Rock and Soil Mechanics

Volume 41 | Issue 2 Article 6

9-27-2020

An asymptotic state constitutive model for saturated clay under partial drainage

Jia-shun LIU
State Key Laboratory for Geomechanics and Deep Underground Engineering, China University of Mining & Technology, Xuzhou, Jiangsu 221116, China

Lai-gui WANG
School of Mechanics & Engineering, Liaoning Technical University, Fuxin, Liaoning 123000, China

Xiang-dong ZHANG
School of Civil Engineering, Liaoning Technical University, Fuxin, Liaoning 123000, China

Jian-jun YANG
School of Civil Engineering, Liaoning Technical University, Fuxin, Liaoning 123000, China

See next page for additional authors

Custom Citation

LIU Jia-shun, WANG Lai-gui, ZHANG Xiang-dong, YANG Jian-jun, SUN Jia-bao, . An asymptotic state constitutive model for saturated clay under partial drainage[J]. Rock and Soil Mechanics, 2020, 41(2): 485-491.

An asymptotic state constitutive model for saturated clay under partial drainage

Authors

Jia-shun LIU, Lai-gui WANG, Xiang-dong ZHANG, Jian-jun YANG, and Jia-bao SUN

Rock and Soil Mechanics 2020 41(2): 485–491 ISSN 1000–7598
https: //doi.org/10.16285/j.rsm.2019.5410 rocksoilmech.researchcommons.org/journal

An asymptotic state constitutive model for saturated clay under partial drainage

LIU Jia-shun[1, 2], WANG Lai-gui[3], ZHANG Xiang-dong[1], YANG Jian-jun[1], SUN Jia-bao[1]

1. School of Civil Engineering, Liaoning Technical University, Fuxin, Liaoning 123000, China
2. State Key Laboratory for Geomechanics and Deep Underground Engineering, China University of Mining & Technology, Xuzhou, Jiangsu 221116, China
3. School of Mechanics & Engineering, Liaoning Technical University, Fuxin, Liaoning 123000, China

Abstract: A series of undrained, partially drained and completely drained triaxial tests were carried out to investigate the influence of drainage boundary conditions on the strength characteristics of saturated clay. The partially drained tests were carried out by controlling the strain increment ratio of the volume and axial strains. The tests investigated the influence of drainage boundary conditions on the mechanical properties of saturated clay including the pore water pressure, the effective stress path and the asymptotic behavior of the p'-q plane. Based on the asymptotic state and dilatancy characteristics, an asymptotic state constitutive model for the saturated clay was established by introducing the strain increment ratio into the stress path constitutive model. The comparison between experimental results of pore water pressure and effective stress path and simulations demonstrates a good predictive ability of the model. The test results show that the strain increment ratio of saturated clay should be less than 0.3. The drainage condition affects the dilatancy of normal consolidated clay, the effective stress path and the shear strength of soil. With the increase of the strain increment ratio, the pore water pressure and the effective stress ratio of saturated clay decrease but the strength increases. The saturated clay specimen will be in a critical state for a long time when the axial strain of the specimen reaches 3%, and the change in drainage conditions can inhibit or accelerate the failure of the soil.

Keywords: soil mechanics; saturated clay; partial drainage; strain increment ratio; strength

1 Introduction

The drainage boundary condition for deformation or instability of soft clay ground in embankment filling, dam and other projects is usually partially drained[1–2]. Due to the limitation of laboratory equipment, the strength index of soft clay is often obtained by the complete drained test (CD) or undrained test (CU). In the theoretical calculation or strength analysis, the use of consolidated drained strength index often leads to a higher safety factor, while the use of consolidated undrained strength index leads to a lower safety factor. Under the same test conditions, Chinese scholars' research on the mechanical characteristics of saturated sand under partially drained condition shows that the drainage condition controls the degree of soil dilatancy, and affects the mechanical properties of soil from the effective stress path and the degree of soil shear strength[3–4]。Therefore, it is of great significance for theories and practical engineering problems to study the mechanical properties of soils under partially drained condition, and to establish corresponding constitutive models.

For saturated soils, the volumetric strain $d\varepsilon_v$ is equal to the volume of water sucked or discharged ΔV from the sample. As a result, during the shear loading process, the influence of drainage degree on the mechanical properties of soil is indicated by controlling the axial strain increment $d\varepsilon_1$ and the corresponding amount of water absorbed (discharged) (i.e. $d\varepsilon_v$) to a certain constant. In the triaxial shear test, the method of controlling the partial drainage is often used to control the rate of volumetric and axial strains, that is, to control the strain increment ratio $\xi=d\varepsilon_v/d\varepsilon_1$ as a constant to achieve the partial drainage. When $\xi = 0$, it is a completely undrained test; when $\xi > 0$, it is a partially drained test, and the larger ξ is, the stronger the drainage is; when $\xi < 0$, it is a partial water absorption test, and the smaller ξ is, the stronger the water absorption is. Under the complete drained condition, the excess pore water pressure caused by loading is 0, and its strain increment ratio is a variable. If the extreme value of the strain increment ratio (ξ_{max}) is defined as the slope of the peak volumetric strain to the origin under complete drained condition, the strain increment ratio ξ in the partially drained condition should be controlled to be smaller than ξ_{max} in the completely

Received：25 February 2019 Revised：8 May 2019
This work was supported by the Program of State Key Laboratory for Geomechanics and Deep Underground Engineering, China University of Mining & Technology (SKLGDUEK1914), the National Natural Science Foundation of China (51774166) and the Natural Science Foundation Guidance Project of Liaoning Province（20180551162).
First author：LIU Jia-shun, male, born in 1986, PhD, Lecturer, mainly engaged in the teaching and research of soil mechanics and foundation. E-mail: liujiashun000@163.com

drained condition.

Various studies have been conducted on the mechanical properties of sand under partially drained condition. Gudehus et al.[5] studied the mechanical properties of sand with a constant strain ratio by constant strain path tests. Topolnicki [6] , Chu[7] and Asaka et al.[8] carried out sand mechanical tests of different strain increment ratios to study the strength characteristics of soils under partially drained condition. Based on the tests of controlling strain increment ratio, asymptotic state constitutive models have been developed. Luo[9] and Yao[10] proposed elastoplastic constitutive models based on the unified hardening model that can reflect the asymptotic behavior of saturated sand. Lu[11] and Cheng et al.[12] used STDTTS standard stress path triaxial apparatus to study the mechanical characteristics of saturated sand under partially drained condition, and proposed an asymptotic state constitutive model for saturated sand by coupling asymptotic state equation and dilatancy equation.

In summary, the experiments and methods for the mechanical properties of sand under partially drained condition are relatively mature, and the asymptotic state constitutive model and UH model of the sand have been established. But there are relatively few studies on the mechanical properties of saturated clay under partially drained condition. Wu et al[13]. studied the pore pressure accumulation and modulus change of red clay under partially drained condition, but the partially drained condition in this paper was caused by poor drainage of the soil (low permeability of red clay). Zhou[14] studied the influence of drainage condition on the shear strength of clay through a triaxial test with constant strain increment ratio. By controlling the strain increment ratio, Liu et al.[15]studied the development laws of the pore pressure, accumulative axial plastic strain, and volumetric strain of saturated silty clay at different stress path slopes and cyclic dynamic stress ratios under partially drained condition. Most of the above studies are descriptions of experimental phenomena, but no clear theoretical system has been formed for the mechanical mechanism and strength characteristics of soft clay under partially drained condition. Therefore, it is necessary to carry out strength tests of saturated clay under partially drained condition by controlling the strain increment ratio ξ (using GDS-DYNTTS dynamic triaxial apparatus) to investigate the influence of drainage boundary condition on the mechanical properties of saturated clay. Moreover, based on the critical state and dilatancy characteristics, an asymptotic state constitutive model is proposed by introducing the strain increment ratio ξ into the complex stress paths based constitutive model, in hope to lay the theoretical foundation for further study on the influence of partially drained condition on mechanical characteristics of soft clay.

2 Experimental program

2.1 Soil sample

The soil samples were taken near the Fuxin Station of the Beijing-Shenyang Passenger Dedicated Line. The depth of the soil samples was 1.5 to 3.5 m below the ground surface. The physical-mechanical properties of the soil samples measured in accordance with the *Code for Soil Test of Railway Engineering* (TB10102－2010)[16] are shown in Table 1.

Table 1 Physical-mechanical properties of tested silty clay

Density ρ / (g/cm^3)	Water content / %	Void ratio e	Liquid limit w_L	Plastic limit w_P	Cohesion c / kPa	Friction angle φ / (°)
1.83	21.8	0.75	26.5	11.9	24.5	26.8

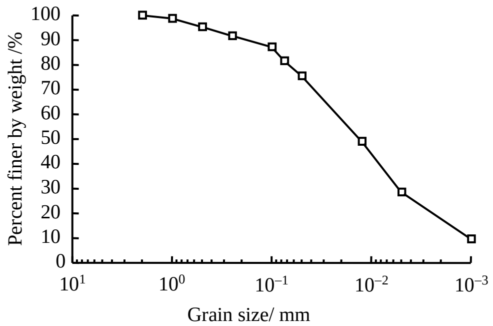


Fig.1 Grain size distribution of tested clay

The result of particle analysis is presented in Fig. 1. It can be seen that the silt content of the soil sample ($d < 0.075$ mm) is 78.0%, and the clay content ($d < 0.005$ mm) is 28.9%. Combined with the results of the limit moisture content test, the tested soil in this study is classified as the silty clay. A cylindrical sample from the undisturbed soil with a diameter of 39.1 mm and a height of 80 mm was prepared for partially drained tests.

2.2 Test program

2.2.1 Laboratory apparatus

GDS-DYNTTS dynamic triaxial apparatus (as shown in Fig.2) is mainly composed of loading system, control system and data acquisition system. It can independently control and measure the confining pressure and back pressure (volume) through the 200 mL/2 MPa water pressure/volume controller. Axial drivers and sensors can control and measure parameters such as axial pressure and axial deformation.

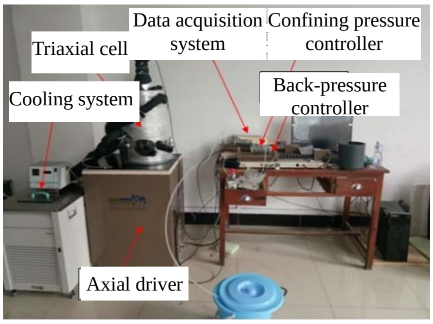


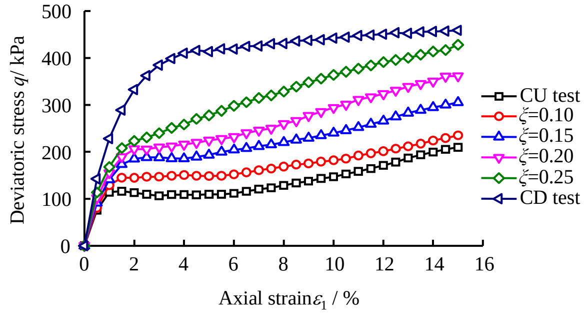
Fig.2 GDS-DYNTTS dynamic triaxial apparatus

2.2.2 Test control condition

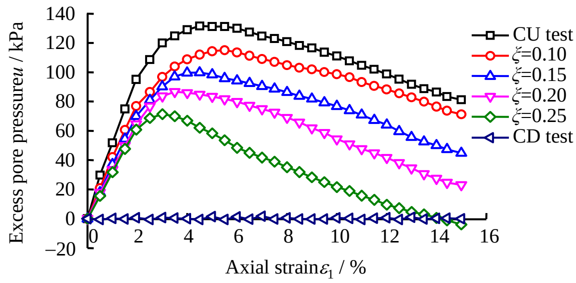
The prepared sample with the rubber film was installed inside the triaxial pressure chamber, connected to the communication pipeline, and the GDSLAB software was started. First, in the saturated consolidation stage, a confining pressure of 100 kPa was applied to the soil sample, and then the additional confining and back pressure of 20 kPa and 30 kPa separately were applied step by step. When the Skempton pore water stress coefficient $B > 0.95$, the sample was considered to be fully saturated. All tests were then isotropically consolidated to 200 kPa. It could be considered that the soil sample has been completely consolidated when the sample displacement was less than 5 mm^3 within 5 minutes. Subsequently, the CD test, CU test and partially drained triaxial shear test can be carried out by the standard triaxial test module. Partially drained conditions were achieved by controlling the back and axial pressure controllers. Set the RAMP command on the back pressure control panel to set the drainage volume rate of the soil sample. Through the Advanced Loading module, set the axial loading rate to 0.01 mm/min, and control the strain increment ratios ξ under the partially drained conditions of 0.1, 0.15, 0.2 and 0.25, to keep the soil sample in a partially drained condition. When the axial strain ε_1 is up to 15%, stop the test.

3 Results and discussions

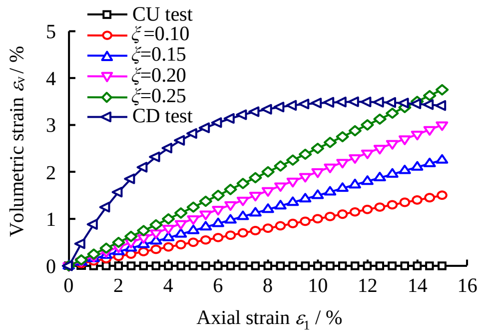
Figure 3 presents the experimental results of saturated clay under different drainage conditions. From the deviatoric stress-axial strain curve (Fig. 3(a)) and the pore pressure-axial strain curve (Fig.3(b)), it shows that for consolidated undrained shear test (CU), the peak stress is about 209.6 kPa when the axial strain is around 12%, and the excess pore water pressure u reaches the maximum ($u_{\max}$ = 131.57 kPa) at 4.0% axial strain. For consolidated drained test (CD), the deviatoric stress–axial strain curve shows strain softening. When the axial strain ε_1 exceeds 10%, the deviatoric stress increment is already small. Taking the corresponding deviatoric stress of 15% of the axial strain as the peak strength, the peak deviatoric stress of the



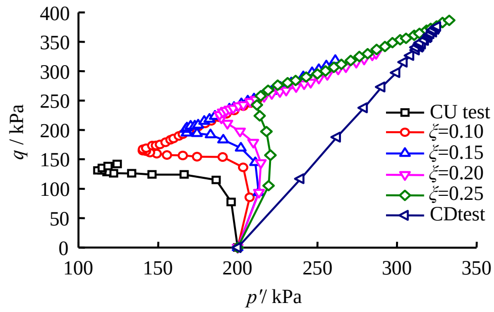
(a) Deviatoric stress–axial strain



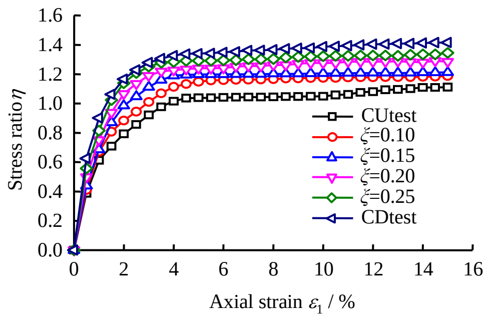
(b) Pore pressure–axial strain



(c) Volumetric–axial strain



(d) Effective stress path



(e) Stress ratio–axial strain

Fig.3 Experimental results of saturated clay under different drainage conditions

saturated clay during the consolidated drained test is around 458.82 kPa. During the test, the excess pore water pressure u is always kept within ± 2 kPa, indicating that the axial load rate of 0.01 mm/min can ensure that the pore pressure is fully dissipated. The axial load rate of 0.01 mm/min is reasonable in this test.

The relationship between the volumetric strain and the axial strain (see Fig. 3(c)) presents that the volumetric strain of the soil first shrinks during the CD test, and then increases with the axial strain. When the axial strain is $\varepsilon_1 = 10\%$, the increment of volumetric strain of the soil is negative value, indicating that the soil exhibits dilatancy. Considering the slope of the peak volumetric strain to the origin as the strain increment ratio of $\xi_{\rm CD} = 0.30$ under completely drained condition, the strain increment ratio $\xi = {\rm d}\varepsilon_{\rm v} / {\rm d}\varepsilon_1$ under partial drainage should be between 0 and 0.3. Therefore, when the strain increment ratios ξ is controlled to be 0.1, 0.15, 0.2, and 0.25 ,respectively under partially drained conditions, and the axial loading rate is 0.01 mm/min, the volumetric strain rate of the back pressure is set to 0, 1.2, 1.8, 2.4, 3.0 mm^3/min, respectively.

Since the drainage rate is always maintained at a constant value under partially drained conditions, the drainage rate of the CD test gradually decreases. And with the increase of the strain increment ratio, the pore pressure can dissipate or even become negative in the later period of the drained test. Its volumetric strain rate can also exceed the volumetric strain value under drainage conditions. Moreover, the larger ξ is, the earlier it exceeds the volumetric strain value under the completely drained condition. For this test, when the axial strain $\varepsilon_1 \approx 13.5\%$, the pore pressure of $\xi = 0.25$ dissipates as a negative value. Its volumetric strain curve exceeds the volumetric strain curve under the completely drained condition. Although the rest of tests do not exceed, there is a tendency to exceed the volumetric strain under the completely drained condition.

It can be seen from the effective stress path curve in Fig.3(d) that with the increase of the strain increment ratio ξ, the deviatoric stress and the volumetric strain gradually increase, but the pore water pressure gradually decreases, and the axial strain at which the peak pore water pressure appears also gradually decreases. The deviatoric stress q, volumetric strain $\varepsilon_{\rm v}$ and pore pressure u of the sample at the later stage of the loading test change linearly with axial strain ε_1. Therefore, when the confining pressure is kept constant, the ratio of the deviatoric stress q to the effective mean stress p' of the sample is close to a linear stress path in the middle and later stages of the test, and its slope is around 1.41 ($q / p' = 1.41$).

From the relationship curve between the stress ratio and the axial strain in Fig.3(e), it can be seen that in the partially drained test, after the axial strain of the soil specimen reaches 3%, although the strength of the specimen continues to increase, the stress ratio is close to the critical stress ratio. If it is suddenly loaded in actual engineering, the drainage conditions of the soil will be suppressed, and the strain increment ratio will be reduced. The internal pore water pressure of the soil then suddenly increases, and the stress path will develop above the critical state line. Subsequently, the soil in the critical state may soon be damaged.

4 Asymptotic state constitutive model

The curve obtained by plotting the peak stress ratio under different stress paths in the ${\rm d}\varepsilon_{\rm v} / {\rm d}\varepsilon_1 - \eta$ plane is the asymptotic state curve. Fig. 4 reflects the asymptotic stress ratio of saturated clay under different drainage conditions. The vertical line (ξ = 0) in the figure indicates the undrained condition. The left and right sides present the conditions of the partial water absorption and drained tests respectively.

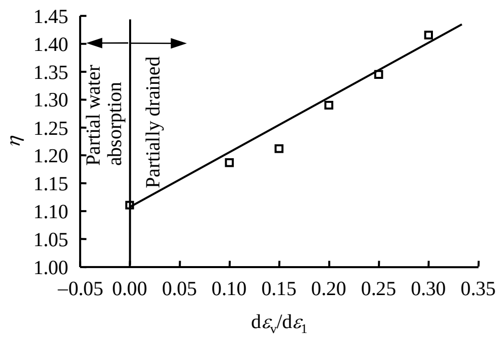


Fig.4 Asymptotic stress ratio under different drainage degrees

From Fig.4, it can be seen that when the strain increment ratio increases, the stress ratio of the clay gradually increases until it reaches the asymptotic state stress ratio. When the control condition is undrained, the dilatancy of the clay is suppressed, which affects the friction structure potential of the clay. It shows that the critical state stress ratio corresponding to the approaching state is small. The stress ratio at this time is defined as the asymptotic state stress ratio M_0 under undrained conditions. When the control condition is free drainage, the volumetric strain of the sample is free. The dilatancy of the clay can be fully exerted, and the corresponding stress ratio is relatively large when the critical state is reached. When the strain increment ratio is controlled, the volumetric strain of the clay gradually weakens with the increase of the strain increment ratio ζ, even if the drainage condition of the specimen is between the free drainage and completely undrained. As a result, the soil strength is fully exerted, and the pore water pressure gradually decreases. When the condition is completely drained, the void ratio of the sample is close to zero, and the critical state stress ratio reaches the maximum, which is defined as the asymptotic state peak stress ratio M_f.

4.1 Asymptotic state constitutive model for clay

Gudehus et al.[5] first proposed the use of asymptotic states to describe the effects of different constraints on the ultimate stress ratio of the soil. Chu et al.[17] proposed an asymptotic state equation reflecting the asymptotic stress ratio and strain increment ratio, and studied the softening characteristics of soil under partially drained conditions. Based on the unified hardening model, Kong[18] and Lü[19] proposed the double hardening elastoplastic constitutive model which reflects the compression-shear coupling effect and the asymptotic state behavior of saturated sand. Luo et al.[20] established a sand constitutive model that can consider the characteristics of particle crushing, dilatancy, shear shrinkage, strain hardening, and softening. This model can also reflect the downward movement of the critical state line of sand under shear stress. However, due to the limitation of geotechnical instruments, the study on the mechanical properties and constitutive models of saturated clay under partially drained conditions has not been widely developed. Therefore, based on the results of the complex stress paths based constitutive model of saturated sand, this paper proposes an asymptotic state constitutive model for the saturated clay considering partially drained conditions.

The complex stress paths based constitutive model of the soil is an elastoplastic incremental mathematical model. The strain increment $d\varepsilon_{ij}$ is the sum of the elastic and plastic strain increments, that is

$$d\varepsilon_{ij} = d\varepsilon_{ij}^{e} + d\varepsilon_{ij}^{p} \tag{1}$$

where $d\varepsilon_{ij}^{e}$ is the elastic strain increment; $d\varepsilon_{ij}^{p}$ is the plastic strain increment. The elastic stress-strain increment relationship is

$$d\varepsilon_{ij}^{e} = \frac{1+\upsilon}{E} d\sigma_{ij} - \frac{\upsilon}{E} d\sigma_{kk}\delta_{ij} \tag{2}$$

The plastic stress-strain increment relationship is

$$d\varepsilon_{ij}^{p} = \frac{1}{3} d\varepsilon_{v}^{p}\delta_{ij} + \frac{3}{2q}(\sigma_{ij} - p'\delta_{ij}) d\varepsilon_{d}^{p} \tag{3}$$

where σ_{ij} is the component of stress tensor; σ_{kk} is the summation mark; δ_{ij} is the Kronecker symbol; q is the deviatoric stress, $q = \sigma_1 - \sigma_3$; p' is the mean effective stress, $p' = (\sigma_1' + 2\sigma_3')/3$; σ_1', σ_3' are the effective maximum and minimum principal stresses; υ is the Poisson's ratio; E is the Young's modulus; $d\varepsilon_{v}^{p}$ is the plastic volumetric strain increment, and $d\varepsilon_{d}^{p}$ is the plastic deviatoric strain increment; The plastic volumetric strain increment $d\varepsilon_{v}^{p}$ and deviatoric strain increment $d\varepsilon_{d}^{p}$ can be obtained by

$$\begin{Bmatrix} d\varepsilon_{v}^{p} \\ d\varepsilon_{d}^{p} \end{Bmatrix} = \frac{c_p M_f^4}{M^4 (M_f^4 - \eta^4)} \cdot \begin{bmatrix} \dfrac{(M^4 - \eta^4)}{p'} & 2\eta(M^2 - \eta^2) \\ \dfrac{2\eta(M^2 + \eta^2)}{p'} & 4\eta^2 \end{bmatrix} \begin{Bmatrix} dp' \\ d\eta \end{Bmatrix} \tag{4}$$

where $c_p = (\lambda - \kappa)/(1 + e_0)$; λ and κ are the slopes of the compression line and the rebound line in the e-ln$p$ plane; e_0 is the initial void ratio; M_f is the peak stress ratio, which is obtained from the drained triaxial test; $\eta(q/p')$ is the shear strss ratio; M is the phase transformation stress ratio from volumetric contraction to dilation. Different from sand, saturated clay only produces volumetric compression during triaxial shear, and its phase transformation stress ratio M is equal to the peak stress ratio M_f, that is $M = M_f$. By substituting $M = M_f$ into Eq. (4), the formula of the plastic volumetric and deviatoric strain increments of saturated clay during triaxial shear test can be obtained:

$$\begin{Bmatrix} d\varepsilon_{v}^{p} \\ d\varepsilon_{d}^{p} \end{Bmatrix} = \frac{c_p}{(M^4 - \eta^4)} \begin{bmatrix} \dfrac{1}{p'} & \dfrac{2\eta}{M^2 + \eta^2} \\ \dfrac{2\eta}{p'(M^2 - \eta^2)} & \dfrac{4\eta^2}{M^4 - \eta^4} \end{bmatrix} \begin{Bmatrix} dp' \\ d\eta \end{Bmatrix} \tag{5}$$

However, the clay is over-consolidated when the loading stress path is $dp' < 0$ and $dq > 0$. At this time, $M \neq M_f$. The plastic volumetric and shear strain increments of the soil should be calculated according to Eq. (4).

Chu and Lo (1994) obtained the asymptotic state characteristics of saturated sand through experimental research, and gave the asymptotic state equation[17]:

$$\frac{d\varepsilon_v}{d\varepsilon_1} = \frac{3(M_0 - \eta_f)}{M_0} \tag{6}$$

where M_0 is the final stress ratio at $\xi = 0$; η_f is the final stress ratio under different drainage conditions. $\xi =$ 1 corresponds to the one-dimensional consolidation stress path ($\eta_{\mathrm{f}} = K_0$); $\xi = 3$ corresponds to the isotropic compressive stress path ($\eta_{\mathrm{f}} = 0$).

Substituting the strain increment ratio $\xi = \mathrm{d}\varepsilon_{\mathrm{v}} / \mathrm{d}\varepsilon_1$ into Eq. (6) gives:

$$\eta_{\mathrm{f}} = \frac{M_0(3-\xi)}{3} \tag{7}$$

With the associated flow rule for the modified Cam-clay model:

$$f = g = q^2 + M^2 p'^2 - Cp' = 0 \tag{8}$$

The dilatancy equation is

$$\frac{\mathrm{d}\varepsilon_{\mathrm{v}}^{\mathrm{p}}}{\mathrm{d}\varepsilon_{\mathrm{d}}^{\mathrm{p}}} = \frac{M^2-\eta^2}{2\eta} \tag{9}$$

where M is the critical stress ratio; C is a constant.

In the asymptotic state, the elastic strain increment can be negligible compared to the plastic one. In other words, the plastic strain increment is equal to the total strain increment, that is, $\mathrm{d}\varepsilon_{\mathrm{v}} = \mathrm{d}\varepsilon_{\mathrm{v}}^{\mathrm{p}}$, $\mathrm{d}\varepsilon_{\mathrm{d}} = \mathrm{d}\varepsilon_{\mathrm{d}}^{\mathrm{p}}$. The following formula can be obtained from the conventional triaxial shear test:

$$\frac{\mathrm{d}\varepsilon_{\mathrm{v}}}{\mathrm{d}\varepsilon_{\mathrm{d}}} = \frac{3\xi}{3-\xi} \tag{10}$$

When the critical state is reached, the dilatancy equation intersects with the asymptotic state equation, that is:

$$\frac{3-\xi}{3\xi} = \frac{M^2-\eta_{\mathrm{f}}^2}{2\eta_{\mathrm{f}}} \tag{11}$$

Substituting Eq.(7) into Eq.(11), the phase transformation stress ratio equation considering partially drained conditions can be obtained as

$$M = \sqrt{\frac{M_0^2(3-\xi)^2}{9} + 2\xi M_0} \tag{12}$$

Eq. (12) is the equation of phase transformation stress ratio of saturated clay when the loading stress path is $\mathrm{d}p' < 0$ and $\mathrm{d}q > 0$.

Substituting Eqs. (12), (4), (3), and (2) into Eq. (1), an asymptotic state constitutive equation for saturated clay can be established.

4.2 Model verification

Isotropic compression and rebound tests were carried out on the undisturbed soil samples, and the isotropic compression and rebound slopes of clay are measured to be 0.093 and 0.025. The initial void ratio e_0 is 0.75, c_{p} is 0.038 9, and the Poisson's ratio v is 0.35. From the test results in Section 3, it is known that the peak stress ratio of saturated clay M_{f} is 1.416, and the asymptotic stress ratio of saturated clay M_0 is 1.111 by the CU test.

The comparsion between test results and model simulations for pore water pressure, effective stress path and stress ratio under different drainage conditions are shown in Fig.5. The scattered points in the figure are the test results, and the curves are the model prediction results.

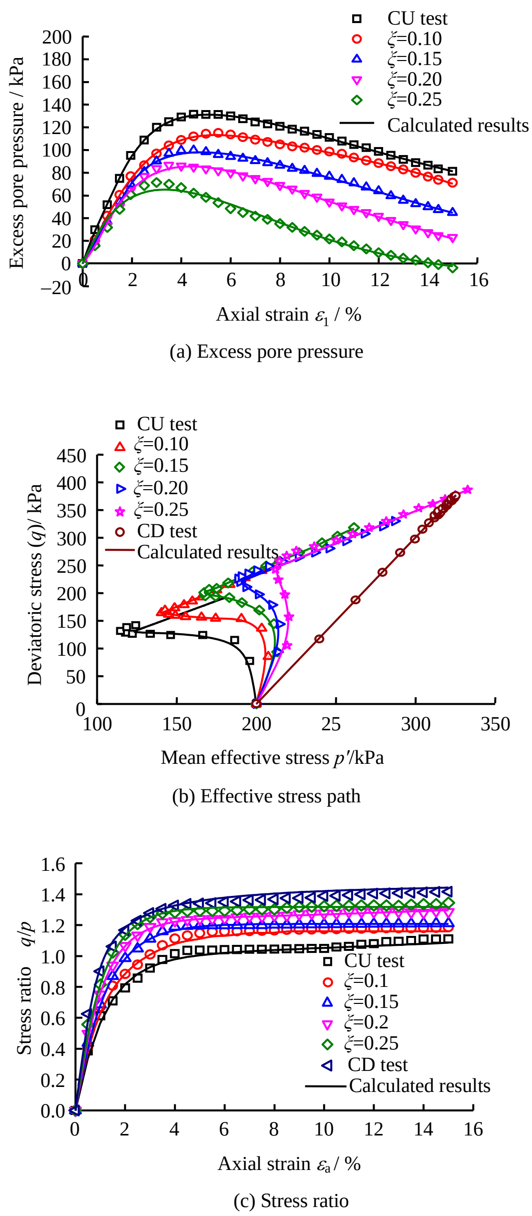


Fig.5 Comparison between calculation results and test results

Figure 5 shows that based on the complex stress paths of sand, the developed asymptotic state constitutive model in this paper can reasonably describe the mechanical properties of saturated clay under different strain increment ratios by phase trnasformation stress ratio equation. This model combines the characteristics of the stress-strain curve of clay and the correlation of the stress paths. The influence of drainage condition on the mechanical properties of clay can be described.

5 Conclusions

In order to study the mechanical properties of saturated clay under partially drained conditions, the undrained, partially drained and free drained triaxial tests on saturated clay were carried out. The effects of drainage boundary condition on the mechanical properties of saturated clay were investigated for the pore water pressure, effective stress path and asymptotic state characteristics on p'-q plane. An asymptotic state constitutive model of saturated clay was then proposed. The key observations are summarized in the following:

(1) Partially drained triaxial test results show that the drainage condition affects the strength of the soil, and the strength of the soil increases with the increase of the strain increment ratio ξ. In the CD test, the strength of the saturated clay is the highest, but in the CU test, the strength is the lowest. The pore water pressure decreases with the increase of the strain increment ratio ξ.

(2) In the partially drained test, when the axial strain reaches 3%, although the strength of the soil increases continuously, its stress ratio is close to the critical stress ratio. If it is suddenly loaded in actual engineering, the drainage conditions of the soil are suppressed, and the stress path will develop above the critical state line. The soil sample in the critical state may be damaged soon.

(3) Considering the characteristics of the stress-strain curve of saturated clay and the correlation of the stress path, an asymptotic state constitutive model of saturated clay under partially drained condition was established by phase transformation stress ratio equation, and the reliability of the model was verified.

Reference

[1] WANATOWSKI D, CHU J. Pre-failure instability behavior of sand in strain path testing under plane-strain conditions[J]. Soils and Foundations, 2011, 51(3): 423–435.

[2] SEKIGUCHI H, SHIBATA T, MIMURA M. Effects of partial drainage on the lateral deformation of clay foundations[J]. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 1997, 27(2): 109.

[3] HUANG Mao-song, YAO Yang-ping, YIN Zhen-yu, et al. An overview on elementary mechanical behaviors, constitutive modeling and failure criterion of soils[C]// The 12th Academic Conference on Soil Mechanics and Geotechnical Engineering of the Civil Engineering Society of China. Shanghai: [s. n.], 2015.

[4] LUO Ting, YAO Yang-ping, HOU Wei. Soil constitutive models[M]. Beijing: China Communications Press, 2010.

[5] GUDEHUS G, GOLDSCHEIDER M, WINTER H. Mechanical properties of sand and clay and numerical integration methods: some sources of errors and bounds of accuracy[M]. [S. l.]: Balkema, 1977: 121–150.

[6] TOPOLNICKI M, GUDEHUS G, MAZURKIEWICZ B K. Observed stress-strain behavior of remoulded saturated clay under plane strain conditions[J]. Geotechnique, 1990, 42(2): 155–187.

[7] CHU J, LO S C R, LEE I K. Strain-softening behavior of granular soil in strain-path testing[J]. Journal of Geotechnical Engineering, 1992, 118(2): 191–208.

[8] ASAKA Y, TOKIMATSU K, IWASAKI K, et al. A simple stress-strain relation based on stress-path behavior in strain-path controlled triaxial tests[J]. Soils and Foundations, 2003, 43(2): 55–68.

[9] LUO Ting, YAO Yang-ping, CHU Jian. Asymptotic behavior of saturated sand and its simulation[J]. Scientia Sinica (Technologica), 2009, 39(1): 39–47.

[10] YAO Yang-ping, LIU Lin, LUO Ting. A constitutive model for granular soils[J]. Science China (Technological Sciences), 2018, 61(10): 120–129.

[11] LU De-chun, CHENG Xing-lei, DU Xiu-li. Mechanical properties of saturated sand under partial drainage[J]. Chinese Journal of Geotechnical Engineering, 2012, 34(12): 2263–2269.

[12] CHENG Xing-lei. Mechanics property research of saturated sand under part drainage condition[D]. Beijing: Beijing University of Technology, 2012.

[13] WU Jian-qi, YANG Xiao, XU Xu, et al. Cyclic triaxial tests on saturated red clay under partially drained condition[J]. Journal of Zhejiang University (Engineering Science), 2017, 51(7): 1309–1316.

[14] ZHOU Xing-chen. Experimental study on saturated clay strength under partial drainage conditions[J]. Science Technology and Engineering, 2017, 17(20): 223–229.

[15] LIU Jia-shun, WANG Lai-gui, ZHANG Xiang-dong, et al. Cyclic triaxial test on saturated silty clay under partial drainage condition with variable confining pressure[J]. Rock and Soil Mechanics, 2019, 40(4): 1413–1419, 1432.

[16] Ministry of Railways of the people's Republic of China. TB10102 — 2010 Code for Soil Test of Railway Engineering[S]. Beijing: China Railway Publishing House, 2010.

[17] CHU J, LO S C R. Asymptotic behavior of a granular soil in strain path testing[J]. Geotechnique, 1994, 44(1): 65–82.

[18] KONG Yu-xia, ZHAO Ji-dong, YAO Yang-ping. A failure criterion for cross-anisotropic soils considering micro-structure[J]. Acta Geotechnica, 2013, 8(6): 665–673.

[19] LU Xi-lin, HUANG Mao-song, ANDRADE J E. Strength criterion for cross-anisotropic sand under general stress conditions[J]. Acta Geotechnica, 2016, 11(6): 1339–1350.

[20] LUO Ting, LIU Lin, YAO Yang-ping. Description of critical state for sands considering particle crushing[J]. Chinese Journal of Geotechnical Engineering, 2017, 39(4): 592–600.