Settlements of Breakwater on Soft Seabed Ground under Ocean Waves

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SYNOPSIS A procedure for predicting the wave induced excess pore water pressure and residual strain of clay using the results of cyclic triaxial tests on the reconstituted Ariake clay is described. Ther reafter, the results of a numerical analysis by a 2-D dynamic effective stress FEM for a breakwater on a soft clay are presented.

INTRODUCTION

The dynamic loading such as earthquake loading, traffic loading and wave loading may cause a decrease in the bearing capacity and the residual displacement or settlement which may lead to the damage of structures founded on soft clay. On the other hand, the situation observed in the seabed deposite beneath nearshore and offshore structures undergoing wave-induced cyclic loading during strong storms in the ocean are different from the ground subjected to earthquakes in that the wave-induced loading continues for the longer periods. Wave-induced settlement and instability of nearshore and offshore structures has recently been recognized as an important problem for oil exploitation (Eide and Andersen, 1984) and contruction of breakwaters (Zen and Umehara, 1985).

In the case of short-term dynamic loading such as in earthquakes, clay may be supposed to be under undrained conditions while in the case of long-term dynamic loading, clay should be taken as in the partially drained situation, and both the generation and dissipation of excess pore water pressure should be considered during dynamic loading and the dissipation of pore water pressure after dynamic loading should also be taken into consideration.

A method for analyzing the behaviour of clay under long-term cyclic loading has been proposed by Hyodo and Yasuhara et al.(1988, 1991), in which the Terzaghi type consolidation equation proposed for analyzing the effect of drainage in liquefaction for sands by Booker et al.(1976) was used.

Strictly speaking, the Terzaghi type consolidation equation is only suitable for one dimensional analysis, because the total mean stress is assumed to be constant in the Terzaghi's thoey, while in the actual field under two or three dimensional condition, the total stress is not constant.

The present paper therefore describes a procedure for predicting the wave-induced excess pore pressure and residual settlements using the results of cyclic triaxial tests on undisturbed clay samples. Thereafter, the resuls of a numerical analysis for two type breakwates on the soft clay are presented.

A MODEL FOR PREDICTING THE CYCLIC BEHAVIOUR OF CLAY

Based on the results of cyclic triaxial tests on the reconstituted Ariake clay (G_s=2.58, W_L=115%, I_p=72) (Yasuhara and Hirao, 1988), the following formulas can be derived:

Cyclic Shear Strength

If the cyclic stress q_c is normalized by the mean stress P_o ($P_o=(\sigma_a+2\sigma_r)/3$), the relation between the cyclic strength and the number of load cycles can be approximated by a straight line in the logarithmic form for both isotropic and anisotropic consoildated conditions(Fig.1):

$$R_f = (q_{cyc}/p'_c)_f = a(N_f)^b \tag{1}$$

where N_f is the number of load cycles required to achieve a 5% double amplitude shear strain for isotropic condition and 5% maximum amplitude shear strain for anisotropic condition respectively, and a and b are experimental constants, equal to 0.553 and -0.058 respectively.

Excess Pore Pressure

The relation between the cyclic stress ratio η^* and relative number of load cycles R_N for both isotropic and anisotropic consolidated conditions as shown in Fig. 2 can be correlated by:

$$\eta^* = \frac{(R_N)^{c_2}}{c_1-(c1-1)(R_N)^{c_2}} \tag{2}$$

where $\eta^*=(\eta_{p,e}-\eta_s)/(\eta_f - \eta_s)$, $\eta_{p,e}[=q_{cyc}/(p'-q_{cyc}/3)$ for the isotropic condition, or $=(q_{cyc}+q_s)/(p'+q_{cyc}/3)$ for the anisotropic condition, respec-tively] is the current effective stress ratio and $\eta_s[=q_{cyc}/(p_o- q_{cyc}/3)$ for the isotropic condition, or $=(q_{cyc}+q_s)/(p_o+q_{cyc}/3)$ for the anisotropic condition, respec-tively] is the initial effec-tive stress ratio in p'-q space, η_f is the

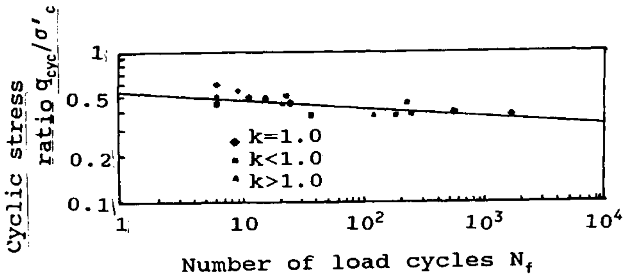

Fig.1 Cyclic stress ratio versus number of load cycles relation at 5% amplitude axiain strain

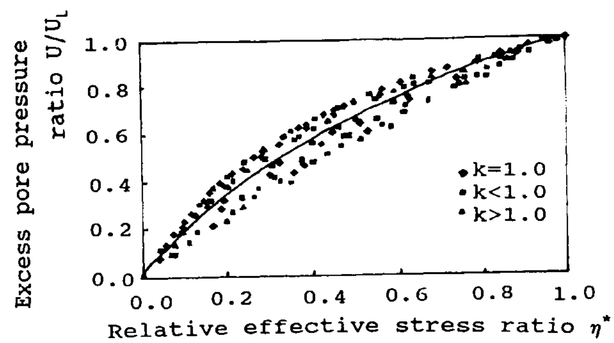

Fig.3 Relation between cyclic-induced pore water pressure ratio and relative effective stress ratio

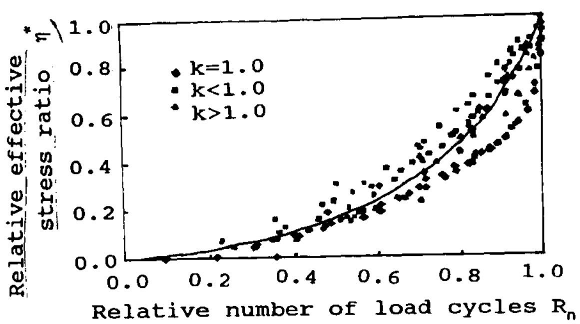

Fig.2 Correlation of relative stress ratio to relative number of load cycles

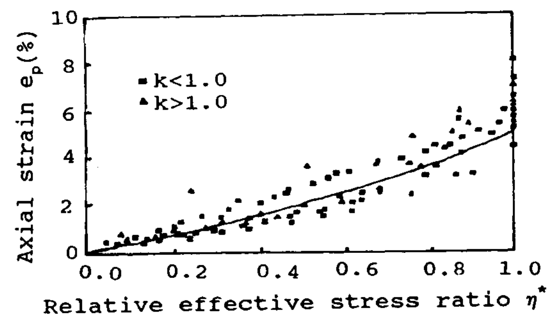

Fig.4 Relation between axial residual strain and relative effective stress ratio

effective stress ratio at failure point, $R_N = \log(N+1)/\log(N_f+1)$, N is the number of load cycles, p_0 is initial mean stress, and c_1 and c_2 are experimental parameters, equal to 2.7 and 1.5 respectively.

A unique relation between the cyclic-induced pore water pressure ratio U/U_f and the cyclic stress ratio η^* for both isotropic and anisotropic consolidated conditions as shown in fig. 3 can be obtained:

$$\frac{U}{U_f} = \frac{\eta^*}{C_3 - (C_3 - 1)\eta^*} \tag{3}$$

where U_f is the ultimate maximum residual pore water pressure while the 5% double amplitude shear strain for isotropic condition and 5% maximum amplitude shear strain for anisotropic condition is reached, c_3 is the experimental parameter, equal to 0.5.

Residual Axial Strain

The relation between the undrained residual axial strain and the cyclic stress ratio for anisotropic consolidated condition as shown in Fig.4 is formulated:

$$\epsilon_p = \frac{\eta^*}{d - (d - 20)\eta^*} \tag{4}$$

where d is the experimental parameter, equal to 0.5

ANALYTICAL PROCEDURES FOR EVALUATION OF SETTLEMENTS UNDER WAVE LOADING

The total settlement of the breakwater on soft seabed ground under ocean waves, S_T, consists of immediated undrained settlement, S_i, and the post-dynamic settlement, S_p, due to dissipation of excess pore water pressure caused by the wave loading. That is, we have

$$S_T = S_i + S_p \tag{5}$$

Methodloge for Evaluation of Settlements due to Dissipation of Wave-induced Pore Pressure

The post-cyclic settlement due to dissipation of wave-induced pore pressure will be calculated by the following governing equations(Zhou, 1991):

$$[L]^T[D][L]\{u\}-[L]^T\{m\}p$$

$$=-[L]^T\{m\}U-\rho\{g\}+\rho(\{\ddot{u}\}+\{\ddot{u}_g\}) \tag{6}$$

$$\{\nabla\}^T\{k\}\{\nabla\}p-\{m\}^T[L]\{u\}=\{\bar{f}\} \tag{7}$$

where [L] = Appropriate differential operator defining strains in terms of displacements
$[L]^T$ = Transposed matrix of [L]
[D] = Tangent modulus
{u} = Displacement vector
$\{m\}^T$ = [1,1,1,0,0,0]
p = Pore water pressure
U = Seismic pore water pressure
ρ = Density of soil
{g} = Gravity acceleration vector
$\{\ddot{u}\}$ = Input earthquake acceleration vector
$\{\ddot{u}_g\}$ = Relative acceleration vector
$\{V\}^T$ = Transposed matrix of Laplacian vector
[k] = $[k]/(\rho g)$
$[\underline{k}]$ = Permeability matrix
$\{\bar{f}\}$ = Seepage discharge vector

Equations (6) and (7) can be solved numerically under given boundary and initial conditions by the finite element method. The weighted residual method and 2-D isoparametic element with 4 nodes are used to formulate the following set of finite equations:

$$[K]\{u\} + [Q]\{p\} + [M]\{\ddot{u}\} = \{F\} \tag{8}$$

$$[Q]^T \{u\} + [H]\{p\} = \{\bar{F}\} \tag{9}$$

where [K] = Stiffness matrix
[Q] = Couple matrix
[M] = Mass matrix
[H] = Permeability matrix
$\{\underline{F}\}$ = Nodal earthquake load vector
$\{\bar{F}\}$ = Nodal seepage discharge vector
{u} = Nodal displacement vector
$\{\ddot{u}\}$ = Acceleration vector
{p} = Nodal pore water pressure vector

Methodloge for Evaluation of Undrained Settlements

In a cyclic triaxial test the radial principal stress is fixed while the axial principal stress is varied where the sample is subjected to the stress difference component. In-situ when the soil is subjected to wave-induced cyclic loading, the principal stresses may reverse and rotate and the soil is subjected to both the horizontal shear stress and the stress different component (plane strain condition). In order to reconcile this difference between laborartory and field conditions the stress difference, $(\sigma_v-\sigma_h)/2$, and the horizontal shear stress component, τ_{vh}, are used here for in-situ situation in the plane strain condition. The vertical strain can be expressed as:

$$\epsilon_v = \epsilon_{vd} + \epsilon_{vs} \tag{10}$$

where ϵ_{vd} is the vertical strain component caused by the stress difference, ϵ_{vs} is the vertical strain componet caused by the horizontal shear stress under initial stress difference condition.

The stress and strain in the triaxial condition can converted into those in the plane strain condition:

$$(\sigma_v - \sigma_h)/2 = \sigma_a/2 \tag{11}$$

$$\tau_{vh} = \sigma_a/2 \tag{12}$$

$$\epsilon_{vd} = \epsilon_{ac} > 0 \qquad (k_o<1,\ \sigma_v>\sigma_h) \tag{13}$$

$$\epsilon_{vd} = \epsilon_{re} = -\epsilon_{ae}/2>0 \quad (k_o<1,\ \sigma_v<\sigma_h) \tag{14}$$

$$\epsilon_{vd} = \epsilon_{ae} < 0 \qquad (k_o>1,\ \sigma_v>\sigma_h) \tag{15}$$

$$\epsilon_{vd} = \epsilon_{rc} = -\epsilon_{ac}/2<0 \quad (k_o>1,\ \sigma_v<\sigma_h) \tag{16}$$

$$\epsilon_{vs} = \epsilon_{ac} > 0 \qquad (ko<1) \tag{17}$$

$$\epsilon_{vs} = \epsilon_{ae} < 0 \qquad (ko>1) \tag{18}$$

Where ϵ_{ac} and ϵ_{rc} are the axial strain and radial strain in triaxial test with $k(=\sigma_{ro}/\sigma_{ao})<1$, ϵ_{ae} and ϵ_{re} are the axial and radial strains in triaxial test with $k(=\sigma_{ro}/\sigma_{ao})>1$, respectively, $ko=\sigma_h/\sigma_v$, is the stress ratio in ground.

Thus the total undrained settlement is evaluated by multiplying ϵ_v, by the depth of the clay layers.

Subsequently, the cyclic-induced settlement is predicted by superimposing the undrained settlement onto the post-cyclic recompression settlement.

RESULTS OF NUMERICAL CALCULATIONS FOR THE SETTLEMENTS OF BREAKWATERS ON ARIAKE CLAY

To predict the behaviour of clays under wave-induced cyclic loading using the proposed method, numerical calculations were performed for two type of breakwaters(inverse T type and inverse π type) founded on Ariake clay deposits 20m deep. The size of the sections of the breakwaters is 16m wide and 6.8m high.

An important point to note in the present calculation is the use of Goda's equation(Goda et al., 1973) for estimatng the peak wave pressure acting on the breakwater. The wave height, period and length were assumed to be 6m, 5.44s and 39m, respectively. the wave-acting duration was supposed to be 24 hours.

Fig.5 shows the results of calculated wave-induced excess pore water pressure in the ground 24 hours after the acting of wave loading for the inverse T type break-water.

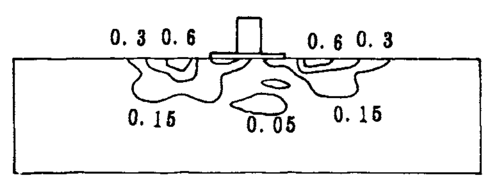

Fig.5 Calculated Wave-induced Excess Pore Pressures in the Ground (Inverse T Type)

Calculations for post-cyclic recompression settle-ment due to the dissipation of wavec-induced pore water pressure superimposed with the wave-induced undrained settlement are shown in Fig. 6 and Fig.7

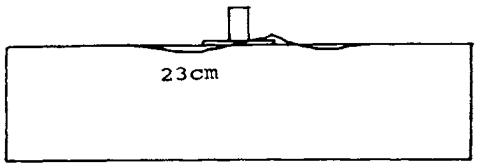

Fig.6 Distribution of Calculated Settlement of Clay under Wave Load (Inverse T Type)

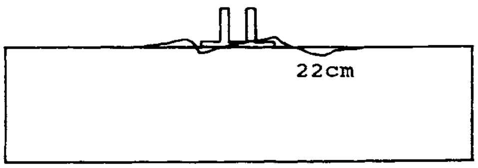

Fig.7 Distribution of Calculated Settlement of Clay under Wave Load (Inverse π Type)

CONCLUSION

The present paper describes the results of numerical computations for the settlement of two type of breakwaters founded on soft Ariake clay deposits, using emperical stress-strain-time relations formulated from the results of cyclic triaxial tests on undisturbed specimens.It is suggested that the location of maximum settlement for the different type of breakwater is different.

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