Normalized Pore Water Pressure Ratio and Post-Cyclic Settlement of Saturated Clay Subjected to Undrained Uni-Directional and Multi-Directional Cyclic Shears

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Abstract

In this paper, normalized excess pore water pressure ratio ($D(U_{dyn}/\sigma'_{v0})$) and normalized post-cyclic settlement in strain ($D\varepsilon_v$), which are defined as the ratio of those induced by multi-directional cyclic shear to the ones induced by uni-directional cyclic shear, were used to show the effect of cyclic shear direction on the post-earthquake deformation of a cohesive soil layer. It is shown from the experimental results that, for the shear strain amplitude (γ) in the range from 0.1 % to 3.0 %, the tendencies of the normalized pore water pressure ratio are different from those of the normalized post-cyclic settlement: the normalized excess pore water pressure ratio reaches its maximum ($D(U_{dyn}/\sigma'_{v0})_{max}$) at γ = 0.1% and then consecutively decreases, meanwhile the normalized post-cyclic settlement firstly increases up to their peaks ($D\varepsilon_{vmax}$) at γ = 0.3 % - 0.4 % then, decreasing. These tendencies are independent of the number of cycles (n). For the Kaolinite clay used in this study, the maximum normalized excess pore water pressure ratio and the maximum normalized post-cyclic settlement are at about $D(U_{dyn}/\sigma'_{v0})_{max}$ = 2.19 - 2.45 and $D\varepsilon_{vmax}$ = 2.25 - 2.73, respectively.

Keywords: Pore water pressure, saturated clay, settlement, undrained cyclic shear

1. Introduction

When a saturated soil specimen is subjected to cyclic shear, the pore water pressure is generally produced. In the case of short-term cyclic loading of an earthquake during which the accelerations mobilize the multi-directional irregular cyclic shear strains with high frequency, cohesive soils can therefore be considered to be under undrained conditions, which leads the pore water pressure accumulation up to very high level. After earthquakes, the dissipation of cyclically induced-pore water pressures may lead to instantaneous and long-term ground settlements which have been typically observed after the Mexico earthquake in 1957 (Zeevaert, 1983), the Miyagi-ken Oki earthquake in 1978 (Suzuki, 1984) or Hyogo-ken Nanbu earthquake in 1995 (Matsuda, 1997). In the case of the ground on which structures are constructed, the stiffness deteriorates and elastic or immediate settlement damaged the structures such as building founded on clay in the 1985 Mexico earthquake (Mendoza and Auvinet, 1988), road and river embankments founded on clay deposits with seated slope failures in Miyagi-ken Oki earthquake in 1978 (Sasaki *et al*., 1980) and Nihon-kai Chubu earthquake in 1983, or major slopes in quick clay area, which were recorded in Alaska earthquake in 1964.

It is well known that the ground vibration during the earthquake is caused by the so-called P-waves and S-waves which propagate vertically and horizontally, respectively. For the soil deposits which is commonly considered to be horizontally stratified, vertical motions produced by P-waves only generate temporary increase in pore water pressure or in other words, no pore water pressure accumulation in saturated soil are caused by such P-waves. Therefore, practically, an attention is usually paid on the effect of one-dimensional S-wave propagation. In such a condition, the behavior of soil element deformation can be directly simulated in the laboratory by undrained cyclic direct simple shear tests, provided that lateral strains are prevented or largely reduced (Matasovic and Vucetic, 1992; 1995). Among which, the multi-directional cyclic shear strains induced by an earthquake can only be simulated by using the multi-directional cyclic simple shear test device.

Therefore, in this study, by using the multi-directional cyclic simple shear test apparatus, normally consolidated Kaolinite clay was subjected to the uni-directional and multi-directional cyclic simple shears with different cyclic shear directions, shear strain amplitudes and number of cycles. The effects of these cyclic shearing conditions on the excess pore water pressure build-up during undrained cyclic shear and the vertical settlement after cyclic shear were observed. The effects of cyclic shear direction on the pore water pressure accumulation and the post-cyclic settlement were investigated in relation to phase difference, shear strain amplitude and number of cycles.

2. Material and Specimen

The soil used in this study is Kaolinite clay, index properties of which are specific gravity G_s = 2.71, liquid limit w_L = 47.80 %, plastic limit w_p = 22.30 %, plasticity index I_p = 25.50 and compression index C_c = 0.31. In order to prepare the specimen, dried Kaolinite powder was mixed with the de-aired water to form slurry having a water content of about 80 %. After keeping the water content constant for one day, the slurry was de-aired in the vacuum cell for about half an hour, and then poured into a rubber membrane placed in the shear box of the multi-directional cyclic simple shear test apparatus.

The slurry was pre-consolidated under the vertical stress σ_{v0} = 49 kPa with the consolidation duration of about 14 minutes. This consolidation period was decided based on the 3t method (JGS, 2000) which has been introduced for closing the consolidation period in laboratory tests and also based on the record of the excess pore water pressure dissipation with time. The dimensions of specimen after the pre-consolidation are about 20 mm in height and 75 mm in diameter. Then the void ratio is in the range of about 1.11 - 1.19. The *B*-value of the specimen before undrained cyclic shear B > 0.95 was confirmed by applying the load increment to satisfy the required saturation degree of undrained cyclic shear test. After pre-consolidation is completed, the specimen was subjected to cyclic shear for pre-determined number of cycles (n), shear strain amplitude (γ) and phase difference (θ) under the undrained conditions. Since the specimen is prevented from the radial and vertical strains during cyclic shear, the constant-volume condition are satisfied. Such a constant-volume test procedure simulates undrained conditions. Following the undrained cyclic shear, drainage was allowed by opening the valve and the pore water pressures generated during cyclic loading were dissipated. The settlements and the pore water pressure were then measured with time.

In this study, the specimens were subjected to the strain-controlled uni-directional and multi-directional cyclic simple shears. The shear strain amplitude was changed in the range from γ = 0.1 % to 3.0 %; the phase difference of multi-directional tests was at θ = 20^0, 45^0, 70^0 and 90^0; and the number of cycles was fixed as n = 10, 20, 50, 100 and 200. The wave form of the cyclic shear strain was sinusoidal (two way cyclic strain) with the period 2 s (frequency of 0.5 Hz) and so the cyclic loading durations were of from 20 seconds to 400 seconds.

3. Results and Discussions

3.1. Excess pore water pressure during undrained uni-directional and multi-directional cyclic simple shears

As a result of undrained cyclic shear, excess pore water pressure (U_{dyn}) increases with the number of cycles. Typical records of the excess pore water pressure ratio which is defined by $U_{\text{dyn}}/\sigma'_{v0}$, where σ'_{v0} is the initial effective stress, during undrained uni-directional and multi-directional cyclic shear tests are shown in Figure 1 for shear strain amplitude γ = 0.1 %, 0.4 % and 2.0 %. The observed results of uni-directional tests in this figure as well as any other figures will be symbolized as “uni”. The differences in the build-up rate and the level of excess pore water pressure are apparently seen in Figure 1, indicating the effects of cyclic shear direction (uni-direction and multi-direction), phase difference (θ = 20^0, 45^0, 70^0 and 90^0), shear strain amplitude (γ) and the number of cycles (n). In addition, since the relations between the cyclically induced-pore water pressure and the cyclic degradation of cohesive soil have been shown by a number of models such as those proposed by Matasovic and Vucetic (1995), the higher level of excess pore water pressure obviously relates to the higher degradation of soil specimens subjected to the multi-directional cyclic shear. Therefore, these results confirm the significant effect of the cyclic shear direction on the undrained behavior of cohesive soils.

As shown in Figure 1, the discrepancies in the excess pore water pressure ratio between uni-directional and multi-directional cyclic shears (θ = 20^0, 45^0, 70^0 and 90^0) are dissimilar for different shear strain amplitude and number of cycles. These discrepancies still remain after large number of cycles as shown in Figure 2, in which although noises caused by the measuring system are seen, the excess pore water pressure ratio induced by uni-directional cyclic shear and multi-directional one with θ = 90^0 are shown for shear strain amplitude γ = 2.0 % and the number of cycles from n = 100 to 500. This behavior is completely different from those of Toyoura sand which shows that the effect of cyclic shear direction on the effective stress reduction becomes negligible when the shear train amplitude is larger than 0.3 % (Matsuda *et al*., 2011 and 2012).

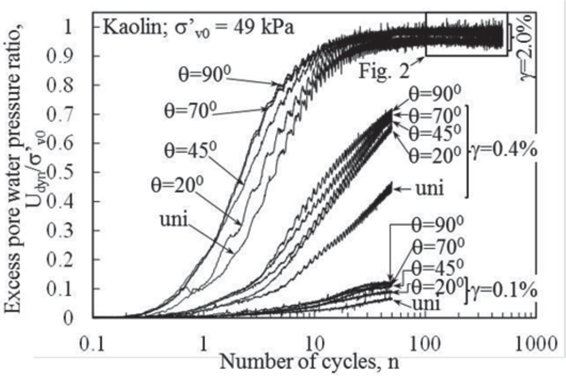

Fig. 1 Relations of excess pore water pressure ratio ($U_{\mathrm{dyn}}/\sigma'_{v0}$) induced by uni-directional and multi-directional cyclic shears versus number of cycles (n) for shear strain amplitude $\gamma = 0.1$ %, 0.4 % and 2.0 %

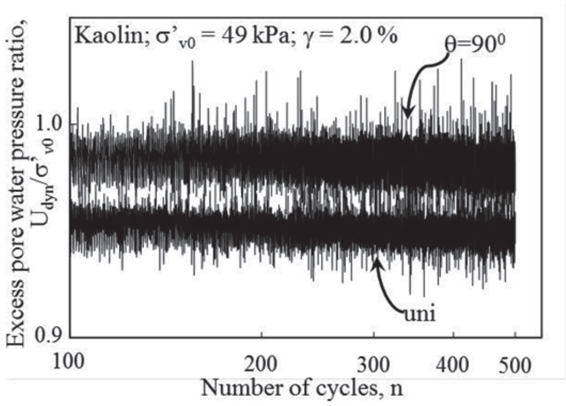

Fig. 2 Excess pore water pressure ratio induced by uni-directional cyclic shear and multi-directional one with the phase difference $\theta = 90^0$ versus number of cycles for $\gamma = 2.0$ %

3.2. The normalized excess pore water pressure ratio of saturated clay subjected to undrained uni-directional and multi-directional cyclic shears

For further detail of the effect of cyclic shear direction on the excess pore water pressure behavior of cohesive soils, the normalized excess pore water pressure ratio, $D(U_{dyn}/\sigma'_{v0})$ is used. This parameter is defined by the ratio of those induced by multi-directional cyclic shear ($(U_{dyn}/\sigma'_{v0})_M$) to the ones induced by the uni-directional cyclic shear ($(U_{dyn}/\sigma'_{v0})_U$) and can be expressed by Equation (1).

$$D(U_{dyn}/\sigma'_{v0}) = (U_{dyn}/\sigma'_{v0})_M \,/ (U_{dyn}/\sigma'_{v0})_U \qquad (1)$$

The changes of $D(U_{dyn}/\sigma'_{v0})$ with shear strain amplitude (γ) are shown in Figures 3(a) to 3(e) for various phase differences and number of cycles.

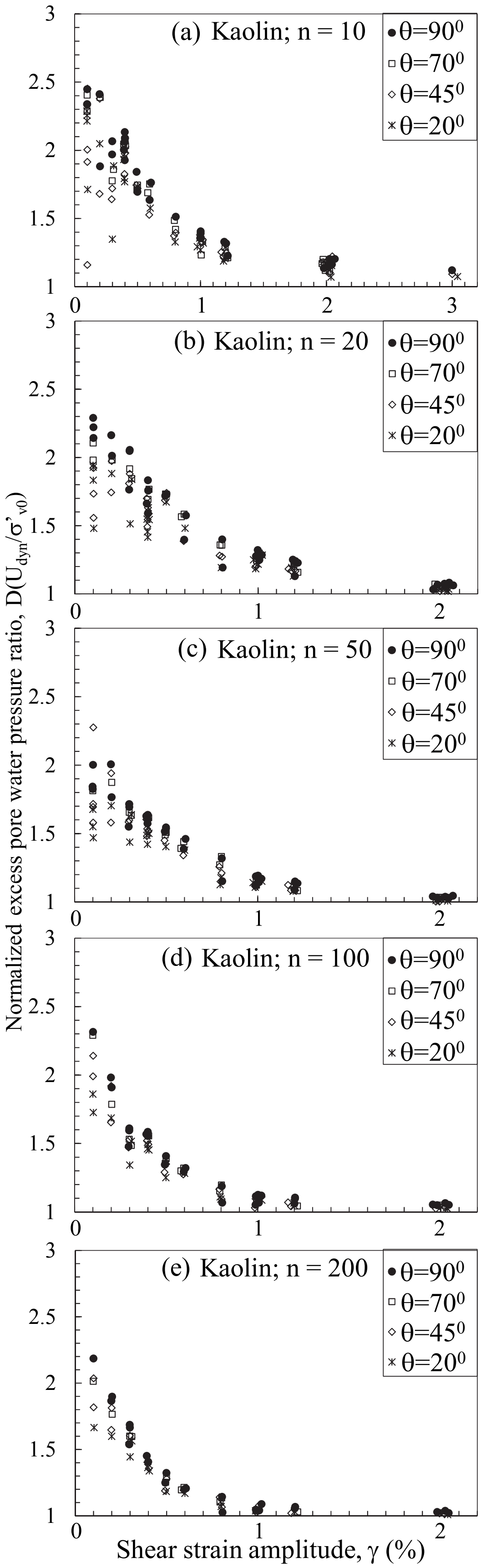

Fig. 3 Normalized excess pore water pressure ratio versus shear strain amplitude for various phase differences and number of cycles

It is seen in these figures that, at the same shear strain amplitude, the normalized excess pore water pressure ratio increases with the phase difference, and this value is significantly affected by the shear strain amplitude as firstly $D(U_{dyn}/\sigma'_{v0})$ reaches its peak at the shear strain amplitude of about γ_{DUmax} = 0.1 % (for simplification, the shear strain amplitude at which $D(U_{dyn}/\sigma'_{v0})$ reaches maximum value $D(U_{dyn}/\sigma'_{v0})_{max}$ is denoted hereinafter as γ_{DUmax}) and secondly consecutively decreases, and this tendency is independent of the number of cycles.

3.3. The normalized settlement of saturated clay pre-subjected to undrained uni-directional and multi-directional cyclic shears

The normalized post-cyclic settlements obtained from multi-directional and uni-directional cyclic shears ($D\varepsilon_v$) by using Equation (2) are plotted against shear strain amplitude in Figures 4(a) to 4(e) for different number of cycles.

$$D\varepsilon_v = \varepsilon_{vM} / \varepsilon_{vU} \quad (2)$$

Where ε_{vM} and ε_{vU} are the post-cyclic settlements induced by undrained multi-directional and uni-directional cyclic shears, respectively.

It is seen from Figures 4(a) to 4(e) that the normalized settlement ($D\varepsilon_v$) generally increases with the phase difference for every number of cycles. The variations of $D\varepsilon_v$ with the shear strain amplitude (γ), however, are definitely different: $D\varepsilon_v$ firstly increases with γ and reaches its peak at the shear strain amplitude of about $\gamma_{D\varepsilon max}$ = 0.3 % - 0.4 % (the shear strain amplitude at which $D\varepsilon_v$ reaches maximum value $D\varepsilon_{vmax}$ is denoted hereinafter as $\gamma_{D\varepsilon max}$) and then, decreasing.

3.4. The tendency of normalized excess pore water pressure ratio and post-cyclic settlement with number of cycles

In order to show the tendencies about the number of cycles for the relation of $D(U_{dyn}/\sigma'_{v0})$ and $D\varepsilon_v$ versus shear strain amplitude, results for the phase difference $\theta = 90^0$ are shown in Figures 5(a) and 5(b), respectively. Although some scatterings are seen, $D(U_{dyn}/\sigma'_{v0})$ and $D\varepsilon_v$ decrease with the number of cycles, regardless of the shear strain amplitude and a tendency that the larger the number of cycles increases, the smaller the value $D(U_{dyn}/\sigma'_{v0})$ and $D\varepsilon_v$, except for the small shear strain amplitude in Figure 5(b) as around 0.3%, where $D\varepsilon_v$ for n = 200 becomes larger compared with small number of cycles. This means that when the shear strain amplitude is small, the effect of the multi-directional shear on the degradation of the structure becomes relatively large.

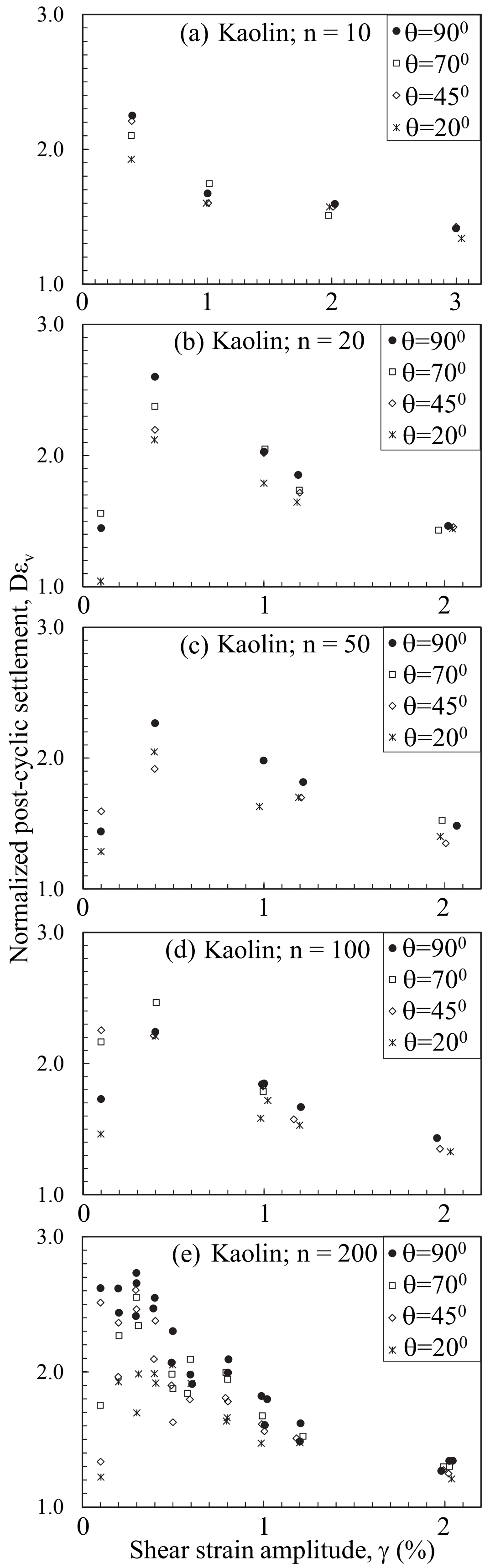

Fig. 4 Normalized post-cyclic settlement versus shear strain amplitude for various phase differences and number of cycles

By using stress-controlled cyclic direct simple shear tests on normally consolidated Drammen clay, Yasuhara and Andersen (1991) confirmed that the soil structure is disturbed even under the uni-directional cyclic shearing conditions and that the settlement of cohesive soil in re-compression stage is affected not only by the level of cyclically induced-pore water pressure but also by this disturbance of soil structure. Therefore, the result in Figure 5(b) suggests that the clay structure would suffer from the higher disturbance under the multi-directional cyclic shearing conditions.

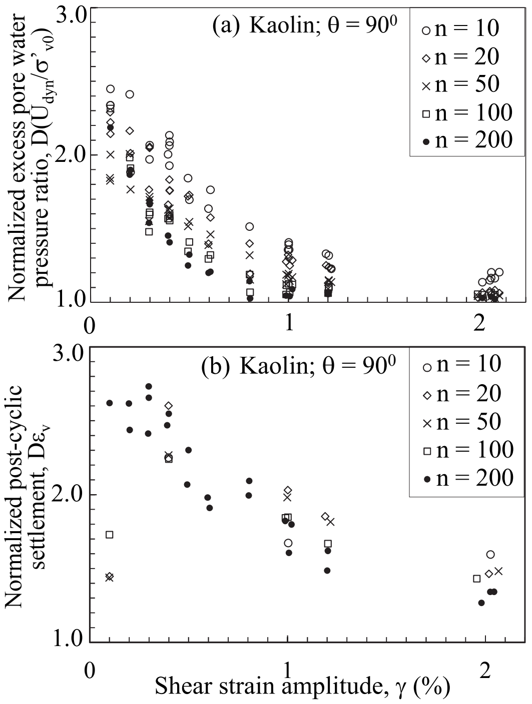

Fig. 5 Normalized excess pore water pressure ratio and post-cyclic settlement between multi-directional cyclic shear with $\theta = 90^0$ and uni-directional cyclic shear versus shear strain amplitude for various number of cycles

Finally, in Figure 6, the maximum normalized excess pore water pressure ratio and post-cyclic settlement and their corresponding shear strain amplitudes (γ_{DUmax} and $\gamma_{D\varepsilon max}$) are plotted against the number of cycles. For the number of cycles from n = 10 to 200, the maximum values of normalized excess pore water pressure ratio fluctuate between $D(U_{dyn}/\sigma'_{v0})_{max}$ = 2.19 and 2.45, being slightly smaller than those of the post-cyclic settlement which are in the range between $D\varepsilon_{vmax}$ = 2.25 and 2.73 (Fig. 6). Meanwhile, the shear strain amplitude for the maximum normalized pore water pressure ratio and post-cyclic settlement are almost unchanged at γ_{DUmax} = 0.1 % and $\gamma_{D\varepsilon max}$ = 0.3 % - 0.4 %, respectively.

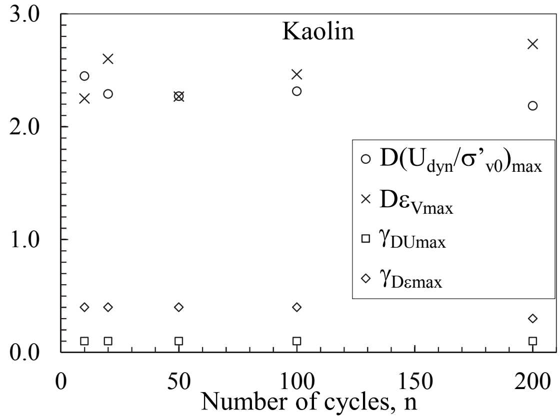

Fig. 6 The tendency of $D(U_{dyn}/\sigma'_{v0})_{max}$, $D\varepsilon_{vmax}$, γ_{DUmax} and $\gamma_{D\varepsilon max}$ with number of cycles

4. Conclusions

By using the multi-directional cyclic simple test apparatus, normally consolidated samples of Kaolinite clay were tested under undrained uni-directional and multi-directional cyclic shears followed by drainage, the effect of cyclic shear direction on the pore water pressure and re-compression characteristics was investigated in relation to another cyclic shearing parameter. The main conclusions are as follows:

The normalized pore water pressure ratio and post-cyclic settlement between multi-directional cyclic shear and uni-directional one generally decrease with the number of cycles. For the number of cycles from n = 10 to 200, the maximum values of normalized pore water pressure ratio are at about 2.19 - 2.45 and slightly smaller than those of post-cyclic settlement being at $D\varepsilon_{vmax}$ = 2.25 - 2.73. These values are obtained at the shear strain amplitude of γ_{DUmax} = 0.1 % and $\gamma_{D\varepsilon max}$ = 0.3 % - 0.4 %, respectively.

Acknowledgements

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