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PORE WATER PRESSURE AND SETTLEMENT INDUCED BY MULTI-DIRECTIONAL CYCLIC SHEAR OF HIGH PLASTICITY CLAY

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Abstract

In this paper, by using the multi-directional cyclic simple shear test apparatus, normally consolidated specimens of Kitakyushu clay (plasticity index, I_p = 63.8) were subjected to the uni-directional and multi-directional cyclic shears (in the multi-directional cyclic shear test, uniform cyclic shear strains were applied to the soil specimen from two perpendicular directions with various phase differences) in which the shear strain amplitude was changed in the range from 0.05% to 2.0%. The cyclic shear induced-pore water pressure (U_{dyn}) and settlement in strain (ε_v, %) were observed and compared with the calculated ones obtained by the conventional estimation method, where the pore water pressure ratio (U_{dyn}/σ'_{v0}) is expressed by equations including the number of cycles (n) and two experimental parameters (α and β). It is shown from the observed and calculated results that the changes of α and β can be expressed by a function of shear strain amplitude.

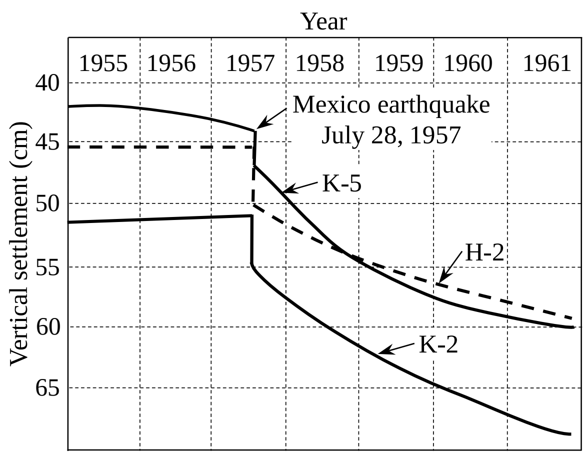
In the recompression stage after undrained cyclic shear, the compression index for the uni-directional cyclic shear C_{dynU} is obtained as 0.233×C_c and for the multi-directional cyclic shear C_{dynM} as 0.250×C_c, respectively, where C_c is the virgin compression index.

In conclusion, the cyclic shear induced-settlement of clay with high plasticity index can be predicted by using these indices.

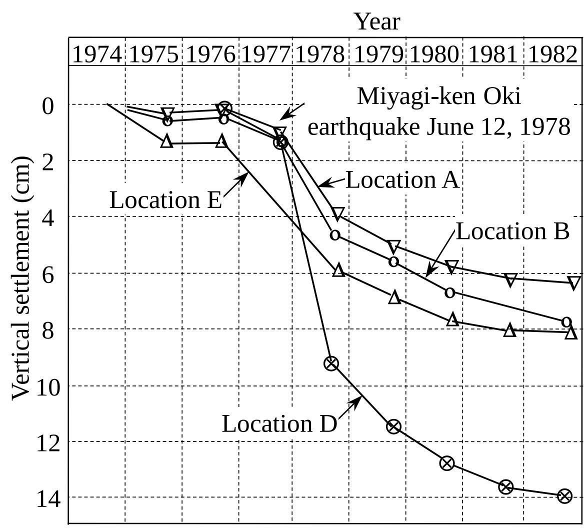
Keywords: cyclic simple shear, sensitive clay, undrained

1. Introduction

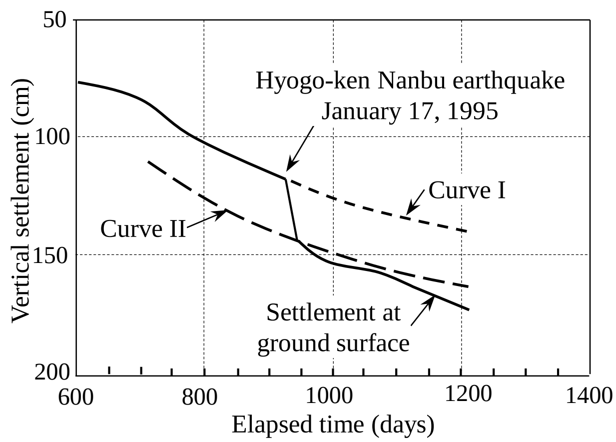
Under strong cyclic loading such as those induced by earthquakes, pore water pressure in saturated sandy soil rapidly increases and then soil liquefaction might occur [1]. The so-called liquefaction-induced settlements after the earthquake have been recorded, then the instantaneous ground settlement and serious damages to structures have been observed [2, 3, 4]. The problems regarding the cyclic shear-induced pore water pressure and settlement of sands soils have been systematically clarified in connection with the mechanism of soil liquefaction [5, 6, 7]. Since the ground shaking during earthquakes induces the multi-directional cyclic shear strain in the soil layer [8], the dynamic behavior of sandy soils including the effect of cyclic shear direction has been extensively studied [1, 8, 9, 10] and the multi-directional characteristics of ground shaking have been implemented as a correction factor and applied to the design specification in the highway bridge construction [11].



(a) Mexico earthquake in 1957 (Zeevaert [19])



(b) Miyagi-ken Oki earthquake in 1978 (Suzuki [20])



(c) Hyogo-ken Nanbu earthquake in 1995 (Matsuda [21])

Fig. 1 – Records of post earthquake settlement-time relations

Saturated fine-grained soil deposits, on the other hand, might be subjected to undrained cyclic loading caused by earthquake, seawave, traffic etc, and under the earthquake in which the loading duration is relatively short the so-called cyclic shear-induced pore water pressure is accumulated leading to the decrease in the effective stress. Although in saturated clays, the liquefaction is considered not to occur even under the strong motion from earthquakes, the cyclic shear-induced pore water pressure can reach 90% of the initial vertical stress [12, 13, 14] and then soft clays may fail, not only by the reduction of strenght but also by the degradation in stiffness [15, 16, 17, 18]. At the same time, clay layers may settle down gradually due to the dissipation of excess pore water pressures after earthquakes. Typical examples in this category have been observed at Mexico earthquake in 1957 [19], Miyagi-ken Oki earthquake in 1978 [20] and Hyogo-ken Nanbu earthquake in 1995 [21] as shown in Figs. 1(a), (b) and (c), respectively. In the case of the soft clay ground on which structures are constructed, the stiffness deterioration and instantaneous settlement which lead to damage the structures like a building were observed in the 1985 Mexico earthquake [22]; road and river embankments founded on clay deposits suffered from slope failures in Miyagi-ken Oki earthquake in 1978 [23] and Nihonkai Chubu earthquake in 1983; or a slope failure in quick clay was recorded in Alaska earthquake in 1964. In fine-grained soils especially including the high portion of small fraction, the bonds between soil particles generated by its high plasticity lead the behavior of soil skeleton more complicated under cyclic loading. Therefore, at the current situation, a curve-fitting method by using the experimental data seems to be one of the most appropriate approach for the estimation of cyclic shear-induced pore water pressure and settlement in cohesive soils [24, 25, 26, 27, 28].

In this study, firstly several series of uni-directional and multi-directional cyclic simple shear tests were carried out on Kitakyushu clay, which has relatively high plasticity index, and the effects of cyclic shear direction and cyclic shear strain amplitude on the pore water pressure accumulation and on the recompression were clarified, and secondly, based on the experimental results, the applicability of the estimation of multi-directional cyclic shear-induced pore water pressure and settlement was confirmed.

2. Undrained uni-directional and multi-directional cyclic shear tests on sensitive clay

2.1 Test apparatus

Fig. 2(a) shows the outline of the multi-directional cyclic simple shear test apparatus. This apparatus can give any types of cyclic displacement at the bottom of specimen from two orthogonal directions by using the electro-hydraulic servo system and at the same time, a predetermined vertical stress can be applied to the specimen by the aero-servo system. Outline of the apparatus and the specimen together with the shear box which is the Kjellman type are shown in Figs. 2(b)-(e). The specimen is enclosed inside a rubber membrane and the outside of which is surrounded by a stack of 20 acrylic rings. Each acrylic ring has 75.4 mm in inside diameter and 2 mm in thickness. In this condition, the specimen is prevented from the lateral deformation but permitted to the simple shear deformation during cyclic shearing.

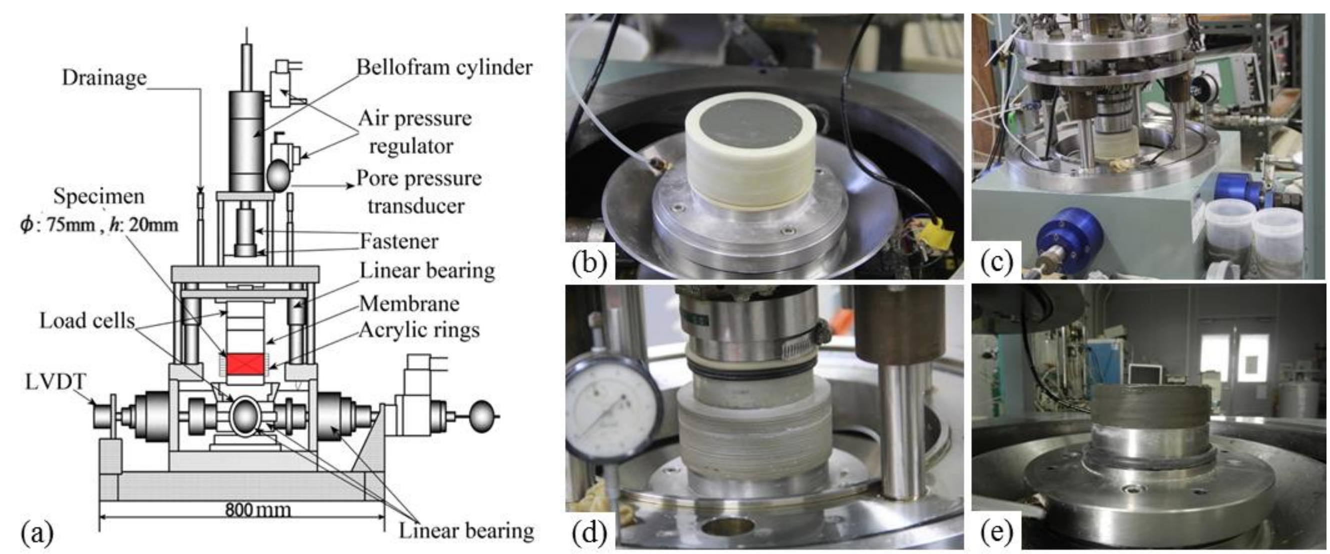


Fig. 2 – (a) Outline of the multi-directional cyclic simple shear test apparatus, (b) slurry of sample after pouring into shear box, (c) soil specimen before pre-consolidation, (d) soil specimen during pre-consolidation and (e) soil specimen after recompression

2.2 Test sample and specimen

The soil used in this study is a marine clay, called Kitakyushu clay, whose grain content is over 40% of silt and 50% of clay-size fraction and Kaolinite clay. The index properties for Kitakyushu clay and Kaolinite clay are specific gravity G_s = 2.63 and 2.71, liquid limit w_L = 98.0% and 47.8%, plastic limit w_P = 34.2% and 22.3%, plasticity index I_p = 63.8 and 25.5, and compression index C_c = 0.60 and 0.31, respectively. In order to prepare the test specimen of each soil, sample was mixed with de-aired water to form slurry having a water content of about $1.5 \times w_L$ which was kept in a big tank under the constant water content. The slurry was then taken out to a smaller box for one day before being de-aired in the vacuum cell. Finally, the slurry was poured into the shear box of the test apparatus (Fig. 2b). Before the cyclic shear test, the slurry was pre-consolidated under the vertical stress σ_{v0} = 49 kPa until the pore water pressure at the bottom surface of the specimen is dissipated. After the pre-consolidation, the dimension of specimen was 75 mm in diameter and about 20 mm in height and the initial void ratio (e_0) was about e_0 = 1.61-1.81 and 1.11-1.19 for Kitakyushu clay and Kaolinite clay, respectively. Since the pore pressure coefficient of soil specimen before undrained cyclic shear was confirmed as B-value > 0.95, the saturation of the specimen can be satisfied.

2.3 Test procedures and conditions

After pre-consolidation is completed, the soil specimen was subjected to the strain-controlled cyclic simple shear under the undrained condition for predetermined cyclic shear direction, shear strain amplitude (γ) and the number of cycles (n). Following the undrained cyclic shear, drainage was allowed. Then the settlement and the pore water pressure at the bottom surface of the specimen were measured with time.

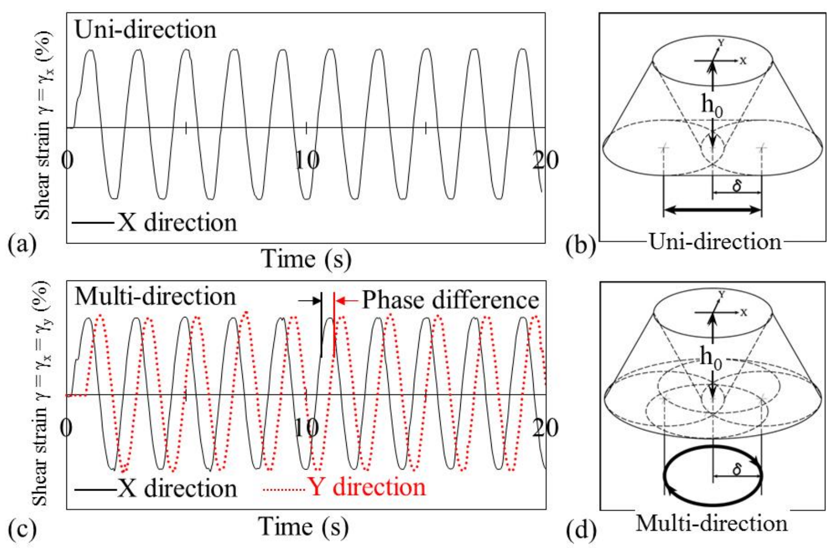


Fig. 3 – (a, c) Typical records of cyclic shear strains and (b, d) respective deformations of specimen under uni-directional and multi-directional cyclic shears

Figs. 3(a)-(d) show the typical records of cyclic shear strains and the deformation modes of the specimen under uni-directional and multi-directional cyclic shears. The shear strain amplitude γ is defined as a ratio of the maximum horizontal displacement (δ) to the initial height (h_0) of specimen (Figs. 3b and d). In uni-directional test, the shear strain was applied to the specimen only in X direction ($\gamma = \gamma_x$) (Figs. 3a and b) and so the orbit of cyclic shear strain forms a linear line (Fig. 3b). In multi-directional test (Figs. 3c and d), the cyclic shear strain was simultaneously applied to the specimen in X direction (γ_x) and Y direction (γ_y) which are perpendicular to each other under the same shear strain amplitude ($\gamma = \gamma_x = \gamma_y$) with various phase differences (θ). Then the orbit shows an elliptical line for $0^0 < \theta < 90^0$ and a circle line for $\theta = 90^0$ which is well known as a gyratory cyclic shear (Fig. 3d). A series of test conditions of uni-directional and multi-directional cyclic shear tests are shown in Table 1. The wave form of the cyclic shear strain was sinusoidal (two way cyclic strain) with the period T = 2 s and the shear strain amplitude was changed in the range from γ = 0.05% to 2.0%. The number of cycles was fixed as n = 200. For the multi-directional tests, the phase difference was changed as $\theta = 20^0, 45^0, 70^0$ and 90^0.

Table 1 – Conditions of uni-directional and multi-directional cyclic simple shear tests

Period *T* (s)	**Number of cycles *n***	**Shear strain amplitude γ (%)**
2	200	0.05, 0.1, 0.2, 0.4, 0.8, 1.0, 1.2, 2.0

3. Results and discussions

3.1 Pore water pressure accumulated in a high plasticity clay subjected to undrained uni-directional and multi-directional cyclic shears

3.1.1 Changes of cyclic shear-induced pore water pressure in saturated clay

As a result of the undrained cyclic shear, pore water pressure (U_{dyn}) increases with the number of cycles (n). For the uni-directional cyclic shear, Matsuda *et al*. [21, 25, 26, 29] proposed an equation based on the results for Kaolinite clay with low plasticity index, which shows the relations between the excess pore water pressure ratio U_{dyn}/σ'_{v0} and the number of cycles n as follows:

$$\frac{U_{dyn}}{\sigma'_{v0}} = \frac{n}{\alpha + \beta n} \tag{1}$$

where σ'_{v0} is the initial effective stress, α and β are the experimental parameters and related to the shear strain amplitude γ as follows:

$$\alpha = A\gamma^m \tag{2}$$

$$\beta = \frac{\gamma}{B + C\gamma} \tag{3}$$

The experimental constants A, B, C and m in Eqs. (2) and (3) can be determined based on the results of multi-directional cyclic simple shear test.

The typical changes of the pore water pressure ratio U_{dyn}/σ'_{v0} on Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears are plotted in Fig. 4(a) for different shear strain amplitudes. In this figure, the observed results of the uni-directional tests are plotted in solid lines, while those of the multi-directional ones with $\theta = 90^0$ are shown by dotted lines. In order to show the changes of U_{dyn}/σ'_{v0} more in detail, Fig. 4(a) was redrawn by changing the vertical scale as shown in Fig. 4(b). In addition, the results on Kaolinite clay are also shown in Fig. 4(c). It is seen from these figures, U_{dyn}/σ'_{v0} gradually increases with n in Kitakyushu clay but in Kaolinite clay, rapidly increases after the cyclic shear starts. When at the same number of cyclic shears and shear strain amplitude, U_{dyn}/σ'_{v0} induced by multi-directional cyclic shear is significantly higher than those generated by the uni-directional one. Since the disturbance of soil structure under uni-directional cyclic shear has been confirmed [30] and the relationship between the pore water pressure accumulation and the cyclic degradation of clayey soil has been clarified [27, 28], it is suggested from the current results that the structure of clayey soils would suffer from the higher disturbance and degradation when subjecting to the multi-directional cyclic shear.

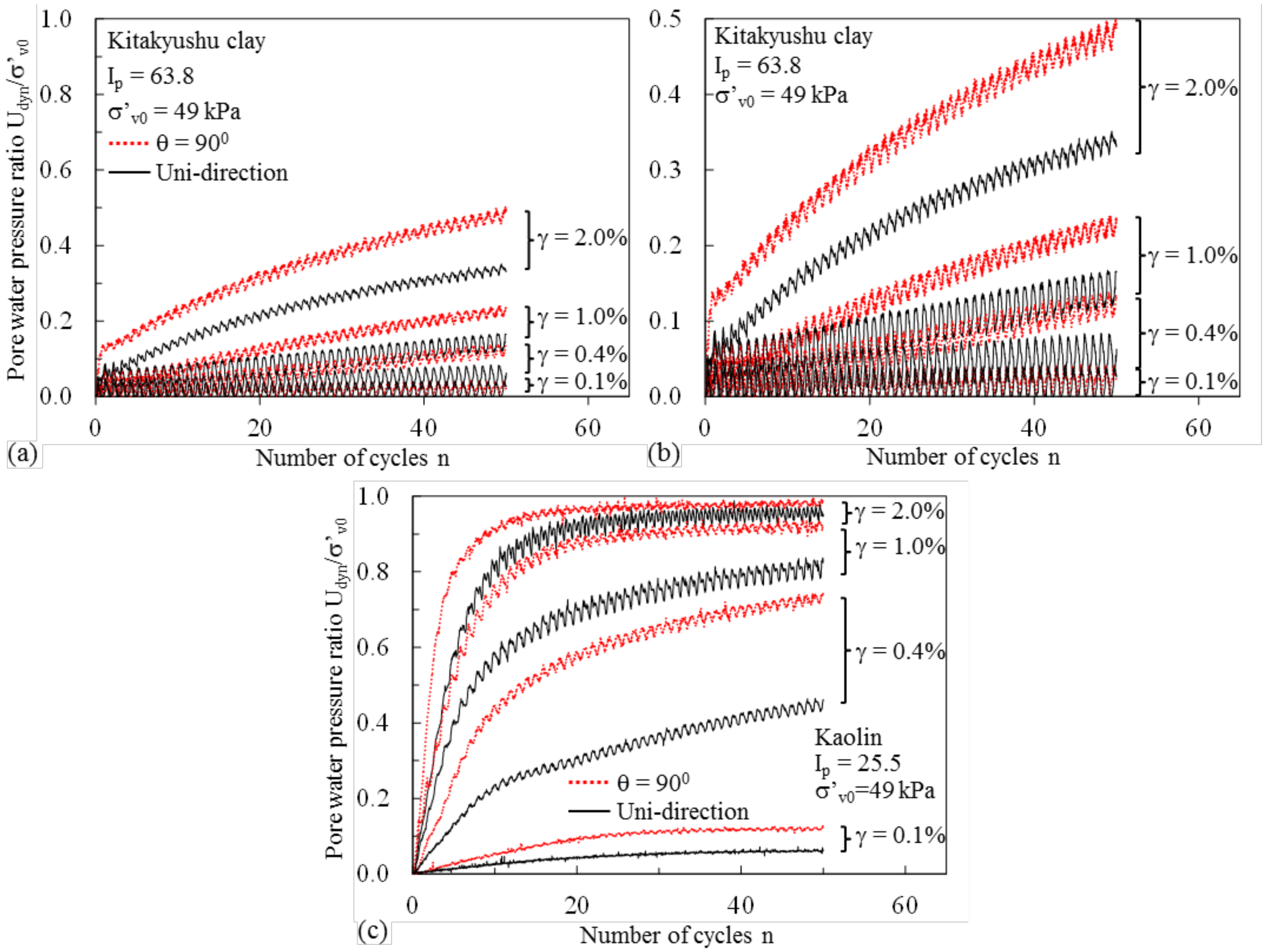


Fig. 4 – Typical changes of U_{dyn}/σ'_{v0} with n in (a and b) Kitakyushu clay and (c) Kaolinite clay during undrained uni-directional and multi-directional cyclic shears

When comparing the results of U_{dyn}/σ'_{v0} between Kitakyushu clay and Kaolinite clay, it is shown that under the same undrained cyclic shear conditions, the higher the Atterberg's limits of clay the lower the level of pore water pressure accumulation. Based on the experimental data on normally and over-consolidated clays and sands, Vucetic and Dobry [31] concluded that plasticity index is one of the most important index properties when evaluating the dynamic behavior of soil, especially for the fine-grained soil deposits. Also, authors indicated that the soil with higher plasticity index shows more linear stress-strain response and less stiffness degradation at a given amplitude of cyclic shear strain. Therefore, results obtained in this study indicate that the level of pore water pressure accumulation and stiffness degradation in Kitakyushu clay are lower than those of Kaolinite clay, and that the effects of uni-directional and multi-directional cyclic shear on the undrained cyclic behavior of clay with a high plasticity index are negiligible when $\gamma \leq 0.1\%$.

3.1.2 Estimation of cyclic shear-induced pore water pressure on clay with high plasticity index

By using Eq. (1), the cyclic shear-induced pore water pressure was estimated. In Fig. 5, relationships between U_{dyn}/σ'_{v0} and the number of cyclic shears are shown, in which symbols show the observed results and the solid curves correspond to the calculated ones by using Eq. (1). Reasonable agreements between them are observed.

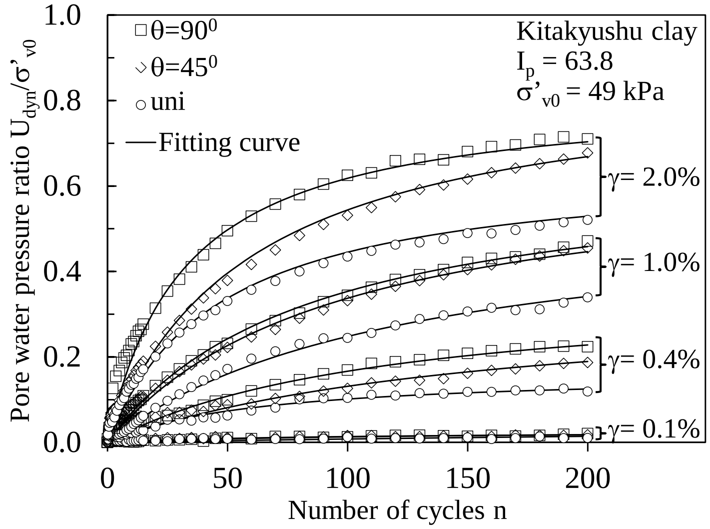


Fig. 5 – Relations between U_{dyn}/σ'_{v0} and n for Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears

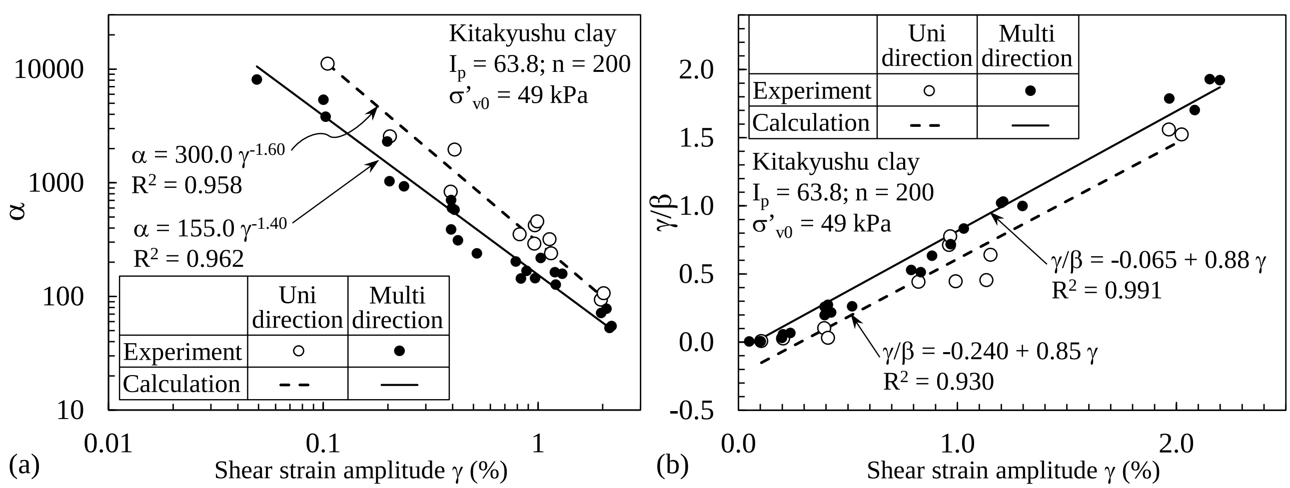


Fig. 6 – Relations of α and β versus γ for Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears

To find the experimental parameters α and β (or A, B, C and m) in Eqs. (2) and (3), relations of α and β versus γ are plotted in Fig. 6(a) and (b) in which dashed and solid lines are fitted lines for uni-directional and multi-directional cyclic shear results, respectively. From these figures, experimental constants are obtained and shown in Table 2.

Table 2 – Experimental constants A, B, C and m obtained for Kitakyushu clay

Cyclic shear direction	***A***	***B***	***C***	***m***
Uni-direction	300.0	-0.240	0.85	-1.60
Multi-direction	155.0	-0.065	0.88	-1.40

In Fig. 7(a), relationships between U_{dyn}/σ'_{v0} and shear strain amplitude γ are plotted for Kitakyushu clay subjected to uni-directional and multi-directional cyclic shears. Symbols in this figure show the experimental results and solid and dashed lines correspond to the calculated ones by using Eq. (1), where the values of *A*, *B*, *C* and *m* in Table 2 were used. To confirm the accuracy of Eq. (1) the observed and calculated results are compared in Fig. 7(b). Reasonable agreements between them are seen and therefore the applicability of the obtained experimental constants in Table 2 is confirmed for Kitakyushu clay.

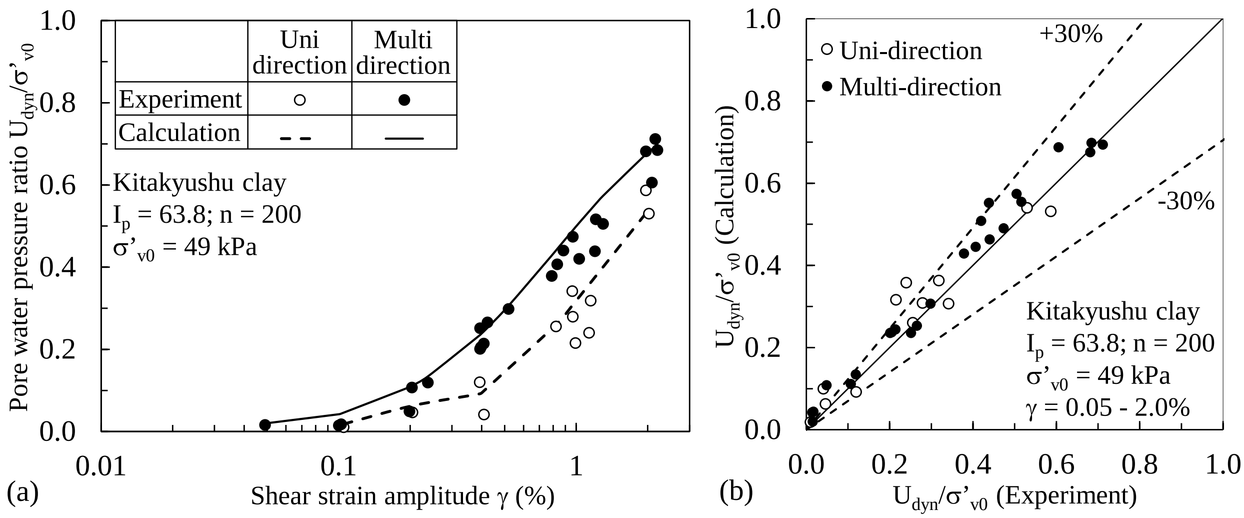


Fig. 7 – Comparisons between observed and calculated results for the relations of U_{dyn}/σ'_{v0} versus γ on Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears

For saturated sands, Matsuda *et al*. [32, 33] indicated that the effective stress reduction ratio, which is defined by $|\Delta\sigma'_v/\sigma'_{v0}|$ where $\Delta\sigma'_v$ is the decrease of effective vertical stress during equivalent undrained cyclic shear (constant volume condition), induced by uni-directional and multi-directional cyclic shears converges to the same level when $\gamma \geq 0.3\%$ and so the effect of cyclic shear direction on $|\Delta\sigma'_v/\sigma'_{v0}|$ is negligible when $\gamma \geq 0.3\%$. Meanwhile, Figs. 5 and 7 suggest that the effect of cyclic shear direction on the pore water pressure accumulation of high plasticity clay is evident and seems to increase even after a long-term application of strong cyclic shear ($\gamma = 2.0\%$ and $n = 200$).

3.2 Cyclic shear-induced settlement of clay with high plasticity index

3.2.1 Estimation of cyclic shear-induced settlement

Although to estimate the post-earthquake settlement of clay layer, many reports have already been published [13, 14, 21, 30, 34, 35, 36], Ohara and Matsuda [26] derived Eq. (4) by which the settlement in strain ε_V (%) after the undrained uni-directional cyclic shear can be estimated.

$$\varepsilon_v = \frac{\Delta h}{h_0} = \frac{\Delta e}{1+e_0} = \frac{C_{dyn}}{1+e_0}\log\left(\frac{1}{1-\dfrac{U_{dyn}}{\sigma'_{v0}}}\right) = \frac{C_{dyn}}{1+e_0}\log SRR \tag{4}$$

where Δh is the settlement, h_0 is the initial height of the clay layer, Δe is the change of void ratio, e_0 is the initial void ratio, C_{dyn} is the cyclic recompression index and *SRR* denotes the stress reduction ratio which is defined by $1/(1-U_{dyn}/\sigma'_{v0})$.

3.2.2 Change of void ratio

Relationships between the change of void ratio Δe and the stress reduction ratio *SRR* are shown in Fig. 8(a) for Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears with a wide range of γ. Symbols in this figure show observed results, and solid and dashed lines show the calculated ones by using Eq. (4). Further comparisons between the observed results and calculated ones are also shown by symbols in Fig. 8(b). The calculated results agree well with the observed ones.

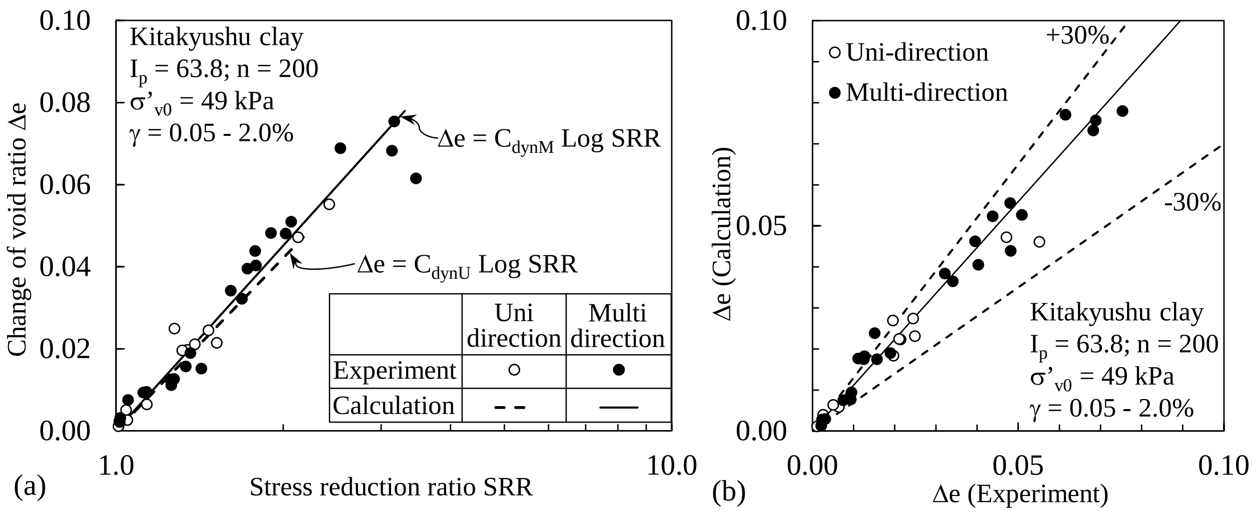


Fig. 8 – Comparisons between observed and calculated results for the relations of Δe versus *SRR* on Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears

It is seen in Fig. 8(a) that Δe increases in proportion to the logarithm of *SRR* and that this relation is independent of shear strain amplitude but being affected by cyclic shear direction. Therefore, the individial values of C_{dyn} are obtained for the case of uni-directional cyclic shear (C_{dynU}) and multi-directional one (C_{dynM}), respectively, and relations as C_{dynU} = 0.14 = 0.233×C_c and C_{dynM} = 0.15 = 0.250×C_c have been obtained, where C_c is the virgin compression index. These relations are very close to C_{dyn} = 0.225×C_c proposed for normally consolidated Drammen clay by Yasuhara and Andersen [30] and Yasuhara *et al*. [35].

3.2.3 Prediction of post-cyclic settlement of clay with high plasticity index subjected to undrained uni-directional and multi-directional cyclic shears

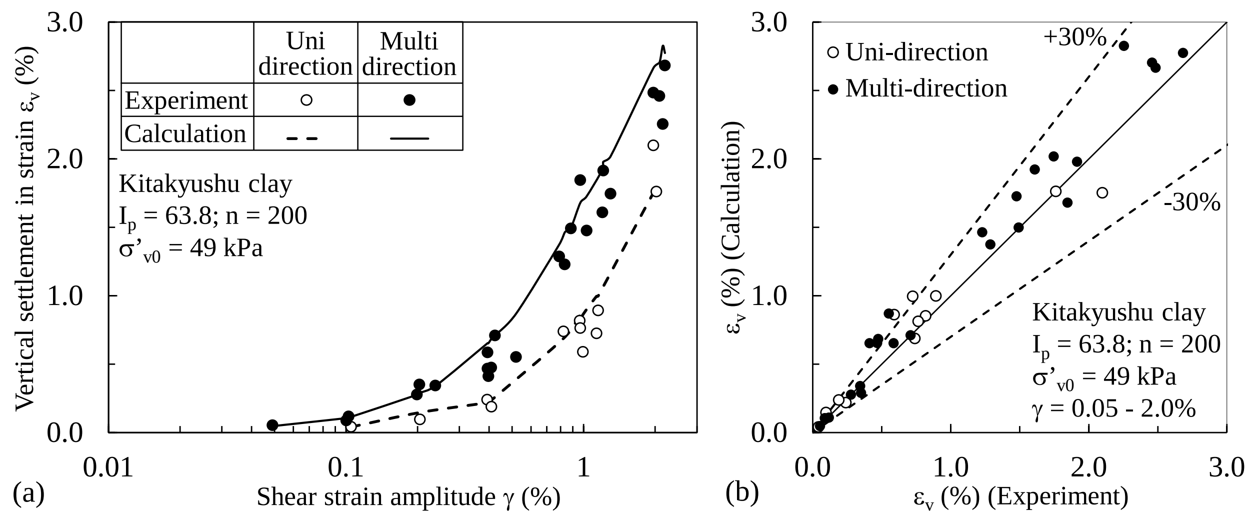


Fig. 9 – Comparisons between observed and calculated results for the relations of ε_v versus γ on Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears

Relationships between post-cyclic settlement in strain ε_v (%) and γ are shown in Fig. 9(a) for Kitakyushu clay subjected to undrained uni-directional and multi-directional cyclic shears. Symbols in this figure show observed results, and solid and dashed lines show the calculated ones by using Eq. (4). When at the same γ, ε_v induced by multi-directional cyclic shear is larger than those generated by the uni-directional one and this means that the cyclic shear direction affects the settlement of high plasticity clay subjected to multi-directional cyclic shear. In addition, the calculated results are plotted against the observed ones in Fig. 9(b). Reasonable agreements are seen for both case of uni-directional and multi-directional cyclic shears and therefore the applicability of the proposed method of post-cylic settlement was confirmed even for a high plasticity clay.

4. Conclusions

By using the multi-directional cyclic simple shear test apparatus, normally consolidated specimens of Kitakyushu clay which is characterized by high Atterberg's limits were subjected to several series of undrained cyclic simple shear. Then the effect of undrained cyclic shearing including the cyclic shear direction on the pore water pressure accumulation and on the recompression was observed. The main conclusions are as follows:

For a clay with high plasticity index like Kitakyushu clay, the cyclic shear direction significantly affects the pore water pressure accumulation and post-cyclic settlement even under long-term application of strong cyclic shearing ($\gamma \geq 2.0\%$ and $n \geq 200$). This behaviour is different from those of Kaolin with lower Atterberg's limit (I_p = 25.5) and sand, in which the effect of cyclic shear direction becomes negligible when $\gamma \geq 2.0\%$ and $\gamma \geq 0.3\%$, respectively.

The conventional estimation method of cyclic shear-induced pore water pressure for a clay with high plasticity index was developed in both cases of uni-directional and multi-directional cyclic shears and the post-cyclic settlement was also predicted by using the cyclic recompression indices.

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