Jour. of the KGS, Vol. 21, No. 2. March 2005, pp. 5~15

Micro In-situ Tests on Overconsolidated Clay Prepared in Chambers

토조내에 준비된 과압밀 점토에 대한 모형 원위치 시험

Cho, Nam-Jun[1] 조 남 준

요 지

모형지반을 이용한 각종 모형말뚝실험에서 자주 문제가 되는 것이 크기효과에 따른 실험의 정량적 분석에 대한 한계성이다. 이러한 문제를 보완하기 위하여 본 연구에서는 대형의 원통형 토조에 인공지반을 제작함으로써 지반 특성에 대한 크기효과를 최소화하도록 노력하였다. 또한 지반의 자연생성과정을 재현하고 빠른 시간내에 압밀을 완료하기 위하여 케올리나이트와 실리카를 1:1로 혼합한 슬러리를 토조에서 압밀하여 인공지반을 생성하였다. 본 연구에서 마련한 모든 인공지반의 특성을 자세히 파악하고 실제의 지반특성과 비교하기 위하여 베인시험기, 피조프로브, 콘관입시험기 등 여러가지의 원위치 소형조사장비를 제작 및 사용하였다. 본 연구에서 행한 원위치 시험 결과, 앞선 연구에서 밝힌 시험결과와 일치하는 결과를 얻었다. 또한 인공지반에서 행한 모형실험결과와 실제 상황과의 차이를 규명함으로써 모형실험결과에 근거하여 실물의 거동을 예측하는 방법을 찾고자 하였다.

Abstract

In this study, model soil deposits are prepared in large test chambers to minimize the scale effects. Also, slurry of mixture containing 50 percent kaolin clay and 50 percent silica has been consolidated to simulate the process of natural soil deposit formation and to reduce the consolidation time. To provide a more detailed description of varying soil properties along the soil profile of model clay deposits and to compare the in-situ test results with those from prototype tests, miniature in-situ tests, including vane shear, piezoprobe, and cone penetration tests were conducted in each of the clay deposits. The current results indicate that consistent soil deposits were prepared for the current and previous test programs. Also, reasonable predicting methods of prototype behavior based on model in-situ test results were suggested in this study by examining differences between the test results from both the model and prototype tests.

Keywords : Miniature in-situ test, Model soil deposits, Prototype tests, Simulation

1. Introduction

Soil deposits were prepared in (a) a short tank, 1.35 m in diameter by 1.21 m high and (b) a tall tank, 1.35 m in diameter by 2.13 m high. The materials and procedures used in this study are similar to those used in former studies at Cornell University (McManus and Kulhawy, 1991; Mayne, et al., 1992).

There are three basic procedures for preparing laboratory clay deposits: (a) soaking, (b) compacting or hand packing technique, and (c) slurry consolidation. Both the soaking and compacting methods can result in a non-uniform deposit with unknown stress histories. Therefore, the slurry consolidation method was chosen for the

1 Member, Associate Prof., School of Civil & Environ. Eng., Kookmin Univ. (mjcho@kookmin.ac.kr)

preparation of 'Cornell Clay' deposits because it is the only process satisfying the required degree of uniformity, saturation, and control over stress history (McManus and Kulhawy, 1991).

Four tall and two short soil deposits were prepared for this experimental study. The preconsolidation stress was 48 kN/m^2 for all of the deposits. During consolidation, the piston displacement was measured to monitor the consolidation process. After completion of primary consolidation, the prestress was removed to allow the soil deposit to rebound for about three days for the short deposits and eight days for the tall deposits before the construction of the drilled shafts. Water contents and miniature vane tests were conducted for each drilled shaft during excavation. After the loading tests, each soil deposit was investigated by miniature cone and piezoprobe tests. The data from the water content, vane, cone, and piezoprobe tests were used to characterize and evaluate the quality and physical properties of each soil deposit. Also, the in-situ test results were compared with those from previous studies.

2. Material Selection

A mixture containing 50 percent kaolin clay and 50 percent silica was chosen for soil deposit preparation by considering several factors, such as (a) cost of material, (b) consistency, (c) gradation, (d) permeability, (e) consolidation time, (f) surcharge required for consolidation, and (g) compressibility of soil (McManus and Kulhawy, 1991). Based on the analysis of grain size distributions for the kaolin and silica mixture, the soil deposit for this study is categorized as a clayey silt in which approximately 33 percent of the particle sizes are clay colloids (smaller than 0.002 mm), 62 percent are silt size (between 0.074 mm and 0.002 mm), and the remaining 5 percent are fine sand (larger than 0.074 mm). Table 1 summarizes the index properties of clay deposits.

Table 1. Index properties of clay deposits

Property	Symbol	Value
Liquid limit	WL	33%
Plastic limit	WP	22%
Plasticity index	PI	11
Specific gravity	Gs	2.65
Percent fines (< #200 sieve)	F	95%
Clay fraction (< 0.002 mm)	CF	33.3%
Activity (PI/CF)	A	0.33

Source: Mayne and Kulhawy (1991), p.3-7

3. Soil Deposit Simulation

Natural soils have been formed mostly under anisotropic consolidation in which different effective principal stresses have been applied. This anisotropic consolidation is assumed commonly to be one-dimensional consolidation without horizontal strain, and is termed usually as consolidation under K_o conditions. Generally, the two horizontal orthogonal stresses are assumed to be equal under K_o conditions.

In the laboratory, these K_o conditions can be simulated by one-dimensional consolidation in fixed-wall chambers in which lateral soil strain is zero. Two major problems associated with soil deposit preparation in laboratory are: (a) there may exist some difficulties in increasing the overburden stresses and (b) the prepared soil deposit may have an extraordinarily high overconsolidation ratio (OCR) profile. The problem associated with the low overburden stresses can be solved partially by prestressing the clay by using high fluid pressure. McManus and Kulhawy (1991) could apply an effective consolidation stress of 110 kN/m^2, which is approximately equivalent to an effective stress under about 5 m high rock.

Some researchers (e.g., Rowe and Barden, 1966; Sulaiman, 1985; McManus and Kulhawy, 1991) have tried to simulate low OCR profile without major increases in soil deposit size. However, some difficulties were encountered in maintaining a surcharge stress during either constructing shafts or test loading. Mayne, et al. (1992) suggested to use large chambers and model shafts to minimize the problem of high OCR profile.

In this study, the clay deposits were prepared in three main procedures; (a) slurry mixing, (b) consolidation, and (c) rebound. Soft and saturated slurry, which was obtained by using a slurry mixing system, was pumped into large fixed-wall chambers. The slurry was consolidated and brought to a soft clay deposit. Subsequently, the clay deposit was allowed to rebound by removal of the overburden stress. These three phases may be currently the most desirable simulation of clay deposits and could provide advantageous soil characteristics, such as homogeneity, uniformity, saturation, consolidation under K_o conditions, and proper strength (Mayne, et al., 1992).

4. Consolidation Results

Four tall and two short soil deposits were prepared for this experimental study. The preconsolidation stress was 48 kN/m^2 for all of the deposits. During consolidation, the piston displacement was measured to monitor the consolidation process. After completion of primary consolidation, the prestress was removed to allow the soil deposit to rebound for about three days for the short deposits and eight days for the tall deposits. Water contents and miniature vane tests were conducted for each drilled shaft during excavation. After the loading tests, each soil deposit was investigated by miniature cone and piezoprobe tests. The data from the water content, vane, cone, and piezoprobe tests were used to characterize and evaluate the quality and physical properties of each soil deposit. Also, the in-situ test results were compared with those from previous studies.

The difference in slurry depth resulted in different consolidation times, and possibly resulted in a different aging effect. However, this possible effect was assumed to be negligible in this study. The coefficient of secondary compression obtained from previous experimental work ($C_{\alpha\varepsilon}$ = 0.0035) showed that Cornell Clay has very low to low secondary compressibility, which implies that the effect of secondary consolidation on soil structure or engineering soil properties would be minor (Mesri, 1973).

The primary consolidation time was predicted using the consolidation coefficient (c_v) given previously (McManus and Kulhawy, 1991), which decreases from 0.2×10^{-6} to 2.0×10^{-6} m^2/sec as the water content increases from 25 to 50 percent. A representative value of c_v during the whole consolidation period was assumed to be 0.4×10^{-6} m^2/sec, because the initial and final coefficients of consolidation were about 0.1×10^{-6} and 0.7×10^{-6} m^2/sec at the corresponding water contents of about 60 and 37.2 percent, respectively. The coefficients of consolidation also were back-calculated using the log time method (Casagrande's method) and the square root of time method (Taylor's method). Table 2 summarizes the consolidation data.

5. In-situ Miniature Tests

To provide a more detailed description of varying soil properties along the soil profile of Cornell clay, miniature in-situ tests, including vane shear, piezoprobe, and cone penetration tests were conducted in each of the clay deposits. The results of these tests are given here to illustrate the in-situ characteristics of model clay deposits.

Table 2. Consolidation time for soil deposits

Deposit No.	Thickness (mm)	Consolidation Time (min)		Coefficient of Consolidation, c_v (m^2/sec)		
		Predicted[a]	Measured	Assumed[b]	Calculated	
					C[c]	T[d]
S1	940	9204	6530	0.4×10^{-6}	0.67×10^{-6}	0.76×10^{-6}
S2	990	10416	12022	0.4×10^{-6}	0.45×10^{-6}	0.68×10^{-6}
T1	1940	39204	30903	0.4×10^{-6}	0.35×10^{-6}	0.38×10^{-6}
T2	1880	36817	20184	0.4×10^{-6}	0.47×10^{-6}	0.58×10^{-6}
T3	1860	36038	19055	0.4×10^{-6}	0.54×10^{-6}	0.62×10^{-6}
T4	1900	37604	27542	0.4×10^{-6}	0.48×10^{-6}	0.53×10^{-6}

a – $t = H^2/c_v$ in which H = half of soil deposit thickness and $c_v = 4.0\times10^{-6}$ m^2/sec
b – assumed representative c_v of whole consolidation period
c – calculated by Casagrande's method
d – calculated by Taylor's method

The equipment and test procedures of vane and cone penetration tests conducted in this study are exactly the same as those in a previous study (Mayne and Kulhawy, 1991). Also, the test results from both the current and previous studies are combined and presented together in this appendix to have a larger and more reliable test database. Since the applied prestress for preparing soil deposits throughout this testing program was constant ($\bar{\sigma}_p$ = 48 kN/m^2), it was necessary to identify the current soil deposits by referring to the early soil deposits which had been prepared using various prestress conditions.

5.1 Water Contents and Vane Shear Tests

Generally, three to six water content and vane shear tests were conducted for each shaft, depending upon the specific design depth, at approximately 90 to 180 mm intervals along the vertical soil profile during excavation of all shaft holes. The number of test sets per hole and intervals were related to the ratio of depth to diameter (D/B). To minimize the effect of delayed concreting time, longer test intervals were selected for deeper shafts.

The average water content and vane strength from 142 test sets were 37.2 percent (S.D. = 1.17 percent) and 8.1 kN/m^2 (1.2 psi) (S.D. = 1.77 kN/m^2 = 0.3 psi), respectively. The coefficient of variation of water contents obtained from this study, which is about 3 percent, indicates relatively consistent and uniform conditions of soil deposits throughout the whole test program. Note the mean value of coefficients of variation from typical site investigations is about 21 percent (Spry, et al., 1986). The observed small variations of water content were caused possibly by several factors (Mayne, et al., 1992): (a) errors in the w_n measurements, (b) increase in effective preconsolidation stress ($\bar{\sigma}_p$) with depth caused by overburden stresses, and (c) incomplete removal of free surface water before excavation.

The vane shear tests are usually used for the in-situ determination of the undrained strength of intact, fully saturated clays. The equipment used in this study consists of a stainless vane of four thin rectangular blades with 6.35 mm in width (d) and 25.4 mm in height (H).

During the excavation for the drilled shafts, vane shear tests (VST) followed by soil sampling for water content determinations were made at about 90 mm to 180 mm intervals from upper to lower levels of the hole. Torque measurement was conducted using a Wykeham-Farrance apparatus with calibrated springs. Shearing occurred at a rate of 0.1 degree per second, and the tests were performed according to ASTM D2573.

The vane apparatus was mounted on the same frame as used for guiding the hand auger to maintain vertical alignment during testing. The frame was positioned and clamped to the load reaction frame by C-clamps. The undrained shear strength was measured by reading calibrated dials at the top of the apparatus. It was not necessary to correct the measurement for the rod friction because the vane tests were conducted at the bottom of the excavated holes. Vane strengths were calculated using standard formulae given in ASTM D2573. For a standard vane with H/B = 2, the undrained shear strength in the unit of kN/m^2 can be calculated by:

$$s_{uv} = 6T/7\pi B^3 = 0.273\ T/B^3 \qquad (1)$$

in which B = 2d = vane diameter (m), and T = maximum measured torque (kN-m). Table 3 lists the test depths, vane shear strengths, and water contents determined for each of the twenty-seven excavated holes in six soil deposits. Also, the effective vertical stress ($\bar{\sigma}_{vo}$), preconsolidation stress ($\bar{\sigma}_p$), and overconsolidation ratio (OCR = $\bar{\sigma}_p/\bar{\sigma}_{vo}$) at each of the test depths are given in Table 3.

The undrained shear strength of clay (s_u) and the water content (w_n), including those from the current and previous study (Mayne and Kulhawy, 1991), are related, as shown in Fig. 1. Regression analysis of these vane shear strength and water content data (n = number of data = 257) is shown in Fig. 1. The statistic analysis showed that the relationship has a coefficient of determination (r^2) of 0.614 and a standard deviation of the water content of 1.61 percent. Alternatively, the relationship is usually rearranged to express the undrained shear strength (s_u) as an exponential function of water content (w_n), as given by:

$$s_{uv} = \exp(8.79 - 0.183\ w_n) \qquad (2)$$

Table 3. Properties of completed clay deposits

Test Series	s_{uvm} (kN/m^2)	Depth (mm)	s_{uv} (kN/m^2)	w_n (%)	$\bar{\sigma}_{vo}$ (kN/m^2)	$\bar{\sigma}_p$ (kN/m^2)	OCR	$s_u/\bar{\sigma}_{vo}$
T1-1	8.24	50.0	8.0	36.7	0.4	48.4	118.0	19.5
		230.0	5.8	38.0	1.9	49.9	26.9	3.1
		320.0	6.5	38.5	2.6	50.6	19.7	2.5
		410.0	8.4	38.3	3.3	51.3	15.6	2.6
		500.0	12.5	33.9	4.3	52.3	12.3	2.9
T1-2	6.30	50.0	6.8	37.0	0.4	48.4	118.6	16.6
		130.0	6.1	37.4	1.1	49.1	46.5	5.8
		160.0	6.0	37.3	1.3	49.3	37.9	4.6
		260.0	6.4	37.3	2.1	50.1	23.7	3.0
T1-3	7.07	50.0	6.5	37.5	0.4	48.4	119.3	16.0
		140.0	6.6	37.7	1.1	49.1	43.4	5.9
		230.0	6.8	38.2	1.8	49.8	27.0	3.7
		400.0	6.1	38.7	3.2	51.2	16.0	1.9
		490.0	9.3	34.4	4.1	52.1	12.6	2.3
T1-4	6.46	50.0	6.9	37.1	0.4	48.4	118.7	16.9
		130.0	6.6	37.5	1.1	49.1	46.5	6.3
		230.0	5.8	38.4	1.8	49.8	27.0	3.2
T1-5	7.27	50.0	7.2	36.9	0.4	48.4	118.4	17.6
		140.0	7.3	37.5	1.1	49.1	43.3	6.4
		230.0	7.3	38.2	1.8	49.8	27.0	4.0
T1-6	8.25	50.0	8.2	37.0	0.4	48.4	118.6	20.1
		140.0	8.2	37.7	1.1	49.1	43.4	7.2
		230.0	6.8	38.3	1.8	49.8	27.0	3.7
		320.0	6.9	38.0	2.6	50.6	19.6	2.7
		410.0	7.4	38.1	3.3	51.3	15.5	2.2
		500.0	12.0	33.6	4.3	52.3	12.2	2.8
S1-1	8.58	50.0	8.5	36.8	0.4	48.4	118.2	20.8
		140.0	8.5	37.1	1.1	49.1	43.0	7.5
		230.0	8.7	37.0	1.9	49.9	26.5	4.6
S1-2	9.12	50.0	8.6	36.8	0.4	48.4	118.3	21.0
		140.0	8.5	36.5	1.2	49.2	42.7	7.4
		230.0	9.1	36.0	1.9	49.9	26.2	4.8
		320.0	8.6	36.8	2.6	50.6	19.3	3.3
		410.0	9.1	36.5	3.4	51.4	15.2	2.7
		500.0	10.8	36.0	4.1	52.1	12.6	2.6
S1-3	8.46	50.0	8.1	37.3	0.4	48.4	119.1	19.8
		120.0	8.9	36.7	1.0	49.0	49.8	9.0
S1-6	8.84	50.0	8.9	36.5	0.4	48.4	117.8	21.8
		140.0	8.7	37.0	1.1	49.1	43.0	7.6
T2-1	10.05	50.0	10.0	36.3	0.4	48.4	117.5	24.3
		140.0	9.8	36.0	1.2	49.2	42.4	8.4
		230.0	10.8	36.0	1.9	49.9	26.2	5.7
		320.0	10.8	36.2	2.6	50.6	19.2	4.1
		410.0	9.0	35.8	3.4	51.4	15.1	2.6
		500.0	9.2	35.9	4.1	52.1	12.6	2.2
		590.0	11.4	36.4	4.9	52.9	10.9	2.4
		680.0	9.4	36.0	5.6	53.6	9.5	1.7
		770.0	10.0	35.9	6.4	54.4	8.5	1.6

Table 3. Properties of completed clay deposits (Continued)

Test Series	s_{uvm} (kN/m^2)	Depth (mm)	s_{uv} (kN/m^2)	w_n (%)	$\bar{\sigma}_{vo}$ (kN/m^2)	$\bar{\sigma}_p$ (kN/m^2)	OCR	$s_u/\bar{\sigma}_{vo}$
T2-2	10.04	50.0	9.0	36.4	0.4	48.4	117.6	21.9
		140.0	9.2	36.0	1.2	49.2	42.4	7.9
		320.0	9.6	36.2	2.6	50.6	19.2	3.6
		590.0	10.3	36.2	4.9	52.9	10.9	2.1
		770.0	12.1	36.0	6.4	54.4	8.5	1.9
T2-3	10.78	50.0	9.1	36.7	0.4	48.4	118.2	22.1
		230.0	10.2	36.4	1.9	49.9	26.4	5.4
		410.0	12.9	36.1	3.4	51.4	15.2	3.8
		590.0	11.1	36.1	4.9	52.9	10.8	2.3
		770.0	10.7	35.9	6.4	54.4	8.5	1.7
T2-4	8.67	50.0	8.1	36.4	0.4	48.4	117.7	19.8
		230.0	8.8	36.2	1.9	49.9	26.3	4.6
		410.0	5.9	36.1	3.4	51.4	15.2	1.7
		770.0	11.9	35.8	6.4	54.4	8.5	1.9
T2-5	11.17	50.0	9.7	37.0	0.4	48.4	118.5	23.7
		250.0	11.0	36.4	2.1	50.1	24.3	5.3
		410.0	11.2	35.7	3.4	51.4	15.1	3.3
		680.0	11.4	36.0	5.6	53.6	9.5	2.0
		770.0	12.6	35.9	6.4	54.4	8.5	2.0
T2-6	10.82	50.0	9.6	37.2	0.4	48.4	118.8	23.6
		230.0	9.9	35.9	1.9	49.9	26.2	5.2
		410.0	12.2	36.2	3.4	51.4	15.2	3.6
		590.0	11.6	36.4	4.9	52.9	10.9	2.4
T3-1	6.57	50.0	6.8	38.5	0.4	48.4	120.9	16.9
		320.0	5.7	39.4	2.5	50.5	20.0	2.2
		410.0	6.4	38.9	3.3	51.3	15.7	2.0
		590.0	6.2	38.7	4.7	52.7	11.2	1.3
		770.0	7.7	38.0	6.2	54.2	8.7	1.2
T3-2	6.16	50.0	7.2	38.8	0.4	48.4	121.5	18.2
		230.0	5.8	39.5	1.8	49.8	27.4	3.2
		410.0	5.5	39.6	3.2	51.2	15.8	1.7
T3-3	5.74	50.0	6.3	38.5	0.4	48.4	120.9	15.7
		320.0	5.4	39.2	2.5	50.5	19.9	2.1
		410.0	5.5	39.3	3.2	51.2	15.8	1.7
T3-4	6.35	50.0	6.8	39.3	0.4	48.4	122.2	17.1
		410.0	6.5	39.4	3.2	51.2	15.8	2.0
		590.0	6.4	38.7	4.7	52.7	11.2	1.4
		770.0	5.8	38.8	6.1	54.1	8.8	0.9
T3-5	6.11	50.0	7.2	38.8	0.4	48.4	121.4	18.2
		230.0	5.5	39.6	1.8	49.8	27.5	3.0
		410.0	6.3	39.8	3.2	51.2	15.9	2.0
		590.0	5.4	38.5	4.7	52.7	11.2	1.1
T3-6	6.37	50.0	7.0	38.5	0.4	48.4	120.9	17.6
		230.0	6.0	39.8	1.8	49.8	27.5	3.3
		590.0	6.1	38.5	4.7	52.7	11.2	1.3
T3-7	6.94	50.0	7.1	38.5	0.4	48.4	120.9	17.8
		230.0	6.8	39.3	1.8	49.8	27.4	3.7

Table 3. Properties of completed clay deposits (Continued)

Test Series	s_{uvm} (kN/m^2)	Depth (mm)	s_{uv} (kN/m^2)	w_n (%)	$\bar{\sigma}_{vo}$ (kN/m^2)	$\bar{\sigma}_p$ (kN/m^2)	OCR	$s_u/\bar{\sigma}_{vo}$
S2-1	8.47	50.0	8.5	36.9	0.4	48.4	118.4	20.9
		150.0	8.1	36.6	1.2	49.2	40.0	6.5
		260.0	8.8	36.3	2.1	50.1	23.4	4.1
S2-2	8.01	50.0	8.3	37.1	0.4	48.4	118.8	20.4
		220.0	7.9	37.0	1.8	49.8	27.7	4.4
		320.0	8.3	36.4	2.6	50.6	19.2	3.1
		450.0	7.6	36.6	3.7	51.7	14.0	2.1
S2-3	8.01	50.0	7.9	36.9	0.4	48.4	118.4	19.2
		140.0	8.3	36.9	1.1	49.1	42.9	7.3
		230.0	7.9	36.8	1.9	49.9	26.5	4.2
S2-4	8.13	50.0	7.9	37.8	0.4	48.4	119.8	19.6
		230.0	8.3	37.0	1.9	49.9	26.6	4.4
		360.0	8.5	36.8	2.9	50.9	17.3	2.9
		460.0	7.8	37.0	3.8	51.8	13.8	2.1
S2-5	8.26	50.0	8.1	37.2	0.4	48.4	118.9	20.0
		140.0	7.9	37.2	1.1	49.1	43.1	6.9
		230.0	8.8	37.0	1.9	49.9	26.6	4.7
S2-6	8.48	50.0	8.7	37.3	0.4	48.4	119.0	21.3
		190.0	8.4	37.4	1.5	49.5	32.1	5.4
		320.0	8.9	36.9	2.6	50.6	19.3	3.4
		450.0	8.0	36.8	3.7	51.7	14.0	2.2
S2-7	9.01	50.0	8.1	37.1	0.4	48.4	118.8	19.9
		130.0	8.6	37.0	1.1	49.1	46.2	8.1
		230.0	10.3	36.7	1.9	49.9	26.5	5.5
T4-1	7.31	50.0	8.8	36.1	0.4	48.4	117.1	21.3
		230.0	7.3	37.4	1.9	49.9	26.7	3.9
		350.0	6.5	38.3	2.8	50.8	18.1	2.3
		470.0	6.6	37.9	3.8	51.8	13.7	1.8
T4-2	8.08	50.0	9.3	36.3	0.4	48.4	117.5	22.7
		330.0	7.0	37.8	2.7	50.7	19.0	2.6
		580.0	7.9	38.0	4.7	52.7	11.3	1.7
T4-3	7.59	50.0	9.0	35.8	0.4	48.4	116.6	21.7
		320.0	6.7	37.7	2.6	50.6	19.5	2.6
		570.0	7.1	38.1	4.6	52.6	11.5	1.6
		730.0	7.5	37.8	5.9	53.9	9.1	1.3
T4-4	6.67	50.0	8.5	35.8	0.4	48.4	116.7	20.6
		320.0	6.4	38.0	2.6	50.6	19.6	2.5
		500.0	5.8	38.4	4.0	52.0	13.0	1.5
		590.0	6.0	37.7	4.8	52.8	11.1	1.2
T4-5	7.23	50.0	8.7	35.9	0.4	48.4	116.8	20.9
		320.0	7.2	37.9	2.6	50.6	19.6	2.8
		580.0	6.3	38.6	4.6	52.6	11.4	1.4
		700.0	6.8	38.7	5.6	53.6	9.6	1.2
T4-6	7.03	50.0	8.5	35.9	0.4	48.4	116.9	20.4
		330.0	6.6	37.9	2.7	50.7	19.0	2.5
		580.0	6.5	38.1	4.7	52.7	11.3	1.4
		720.0	6.6	38.1	5.8	53.8	9.3	1.1

Fig. 2 shows the relationship between s_{uv} and $\bar{\sigma}_p$. Least square regression analysis of these data with a forced intercept through the origin gives (r^2 = 0.601):

$$s_{uv} = 0.157\ \bar{\sigma}_p \tag{3}$$

Mayne and Kulhawy (1991) showed that this linear relationship between the undrained vane shear strength and preconsolidation pressure is consistent with Chandler's state-of-the-art review of the VST (1987). Also, Chandler (1987) suggested that the general relationship between the undrained vane shear strength (s_{uv}) and the plasticity index (PI) of NC clays developed by Skempton (1957) may be valid for OC clays. Therefore, the preconsolidation ($\bar{\sigma}_p$) can replace the current effective vertical stress ($\bar{\sigma}_{vo}$) in Skempton's equation as follows:

$$s_{uv}/\bar{\sigma}_p = 0.11 + 0.0037\ \mathrm{PI} \tag{4}$$

The value of $s_{uv}/\bar{\sigma}_p$ was 0.151 for the plasticity index (PI) of Cornell clay is about 11. The accuracy of the estimated value is on the order of 4 percent, which is inside the boundary (± 25 percent) predicted by Chandler.

The ratio of undrained strength to vertical effective stress ($s_{uv}/\bar{\sigma}_{vo}$) varies with OCR. Many researchers (e.g., Mayne and Mitchel (1987), Chandler (1987), Mesri (1988), etc.) have found that $s_{uv}/\bar{\sigma}_{vo}$ varies linearly or almost linearly with OCR. For Cornell clay, nonlinear regression gives (r^2 = 0.949) an almost linear relationship between $s_{uv}/\bar{\sigma}_{vo}$ and OCR as follows:

$$s_{uv}/\bar{\sigma}_{vo} = 0.158\ \mathrm{OCR}^{1.00} \tag{5}$$

Mayne and Mitchell (1988) also showed a linear relationship between $s_{uv}/\bar{\sigma}_{vo}$ and OCR by analyzing 263 individual data points from 96 different clays. In their linear regression analysis, they used a constant term of the OCR adjustment coefficient (α_{FV}), which is a ratio of OCR to $s_{uv}/\bar{\sigma}_{vo}$, and showed that general values of α_{FV} range from 1.8 to 20. In this study, α_{FV} is 6.33 (= 1/0.158), which is in the range of their findings. Hansbo (1957) proposed a correlation between α_{FV} and plasticity index (PI) as follows:

$$\alpha_{FV} = 22\ (\mathrm{PI})^{-0.48} \tag{6}$$

For Cornell clay, the PI = 11, and the estimated value of α_{FV} from Hansbo's correlation is 6.96, which is close to the α_{FV} obtained in this study. Also, Larsson (1980) alternatively expressed the α_{FV}, by adding more data to Hansbo's database, as follows:

$$\alpha_{FV} = 1/(0.08 + 0.0055\ \mathrm{PI}) \tag{7}$$

The estimated value of α_{FV} from Equation 7 for Cornell clay is 7.12.

Bjerrum (1972) introduced a correction factor (μ) with which a vane shear strength should be corrected before it is used in stability analysis of embankments and foundations in clay. This correction factor (μ) decreases as the plasticity index (PI) increases. He also noted that the correction of vane shear strength only removes a partial uncertainties involved in the applied testing method.

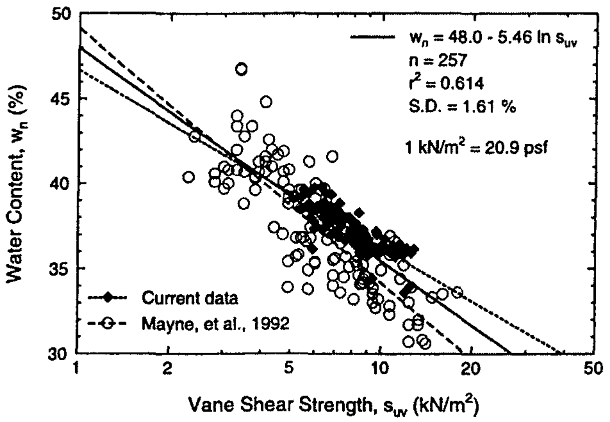

Fig. 1. Water content versus vane shear strength

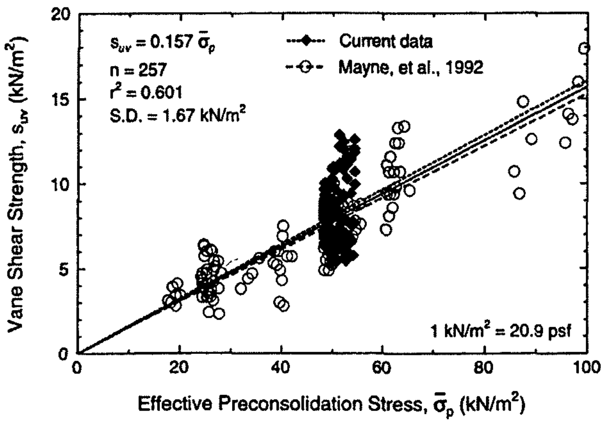

Fig. 2. Vane shear strength versus effective preconsolidation stress

For Cornell clay, the correction factor (μ) can be assumed to be a unit using his suggested correction analysis.

5.2 Cone Penetration and Piezoprobe Tests

Continuous measurements of the tip resistance (q_c) and excess pore water pressure (u_t or u_{bt}) are attractive of their capability to quantify soil properties and stress history. For this study, a miniature electric cone penetrometer developed by Sweeney (1987) and two types of piezoprobes built by Mayne and Kulhawy (1991) were used.

The electric cone penetrometer has a 60 degree apex and a 23.3 mm diameter, giving a projected tip area of 426 mm^2, while the standard cone has a tip area of 1000 mm^2. The miniature cone was used to provide measurements of cone tip resistance (q_c) during penetration into the clay.

Piezoprobe soundings were conducted using brass cones with 60 degree apex angles and 19 mm diameter that were fitted with pore water stress transducers. The first piezocone was designed to measure pore water stresses at the cone tip or face (u_t) to provide optimal profiling. The other one was designed to monitor pore-water stresses on the shaft of the cone, just behind the tip (u_{bt}) for the correction of the measured cone tip resistance (q_c). Both piezoprobes were machined from solid brass rods. The porous elements were made of sintered brass and attached to the cones. To ensure proper de-airing of the filters, the piezoprobe tips were boiled in water at least thirty minutes prior to testing. After de-airing, the piezoprobes were assembled under water to maintain saturation of the porous elements.

The cone penetration tests (CPTs) are able to measure the cone resistance (q_c) and optionally sleeve friction behind the cone (f_s). From the measurements of tip resistance (q_c), the undrained strength can be calculated using the plane-strain bearing capacity solution, cavity expansion theory, or steady penetration approach (Mohsen, et al., 1980).

Piezocones can measure total penetration pore-water stresses in clays. These measured pore-water stresses (u_m) is the summation of the hydrostatic head (u_o), excess pore-water stresses (Δu_{oct} and Δu_s) caused by changes in octahedral normal and shear-induced stresses. The hydrostatic stress can be calculated from the groundwater table position. The excess pore-water stresses from both the octahedral normal and shear stress invariant have been evaluated by either cavity expansion theory or the strain path method (Mayne and Kulhawy, 1990).

The cone penetration tests and piezoprobes were advanced into the clay using an electro-mechanical actuator. This device provided a constant rate of penetration of 16 mm/sec, which is slightly less than the recommended rate of 20 mm/sec given by ASTM D 3441 (1990). All data were recorded using the HP 3852A data acquisition unit and HP 1000 computer system. This system actually records the automatically-counted number of the rotating wheel's revolution, which were converted to depth by multiplication of the known penetration depth per revolution.

The complete records of cone tip resistance (q_c) and pore water stress measurements (u_t and u_{bt}) from both types of piezoprobes yield to the equivalence of a piezocone sounding. The tests indicate the general uniformity of the deposits.

It is needed to correct q_c for pore water stresses acting on unequal areas of the cone using the obtained piezoprobe measurements, so that the corrected cone tip resistance (q_T) is given by:

$$q_T = q_c + (1 - a)\ u_{bt} \tag{8}$$

in which a = net area ratio. For this mini-cone, a = 0.88. Also, u_{bt} measurements generally were small because of the high OCR values of the deposits in which the clay is fissured. The readings of u_{bt} initially were negative and then became increasingly positive with depth, which was also observed in previous study. Therefore, the difference between the corrected cone tip resistance (q_T) and uncorrected tip resistance (q_c) was negligible. As an example, Fig. 3 indicates only minor differences between uncorrected q_c and corrected q_T for deposit T-3.

Measurements of q_c, u_t, and u_{bt} can also allow quantification of stress history and strength in clays (Kulhawy and Mayne, 1990). Examples of measuring q_c, u_t, and u_{bt}

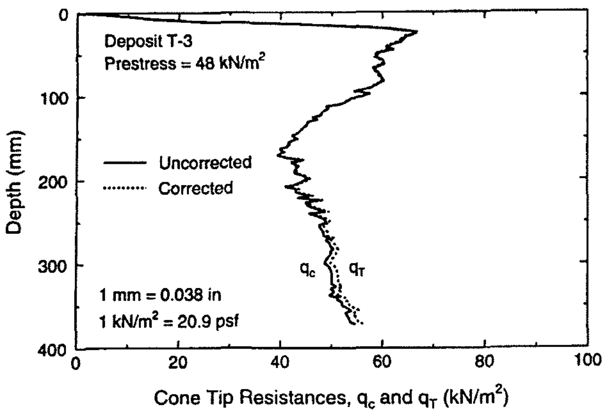

Fig. 3. Comparison of cone tip resistances in deposit T-3

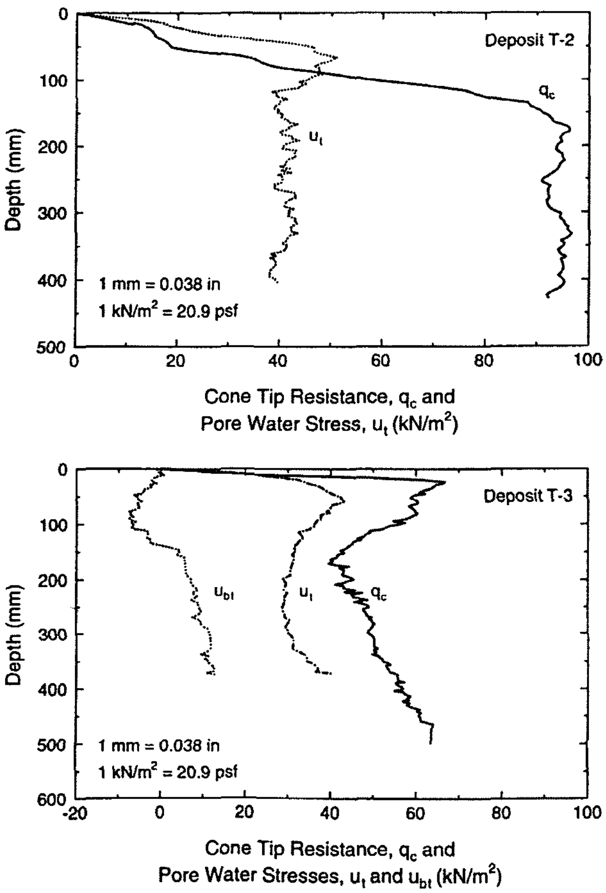

Fig. 4. Soundings in deposits T-2 and T-3

are shown Fig. 4. Details in evaluation of stress history and strength in Cornell clay have been given elsewhere (Mayne and Kulhawy, 1991).

6. Conclusion

Deposits of Cornell clay were characterized using a variety of in-situ tests, including water content determinations, vane shear, cone penetration, and piezoprobe tests. These tests were useful in assessing the uniformity, homogeneity, and variability of the six clay deposits. The findings from the in-situ test measurements are as follows:

(1) The undrained shear strengths measured by VST in this study were consistent with those measured in previous studies and can be expressed as an exponential function of water contents.

(2) There is a linear relationship between the vane shear strength and preconsolidation pressre. This fact is consistent with field data (Chandler, 1987).

(3) The general relationship between the vane shear strength and the plasticity index of NC clays is valid for OC clays. Therefore, the preconsolidation stress can replace the current effective vertical stress in Skempton's equation.

(4) The ratio of undrained shear strength to vertical effective stress varies almost linearly with OCR. This fact has been found also for different prototype clays.

(5) The OCR adjustment coefficient determined in this study is in the range of those in natural clay deposits.

(6) The measurements of cone tip resistance and pore water stresses indicate the general uniformity of the deposits.

(7) The u_{bt} measurements are generally small because of the high OCR values of the clay deposit in which micro-fissures might exist. The readings of u_{bt} initially were negative and then became increasingly positive with depth.

(8) Measurement of cone tip resistance and pore water stresses can allow quantification of stress history and strength in clays.

References

1. American Society for Testing and Materials (1993), "Standard Test Methods for Field Vane Shear Test in Cohesive Soil (D2573-72)", *Annual Book of Standards*, Vol.4.08, ASTM, Philadelphia, pp. 346-348.
2. American Society for Testing and Materials (1993), "Standard Test Methods for Deep Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil (D3441-86)", *Annual Book of Standards*, Vol.4.08, ASTM, Philadelphia, pp.469-474.

3. Baligh, M. M., Azzouz, A. S., and Martin, R. T. (1980), "Cone Penetration Tests Offshore the Venezuelan Coast", *Report No. MITSG 80-21*, Massachusetts Institute of Technology, Cambridge, MA, 163 p.
4. Bjerrum, L. (1988), "Embankments on Soft Ground", *Proceedings*, Performance of Earth and Earth-Supported Structures, ASCE, Lafayette, IN, Vol.2, pp.1-54.
5. Campanella, R. G. and Robertson, P. K., "Current Status of the piezocone Test", *Proceedings*, International Symposium on Penetration Testing (ISOPT1), Vol.1, Orlando, pp.93-116.
6. Chandler, R. J. (1987), "The In-Situ Measurement of Undrained Shear Strength of Clays Using The Field Vane", *Vane Shear Strength Testing in Soils*: Field and Laboratory Studies (STP 1014), Ed. A.F. Richards, ASTM, Philadelphia, pp.13-45.
7. Kulhawy, F. H. and Mayne, P. W. (1990), "Manual on Estimating Soil Properties for Foundation Design", *Report EL-6800*, Electric Power Research Institute, Palo Alto, 306 p.
8. Kulhawy, F. H., Beech, J. F., and Trautmann, C. H. (1989), "Influence of Geologic Development on Horizontal Stress in Soil", *Foundation Engineering*: Current Principles and Practices (GSP 22), Vol.1, Ed. F.H. Kulhawy, ASCE, New York, pp.43-57
9. Mayne, P. W. and Mitchell, J. K. (1988), "Profiling OCR in Clays by Field Vane", *Canadian Geotechnical Journal*, Vol.25, No.1, pp.150-157.
10. Mayne, P. W. (1991), "Determination of OCR in Clays by Piezocone Tests Using Cavity Expansion and Critical State Concepts", *Soils and Foundations*, Vol.31, No.2, pp.65-76.
11. Mayne, P. W., Kulhawy, F. H., and Kay, J. N. (1990), "Observations on Development of Pore Water Stresses During Piezocone Penetration in Clays", *Canadian Geotechnical Journal*, Vol.27, No.4, pp.418-428.
12. Mesri, G. (1989), "A Reevaluation of $s_{u(mob)} = 0.22\ \bar{\sigma}_p$ Using Laboratory Shear Tests", *Canadian Geotechnical Journal*, Vol.26, No.1, pp.162-164.
13. Larsson, R. (1980), "Undrained Shear Strength in Stability Calculation of Embankments and Foundations on Clays", *Canadian Geotechnical Journal*, Vol.17, pp.591-602.
14. Skempton, A. and Henkel, D. (1957), "Tests on London Clay from Deep Borings", *Proceedings*, 4th International Conference on Soil Mechnics and Foundation Engineering, London, pp.100-105.
15. Sweeny, B. P. (1987), "Liquefaction Evaluation Using a Miniature Cone Penetrometer and a Large Scale Calibration Chamber", *Ph.D. Thesis*, Stanford University, Palo Alto, 281 p.

(received on May 16, 2002, accepted on Feb. 10, 2005)