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PORE PRESSURE CHARACTERISTICS IN ISOTROPIC CONSOLIDATED SATURATED CLAY UNDER UNLOADING CONDITIONS

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Key words: saturated soft clay, uplift, negative excess pore pressure, modified Cambridge model, triaxial extension test.

ABSTRACT

When foundations, such as caissons, spudcans, and mudmats, are pulled out from the saturated soft clay foundation, negative excess pore pressure will develop in the interface between the base and the soil. This phenomenon will significantly increase the uplift force. Therefore, estimating the developed negative excess pore pressure in the interface under unloading conditions is important. The development of excess pore pressure was investigated based on the triaxial extension test under isotropic consolidation condition. The method of using the modified Cambridge model to predict the excess pore pressure was programmed and has been proven to be feasible. By associating Henkel's theory in *p-q* space, the generation mechanism of negative pore pressure was revealed. The research indicates that excess pore pressure is initially negative and then becomes positive in triaxial extension test under isotropic consolidation. The generated excess pore pressure can be divided into two parts. The first part generated from the change in the mean total stress is negative, whereas the second part generated from the change in deviatoric stress is positive. However, these two parts are unbalanced. Research results provide theoretical basis for the development of excess pore pressure in soft clay to analyze the pull-out of structure and the excavation of foundation pit.

I. INTRODUCTION

After completing construction, certain offshore foundations must be decommissioned and removed from the seabed to comply with environmental regulations. In the removal procedure, suction develops at the interface between the foundation base and the underlying clay soil. In the extraction test of spudcans, the increase in base suction is established to be the key contributor for the larger breakout force (Purawana et al., 2005). On soft soil, removing mudmats during maintenance or decommissioning is difficult and costly because of the significant suction that develops at the mudmat–soil interface (Chen et al., 2012; Li et al., 2014). Suction is an important part of uplift capacity. Therefore, the suction generation mechanism and the calculation method for the uplift analysis of bottom-supported foundation in saturated soft clay are significant for offshore design guidelines.

The change of pore water pressure is determined by the external load. Skempton (1954) proposed the relationship between excess pore pressure and total pressure in the axisymmetric stress situation, and this theory is widely used in the engineering calculation. Pore pressure coefficient can be easily determined through triaxial test. However, on the one hand, pore pressure coefficient A is constantly changing as soil stress state varies, which is inconvenience in practical application. On the other hand, this theory disregarded the influence of the intermediate principal stress. Henkel (1966) proposed a pore pressure calculation method generalized to general three-dimensional stress state. This method has a few advantages, such as considering the shear dilatancy and definite physical meaning. Zhou (2002) presented a method to predict excess pore pressure in triaxial extension test using modified Cambridge model, but the negative pore pressure is ignored when analyzing pore pressure characteristics. Triaxial compression and tension tests were first modeled by Li et al. (2015) to develop a rigorous understanding of the pore pressure responses, but lack test evidence.

In the present study, the development of excess pore pressure was investigated based on the triaxial extension test in saturated clay under isotropic consolidation condition. The modified Cambridge model was used to simulate triaxial extension test to find a method to predict pore pressure. The generation mechanism of negative excess pore pressure was then analyzed based on Henkel's theory.

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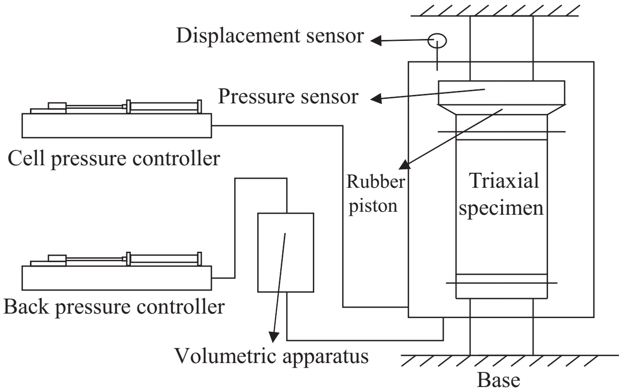

Fig. 1. Stress path triaxial apparatus.

Table 1. Parameters of Tianjin soft clay.

Parameter	Value
plastic index, I_p	23.7
Liquid limit, w_L /%	46.5
Specific gravity, Gs	2.73
Coefficient of consolidation cv/(m^2/year)	42.5
Coefficient of permeability, k/(m/s)	3.0 × 10-8
Void ratio at p_0 = 1 kPa on critical state line, e_0	1.22
Elasticity modulus E/MPa	11
Poisson's ratio, ν	0.3
Slope of normally consolidated line in $e - \ln p_0$ space, λ	0.15
Slope of swelling and recompression line in $e - \ln p_0$ space, κ	0.03
Slope of critical state line in p'-q space, M	0.658

II. TRIAXIAL EXTENSION TEST

According to the unloading path, triaxial extension test is divided into relief triaxial extension (RTE), common triaxial extension, and average stress p for constant triaxial extension (Zhang and Yan, 2004). When foundation is extracted, the bottom soil of foundation is in the unloading state in which cell pressure is constant and the vertical stress is reduced. The question of foundation extraction can be approximately equivalent to RTE.

1. Test Apparatus

Fig. 1. shows the sketch of the stress path triaxial apparatus, which is composed of a cell pressure controller, a back pressure controller, and a triaxial test chamber. The back pressure controller is connected to a volumetric apparatus that is used to measure volume deformation. The triaxial specimen is installed in the triaxial test chamber. The top of the triaxial specimen is fixed through the rubber piston connecting loading cap and loading rod, whereas the bottom can be activated by adjusting the rigidity base. In addition, the apparatus is equipped with the displacement sensor and pressure sensor on the upper portion of the chamber, which are used to measure the axial deformation and axial deviatoric stress.

2. Sample Properties

The soft clay used in this study was constituted from clay in Tianjin Binhai New Area. Undisturbed soil sample was coated by iron bucket and was then sealed with wax to prevent moisture loss. The properties of the soft clay have been determined by geo-technical testing, and are listed in Table 1.

3. Test Procedure

Isotropic consolidation pressures are 100, 200, and 300 kPa, and the test is subject to SL237-1999 Specification of Soil Test (China, 1999). The test steps are as follows:

1) Soil Sample Preparation

According to the Specification of Soil Test and the requirements of stress path triaxial apparatus, triaxial specimen size of 7 cm in diameter and 14 cm in height is determined. Undisturbed soil sample is cut into specified size, and the weight quality of triaxial specimen is used to measure the density.

2) Soil Sample Saturation

Saturation process is divided into two steps. The first step is vacuum saturation conducted in airtight container. Quality can be repeatedly weighed after this step to evaluate the uniformity of soil sample. Triaxial specimen is then installed in triaxial test chamber, and the method of back pressure saturation is adopted to fully saturate and detect saturability simultaneously. A 95% saturation is used as back pressure saturated termination condition.

3) Isotropic Consolidation

Cell pressure and back pressure are slowly applied on the triaxial specimen. These pressures are made equal to the consolidation pressure, also called the effective cell pressure. Under current stress state, saturated soil samples will be isotropically consolidated, while excess pore pressure dissipates over time. The degree of consolidation reaches 97%, which denotes the end of consolidation.

4) RTE

RTE is applied using strain control, and unloading shear strain rate is 0.05%/min. The test records are shown in Table 2.

III. THEORETICAL PREDICTION METHOD

Cambridge model is based on a large number of experiments in the normal consolidation of clay and weak over-consolidated clay, including isotropic consolidation and expansion test and the triaxial drained and undrained tests of different consolidation pressures. Roscoe and Burland (1968) have modified Cambridge model and is referred to as the modified Cam clay (denoted as MCC hereafter) constitutive model, in which yield surface is modified to oval. In the present study, MCC constitutive model was used to simulate RTE

Table 2. Test records.

Test number	Consolidation pressure P_0/kpa	Density γ/(kg · m^{-3})	Saturation S_r /%	Consolidation time T/h	Degree of consolidation U_t/%	Unloading rate v/(%/min）
1	100	18.7	99	24.7	97.9	0.05
2	200	18.6	98	26.0	99.1	0.05
3	300	18.8	100	25.1	98.6	0.05

and predict excess pore pressure. This task is conducted to solve the effective stress path such that pore pressure can be calculated according to the principle of effective stress. Completely undrained situation is assumed, $\sigma_2 = \sigma_3$; and the strain of each computing step in the direction of σ_1 is known.

1. Theoretical Arithmetic

The equation of Modified Cambridge model yield surface is

$$f\left(p',q,p_c\right)=\left(\frac{p'}{p_c}\right)^2+\left(\frac{q}{Mp_c}\right)^2-\frac{p'}{p_c}=0 \tag{1}$$

where p' = effective mean principal stress; q = generalized shear stress; p_c = hardening parameter; M = slope of critical state line, (= 6sinφ'/(3 ±sinφ'), where φ' = effective internal friction angle, the denominator of triaxial compression taking minus sign, take plus triaxial extension).

For brevity, p' and q are expressed as

$$p'=\delta_o p_c$$

$$q=\delta_o \eta p_c \tag{2}$$

where η = shear compression ratio, $\eta = q/p'$; δ_o = 1/2[1 + $(\eta/M)^2$].

Hardening rule is

$$p_c=\exp\left(\frac{1+e_o}{\lambda-\kappa}\varepsilon_v^p\right) \tag{3}$$

where ε_v^p = plastic volumetric strain; e_o = initial void ratio; λ = slope of normally consolidated line, and k = slope of swelling and recompression line.

If the associated flow rule is adopted, then the plastic potential function is equal to the yield surface function. The plastic potential flow theory is expressed as

$$d\varepsilon_v^p=d\lambda\frac{\partial f}{\partial p'} \tag{4}$$

$$d\varepsilon_s^p=d\lambda\frac{\partial f}{\partial q} \tag{5}$$

where ε_s^p = plastic shear strain. If $d\lambda$ is known, then plastic volumetric strain increment and plastic shear strain increment can be calculated.

Elastic deformation can be determined by the Hooke's law. The constitutive relation is expressed as

$$\begin{cases} dp'=K(d\varepsilon_v-d\varepsilon_v^p) \\ dq=3G(d\varepsilon_s-d\varepsilon_s^p) \end{cases} \tag{6}$$

where K = bulk modulus; G = shear modulus; ε_s = shearing strain.

Eq. (1) is formulated using total differentiation and is taken into Eq. (4). Therefore, $d\lambda$ can be calculated as

$$d\lambda=\frac{1}{H_p}\frac{\left(\frac{\partial f}{\partial p'}dp'+\frac{\partial f}{\partial q}dq\right)}{\left(\frac{\partial f}{\partial p'}\right)^2+\left(\frac{\partial f}{\partial q}\right)^2} \tag{7}$$

where

$$H_p=\frac{1+e_0}{\lambda-k}\frac{p_c}{2}\frac{1}{m^2}\left(\delta_o-\frac{1}{2}\right)\delta_o;$$

$$m=\sqrt{\left(\delta_o-\frac{1}{2}\right)^2+\eta^2\delta_o^{\,2}\left(\frac{1}{M}\right)^4}.$$

Take Eq. (7) into Eq. (4) and Eq. (5). Plastic body strain increment and the plastic shear strain increment can be expressed as

$$\begin{cases} d\varepsilon_v^p=\frac{1}{H_P}(dp'n_p+dqn_q)n_p \\ d\varepsilon_s^p=\frac{1}{H_P}(dp'n_p+dqn_q)n_q \end{cases} \tag{8}$$

where

$$n_p=\frac{\frac{\partial f}{\partial p'}}{\sqrt{\left(\frac{\partial f}{\partial p'}\right)^2+\left(\frac{\partial f}{\partial q}\right)^2}}=\frac{1}{m}\left(\delta_o-\frac{1}{2}\right);$$

$$n_q = \frac{\frac{\partial f}{\partial q}}{\sqrt{\left(\frac{\partial f}{\partial p'}\right)^2 + \left(\frac{\partial f}{\partial q}\right)^2}} = \frac{1}{m}\eta\delta_o\left(\frac{1}{M}\right)^2.$$

Take Eq. (8) into constitutive equation Eq. (6). Thus,

$$\begin{cases} dp' = \dfrac{3Gn_q d\varepsilon_s}{H_p + Kn_p^2 + 3Gn_q^2}(-Kn_p) \\ dq = 3G(d\varepsilon_s - \dfrac{3Gn_q d\varepsilon_s}{H_p + Kn_p^2 + 3Gn_q^2} n_q) \end{cases} \tag{9}$$

According to the principle of effective stress, pore pressure increment is expressed as

$$du = dp - dp' \tag{10}$$

where p = total mean stress and du = excess pore pressure increment.

2. Realization Process

Under completely undrained conditions, the total volume strain is zero. The shear strain increment is expressed as

$$\begin{cases} d\varepsilon_s = \dfrac{\sqrt{2}}{3}[(d\varepsilon_1 - d\varepsilon_2)^2 + (d\varepsilon_2 - d\varepsilon_3)^2 + (d\varepsilon_3 - d\varepsilon_1)^2]^{1/2} \\ d\varepsilon_1 + d\varepsilon_2 + d\varepsilon_3 = 0 \end{cases} \tag{11}$$

If $\sigma_2 = \sigma_3$, then $d\varepsilon_2 = d\varepsilon_3$. Therefore, $d\varepsilon_s = d\varepsilon_1$, which means the generalized shear strain increment is equal to the principal strain increment.

According to 2.1, excess pore pressure increment can be calculated, if each calculation step strain increment $d\varepsilon_s$ is known. The specific steps are as follows:

(1) Determine the Cambridge model parameters M, λ, k, and soil property parameters e_o, K, G;
(2) Assume an initial condition $q = 0$, $p' = p_0$ (the consolidation pressure noted by p_0), $d\varepsilon = 0$, total strain ε_s, total steps N;
(3) In every calculating step, the strain increment $d\varepsilon_s$ per step equals to ε_s/N. Then, dq, dp' can be calculated using Eq. (9);
(4) Isotropic consolidation soil is in the initial yield surface at first, and then the stress state under unloading is constantly changing along with yield surface affected by hardening rule. To ensure stress points on the yield surface in the entire calculation process, stress has been amended along the yield surface normal direction (Gao et al., 2010). Correction formula is as follows:

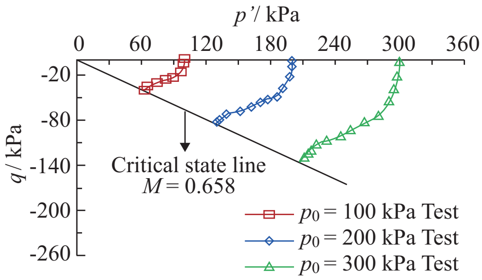

Fig. 2. Critical state of triaxial extension test.

$$\begin{cases} \delta p' = -\dfrac{\frac{\partial f}{\partial p'} f}{\left(\frac{\partial f}{\partial p'}\right)^2 + \left(\frac{\partial f}{\partial q}\right)^2} \\ \delta q = -\dfrac{\frac{\partial f}{\partial q} f}{\left(\frac{\partial f}{\partial p'}\right)^2 + \left(\frac{\partial f}{\partial q}\right)^2} \end{cases} \tag{12}$$

where $\delta p'$ = modified variable effective mean stress and δq = modified variable deviatoric stress.
(5) Considering the correction formula of stress, calculate pore pressure increment du, cumulative excess pore pressure u, deviatoric stress q, effective mean stress p', and total mean stress p;
(6) In every calculation step, conduct cycle steps (3) to (5) until $n = N$.

IV. COMPARISON OF TEST AND THEORY

According to triaxial extension test, the development of excess pore pressure under unloading condition was obtained. The modified Cambridge model was used to simulate triaxial tests and to confirm the feasibility of pore pressure prediction method. To easily distinguish the extension test and the compression test in the coordinate system, the generalized shear stress q in the compression test is represented in positive half shaft, whereas in negative half shaft in the extension test.

1. Stress Path

Three triaxial extension tests were consolidated in pressures 100, 200, and 300 kPa. Effective stress paths were plot in p-q space, in which the critical state line can be determined, as shown in Fig. 2. The critical state line slope is the Cambridge parameter M, which is 0.658.

Effective stress path of the test and theory are compared, as shown in Fig. 3. This figure shows that the values of both test and theory are very close. In other words, the two curves of effective stress path coincide basically. Shear stress predicted by modified Cambridge model is slightly lower than that of the

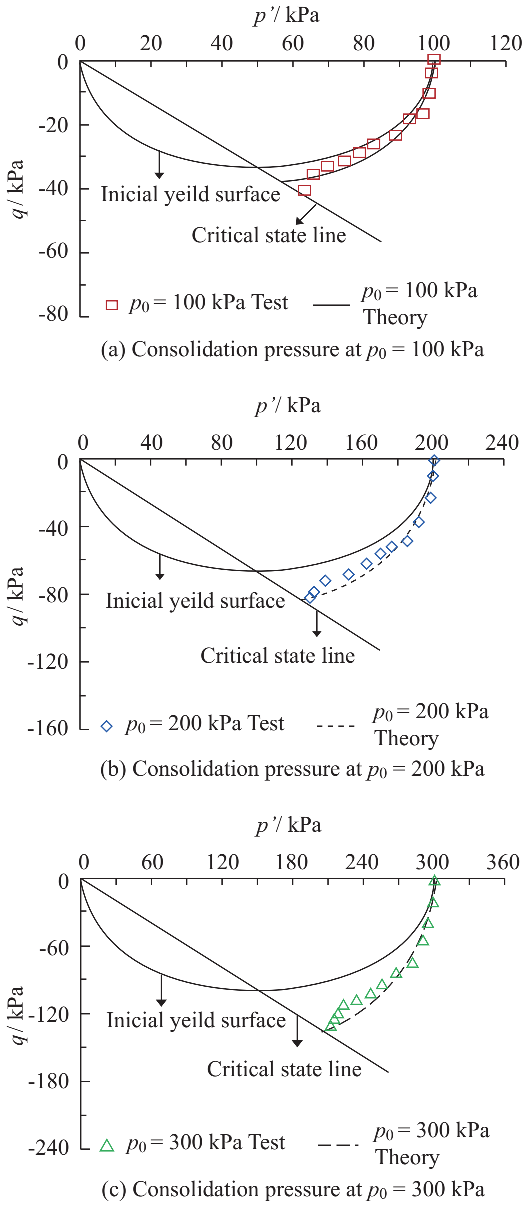

Fig. 3. Effective stress path test of triaxial extension test.

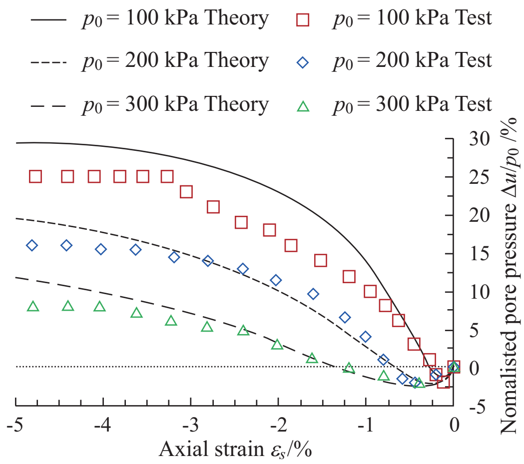

Fig. 4. Development of excess pore pressure.

test, but the error is within 4%. In theoretical prediction method, the excess pore pressure equals to the difference between total stress and effective stress. Hence, the closer the theoretical stress path compared with experimental, the more accurate the predicted excess pore pressure.

2. Development of Excess Pore Pressure

To facilitate the analysis of the development of pore pressure under different consolidation pressures, excess pore pressure was normally processed. The comparison of excess pore pressure between triaxial extension test and theoretical prediction with axial strain is shown in Fig. 4. The development of pore pressure is consistent and close in test and theory, and the predicted error is within 6.3%. Excess pore pressure is negative in the early unloading and increases with the development of the strain. The negative pore pressure increases to a maximum, and then decreases gradually and develops positive value after zero. Finally, the pore pressure achieves the positive maximum number and stops.

As shown in Fig. 4, under consolidation pressures of 100, 200, and 300 kPa, the maximum values of negative pore pressure are 2, 5, and 7.5 kPa in the test results, respectively. The maximum negative value increases with varying consolidation pressure. Meanwhile, its corresponding strain value also increases. Notably, the theoretical prediction method proposed in this study can also predict this rule.

The comparison of the test and theoretical calculation on stress path and the development of the pore pressure indicate that this prediction method is feasible in uplift question of foundation.

V. GENERATION MECHANISM OF PORE PRESSURE IN TENSION

Both test and theory results demonstrate that excess pore pressure is negative in the early unloading state, and then gradually becomes positive with the development of axial strain. Combining with the modified Cambridge model (Roscoe and Burland, 1968) and Henkel pore pressure theory (Henkel and Wade, 1966), this change rule and the generation mechanism of excess pore pressure in unloading state can be explained.

In p-q space, p and q can be noted by principal stress σ_1, σ_2, σ_3:

$$\begin{cases} p = (\sigma_1 + \sigma_2 + \sigma_3)/3 \\ q = \dfrac{1}{\sqrt{2}}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2} \end{cases} \tag{13}$$

When $\sigma_2 = \sigma_3$, the generalized shear stress is given by $q = |\sigma_1 - \sigma_3|$. When isotropic consolidation has been completed,

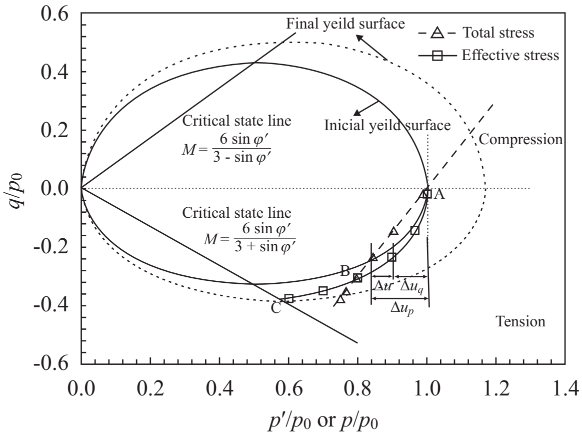

Fig. 5. Generation mechanism of pore pressure in tension state.

$$q = 3(p - p_0) \tag{14}$$

which means that $q = 0$ and $p = p_0$, axial unloading test is conducted in undrained conditions along with constant cell pressure. The total stress path can be expressed as

According to Henkel and Wade (1968), the generated excess pore pressures can be divided into two parts: the first part is generated from the change in the mean total stress, and the second part is generated from the change in deviatoric (or shear) stress. Henkel's theory is as follows:

$$\Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + \frac{a_0}{3}\Delta[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2} \tag{15}$$

Taking formula (13) into the formula (15) obtains

$$\Delta u = \Delta p + aq = \Delta u_p + \Delta u_q \tag{16}$$

where Δp = excess pore pressure generated from the change in the mean total stress, noted by Δu_p; $a\Delta q$ = excess pore pressure generated from deviatoric stress, noted by Δu_q.

Considering

$$\Delta u_p = p - p_0 \tag{17}$$

According to the principle of effective stress $\Delta u = \Delta p - \Delta p'$,

$$\Delta u_q = p_0 - p' \tag{18}$$

Fig. 5 presents the total and effective stress paths for triaxial compression and tension from the initial isotropic conditions. Point A denotes the stress state after the completion of isotropic consolidation, wherein the shear stress is zero. Point B is the intersection of effective stress path and total stress path, whereas point C is the intersection of effective stress path and the critical state line, that is, the damage state points. In *p-q* space, total stress and effective stress value difference is excess pore pressure, noted by Δu. The excess pore pressures generated from the change in the mean total stress (Δu_p) can be determined by Eq. (17), whereas that generated from the change in deviatoric (or shear) stress (Δu_q) can be calculated by Eq. (18). In tension, Δu_p is negative, whereas Δu_q is positive. Hence, these parts partially cancel each other. This cancellation results in the generation of negative excess pore pressures in section AB of tension ($|\Delta u_p| > \Delta u_q$), but positive excess pore pressures in section BC of tension ($|\Delta u_p| < \Delta u_q$). This observation suggests the generation mechanism of negative pore pressure and why excess pore pressure changes from negative to positive in this study.

VI. LIMITATIONS OF THE STUDY

Many Centrifuge model tests only obtain negative pore pressure under unloading state (Purawana et al., 2005; Chen et al., 2012; Li et al., 2014a; Liu et al., 2015). In this paper, the process of pore pressure existed from negative to positive. The present study focuses on the isotropic consolidation state, whereas the stress state of natural foundation is for K_0 consolidation. This phenomenon maybe influenced the difference of these tests. Several parameters including uplift velocity, loading history, and soil degree of consolidation influence the excess pore pressure variation (Gourvenec et al., 2009; Li et al., 2014b; Li et al., 2014c). However, a more advanced research is necessary to realistically simulate the stress state and to explore the influencing parameters of excess pore pressure.

VII. CONCLUSIONS

This paper presents the results from triaxial extension test at the initial isotropic conditions and focuses on the development of excess pore pressure with strain. Furthermore, modified Cambridge model was used to simulate RTE and to predict excess pore pressure. Hence, the feasibility of this prediction method has been confirmed through comparing theory prediction results and test results. Based on Henkel's theory, the generation mechanism of negative pore pressure was determined and the change of pore pressure from negative to positive was revealed. The following conclusions are drawn:

(a) In unloading condition, the shear stress generates positive pore pressure, whereas the contribution of the total mean stress increment to the excess pore pressure is negative. Therefore, the generation of negative excess pore pressure in the unloading condition is due to that these parts can incompletely cancel each other;
(b) Under isotropic conditions, excess pore pressure initially presents the negative and then becomes positive. The higher the consolidation pressure is, the greater the maximum negative pore pressure and corresponding axial strain. Meanwhile, modified Cambridge model can efficiently predict this rule.

This study has realized the development of excess pore pressure in isotropic condition and has validated the use of the MCC constitutive model. The research results provide theoretical basis for the development of excess pore pressure in saturated soft clay to analyze uplift of foundation and excavation of foundation pit. Further work is in progress both numerically and experimentally (i) to develop a more realistic consolidation ($k_0 \neq 1$) to investigate the effect to excess pore pressure, and (ii) to thoroughly explore the development of excess pore pressure with various influencing factors under unloading condition.

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