PREDICTION OF CLAY BEHAVIOUR IN UNDRAINED AND PARTIALLY DRAINED CYCLIC TRIAXIAL TESTS

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ABSTRACT

Results from undrained cyclic triaxial tests followed by drainage on a reconstituted marine clay with high plasticity have been formulated to model the generation and dissipation process of cyclic induced excess pore pressure in clay. Through the introduction of two stress parameters in the formulation related to the relative cyclic strength and position in the effective $p'-q$ stress space a unique relationship was found between the peak stress ratio on the extension side and the double amplitude shear strain under undrained cyclic loading. The excess pore pressure model proposed using this unique relation between the two defined parameters was then combined with the theory of axisymmetrical consolidation with radial drainage to predict the behaviour of clay under partially drained cyclic loading. The results of analyzing the behaviour during partially drained conditions were compared with the observed values of pore pressures and shear strains in drained-cyclic triaxial tests. It is suggested from comparisons that the proposed model for cyclic-induced pore pressures and shear strains should provide a practical tool for evaluating the settlements and stability of clay under partially drained cyclic loading.

Key words: clay, cyclic load, partial drainage, pore pressure, shear strain (IGC: D7/D6)

INTRODUCTION

Cyclic loading situations such as earthquakes, traffic and wave loading may induce a decrease in bearing capacity and settlement or displacement which will lead to the damage of structures founded on soft clays. The earthquakes at Mexico City in 1957 (Zeevearts, 1983) and 1985 and at Miyagioki in Japan in 1978 (Suzuki, 1984) are typical examples pertaining to this category.

On the other hand, the situations observed in seabed clays beneath nearshore and

offshore structures are different from those in ground during earthquakes in that cyclic loading is more long-term. Typical case histories involved in this category of cyclic loading are those such as the landslide on the Mississippi delta (Henkel, 1970) and the bearing capacity or settlement of seabed clay at offshore platforms for oil exploitation in the North Sea (Eide and Andersen, 1984; Andersen et al., 1988, 1989).

Soft clay subgrades of low highway and railway embankments have suffered from differential settlements which may also be

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counted as one of the long-term cyclic loading problems of clays (Yamanouchi and Yasuhara, 1975; Kutara and Miki, 1980; Hyodo and Yasuhara, 1988).

In the case of short-term cyclic loading such as in earthquakes, clay will be under undrained conditions while in the case of a long-term cyclic loading clays may be assumed to be in a partially-drained situation, in which the generation and dissipation of excess pore pressure during each cycle are alternately repeated (Yasuhara, Hirao and Hyodo, 1988).

To examine the behaviour of clay under long-term or so called partially-drained cyclic loading, a model for predicting the residual strain and excess pore pressure has been formulated on the basis of the results from undrained triaxial tests on a reconstituted Ariake clay. Thereafter an attempt has been made to explain the behaviour of the time-dependent variations of shear strains and excess pore pressures observed during partially-drained cyclic loading on the same clay as was employed in the undrained cyclic triaxial tests.

TESTING PROCEDURE

Reconstituted Ariake clay was used for the cyclic triaxial tests. The index properties of this clay were: $G_s=2.58$, $w_L=123\%$, $I_p=69$, $C_c=0.700$, $C_s=0.163$, $M_c=1.60$ and $M_E=1.44$. The clay slurry mixed to an initial water content of 200% was poured into a large consolidation vessel and was then preconsolidated by a vertical pressure of about 60 kPa. After the full preconsolidation was attained a specimen for triaxial testing with 3.5 cm diameter and 8.75 cm height was trimmed from the clay block which was formed in the consolidation vessel. The average water content of each specimen was 90% to 95% as listed in Table 1. Excess pore water pressure was measured through a porous stone with 3 mm diameter buried at the center of the lower pedestal (Yasuhara, Hirao and Hyde, 1992).

Pore water was drained through filter paper surrounding the periphery of the specimen during preconsolidation and drained-cyclic shear. Volume change was measured by a double-

Table 1. Cyclic triaxial testing conditions

Test No.	Drainage condition	ω_f (%)	ω_c (%)	q_{cyc} (kPa)	$\frac{q_{cyc}}{p'_c}$	N_f (cycles)
K-26	Undrained	90.8	65.8	118	0.60	6
K-27	"	90.3	66.4	108	0.55	12
K-16	"	95.2	66.5	98	0.50	11
K-20	"	91.9	66.8	88	0.45	24
K-25	"	90.3	65.8	78	0.40	543
K-12	"	88.8	64.8	74	0.375	1674
D-1	Drained	90.7	66.8	108	0.55	11
D-2	"	92.2	67.8	98	0.50	36
D-3	"	90.9	66.5	88	0.45	—
D-4	"	91.1	66.9	78	0.40	—

($p'_c=196$ kPa, $f=0.1$ Hz)

tube burette.

The cyclic triaxial testing conditions are summarized in Table 1. The cyclic undrained and partially-drained triaxial tests are labeled K and D, respectively. After each specimen was isotropically consolidated for 24 hrs under 2.0 kgf/cm^2 (196 kPa) confining pressure, a cyclic shear stress whose shape was sinusoidal and of frequency 0.1 Hz was applied under stress controlled conditions.

In the drained cyclic tests, a cyclic shear stress was applied to a specimen while keeping the drainage line open throughout the duration of cyclic loading. This is therefore regarded as a sort of model test because the distribution of excess pore pressure through a specimen is not homogeneous during drained-cyclic loading. The method proposed on the basis of the results from undrained cyclic tests was applied to the analysis of the behaviour of clay under these partially-drained conditions. The interrelationship between each type of cyclic triaxial test is schematically illustrated in Fig. 1 using the $e-\log p'$ plot (Yasuhara, Hirao and Hyodo, 1988).

FORMULATION OF THE RESULTS FROM UNDRAINED CYCLIC TRIAXIAL TESTS

In the undrained cyclic triaxial test on a soil specimen, excess pore pressure builds up and shear strain amplitude develops with the number of load cycles, and then the specimen

Fig. 1. $e-\log p'$ representation for (a) cyclic loading followed by drainage and (b) partially drained cyclic loading

Fig. 2. Cyclic stress ratio versus number of load cycles relation from undrained and drained cyclic triaxial tests on reconstituted Ariake clay

may eventually undergo a cyclic failure.

Using logarithmic scales, Fig. 2 illustrates a relation between the cyclic stress ratio, $(q_{cyc}/p'_c)_f$ ($q_{cyc} = \sigma'_d$, $p'_c = \sigma'_c$) required for a 5% double amplitude shear strain. This relation is then formulated into:

$$R_f = (q_{cyc}/p'_c)_f = aN_f^b \quad (1)$$

where a and b are experimental constants equal to 0.624 and -0.071 , respectively, for the reconstituted Ariake clay used in the present study.

In addition to formulating the excess pore pressure, we also attempted to formulate the variation of cyclic shear strain with the number of load cycles. The double amplitude shear strain ε_{DA} in each cycle for every stress ratio was plotted against the residual excess pore pressure in Fig. 3. It cannot be said, however, from Fig. 3 that there is a good correlation between both values. Thus for correlating with ε_{DA} we adopted instead of the residual pore pressure, the effective stress ratio, $\eta_{p,e} = (q_{cyc}/p'_c)_{p,e}$, defined at the peak shear stress in each cyclic loading. The reason we adopted $\eta_{p,e}$ is that the cyclic failure of clay in triaxial tests is likely to occur on the extension side rather than on the compression side.

Fig. 4 shows the unique relation between ε_{DA} and $\eta_{p,e}$ in undrained cyclic triaxial tests, which does not depend on the magnitude of the cyclic shear stress. The results in Fig. 4 are formulated into:

$$\varepsilon_{DA} = \frac{\eta_{p,e}}{c - \eta_{p,e}} \quad (2)$$

Fig. 3. Relation between excess pore pressure and double amplitude axial strain from undrained cyclic triaxial tests

Fig. 4. Double amplitude axial strain plotted against peak effective stress ratio from undrained cyclic triaxial tests

Fig. 5. Double amplitude axial strain plotted against peak effective stress ratio from drained cyclic triaxial tests

where c is an experimental parameter. As failure is defined as $\epsilon_{DA} = 5\%$ in this study, the parameter, c , corresponds to the effective stress ratio when failure is reached and is taken to be 1.30 for the specimens used in the present study. This value was smaller than the critical state parameter 1.44 observed in undrained extension static tests on Ariake clay. For comparison the results from drained-cyclic triaxial tests are plotted in Fig. 5 against the cyclic stress ratio, $\eta_{p,e}$, in the same way as was used for undrained cyclic tests. The best fit curve drawn as a thick line is almost the same as that in Fig. 4 and given by Eq. (2). Therefore, it should be noted that there is a unique relationship for the double amplitude shear strain plotted against the effective stress ratio in spite of the difference in drainage conditions. During drained cyclic loading illustrated in Fig. 5, we should pay particular attention to the fact that the excess pore pressures must vary from time to time depending on the distance from the drainage boundary. To determine the effective stress ratio, $\eta_{p,e}$, in Fig. 5, therefore, the pore pressures measured at the center of the base of the specimen were adopted. This pore pressure is assumed to be the maximum one in

the specimen during drained cycling. Hence, $\eta_{p,e}$, is nearest to the failure envelope in each cycle. We considered that the development of double amplitude strains might start to occur from the most degraded part of the specimen. It will be shown later in the present paper that the variations of double amplitude of shear strains, ϵ_{DA} , increase and decrease in accordance with the variations of these excess pore pressures with the number of load cycles during drained cyclic loading.

In order to present the time-dependent behaviour of clay during undrained cyclic loading, let us introduce the following two stress parameters which are defined in Fig. 6 and Fig. 7. The first parameter, $RR(N)$, defined is an index related to the possibility of cyclic failure and is given by:

$$RR(N) = \frac{R_f(1) - R_f(N)}{R_f(1) - R} \quad (3)$$

where $R_f(1)$ and $R_f(N)$ respectively are cyclic strengths at the 1st and N th load cycles given by Eq. (1), and R is the ratio of the applied cyclic shear stress to the initial mean effective principal stresses. When failure is expected to be achieved by applying K cycles of normal-

ized cyclic shear stress, R , as shown in Fig. 6, then $R_f(N)$ varies from $R_f(1)$ to $R_f(K)$ from the first cycle to the cycle at failure while the values of $R_f(1)$ and R are fixed. Therefore, this parameter given by Eq. (3) varies from zero to unity. Another parameter, η^* , defined as:

$$\eta^* = \frac{\eta_{p,e} - \eta_s}{\eta_f - \eta_s} \quad (4)$$

where $\eta_{p,e}$ is the current effective stress ratio at the peak shear stress on the extension side of the current cycle and η_s is the effective stress ratio at the intersection of the total stress path ($dq/dp' = 3$) with the straight line $q = -q_{cyc}$ as explained in Fig. 7. This parameter, η^* , given by Eq. (4) indicates the relative stress ratio between the initial point and the final or failure point on the extension side in the $p' - q$ space. These parameters were originally introduced

Fig. 6. Definition of relative effective stress ratio

Fig. 7. Definition of relative cyclic strength ratio

for sands by Hyodo, et al. (1989, 1991). By correlating the values of both parameters calculated from a set of cyclic triaxial tests, we obtain Fig. 8. The plots shown in Fig. 8 enable us to assume the relation:

$$\eta^* = \frac{RR}{c_1 - (c_1 - 1)RR} \quad (5)$$

where c_1 is an experimental constant equal to 5.0 for reconstituted Ariake clay. Using the interrelationship between the above-mentioned parameters which is illustrated in Fig. 6 and Fig. 7, we carried out the predictions of the variation with number of load cycles of the excess pore pressure on the extension side and the associated double amplitude of axial strain during undrained cyclic loading. The procedure is as follows:

- 1) If a cyclic stress ratio, R , and a number of load cycles, N , are given, the cyclic strength $R_f(N)$ is estimated from Eq. (1). And then, the value of RR is calculated by using Eq. (3):
- 2) Since substitution of this value of RR into Eq. (5) gives the value of η^* , $\eta_{p,e}$ is determined from Eq. (4):
- 3) Thus the excess pore pressure, u_p , at the time when the peak cyclic shear stress is attained on the extension side is given by:

$$u_p = q_{cyc} \left(\frac{1}{\eta_s} - \frac{1}{\eta_{p,e}} \right) \quad (6)$$

Fig. 8. Interrelationship between relative effective stress ratio and relative cyclic strength

4) By substituting the value of $\eta_{p,e}$ determined from Eq. (4) into $\eta_{p,e}$ in Eq. (6), we obtain:

$$u_p = q_{cyc} \left[\frac{1}{\eta_s} - \frac{1}{\eta^*(\eta_f - \eta_s) + \eta_s} \right] \quad (7)$$

5) Substituting Eq. (4) into Eq. (2) also gives:

$$\varepsilon_{DA} = \frac{\eta^*(\eta_f - \eta_s) + \eta_s}{c_1 - \{\eta^*(\eta_f - \eta_s) + \eta_s\}} \quad (8)$$

The above procedures should be repeated from $N=1$ to a certain number of load cycles. The values of excess pore pressure and double amplitude shear strain calculated using Eq. (7) and Eq. (8) are plotted in Fig. 9 and Fig. 10 and compared with the results observed in undrained triaxial tests on reconstituted Ariake clay. The calculated values in both figures are in good correspondence with the observed values. Then, some modifications were introduced to find a correlation between pore pressures and strain amplitudes. By substituting Eq. (3) into Eq. (5), η^* can be represented by:

$$\eta^* = \frac{R_f(1) - R_f(N)}{c_1 \{R_f(1) - R\} - (c_1 - 1) \{R_f(1) - R_f(N)\}} \quad (9)$$

For the interrelationship between u_p/p'_c and ε_{DA} , combination of Eq. (7) with Eq. (8) gives:

$$\frac{u_p}{p'_c} = \frac{q_{cyc}}{p'_c} \left(\frac{1}{\eta_s} - \frac{1 + \varepsilon_{DA}}{c_1 \cdot \varepsilon_{DA}} \right) \quad (10)$$

Fig. 9. Measured and predicted variations of excess pore pressures with number of load cycles for undrained cyclic triaxial tests

Fig. 10. Measured and predicted variations of double amplitude axial strains with number of load cycles for undrained cyclic triaxial tests

This equation indicates that the u_p/p'_c versus ε_{DA} relation depends on the normalized cyclic shear stress and the double amplitude shear strain.

EVALUATION OF PORE PRESSURE AND STRAIN AMPLITUDE DURING PARTIALLY-DRAINED CYCLIC LOADING

As mentioned above, the excess pore pressure within a clay specimen is not uniformly distributed during the drained cyclic test since the generation and dissipation of excess pore pressures occurring simultaneously during cycling depend on the distance from the drainage boundary. A clay specimen in the drained-cyclic triaxial condition cannot, therefore, be regarded as an element. Rather the clay specimen under this situation must be a kind of model test whose boundary conditions are clear. Subsequently, it can be said that the drained triaxial test under this situation is a test which is best suited for investigating the applicability of the proposed model to the prediction of clay behaviour under long-term cyclic loading. The authors in the present paper perform the numerical simulation for clay behaviour using the *FEM* adopting the same procedure as outlined in a previous

The first step is to assume the process of generation and dissipation of excess pore pressures within a clay specimen subjected to partially-drained cyclic loading under triaxial stress conditions as shown in Fig. 11. The curves in Fig. 12 schematically represent the variations of excess pore pressures and volumetric strains with elapsed time, respectively (Hyodo et al., 1988). Behaviour of excess pore pressures during partially-drained cyclic loading must trace the path A'B from an arbitrary time, t , to an elapsed time $t + \Delta t$ (Δt : time increment) as described in Fig. 12. It is postulated in the analytical procedure that the incremental excess pore pressure, u_g , generated during a time increment, Δt , is added to the excess pore pressure, Δu , and at the same time the total excess pore pressure is partly dissipated. This means that the excess pore pressures trace a path A'C'B in Fig. 12. In addition an increment in volumetric strain

Fig. 11. Outline of analytical procedure in the present study for predicting the behaviour of clay cylinder under cyclic loading

Fig. 12. A key sketch for partially drained cyclic behaviour of soils

during C'B path is assumed to occur as the path moves from C' to B due to the dissipation of excess pore pressure, Δu_d , or the increase of effective stress. This is given by:

$$\varepsilon_{vr} = m_{vr} \Delta u_d \quad (11)$$

where m_{vr} is the coefficient of volume compressibility during post-cyclic recompression.

Generation and partial dissipation of excess pore pressures in clay are analyzed following the *FE* model using the Terzaghi type consolidation equation which was proposed for analysing the effect of drainage in liquefaction for sands by Booker et al. (1976). The basic equation for it is given by:

$$\{\nabla^T\}\{k_r\}\{\nabla u_d/\gamma_w\}=m_{vr}\left(\frac{\partial u_d}{\partial t}-\psi\right) \quad (12)$$

where u_d is the pore pressure, k_r is the permeability coefficient matrix, γ_w is unit weight of water, ∇ is the differential operation and ψ is the rate of generation of excess pore pressure, u_g , during undrained cyclic loading, converted to the time domain and is given by:

$$\psi = \left(\frac{\partial u_g}{\partial N} \right) \left(\frac{dN}{dt} \right). \quad (13)$$

Both coefficients of permeability and volume

compressibility were determined from the characteristics of post-cyclic recompression volume change. The volumetric strain due to dissipation of cyclically induced excess pore pressures is plotted in Fig. 13 against the maximum residual excess pore pressure generated in undrained cycling normalized by the confining pressure, u_r/p'_c . The thick line in Fig. 13 is calculated using:

$$\varepsilon_{vr} = 1.5 \frac{C_r}{1+e_0} \log \left(\frac{1}{1-u_r/p'_c} \right) \quad (14)$$

where C_r is the recompression index. Eq. (14) was formerly proposed by Yasuhara and Andersen (1990, 1991). The characteristic feature observed in Fig. 13 is that the post-cyclic recompression volumetric strain is primarily governed by the normalized excess pore pressure during undrained cyclic loading. The coefficient of volume compressibility was determined by:

$$m_{vr} = \frac{d\varepsilon_{vr}}{du_r} \quad (15)$$

Adopting the same procedure as used in the previous study (Yasuhara, Hirao and Hyodo,

Fig. 13. Recompression volumetric strain due to dissipation of excess pore pressure generated during undrained cyclic loading

1988), the coefficient of permeability needed for the analysis of partially-drained behaviour was determined using the theoretical solution of consolidation of a clay cylinder with radial drainage which was given by Yoshikuni and Nakanodo (1975). We selected $0.027 \text{ cm}^2/\text{kgf}$ ($0.00028 \text{ cm}^2/\text{kN}$) and $1.61 \times 10^{-8} \text{ cm/s}$ ($1.61 \times 10^{-10} \text{ m/s}$) as the average values of m_{vr} and k_r , respectively, for the present analysis of behaviour in drained cyclic triaxial tests. The object of the present analysis was a clay cylinder with radial drainage. The finite element mesh for this analysis is shown in Fig. 14. The first step in the procedure was to analyze the reconsolidation due to dissipation of excess pore pressure generated by undrained cyclic loading, using the model shown in Fig. 14. Fig. 15(a) and Fig. 15(b) compare measured and calculated variations of excess pore pressures and volumetric shear strains with the elapsed time in which the plots and the thick lines correspond to the measured and calculated values, respectively. Both pore pressures and volumetric strains were calculated by solving the equation:

$$\frac{\partial u}{\partial t} = \frac{k_r}{m_{vr}\gamma_w} \frac{\partial^2 u}{\partial r^2} \equiv c_{vr} \frac{\partial^2 u}{\partial r^2} \quad (16)$$

The reason we used this type of consolidation is that Eq. (16) is convenient not only in pursuing the analysis but also in evaluating the parameters included in the equation.

The comparisons between calculated and ob-

Fig. 14. Finite element model for analysing behaviour of a clay specimen under drained cyclic loading

Fig. 15. Measured and predicted variations of excess pore pressure and volumetric strain after undrained cyclic loading

served values as shown in Fig. 15(a) and Fig. 15(b) indicate that from the beginning to the final stages calculated values of both variation of pore pressure and volumetric strain with elapsed time approximate well to the measured values.

For the next stage of the analysis, the behaviour of clay specimens under partially-drained cyclic loading situations was predicted using the proposed model. Fig. 16 illustrates an example of the analytical results for time-dependent variation of excess pore pressure distribution within a clay specimen during partially-drained cyclic loading where drainage is allowed in the radial direction throughout the test.

It is indicated in Fig. 16 that except in the vicinity of the drainage boundary the excess pore pressure homogeneously increases with the number of load cycles until the isochrone of pore pressure is of a trapezoidal shape. Following this, the pore pressure distribution within a specimen is of a parabolic shape when

Fig. 16. Generation and dissipation processes during drained cyclic loading predicted using the proposed model

the pore pressure at the center of the specimen reaches a peak value. Thereafter, it gradually dissipates while keeping a parabolic shape, finally becoming zero completely throughout the whole specimen.

Fig. 17 and Fig. 18 compare the observed and calculated variations of the peak excess pore pressure on the extension side and double strain amplitude at each cycle with the number of load cycles. The thick lines and plots demonstrate the calculated and the observed values, respectively. Estimated excess pore pressures are the *FE* values at the nodal point

Fig. 17. Measured and predicted variations of excess pore pressure with number of load cycles for drained triaxial tests

Fig. 18. Measured and predicted variations of double amplitude axial strain with number of load cycles for drained cyclic loading

at the centre of the specimen. The double amplitude shear strains were calculated by applying the calculated excess pore pressure to the relation given by Eq. (2). The experimental results indicate that the excess pore pressure in two of the specimens rose until a certain value was reached and then dissipated while the other two specimens went to failure, despite being under partial-drained cyclic loading. At the same time, the strain amplitude increased or decreased keeping pace with the tendency of the excess pore pressure. When comparing these results in Fig. 17 and Fig. 18 with those in Fig. 9 and Fig. 10, allowing drainage during cyclic loading has a favourable effect in restraining the cyclic failure of normally consolidated clay.

CONCLUSIONS

The results from undrained cyclic triaxial tests on a highly plastic marine clay were formulated to predict the time-dependent variations of excess pore pressures and axial strains during partially-drained cyclic loading.

It was firstly found that on reaching the failure line on the extension side there is a unique relation between the effective stress ratio, $\eta_{p,e}$, and the double amplitude of cyclic shear strains, ϵ_{DA} during undrained loading.

We also found that the two parameters, the

relative stress ratio, η^* , and the relative cyclic strength, RR , used in the present paper are uniquely correlated with each other independent of the intensity of cyclic load.

These two findings enable us to evaluate the behaviour of clay under cyclic loading, that is, the variations of excess pore pressure and axial strain amplitude with the number of load cycles.

Through a combination of the formula for predicting undrained behaviour during cyclic loading with the post-cyclic reconsolidation characteristics, an attempt has been made to explain the partially-drained behaviour of the same marine clay as used in undrained tests. It is suggested from the results of a numerical simulation using a finite element analysis that the proposed model is a useful and practical tool to predict clay behaviour under partial drained cyclic loading.

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