Research Article

Study on Cyclic Cumulative Deformation Characteristics and the Equivalent Cyclic Creep Model of Soft Clay

Wenbo Zhu1,2, Guoliang Dai1,2 and Weiming Gong1,2

1Key Laboratory of Concrete and Prestressed Concrete Structure of Ministry of Education, Southeast University, Nanjing, Jiangsu 211189, China
2School of Civil Engineering, Southeast University, Nanjing, Jiangsu 211189, China

Correspondence should be addressed to Wenbo Zhu; 763566305@qq.com

Received 5 February 2021; Revised 24 February 2021; Accepted 2 March 2021; Published 12 March 2021

1.Introduction

Suction caisson foundations can be used as anchoring foundations for tension leg platforms (TLPs). Under an uplift load, the soil at the bottom of the caissons is subjected to both axial unloading and cyclic loading superimposed by wind and wave loads. However, the problem of cyclic cumulative deformation of soft clay under unloading has rarely been addressed. The soil at the bottom of the caisson undergoes both unloading and cyclic loading under wind and wave loads. However, the problem of cyclic cumulative deformation of soft clay under unloading has rarely been addressed. So, the strain cumulative deformation and strain softening characteristics of soft clay are studied by cyclic triaxial tests. The test results show that under low static deviator stress ratios and dynamic deviator stress ratios, the soil has a low level of strain accumulation and softening. As the dynamic deviator stress ratios increase, the cumulative cyclic deformation gradually increases, which rapidly develops in the early stage and tends to stabilize in the later stage. Moreover, the softening index gradually increases and is linearly related to the logarithm of the number of cycles. The cyclic cumulative deformation of the soil increases with increases in unloading stress and dynamic deviator stress, showing a creep characteristic of attenuation and then stabilization. Based on the tests, an equivalent cyclic creep model is established to describe the strain accumulation and softening of soil and verified through comparison with the test results. Then, the model is extended to a three-dimension model, and a finite element subroutine is developed for studying the strain cumulative deformation and strain softening characteristics of soft clay.

1. Introduction

Suction caisson foundations can be used as anchoring foundations for tension leg platforms (TLPs). Under an uplift load, the soil at the bottom of the caissons is subjected to both axial unloading and cyclic loading superimposed by wind and wave loads, with cyclic loading inevitably leading to strain softening and cumulative cyclic deformation of the soft clay. Most of studies [1–8] have focused on the case where the initial deviatoric stress is greater than or equal to zero but rarely investigated the cyclic cumulation deformation of soft clay during unloading. Additionally, empirical models are mostly used for cyclic accumulation modeling but have difficulty describing soil strain softening, which is a problem shared by element creep models.

Many studies have been conducted with the aim to accurately understand the characteristics of strain softening and the cumulative cyclic deformation of soft clay. In terms of soil strain softening, Andersen [9], Lee [10], Mitchell and King [11], Hyodo et al. [12], and Yasuhara [13] studied the dynamic properties of different types of undisturbed and remolded soil samples under cyclic loading and analyzed the effects of different influencing factors, such as cyclic stress ratio, overconsolidation ratio, frequency, and number of loading cycles on cumulative deformation and soil softening. Zhou and Gong [14] found that the higher the frequency was, the lower the degree of soil strain softening and the faster the strain development. Matasovic and Vucetic [15] established a relationship between softening parameters and cyclic strength to reflect the softening of soil by the change in softening parameters. According to the softening index defined by Idriss et al. and considering the effect of the initial consolidation shear stress, Yao and Nie [16] and Cai et al. [17] redefined the softening index to describe the degree of soil softening. Wang
and Cai [18] investigated the influences of the cyclic stress ratio, vibration frequency, overconsolidation ratio, and consolidation ratio on the strain softening characteristics of saturated soft clay with cumulative plastic strain. In terms of strain accumulation, Monismith et al. [19] established a relationship between the cyclic cumulative strain of soil elements and the number of cycles through triaxial tests. On this basis, Li and Selig [20] introduced the static strength parameter of soil and proposed an empirical relationship for the cyclic cumulative strain. Based on the relationship proposed by Li and Selig [20], Chai and Miura [21] further established an empirical exponential relationship considering the effect of the initial static deviatoric stress of the soil element on the cyclic cumulative strain. For the first time, Huang et al. [22] introduced the concept of relative deviatoric stress level $D^\ast$, considering the cyclic cumulative strain resulting from the coupling effect of initial static deviatoric stress and cyclic stress. Hyde and Brown [23] and Yasuhara et al. [24] found that the behavior of the residual strain of clay generated under cyclic loading is very similar to that under creep loading and established a method for calculating the cumulative cyclic strain by making the number of loading cycles equivalent to time. Based on the theory of static creep, Wang et al. [25] proposed an empirical model for cumulative cyclic plastic axial strain of saturated soft clay under long-term cyclic loading. Moreover, Fujiwara et al. [28], Zhu et al. [29], Li and Huang [30], and Guo et al. [31] conducted long-term undrained cyclic loading tests on soft clays in different regions and analyzed the cyclic creep behavior of soft clay in depth under long-term repeated loading.

The above studies are all focused on the case in which the axial deviator stress is greater than the radial deviator stress, whereas the cumulative cyclic deformation of soft clay during unloading has rarely been investigated. So, the strain accumulation and deformation characteristics of soft clay under axial unloading and cyclic unloading were investigated by the GDS dynamic triaxial test. Based on the tests, an equivalent cyclic creep model is established to describe the strain accumulation and softening of soil and verified through comparison with the test results. Then, the model is extended to a three-dimension model, and a finite element subroutine is developed for studying the strain cumulative deformation and strain softening characteristics of soft clay.

2. Cyclic Triaxial Tests

The test soil samples were prepared by the vacuum preloading method and the surcharge preloading method. As shown in Figure 1(a), the soil sample preparation chamber is a bucket made of acrylic plexiglass, which has dimensions of $300\,\text{mm} \times 600\,\text{mm} \times 10\,\text{mm}$, to facilitate the observation of slurry settlement. After the soil sample molding, the soil samples were tested on a GDS dynamic triaxial test apparatus (Figure 1(b)). The test procedure is described as follows: (1) the B test was first performed; since the samples were prepared using the slurry method, the B values were above 98%, and there was no need for backpressure saturation; (2) isotropic consolidation was carried out according to the differently set confining pressures; (3) after consolidation was completed, the unloading was carried out at a rate of 0.1 kPa/min until the preset deviatoric stress was reached; and (4) cyclic loading was performed at a frequency of 0.1 Hz and with a sine waveform in the presence of the deviatoric stress. The test parameters are shown in Table 1.

To determine the soil strength, the triaxial compression and extension tests of soft clay were performed under confining pressures of 50 kPa, 100 kPa, and 150 kPa. The stress-strain curves of saturated soft clay subjected to consolidated undrained (CU) triaxial tests are shown in Figure 2. The Mohr–Coulomb ultimate strength indices of saturated clay were calculated to be $c = 18.3$ kPa and $\varphi = 19.6^\circ$.

3. Test Results

3.1. Cyclic Cumulative Strain. Figures 3–5 show the cyclic cumulative strain curves under different static and dynamic deviator stresses. Figure 3(a) presents the cyclic cumulative strain curves under different dynamic deviator stress ratios. Under the dynamic deviator stress $q_d = 0.2q_f$, there was no significant strain accumulation due to the small static deviator stress. Under the dynamic deviator stress $q_d = 0.3q_f$, the cyclic cumulative strain increased rapidly at the beginning of the loading, and then, the rate of strain accumulation started to rapidly decrease, and the development of cumulative strain entered an attenuation stage and eventually reached a stabilization stage. Under the same confining pressure and the same static deviator stress, the cyclic cumulative strain increased with the increase of unloading stress, and as the dynamic deviator stress increased, the cyclic cumulative strain increased.

Figure 3(c) shows the cyclic cumulative strain curves under different dynamic stress ratios and a static stress ratio $q_s = 0.6q_f$. With a dynamic deviator stress $q_d = 0.1q_f$ and $0.2q_f$, all the curves exhibit the cyclic creep characteristics. With $q_d = 0.3q_f$, at the later stage, the slope of the cyclic cumulative strain curve did not tend to stabilize. Figure 3(d) shows the cyclic cumulative strain curves obtained under $q_s = 100$ kPa, $q_s = 0.8q_f$, and $q_d = 0.1q_f$. Under high static deviator stress and low dynamic deviator stress, the cyclic cumulative strain increased rapidly with the increase in the number of loading cycles; at the later stage, the development of cumulative strain entered an attenuation stage and eventually reached stabilization. So, there was a large cyclic cumulative strain under high static deviator stress and low dynamic deviator stress.
From the above test results, it can be seen that under low static deviator stress and low dynamic deviator stress ratio, the cyclic cumulative strain was small and tended to eventually stabilize with an increasing number of cycles. Under low static deviator stress and a high dynamic deviator stress ratio, the cyclic cumulative strain was mainly concentrated in the early stage with relatively fast cumulative cyclic deformation, and then, it gradually stabilized in the later stage. Under high static deviator stress and low dynamic deviator stress ratio, the cyclic cumulative strain of soil developed quickly in the early stage and gradually stabilized in the later stage.

### 3.2 Strain Softening Index

As the number of cycles increased, the hysteresis loops were gradually inclined toward the strain direction; as a result, the cyclic cumulative strain gradually increased, and the strength of the soil softened (Figure 6). The soil shear modulus of the $N^{th}$ cycle was calculated using the following equation.

$$G_N = \frac{q_{N,\text{max}} - q_{N,\text{min}}}{\varepsilon_{N,\text{max}} - \varepsilon_{N,\text{min}}}$$

where $q_{N,\text{max}}$ and $q_{N,\text{min}}$ are the maximum and minimum deviator stresses, respectively, of the $N^{th}$ cycle; and $\varepsilon_{N,\text{max}}$ and $\varepsilon_{N,\text{min}}$ are the maximum and minimum deviator strains, respectively, of the $N^{th}$ cycle.

Idriss et al. [32] were the first to introduce the concept of the softening index $\delta$, which is defined as the ratio of the shear modulus of the $N^{th}$ cycle to the shear modulus of the first cycle, that is,

$$\delta = \frac{G_N}{G_1} = \frac{\tau_{N,\gamma}}{\tau_{1,\gamma}} = \frac{\tau_N}{\tau_1}$$

Andersen [9] redefined the softening index based on equation (2) by considering the initial deviator strain and also established the relationship between the softening index and the number of cycles:

$$\delta = \frac{G_N}{G_1} = \frac{\varepsilon_{N,\text{max}} - \varepsilon_{N,\text{min}}}{\varepsilon_{1,\text{max}} - \varepsilon_{1,\text{min}}}$$

The test results show that the strain softening was low under a small static deviator stress ratio and a small dynamic deviator stress ratio. As the number of cycles increased, the

### Table 1: Test parameters.

| Type           | Confining pressure (kPa) | Unloading rate (kPa/min) | Frequency (Hz) | Static deviator stress | Dynamic deviator stress |
|----------------|--------------------------|--------------------------|----------------|------------------------|-------------------------|
| Unloading      | 50/100/150               | 0.1                      |                | 0/0.4/0.5/0.6/0.8      | 0.1/0.2/0.3             |
| Cyclic loading | 50/100/150               | —                        | 0.1            | 0.8                    | 0.1                     |

Figure 1: Dynamic triaxial tests. (a) The preloading device. (b) The GDS dynamic triaxial test system.

![Strength characteristics of soil](image-url)

Figure 2: Strength characteristics of soil.
Figure 3: Strain-time relationships ($\sigma_3 = 100$ kPa). (a) $q_s = 0.4q_f$, (b) $q_s = 0.5q_f$, (c) $q_s = 0.6q_f$, and (d) $q_s = 0.8q_f$.

Figure 4: Strain-time relationships ($\sigma_3 = 50$ kPa). (a) $q_s = 0.4q_f$ and (b) $q_s = 0.5q_f$. 
strain softening index gradually approached a constant value. A large dynamic deviator stress ratio led to a high degree of strain softening. Under the same dynamic deviator stress ratio, the softening index was smaller when the static deviator stress was higher. Therefore, a strain softening index equation, as shown in equation (4), was established in the present study based on the test data. The theoretically calculated values and the test values are compared in Figure 7. It can be seen that the fitting result is good. Moreover, the fitting parameters being $a = 0.16$, $n_1 = 3.6$, $n_2 = 7.64$, and $n_3 = 0.02$. So, the process of soil strain softening under cyclic loading can be described by the following equation:

$$\delta = 1 - a \left( \frac{q_d}{q_f} \right)^{n_1} \left( 1 + \frac{q_s}{q_f} \right)^{n_2} \left( \frac{P}{P_a} \right)^{n_3} \log t, \quad (4)$$

where $P$ is the confining pressure, $q_s$ is the static deviator stress, $q_d$ is the dynamic deviator stress, and $P_a$ is the reference pressure, which is taken as 100 kPa here.

### 4. Equivalent Cyclic Creep Model

Figure 8 shows the stress-strain relationship. During the cyclic loading process, there is not only cumulative cyclic deformation but also reversible deformation at a peak strain point $\varepsilon_p$ and a valley strain point $\varepsilon_v$. However, the stress in the Merchant creep model is the static deviatoric stress, and there is no peak or valley point of the creep curve. Therefore, the stress in the Merchant creep model under cyclic loading should be the sum of static and dynamic deviator stresses, as expressed in the following equation:

$$\sigma = \sigma_s + \sigma_{cyc} \sin \omega t. \quad (5)$$

Figure 9(a) shows the strain softening process by the test. The test cyclic creep curve had already accounted for the influence of strain softening, which was reflected by the increase in the difference between the peak and valley strain values in each cycle as the number of cycles increased. Figure 9(b) shows the strain softening process obtained by the Merchant model. The difference between $\varepsilon_p$ and $\varepsilon_v$ remained the same in the cyclic accumulation process. Therefore, the soil strain softening index cannot be reflected in the Merchant model. Note that in the test, the ratio of the difference between $\varepsilon_p$ and $\varepsilon_v$ in the first cycle to that in the $N^{th}$ cycle of the cyclic accumulation process is the softening index. The cumulative cyclic deformation and strain softening can be more accurately reflected by multiplying the parameters in the Merchant model by the softening index. So, the equivalent cyclic creep model can describe the cyclic accumulation of soil, and the softening index can describe strain weakening characteristics of soft clay. Therefore, the improved Merchant model for equivalent cyclic creep is established as follows:

$$\varepsilon(t) = \sigma_s + \sigma_{cyc} \sin \omega t + \sigma_s + \sigma_{cyc} \left( 1 - e^{-\left( E_s/\eta_1 \right) t} \right). \quad (6)$$

Figure 10 shows the equivalent cyclic creep model. The equivalent cyclic creep model is different from the Merchant model. The load applied on the Merchant model is static load, while the load applied on the equivalent cyclic creep Merchant model is dynamic load. The cyclic cumulative deformation can be regarded as the creep process of soil. A stress control switch can be paralleled on the Kelvin body.
Figure 7: Curves of softening index-number of cycles. (a) $\sigma_3 = 50$ kPa, (b) $\sigma_3 = 100$ kPa, and (c) $\sigma_3 = 150$ kPa.

Figure 8: Stress time curve.
Figure 9: Strain curves—number of cycles. (a) Actual curve. (b) Traditional Merchant model.

Figure 10: Equivalent cyclic creep model.

Figure 11: Cumulative cyclic-strain curves ($\sigma_3 = 50$ kPa). (a) $q_s = 0.4q_f$, $q_d = 0.2q_f$ (b) $q_s = 0.5q_f$, $q_d = 0.3q_f$.
Figure 12: Cumulative cyclic-strain curves ($\sigma_3 = 100$ kPa). (a) $q_s = 0.4 \ q_f$, (b) $q_s = 0.5 \ q_f$.

Figure 13: Cumulative cyclic-strain curves ($\sigma_3 = 150$ kPa). (a) $q_s = 0.4 \ q_f$, (b) $q_s = 0.5 \ q_f$. 
The stress control switch can ensure the maximum cyclic stress. It can ensure that the stress applied on the Kelvin body is the maximum cyclic stress, and the stress applied on the hooker body is the sum of static deviator stress and dynamic deviator stress. So, the equivalent cyclic creep model can not only describe the strain cumulative deformation characteristics but also describe strain weakening characteristics of soft clay.

The 3D constitutive equations of the equivalent cyclic creep model extended from its 1D equations are given in the following equation:

\[
\begin{align*}
\varepsilon_n &= \varepsilon_n^e + \varepsilon_n^{ve}, \\
\sigma_n^e &= \sigma_n, \\
\sigma_n^{ve} &= \sigma_n - \sigma_{cy} \sin \omega t + \sigma_{cy}, \\
\sigma_n^{w} &= E_1 A^{-1} \varepsilon_n^{w} + \eta_1 A^{-1} \varepsilon_n^{v}, \\
\varepsilon_n^{ve} &= \frac{1}{\eta_1} A \sigma_n^{ve} - \frac{E_1}{\eta_1} \varepsilon_n^{ve}, \\
A &= \begin{bmatrix} 1 & -\mu & -\mu \\ -\mu & 1 & -\mu \\ -\mu & -\mu & 2(1 + \mu) \end{bmatrix}, \\
\Delta \varepsilon_n = \frac{\theta \Delta t_n A \Delta \sigma_n^{ve} + \varepsilon_n^{ve} \Delta t_n}{\eta_1} (1 - \theta E_1 \Delta t_n), \\
\Delta \sigma_n^{ve} &= (\Delta \sigma_n - \Delta t w \sigma_{cy} \cos \omega t), \\
\Delta \varepsilon_n^{ve} &= B \Delta t_n \varepsilon_n^{ve} + C_n \Delta \sigma_n^{ve},
\end{align*}
\]

where \(\varepsilon_n\) is the total strain at step \(n\), \(\varepsilon_n^e\) is the instantaneous elastic strain of the Hookean body at step \(n\), \(\varepsilon_n^{ve}\) is the viscoelastic strain of the Kelvin body at step \(n\), \(\sigma_n^{ve}\) is the viscoelastic strain rate of the Kelvin body at step \(n\), and \(\sigma_n, \sigma_n^e, \sigma_n^{ve}\) are the total stress, the stress of the Hookean body, and the stress of the Kelvin body, respectively, at step \(n\).

To implement the 3D constitutive model of the equivalent cyclic Merchant model through an FE subroutine, it is necessary to derive the stress-strain relationship in an incremental form for the equivalent cyclic Merchant model, i.e.,

\[
\begin{align*}
\Delta \varepsilon_n^{ve} &= \frac{\theta \Delta t_n A \Delta \sigma_n^{ve} + \varepsilon_n^{ve} \Delta t_n}{\eta_1} (1 - \theta E_1 \Delta t_n), \\
\Delta \sigma_n^{ve} &= (\Delta \sigma_n - \Delta t w \sigma_{cy} \cos \omega t), \\
\Delta \varepsilon_n^{ve} &= B \Delta t_n \varepsilon_n^{ve} + C_n \Delta \sigma_n^{ve},
\end{align*}
\]

where \(B = 1 - (\theta \Delta t_n E_1 / \eta_1)\) and \(C_n = (\theta t w / \eta_1)\)A.

For the current incremental step, there is

\[
\begin{align*}
\Delta \varepsilon_n &= E_0 A^{-1} \varepsilon_n, \\
\Delta \sigma_n^e &= \frac{1}{E_0} A \Delta \sigma_n, \\
\Delta \varepsilon_n^e &= \Delta \varepsilon_n + B \Delta t_n \varepsilon_n^{ve} + C_n (\Delta \sigma_n - \Delta t w \sigma_{cy} \cos \omega t), \\
\Delta \sigma_n &= \tilde{D} (\Delta \varepsilon_n - B \Delta t_n \varepsilon_n^{ve} + C_n \Delta t w \sigma_{cy} \cos \omega t), \\
\tilde{D} &= \left( C_n + \frac{1}{E_0} A \right)^{-1},
\end{align*}
\]

where \(\tilde{D}\) is the Jacobian matrix in ABAQUS. According to the Jacobian matrix \(\tilde{D}\), the equivalent cyclic Merchant model can be obtained to analyze the stress-strain relationship.

In ABAQUS, the 3D creep FE subroutine was validated through two analysis steps, i.e., application of the confining pressure and cyclic loading and using the strain-time curves for model validation. The total strain E33 was extracted, as shown in Figures 11–13. A comparison of FE calculation results and experimental results found generally a good agreement between the two results. Therefore, the equivalent cyclic creep model can be used to analyze the strain accumulation and softening process of soft clay during unloading.

5. Conclusions

In this study, the strain softening and strain cumulative deformation of soft clay were characterized through cyclic triaxial tests. An equivalent cyclic creep model was established to describe the cyclic cumulative strain of saturated soft clay and then used to analyze the strain cumulative deformation characteristics and strain softening characteristics of soft clay. The main conclusions are described as follows:

1. The strain accumulation and strain softening of the soil were low under low static stress ratio and low cyclic stress ratio. With the increase in the number of cycles and the cyclic stress ratio, the cumulative cyclic deformation gradually increased, developing quickly in the early stage and tending to stabilize in the later stage. The cyclic cumulative deformation of the soil increased with the increase in unloading stress and dynamic deviatoric stress, which was the same as the creep characteristics of the curve from the static unloading creep test under undrained conditions.
The degree of soil strain softening was low under low static deviatoric stress and the dynamic deviatoric stress ratio. With the increase in the number of cycles and the dynamic deviatoric stress ratio, the softening index gradually increased and was approximately linearly related to the logarithm of the number of cycles. Based on the experimental results, a softening index equation considering static and dynamic deviatoric stresses was proposed.

By introducing the Merchant model and considering the effects of dynamic deviatoric stress and the softening index, an equivalent cyclic creep model was established to describe the cyclic cumulative strain of saturated soft clay and validated through a comparison with the test results. Then, the model was extended to the 3D case, and the corresponding FE subroutine was developed and validated by the test results. Therefore, the equivalent cyclic creep model can be used to analyze the strain accumulation and strain softening process of soft clay.

Data Availability
The data used to support the findings of this study are included within the article.

Conflicts of Interest
The authors declare that there are no conflicts of interest.

Acknowledgments
The research was supported by the National Natural Science Foundation of China (51878160, 51678145, and 52078128).

References
[1] Y. Wang, C. H. Li, H. Liu, and J. Q. Han, "Fracture failure analysis of freeze–thawed granite containing natural fracture under uniaxial multi-level cyclic loads," Theoretical and Applied Fracture Mechanics, vol. 110, p. 102782, 2020.
[2] Q. X. Meng, W. Y. Xu, H. L. Wang, X. Y. Zhuang, W. C. Xie, and T. Rabczuk, "DigiSim - an open source software package for heterogeneous material modeling based on digital image processing." Advances in Engineering Software, vol. 148, p. 102836, 2020.
[3] Z. G. Tao, C. Zhu, M. C. He, and M. Karakus, "A physical modeling-based study on the control mechanisms of Negative Poisson’s ratio anchor cable on the stratified toppling deformation of anti-inclined slopes," International Journal of Rock Mechanics and Mining Sciences, vol. 138, p. 104632, 2021.
[4] R. Jiang, F. Dai, Y. Liu, and A. Li, "Fast marching method for microseismic source location in cavern-containing rockmass: performance analysis and engineering application," Engineering, vol. 4, 2020.
[5] Z. Tao, C. Zhu, X. Zheng et al., "Failure mechanisms of soft rock roadways in steeply inclined layered rock formations," Geomatics, Natural Hazards and Risk, vol. 9, no. 1, pp. 1186–1206, 2018.
[6] B. Li, R. Bao, Y. Wang, R. Liu, and C. Zhao, "Permeability evolution of two-dimensional fracture networks during shear under constant normal stiffness boundary conditions," Rock Mechanics and Rock Engineering, vol. 54, no. 3, pp. 1–20, 2021.
[7] Q. Wang, H. K. Gao, B. Jiang, S. C. Li, M. C. He, and Q. Qin, "In-situ test and bolt-grouting design evaluation method of underground engineering based on digital drilling.," International Journal of Rock Mechanics and Mining Sciences, vol. 138, p. 104575, 2021.
[8] Q. Wang, Q. Qin, B. Jiang et al., "Mechanized construction of fabricated arches for large-diameter tunnels," Automation in Construction, vol. 124, p. 103583, 2021.
[9] K. H. Andersen, "Bearing capacity under cyclic loading - offshore, along the coast, and on land," Canadian Geotechnical Journal, vol. 46, no. 5, pp. 513–535, 2009.
[10] K. L. Lee, "Cyclic strength of a sensitive clay of eastern Canada," Canadian Geotechnical Journal, vol. 16, no. 1, pp. 163–176, 1979.
[11] R. J. Mitchell and R. D. King, "Cyclic loading of an Ottawa area champlain sea clay," Canadian Geotechnical Journal, vol. 14, no. 1, pp. 52–63, 2011.
[12] M. Hyodo, K. Yasuhara, and K. Hirao, "Prediction of clay behaviour in undrained and partially drained cyclic triaxial tests," Soils and Foundations, vol. 32, no. 4, pp. 117–127, 1992.
[13] K. Yasuhara, "Postcyclic undrained strength for cohesive soils," Journal of Geotechnical Engineering, vol. 120, no. 11, pp. 1961–1979, 1994.
[14] J. Zhou and X. N. Gong, "Study on strain softening unsaturated soft clay under cyclic loading," China Civil Engineering Journal, vol. 33, no. 6, pp. 75–78, 2000.
[15] N. Matasovic and M. Vucetic, "A pore pressure model for cyclic straining of clay," Soils and Foundations, vol. 32, no. 3, pp. 156–173, 1992.
[16] M. L. Yao and S. L. L. Nie, "A model for calculation deformation of saturated soft clay," Chinese Journal of Hydraulic Engineering, vol. 25, no. 7, pp. 51–55, 1994.
[17] Y. Q. Cai, W. Liu, C. J. Xu, and H. C. Huang, "Dynamic stress-strain relationship of soft clay based on modified Iwan model," Chinese Journal of Geotechnical Engineering, vol. 29, no. 9, pp. 1314–1319, 2007.
[18] J. Wang and Y. Q. Cai, "Study on accumulative plastic under cyclic strain model of soft clay loading," Chinese Journal of Rock Mechanics and Engineering, vol. 27, no. 2, pp. 331–338, 2008.
[19] C. L. Monismith, N. Ogawa, and C. R. Freeme, "Permanent deformation characteristics of subgrade soils due to repeated loading," Transport Research Record, vol. 537, pp. 1–17, 1975.
[20] D. Li and E. T. Selig, "Cumulative plastic deformation for fine-grained subgrade soils," Journal of Geotechnical Engineering, vol. 122, no. 12, pp. 1006–1013, 1996.
[21] J.-C. Chai and N. Miura, "Traffic-load-induced permanent deformation of road on soft subsoil," Journal of Geotechnical and Geoenvironmental Engineering, vol. 128, no. 11, pp. 907–916, 2002.
[22] M. S. Huang, J. J. Li, and X. Z. Li, "Cumulative deformation behavior of soft clay in cyclic undrained tests," Chinese Journal of Geotechnical Engineering, vol. 28, no. 7, pp. 891–895, 2006.
[23] A. F. L. Hyde and S. F. Brown, "The plastic deformation of a silty clay under creep and repeated loading," Géotechnique, vol. 26, no. 1, pp. 173–184, 1976.
[24] K. Yasuhara, K. Hirao, and A. F. L. Hyde, "Effects of cyclic loading on undrained strength and compressibility of clay," Soils and Foundations, vol. 32, no. 1, pp. 100–116, 1992.
[25] Y. Z. Wang, J. L. Qi, Y. H. Dong, and Y. C. Long, "Research on cumulative deformation of saturated soft clay under cyclic..."
loads,” *Journal of Waterway and Harbor*, vol. 38, no. 3, pp. 286–290, 2017.

[26] Y. H. Dong, *Creep Behavior of Saturated Soft Clay under Long-Term Cyclic Loading*, Tianjin University, Tianjin, China, 2013.

[27] J. L. Qi, *Research on Cumulative Deformation of Saturated Soft Clay Considering Creep Characteristics under Long Term Cyclic Loads*, Tianjin University, Tianjin, China, 2016.

[28] H. Fujiwara, T. Yamanouchi, K. Yasuhara, and S. Ue, "Consolidation of alluvial clay under repeated loading," *Soils and Foundations*, vol. 25, no. 3, pp. 19–30, 2008.

[29] D. F. Zhu, H. W. Huang, and J. H. Yin, “Cyclic creep behavior of saturated soft clay,” *Chinese Journal of Geotechnical Engineering*, vol. 9, pp. 1060–1064, 2005.

[30] X. Z. Li and M. S. Huang, “A bounding surface model for creeping soft clays under cyclic loading,” *Chinese Journal of Geotechnical Engineering*, vol. 29, no. 2, pp. 251–254, 2007.

[31] L. Guo, Y. Q. Cai, J. Wang, and C. Gu, “Long-term cyclic strain behavior of Wenzhou structural soft clay,” *Chinese Journal of Geotechnical Engineering*, vol. 34, no. 12, pp. 2249–2254, 2012.

[32] I. M. Idriss, R. Dobry, and R. D. Singh, "Nonlinear behavior of soft clays during cyclic loading," *Journal of the Geotechnical Engineering Division*, vol. 104, no. 12, pp. 1427–1447, 1978.