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Generalized Plasticity and Cyclic Pressuremeter Test Modelling

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SYNOPSIS : A constitutive model based on generalized elastoplasticity (Zienkiewicz et al., 1985) is used for the modelling of monotonic and cyclic pressuremeter tests in a clay. The permeability of the material is taken into account for the modelling of the excess pore water pressure generation during the test (combination of pore pressure build up and dissipation). It is shown how this type of model can simply represent the main features observed during a cyclic pressuremeter test in a clay, particularly the accumulation of excess pore water pressure during the cycles of loading, and the importance of dissipation on the excess pore water pressure build up.

INTRODUCTION

The determination of soil properties using pressuremeter tests constitutes an important issue in soil mechanics, particularly in the case of cyclic behaviour (earthquakes, wave action...). The interpretation of such tests, in most cases, should take into account the biphasic aspect (water + squeleton) of the material. The modelling of the soil behaviour under monotonic and cyclic sollicitation may be done based on the "generalized plasticity" theory (Zienkiewicz et al., 1985, Pastor et al., 1985). The mechanical equations of the coupled problem are first given and then are applied to the cyclic pressuremeter sollicitation. After giving some elements on the numerical problem resolution, this communication shows the ability of such a model to simulate a cyclic pressuremeter test. In particular, we analyze the excess pore water pressure build up obtained with the generalized plasticity theory in the case of a clayey soil.

PRESENTATION OF THE MODEL

Within the frame of generalized plasticity (Zienkiewicz and Mroz, 1984), two models have been proposed. In the case of a clayey soil, Zienkiewicz et al. (1985) have proposed a first version. In order to simulate cyclic phenomena such as accomodation, adaptation or ratchet, the concept of bounding surface (Dafalias and Popov, 1975, Dafalias and Hermann, 1982) was introduced. This concept allows for plastic strains development inside the "yield surface", which then becomes a "bounding surface". For states of stress located on the bounding surface, strain increments are determined based on the classical elastoplasticity theory. For states of stress located inside the bounding surface, plastic strains are evaluated based on an interpolation rule. This rule, proposed by Mroz (1982), different from the one described by Dafalias and Herrmann (1982), introduces an additional parameter denoted γ which controls the amount of irreversibilities inside the bounding surface. This model is based on an associative flow rule with an elliptic bounding surface similar to the well-known "modified cam-clay" model (figure 1). In order to represent characteristic features of sand behaviour such as dilatancy of medium/dense sands or liquefaction of loose sands, Pastor et al. (1985) have proposed a second version of the initial model.

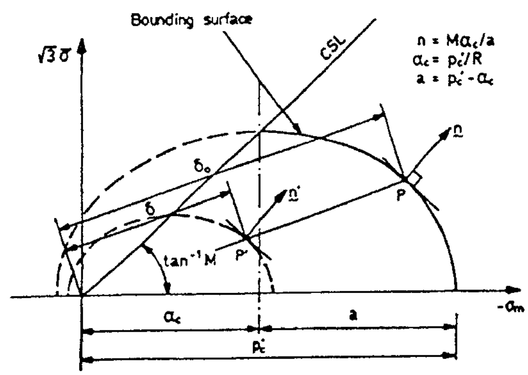

Fig. 1. Bounding surface in the (q, p') stress space (after Zienkiewicz et al., 1985).

MODELLING THE PRESSUREMETER SOLLICITATION

The constitutive models described above have been implemented in a finite difference code allowing to simulate the monotonic and cyclic pressuremeter sollicitation. It is assumed that this sollicitation may be approximated by the expansion of an infinitely long cylindrical cavity, allowing to study the problem under plane strains conditions. The mechanical problem is then one-dimensionnal along the radius r (figure 2). Dynamic terms are neglected in the equations, and only quasi-static equilibriums are considered. Small strains and small displacements are also assumed.

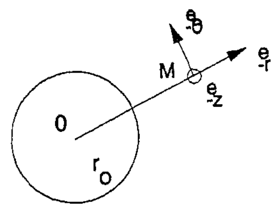

Fig. 2. Expansion of a cylindrical cavity.

The soil is supposed only biphasic (water + squeleton). The water and the solid matrix are supposed incompressible. In this case, the general equations proposed by Zienkiewicz and Bettess (1982) reduce as follows :

$$\begin{cases} \underline{\sigma} = \underline{\sigma}' - p.\underline{Id} \\ d\underline{\sigma}' = \mathbf{L} : d\underline{\varepsilon} \\ \underline{\varepsilon} = \frac{1}{2}(\underline{grad}(\underline{u}) + {}^t\underline{grad}(\underline{u})) \\ \underline{div}(\underline{\sigma}' - p.\underline{Id}) = \underline{0} \\ \frac{k}{\gamma_w}\Delta p - tr(\dot{\underline{\varepsilon}}) = 0 \end{cases} \quad (s1)$$

with : $\underline{\sigma}$: total stress tensor
$\underline{\sigma}'$: effective stress tensor
p : excess pore water pressure
$\underline{\varepsilon}$: strain tensor
$\underline{u}$: radial displacement vector of the solid matrix
L : fourth order tensor representing the soil behaviour
γ_w : unit weigth of water
k : permeability coefficient
$\dot{\underline{\varepsilon}}$: strain rate tensor

The boundary conditions in the case of the pressuremeter problem are :

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+---+------------------------------->
0   r0                              ∞
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$$\begin{cases} \forall t: & (\partial_r p)_{r_0} = 0 & p_{r_\infty} = 0 \\ & u_{r_0} = \bar{u}\,(1 - \cos(\omega t)) & u_{r_\infty} = 0 \\ t=0, \forall r: & u = 0,\ p=0,\ \partial_t u = 0 \text{ et } \partial_t p = 0 & \end{cases} \quad (s2)$$

The displacement vector $\underline{u}$ in the case of the pressuremeter sollicitation is supposed to have only a radial component :

$$\underline{u} = u(r,t).\underline{e}_r \ , \ p = p(r,t)$$

After developments of the system (s1), the mass balance of flow (last equation of s1) can be integrated and this system becomes :

$$\begin{cases} d\underline{\sigma}' = \mathbf{L} : d\underline{\varepsilon} \\ \underline{\varepsilon} = (\partial_r u\ , \frac{u}{r}\ , 0) \\ \partial_r p = \frac{\gamma_w}{k}(\ \partial_t u - \frac{r_0}{r}(\partial_t u)_{r_0}\ ) \\ \partial_r \sigma_{rr}^{\cdot} + \frac{\sigma_{rr}^{\cdot} - \sigma_{\theta\theta}^{\cdot}}{r} = \frac{\gamma_w}{k}(\ \partial_t u - \frac{r_0}{r}(\partial_t u)_{r_0}\ ) \end{cases} \quad (s3)$$

in which : $\underline{\sigma}' = \begin{bmatrix} \sigma'_{rr} \\ \sigma'_{\theta\theta} \\ \sigma'_{zz} \end{bmatrix}$, $\underline{\varepsilon} = \begin{bmatrix} \varepsilon_{rr} \\ \varepsilon_{\theta\theta} \\ \varepsilon_{zz} \end{bmatrix}$, $\partial_\alpha(\ ) \equiv \frac{\partial}{\partial\alpha}(\ )$

NUMERICAL RESOLUTION

In order to implement the soil behaviour in the mass balance of flow, this last equation is derivate and the variables become increments in time of the radial displacement u(r,t). The time partial derivatives are approximated to first order and a time implicit scheme is adopted. Spatial discretisation along the radius is done following a geometric series. First order centered finite differences are adopted to approximate the radial partial derivatives and the classical matricial method can be used to solve the problem in which the variable vector is composed of the increments in time of the radial displacements at all radius. The rigidity matrix obtained is tridiagional and can be simply inverted by the Cholesky method. Knowing the increments in time of the displacements, all other variables as excess pore pressure, effective stresses can be determined. Fictive points are added at $r=r_0$ in order to accelerate the code convergency. More details on the numerical resolution are given in Saïtta (1994).

PRESENTATION OF THE RESULTS

This section presents monotonic and cyclic simulations done using the Zienkiewicz et al. (1985) adapted to fine soils. The parameters used are given in table 1.

N° Simul.	ν	λ	κ	φ	Pc_0	e_0	k(m/s)	t_f(s)	T(s)	Amp.cyc.
SM1							Drained			
SM2							2.10^{-5}			
SM3	0.3	0.1	0.02	30°	24kPa	1.5	2.10^{-7}	150		
SM4							10^{-9}			
SM5							Undrained			
SM6	0.3	0.15	0.02	31°	24kPa	2	5.10^{-8}	150		
SM7	0.3	0.1	0.02	30°	20kPa	1.5	Undrained			
SM8							5.10^{-8}		60	2%
SM9	0.3	0.2	0.04	30°	50kPa	1.1	Undrained			

TABLE I. Parameters used in the simulations

Figure 3 presents a typical simulation of a monotonic expansion test (test SM4). In particular, it can be observed that the excess pore pressure is relatively high in this case, accounting for an important soil contractancy. Figure 4 shows the stress distributions with respect to the normalized radius. The rapid decrease of the radial stress and excess pore water pressure values within the soil mass may be noticed.

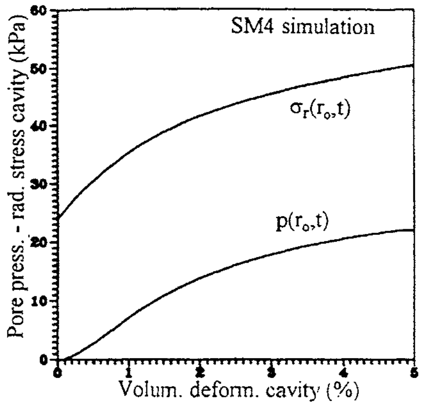

Fig. 3. Typical simulation of a monotonic cavity expansion.

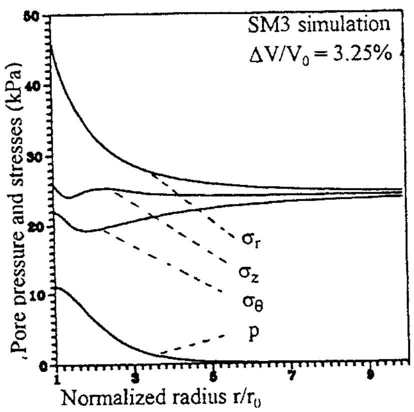

Fig. 4. Total stresses and excess pore water pressure distributions for $\Delta V/V_0 = 3.25\%$.

Figure 5 shows a simulation of a typical relaxation test for which, at the end of the expansion, the cavity volume is maintained constant. The dissipation with time of excess pore pressure generated at the cavity is observed, as well as a relaxation of the total radial stress. It is important to note the increase in radial effective stress in the soil near the cavity. Figure 6 shows the excess pore pressure generated in the soil at different moments of the consolidation process. The pore pressure first increases for radii far from the cavity (due to the diffusion phenomenon), followed by a decrease down to the initial hydrostatic value.

Comparison of the simulation with experimental results carried out at the Cran site by the Laboratoire Régional des Ponts et Chaussées of St-Brieuc (provided by M. Jézéquel) is shown on the figure 7. The main characteristics of the simulation are given in table I. While the expansion curve is well represented by the model, the simulation overestimates the excess pore pressure measured. This is essentially due to an anisotropic permeability coefficient existing in reality and which is not taken into account in the simulation. Moreover,

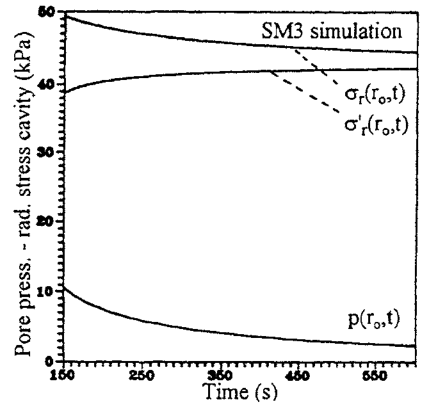

Fig. 5. Dissipation at the end of cavity expansion.

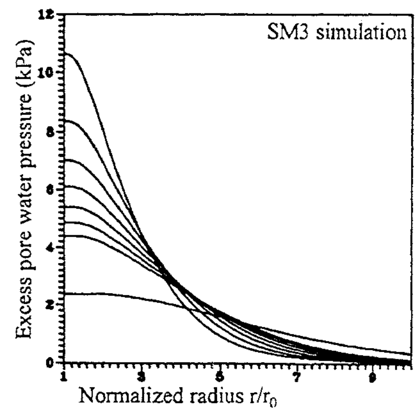

Fig. 6. Excess pore pressure distribution every 30 seconds during consolidation process.

the yield surface of the model is elliptic like in the modified cam-clay model, and this contributes to overestimate the soil contractancy (and then, the excess pore pressure build up).

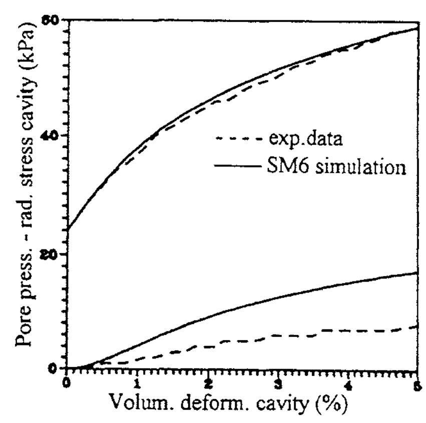

Fig. 7. Simulation of a test carried out at Cran site.

Figure 8 shows two simulations of cyclic expansion test (maximum value of the relative cavity volume $\Delta V/V_0$=2%). It is interesting to note the accumulation of excess pore pressure simulated by the model during the cycles. Figure 8 shows the effects of the permeability on this accumulation by comparing a perfectly undrained test and a material with a permeability coefficient k=5e-8 m/s. In the undrained case, the accumulation of excess pore pressure stabilizes during the cycles, while the average value decreases due to dissipation along the radius in the second case.

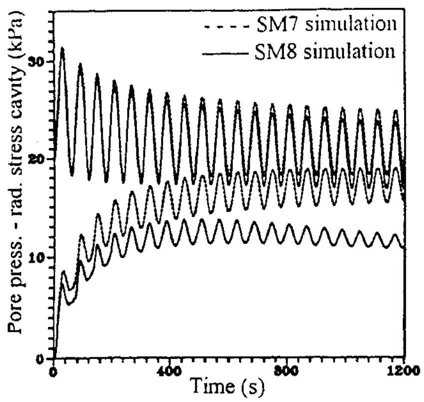

Fig. 8. Influence of permeability coefficient during cyclic expansion test.

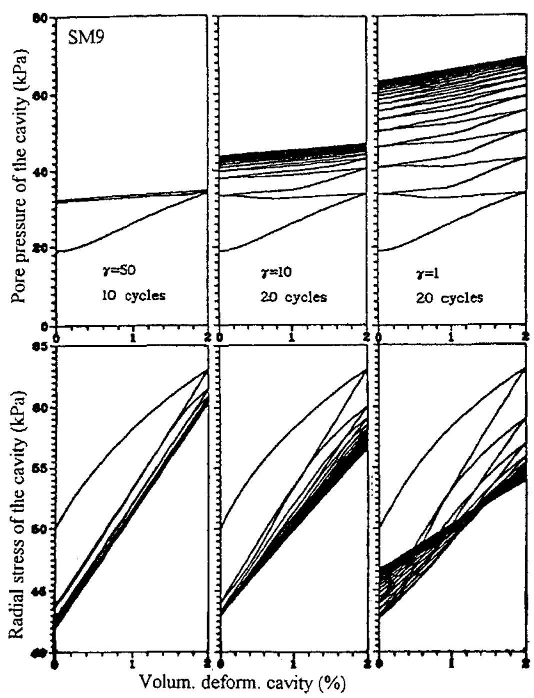

Fig. 9. Influence of γ on the simulations for cycling loading.

In the model, the accumulation of irreversibles strains during cyclic loading is governed by the interpolation parameter γ. Figure 9 shows the influence of this parameter on the simulations obtained. It is interesting to note that, for high values of γ (no plastic strains inside the bounding surface), the material immediatly reaches adaptation. For low values of γ, the accumulation of excess pore pressure during the cycles is high. This model can consequently account in a simple manner for these cyclic phenomena observed in-situ particularly by Canou (1984).

CONCLUSION

The interest in using such models as the one presented is to simulate monotonic and cyclic phenomena observed in geomechanics in a simple manner. Moreover, these models require a minimum number of parameters particularly for the modelling of cyclic loading. Concerning the pressuremeter test, this communication has shown the ability of such models to simulate this sollicitation. In particular, the cyclic pressuremeter test can be simulated, in a clayey soil, accounting for coupled excess pore water pressure build up and dissipation. Since there is only one parameter accounting for the cyclic part of the behaviour (γ), the inverse problem may be solved to identify γ. This parameter may then be very useful to quantify the response of the soil to a given cyclic sollicitation, in particular in terms of excess pore water generation.

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