Rock and Soil Mechanics

Volume 42 | Issue 4 Article 5

8-25-2021

Experimental study on the one-dimensional nonlinear consolidation and seepage of saturated clay considering stress history under ramp loading

Le ZHANG

Fa-ning DANG
dangfn@mail.xaut.edu.cn

Jun GAO

Jiu-long DING

Custom Citation

ZHANG Le, DANG Fa-ning, GAO Jun, DING Jiu-long. Experimental study on the one-dimensional nonlinear consolidation and seepage of saturated clay considering stress history under ramp loading[J]. Rock and Soil Mechanics, 2021, 42(4): 1078-1087.

Rock and Soil Mechanics 2021 42(4): 1078−1087 ISSN 1000−7598
https: //doi.org/10.16285/j.rsm.2020.6227 rocksoilmech.researchcommons.org/journal

Experimental study on the one-dimensional nonlinear consolidation and seepage of saturated clay considering stress history under ramp loading

ZHANG Le, DANG Fa-ning, GAO Jun, DING Jiu-long
State Key Laboratory of Eco-hydraulics in Northwest Arid Region, Xi'an University of Technology, Xi'an, Shaanxi 710048, China

Abstract: To study the permeability of saturated clays with different stress histories under ramp loading, the one-dimensional consolidation and seepage tests are carried out using modified GDS triaxial apparatus, and the change of the permeability coefficient of remolded silty clay in Luochuan, Shaanxi Province is studied. The results show that permeability coefficients of normally and over-consolidated saturated clays decrease nonlinearly with the increase of the consolidation stress, and the void ratios of these two soils are consistent with the trend of change of the permeability coefficients with the consolidation stress. The compressibility and permeability of soil in the overconsolidated state are much smaller than that in the normally consolidated state. The permeability coefficient and void ratio decrease with the increase of the osmotic pressure difference under a constant consolidation stress. Finally, the measured values are compared with the permeability coefficients calculated by the modified Darcy permeability coefficient formula, modified Kozeny-Carman formula, Stokes porosity permeability coefficient formula and degree of consolidation and seepage formula in Ref. [12]. The results reveal that the permeability coefficients calculated using the modified Kozeny-Carman formula are in good agreement with the measured values. Therefore, the modified Kozeny-Carman formula is recommended to predict the permeability coefficient of Luochuan saturated clay.

Keywords: ramp loading; normally consolidated saturated clay; overconsolidated saturated clay; permeability coefficient; void ratio; nonlinear consolidation and seepage

1 Introduction

With the continuous and rapid development of China's national economy, many sections of the expressway have to be built on soft foundations (saturated clay). There are more and more pavement disasters caused by settlement. Therefore, being able to predict the subsequent load induced settlement on the soft foundation accurately becomes an important and urgent issue. As early as 1925, Terzaghi established the theory of unidirectional consolidation theory of saturated soils and obtained mathematical solutions for certain initial and boundary conditions. This law is still widely used. In Terzaghi's consolidation theory, it is first assumed that the consolidation coefficient C_v of the soil is constant, but in fact, when calculating the consolidation coefficient C_v, the permeability coefficient k, void ratio e, and compression coefficient a_v will all vary with the change of stress. Furthermore, Terzaghi also assumed that an external load was applied to the soil instantaneously. However, in practical engineering projects, the external load is often linear or cyclic load, etc. [1]

Over the years, the one-dimensional consolidation theory has also made great progress, and many scholars have revised the basic hypothesis of Terzaghi. Davis et al.[2] noticed that the compressibility and permeability coefficient changed nonlinearly with the effective stress, and thus obtained the analytical solution of the one-dimensional nonlinear consolidation equation. Based on Davis's findings, Xie et al.[3] proposed an analytical solution for one-dimensional nonlinear consolidation of double-layered soils. Xia et al.[4] and Hu et al.[5] assumed that the compressibility and seepage of the soil during the consolidation process were proportional to the change and deduced an analytical solution to one-dimensional nonlinear consolidation of saturated soft soil layered foundations. Li et al.[6] assumed that both the compressibility and permeability coefficients changed and established a one-dimensional nonlinear consolidation equation of clay during linear loading. Shi et al.[7] studied the consolidation and deformation laws of soft soil foundation under linear unloading by using the method of Terzaghi's one-dimensional consolidation theory and effective stress principle. Lee et al.[8] proposed a more explicit analytical solution for the one-dimensional consolidation of layered soils based on findings from Schiffman et al.[9]. They also pointed

Received: 16 August 2020 Revised: 29 September 2020
This work was supported by the National Natural Science Foundation of China (51979225, 51679199), the Special Funds for Public Industry Research Projects of the Ministry of Water Resources (201501034-04) and the Key Laboratory for Science and Technology Coordination & Innovation Projects of Shaanxi Province (2014SZS15-Z01).
First author: ZHANG Le, female, born in 1993, PhD candidate, specializing in clay soil mechanics and retaining wall earth pressure and geotechnical test. E-mail: lezhang_1993@163.com
Corresponding author: DANG Fa-ning, male, born in 1962, PhD, Professor, PhD supervisor, mainly engaged in the teaching and research of numerical analysis and computational mechanics of geotechnical engineering. E-mail: dangfn@mail.xaut.edu.cn

out that the influence of soil permeability coefficient and volume compressibility coefficient on consolidation was different, and the changes of the two could not be reflected in the consolidation coefficient of soil. Sun et al.[10] considered the consolidation coefficient as a variable and improved the Baron's consolidation theory on this basis. In addition, some scholars have also taken into account the influence of stress history on the seepage of one-dimensional consolidation of soil[11−17] and proposed a one-dimensional nonlinear consolidation equation that considers the stress history. In order to study the change law of permeability coefficient and compressibility coefficient in the process of one-dimensional consolidation and seepage, some scholars have conducted laboratory experiments on soft clay. Zou et al.[18] conducted experiments on the change law of soil permeability coefficient under different head pressures and found that the permeability coefficient of soft soil decreases with the decrease of the void ratio. Qi[19] conducted a one-dimensional consolidation and seepage test of over-consolidated soil on Xiaoshan clay, and the results showed that the compressibility and seepage of over-consolidated soil change nonlinearly with the consolidation pressure during the consolidation process.

Although there have been a lot of theoretical studies on one-dimensional nonlinear consolidation seepage, most of them assume that the consolidation coefficient is constant or that the permeability coefficient has nothing to do with the hydraulic gradient, and there are few systematic laboratory tests to study the one-dimensional consolidation and seepage theory of saturated clay under different initial consolidation states. In view of this, this paper uses the GDS triaxial apparatus to carry out a series of one-dimensional consolidation permeability tests on the Luochuan loess (silty clay) in Shaanxi under linear loading conditions, considering the stress history. The nonlinear seepage laws of silty clay under different consolidation stresses and multiple sets of water heads were studied. On this basis, the applicability of the permeability coefficient prediction formula proposed in literature [12] considering the influence of the initial consolidation state is verified.

2 Test plan and method

2.1 Sample preparation

The test soil is Q_3 loess from Luochuan, Shaanxi, with a depth of 7−8 m. The physical properties of the intact loess are provided in Table 1.

Table 1 Physical properties of intact Q_3 loess

Specific gravity G_s	Water content w /%	Dry density ρ_d /(g • cm^{-3})	Liquid limit w_L /%	Plastic limit w_P /%	Particle size distribution /%		
					>0.075 mm	0.075−0.005 mm	<0.005 mm
2.70	13.3	1.34	28.4	19.2	4	73	23

The soil samples used in this paper are all saturated remolded soil samples. The preparation process is as follows:

(1) After crushing the air-dried soil, pass it through a 2 mm sieve, and measure its air-dried water content after it is fully mixed. Then the soil is prepared to a water content that can be used in the sample preparation process, and the water content of the sample in this paper is 11.6%. Put the prepared soil in a plastic bag and seal it for 24 hours.

(2) Measure the water content of the prepared soil sample and calculate the mass of the soil required for sample preparation. The sample diameter is 3.91 cm and the height is 8 cm. This article adopts the compaction method for sample preparation. When preparing the sample, the sample is evenly divided into 5 layers. In order to make better contact between the sample layers, the contact surface is roughened before each compaction. Press the sample 5 times continuously to reach the target dry density. According to the e-lg$p$ curve, the preconsolidation pressure σ_c of Luochuan silty clay is 115 kPa, and it is considered that the soil is in a normal consolidation state at this time. As a result, the reshaped soil sample is consolidated under a pressure of 115 kPa. After the consolidation is stable, the sample is considered to be in a normally consolidated state.

2.2 Test apparatus

The test instrument is a GDS triaxial instrument made in the UK, as shown in Fig. 1. The GDS triaxial instrument can not only perform traditional consolidation tests, but also conduct seepage tests under the condition of keeping the consolidation stress constant, and directly determine the permeability coefficient when consolidation is completed under each level of load. Thereby, the defects of the existing test instruments in the consolidation and seepage test are solved. In order to carry out consolidation and seepage tests on saturated remolded soil samples, the GDS triaxial apparatus was modified. By redesigning and processing a set of saturated soil base and dual-channel sample cap, the connection between the back-pressure controller and the base of the sample and the sample cap is achieved. This ensures that the upper and lower ends of the sample are saturated and consolidated. Water drained is merged into the back pressure controller, which can accurately measure the volumetric change of the test sample.

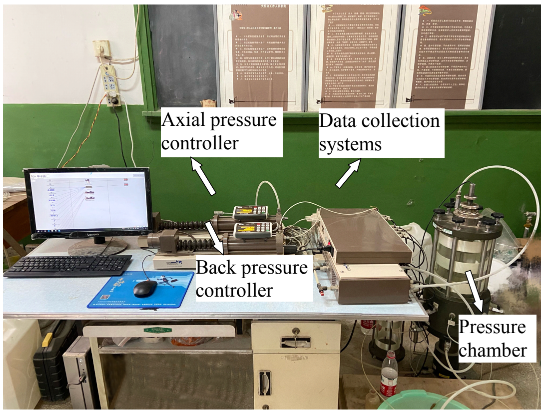

Fig. 1 GDS triaxial apparatus

2.3 Test plan and method

In order to study the one-dimensional consolidation and seepage law of saturated clay considering the initial consolidation state, one-dimensional consolidation seepage tests were conducted. In order to increase the degree of saturation of the sample, before loading the sample, the sample was vacuum-saturated first, and then the GDS was used for back pressure saturation. When the back pressure was exerted to saturate sample, the back pressure and confining pressure were applied on the sample simultaneously, and the back pressure always was kept to be 2−5 kPa lower than the confining pressure (in this test, the back pressure was set to be 5 kPa lower than the confining pressure). The purpose of this is to prevent the sample from swelling. When the ratio of the increase in pore water pressure to the increase in confining pressure was greater than 0.98, the sample was regarded as to be saturated. After the sample was saturated, the GDS advanced loading module was utilized to perform consolidation tests on the normal consolidated soil at 140, 165, 190, and 215 kPa, and perform consolidation tests on the overconsolidated soil at 25, 50, 75 and 100 kPa. Figure 2 is a schematic diagram of multi-stage ramp loading. In the figure, $t_{j1}-t_{j4}$ represent the time required to reach each level of load. In this paper, $t_{j1}=t_{j2}=t_{j3}=t_{j4}=30$ min, and t_1-t_4 represent the total duration of each level of consolidation stress. This article sets $t_1=t_2=t_3=t_4=24$ h. After the completion of each group of consolidation tests, keeping the consolidation stress constant, a seepage test was conducted under multi-level heads, and the seepage time of each level was 1 h. Each sample under the initial consolidation state was subjected to 3 sets of parallel tests, and a total of 9 sets of tests were carried out. The specific test plan is shown in Table 2 using normal consolidated soil as an example.

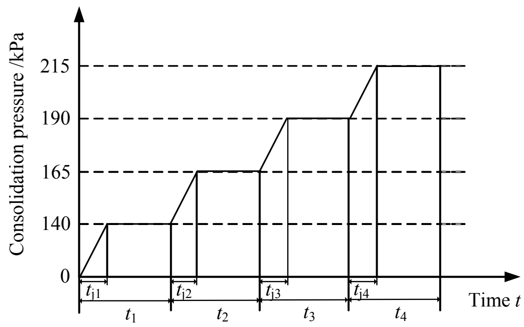

Fig. 2 Schematic diagram of multi-stage ramp loading

Table 2 One-dimensional consolidation and seepage joint test program

Procedure	Test type	Confining pressure /kPa	Back pressure /kPa	Base pressure /kPa	Remarks
1	Consolidation	140	0	—	Consolidation stress 140 kPa
2	Seepage	140	20	0	Osmotic pressure difference 20 kPa
3	Seepage	140	40	0	Osmotic pressure difference 40 kPa
4	Seepage	140	60	0	Osmotic pressure difference 60 kPa
5	Consolidation	165	0	—	Consolidation stress 165 kPa
6	Seepage	165	20	0	Osmotic pressure difference 20 kPa
7	Seepage	165	40	0	Osmotic pressure difference 40 kPa
8	Seepage	165	60	0	Osmotic pressure difference 60 kPa
9	Seepage	165	80	0	Osmotic pressure difference 80 kPa
10	Consolidation	190	0	—	Consolidation stress 190 kPa
11	Seepage	190	20	0	Osmotic pressure difference 20 kPa
12	Seepage	190	40	0	Osmotic pressure difference 40 kPa
13	Seepage	190	60	0	Osmotic pressure difference 60 kPa
14	Seepage	190	80	0	Osmotic pressure difference 80 kPa
15	Consolidation	215	0	—	Consolidation stress 215 kPa
16	Seepage	215	20	0	Osmotic pressure difference 20 kPa
17	Seepage	215	40	0	Osmotic pressure difference 40 kPa
18	Seepage	215	60	0	Osmotic pressure difference 60 kPa
19	Seepage	215	80	0	Osmotic pressure difference 80 kPa
20	Seepage	215	100	0	Osmotic pressure difference 100 kPa

In the consolidation phase, the consolidation stress of 140 kPa was taken as an example to test. Open the upper and lower back pressure valves that communicate with the back pressure, and close the lower valve that communicates with the outside world. The confining pressure was set to 140 kPa and the back pressure was set to 0 kPa, respectively, and the consolidation test was carried out for 24 hours. When the pore pressure dissipated completely, the axial deformation was basically unchanged, and the volumetric deformation remained stable, the consolidation was considered complete.

In the seepage stage, take the first stage (consolidation stress of 140 kPa) as an example. Close the lower back pressure valve that communicates with the back pressure, open the lower interface that communicates with the outside world and the back pressure upper valve that communicates with the back pressure, and thus form a seepage path from top to bottom using the head difference caused by the difference between the back pressure and base pressure. Under the condition that the consolidation stress is kept constant at 140 kPa, the osmotic pressure difference is set to 20, 40, and 60 kPa for infiltration, respectively. The selection of osmotic pressure difference refers to [19]. Figure 3 shows how the confining pressure, back pressure and bottom pressure of the consolidation and seepage test are applied.

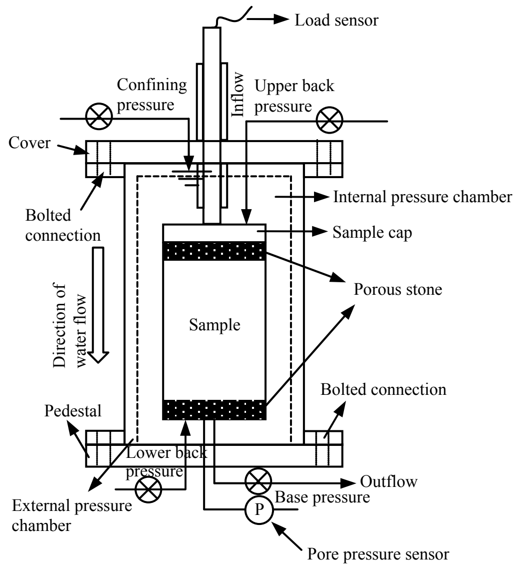

Fig. 3 Schematic diagram of pressure application for consolidation and seepage tests

3 Analysis and verification of test results

3.1 Normally consolidated soil

3.1.1 Relationship between k, e and σ

Since underconsolidated soils are essentially normally consolidated soils (NC soils), this paper only uses normally consolidated and overconsolidated clays (OC clays) to explore the influence of stress history on the seepage of saturated clays.

Under the condition of keeping the consolidation stress σ constant, the permeability coefficient k of normally consolidated clay at each level of osmotic pressure difference is measured, as listed in Table 3.

Table 3 Measured values of permeability coefficient of normally consolidated clay under various consolidation pressures

Consolidation stress /kPa	Osmotic pressure difference /kPa	Permeability coefficient /(10^{-5} cm • s^{-1})	Average permeability coefficient /(10^{-5} cm • s^{-1})
	20	0.96	
140	40	0.83	0.86
	60	0.78	
	20	0.85	
165	40	0.74	0.74
	60	0.70	
	80	0.66	
	20	0.64	
195	40	0.56	0.56
	60	0.54	
	80	0.51	
	20	0.57	
	40	0.50	
215	60	0.48	0.49
	80	0.46	
	100	0.45	

According to Table 3, the relationships of permeability coefficient k and void ratio e with the change of consolidation stress are plotted in Fig. 4 and Fig. 5, respectively. As seen from Fig. 4, the permeability coefficient k shows a nonlinear descending trend with the increase of consolidation stress σ. Before the consolidation stress reaches 165 kPa, the permeability coefficient decreases faster, and then the permeability coefficient decreases slowly and gradually stabilizes. According to Fig. 5, it can be observed that the void ratio e decreases with the increase of the consolidation stress. When the consolidation stress is relatively small, the reduction rate in the void ratio is larger, and with the increase of the consolidation stress, the reduction rate gradually decreases and tends to be gentle. By linearly fitting the e-lg σ curve, the compressibility index of the sample C_c=0.88 can be obtained. Due to space limitations, the e-lg σ fitting curve will not be shown in detail here.

Comparing Fig. 4 and Fig. 5, it is found that the trends of variation in permeability coefficient k and void ratio e with the consolidation stress σ are basically the same, because the permeability coefficient is mainly affected by the void ratio.

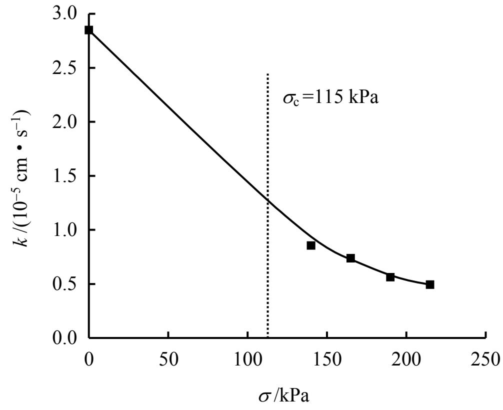

Fig. 4 Curve of permeability coefficient changing with consolidation stress for normally consolidated clay

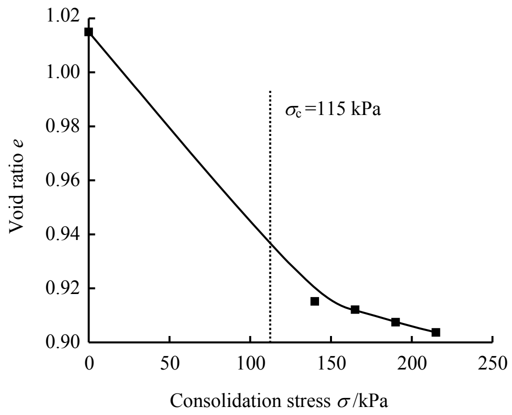

Fig. 5 Curve of void ratio changing with consolidation stress for normally consolidated clay

3.1.2 Relationship between k and e and osmotic pressure difference

To examine the influence of osmotic pressure difference (hydraulic gradient) on the seepage of NC clay, curves of permeability coefficient k and void ratio e changing with osmotic pressure difference are depicted according to data in Table 2 and they are displayed in Figs. 6 and 7. As seen from figures, under different osmotic pressure differences with a certain consolidation stress, the permeability coefficient and void ratio both show a decreasing trend with the increase of osmotic pressure difference. Song et al.[20] also studied the influence of hydraulic gradient on the seepage of clay, and got the same conclusion as this paper, that is, the void ratio and permeability coefficient both decrease with the increase of hydraulic gradient. This is because the increase in osmotic pressure difference causes the hydraulic gradient to gradually increase, resulting in a decreasing trend of the void ratio of the sample. This is what Li[21] said, under the action of osmotic pressure difference, seepage is formed inside the soil; the seepage force acts on the particles in the soil framework cause the soil to compress, which leads to a decrease in the coefficient of permeability of the soil. In addition, the reduction range of both decreases with the increase of consolidation stress.

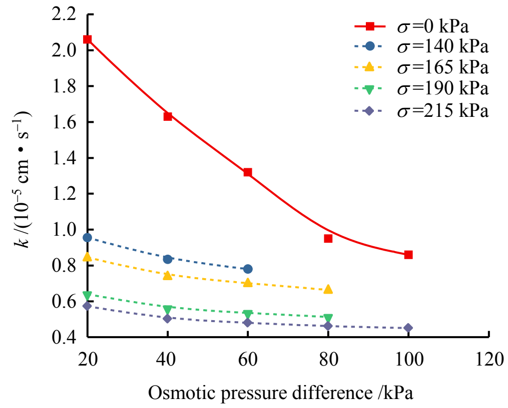

Fig. 6 Curves of permeability coefficient k changing with osmotic pressure difference for NC clay

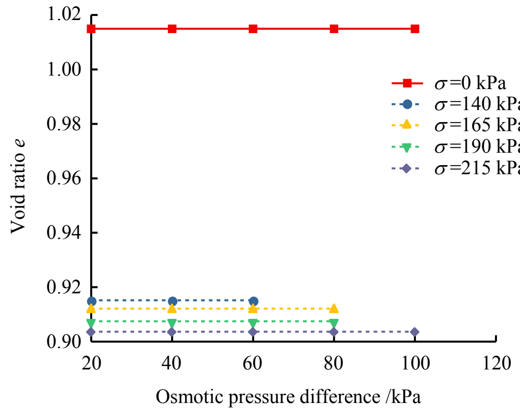

Fig. 7 Curves of void ratio e changing with osmotic pressure difference for NC clay

3.1.3 Relationship between k and e

From the test results in Table 3, the values of permeability coefficient k are plotted against the void ratio e for different consolidation stresses, as shown in Fig. 8.

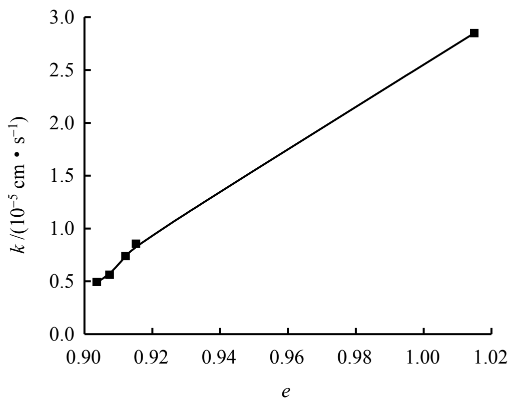

Fig. 8 Curve of permeability coefficient k changing with void ratio e for NC clay

Figure 8 shows that the void ratio e has a great influence on the permeability coefficient k. The permeability coefficient k decreases correspondingly from 0.856×10^{-5} cm/s to 0.493×10^{-5} cm/s as the void ratio e of the sample decreases from 1.015 to 0.903. After investigating the reason, the author believes that the soil sample is compressed during the consolidation stage, and the soil becomes denser, which leads to a decrease in the void ratio e of the soil. Therefore, the corresponding permeability coefficient is also decreasing. The permeability coefficient k changes nonlinearly with the change of the void ratio e.

3.2 Overconsolidated soil

3.2.1 Relationship between k, e and σ

Under the condition of keeping the consolidation stress σ constant, the measurements of permeability coefficient k of overconsolidated clay at each level of osmotic pressure difference are listed in Table 4.

Table 4 Measured values of permeability coefficient of overconsolidated clay under various consolidation pressures

Consolidation pressure /kPa	Osmotic pressure difference /kPa	Permeability coefficient /(10^{-5} cm • s^{-1})	Average permeability coefficient /(10^{-5} cm • s^{-1})
25	5	1.72	1.51
	10	1.29	
50	10	1.22	1.14
	20	1.06	
75	10	1.18	1.10
	20	1.03	
100	20	0.91	0.85
	40	0.80	

According to data in Table 4, the relationships of k, e and σ with consolidation stress are depicted in Figs. 9 and 10.

It can be seen from Fig. 9 that the permeability coefficient k of the overconsolidated saturated clay decreases with the increase of the consolidation stress σ and shows an obvious non-linear trend. When $\sigma\leqslant 50$ kPa, the permeability coefficient decreases faster, and then gradually slows down. The range of the corresponding permeability coefficient k under the action of consolidation stress ranges from 1.51×10^{-5} to 0.85×10^{-5} cm/s. Comparing Fig. 4 and Fig. 9, it can be seen that the permeability coefficient k of normally consolidated and overconsolidated saturated clays varies similar as subjected to the consolidation stress.

From Fig. 10, it is observed that the void ratio e of overconsolidated saturated clay decreases with the increase of consolidation pressure σ. This represents that after the soil is subjected to consolidation stress, the volume of pores in the soil compressed gradually with the drainage of the pore water.

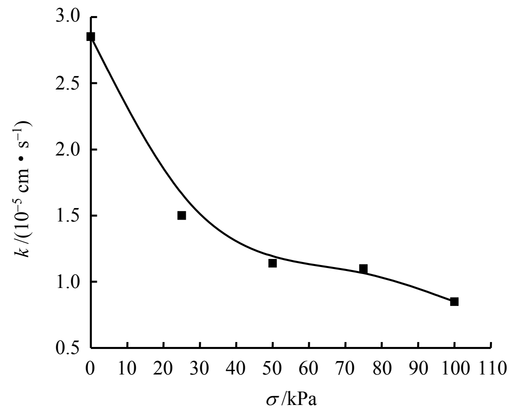

Fig. 9 Curve of permeability coefficient changing with consolidation stress for OC clay

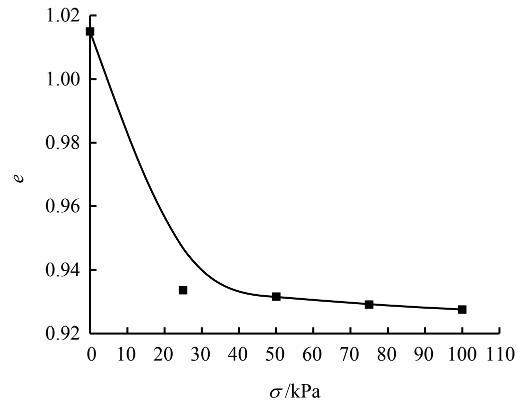

Fig. 10 Curve of void ratio changing with consolidation stress for OC clay

3.2.2 Relationship between k and e and osmotic pressure difference

Figures 11 and 12 show curves of permeability coefficient k and void ratio e of OC saturated clay changing against osmotic pressure difference, respectively. It is seen from Fig. 11 that as the osmotic pressure difference increases, when the soil is not subjected to consolidation pressure, the permeability coefficient k decreases faster. As the consolidation stress increases, the magnitude of decrease of permeability coefficient with an increase in the osmotic pressure difference is significantly reduced.

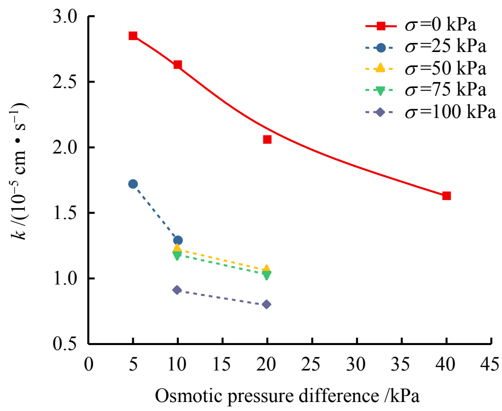

Fig. 11 Curves of permeability coefficient changing with osmotic pressure difference for OC clay

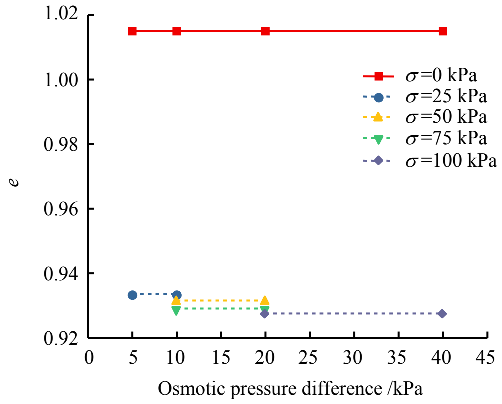

Fig. 12 Curves of void ratio changing with osmotic pressure difference for OC clay

It can be seen from Fig.12 that the void ratio e decreases slightly with the increase of the osmotic pressure difference. Under the same osmotic pressure difference, a greater consolidation stress leads to a smaller the void ratio. The author believes that the osmotic pressure causes the soil to compress, which in turn causes the soil void ratio to decrease.

It can be seen from Figures 11 and 12 that the permeability coefficient k and void ratio e change with the osmotic pressure difference in the same way. Both of them decrease with the increase of osmotic pressure difference. This further confirms the nonlinearity of saturated clay in one-dimensional consolidation and seepage.

3.2.3 Relationship between k and void ratio e

In order to analyze the relationship between the permeability coefficient k and the void ratio e of OC clay in one-dimensional consolidation and seepage, the e–k curve is drawn according to Table 4 as shown in Fig. 13.

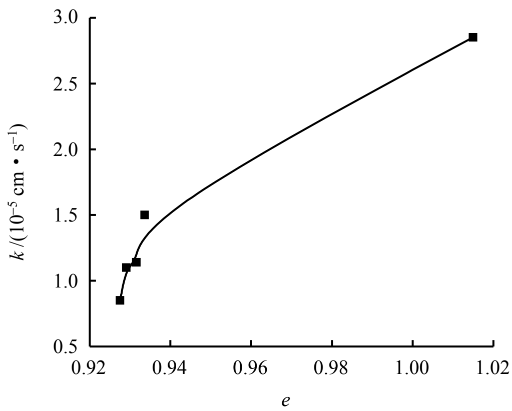

Fig. 13 Curve of permeability coefficient k changing with void ratio e for OC clay

Figure 13 illustrates that the permeability coefficient k changes nonlinearly with the change of the void ratio e. The permeability coefficient k decreases with the decrease of the void ratio e, which is consistent with the trend of the permeability coefficient k of the NC clay changing with the void ratio e, and will not be repeated here.

3.3 Influence of stress history

3.3.1 Relationships between k, e and σ of NC and OC clays

In order to investigate the influence of stress history on the change of permeability coefficient k and void ratio e with consolidation stress σ in the process of one-dimensional consolidation and seepage of saturated clay, this paper selects the same consolidation stress increment to carry out one-dimensional consolidation-seepage tests of normally-and over-consolidated clays. According to Tables 3 and 4, the relationships of k−σ and e−σ for NC and OC saturated clays are plotted as shown in Figs. 14 and 15.

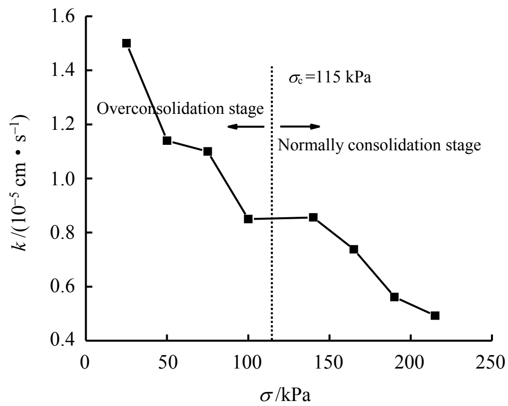

Fig. 14 Curve of permeability coefficient of NC and OC saturated clays changing with consolidation stress

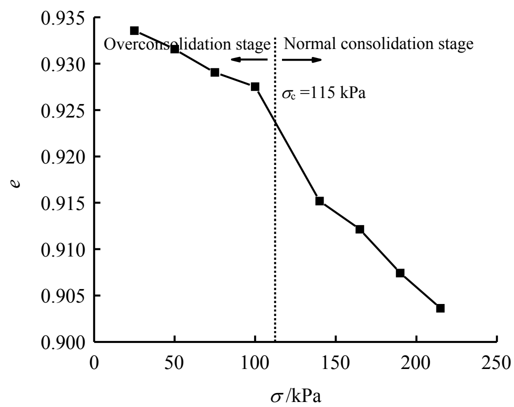

Fig. 15 Curve of void ratio of NCand OC saturated clays changing with consolidation stress

It can be seen from Figs. 14 and 15 that the overconsolidation stage is before the preconsolidation stage where preconsolidation pressure σ_c = 115 kPa, and the normal consolidation stage is afterwards. The permeability coefficient k and void ratio e of NC saturated clay and OC saturated clay decrease monotonously with the increase of consolidation stress. The permeability coefficients of normally NC and OC clays have a one-to-one relationship with the void ratio, that is, the permeability coefficient decreases as the void ratio decreases.

3.3.2 Relationship between *k* and void ratio *e*

Figure 16 shows the relationship between the permeability coefficient *k* and the void ratio *e* in the process of one-dimensional consolidation and seepage of normally consolidated and overconsolidated saturated clays.

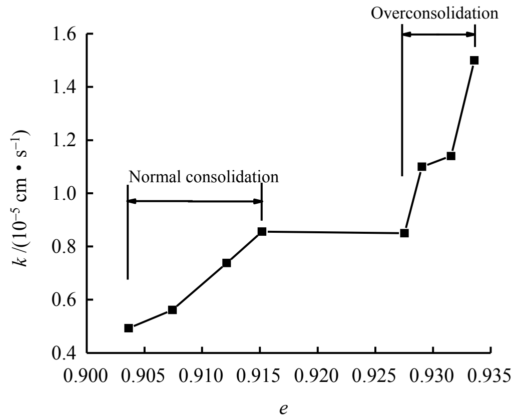

Fig. 16 Curve of permeability coefficients of NC and OC saturated clays changing with void ratio

It is found that the *k* values of NC and OC saturated clays in one-dimensional consolidation seepage decrease nonlinearly with the decrease of values of *e*. The almost straight line in the figure is because one-dimensional consolidation and seepage tests were carried out on NC and OC clays in this study. The void ratio of NC saturated clay decreases from 1.014 to 0.904, with a reduction of 10.85%. At the same time, the permeability coefficient decreases from 0.856×10^{-5} cm/s to 0.493×10^{-5} cm/s, with a reduction of 42.41%. The void ratio of OC saturated clay decreases from 0.934 to 0.928, with a reduction of 0.648%. Meanwhile, the permeability coefficient decreases from 1.049×10^{-5} cm/s to 0.850×10^{-5} cm/s, with a reduction of 43.33%. In summary, the compressibility and seepage of the soil in the overconsolidation stage are much smaller than those in the normal consolidation stage. It can also be seen from the figure that the stress history only affects the value of the permeability coefficient but has no effect on the relationship between the permeability coefficient and the void ratio. The literature [22] also confirms that the stress history has no effect on the relationship between the permeability coefficient and the void ratio, which is also shown in this research.

4 Model prediction

Usually in engineering practice, it is particularly important to predict the permeability coefficient of saturated clay accurately and quickly. Section 3 points out that the stress history has no effect on the relationship between the permeability coefficient and the void ratio, so this section only predicts the permeability coefficient of normally consolidated saturated clay. This paper selects the prediction formula for permeability coefficient considering the influence of the initial consolidation state, Stokes pore flow permeability coefficient formula and the degree of consolidation-seepage formula proposed by literature [12] to predict the permeability coefficient of Luochuan saturated clay in Shaanxi.

(1) When expressed by the permeability coefficient prediction formula proposed in literature [12], we can get

$$k = k_0 \frac{1+e_0}{e_0^2} \frac{\left(e_0 - C_c \lg \dfrac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)^2}{1 + e_0 - C_c \lg \dfrac{\sigma_0 + \Delta\sigma}{\sigma_0}} \tag{1}$$

$$k = k_0 \frac{1+e_0}{e_0^3} \frac{\left(e_0 - C_c \lg \dfrac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)^3}{1 + e_0 - C_c \lg \dfrac{\sigma_0 + \Delta\sigma}{\sigma_0}} \tag{2}$$

where k_0 and e_0 are the initial permeability coefficient and initial void ratio of the soil, respectively.

Equations (1) and (2) are the permeability coefficient prediction formulas of normally consolidated soil based on the Darcy permeability coefficient formula and the Kozeny-Caman permeability coefficient formula, respectively.

(2) When expressed by the Stokes pore flow permeability coefficient formula, we can get

$$k = \frac{\gamma_{wz} R^2 e}{8\eta(1+e)} \tag{3}$$

where *R* is the radius of the capillary tube (cm); η is the dynamic viscosity coefficient of free water (g • s • cm^{-2}); γ_{wz} is the unit weight of free water; *e* is the void ratio of the soil. Using the method proposed in literature [12], in the process of soil compression, the basic physical parameters such as the dynamic viscosity coefficient of water and the unit weight of free water are all constants, and only the permeability coefficient and void ratio are changing. Therefore, we can use the initial permeability coefficient k_0 and the initial void ratio e_0 to represent these constant physical parameters. Then formula (3) becomes

$$\frac{\gamma_{wz} R^2}{8\eta} = k_0 \frac{1+e_0}{e_0} \tag{4}$$

Substituting formula (4) into formula (3) gives the permeability coefficient of the soil:

$$k = k_0 \frac{1+e_0}{e_0}\frac{e}{1+e} \tag{5}$$

For a normally consolidated soil, assuming 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 on this point is $\Delta\sigma$, then the actual stress is ($\sigma_0+\Delta\sigma$), and the corresponding void ratio e is

$$e = e_0 - C_c \lg\frac{\sigma_0+\Delta\sigma}{\sigma_0} \tag{6}$$

By substituting formula (6) into formula (5), the permeability coefficient formula of normally consolidated soil under one-dimensional consolidation and seepage is obtained as

$$k = k_0 \frac{1+e_0}{e_0}\frac{e_0 - C_c \lg\frac{\sigma_0+\Delta\sigma}{\sigma_0}}{1+e_0 - C_c \lg\frac{\sigma_0+\Delta\sigma}{\sigma_0}} \tag{7}$$

(3) When expressed using the degree of consolidation and seepage formula, we have

$$k = C_v\gamma_{wz}\frac{a_v}{1+e} \tag{8}$$

In the same way as the Stokes pore flow permeability coefficient formula, the permeability coefficient formula for one-dimensional consolidation and seepage of normally consolidated soil can be obtained (see the following formula), which will not be repeated here.

$$k = k_0 \frac{1+e_0}{1+e_0 - C_c \lg\frac{\sigma_0+\Delta\sigma}{\sigma_0}} \tag{9}$$

According to the measured data and formulas (1), (2), (7), and (9) of the one-dimensional consolidation seepage test, the predicted value of the permeability coefficient of the normally consolidated saturated clay can be compared with the actual measured value as shown in Fig. 17.

By observing Fig. 17, it is found that changes in predicted values and measured values of the permeability coefficients of the three samples with the consolidation pressure are consistent. The measured values of permeability coefficient are in the order of 10^{-5}–10^{-6}, while the permeability coefficients calculated using the modified Darcy permeability formula, Stokes pore flow permeability formula, and degree of consolidation and seepage formula are all in the order of 10^{-5}, which is quite different from the measured value. The predicted value calculated using the modified Kozeny-Carman permeability coefficient formula proposed in literature [12] is closer to the actual measured value. Therefore, it is more appropriate to use the modified Kozeny-Carman permeability coefficient formula to predict the permeability coefficient of saturated clay in this paper.

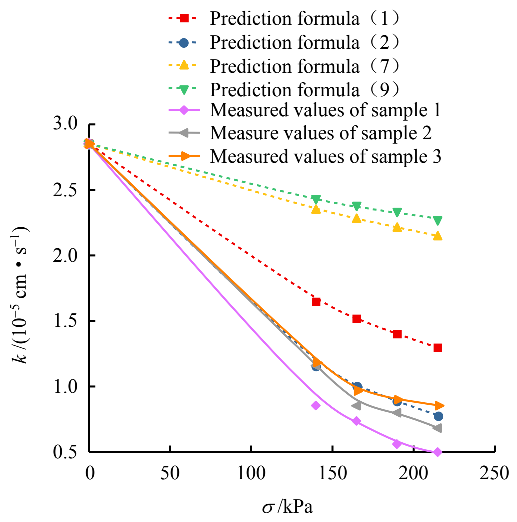

Fig. 17 Curves of permeability coefficient of NC saturated clay changing with consolidation stress

5 Conclusion

The improved GDS triaxial instrument was used to conduct one-dimensional consolidation−seepage tests on saturated clay samples under different stress histories. Some conclusions are drawn as follows:

(1) The one-dimensional consolidation-seepage test was used to determine the permeability coefficients of normally consolidated and overconsolidated saturated clays during the one-dimensional consolidation and seepage process. The permeability coefficient and void ratio of saturated clay under the two stress histories both decrease with the increase of consolidation pressure.

(2) The compressibility and seepage of the soil in the overconsolidated state are much smaller than those in the normal consolidated state. The relationship between the permeability coefficient and the void ratio has nothing to do with the stress history. It shows that the permeability coefficient of the same kind of soil is only related to the void ratio.

(3) The modified Darcy permeability formula, the modified Kozeny-Carman permeability coefficient formula, the Stokes pore flow permeability coefficient formula and the degree of consolidation and seepage formula were used to calculate the permeability coefficient of the saturated clay in Luochuan, Shaanxi. The results show that the permeability coefficient calculated using the modified Kozeny-Carman permeability coefficient formula is in good agreement with the measurements. Therefore, it is recommended to use the modified Kozeny-Carman formula to predict the permeability coefficient

of the Luochuan saturated clay in Shaanxi Province during the one-dimensional consolidation seepage process.

References

[1] MA B H, HU Z Y, LI Z, et al. Finite difference method for the one-dimensional non-linear consolidation of soft ground under uniform load[J]. Frontiers in Earth Science, 2020, 8: 111-1-111-9.

[2] DAVIS E H, RAYMOND G P. A non-linear theory of consolidation[J]. Geotechnique, 1965, 15(2): 161−173.

[3] XIE K H, XIE X Y, JIANG W. A study on one-dimensional nonlinear consolidation of double-layered soil[J]. Computers and Geotechnics, 2002, 29(2): 151−168.

[4] XIA Chang-qing, HU An-feng, CUI Jun, et al. Analytical solutions for one-dimensional nonlinear consolidation of saturated soft layered soils[J]. Rock and Soil Mechanics, 2018, 39(8): 2859−2864.

[5] HU An-feng, ZHOU Yu-shan, CHEN Yuan, et al. Semi-analytical solutions for one-dimensional nonlinear large strain consolidation of structured soft clay[J]. Rock and Soil Mechanics, 2020, 41(8): 2583−2591.

[6] LI C X, HUANG J S, WU L Z, et al. Approximate analytical solutions for one-dimensional consolidation of a clay layer with variable compressibility and permeability under a ramp loading[J]. International Journal of Geomechanics, 2018, 18(11): 06018032.

[7] SHI Xu-chao, SUN Yun-de. Analysis of the evolution of excess pore water pressure in soft soil under linear unloading[J]. Rock and Soil Mechanics, 2020, 41(4): 1333−1338.

[8] LEE P K K, XIE K H, CHEUNG Y K. A study on one-dimensional consolidation of layered systems[J]. International Journal for Numerical and Analytical Methods in Geomechanics, 1992, 16(11): 815−831.

[9] SCHIFFMAN R L, STEIN J R. One-dimensional consolidation of layered systems[J]. Mechanics and Foundation Division Journal, ASCE, 1970, 96(4): 1499−1504.

[10] SUN Li-qiang, YAN Shu-wang, QIU Chang-lin. Consolidation theory considering coefficent of consolidation of hydraulic fill as variable[J]. Chinese Journal of Geotechnical Engineering, 2013, 35(Suppl. 1): 312−316.

[11] LI Xi-bin, XIE Kang-he, CHEN Fu-quan. One dimensional consolidation and permeability tests considering stress history and rheological characteristic of soft soils[J]. Journal of Hydraulic Engineering, 2013, 44(1): 18−25.

[12] GAO Jun, DANG Fa-ning, DING Jiu-long, et al. Research on soft soil consolidation calculation method considering the impacts of initial consolidation state[J]. Chinese Journal of Rock Mechanics and Engineering, 2019, 38(Suppl. 1): 3189−3196.

[13] QI Tian, XIE Kang-he, TANG Hu-le. Computational parameters on one-dimensional nonlinear consolidation of over-consolidation soil and its experimental determination[J]. Chinese Journal of Engineering Geophysics, 2008, 5(6): 735−741.

[14] WEN Jie-bang. Study on one-dimensional consolidation theory of layered soils considering the effect of stress history[D]. Hangzhou: Zhejiang University, 2007.

[15] LÜ Wei-qing, DONG Zhi-liang, CHEN Ping-shan, et al. Research on relationship between permeability coefficient and consolidation stress of normal consolidation clay[J]. Rock and Soil Mechanics, 2009, 30(3): 769−773.

[16] LIN Peng, XU Zhen-hong, XU Peng, et al. Research on coefficient of consolidation of soft clay under compression[J]. Rock and Soil Mechanics, 2003, 24(1): 106−112.

[17] ZHUANG Ying-chun, LIU Shi-ming, XIE Kang-he. Study on nonlinearity of one-dimension consolidation coefficient of Xiaoshan clay[J]. Chinese Journal of Rock Mechanics and Engineering, 2005, 24(24): 4565−4569.

[18] ZOU Sheng-feng, LI Jin-zhu, WANG Zhong-jin, et al. Seepage test and empirical models for soils based on GDS apparatus[J]. Journal of Zhejiang University (Engineering Science), 2017, 51(5): 856−862.

[19] QI Tian. Theoretical and experimental studies on one-dimensional nonlinear consolidation of soft soil[D]. Hangzhou: Zhejiang University, 2008.

[20] SONG Lin-hui, HUANG Qiang, YAN Di, et al. Experimental study on effect of hydraulic gradient on permeability of clay[J]. Chinese Journal of Geotechnical Engineering, 2018, 40(9): 1635−1641.

[21] LI Guang-xin. On soil skeleton and seepage force[J]. Chinese Journal of Geotechnical Engineering, 2016, 38(8): 1522−1528.

[22] LIU Wei-zheng, SHI Ming-lei, MIU Lin-chang. Experimental study of permeability coefficient of natural saturated clay and its prediction model[J]. Rock and Soil Mechanics, 2013, 34(9): 2501−2514.