PARTIALLY-DRAINED CYCLIC BEHAVIOR AND ITS APPLICATION TO THE SETTLEMENT OF A LOW EMBANKMENT ROAD ON SILTY-CLAY

AKIRA SAKAI(i), LAWALENNA SAMANC(ii) and NORIHKO MIURA(iii)

ABSTRACT

This study examines the phenomenon of settlement induced by traffic loading on low embankment roads built on soft cohesive soils deposited in a reclaimed lowland. A series of cyclic triaxial tests were conducted on undisturbed soft Ariake silty-clay under undrained and partially-drained conditions. A model based on the test results is proposed to simulate the partially drained behavior of silty-clay under a variety of lateral stresses. It is shown that the maximum excess pore pressure generation ratios and volumetric changes increase until the cyclic stress ratio reaches a value of 0.73, which is independent of the effects of lateral stresses under the same anisotropic consolidation ratio. Numerical analyses applying the proposed model were performed to compare the effects of soil thickness and cyclic loading time during one day and one cycle on partially drained cyclic behavior. It was found that a large ratio of cyclic to non-cyclic loading during one cycle exerts a smaller influence on volumetric strain than the total time of cyclic loading during one day. Observed settlement results of a low embankment road are analyzed to verify the applicability of the proposed model.

Key words: anisotropic consolidation, cyclic loading, cyclic triaxial test, low embankment, partially drained, settlement, silty-clay, undisturbed (IGC: D6/D7)

INTRODUCTION

Data have been presented that suggests that settlement of low embankment roads due to traffic loading is greater than that due to static loading (Yamanouchi et al., 1975; Kutara et al., 1980; Yasuhara, 1995). An equivalent traffic load-induced incremental load model has been proposed in the practical design of low embankment roads built on soft deposits in Japan (Kutara et al., 1980) to estimate settlement due to traffic loading. Several methods have been proposed to model soft subgrades in partially drained conditions (Hyodo et al., 1988, 1992) because the mechanisms of settlement induced by traffic loading are complex due to the inclusion of drainage. Characteristic cyclic loading due to traffic flow is associated with one-way loading without stress reversal, which lasts for a long period of time (Yasuhara et al., 1995) and generally takes place under partially drained conditions, in which generation and dissipation of excess pore pressure occur simultaneously. Terzaghi’s consolidation theory with the inclusion of undrained excess pore pressure generation has been applied as an analytical method for coupling these two effects for analyzing the settlement induced by liquefaction of sands (Seed et al., 1976). Hyodo et al. (1988, 1992) and Yasuhara et al. (1988) proposed a partially-drained cyclic model by formulating the same incremental excess pore pressure generation with a number of cycles in undrained cyclic triaxial tests to analyze the partially drained cyclic behavior of soft clay. The cyclic triaxial test is advantageous for examining partially-drained cyclic behavior because it permits the sampling of excess pore pressure measurements at the core of a specimen. Cyclic triaxial tests have been examined under isotropic stress conditions; however, the non-linear characteristics of soil compression caused by dissipation of excess pore pressure have not been considered.

A low embankment road was constructed on soft, compressible, and highly sensitive marine silty-clay in a lowland area of Saga, Japan and the settlement monitored during and after construction. Some observed data has already been reported by Miura et al. (1995), who concluded that an additional settlement of approximately 15 cm was caused by 2 years of traffic loading. In this study, a series of cyclic triaxial tests were performed on specimens from the near site to examine the behavior of undisturbed silty-clay under both undrained and partially-drained conditions with various anisotropic consolidations. Test results were modeled by taking non-linear soil compressibility during excess pore pressure dissipation into account to predict volumetric strain under partially drained cyclic conditions. Numerical analyses using the partially drained cyclic model were performed...
to clarify the effects of soil thickness and cyclic loading time during one day or one cycle. The long-term settlement of a low embankment under partially drained conditions was analyzed to verify the applicability of the proposed model.

TESTING PROCEDURE

Undisturbed specimens of silty-clay were sampled from a depth of 2.5 m to 3.2 m, from the construction site at Saga Airport, where the water table was at a depth of 1.0 m. The specimens were taken from a sample of size 15 cm to 20 cm in diameter and 20 cm in height. The soil properties were as follows: natural water content $\omega_n = 71\%$, liquid limit $\omega_L = 53\%$, plasticity index $I_p = 32\%$, initial void ratio $e_i = 1.81$, compression index $C_c = 0.890$, yield stress $\sigma_y^* = 32$ kPa. The grain size distribution showed a clay fraction of 34% ($d < 5 \mu m$) and a silt fraction of 48% ($5 \mu m \leq d \leq 74 \mu m$).

A specimen 5 cm in diameter and 10 cm in height was used in the triaxial test and subjected to a back pressure of 196 kPa for 24 hours under an isotropic effective confining pressure of 13 kPa (point O' in Fig. 1). B values in all tests were observed to be approximately 0.97. A vertical stress was applied at a rate of 2.45 kPa per hour to reach the K-line, point A in Fig. 1. K-consolidation along path AB was applied for a K value of 0.5 using the stress control method at the rate of vertical stress described above. Consolidation stress ratio $K$ is defined as $K = \sigma_0^* / \sigma_{00}$, where $\sigma_0^*$ and $\sigma_{00}$ are the effective vertical and lateral stresses after consolidation respectively. Specimens were subjected to a constant K value of 0.5 in the normally consolidated area with lateral consolidated stresses $\sigma_0^*$ of 49, 74, 98, and 147 kPa prior to cyclic loading. A sinusoidal cyclic load was applied to the specimen under one-way stress-controlled conditions at a frequency of 0.1 Hz. The measurement of pore water pressure for both undrained and partially drained cyclic tests was made at the center and top of the specimen through the pedestal with a 3 mm diameter porous stone. The drainage boundary of specimens during partially drained cyclic tests and post-reconsolidation was in the lateral direction. The undrained and partially drained cyclic test conditions are summarized in Table 1. A uniform distribution of excess pore pressure after undrained cyclic loading, path BC, was achieved by allowing the specimen to stand for two hours. Further, the post-cyclic reconsolidation test, path CB', was performed by allowing drainage.

The stress parameters $p' = (\sigma_1^* + 2\sigma_3^*)/3$ and $q = (\sigma_1^* - \sigma_3^*)$ were used to represent the test results, where $\sigma_1^*$ and $\sigma_3^*$ are the effective vertical and lateral stresses respectively. The testing program covered a wide range of different values of cyclic stress ratio $q_\sigma / p_\sigma^*$, where $q_\sigma$ and $p_\sigma^*$ are the cyclic deviator and mean effective principal stresses after consolidation, respectively.

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UNDRAINED AND PARTIALLY DRAINED CYCLIC BEHAVIOR

Comparison of Undrained and Partially Drained Cyclic Behavior

Cyclically induced axial strain, excess pore pressure, and volumetric strain responses with loading cycle N are shown in Fig. 2. The results show a typical pattern for the observed responses in all tests. As shown in Fig. 2(a), the axial strain εₐ curves, both in undrained and partially drained cyclic tests, increase monotonically with the number of loading cycles. The increment of εₐ under partially drained conditions was small due to the dissipation of excess pore pressure during cyclic loading.

The typical excess pore pressure generation and dissipation curves, as shown in Fig. 2(b), indicate that drainage effects caused a peak in the excess pore pressure curve under partially drained conditions. Both curves show that the excess pore pressure rate was rapidly generated in the stage with the early loading cycles. In the partially drained cyclic tests, the volumetric strain curve response is associated with the dissipation of excess pore pressure, as shown in Fig. 2(c).

The cyclic stress path and cyclic stress strain relationships of typical test results under undrained and partially drained cyclic conditions are presented in Figs. 3(a) and 4(a). A static failure envelope M was determined from undrained static compression tests. The characteristic undrained cyclic stress path (Fig. 3(a)) not only moves until the upper end touches the static failure line but finally crosses it at the final cyclic loading stage under the peak axial strain εₚ₀ = 10%. For partially drained conditions, (Fig. 4(a)), the effective stress path does not reach failure line M. This is due to the fact that the excess pore pressure peak for each cycle will not be as high as it is under undrained conditions, even though the applied cyclic stress ratio is relatively high. Figures 3(b) and 4(b) show the characteristics of cyclic stress strain relationships under undrained and partially drained conditions. The results are similar to that in the presence of sand (Hyodo et al., 1991) and silty-clay (Sakai et al., 1996) under two-way cyclic loading conditions, i.e., the cyclic axial strain increments decrease with the loading cycles. The residual
Fig. 5. Undrained cyclic shear strengths in an $R_t - \log (N_t)$ plot

Fig. 6. Variations of stress ratio $\eta_i$ at a peak axial strain $\varepsilon_{pa}$ of 10% in undrained cyclic stress path $q - p'$ space

axial strain under partially drained conditions was smaller than that under undrained conditions.

Undrained Cyclic Strength and Deformation

A peak axial strain of $\varepsilon_{pa} = 10\%$ was used as the failure condition to evaluate the undrained cyclic shear strength of the soil. Based on this criterion, the peak cyclic stress ratio $R_t = (q_{\sigma_0} + q_0)/\sigma_{00}$ versus the number of loading cycles at failure $N_t$ is shown in Fig. 5. The curve is expressed as a linear relationship in $R_t - \log N_t$ space for various lateral stress $\sigma_{00}$ values as follows:

$$R_t = a(N_t)^b$$  \hspace{1cm} (1)

where $R_t = (q_{\sigma_0} + q_0)/\sigma_{00}$ is the cyclic stress ratio at the peak deviator stress, $q_0 = (\sigma_{00} - \sigma_{0})$ represents the initial static deviator stress. Experimental parameters $a$ and $b$ were approximated as 1.60 and -0.045, respectively. Figure 6 presents the stress state at failure conditions for different levels of lateral stress. The cyclic stress ratio $\eta_i$ at failure can be represented by a single line passing through the origin with a slope of 1.75 in the $q - p'$ plot.

Figure 7(a) shows the correlation between the normalized peak excess pore pressure ratio $u_e/u_i$ and the normalized cyclic loading $N/N_t$ in undrained cyclic loading tests. The terms $u_e$ and $u_i$ are the excess pore pressures at peak deviator stresses corresponding to the loading cycles $N$ and $N_t$, respectively. The test curves show a unique nonlinear relationship, independent of cyclic stress ratio $q_{\sigma_0}/p_{00}$ and lateral stress $\sigma_{00}$. This relationship can be expressed as follows:

$$u_e/u_i = 4/\pi \arctan (N/N_t)^{1/\alpha}$$  \hspace{1cm} (2)

where $\alpha$ is an experimental parameter (Sakai et al., 1996). A unique relationship between the peak axial strain $\varepsilon_{pa}$ and the normalized number of cycles $N/N_t$ is also observed as shown in Fig. 7(b); this relationship is approximated by the following equation:

$$\varepsilon_{pa} / \varepsilon_{fs} = \ln (1 + \beta N/N_t) / \ln (1 + \beta)$$  \hspace{1cm} (3)

where $\beta$ is an experimental parameter and the peak axial strain at failure is defined as $\varepsilon_{fs} = 10\%$.

Partially Drained Cyclic Behavior

Figure 8 shows the variation in peak excess pore pressure ratio $(u_e/\sigma_{00})$ for each cycle and volumetric strain ($\varepsilon_v$) for various cyclic stress ratios $q_{\sigma_0}/p_{00}$ under lateral stress $\sigma_{00} = 49$ kPa conditions. The peak excess pore pressure reached a maximum value, which increased with $q_{\sigma_0}/p_{00}$; the maximum value was achieved after 10 to 20 cycles and then dissipated gradually to a small $u_e/\sigma_{00}$ value at $N > 200$ cycles. The volumetric strain, $\varepsilon_v$, curves that increased with loading cycle $N$ was greatly affected by the maximum values of peak excess pore pressure.

The characteristics of the test results for various cyclic stress ratios of $q_{\sigma_0}/p_{00}$ and lateral stresses $\sigma_{00}$ of 49, 74, 98, and 147 kPa are presented in Fig. 9. The maximum values of peak excess pore pressure ratio, $(u_e/\sigma_{00})_{max}$, in-
PARTIALLY-DRAINED CYCLIC BEHAVIOR

Fig. 8. Relationship between peak excess pore pressure ratio \( u_e / \sigma_{in} \) and volumetric strain \( \varepsilon_v \) in undrained and partially drained cyclic loading tests for \( \sigma_{in} = 49 \) kPa

Fig. 9. Effect of cyclic stress ratio \( q_{cy} / p_{mo} \) on partially drained cyclic behavior: (a) maximum peak of excess pore pressure ratio \( u_e / \sigma_{in} \) and (b) cyclic volumetric strain \( \varepsilon_v \) at \( N = 200 \) cycles

Fig. 10. A procedure for evaluating the behavior of soil under partially drained conditions of cyclic loading

MODELING OF PARTIALLY DRAINED CYCLIC BEHAVIOR

Diagram for the Modeling of Test Results

A model of the generation and dissipation of excess pore pressures under partially drained cyclic conditions to simulate the responses of the cyclic triaxial test results was proposed by Hyodo et al. (1988). The model assumes that the generation of internal excess pore pressure \( (\Delta u_e)_b \) at each number of cycles \( N \) is the same value as that \( (\Delta u_e)_b \) obtained during undrained cyclic loading. A schematic modeling of the test results is shown in Fig. 10. The excess pore pressure during partially drained cyclic loading tests undergoes a change \( \Delta u_e \), via path A'C, during time interval \( \Delta t \). The soil element will also be subjected to a cyclic shear stress, which induces an internal excess pore pressure \( \Delta u_e \) by path A'B'. Assuming that the process of excess pore pressure generation and its dissipation traces along path A'B'C, the excess pore pressure \( \Delta u_e = \Delta u_e + \Delta u_d \) is partly dissipated during loading period \( \Delta t \). The magnitude of internal pore pressure generation \( \Delta u_e \) during partially drained cyclic loading tests may be affected by cyclic stress history, volumetric strain, dissipation, and distribution of excess pore pressure. However, it is difficult to evaluate the effects on the change in internal excess pore pressure generation \( \Delta u_e \). In this study, the simple assumption of \( \Delta u_e = \Delta u_e \), suggested by Hyodo et al. (1988), is adopted for the comparison with test results.

When the magnitude of \( \Delta u_d \) partially dissipates within period \( \Delta t \), the volume change along path DE depends on the coefficients of volume compressibility during loading and unloading. It has been assumed that the coefficient of volume compressibility of clay under reloading is slightly larger than that under unloading and decreases with cy-

increased with an increase in cyclic stress ratio in the range of 0.23 – 0.73. However, there was no significant difference between the maximum values and the lateral stresses except \( \sigma_{lo} \) of 147 kPa. Conversely, the volumetric strain, \( (\varepsilon_v)_{200} \), at \( N = 200 \) cycles increased with both cyclic stress ratio \( q_{cy} / p_{mo} \) and lateral stress \( \sigma_{lo} \).
cyclic numbers in the modeling for one-dimensional cyclic consolidation, which is essentially different from that of static consolidation (Sakai, 1997). Adopting the same assumption to express the effects of cyclic loading on the volumetric strain of soil in partially drained cyclic loading tests, the incremental volumetric strain, $\Delta e_v$, for the dissipation of excess pore pressure generation $\Delta u_r$, during time interval $\Delta t$, is given by the following relationship:

$$\Delta e_v = m_c \Delta u_r = \chi N^m m'_c \Delta u_r$$

(4)

where the soil compressibility $m'_c$ is defined as the coefficient of volume compressibility during post-cyclic recompression; $N$ is the number of cycles; and $\chi$ and $\zeta$ are experimental parameters where $\chi > 1.0$ and $\zeta < 0$.

Consolidation with Excess Pore Pressure Generation

The effects of excess pore pressure generated by cyclic loading were incorporated into a one-dimensional consolidation analysis to assess the effect of drainage in the liquefaction of saturated sands (Seed et al., 1976). The equation of this consolidation diffusion problem (Booker et al., 1976) is given by

$$\frac{\partial u}{\partial t} - \varphi = \frac{k_r}{m_r \gamma_r} \frac{\partial^2 u}{\partial z^2}$$

(5)

where $u$ is the excess pore pressure, $\varphi$, is the coefficient of consolidation, $k_r$ is the permeability coefficient, $m_r$ is the coefficient of volume compressibility, and $\gamma_r$ is the unit weight of water. In the present study, these coefficients were determined from the results of the post-cyclic recompression.

The change in peak excess pore pressure generation $\Delta u_r$ corresponds to the generation of peak excess pore pressure $\Delta u_r$ that occurs during undrained cyclic loading. If we assume that $\Delta u_r = \Delta u_r$, the source term $\varphi$ is expressed in terms of a time domain as follows:

$$\varphi = \left( \frac{\partial u_r}{\partial N} \right) \left( \frac{dN}{dt} \right) = \left( \frac{\partial u_r}{\partial N} \right) \left( \frac{dN}{dt} \right)$$

(6)

where $dN/dt$ corresponds to the loading frequency. Furthermore, $\partial u_r/\partial N$ is derived from Eq. (1) and Eq. (2) as follows:

$$\frac{\partial u_r}{\partial N} = \frac{4u_r}{\alpha \pi N} \left\{ (N/N_0)^{1/\alpha - 1} \right\}$$

(7)

where $u_r$ is the excess pore pressure at the point of peak deviator stress corresponding to the loading cycles $N_t$ and expressed as

$$u_r = \frac{1 - 3(1 - K)}{1 + 2K} N_t + \frac{1}{3} \frac{q_{CS}}{p'_{so}}$$

(8)

in which $N_t$ is the cyclic stress ratio, $(q_{CS} + q_i)/p'_{so}$, at failure.

The magnitude of volumetric strain $e_v$ in partially drained cyclic tests, as shown in Fig. 9, is greatly influenced by the applied cyclic stress ratio $q_{CS}/p'_{so}$. The coefficient of volume compressibility $m'_c$ value is determined by Eq. (4) using $m'_c = du_{el}/du_r$, based on the test results of post-cyclic reconsolidation to calculate the volume change in partially drained cyclic tests. In post-cyclic reconsolidation tests, the volumetric strain $e_{vr}$ is correlated with the magnitude of excess pore pressure ratio $u_r/p'_{so}$, which is developed during the undrained cyclic tests, as shown in Fig. 11(a). The calculated lines, which were proposed by Yasuhara et al. (1991), are expressed as follows:

$$e_{vr} = \alpha_e c_{so} \frac{1}{1 + e_r} \frac{1}{1 - u_r/p'_{so}}$$

(9)

where $\alpha_e$ is an experiment parameter and $c_{so}$ is the swelling index ($\alpha_e = 2.5, c_{so} = 0.076$). Therefore, the coefficient of volume compressibility $m'_c$ may be given by using the peak excess pore pressure $u_r$ instead of $u_r$ in post-cyclic reconsolidation:

$$m'_c = \frac{0.434 \alpha_e c_{so}}{(1 + e_r)(1 - u_r/p'_{so}) p'_{so}}$$

(10)

The $m'_c$ values obtained by Eq. (10) in Fig. 11(b) increase with peak excess pore pressure ratio $u_r/p'_{so}$. When $u_r/p'_{so}$ is 0.5, the $m'_c$ value is twice that of when $u_r = 0$, because it is inversely proportional to $(1 - u_r/p'_{so})$.

Permeability in partially drained cyclic triaxial tests was evaluated using the method proposed by Yoshikuni et al. (1975) for the analysis of the consolidation of a clay cylinder with external drainage. The estimates of the coefficient of consolidation $c_{vr}$ for various effective verti-
cal pressures were obtained from the post-cyclic reconsolidation test curves, as shown in Fig. 12, which shows that the values of $c_v$ in the post-cyclic reconsolidation tests were smaller than those in $K$-consolidation tests ($K = 0.5$). In this paper, it is assumed that the $c_v$ values were constant during partially drained cyclic loading, independent of effective vertical pressures, whereas the coefficient of permeability $k_c$ changes with $m^*_h$ governed by excess pore pressure generation $u_c$, as shown in Fig. 11(b). Based on these results, parameters required for the analysis of partially drained cyclic tests are the values in Fig. 11(b) and $3 \times 10^{-3}$ cm$^3$/sec, corresponding to $m^*_h$ and $c_v$, respectively.

**Experimental Verification of the Proposed Model**

A numerical simulation was performed to examine the partially drained behavior obtained in the test results and hence verify the validity of the proposed method. The GADFLX2 computer program (Booker et al., 1976) was modified to solve Eq. (5) for the non-linear characteristics in the source term for Eqs. (4), (7), (8), and (10).

A triaxial specimen 10 cm in height and 5 cm in diameter was modeled as an axisymmetrical problem under conditions of consolidation diffusion. Five four-node quadrilateral elements (0.5 cm × 0.5 cm) were installed along the radial drainage direction from the center to the boundaries of the specimen (0.5 cm × 5 = 2.5 cm) as the finite element mesh of the triaxial specimen. Excess pore pressure $u_c$ was assumed to be zero at the specimen boundaries. Cyclic stress $q_{cy}$ was specified as a uniform pressure at each node. The calculated excess pore pressures were taken at the nodal points in the specimen for each stage of the loading cycle $N$. Additional volumetric strain $(\Delta \varepsilon_v)_\text{cal}$ at these points were calculated from Eqs. (5), (10) once the excess pore pressure increments $\Delta u_c$ and $\Delta u_0$ were known. Volumetric strain $(\varepsilon_v)_\text{cal}$ was calculated from accumulation $(\Delta \varepsilon_v)_\text{cal}$ at each time increment $\Delta t$. The experimental volumetric strain $(\varepsilon_v)_\text{exp}$ was compared with the volumetric strain $(\varepsilon_v)_\text{cal}$ corresponding to the analytical results of the simulation in partially drained cyclic tests; the relationship between peak axial strain $\varepsilon_{pa}$ and peak lateral strain $\varepsilon_{pl}$ is shown in Fig. 13, including the undrained cyclic deformation line $(\varepsilon_{pa} + 2\varepsilon_{pl} = 0)$. It is of note that these strains are not equivalent values, because the experimental volumetric strain is the value in cyclic triaxial tests under radial drainage conditions, whereas the value of volumetric strain obtained from the simulation is based on a one-dimensional consolidation analysis. However, it may be assumed that the experimental volume strain $(\varepsilon_v)_h$ in partially drained cyclic tests is approximately equal to the calculated volumetric strain $(\varepsilon_v)_\text{cal}$, so that $(\varepsilon_v)_h = (\varepsilon_v)_\text{cal} + 2(\varepsilon_{pl}) = 2(\varepsilon_{pl})_h = (\varepsilon_v)_\text{cal}$. The post-cyclic reconsolidation test results suggest that the axial strain $(\varepsilon_v)_h$ was much smaller than the volumetric strain $(\varepsilon_v)_h$, as shown in Fig. 14.

Figure 15 illustrates typical analytical results for the peak excess pore pressure ratio $\mu_c/\sigma_0$ at the center of the specimen and the volumetric strain $\varepsilon_v$ with a lateral stress of $\sigma_0 = 74$ kPa and a cyclic stress ratio of $q_{cy}/p_0 = 0.445$, all which were calculated using Eqs. (4) - (10) with a constant value of the coefficient of consolidation $c_v$ under conditions of partial drainage and cyclic loading. The volumetric strains were calculated for three cases in which the respective coefficients of volume compressibility were $m^*_v$, $\chi V m^*_v$, and $\chi V^2 m^*_v$. The dotted line used for the coefficient of volume compressibility, $m^*_v$, during post-cyclic reconsolidation is smaller than the observed volumet-
Fig. 15. Typical analytical results for peak excess pore pressure ratio \( \frac{u_e}{\sigma_{00}} \) and volumetric strain \( \varepsilon \), with various coefficients of compressibility

Fig. 16. Typical analytical results for the change in peak excess pore pressure ratio \( \frac{u_e}{\sigma_{00}} \) and volumetric strain \( \varepsilon \), at each distance ratio \( r/R \) from the center of the specimen

fig. 17. Typical analytical results for the distribution of volumetric strain \( \varepsilon \), within a model triaxial specimen

from the center of the specimen, corresponding to distance ratios \( r/R (R = 2.5 \text{ cm}) \) of 0.0, 0.2, 0.4, 0.6, 0.8 and 1.0. The peak excess pore pressure curves at each distance ratio developed rapidly until they reached peak values, which arise in earlier cycles and decrease with an increase in distance ratio. Conversely, the change in volumetric strains for each distance ratio is complicated by the number of cycles. The time dependent volumetric strain distribution is shown in Fig. 17. The volumetric strains \( \varepsilon \), induced near the specimen boundaries \( (r/R = 1) \) were larger than at other points during the first 10 cycles. After the first 10 cycles, the incremental rate of volumetric strain near the specimen boundaries decreased with an increase in the number of cycles; this latter effect is due to the smaller value of maximum peak excess pore pressure, which is in turn due to the dissipation of the excess pore pressure at adjacent specimen boundaries.

Figures 18(a) to 18(d) show the numerical simulation and test curves of peak excess pore pressure ratio \( \frac{u_e}{\sigma_{00}} \) and volumetric strain \( \varepsilon \), versus loading cycle \( N \) for \( \sigma_{00} = 49, 74, 98, \) and 147 kPa. The predicted values of \( u_e/\sigma_{00} \) and \( \varepsilon \) show good agreement with observed curves for various cyclic stress ratios \( q_{\alpha}/p_{\alpha} \).

The analytical results are plotted with the observed values for comparison in Fig. 9 to clarify the characteristics of the proposed model for various cyclic stress ratios \( q_{\alpha}/p_{\alpha} \) and lateral stresses \( \sigma_{00} \). The calculated maximum values of peak excess pore pressure ratio, \( (u_e/\sigma_{00})_{\text{max}} \), increase with increasing cyclic stress ratio (less than 0.75), but are independent of lateral stresses. Conversely, the calculated volumetric strain increases with both cyclic stress ratio and lateral stress. The observed values also increased, until the cyclic stress ratio was approximately 0.75. When the cyclic stress ratio exceeded 0.75, however, both the calculated peak excess pore pressure and its corresponding the calculated volumetric strain decreased due to the decrease of \( u_e \) in Eq. (8).
Effect of Soil Thickness and Cyclic Loading Time on the Partially Drained Cyclic Behavior over One Day or One Cycle

An analytical investigation was performed on specimens of cyclic triaxial tests with various radii and cyclic loading times to clarify the effects of soil thickness and cyclic loading time on partially drained cyclic behavior. The analytical conditions were as follows: the radii of specimens $R$ were 2.5, 25, and 100 cm, $c = 3 \times 10^{-3} \text{cm}^2/\text{sec}$, $\varepsilon = 1.56$, $\alpha = 2.5$, $\zeta = 0.076$, $x = 2.0$ and $\chi = -0.05$. The cyclic stress ratio $q_{\text{cy}}/p_0$ was 0.445 under an initial vertical stress $\sigma_0$ of 147 kPa and lateral stress $\sigma_0$ of 74 kPa. The loading time period (one day) was subdivided into $t_{\text{cy}}$ and $t_{\text{non-cyc}}$, where $t_{\text{cy}}$ is the continuous total time of each cycle including both cyclic and non-cyclic loading, consisting of two periods of cyclic loading, $T_{\text{cy}}$ and non-cyclic loading $T_{\text{non-cyc}}$. The parameter $t_{\text{cy}}$ is expressed by:

$$t_{\text{cy}} = N_{t} (T_{\text{cy}} + T_{\text{non-cyc}}) = N_{t} T_{\text{cy}} + t_{\text{cy}} + t_{\text{non-cyc}} = 24 \text{ hours},$$

(11)

where $N_{t}$ is the number of cycles per day and $T_{\text{cy}}$ is defined as a period of one cycle including both cyclic and non-cyclic loading time. In this analysis, the values of $t_{\text{cy}}$ were 1, 6, 12, and 24 hours and the values of $T_{\text{cy}}$ were 10, 30, and 60 sec under a $T_{\text{cy}}$ of 60 sec.

Figure 19 shows the analytical results of excess pore pressure ratio at the center of the specimen and volumetric strain under a continuous cyclic loading of $T_{\text{cy}} = 10$ sec with specimen radii of 2.5, 25 and 100 cm. The maximum values of excess pore pressure increased with speci-
men radius. This effect is due to the identical values of the coefficient of consolidation and the term $\varphi$ corresponding to the excess pore pressure generation $u_p$ given by Eqs. (5), (6), and (7). As with the maximum values of excess pore pressure, the volumetric strains also increased with radius, even though soil compressibility under cyclic loading decreased with number of cycles as follows: $m_\varphi = \chi N/m_s$ ($\zeta \leq 0$). It is suggested that the difference in volumetric strain with the increase in the radius of the specimen becomes small or has the possibility to become negative as the value of $\zeta$ decreases.

It is well known that traffic loading is not continuous or constant. Hence, it is important to clarify the effects of loading time $t_{cyc}$ on volumetric strain over one day and the period of cyclic loading $T_{cyc}$ over one cycle. First, the excess pore pressure and volumetric strain for four different loading times $t_{cyc}$ of 1, 6, 12, and 24 hours were calculated under conditions in which the value of $T_{cyc}$ is 60 sec, without the non-cyclic loading time. As shown in Fig. 20, longer loading times result in larger volumetric strains, although this difference is not much larger than that of volumetric strain produced over a $t_{cyc}$ of 12 hours. Additional numerical analyses were performed on three cases with various combinations of $T_{cyc}$ and $T_{non-cyc}$ (60–0 sec, 30–30 sec, and 10–50 sec) to clarify the effects of the period of non-cyclic loading $T_{non-cyc}$ on volumetric strain. However, no significant change in volumetric strain was recognized under the condition in which the total value of $T_{cyc}$ and $T_{non-cyc}$ was a constant value. Conversely, Fig. 21 shows that the value of $T_{cyc}$, with the period of non-cyclic loading $T_{non-cyc}$ of 0 sec, exerts an important effect on volumetric strain. As the number of cycles per day increased, the volumetric strain increased due to the increase in the excess pore pressure generation with a decrease in the value of $T_{cyc}$. Therefore, the period of cyclic loading $T_{cyc}$ related to the number of cycles per day is

a significant factor affecting the magnitude of volumetric strain, which is identical to the total loading time $t_{cyc}$ for one day.

**APPLICATION OF PROPOSED MODEL TO PREDICT SETTLEMENT INDUCED BY TRAFFIC LOADING**

The proposed method is highly applicable for evaluating the time-dependent response of soft clay layers under partially drained conditions of cyclic loading due to traffic loading. One of the observed values of total settlements of the Saga Airport access road was used in the
analysis. It should be noted, however, that consolidation settlement due to embankment loading occurs in addition to the settlement induced by traffic loading. In this paper, the settlement prediction was analyzed by the summation of settlements under both embankment and traffic loading. The extent of settlement was calculated using a one-dimensional consolidation analysis; the proposed model takes into account the effects of cyclic loading time.

The basic concept of the excess pore pressures under partially drained cyclic conditions, as shown in Fig. 10, is the same as that in the model suggested by Hyodo et al. (1988). This concept is based on the assumption that the generation of internal excess pore pressure is the same value as that obtained during undrained cyclic loading, and that the excess pore pressure is dissipated following the post-cyclic reconsolidation. A different point concerns the method of evaluating the effects of the non-linear characteristics of soil compression and cyclic numbers during partially drained cyclic loading on volumetric strain. Here, there is a problem concerning the determination of the excess pore pressure generation under undrained cyclic conditions which assigns a large effect to the excess pore pressure during partially drained cyclic loading, when the proposed model is applied to the prediction of the soft ground settlement due to traffic loading. It is desirable to use the excess pore pressure generation under the stress path in combination with the principal stress rotation caused by traffic loading. However, the undrained cyclic triaxial test results under a one-way stress condition at a frequency of 0.1 Hz, as described before, were used to determine the excess pore pressure generation induced by traffic loading in normal consolidated ground, instead of test results in conjunction with the stress path.

Field Measurement of Saga Airport Access Road Settlement

The Saga Airport access road was constructed on soft Ariake deposit and opened to traffic in 1992. The road is a typical low embankment road on a lowland plain in Japan. Field measurements were performed along location A within 250 m to investigate post-construction settlement. The geological profile of point No. 1 is presented in Fig. 22, which shows that the alluvial deposit consists of two layers, Ac1 and Ac2, with a marine silty-clay and a sand layer, which is sandwiched at a depth of 5.8 m to 9 m. A diluvial sand layer exists below a depth of 19.4 m. All of these sand layers may act as drainage layers. The overburden stresses on the ground are normally consolidation states compared with consolidation yield stresses along the depth direction. The water table was at a depth of approximately 1.0 m.

As shown in Fig. 23, the road pavement was designed for two-way traffic and is 11 m wide. A 1 m landscaping zone and a 3.5 m non-asphalt zone is situated on both sides. The subgrade was lime-stabilized clay with a thickness of 0.4 m, below which a 0.3 m thick material (masado) was placed. The subbase and base of the pavement consisted of crushed and graded grain materials with a total thickness of 0.3 m. The pavement surface was an asphalt concrete that was 5 cm thick.

The settlements measured along location A are shown in Fig. 24. It can be seen that the pavement surface shows a large differential settlement in-between the two box culverts supported by end bearing piles. A separation at the interface between the box culvert and the road pavement causes a dynamic impact on the pavement surface due to traffic loading. Cross-sectional profiles at point No. 1 show a settlement of approximately 30 cm on both sides of the center. It is recognized that the settlement profiles were not significant when the distances from the center of the embankment were larger than 25 m. Also, ground subsidence in the plain around location A occurred in small increments of 4 mm over the two year period 1992 to 1993. Thus, the effects of ground subsidence due to withdrawal of the groundwater table on the settlement of the Saga Airport access road are considered to be negligible.

Settlement due to Embankment and Traffic Loading

The observed settlement at the center of point No. 1 is presented in Fig. 25. However, it should be noted that measurements were performed 48 days after construction, during which time a calculated settlement of 10 cm took place. The amount of settlement suddenly increased once the road was opened to construction traffic. The settlement induced by traffic loading may have been approximately 30% of the total settlement of 45 cm during the two years after the road was opened to traffic.

Total settlement was obtained from the summation of the predicted settlements due to both embankment and traffic loading to predict the settlement of the low embankment road. As shown in Fig. 22, two silty-clay layers were subjected to settlement due to embankment loading. The settlement was calculated using a one-dimensional consolidation analysis, in which the parameters were determined from the data shown in Fig. 22; the coefficients of consolidation were \( c_v = 8.1 \times 10^{-4} \text{cm}^2/\text{s} \) in Ac1 layer and \( 6.4 \times 10^{-3} \text{cm}^2/\text{s} \) in Ac2 layer. The initial excess pore pressure was set to zero at the surface layer, where the groundwater level was at GL-1 m. The additional vertical stress due to embankment loading was 12 kPa, a value obtained from the pavement composition described above. The calculated settlement for each layer and in total due to embankment loading is presented in Fig. 25. The total consolidation settlement gradually increased to a value of 32 cm after 800 days.

The proposed model assumes that settlement under traffic loading occurs under conditions of partially-drained cyclic loading, which produces nearly one-dimensional deformation with a drainage direction identical to that of post-cyclic reconsolidation. However, it is difficult to assess incremental stresses in the ground due to traffic loading. For example, the principal stress rotation under a travelling load affects the cyclic deformation of soil (Ishihara, 1983), and the dynamic characteristics reported by Hyodo et al. (1988, 1989) indicate that incremental vertical stresses under a low embankment were approxi-
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Material in soil (No.1)</th>
<th>Atterberg limits, (%)</th>
<th>Yield &amp; overburden stresses, (kPa)</th>
<th>Compression index and void ratio</th>
<th>$m_c$ ($m^2/kN$)</th>
<th>$k$(cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Surface soil</td>
<td>$\omega_b \omega_a \omega_h$</td>
<td>$\omega_b \omega_a \omega_h$</td>
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<tr>
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<td></td>
<td></td>
<td>$10^{-2}$</td>
<td>$10^{-1}$</td>
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<tr>
<td></td>
<td>Silty clay (Ac2)</td>
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<td>$10^{-3}$</td>
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<td>15</td>
<td>Sand</td>
<td></td>
<td></td>
<td></td>
<td>$10^{-5}$</td>
<td>$10^{-4}$</td>
</tr>
</tbody>
</table>

Fig. 22. Geological properties of the Saga Airport access road at point No. 1

approximately 3 to 4 times larger than those due to the truck dead weight; furthermore, these stresses were deeply transmitted. The incremental vertical stress $(\Delta \sigma_z)$ (kPa) due to traffic loading in the present analysis was determined by using a simple assumption with a linear function of depth $z$ (m), from the pavement surface (Fujikawa et al., 1996; Miura et al., 2000):

$$(\Delta \sigma_z)_z = (\Delta \sigma_z)_z - \psi \cdot z$$

where $(\Delta \sigma_z)_z$ (kPa) is an incremental vertical stress at the pavement surface, and parameter $\psi$ (kPa/m) expresses the decreasing rate of the vertical stress. As shown in Fig. 26, the value of $(\Delta \sigma_z)_z$ and parameter $\psi$ in this analysis were fixed at 35 kPa and 5 kPa/m, respectively, because the effect region of the traffic loading under the 1 m thickness of the low embankment is to the depth of about 6~7 m (Fujikawa et al., 1996). The value of the stress ratio $\Delta \sigma_z/p_0$ was assumed to correspond to the cyclic shear stress ratio $q_{\alpha}/p_0$ induced by traffic loading. The incremental vertical stress $\Delta \sigma_z$ was zero at a depth of 7 m, which suggests that only layer Ac1 was subject to settlement due to traffic loading. The volume of heavy trucks (1992-1995) was approximately 800 vehicles per day. The values of $(\Delta \sigma_z)_z$ and number of cycles $N_i$ for one day were fixed at 35 kPa and 840, respectively, with a period of cyclic loading $T_{cyc}$ of 5 sec and a non-cyclic loading period $T_{non-cyc}$ of 25 sec. The total times with and without traffic loading for one day was $T_{cyc} = 7$ hours and $T_{non-cyc} = 17$ hours, respectively. The parameters used in the proposed model for partially-drained cyclic behavior include the values obtained from undrained and partially-drained triaxial tests shown in Table 1.

The GADFEA computer program (Booker et al., 1976) was modified to predict the long-term cyclic loading phenomena of soft cohesive soils. The calculated total settlement was obtained from a superposition of settlement induced by both embankment and traffic loading, as shown in Fig. 25. The calculated curves show a similar tendency to the measured values. The numerical results of excess pore pressure ratio $u_p/\sigma_{0z}$ and vertical strain $\varepsilon_{za}$ at various depths are shown in Fig. 27. The excess pore pressure curves almost reached a peak after 60 days and
PARTIALLY-DRAINED CYCLIC BEHAVIOR

20.00 m

4.50 m  11.00 m  4.50 m

3.50 m  2.25 m  2.25 m  3.50 m

3.25 m  3.25 m

0.55 m

2%  2%  2%  2%

Replaced materials, t=30cm
Asphalt concrete, t=5cm
Based graded grain materials, t=15cm
Subbase crushed materials, t=15cm
Subgrade improved by lime stabilized t=40cm

Fig. 23. Road sectional profiles

Profile of location A (m)

Settlement (cm)

0 10 20 30 40

Box culvert

150 days

Box culvert

380 days

615 days

No.1

Fig. 24. Settlement profiles measured at the center of the road along location A

Incremental vertical stress, $\Delta \sigma_v$ (kPa)

0 10 20 30 40 50

Depth, Z (m)

$(\Delta \sigma_v)_z = (\Delta \sigma_v)_{z=0} - \psi Z$

$\psi = 5 \text{kPa/m (present study)}$

Fig. 26. Depth distribution of incremental vertical stress due to traffic loading for analysis

the elapsed times at a peak pressure ratio increased with depth. The results show that the magnitude of both excess pore pressure ratio and vertical strain was larger at the middle of the layer Ac1 than at the top of that layer.

CONCLUSIONS

A modified partially drained cyclic model was proposed on the basis of the behavior of undisturbed silty-clay in undrained and partially drained cyclic triaxial tests under conditions of one-way loading. The model was applied to predict the settlement of low embankment
roads. The results are summarized as follows.

(1) The generation and dissipation of excess pore pressure and volumetric change during partially drained cyclic tests increased with the cyclic stress ratio $q_{cy}/p_{0}$ and effective lateral pressure $\sigma_{0}$. The maximum values of peak excess pore pressure ratio, $(h_{p}/\sigma_{0})_{max}$, increased with $q_{cy}/p_{0}$ to a value of 0.73, independently of $\sigma_{0}$. The coefficient of volume compressibility $m_{v}$ reflects nonlinear behavior under partially drained cyclic conditions. The magnitude of volumetric strain $\varepsilon_{v}$ in the partially drained cyclic tests was greatly influenced by $q_{cy}/p_{0}$ and $\sigma_{0}$.

(2) The model suggested by Hyodo et al. (1988) was modified to evaluate the partially drained cyclic behavior of silty-clay. The proposed model shows a reasonable agreement with the observed responses in the partially drained cyclic triaxial tests. A new aspect of the proposed model is that the non-linear characteristics of soil compressibility have been incorporated in the numerical analysis.

(3) From the numerical analysis using the proposed model, the maximum values of volumetric strain increased with soil drainage distance under partially drained cyclic loading conditions. Also, it was found that the loading time $t_{cy}$ and the period of cyclic loading $T_{cy}$ correlated with the number of cycles per day, and are important factors affecting the magnitude of the volumetric strain.

(4) The settlement of a low embankment road was observed. The analysis demonstrated that the settlement induced by traffic loading was approximately 30% of the total settlement during two years. The proposed model was adapted to predict such settlement; this model is applicable for analyzing this type of case, which is an example of a time-dependent response of cohesive soil layers to partially drained traffic loading conditions.

REFERENCES


