Pore pressure behaviour of soft clay

Comportement de la pression interstitielle dans les argiles molles

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SYNOPSIS Undrained shear tests under various stress path were carried out in triaxial, plane strain and hollow cylinder apparatuses with artificially prepared soft clay samples, and pore pressures were measured, analyzed and compared. It is shown by test results that the normally consolidated soft clay is always dilatant negatively. However, this essential behaviour of soft clay can not be considered in many linear or nonlinear elastic constitutive models of soil available presently so that the results of prediction with these models are often unsatisfactory. A model is presented on basis of the experimental data, and two case histories are analyzed with the proposed model together with other two models in order to compare the pore pressures calculated with or without consideration of negative dilatancy of the soft clay.

INTRODUCTION

Soil is different from many other materials in that volume change (drained) or pore pressure (undrained) occurs also under the action of shear stress. This behaviour of soil is called dilatancy (Wei, 1963). If the soil mass contracts or positive pore pressure is built up under the shear stress, then it is dilatant negatively.

The pore pressure behaviour of soil under shear stress is influenced by many factors, such as the initial state of consolidation, stress history, mode of failure and reorientation of principal stress. These factors can be reduced primarily to the effect of different stress condition on the pore pressure behaviour of soil. In order to study the pore pressure behaviour of soft clay under the action of shear stress, undrained shear tests under various stress path were carried out in triaxial, plane strain and hollow cylinder apparatuses with artificially prepared soft clay samples(w_L=32.5%; I_P=12.2; w= 35% and ρ=1.80 g/cm^3 after consolidation under vertical pressure of 50 kPa), and the pore pressures in these tests were measured, analyzed and compared.

It is shown by test results that the normally consolidated soft clay is dilatant negatively, and it is dangerous to overlook the negative dilatancy of soft clay in the engineering design. For instance, according to the principle of effective stress, the factor of safety in the stability problem decreases with the increasing pore pressure. If the pore pressure caused by negative dilatancy is neglected, then the calculated pore pressure will be underestimated and the factor of safety is hence overestimated.

However, the negative dilatancy of soft clay can not be considered in many linear or nonlinear elastic constitutive models of soil available presently so that the results of prediction with these models are often unsatisfactory. Therefore it is necessary to establish a nonlinear elastic model that is not only relatively simple but also able to reflect on the essential characteristics of soft clay including its negative dilatancy. Such a model is proposed in this paper on basis of experimental data. Then, two case histories are analyzed with this model together with other two models in order to compare the pore pressures calculated with or without consideration of the negative dilatancy of soft clay.

NORMALIZATION OF PORE PRESSURE

In order to study the relationship between the pore pressure and the components of stress or strain, the pore pressure may be normalized with the mean effective stress p or the average consolidation pressure p_c. It can be seen from Fig. 1 that, after the pore pressure is normalized with p_c, the u/p_c versus ε_1 curve is a hyperbola, and there is a unique relation between u/p_c and ε_1 for the same type of stress condition independent of the magnitude of consolidation pressure. The following idea seems to be verified that the pore pressure is a strain effect instead of a stress effect (Lo, 1960). But, it is not reasonable to say that the

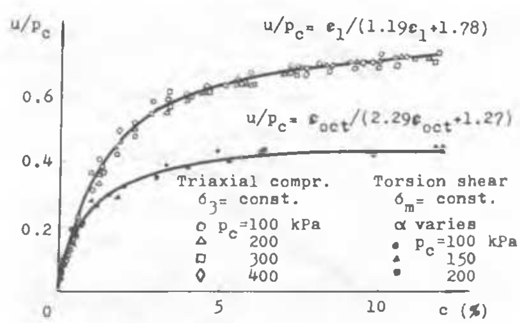

Fig. 1 Normalized pore pressure curves under different stress conditions.

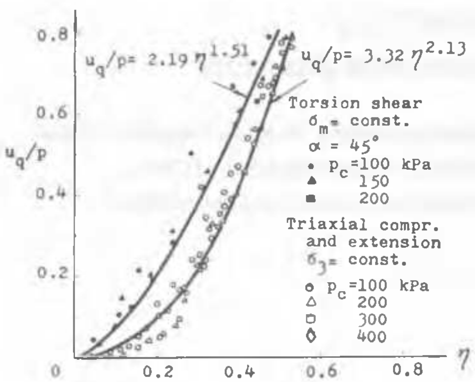

Fig. 2 Normalized pore pressure caused by negative dilatancy under different stress condition.

pore pressure depends uniquely on the strain, because the relationships between u/p_c and ε_1 obtained under various type of stress condition are although all hyperbolic, the fitting parameters vary with the type of stress condition. That is to say, the type of stress condition still has an influence on the pore pressure behaviour of soft clay. Therefore, it is preferable to say that the pore pressure ratio u/p_c depends uniquely on the major principal strain ε_1 under the given type of stress condition.

The pore pressure built up in the soft clay may be divided into two parts: u_p induced by normal pressure p and u_q by shear stress q. It can be seen from Fig. 2 that the relationships between u_q/p and η (η=q/p is the stress ratio) under different stress condition are all power functions, but the fitting parameters vary with the stress condition too.

EFFECTIVE STRESS PATH AND STATE BOUNDARY SURFACE

It is shown by the triaxial compression tests that the effect of initial consolidation condition (isotropic or K_o-consolidation) on the shape of effective stress path is not remarkable

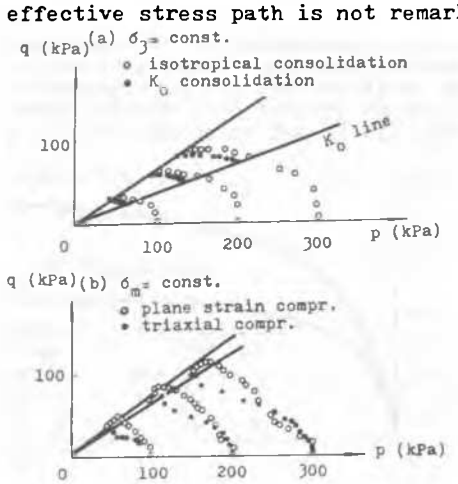

Fig. 3 Effective stress paths under different stress condition.

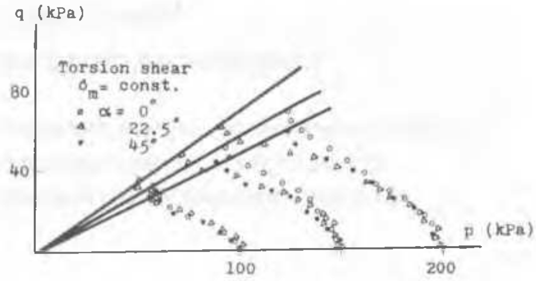

Fig. 4 Effective stress paths under different direction of principal stress.

as shown in Fig. 3(a). It can be seen from Fig. 3(b) that the effective stress paths of compression tests in triaxial and plane strain apparatuses essentially coincide with each other under low shear stress , and deviate from each other when the shear stress increases. The reason responsible for this difference is that the effect of intermediate principal stress acting on the plane strain specimen becomes more and more pronounced in the last stage of test.

It is shown in Fig. 4 that different effective stress path or different state boundary line may be obtained from undrained torsion tests with different direction of principal stress. In order to describe the effect of reorientation of principal stress on the pore pressure behaviour of soft clay, a modified concept of state boundary surface (Symes et al., 1984) is introduced. The state boundary surface was originally defined in the p-q-e space (Roscoe et al., 1958), where e is the void ratio of soil. In this paper, the results of various torsion test with different direction of principal stress α=const. are defined in the p-q-α space, and the curved surface formed by the data points represents the state boundary surface at e=const. or p_c=const. The state boundary surface shown in Fig. 5 is one defined by the results of three torsion tests under p_c=100 kPa with the major principal stress fixed at α=0°, 22.5° and 45° respectively. The result of another torsion test under the same consolidation condition but with varying direction of principal stress during the test is also shown in this figure, and it can be seen that the data points (p,q,α) of this test locate basically on the state boundary surface defined above. Therefore, using this modified state boundary surface, we can predict the pore pressure variation with varying direction of principal stress on basis of the results of torsion tests with fixed direction of principal stress.

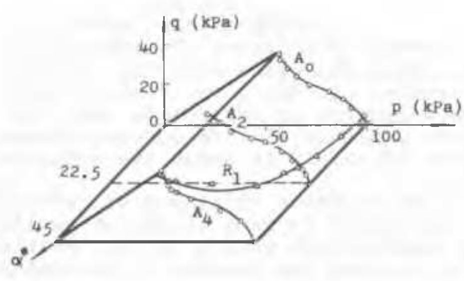

Fig. 5 State boundary surface under p_c=100 kPa.

PORE PRESSURE PARAMETER AT FAILURE UNDER DIFFERENT STRESS CONDITION

The soil in-situ is always K_o-consolidated and fails under the plane strain condition. For sake of that the characteristics of soil determined in the laboratory can realistically represent its behaviour in the field, the laboratory test should simulate the in-situ stress condition and the mode of failure as closely as possible. In order that the pore pressure parameter at failure under the field condition can be deduced approximately from the conventional triaxial test data, attemps were made to find out the relation between the pore pressure parameters at failure determined with isotropically and K_o-consolidated samples in the triaxial and plane strain compression tests.

It is shown by test that for a given soft clay, the ratio between stress ratios at failure in K_o- and isotropically consolidated triaxial and plane strain tests is a constant. Then according to the Mohr-Coulomb failure criterion and the principle of effective stress , the relation between the pore pressure parameters at failure of K_o- and isotropically consolidated samples in triaxial compression tests or between those in plane strain and triaxial compression test can be readily obtained. Finally the following relations are determined for the artificially prepared samples:

$$A_{Kof} = 1.9\, A_{If}$$

$$\alpha_{Pf} = 0.75\, \alpha_{Tf}$$

where $A_{Kof}=u_{Kof}/(\sigma_1-\sigma_3)_{Kof}$, $A_{If}=u_{If}/(\sigma_1-\sigma_3)_{If}$, and $\alpha_{Pf}=u_{Pf}/(\tau_{oct})_{Pf}$, $\alpha_{Tf}=u_{Tf}/(\tau_{oct})_{Tf}$ defined similarly as Skempton's and Henkel's pore pressure parameter respectively.

GENERAL EQUATION OF PORE PRESSURE

The effects of the anisotropy of clay and the reorientation of principal stress can not be considered in Skempton's and Henkel's pore pressure equation. However, these effects may be considered in terms of anisotropic parameters of pore pressure, and the general equation of pore pressure is expressed as follows (Baker, 1984):

$$\begin{aligned}\Delta u=&(\Delta\sigma_1+\Delta\sigma_2+\Delta\sigma_3)/3+[a_1^2(\Delta\sigma_1\sin^2\alpha+\Delta\sigma_3\cos^2\alpha-\Delta\sigma_2)\\ &+a_2^2((\Delta\sigma_2-\Delta\sigma_1\cos^2\alpha-\Delta\sigma_3\sin^2\alpha)^2\\ &+(\Delta\sigma_1\cos^2\alpha+\Delta\sigma_3\sin^2c-\Delta\sigma_1\sin^2\alpha-\Delta\sigma_3\cos^2\alpha)^2)\\ &+6d_2^2(\Delta\sigma_1-\Delta\sigma_3)^2\sin^2\alpha\cos^2\alpha\,]^{1/2}/3 \qquad (1)\end{aligned}$$

where a_1, a_2 and d_2 are anisotropic parameters of pore pressure, and their values not only increase with the increasing shear stress, but also vary with different soil type, consolidation condition and stress history.

According to the modified concept of state boundary surface and based on the stress and pore pressure at failure of the artificially prepared soft clay samples in the torsion tests with direction of the principal stress fixed at 0°, 22.5° and 45°, the pore pressure parameters at failure in eq. (1) are determined as follows: $a_{1f}=5.26$, $a_{2f}=1.17$ and $d_2=1.46$. Then these values are used together with eq. (1) to predict the pore pressure of the sample approaching failure in the torsion test with the direction of principal stress varying continuously during the test. In comparison of the calculated pore pressures with the measured values, it is found that the relative errors of the prediction are all within 8% (Table I).

Table I Comparison between the predicted and measured pore pressures.

Consolidation pressure (kPa)	100		150		200	
Measured pore pressure in torsion test(kPa)	44	45	65	68	85	87
Predicted pore pressure with eq. (1) (kPa)	43	45	60	65	79	87

NONLINEAR ELASTIC MODEL WITH CONSIDERATION OF NEGATIVE DILATANCY

It is shown by the laboratory tests that: (a) the confined compression and rebound curves may be fitted with semilogarithmic lines, where the volumetric strain caused by negative dilatancy is irreversible and can be represented with a power function of shear stress; (b) the shear stress-strain relation may be described as a hyperbolic curve, and its unloading branch is a straight line parallel to the initial tangent, while the value of the initial tangent modulus is a power function of the mean effective stress. Therefore the stress-strain relations may be expressed as

$$\varepsilon = \lambda \ln p + c\eta^m + \varepsilon_o \qquad (2)$$

$$\gamma = \frac{aq}{1-\eta/M} \qquad (3)$$

where ε and γ is the volumetric and shear strain; λ is the bulk compression index; c--coefficient of the negative dilatancy; m--power exponent of the negative dilatancy; a--reciprocal of the initial modulus, $a=\bar{a}(p/p_a)^n$, n is power exponent of pressure and p_a is the atmospheric pressure; ε_o--a constant; $M \doteq q_u/p$--the ultimate stress ratio; and q_u is the ultimate shear stress.

By differentiation of eqs. (2) and (3), and introducing the factors of loading α_1 and α_2, the stress-strain relations in incremental form are established as follows:

$$\delta\varepsilon = \delta p/K_p + \delta q/K_q \qquad (4)$$

$$\delta\gamma = \delta p/G_p + \delta q/G_q \qquad (5)$$

where

$$1/K_p=(\kappa+\alpha_1(\lambda-\kappa)-\alpha_2 cm\eta^m)/p$$

$$1/K_q=\alpha_2 cm\eta^{m-1}/p$$

$$1/G_p=a\eta(\alpha_1\alpha_2(M+n\eta)(M-\eta)+\alpha_1 n(M-\eta)^2-\alpha_2 M^2)/(M-\eta)^2$$

$$1/G_q=a((M-\eta)^2+\alpha_2(2M-\eta)\eta)/(M-\eta)^2$$

$$\alpha_1=\begin{cases}1 & \text{when } \delta p \quad 0\\ 0 & \text{when } \delta p \quad 0\end{cases}$$

$$\alpha_2=\begin{cases}1 & \text{when } \delta\eta \quad 0\\ 0 & \text{when } \delta\eta \quad 0\end{cases}$$

$$a = \bar{a}(p/p_a)^n$$

K_p, K_q, G_p and G_q is called compression modulus, negative dilatancy modulus, compression hardening modulus and shear modulus respectively; κ is the bulk rebound index. There are altogether 7 parameters in the proposed model: λ, κ, c, m, $\bar{a}$, n and M, all of them can be determined from the isotropic consolidation test and drained triaxial compression test with p=const.

Using the proposed model and the values of above mentioned parameters determined with the artificially prepared samples, the pore pressures caused by negative dilatancy in the undrained tests are predicted and compared with the measured values. It is well known that in the undrained test of saturated clay, $\delta\varepsilon$=0 and δp=$-\delta u_q$. Then from eq.(4), we get

$$\delta u_q/p = cm\eta^{m-1}\delta\eta/\lambda \qquad (6)$$

This means that the relation between the normalized pore pressure and the stress ratio calculated with the proposed model is a power function. It agrees with the measured results as shown in Fig. 2.

ANALYSIS OF TWO CASE HISTORIES

A computer program for calculating the consolidation of soft clay foundation with finite element method is compiled, where the elastic or elasto-plastic theory is coupled with Biot's consolidation theory, while the space is discretized with finite element method and the time is discretized with finite difference method, and a set of algebraic equations with the nodal displacements and pore pressure increments as unknowns are finally established and solved to determine the stress, strain, displacement and pore pressure in the soft clay. This program has been used to analyze the soft clay foundation in two case histories: an oil tank and a seawall. The models used in the calculation are: Duncan-Chang's model, the elasto-plastic model proposed by the first author(Wei, 1981) and the model proposed in this paper. The negative dilatancy of soft clay can be taken into account in the elasto-plastic model as well as in the proposed model with the help of the initial stress method, while it can not be considered in Duncan-Chang's model. When the calculated result is compared with the measured value, the merits and demerits of these models can be seen obviously. The variation of pore pressure in the soft clay foundation during the process of water filling of an oil tank with 20,000 m^3 capacity is shown in Fig. 6(a), and the time history of pore pressure in the foundation of a seawall during construction is shown in Fig. 6(b). It can be seen from these figures that, when the model without consideration of the negative dilatancy is used, the calculated pore pressure is always smaller than the measured value, and the deviation is especially remarkable at places where the shear stress or the rotation of principal stress is great, such

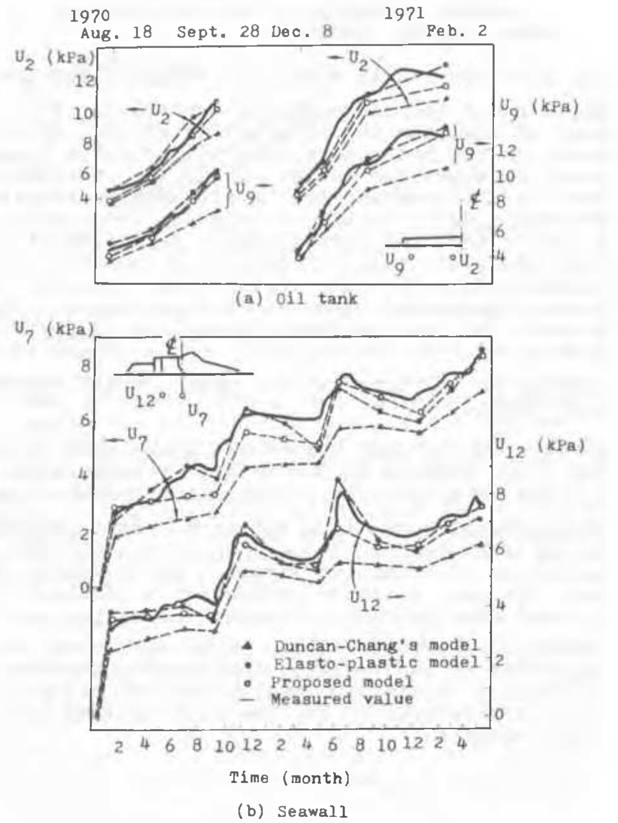

Fig. 6 Time histories of predicted and measured pore pressure.

as U_9 under the oil tank and U_{12} under the seawall, the maximum deviation amounts to 70% approximately. However, when the models with consideration of the negative dilatancy are used, the calculated pore pressures agree fairly well with the measured values. Hence, the importance of consideration of negative dilatancy in the calculation of pore pressures for the soft clay foundation is obvious therein.

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