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STRENGTH AND DEFORMATION CHARACTERISTICS OF LIME-TREATED SOFT CLAYS

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SYNOPSIS

The effect of lime on the strength and deformation characteristics of Soft Bangkok Clay has been investigated by unconfined compression tests, oedometer tests and undrained triaxial tests. The unconfined compression tests revealed that the undrained strength can be increased nearly tenfold by the addition of 5% lime. Under the oedometer test condition the preconsolidation pressure of the lime-treated clays showed a marked increase and the overconsolidation ratio increased, thus improving the compressibility characteristics. The reduction in compressibility was also accompanied by an increase in the coefficient of consolidation. The undrained triaxial tests further confirm the overconsolidated nature of the lime treated clay in the effective stress paths, stress-strain curves and pore pressure characteristics. Both the cohesion and the angle of shearing resistance are also increased in a substantial manner with the addition of lime. These results form an intersting basis to develop the appropriate stress-strain theory for lime-treated clays.

INTRODUCTION

The strength and deformation characteristics of Bangkok Clays have been investigated extensively (see Balasubramaniam & Hwang, 1980; Balasubramaniam & Waheed, 1977; Balasubramaniam & Chaudhry, 1977; and Balasubramaniam & Li, 1977). These studies formed a useful source of information for the solution of boundary value problems in practice, i.e. embankments and excavation on soft clay deposits (see Balasubramaniam et al., 1979). In contrast to such studies, the recent emphasis has been to identify a suitable ground improvement technique that can be successfully applied to improve the engineering properties of the Soft Bangkok Clay which is of high compressibility and low shear strength.

The lime column technique has been claimed to have been successfully applied to improve the engineering properties of soft clays in Japan and Scandinavia (see

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Terashi & Tanaka, 1981; Matsuo & Kamon, 1981; Kawasaki et al., 1984; Brandl, 1981; and Broms, 1984). Methods for design for lime colums have been proposed (see Broms & Boman, 1977; Broms, 1984) for settlement analysis and bearing capacity of foundations and for the design of deep trenches in improved ground. As of 1984, lime columns of 0.5 m diameter and up to 15 m lengths had been installed in Sweden, using light mobile equipment, as foundations for light structures and embankments, as lateral support in excavations and trenches, retaining walls or slurry walls, and as a barrier to reduce vibrations caused by traffic, blasting and pile driving. Piled foundations are traditionally used for all structural foundations in the Bangkok Plain (see Balasubramaniam et al., 1985) and it seems there are merits in looking for alternative, more economical foundation techniques which could be suited to a number of lightly loaded structures including road embankments for approach roads.

With this view in mind laboratory and field tests were conducted to study the improvement of lime-treated Soft Bangkok Clay.

LIME STABILIZATION

Normally, finely pulverised quicklime is used for soil stabilization of clays with moderately high water contents, as reported by Broms & Boman (1977). A large amount of heat is released upon the hydration of quicklime, which causes a rise in temperature and this, combined with an increase in pH, is favourable to long term chemical reactions in the soil-lime mixture (Broms, 1984). Also, the hydration of the quicklime causes a reduction in the water content of soft clays (Assarson et al., 1974). The ion exchange and the flocculation mechanism is described by Kezdi (1979) and Herrin & Mitchell (1961). Further details related to pozzolanic reactions of the calcium hydroxide in the pore water with the silicates and aluminates of the clay in forming cementing materials is discussed by Diamond & Kinter (1966). Eades et al. (1962), have demonstrated that the strength gain in soils due to cementation caused by carbonation is rather negligible.

Kezdi (1979) indicated that in dry lime stabilization, the finest soil particles agglomerate to form a coarser structure, and the resulting increase in permeability of lime-treated clays has been pointed out by Brandl (1981), Evans & Bell (1981) and Broms & Boman (1977). The critical lime content referred to as "lime fixation point" was defined by Pietsch & Davidson (1965). The plastic limit of the clay generally increases up to the lime fixation point. Herrin & Mitchell (1961) showed that the liquid limit of highly plastic clays decreases with lime stabilization, whereas in the low plastic clays it has a tendency to increase. Broms & Boman (1977) showed that the strength and deformation properties of lime columns are similar to those of a heavily overconsolidated stiff fissured clay. Similar observations were noted by Matsuo & Kamon (1981), where the compressibility of very soft marine clays, when treated with multivalent cations, showed less compressibility at lower consolidation pressures; however, at higher consolidation pressures the compressibility is somewhat increased.

In ascertaining the optimum lime content for strength increase, Eades & Grim (1966) suggest that the lowest percentage of lime required to maintain a pH of 12.4 is the percentage required to stabilize the soil. However, a strength test is still needed to show the percentage of strength increase. Halton et al. (1969) found that quicklime and calcitic hydrated lime are superior in stiffening soft wet soils. Taylor & Arman (1960) observed that lime has an initial reaction with soil taking place during the first 48-72 hours after mixing, and a secondary reaction which starts after that period and continues indefinitely. Brandl (1981) and Okamura & Terashi (1975) expressed the time-dependent increase in shear strength as a linear function with the logarithm of time.

DETAILED SCOPE OF PRESENT WORK

The laboratory program of work on lime-treated clays and natural clays reported here was as follows.

(i) Unconfined compression tests; mainly to determine the strength gain with time, using lime contents of from 2.5% to 15%, and curing periods of 7, 14, 28 and 56 days.

(ii) Oedometer consolidation tests: mainly to determine the consolidation characteristics of lime-treated clay and the effects of lime treatment on the consolidation properties of the clay. Lime contents were varied from 2.5 to 15% using a curing period of 28 days only.

(iii) Consolidated undrained triaxial compression tests: mainly to investigate the triaxial strength characteristics of lime-treated clay and to determine the effects of effective consolidation pressure and lime content. Lime contents were varied from 2.5 to 15% and curing periods from 27 to 29 days. Effective consolidation pressures were from 5 to 60 t/m^2.

Since quicklime has been found to be more effective than hydrated lime in its reactions with the Soft Bangkok Clay (Yu Kuen, 1975), quicklime is used for the tests.

EXPERIMENTAL INVESTIGATIONS

The experimental investigations in the study were focused mainly on consolidated undrained triaxial compression tests in order to study the strength and compressibility characteristics of lime-treated soft clay. The soft clay samples were taken from a test embankment site on the campus of the Asian Institute of Technology from a depth of 3 to 4 m to obtain uniform samples in terms of maximum past pressures, water contents, compressibility and shear strengths. Piston tube samples of 250 mm diameter and 500 mm long were obtained. The typical properties of the soft clay are;

Water content, w_n, %	76 – 88
Total unit weight, t/m^3	1.49 – 1.53
Specific gravity	2.68 – 2.69
Sand, %	3
Silt, %	27
Clay, %	70
Liquid limit, %	104
Plastic limit, %	41
Plasticity Index, %	63
Liquidity Index	0.62
Activity	0.90
Sensitivity	7.3

Commercially available quicklime or Calcium Oxide (CaO) powder with a CaO content more than 95% was used. The loss on ignition was less than 10% and the Chloride content less than 0.02%. The lime content in this study is defined as the ratio of the weight of lime to the dry weight of clay expressed in percent, and the values used in the laboratory were 2.5, 5.0, 7.5, 10.0, 12.5 and 15.0%.

The soft clay in the 250 mm diameter tube was extruded and cut to approximately 150 mm lengths. These samples were then waxed and stored in a constant temperature humid room. A small amount of the clay was used for moisture content determination. At the time of sample preparation, a steel mould was placed around the sample, the top wax cover of the sample was removed and the sample trimmed to a height of approximately 125 mm. Three specimens were then taken from each mould using a thin walled tube with an inner diameter of 63.5 mm. The unit weight of the clay was determined. The quicklime was mixed with these three specimens of clay at the desired lime content, until a uniform mix was obtained. The lime-clay mix was then put back into the pre-drilled holes in approximately five equal layers. Each layer was compacted using a 25.4 mm diameter steel rod, and applying 30 blows per layer. This was done to remove entrapped air and to obtain uniform unit weight of the samples. A top cover made of plastic with a 63.5 mm diameter guide was placed over the mould during compaction to prevent heaving. This procedure produced a column of approximately 100 mm height and a unit weight of 1.52 t/m^3; the unit weight being a little higher than the values for the untreated clay. This procedure was repeated until three lime columns were prepared in each mould. The steel mould was then removed, and the samples waxed again and cured for 27 to 29 days inside the humid room. At the approximate time of testing the lime columns were cut out of the clay "cake" using cutting wires and trimmed to the size for each type of test to be conducted, i.e. 35.5 mm × 71 mm for triaxial and unconfined compression tests and 63.5 mm × 19.0 mm for oedometer tests. The water content of the treated clay was determined from the trimmed portion of the clay. The unit weight of the samples to be tested was also determined prior to testing.

RESULTS AND DISCUSSION

All lime-treated samples were prepared using the procedure discussed above. The water content, void ratio and unit weight of the samples with various lime contents are as follows.

Lime content (%)	Water content (%)	Void Ratio	Dry unit weight (t/m^3)
2.5	75.6-79.7	2.13-2.20	0.83-0.84
5.0	75.1-77.5	2.05-2.12	0.85-0.87
7.5	75.9-77.1	2.03-2.07	0.86-0.87
10.0	72.0-75.9	1.91-2.05	0.86-0.90
12.5	71.1-77.0	1.90-2.03	0.86-0.90
15.0	68.0-70.8	1.86-1.92	0.89-0.91

Figure 1 shows the effect of lime treatment on water content.

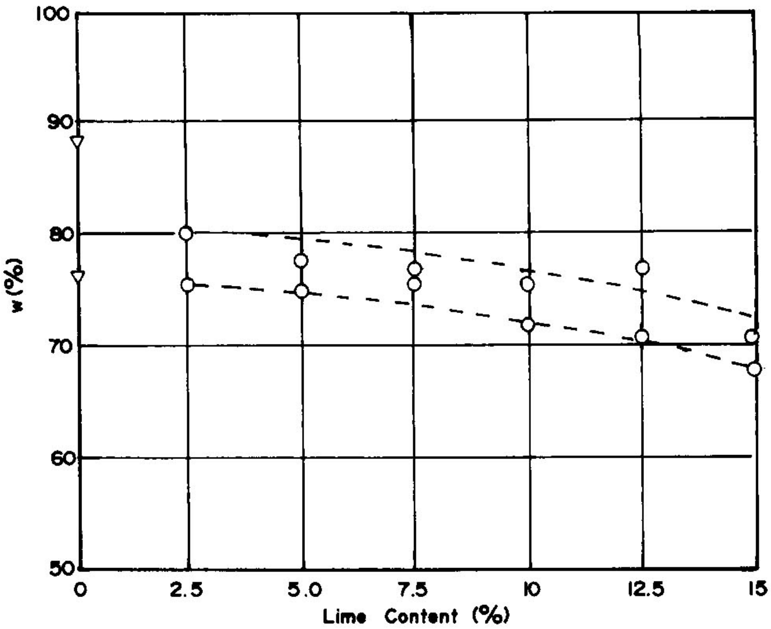

Fig. 1 Effect of Lime Treatment on Water Content (After 28 days Curing Period)

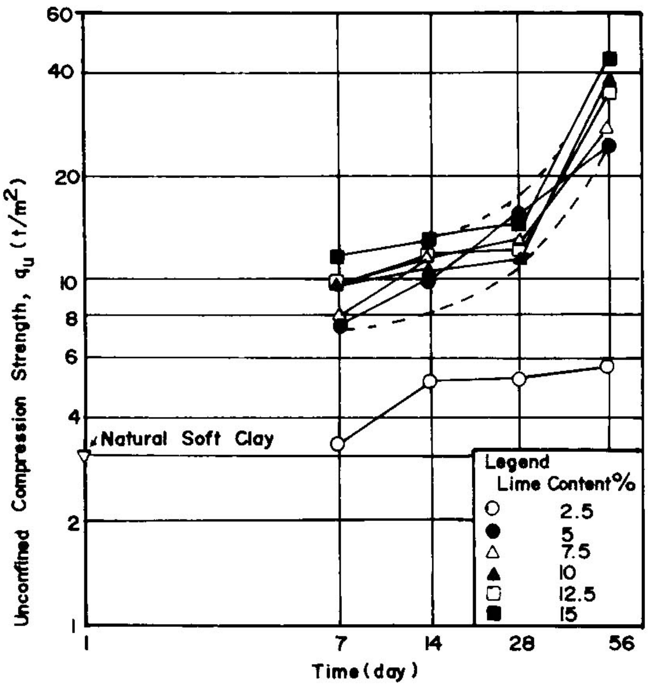

Fig. 2 Strength Gain with Time for Various Lime Contents

Unconfined Compression Tests

The strength gain with time of treated clays with various lime contents is illustrated in Fig. 2. The results show that the strength of all the lime-treated samples increases with time, although 2.5% lime content obviously does not cause substantial strength gain. With a 28 day curing period, the strength gain for the samples with 5 to 15% lime was about 4 to 5 times the strength of natural clay. Based on these results the 5% lime content appears to be enough to attain the same strength gain as the 15% lime content.

Oedometer Consolidation Tests

Oedometer consolidation tests were carried out to study the consolidation characteristics of lime treated clay and the improvement over that of natural clay. Of particular interest were the effects of stabilization on compression index (C_c), preconsolidation pressure ($\bar{\sigma}_{vm}$) and the coefficient of consolidation. The void ratio-axial stress relationships for samples with various lime contents are illustrated in Fig. 3. It appears that a 2.5% lime content does not result is substantial improvement, whereas for a 5% lime content, the virgin compression characteristics are not evident even up to a vertical stress of 64 t/m^2. Thus the apparent preconsolidation pressure has increased manyfold due to lime stabilization and the soft clay has become overconsolidated. Figure 4 illustrates the effect of lime treatment on the preconsolidation pressure. The $\bar{\sigma}_{vm}$ for natural clay is 7t/m^2, while with a 5% lime content the $\bar{\sigma}_{vm}$ can be increased to 22 t/m^2 , nearly a threefold increase in value. The effect of lime treatment on the compression index is shown in Fig. 5.

Another interesting aspect of the lime treatment is the increase in the coefficient of consolidation, c_v. At axial stresses below 7 t/m^2 and with 5 to 15% lime content, the c_v values increased by 5 to 10 times that of the natural clay. The corresponding values at stresses higher than 7 t/m^2 were 15 to 40 times (see Fig. 6).

Consolidated Undrained Triaxial Compression Tests

The stress-strain behaviour and strength characteristics of the lime-treated clay were studied in six series of consolidated undrained triaxial tests with lime contents ranging from 2.5 to 15%. The effective pre-shear consolidation pressures of 10, 20 and 40 t/m^2 were applied to the natural clay while a wide range of consolidation stresses of 5, 10, 15, 20, 40 and 60 t/m^2 were used on the samples with 5 to 15% lime contents. For the 2.5% lime content consolidation pressures were 5, 7.5, 10, 20 40 and 60 t/m^2.

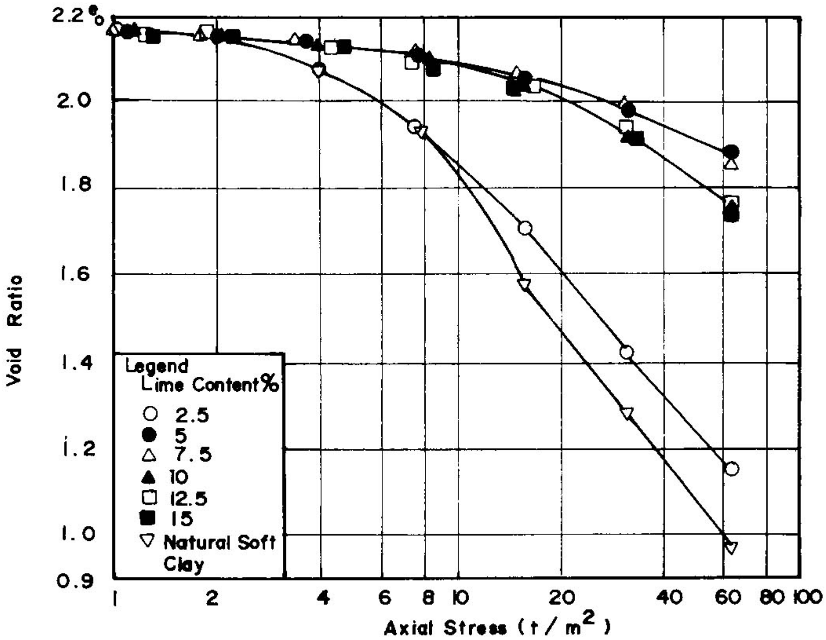

Fig. 3 Void Ratio-Axial Stress Relationship (e-log $\overline{\sigma}_v$ curve)

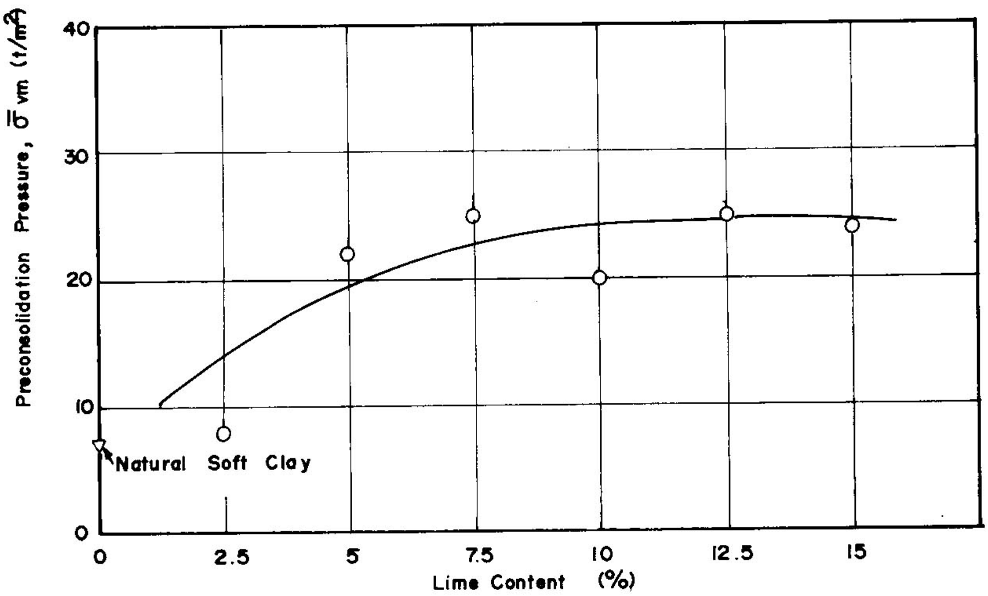

Fig. 4 Effect of Lime Treatment on Preconsolidation Pressure

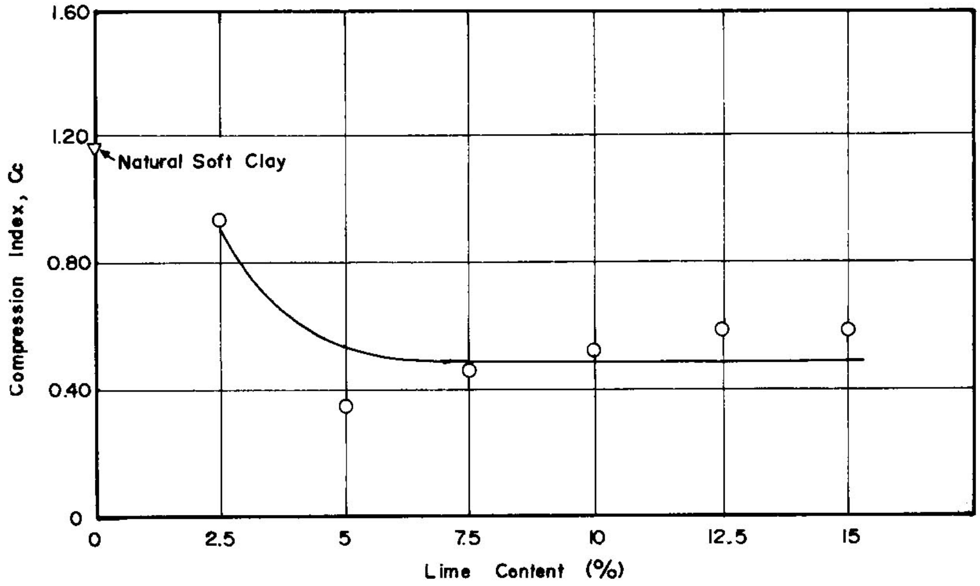

Fig. 5 Effect of Lime Treatment on Compression index for Maximum Stress Level

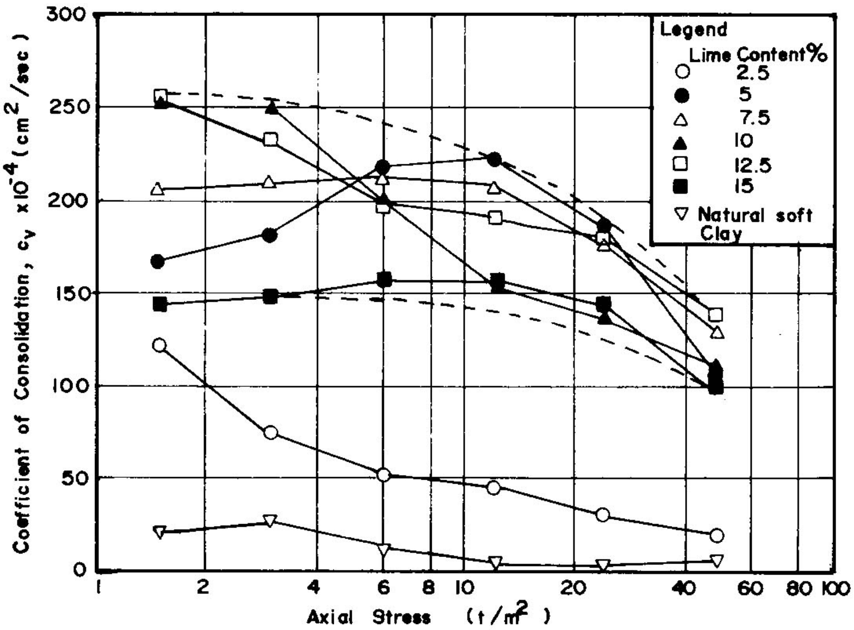

Fig. 6 Effect of Lime Treatment on Coefficient of Consolidation

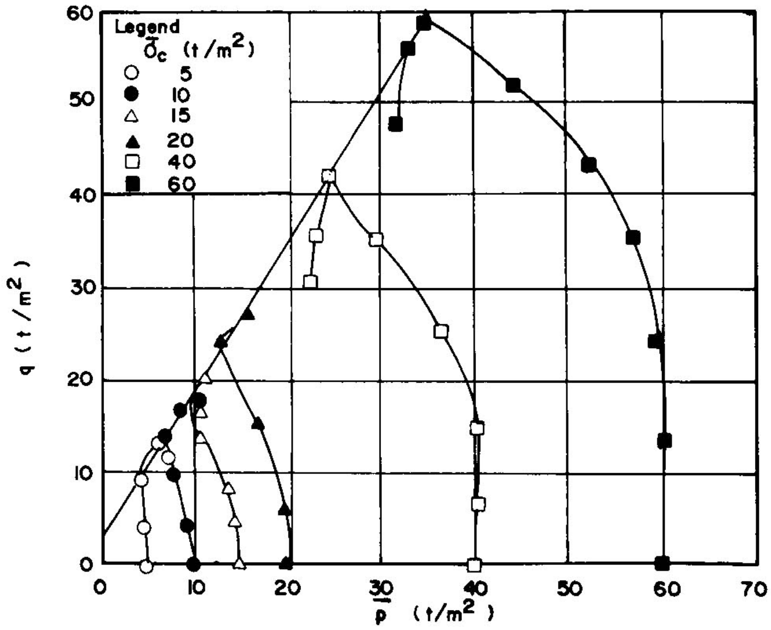

Fig. 7 Effective Stress Paths for Lime-Treated Clay (5% Lime Content)

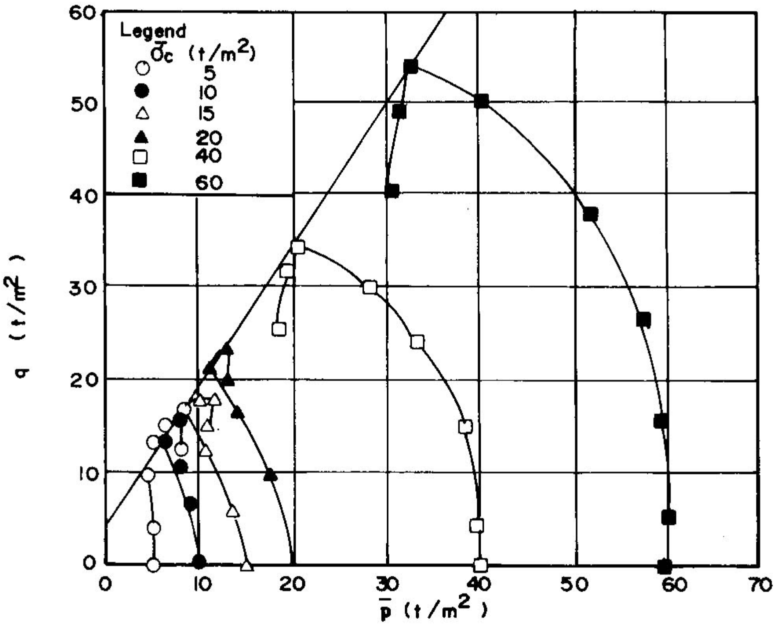

Fig. 8 Effective Stress Paths for Lime-Treated Clay (15% Lime Content)

Effective Stress Paths

Typical effective stress paths for lime contents of 5% and 15% are shown in Figs. 7 and 8. For natural clay, all the stress paths are very similar to normally consolidated clays, while those for the treated clays are similar to overconsolidated samples in that less pore pressure developed and the stress paths were less rounded. The stress paths tend to normalize to a unique curve only when the pre-shear consolidation pressures are higher than the apparent preconsolidation pressures which resulted from lime treatment.

Deviator Stress-Shear Strain Relationships

Figures 9 to 11 illustrate the deviator stress-shear strain relationships under undrained conditions. The behaviour of the lime-treated samples is similar to overconsolidated clays at lower pre-shear consolidation pressures. These results are untypical of the behaviour of untreated clay samples. There are substantial differences between the untreated clay and the samples treated with 5% and higher lime contents.

Pore Pressure-Shear Strain Relationships

Figures 12 and 13 illustrate the pore pressure-shear strain relationships. Here again the behaviour of the lime-treated clays is similar to overconsolidated clays.

Failure Envelopes

The strength envelopes of natural clay and some of the treated clays are shown in Figs. 14 to 16. The strength envelopes have been drawn on the basis of maximum deviator stress failure criterion. The values of the effective stress parameters are presented below.

Lime content (%)	$\bar{c}$ (t/m^2)	$\bar{\phi}$ (Degrees)
2.5	1.43	29.1
5.0	1.71	38.0
7.5	2.57	36.6
10.0	2.33	35.8
12.5	3.29	35.3
15.0	0.97	40.1
Natural clay	0	24.3

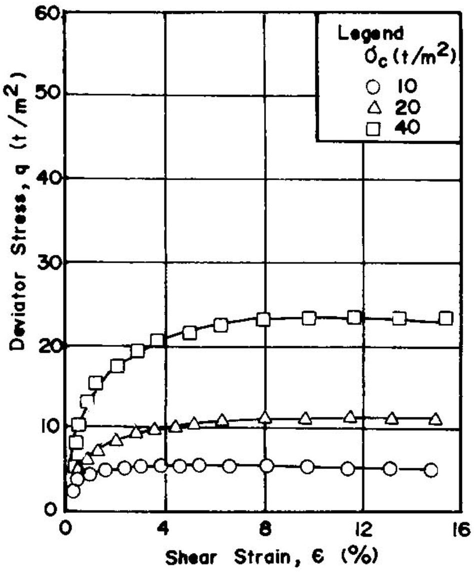

Fig. 9 Deviator Stress-Shear Strain Relationships for Natural Soft Clay

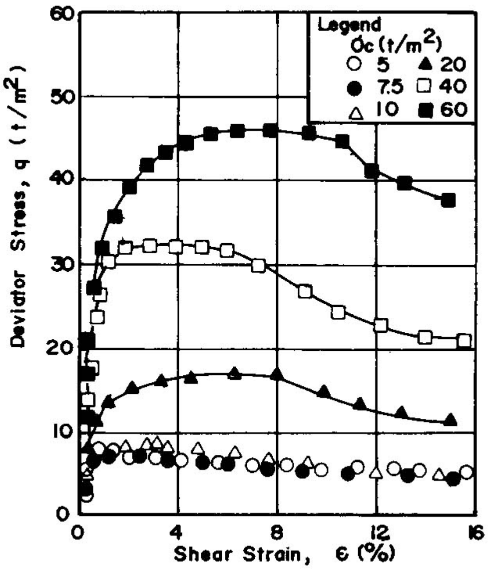

Fig. 10 Deviator Stress-Shear Strain Relationships for Lime-Treated Clay (2.5% Lime Content)

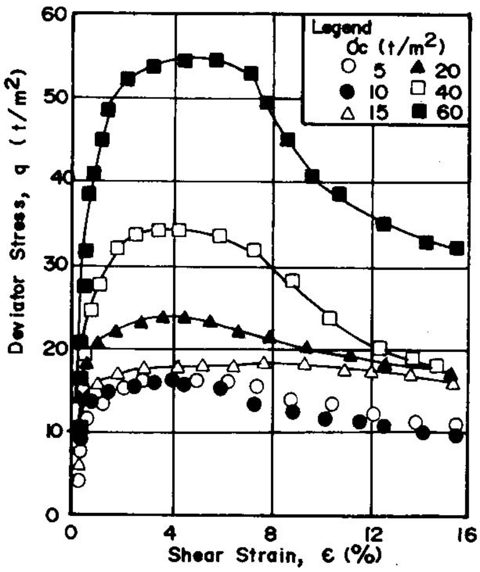

Fig. 11 Deviator Stress-Shear Strain Relationships for Lime-Treated Clay (5% Lime Content)

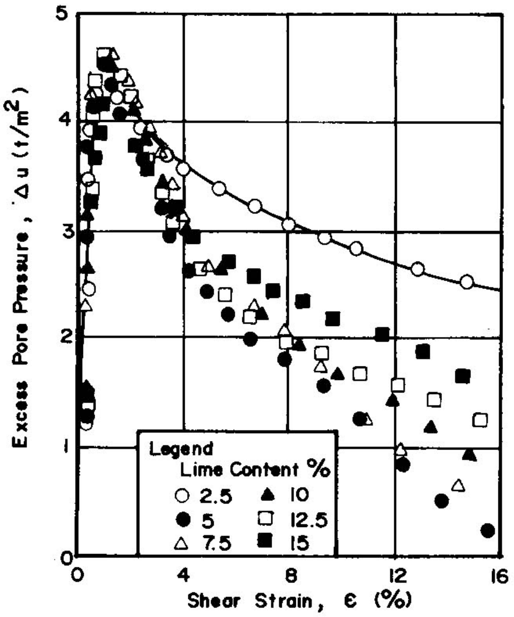

Fig. 12 Excess Pore Pressure-Shear Strain Relationships of Various Lime Contents for $\sigma_c = 5$ t/m^2

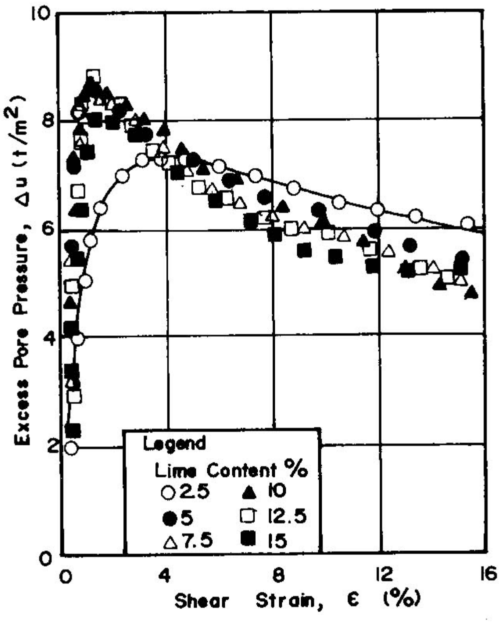

Fig. 13 Excess Pore Pressure-Shear Strain Relationships of Various Lime Contents for $\sigma_c = 10$ t/m^2

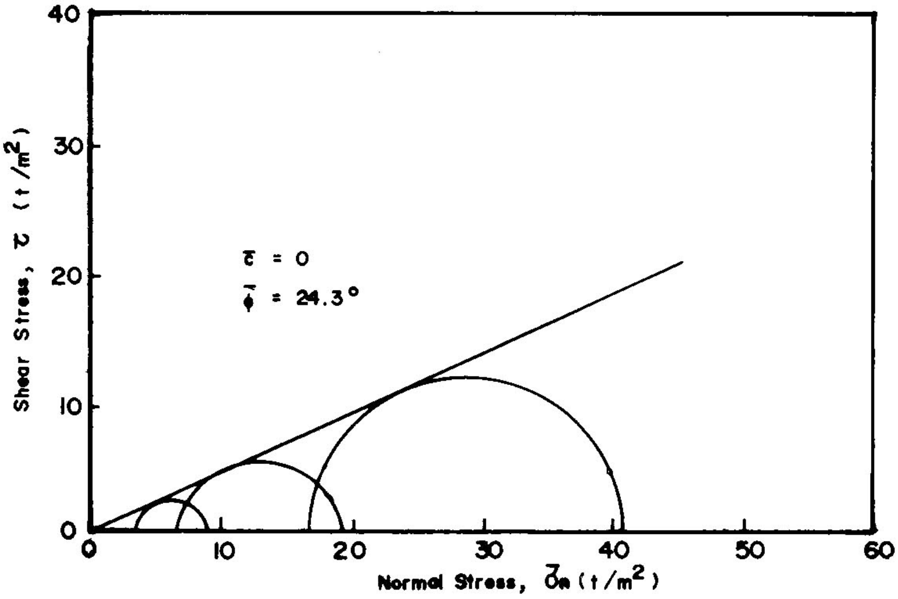

Fig. 14 Mohr-Coulomb Failure Envelope for Natural Soft Clay

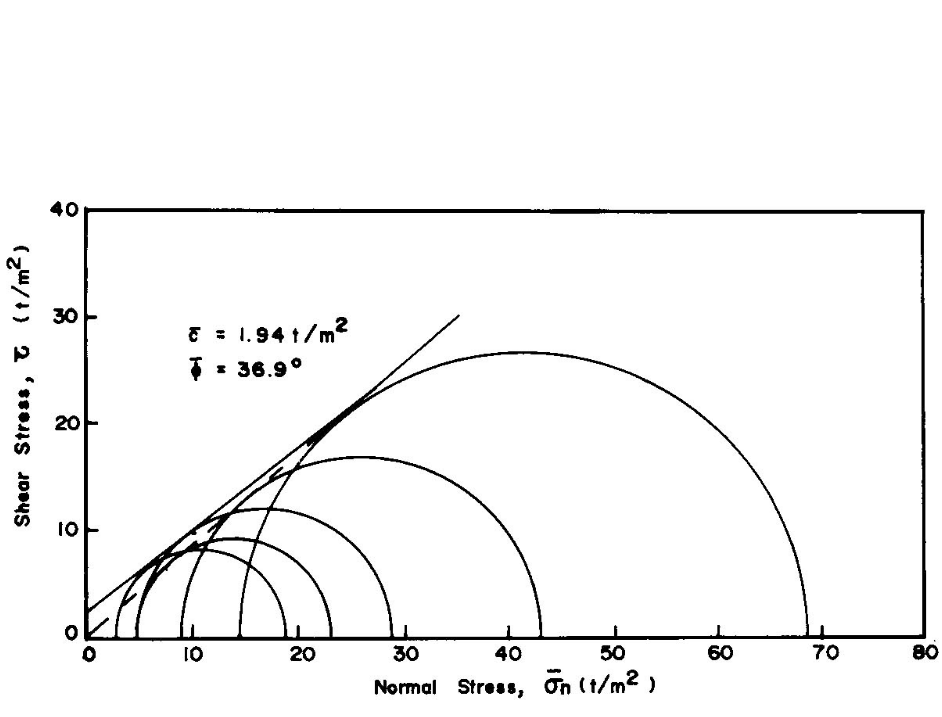

Fig. 15 Mohr-Coulomb Failure Envelope for Lime-Treated Clay (5% Lime Content)

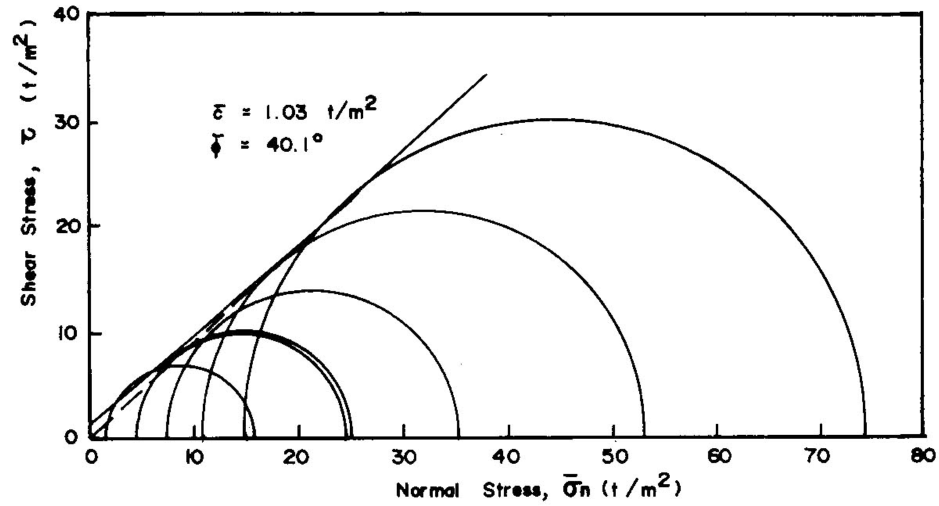

Fig. 16 Mohr-Coulamb Failure Envelope for Lime-Treated Clay (15% Lime Content)

The $\bar{c}$ value increases with increase in lime content. At 5% lime content, a $\bar{c}$ value of 1.71 t/m^2 is observed as compared to a zero value for untreated samples. The increase in friction angle is very substantial from 24.3° for untreated samples to 36.9° for samples with 5% lime content. This observation is very interesting in the sense that the addition of lime seems to improve the frictional behaviour of clays. Thus the long term stability and bearing capacity of lime-treated clays would be much better than the untreated clay.

CONCLUSIONS

Based on the experimental study, the following conclusions can be made:

(i) A lime content of 5% is sufficient to increase the unconfined strength 4 to 5 times that of the untreated clay.

(ii) A 5% lime content increased the preconsolidation pressure from 7 t/m^2 to 22t/m^2.

(iii) The coefficient of consolidation of the lime-treated clay increased about 10 to 40 times depending on the stress level.

(iv) The effective stress and the stress-strain behaviour of lime-treated clay exhibit relations which are similar to overconsolidated clay. Thus, less pore pressures are developed and higher undrained moduli are achieved.

(v) The lime-treated clays show a substantial increase in the effective strength parameters, especially the angle of friction, which increased from 24° to 40° by lime treatment.

NOTATION

C_c = compression index
$\bar{c}$ = cohesion
c_v = coefficient of consolidation
e = void ration
$\bar{p}$ = mean normal stress = $(\bar{\sigma}_1+2\bar{\sigma}_3)/3$
q = deviator stress = $(\sigma_1 - \sigma_3)$
q_u = unconfined compressive strength
u = pore pressure
Δu = excess pore pressure
w = water content
$\bar{\sigma}_1$ = effective major principal stress
$\bar{\sigma}_3$ = effective minor principal stress
$\bar{\sigma}_{vm}$ = preconsolidation pressure
σ_c = pre-shear consolidation pressure
ϵ = axial strain
$\bar{\phi}$ = angle of internal friction

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