UNDRAINED CYCLIC SHEAR BEHAVIOUR OF NORMALLY CONSOLIDATED CLAY SUBJECTED TO INITIAL STATIC SHEAR STRESS

MASAYUKI HYODO(0), YOICHI YAMAMOTO(0) and MOTOHIRO SUGIYAMA(0)

ABSTRACT

A series of undrained cyclic triaxial compression tests has been performed on a high plasticity marine clay. Testing was performed using various combinations of initial static and subsequent cyclic shear stress on isotropically and anisotropically consolidated specimens. Additionally, monotonic triaxial compression and extension tests were also performed on the same clay with the same initial condition. The cyclic shear strength was considered first, and then, the accumulated shear strain was investigated related to the effective stress ratio at the peak cyclic stress. Based on the experimental results, a semi-empirical model is proposed for evaluating the development of pore pressure and residual shear strain during cyclic loading. The model successfully explains the behaviour of clay subjected to various magnitudes of initial static and subsequent cyclic shear stresses.

Key words: clay, cyclic strength, deformation, pore pressure, repeated load, static shear, stress path, triaxial compression test (IGC: D7/D6)

INTRODUCTION

In considering earthquake design problems, interest has focused on the liquefaction of saturated sands. During earthquakes, clays have been considered to be stable in comparison with sands. Serious damage to structures sited on thick clay layers, however, was reported in the 1985 Mexico Earthquake (Seed et al., 1987; Mendoza et al., 1988). Large ground deformations due to the amplification of seismic motion was recognized as a characteristic of clay behaviour during earthquakes. In addition, many fill collapses were caused in Japan due to the failure of clay base layers in the 1964 Niigata Earthquake and 1978 Miyagiken-oki Earthquake (Sasaki et al., 1980).

Such case histories from the previous earthquakes raise the question of whether or not clays are really more stable than sands during earthquakes. A comparison between the cyclic strengths for sand and clay has been made using laboratory tests, such as cyclic triaxial tests on isotropically consolidated specimens which simulate the horizontal layers. In this condition, clay may exhibit stability and the pore pressure in the clay may not develop up to the initial effective confining stress. Practically, however, most designs have been concerned with the base subsoils below structures or elements in slopes whose soil elements are generally subjected to the initial static shear stresses. In order to simulate these situations, the cyclic triaxial tests on anisotropically consolidated specimens or cyclic simple shear tests subjected to initial static shear stresses were performed.

In the design of offshore platforms to sustain wave loads, the cyclic properties of clays have been investigated and used in practical design methods for the stability of offshore platforms (Andersen et al., 1988a, b). Although clay dynamics has not been investigated as widely as the liquefaction of sands, many dynamic problems could potentially arise especially with the increasing construction of various structures founded on clay layers.

A series of undrained cyclic triaxial tests was performed on a plastic marine clay for this study. Testing was carried out for various combinations of initial static and cyclic shear stresses which are those expected on soil elements subjected to cyclic loading in the vicinity of structures. Based on the experimental results, a semi-empirical model for predicting pore pressure and residual shear strain developed during cyclic loading is proposed which will be a useful tool for practical design methods.

TESTING PROCEDURE

Monotonic and cyclic triaxial compression tests were

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1
carried out on both undisturbed and remoulded Itsukaichi clay, a marine clay sampled from a depth of 8–10 m below the seabed at Itsukaichi harbour, on the southwest coast of Hiroshima in Japan. The soil was quite homogeneous. Its index properties are $G_s = 2.532$, $w_r = 124.2\%$, $w_p = 51.4\%$ and $I_p = 72.8$. The undisturbed sample was taken with a thin walled tube 1 m long, 75 mm in diameter. In order to prepare the remoulded sample, the clay slurry mixed with an initial water content of 260% was poured into a consolidation vessel and was then preconsolidated with a vertical pressure of 50 kPa. The specimens with initial dimensions of 50 mm diameter and 100 mm height were trimmed from the clay block. In order to obtain a high degree of saturation, a back pressure of 100 kPa was applied for 1 hour. $B$-values of more than 0.97 were observed in all specimens used in the tests. To reduce the effect of friction on the clay behaviour during the test, two thin rubber sheets coated with silicone grease were placed between the lower and upper porous stones and the specimen and a hole 3 mm in diameter was made at the center of the lower membrane. Pore pressure was measured through the porous stone 3 mm in diameter in the center of the lower pedestal. To accelerate consolidation a filter paper side drain was placed along the circumference of the sample to cover about 40%. Drainage occurred through the filter paper surrounding the specimen which was attached to the side of both porous stones. The thickness of rubber membrane surrounding the specimen was 0.2 mm and its extension force was confirmed in summarizing the data although the effect was not significant. Normally consolidated specimens were initially consolidated isotropically and after a period of 24 hours consolidated anisotropically by applying a static deviator stress at a constant mean principal stress of 200 kPa in the triaxial cell.

A sinusoidal cyclic axial load was applied at a frequency of 0.02 Hz, which was determined after investigating the homogeneity of the generation of pore pressures in specimens under undrained cyclic loading (Yamamoto et al., 1991; Hyodo et al., 1993). The conditions for the cyclic triaxial tests are summarized in Table 1. Test Nos. DN10 to DN12 in the table are tests on undisturbed samples and the others are those on remoulded ones. Stress parameters $p$ and $q$ are used to represent the effective mean principal stress, $p=(\sigma_1^e+2\sigma_3^e)/3$, and deviator stress, $q=\sigma_3^e-\sigma_1^e$, respectively. The symbols $\sigma_1^e$ and $p_e$ represent the lateral pressure and the value of $p$ after consolidation, respectively in which $\sigma_1^e$ and $\sigma_3^e$ are axial and lateral stresses. Cyclic loading tests were performed over a range of initial static deviator stress ratio $q_e/p_e$ varying from 0 to 1.2 at 0.3 intervals. From three to four magnitudes of cyclic deviator stress $q_{cyc}$ were combined with each static deviator stress level so that both reversal and non-reversal of cyclic shear stresses were simulated.

Strain controlled undrained monotonic triaxial compression and extension tests were also performed on normally consolidated samples with the same initial condition after isotropic and anisotropic consolidation. They were executed by applying a monotonic axial load with a strain rate of 0.1%/min. These tests were performed in order to compare their behaviour with the results of cyclic triaxial tests. The conditions for the monotonic triaxial tests are shown in Table 2 in which undisturbed samples are represented as Tests No. DN01C and DN02E while the others are remoulded ones.

### Table 1. Conditions of cyclic triaxial tests on clay

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$p_e$ (kPa)</th>
<th>$\sigma^e_1$ (kPa)</th>
<th>$q_e$ (kPa)</th>
<th>$f$ (Hz)</th>
<th>$w_e$ (%)</th>
<th>$e_1$</th>
<th>$e_e$</th>
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### Table 2. Conditions of monotonic triaxial tests on clay

<table>
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<th>Test No.</th>
<th>$p_e$ (kPa)</th>
<th>$\sigma^e_1$ (kPa)</th>
<th>$q_e$ (kPa)</th>
<th>$f$ (Hz)</th>
<th>$w_e$ (%)</th>
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<td>2.246</td>
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**COMPARISON BETWEEN UNDISTURBED AND REMOULDED CLAYS**

In order to make a comparison between the cyclic and monotonic behaviour of undisturbed and remoulded samples, undrained cyclic and monotonic triaxial tests were performed on both types of specimen normally consolidated to 200 kPa. Fig. 1 shows the relationship between the cyclic deviator stress ratio and the number of cycles required to induce a 10% double amplitude axial strain, $DA$, which is considered to be a failure in cyclic triaxial tests as will be explained later. Furthermore, Figs. 2(a), (b) present the relationship between deviator stress and axial strain and the effective stress path, respectively, which were obtained by monotonic compression and extension tests on undisturbed and remoulded samples. As
such as chemical bonding or secondary compression for undisturbed samples was destroyed by applying confining stress to a normally consolidated state and that the characteristics changed to equal those of young clay remoulded in the consolidation vessel, is considered to be the reason for this. Only the results of remoulded samples are therefore considered below.

**CYCLIC BEHAVIOUR OF ISOTROPICALLY AND ANISOTROPICALLY CONSOLIDATED CLAY**

Cyclic triaxial tests were performed using both reversal and non-reversal of cyclic shear stresses. Typical effective stress paths during cyclic loading for normally consolidated clay for each loading pattern are presented in Fig. 3. The failure envelopes obtained from undrained monotonic tests on isotropically consolidated specimens are also drawn in each figure. The slopes of the envelopes for compression and extension are $M_e = 1.560$ and $M_e = 1.456$, respectively. In the isotropically consolidated case, as shown in Fig. 3(a), the effective stress path moved towards the failure envelopes during cyclic loading and finally traced a steady loop which reached the failure envelopes for both the compression and extension sides. Pore pressure measured at the bottom of the specimen through the pedestal did not develop up to the initial confining stress. In the anisotropically consolidated results, as shown in Figs. 3(b), (c), and (d) and (e) the effective stress paths moved until their upper part intersected the failure line on the compression side. In addition, it was found that the stress paths crossed the failure envelopes at the final stage of cyclic loading except in the case of the highest normalized initial static deviator stress $q_s/p_c = 1.2$. The phenomenon was also observed by Hyde and Ward (1985) and was explained as a condition where the samples became more heavily overconsolidated under undrained conditions.

Typical results demonstrating the relationship between cyclic deviator stress and axial strain are presented in Fig. 4. Smoothly expanding stress-strain hysteresis curves appeared for the isotropically consolidated clay as shown in Fig. 4(a). In the case of stress reversal from the compression to the extension side in the results presented in Figs. 4(a) and (b), significant magnitude of strain amplitude developed near the failure stage of cyclic loadings while in the non-reversal case, as illustrated in Figs. 4(c), (d) and (e) residual strain was predominant instead of the cyclic component. Furthermore, it should be noted that an increase in the residual strain of clay during a cycle was triggered at the final stage of cycling when the stress paths approached the critical state line.

**UNDRAINED MONOTONIC TRIAXIAL BEHAVIOUR**

Undrained monotonic compression and extension tests were performed with the conditions shown in Table 2. All test results are presented together in Figs. 5 and 6 which illustrate the effective stress paths and the relation-
ship between deviator stress and axial strain, respectively. While both compression and extension tests were carried out from several different anisotropically consolidated conditions with various initial deviator stresses, these initial deviator stresses were applied only on the compression side. Considering the effective stress path on the compression side, after reaching a local peak deviator stress, strain softening behaviour occurred. The tendency of strain softening becomes more noticeable with increasing initial deviator stress and finally the critical state strength became smaller than the initial deviator stress at the high initial deviator stress condition. On the extension side, stress paths for anisotropically consolidated specimens trace inner paths with increasing initial static deviator stress. Only the results for the highest initial static deviator stress ratio $q_s/p_s=1.2$ showed a dilative manner at the final stage.

The stress-strain curves for compression tests show that the softening behaviour appears after reaching the peak strength at a fairly small strain stage. It was found that the peak strength increases with increasing initial deviator stress. The degree of stress decrement due to softening, however, is more rapid with increasing initial deviator stress. The extension tests exhibited only hardening behaviour.

**EVALUATION OF ACCUMULATED AXIAL STRAIN DURING CYCLIC LOADING**

Attempts were made to quantify the accumulated axial strain corresponding to the peak cyclic stress on the compression side for each loading cycle. The peak axial strains $\varepsilon_p$ from all tests were compared using an effective stress ratio $\eta_p (= q_s/p)$ which is mobilized during cyclic loading and represents the ratio of the peak deviator stress divided by the corresponding mean effective prin-
cipal stress at the peak value. The schematic diagram for presenting $\eta_p$ at an arbitrary number of cycles and the corresponding $\varepsilon_p$ is illustrated in Fig. 7. Fig. 8 presents the relationship between peak axial strain $\varepsilon_p$ and effective stress ratio $\eta_p$ for cyclic tests on normally consolidated specimens. Although some scatter occurs, generally in the figures, there is a unique relationship between peak axial strain and effective stress ratio although the magnitudes of initial static and applied cyclic deviator stresses are different from each other. Furthermore, it was found as shown in the figure that although the starting points are different for each initial static deviator stress, most of the relationship can be approximated by a unique hyperbola given by the following equation.

$$\varepsilon_p = \frac{a_1 \eta_p}{1 - \eta_p/\eta_{\text{sat}}}$$

where $1/a_1$ means the slope of the initial tangential line of $\varepsilon_p - \eta_p$ curve and $\eta_{\text{sat}}$ is the value of $\eta$ at the asymptote of the hyperbola. The parameters were determined to be $a_1=0.5$ and $\eta_{\text{sat}}=2.0$ based on these results, respectively.

A similar relationship was also observed in anisotropically consolidated sand (Hyodo et al., 1991).

**CYCLIC SHEAR STRENGTH**

It is convenient to make a unified definition of cyclic shear strength for both reversal and non-reversal stress conditions. In the relationship shown in Fig. 8, the cyclic failure is defined as the peak accumulated axial strain $\varepsilon_p=10\%$ because the hyperbola shown in this figure approaches asymptotically at about $\varepsilon_p=10\%$. Considering this failure criterion for both the reversal and non-reversal regions, the relationship between the cyclic deviator stress ratio and the number of cycles required to cause failure for each initial static deviator stress $q_s$ is represent-
ed as shown in Fig. 9. The cyclic strength for the clay with zero initial static deviator stress ratio was determined to be 10% of the double amplitude of axial strain, $DA$, instead of $e_0$, although it is not certain whether both failure criteria are equivalent to each other or not. It can be observed in this figure that the cyclic shear strength for clay decreases with increasing initial static deviator stress. In order to compare the cyclic shear strength for clay with that for sand, the cyclic shear strength curves for isotropically consolidated clay and Toyoura sand with $Dr=50$% and 70$\%$ (Hyodo et al., 1991) are presented in Fig. 10. In the isotropically consolidated condition, the cyclic strength for clay is about two or three times as high as that for sand.

The variations of cyclic deviator stress ratio required to cause $e_0=10\%$ in 20 cycles with initial static deviator stress ratio for both clay and sand are summarized in Fig. 11. It can be seen in both of these figures that the cyclic deviator stress ratio to cause failure in clay decreases with increasing initial static deviator stress. Cyclic shear strength for sand increases with increasing initial devia-
Fig. 10. Relationship between cyclic deviator stress ratio \( q_{ov}/p_e \) and number of cycles to develop double amplitude of axial strain \( DA=10\% \) for isotropically consolidated clay and sand

Fig. 11. Relationship between normalized cyclic deviator stress and initial static deviator stress to cause \( \varepsilon_f=10\% \) at 20 cycles for clay and sand

Fig. 12. Relationship between cyclic deviator stress ratio \( (q_i+q_{ov})/p_e \) and number of cycles to cause peak axial strain \( \varepsilon_f=10\% \)

Fig. 13. Relationships between cyclic strength \( R_i \) and initial static deviator stress ratio to cause \( \varepsilon_f=10\% \) at 5, 20, 100 cycles

In the isotropically consolidated condition, the cyclic strength for clay is certainly greater than that for sand. The inequality is reversed however at a certain initial static deviator stress. Especially in the non-reversal region, cyclic shear strength for clay is far lower than that for sand. It appeared therefore that the clay subjected to initial shear stress, such as a layer upon which the structure is founded, is more unstable than sand during cyclic loading.

For the purpose of development of the empirical model, redefining the cyclic strength as \( R_i=((q_{ov}+q_i)/p_e)/f \), where subscript \( f \) means the state of failure, it can be approximated by straight lines in logarithmic form. These are parallel straight lines for each initial static deviator stress as shown in Fig. 12 in which the cyclic strength for \( q_i/p_e=0 \) is also illustrated and formulated by the following equation:

\[
R_i=\left(\frac{q_{ov}+q_i}{p_e}\right) = \kappa N^\beta
\]

where \( \beta \) is slope of each strength line given \( \beta=-0.088 \) and \( \kappa \) is cyclic strength for the first cycle which is related by normalized initial deviator stress as \( \kappa=1.0+1.5 q_i/p_e \).

The variations in cyclic strengths with initial static deviator stress ratio for 5, 20 and 100 cycles with initial static deviator stress ratio are illustrated in Fig. 13.

RELATIVE EFFECTIVE STRESS RATIO AND RELATIVE CYCLIC SHEAR STRESS

In the previous section of this paper, a unified cyclic shear strength, \( R_i=((q_{ov}+q_i)/p_e)/f \), applicable to all the loading patterns was defined. In order to represent the undrained cyclic behaviour of clay, the following two parameters are introduced. The first parameter is defined as an index of the possibility of cyclic failure, \( RR(N) = R/R_i \) which is the ratio of the peak cyclic deviator stress, \( R=q_i+q_{ov} \), to the cyclic shear strength, \( R_i=(q_{ov}+q_i)/p_e \) in a given number of cycles. \( RR(N) \), named relative cyclic shear stress, whose schematic diagram is shown in Fig. 14, is equivalent to the reciprocal of the safety factor against cyclic failure. When the mag-
nitude of $R$ is constant, $RR(N) = R/R_f$ increases with increasing number of cycles and varies from zero at nonloading to unity at failure.

The second parameter is defined as:

$$\eta^* = \frac{\eta_p - \eta_0}{\eta_f - \eta_0}$$

(3)

where $\eta_p$ is the effective stress ratio at the peak of the cyclic stress in each cycle, $\eta_f$ is the effective stress ratio for initially consolidated condition and $\eta_f$ is the effective stress ratio at failure. This parameter, $\eta^*$, therefore, indicates the relative effective stress ratio between the initial point and the final point in $p-q$ space as shown in Fig. 15. These parameters were originally introduced for sand by Hyodo et al. (1988, 1991) and also applied to isotropically consolidated clay (Hyodo et al., 1992, 1993). By correlating the values of both parameters, we obtain Fig. 16 for each $q_s/p_c$. Despite the difference in initial static and subsequent cyclic deviator stresses, the best fit curve for each relation is given by a unique curve formulated by the following equation:

$$\eta^* = \frac{R/R_f}{\{a_2 - (a_2 - 1)R/R_f\}}$$

(4)

where $a_2$ was determined experimentally as 6.5.

Cyclic-induced peak axial strain is calculated using the following procedure:

1. The cyclic shear strength $R_f$ for the desired initial static deviator stress and number of stress cycles is determined from the relationship given by Eq. (2). Then the relative cyclic shear stress $R/R_f$ is obtained by dividing the applied stress ratio $R$ by the strength $R_f$.

2. The relative effective stress ratio $\eta^*$ is obtained by substituting $R/R_f$ into the relationship between $\eta^*$ and $R/R_f$ given by Eq. (4).

3. The effective stress ratio $\eta_p$ at the peak cyclic stress of a given stress cycle is calculated by the following rewritten form of Eq. (3).

$$\eta_p = \eta^*(\eta_f - \eta_0) + \eta_0$$

(5)

4. The pore pressure at the peak axial strain is calculated substituting $\eta_p$ into the following equation.

$$u_p = p_c + q_{cyc}/3 - (q_s + q_{cyc})/\eta_p$$

(6)

The peak axial strain is evaluated by substituting $\eta_p$ into Eq. (1).

If these steps are repeated from the first to the last stress cycle, the accumulated peak pore pressures and peak axial strains can be predicted for the number of cycles during cyclic loading. The predicted and observed pore pressures and axial strains are presented in Figs. 17 and 18 in which the results for each initial static and subsequent cyclic shear stresses are illustrated. In these figures, the predicted and observed results correspond to solid lines and plots, respectively.

Fairly good correspondence is recognized between the predicted and experimental results in spite of very complicated initial conditions. The difference which appeared in a few cases as shown in the figures is mainly due to the deviation of the cyclic stress plots from their best fit curves as shown in Fig. 12. It was confirmed that the proposed model is a reasonable method for accumulating...
Fig. 17. Predicted and experimental pore pressure at the peak cyclic stress

Fig. 18. Predicted and experimental accumulated peak axial strain

the cyclic-induced pore pressure and shear strain of clay subjected to various magnitudes of initial static and subsequent cyclic shear stresses.

POST-CYCLED RECOMPRESSURE OF ISOTROPICALLY AND ANISOTROPICALLY CONSOLIDATED CLAY

Samples which failed under undrained cyclic loading were allowed to be reconsolidated when measurements of volumetric strain were taken after full dissipation of the residual pore water pressure. Figure 19 shows the relationship between the post cyclic volumetric strain and the normalized accumulated peak pore water pressure. Similar relationship was investigated and formulated on isotropically consolidated clay by Yasuhara et al. (1992). It was found that there is a unique non-linear relationship which includes points for all initial isotropic and anisotropic consolidation conditions. The curve in this figure was drawn based on the following equation.
Fig. 19. Relationship between post-cyclic volumetric strain and normalized peak pore pressure at the end of cyclic loading

$$\varepsilon_v = \frac{C_r}{1 + e_0} \log \left( \frac{1}{1 - u_p / p_c} \right)$$  \hspace{1cm} (7)

where $C_r$ is coefficient of recompressibility and is obtained as $C_r = 0.243$ for Itsukaichi clay in the experiments and $e_0$ is initial void ratio before cyclic loading. The post-cyclic volumetric strain $\varepsilon_v$ can be evaluated by substituting the magnitude of peak pore pressure at the end of each applied cyclic loading obtained from Eq. (6) into Eq. (7).

CONCLUSIONS

A series of cyclic triaxial compression tests with various initial static and subsequent cyclic shear stresses was carried out on a high plasticity marine clay (Itsukaichi clay). At the same time, monotonic triaxial compression and extension tests were performed. The cyclic behaviour of the clay was compared with that of Toyoura sand. The following conclusions were reached based on the experimental investigation.

1. In cyclic tests for clay, large deformations were observed not only in reversal but also non-reversal cyclic loading and especially at the final stage of cycling, a large increment of strain occurred.

2. Cyclic shear strength of clay was investigated and compared with that of Toyoura sand. The cyclic strength defined by development of a specified amount of peak axial strain was used to include the whole initial static shear stress region for both reversal and non-reversal cyclic stress conditions. The cyclic shear strength based on this definition tended to decrease with increasing initial static shear stress. It was the opposite tendency from that for Toyoura sand. It was observed that cyclic strength of clay decreased below that of sand in the high initial static shear stress region.

3. A unique relationship was recognized between peak axial strain and mobilized effective stress ratio measured for each stress cycle during cyclic loading, despite various initial static and subsequent cyclic shear stresses. The best fit curve for this relationship was a hyperbola.

4. We found that the new parameters, the relative effective stress ratio $\eta^*$ and the relative cyclic shear stress, $RR(N) = R / R_0$, used in the present paper are uniquely correlated with each other independent of the magnitude of initial static deviator stress and intensity of cyclic load.

5. By using the relationships noted in (3) and (4), it was possible to predict the accumulated pore pressure and peak axial strain of clay with initial static shear stress subjected to cyclic loading.

6. A unique non-linear relationship was found between post cyclic volumetric strain and cyclic induced pore pressure although the magnitude of initial static and subsequent cyclic deviator stresses were different.

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