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Relationship between monotonic and cyclic behavior of saturated soft clay in undrained triaxial compression tests

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Abstract

Cyclic loading-induced deformation of soil is a common problem in the engineering practice. In the current practice, however, monotonic triaxial tests are more commonly used in the practice, due to the availability of apparatus and ease of operation. Thus, it will be very useful and practical if the monotonic triaxial tests can be used to evaluate the behavior of soil under cyclic loading. This study aims to find an explicit relationship between monotonic and cyclic behavior of saturated soft clay. Six monotonic and nine cyclic triaxial compression tests were conducted on undisturbed saturated soft clay under an undrained condition. The test results showed that the

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13 monotonic and cyclic tests shared the same stress-strain surface in a three-dimensional
14 space $p' - q - \varepsilon_a$. It is also found possible to evaluate the effective stress states of
15 cyclic tests at two specific numbers of cycles, using corresponding monotonic tests.
16 Based on these two findings, a simple procedure was then proposed to predict the peak
17 axial strain for the saturated soft clay under different cyclic loadings based on the
18 monotonic tests and only one cyclic test, which was further verified against more test
19 data from the previous literature.

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21 **Key words:** soft clay; cyclic loading; triaxial tests; relationship; axial strain

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1. Introduction

Cyclic loading-induced deformation of soil is a common problem in the engineering practice. Traffic loading and wave loading are two examples of cyclic loading. After long-term cyclic loading, excess settlement sometimes occurs on the infrastructures. Take the Shanghai Metro Line-1 as an example, the metro line settled very little (smaller than 6 mm) at the end of the construction and prior to the operational service (totally 27 months). Nonetheless, excessive settlement of 60 mm developed during the first eight months of service due to the cyclic loading (Xiao et al. 2014). The existence of this kind of problems is closely related to the detrimental effect of cyclic loading, which has yet to be fully considered during design or numerical analysis in the geotechnical practices. Therefore, the effect of cyclic loading on the soil behavior and the performance of infrastructures cannot be ignored in engineering practice.

In recent years, the cyclic behavior of soils has received significant attention among researchers. With better understanding of soil behavior and improved experimental and numerical capabilities, researchers were capable of simulating and analyzing more complex problems under cyclic loading (Cai et al. 2018; Cai et al. 2012; Guo et al. 2017; Matešić and Vucetic 2003; Moses et al. 2003; Wichtmann et al. 2013; Wu et al. 2017). For example, Andersen (1980) conducted cyclic triaxial tests on Drammen clay to investigate the adverse effects of wave loading using a trapezoidal loading pulse. Thirty years later, Tong et al. (2010) were able to control the continuous principal stress rotation on soils by a cyclic torsional shear device, which maintained the constant value

of deviatoric stress. The stress states simulated are much closer to those under wave loading in engineering practice. In the current practice, however, monotonic triaxial tests, such as the consolidated undrained triaxial tests, are more commonly used in the practice due to the availability of apparatus and ease of operation (Bolton and Wilson 1989; Li et al. 2017), and their results are also useful and more accessible for engineering practice including design and numerical analysis. Therefore, it will be very useful and practical if the monotonic triaxial tests can be used to evaluate the behavior of soil under cyclic loading.

Under most circumstances, the cyclic deformation behavior of soils has been investigated independently of the monotonic deformation behavior (Cai et al. 2012; Guo et al. 2013; Moses et al. 2003; Vucetic and Dobry 1988). Limited studies have been conducted to evaluate the relationship between the monotonic and cyclic test results, especially for the soft clays. Bolton and Wilson (1989) introduced the parameters determined from the monotonic tests into the constitutive model for cyclic loading of sand. However, this model does not simulate the behavior of sand well at a large number of cycles (Niemunis et al. 2005). Yasuhara et al. (1982) found the curves of shear strain versus effective stress ratio in monotonic triaxial tests and those of staged cyclic tests on Ariake clay have similar patterns. Tatsuoka et al. (1995) and Konstadinou and Georgiannou (2013) found the curves of shear strain versus shear modulus in the monotonic and cyclic tests on sand are close to each other. Mao and Fahey (2003) and Li et al. (2017) found some correlations between the stress paths of

64 cyclic and monotonic tests on different soils. The above researches were rather
65 qualitative, but shows that it is possible to establish an explicit relationship between
66 cyclic and monotonic test results.

67 This study aimed to establish the relationship between the monotonic and cyclic
68 test results for clays. Therefore, both monotonic and cyclic triaxial tests were conducted
69 on a saturated soft clay under an undrained condition. Based on the test results, an
70 explicit relationship between monotonic and cyclic test results for the clay was
71 established. And it was further verified against more test data from the previous
72 literature.

73 **2. Sample preparation and test apparatus**

74 ***2.1. Soil properties***

75 The clay samples were obtained from the bottom of an excavation pit at a depth of
76 approximately 3 m in Wenzhou, China. The ground water level is -0.8 m. They were
77 obtained by gently pressing thin-wall tubes (160 mm in diameter and 300 mm in length)
78 into a level ground inside the excavation pit (see Figure 1). Each sample was carefully
79 excavated out and sealed on both ends. Then the samples were transported to the
80 laboratory and stored in a moist room. Similar methods of sampling have been adopted
81 by other researchers to obtain high-quality in-situ soil samples (Guo et al. 2013; Li et
82 al. 2011). Table 1 shows the basic properties of the clay obtained from this site.

83 ***2.2. Tests procedures***

An electromechanical dynamic triaxial testing system (DYNTTS) manufactured by GDS Instruments Ltd., UK, was employed in this study. In this apparatus, the deviatoric stress is applied by a servo-loading system and the confining pressure is applied through an oil pressure-type piston. Figure 2 shows a schematic diagram of the apparatus.

The deviatoric stress and confining pressure can vary simultaneously with a range of frequencies between 0.001 and 10 Hz. This apparatus has good control, measurement precision, and real-time data acquisition function and can be used to perform cyclic loading tests for a large number of cycles.

All the triaxial specimens used in this study had a dimension of 100 mm in height and 50 mm in diameter. Each specimen was installed on a base inside a chamber. At the beginning of the test, the specimen was saturated at a back pressure of 300 kPa with an effective confining pressure of 20 kPa for 24 hours, and the Skempton parameter B (Skempton 1954) was greater than 0.97 at the end of the preparation stage. After that, the test specimen was isotropically consolidated under a desired effective confining pressure for approximately 24 hours until the volumetric strain of the specimen became less than 0.006% per hour, which was small enough to affect the overall measurement. After that, both the monotonic and cyclic triaxial tests were conducted on the consolidated specimens under an undrained condition.

Table 2 presents six strain-controlled monotonic triaxial tests carried out at a strain rate of 0.01 mm/min, under different confining pressures. Nine cyclic tests were also

conducted. The loading waveform of the axial stress was a positive semi-sinusoidal function at a frequency of 1 Hz, which is considered as representative for typical traffic loading (Chazallon et al. 2006; Guo et al. 2013). The confining pressure was kept constant during each test. Table 3 shows the scheme of cyclic triaxial test. The device recorded 50 data per cycle (i.e., one data per 0.02 s). All specimens were subjected up to 50,000 loading cycles and did not fail even to the end. In this paper, each monotonic test is labeled as “M” (for the monotonic test) followed by three digits for the initial effective confining pressure. Each cyclic test is labeled as “C” (for the cyclic test) followed by a series symbol (“A”, “B”, or “C”) and two digits for the cyclic deviatoric stress amplitude.

3. Test results

3.1. Monotonic triaxial tests

As compared with a stress-controlled test, the strain-controlled test allows for easier identification of the failure point for each test. The stress state of a soil specimen is often described in terms of the deviatoric stress q and the effective mean principal stress p' :

$$q = \sigma_1' - \sigma_3' \quad (1)$$

$$p' = \frac{1}{3}(\sigma_1' + 2\sigma_3') \quad (2)$$

where σ_1' is the effective axial stress and σ_3' is the effective confining pressure in triaxial tests.

Figure 3 shows the development of deviatoric stress q and pore water pressure versus the axial strain for each specimen in the monotonic test. The test data corresponding to the axial strain higher than 20 % are not presented in this figure since significant bulging of the specimen happened and the test data are no longer reliable. For each test, under different confining pressure, the deviatoric stress increased quickly in the initial stage of shearing, and then peaked at an axial strain of approximately 10 %, followed by slight softening behavior. In this study, the peak stress state is considered as the failure point for each specimen, which is marked with an arrow on each curve in Figure 3 (a). The axial strain at the failure in this study matches well with the criterion ($\varepsilon_a = 10\%$) for the peak stress state of undisturbed natural clays suggested by other researchers (Hyodo et al. 1999; Wichtmann et al. 2013). Table 2 summarizes the deviatoric stresses at failure q_f , as well as the undrained strength ratios $S_u/\sigma'_{vo} = q_f/2p_o'$, for these six monotonic tests.

Figure 4 presents the effective stress paths (ESPs) of the monotonic triaxial tests in the space $p' - q$. The failure envelop in this study, however, is non-linear at lower confining pressures (approximately lower than 100 kPa), due to the soil structure in natural sedimentary clays (Panda and Rao 1998). To simplify the procedure proposed in this study, the critical state line (CSL) was plotted as a straight line connecting the failure points of tests at higher confining pressures (higher than 100 kPa). The CSL had a slope of 1.07 ($\phi' = 27^\circ$).

3.2. Cyclic triaxial tests

Three series of cyclic triaxial tests were then conducted under different confining pressures at a frequency of 1 Hz. Figure 5 presents the typical results of the cyclic tests at the confining pressure $p_o' = 100$ kPa. Figure 5 (a) shows the development of the axial strain with the number of cycles and defines the permanent axial strain, ε^p , the resilient axial strain, ε^r , and the peak axial strain, ε^{peak} . With the increasing number of cycles, both the peak and permanent axial strains increased rapidly at the beginning, but their rates of the increase gradually decreased until reaching stable values (see Figure 5 (c) (d)). The resilient strain, however, underwent little change during the entire process as shown in Figure 5 (b). Therefore, the peak axial strain not only shows the upper strain limit during each loading cycle but also represents the permanent axial strain to some extent.

Figure 6 presents the peak axial strain versus number of cycles at different confining pressures in a log-log plot. The relationship between the peak axial strain and the number of cycles is approximately a straight line, especially when $N > 1000$. This finding is similar to that for the permanent axial strain obtained by other researchers (Cai et al. 2015; Guo et al. 2013; Sweere 1990). In their studies, the relationship between the permanent axial strain and the number of cycles for a soil under cyclic loading is a simple power function. Several prediction models were developed based on this function by previous researchers (Cai et al. 2015; Guo et al. 2013). Therefore, for a large number of loading cycles, a power function could also be used to express the peak axial strain as follows:

$$\varepsilon^{peak} = aN^b \quad (3)$$

where a and b are the parameters depend on the stress conditions and soil properties.

3.3. Relationship between monotonic and cyclic test results

3.3.1 Effective stress paths

Figure 7 presents the development of pore water pressure versus number of loading cycles in the cyclic tests under different confining pressures. The pore pressure increases rapidly in the beginning and increases slowly in the following cycles. Figure 8 shows the relationship between the peak pore water pressure versus cyclic deviatoric stress after a certain number of cycles. Regardless of the confining pressure, the pore water pressure increases almost linearly with the cyclic stress amplitudes, especially for a larger number of cycles (i.e., the data for the 1000th cycle are more scattered than those for the 50,000th cycle). Similar observation has also been reported in the literature for clay specimens in undrained cyclic tests (Ansal and Erken 1989; Sangrey et al. 1969).

The linear relationship between the pore pressure and deviatoric cyclic stress results in a similar linear relationship between the effective mean principal stress and the deviatoric cyclic stress. For example, Figure 9 presents the effective stress paths for the 5,000th and 50,000th cycles in each cyclic test under the confining pressure of 100 kPa. The top points of the ESP loops (corresponding to the peak pore water pressure) for each test at the same number of cycles are aligned on the same line in the space $p' - q$. The line for the 5,000th cycles is almost parallel to that for the 50,000th cycles. The

increase of the number of cycles moved the line to the left. [Sangrey et al. \(1969\)](#) had found the existence of this line for the saturated clay after a large number of loading cycles. They termed this line as an “equilibrium line”. [Shahin et al. \(2011\)](#) also found the “equilibrium line” exist in the one-way cyclic triaxial tests on kaolinite clay. In this study, the linearity was found exist in the cyclic tests at different number of cycles, not necessarily the cycle at equilibrium state. These straight lines are termed as “quasi equilibrium line”.

Figure 10 shows the top points for the 1000th and 50,000th cycles under three different confining pressures in the cyclic tests, and also the ESP of monotonic triaxial tests under the corresponding confining pressures. The quasi equilibrium line for the 50,000th cycle connects the start point and the failure point of the ESP for the corresponding monotonic test, regardless of the confining pressure. Also, the top points for the 1000th cycles in the cyclic test lie on the ESP in the corresponding monotonic test for all the confining pressures. Namely, the pore water pressure in the cyclic test before 1,000 cycles is always lower than that in the monotonic test at the same deviatoric stress. Take the specimen CC45 as an example, the ESP in the cyclic test moved up and down across the ESP in the monotonic test within the first 1000 cycles as shown in Figure 10, indicating that the pore water pressure generated under cyclic loading was sometimes lower than that under monotonic loading. After 1000 cycles, the pore water pressure developed sufficiently and is always larger than that of corresponding monotonic test at the same deviatoric stress, and the strain became linear with the

number of cycles in a log-log plot as shown in Figure 6. Similar observation was reported in the previous study (Guo et al. 2013).

At $N_1 = 1000$ and $N_2 = 50,000$, the stress states of the soil specimens in the cyclic tests can be estimated from the ESPs of the monotonic tests. Please note the values of N_1 and N_2 may change for a different soil. To determine these values, we only need to conduct one or two cyclic tests and compare the results with corresponding monotonic test. The peak pore water pressure in each cyclic test after N_1 cycles, u_1^{peak} , is equal to the pore water pressure in the monotonic test when q reaches the same value as q^{cyc} :

$$u_1^{peak} = u|_{q=q^{cyc}} \quad (4)$$

where u and q are the pore water pressure and the deviatoric stress for an arbitrary point on the ESP in the monotonic test, respectively.

The peak pore water pressure in each cyclic test after N_2 cycles, u_2^{peak} , can be expressed as follows:

$$u_2^{peak} = \frac{u_f}{q_f} q^{cyc} \quad (5)$$

where u_f and q_f are the pore water pressure and the deviatoric stress, when the specimen failed in the corresponding monotonic test, respectively.

3.3.2 Stress-strain relationship

The generation of pore water pressure and the deformation behavior of soil are closely related and influence each other. Yasuhara et al. (1982) found that the relationship between pore water pressure and strain of clay is independent of the loading

230 method (monotonic or cyclic). [Hyde et al. \(1993\)](#) proposed a unique relationship
231 between normalized pore water pressure rate and strain rate of clay in cyclic tests.
232 Figure 11 shows the relationships between the (peak) pore water pressure ratio ($r_u =$
233 u/p_o') and the (peak) axial strain of monotonic and cyclic tests in this study. The curves
234 of cyclic tests are close to those of monotonic tests, but the match is not enough to
235 establish an explicit relationship between the pore water pressure and the strain.

236 In the past studies, the critical state line (CSL) connecting the failure points of
237 monotonic tests was often used to determine the dynamic strength of soil ([Gu 2012](#);
238 [Hyde and Brown 1976](#); [Hyde and Ward 1985](#); [Hyde et al. 1993](#); [Moses and Rao 2003](#)).
239 In terms of the geometrical relationship between ESP and CSL in the space $p' - q$,
240 [Kaggwa et al. \(1991\)](#) related the cyclic shear modulus and the effective stress paths of
241 calcareous sand under multistage cyclic loading, and [Hyde et al. \(1993\)](#) proposed a
242 failure criterion for the pore water pressure in the saturated clay in cyclic tests. As a
243 generalization of the criterion, [Wang et al. \(2013\)](#) developed another model that
244 describes and predicts the stress-strain relationship of clay under cyclic loading. The
245 horizontal distance from the top point of ESP at each cycle to CSL is defined as the
246 critical distance d . Then a unique exponential relationship between d and the peak axial
247 strain was established for a given confining pressure, regardless of the cyclic stress level.
248 Based on this model, similar investigation was also conducted on both cyclic and
249 monotonic tests in this study.

Figure 12 presents the horizontal distance from an arbitrary locus on the ESP in the space $p' - q$ to the CSL, which is here defined as an “allowable stress”, σ^a , and can be expressed as follows:

$$\sigma_m^{as} = p_o' + \left(\frac{1}{3} - \frac{u}{q} - \frac{1}{M}\right)q, \quad \text{for monotonic tests} \quad (6)$$

$$\sigma_c^{as} = p_o' + \left(\frac{1}{3} - \frac{u^{peak}}{q^{cyc}} - \frac{1}{M}\right)q^{cyc}, \quad \text{for cyclic tests} \quad (7)$$

Figure 13 presents the relationships between the allowable stress and the peak axial strain for both cyclic and monotonic tests in the semi-log plot. For both the monotonic and cyclic tests, as the strain develops, the allowable stress decreases. And the data of cyclic tests merged well with the data of corresponding monotonic tests for all the number of cycles. This result indicates that for a specimen with a certain amount of peak axial strain caused by cyclic loading, the corresponding monotonic loading can cause an equal axial strain of another specimen, as long as their allowable stresses are the same, even though the ESPs of the monotonic and cyclic tests are totally different. Therefore, the axial strains developing in the monotonic and cyclic tests shared the same surface in the three-dimensional space $p' - q - \varepsilon_a$, as shown in Figure 14. This surface is approximately parallel to the CSL. The space $\frac{M}{\sqrt{M^2 + 1}}\sigma_m^{as} - \varepsilon_a$, or $\frac{M}{\sqrt{M^2 + 1}}\sigma_c^{as} - \varepsilon_a^{peak}$ (similar to the coordinates of Figure 13) can be considered as one section perpendicular to the CSL that has all projection data.

Since the excellent relationship found between the monotonic test results and cyclic test results, the solid line represents the monotonic test in Figure 13 is referred to as the

270 “feature line” for the cyclic test. And the peak axial strain of the soil after cyclic loading
 271 can be predicted by utilizing the monotonic test results, in the following steps:

272 For $N_1 = 1000$, we have $\sigma_c^{as} = \sigma_m^{as}$, then $\varepsilon_{a,1}^{peak} = \varepsilon_a|_{q=q^{cyc}}$, i.e., $\varepsilon_{a,1}^{peak}$ is equal to
 273 the ε_a in the monotonic test as its q reaches the value of q^{cyc} .

274 For $N_2 = 50,000$, substituting Equation (5) into Equation (7) leads to the allowable
 275 stress expressed by the results of the monotonic test as follows:

$$276 \quad \sigma_{c,2}^{as} = p_o' + \left(\frac{1}{3} - \frac{u_f}{q_f} - \frac{1}{M} \right) q^{cyc} \quad (8)$$

277 Then the peak axial strain $\varepsilon_{a,2}^{peak}$ can be obtained from the ε_a in the monotonic
 278 tests as its σ_m^{as} reaches the value of $\sigma_{c,2}^{as}$.

279 Since a good power function relationship exists between the peak axial strain and
 280 the number of cycles, the parameters a and b in Equation (3) can be expressed as follows:

$$281 \quad a = \frac{\varepsilon_{a,1}^{peak}}{N_1^b} \quad (9)$$

$$282 \quad b = \frac{\ln \varepsilon_{a,1}^{peak} - \ln \varepsilon_{a,2}^{peak}}{\ln N_1 - \ln N_2} \quad (10)$$

283 where $\varepsilon_{a,1}^{peak}$ and $\varepsilon_{a,2}^{peak}$ are obtained only from the monotonic test results, and N_1
 284 and N_2 can be determined by comparing one cyclic test and corresponding monotonic
 285 test. Once determined, these two N values are universal to all stress levels and all
 286 confining pressures for one specific soil. In this study, $N_1 = 1,000$ and $N_2 = 50,000$. The
 287 N_1 and N_2 values for different soils will be investigated in the future and beyond the
 288 scope of this study.

289 3.3.3 Further generalization

Figure 15 shows that the relationship between the allowable stress and the axial strain for all six monotonic tests under different confining pressures. The relationship curve, namely the “feature line”, becomes flatter as the confining pressure increases. Note that, to simplify the method, the CSL was plotted as a straight line. For M020, M050 and M100, the specimens was slightly over-consolidated, and therefore, has not failed as they reaching the straight CSL. However, the shape of CSL has little effect on the relationships between monotonic and cyclic tests, as long as they share the same CSL. Therefore, the findings also apply to the tests of slightly overconsolidated clay. The feature line can be approximately expressed in the following exponential function:

$$\varepsilon_a = e^{\frac{\alpha - \sigma_m^{as}}{\beta}} \quad (11)$$

where α represents the stress on the “feature line” at $\varepsilon_a = 1\%$, and $1/\beta$ denotes the slope of the “feature line”.

Figure 16 plots the data points that represent each monotonic test result and the feature line in the space $\alpha - \beta - p'_o$. The paired monotonic and cyclic data points nearly coincide, indicating that the soil behavior under cyclic loading can be evaluated by that under monotonic loading. Therefore, the parameters for the feature line of an unknown monotonic test can also be calculated by linear interpolation base on the data of other known monotonic test results. Figure 15 shows that the parameters for the feature line can be calculated using the following expression:

$$p'_o = 14.85 + 1.80\alpha = -5.94 + 4.08\beta \quad (12)$$

The linear expression between the parameters α and β makes easier for the application of this proposed method to actual projects.

3.4. Procedure and Verification

To predict the peak axial strain of a specific clay after a large number of loading cycles under different confining pressures and cyclic stress levels, only a series of undrained monotonic triaxial tests need to be conducted, and one cyclic test is necessary to calibrate the model. Then, the peak axial strain under any cyclic stress level, q^{cyc} , and confining pressure, p'_o , can be obtained using the following procedure:

1) Determine N_1 and N_2 by comparing the ESPs of the cyclic test and the corresponding monotonic test. N_1 is the number of cycle as the ESP of cyclic loading tests crossing the ESP of the corresponding monotonic test. N_2 is the number of cycle as the ESP of cyclic loading tests crossing the connection line between the start point and the failure point of corresponding monotonic test.

2) Find the $\varepsilon_{a,1}^{peak}$ which is equal to ε_a in the monotonic test as its q reaches the value of q^{cyc} .

3) Calculate the allowable stress $\sigma_{c,2}^{as}$ by Equation (9), then find the corresponding $\varepsilon_{a,2}^{peak}$ by linear interpolation on the “feature line” or using Equation (11).

4) Determine the parameters a and b in the prediction model (i.e., Equation (3)) using Equations (9) and (10). Generate a predicted curve of axial strain versus number of cycles.

In addition, if there is no test data available for a certain confining pressure, the “feature line” can be determined by Equation (12) according to the results from the monotonic tests at other confining pressures.

To verify the proposed method, the experimental data from some previous researches were used:

(1) [Wang et al. \(2013\)](#) conducted series of monotonic triaxial tests (strain rate: 0.01mm/min) and cyclic triaxial tests (frequency: 1 Hz) in undrained condition on saturated Wenzhou clay. The method we proposed applied to the available data in their paper. Table 4 presents the calculations in the process of predicting peak axial strain of cyclic tests. Figure 17 presents the predicted peak axial strains for cyclic tests based on the results of four monotonic triaxial tests and one cyclic triaxial tests. The predicted curves match well with the measured data.

(2) [Huang et al. \(2020\)](#) conducted three cyclic triaxial tests (frequency: 1 Hz) in undrained condition on saturated Ningbo clay (Ningbo is also a coastal city that lies to the north of Wenzhou). Based on the available data, Figure 18 shows that there is also an exponential relationship between the allowable stress and axial strain for cyclic tests on Ningbo clay, and the curve almost coincides with that of monotonic test at the same confining pressure on Wenzhou clay.

(3) [Gao et al. \(2018\)](#) conducted series of monotonic triaxial tests (strain rate: 0.0057mm/min) and cyclic triaxial tests (frequency: 1 Hz) in undrained

condition on saturated Kunming sapric peats with different organic contents (Kunming is an inland city on the plateau of west China). Figure 19 also shows fair coincidence between the monotonic test and cyclic tests at a certain confining pressure with certain organic content.

4. Discussion

The generalization of this method is worth to further study. According to Sangrey et al. (1969), the “equilibrium line” exists not only for isotropically consolidated specimens, but also for anisotropically consolidated specimens and over-consolidated specimens. More experimental studies should be done in the future to further investigate the existence of the relationships between monotonic and cyclic behavior of soils under other consolidation conditions. Furthermore, since the relationship we proposed is independent on the loading types, it is also possible that the relationship could apply to the other cyclic loading types, such as traffic loading considering principal stress rotation. Further investigation can be done.

5. Conclusions

This study aimed to establish the relationship between the monotonic and cyclic test results on soft clay. A series of monotonic tests and cyclic triaxial tests were conducted on the undrained saturated undisturbed soft clay.

Based on the test results, it is found that the axial strains in the monotonic and cyclic tests shared the same stress-strain surface in the three-dimensional space $p' - q - \varepsilon_a$. Also, it is found possible to evaluate two special effective stress states of a

cyclic test using the corresponding monotonic test. Therefore, this study established an explicit relationship between cyclic and monotonic test results. A simple procedure was proposed, by which the peak axial strain for the undisturbed saturated soft clay under different one-way cyclic loading levels can be predicted based on several monotonic tests and only one cyclic test. The prediction of this method was verified by the test data from the previous researches.

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Table Captions

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Table 3 Summary of cyclic triaxial tests

Table 4 Calculations in the process of predicting peak axial strain of cyclic tests
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Table 1 Basic properties of the Wenzhou clay

Basic Properties		Value
Specific gravity	G_s	2.75
Natural density	ρ (g/cm ³)	1.68~1.71
Natural water content	w (%)	56-59
Liquid limit	w_L (%)	64
Plasticity index	I_p	36
Initial void ratio	e_0	1.55~1.59
Clay fraction	(%)	55
Silt fraction	(%)	41
Sensitivity	S_t	5.9
Pre-consolidation Pressure	p_c (kPa)	67

Table 2 Summary of monotonic triaxial tests

Test ID	Initial effective confining pressure	Deviatoric stress at failure	Undrained strength ratios
	p_o' (kPa)	q_f (kPa)	$S_u/\sigma'_{vo} = q_f/2p_o'$
M020	20	29.3	0.733
M050	50	44.7	0.447
M100	100	71.8	0.359
M200	200	116.0	0.290
M250	250	138.4	0.277
M300	300	164.6	0.274

Table 3 Summary of cyclic triaxial tests

Test ID		Cyclic deviatoric stress	Initial effective confining pressure	Cyclic deviatoric stress ratios
		q^{cyc} (kPa)	p_o' (kPa)	$CSR = q^{cyc}/2p_o'$
Series A	CA22	22	50	0.220
	CA27	27		0.270
	CA30	30		0.300
Series B	CB29	29	100	0.145
	CB43	43		0.215
	CB55	55		0.275
Series C	CC45	45	200	0.113
	CC54	54		0.135
	CC86	86		0.215

Table 4 Calculations in the process of predicting peak axial strain of cyclic tests in Wang et al. (2013)

Given data					Process				Results	
p'_o (kPa)	q_f (kPa)	q^{cyc} (kPa)	CSR_f	CSR	Monotonic tests				Cyclic tests	
					$\sigma_{c,1}^{as}$ (kPa)	$\sigma_{c,2}^{as}$ (kPa)	$\varepsilon_{a,1}^{peak}$ (%)	$\varepsilon_{a,2}^{peak}$ (%)	a	b
50	43.35	13.87	0.16	0.139	36.16	29.24	0.2010	0.3869	0.0632	0.1675
		21.50	0.25	0.215	26.71	17.83	0.4999	1.0120	0.1439	0.1803
		27.74	0.32	0.277	17.89	8.49	1.0205	1.9581	0.3229	0.1666
100	69.25	23.00	0.14	0.115	79.89	60.49	0.1893	0.5548	0.0284	0.2748
		27.70	0.20	0.139	74.53	52.42	0.2720	0.7894	0.0414	0.2724
		38.78	0.28	0.194	58.12	33.39	0.6273	1.6328	0.1158	0.2445
		45.71	0.33	0.229	46.46	21.49	1.0135	2.4745	0.2096	0.2282
		55.40	0.40	0.277	26.40	4.84	2.0973	4.6978	0.5049	0.2061
		58.17	0.42	0.291	20.29	0.08	2.5990	5.8054	0.6288	0.2054
200	112.46	42.73	0.19	0.107	160.71	125.39	0.2196	0.5916	0.0382	0.2533
		71.97	0.32	0.180	116.93	74.33	0.7260	1.7313	0.1565	0.2222
		85.47	0.38	0.214	90.14	50.77	1.2620	2.7899	0.3110	0.2028

Notes: (1) $CSR_f = q^{cyc}/2q_f$ (used in Wang et al. (2013)); $CSR = q^{cyc}/2p'_o$ (used in this study).

(2) $N_1 = 1000$ and $N_2 = 50,000$ (obtained by comparing the monotonic tests and a cyclic test).

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