MULTI-DIRECTIONAL CYCLIC SHEAR-INDUCED PORE PRESSURE AND SETTLEMENT OF UNDISTURBED CLAY

H. Matsuda ¹, TT. Nhan ², H. Sato ³ and H. Hara ⁴

ABSTRACT

When a cohesive soil layer is subjected to the multi-directional cyclic shear by the earthquake, an additional settlement would occur after the earthquake. In this study, by using the multi-directional cyclic simple shear test apparatus, undisturbed specimens of a clay were tested under a wide range of shear strain amplitude and number of cycles, and the results about the pore water pressure accumulation and the cyclic shear-induced settlement were compared with those of disturbed specimen. Then, effects of disturbance of clay on the dynamic properties and on the $e$-$\log p$ relations were investigated. In conclusion, (1) under the same cyclic shearing condition, the pore water pressure induced in the disturbed specimen increases to a higher level than those in the undisturbed one. Meanwhile, the differences of the cyclic shear-induced settlement between the undisturbed and disturbed specimens are negligible when the accumulated pore water pressure is the same, (2) when the effective stress increases by the dissipation of accumulated excess pore water pressure, $e$-$\log p$ curve does not meet with the virgin compression line even at larger vertical effective stress than those of pre-consolidation, (3) the compression index decreases with the shear strain amplitude and the level of cyclic shear-induced pore water pressure.

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ABSTRACT

When a cohesive soil layer is subjected to the multi-directional cyclic shear by the earthquake, an additional settlement would occur after the earthquake. In this study, by using the multi-directional cyclic simple shear test apparatus, undisturbed specimens of a clay were tested under a wide range of shear strain amplitude and number of cycles, and the results about the pore water pressure accumulation and the cyclic shear-induced settlement were compared with those of disturbed specimen. Then, effects of disturbance of clay on the dynamic properties and on the e-logp relations were investigated. In conclusion, (1) under the same cyclic shearing condition, the pore water pressure induced in the disturbed specimen increases to a higher level than those in the undisturbed one. Meanwhile, the differences of the cyclic shear-induced settlement between the undisturbed and disturbed specimens are negligible when the accumulated pore water pressure is the same, (2) when the effective stress increases by the dissipation of accumulated excess pore water pressure, e-logp curve does not meet with the virgin compression line even at larger vertical effective stress than those of pre-consolidation, (3) the compression index decreases with the shear strain amplitude and the level of cyclic shear-induced pore water pressure.

Introduction

Saturated soil deposits might be subjected to the undrained cyclic loading due to earthquakes, traffic load, pile driving, explosions and other sources. Although clayey soils have been considered as relatively stable materials for the earthquake, cyclic shear-induced pore water pressure might accumulate to a high level and a degradation of the soil has been confirmed through laboratory tests [1] and major earthquakes [2]. In addition, during the earthquake, soil layers are subjected to multi-directional cyclic shear showing the irregularity of shear strain

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amplitude and frequency [3] and such conditions can be simulated by the laboratory multidirectional cyclic simple shear test. For the dynamic properties of saturated sand under the multidirectional cyclic shear have been extensively studied [3] and taken into account in the design specification [4]. Meanwhile, the situation of such problems on cohesive soils under cyclic loading needs to be clarified, including the undisturbed clays.

When investigating the dynamic properties of soils, laboratory experiments are usually carried out based on the stress-controlled approach, e.g. the stress-controlled cyclic simple shear tests for clays were performed by Andersen et al. [5]. As an alternative approach, the strain-controlled tests were introduced by Dobry et al. [6] who indicated that the shear strain is the main parameter controlling the settlement and pore water pressure generation of sand during cyclic loading. When focusing on an arbitrary cycle during the cyclic shear test, the deformation of soil microstructure might increase with the applied cyclic shear strain amplitude ($\gamma$). From a literature review on the cyclic behavior of clays, Matsui et al. [7] concluded that the stress-deformation characteristics of clays depend on the magnitude of applied shear strain. On the other hand, Matasovic and Vucetic [8] clarified that the generation and buildup of pore water pressure is a consequence of the tendency of saturated soil to change in volume and that the volume change depends directly on the shear deformation of the soil [9]. These observations mean that the shear strain amplitude is a more fundamental parameter when investigating the pore water pressure during undrained cyclic shear and so, the cyclic strain-controlled tests instead of the stress-controlled ones are meaningful for studying the pore water pressure accumulation during cyclic loading [8].

In this study, several series of strain-controlled uniform and irregular multi-directional cyclic shear tests were carried out on undisturbed and disturbed clay specimens. After the dissipation of the cyclic shear-induced pore water pressure, undisturbed specimens were further consolidated under the larger vertical stress. Then following points were mainly discussed, (1) the effect of the disturbance of clayey soil on the pore water pressure accumulation during multi-directional cyclic simple shear, (2) the settlements due to the dissipation of the cyclic shear-induced pore water pressure and (3) the effect of uniform and irregular multi-directional cyclic shears on the post-cyclic $e$-log$p$ relations of the undisturbed clay.

**Experimental Aspects**

**Sample and Specimens**

Used sample is the undisturbed Tohoku clay which was taken from the borehole at GL-18.0m in Ishinomaki city, Tohoku area, Japan. The index properties of the soil are: specific gravity $G_s = 2.607$, liquid limit $w_L = 124.7\%$, plasticity index $I_p = 84.2$, and compression index $C_c = 1.295$. Disturbed sample was made directly from the undisturbed one by mixing with de-aired water to form slurry having water content of about 150% and kept in vessel at least for one day. The slurry was then de-airied in the vacuum cell and poured into the shear box and pre-consolidated under the same vertical stress as the in-situ vertical effective stress ($p'c \approx 157$ kPa) until the dissipation with 99% of the excess pore water pressure at the bottom of specimen was confirmed. After the pre-consolidation, the dimensions of specimen were 75 mm in diameter and 20 mm in height and the initial void ratio was about $e_0 = 1.46 - 1.57$. 
Test Apparatus and Procedures

After the pre-consolidation, soil specimen was subjected to the strain-controlled uniform and irregular multi-directional cyclic shear by the multi-directional cyclic simple shear test apparatus which was developed at Yamaguchi University, Japan [4]. This apparatus can give any types of horizontal displacement independently from two orthogonal directions at the bottom of specimen by using the electro-hydraulic servo system. A predetermined vertical stress can be applied to the specimen by the aero-servo system. The shear box is the Kjellman type in which the specimen is enclosed in a rubber membrane. The membrane-enclosed specimen is surrounded by a stack of acrylic rings and in this situation the specimen is subjected to simple shear deformation keeping the constant cross-sectional area. During the cyclic shear test, the height of specimen was kept constant and then the change of vertical stress and the pore water pressure at the bottom of the specimen were measured. This means that such a condition simulates the undrained cyclic shearing.

Experiments were carried out on 15 strain-controlled uniform and irregular multi-directional cyclic shear tests (5 uniform multi-directional cyclic shear tests on undisturbed sample, 6 uniform multi-directional cyclic shear tests on disturbed sample and 4 irregular multi-directional cyclic shear tests on undisturbed sample). In the multi-directional cyclic shear tests, the cyclic shear strain was applied to the specimen simultaneously in $X$ direction ($\gamma_x$) and $Y$ direction ($\gamma_y$) which are perpendicular to each other under the same amplitude ($\gamma = \gamma_x = \gamma_y$) with the largest phase difference $\theta = 90^\circ$, where the multi-directional cyclic simple shear is identified by the phase difference from $\theta = 0^\circ$ to $\theta = 90^\circ$ and the effect of the cyclic shear direction on the pore water pressure and the post-cyclic settlement has been confirmed by changing the value of $\theta$ [10,11]. In this study, because of limited number of undisturbed samples, the multi-directional cyclic shear tests were performed under the largest degree of phase difference to obtain the maximum effect of the cyclic shear direction. The cyclic shear strain was sinusoidal with the period of 2s and the shear strain amplitude was fixed as $n = 200$. Following the cyclic shear test, the drainage from the top surface of specimen was allowed and then the pore water pressure at the bottom surface of specimen and the settlement were measured with time. The irregular multi-directional cyclic shear tests were also carried out by using the strain-time histories at the Hyogo-ken Nanbu earthquake 1995. The detail of these tests will be shown later. Furthermore, in the case of the undisturbed specimen, after the dissipation of cyclic shear-induced pore water pressure, the consolidation was continued by increasing the vertical stress up to several times higher than the effective stress at the start of cyclic loading.

Results and Discussions

Pore Water Pressure Accumulated in Undisturbed and Disturbed Clays under Multi-Directional Cyclic Shear

Pore Water Pressure during Cyclic Shearing

As a result of cyclic loading under the constant volume condition, the vertical effective stress ($\sigma'_vo$) decreases by the application of cyclic shear strain. Then the decrease of the effective stress ($|\Delta \sigma'_v|$) is assumed to be equal to the increment of the pore water pressure ($U_{dyn}$). For undisturbed and disturbed specimens which were subjected to the multi-directional cyclic shear with a wide range of shear strain amplitude, typical changes of the pore water pressure ratio which is defined by $U_{dyn}/\sigma'_vo$, are shown in Fig. 1. It is seen that $U_{dyn}/\sigma'_vo$ increases with the number of cycles and
at the same number of cycles, the larger shear strain amplitude results in the higher pore water pressure ratio. Under similar cyclic shear conditions, the pore water pressures in the disturbed specimen increases to a higher level than those in undisturbed one. Since the cyclic resistance of soil deposits is significantly affected by the level of cyclic shear-induced pore water pressure accumulation [8], the higher pore water pressure accumulation indicates the occurrence degradation of clay.

Figure 1. Typical changes in pore water pressure of undisturbed and disturbed clays subjected to multi-directional cyclic shear.

**Estimation of Cyclic Shear-Induced Pore Water Pressure Accumulation in Undisturbed and Disturbed Clays**

In the cyclic simple shearing model, the length along the strain path of the specimen can be denoted by the cumulative shear strain ($G^*$) which was proposed by Fukutake and Matsuoka [12] to estimate the volumetric strain of granular materials subjected to drained cyclic simple shear (Eq. 1). As to uniform strain reversals, the cumulative deformation of the soil microstructure is in proportion to the shear strain amplitude ($\gamma$) and number of cycles ($n$), and therefore $G^*$ can be shown as a function of $\gamma$ and $n$. Matsuda et al. [10] has proposed $G^*$ - $\gamma$ - $n$ relations in the multi-directional cyclic simple shear test, as expressed by Eq. 2.

\[
G^* = \sum \Delta G^* = \sum \sqrt{\Delta \gamma^2_x + \Delta \gamma^2_y}
\]

\[
G^* = n \times (5.995 \times \gamma + 0.351)
\]

where $\Delta \gamma_x$ and $\Delta \gamma_y$ are the shear strain increment in $X$ and $Y$ directions on the horizontal plane, respectively. Based on the relationships between the volumetric strain in the drained condition and the pore water pressure accumulation under the undrained condition, Matsuda et al. [4,10] developed an estimation method of the pore water pressure accumulation for the saturated sand and clay by using the cumulative shear strain $G^*$ as follows:

\[
\frac{\Delta \sigma_v}{\sigma_{vo}} = \frac{U_{dum}}{\sigma_{vo}} = \frac{G^*}{\alpha + \beta G^*}
\]

where $\alpha$ and $\beta$ are the experimental parameters and defined by a function of $\gamma$ as $\alpha = A(\gamma)^m$ and $\beta = \gamma(B + C\gamma)$. Where $A$, $B$, $C$ and $m$ are experimental constants and can be determined based on the
results of cyclic simple shear tests. Relationships between \( U_{\text{dyn}}/\sigma'_{\text{vo}} \) and \( G^* \) are shown in Fig. 2(a) for undisturbed and disturbed specimens, in which symbols show the observed results and solids lines show the calculated ones by using Eq. 3. In Fig. 2(b), relationships between \( G^*/(U_{\text{dyn}}/\sigma'_{\text{vo}}) \) and \( G^* \) are shown, by which the experimental parameters \( \alpha \) and \( \beta \) are given as a function of \( \gamma \), and so, it is possible to decide the experimental constants \( A, B, C \) and \( m \). In Fig. 2, reasonable agreements are seen and the experimental constants \( A, B, C \) and \( m \) for undisturbed and disturbed specimens are summarized in Table 1.

In Fig. 3(a), relationships between \( U_{\text{dyn}}/\sigma'_{\text{vo}} \) and \( G^* \) after 200 cycles of cyclic shears with \( \gamma = 0.1\% \sim 3.0\% \) are shown for undisturbed and disturbed specimens, in which solid and dashed lines show the observed results and calculated ones, respectively.

![Figure 2](image1.png)

![Figure 3](image2.png)

Table 1. Experimental constants \( A, B, C \) and \( m \).

<table>
<thead>
<tr>
<th>Experimental Constants</th>
<th>( A )</th>
<th>( B )</th>
<th>( C )</th>
<th>( m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undisturbed specimen</td>
<td>428.1</td>
<td>-0.2558</td>
<td>0.7801</td>
<td>-1.091</td>
</tr>
<tr>
<td>Disturbed specimen</td>
<td>179.5</td>
<td>-0.2776</td>
<td>0.8882</td>
<td>-1.393</td>
</tr>
</tbody>
</table>

Figure 3. Comparisons between observed and calculated results for the pore water pressure accumulation in undisturbed and disturbed clays under multi-directional cyclic shear.
lines show the calculated results by using Eq. 3. Further comparisons between calculated and observed results are shown in Fig. 3(b). Calculated results agree well with the observed ones and therefore, Eq. 3 is applicable for estimating the cyclic shear-induced pore water pressure of undisturbed and disturbed clays.

**Recompression due to the Dissipation of Cyclic Shear-Induced Pore Water Pressure**

After undrained cyclic loading, the dissipation of the pore water pressure results in the volumetric reduction of a clay layer. Since the post-cyclic settlement was confirmed to be correlated with and influenced by the cyclically induced pore water pressure [1,13,14], the settlement in strain $\varepsilon_v$ (%) was calculated by using Eq. 4 as follows [1,14].

$$\varepsilon_v = \frac{\Delta h}{h_0} \frac{\Delta e}{1+e_0} = \frac{C_{dyn}}{1+e_0} \log(\frac{1}{1-\frac{U_{dyn}}{\sigma'}}) = C_{dyn} \log SRR$$

where $C_{dyn}$ is the cyclic shear-induced recompression index (in the case of undisturbed sample, $C_{dyn}$ is replaced by $C_{dynU}$ and in the case of disturbed one, $C_{dyn}$ is replaced by $C_{dynD}$), $SRR$ is the stress reduction ratio, $\Delta h$ is the settlement of specimen, $h_0$ and $e_0$ are the height and the void ratio at the start of cyclic shear, $\Delta e$ is the change of void ratio.

![Figure 4](image_url)

Figure 4. Change of the void ratio and the post-cyclic settlement of undisturbed and disturbed clays in recompression stage after the multi-directional cyclic shear.

Relations between $\Delta e$ and $SRR$ are shown in Fig. 4(a) indicating linear increasing tendencies of $\Delta e$ with log$SRR$, irrespective of sample disturbance and the shear strain amplitude. The cyclic shear-induced recompression index, designated by the gradients of solid and dashed lines are affected by the sample disturbance and therefore, different values of $C_{dynU} = 0.234$ and $C_{dynD} = 0.167$ were obtained for undisturbed and disturbed clays, respectively. The observed results for the relations between $\varepsilon_v$ and $G^*$ are plotted by symbols in Fig. 4(b), and solid and dashed lines show the calculated ones by using Eq. 4, in which $SRR$ was determined by using Eq. 3 and the experimental constants in Table 1. Reasonable agreements in Fig. 4(b) between observed and calculated results are seen and so, the applicability of estimating the post-cyclic settlement of undisturbed and disturbed clays was confirmed.
Compressibility of Undisturbed Clay Pre-Subjected to Uniform and Irregular Multi-Directional Cyclic Shear

After the dissipation of cyclic shear-induced pore water pressure, undisturbed specimens were re-consolidated by the step loading. The dissipation of the pore water pressure was confirmed by the record of the pore water pressure at the bottom surface of the specimen and also by the 3t-method. The $e$-log$p$ relations following cyclic shear were then compared with those of specimens without cyclic shearing histories. The $e$-log$t$ relations in the re-consolidation stage on undisturbed Tohoku clay and the corresponding $e$-log$p$ relations are shown in Figs. 5(a) and (b), respectively. It is seen that the recompression curves after the multi-directional cyclic shear do not come in contact with the virgin compression line even when the vertical effective stress increases up to 2.5 times higher than those at the start of cyclic loading. This tendency is similar to those on reconstituted Kaolin and undisturbed Drammen clay subjected to the uni-directional cyclic simple shear [1,14].

In order to investigate the characteristics of the post-earthquake settlement of undisturbed clay more in detail, several multi-directional cyclic shear tests were carried out by using irregular strain-time histories at the Hyogo-ken Nanbu earthquake 1995. The original calculated strain-time histories are shown in Fig. 6(a) and when using these irregular waves in the tests, maximum shear strain amplitude in $NS$ and $EW$ directions was changed and since the maximum shear strain amplitude in $NS$ direction is larger than those in $EW$ direction, i.e. $\gamma_{maxNS} > \gamma_{maxEW}$, $\gamma_{maxNS}$ was referred as the maximum shear strain amplitude in the earthquake, i.e. $\gamma_{max} = \gamma_{maxNS}$. The changes of pore water pressure during the irregular multi-directional cyclic shear and the settlement in the recompression stage are shown in Figs. 6(b) and (c) for different levels of maximum shear strain amplitude. It is evident that the pore water pressure accumulates during the earthquake and that the larger the maximum shear strain amplitude, the higher the pore water pressure accumulation and the post-earthquake settlement.

Figure 5. Settlement versus elapsed time and the compressibility characteristics of undisturbed clay pre-subjected to uniform multi-directional cyclic shear.
Figure 6. (a) Strain-time histories of Hyogo-ken Nanbu earthquake 1995, (b) pore water pressure accumulation and (c) the post-cyclic settlement (in strain) of undisturbed clay under irregular multi-directional cyclic shear.

Figure 7. (a) Settlement versus elapsed time and (b) e-log\(p\) relations and (c) the change of \(C_{dyn}\), \(C_{cdyn}\), \(C_{sdyn}\) with \(\gamma\) in the consolidation tests.
After the dissipation of the irregular cyclic shear-induced pore water pressure, the consolidation tests were performed by applying the step loading. In Fig. 7(a), the settlement-time relations are shown for each loading step and $e$-$\log p$ relations were then obtained as shown in Fig. 7(b). Since the equivalent uniform number of cycles was confirmed as $N_{eq} \approx 8.9$ [15] and even under this relatively small number of cycles, the recompression curves of undisturbed clay do not meet with the virgin consolidation line at the vertical stress several times higher than the pre-consolidation one. The changes in cyclic shear-induced recompression index ($C_{dyh}$), post-cyclic compression index ($C_{cdy}$) and post-cyclic swelling index ($C_{sdy}$) with $\gamma$ are shown in Fig. 7(c) for both cases of uniform and irregular cyclic shears. In the same figure, the horizontal axis shows the range of shear strain amplitude in the uniform cyclic shear tests and therefore, the maximum amplitude of the irregular cyclic shear strain ($\gamma_{\text{max}}$) should be transferred into the equivalent uniform ones ($\gamma_{\text{dyh}}$). Several studies on liquefaction potential analyses confirmed that the equivalent shear strain amplitude was referred as 65% of the maximum shear strain amplitude [15], i.e. $\gamma_{\text{dyh}} = 0.65 \gamma_{\text{max}}$ and this definition was used in this study. It is evident in Fig. 7(c) that, the values of $C_{dyh}$ and $C_{sdy}$ are generally kept constant, meanwhile $C_{cdy}$ slightly decreases with the cyclic shear strain amplitude and since the cyclic shear-induced pore water pressure increases with $\gamma$, it is suggested that $C_{dyh}$ decreases with $U_{\text{dyh}}/\sigma'_{\nu}$ (Figs. 5(b) and 7(b)).

Vucetic and Dobry [16] concluded that the plasticity index ($I_p$) is one of the most important index properties governing the dynamic behavior of normally and over-consolidated clays. The soil with higher plasticity index shows higher cyclic resistances due to the lower accumulated pore water pressure and settlement [11]. Therefore, the level of pore water pressure accumulation and the post-cyclic settlement of undisturbed Tohoku clay ($I_p = 84.2$) are relatively low even when the soil is subjected to strong cyclic shear and the differences of the $e$-$\log p$ relations before and after cyclic shearing are not definite. However, the decreasing tendency of $C_{dyh}$ with $\gamma$ and $U_{\text{dyh}}/\sigma'_{\nu}$ in Fig. 7(c) suggests the influence of the multi-directional cyclic shear on the compressibility of undisturbed clay. For the soils with lower plasticity index, such as Kaolinite clay ($I_p = 25.0$) and Drammen clay ($I_p = 27.0$), the effect of uni-directional cyclic shear on the compressibility of clayey soils has been evidently confirmed [1,14].

**Conclusions**

The main conclusions are as follows:

Under the similar multi-directional cyclic shearing conditions, the pore water pressure accumulation and the post-cyclic settlement of disturbed specimen are higher than those obtained on the undisturbed specimen and therefore the effect of the disturbance of the soil structure on these properties were confirmed.

The $e$-$\log p$ curves on the undisturbed specimen obtained after the dissipation of the pore water pressure induced by uniform and irregular cyclic shear do not contact the virgin compression line even at the consolidation stress 2.5 times higher than those at the start of cyclic loading.

$C_{sdy}$ and $C_{dyh}$ of the undisturbed clay are not affected by the shear strain amplitude, but $C_{dyh}$ decreases with $\gamma$ and $U_{\text{dyh}}/\sigma'_{\nu}$. 
Acknowledgments

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