Some important aspects in evaluating cyclic triaxial tests on clayey soils

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ABSTRACT: Compared to static test, cyclic triaxial test on saturated clayey soils under partly drained condition can not be seen as element test, as even under ideal technical condition the distributions of stress and excess pore pressure in soil sample are inhomogeneous. In this paper, cyclic triaxial test is numerically studied by using a quasi-static model which is developed for describing long-term plastic deformation of saturated clayey soils under cyclic loading condition. In the investigation, real boundary conditions were taken into consideration and their influence was demonstrated.

1 INTRODUCTION

Cyclic triaxial test is a conventional method in investigating the mechanical behavior of soils under cyclic loading. It is well known, however, that cyclic triaxial tests on clayey soils under partly drained condition can not be seen as element tests, as the distributions of stress and excess pore pressure in soil sample are normally inhomogeneous due to the low permeability of clayey soil and applied load velocity (frequency). A direct derivation of constitutive relationship from tests is therefore impossible.

Based on the physical description of excess pore pressure and plastic deformation of saturated clayey soils under cyclic loading, a conceptual model is proposed upon quasi-static approach. Using some typical test results under cyclic undrained condition as well as under cyclic undrained-subsequent drained tests, this model is formulated and numerically implemented by using finite element method.

Using this calculation model, cyclic triaxial test is numerically simulated for clarifying the real distribution and development of stress and excess pore pressure as well as resulting permanent deformation of soil sample.

2 QUASI-STATIC MODEL

2.1 *General description and conceptual model*

The mechanical behavior of saturated clayey soils under cyclic loading is dependent on many factors, above all soil type. Here, it is focused on saturated clayey soils that show contracting behavior under shear loading. For such soils, excess pore pressure is expected under cyclic loading. Depending on cyclic number, excess pore pressure rises in the first stage and consists of reversible and "permanent" components ($u = u^c + u^b$). The "permanent" part results from the undrained plastic shear deformation and remains when load is removed. After some time (t_p see Figure 1) which depends on e.g. permeability, drainage path, loading and boundary condition, excess pore pressure reaches peak value and begins to dissipate in the second stage. Due to the dissipation of excess pore pressure, plastic volume strain occurs. Compared to "permanent" part, however, reversible part of excess pore pressure is normally much smaller than "permanent" part. Furthermore, this part of excess pore pressure can not dissipate due to its oscillating character.

The research in (Hu 2000) showed that "permanent" excess pore pressure is a decisive parameter for plastic deformation of normally consolidated clayey soils under cyclic loading condition. Both plastic deformations, 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 was proposed describing plastic deformation behavior of saturated clayey soils under cyclic loading, see Figure 1 from (Hu 2000). In this model, maximum of cyclic stress is applied as loading parameter. The attention is focused on permanent deformation and individual cyclic loop is therefore not followed.

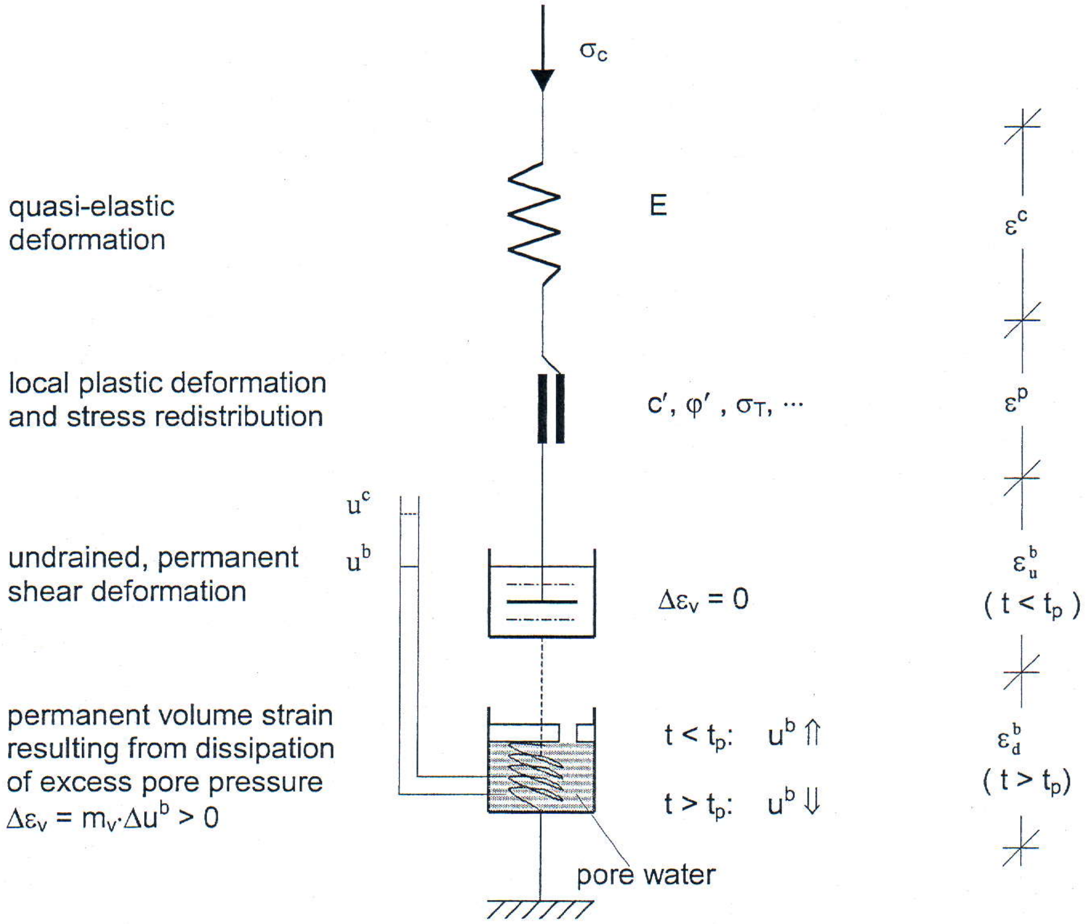

Figure 1. Conceptual model (quasi-static) describing deformation behavior of saturated clayey soils under cyclic loading.

The elastic module E is introduced to simulate 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 in Figure 1 represents shear and tension limits of saturated clayey soils under cyclic loading condition. Some test results in the literature showed that dynamic frequency has an influence on shearing strength limit. Under cyclic loading condition, static shear strength of clayey soils can be exceeded. However, this occurs mostly in the beginning of cyclic loading. The investigation of (Yasuhara et al. 1982) using effective stress strength parameters indicated that for normally consolidated clays shear strength under undrained condition is not significantly affected by cyclic frequency and loading duration. For simplification, it is assumed that static strength parameters can be applied for the case of lasting cyclic loading.

In the beginning of cyclic loading, most soil elements remains undrained and excess pore pressure rises depending on cycle number. The reversible part results from elastic deformation of soils and can be approximately assessed using $u^c = A_c \cdot (\sigma_{x,c} + \sigma_{x,c} + \sigma_{z,c})$ ($\sigma_{x,c}$, $\sigma_{x,c}$ and $\sigma_{z,c}$ stress components from 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. With the help of excess pore pressure and total stress (= static + quasi-static stress), possible strength failure upon effective stress principle can be checked and plastic deformation ε^p occurs if strength limit exceeded.

Under cyclic loading condition, a particular aspect is the occurring plastic deformation of soils in the case that cyclic loading is inside strength failure envelope. Here, plastic deformation consists of two parts. The first part $\varepsilon^b{}_u$ results from undrained and the second $\varepsilon^b{}_d$ drained part (non-linear viscous dashpot and consolidation dashpot in Figure 1). As mentioned above, both of them are dependent on "permanent" excess pore pressure.

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:

$$\frac{\partial}{\partial x}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial x}\right)+\frac{\partial}{\partial y}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial y}\right)+\frac{\partial}{\partial z}\left(\frac{k}{\gamma_w}\frac{\partial u^b}{\partial z}\right)= m_v\left(\frac{\partial u^b}{\partial t}-\psi\right) \tag{1}$$

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 undränied plastic shear deformation through an empiric relationship (see section 2.3). The subsequent dissipation ($\Delta u^b < 0$) leads to plastic drained strain and can be determined by using:

$$\Delta\varepsilon_d^b = m_v \cdot \Delta u^b. \tag{2}$$

The parameter m_v is volume compressive coefficient under cyclic loading

2.2 *Excess pore water pressure under undrained cyclic loading condition*

Lots of results of cyclic triaxial tests on clayey soils under undrained condition have been conducted and reported in the literature. For example, *Yasuhara (1988) et al* proposed that excess pore pressure u^b_g under cyclic undrained condition can be evaluated using the following empiric relationship:

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

where ρ, α_G and θ are regression parameters obtained from undrained cyclic tests.

The test results of (Matsui et al. 1980) using different frequencies (f = 0.02/0.05/0.2/0.5 Hz) indicated that in reference to cycle number higher excess pore pressure appears at low frequency condition. This difference may be probably from different loading time, because "permanent" excess pore pressure is caused by plastic shear deformation of soil skeleton and therefore closely dependent on total loading time. If test results are analyzed using absolute loading time instead of cycle number, the influence of frequency could be secondary. Based on this, the empiric relationship (4) is rewritten as follows:

$$\frac{u_g^b}{\sigma_{3,0}} = \rho \cdot \left(\frac{q_c}{\sigma_{3,0}}\right)^{\alpha_G} \cdot (1+\log t)^{\theta}. \tag{4}$$

2.3 *Undrained plastic shear deformation*

In strain theory it is assumed that under undrained condition shear sliding as well as partly collapse of soil skeleton leads to redistribution of loads between pore water and soil skeleton, see (Lo 1969 a, b). The load originally carried by contact points of soil grains is partly transferred to pore water. As this shear deformation is predominately plastic, resulted excess pore pressure is mostly "permanent". It can be therefore concluded that there exists a close relationship between undrained plastic shear strain and "permanent" excess pore pressure. For normally consolidated clayey soils, (Yasuhara et al. 1982) found out that independent of loading form and frequency an hyperbolic relationship exists between the ratio of excess pore pressure to consolidation pressure and the first principal strain:

$$\frac{u}{p_0'} = \frac{\varepsilon_{1,u}}{a+b\cdot\varepsilon_{1,u}}, \tag{5}$$

where a and b are two experimental parameters.

Though the relationship shown in equation (5) includes reversible part of excess pore pressure and strain, it could be concluded from the strain theory that there is a similar relationship for "permanent" parameters. Therefore, the following relationship is used in the proposed model:

$$\frac{u^b}{p_0'} = \frac{\varepsilon_{1,u}^b}{a+b\cdot\varepsilon_{1,u}^b} \tag{6}$$

2.4 *Drained plastic volume strain*

The parameter m_v in equation (1) and (2) represents volume compressive coefficient under cyclic loading condition and should not be the same as that under static loading condition. It is a common way to get this parameter using undrained cyclic triaxial tests with subsequent dissipation. In this way, an empiric relationship between "permanent" excess pore pressure u^b after undrained cyclic loading and plastic volume strain from dissipation can be established. The tests on Drammen clay (OCR=1) by (Yasuhara & Andersen 1991) showed that permanent volume strain resulted from cyclic loading could be assessed using the following relationship:

$$\varepsilon_v^b = 1{,}5\frac{C_r}{1+e_0}\log\left(\frac{1}{1-u^b/p_0'}\right). \tag{7}$$

C_r is the compressive parameter of conventional static oedometer test in reloading stage. Upon this, the parameter m_v can be calculated using

$$m_v = d\varepsilon_v^b/du^b = 1{,}5\cdot\frac{C_r}{1+e_0}\cdot\frac{1}{\ln 10}\cdot\frac{1}{p_0'-u^b}. \tag{8}$$

Based on the relationships shown above, the proposed model in Figure 1 was numerically implemented using finite element method (called Geocycl).

3 NUMERICAL STUDY OF CYCLIC TRIAXIAL TESTS

3.1 *Generally*

Using Geocycl, the mechanical behavior of saturated clayey soil under cyclic triaxial condition was numerically investigated. This research consists of two parts. In order to demonstrate the applicability of the model in predicting the development of excess pore pressure in the course of cyclic loading, the cyclic triaxial tests of (Yasuhara et al. 1988) on Ariake clay were reanalyzed. Then, a parametric study was conducted on plastic deformation of a medium-plastic clay under cyclic triaxial condition.

3.2 *Numerical simulation of excess pore pressure under cyclic triaxial condition in the literature*

Based on (Yasuhara et al. 1988), Ariake clay shows high plasticity. The test material was fully consolidated under a vertical pressure of 59 kN/m². The trimmed soil samples (35 mm in diameter and 87,5 mm in height) had an initial water content between 90% and 95%. After installation in triaxial cell, they were statically loaded by an around pressure of 100, 200 and 300 kN/m² respectively and consolidated for 24 hours. Then, soil samples were cyclically loaded by an harmonic vertical stress. In this stage, drainage was allowed laterally. In the course of cyclic loading, excess pore pressure was measured at the middle point of sample bottom. In terms of the literatures, the soil mechanical parameters and the parameters necessary for the numerical modeling are given in the table 1.

In Figure 2, the calculated results are illustrated in comparison with the test results of (Yasuhara et al. 1988). It can be concluded that the presented model can predict the development of excess pore pressure in the course of cyclic loading properly.

Table 1. Soil mechanical parameters of Ariake clay and the parameter necessary for numerical modeling after (Yasuhara et al. 1988).

Soil mechanical parameters:
$\rho_s = 2.65$ g/cm³; $w_L = 123$ %; $I_P = 69$ %;
$C_c = 0.7$; $C_s = 0.163$; $c' = 0.0$; $\varphi' = 39°$.

Parameter for numerical modeling:
$\rho = 0.064$; $\alpha_G = 1.412$; $\theta = 1.535$; $c' = 0.0$;
$\varphi' = 39°$; $k_r = k_z = 1{,}2 \times 10^{-10}$ m/s;
$m_v = 1.26 \times 10^{-4}$ m²/kN; $a = 1.48$; $b = 0.57$.

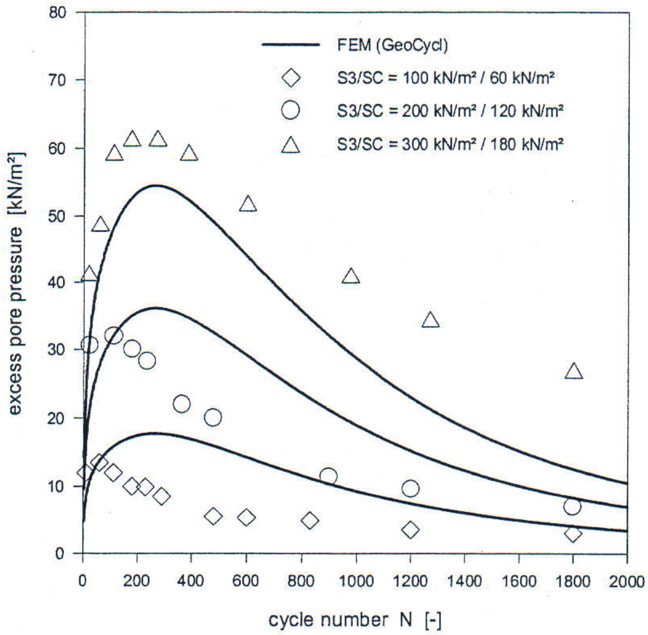

Figure 2. Comparison of excess pore pressure between the calculation using GeoCycl and the test results of (Yasuhara et al. 1988).

3.3 *Deformation behavior of clayey soils under cyclic triaxial condition*

The details of the problem studied are shown in Figure 3 including load, boundary condition, FE-mesh and FD-grid. The parameters used are also given in the figure. The soil is a medium-plastic clay. Soil samples are saturated and 9.95 cm in diameter and 10 cm in height. Soil samples is statically consolidated under $\sigma_{3,0}$ and then cyclically loaded by $P = \pi \cdot R^2 \cdot \sigma_c/2 \cdot [1 - \cos(2\pi f t)]$ ($f = 1$ Hz). It is assumed that the surface of soil sample is kept being drained during cyclic loading.

For simulating the real load transferring, top platen is included in the calculation model, see Figure 3. As smearing technique is usually applied in triaxial test for reducing friction between top platen and soil sample, a frictionless horizontal joint surface is assumed in between-zone.

For the case of $\sigma_{3,0} = 50$ kN/m² and $\sigma_c = 50$ kN/m² the principal stress at N = 0 and 10^5 as well as displacement are illustrated in Figure 4. It can be clearly seen that the stress distribution on the top of soil sample is initially equal and gradually becomes unequal depending on cycle number. The shear failure begins from outside ring. Compared to this, the vertical displacement at the top of sample is always equal because of forced displacement condition induced by stiff top platen, whereas the distribution of lateral displacement is not equal along sample height in spite of assumed frictionless surface.

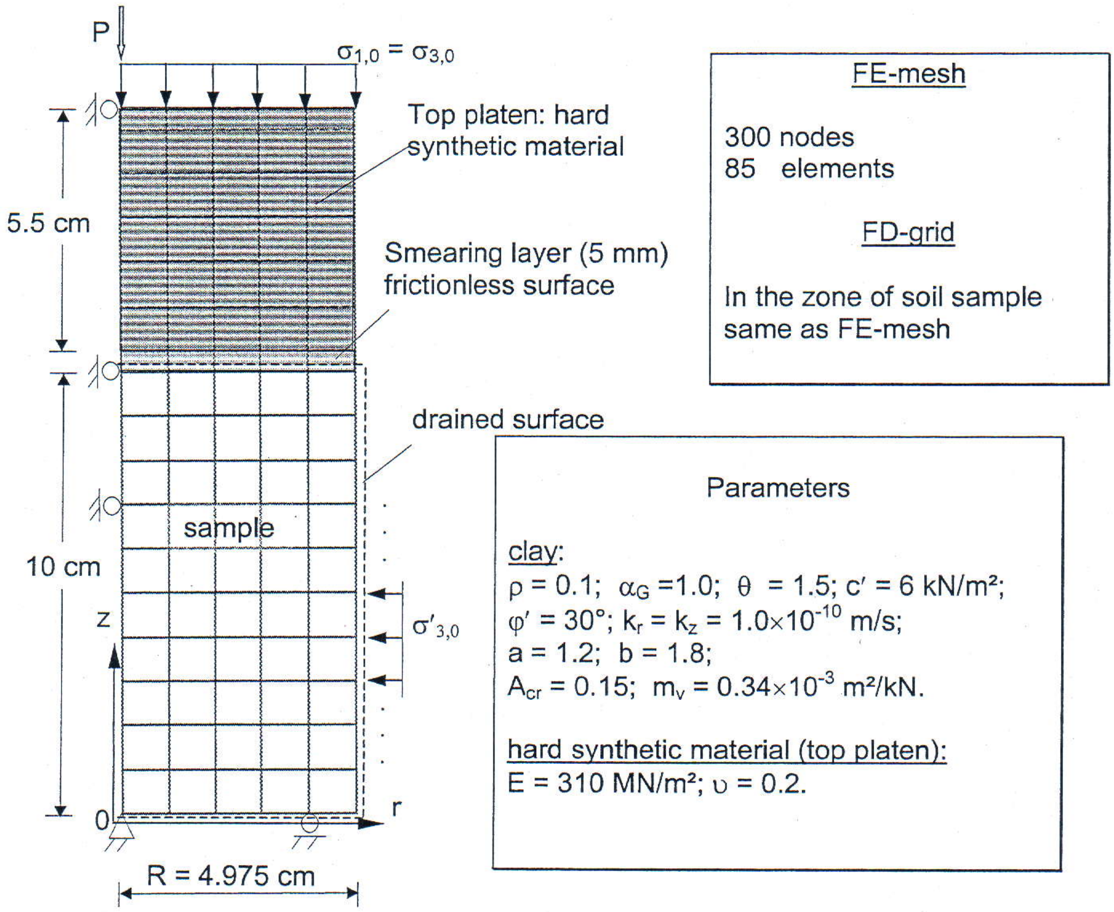

Figure 3. Computational section, FE-mesh, boundary conditions and parameter used for the parametric study of saturated clay under cyclic triaxial condition.

The calculated results can be explained as follows: near the assumed drained surface excess pore pressure changes rapidly and is smaller than that inside. As permanent undrained and subsequent drained strains are dependent on "permanent" excess pore pressure, resulted plastic strain must be unequal at the top of sample surface. This phenomena can however not be observed in the investigated case due to the forced equal displacement condition of top platen. Arising from this, load redistribution takes place in the way that the load in the outside ring increases from 90 kN/m² (N=0) up to 130 kN/m² ($N=10^5$). At the same time the stress in the middle reduces to 47 kN/m². As a result, shear failure begins in the outside ring and extends gradually into the inside.

To find out the critical load at which plastic flow of soil sample occurs, parametric study was carried out by varying maximum of cyclic stress. The results are put together with the test results of (Hu 2000) in Figure 5. It can be seen that plastic flow begins at σ_c = 45 kN/m² computationally. Up to 40 kN/m² (2. stage), the computational and experimental results are in good agreement, whereas a deviation appears after 45 kN/m². The test results show clearly stiffer behavior than calculated. This can be put down to the compacting effect in the preceding cyclic loading stages which can not be considered using the model.

4 CONCLUSIONS

This paper presents a quas-static constitutive model describing development of excess pore pressure and plastic deformation of saturated clayey soils under cyclic loading condition. For this purpose, some experimental results as well as correlation from the literature were used. The implementation of the equation by using finite element method makes the practical application possible.

The re-analysis of the measured excess pore pressure under cyclic triaxial condition in the literature indicated the applicability of the developed model.

Using the numerical model, a study of plastic deformation behavior was made for a saturated medium-plastic clay. The real loading and boundary

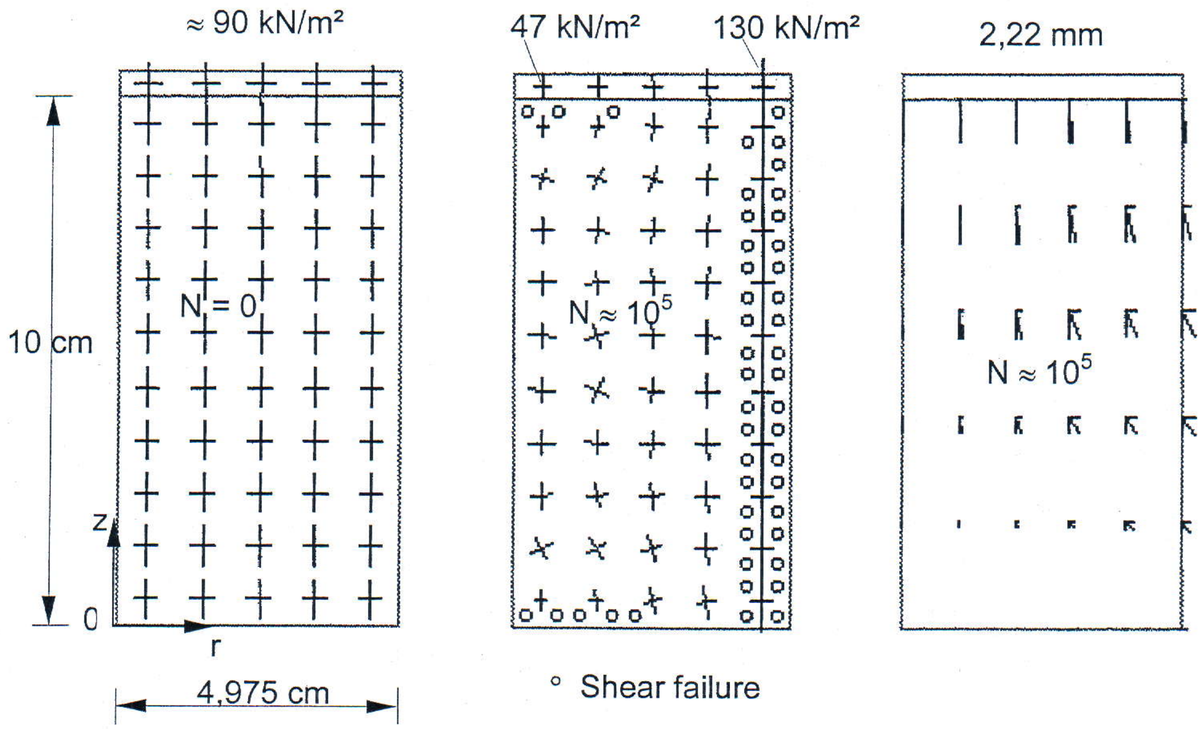

Figure 4. Stress distributions at $N = 0$ and 10^5 as well as displacement distribution at $N = 10^5$ for the case of $\sigma_{3,0} = 50$ kN/m² and $\sigma_c = 50$ kN/m².

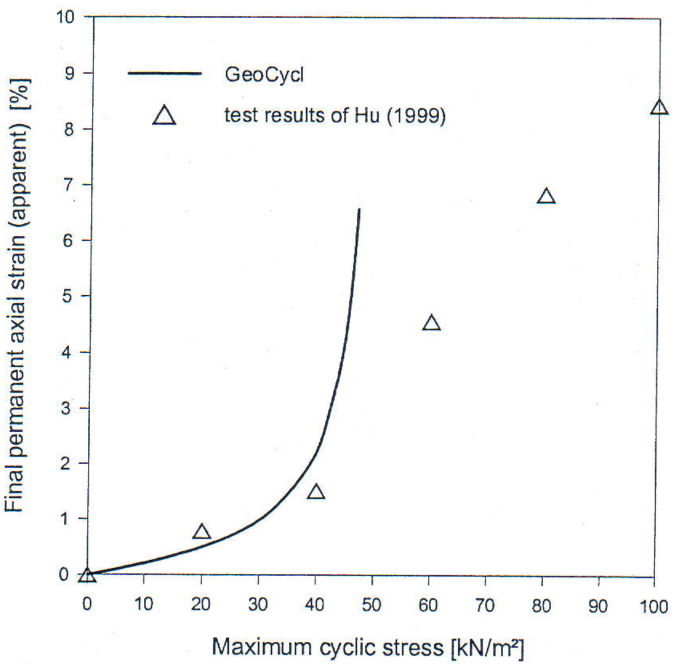

Figure 5. Final permanent axial strain depending on maximum cyclic stress.

conditions were simulated. The numerical results illustrated the development of plastic deformation depending on cycle number, the influence of displacement boundary condition on loading redistribution as well as plastic failure. This is helpful in evaluating cyclic triaxial tests on saturated clayey soils.

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