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**MODELLING THE EFFECTS OF LIME-FLY ASH COLUMNS
INSTALLED IN SOFT CLAYS**

by
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B.Sc.(Hons), Taiyuan University of Technology, China
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A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of
DOCTOR OF PHILOSOPHY
in
Department of Civil and Environmental Engineering*

Carleton University
Ottawa, Canada K1S 5B6
July 21, 1995

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Modelling the Effects of Lime-Fly Ash Columns Installed in Soft Clays

submitted by

Pingan Chen

in partial fulfilment of the requirements
for the degree of Doctor of Philosophy

Chairman

Department of Civil and Environmental Engineering

Thesis Supervisor

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Abstract

An analytical model for the lime column expansion has been developed on the basis of cylindrical expansion theory for an elasto-plastic soil material. The coupled effects of lime column expansion and consolidation are considered through the boundary conditions of the lime column expansion. A numerical solution for estimating the increases in effective stress, static shear strength and dynamic shear modulus has been developed. A parametric study has been conducted to study the relative importance of soil parameters and lime column parameters used in the proposed model. Two case studies were carried out to demonstrate the validity of the model. A full-scale experimental study involving the installation of 56 lime-fly ash columns was conducted to verify the method developed for estimating the increases in effective stress, static shear strength, and dynamic shear modulus. A laboratory investigation was also carried out to determine the lime parameters required for the analysis and the comparative study.

The parametric study shows that the excess pore pressure distribution along the radial direction is non-linear and the magnitude of the undrained pore pressure is a function of the undrained shear strength, the drained Young's modulus and the

drained Poisson's ratio of the soil, and the spacing of the lime columns.

The case studies suggest that the model developed could reasonably represent the process of lime column expansion, and generation and concurrent dissipation of excess pore pressures.

The full-scale experimental study shows that the lime-fly ash column produces a substantial increase in horizontal effective stress, static shear strength, and dynamic shear modulus, and therefore is applicable to improve not only the static shear strength but also the dynamic shear modulus of soft sensitive clays.

Comparisons between the estimated and the measured from the full-scale experimental study and two case studies show that the increases in horizontal effective stress, static shear strength, and dynamic shear modulus can be reasonably calculated from the proposed method.

The results of this study suggest that the proposed model has a high potential of becoming a useful tool for the design of lime-fly ash columns to improve soft soils, including soft sensitive clays.

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Chapter 1

Introduction

1.1 General

Sensitive clay deposits are widespread in the eastern Canada and appear to be very troublesome in general engineering applications. Increased construction of earth dams, embankments, highways, bridges, retaining walls, and light buildings on these sites has generated an interest in improving the engineering characteristics of these soft soils. It has been known that lime-fly ash column may be successfully used in improving the static performance of the soil. At a site, moreover, especially in earthquake zones, it may be of interest whether the dynamic performance of the site can also be improved by the use of the lime column method.

Although the application of the lime-fly ash column method has been employed extensively in Sweden, China and Japan, still many practical uncertainties remain

concerning this treatment. The most important is the static and dynamic strength increases of the treated soil. The improvement of the soil by the use of lime-fly ash column is mainly due to the expansion effect of the lime columns. The expansion of lime columns initiated by lime slaking causes the soil to consolidate horizontally. This quick soil consolidation process results in an increase of the effective stress. Since the magnitude of the increase in soil strength depends largely on the increase in effective stress, it is essential to study the increase in effective stress resulting from the dissipation of excess pore pressures during the lime column expansion.

The ability to determine the increases in effective stress plays an important role in engineering applications. In order to achieve such estimations, a comprehensive analytical approach taking into account the true behaviour of the treatment is required.

In the present study, the expansion of lime columns for strengthening a soft soil has been modelled by the cylindrical cavity expansion in an elasto-plastic material. This soil model couples the consolidation theory in describing the time-dependent process of increasing total stress with the generation and concurrent dissipation of excess pore water pressure. In order to obtain the accurate distributions of total stress and excess pore pressure as a function of time and space, equations have been derived and solved by a finite difference scheme.

Based on the effective stress changes in the treated soil, a method is proposed for the estimation of static shear strength and dynamic shear modulus of soil in terms of stress changes.

A full-scale experimental study, involving the installation of 56 lime-fly ash columns in a soft sensitive clay in Ottawa, Ontario, was carried out. The objectives were to study the feasibility of using the lime-fly ash column to improve the soft sensitive clay at the site and to serve as a case study for assessing the proposed numerical solution. Comprehensive field tests consisting of seismic cone tests, piezo-cone penetrometer tests, and Menard pressuremeter tests were carried out at the test site before the lime column treatment and at different time intervals up to two years after the treatment. Resonant column tests were performed in the laboratory on undisturbed soil samples collected from the test site prior to and after the lime column treatment.

1.2 Scope of Research

1.2.1 Objectives

The main objective of this research is to develop a mathematical model to study the effect of expansion of a lime-fly ash column, including the effect of increases in both the static shear strength and the dynamic strength of the soils treated. The objectives can be summarized as follows:

1. to study the feasibility of using lime-fly ash columns for the improvement of a soft sensitive clay;

2. to investigate the quantity of the increase in shear strength and dynamic shear modulus of the soil induced by the use of lime-fly ash columns:
3. to study the degree of soil improvement in terms of soil properties' changes and lime column characteristics:
4. to develop an analytical solution for estimating the static shear strength and dynamic shear modulus of the improved soil; the solution will take into account the mechanics of the improvement process and the behaviour of the soil.

1.2.2 Outline of thesis

A comprehensive review of the improvement effects and applications of the use of lime columns is presented in Chapter 2. The methods for estimating the increases in strength are also included.

Chapter 3 presents the proposed soil model for the lime column expansion simulation and a numerical solution for estimating pore pressure generation, effective stress increase, and strength and dynamic modulus increases in the treated soil. The fundamental expression for radial consolidation in an elastic soil due to an expanding column is also derived.

A parametric study is presented in Chapter 4, in which the relative importance of various soil and lime column parameters is investigated. The proposed soil model is also examined by two case studies.

Chapter 5 focuses on the experimental study at the test site which has been treated by 56 lime-fly ash columns. Results of piezocone tests, Menard pressuremeter tests and seismic cone tests performed before and after the treatment are also reported.

The laboratory testing programme for the determination and comparison of the soil, lime and fly ash parameters used in the analyses is detailed in Chapter 6. Laboratory tests carried out at the Institute for Research in Construction (IRC), National Research Council of Canada (NRC) on soil samples collected from the test site before and after the treatment are also presented.

Chapter 7 highlights the evaluation of the proposed analytical approach for estimating the static shear strength and dynamic shear modulus increases of the treated soil, based on the comparison of the estimated and the experimental results.

Finally, Chapter 8 concludes the achievements of this research.

Chapter 2

Literature Review

2.1 General

Lime columns have been widely used for the improvement of soft soils in Sweden, Finland, Norway, China, Japan, India, etc. Broms (1991) reported that, in Sweden and Finland, about 25,000 lime columns were manufactured per year and almost 2 million metres had been installed since 1975, while in China, about 300,000 lime columns had been produced in the 1980s (Shi, 1989). The lime column method has, therefore, been proven to be technically and economically advantageous in various types of soils and applications.

2.2 Types of Lime Column Installation Methods

There are two major types of lime column installation methods, i.e., the in-situ compacted lime column method (compacted lime columns) and the in-situ mixed

lime column method (mixed lime columns).

The compacted lime column method is widely used in China, India, and Taiwan (Chiu and Chin, 1963; Holeyman et al., 1988; Shi et al., 1989). The column may be readily installed in soft soils using a special metal tube with a closed tip, driven to design depth, up to 8 m, by simple equipment. As a result, a hole is formed as the tube is withdrawn from the soil. The ready-made quicklime is placed and compacted in the holes rather than mixing the lime with the surrounding soil. The compacted lime column will then expand due to lime slaking. Inspection of the column one month after installation shows that the expansion results in consolidation of the soil surrounding the columns. The diameter of the compacted lime column can be 15 to 20 percent larger than the diameter of the lime column prior to the lime slaking. Shi et al. (1989) reported that the diameter of a column swells from 220 mm immediately after installation to 280 mm one month after installation. This volume increase in the lime column, together with a decrease in water content of the soil, are the major effects of the compacted lime column method. The length of a compacted lime column is usually limited to 2 m and the diameter ranges from 200 to 400 mm.

The mixed lime column method is well known in Sweden, Japan, and France (Broms and Boman, 1976 and 1977a, 1979b; Kitsugi and Azakami, 1982; Holm et al., 1983; Aboshi, 1984). The lime columns may be installed by means of special equipment as described by Broms and Boman (1979b). Figure 2.1 shows the mast and the rotary table mounted on a front wheel-loader. The unslaked lime is stored

in a container attached to the loader. The auger is screwed into the soil to a desired depth, then the unslaked lime is forced into the soil by compressed air, and mixed with the clay by a mixing tool as shown in Figure 2.2. The Japanese mixing bit is generally larger and more complex than the Swedish one. Thus a larger diameter (up to 175 mm) and a longer column (up to 60 m) can be obtained in Japan while the standard Swedish lime column normally has a diameter of 50 mm and a length of 10 m or less and is installed by means of light mobile equipment.

A new version of a quicklime column, the quicklime consolidated briquette (QCB), has been introduced to improve not only soft cohesive soils but also loose sandy soils (Ito et al., 1990, 1992). The QCB is a quicklime-cement mixture soil stabilizer. It is cast in the briquette machine under high pressure, as shown in Figure 2.3. The QCB method has been proven to be successful to improve a site of loose liquefiable sandy soils (Ito et al., 1992). One of the advantages of using the QCB is the reduction of noise due to vibro-compaction and dynamic compaction which are the common methods of soil improvement.

The effects of the improvement on both the lime column itself and the soil between the columns can be enhanced by adding silica; fly ash is the most often employed (Broms and Boman, 1976; Shi, et al., 1989). Similarly, rice husk ash (RHA) can also be considered as an excellent additive, since it consists of a high percentage (82-87%) of silica (Bartha and Huppertz, 1974; Ali, 1992; Ali et al., 1992). It is a major agricultural by-product obtained from the food-crop of rice paddy. Suprenant et al. (1990) reported that cement and fly ash (with a high lime content) can be

successfully applied to the treatment of oil pits or the most hazardous waste pits. The mixing of cementitious materials (cement, lime or fly ash) with waste can immobilize the oil wastes. Since the cement and fly ash or lime can maintain the waste at a high pH, in the range of 9-11, they lower the solubility of most multivalent cations (toxic heavy metals) in the waste, reduce the waste surface area exposed to the environment, and improve the handling characteristics and physical properties of the waste. The resulted hydrated cementitious products will chemically and physically bind the metal ions.

2.2.1 Effects of lime columns

The effects of the improvement are influenced not only by the type of soil but also by the type of lime, column geometry, and column spacing. To estimate the strength increase of the treated soil, a good understanding of the effects by the use of lime columns has to be obtained. The following are the basic aspects involved in lime column improvement:

1. Absorption of water from the soil during the hydration of the quicklime and filling its voids. The process starts immediately and takes about 2 to 10 days to complete (Shi et al., 1989; Holeyman et al., 1983). The resulting reduction of the water content of the soil around the bore-holes increases the strength and stiffness of the soil.

2. Substantial heat released during the slaking of the lime. The ground temperature within the treated soil may rise to about 50°C a few days after installation. This phenomenon was observed by Chen (1989) during an excavation of lime columns 7 days after installation. At the centre of the column, the recorded temperature was close to 300 °C within a few hours after the installation of the column as shown in Figures 2.4 and 2.5 (Kitsugi and Azakami, 1982). This increase in ground temperature may speed up the chemical reactions of both the lime and the clay. The high temperature also accelerates the evaporating process which results in a water loss.
3. Lime slaking leads to an apparent volume expansion (as shown in Table 2.1) which causes horizontal consolidation of the soil between columns. The microscopic study on soil improved by means of the lime-fly ash column method using mercury intrusion porosimetry and scanning electron microscopy techniques by Chen (1989) indicated that the volume decrease in pores due to the consolidation of soft soils caused by the treatment is mainly due to the changes in the volume of the largest pores. The changes represent a reduction in total porosity and a shift towards smaller pores. This consolidation is considered to be the major effect of the lime column method on the soil properties between the columns. Chen et al. (1989) and Shi et al. (1989) presented the enhanced consolidation effect on the treated soil due to the heat generation and the expansion of the column.

4. The chemical (pozzolanic) reaction (i.e., the reaction of the lime with the silicates and aluminum) results in products such as aluminum silicate and calcium silicon hydrates which are similar to the concrete paste that binds aggregates together. This process takes place in the clay adjacent to the column surface which forms a hardened zone around the column. It may also occur in the column itself when it contains silicates. This effect, however, is considered negligible since the migration of lime through an intact clay is a very slow process (about 10 mm/year)(Holeyman et al. 1983; Fang 1991).
5. Ion exchanges occur in the clay with cations such as sodium. The sodium absorbed by the clay mineral surfaces is exchanged with calcium. This process reduces the thickness of the double layer and hence improves the characteristics of the soil. It occurs only if the lime is in direct contact with the clay. Therefore, this effect is only significant for the soil adjacent to the column surface.
6. Carbonation is the reaction of quicklime with carbon dioxide in the open air or in the voids of the soil. The calcium carbonate (CaCO_3) formed is a water-stable aggregate. It has an additional effect on the soil since it prevents further pozzolanic reactions between lime and silicates or aluminum. This effect may be significant if there is a plenty of carbon dioxide presented in the soil. This carbonation is negligible since the lime column

method is only applied in the soil under the groundwater table, where little air exists in the voids of the soil, and also because an additive with high contents of silicates and aluminum, such as fly ash, is usually added to the lime column.

7. Diffusion of the slaked lime solution into the clay around the columns makes it possible to have carbonation effects and chemical reactions in the soil between columns.
8. Replacement of the soft soil by the lime column with a higher strength plays an important role in the overall improvement effect.

Since the processes mentioned above occur simultaneously, it is a complex problem to analyze. Therefore, some assumptions have to be made to facilitate analysis. These processes may be divided into three major effects.

First is the column expansion during the process of lime slaking which generates an excess pore water pressure in the soil between the columns, causing the soil adjacent to the columns to consolidate in the horizontal direction with the water taken from the soil into the column. This consists of processes 1, 2, and 3 as summarized in Table 2.1. The effect of increase in ground temperature and water loss is assumed to be negligible since substantial evaporation can take place only under a ventilated condition. The effect of temperature on the permeability of soil, however, should be taken into account and will be fully described in Chapter 7 of this thesis.

Chemical reaction	CaO	+	H ₂ O	=	Ca(OH) ₂
Molecular weight	56		18		74
Specific gravity	3.37		1.0		2.24
Weight ratio	1.0		0.32		1.32
Solid volume ratio	1.0		1.08		1.99
Apparent volume ratio	1.0		0.36		1.47-1.65

Table 2.1: Summary of lime column expansion

Note: (1). 1166 kJ of heat is generated by 1 kg of quicklime during slaking (Clark and Morriss, 1990);

(2). The apparent volume ratio depends on compacted unit weight of a lime column and its unit weight of slaked lime column. This ratio is an important quantity as it measures the actual volume expansion of a lime column. Before slaking, a unit apparent volume of quicklime contains a solid volume of 0.326 and a void volume of 0.673 based on a common design lime dry unit weight of 11 kN/m^3 . This will react with 0.36 volume of water to form a solid slaked lime volume of 0.69. This slaked lime however always exists with a certain void volume. For a design slaked lime unit weight (with the void volume) of 10 kN/m^3 , the void volume is 0.796 (with a void ratio of 1.20). Hence, the apparent volume of

the slaked lime increases to 1.47 (with the range of 1.47 - 1.65 if it is assumed that the compacted unit weight of the lime column is 11 kN/m^3 and the unit weight of the slaked lime column ranges from 10 kN/m^3 to 8 kN/m^3).

The cementation effect is the second major effect involved in processes 4, 6, and 7.

The third is the replacement effect involved in processes 5, 6, and 8, since the hardening of the annular zone around the column can be considered as part of the column. The replacement effects depend on the column stiffness and the replacement ratio defined as the ratio of the cross-sectional area of a lime column to the cross-sectional area of the influence of column.

The improved characteristics of the soil between the columns are mainly due to the first and second effects (Chen et al. 1989; Shi et al. 1989). The cementation effect is assumed to be negligible since it takes a long time to achieve, and the effect is only localized within a small zone near the column surface.

2.2.2 Applications of lime columns

The mixed lime column method was widely used in Sweden and Finland as column walls in deep excavations in soft soils instead of sheet-pile or support embankments or fills (Broms and Boman 1981; Sahlberg 1979). The mixed column method used in Japan is mainly applied to ports and harbours (Okumura and Terashi 1975; Terashi

and Tanaka 1981, 1983). The main applications of the compacted lime column method in China, Singapore and India are to improve the soft soil for supporting high-rise building, oil tanks, or chimneys (Shi et al. 1989). The purposes of the application of lime columns are summarized below:

1. to prevent bottom heaving of an excavation (Figure 2.6);
2. to prevent sliding failure of an excavated slope (Figure 2.7);
3. to increase the bearing capacity and reduce the total and differential settlements of soft soil in supporting tall buildings ranging from 8 to 12 storey high (Shi, 1989) (Figure 2.8);
4. to improve the slope stability of embankments (Figure 2.9);
5. to act as a vertical drain (Broms and Boman, 1977) considering that the permeability of the mixed lime column is about 100 to 1000 times higher than the untreated soil; and
6. to reduce both the total and differential settlements (Bredenberg, 1983; Holm et al. 1981, 1983).

2.3 Bearing Capacity of a Single Lime Column

The bearing capacity of a single lime column can be determined on the basis of the assumption of failure occurring either in the soil or in the column.

2.3.1 Soil failure mode

The bearing capacity of a single column is taken as the summation of both the toe resistance and the shaft resistance along the surface of a lime column in a soil failure mode, as shown in Figure 2.10. According to Broms et al. (1991), the shaft resistance is assumed to be equal to the undrained shear strength of the soil, S_u , and the toe resistance is assumed to be given by $9S_u$, then the bearing capacity of a single lime column, Q_{col} , is given by:

$$Q_{col} = (\pi d H_{col} + 2.25\pi d^2) S_u \quad (2.1)$$

where d is the diameter of the column, H_{col} is the length of the column and S_u is the average undrained shear strength of the soil surrounding the lime column.

2.3.2 Column failure mode

The bearing capacity of a single lime column in column failure mode is assumed to be dominated by the shear strength of the column material.

Assuming an angle of internal friction for the soil being 30° ,

Broms et al. (1990) proposed the following expression for estimating the bearing capacity Q_{col} at a depth of z :

$$Q_{col} = A_{col}(3.5c_{col} + 3\sigma_h) \quad (2.2)$$

where A_{col} is the cross-sectional area of a lime column, c_{col} is the cohesion of the

column and σ_h is the total lateral pressure at depth z .

2.4 Bearing Capacity of a Lime Column Group

Three possible assumptions may be made in determining the bearing capacity of a lime column group:

1. assume that the capacity of a lime column group is defined as the sum of the capacities of the individual columns in the group:
2. assume that the capacity of a lime column group is governed by the bearing capacity of the lime column block, as shown in Figure 2.11;
3. assume that the capacity of a lime column group depends on the local bearing capacity along the edge of the block.

In practice, assumption No. 1 is commonly applied to the determination of the capacity of a lime column group, which is given by:

$$Q_{group} = NQ_{col} \quad (2.3)$$

where N is the number of the columns in the group and Q_{group} is the bearing capacity of the group.

2.4.1 Block failure mode

The block failure mode of a lime column group may be assumed if the spacing of the lime columns is small (Figure 2.11). The bearing capacity of a lime column group failing in this mode is equal to the sum of the shaft resistance along the perimeter of the lime column group and the toe resistance of the block (Broms et al. 1991):

$$Q_{group} = 2S_u H_{col}(B + L) + (6to9)S_u BL \quad (2.4)$$

where B , H_{col} and L are the width, height and length of the column group, respectively.

2.4.2 Local failure mode

The bearing capacity with respect to the local failure mode, as shown in Figure 2.12, is governed by the average shear strength of the soil along the assumed circular failure surface and is given by (Broms et al. 1991):

$$q_{ult} = 5.5(S_u)_{ave}(1 + 0.2b/l) \quad (2.5)$$

where b and l are the width and length of the locally loaded area, and $(S_u)_{ave}$ is the average shear strength along the assumed failure surface.

2.5 Shear Strength of the Treated Soil

As shown in Equations 2.1, 2.4 and 2.5, the shear strength of the treated soil between columns has to be properly estimated in order to determine the bearing capacity of a lime column group.

Studies have been conducted on measuring the strength and deformational parameters of the lime column (Broms and Boman, 1979a, 1979b; Ahnberg et al., 1989; Shi et al., 1989). However, only little information is available from the literature on estimating the behaviour of the treated soil.

2.5.1 Existing method

According to Kitsugi and Azakami (1982), the cohesion of the soil after improvement can be determined as follows:

$$c' = c_0 + \alpha \Delta p \quad (2.6)$$

where c' = cohesion of the soil after improvement (kPa);

c_0 = cohesion of the soil before improvement (kPa);

α = coefficient of strengthening of soil ranging from 0.2 to 0.4;

Δp = estimated value of consolidation pressure from the $e - \log p$ curve based on the reduction of void ratio (Δe) in the soil due to lime column treatment (kPa).

$\Delta\epsilon$ may be estimated from the reduction in water content of the soil (Δw in %) expressed as:

$$\Delta\epsilon = \frac{G_s \times \Delta w}{100} \quad (2.7)$$

where G_s is the specific density of soil and Δw is given by:

$$\Delta w = \frac{1 + \frac{w_0}{100}}{\gamma} a_s [h \cdot \gamma_c (\frac{n'}{100}) (1 + \frac{\epsilon_v}{100}) (\frac{S'_r}{100}) \gamma_w] \cdot 100\% \quad (2.8)$$

where $\Delta w = w_0 - w'$ (%);

γ = unit weight of soil before improvement (kN/m^3);

w_0 = water content (%) of soil before improvement;

w' = average water content of improved soil (%);

a_s = area ratio of the lime column;

$$a_s = \frac{r_1^2}{r_2^2};$$

r_1 = radius of the lime column;

r_2 = radius of influence zone of the lime column;

h = equivalent of water absorption of the lime column;

γ_c = unit weight of the lime column (kN/m^3);

ϵ_v = coefficient of volumetric expansion of lime slaking (%);

S_r' = degree of saturation of the slaked lime column (%);

γ_w = unit weight of water (9.81 kN/m³);

n' = porosity of the slaked lime column (%).

2.5.2 Limitations of the existing method

The method proposed by Kitsugi and Azakami (1982) only gives an average value of the increased strength in the soil over the distance among the columns and cannot be used to estimate the dynamic shear modulus of the treated soils. However, the field measurements (Kitsugi and Azakami, 1982), as shown in Figures 2.13 and 2.14, indicate that both the improvement of the soil and the reduction in water content vary with the distance from the surface of a lime column. On the basis of the triaxial tests on undisturbed soil samples taken at various distances from the column surface, Shi et al. (1989) found that the closer to the column surface, the larger was the increase in strength of the treated soil.

Based on the above findings, one may conclude that it is essential to develop a method to estimate the increased shear strength and the increased dynamic shear modulus of the soil improved by the lime column method that can overcome all the limitations in the existing methods. This is one of the objectives of this study.

Figure 2.1: Lime column machine – mast folded for transportation (After Broms and Boman, 1979b)

Figure 2.2: Manufacture of the lime columns (After Broms and Boman, 1979b)

Figure 2.3: Briquette systems and dimensions (After Ito et al., 1990)

Figure 2.4: Time-temperature curve in peat (Based on data from Kitsugi and Azakami, 1982)

Figure 2.5: Time-temperature curve in sandy silt (Based on data from Kitsugi and Azakami, 1982)

Figure 2.6: Prevention of bottom heaving in excavation (Replotted from Kitsugi and Azakami, 1982)

Figure 2.7: Prevention of slope failure in excavation (Replotted from Kitsugi and Azakami, 1982)

Figure 2.8: Lime columns below building (Replotted from Broms and Boman, 1979)

Figure 2.9: Lime columns below embankment (Replotted from Kitsugi and Azakami, 1982)

Figure 2.10: Soil failure mode

Figure 2.11: Block failure mode

Figure 2.12: Local failure mode

Figure 2.13: Distribution of dry unit weight of the soil and cone index (Data from Kitsugi and Azakami, 1982)

Figure 2.14: Water content of soil with respect to the distance from column walls
(Data from Kitsugi and Azakami, 1982)

Chapter 3

Proposed Mathematical Model

3.1 General

As explained in Section 2.2.1, a lime column will expand into the soil due to a slaking process. The process of lime column expansion is similar to that of the pressuremeter test involving the expansion of a pre-existing cavity. Observations show that there is a substantial increase in both the pore water pressure and the stress in the treated soil as a result of lime column slaking. The magnitude of the excess pore pressure may even exceed the total overburden pressure (Shi et al. 1989). The excess pore pressure dissipates rapidly due to the suction effect of quicklime column slaking. The subsequent radial consolidation causes the effective stress to increase, which in turn leads to the strength increase of the soil as shown in Figure 2.13 (Kitsugi and Azakami, 1982). A good estimate of excess pore pressure and effective stress increases will enhance the ability to predict the strength increase of the treated soil. A cylindrical expansion theory has been used by Holeyman et al. (1983) to

model the process of lime column expansion in an elastic soil ring. The authors assumed a constant distribution of excess pore pressure along the radial direction. This assumption is far from the actual excess pore pressure generation during pile driving or pressuremeter tests. Lo and Stermac (1965) and Koizumi and Ito (1967) have reported that the actual excess pore pressure decreases approximately linearly with the logarithm of the radius from the pile axis.

To better describe the expansion of a lime column, a more complex but more realistic soil model (Chen and Law, 1993) was developed to represent the soil behaviour during and after column installation. Furthermore, it is assumed that the excess pore water pressure in the soil at a given point is a function of the total stress increase at that point.

3.2 Expansion of a Lime Column

A relationship between the radial stress increase at the soil-lime column interface and the volumetric strain of the cavity is first derived. A stiffness coefficient, K_1 , is introduced to express this relationship. As the radial stress at the wall of the cavity increases in response to the lime column expansion, a plastic zone may develop. The width of the plastic zone depends on the stress level at the cavity wall and can be estimated using an elasto-plastic model to be presented in Section 3.2.3.

Since the elastic solution is simpler, it is useful to compare the results between the elastic and the elasto-plastic models. Such a comparison on the radial stress

increments at the cavity wall has been made. The comparison suggests that an elasto-plastic model should be used to determine the stresses within the soil ring and the stiffness coefficient, K_1 , should take into account the plastic zone effect.

3.2.1 Radial stress increase at cavity wall

The following assumptions are made for modelling the quicklime column behaviour:

1. The quicklime column is considered infinitely long.
2. The surrounding soil is saturated, weightless, isotropic, and homogeneous.
3. The displacement at the symmetrical surface among the columns is zero.

Assumptions 1 and 2 suggest that a long cylindrical cavity expands in a saturated isotropic and homogeneous soil subjected to a plane strain deformation. Consequently, the expansion of a quicklime column considered herein is an overall axisymmetrical problem with the radial, circumferential, and vertical stresses on a soil element all being principal stresses. The zero displacement in assumption 3 occurs at a radius of r_2 defined as the equivalent radius of the influence of a column. The value of r_2 can be approximated by equating the area affected by the column to the equivalent circular area as shown in Figure 3.1.

For a square arrangement:

$$r_2 = \sqrt{\frac{1}{\pi}} s = 0.564s \quad (3.1)$$

For a triangular arrangement:

$$r_2 = \sqrt{\frac{\sqrt{3}}{2\pi}} s = 0.525s \quad (3.2)$$

For a hexagonal arrangement:

$$r_2 = \frac{1}{2} \sqrt{\frac{3\sqrt{3}}{\pi}} s = 0.643s \quad (3.3)$$

where s is the spacing of columns.

The relationship between the radial stress increment at the interface of the lime column and the soil to the volumetric strain of the cavity is derived on the basis of the above assumptions and with an initial assumption of a linear elastic soil behaviour. The relationship is expressed as:

$$\Delta\epsilon_{vc} = -\frac{\Delta\sigma_r(r_1, \Delta t)}{K_1} \quad (3.4)$$

where $\Delta\epsilon_{vc}$ = the volumetric strain of the cavity over the time interval, Δt ; $\Delta\sigma_r(r_1, \Delta t)$ = the radial stress increment at the soil-column interface within Δt ; K_1 = stiffness coefficient for the elastic case and can be modified later to incorporate the elastoplastic effects.

Note that the negative value of the volumetric strain of the cavity refers to the expansion of the lime column.

The expression for the radial stress increment at the cavity wall over the time interval, Δt , is derived from Equation 3.4 and Figure 3.2 based on deformation compatibility and is given by:

$$\Delta\sigma_r(r_1, \Delta t) = \frac{C_1 \Delta\mu}{(1 + \mu_p C_1) C_s \mu + (1 - \mu) C_q + \frac{1}{K_1}} \quad (3.5)$$

where C_1 = the free volumetric expansion coefficient;

C_q = the compressibility of the quicklime;

C_s = the compressibility of the slaked lime;

μ = the volume ratio of the quicklime that has reacted to the initial volume of the quicklime;

μ_p = the ratio of the actual expansion to the unrestrained expansion at the same stage of hydration;

K_1 = the stiffness coefficient of soil ring.

K_1 is given by:

$$K_1 = \frac{G(\frac{r_2^2}{r_1^2} + 1) + \lambda}{\frac{r_2^2}{r_1^2} - 1} \quad (3.6)$$

where

$$\lambda = \frac{E\nu}{(1 + \nu)(1 - 2\nu)} \quad (3.7)$$

$$G = \frac{E}{2(1 + \nu)} \quad (3.8)$$

and $\Delta\mu$ = the volume ratio of quicklime that has reacted with water within Δt and can be written as:

$$\Delta\mu = \frac{\Delta Q}{C_2 V_q} \quad (3.9)$$

where V_q = the volume of the initial quicklime column;

C_2 = the volume of water needed to react with a unit volume of a quicklime

(Holeyman, et al., 1983) with a typical value of 0.3;

ΔQ = the volume of water reacting with quicklime within Δt ;

E = Young's modulus of the soil ;

ν = Poisson's ratio of the soil.

For a given type of quicklime and diameter of lime column, C_2 and V_q are constant, hence $\Delta\mu$ is only a function of ΔQ which depends not only on the permeability of soil in the horizontal direction but also on the speed of the reaction front of lime slaking. ΔQ can be calculated from the expression:

$$\Delta Q = 2\pi r_1 \cdot v_s \cdot \Delta t \quad (3.10)$$

where v_s = the rate of slaking equal to the smaller value of the limits, v_{max} and v_h . v_{max} is the speed of the reacted front at which the slaking can proceed and v_h is the speed of lime column slaking based on the hydraulic gradient at the cavity wall.

The expressions for v_{max} and v_h are given in the form (Holeyman, et al., 1983):

$$v_h = -\frac{k_h}{\gamma_w} \cdot \left(\frac{\partial u_p}{\partial r} \right)_{r=r_1} \quad (3.11)$$

and

$$v_{max} = \sqrt{1 - \mu} \cdot v_{lim} \quad (3.12)$$

where k_h is the permeability of the soil; u_p is the excess pore water pressure; μ is the volume ratio of the quicklime that has reacted to the initial volume of quicklime as defined previously and v_{lim} is the constant limiting speed of the reacting front separating the slaked lime from the quicklime in a mass of lime in the presence of excess water.

The value of v_{lim} , as noted by Holeyman, et al. (1983), is a constant value of about 0.02 mm/sec.

The total radial stress increase at the wall of cavity at any time, t , is given by:

$$\Delta\sigma_r(r_1, t) = \int_0^{\mu_t} \frac{C_1 d\mu}{(1 + \mu_p C_1) C_s \mu + (1 - \mu) C_q + \frac{1}{K_1}} \quad (3.13)$$

where μ_t = the volume ratio of the quicklime that has reacted to the initial volume of the cavity at the time, t .

3.2.2 Stress distribution

The radial stress distributions within the soil ring depend on the magnitude of radial stress at the soil-column interface and the soil skeleton models used for the analysis.

Two models are considered here.

a. Linear elastic model

The soil with linear elastic, isotropic and homogeneous characteristics can be described by two parameters: Young's modulus, E , and Poisson's ratio, ν .

The equations for the radial stress and the circumferential stress increments have been derived using the linear elastic model and are given in the form:

$$\begin{aligned}\Delta\sigma_r(r,t) &= \frac{\Delta\sigma_r(r_1,t)}{K_2} \cdot [G(\frac{r_2^2}{r^2} + 1) + \lambda] \\ \Delta\sigma_\theta(r,t) &= -\frac{\Delta\sigma_r(r_1,t)}{K_2} \cdot [G(\frac{r_2^2}{r^2} - 1) - \lambda]\end{aligned}\quad (3.14)$$

where $\Delta\sigma_r(r_1,t)$ = the total radial stress increase at the soil-column interface and

$$K_2 = K_1 \cdot (\frac{r_2^2}{r_1^2} - 1) = G(\frac{r_2^2}{r_1^2} + 1) + \lambda \quad (3.15)$$

b. Elasto-plastic model

As the radial stress on the cavity wall increases in response to the lime column expansion, the soil adjacent to the wall may yield. Yielding first occurs around the cavity wall as a ring of yield or plastic zone. The plastic zone widens as the radial stress on the wall continues to increase. Since improvement by the lime column

method is mainly applied to saturated cohesive soils, it is appropriate to use the Tresca failure criterion:

$$\frac{\sigma_1 - \sigma_3}{2} = S_u \quad (3.16)$$

where σ_1 and σ_3 are the major and the minor principal total stresses, respectively and S_u is the undrained shear strength of cohesive soil. In this case, the major and the minor principal stresses are $\sigma_r(r, t)$ and $\sigma_\theta(r, t)$, respectively. Therefore, yield will occur when:

$$\begin{aligned} \sigma_r(r, t) - \sigma_\theta(r, t) &= [\sigma_r(r, t_0) + \Delta\sigma_r(r, t)] - [\sigma_\theta(r, t_0) + \Delta\sigma_\theta(r, t)] \\ &= \Delta\sigma_r(r, t) - \Delta\sigma_\theta(r, t) \\ &= 2S_u \end{aligned} \quad (3.17)$$

where $\sigma_r(r, t_0)$ and $\sigma_\theta(r, t_0)$ are the in-situ horizontal stress at time t_0 just prior to the column installation. In the plastic zone, both the yield criterion and the equilibrium equation for small deformation apply. This leads to the following differential equation:

$$\frac{\partial \Delta\sigma_r}{\partial r} + \frac{2S_u}{r} = 0 \quad (3.18)$$

The integration of the above equation gives:

$$\Delta\sigma_r(r, t) = D - 2S_u \ln(r) \quad (3.19)$$

where D is the integration constant and can be determined from the elastic-plastic boundary condition:

$$\Delta\sigma_r(r_u, t) = \Delta\sigma_{r_u} \quad (3.20)$$

in which $\Delta\sigma_{r_u}$ is the total radial stress increase at r_u , the radius of the elastic-plastic boundary.

Therefore, the expressions for the total radial stress increase and the total circumferential stress increase anywhere in the plastic zone become:

$$\begin{aligned} \Delta\sigma_r(r, t) &= \Delta\sigma_{r_u} + 2S_u \ln\left(\frac{r_u}{r}\right) \\ \Delta\sigma_\theta(r, t) &= \Delta\sigma_r(r, t) - 2S_u \end{aligned} \quad (3.21)$$

In the elastic zone, the expressions for the total radial stress increase and the total circumferential stress increase are the same as Equation 3.14.

The expression for $\Delta\sigma_{r_u}$ is derived here.

At the elastic-plastic boundary, the total radial stress and the circumferential

stress increases can be obtained from Equation 3.14:

$$\begin{aligned}\Delta\sigma_r(r_u, t) &= \frac{\Delta\sigma_r(r_1, t)}{K_2} \cdot [G(\frac{r_2^2}{r_u^2} + 1) + \lambda] \\ \Delta\sigma_\theta(r_u, t) &= -\frac{\Delta\sigma_r(r_1, t)}{K_2} \cdot [G(\frac{r_2^2}{r_u^2} - 1) - \lambda]\end{aligned}\quad (3.22)$$

Since the yield condition is reached at the interface of the elastic and the plastic zones, one can obtain the relationship between the undrained shear strength of the soil and the radial stress increase at r_u :

$$\Delta\sigma_{ru} = S_u \cdot (1 + \frac{\zeta^2}{1 - 2\nu}) \quad (3.23)$$

where $\zeta = r_u/r_2$. The value of ζ depends on the stress level at the cavity wall and on the shear strength of the soil:

$$\zeta = \sqrt{\frac{\Delta\sigma_r(r_1, t) \cdot G}{S_u \cdot K_2}} \quad (3.24)$$

Note that soil will not yield if the value of ζ is less than the ratio of r_1 to r_2 .

Substituting Equation 3.23 into Equation 3.21 leads to the following expressions for the total radial and the total circumferential stress increases in the plastic zone:

$$\begin{aligned}\Delta\sigma_r(r, t) &= S_u [\frac{\zeta^2}{1 - 2\nu} + 2\ln(\frac{r_u}{r}) + 1] \\ \Delta\sigma_\theta(r, t) &= S_u [\frac{\zeta^2}{1 - 2\nu} + 2\ln(\frac{r_u}{r}) - 1]\end{aligned}\quad (3.25)$$

The whole radial stress distribution can be obtained by means of Equation 3.14 and Equation 3.25 with the value of ζ determined from Equation 3.24. Figure 3.3 shows the radial stress distribution for the purely cohesive soils for both elastic and elasto-plastic models for two values of ζ . It was assumed that the soil parameters, Young's modulus, E , and Poisson's ratio, are 0.4 MPa and 0.4, respectively, and the value of ζ varies from 0.4 to 0.8. It can be seen that the radial stress increment at the cavity wall for the $r_1/r_2 = 0.20$ is significantly different depending on the model used. The radial stress increment at the soil-column interface based on the elastic model, $(\Delta\sigma_r)_e$, is about three times greater than that based on the elasto-plastic model, $(\Delta\sigma_r)_{ep}$, when the plastic zone widens from a value of ζ of 0.4 to 0.8. Therefore the linear elastic model represents poorly the soil behaviour after the soil yields. To account for the yielding of the soil, the stiffness of the soil ring, K_1 , is modified to provide a better estimate of the radial stress increment at the cavity wall. This can be achieved by introducing a reduction factor, η . As the Young's modulus within the plastic zone for further strain is zero, the reduction factor is related to the area ratio of the elastic zone to the influence zone of a column and is given by:

$$\eta = \frac{1 - \zeta^2}{1 - \frac{r_1^2}{r_2^2}} \quad (3.26)$$

where ζ is estimated from Equation 3.24. Hence, the stiffness of the soil ring with

the presence of a plastic zone, $(K_1)_{e,p}$ can be expressed as:

$$(K_1)_{e,p} = \eta K_1 \quad (3.27)$$

where $0 \leq \eta \leq 1$.

3.3 Radial Consolidation

3.3.1 Equations governing consolidation

The basic equation governing the radial consolidation in an elastic soil due to an expanding column is derived here. The soil is assumed to deform elastically during consolidation characterized by the drained elastic parameters, E' , and ν' .

1). Effective stress - strain relationship

For the case of plane strain under consideration, the incremental stress - strain relationship in terms of effective stress is given by:

$$\begin{aligned} \Delta \sigma'_r &= (\lambda + 2G)\epsilon_r + \lambda \epsilon_\theta \\ \Delta \sigma'_\theta &= (\lambda + 2G)\epsilon_\theta + \lambda \epsilon_r \end{aligned} \quad (3.28)$$

where

$$\lambda = \frac{E'\nu'}{(1 + \nu')(1 - 2\nu')} \quad (3.29)$$

$$G = \frac{E'}{2(1 + \nu')} \quad (3.30)$$

The radial strain, ε_r , and the circumferential strain, ε_θ , can be determined from the radial displacement field, $u = u(r, t)$ and

$$\varepsilon_r = -\frac{du}{dr} \quad (3.31)$$

and

$$\varepsilon_\theta = -\left[\frac{(r + u)d\theta - rd\theta}{rd\theta}\right] = -\frac{u}{r} \quad (3.32)$$

2). Radial stress equilibrium

The equation of radial stress equilibrium in terms of total stress increments is expressed as:

$$\frac{\partial \Delta \sigma_r}{\partial r} + \frac{\Delta \sigma_r - \Delta \sigma_\theta}{r} = 0 \quad (3.33)$$

The total stress increments may be expressed as the sum of the effective stress increments and the excess pore pressure, i.e.,

$$\begin{aligned} \Delta \sigma_r &= \Delta \sigma'_r + \Delta u_p \\ &= \Delta \sigma'_r + u_p(r, t) - u_p(r, t_0) - u_e(r, t) \\ \Delta \sigma_\theta &= \Delta \sigma'_\theta + \Delta u_p \end{aligned} \quad (3.34)$$

$$= \Delta\sigma'_z + u_p(r, t) - u_p(r, t_0) - u_e(r, t)$$

where $u_p(r, t)$ or u_p represents the pore pressure at a given time, t , and radius, r :

$u_p(r, t_0)$ or u_{p0} denotes the initial pore pressure at the same radius as u_p :

$u_e(r, t)$ or u_e denotes the excess pore pressure generated in the time interval between t and t_0 .

Substitution of Equation 3.35 into Equation 3.33 leads to:

$$\frac{\partial u_p}{\partial r} = \frac{\partial}{\partial r}(u_{p0} + u_e) - \frac{\partial \Delta\sigma'_r}{\partial r} + \frac{\Delta\sigma'_\theta - \Delta\sigma'_r}{r} \quad (3.35)$$

Substituting Equation 3.28 into Equation 3.35 yields:

$$\frac{\partial u_p}{\partial r} = \frac{\partial}{\partial r}(u_{p0} + u_e) - (\lambda + 2G) \frac{\partial}{\partial r}(\varepsilon_r + \varepsilon_\theta) \quad (3.36)$$

3). Pore water flow behaviour

The movement of the pore water through the soil is assumed to be governed by Darcy's law. The horizontal velocity of the pore water relative to the soil particles is proportional to the radial hydraulic gradient, i.e.,

$$v = -\frac{k_h}{\gamma_w} \cdot \frac{\partial u_p}{\partial r} \quad (3.37)$$

in which k_h is the permeability of the soil in the horizontal direction and γ_w is the unit weight of water.

4). Continuity of volume strain rate

Based on volume continuity, the net volume of water into a region within any time interval must be equal to the volume change of soil element, i.e.,

$$-\frac{k_h}{\gamma_w} \cdot \left(\frac{1}{r} \frac{\partial u_p}{\partial r} + \frac{\partial^2 u_p}{\partial r^2} \right) = \frac{\partial}{\partial t} (\varepsilon_r + \varepsilon_\theta + \varepsilon_z) \quad (3.38)$$

Substituting Equations 3.31 and 3.32 into Equation 3.38 and taking into consideration the condition of plane strain lead to:

$$\frac{k_h}{\gamma_w} \cdot \left(\frac{1}{r} \frac{\partial u_p}{\partial r} + \frac{\partial^2 u_p}{\partial r^2} \right) = \frac{\partial}{\partial t} \left(\frac{u}{r} + \frac{\partial u}{\partial r} \right) \quad (3.39)$$

Changing the order of partial differentiation on the right side of Equation 3.39 and integrating Equation 3.39 with respect to r yield:

$$\frac{k_h}{\gamma_w} \cdot \left(\frac{\partial u_p}{\partial r} \right) = \frac{\partial u_p}{\partial t} + \frac{f(t)}{r} \quad (3.40)$$

in which $f(t)$ is a constant of integration depending on the boundary conditions.

The governing equation for the pore water pressure may be obtained by eliminating the radial displacement, u , between Equation 3.36 and Equation 3.40. Hence:

$$\frac{\partial u_p}{\partial t} = \frac{\partial}{\partial t} (u_{p0} + u_r) + C_h \left(\frac{1}{r} \frac{\partial u_p}{\partial r} + \frac{\partial^2 u_p}{\partial r^2} \right) + g(t) \quad (3.41)$$

where

$$C_h = \frac{k_h}{\gamma_w} (\lambda + 2G) \quad (3.42)$$

u_e denotes excess pore pressure generated in the time interval between t and t_0 , and $g(t)$ is a constant of integration depending on the boundary conditions. It is interesting to note that, except for the terms $g(t)$ and $\frac{1}{\eta}(u_{p0} + u_e)$, Equation 3.41 is similar to Terzaghi's one-dimensional consolidation equation. The coefficient of consolidation C_h is the same as that for the one-dimensional consolidation C_v with the volume compressibility, m_v , corresponding to the one-dimensional consolidation replaced by $1/(\lambda + 2C_r)$.

3.3.2 Boundary conditions

A negative pore water pressure is created within the soil near the column surface due to lime slaking. This effect, known as a suction force, is assumed to be a boundary condition at the soil-column interface. A symbol " u_s " is used to represent the absolute value of pore water pressure at the beginning of lime slaking; u_s is always negative. Its value can be determined from laboratory tests. The suction effect changes from a maximum value of u_s , at the beginning of lime slaking, to zero at the end of slaking. It is necessary to know the relationship between the negative pore water pressure at the soil-column interface, $u_{r=r_1}$, and at any time during the process of lime slaking. A linear relationship between $u_{r=r_1}$ and μ_t is assumed as shown in Figure 3.4.

The boundary conditions for the expansion of a lime column and the subsequent consolidation of soil under consideration are listed here and illustrated in Figure 3.5:

- (1) $u_p = (1 - \mu_t) u_s$ at $r = r_1$ for $t \leq t_e$
- (2) $\frac{\partial u_p}{\partial r} = 0$ at $r = r_2$ for $t \geq 0$
- (3) $u_p = u_{p0}$ at $r_1 < r \leq r_2$ for $t = 0$
- (4) $u_p = u_{p0}$ at $0 \leq r \leq r_2$ for $t = \infty$

where u_{p0} is the initial pore pressure before installing the lime column; t_e is time required to complete lime column slaking and u_s is the maximum negative pore water pressure at the soil-column interface at the beginning of lime slaking.

3.3.3 Approximate solution

Equation 3.41 is an exact expression governing the consolidation behaviour of a lime column in an elastic soil. Generally, it is impossible to obtain the solution for this equation by means of an analytical method unless major simplifying assumptions are made. However, one of the earliest numerical techniques, the finite-difference method, may be used to find the solution to this equation with sufficient accuracy and simplicity. The simplest form of finite-difference approximation to Equation 3.41 is adequate here. It is shown schematically in Figure 3.6 on the two dimensional plane, t and r , and is written as:

$$\begin{aligned}
 u_{pi,t+\Delta t} &= u_{pi,t} + u_{ei,t} \\
 &+ C_h \left[\frac{1}{r_i} \left(\frac{u_{pi+1,t} - u_{pi-1,t}}{2\Delta r} \right) + \frac{u_{pi+1,t} - 2u_{pi,t} + u_{pi-1,t}}{\Delta r^2} \right] \Delta t \quad (3.43)
 \end{aligned}$$

where the first suffix refers to the position in space and the second refers to time, i.e. $u_{pi,t}$ is the pore pressure at radius $r_1 + (i-1)\Delta r$ and at time, t , while $u_{e,i,t}$ is the excess pore pressure induced at radius $r_1 + (i-1)\Delta r$ in the time interval Δt . The nodal point number is denoted by i and Δr is equal to $(r_2 - r_1)/(i - 1)$.

The process of a lime column expansion due to the lime slaking is coupled with the process of the time-dependent increases of total stress and the generation and concurrent dissipation of excess pore water pressure. A common method used in soil mechanics is to assume that the external load is applied instantaneously at time t , and the consolidation takes place with a constant value of the load within Δt . This assumption implies that for the problem considered herein, the radial stress increment at the soil-column interface is applied instantaneously at the end of each time step and is kept constant within Δt as shown in an exaggerated manner in Figure 3.7a, and the excess pore pressure resulting from this load can be considered under undrained conditions at this moment as shown in Figure 3.7b. A good estimation can be obtained if Δt is sufficiently small.

The expressions for the excess pore water pressure within Δt may be estimated using Henkel's pore pressure equation (Henkel 1960):

$$\Delta u = \frac{\beta}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + \alpha\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \quad (3.44)$$

where α and β are the Henkel pore pressure parameters, $\Delta\sigma_1$, $\Delta\sigma_2$, and $\Delta\sigma_3$ are

the change in major, intermediate and minor principal stresses, respectively.

For the saturated soil considered here, the value of β is equal to unity. The Henkel pore pressure parameter, α , depends on test conditions and stress levels. Law and Bozozuk (1979) carried out triaxial and plane strain tests on soft marine clay from sites in the Ottawa-Carleton region, Ontario. They suggest that a below-elastic pore pressure response ($\alpha < 0$) prevails at the pre-yield stage of the stress-strain relationship and an elastic response at the yield ($\alpha = 0$), as described by:

$$\alpha = 0.24(R - 1) \quad (3.45)$$

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

For the case of plane strain under undrained conditions,

$$\begin{aligned} \Delta\sigma_z(r, \Delta t) &= \nu(\Delta\sigma_r(r, \Delta t) + \Delta\sigma_\theta(r, \Delta t)) \\ &= \frac{\Delta\sigma_r(r, \Delta t) + \Delta\sigma_\theta(r, \Delta t)}{2} \end{aligned} \quad (3.46)$$

Therefore the general equation for the excess pore pressure generated within time interval, Δt , becomes:

$$\begin{aligned} \Delta u(r, \Delta t) &= \frac{\Delta\sigma_r(r, \Delta t) + \Delta\sigma_\theta(r, \Delta t)}{2} \\ &\quad + \alpha \frac{\sqrt{6}}{2} [\Delta\sigma_r(r, \Delta t) - \Delta\sigma_\theta(r, \Delta t)] \end{aligned} \quad (3.47)$$

The excess pore pressure before yield, or outside the plastic zone, can be derived from Equation 3.14 and is equal to:

$$\Delta u(r, \Delta t) = \frac{\Delta \sigma_r(r_1, \Delta t)}{K_2} \cdot [\lambda + G(1 + \alpha \sqrt{6} \cdot \frac{r_2^2}{r^2})] \quad (3.48)$$

where

$$\alpha = 0.24 \left[\frac{G}{S_u} \cdot \frac{\Delta \sigma_r(r_1, t)}{K_2} \cdot \frac{r_2^2}{r^2} - 1 \right] \quad (3.49)$$

Within the plastic zone, the excess pore pressure, taking into account the zero value of α at the yield or post-yield state (Law and Bozozuk, 1979), is equal to:

$$\Delta u(r, \Delta t) = \frac{G}{(1 - 2\nu)} \cdot \frac{\Delta \sigma_r(r_1, \Delta t)}{K_2} - S_u \cdot \ln \left[1 - \frac{\Delta \sigma_r(r_1, \Delta t)}{\Delta \sigma_r(r_1, t)} \right] \quad (3.50)$$

where $\Delta \sigma_r(r_1, \Delta t)$ and $\Delta \sigma_r(r_1, t)$ are given by Equations 3.5 and 3.13.

When the slaking process is completed, the excess pore pressure generated will cease but the radial consolidation continues until all the excess pore pressure is completely dissipated as illustrated in Figure 3.5(d).

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

The excess pore water pressure generated by lime column slaking can completely dissipate within a short time of less than 10 days depending on the hydraulic conductivity of the soil as indicated by the measurements of pore water pressure during

and two months after lime column installations (Chen, 1989; Shi, et al., 1989). Because of the rapidity of the consolidation, the measurements of the soil strength and stiffness are normally made at one month after the lime column treatment and used for the design.

As the excess pore water pressure is completely dissipated, there will be an effective stress increase equal to the total stress increase. Assuming that the total stress increase can be obtained from the proposed model and remains unchanged after the complete slaking of the lime, the effective stress increases can be estimated. Such an estimation will be made and compared with the results of a full-scale experimental study and another case record to be reported in Chapter 7. Hence, the horizontal effective stress increase after complete dissipation of the excess pore pressure can be given by following two forms:

- a. Before the yielding of the soil or in the elastic zone,

$$\Delta\sigma'_h(r) = \Delta\sigma_r(r, t_e) = \frac{\Delta\sigma_r(r_1, t_e)}{K_2} \cdot [G(\frac{r_1^2}{r^2} + 1) + \lambda] \quad (3.51)$$

where t_e = the time required to complete the lime column slaking and $\Delta\sigma_r(r_1, t_e)$ = the total radial stress increase at the soil-column interface at the end of the lime column slaking.

- b. After the yielding of the soil,

$$\Delta\sigma'_h(r) = \Delta\sigma_r(r, t_e) = S_u[\frac{\zeta_e^2}{1 - 2\nu} + 2\ln(\frac{r_{ue}}{r})] \quad (3.52)$$

where r_{ue} = the radius of the plastic and elastic boundary at the end of lime slaking and

$$\zeta_e = \frac{r_{ue}}{r_2} = \sqrt{\frac{\Delta\sigma_r(r_1, t_e) \cdot G}{S_u \cdot K_2}} \quad (3.53)$$

3.5 Basic Equation for Strength Increase

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 as well as the Skempton's pore pressure parameter at failure, A_f , was given by Leonards (1962) as:

$$S_u = \frac{c' \cos \phi' + \sigma'_{v0} [K_0 + A_f(1 - K_0)]}{1 + (2A_f - 1) \sin \phi'} \quad (3.54)$$

where K_0 = coefficient of earth pressure at rest, and σ'_{v0} = vertical effective stress.

Substitution of K_0 in terms of σ'_{v0} and σ'_{h0} into Equation 3.54 yields:

$$S_u = \frac{c' \cos \phi' + \sin \phi' [A_f \sigma'_{v0} + (1 - A_f) \sigma'_{h0}]}{1 + (2A_f - 1) \sin \phi'} \quad (3.55)$$

where σ'_{h0} = horizontal effective stress.

It can be seen from Equation 3.55 that the undrained shear strength of the soil at a certain depth is dependent on the values of A_f , c' and ϕ' , and stress state of the soil.

The lime column treatment induces an effective stress increase which in turn causes changes in A_f , c' , and σ' . The net result usually leads to an increase in undrained shear strength.

The undrained shear strength increase is equal to the difference of strengths obtained from Equation 3.55 using appropriate values of c' , σ' , σ'_{i0} , σ'_{h0} and A_f corresponding to conditions after and before the treatment. Before treatment, $\sigma'_{h0} = K_0 \sigma'_{i0}$, and after the treatment, $\sigma'_{h0} = (K_0 \sigma'_{i0} + \Delta \sigma'_h)$. The values of c' and σ' corresponding to conditions before and after the treatment can be determined from the laboratory tests on soil samples taken prior to the treatment. A_f depends on stress history of the soil and test conditions. The horizontal effective stress increase is given in Section 3.4.

3.6 Basic Equation for Dynamic Modulus Increase

A number of empirical equations to estimate the initial shear modulus (i.e. strains at or below 0.001%) are available in the literature (Humphries and Wahls, 1968; Hardin and Blank, 1968; Hardin and Drnevich, 1972a, 1972b; Weiler and William, 1988). Weiler and William adopted the SHANSEP (stress history and normalized soil engineering parameters) methodology to express the relationship for G_0 for a particular clay in the form:

$$G_0 = K \sigma_c^n (OCR)^m \quad (3.56)$$

where G_0 = initial shear modulus in MPa :

K , n , and m are constants for a soil while m and n are pure numbers and K has a unit of MPa :

σ_c = mean consolidation pressure in kPa applied on the sample:

OCR = overconsolidation ratio as determined from standard one-dimensional consolidation tests.

The constants, K , n , and m can be obtained from laboratory tests on undisturbed soil samples taken from the site before the lime-fly ash column installation. Resonant column tests are usually used to determine these parameters.

The dynamic shear modulus of the treated soil can be obtained from Equation 3.56 in which the mean consolidation pressure applied to the treated soil is equal to $(\sigma_c + \Delta\sigma_c)$ and will be given in Section 3.6.1. σ_c is the mean effective stress of the soil before the treatment and $\Delta\sigma_c$ is the increase in mean consolidation pressure due to the treatment.

3.6.1 Mean consolidation pressure increase ($\Delta\sigma_c$)

$\Delta\sigma_c$ is a function of effective stress changes. Under the plane strain conditions, it can be expressed as:

$$\Delta\sigma_c = \frac{1 + \nu'}{3} \cdot (\Delta\sigma'_h + \Delta\sigma'_\theta) \quad (3.57)$$

where ν' is the drained Poisson's ratio. $\Delta\sigma'_h$ and $\Delta\sigma'_\theta$ are the horizontal effective stress increase and the circumferential effective stress increase due to the treatment, respectively. The expressions for the stress increases were given by Equations 3.14 and 3.25.

Substituting Equation 3.14 into Equation 3.57 leads to the expression for mean effective stress in the elastic zone:

$$\Delta\sigma_c = \frac{2(1 + \nu')(\lambda + G)}{3K_2} \cdot \Delta\sigma_r(r_1, t_e) \quad (r_{ue} \leq r \leq r_2) \quad (3.58)$$

It can be seen from Equation 3.58 that the mean effective stress increase within the elastic zone does not vary along the radial direction.

The stress increases within yielding zone at the end of complete consolidation can be obtained from Equation 3.25 and are given by:

$$\begin{aligned} \Delta\sigma'_h &= S_u \cdot \left[\frac{\zeta_e^2}{1 - 2\nu} + 2\ln\left(\frac{r_{ue}}{r}\right) + 1 \right] & (r_1 \leq r \leq r_{ue}) \\ \Delta\sigma'_\theta &= S_u \cdot \left[\frac{\zeta_e^2}{1 - 2\nu} + 2\ln\left(\frac{r_{ue}}{r}\right) - 1 \right] & (r_1 \leq r \leq r_{ue}) \end{aligned} \quad (3.59)$$

where S_u = the initial undrained shear strength of the soft soil and

$$\begin{aligned} \zeta_e &= \frac{r_{ue}}{r_2} \\ &= \sqrt{\frac{\Delta\sigma_r(r_1, t_e) \cdot G}{S_u \cdot K_2}} \end{aligned} \quad (3.60)$$

Substitution of Equation 3.59 into Equation 3.57 results in following equation for mean effective stress increase in the plastic zone:

$$\Delta\sigma_c = \frac{2(1+\nu')S_u}{3} \cdot \left[\frac{\zeta_e^2}{1-2\nu'} + 2\ln\left(\frac{r_{ue}}{r}\right) \right] \quad (r_1 \leq r \leq r_{ue}) \quad (3.61)$$

3.6.2 Radial distribution of dynamic shear modulus

From Equations 3.56, 3.58 and 3.61, one can derive following expressions for the dynamic shear modulus at any point between column wall and the influence of a column.

Throughout the elastic zone, $G_0(r)$ is constant if (OCR) remains unchanged before and after the treatment, and is given by:

$$\begin{aligned} G_0(r) &= K \left[\sigma_c + \frac{2(1+\nu')(\lambda+G)}{3K_2} \cdot \Delta\sigma_r(r_1, t_e) \right]^n (OCR)^m \\ &= g \quad (r_{ue} \leq r \leq r_2) \quad (3.62) \end{aligned}$$

where g is a constant.

For the plastic zone:

$$G_0(r) = K \left[\sigma_c + \frac{2(1+\nu')S_u}{3} \cdot \left(\frac{\zeta_e^2}{1-2\nu'} + 2\ln\left(\frac{r_{ue}}{r}\right) \right) \right]^n (OCR)^m \quad (r_1 \leq r \leq r_{ue}) \quad (3.63)$$

The average shear modulus, $(G_0)_{ave}$, of the treated soil between the columns is

assumed to be equal to the volume weighted mean value and is expressed as:

$$(G_0)_{ave} = \frac{\int_{r_1}^{r_2} G_0(r) \cdot dv}{\int_{r_1}^{r_2} dv} \quad (3.64)$$

In order to facilitate integration without losing much accuracy, Equation 3.62 can be approximated by a polynomial expression as:

$$G_0(r) = g_2 r^2 + g_1 r + g_0 \quad (3.65)$$

where g_2 , g_1 and g_0 are constants to be determined from Equation 3.62 when three $G_0(r)$ values are those corresponding to r_1 , r_{ue} and $\frac{(r_1 + r_{ue})}{2}$

Substituting Equation 3.61 into Equation 3.65 and integrating Equation 3.65 lead to the following expression for $(G_0)_{ave}$:

$$(G_0)_{ave} = \frac{3g_2(r_{ue}^4 - r_1^4) + 4g_1(r_{ue}^3 - r_1^3) + 6g_0(r_{ue}^2 - r_1^2) + 6g(r_2^2 - r_{ue}^2)}{6(r_2^2 - r_1^2)} \quad (3.66)$$

3.7 Summary

In this chapter, a model is proposed to estimate the excess pore pressure due to lime column slaking. The excess pore pressure is expressed in terms of the stress level generated at any point and at any time after lime column installation. An analytical approach is developed to estimate the increases in the static shear strength

and dynamic shear modulus of the treated soil in terms of the increases in effective stresses.

(a) Square arrangement

(b) Triangular arrangement

(c) Hexagonal arrangement

Figure 3.1: Lime column arrangements

Figure 3.2: Schematic diagram showing lime column expansion at different time

Figure 3.3: Comparison of elastic and elasto-plastic solutions for radial stress increases

Figure 3.4: Assumed suction condition at the soil-column interface during lime slaking

Figure 3.5: Schematic diagram showing distributions of pore pressure at various stages

Figure 3.6: A method with constant increments of radius

Figure 3.7: Exaggerated approximation, for numerical solution, of the radial stress increase at the cavity wall and the corresponding variations of pore pressure at radius r

Chapter 4

Parametric Study and Case Studies

4.1 General

As discussed in Section 3.4, the excess pore pressure resulting from lime column slaking depends on a number of factors. For a given type of quicklime, the excess pore pressure generated by lime column slaking is influenced by the type of soil, column geometry, as well as column spacing. In this chapter, a parametric study will be conducted to examine the relative importance of different parameters including drained Young's modulus, E' , drained Poisson's ratio, ν' , undrained shear strength, S_u , and coefficient of horizontal permeability, k_h , as well as column spacing, s . Two case studies are also used to verify the proposed method for estimating excess pore pressure generated due to lime column slaking.

4.2 Parametric Study

Only one type of quicklime is considered here. The column is composed of a mixture of quicklime and fly ash at a volume ratio of 7 to 3. The column diameter is 280 mm. The lime-fly ash mixture is characterized by $C_1 = 0.5$, $C_2 = 0.3$, $C_3 = 2.0 \text{ MPa}^{-1}$, and $C_4 = 0.4 \text{ MPa}^{-1}$.

4.2.1 Effects of drained Young's modulus (E')

Figure 4.1 shows the distribution of the normalized excess pore pressure for three values of E' : 10 MPa, 1 MPa and 0.2 MPa with the other parameters kept constant as shown in Figure 4.1. The X-axis represents the ratio of radial distance, r , from the column axis to the column diameter, d . The Y-axis represents the undrained excess pore pressure resulting from the expansion of lime column slaking, normalized by the undrained shear strength. It can be seen that the influence of the drained Young's modulus on the excess pore pressure is significant: higher excess pore pressures are generated in a soil with higher values of drained Young's modulus. It is important, therefore, to choose the correct value of Young's modulus.

4.2.2 Effects of drained Poisson's ratio (ν')

The normalized excess pore pressures generated due to lime slaking have been calculated for seven different values of ν' : 0.05, 0.1, 0.2, 0.3, 0.35, 0.4 and 0.45. The results show that the excess pore pressure increases slightly with increasing ν' when

ν' is less than 0.3, but increases significantly when ν' is greater than 0.3. The results are shown in Figure 4.2. For soils with ν' greater than 0.3, therefore, the value of ν' should be accurately determined in order to adequately reflect the response of excess pore pressure.

4.2.3 Effects of column spacing (s)

The influence of column spacing on the normalized excess pore pressure is shown in Figure 4.3. It can be seen that the excess pore pressure in the treated soil with smaller spacing is significantly higher than that with larger column spacing. If the spacing is too small, very high excess pore pressure will be generated with the possible undesirable consequence of ground heave. Therefore, it is wise to adopt an appropriate column spacing in order to achieve the best improvement effect without waste of energy.

4.2.4 Effects of undrained soil strength (S_u)

Figure 4.4 represents the distribution of undrained excess pore pressure without normalization along the radial direction for three different values of undrained shear strength: $10kPa$, $20kPa$, and $30kPa$. It can be seen that higher pore pressure is generated near the column for higher soil strength. However, an opposite phenomenon was observed at a distance further from the column. The reason for this behaviour is due to different sizes of plastic zones developed in soils with different strengths.

4.2.5 Effects of permeability (k_h)

Four values of coefficient of horizontal permeability, 1×10^{-7} , 1×10^{-8} , 1×10^{-9} , and $1 \times 10^{-10} \text{ m/s}$, were used in the study. The results indicate that there is no influence of permeability of soil on the magnitude of undrained pore pressure induced by lime slaking. This is a direct consequence of the assumption of undrained condition. The permeability of soil, however, will affect the duration of lime slaking. The column installed in the soil with low permeability takes more time to complete the lime column reaction with water.

4.3 Case Studies

The above parametric study gives a clear indication that excess pore pressure is a function of E' , ν' , S_u , and s and the pore pressure distribution is non-linear along the radial direction (Law and Chen, 1992). Although complete sets of field measurements on the radial distribution of excess pore pressure are not available in the literature, some field data on excess pore pressure have been reported for limited locations. In order to examine the validity of the proposed method for estimating excess pore pressure, data from field measurements at two sites in China (Chen, 1986; Shi et al., 1989) were considered. The sites were strengthened by the lime-fly ash column method. One was at the Xinhua district test site, the other was at the Locomotive test site. They were located in Taiyuan, Shanxi, China. Excess pore pressures at each site were monitored using electric piezometers, located at different

depths, during the formation of lime column and at different durations up to two months after lime column installation.

4.3.1 Case I — Excess pore water pressure

A comprehensive field study at the Xinhua district site subjected to lime-fly ash column improvement was conducted (Chen, 1986; Shi et al., 1989). The soil is composed of a 4.1 m of clayey silt followed by a layer of 1.7 m of silty clay underlain by clayey silt again. In-situ measurements (including vane test, plate loading test, and standard penetration test, etc.) and laboratory tests show the following results: groundwater table at 1.67 m below ground surface, the natural water content, w , ranging from 28 to 35 per cent, liquid limit, w_L , =33 per cent, void ratio, e , = 1.0, undrained shear strength, S_u , =30 *kPa*, drained modulus of deformation, $E' = 2.08 \text{ MPa}$, permeability, $k_h = 8.6 \times 10^{-8} \text{ m/s}$. Quicklime-fly ash (quicklime : fly ash = 7:3) column with a design diameter of 200 mm and a length of 5 m including 1 m clay plug was used at this site. The clay plug was used to prevent energy loss during the process of lime slaking. Four test zones with different spacings were studied. The column installation method involved a predrilled and cased hole into which the lime-fly ash was compacted in-situ. Due to the compaction effort, the actual diameter of the column at the formation of column was estimated to be 230 mm. The diameter of the column one month after installation of the column was observed to be 280 mm. The dry unit weight of the quicklime-fly ash column was 11 *kN/m³*. Two electric piezometers were installed at different depths in the zone with

columns arranged in an equilateral triangular with 0.70 m length for each side. The piezometers were both located at the centroid of the triangle, about 0.40 m from any of the columns. Pore water pressure was measured before and after boring of the hole, and at different durations up to two months after the column installation. The results showed that the pore water pressure reached its peak within two hours after the columns were completely installed. The change in pore water pressure due to casing, column forming, and column expansion resulting from slaking could be separately identified in this method of column installation. The maximum measured excess water pressure due to quicklime slaking at a depth of 3 m was found to be 105 *kPa*.

The following parameters, as measured by appropriate techniques similar to those described in Chapter 6 of this thesis, have been used for estimating the excess pore pressure based on the method proposed in Chapter 3:

Quicklime:

$$C_1 = 0.56, C_2 = 0.21, C_q = 1.0 \text{ MPa}^{-1}, C_s = 0.2 \text{ MPa}^{-1}$$

Soil:

$$E' = 2.08 \text{ MPa}, \text{ Poisson's ratio} = 0.38$$

$$S_u = 30 \text{ kPa}, k_h = 8.6 \times 10^{-8} \text{ m/s},$$

Lime column:

$$r_1 = 115 \text{ mm}, r_2 = 370 \text{ mm}.$$

The calculated excess pore water pressure at the centroid of the triangle for the same location is 104 *kPa*. This is in good agreement with the measured value.

It is of interest to note that, if the elastic analysis is used, the same set of input parameters will give an excess pore water pressure about 50% higher than the measured values.

4.3.2 Case II – Process of consolidation

Shi et al. (1989) report a comprehensive field study at a test site strengthened by the lime-fly ash column method. Some of the pertinent test results are given here. The soil at the test site consists of a 1.7 m of top loose soil followed by a layer of 4 m of clayey silt underlain by silty clay. The groundwater table is at 1.65 m below the ground surface. The range of natural water content is from 30 to 36 per cent. The liquid limit is about 35 per cent. The average undrained shear strength of the soil below the water table down to a depth of 5 m is 25 *kPa*. The drained modulus of deformation is 1.5 *MPa* and the horizontal coefficient of permeability of the soil is $1 \times 10^{-8} \text{ m/s}$.

Quicklime-fly ash columns with a diameter of 340 mm, 6 m length including 1 m top clay plug and a spacing of 1.2 m at a rectangular arrangement were installed at the site.

The volume ratio of quicklime to fly ash is 7 to 3. A displacement method was used in installing the lime-fly ash columns. A closed-end steel tube with a diameter of 325 mm was pushed into the ground to the design depth. The tube was then retrieved leaving a hole in the soil. A quantity of lime-fly ash mixture was placed into the hole and compacted by pushing the tube against the mixture to form the

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bottom section of the column. The procedure was repeated section by section until the desired column length was obtained. The compacted dry unit weight of the lime-fly ash column was 11 kN/m^3 .

Pore water pressures were measured with an electric piezometer at a depth of 3 m and midway between columns. The horizontal stress change at the same location was measured with a stress gauge. Readings were taken before and after boring of the hole, and at different times up to two months after the column installation. The results as shown in Figure 4.5 indicated that both the pore water pressure and horizontal stress increase at the middle between columns reached their peaks within two hours after the column installation. The excess pore water pressure dissipated completely within 170 hours later. However, the horizontal stress increase dropped about 10 kPa from the peak value and stabilized at a value of 69 kPa over two months after the lime column installation.

The pore water pressure and stress increase at this location due to the lime column treatment can be estimated by the proposed method. The parameters used in the calculation will be described as follows:

The value of C_1 for pure quicklime is estimated from Table 2.1. For a lime column with a dry unit weight of 11 kN/m^3 and an estimated dry unit weight of the slaked column of 8 kN/m^3 , a value of C_1 of 0.7 is obtained. Taking into consideration of the lime-fly ash column with a volume ratio of 7 to 3 and the soil displaced during the installation process, C_1 is estimated to be 1.49.

It has been found C_q ranges from 0.4 to 4 MPa^{-1} and C_s from 0.08 to 1.2

MPa^{-1} and both decrease with the increase in the compacted dry unit weight of a lime column. The values of C_γ and C_s are assumed to be 2.0 MPa^{-1} and 0.4 MPa^{-1} , respectively. Such low values of C_γ and C_s were used herein since a relatively high compaction effort was employed in the field.

The value of E' for the soil was obtained from a plate loading test conducted at the site and was equal to 1.5 MPa .

The undrained shear strength of the soil was obtained from the vane test at 25 kPa . The permeability of the soil was determined from laboratory tests on undisturbed soil samples taken from the site prior to the treatment.

The other parameters used in the analysis are: $\nu' = 0.30$, $u_s = 10 \text{ kPa}$, $C_2 = 0.3$, $k_h = 1.0 \times 10^{-8} \text{ m/s}$, $r_1 = 170 \text{ mm}$, and $r_2 = 680 \text{ mm}$. $\Delta t = 20 \text{ s}$, $\Delta r = 20 \text{ mm}$.

The total excess pore water pressure without consolidation and pore water pressure with consolidation as well as stress increase at any point and any time can be calculated by the proposed method using the above parameters. The variations of pore water pressure with consolidation and horizontal stress increase at this location are illustrated in Figures 4.5(a) and 4.5(b), respectively. The solid line and the solid circles in Figure 4.5(a) represent the calculated and the measured total horizontal stress increases, respectively. It indicates that the calculated total horizontal stress increase agrees well with the measured values. The calculated horizontal effective stress increase after complete consolidation is 75 kPa , about 6 kPa higher than the measured value. One reason for this discrepancy may be due to ground heave which was observed during the experiment. The ground heave could lead to some degree

of stress release. It should be mentioned that the horizontal effective stress increase was about two times higher than the measured value if the analysis was based on the elastic model as discussed in Chapter 3. This further supports the proposed method for estimating the horizontal effective stress increase.

A comparison of the variations of excess pore water pressure with time is made between the measured and the calculated. The calculated excess pore pressure represented by a solid line in Figure 4.5(b) is in a close agreement with the measured values illustrated in Figure 4.5(b) by the hollow circles. The estimated time required to complete dissipation of excess pore water pressure due to the lime column treatment is 174.5 hours. This is close to the measured value of 170 hours.

The two case records therefore show that the calculated values of pore water pressure induced by quicklime column with different methods of column installation are reasonably close to those measured from the field. The difference between the calculated and measured is less than 15 % if ground heave occurs. If there is no ground heave, the method yields very good results in comparison with the measurements.

4.4 Summary

This chapter reports a parametric study and two case studies involving loose sandy soils. The following conclusions can be drawn: (a) the excess pore pressure distribution along the radial direction is non-linear; (b) the magnitude of the undrained

pore pressure is a function of the undrained shear strength, modulus and Poisson's ratio of the soil, and the spacing of the columns: (c) the proposed model gives a good estimate of excess pore pressure induced by the expansion of the column; and (d) the proposed method also provides us with a useful tool to estimate total stress increase, excess pore water pressure, and generation and dissipation of the excess pore water pressure and hence to calculate and predict the effective stress increase of the treated soil.

Figure 4.1: Excess pore pressure distributions for various values of E'

Figure 4.2: Excess pore pressure distributions for various values of ν'

Figure 4.3: Excess pore pressure distributions for different spacings

Figure 4.4: Excess pore pressure distributions for various values of S_u

Figure 4.5: Variations of total horizontal stress increase and excess pore water pressure during and after lime column installation

Chapter 5

Full-Scale Experimental Study

5.1 General

An experimental study was conducted to study the validity of the proposed method for estimating the stress increase, the strength increase and the dynamic shear modulus increase of the treated soil. The experimental study was carried out at a soft sensitive clay site subjected to soil strengthening by means of the use of lime-fly ash column method. This chapter describes the test programme and results. A comparison between these results and the estimated values based on the model described in Chapter 3 will be presented in Chapter 7.

5.2 Test Site and Test Program

The test site is located 20 km south-east of Ottawa, Ontario, where this Champlain Sea clay has been previously studied by a number of researchers (Bozozuk and

Leonards, 1972; Lo et al., 1976; Bauer et al., 1980; Lo et al., 1990). The field test area was approximately 60 m north west of the test embankment as shown in Figure 5.1.

At the test site, the soil profile consists of 0.2 m thick top soil followed by a crust of silty clay to a depth of 1.4 m. Underlying the crust is a sensitive clay layer extending to a depth of 20 m. A hard layer of till lies below 20 m. The plasticity index of the clay ranges from 20 % to 30 % and the average clay content is about 80 %. The geotechnical profile of the test site (Bozozuk and Leonards, 1972) is shown in Figure 5.2. The clay is slightly overconsolidated with a ratio (*OCR*) of 1.5. The average natural water content of the soft clay below watertable down to a depth of 5.5 m is about 60 %, which is well above the liquid limit of the soft clay. The average vane shear strength of the soil above the depth of 5.5 m is smaller than 20 *kPa* while that below this level is in excess of 30 *kPa*.

Based on the above information, it was decided to treat the soft soil down to a depth of 5.5 m below the ground surface. The total length of the lime-fly ash column was 5.3 m with a diameter of 280 mm including a 1.3 m top clay plug. Fly ash was added not only to make use of the waste from a coal burning power plant but also to improve the lime column behaviour. Three square test zones (Figure 5.3) with lime-fly ash (7:3) column method were chosen to study the effect of spacing on the results of improvement. Three selected spacings of 0.70 m, 0.84 m, and 0.98 m for Zone 1, Zone 2, and Zone 3, were used, respectively.

The ground surface settlement and the lateral movement resulting from the in-

stallation and expansion of lime-fly ash columns were measured before, during, and one month after the treatment. Ground surface settlements were monitored with a steel settlement auger of 1.3 m in length embedded 1.0 m into the soil. The settlement was measured by means of a survey level. An inclinometer was installed for the measurement of the lateral movement due to the expansion of the lime-fly ash column during and after the installation. The general layout of the lime-fly ash column, and the instrumentation layout are shown in Figure 5.3.

The site was investigated before treatment, one month, and 8 months after the treatment by means of Menard pressuremeter tests, piezocone tests, and seismic cone tests. The general layout of the Menard pressuremeter, piezocone and seismic cone tests is shown in Figure 5.4.

The fly ash used was obtained from the Nanticoke coal-fired generating station. It is a silt-sized material composed mainly of glass spheres with some crystalline and carbonaceous matter. Its average specific density is 2.5, which is lower than the normal value for soil solids. The grain size distribution is quite uniform as shown in Figure 5.5 (Toth et al., 1988). Because of its spherical shape, small surface area, and uniform silt size of individual particles, the fly ash is defined as with no plasticity. A typical value of the friction angle for fly ash is 35° (Toth et al., 1988). The principal chemical constituents are silica (SiO_2), alumina (Al_2O_3), iron oxide (Fe_2O_3), and calcium oxide (CaO).

5.2.1 Description of installation of lime column

As shown in Figure 5.6, the lime-fly ash columns were installed in soft soils under the following steps:

Step 1. Pre-drill a hole to the desired depth (Figures 5.6(a) and (b) and 5.7) using a mobile drilling machine (Figure 5.8):

Step 2. Prepare a dry lime-fly ash mixture (Figure 5.9) with a volume ratio of 7 to 3 (i.e. lime : fly ash = 7:3) using a mixture apparatus (Figure 5.10):

Step 3. Backfill the hole with 1-m thick lift of lime-fly ash mixture (Figures 5.6(c) and 5.11):

Step 4. Compact each lift with a compactor (Figure 5.12) to the designed dry unit weight (Figures 5.6(d) and 5.13) :

Step 5. Repeat Step 2 and Step 3 until the hole is filled to 1.3 m below the ground surface (Figure 5.6(f)):

Step 6. Backfill with the excavated clay and compact it to form a clay plug (Figure 5.6(g)).

5.2.2 Inclinator data

An inclinometer casing tube was monitored before the column installation and up to 6 months following the installation.

The inclinometer casing was installed Aug. 7, 1990 at a distance from 0.4 m in Zone 1 where no horizontal displacements were expected to occur.

The results showed that there was a nominal horizontal displacement due to the lime-fly ash column treatment. This observation supports Assumption 3 in Section 3.2 for the use of modelling the lime column expansion.

5.2.3 Settlement data

The vertical movement of augers was monitored using a surveying level. Readings were obtained before and after lime-fly ash column installation and up to six months afterwards. The variation of settlement with time for the settlement augers *s1*, *s2* and *s3* are presented in Figure 5.14. The plots indicate that there is a slight ground heave due to the lime-fly ash column treatment. This observation suggests that the energy loss due to this ground heave is negligible.

5.3 In-situ Tests

5.3.1 Menard pressuremeter test

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

A total of 6 Menard pressuremeter tests were conducted at the test locations shown in Figure 5.4. Test equipment and detailed test procedure have been described by Baguelin et al. (1972). Pressuremeter tests were conducted according to *ASTM Standard D4719-87* at depth interval of 0.75 m starting from approximately 1.5 m below surface and terminating at 5 m.

A typical pressuremeter test result is shown in Figure 5.15 where the corrected pressure is plotted against the corrected volume change. The total in-situ horizontal stress (σ_{h0}) is estimated from the corrected pressuremeter curve based on the standard method of assuming that σ_{h0} equals to the start of the linear region of the pressure-volume curve (P_{0M}). The coefficient of earth pressure at rest, K_0 , is obtained from:

$$K_0 = \frac{\sigma_{h0} - u_0}{\sigma_{v0} - u_0} \quad (5.1)$$

where σ_{v0} 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* and is given as follows:

$$E_p = 2(1 + \nu)(V_0 + V_m) \frac{\Delta P}{\Delta V} \quad (5.2)$$

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 un-inflated probe at zero reading at ground surface (535 cm³ for this test), V = corrected volume reading of the measuring portion of the probe, P = corrected pressure increase, $\frac{\Delta P}{\Delta V}$ = slope of the pressuremeter curve between P_{0M} and P_f as shown in Figure 5.15, and V_m = corrected volume reading under the pressure $P_m = (P_{0M} + P_f)/2$.

The drained Young's modulus, E' , was calculated by the following equation

Depth (m)	σ'_{ho} (kPa)	K_0	E_M (MPa)	E' (MPa)	σ_{ho} (kPa)
2.25	29.5	1.02	0.45	0.40	37.0
3.00	22.0	0.67	0.67	0.59	37.0
3.50	26.0	0.72	0.69	0.61	46.0

Table 5.1: Menard pressuremeter test results before the treatment

(Small et al., 1976):

$$E' = \frac{2(1 + \nu')E_u}{3} = \frac{2(1 + \nu')E_p}{3} \quad (5.3)$$

where ν' is the drained Poisson's ratio, and E_p is the pressuremeter modulus or the Menard modulus estimated from Equation 5.2.

Figures 5.16, 5.17, 5.18, 5.19, and 5.20 show the other results for the Menard pressuremeter tests.

A summary of these results is illustrated in Tables 5.1 and 5.2. Table 5.1 shows the measured horizontal effective stresses and the drained Young's modulus before treatment and Table 5.2 shows those after treatment.

where σ'_{ho} = horizontal effective stress; σ_{ho} = the total horizontal stress; K_0 = the

Depth (m)	σ'_{h0} (kPa)	K_0	E_M (MPa)	E' (MPa)	σ_{h0} (kPa)
2.25	41.4	1.46	1.22	1.09	50.0
3.00	37.0	1.12	2.34	2.08	52.0
3.50	41.0	1.03	0.93	0.83	61.0

Table 5.2: Menard pressuremeter test results after the treatment

coefficient of earth pressure at rest; E_M = the Menard modulus or the pressuremeter modulus; E' = the drained Young's modulus

K_0 values obtained from the present study are compared with the published data. Results reported by Bozozuk (1974) showed that the values of K_0 obtained from hydraulic fracture tests for this site are not a constant while they vary from greater than one at a depth of 2.4 m to 0.64 at a depth of 5.3 m. Figure 5.21 shows the comparison of the distributions of measured K_0 according to the two different methods. It can be seen that the two methods yield comparable results within a depth range of 3.5 m. However, it should be mentioned that some difficulties were met when the Menard pressuremeter tests were carried out at a depth of 4.0 m. Self-boring pressuremeter apparatus should be employed if it were available at that

time.

5.3.2 Piezocone penetrometer test

A total of 11 piezocone tests were conducted to determine the undrained shear strength of the soil at the test site and their locations are shown in Figure 5.4. Three of them, pc1, pc2 and pc3 as shown in Figure 5.4, were carried out prior to the treatment. The average data of these test results will be used as a reference for the comparison in Chapter 7.

The undrained shear strength (S_{uc}) from the piezocone test can be estimated according to Konrad and Law's model (1987):

$$S_{uc} = \frac{q_t - (1 + F)\sigma_i + F\alpha u}{\frac{4}{3}(1 + F)(\ln I_R + 1)} \quad (5.4)$$

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 Konrad and Law (1987), the following parameters were chosen for the site: $a = 0.64$, $M = 0.75$, $\phi' = 27^\circ$, $\theta = 30^\circ$, $\alpha = 1.1$, $I_R = 130$, $\sigma_i = \sigma_{v0}$.

The strength profiles are determined from the above parameters.

Figure 5.22 shows the strength profiles of piezocone tests prior to the treatment. The solid line in Figure 5.22 represents the average value of the test results.

Figure 5.23 shows the strength profiles at midway between the columns at Zone

1 before the treatment, one month and 8 months after the treatment. It indicates that there is a substantial increase in shear strength after the treatment. The major increase in shear strength was noted within one month after the treatment. This is consistent with other observations reported by Chen, 1989 and Shi et al., 1989. This supports that the increase in shear strength is mainly due to the expansion of lime columns. The strength gain one month after the treatment is small and therefore it is in practice to design a lime-fly ash column project on the basis of the strength increase one month after the treatment.

The same observations were noted for Zone 2 and Zone 3. The strength profiles for Zone 2 and Zone 3 are plotted in Figures 5.24 and 5.25, respectively.

Figure 5.26 shows the strength profiles before the treatment, at midway between columns one month after the treatment and at a distance of one fourth of the column spacing from the column wall one month after the treatment. It can be seen that the increase in strength at a distance closer to the column wall is larger.

5.3.3 Seismic cone test

The seismic cone is a cone penetrometer fitted with a seismometer which facilitates the measurements of shear wave velocity in soil. Thus, a downhole seismic test can be conducted during the brief pauses in the cone penetration. In addition, a surface signal source, an electronic mechanism triggering, and a data acquisition system are employed during the downhole seismic test. The signal source in this study was generated by a rigid platform, weighted down by the pads of the vehicle and struck

with a sledge hammer. The seismic wave traces were detected by the seismometer and recorded by a data acquisition system with the capability of recording the time accurately and the shear waves clearly. A multi-channel signal enhancement seismograph was also employed in this study. Shear waves were recorded digitally and displayed on a thermal sensitive paper on site.

The times for the first shear wave peak arrival from the forward and reverse shear waves can easily be determined and are considered as two reference arrival times for the forward and reverse signals, respectively. The average arrival time from the forward and reverse signals is considered as the time at which the shear wave travels from the middle point of the platform to the seismometer. To eliminate the effect of different soil layers with depth on the measured results, the shear wave velocity between two successive test depths was taken. This velocity was obtained by the ratio of the difference between successive travel distance to the difference of successive average travel time. The depth interval for the seismic cone tests in this study was 0.5 m.

The shear wave velocities calculated from the difference of arrival times are shown in Figure 5.27. There is a significant increase in the shear wave velocity (V_s) in the treated soil at depths ranging from 1.3 to 6.0 m. The initial shear modulus (G_0) is obtained by substituting the mass density (ρ) of the soil and the shear wave velocity into the equation:

$$G_0 = \rho V_s^2 \quad (5.5)$$

Based on the laboratory tests on undisturbed soil samples, the average value of ρ of soil before treatment is 1550 kg/m^3 , and that after treatment is 1600 kg/m^3 . A comparison of G_0 estimated from Equation 5.5 is shown in Figure 5.28. The values of G_0 within the depth range of 1.3 to 6.0 m before treatment vary from 4.5 to 12.9 MPa with an average of 8.1 MPa . Within the same depth range after treatment, the values of G_0 vary from 11 to 33.1 MPa with an average of 21.1 MPa . Therefore, there is more than 160 % increase of the maximum shear modulus in this range.

Figure 5.1: General layout of the test site

Figure 5.2: Geotechnical profile of the test site (After Bozozuk and Leonards, 1972)

Figure 5.3: Layout of lime-fly ash column and instrumentation

Figure 5.4: Layout of Menard pressuremeter, piezocone and seismic cone tests

Figure 5.5: Summary of the grain size distributions of fly ash and bottom ash (After Toth et al., 1988)

Figure 5.6: Process of installation of lime-fly ash column

Figure 5.7: The hole ready for filling with the lime-fly ash mixture

Figure 5.8: Mobile drill for hole preparation

Figure 5.9: Lime-fly ash mixture ready to fill the hole

Figure 5.10: The mixture apparatus for lime-fly ash mixing

Figure 5.11: Filling the hole with the lime-fly ash mixture

Figure 5.12: The compactor for manual compaction

Figure 5.13: Process of manual compaction

Figure 5.14: Summary of settlement results

Figure 5.15: Typical results of a Menard pressuremeter test

Figure 5.16: Menard pressuremeter test results at depth of 2.25m before treatment

Figure 5.17: Menard pressuremeter test results at depth of 3.0m before treatment

Figure 5.18: Menard pressuremeter test results at depth of 3.5m before treatment

Figure 5.19: Menard pressuremeter test results at depth of 2.25m after treatment

Figure 5.20: Menard pressuremeter test results at depth of 3.5m after treatment

Figure 5.21: Comparison of K_0 with published data

Figure 5.22: Strength profiles prior to the treatment

Figure 5.23: Strength profiles at Zone 1 prior to, one month and 8 months after the treatment

Figure 5.24: Strength profiles at Zone 2 prior to, one month and 8 months after the treatment

Figure 5.25: Strength profiles at Zone 3 prior to, one month and 8 months after the treatment

Figure 5.26: Comparison of strength increases at different distances from the column surface

Figure 5.27: Comparison of V_s prior to and after the treatment

Figure 5.28: Comparison of G_0 prior to and after the treatment

Chapter 6

Laboratory Investigation

6.1 General

In addition to the full-scale field study described in Chapter 5, a laboratory testing programme was also included in determining the additional soil, lime, and fly ash parameters for analyses and comparisons. The laboratory tests were conducted at the Institute for Research in Construction, National Research Council of Canada, and consisted of the following aspects:

1. plane strain compression tests for determining the lime parameters:
2. resonant column tests for determining the dynamic shear modulus of the soft sensitive clay.

6.2 Determination of Free Volumetric Expansion Coefficient, (C_1)

C_1 of a pure quicklime column depends on the compacted dry unit weight of the lime column and the dry unit weight of the slaked column. Its value can be determined either by means of a laboratory test or by a theoretical calculation if the values of the compacted dry unit weight of a lime column and the dry unit weight of the slaked column are known. The latter is given in Table 2.1.

In practice, it is more convenient to determine the value of C_1 by laboratory tests. The volume of a quicklime compacted to a required dry unit weight (V_q) and the resulting volume of the lime after slaking under the atmospheric pressure are measured respectively. Thus, C_1 is obtained from:

$$C_1 = \frac{\Delta V}{V_q} \quad (6.1)$$

where ΔV is the volume increase due to the lime slaking.

The calculated C_1 is applicable to the pure quicklime column installed by a non-displacement method. For a non-displacement lime column with an additive, such as fly ash, and in-situ mixed column, C_1 should be multiplied by a factor equal to the volume ratio of the quicklime to the column. For the displacement lime column, C_1 is increased by 1.0.

6.3 Plane Strain Compression Test

In order to determine the compressibility of unslaked lime and slaked lime or lime-fly ash mixture, plane strain compression tests were performed on specimens prepared in the laboratory. Specimens of lime or slaked lime are formed in a mold. The top cap and base are fixed to provide a plane strain condition.

The test is performed by applying incremental cell pressure at constant time intervals. Volume readings are taken at each increment. The compression data are presented by a pressure versus volumetric strain curve. The volumetric strain is defined as the ratio of the corrected volume increase, which takes into account the volume change due to expansion of the test system, to the initial volume of the quicklime. A typical curve of the pressure versus volumetric strain is illustrated in Figure 6.1.

The compressibility of the quicklime, C_q , is defined as the slope of the curve from the tests on the quicklime and is written as:

$$C_q = \frac{\Delta \epsilon_{vq}}{\Delta p} \quad (6.2)$$

where $\Delta \epsilon_{vq}$ = the change in the volumetric strain of the quicklime within a pressure increase of Δp .

Similarly, the compressibility of the slaked lime, C_s , is defined as the slope of the curve from tests on slaked lime.

6.3.1 Compressibility of quicklime and slaked lime

A typical compression test result is shown in Figure 6.1 where the pressure is plotted against the corrected volumetric strain. The coefficient of compressibility of quicklime-fly ash mixture is estimated from the curve. The maximum pressure in the field is assumed to be 150 kPa for this case. A series of compression tests were conducted on samples of both quicklime-fly ash mixture and slaked lime-fly ash mixtures which were compacted at various dry unit weights. A relationship between C_v/C_s and dry unit weight of the mixture is shown in Figure 6.2. It can be seen that C_v and C_s corresponding to the dry unit weight of 8 kN/m^3 are 2.74 MPa^{-1} and 0.685 MPa^{-1} , respectively.

6.4 Resonant Column Tests

Resonant column tests were conducted on undisturbed soil samples taken from the site prior to the treatment for determining the constants, K , n and m as shown in Figure 6.3. These constants are used for the estimation of the increase in dynamic shear modulus which is presented in Chapter 7. Field test results reported in Chapter 6 showed that the soil within 5 m of depth are grouped into two types and separated at a depth of 3 m. Undisturbed soil samples were collected both at depths of 2.2 m and 3.5 m.

Resonant column tests were also conducted on undisturbed soil samples taken from Zone 1 at the middle between columns at one month after the treatment.

The results from a series of tests are taken as the reference for the comparing with analytical solution.

6.4.1 Stokoe resonant column apparatus

The Stokoe resonant column test apparatus was used to measure the dynamic shearing properties of soils. A schematic diagram of the resonant column test apparatus is shown in Figure 6.4 (Chen, et al., 1995). The basic principle of the resonant column test is to excite a cylindrical specimen configured to a fixed-free type column at first-mode torsional resonance. The bottom end of a soil specimen is rigidly fixed to the base while the top end is connected to a coil-magnetic drive system which is used to vibrate the specimen in torsional motion. This drive system consists of a drive plate, drive coils, a sine wave generator, and a power amplifier. The first-mode resonance is defined as the frequency at which maximum motion at the top of the specimen is obtained during a sweep of frequencies starting at 5 Hz. The shear modulus of the soil is determined from the resonant frequency, specimen properties and as well as system constants.

In the resonant column test, the calculation of shear wave velocity, V_s , is based on the theory of elasticity.

V_s is calculated from:

$$V_s = \frac{2\pi h}{\beta p} \quad (6.3)$$

where h is height of soil specimen; p is period of vibration at resonance in milliseconds and β is calculated by solving the following expression:

$$\frac{I}{I_0} = \beta \tan(\beta) \quad (6.4)$$

in which I is the mass polar moment of inertia of soil specimen and I_0 is the mass polar moment of inertia of resonant column drive system.

Once the value of shear wave velocity is determined from Equation 6.3, shear modulus, G , can be calculated:

$$G = \rho V_s^2 \quad (6.5)$$

in which ρ is the mass density of soil.

6.4.2 Results of resonant column tests

Five resonant column tests were performed on undisturbed soil samples collected at different depths one month after treatment. The samples were taken from Zone i at the subject test site at midway between cc' imns. The test results are shown in Figure 6.5 and will be fully discussed in next chapter.

Figure 6.1: Determination of compressibility of lime

Figure 6.2: Relationship between compressibility and dry unit weight

Figure 6.3: Determination of K and n

Figure 6.4: Schematic diagram of resonant column apparatus (After Chen, et al., 1995)

Figure 6.5: Dynamic shear modulus profile of the treated soil at Zone 1

Chapter 7

Comparison of Estimated and Measured Results

7.1 General

The validity of the approach developed for estimating the increases in the horizontal effective stress, static shear strength, and dynamic shear modulus of the treated soil may be assessed by a comparison of the estimated and the measured values. This comparison is presented in this chapter.

7.2 Selection of Parameters

7.2.1 Lime-fly ash column parameters

The free volumetric expansion coefficient of quicklime, determined from the laboratory tests as presented in Chapter 6, was equal to 0.35 for the non-displacement

column with a mixture of 7 to 3. C_v and C_s values were selected on the basis of the results from the compression plane strain tests, as reported in Chapter 6. C_v and C_s are equal to 2.74 MPa^{-1} and 0.685 MPa^{-1} , respectively. The maximum suction of lime-fly ash column at the beginning of the slaking is assumed to be 10 kPa .

7.2.2 Soil parameters

The drained Young's moduli for the analysis are shown in Tables 5.1 and 5.2. These values were selected from the results of the Menard pressuremeter test, as interpreted in Chapter 6. The drained Poisson's ratio is assumed to be 0.3.

The coefficient of permeability of soils is affected not only by the grain size, grain size distribution, but also by the viscosity of the permeating water. Since the viscosity of water is a function of its temperature, it is necessary to take into account the influence of temperature increase due to lime slaking on the permeability of soils. The influence of water temperature on the coefficient of permeability of soil can be estimated from the expression:

$$k_{20} = k_t \frac{\eta_t}{\eta_{20}} \frac{\gamma_{w20}}{\gamma_{wt}} \quad (7.1)$$

where k_{20} , γ_{20} , and η_{20} are the standard coefficient of permeability, the unit weight of water and the coefficient of viscosity for water at a temperature of 20°C , respectively; k_t , γ_{wt} , and η_t are those at a temperature of t degree, respectively.

Suppose the soil temperature increases from 20°C to 40°C as reported in Chapter

2. the coefficient of permeability corresponding to the rise in the temperature of soils due to heat release of lime slaking, k_{40} , can be obtained from Equation 7.1.

$$k_{40} = k_{20} \frac{\eta_{20} \gamma_{w40}}{\eta_{40} \gamma_{w20}} = k_{20} \times \frac{1}{0.68} \times \frac{0.993}{0.9986} = 1.5k_{20} \quad (7.2)$$

Substituting the value of permeability of the soil into Equation 7.2 and considering the process of lime slaking yield a value of $1 \times 10^{-9} m/s$.

The initial shear strengths before treatment of the site are based on the work of Bozozuk and Leonards (1972) as shown in Figure 5.2.

7.3 Horizontal Effective Stress Increase

The increased values of horizontal effective stress at different depths have been estimated using the proposed method in Section 3.6.

Figure 7.1 shows the estimated horizontal effective stress profiles after the treatment for both the elastic and the elasto-plastic models. The dotted line in Figure 7.1 represents the estimated effective stresses from the elastic model while the solid line denotes those from the elasto-plastic model. The results show that the values from the elasto-plastic solution are lower than those from the elastic solution.

A comparison of the estimated and the measured effective stresses after the treatment is also plotted on Figure 7.1. A similar trend is noted between the estimated and the measured horizontal effective stresses. The estimated values from the elasto-plastic model are slightly higher than the measured values while those from

the elastic model yields a significant overestimation. The comparison suggested that the elasto-plastic model should be employed in the analysis.

In general, the estimated values from the elasto-plastic model and the measured results are in good agreement. However, the horizontal effective stress near the bottom of the column tends to be overestimated. This discrepancy may be due to the bounding effect, i.e., energy loss due to spherical expansion near the bottom of the column and/or due to soil relaxation during the consolidation of soils.

From the practical point of view, however, the proposed analysis' method provides a reasonable estimate of the horizontal effective stress after the lime column treatment. This method, therefore, has its potential application for estimating the strength of treated soil as illustrated in following sections.

7.4 Shear Strength of the Treated Soil

The strength increases at different depths have been estimated using the proposed approach. The values of c' and ϕ' were obtained from the strength envelope shown in Figure 7.2 (Law, 1974).

The profiles of strength increase obtained from the analysis for Zone 1, Zone 2, and Zone 3 are compared with the measured values from the piezocone tests, as shown in Figure 7.3, Figure 7.4, Figure 7.5, and Figure 7.6, respectively.

The solid line, dotted line and dashed line in Figure 7.3 represent the estimated values at midway between the columns, and the measured values from the piezocone

test one month and 8 months after the treatment, respectively. It can be seen that the estimated values at midway between the columns are in good agreement with the measured values. At a distance equal to one-fourth of the column spacing, the correlation between the estimated, as dotted in Figure 7.4 and the measured values, as solid line in Figure 7.4, is still reliable even though they are not as good as those at midway between the columns. Figure 7.4 also indicates that the proposed model slightly underestimated the shear strength of the treated soil. Such higher underestimation compared with that at midway between the columns may be caused by the more cementation effect and/or cation exchange effect closer to the surface of a column.

Figures 7.5 and 7.6 present a comparison of the strength increase between the estimated and the measured values for Zone 2 and Zone 3, respectively. The comparison again shows a good agreement between the estimated and the measured values within the treated area, except for that near the base and the top of the column. This discrepancy may be due to the assumption of plane strain condition as discussed previously.

The overall estimated values are about 17 % lower than the measured values. This minor underestimation errs on the safe side and is acceptable from the practical point of view. Therefore, this method is suggested to be a useful tool for designing lime columns for strengthening soft soils.

7.5 Dynamic Shear Modulus of the Treated Soil

7.5.1 Dynamic shear modulus

Two sets of constants, K , n , and m were obtained from the resonant column test results, as described in Chapter 6. Substituting the values of K , n and m into Equation 3.56 yields the following equations:

$$\begin{aligned} G_0 &= 2.5\sigma_c^{0.52}(OCR)^{0.24} & (\text{depth} < 3m) \\ G_0 &= 2.3\sigma_c^{0.51}(OCR)^{0.24} & (\text{depth} > 3m) \end{aligned} \quad (7.3)$$

An evaluation of the dynamic shear modulus of the strengthened soil has been made from the above equations. The estimated values of G_0 at midway between the columns at Zone 1 are illustrated in Figure 7.7, as solid lines.

The measured values of initial dynamic shear moduli from resonant column tests are shown in Figure 7.7, as hollow circles. It can be seen that the profile of G_0 from the proposed method agrees well with that from the resonant column tests. The estimated values are slightly greater at the bottom of the column. This discrepancy may be due to the end effects of the column.

The estimations could be improved by taking into account the three-dimensional effect and chemical effect in modelling the lime column expansion. Additional investigation regarding to the in-situ Poisson's ratio can also improve the overall estimations.

7.5.2 Average dynamic shear modulus

The average shear moduli of the treated soil are calculated from Equation 3.66. A comparison between the estimated and the measured $(G_0)_{ave}$ from the in-situ seismic cone tests conducted at midway between columns is shown in Figure 7.8. The solid line in Figure 7.8 represents the estimated values and the filled circles represent the measured values. The comparison shows that the proposed method yields a reasonable overall estimation of the dynamic shear modulus of the treated soil. The estimated average value of $(G_0)_{ave}$ around the column bottom is larger than the measured value. This discrepancy may be due to the spherical expansion near the bottom of the column while the model assumes radial expansion only. The estimated $(G_0)_{ave}$ within the middle section is smaller than the measured value. This is due to the effects of the rigidity of the column. The overall estimated $(G_0)_{ave}$ values are slightly lower than the measured values. Hence, the estimate is slightly conservative.

7.6 Summary

The comparisons of estimated and experimental test results on horizontal effective stress, static strength, and dynamic modulus of the treated soils are presented in this chapter. It is shown that the prediction from the proposed method gives adequate estimates of horizontal stress increase, static strength increase, and dynamic modulus increase. This method has a high potential to become a useful tool for designing lime-fly ash columns for improving the soft soils.

Figure 7.1: Comparison of estimated and measured horizontal effective stress

Figure 7.2: Strength envelope at the test site (After Law, 1974)

Figure 7.3: Comparison of estimated and measured shear strength for Zone 1

Figure 7.4: Comparison of estimated and measured shear strength for Zone 1

Figure 7.5: Comparison of estimated and measured shear strength for Zone 2

Figure 7.6: Comparison of estimated and measured shear strength for Zone 3

Figure 7.7: Comparison of estimated and measured G_0

Figure 7.8: Comparison of estimated and measured $(G_0)_{ave}$

Chapter 8

Conclusions and Recommendations

This chapter represents its conclusions along with recommendations for the further work.

8.1 Conclusions

The expansion of the lime columns for strengthening soft soils has been modelled by means of the theory of cylindrical cavity expansion in an elasto-plastic material. This model considers consolidation theory taking into account the process of the time-dependent changes in total stress, and the generation and concurrent dissipation of excess pore water pressure. Expressions for the increase in total stress and excess pore pressure in the soil among the columns have been derived in terms of the radial stress increase at the column wall and solved by a finite difference scheme.

A parametric study has been conducted to determine the relative importance of lime parameters and soil parameters used in the proposed model.

Equations have been derived for estimating the increases in both static shear strength and dynamic shear modulus of the treated soils in terms of the changes in effective stresses.

The model has been examined with two case studies and a full-scale experimental study.

The conclusions of this study can be summarized in the following:

a. Parametric Study

The parametric study shows:

1. The excess pore pressure distribution along the radial direction is non-linear. i.e., the effects of improvement of lime columns on the soil vary with the distance from the column surface.
2. The magnitude of the undrained pore pressure is a function of the undrained shear strength, Young's modulus and Poisson's ratio of the soil, and the spacing of the columns.

b. Case Records

The study on two case records involving clayey silt and silty clay shows:

1. The proposed method gives a good estimate of maximum excess pore water pressures due to lime column expansion.
2. The proposed method can be used successfully to estimate the total stress increase, and generation and dissipation of the excess pore water pressure.

Hence, the proposed method is applicable for estimating changes in the effective stresses.

c. Full-scale Experimental Study

The full-scale experimental study in a soft Champlain Sea clay shows:

1. The horizontal effective stress of the treated soil has increased significantly.
2. The elasto-plastic model can be employed for estimating the increase in horizontal effective stress, as given by Equations 3.51 and 3.52. This model gives much better results than the elastic model.
3. There are substantial increases in both the static shear strength and the dynamic shear modulus of the treated soil. Therefore, the lime-fly ash column method is effective in increasing the static shear strength and dynamic shear modulus of the soft sensitive clay.
4. For the in-situ compacted lime-fly ash column method, the increases in shear strength of the treated soil are mainly due to the expansion effect of lime column initiated by lime slaking.
5. The proposed method is applicable to estimate the different shear strength increases corresponding to different spacings of the lime columns.
6. The estimated and measured static shear strength and dynamic shear modulus are in good agreement. It is suggested that the approach de-

veloped here can be used to calculate and predict the shear strength and dynamic shear modulus.

7. The results of this study suggest that the proposed method has a high potential of becoming a useful tool for the design of lime-fly ash columns to improve soft soils, including soft sensitive clays.

8.2 Recommendations for Further Study

1. A graphical user interface tool will be desirable for designing lime-fly ash columns.
2. An analytical solution for the prediction of settlement is worth developing.
3. The proposed method can be extended to estimate the increase in the liquefaction resistance of a saturated granular soil with significant amount of fines.
4. The estimations could be further improved by modelling the lime column expansion in 3-D analysis and/or taking account of the chemical effect and cementation effect.
5. The effects of freezing and thawing on the long-term behaviour of lime column should be investigated
6. For the analysis of dynamic performance of a treated soil, the dynamic

properties of the lime-fly ash column should be studied.

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