International Journal of Technology (2015) 4: 709-717
ISSN 2086-9614

IMPROVING SHEAR STRENGTH OF CLAY BY USING CEMENT COLUMN REINFORCEMENT UNDER CONSOLIDATED UNDRAINED TEST

Damrizal Damoerin[1*], Widjojo A. Prakoso[1], Yunita Utami[1]

[1] *Department of Civil Engineering, Faculty of Engineering, Universitas Indonesia, Kampus Baru UI Depok, Depok 16424, Indonesia*

(Received: June 2015 / Revised: August 2015 / Accepted: October 2015)

ABSTRACT

Cement column reinforcement is a method of soil reinforcement used in the field to increase soil's shear strength and decrease soil's compressibility. A set of laboratory studies of the effect of cement column reinforcement on shear strength in an undrained condition was conducted on kaolinite clay by using a triaxial apparatus to simulate real conditions. For triaxial testing, the soil samples were made using an extruder. Afterward, to make the composite samples, the soil sample cores were bored to create holes 5 mm in diameter and 50 mm long, and the holes were filled with cement slurry. The soil samples' cement column reinforcements were cured for seven, 14, and 21 days. Then, the soil and composite samples were saturated, consolidated, and applied to the loading. In this test, applying shear force to the soil sample and composite samples was carried out until the maximum stress and a strain of 12% were reached. The results from this test indicated that the cohesion parameter and angle of the internal friction of composite samples are higher and lower, respectively, than the unreinforced soil samples in the consolidated, undrained triaxial test. It was found that a cement column reinforcement system can improve soil shear strength.

Keywords: Cement column; Consolidated undrained; Kaolinite clay; Reinforcement; Shear strength

1. INTRODUCTION

Building foundations on soft soil should be safe and experience settlement within acceptable parameters. One of the methods to improve the shear strength of soft soils in the field is the application of cement column reinforcement. This method is used to increase the shear strength capacity of soft soils by using a bearing layer.

Full-scale loads tests of granular piles on soft clay were conducted, and the results indicate an increase in bearing capacity and a reduction of settlement. Using a cement column in the field can improve the stability of slope, trenches, and deep excavations and can increase bearing capacity and reduce the total and differential settlement (Bergado et al., 1996). In addition to these tests, the behavior of weak subsoil treated with granular piles was examined. Different model-based tests were also carried out in laboratories. The results show that the group action of piles can reduce settlement from 35–40% and increase the bearing capacity of untreated weak subsoil (Bayan, 2003). The effect of sand pile on the clay's shear strength, which was used as a composite soil, was studied in a consolidated undrained triaxial test. The research indicates an increase in the shear strength of composite soils (Park et al., 2003). Unconfined compression

* Corresponding author's email: damrizal@eng.ui.ac.id, Tel. +62-21-7270029, Fax. +62-21-7270028
Permalink/DOI: http://dx.doi.org/10.14716/ijtech.v6i4.1206

strength and consolidated drained triaxial tests were conducted on composite reinforced soil by using geosyntetics materials. The test results show that the inclusion of geotextiles significantly increased strength (Chang et al., 2003). Embankment construction over soft cohesive soil by using bamboo piles as raft reinforcement was carried out in the field to increase shear strength capacity and decrease settlement (Irsyam et al., 2008). The investigation of the effect of timber pile reinforcement on the shear strength of clay in a consolidated undrained triaxial test was conducted. The test results indicate that the shear strength of composite samples is higher than that of unreinforced soil samples (Damoerin et al., 2011). In this study, the consolidated undrained triaxial test was conducted on the unreinforced and reinforced cement columns of soil samples to find the composite samples' effect on shear strength.

2. EXPERIMENTAL

A physical properties test of kaolinite clay was carried out first, and then the triaxial tests were conducted on soil and composite samples. These tests were performed according to the American Society for Testing and Materials (ASTM) method (ASTM, 1995).

2.1. Sample Preparation

2.1.1. Slurry sample

The kaolinite clay slurry was prepared by adding water, equivalent to 100% of its dry weight, into the dry kaolinite clay powder. Afterward, the slurry was mixed for 10 minutes by using a mixer. After this preparation, the mixed slurry-like cement paste was ready for the consolidation phase.

2.1.2. Triaxial sample

In the first step of triaxial sample preparation, the kaolinite clay slurry was consolidated in the ROWE CELL consolidometer apparatus under 100 kPa pressure for eight days. The soil samples for triaxial testing were made by using an extruder with a 38 mm diameter and 76 mm height. The soil samples reinforced by a cement column were termed "composite samples." To make the composite soil samples, the soil sample cores were bored to create holes 5 mm in diameter and 50 mm long. The holes were filled slowly with the cement slurry at a cement-water ratio of 0.5 by using a plastic pipe. In the second step, the composite samples were cured for seven, 14, and 21 days. In the third step, the soil and composite samples were saturated in the triaxaial test until the coefficient of B was above 0.97.

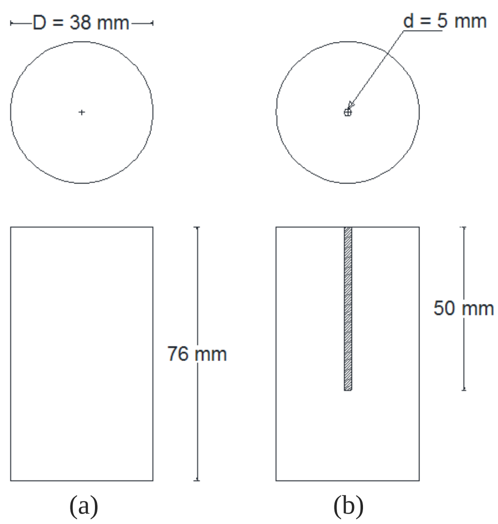

Figure 1 Sample configuration: (a) soil sample; (b) composite sample with cement column

After saturation, the soil and composite samples were consolidated under 100, 140, and 180 kPa cell pressure, which was applied within 24 hours. The configuration of the soil and composite samples can be seen in Figure 1, and the number of soil and composite samples are shown in Table 1.

Table 1 Amount of soil and composite samples

Sample Code	Curing Time (days)	Number of cement columns			Number of Samples
		Diameter	Length	Number	
1 (soil sample)	0	-	-	-	3
2 (composite sample)	7	5 mm	50 mm	1	3
3 (composite sample)	14	5 mm	50 mm	1	3
4 (composite sample)	21	5 mm	50 mm	1	3
			Total		12

2.2. Triaxial Test

The sample preparation was finished first, and then the consolidated undrained triaxial test was conducted on soil and composite samples. Before testing, the composite samples were cured for seven, 14 and 21 days to find the biggest deviator stress value. In this test, the shearing of the soil sample and composite samples was carried out at 0.05 mm/minute or 0.07%/minute (Bishop et al., 1982). Application of the shearing force to the sample and composite samples was conducted until reaching a maximum deviator stress and a strain of 11%. The pore pressure was measured during this test.

3. RESULTS

3.1. Physical Properties

3.1.1. Pattern making

The physical property test results of kaolinite clay are shown in Table 2. Based on Casagrande's (1948) Plasticity Chart, the kaolinite clay is classified as CH and is inorganic and of high plasticity.

Table 2 Physical properties of kaolinite clay

No.	Physical parameters	Value
1.	Color	White
2.	Natural water content (%)	1.58
3.	Specific gravity	2.59
4.	Atterberg Limits:	
	• Liquid Limit, LL (%)	77.93
	• Plastic Limit, PL (%)	39.10
	• Plastic Index, PI (%)	38.83
5.	Sieve Analysis:	
	• Sand (%)	0
	• Silt (%)	47
	• Clay (%)	53

3.2. Engineering Properties

Generally, engineering properties, such as shear strength parameters, are necessary to analyze the bearing capacity of the foundation, retaining wall, and slope stability.

3.2.1. Stress-strain relationship

Consolidated undrained triaxial tests were performed to obtain the engineering properties of soil and composite samples, such as shear strength parameters. The shearing force was applied until reaching a maximum deviator stress and a strain between 5–11%. Pore pressure was measured during this test. In this test, the effective stress was influenced by excess pore water pressure. The maximum values found in the test results of deviator stress, strain, and excess pore water pressure on the soil sample and composite samples are shown in Table 3. The relationship between deviator stress and strain on soil and composite samples is shown in Figures 2–5.

Table 3 Maximum deviator stress, strain, and excess pore water pressure values

Sample Code	Curing Time (days)	σ_3 (kPa)	ε (%)	$\epsilon_{average}$ (%)	Δu (kPa)	max q (kPa)
		100	10.53		50	101.14
1 (soil sample)	0	140	6.91	9.21	70	117.90
		180	10.20		95	133.72
		100	10.53		55	100.03
2 (composite sample)	7	140	9.54	9.98	70	119.61
		180	9.87		100	136.45
		100	10.20		55	104.84
3 (composite sample)	14	140	9.21	9.43	90	129.56
		180	8.88		100	139.62
		100	10.20		70	112.59
4 (composite sample)	21	140	8.88	8.22	90	137.91
		180	5.59		105	142.32

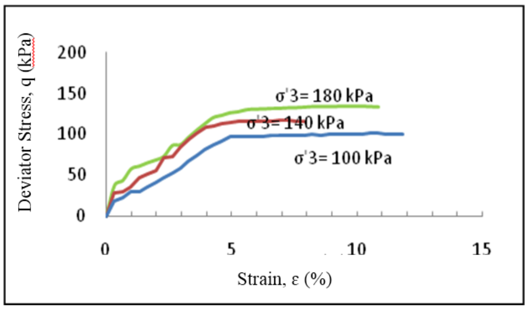

Figure 2 Relationship between deviator stress (q) and strain (ϵ) on soil sample without curing time

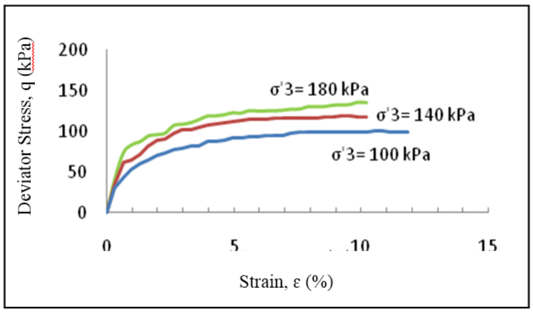

Figure 3 Relationship between deviator stress (q) and strain (ϵ) on composite sample, curing time 7 days

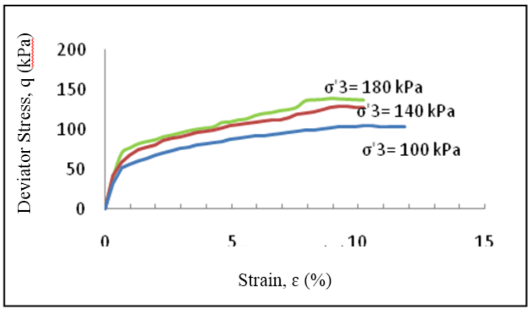

Figure 4 Relationship between deviator stress (q) and strain (ϵ) on composite sample, curing time 14 days

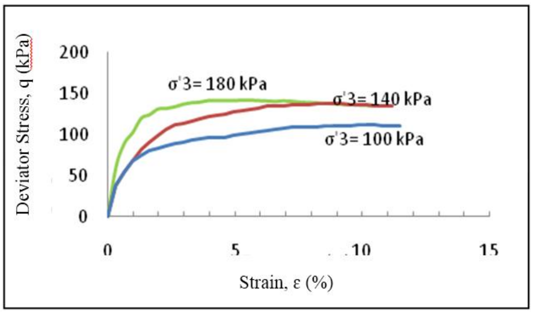

Figure 5 Relationship between deviator stress (q) and strain (ϵ) on composite sample, curing time 21 days

In general, the excess pore water pressure on composite samples was higher than the unreinforced soil sample. As shown in Figures 2–4, the maximum deviator stress on composite samples that were cured for seven and 14 days are similar to the unreinforced soil sample. Nevertheless, as seen in Figure 5, the maximum deviator stress on composite samples that were cured for 21 days is higher than that on unreinforced soil samples. This indicates that the composite samples cured for 21 days can increase the soil's shear strength. The bonding of particles of cement-treated clay occurred, and this was expected to increase over time (Bergado et al. 1996).

3.2.2. M Parameter & q_o Value

The critical state path method, with an effective stress of p' and deviator stress of $q' = q = (\sigma_1 - \sigma_3) = \Delta\sigma$, illustrated the relationship between effective and deviator stress, as shown in Figures 6–9. The effective and deviator stress can be calculated by using the formula (Bardet, 1997):

$$p' = [1/3(\sigma_1' + 2\sigma_3')] = [1/3(\Delta\sigma + 3\sigma_3')] = [(\Delta\sigma/3) + \sigma_3'] = [1/3(\sigma_1 + 2\sigma_3)] - \Delta u = p - \Delta u \qquad (1)$$

$$q' = q = (\sigma'_1 - \sigma'_3) = (\sigma_1 - \sigma_3) = \Delta\sigma \qquad (2)$$

$$q' = M\,p' + q_o \qquad (3)$$

where, $M = \{(6 \sin Ø) / (3 - \sin Ø)\}$

$q_o = \{(6 \cos Ø) / (3 - \sin Ø)\} \times C'$

The stress-strain relationships are shown in Figures 6–9.

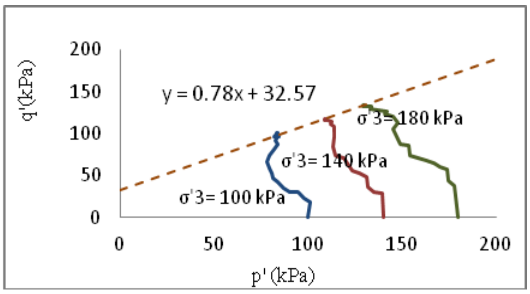

Figure 6 Relationship between deviator stress (q) and effective stress (p') on soil sample without curing time

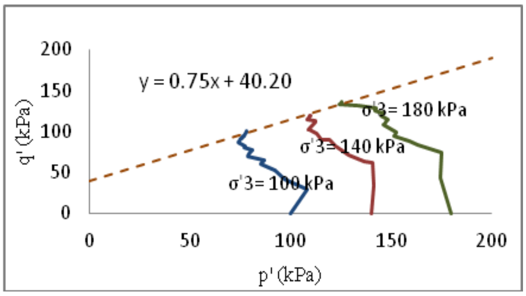

Figure 7 Relationship between deviator stress (q) and effective stress (p') on composite sample, curing time 7 days

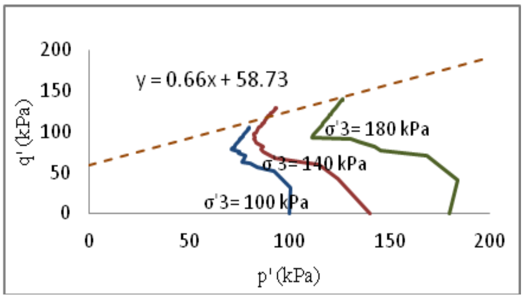

Figure 8 Relationship between deviator stress (q) and effective stress (p') on composite sample, curing time 14 days

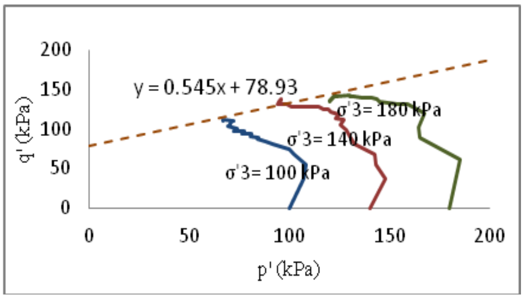

Figure 9 Relationship between deviator stress (q) and effective stress (p') on composite sample, curing time 21 days

The M parameter and q_o value can be obtained. The M parameter can be determined as a gradient between the q and p' values at peak failure condition; the q_o value is an intersection between the gradient and ordinate line.

3.2.3. *Shear strength parameter*

The shear strength parameter consists of cohesion (C') and internal friction angle (Ǿ). This parameter can be obtained by using Equations 4 and 5 (Bardet, 1997):

$$\varphi' = \sin^{-1}\left[\frac{3\times M}{6+M}\right] \tag{4}$$

$$c' = \left[\frac{3-\sin\varphi'}{6\cdot\cos\varphi'}\right] q_o \tag{5}$$

Based on Equations 4 and 5, the value of C' and Ø' can be calculated. The M parameter, q_o value, and parameters of C' and Ǿ in soil sample and composite samples can be seen in Table 4.

Table 4 M parameter and q_o value and C' and Ǿ parameter

Sample Code	Curing Time (days)	M	q_o	C' (kPa)	Ø' (°)
1 (soil sample)	0	0.780	32.57	15.37	20.19
2 (composite sample)	7	0.750	40.20	18.95	19.47
3 (composite sample)	14	0.660	58.73	27.71	17.30
4 (composite sample)	21	0.545	78.93	37.36	14.47

The test results shown in Table 4 indicate that the cohesion parameter and the internal friction angle of composite samples are higher and lower, respectively, than the unreinforced soil samples, especially in the composite samples that were cured for 21 days.

3.2.4. *Parameter λ_{NCL}, N, λ_{CSL}, Γ*

The parameters λ_{NCL}, N, λ_{CSL}, and Γ are volume parameters and can be found from:

(i) Graph equation of specific volume (v = 1+e) vs. ln (effective stress, p') from the original equation of the normal consolidation line (NCL) and critical state line (CSL) as shown in Figures 10–13.

(ii) NCL with $v = N - \lambda \ln p'$ (6)

(iii) CSL with $v = \Gamma - \lambda \ln p'$ (7)

CSL is located to the left of NCL. By using Equations 6 and 7, and then filling in the value of p’ = 1 kPa, the values of N, λ, and Γ can be obtained (Atkinson et al., 1982). The parameters of λ_{NCL}, N, λ_{CSL} , Γ can be seen in Table 5.

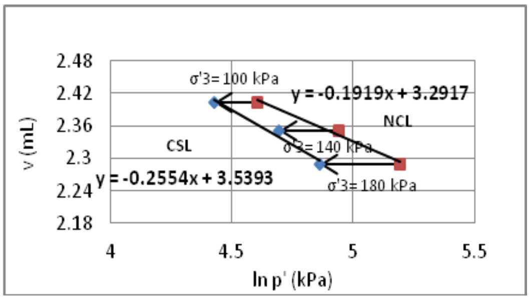

Figure 10 Relationship between specific volume (v) and ln (effective stress, p’) on soil sample without curing time

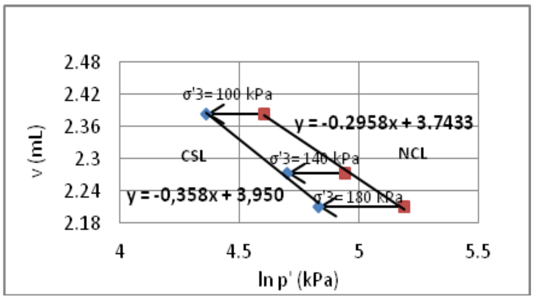

Figure 11 Relationship between specific volume (v) and ln (effective stress, p’) on composite sample, curing time 7 days

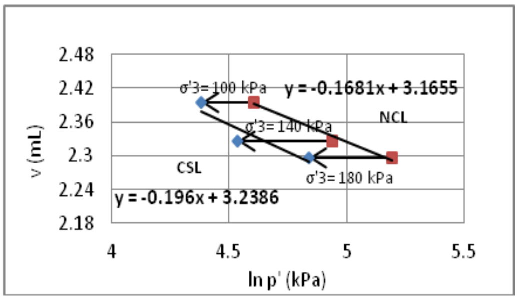

Figure 12 Relationship between specific volume (v) and ln (effective stress, p’) on composite sample, curing time 14 days

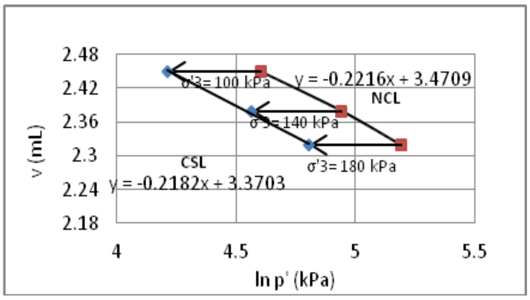

Figure 13 Relationship between specific volume (v) and ln (effective stress, p’) on composite sample, curing time 21 days

Table 5 Parameters of λ_{NCL}, N, λ_{CSL} , Γ

Sample Code	**Curing Time (days)**	**Critical State Parameter**			
		NCL		**CSL**	
		λ_{NCL}	**N**	λ_{CSL}	**Γ**
1 (soil sample)	0	-0.191	3.291	-0.255	3.539
2 (composite sample)	7	-0.295	3.743	-0.358	3.950
3 (composite sample)	14	-0.168	3.165	-0.196	3.238
4 (composite sample)	21	-0.221	3.470	-0.205	3.316

3.3. Discussion

The results of the consolidated undrained triaxial test have been evaluated by using Bardet’s formula. The M parameter, q_o value, cohesion (C’), and internal friction angle (Ø) can be obtained by using Equations (3), (4), and (5).

Bardet offered an explanation that soils are cohesionless when the curved failure envelope passes through the stress origin (i.e., $\sigma' = \tau = 0$), and they are cohesive when their curved failure envelope intercepts the τ axis above the origin. Coarse grained soils without plastic fines are usually cohesionless. For the most cohesive soils—with the exception of cemented soils, partially saturated soils, and heavily consolidated clay—the intercept of the curved failure envelope with the τ axis is generally small (Bardet, 1997). Thus, the composite samples can be assumed to be cemented soils.

The values of C’ and Ǿ can be calculated by using the stress path method (p’–q diagram), and this method represents the failure circle (Mohr τ–σ' diagram). The results indicate that the average values of C’ and Ǿ decrease by about 97% and 96%, respectively, using Bardet’s formula. However, the range of values is still good.

4. CONCLUSION

The soil samples used for cement column reinforcement should be cured for 21 days to reach the biggest deviator stress value, especially for the soil shear strength parameter, and to get the smallest volume parameter, such as the values of λ_{CSL} and Γ.

This study shows that for 21 days of curing time, the cohesion parameter and the internal friction angle of the composite samples increased about 2.4 times, from 15.37 kPa to 37.36 kPa, and decreased by about 28%, from 20.19 degrees to 14.47 degrees, respectively, compared to unreinforced soil samples.

5. ACKNOWLEDGEMENT

The authors would like to express their gratitude to the Soil Mechanics Laboratory and Civil Engineering Department, Faculty of Engineering, University of Indonesia for supporting this study.

6. NOMENCLATURE

σ_3	Minor principle stress or cell pressure
ε	Strain
Δu	Excess pore water pressure
$q' = q = \Delta\sigma$	Deviator stress
$\sigma_1 = (\sigma_3 + \Delta\sigma)$	Major principle stress
$\sigma_{3'} = (\sigma_3 - \Delta u)$	Effective minor principle stress or effective cell pressure
$\sigma_{1'} = (\sigma_1 - \Delta u)$	Effective major principle stress
$p' = [1/3(\sigma_1' + 2\sigma_3')] = (p - \Delta u)$	Effective stress
M	Gradient of q’ equation ($q' = M\,p' + q_o$)
q_o	Intersection between gradient and ordinate line
C′	Cohesion
Ø′	Internal friction angle
$v = (1 + e)$	Specific volume
$v = N - \lambda \ln p'$	Equation of normal consolidation line (NCL)
$v = \Gamma - \lambda \ln p'$.	Equation of critical state line (CSL)
λ_{NCL}	Slope of normal consolidation line (negative)
N	Specific volume of isotropically normally consolidated soil at p’ = 1 kPa
λ_{CSL}	Slope of critical state line (negative)
Γ	Specific volume of soil at critical state at p’ = 1 kPa

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