Pore Pressure Behavior of Normally Consolidated Deep Sea Clay

정규압밀된 심해점토의 간극수압 거동

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요 지

본 연구는 깊은 바다에 퇴적되어 있는 정규압밀된 점토 시료에 대한 등방 및 이방 압밀 비배수 삼축압축 시험을 시행하고, 압밀응력의 이방성이 간극수압 거동에 미치는 영향을 분석하였다. 등방압밀된 시료와 이방압밀된 시료의 비배수 전단시험 시의 간극수압은 모두 최대주변형인 축변형율의 쌍곡선 함수로 표시되었다. 그리고 두가지 시험의 유효응력 경로를 이용하여 간극수압의 차이를 해석하여 그 두 경우의 간극수압을 관련짓는 계수를 도출하였다.

Abstract

This paper presents triaxial test (CIUC and CK_0UC) results on normally consolidated deep sea clay samples. Based on the test results the pore pressure-strain relations for both isotropicaly and anisotropicaly consólidated samples are expressed with hyperbolic functions of the major princpal strain. The analysis of the difference in pore pressure behavior due to the anisotropy in consolidation stress is carried out with the effective stress pathes of CIUC and CK_0UC and finds a factor which correlates the pore pressure of two types of test.

INTRODUCTION

In fine grained soil under rapid shear excess pore water pressure developes due to its low permeability. And the constitutive behavior of such soil largely depends on the excess pore pressure during shear. That is the reason why many researchers have tried to develope the way of measuring and predicting shear induced pore pressure since the first measurement of pore pressure was performed by Rendulic in 1936 using triaxial test. In 1948, Skempton developed firstly an analytical approach to express the pore pessure in an axisymmetrical triaxial compression test

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sample using so called "$\lambda-$theory" which correlates pore pressure and principal stress changes with the compression ratio λ. Afterwards he modified the "$\lambda-$theory" and derived a new pore pressure function with the well known pore pressure parameters A and B. Many other researchers reported some pore pressure functions of applied stress changes. Such approaches are called "stress-theory" of pore pressure, which is being used widely to predict the excess pore pressure during undrained shear. However, the pore pressure parameters of the "stress-theory" functions are not constant but vary with strain. So these functions are not suitable to express the nonlinear pore pressure-strain relationship. Lo(1969a)[2] suggested the "strain-theory" of pore pressure which considers the shear induced pore pressure to be a function of major principal strain.

$$\Delta u_s / p = f(\varepsilon_1) \tag{1}$$

where, Δu_s is excess pore pressure due to shear, p is pressure for the use of equating the dimensions of both sides of the equation, the vertical consolidation pressure σ_{1c} is used as p in this paper. ε_1 is major principal strain. Lo(1969b)[3] reported some triaxial test results showing that pore pressure depends largely on axial strain. The author have reported the relation between the shear induced pore pressure and axial strain in CIUC test with a hyperbolic function based on the triaxial test results on some undisturbed and remoulded clays(Park, 1985)[4].

Natural clays typically exist under anisotropic in situ stress states. The pore pressure of anisotropically consolidated soil are to be predicted by CK_0UC triaxial test. CK_0UC test, however, is not easy to perform due to its time and economic problem. In this paper the pore pressure in CIUC and CK_0UC tests are compared and correlated using a few factors. The correlating factors can be used to predict the pore pressure in CK_0UC test sample with the conventional CIUC triaxial test results.

Triaxial Test

Soil Tested :

Tests are performed on undisturbed samples of normally consolidated illite rich clay taken from deep sea of Pacific Ocean. Thirty three percents by weight of the material is silt and sixty seven percents is clay. And the Atterberg limits and specific gravity are LL=88%, PI=39%, LI=1.2 and Gs=2.71 relatively.

Methodology and Procedures :

Tests are performed at the University of Rhode Island Marine Geomechanics Laboratory. Three samples are tested by isotropically consolidated undrained compression mode(CIUC) and the other three samples are tested by anisotropically consolidated undrained compression test mode (CK_0UC). The particular aspects of the apparatus at URI / MGL and the detailed procedures for the CK_0UC tests are described by Siciliano(1984)[5] and Zizza(1987)[7]. In short, the process of K_0 consolidation depends on a sensitive lateral strain gauge which is continually monitered by an electronic servomechanism. Upon consolidation, the decrease in the sample volume is detected by the lateral strain gauge that triggers the servomecanism which then activates a vertical motion actuator: increasing the axial stress until the sample's sides return to their original position. When the zero lateral strain is satisfied, the actuator is stopped. The lateral motion sensitivity of the lateral strain gauge is about 0.0014cm or strain of about 0.04% lateral strain (Zizza, 1987)[7]. In order to raise the reliability of the test results samples are back pressured by not less than 400 kPa for at least 24 hours before consolidation process starts, and pore pressures are measured with a pressure transducer. A strain rate of 0.003 cm / min are used during the shear phase.

Table 1. Triaxial Compression Test Data

Test model	Test No.	σ_{1c} (kPa)	K_0	at $(\sigma_1-\sigma_3)_{max}$					at $(\sigma_1/\sigma_3)_{max}$					
				ε_1(%)	$^*r_s$	$^{**}r_u$	A_f	ϕ°	ε_1(%)	$^*r_s$	$^{**}r_u$	σ_1'/σ_3'	A_f	ϕ°
CIUC	I−1	138	1.0	19.4	.802	.680	.85	33.8	19.0	.795	.689	3.555	.87	33.9
	I−2	209		19.4	.792	.673	.85		17.2	.792	.675	3.434	.85	
	I−3	279		13.4	.803	.689	.86		14.6	.803	.693	3.611	.86	
CK_0UC	A−1	155	.491	0.4	.728	.100	.14	28.2	8.0	.627	.242	3.524	.36	33.3
	A−2	194	.555	0.8	.740	.136	.18		7.0	.649	.278	3.349	.43	
	A−3	256	.534	0.4	.746	.106	.14		8.8	.676	.266	3.521	.39	

$^*r_s=(\sigma_1-\sigma_3)/\sigma_{1c}$, $^{**}r_u=\Delta u_s/\sigma_{1c}$

Test Results and Analysis

Test data are given in Table−1. All stress and pore pressure are normalized to vertical consolidation pressure σ_{1c}, then become dimensionless ratios.

$$r_s=(\sigma_1-\sigma_3)/\sigma_{1c} \tag{2}$$

$$r_u=\Delta u_s/\sigma_{1c} \tag{3}$$

where r_s is deviator stress ratio and r_u is pore pressure ratio. Curves in Fig. 1 are showing very good normalization on both for deviator stress and pore pressure. Effective stress pathes are plotted in q vs. p′ and σ_1' vs. σ_3' coordinate systems. In Fig. 2, all $(\sigma_1'/\sigma_3')_{max}$ points of the stress pathes construct a straight failure line, which means that the shear strengthes of CIUC and CK_0 UC test are same in the maximum principal stress ratio failure criterion.

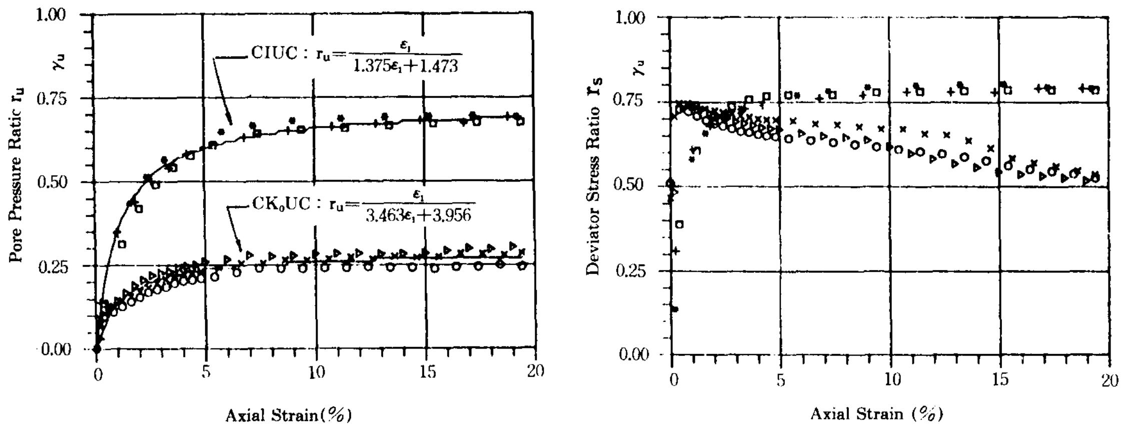
The pore pressure ratio-strain data plotted in Fig. 3 using ε_1/r_u vs. ε_1 coordinate system form a straight line which is expressed by a linear function as follows,

$$\varepsilon_1/r_u=a\varepsilon_1+b \tag{4}$$

And it can be transformed to,

$$r_u=\frac{\varepsilon_1}{a\varepsilon_1+b} \tag{5}$$

where, a and b are parameters for the function as shown in Fig. 4. It is shown in Fig. 3 that pore pressure-strain relations can be formulated by a hyperbola sucessfully. The parameters a and b of every curves are calculated by linear regression using least square method and given in Table−2. The value of 1 / a is the asymptote of the hyperbola and 1 / b is the slope coefficient of the initial tangent line at very small strain. The values in Table−2 show better normalizing in parameter a than b. And the values of r_u show that the asymptotes of the pore pressure hyperbolas are about 1.05 times larger than the ultimate values of $r_{u,\,ult}$. This hyperbolic function has been already being used for modeling stress-strain behavior since Kondner(1963)[1]. In Table−2 the hyperbolic parameters for stress-strain behavior are also given except CK_0UC case. Vaid(1985)[6] reported that the stress-strain curves of CK_0UC test can be formulated by the hyperbolic function, however, it is limited in the range of very small strain.



(a) Normalized stress-strain curves (b) Normalized pore pressure-strain curves

Fig. 1 Triaxial test result

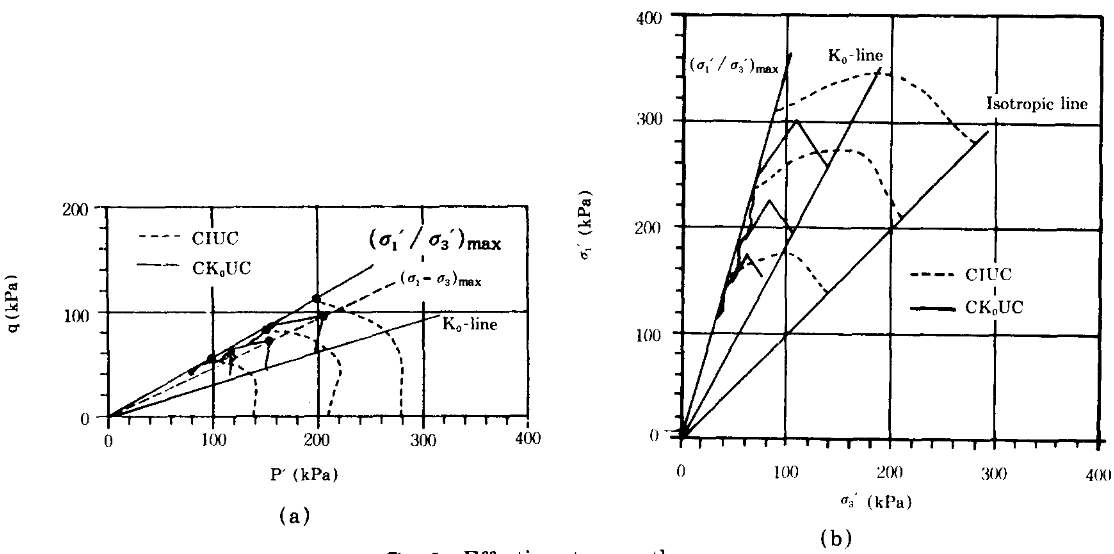


Fig. 2 Effective stress pathes

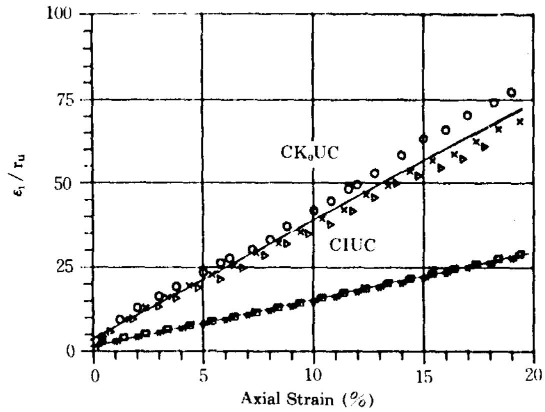
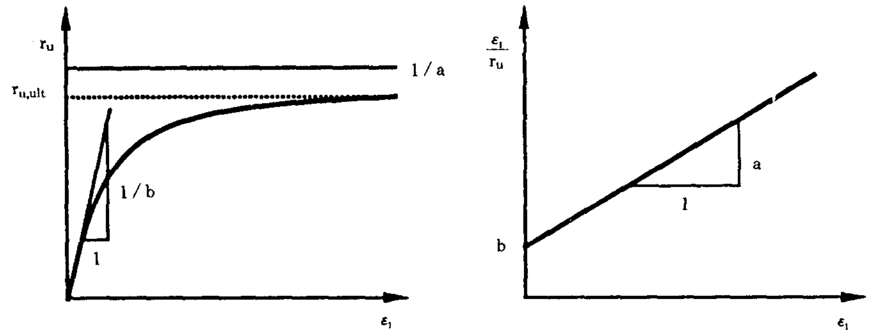


Fig. 3 Pore pressure-strain data in transformed hyperbola



(a) hyperbola (b) transformed hyperbola

Fig. 4 Hyperbolic model for pore pressure-strain behavior

Table 2. Parameters for Hyperbolic Model

Test No.	(1) r_u				(2) r_s			
	a_u	b_u	$r_{u,ult}$	*Ru	a_s	b_s	$r_{s,ult}$	**Rs
I−1	1.387	1.396	0.689	1.05	1.249	0.397	0.803	1.00
I−2	1.386	1.633	0.675	1.07	1.255	0.306	0.792	1.01
I−3	1.365	1.213	0.693	1.05	1.227	0.409	0.802	1.02
A−1	3.853	3.805	0.252	1.03	$^*Ru = (1 / a_u) / r_{u,ult}$			
A−2	3.202	3.506	0.302	1.03	$^{**}R_s = (1 / a_s) / r_{s,ult}$			
A−3	3.363	4.107	0.284	1.05				

(1) subscript u is for pore pressure.
(2) subscript s is for deviator stress.

Comparison of Pore Pressure from CIUC and CK_0UC tests

In general, CIUC tests are often used to characterize the pore pressure behavior. However, anisotropy in consolidation stress influences to the pore pressure behavior as shown in Fig. 1(b). To account for the effect of preshear in anisotropically consolidated soil Lo(1969, b)[3] suggested the use of the pore pressure ratio $(1-K)+r_{u,CK_0UC}$. However, the ratio is not zero at zero strain. In this paper in order to compare the pore pressure of both cases σ_1' vs. σ_3' effective stress pathes are used in Fig. 5. Two stress pathes of same vertical consolidation pressure visualize the difference in pore pressure due to preshear effect.

$$\rho=\frac{r_{u,CK_0UC}}{r_{u,CIUC}}=\frac{F'B'}{E'A'}=\frac{C'B'}{E'A'}+\frac{F'C'}{E'A'} \tag{6}$$

As points C′ and F′ are very close to each other, F′C′ / E′A′ can be thought to be zero, then,

$$\rho=\frac{C'B'}{E'A'}=\frac{C'B'}{(E'A'/C'A')C'A'} \tag{7}$$

where,

$$\begin{aligned}
&OA'=AA'=\sigma_{1c}\\
&OB'=K_0(BB')=K_0\sigma_{1c}\\
&OC'=CC'/(\sigma_1'/\sigma_3')_{max.}=\sigma_{1c}/(\sigma_1'/\sigma_3')_{max}\\
&E'A'=\Delta u_{sf,\,CIUC}\\
&C'A'=CA=CC''=EE''=(\sigma_1-\sigma_3)_{f,\,CIUC}\\
&C'B'=OB'-OC'=\sigma_{1c}[K_0-1/(\sigma_1'/\sigma_3')_{max.}]\\
&C'A'=OA'-OC'=\sigma_{1c}[1-1/(\sigma_1'/\sigma_3')_{max.}]
\end{aligned} \tag{8}$$

substituting eq. (8) in eq. (7) becomes,

$$\rho=\frac{K_0(\sigma_1'/\sigma_3')_{max.}-1}{A_f[(\sigma_1'/\sigma_3')_{max.}-1]} \tag{9}$$

where, A_f is the Skempton's pore pressure parameter at $(\sigma_1'/\sigma_3')_{max.}$ in CIUC test. And

$$(\sigma_1'/\sigma_3')_{max.}=(1+\phi')/(1-\phi') \tag{10}$$

where, ϕ' is the angle of shearing resistance from $(\sigma_1'/\sigma_3')_{max.}$ failure criterion. From eq. (9) and eq. (10),

$$\rho=\frac{\sin\phi'(1+K_0)-(1-K_0)}{2\,A_f\sin\phi'} \tag{11}$$

So, the pore pressure of CK_0UC test can be predicted from the CIUC test results as follows,

$$r_{u,CK_0UC}=\rho\cdot r_{u,CIUC} \tag{12}$$

In Fig. 6 predicted and measured pore pressures of a CK_0UC test are given to demonstrate that the conversion factor ρ works well.

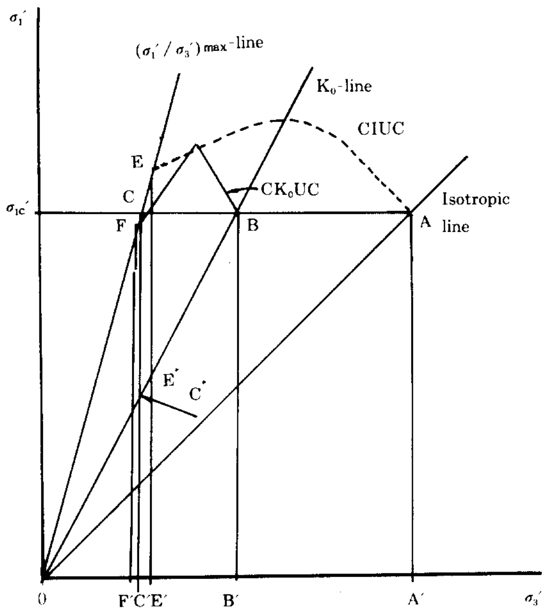


Fig. 5 Comparison of pore pressure behavior of CIUC and CK_0UC test with effective stress pathes.

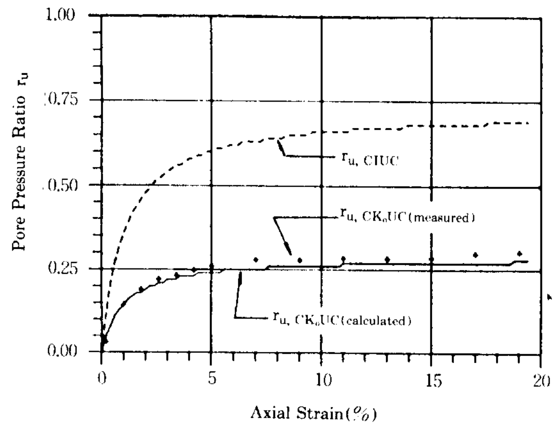


Fig. 6 Measured and calculated pore pressure of CK_0UC test.

Conclusions

This paper presents CIUC and CK_0UC triaxial tests on deep sea clay and expresses the shear induced excess pore pressure-axial strain relationship with a hyperbolic function. This hyperbolic function for CK_0UC test shows good expression of the test results as well as for CIUC test. Two parameters a and b of the hyperbolic function can be determined from the triaxial test results by linear regression.

This paper also presents a comparison of the pore pressure of CIUC and CK_0UC tests and suggests a way of estimating the pore pressure of CK_0UC test from the results of CIUC test. This estimation is done with three well known parameters A_f, ϕ' and K_0.

The understanding of the relations between the hyperbolic parameters and the mechanical properties of soil needs more tests and studies on various types of soil.

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