NUMERICAL ANALYSIS OF EXCESS PORE WATER PRESSURE AND PLASTIC DEFORMATION FOR SATURATED CLAYS UNDER CYCLIC LOADING CONDITION

ANALYSE NUMÉRIQUE DE SURPRESSION INTERSTITIELLE ET DÉFORMATION PLASTIQUE D' ARGILE SATURÉ DANS LES CONDITIONS DE CHARGE PÉRIODIQUE

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ABSTRACT. This paper presents a numerical procedure of modeling the development of excess pore water pressure and corresponding plastic deformation for normally consolidated or lightly overconsolidated saturated clays under cyclic loading condition. Two calculation examples, the first for undrained, the second for partly drained triaxial condition, are given. The results show the applicability of the numerical model for clays under cyclic triaxial condition.

RESUME.

1. Introduction

It is well-known that under dynamic-cyclic loading excess pore water pressure can be induced in normally consolidated or lightly overconsolidated clays. The resulted excess pore water pressure may be divided into cyclic and permanent parts. The permanent part of excess pore water pressure arises mainly from undrained plastic shear deformation of clays. Its prediction is essential for the stability and deformation analysis of foundations. The subsequent dissipation of the permanent excess pore water pressure results in plastic volume strain and so further plastic deformation.

For modeling this behavior, a conceptual model has been proposed upon quasi-static approach (Hu 2000). Based on some relationships deduced from literature, a semi-theoretical model has been developed for a quantitative description of stress − excess pore water pressure - permanent strain - cycle number - relationship of normally consolidated or lightly overconsolidated clays under cyclic loading condition.

This model has been implemented by using the finite element method and the finite difference method. To show its applicability, two numerical calculation examples are presented.

2. Quasi-static model

The research by Hu (2000) showed that permanent excess pore pressure is a deciding parameter for plastic deformation of normally consolidated or lightly overconsolidated clays under cyclic loading condition. Plastic deformation including undrained plastic shear deformation in the first stage and drained plastic volume strain in the second stage, are exclusively dependent on this parameter. Based on this, a quasi-static model has been proposed, see Figure 1.
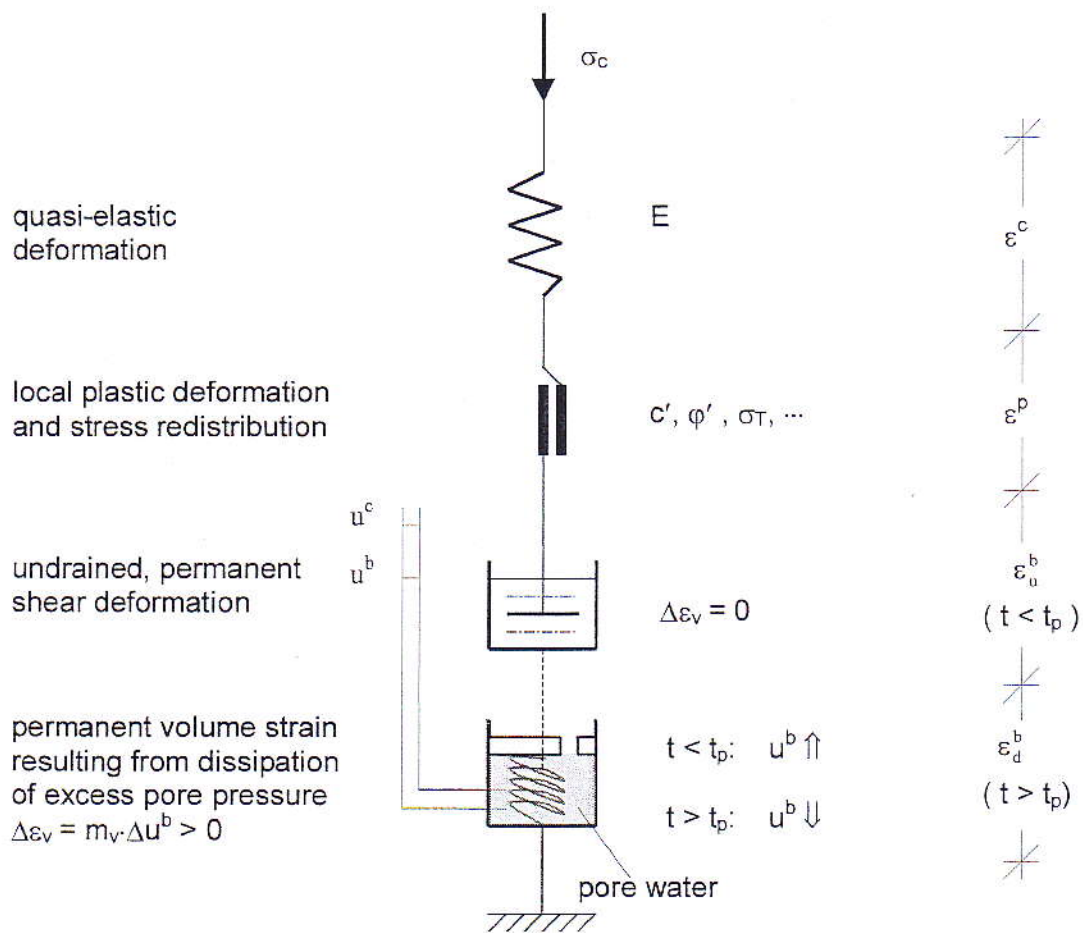
σ_c

quasi-elastic
deformation

E

ε^c

local plastic deformation
and stress redistribution

c', φ', σ_T, $\cdots$

ε^p

u^c

undrained, permanent
shear deformation

u^b

$\Delta\varepsilon_v = 0$

ε_u^b

$(t < t_p)$

permanent volume strain
resulting from dissipation
of excess pore pressure
$\Delta\varepsilon_v = m_v \cdot \Delta u^b > 0$

$t < t_p$: $u^b \Uparrow$

$t > t_p$: $u^b \Downarrow$

ε_d^b

$(t > t_p)$

pore water

Figure 1. Quasi-static model describing deformation behavior of normally consolidated
or lightly overconsolidated clays under cyclic loading

The elastic module E (Figure 1) is used for simulating maximum reversible strain under cyclic loading. In principle, this part of strain also changes in the course of cyclic loading. However, it is normally much smaller than permanent strain. As a first approximation, E-module is therefore assumed to be independent of cycle number. The sliding element represents shear and tension limits of clays under cyclic loading condition. Yasuhara et al. (1982) proven that for normally consolidated clays effective shear strength parameters under undrained condition is not significantly affected by cyclic frequency and loading duration. It was therefore assumed that static strength parameters could be applied for the case of cyclic loading condition.

In the beginning of cyclic loading, saturated clays remains nearly undrained and excess pore water pressure rises depending on cycle number. The reversible part results from elastic deformation of soils and can be approximately assessed using $u^c = A_c \cdot (\sigma_{x,c} + \sigma_{y,c} + \sigma_{z,c})$, where $\sigma_{x,c}$, $\sigma_{y,c}$ and $\sigma_{z,c}$ are stress components of cyclic loading. The parameter A_c can be determined from cyclic undrained test. In comparison with u^c, permanent excess pore pressure u^b is caused by plastic shear deformation of soil skeleton and shows time-dependent. Lots of results of cyclic triaxial tests on clayey soils under undrained condition showed that excess pore pressure u_g^b under cyclic undrained condition can be approximatively evaluated using the following empiric relationship:

$$\frac{u_g^b}{\sigma_{3,0}} = \rho \cdot \left(\frac{q_c}{\sigma_{3,0}}\right)^{\alpha_G} \cdot (1 + \log t)^{\theta} \tag{1}$$

where ρ, α_G and θ are regression parameters.

The permanent excess pore pressure generated under undrained condition is treated as a pressure spring $\psi = \partial u_g^b / \partial t$ and added to consolidation equation. Under consideration of simultaneous dissipation, resulting equation can be expressed as follows:

$$\frac{\partial}{\partial x}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial x}\right) + \frac{\partial}{\partial y}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial y}\right) + \frac{\partial}{\partial z}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial z}\right) = m_v\left(\frac{\partial u^b}{\partial t} - \psi\right) \tag{2}$$

The solution of the consolidation equation provides increasing excess pore pressure in the beginning of cyclic loading ($\Delta u^b > 0$), which is used for evaluating undrained plastic shear deformation through an empiric relationship. The research by Lo (1969 a, b) as well as by Yasuhara et al. (1982) showed that independent of loading form and frequency an hyperbolic relationship exists between the ratio of permanent excess pore pressure to consolidation pressure and the first principal strain:

$$\frac{u^b}{\sigma_{3,0}} = \frac{\varepsilon_{1,u}^b}{a + b \cdot \varepsilon_{1,u}^b} \tag{3}$$

The subsequent dissipation ($\Delta u^b < 0$) leads to plastic volume strain and can be determined by using:

$$\Delta \varepsilon_d^b = m_v \cdot \Delta u^b \tag{4}$$

The parameter m_v is the volume compressive coefficient under cyclic loading. The tests on Drammen clay (OCR=1) by Yasuhara & Andersen (1991) showed that the parameter m_v can be approximately assessed using the following relationship:

$$m_v = 1,5 \cdot \frac{C_r}{1 + e_0} \cdot \frac{1}{\ln 10} \cdot \frac{1}{\sigma_{3,0} - u^b} \tag{5}$$

C_r is the compressive parameter of conventional static oedometer test in reloading stage.

3. Numerical study of undrained cyclic triaxial tests

In Brown et al (1975), some undrained cyclic triaxial tests ($f = 10$ Hz) on samples of Keuper marl are presented. The soil was reconstituted from a slurry and prepared to overconsolidation rations between 2 and 20. The soil samples were prepared to the magnitude of $D = 9.95$ cm and $H = 10$ cm. The basic soil mechanical parameters are as follows:

$$w_L = 32\ \%; \quad I_P = 18\ \%; \quad \rho_s = 2.74; \quad \text{clay content } 18\%.$$

By using the numerical model described above, the undrained cyclic tests on samples of lightly overconsolidated samples (OCR = 2) were simulated. For this purpose, the necessary

parameters in the model were assessed by using the results of static triaxial tests at OCR = 2 in Brown et al (1975) with some simplifications:

$$E = 170 \text{ MN/m}^2 \ (\sigma_{3,0} = 380 \text{ kN/m}^2, \ q_r = q_c = 195 \text{ kN/m}^2);$$
$$E = 130 \text{ MN/m}^2 \ (\sigma_{3,0} = 380 \text{ kN/m}^2, \ q_r = q_c = 280 \text{ kN/m}^2);$$
$$\rho = 0.06; \ \alpha_G = 1.88; \ \theta = 1.3; \ c' = 5 \text{ to } 10 \text{ kN/m}^2; \ \varphi' = 25 \text{ to } 30°;$$
$$a = 9.78; \ b = 2.55; \ A_c = 0.05 \text{ to } 0.10.$$

In Figure 2, the numerical calculation and the test results of Brown et al (1975) are put together for the case under q_c = 195 kN/m² as well as 280 kN/m². The calculated permanent axial strain and the test results are generally in good agreement, if we consider that some simplifications were made in determining the input parameters.
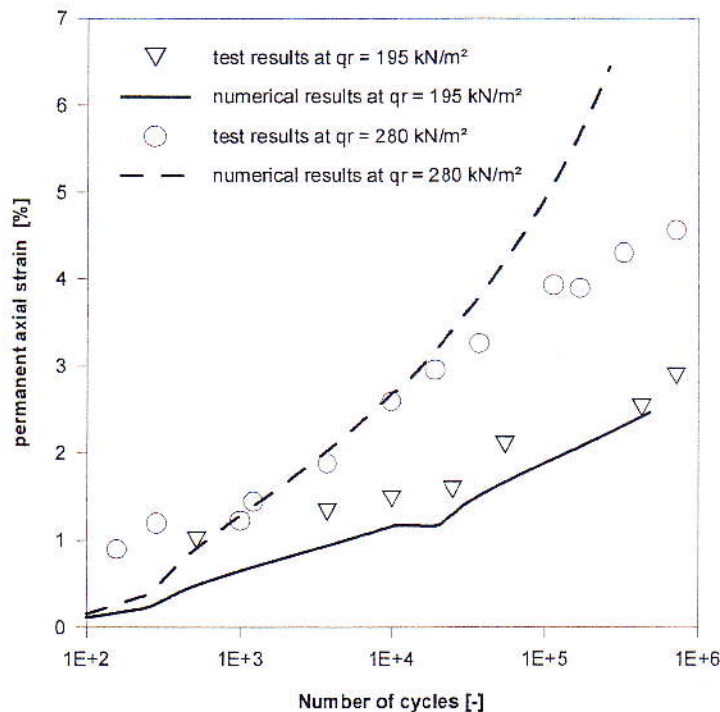

Figure 2. Comparison between numerical calculations and the test results
of Brown et al (1975), OCR = 2

4. Numerical study of cyclic triaxial tests under partly drained condition

To study the behavior of a medium-plastic clay (w_L = 41.9%, I_P = 23.5%, G_s = 2.75, I_c = 0.82) under partly drained condition, the outer surface of soil samples (D = H = 10 cm) is assumed to be drained during cyclic loading. Soil samples are statically consolidated under $\sigma_{3,0}$ = 50 kN/m² and then cyclically loaded by $P = \pi \cdot R^2 \cdot \sigma_c / 2 \cdot [1 - \cos(2\pi f t)]$ (f = 1 Hz). The applied parameters were determined from conventional static triaxial and oedometer tests:

$$\rho = 0.1; \ \alpha_G = 1.0; \ \theta = 1.5; \ c' = 6 \text{ kN/m}^2; \ \varphi' = 30°; \ k_r = k_z = 1.0 \times 10^{-10} \text{ m/s};$$
$$a = 1.2; \ b = 1.8; \ A_{cr} = 0.15; \ m_v = 0.34 \times 10^{-3} \text{ m}^2/\text{kN}.$$

Totally, 9 cases were numerically studied under different cyclic stresses. In Figure 3, the development and distribution of excess pore water pressure are illustrated for the case $\sigma_{3,0} = 50$ kN/m² and $\sigma_c = 40$ kN/m². The permanent axial and radial strains as well as the principal stress and plastic zones for $\sigma_c \geq 30$ kN/m² at $N = 10^5$ are shown in Figure 4 and 5.

5. Conclusions

The numerical study for cyclic triaxial tests of normally consolidated or lightly overconsolidated saturated clays under undrained as well as partly drained condition showed, that the development of excess pore water pressure and plastic strain can be well predicted by using the developed quasi-static model. It is expected that the model be applied and confirmed by in situ test conditions in the future.

6. References

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Yasuhara K., Yamanouchi T., Hirao K. (1982), Cyclic strength and deformation of normally consolidated clay. *Soils and Foundations 22(3)*, 77 - 91.

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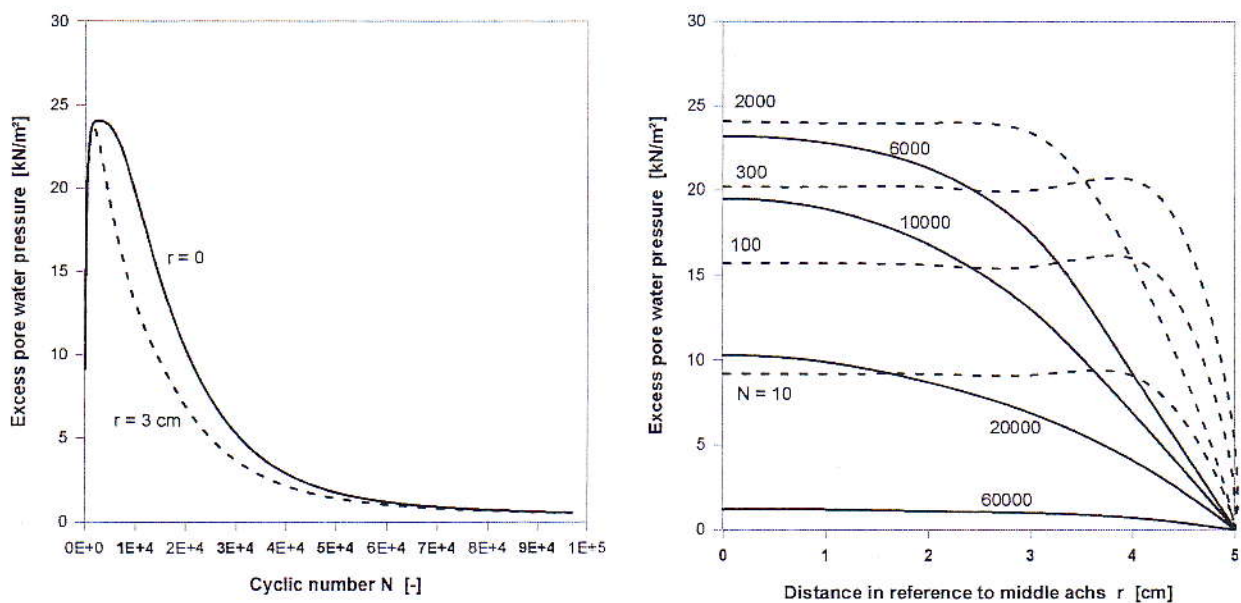
Figure 3. Excess pore water pressure dependent on cyclic number and distribution, in the case $\sigma_{3,0} = 50$ kN/m² and $\sigma_c = 40$ kN/m²
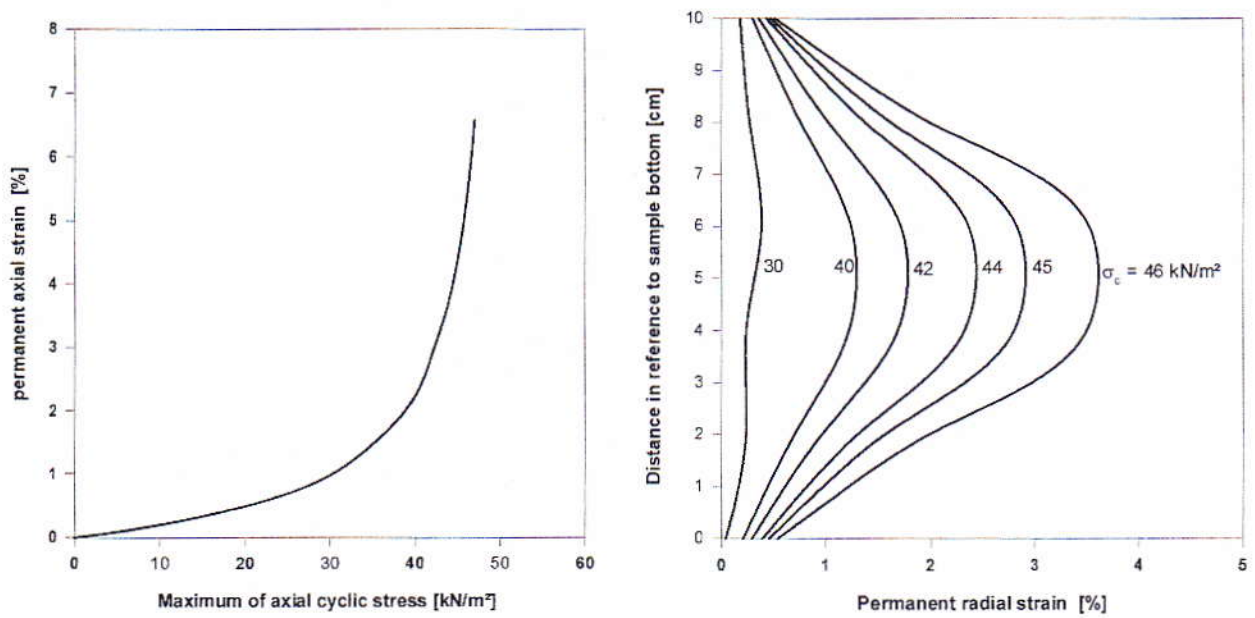
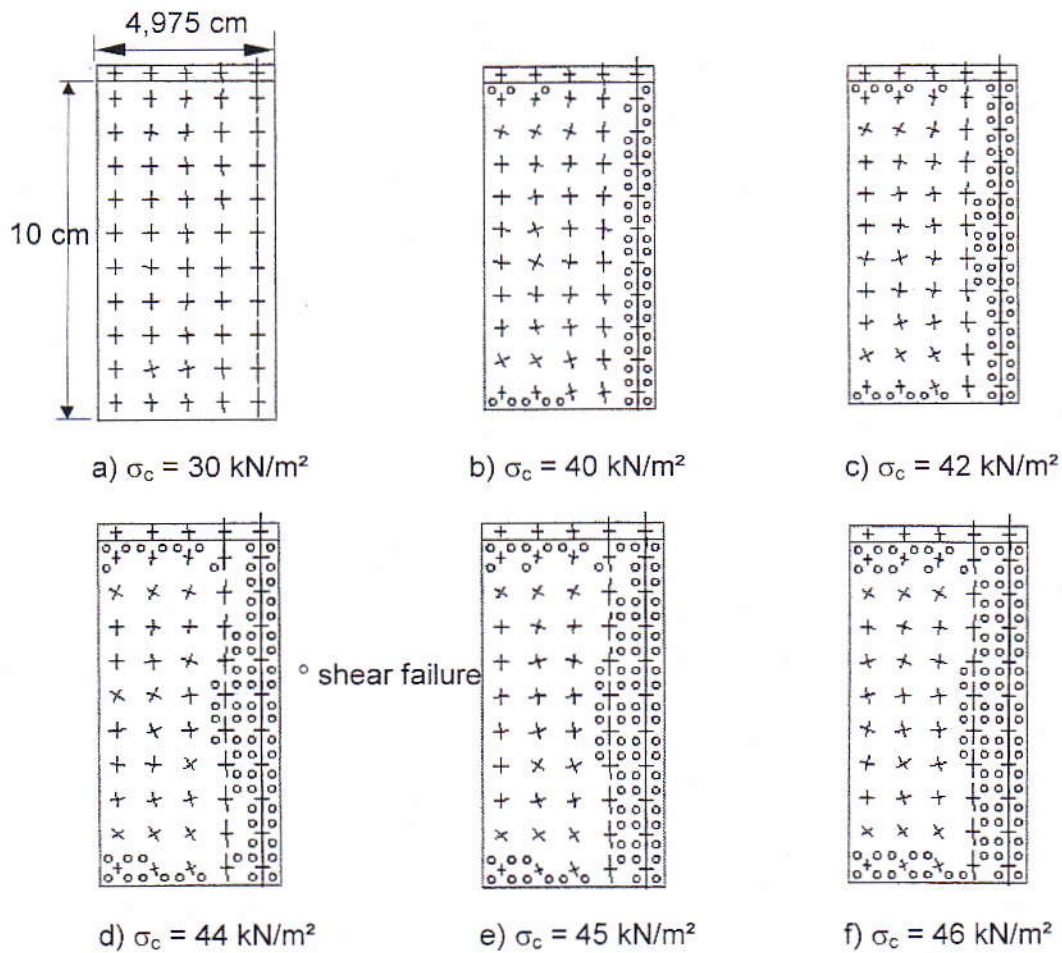Figure 4. Permanent axial strain dependent on σ_c and radial strain distribution

a) $\sigma_c = 30$ kN/m²

b) $\sigma_c = 40$ kN/m²

c) $\sigma_c = 42$ kN/m²

d) $\sigma_c = 44$ kN/m²

e) $\sigma_c = 45$ kN/m²

f) $\sigma_c = 46$ kN/m²

Figure 5. The principal stress and plastic zones at $N = 10^5$

Paris, Friday, June 14, 2002

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