PREDICTING STRENGTH INCREASE OF SOFT SOIL BY LIME-FLY ASH PILES

A PREDIRE L'AUGMENTATION DE LA FORCE DU SOL DOUX PAR DES PIEUX DE CENDRES VOLANTES CALCAIRES

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SYNOPSIS: This paper describes a method to predict the strength increase of soft soils by means of lime-fly ash piles. The method is based on an analytical model of soil behaviour and demonstrated by a full scale experiment involving 56 lime-fly ash piles installed in a soft sensitive clay near Ottawa, Ontario. The model uses an elasto-plastic stress-strain relationship and the associated physical phenomena related to pore pressure generation and induced horizontal stress. One month after the lime pile installation, the strength increase in the field was measured using a piezocone. Other test devices including a Menard pressuremeter have been used to obtain parameters to predict the strength increase. Comparison of the predicted and measured strength increases shows the following:

1) The proposed method of prediction gives good estimates of the horizontal stress increase as a function of the radial distance from the interface between the pile and the soil.

2) The proposed method also gives good estimates of the strength increase as a result of the increase in the effective horizontal stress.

3) This method of prediction has the potential of giving a reasonable tool for designing lime-fly ash piles to improve the strength of soft soils.

INTRODUCTION

There is an increasing interest in improving soft soils as more and more earth dams, embankments, highways, bridges, retaining walls and light buildings have to be built at sites with soils too soft for the engineering applications. The lime column or pile method has been proven to be technically and economically advantageous in several types of soft soil improvement. Although the fundamental aspects of lime pile treatment are not completely known, it has been widely used in soft soil improvement. There are two types of lime columns based on the method of installation. One type of lime column, the in-situ mixing lime column, can be produced by mixing unslaked lime with the soft soil by means of special equipment (Broms 1982, Holm et al. 1983). Another type of lime column, the in-situ compacted lime column, is used in China, India, Japan, and Taiwan, where unslaked lime is placed and compacted in predrilled holes rather than mixing the lime with the surrounding clay (Shi et al. 1989).

An important question in designing a lime pile project is to predict the shear strength increase in the treated soils. The strength increase results from two major effects. The first is the expansion of pile causing consolidation and hence strength increase in the surrounding soil. Second is the cementation effect between the soil particles. This takes a long time to achieve (Clark and Morriss 1990). It has been found that the strength gain due to expansion effect is one of the major benefits of in-situ compacted lime pile for improving soft soils. Since the magnitude of strength increase of soft soil depends largely on that of effective stress increase, it is important to predict the horizontal effective stress increase resulting from the dissipation of excess pore pressure generated by the pile expansion. A model proposed by Law and Chen (1993) has been shown to give a good estimate of excess pore pressure due to lime pile slaking. The model is extended in this paper to predict the strength increase of treated soil due to the expansion effect of lime pile treatment. A full scale experimental study was carried out involving 56 lime-fly ash piles installed in a soft sensitive clay. The proposed method is examined by field tests including piezocone and Menard pressuremeter tests.

PROPOSED METHOD TO DETERMINE STRENGTH INCREASE (ΔS_u)

A relationship between the undrained shear strength (S_u) and the drained strength parameters, c', cohesion in terms of effective stress, and ϕ', angle of internal friction in terms of effective stress, was given by Leonards (1962) as:

$$S_u=\frac{c'\cos\phi'+\sigma'_{vo}\sin\phi'[K_0+A_f(1-K_0)]}{1+(2A_f-1)\sin\phi'} \tag{1}$$

where K_0 = the coefficient of earth pressure at rest, σ'_{vo} = the effective overburden pressure, and A_f = pore pressure coefficient at failure.

It can be seen from Eq.1 that the undrained shear strength of the soil at a certain depth, for a given value of A_f, is dependent on the values of c' and ϕ', and the magnitude of the horizontal effective stress. If the horizontal effective stress increases as a result of some processes, such as lime pile slaking or pile driving, there will be an increase in undrained shear strength (ΔS_u). The expansion due to lime pile slaking occurs under the plane strain condition in the vertical direction. For the soil studied here, this condition leads to A_f = 0.5 (Law and Bozozuk 1979). The expression for S_u for A_f = 0.5 can be derived from Eq.1 as follows:

$$S_u=c'\cos\phi'+p'\sin\phi' \tag{2}$$

where $p' = (\sigma'_1+\sigma'_3)/2$; σ_3' = the horizontal effective stress; and σ'_1 = the effective overburden pressure (σ'_{vo}) assumed unchanged before and after treatment. Before treatment, $\sigma'_3 = K_0\,\sigma'_{vo}$, and after treatment, $\sigma_3' = (K_0\,\sigma'_{vo} + \Delta\sigma'_h)$. The values of c' and ϕ' can be easily determined from the laboratory tests prior to treatment. The horizontal effective stress increase due to lime pile slaking may be estimated from following proposed method.

Horizontal Effective Stress Increase ($\Delta\sigma'_h$)

The horizontal effective stress increase is assumed to be equal to the dissipated portion of the excess pore pressure (Δu) generated by lime pile slaking. Based on the expressions for Δu derived by Law and Chen (1993), the final $\Delta\sigma'_h$ after complete excess pore pressure dissipation can be estimated in the following for two different situations. Before soil yielding or during elastic deformation, $\Delta\sigma'_h$ is given by:

$$\Delta\sigma'_h=\Delta u=\frac{\Delta\sigma_r(r_1,t)}{G(\frac{r_2^2}{r_1^2}+1)+\lambda}\cdot[\lambda+G(1+\alpha\frac{\sqrt{6}}{3}\cdot\frac{r_2^2}{r^2})] \qquad (3)$$

where r_1 = the radius of lime pile, r_2 = the radius of influence of pile, $\Delta\sigma_r(r_1, t)$ = the total radial stress increment at the wall of cavity, α = the Henkel pore pressure parameter depending on the loading conditions and stress levels, $G = 1/2(E'/(1+\nu'))$, and $\lambda = E\nu'/((1+\nu')(1-2\nu'))$, E' and ν' = drained Young's modulus, and drained Poisson's ratio, respectively. Law and Bozozuk (1979) carried out triaxial and plane strain tests on soft sensitive marine clay from sites around the Ottawa valley, Ontario. They suggested that a below-elastic pore pressure response ($\alpha < 0$) prevails at the prepeak stage of shearing and an elastic response ($\alpha = 0$) at the peak. The functional relationship can be written as, using α as defined here:

$$\alpha=0.72(R-1) \qquad (4)$$

where R is the shear stress level defined by the ratio of shear stress to the shear strength.

If the soil reaches yield, a plastic or yielded zone will be created within which $\Delta\sigma'_h$ is given by:

$$\Delta\sigma'_h=\Delta u=S_u\cdot[\frac{\zeta^2}{1-2\nu'}+2\ln(\frac{r_u}{r})] \qquad (5)$$

where S_u is the undrained shear strength of the soft soil, and r_u is the radial distance from the pile axis to the elastic-plastic boundary. The value of r_u can be estimated from:

$$\zeta=\sqrt{\frac{\Delta\sigma_r(r_1,t)}{S_u\cdot[G(\frac{r_2^2}{r_1^2}+1)+\lambda]}} \qquad (6)$$

where $\zeta = r_u/r_2$.

The total radial stress increase is estimated from:

$$\Delta\sigma_r(r_1,t)=\sum_{t=0}^{t=t}\Delta\sigma_r(r_1,\Delta t) \qquad (7)$$

where $\Delta\sigma_r(r_1, \Delta t)$ represents the relationship between the radial stress increment at the wall cavity and the volumetric strain increment of the cavity due to lime pile expansion over a time interval of Δt and is given by:

$$\Delta\sigma_r(r_1,\Delta t)=\frac{C_1\Delta\mu}{C_3[(1+\mu_p C_1)C_5\mu+(1-\mu)]+\frac{1}{K_1}} \qquad (8)$$

where C_1 = the volumetric expansion under atmospheric conditions; C_3 = the compressibility of quicklime; C_5 = the ratio of the compressibility of the slaked lime to the quicklime; μ = the volume ratio of quicklime which has reacted with water to the initial volume of the cavity; $\Delta\mu$ = the volume ratio of quicklime which has reacted with water within Δt to the initial volume of the cavity; μ_p = the ratio of the actual expansion to the unrestrained expansion at the same stage of hydration; and K_1 = the stiffness coefficient of soil ring given by:

$$K_1=\frac{G(\frac{r_2^2}{r_1^2}+1)+\lambda}{\frac{r_2^2}{r_1^2}-1} \qquad (9)$$

where r_1, r_2, G and λ are given as above. The soil ring is the annular zone of soil affected by a pile within the pile group.

EXPERIMENTAL STUDY

Test Site and Test Program

An experimental study at a soft sensitive clay site subjected to strengthening by means of the lime-fly ash pile method was conducted to study the use of the proposed method for predicting the strength increase of treated soil. The test site is located at 21 km south-east of Ottawa. The soil profile at the site consists of 0.2 m thick top soil followed by a crust of silty sand and silty clay to a depth of 1.4 m. Underlying the crust is a soft sensitive clay extending to a depth of 20 m. Below this depth lies a hard layer of fill. The range of plasticity index is from 20 to 30 % and the average clay content is about 80 %. The watertable is at 1.0 m below the ground level. Geotechnical profile of this site (Bozozuk and Leonards 1972) is shown in Fig.1. The clay is slightly overconsolidated with a ratio (OCR) of 1.5. The average water content of soft clay below the watertable up to a depth of 5.5 m is about

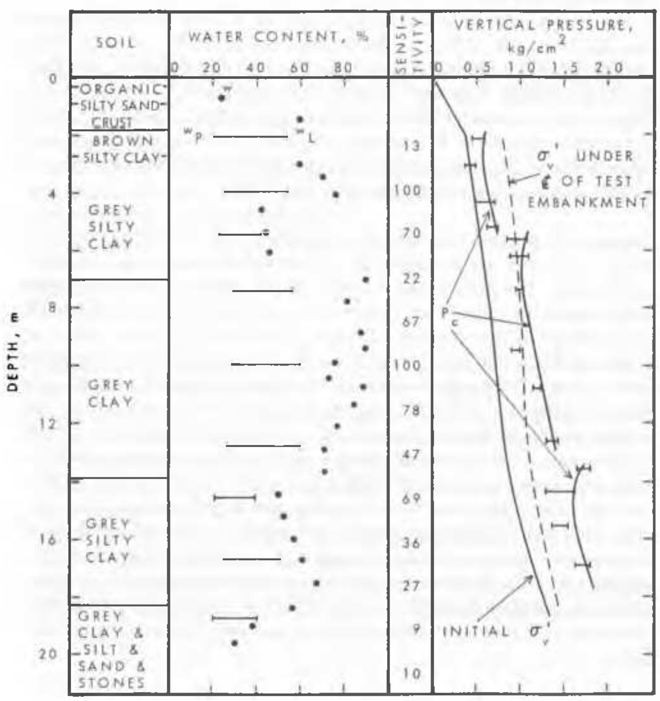

Fig.1. Geotechnical profile of test site

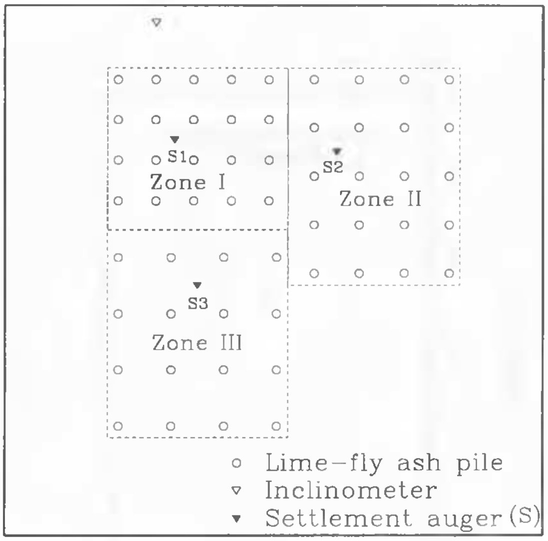

Fig.2. Layout of lime-fly ash pile and instrumentation

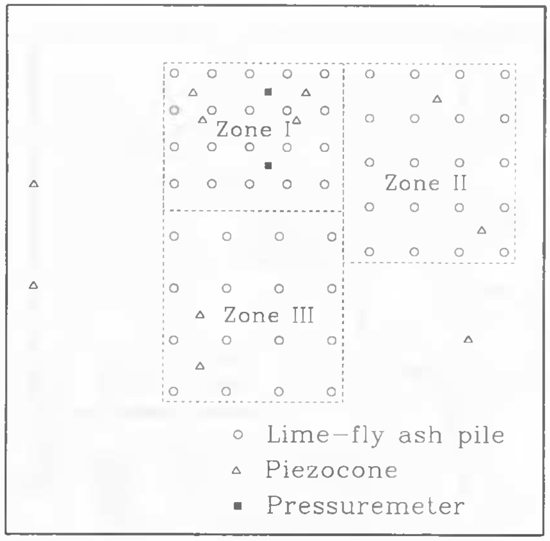

Fig.3. Layout of piezocone and pressuremeter tests

60%, which is well above the liquid limit of the clay, and the average vane shear strength of the soil above 5.5 m is lower than 20 kPa while that below this level is higher than 30 kPa.

Based on the above information, it was decided to strengthen the soft soil down to 5.3 m below the ground surface. The total length of the lime-fly ash pile with a diameter of 0.28 m was 5.3 m including a 1.3 m thick top clay plug. Fly ash was added not only to improve the effects of strengthening weak soils by the lime pile method but also to develop potentially beneficial uses for coal ashes as positive alternatives to conventional waste disposal. Fig.2 shows the plan of the site which is composed of three square zones, each with a different spacing of the lime-fly ash pile (at a volume of ratio of 7 to 3). The in-situ compacted lime pile method was employed here.

Field Tests

Menard pressuremeter test

A Menard pressuremeter was employed in order to determine the horizontal effective stresses and the drained Young's modulus both before and after treatment.

A total of 6 Menard pressuremeter tests were conducted with the test locations shown in Fig.3. Test equipment and test procedure have been fully described by Baguelin et al. (1978). The pressuremeter test was conducted according to ASTM Standard D4719-87. The pressuremeter tests were carried out at an interval of 0.75 m of depth starting from about 1.5 m to 5 m.

The experience on determining the total horizontal stress at rest from Menard pressuremeter (Baguelin et al. 1978, Mair and Wood 1987) suggested that a borehole should be prepared properly with the disturbance of the borehole wall kept to a minimum and that the diameter of the borehole should lie within a certain limit. To achieve these, the following measures have been taken during the test: a) a thin-wall sampling tube with a diameter slightly larger than that of the pressuremeter was used for forming the hole for the pressuremeter tests, b) the sampling tube was carefully pushed by hand to the desired depth to reduce the disturbance to the soil around the hole, c) the time between completion of the hole and insertion of the pressuremeter was minimized to prevent any potential problem caused by partial collapse or other forms of deterioration of the borehole wall, and d) a small pressure increment of 5 kPa was used at the beginning of the test in order to obtain the best estimate of total horizontal stress.

The result of a typical pressuremeter test is shown in Fig.4 where the corrected pressure is plotted against the corrected volume change. The total in situ horizontal stress (σ_{ho}) is estimated from the corrected pressuremeter curve based on the standard method of assuming σ_{ho} equal to the start of the linear region of the pressure-volume curve (P_{oM}). The coefficient of earth pressure at rest, K_0, is obtained from:

$$K_0=\frac{\sigma_{ho}-u_0}{\sigma_{vo}-u_0} \tag{10}$$

where σ_{vo} is the total vertical stress at rest and u_0 is the in situ hydrostatic pore water pressure.

The pressuremeter modulus is determined according to ASTM Standard D4719-87 as follows:

$$E_p=2\,(1+\nu)\,(V_0+V_m)\,\frac{\Delta P}{\Delta V} \tag{11}$$

where E_p = pressuremeter modulus, ν = Poisson's ratio, a value of 0.33 is recommended, V_0 = volume of the measuring portion of the uninflated probe at zero reading at ground surface (535 cm^3 for this test), V = corrected volume reading of the measuring portion of the probe, P = corrected pressure increase, $\Delta P/\Delta V$ = slope of the pressuremeter curve between P_{oM} and P_f, (see Fig.4), and V_m = corrected volume reading under the pressure $P_m = (P_{oM} + P_f)/2$.

The drained Young's modulus, E', was calculated using the following equations:

$$E'=\frac{2\,(1+\nu')\,E_u}{3}=\frac{2\,(1+\nu')\,E_p}{3} \tag{12}$$

where E_p is the pressuremeter modulus or the Menard modulus estimated from Eq. 11.

The results of the Menard pressuremeter test are summarized in Table 1.

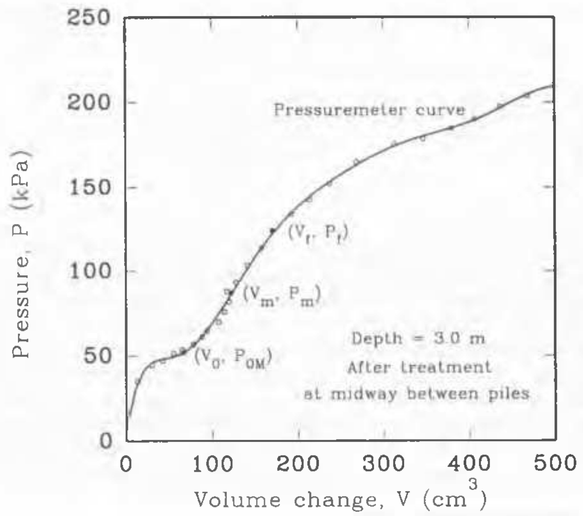

Fig.4. Result of a typical Menard pressuremeter test

Table 1. Summary of Pressuremeter Test Results

Depth (m)	σ'_{ho} (kPa)	K_0	G_M (MPa)	E_M (MPa)	E' (MPa)	σ_{ho} (kPa)
Before treatment						
2.25	29.5	1.02	0.17	0.45	0.40	37.0
3.0	22.0	0.67	0.25	0.67	0.59	37.0
3.5	26.0	0.72	0.26	0.69	0.61	46.0
After treatment						
2.25	41.4	1.46	0.46	1.22	1.09	50.0
3.0	37.0	1.12	0.88	2.34	2.08	52.0
3.5	41.0	1.03	0.35	0.93	0.83	61.0

When dealing with pressuremeter test result analysis, difficulty was encountered in estimating the total horizontal stress below a depth of 4 m at the site. This was possibly due to partial collapse of the borehole at such depths. However, the values of K_0 within a depth of 3.5 m obtained in this method are reasonable and consistent with the results of hydraulic fracture tests at the same site conducted by Bozozuk (1974). His results show that K_0 is not a constant but varies from greater than one at a depth of 2.4 m to 0.64 at a depth of 5.3 m. A comparison of K_0 between the present test results and the Bozozuk's data is shown in Fig. 5. It can be seen that the two different test methods for measuring in situ horizontal effective stress yield almost the same results within a certain limit of depth.

Piezocone penetrometer test

A total of 11 piezocone tests have been used to determine the undrained shear strength of the soil at the site and their locations are shown in Fig. 3. The tests were carried out at different times. The results of the tests one month after treatment were used in this paper to examine the strength increase as a result of the expansion effect.

The undrained shear strength (S_{uc}) from the piezocone test may be estimated from Konard and Law's model (1987):

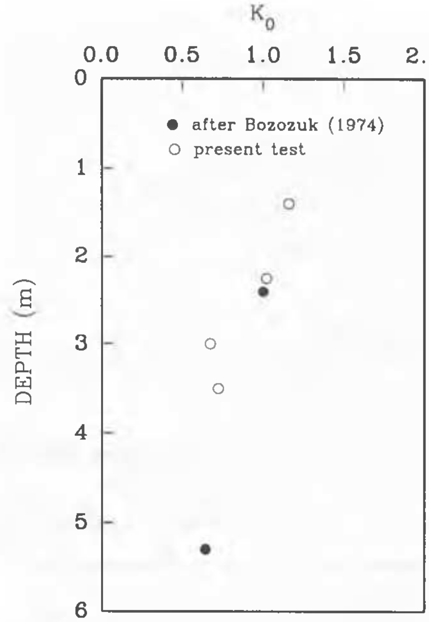

Fig.5. Comparison of K_0 with published data

$$S_{uc}=\frac{q_t-(1+F)\,\sigma_i+F\alpha u}{\frac{4}{3}(1+F)\,(\ln I_R+1)} \tag{13}$$

where $q_t = q_c + (1 - a)u$, $F = M \tan\phi' \cot\theta$, σ_i = the in situ total octahedral stress, α = a factor in the range of 1 to 1.1 for soft soils, I_R = the rigidity index, q_c = the measured tip resistance, u = the measured pore pressure. According to Konard and Law (1987), the following parameters were chosen for the site:
$a = 0.64$, $M = 0.75$, $\phi' = 27^0$, $\theta = 30^0$, $\alpha = 1.1$, $I_R = 130$, $\sigma_i = \sigma_{vo}$.
Fig.6 shows the strength profiles determined based on the above parameters for both before and one month after treatment. It can be seen that the strength of soil increases after treatment and the strength gain at a distance of one-fourth of pile spacing from the pile wall is greater than that at midway between piles.

PREDICTION AND COMPARISON

The values of horizontal effective stress increase at different depths have been estimated based on the method proposed earlier in this paper using the following parameters determined from appropriate techniques:
$C_1 = 0.35$, $C_2 = 0.21$, $C_3 = 2.74$ (MPa)$^{-1}$, $C_5 = 0.25$, $k_h = 1 \times 10^{-9}$ m/s, Poisson's ratio = 0.3, $r_1 = 0.14$ m, $r_2 = 0.40$ m. Other parameters such as the drained Young's modulus are shown in Table 1. The initial shear strengths before treatment at the site are based on the work of Bozozuk and Leonards (1972). Fig. 7 shows a comparison of horizontal effective stress increase between the predicted and the measured. It indicates that the predicted is in very good agreement with the measured. The difference between the predicted and the measured is less than 7%.

The strength increase for different depths have also been estimated using the proposed method. The values of c' and ϕ' were obtained from strength envelope shown in Fig. 8 (Law 1974). Fig. 9 shows a comparison of the strength increase between the predicted and the measured for the pile spacing of 70 cm. It can be seen that the predicted values at midway between piles are very close to the measured values. At a distance equal to one-fourth of the pile spacing, the agreement between the predicted and the measured

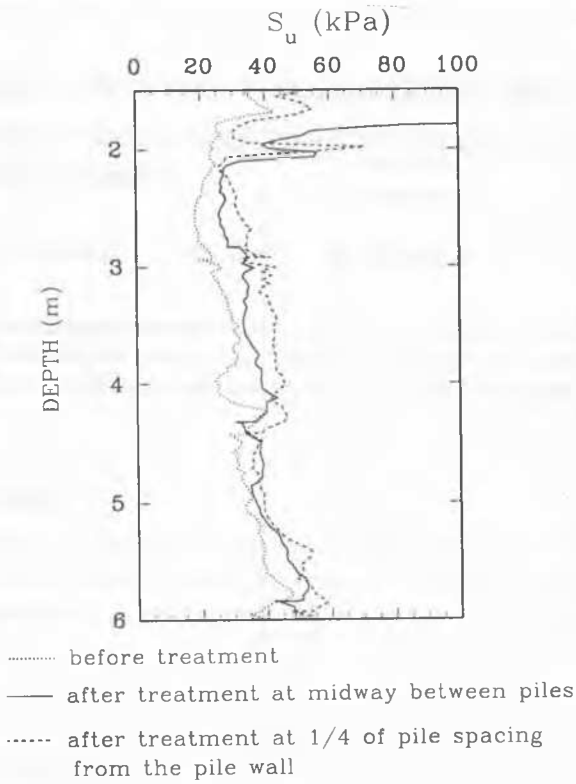

Fig.6. Comparisons of strength profiles between before and after treatment

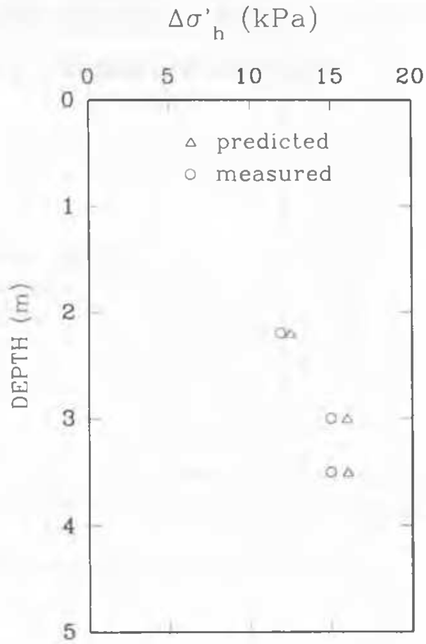

Fig.7. Measured and predicted $\Delta\sigma'_h$

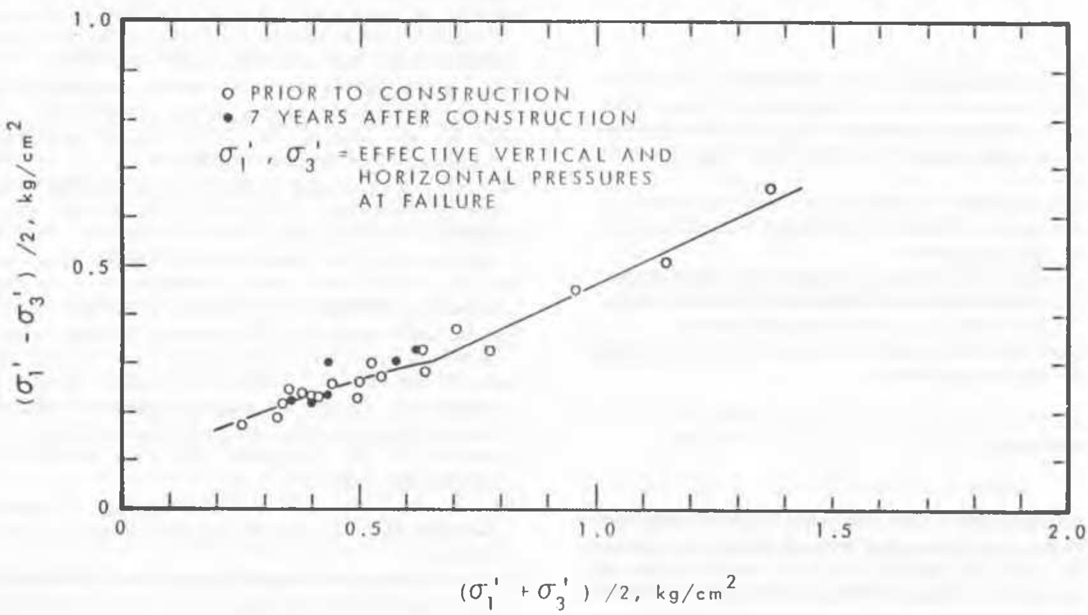

Fig. 8. Strength envelope for soil layer between depths 2.3 m and 5.3 m at the site (after Law 1974)

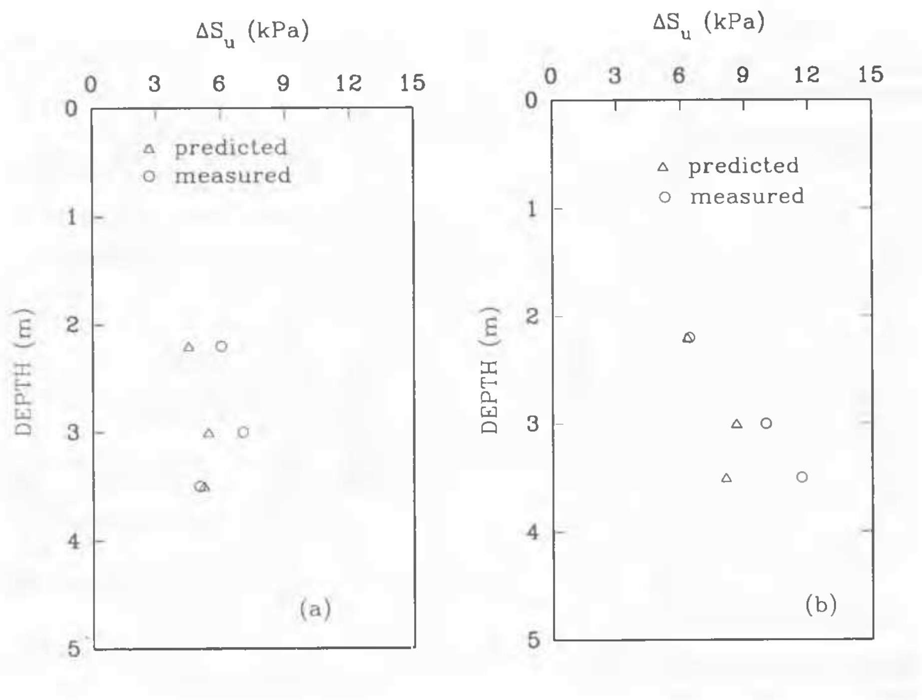

Fig.9. Comparison of predicted and measured ΔS_u at (a) midway between piles and (b) quarter of 70 cm pile spacing from the pile wall

values is acceptable though not as good as those at midway between piles. The overall predicted value is about 17% lower than the measured value. This slight underestimation errs on the safe side and is acceptable from a practical point of view. Therefore this method has the potential to become a useful tool for designing lime piles for strengthening soft soils.

CONCLUSIONS

A method based on an analytic model of soil behaviour is proposed for predicting the strength increase of soft soil strengthened by lime-fly ash piles. The validity of the method was substantiated by a full scale experiment involving 56 lime-fly ash piles installed in a soft sensitive clay. The study shows the following:

1) The proposed method of prediction gives good estimates of the horizontal stress increase as a function of the radial distance from the interface between the pile and the soil.

2) Since the component of strength increase due to expansion effect is caused by the horizontal effective stress increase, the proposed method also gives good estimates of this component of strength increase.

3) This method of prediction provides a reasonable tool for designing lime-fly ash piles for improving soft soils.

ACKNOWLEDGEMENTS

The work described in this paper is partly supported by an operating grant of the Natural Science and Engineering Research Council of Canada (No.8741). The field work and laboratory tests were conducted with the assistance and resources of the Institute for Research in Construction (IRC), National Research Council Canada. The authors are particularly indebted to T. Hoogeveen and A. Laberge, both of IRC, for their conscientious help in the field and in the laboratory. Dr. H. Rainer, Head of Structures Laboratory, IRC, gave invaluable support during the study.

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