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Deformation Behavior of Saturated Marine Silt under Principal Stress Rotation as Induced by Wave Loading

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Abstract: An experimental study aimed at providing insights into the cyclic deformation behavior of saturated marine silt under principal rotation, as induced by wave loading, is presented. Using the GDS hollow cylinder apparatus, a series of undrained tests are performed and the specimens at identical initial states are subjected to combined axial–torsional cyclic loading that imposes different levels of stress rotation. The cumulative generalized shear strain γ_g is used to describe the deformation of the silt under complex stress paths. The test results show that the cumulative generalized shear strain is significantly dependent on the cyclic stress ratio (CSR) and cyclic loading amplitude ratio δ . The cumulative generalized shear strain increases with the increase in CSR and decreases with the increase in δ . The development trend of γ_g can be well predicted through the correct Monismith model in the non-liquefaction silt, with a low error that is generally less than 10%.

Keywords: wave loading; saturated silt; cumulative generalized shear strain; cyclic stress ratio; cyclic loading amplitude ratio

Citation: Cui, L.; Sheng, Q.; Niu, Z.; Chang, L. Deformation Behavior of Saturated Marine Silt under Principal Stress Rotation as Induced by Wave Loading. *Appl. Sci.* **2021**, *11*, 9458. <https://doi.org/10.3390/app11209458>

Academic Editor: Daniel Dias

Received: 6 August 2021

Accepted: 29 September 2021

Published: 12 October 2021

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1. Introduction

Soft silty soils are widespread in deltas, estuaries and coastal zones, and are prone to liquefaction under cyclic wave loading because of their high natural water content and weak permeability [1,2], which may result in serious damage to marine structures, such as the instability of gravity platforms [3], large horizontal displacements of immersed tunnels [4], tilting of caissons [5], shear failure of breakwater slopes [6], shear and tensile failures of bolts [7], and floating up of pipelines [8]. Such silty soils are also characterized by high anisotropy of strength properties, mainly manifested as stress path dependence. In nature, the wave-induced cyclic loading is non-proportional and multiaxial in a saturated seabed of finite thickness, accompanied by simultaneous changeover in the magnitude and direction of the principal stress. It has been recognized that the rotation of the principal stress axis plays a crucial role in wave-induced liquefaction of sand beds; for example, the resistance to liquefaction under progressive waves is considerably smaller than the resistance exhibited under standing waves for sand beds [9]. However, the effects of fabric anisotropy and non-proportional loading on silt beds still require further study.

Soil anisotropy has been widely examined in numerous laboratory tests, under proportional and non-proportional loading, and has also been considered in a variety of soil constitutive models [10–16]. It also has been found that sandy and silty soils exhibit more complex undrained behavior under cyclic loading than under monotonic loading [17]; for example, ref. [18] carried out a string of triaxial tests and concluded that with the increase

in the degree of soil anisotropy, the shear strength of sand increases in the monotonic triaxial test, while the liquefaction resistance decreases in the cyclic triaxial test. This can be interpreted by the fact that under axial stress-controlled cyclic loading, sand exhibits a considerably higher contraction on the triaxial extension side, which is definitive evidence of soil anisotropy being related to the orientation of the major principal stress direction. However, the cyclic triaxial test is a typical case of proportional loading, which cannot involve either continuous principal stress rotations or their orientation with regard to the axis of anisotropy.

In most practical cases, the stress path tends to deviate from the proportional loads significantly, such as wave loads on seabed sediments [19], multi-directional seismic loads on the ground [20], and train moving loads in railway track foundations, involving continuous rotation of the principal stress axes during cyclic shear deformation. At the same time, the use of hollow cylinder torsional shear tests offers an effective way to investigate soil anisotropic behavior, considering that the orientation and magnitude of the principal stress can be independently controlled in these tests. Most of the previous studies on the torsional shear testing of hollow cylinders have concentrated on situations where the major principal stress direction was fixed at specific values, or the deviatoric stress was continuously rotated during the cyclic shear process. However, when allowing for soil that is highly anisotropic and nonlinear, the study of the coupling effect of the principal stress magnitude and principal stress direction is still in its infancy. Indeed, the stress path involving simultaneous changeover in the magnitude and directions of the principal stress is an even more general case of non-proportional loading, as compared with rotational shear, defined as the shear stress that is perpendicular to the mirror image generated by a thin-walled hollow cylindrical specimen, caused by torsion. This condition can be best demonstrated by the fact that the stress path of finite seabed deposits is related to wave loading [21].

This paper aims to explore the influence of wave-induced non-proportional loading patterns on the undrained deformation behavior of saturated marine silt, based on a series of undrained cyclic tests using a hollow cylinder apparatus. The test samples are from the coastal area of Nantong, Jiangsu Province (Figure 1). The correspondent cyclic shear mode and stress path are replicated by simultaneous changeover in the magnitudes and directions of the principal stress. The emphasis of this study is placed on the effects of the changeover in the magnitudes and directions of the principal stress on the cyclic behavior of saturated silt, characterized by the cyclic stress ratio (CSR) and the ratio (δ) of the amplitude of deviatoric stress components (i.e., deviator stress and torsional shear stress).

Figure 1. The area for obtaining the test samples.

2. Test Procedures

2.1. Experiment Equipment

A hollow cylinder apparatus (HCA) manufactured by GDS Instruments is adopted in this study (Figure 2). The capability of GDS HCA to accurately measure and control such small stress differences has been discussed in detail and fully confirmed [22,23]. The HCA can cyclically and independently control the vertical/torque load (W/M_T) and outer/inner cell pressure p_o/p_i (Figure 3a). Inner wall radial displacement u_i and outer wall radial displacement u_o are calculated from the volume change in the inner and outer pressure, respectively, axial displacement w and rotation angle θ are measured by an axial displacement sensor and a rotation angle displacement sensor, respectively. The application of W , p_o , and p_i on the specimen contributed to developing the stress components of vertical (axial) stress σ_z , and M_T contributed to developing the stress components of torsional shear stress τ ; whereas, simultaneous application of loads p_o and p_i was used to control the stress components of radial stress σ_r and circumferential or tangential stress σ_θ in the hollow cylindrical specimen element. The stress components of σ_z , σ_r , σ_θ , and $\tau_{z\theta}$ in an element of a hollow cylinder specimen are illustrated in Figure 3b. By adjusting these stresses, the major principal stress σ_1 , intermediate principal stress σ_2 , and minor principal stress σ_3 can be independently controlled (Figure 3c). Furthermore, the corresponding vertical strain ε_z , radial strain ε_r , circumferential strain ε_θ , and shear strain $\gamma_{z\theta}$ can be determined by the application of the formulas, assuming that the outer and inner surfaces of specimens retain their correct cylinder shape during torsion [24], compressive stresses and contractive strains are positive in accordance with the geomechanics sign convention.

2.2. Tested Material, Specimen Preparation and Saturation Method

The quartzitic silt soil in powdered form used in the tests was taken from Nantong tidal flat area, China. The Nantong silt was taken from within 2~5 m below the seabed. The detailed location is shown in Figure 3. Figure 4 shows the particle size distribution of Nantong silt, including 57.7% silt, 1.3% clay, and 41.0% fine sand particles. The natural void ratio is 0.935. The index properties of the silt are summarised in Table 1. Specimens with an outer radius of $r_o = 50$ mm, inner radius of $r_i = 30$ mm, and height of $H = 200$ mm were tested. The specimens were prepared by dry deposition method in eight equal-mass layers; a more detailed process of sample preparation was presented in [25]. All specimens were tested under saturated, carbon dioxide flushing, and de-aired water flushing was carried out first, then back pressure saturation followed [25]. When Skempton's B -value > 0.95 , the specimen was considered fully saturated [26]. Following saturation, $p'_0 = 100$ kPa was applied to all specimens to complete the isotropic consolidation.

Figure 2. GDS hollow cylinder apparatus test system.

Figure 3. Idealized stress and strain components within HCA subjected to axial and torsional loads. (a) The HCA can cyclically and independently control the vertical/torque load (W/M_T) and outer/inner cell pressure p_o/p_i ; (b) The stress components of σ_z , σ_r , σ_θ , and $\tau_{z\theta}$ in an element of a hollow cylinder specimen; (c) The stress components of σ_1 , σ_2 and σ_3 in an element of a hollow cylinder specimen.

Figure 4. The grain size distribution of silt soil.

Table 1. Physical properties of the silt soil tested in this study.

Physical Parameters	Symbol	Value
Specific gravity	G_s	2.70
Plastic Index	PI	3
Maximum void ratio	$e_{\max}$	1.22
Minimum void ratio	$e_{\min}$	0.65

2.3. Test Program

The wave-induced stress path in the seabed is traditionally considered as a circle, which only occurs in an infinite seabed [11]. A nonstandard elliptical rotation stress path is shown to be a more common stress state based on the analytical solutions for a finite seabed [27]. The elliptical rotation stress path and the loading waveform are shown in Figure 5. For both the circular and elliptical stress paths, the mean total stress, p , is fixed at

a constant within one loading cycle, but the intermediate principal stress ratio, b , changes in a sinusoidal form between 0 and 1. The cyclic stress ratio (CSR) is defined as $q_{\max}/p'_0$, where $q_{\max}$ is $\max(\sigma_{\max}, \tau_{\max})$ and p'_0 is the initial confining pressure. The cyclic loading amplitude ratio is $\delta = \sigma_{\max}/\tau_{\max}$. Considering that the period of a typical ocean wave ranges from 5 to 20 s [28], the loading frequency is set as 0.1. Table 2 details the case of 15 undrained cyclic shears.

Figure 5. Stress path for cyclic shear loading.

Table 2. Undrained cyclic shear tests.

Case No.	$\sigma_{\max}$ (kPa)	$\tau_{\max}$ (kPa)	Δ	CSR	Stress Path	Case No.	$\sigma_{\max}$ (kPa)	$\tau_{\max}$ (kPa)	δ	CSR	Stress Path
S1	3	3	1	0.03	Circle	S9	5	2.5	2	0.05	Oval
S2	3.5	3.5	1	0.035	Circle	S10	6.5	3.25	2	0.065	Oval
S3	4	4	1	0.04	Circle	S11	8	4	2	0.08	Oval
S4	5	5	1	0.05	Circle	S12	3	0.75	4	0.03	Oval
S5	6.5	6.5	1	0.065	Circle	S13	5	1.25	4	0.05	Oval
S6	8	8	1	0.08	Circle	S14	6.5	1.625	4	0.065	Oval
S7	3	1.5	2	0.03	Oval	S15	8	2	4	0.08	Oval
S8	4	2	2	0.04	Oval						

3. Test Results and Analysis

3.1. Typical Test Results of Saturated Silt Sample under Cyclic Axial–Torsional Combined Loading

The typical results of the silt tested under wave loading ($CSR = 0.05$, $\delta = 1$) are shown in Figure 6; the excess pore pressure u rises more rapidly at the beginning of loading. Then, the increasing rate gradually decreases to zero. When the number of cycles N is greater than 1600, u gradually stabilizes and does not reach the effective confining pressure. Under the circular stress path, the specimen produces shear, axial, radial and other strains in different directions. The shear strain γ changes a little with the increase in N , and the axial strain ε_z develops rapidly in the initial stage of loading, and then ε_z gradually slows down with the increase in N . This test specifies that the direction of radial strain ε_r is consistent with the direction of radial stress σ_r .

During axial–torsional cyclic loading, the silt sample will simultaneously generate shear strain γ , axial strain ε_z , hoop strain ε_θ , and radial strain ε_r . Therefore, a single strain parameter as described with deformation has a certain one-sidedness. Based on the Mohr circle, the major, intermediate, and minor principal strains, ε_1 , ε_2 , and ε_3 , respectively, corresponding to large, medium, and small principal stresses, σ_1 , σ_2 , and σ_3 , respectively, can be calculated. In order to comprehensively reflect the development of the sample

deformation, the cumulative generalized shear strain γ_g is introduced to describe the deformation of the silt [29]. The formula of γ_g is as follows:

$$\gamma_g = \frac{\sqrt{2}}{3} \sqrt{(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_1 - \varepsilon_3)^2 + (\varepsilon_2 - \varepsilon_3)^2} \quad (1)$$

Figure 6. Typical test results of saturated silt sample under cyclic axial–torsional combined loading. (a) The relationship between excess pore water pressure and number of cycles; (b) The relationship between axial strain and number of cycles; (c) The relationship between shear strain and number of cycles; (d) The relationship between radial strain and number of cycles.

3.2. Effect of CSR on the Development of Cumulative Generalized Shear Strain

To predict the rate of cumulative strain, Ref. [30] developed an empirical model that is given in Equation (2), as follows:

$$\gamma_g = aN^b \quad (2)$$

where a and b are the stress-specific fitting parameters. Figure 7 presents the test data points and the best-fit line of the Monismith model; in view of the γ_g variations within a loading cycle, showing an absolute sine form wave-like growth, the value of γ_g within the loading cycle shown in Figure 6 is the maximum value within a loading cycle. As illustrated in Figure 7, the Monismith model can appropriately describe the development law of γ_g . Under various stress paths, γ_g increases with increasing N , and CSR and δ have significant effects on the rate of increase. For the circular stress path, when $CSR = 0.03$, γ_g increases almost linearly with increasing N . When $CSR = 0.04$ and 0.05 , the growth rate of γ_g decreases with the increase in N . For elliptical stress paths, when $CSR = 0.03$, 0.05 , and 0.065 , γ_g increases almost linearly with increasing N . When $CSR = 0.08$ and 0.05 , the growth rate of γ_g gradually slows down with the increase in N . It is observed that γ_g increases with increasing CSR , when δ is the same. Of the many criteria used in the undrained dynamic tests, to determine when sand liquefaction occurs, the most commonly used is initial liquefaction, which is the state of the R_u at 100% [31], and the more widely used 2–3% SA or 5% DA axial strain [32]. However, in the axial–torsion coupling cyclic loading test, the sample will not only produce γ , but also ε_z and ε_r . It is somewhat one-sided to use the development of a single strain parameter as the criterion for liquefaction. Therefore, this study uses the generalized shear strain γ_g to describe the deformation of the soil, and “ $\gamma_g = 5\%$ ” is used as the criterion for the liquefaction failure of the specimen in this article. Figure 8 presents the relationship between γ_g and N under various stress paths.

Note that the development of γ_g is different under different combinations of CSR and δ . According to Figure 8, when $CSR \leq 0.05$, regardless of whether $\delta = 1$ or 2, γ_g is less than 0.2%, the plastic deformation of the sample is very small, and γ_g does not meet the cyclic failure. When $CSR = 0.065$, $\delta = 1$ and $N > 460$, γ_g develops rapidly, and γ_g reaches 5% for $N = 478$. The critical cycle stress ratio CSR_{th} is defined as the minimum CSR required for the specimen to reach liquefaction failure. When $\delta = 1$, $0.05 < CSR_{th} \leq 0.065$; when $\delta = 2$, $0.065 < CSR_{th} \leq 0.08$; and when $\delta = 4$, no liquefaction occurs in the silt samples.

Figure 7. (a–c) Relationship between cumulative generalized shear strain and number of cycles under the same δ .

Figure 8. (a,b) Relationship between cumulative generalized shear strain and number of cycles under different stress paths.

3.3. Effect of δ on the Development of Cumulative Generalized Shear Strain

Figure 9 illustrates the relationship between cumulative generalized shear strain and the number of cycles under the same CSR. According to Figure 9a, it is noted that γ_g develops almost linearly with N for a different value of δ when $CSR = 0.03$. When $CSR = 0.05$, γ_g increases almost linearly with the increase in N for $\delta = 2$ and 4, while γ_g

increases nonlinearly with the increase in N for $\delta = 1$. When the sample is subjected to the small external dynamic stress level, the soil presents an inelastic working state and the cumulative generalized shear strain exhibits linear growth; however, when the sample is subjected to the large external dynamic stress level, the soil presents an elasto-plastic working state in the initial loading stage, the specimen undergoes plastic failure locally, and the axial deformation gradually increases. However, as the loading continues, the skeleton reorganizes after the failure of the specimen to form a relatively stable fabric, which leads to an increase in the rate of axial deformation, slowing shrinking. Furthermore, when $CSR = 0.065$, nonlinear developing trends of γ_g are observed with the increase in N for $\delta = 2$ and 4. It can be observed that the value of generalized shear strain decreases with increasing δ . This is because when $\delta = 1$, the stress path is a circular one. At this time, the seabed soil undergoes the highest stress level, which shows that the circular stress path generated by the wave load causes the deformation of the soil to be greater than that caused by a single normal stress or shear stress.

Figure 9. Relationship between cumulative generalized shear strain and number of cycles under the same CSR. (a) The relationship between cumulative generalized shear strain and number of cycles when $CSR = 0.03$; (b) The relationship between cumulative generalized shear strain and number of cycles when $CSR = 0.05$; (c) The relationship between cumulative generalized shear strain and number of cycles when $CSR = 0.065$.

When N reaches 2000, the cumulative generalized shear strain of the specimen is defined as the terminal generalized shear strain γ_{g2000} . Figure 10 presents the variation in the terminal generalized shear strain versus the cyclic stress ratio of saturated silt under different stress paths. According to Figure 10, when the CSR is the same, γ_{g2000} decreases in sequence as δ increases. For $\delta = 1$, the stress path is circular, and γ_{g2000} is the largest. For δ , the stress path is the same, and γ_{g2000} increases with the increase in CSR. It is noteworthy that the above tendency is only correct when $CSR < CSR_{th}$; when CSR approaches or exceeds CSR_{th} , liquefaction occurs and the silt appears to be unstable, and the terminal generalized shear strain will lose its physical meaning.

Figure 10. Relationship between terminal generalized shear strain and cyclic stress ratio.

3.4. The Predict Model of Cumulative Generalized Shear Strain

Figures 11 and 12 show the variation curves of the fitting parameters a and b with CSR. For different δ (i.e., different stress path), a increases with increasing CSR. The values of the Monismith model parameters a and b in Figures 11 and 12 have been used in Figure 6. The growth rate gradually increases with increasing CSR, and a has a good power function relationship with CSR. b decreases with increasing CSR, and the rate gradually decreases with increasing CSR. b has a good negative power function relationship with CSR. γ_g , calculated by the modified Monismith model, is defined as the cumulative generalized shear strain prediction γ_{gp} . Then, the empirical relationship between a/b and CSR is as follows:

$$a = A_1 \times CSR^{A_2} \quad (3)$$

$$b = B_1 \times CSR^{B_2} \quad (4)$$

Figure 11. Relationship between parameter a and CSR in Monismith model. (a) The relationship between parameter a and CSR when $\delta = 1$; (b) The relationship between parameter a and CSR when $\delta = 2$; (c) The relationship between parameter a and CSR when $\delta = 4$.

Figure 12. Relationship between parameter b and CSR in Monismith model. (a) The relationship between parameter b and CSR when $\delta = 1$; (b) The relationship between parameter b and CSR when $\delta = 2$; (c) The relationship between parameter b and CSR when $\delta = 4$.

In summary, the cumulative deformation of seabed silt soil under various stress levels and loading models can be predicted by combining Equations (2)–(4).

Figure 13 shows the relationship between the measured value of γ_g and the predicted value calculated by the modified Monismith model. It can be observed that the error between the predicted value and the measured value is roughly within 10%. The results show that the modified Monismith model can reasonably describe the γ_{gp} and CSR of saturated silt under different cyclic stress paths.

Figure 13. (a–c) Comparison of measured values and predicted values of cumulative generalized shear strain.

4. Conclusions

An experimental comparison between the circular stress path and the elliptical stress path was conducted, and the experimental results demonstrated the necessity of adopting the elliptical rotation stress path to evaluate the cyclic deformation of marine silt. In addition, the following conclusions can be drawn from the results:

- (1) When CSR is small, the generalized shear strain of marine silt is small under wave loading. However, with the increase in CSR, the value of generalized shear strain is larger under the same confining pressure.
- (2) The influence of the cyclic loading amplitude ratio δ on the generalized shear strain of marine silt is significant. With the increase in δ , the value of generalized shear strain is smaller. When the $\delta = 1$, the generalized shear strain is the most significant.
- (3) The modified Monismith model exerts a significant advantage in the evaluation of the generalized shear strain behavior of the non-liquefaction marine silt in geotechnical engineering practice. The variables a and b both present a moderately strong power correlation with CSR. In addition, the proposed prediction method, based on the modified Monismith model, is in good agreement with our measurements, with an error that is generally less than 10%.

Author Contributions: L.C. (Lan Cui) proposed the general framework of the paper, was responsible for drafting the paper, and sorted out and analyzed the subsequent data; Q.S. mainly made important modifications to the paper and gave some key opinions. Z.N. mainly sorted out the experimental materials and literature, and approved the final version of the paper to be published. L.C. (Liuming Chang) was mainly involved in the experiment process and helped with the drawing. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by [National Natural Science Foundation of China] grant number [52009129].

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The data presented in this study are available on request from the corresponding author.

Acknowledgments: The authors acknowledge the financial support provided by National Natural Science Foundation of China (Grant No. 52009129). The authors would also like to thank referees for their valuable comments and contributions.

Conflicts of Interest: The authors declare no conflict of interest.

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