MODELLING OF PARTIALLY DRAINED CYCLIC BEHAVIOUR OF SOFT COHESIVE SOILS

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1. INTRODUCTION

Deformation of soft cohesive foundations under long term cyclic loading is recognized to be partially drained condition, in which the generation and dissipation of excess pore pressure occur simultaneously during the loading. Terzaghi's consolidation theory with the inclusion of undrained excess pore pressure generation is widely used for evaluating such phenomenon. Using this concept, Hyodo et al. (1988) developed a procedure for analysing the partially drained cyclic behaviour of soft clays.

This study investigates the behaviour of undisturbed soft cohesive soils, silty clay, in undrained and partially drained cyclic triaxial tests with consolidation stress ratio K=0.5 for various cyclic deviator stress ratios q_{cy}/p_{co}' and effective lateral stresses σ_{30}'. The results are used to model excess pore pressure generating effect due to cyclic loading, considering a non linearity characteristic of soil compressibility.

2. EVALUATION OF PARTIALLY DRAINED CYCLIC BEHAVIOUR

To model the response of test results, a procedure suggested by Hyodo et al. (1988) is extended. A proposed diagram is illustrated in Fig. 1. Suppose that during an interval of time Δt, the excess pore pressure during partially drained cyclic loading test undergoes a change Δu (path A'C in Fig. 1), the element will also be subjected to cyclic shear stress which induces an additional excess pore pressure generation $\Delta u_g'$ (path A'B''). Assuming that the process surpassing of generation and dissipation traces the path A'B''C, the magnitude of excess pore pressure $\Delta u_d'(=\Delta u_g'+\Delta u)$ would be partly dissipated during the loading period Δt.

To simulate such behaviour, this study suggests that the change of internal pore pressure generation $\Delta u_g'$ corresponding to the excess pore pressure generated during undrained cyclic loading Δu_g (path AB or A'B''), may be expressed as

$$\Delta u_g' = \frac{1}{1+\chi*\varepsilon_v}*\Delta u_g \qquad (1)$$

where parameter χ is an experimental parameter related with the internal excess pore pressure generation. ε_v is the accumulated volumetric strain at arbitrary time t.

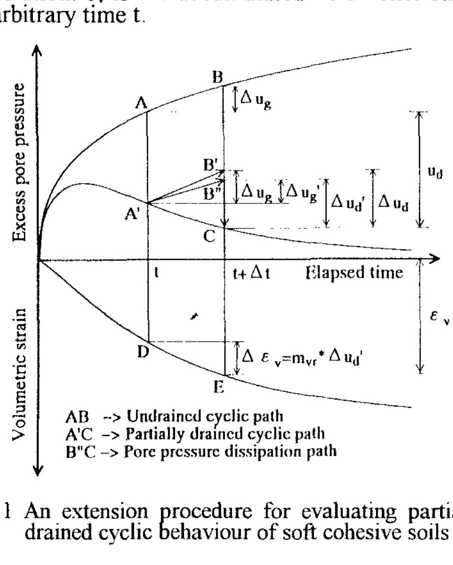

Fig.1 An extension procedure for evaluating partially drained cyclic behaviour of soft cohesive soils

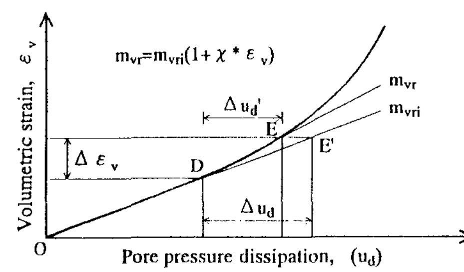

Fig.2 Diagram for evaluating soil compressibility

Experimental results, schematically shown in Fig. 2, indicate that the compressibility of soft cohesive soils under partially drained cyclic and post cyclic reconsolidation conditions would be a non linear response following the path ODE. When the magnitude of $\Delta u_d'$ dissipates partially within a period Δt, the volume change along the path DE is approximated by a relation $\Delta\varepsilon_v=m_{vr}*\Delta u_d'$. The soil compressibility m_{vr} is defined as a secant of the curve (path DE). Further, a simple relation is introduced to depict a non linearity characteristic of m_{vr} as follows;

$$m_{vr} = f(\varepsilon_v)*m_{vri} \qquad (2)$$

where m_{vri} is the soil compressibility after consolidation prior to cyclic loading, it is defined as a tangent in ε_v -u_d relationship along the path ODE' (Fig. 2). The function $f(\varepsilon_v)$ reflects a tendency of the volumetric strain increments increases with the increasing of the pore pressure dissipation, approximating by $f(\varepsilon_v)=1+\chi*\varepsilon_v$). From Eq. (2), the change of volumetric strain $\Delta\varepsilon_v$ along the path DE (Figs. 1 and 2) at time increment Δt is expressed as

$$\Delta\varepsilon_v = m_{vri}(1+\chi*\varepsilon_v)*\Delta u_d \qquad (3)$$

3. MODELLING OF EXCESS PORE PRESSURE GENERATION AND DISSIPATION

The governing equation of the consolidation diffusion problem (Booker et al., 1976) is given by:

$$\{\nabla\}^T[k_r]\left\{\nabla\frac{u}{\gamma_w}\right\} = m_{vr}\left(\frac{\partial u}{\partial t}-\varphi\right) \qquad (4)$$

where u is the excess pore pressure, $[k_r]$ is the matrix of permeability coefficients, γ_w is the unit weight of water, ∇ is the differential operation. A

$$\varphi = \frac{1}{(1+\chi^{*}\varepsilon_v)} * \left(\frac{\partial u_g}{\partial N}\right)\left(\frac{dN}{dt}\right) \tag{5}$$

where $\partial u_g/\partial N$ is the partial rate of excess pore pressure generation. Under regular loading, dN/dt is obtained corresponding to a loading frequency.

When the analysis concerns with peak values of u_g during the tests, the component $\partial u_g/\partial N$ is expressed by $\partial u_p/\partial N$ deriving from undrained cyclic test results (Samang, 1997) as

$$\frac{\partial u_p}{\partial N} = \frac{4u_f}{\alpha\pi N_f}\left\{\frac{1}{\tan^{\alpha-1}(\pi r_u/4) * \sec^2(\pi r_u/4)}\right\} \tag{6}$$

where $r_u = u_p/u_f$ is the normalized excess pore pressure in undrained cyclic tests.

In this analysis, a FEM computer code GADFLEA (Booker et al, 1976) was modified to incorporate the above requirement.

4. VERIFICATION OF PROPOSED METHOD

The geometry of a triaxial specimen and finite element mesh are shown in the Figs. 3(a) and 3(b). Because of symmetry, only one half of the specimen is modelled by using 5 four-noded quadrilateral elements with radial drainage. Excess pore pressure u is assumed to be zero at the specimen boundary. The cyclic stress q_{cy} is specified as an uniform pressure at each nodes.

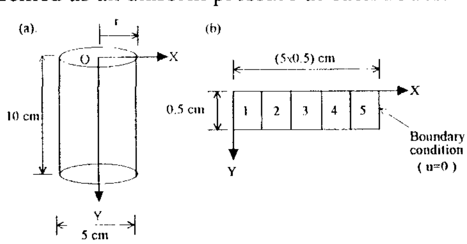

Fig.3 Finite element model of a triaxial specimen

The parameters necessary to perform the analysis such as m_{vri} and k_r are summarised in Table 1. The value of $m_{vri}/m_{vro}=0.345$ and $\chi=25$ are used to simulate the response on test results.

Table 1. Soil properties used in the analysis

σ_{10}' (kPa)	e_c	m_{vro} (m^2/kN)	m_{vri}/m_{vro}	m_{vri} (m^2/kN)	k_r (m/sec)
98	1.675	1.28×10^{-3}	0.345	0.45×10^{-3}	2.39×10^{-9}
147	1.532	1.06×10^{-3}	0.345	0.37×10^{-3}	9.33×10^{-10}
196	1.395	0.75×10^{-3}	0.345	0.26×10^{-3}	4.83×10^{-10}
294	1.352	0.55×10^{-3}	0.345	0.19×10^{-3}	2.67×10^{-10}

The predicted curves of u_p/σ_{30}' for $\sigma_{30}'=49$ kPa as shown in Fig. 4(a) indicate that the calculated pore pressure dissipate faster than the observed curves. While the calculated curves for ε_v almost depict the test curves, except for test with $q_{cy}/p_{co}'=0.729$. A comparison between the calculated curves and observed curves for $q_{cy}/p_{co}'=0.623$ and various lateral stresses is shown in Fig. 4(b). The excess pore pressure and their corresponding volumetric strain are affected by the applied lateral stress. As the lateral stresses increase, the peaks of excess pore pressure as well as volumetric strain also increase. Both figures reveal a good agreement between the calculated and observed curves for various q_{cy}/p_{co}'.

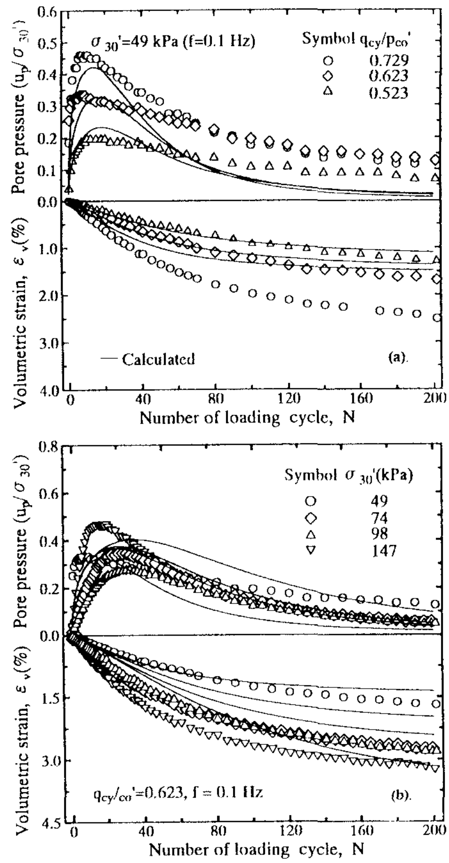

Fig.4 Predicted and observed curves of excess pore pressure and volumetric strain with time

5. CONCLUSIONS

Test results characterize that the coefficient of volume compressibility m_{vr} reflects non linear behaviour under partially drained cyclic conditions, about which present model is concerned.

Numerical results show a good agreement with the observed responses in partially drained cyclic triaxial tests.

The proposed method provides a useful means for evaluating the time dependent response of soft subgrade layers as partially drained cyclic condition such as due to traffic loading.

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