress (kPa)	Sample's compression (mm)	Sample's height after compression (mm)	Void ratio e	Compression coefficient (MPa^{-1})	Compression modulus (MPa)
1	25	3.23	76.76	0.934	3.25	0.62
	50	3.31	76.68	0.932	0.08	25.00
	75	3.41	76.58	0.929	0.10	20.00
	100	3.53	76.46	0.926	0.12	16.66
1.3	25	3.51	76.48	0.927	3.53	0.57
	50	3.61	76.38	0.924	0.10	19.38
	75	3.74	76.25	0.921	0.13	15.04
	100	3.92	76.07	0.916	0.18	11.20
1.5	25	3.72	76.27	0.921	3.75	0.54
	50	3.87	76.12	0.918	0.15	13.62
	75	4.10	75.89	0.912	0.23	8.61
	100	4.57	75.42	0.900	0.48	4.22

FIGURE 15: The curve of overconsolidation clay compression with time at different consolidation stress ratios.

FIGURE 16: The curve of overconsolidation void ratio with time at different consolidation stress ratios.

TABLE 9: Stress calculations of overconsolidation clay.

Stress state	σ (kPa)	σ_{cz} (kPa)	σ_{dv} (kPa)
Overconsolidation	25	115	-90
	50		-65
	75		-40
	100		-15

$$k = k_0 \frac{1 + e_0}{e_0} \frac{e}{1 + e}. \quad (13)$$

For normally consolidated soil, it is assumed that the self-weight stress acting on the midpoint of the soil layer is σ_0 , the corresponding initial void ratio is e_0 , and the additional stress is $\Delta\sigma$; then actual stress is $(e_0 + \Delta\sigma)$, and the corresponding void ratio is

$$e = e_0 - C_c \lg \frac{\sigma_0 + \Delta\sigma}{\sigma_0}. \quad (14)$$

Submitting Equation (14) into Equation (13), the permeability coefficient equation for one-dimensional consolidation of normally consolidated soil was obtained as

$$k = k_0 \frac{1 + e_0}{e_0} \frac{e_0 - C_c \lg ((\sigma_0 + \Delta\sigma)/\sigma_0)}{1 + e_0 - C_c \lg ((\sigma_0 + \Delta\sigma)/\sigma_0)} \quad (15)$$

(3) When expressed by the consolidation degree permeability equation, there was

$$k = C_v \gamma_{wz} \frac{a_v}{1 + e}, \quad (16)$$

where C_v is the consolidation coefficient (cm^2/s); γ_{wz} is the weight of free water (kN/m^3); and a_v is the compression coefficient (MPa^{-1})

In the same way as the Stokes equation, the equation of the permeability coefficient of normally consolidated soil under one-dimensional consolidation can be obtained as

$$k = k_0 \frac{1 + e_0}{1 + e_0 - C_c \lg ((\sigma_0 + \Delta\sigma)/\sigma_0)}. \quad (17)$$

According to the measured results of one-dimensional consolidation permeability tests and Equations (9), (10), (15), and (17), the comparison between the predicted value and the measured value of the permeability coefficient of normally consolidated saturated clay can be obtained as shown in Figure 21.

FIGURE 17: The relation curve between overconsolidation clay ultimate compression and stress difference.

FIGURE 18: The curve of compression with time considering the influence of stress history.

FIGURE 19: Variation curve of the void ratio with consolidation pressure during one-dimensional consolidation of normally and overconsolidation saturated clays.

FIGURE 20: Variation curve of permeability coefficient with consolidation pressure during one-dimensional consolidation of normally and overconsolidation saturated clays.

It was found in Figure 21 that the predicted values of the permeability coefficient and the measured value were consistent with the change of the consolidation stress. The measured value of permeability coefficient was in the order of $10^{-5} \sim 10^{-6}$, while the permeability coefficient calculated by Stokes equation and consolidation degree equation was in the order of 10^{-5} , which was quite different from the measured value. The calculated values of the modified Darcy's permeability coefficient equation and

modified Kozeny-Carman's permeability coefficient equation proposed by Gao et al. [40] were close to the measured values. Besides, it can be seen from the figure that the permeability coefficient calculated by the modified Kozeny-Carman's permeability coefficient equation is almost coincided with the measured value. Therefore, it was more appropriate to use the modified Kozeny-Carman's permeability coefficient equation to predict the saturated clay permeability coefficient in Luochuan.

FIGURE 21: The comparison of the measured and predicted values of one-dimensional consolidation permeability coefficient of normally consolidated soil.

5. Conclusion

In this study, a series of one-dimensional consolidation permeability tests under different consolidation stress ratio K_c and different stress history were carried out. Moreover, a model for predicting the permeability coefficient of the saturated clay in Luochuan, Shaanxi, was recommended by comparing several models. The following conclusions can be drawn:

- (1) The permeability coefficient and void ratio of normally and overconsolidation saturated clay decreased with the increase of K_c , and the amount of sample's compression increased with the increase of K_c . When $K_c = 1, 1.3$, and 1.5 , the corresponding amounts of sample's final compression of normally consolidated soil were 4.43, 5.52, and 7.17 mm, and those of overconsolidated soil were 3.53, 3.92, and 4.57 mm
- (2) Fitting the difference σ_{dv} between the consolidation pressure σ and the soil self-weight stress σ_{cz} with the amount of final sample's compression s_f , it was concluded that the stress difference σ_{dv} between the consolidation pressure and the self-weight stress of the soil layer and the amount of final compression under each consolidation pressure had a good linear relationship. After knowing the self-weight stress of the soil, the amount of final sample's compression can be predicted more accurately, which had an important application value for the engineering

- (3) The compressibility and permeability of the soil in the overconsolidation stage are much smaller than that in the normally consolidation state under the same consolidation stress ratio but different consolidation states. Meanwhile, the amount of compression of overconsolidated saturated clay was less than that of normally consolidated saturated clay during the same consolidation time; that is, the amount of final sample's compression of overconsolidated soil was 3.53 mm, while that of normally consolidated soil was 4.43 mm under the condition of $K_c = 1$
- (4) The permeability coefficient of Luochuan saturated clay in Shaanxi was predicted using the modified Darcy's permeability coefficient equation, the modified Kozeny-Carman's permeability coefficient equation, the Stokes flow permeability coefficient equation, and the consolidation degree permeability equation. The results showed that the calculated permeability coefficient of the modified Kozeny-Carman's permeability coefficient equation was in good agreement with the measured value. Therefore, it was recommended to use the modified Kozeny-Carman's equation to predict the permeability coefficient of Luochuan saturated clay during one-dimensional consolidation

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 they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Authors' Contributions

Le Zhang is responsible for the investigation, testing, conceptualization, software, and writing of the original draft. Faning Dang is involved in the conceptualization and methodology. Jun Gao is also involved in the methodology and analyzed the data. Jiulong Ding performed the tests.

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