THE DESIGN AND BEHAVIOUR OF CEMENT COLUMN IN SOFT BANGKOK CLAY

Engineering

By

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Statement of Originality

This thesis contains no material, which has been accepted for the award of any other degree or diploma in any tertiary institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person, except when due reference is made in the text.

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ABSTRACT

This thesis involves a general review of deep soil treatment methods, which leads to a detailed review of cement columns as cost effective medium depth foundations. A field study involving the improvement of soft Bangkok clay by cement jet grouting was undertaken using cement columns in the range of 5m to 15m. The columns were loaded to ultimate to obtain data for verification of the design process. Subsequent to testing the columns were cored to allow correlation between mix design and in situ column behaviour.

It was found that the existing design process was inadequate due to high variation between laboratory mix design and installed column strength. The high variation resulted in piles crushing before ultimate bearing capacity could be achieved. A new design process has been developed that will help reduce the likelihood of poor field behaviour.

Finally a parametric study using a Finite Element Method was undertaken and the results compared with the data from the field testing. The computer model for the cement columns was then calibrated to allow its use to predict the ultimate load of the cement column without the need for further expensive full scale tests.

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INTRODUCTION

1.1. General Background

Cement columns are used to improve soils at depth by using cementitious materials (usually using cement or/and lime) which are mixed with the soil in situ using a deep mixing method. Soil improvement by these method results in a new material with higher strength, reduced compressibility and lower permeability compared to the native soils.

Generally, construction in soft clays means that the soil must be improved before loading, otherwise the soft clays will undergo excessive settlement, often causing darmage to the surcharging structure. There are many methods available to improve soft clays including prestressed concrete piles, bored piles, band drains and cement columns. The optimum method for a specific project depends on both the structures being supported and the construction process. Bored piles are the most expensive and driven prestressed concrete piles are the second most expensive, with both being suitable for high buildings (typically more than four stories). Band drains and cement columns have a similar construction cost and are cheaper than the previous mentioned pile methods. Band drains and cement columns are suitable for light structures such as houses, factories, low rise building, roads and airfields. Even though band drains are similar in cost to cement columns, band drains take a longer time to improve soft clays than cement columns as strength gain using the band drains is a function of soil permeability and load surcharge. They take typically about six months to one year to achieve the design level of consolidation whereas cement columns take only one month to install and reach working capacity.

1.2. Scope and Objectives of the study

This thesis involves a general review of deep soil treatment methods, a detailed review of cement columns and a field study involving the improvement of a soft Bangkok clay by cement jet grouting. The research on the improvement of soft Bangkok clay was undertaken with the following objectives:

1. to study the strength characteristics of a natural soft Bangkok clay
2. to study the strength characteristics of a cement column using the unconfined compressive strength test of cored samples as an indicator
3. to determine cement columns ultimate bearing capacity
4. to study the settlement characteristics of the cement columns during static load testing
5. to determine the most appropriate factor of safety for design of cement columns in soft Bangkok clay
6. to undertake a parametric study using a Finite Element Method

In order to achieve these objectives, a systematic investigation of the characteristic of the in situ soil was carried out. Natural soft Bangkok clay from a selected construction site was taken as the base clay for treatment. Compression (UC) tests, Oedometer tests, Drained and Undrained Triaxtial tests were conducted on both treated and untreated clay.

In the first part of the testing program, the basic engineering properties, field vane shear test and the compressibility and consolidation characteristics from Oedometer tests were determined for the native soil. The native soil was then treated with cement at 200kgs/m^3 and cured for a period for 2 months. The basic engineering properties and the unconfined strength of the cured modified soil were then determined.

The second part of the testing program involved a comprehensive program of triaxial testing of drained and undrained testing (constant stress ratio) of native soil.

The third part of this study was to investigate the bearing capacity of the cement columns, which were installed in the testing area. The length of the cement columns was selected as 5 m, 7 m, 9m, 11 m, 13 m, 15m, respectively. To determine the ultimate bearing capacity of these tested columns, static pile load testing was employed

The fourth part of the testing program was to investigate the strength of cement columns from cored samples by using the unconfined strength test.

1.3 Thesis Layout

To achieve the aims of this thesis, an extensive literature review has been undertaken focussing on the areas of characteristics of cement treated clay material, field application of cement treated clays, cement column installation methods, cement column design methods and modelling using the PLAXIS program. The summary of the literature review is contained in Chapter 2 and 3.

Chapter 2 provides a review of the literature ranging from the modification of the clay soil matrix by cement treatment to the effect of low cement additions on strength and stiffness of soil cement columns. In Chapter 3 the methods used to install cement columns and their field applications are outlined. Also introduces the design methods for cement columns and outlines the different methods used to determine the ultimate load capacity of a single pile. Moreover an introduction to the PLAXIS program is provided.

Based on the outcomes of the literature review, a program of experimental work was developed to monitor the behaviour of cement columns and to evaluate surrounding soil parameters. This discussion forms the basis of Chapter 4.

The experimental results are discussed in Chapter 5 related to the laboratory and the field experimental work, described in Chapter 4.

An analyse of the cement column/soil interaction and ultimate load prediction using the PLAXIS program is contained in Chapter 6. Also a comparison of the ultimate load obtained in the field and that predicted by program is provided.

There are many design parameters that effect to the prediction of column strength and column capacity. These are discussed in Chapter 7.

Chapter 8 provides a step-by-step description of the major process and each step has some briefly explanation for the inexperienced designers.

A summary of the findings and recommendations for further research are presented in Chapter 9.

Chapter 2

LITERATURE REVIEW

2.1. Introduction

According to Davidson (1963), the instance of first soil improvement using a cement/soil mix occurred in 1915 because the concrete mixing machine had broken. Oak Road in Saratosa, Florida was constructed by digging shell up from a pond, mixing with sand and cement and then compacting the road with steamrollers.

In 1932, Dr. C.H. Moorefield of South Carolina Sate Highway Department used soil-cement to construct many roads and concluded that soil-cement was an effective cost material to construct roads. This work by Moorefield stimulated much research into the behaviour and benefits of soil-cement. (Davidson, 1963).

In 1935, the South Carolina State Highway Department and Bureau of Public Roads and the Portland Cement Association cooperated to construct 1.5 miles of a soil-cement road as an experiment. Following this success, the Portland Cement Association constructed many roads in Michigan, Missouri and Wisconsin. (Mitchell, Veng and Monismith (1974)

More recently in the late 1960's, Japan introduced soil improvement techniques based on soil-cement by using deep mixing method. (Somchai, 1995)

There are now many countries that have researched and successfully developed techniques for the design and installation of cement columns across a range of applications. For example, cement columns are used for temporary or permanent earth retaining structures, excavation support walls, strengthening floors against base heaving in excavation, stabilizing soil along river banks, strengthening existing retaining structures, the construction of impervious cut-off walls and environmental remediation (Kasem and Pinit, 1998).

2.2. Fundamental Concepts of Cement Stabilization

2.2.1 Type of Cement

The American Society for Testing and Material (1986) divided cements into five types as follows.

2.2.1.1 Type One (Ordinary Portland Cement-OPC) is the most popularly produced and used. It is appropriate for general concrete work, which requires no special properties of cement in the construction process.
2.2.1.2 Type Two (Modified Portland Cement) can increase the resistance to sulphate attack and is used with concrete work that requires high heat.
2.2.1.3 Type Three (High-Early Strength Portland Cement) has a very short setting time and gives high strength in short period. It is this suitable for urgent construction work and repair.
2.2.1.4 Type Four (Low-Heat Portland Cement) gives low-heat when it is setting. It is appropriate with mass concrete work. Now there is no Type Four produced in Thailand.
2.2.1.5 Type Five (Sulphate Resistance Portland Cement) has low Tri Calcium Aluminate for protecting concrete against sulphate attack.

The type of cement mainly used in soil mixes in Thailand is OPC (Type One) due to cost implications and the lack of need for special cements.

2.2.2 Soil–Cement Action

The cementation process of the compacted soil-cement depends on the overall hydration reactions within the soil mass. Soil particles will be grouped together but the grouping is not only occurring between soil particle and cement particle but also between cement particle and cement particle. The hydration reaction between soil particle and cement particle can be categorized as follows (Rubright and Welsh, 1993).

(A) Fine soil particles - Clayey soils and silty soils can be used to make soil-cement however these soils require large amounts of cement compared to coarse soil particles. This is because fine soil particles have a much larger contact surface than coarse soil particles and thus more cement is required to bind the soil. Cement not only reduces plasticity but also increases the shear strength of the soil because of the cement reaction around the soil particle. The absorbed water around the soil particles will be replaced by cement, and the replacement will reduce swelling and softness of soil.

(B) Coarse soil particles - The reaction between cement and coarse-rough soil particles will act similar as concrete except that the cement paste will not fill up the voids between the soil particles. The cementation process involves both mechanical interlock and chemical cementation. In the case of sand, the cementation happens around the contact surface area. When sand is compacted, the voids between the sand particles are reduced and contact surface area is decreased. The end result is that the cemented mass has a higher strength. If the sand is uniformly graded, the contact surface area is less than well-graded sand. Thus treatment of uniform graded sand will involve the use of more cement than well-graded sand.

The main factors of mixing control between cement and soil are :

-the condition of natural soil

-the properties of cement

-the moisture content of compacted soil-cement

-the required density of compacted soil-cement

The moisture content and density of compacted soil-cement can be controlled according to AASHO-T 134 and ASTM D 558 (Rubright and Welsh, 1993), assuming that mixing and curing are done correctly. The factor of mixing control will depend on the condition of natural soil and cement volume.

2.2.3 Mechanical of Soil-Cement Stabilization

As indicated earlier Type One Portland cement is the most widely used in soil stabilization. The composition of the Type One Portland cement used in this study is shown in Table 2.1 (Kasem and Pinit, 1998). A Portland cement particle is a heterogeneous substance, containing tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A) and a solid solution described as tetracalcium alumino-ferrite (C_4AF) (Lee, 1956). These four main constituents are the major strength producing compounds. When the pore water of the soil encounters with the cement, hydration of the cement occurs rapidly and the major hydration (primary cementitous) products are hydrated calcium silicates (C_2Sh_x ,$C_3S_2H_x$), hydrated calcium aluminates (C_3AH_x,C_3AH_x) and hydrated lime $Ca(OH)_2$ (Broms, 1974). The first two of the hydration products listed above are the main cementitious products formed and the hydrated lime is deposited as a separate crystalline solid phase. These cement particles bind the adjacent cement grains together during hardening and form a hardened skeleton matrix, which encloses unaltered soil particles.

Table 2.1 Properties of Type One Portland Cement (Kasem and Pinit, 1998)

Chemical Composition	%(by weight)
Silicon dioxide (SiO_2)	21.63
Aluminum oxide (Al_2O_3)	5.09
Ferric oxide (Fe_2O_3)	2.92
Magnesium oxide (MgO)	0.91
Sulphur trioxide (SO_3)	1.68
Loss of ignition	0.82
Insoluble residue	0.11
Tricalcium silicate ($3Ca,SiO_2$)	58.00
Tricalcium aluminate ($3CaO,Al_2O_3$)	8.60
Fineness, specific surface (Blaine)	3000 cm^2/g

In addition, the hydration of cement results in a rise of pH value of the pore water, which is caused by the dissociation of the hydrated lime. The strong bases dissolve the soil silica and alumina (which are inherently acidic) from both the clay minerals and amorphous materials on the clay particle surfaces, in a manner similar to the reaction between a weak acid and strong base (Broms, 1974). The hydrous silica and alumina then gradually react with the calcium ions liberated from the hydrolysis of cement to form insoluble compounds (secondary cementitious products), which hardens when cured to stabilise the soil. This secondary reaction is known as the pozzolanic reaction (Broms, 1974)

The reactions, which take place in soil-cement stabilization, can be represented in the equation given below. The reactions given here are for tricalcium silicate (CS) only, because they are the most importance constituents of Portland cement:

$$C_3S + \text{water } (H_2O) \longrightarrow C_3S_2H_x(\text{Hydrated gel}) + Ca(OH)_2 \text{ ---- } 2.1$$

(Primary cementitious products)

$$Ca(OH)_2 \longrightarrow Ca^{++} + 2(OH)^- \text{ ------ } 2.2$$

$$Ca^{++} + 2(OH)^- + \text{Soil Silica } (SiO_2) \longrightarrow CSH \text{ ------ } 2.3$$

(Secondary cementitious product)

$$Ca^{++} + 2(OH) + \text{Soil Alumina } (Al_2O_2) \longrightarrow CAH \text{ ------ } 2.4$$

(Secondary cementitious product)

When pH<12.6, then the following reaction occurs:

$$C_3S_2H_x \longrightarrow C_3S_2H_x \text{ (hydrated gel)} + Ca(OH)_2 \text{ ------- } 2.5$$

However, the pH drops during pozzolanic reaction and this drop in the pH tends to promote the hydrolysis of CSH. The formation of CSH is beneficial only if it is formed by the pozzolanic reaction of lime and soil particles, but it is detrimental when CSH is formed at the expense of the formation of the CAH, whose strength generating characteristics are superior to those of CSH (Broms, 1984). The cement hydration and pozzolanic reaction can last for months, or even years, after the mixing and the strength of cement treated clay is expected to increase with time.

2.2.4 Structure of Clay Cement Skeleton Matrix

It can be seen that the five equations (2.1-2.5) that the cementation process creates CSH and CAH which help bond and consolidate the soil. The bonding modification is shown below in figure 2.1 (Herzog, 1964).

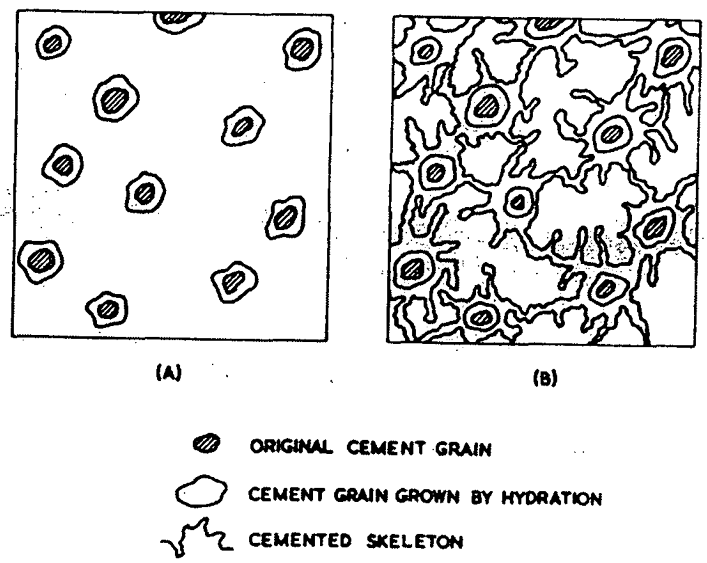


Figure 2.1 Comparison of clay cement models (A)Cement grains 'floating' in clay matrix. (B)Skeleton matrix structure formed by cement- soil interaction. (Herzog, 1964)

2.3 Effects of Cement on the Physical Properties of Soils

2.3.1 Grain Size Distribution

The soluble products of cement hydration cause the electrolyte concentration of the pore water and the pH value to be increased. The dissolved bivalent calcium ions (Ca^{++}) replace the monovalent ions, which are normally attracted to the surface of the negatively charged clay particles (Assarson et al, 1974). The crowding of Ca^{++} ions onto the surface of the clay particles brings about the flocculation of the clay (Herrin and Mitchell, 1961). The flocculation can also be brought about by the hydration of cement, resulting in a change to a coarser grain size distribution of the soil particles and an ensuring change in materiel properties.

2.3.2 Permeability

The addition of cement to clay increases the permeability of the soils, due to the flocculation of the soil particles. The permeability of the cement-treated clays however is reduced with increasing cement content and curing period as pores are progressively eliminated. This is most probably due to the impervious hardened cement hydrates, which hinder the movement of the pore water in the enclosed matrix. However, the permeability of lime-treated clays has been shown to increase with increasing lime content and age (Broms, 1986).

2.3.3 Plasticity

The plastic limit of the soil generally increases with an increase in the cement content, while the plasticity index reduces. The liquid limit, on the other hand, is not usually affected, or is only slightly affected. As a result of the clay's change in plasticity, the shear strength is increased, while the compressibility of the treated soils is reduced (Broms, 1986).

2.3.4 Compressibility

The consolidation yield stress of the treated soil increases with increasing cement content. The compression indices, at consolidation stress conditions less than the consolidation yield stress, are extremely small. The compression indices at consolidation stress conditions greater that the consolidation yield stress however remain the same. (Suzuki, 1982). Law (1989) reported that cement contents of 10% provides the best improvement in term of compression index (C_c) and coefficient of consolidation (C_v) for clays. Bangkok clay with cement contents of 7.5% and below does not show significant improvement of consolidation characteristics.

2.3.5 Strength

The increase of the shear strength of soft clays is partly caused by ion exchanges when monovalent ions, (e.g. Na^+ and K^+) are replaced by the multivalent ion (Ca^{++}). Part of the immediate increase in shear strength is caused by the flocculation of the clay and partly by the reduction of the water content (Broms, 1986). The strength increase in the cement–clay skeleton matrix structure (Herzog, 1963,1967) is a result of the increases in the frictional resistance and cohesion of the aggregation of hydrated cement cores and surrounding clay particles. These form a cement-clay skeleton-matrix structure, which contributes towards the major strength contribution by significant interlocking. This then contributes to the improvement of the frictional component (ϕ') of the shear strength. The second process involves the reduction of the thickness of the double-layered water, caused by the ion exchange and the flocculation of the clay particles. This reduction in the interparticle space increases the inter-particle bond strength (Broms, 1986). The inter-particle bond strength is also increased with the flocculation by the secondary cementitious material. All of the steps of the second process contributes to the improvement of the cohesive component (C') of the shear strength.

It is well known that formation of the primary and secondary cementitious materials proceeds slowly and continuously and many extend to years. This is because the strength of the cement-treated clays will generally increase with time until the completion of the reaction. Broms (1986) investigated the applications of cement columns in soft clays in the Southeast Asian region, in which the general properties of the clays are relatively similar to Bangkok clay. He reported maximum strengths of soil-cement as listed below:

(a) For 16% cement treatment, the 1-month, 2-month and 4-month strengths are 410kPa, 660kPa and 700kPa respectively.

(b) For 10% cement treatment, the 1-month, 2-month and 4-month strengths are 230kPa, 320kPa and 460kPa, respectively.

The Dry Jet Mixing (DJM) Research Group (1984) also reported that an average value of 67% of the 28-day strength can be achieved at the 7-day age for all cement contents for clays.

2.3.6 Strength Development Index

A Strength Development Index (SDI) can be defined where:

$$SDI = \frac{\text{Strength of stabilized soil specimen - Strength of virgin soil specimen}}{\text{Strength of virgin soil specimen}} \quad \text{-------- 2.6}$$

For example, the strength of soft Bangkok clay is 60kPa. After stabilized the soil, its strength is 600kPa. Thus SDI is (600-60)/60)=9. The SDI provides a good indication of the degree of relative strength improvement that can result from cement treatment taking into account variables such as cement content and curing time. Kamaluddin (1995) reported that increasing cement content and time SDI also increased.

2.4 Predominant factors that Controls Hardening Characteristics of Cement Treated Clay Material

The hardening characteristics of cement treated soil mixtures are dependent on a number of factors. Owing to the large number of alternatives and combinations, it is impossible to tabulate the change in the various mechanical properties as a function of these factors. As a result experimental determination is indispensable in most cases. There are, nevertheless, some predominant factors and these are described in this section. They only however provide information regarding an approximate order of dominance value, and illustrate the relative effect of these factors on the strength and stiffness of the cemented clay.

2.4.1 Type of Cement

Several researchers have investigated the differences in improvement of cement treated clays by using different types of Portland cement (e.g., Clare, 1951,1956, Felt, 1955) and reported that stabilisation by Type Three Portland cement provides better improvement of clayey soil than Type One cement. However, Type One Portland cement is the most popular cement used in soil stabilisation, because it is readily available and of relatively low cost compared with other types of cement.

2.4.2 Cement Content

In general, it has been found that the higher the cement content, the greater the strength of the cement treated clay (Broms, 1986). This behavior is different to the case of lime treated clay. In the case of lime, there is a maximum strength limit that can be obtained at the optimum lime content. Further increase in the lime content will cause a reduction in the improved strength.

2.4.3 Curing Time

In a manner similar to that of concrete and lime treated soils, the shear strength is generally rapid in the early stages of the curing period. Thereafter, the rate of increase in strength decreases with time. The rate of strength gain for cement treated clay is greater than that of lime treated clay in the early stages (Broms, 1986).

2.4.4 Soil Type

The effectiveness of cement decreases with increasing water content and organic content. The improvement decreases generally with increasing plasticity index of the clay (Broms, 1986). The strength increase of cement treated organic soils is often very low. However in spite of the low gains, cement is more effective than lime in the stabilization of organic soils (Miura et al, 1986).

The effects of cement on clayey soils gradually decrease with increasing clay content and increasing plasticity index (Woo and Moh, 1972). In general, when the activity (the degree of plasticity of the clay size fraction of a soil is expressed by the ratio of the plasticity index to the percentage of clay size particles in the soil) is very high, the increase in strength of the soil treated with cement is low. These effect are opposite to the case with lime, since the strength of the lime treated clay depends mainly on the participation of the clay particles in the pozzolanic reactions. For cement treated clay, strength improvement depends mainly on the cementation resulting from the cement hydration. The increase of the shear strength due to the flocculation is often relatively small for marine clays deposited in salt water, since these clays already have a flocculated structure (Broms, 1986).

2.4.5 Curing Temperature

An increase in the soil mass temperature accelerates the chemical reactions and solubility of the silicates and aluminates, resulting in an increase in the rate of strength gain of the treated soil.

2.4.6 Soil Minerals

The strength characteristic of a treated soil is governed by the strength behavior of the hardened cement bodies. In the case of soils having lower pozzolanic reactivity, the strength characteristics of the treated soils are governed by the strength characteristics of the hardened soil bodies (Saitoh, 1985). It then becomes obvious that if improvement conditions are equal, greater strength is obtained from a soil with higher pozzolanic reactivity.

Davison and Gill (1963) found that montmorillonitic and kaolinitic clayey soils were more effective pozzolanic agents, in comparison to clays which contained illite, chlorite or vermiculite. Wissa and Ladd (1964) came to a similar conclusion noting that the amount of secondary cementitious materials produced during pozzolanic reaction of the clay particles and hydrated lime $Ca(OH)_2$ were dependent on the amount and mineral composition of the clay fraction as well as the amorphous silica and the alumina present in the soil. They suggested that the montmorillonite clay mineral probably react more readily than the illites and kaolins because of poorly defined crystallinity (Wissa and Ladd, 1964).

2.4.7 Soil pH

The long-term pozzolanic reactions are improved by high pH values, since the reactions are accelerated due to the increased solubility of the silicates and the aluminates of the clay particles. When the pH value of treated clay becomes lower than about 12.6, the reaction shown earlier in equation 2.5 takes place. The consumed $C_3S_2H_x$ is to produce the CHS and the hydrated lime $Ca(OH)_2$. This reaction reduces the strength of the treated clay at the expense of stronger cementitious material, $C_3S_2H_x$ by producing the weaker cementitious material, CSH (Broms and Boman, 1975).

2.5 The Engineering Characteristics of Improved Soil

Uddin (1995) indicated that the main effect of cement treatment was to modify the behavior of soft clay such that it would change from a normally consolidated to an overconsolidated state. The degree of "overconsolidation" achieved is influenced by the cement content. The stress-strain behavior of the treated soil then becomes similar to a heavily stiff fissured clay in a desiccated dry zone (dry crust) above the ground water level (Broms, 1984). This behavior is similar to that observed in overconsolidated clay or in strongly (self) cemented natural soils (Terashi et al., 1980).

2.5.1 Influence of Confining Pressure on Cement Treated Soils

Broms (1984) reported that both triaxial undrained tests and direct shear tests on treated soils demonstrate that shear strength will increase with increasing confining pressure or normal pressure up to a limiting value. Terashi et al., (1980) conducted triaxial undrained tests to investigate the brittle behavior of lime stabilized clays and formed similar conclusion (Figure 2.2). Broms (1984) plotted the undrained shear strength, C_u, against consolidation pressure, σ_c, and found that for untreated soils, a linear relationship exists between log C_u versus log σ_c. For lime treated soil, the undrained strength is constant up to a consolidation pressure, σ_c where the constant line intersects the linear plot for untreated soil. At consolidation greater than σ_c, C_u increases with increasing σ_c, with C_u vs. σ_c relationship of treated soil the same as that of untreated soils Fig. 2.2.

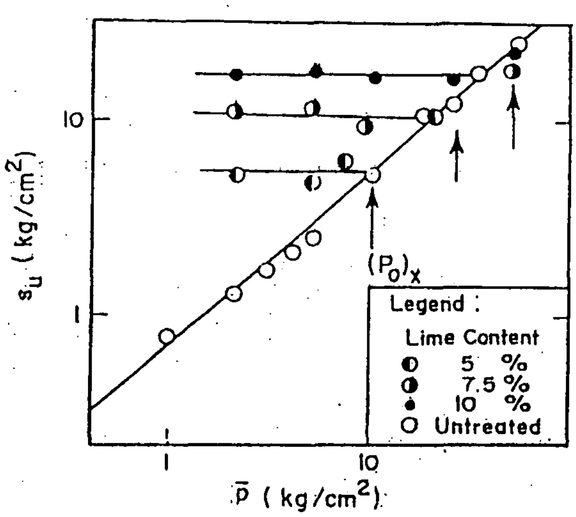


Figure 2.2 Relationship between Strength and Consolidation Pressure for Lime Treated Clay (Terashi et al., 1980)

The nominal preconsolidation pressure of treated soil is about 1.3 times the unconfined compressive strength of the material. Also the compressibility of treated soil is very small for pressures below the consolidation yield stress, but becomes about 1.5 times that of remolded soil over the yield stress (Okumura and Terashi, 1975).

2.5.2 Unconfined Compressive Strength

The unconfined compressive strength, q_u, of cement improved soil is very often used as a measure of the strength of stabilized ground (Suzuki, 1982). Figure 2.3 shows the relationships of the strength, cement contents, the age of improved soil for a number of clays at various sites in Japan. It can be seen from the figure that q_u increases with increasing cement content and age, while decreasing with increasing water content of the soil. Broms, (1984) stated that the unconfined compressive strength of treated clays is related to the strength along the cracks and microfissures present in a stabilised soil sample. Thus UCS at the micro level may not truly effect the actual shear strength of the stabilised soil mass at a macro level.

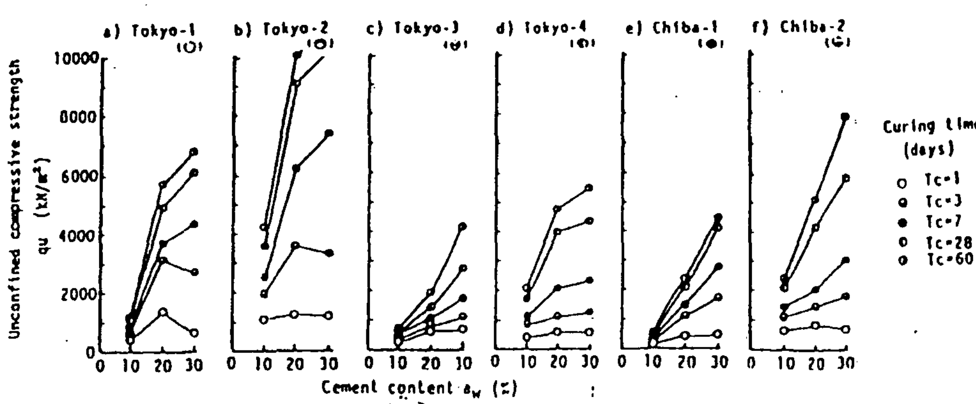


Figure 2.3 Relationships of q_u, Cement Content and Age of Improved Soil (After Kawasaki et al, 1981)

Uddin (1995) found that stress strain curves for cement treated samples increased abruptly until a peak compressive strength and, then suddenly decreased to very low residual values upon further straining. The properties of treated clay exhibited significant increase in strength and modulus of deformation. Unfortunately the clay material was also changed mainly to a brittle and quasi-brittle material. The zonal demarcation of the effectiveness of cement treatment determined by Uddin (1995) is presented in Figure 2.4. The unconfined compressive strength, q_u percentage of cement by weight, A_w plane is subdivided into 3 regions (A, B and C) on the basis of gradient development of the strength gain curve. Based on unconfined compressive strength, 120 and 150 kg/m^3 of cement content and 1 to 2 month curing period were regarded as optimum by Uddin. The (ε_f, A_w) relationships showed that substantial reduction of failure strain occurred up to cement content of 150 kg/m^3

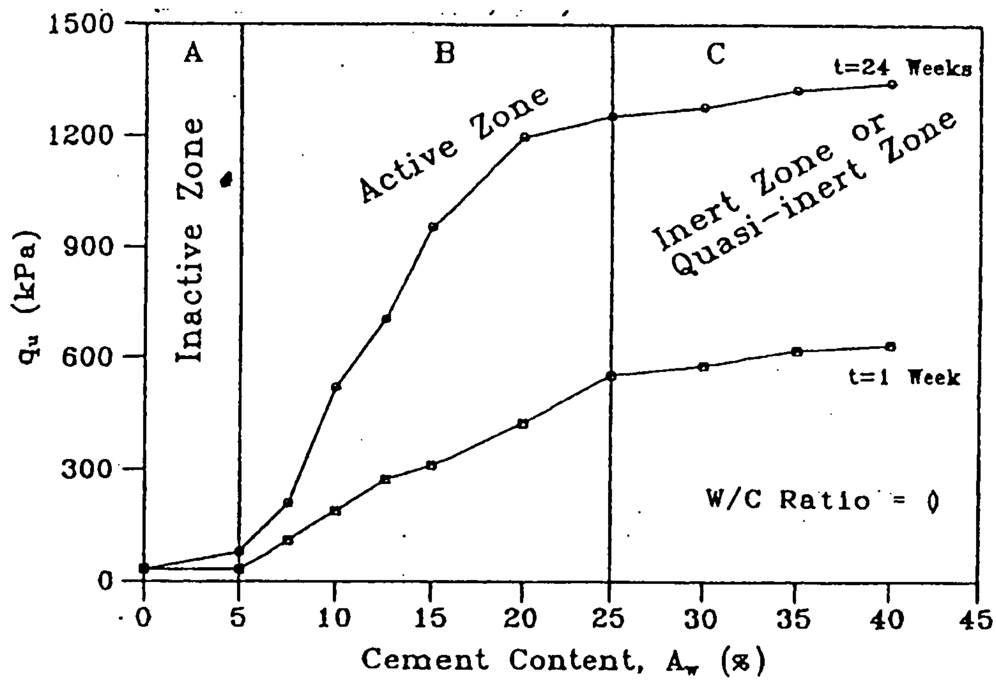


Figure 2.4 Influence of Cement Content on Unconfined Compressive Strength (After Uddin, 1995)

2.5.3 Modulus of Elasticity

Various forms of the Elastic Modulus (Young's Modulus), E, are generally used to express the stress strain relationship of improved soil. Figure 2.5 shows the relationship between q of improved soil and the secant modulus, E_{50} (Kawazaki et al., 1981). The stiffness of treated soil increases exponentially to the strength (Terashi et al., 1983) and this has some importance in elastic soil deflection. Wissa et al. (1965) observed that the initial tangent modulus of treated soil increased with an increase in confining pressure and time. This work was based on results from undrained triaxial tests. Testing conducted by Uddin (1995) showed that the initial tangent modulus versus unconfined compressive strength (E_u, q_u) relationships produced a straight band with narrow scatter. The higher the cement content and curing time, the greater is the degree of stiffness and brittleness of the treated clay.

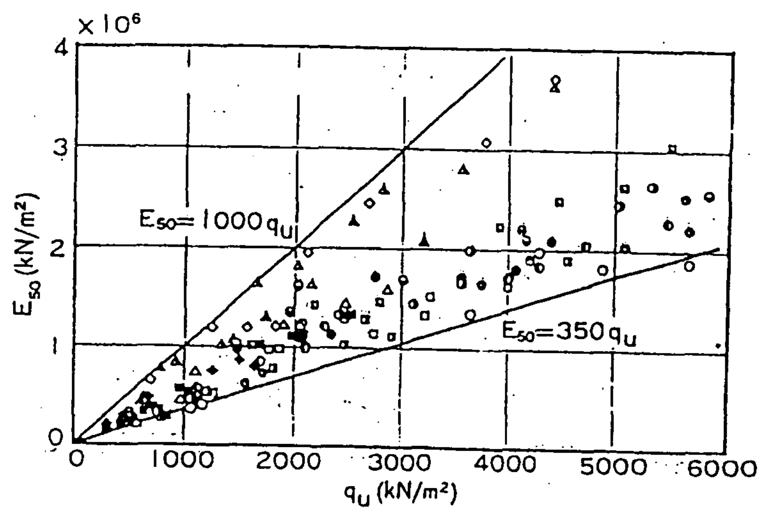


Figure 2.5 Relationship of q_u and E_{50} (Kawasaki et al., 1981)

2.5.4 Shear Strength

Brandl (1981) observed a relatively high angle of internal friction (30 to 45 degrees) from triaxial tests and direct shear tests for clay stabilised with lime and gypsum at low cell pressures and low normal pressures. The increase of the shear strength with increasing normal pressure at direct shear and triaxial reflects the dilatancy of the stabilized soil when the confining or the normal pressure is low (Broms, 1986). Effective stress paths obtained by Wissa et al., (1965) from consolidated triaxial compression tests on both untreated and lime stabilized clays are shown in Figure 2.6. For both untreated and stabilized samples, an essentially straight line can be drawn tangential to the effective stress path for pressures ranging from 0 to 5000Pa, from which the effective shear strength parameters ϕ and C can be obtained.

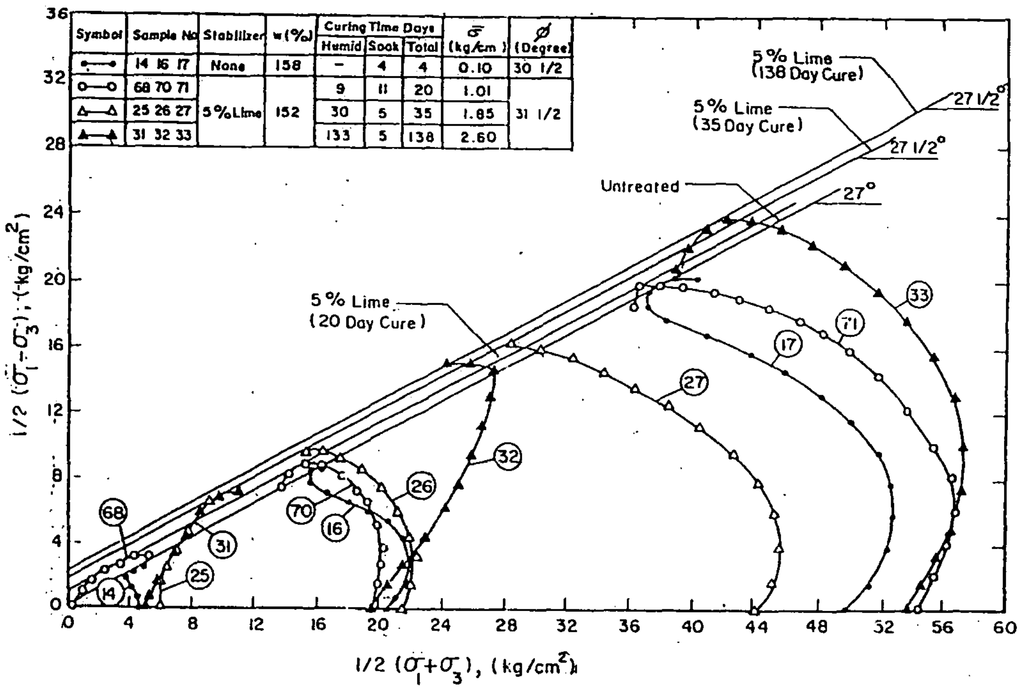


Figure 2.6 Effective Stress Paths and Failure Envelops for Lime Treated Clays from CIU Triaxial Test (Wissa et al., 1965)

In general, the shear strength (as determined by direct shear test) is between 0.5 and 0.3 of the unconfined compressive strength of treated soil (Suzuki, 1982). As shown in Figure 2.7 by Suzuki (1982), in the lower strength range, the shear strength tends to approach 0.5 times the unconfined compressive strength. In the higher strength range the shear strength tends to be closer to 0.3 times the unconfined compressive strength.

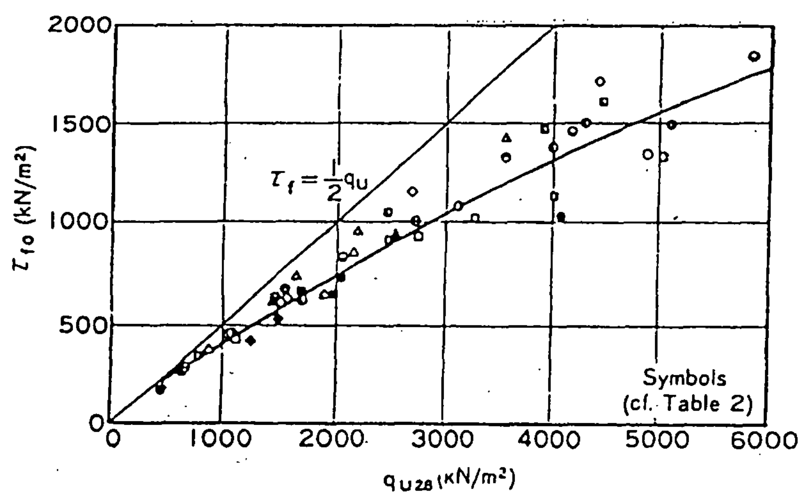


Figure 2.7 Shear Strength Unconfined Compressive Strength Relationship (Suzuki, 1982)

As a simple model Broms (1984) assumed that the stress strain relationship for lime stabilized soil is linear up to creep limit. The slope of the linear portion is then taken as the compression modulus of the treated column material. The creep limit is dependent on the in situ confining pressure and usually determined by in situ load testing. The assumed stress strain relationship is illustrated in Figure 2.8. The bearing capacity of a single excavated lime column with respect to column failure depends on the shear strength of the column material. Figure 2.9 shows the plot of the corresponding failure envelope curve (Broms, 1986). The data is mainly based on results of direct shear tests.

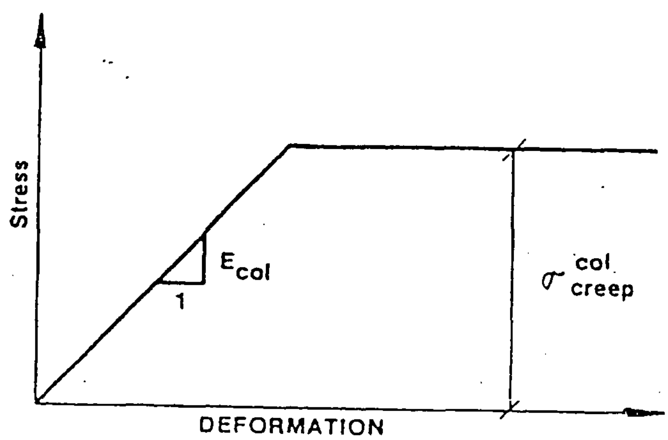


Figure 2.8 Assumed Load Deformation Relationship of Lime Column (After Broms, 1996)

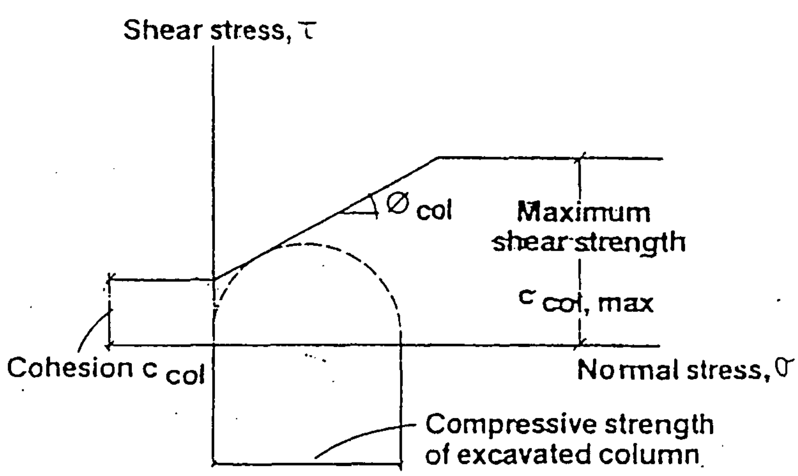


Figure 2.9 Assumed Rupture Diagram for Lime Stabilized Soil (After Broms, 1996)

Figure 2.10 shows the relationship between normal stress, and strength, in the rapid shearing test conducted by Suzuki (1982). In the figure, average unconfined compressive strength and average tensile strength of each improved soil are indicated with Mohr's stress circles. Figure 2.11 shows shear strength versus normal stress relationship form consolidated equal volume shear test. It can be seen from both figures that shear strength increase with increase in normal stress and the change in curvature point of the curve corresponds to the preconsolidation pressure of improved soil. Shear strength increases as the cement content increases.

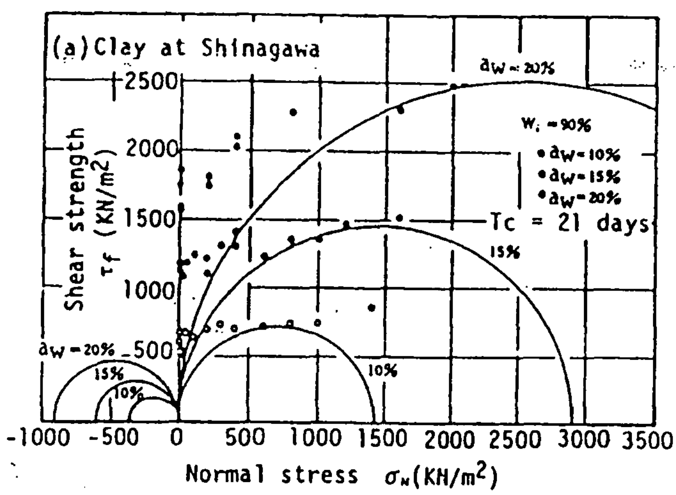


Figure 2.10 Relation between Shear Strength, Unconfined Compressive Strength and Simple Tensile Strength (Suzuki, 1982)

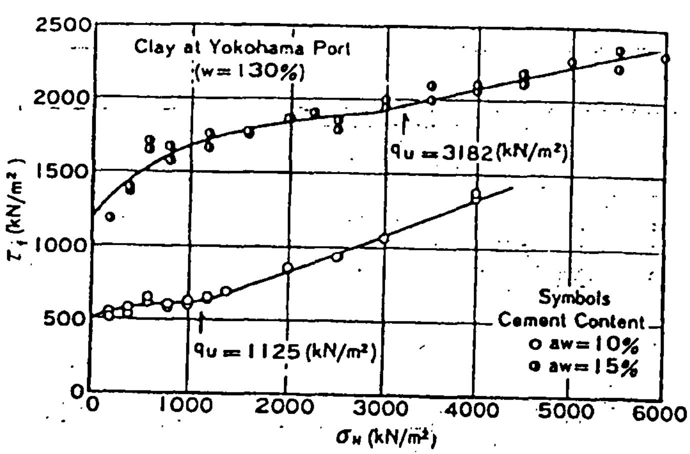


Figure 2.11 Relationship between Shear Strength τ_f and Normal Strength σ_n (Suzuki, 1982)

Figure 2.12 and 2.13 illustrate the Mohr-Coulomb failure envelope curve for cement treated clay triaxial testing conducted by Law (1989) and Uddin (1995). It can be clearly seen that both failure envelope showed nonlinearity and failure strength increase with increase in confining pressure. Broms, (1986) results that were based on direct shear test indicated that the shear strength increased up to certain limit as a function of normal pressure. At higher normal pressure (about 1200kPa) the shear strength of the treated soil became constant. Both C' and ϕ'of the clay were increased by cement treatment but no evident pattern of the increased was observed or reported (Law, 1989)

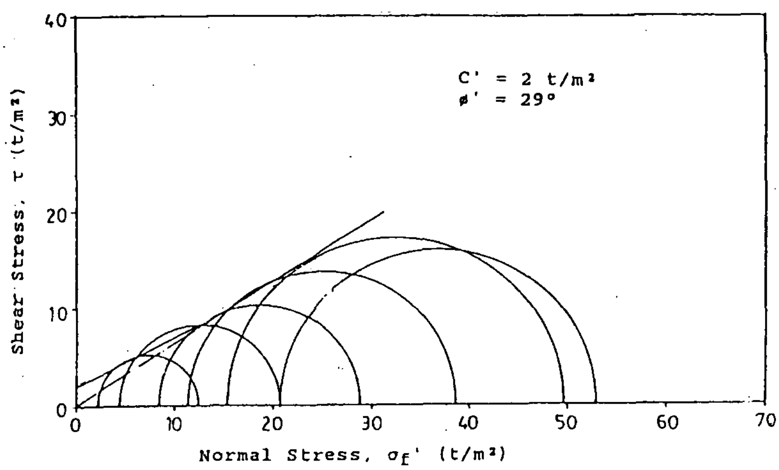


Figure 2.12 Mohr Coulomb Failure Envelope for Cement Treated Clay (Dry mixing, Cement Content 80 kg/m^3) (Law, 1989)

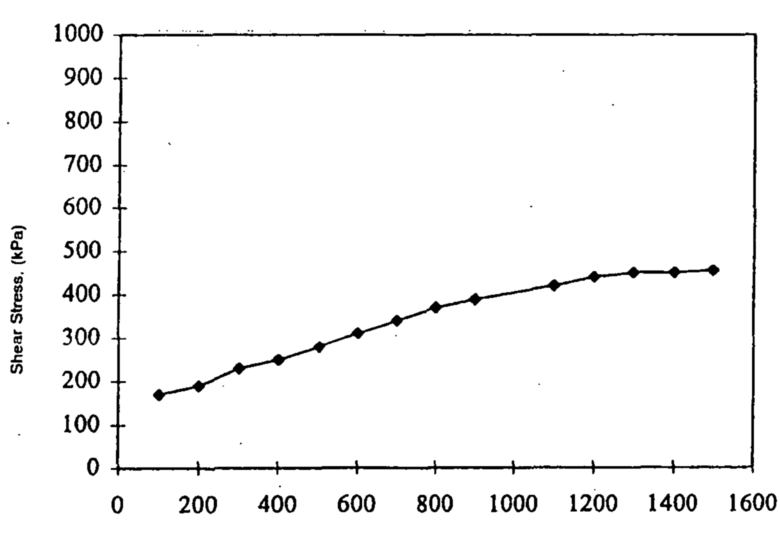


Figure 2.13 Mohr Coulomb Failure Envelope for Cement Treated Clay (Slurry mixing, w/c ratio-0.25, Cement Content-80 kg/m3) (Uddin, 1995)

2.5.5 Deviator Stress-Shear Strain Relationship

The plot of deviator stress-shear strain relationship for cement treated with various cement contents conducted by Law (1989) is illustrated in Figure 2.14. It can be seen that the deviator stress of the cement treated clays is dependent on the pre-shear consolidation pressure. It was found that σ'_c had no significant effects on the stress-shear strain relationship. Also Uddin (1995) found that the plot of q_u versus shear strain, ε_s was significantly affected by the σ'_c when it is at higher range. ε_s was found to be linearly dependent on deviator stress of the value of q up to 60% to 70% of q_{max}. Both ε_s and q_{max} are reduced when the cement content is increased. It was found that low cement contents of 40 to 80kg/m^3 resulted in mild peak and low strain softening beyond that peak.

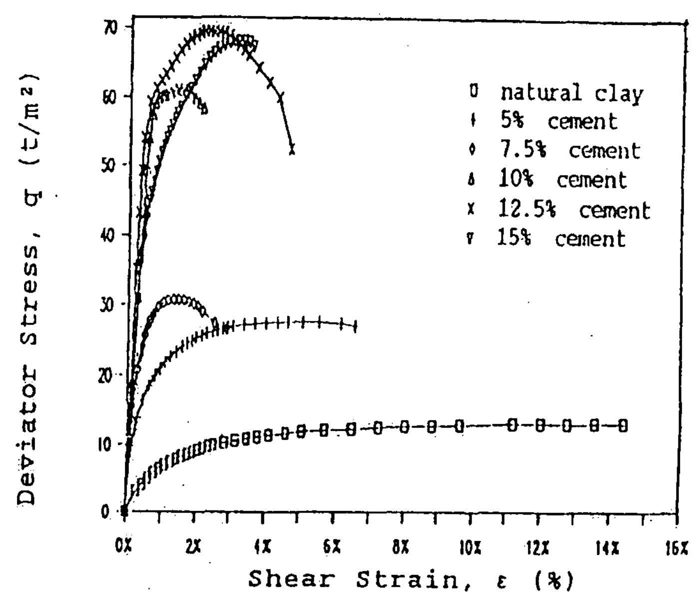


Figure 2.14 Deviator Stress vs. Shear Strain, Dry Mixing (Law, 1989)

2.5.6 Consolidation Behavior

The relationship between void ratio and consolidation pressure for improved soil over a range of different cement content conducted by Suzuki, (1982) is illustrated in Figure 2.15. The preconsolidation pressure was approximately equal to the unconfined compressive strength of the improved soil. Increasing cement content causes the consolidation pressure to increase. This is important since it indicates the limit of loading before rapid and permanent deformation of the Deep Mixing Method (DMM) improved soil will take place (Suzuki, 1982).

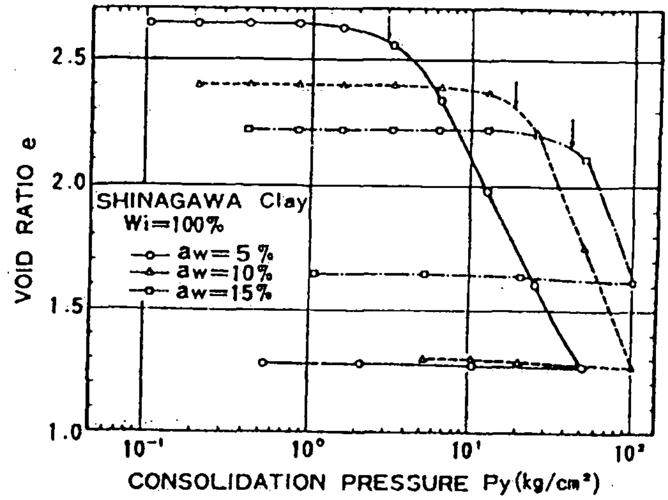


Figure 2.15 Void Ratio-Consolidation Pressure Relationship (Suzuki, 1982)

Figure 2.16 shows the relationship between void ratio and consolidation pressure for a treated clay conducted by Uddin, 1995. It can be see in the plot that the treated curve crosses the untreated curve much before its preconsolidation pressure and then becomes displaced from the untreated curve with increasing values of axial pressure σ_v. This indicates that the treated soil has lower compressibility than the untreated one. Uddin (1995) suggested that for low values of cement content of 40 kg/m^3, an increase of curing period would not help to develop any significant hardening effect in the clay matrix. The swelling ratio (SR) was reduced due to cement treatment and Uddin suggested that this implied that a rigid mechanism occurred during the swelling process. The high pressure test of $\sigma_{v\,(max)}$ equal to 600kPa showed that e-log σ_v plot at the highest range of stress tends to proceed nearly parallel with the normally consolidated line of untreated clay. At sufficiently higher levels of axial pressure, the plot of e-log σ_v of treated clay tends to run parallel to that of untreated clay.

One important effect of cement treatment is to increase the values of coefficient of consolidation C_v. The C_v value generally decreases approximately linearly with increasing consolidation pressure. From the results of Uddin, (1995) and Law (1989) , the highest enhancement of C_v values with cement content occurred in the range of 80 to 120 kg/m^3 of cement content. The compression index C_c value was found to decrease with cement content, thus verifying that cement treatment reduces overall settlement.

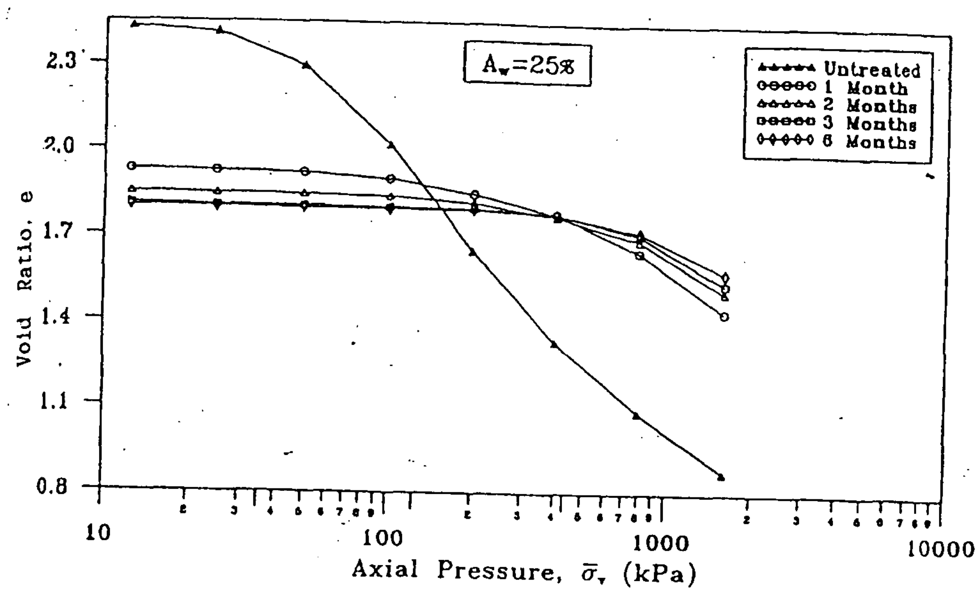


Figure 2.16 Void Ratio-Consolidation Pressure Relationship (Uddin, 1995)

2.5.7 Laboratory Strength Versus Field Strength of Cement Treated Soil

The shear strength of the stabilised soil in the columns may occasionally be less than that of samples stabilized in the laboratory. In most cases however the undrained shear strength of the column will be equal to or larger than that determined by unconfined compression tests on laboratory samples due to material confinement within the columns.

The shear strength of the stabilised soil in the columns is not uniform within the column even when the mixing of the lime/cement with the clay has been conducted very carefully. Small lumps form in the columns during the mixing and these cause the measured shear strength to vary both with the testing method and with the size of the tested sample. The shear strength determined by, for example, falling cone or laboratory vane tests will generally be higher than half the unconfined compression strength of samples cut from the columns. This is due to cracks and fissures in the soil mass (Broms, 1986) and testing in the laboratory at the micro level.

There are also some indications that the increase of the shear strength with time is slower in the field than in the laboratory, possibly due to the low ground temperature at the site and not as efficient mixing of the cement with clay in the field (Broms, 1986). Broms (1984) has reported that the shear strength of lime columns was higher than that of laboratory samples, while for columns stabilized with lime and gypsum the average shear strength was lower than that from the laboratory tests.

Other considerations to take into account when transferring laboratory data to the field are the inherent differences in various strength tests and their limitations, the effects of differences in curing conditions, sample preparation and specimen size. More detailed discussions on these factors have been presented by Broms (1984) and the reader is referred to that reference for more details.

For the design of foundations on the Ariake clay ground improved by Dry Jet Mixing (DJM) method, a tentative code is being used (Research Group on DJM, 1984). The code stipulates that the unconfined compressive strength of improved soil in situ should be greater than 400 kPa after 28 days curing (Buensuceso, 1990). It has been found that the strength ratio of in situ strength to the laboratory strength may be expected to be greater than $\frac{1}{4}$ (Miura et al., 1987). When the laboratory strength exceeds 4 times the design strength for a certain amount of cement, it can be expected that this cement content would give the design strength in situ (Miura, et al., 1997).

Chapter 3

FIELD APPLICATIONS AND DESIGN ASPECTS IN CEMENT TREATED CLAY

3.1 General

This chapter reviews the various processes used for the in situ treatment of native soils. As described in Chapter 2, Portland cement and blended cements are effective stabilising agents applicable to wide range of soils and situations. In terms of this project Cement has two important effects on soil behavior as described below.

It greatly reduces the moisture susceptibility of the native soil, providing enhanced volume and strength stability under variable moisture conditions.

It can cause the development of interparticle bonds in endowing the stabilized material with increased strength and a higher elastic modulus.

3.2 Shallow Stabilisation

Shallow stabilization may be used to describe the techniques that are used for the improvement of subgrade and base course material for road construction, with effective treatment depths to about 30 cm or so of the ground (Buensuceso, 1990). Surface stabilization usually involves the mixing in of borrow soils and binders (such as bitumen, Portland cement, or lime plus pozzolans) and compaction of the mixture at optimum water content. If the compaction cannot be carried out at the optimum water content to achieve maximum strength gain, the procedure is often called modification of the in-place soil (Assarson et al., 1974). Most commonly chemical admixtures such as lime and cement are used to improve the properties of soils by ion exchange and cementation reactions. Lime has mainly been used to stabilize cohesive soils, while cement been applied to cohesionless soils (Broms and Anttikoski, 1983). A large amount of information on the mechanism of lime stabilization and its physical chemical effects on soil are available. These are often based on studies published by the Highway Research Board, USA and other agencies concerned with road research (Diamond and Kinter, 1965 and Eades and Grim, 1966).

3.3 Deep Mixing Method

The Japanese method, which is similar to the lime and cement column method used in Sweden, is the so-called Deep Mixing Method (DMM). In this case unslaked lime or cement is mixed in situ with soft clay (Okumura and Terashi, 1975, Terashi and Tanaka, 1981; Kawasaki et al., 1981 and Saitoh et al., 1985). This method has been used successfully in Japan since about 1967.

In this method, a clayey soil deposit is thoroughly mixed in situ with a hardening agent and the chemical consolidation action between them is utilized to achieve ground stabilization deep below ground surface (Suzuki, 1982). A schematic diagram of stabilization work is shown in Figure 3.1.

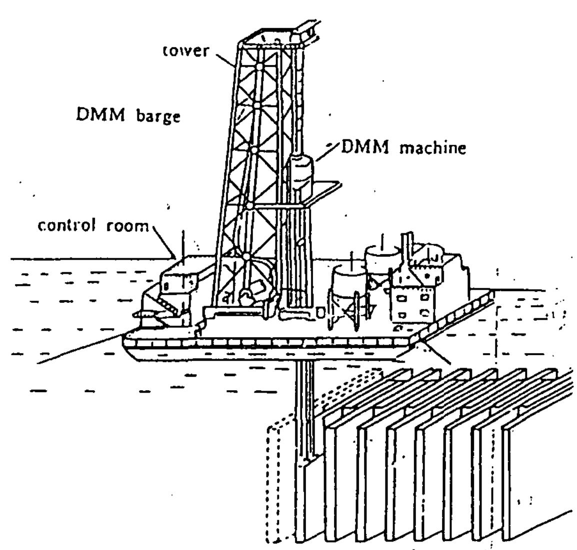


Figure 3.1 Deep Stabilization work (Kawasaki et al., 1981)

DMM has mainly been applied in harbor construction, for ports and quays in order to increase the shear strength and stability of silt and soft clay. Clay with a water content of over 100% has been stabilised successfully with cement and shear strength can often be increased up to about 1MPa depending on the cement content (Broms, 1986). Hydraulically active blast furnace slag and flyash are also often used in combination with the cement (Saitoh et al., 1985). Deep mixing improvement methods may be classified into two categories: (a) mechanical mixing method and (b) slurry jet mixing method. Most of these methods use cement in a slurry state as admixture.

3.4 Dry Jet Mixing Method

The Dry Jet Mixing (DJM) method, which was developed fairly recently, uses cement or quicklime in powder form, instead of in a slurry form (Chida, 1982). The cement or quicklime is transported deep into the ground through a pipe with the aid of compressed air (under pressure of several hundred kPa) and mixed with the clay mechanically by rotating wings. In this method, no water is added to the ground, hence, the improvement effect is expected to be much higher than that of the case using slurry. Especially when quicklime is used, the hydration process in the soil generates some amount of heat resulting in an additional drying effect to the surrounding clay (Yamanouchi et al, 1982), hence the improvement can be done very effectively. For a clay of high organic matter content more than 8% use of cement instead of quicklime becomes advantageous (Miura et al., 1987). Figure 3.2 shows some details of the Deep Mixing Machine.

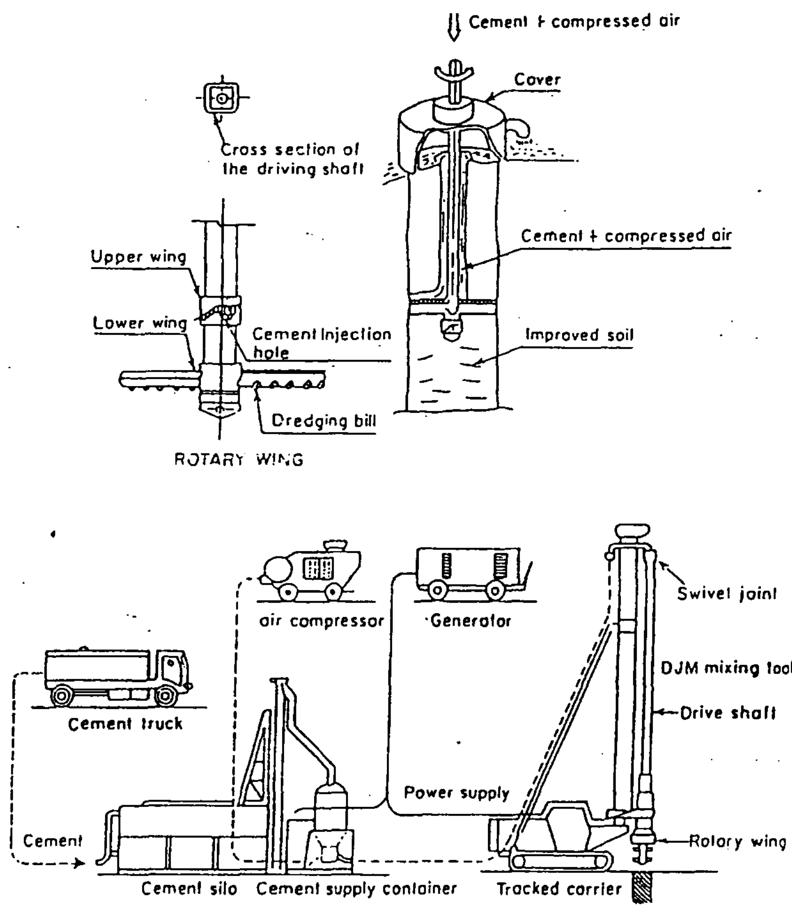


Figure 3.2 Schematic flow of DJM (DJM Research Group, 1984)

3.5 Wet Jet Mixing Method

The "Wet Jet Mixing Method" (WJM Method), involves jetting slurry cement into clay at a nominal pressure of 20kPa from a rotating nozzle (Chida, 1982). This method's advantages are that the machine is relatively light and easy to carry to the project site. The main disadvantage is that the diameter of the improved column tends to vary with depth according to the variation of the subsoil shear strengths and soil structure. The jet grouting system is shown in Figure 3.3.

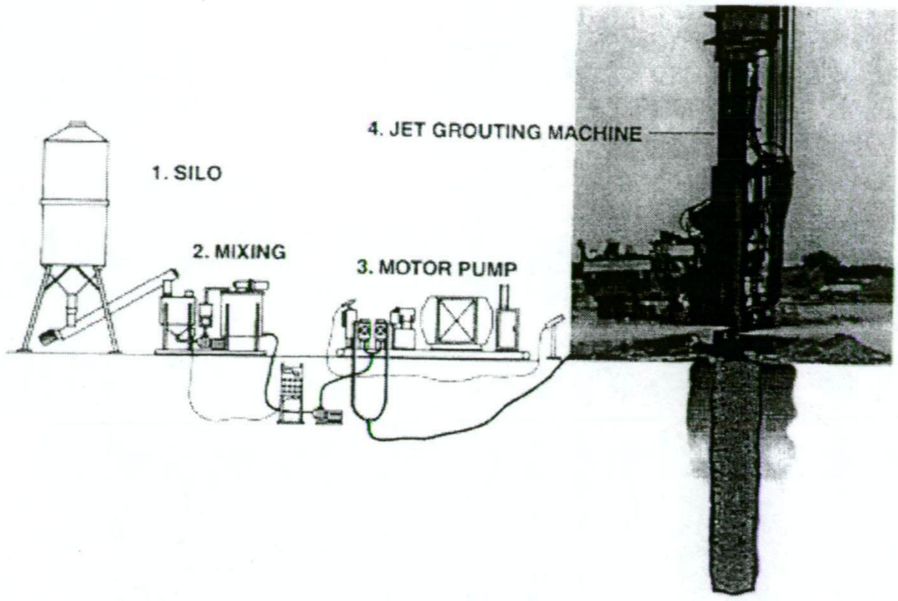


Figure 3.3 Schematic flow of WJM (Kesam and Pinit, 1998)

3.6 Optimum Cement Content

Materials used in the DMM include a hardening agent, mixing water and an admixture. For the hardening agent normal Portland cement is used depending upon the ground to be stabilised, and the dosages of the cement are determined in the following procedure.

First, the soil to be stabilised is sampled prior the taking up the work and the laboratory mixing proportion test is carried out on the soil samples. The relation between the dosage of the hardening agent and unconfined compressive strength of improved soil is obtained, and the dosage of hardening agent necessary to obtain the design strength of the improved soil is predicted. Next, investigation is carried out on the ground to be treated by selecting two or three dosages of the hardening agent on the basis of the results of the laboratory mixing proportion test and under the same conditions as employed in the actual stabilisation work. From the unconfined compression test results of treated soil samples collected after completion of the stabilised work, the required dosage of the hardening agents are finally determined. From the records of testing works conducted, cement contents of between 120 to 210kg have been added to $1m^3$ of soil. (Suzuki, 1982).

3.7 Predicting Unconfined Compressive Strength

The unconfined compressive strength of treated soil prepared in the laboratory (q_u), assuming adequate mixing and constant curing conditions, is a function of soil type, stabilizing agent, stabilizing agent-total water ratio, organic matter content, fine granular content and curing time (JGS, 1996). The unconfined compressive strength of in situ treated soil is a function of laboratory strength, curing time, curing temperature, and mixing efficiency. Mitchell (1974) presented a comprehensive review of the strength properties of cement stabilisation and developed equation 3.1. The unconfined compressive strength q_u is generally described as increasing linearly with the cement content percentage A_w. This increase is more pronounced for coarse-grained soil than for silt and clay. Like q_u other strength parameters such as cohesion intercept (C) and friction angle (ϕ) increase with A_w and curing time. Mitchell (1974) gave the following relationships between curing time and q

$$q_u(t) = q_u(t_0) + K.\log \frac{t}{t_0} \qquad \text{------- } 3.1$$

where

$q_u(t)$ = Unconfined compressive strength at t days, kPa

$q_u(t_0)$ = Unconfined compressive strength at t_0 days, kPa

K = 480 A_w for granular soils and 70 a for fine grain soil

A_w = Cement content % by mass

t, t_o = Curing time

3.8 Cement column installation methods

3.8.1. General

Before cement columns are installed, the designer must carry out a detailed site investigation including sampling of the soil in the area where cement columns will be installed. Typically natural Water Content, Liquid Limit, Plastic Limit, Shear Strength, Unit Weight and Particle Size Distribution testing is carried in the laboratory to classify the type(s) of the soil to be treated. This process will be repreated for each significant soil type in the soil profile to be treated. After classification, soil from the area is mixed with various proportions of cement by using a w/c ratio of 1.1/1. This w/c ratio is also used in the field installation because of the ease of injecting slurry. After thorough mixing the soil-cement is cast in containers with 35 mm. diameter and 70 mm. height. Curing is carried out up to 7 days, 14 days, and usually 28 days as an upper limit and unconfined compression testing is undertaken to determine the mix that will provide the same strength as the preliminary design.

With Jet Mixing, different pressures and rising speeds are evaluated to find out which pressures and rising speeds are able to create the same strength as the preliminary design. Similar to Rotary Mixing, a variety of rotary speeds, rising speeds and feeding rates are evaluated to determine the field installation conditions to produce cement columns that have the same characteristics as the preliminary design.

Checking of in situ strength is carried out by coring the whole cement columns as shown in Figure 3.4 and subsequently determining the unconfined compressive strength of the cores. The cement columns that produce the same strength as the preliminary design are identified and the pressure and the rising speed (for Jet Mixing) or rotary speed, rising speed and feeding rate (for Rotary Mixed) of producing those cement columns are then used for the whole project. Finally pull out tests as shown in Figure 3.5 may be undertaken to check the completeness of the design of the cement columns.

In addition to coring for testing the unconfined compressive strength, a pile load test is also carried out for one in every 100 cement columns.

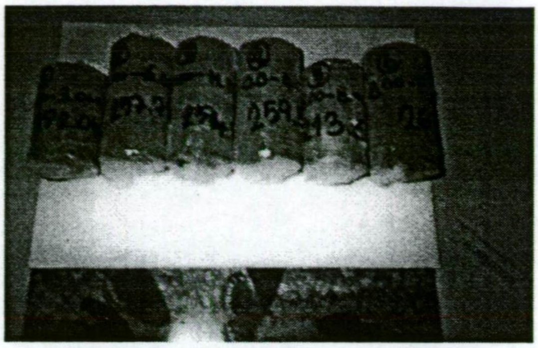


Figure 3.4 Coring for testing Unconfined Compressive Strength (Kasem and Pinit, 1998)

Figure 3.5 Pull out test of cement column (Kasem and Pinit, 1998)

3.8.2 Installation

Cement columns are usually installed by using deep mixing methods. Generally, there are two categories of installing cement columns ; high-pressured grout mixing or jet mixing and mechanical mixing or rotary mixing. Both categories have quite similar processes and use similar batching equipment. The batch plant typically consists of colloidal mixers, volumetric screw feeders and flow controllers. Dry materials (cement) are stored in silos and fed by screw feeders to the colloidal mixers for agitation and circulation. The resulting (grout slurry) is transferred by hose to the deep soil-mixing rig.

A high-pressured pump required for Jet Mixing may be added for delivering the grout slurry into the soil by injection.

Slurry (grout) proportions are monitored and controlled closely to ensure proper proportioning and density measurements made to verify mix proportions. Cement proportions are mixed by weight to obtain a predetermined final mix density. The method of injecting the grout slurry into the soil depends on the design mix. Positive displacement pumps are used to transfer the grout slurry from the mix plant to the rig. The rate of application may be controlled and monitored by various methods. The first method uses calibrated displacement grout pumps to produce a time against flow correlation and the pump output is adjusted to obtain that required for the penetration rate. The second method involves using transfer pumps will deliver the grout slurry to the rig where a return line can be adjusted to bleed off any overflow back to the surge tank. The surge tank is usually a cylindrical vessel with a constant depth-volume relationship that can be monitored, while the return line is adjusted to ensure a constant injection rate. The third method employs a programmable electronic controller, flow metering and pressure regulating robot to ensure consistent mix quality.

3.8.3. High-pressured grout mixing

This method can be used with almost any soil type as shown in Figure 3.6, including silts and some clays.

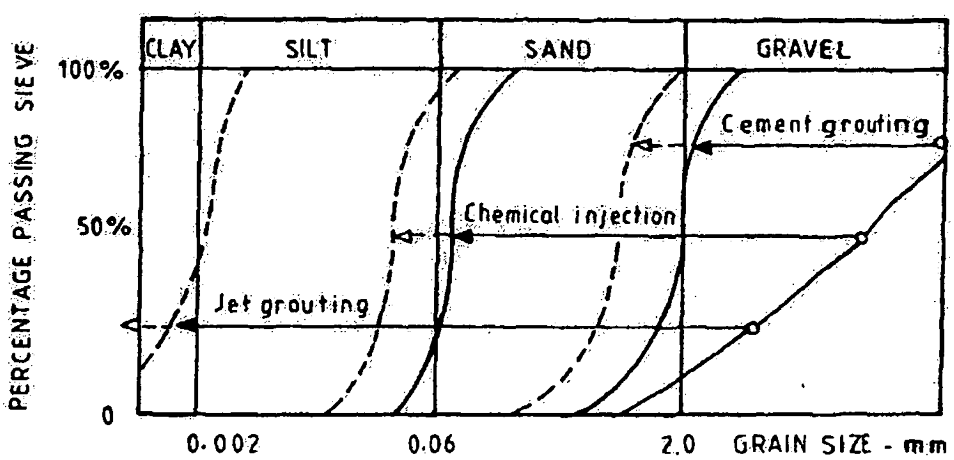


Figure 3.6 Jet Mixing is effective across the widest range of soil types.(Bell, 1993)

High-pressured grout mixing or jet mixing systems injects the grout slurry into soils at high flow rates. As a result, the soil is "de-structured" and then slurry is mixed together with the in situ soil to create soil-cement columns. The resultant columns have much better mechanical and physical characteristics than the original native soils. A typical schematic flow of production is shown in Figure 3.7 and the working procedure of jet mixing is shown in Figure 3.8.

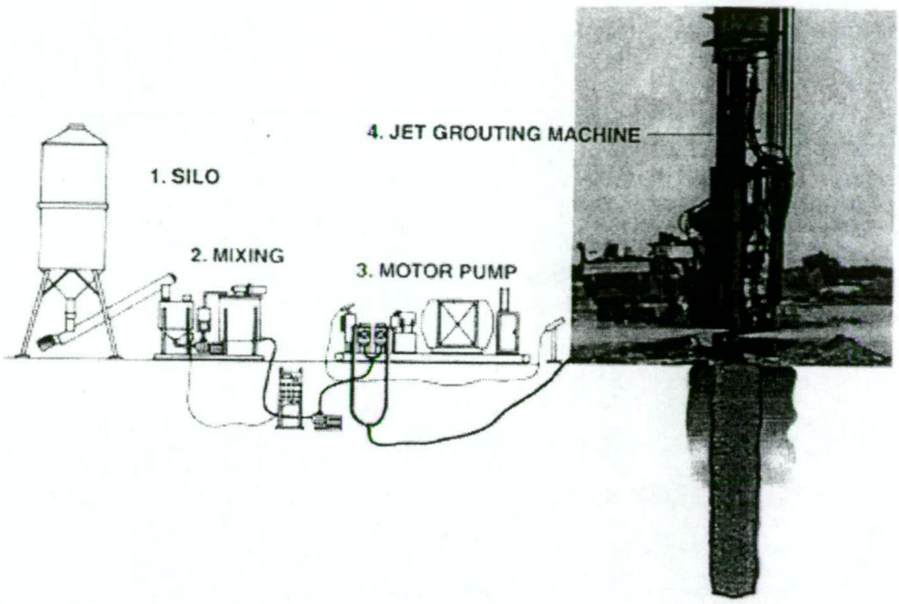


Figure 3.7 A typical schematic flow of Jet Mixing.(Kesam and Pinit, 1998)

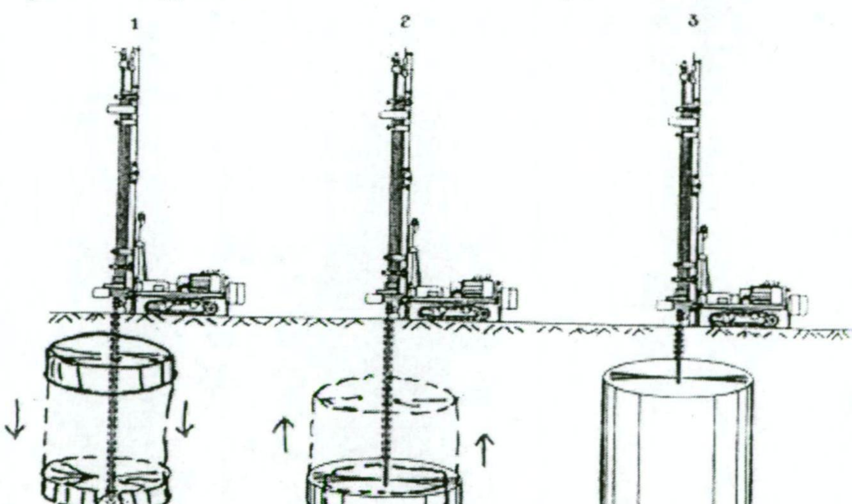


Figure 3.8 Working procedure of Jet Mixing.(Kesam and Pinit,1998)

Figure 3.8 (1) shows pre-jetting with water using a pressure of about 150bars or 15000kPa. Pre-jetting is carried out from pile top until pile tip of design. (2) shows jetting under very high pressure (about 200bars or 20000kPa) of a cement grout by simultaneous rotating and rising of the rod part. (3) shows jetting under high pressure until the top of preliminary design of the treatment.

High-pressured grout mixing or jet mixing systems

There are three traditional jet grouting systems as shown in Figure 3.9 and Figure 3.10 The selection of the most appropriate system is generally a function of the in situ soil, the application, and the physical characteristics of cement columns required for that application (Bell, 1993). However, any system can be used for almost any application providing that the right design and operating procedures are used.

Single Rod Jet Grouting

This is the simplest form of jet grouting involving a hollow stem grout pipe fitted with one or several nozzles of 2.0-4.0mm. The tip is positioned at the treatment depths (Figure 3.9) and grout is pumped through the rod and exits the horizontal nozzles in the monitor with a high velocity, approximately 200m/sec. This energy causes the erosion of the surrounding ground and mixes the grout with in the in situ soil. Single rod jet grouting is generally less effective than multiple rods jet grouting due to mixing efficiency.

Double Rod Jet Grouting

This a more advanced form of the single system in which the erosive effect of the grouting jet is greatly increased. A two-phase internal rod system is employed for the separate supply of grout and air down to different, concentric nozzles. The inner rod carries grout and the outer rod carries air (Figure 3.9). The grout is used for eroding and mixing with the soil while the air shrouds the grout jet and increases erosion and mixing efficiency. The double rod system is more effective in cohesive soils than the single rod system.

Triple Rod Jet Grouting

The triple system uses air-shrouded water jetting for soil erosion and separate injection nozzles for placement of the grout. Grout, air and water are pumped separately through concentric triple pipe (Figure 3.9), maintains the advantages of a highly efficient erosive jet but enables achievement of a wider range of geotechnical performance from treated soil. Pressure and flow rate of grout water and air may all be varied independently to give the desired geometry and in situ mix of grout jetting water and eroded soil to form a cement column left in the ground.

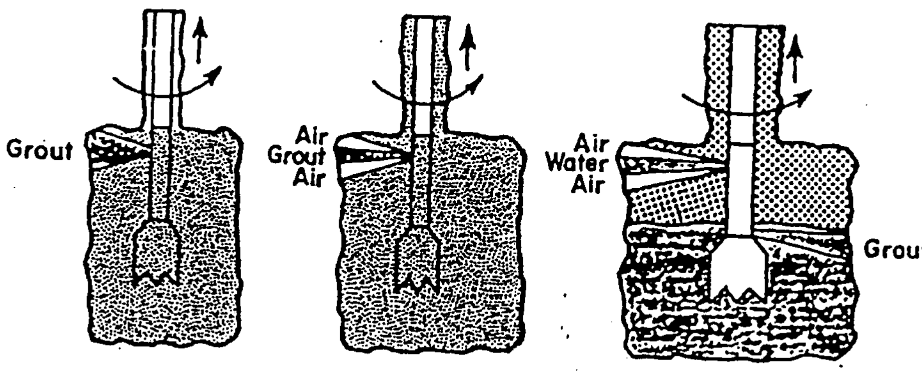
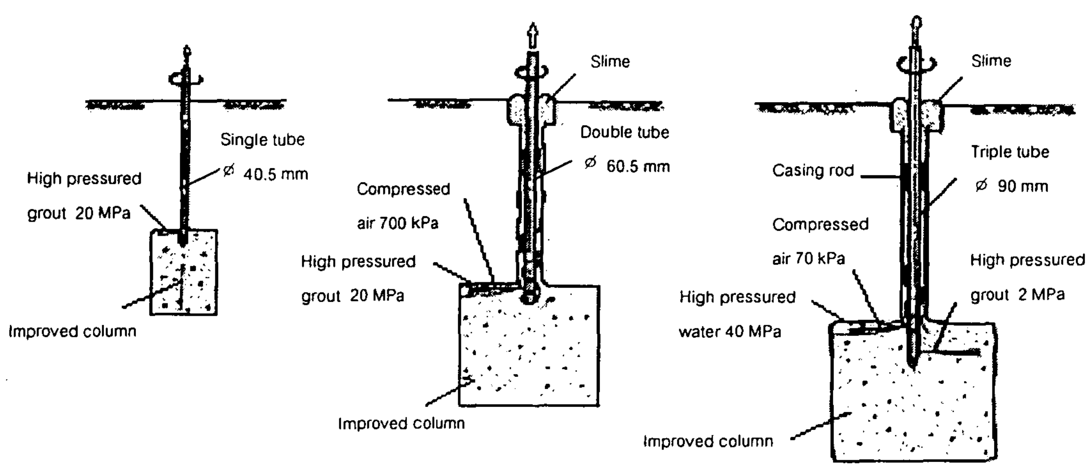


Figure 3.9 Three traditional types of Jet Grouting.(Bell, 1993)



(a) Mixed by pressured grout

(b) Mixed by pressured grout with compressed air

(c) Mixed by pressured grout with compressed air and pressured water

Figure 3.10 The detail of the three traditional Jet Grouting.(Masashi,1996)

3.8.4. Rotary Mixed

This method is similar to the Jet Mixing Method in both process and production, there are however, some differences in the two procedures.

Figure 3.11 shows a typical schematic flow of process for rotary mixing. Compressed air is fed into the tank which containing the binder. Consequently, the binder is moved downward easily, in a process called fluidization. The air them leaves the tank from a pipe at the top. This external pipe goes down to the tank bottom, where the binder is fed into the air stream by a cell feeder. The air and the binder are transported through the hollow hose down to a mixing tool. The air and the binder are blown horizontally out into the surrounding soil and mix with the soil. The compressed air from the mixing tool penetrates in cracks and voids in the soil and helps with the overall mixing process. The binder is mixed with the soil by the lifting and rotation movements of the mixing tool.

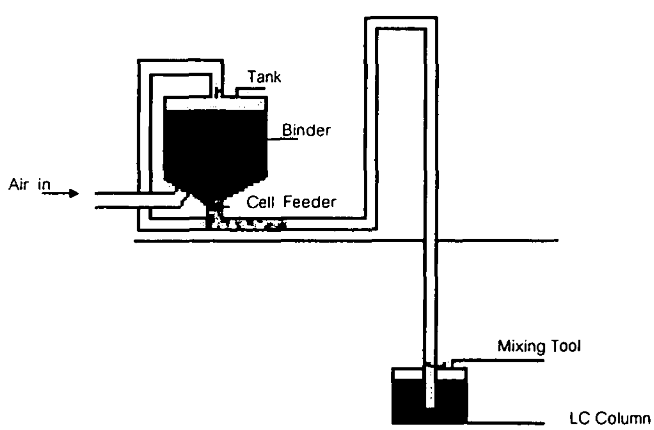


Figure 3.11 Schematic of Rotary Mixed.(Bredenderg,1999)

The mechanical mixing method or rotary mixed method can use both a slurry state or a dry powder state stabiliser, as shown in Figure 3.12.

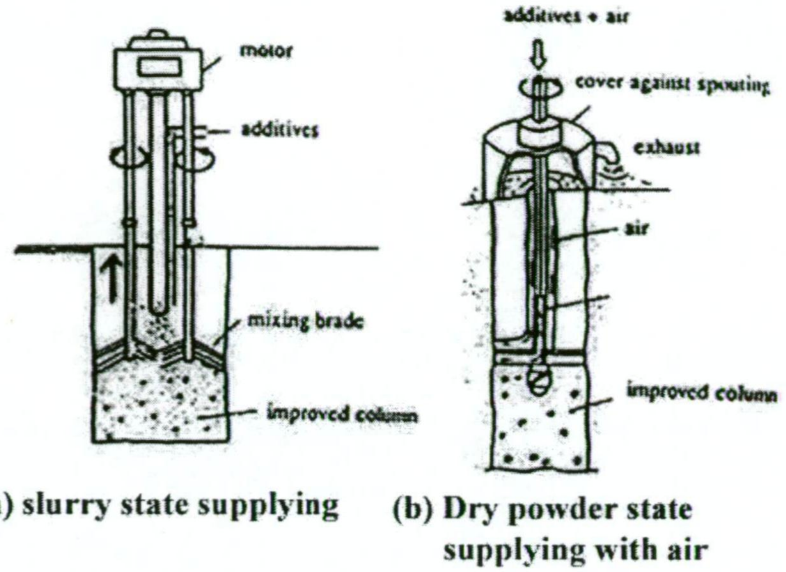


Figure 3.12 Details of mechanically mixing methods (Kamon, 1991)

Figure 3.13 shows the working procedure of the Rotary Mixed method. The rig goes to the position and the process is started by penetrating and rotating the mixing blade simultaneously into the ground until the predetermine depth. After the rig completes the penetration to the required depth, it is withdrawn while rotating the mixing blade and injecting the slurry simultaneously.

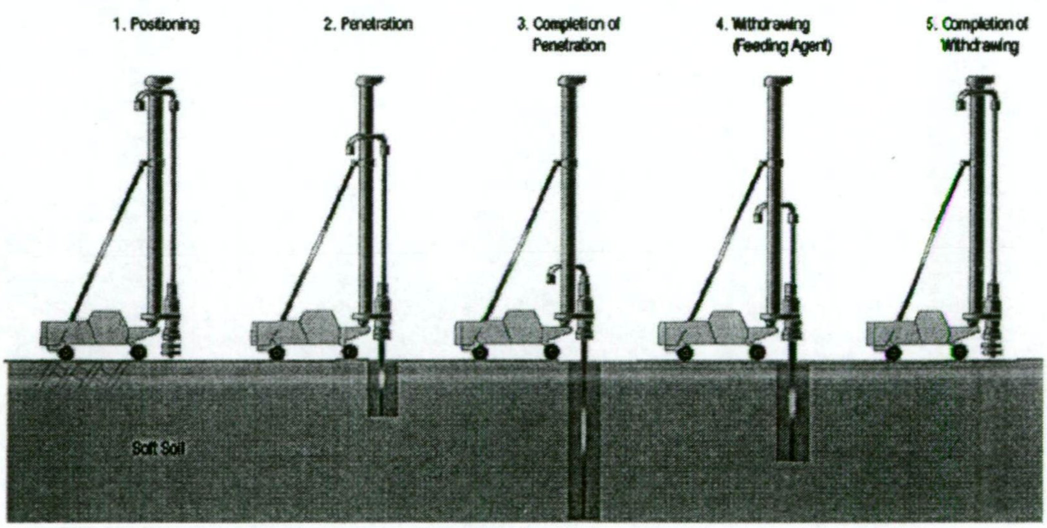


Figure 3.13 Working procedure (DJM Research Group, 1984)

3.9 Cement Columns Design Methods

The failure mechanisms associated with cement column are essentially the same as those for concrete piles. They can be termed surrounding soil failure and column failure.

3.9.1 Surrounding soil capacity load (in case of soil failure)

Whitaker and Cooke (1961) expressed the ultimate column capacity, based on soil failure in equation 3.2 to 3.4. The design uses a Holmberg Adhesion Factor (α) and a bearing capacity factor.

$$Q_{ult} = Q_s + Q_b \qquad \text{———} \; 3.2$$

$$Q_b = N_c\, C_b\, A_b \qquad \text{———} \; 3.3$$

$$Q_s = \alpha\, C_v\, A_s \qquad \text{———} \; 3.4$$

For Q_{ult} = Ultimate load (ton)

N_c = bearing capacity factor = 9 (Skempton, 1951)

C_b = Undrained Shear Strength at the column tip (t/m^2)

C_v = Average Undrained Shear Strength (t/m^2)

A_b = cross section area of column (m^2)

A_s = surface area

α = Holmberg Adhesion Factor (1970) (1 for soft Bangkok Clay)

In the short term case N_c = 9 (Skempton, 1951) and at this stage the Adhesion Factor may be assumed as 1 for Bangkok clay. This value is to be verified during the experimental stage of the thesis.

3.9.2 Pile load capacity (in case of pile failure)

The ultimate failure load (Q^{Col}_{ult}) of the cement column is shown in equation 3.5

$$Q^{Col}_{ult} = (2C^{Col}_u + 3\sigma_h)\, A_b \qquad \text{———} \; 3.5$$

When C_u^{Col} = Undrained Shear Strength of the cement column = $q_u/2$

q_u = Unconfined Compression Strength

σ_h = horizontal pressure

A_b = cross section area of the cement column

When cement columns carry load in the long term, the long-term ultimate strength of a single column (C^{Col}_{creep}) will less than short-term ultimate strength (C^{Col}_{ult}). It is typically about 65% to 80% of short-term ultimate strength (C^{Col}_{ult}) (Broms and Sweroad, 1992).

So $$C^{Col}_{creep} = (0.65 \text{ to } 0.80)\ C^{Col}_{ult} \qquad \text{———— 3.6}$$

If we allow for Creep in a group of piles, the load of cement columns (Q^{creep}) is shown in equation 3.7

$$Q^{creep} = (0.65 \text{ to } 0.80)\ a\ C^{Col}_{ult} \qquad \text{———— 3.7}$$

When $a = A/S^2$

A = cross section area of cement columns

S = Spacing of cement columns in square grid

When designing cement columns, the designers must also consider the axial stress in the column. Column stress (σ_{col}) must less than creep limit of the columns. According to Swedish National Road Administration (Sweroad), the column stress (σ_{col}) is as follows in equation 3.8

$$(\sigma_{col}) = Q_{col}/A_{col} = q/\ [a+(1-a)E_{soil}/E_{col}] \qquad \text{———— 3.8}$$

Where q = Average unit load = $\dfrac{W}{BxL}$

a = Relative column area = $(N^*A_{col})/(BxL)$

N = Number of cement columns in area BxL

E_{soil} = Modulus of Elasticity of soil

E_{col} = Modulus of Elasticity of cement columns

3.9.3 Slope Stability (local shear failure.)

When the carrying load of cement column groups in embankments or excavations is considered then local shear failure or stability of slope issues arise. According to Broms (1993) and Sweroad (1992), the undrained shear strength of an in situ soil and undrained shear strength of cement columns are averaged together to become a new undrained shear strength of a new soil (C_{ave}) by using the empirical formula as follow.

$$C_{ave} = C_u^{Soil}(1\text{-}a) + C_u^{Col} * a \qquad \text{——— 3.9}$$

Where C_{ave} = the new Shear Strength

C_u^{Soil} = Undrained Shear Strength of an in situ soil

C_u^{Col} = Undrained Shear Strength of cement columns

$a = A/S^2$

A = cross section of cement columns

S = Spacing of cement columns in square grid.

Figure 3.14 and 3.15 show the procedure for the transforming of soil and cement columns of embankment before calculating slope stability factor by the Swedish method, the Bishop method or the Spencer method.

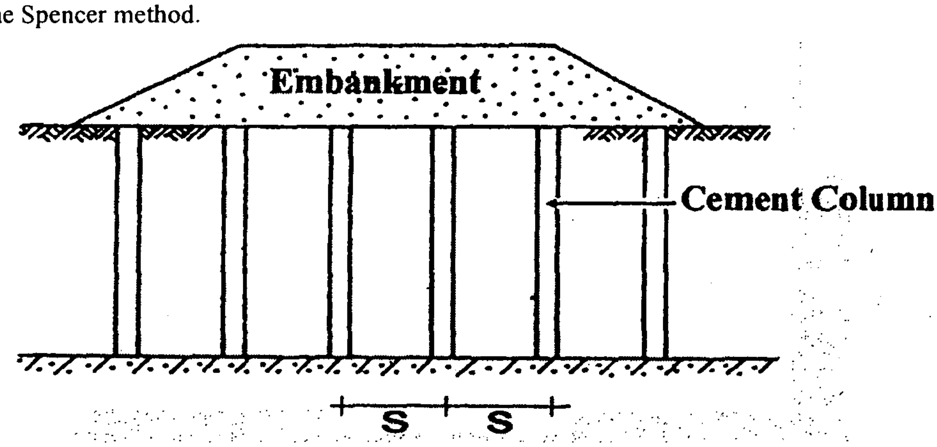


Figure 3.14 cement columns under an embankment (Sweroad, 1992)

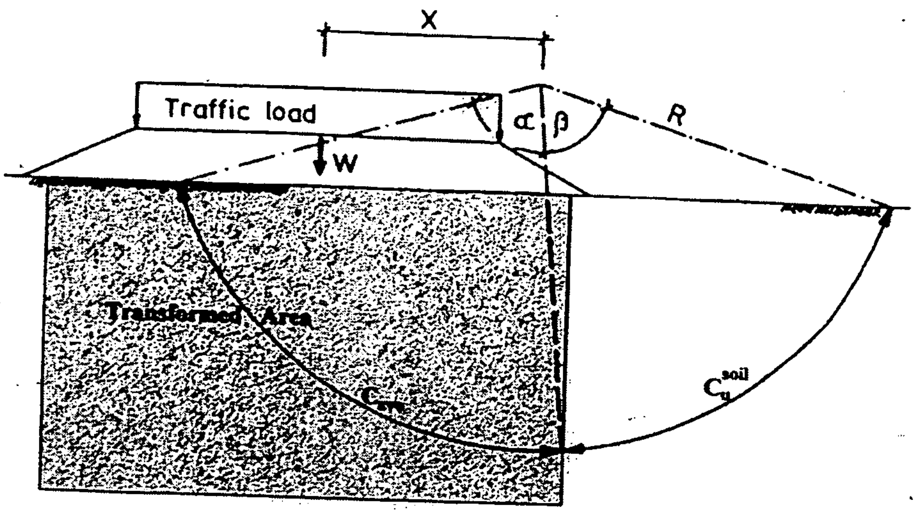


Figure 3.15 Transformed Area for calculating factor of slope stability (Sweroad, 1992)

From Figure 3.14, the spacing between cement columns can be found from equation 3.10

$$[a^*C_u^{Col}+(1-a)C_u^{Soil}]^*R^2{}^*\alpha+R^2{}^*\beta^*C_u^{Soil}=W^*X^*FS \qquad 3.10$$

When R = Radius of the circle

α, β = The angle of the center of rotation (Figure 4.2), (radial)

W = total weight

X = Lever arm of the load

FS = safety factor

C_u^{Soil} = Undrained Shear Strength of soil

C_u^{Col} = Undrained Shear Strength of cement columns

a = A/C^2

A = cross section area

C = spacing of cement columns

In case of where there are many soil layers (Figure 4.3), the average of undrained shear strength (C_{ave}) will be found from the formula below in equation 3.11

$$C_{ave}=\frac{\sum_A^C[a^*C_u^{Col}+(1-a)C_u^{Soil}]\beta_n+C_u^{Soil}{}^*\beta_L}{\sum\beta_n} \qquad 3.11$$

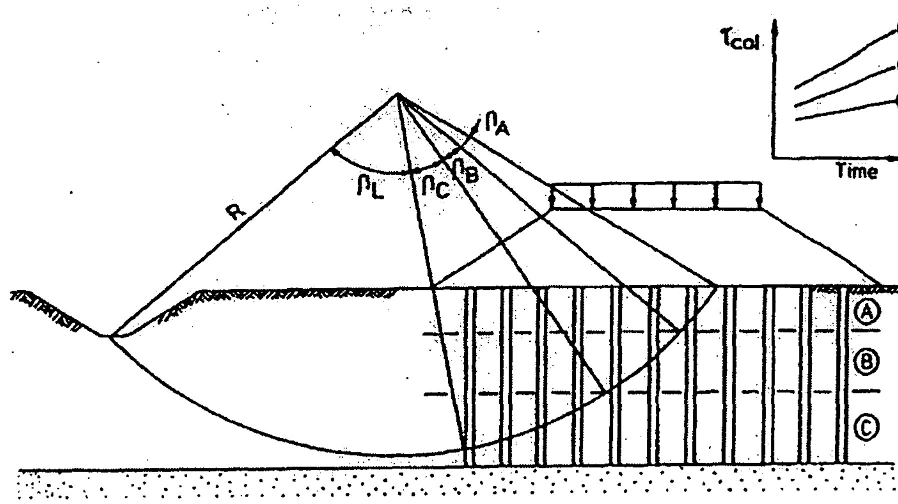


Figure 3.16 Calculation Model of many soil layers (Sweroad, 1992)

3.10 Ultimate load capacity of a single pile by static load test

Once the load-settlement curve has been derived, it is necessary to determine the ultimate load capacity, which means it must be defined, where "failure" occurs. There are many different methods that have been proposed to interpret the ultimate load capacity. This is an issue that must be addressed for this thesis.

Tomlinson (1975) reported that when a pile is loaded continuously in a vertical line, the relationship between load and settlement that will occur is shown in Figure 3.17.

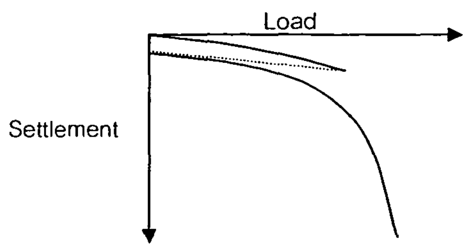


Figure 3.17 Load-settlement curve for compressive load to failure on pile (Tomlinson, 1975)

Normally, the limiting settlement for maximum skin friction is about 0.3-1.0% of a diameter of a pile and the settlement that represents pile failure is about 10-20% of a diameter at pile tip (Tomlinson, 1975).

Sidney and Thomas (1968) reported that the total settlement including elastic deformation, which corresponds to an ultimate load is 0.254mm/ton for their study.

According to De beer (1967) and Fellinius (1975), the ultimate load can be determined from the intersection point in the logarithmic scale graph between load and settlement. This is illustrated in Figure 3.18.

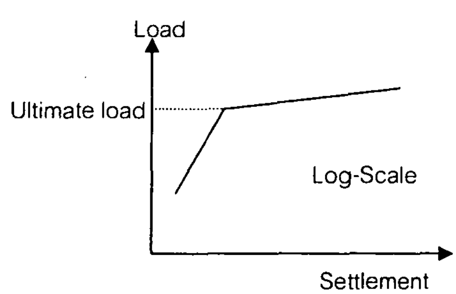


Figure 3.18 Load-settlement curve in log-scale for determining the ultimate load (Fellinius, 1975)

AASHTO (1989) reported that the vertical load, which causes a pile to settle more than 6.44mm (0.25inch), is the ultimate load.

Davidson (1973) defined the ultimate load capacity as that which occurs at a settlement of 4mm + B/120 + PD/(AE) as shown in Figure 3.19

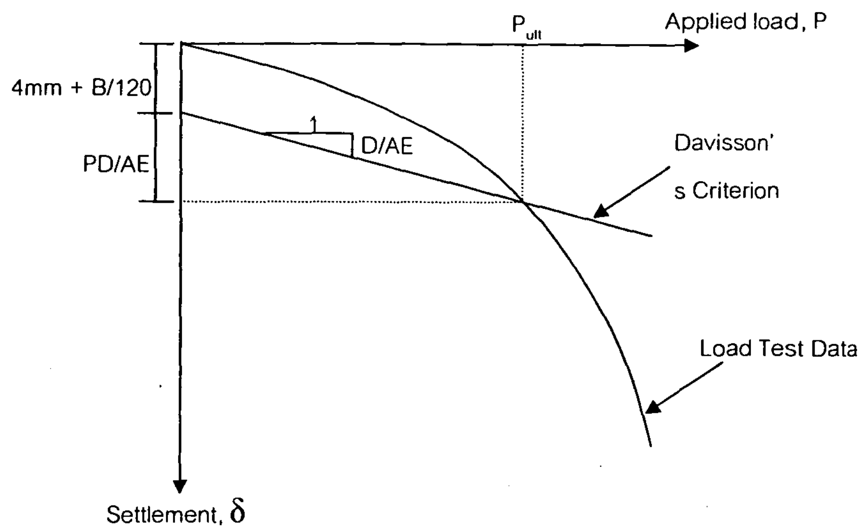


Figure 3.19 Davidson's method of interpreting pile static load test data

Where B = Foundation Diameter

P = Applied Load

δ = Settlement

D = Foundation Depth

A = Foundation Cross-Section Area

E = Foundation Modulus of Elasticity

The methods, which have been described above, will be used later in the thesis to determine the nominal ultimate load capacity of cement columns installed. The Tomlinson method, De beer and Fellinius method and Davisson method will be used to estimate the pile capacity while Sidney and Thomas method and AASHTO method will be not applied. These methods are not suitable to determine the pile capacity of cement columns because both are reported to give results well below the actual ultimate load capacity of cement columns (Kasem and Pinit, 1998).

Chapter 4

FIELD AND LABORATORY INVESTIGATIONS

4.1 Introduction

This chapter contains relevant details of the various experimental investigations carried out in the study. The various phases of the linked series of investigation may be summarised as show below. The iterative process is also shown schematically.

-Site Investigation and sampling

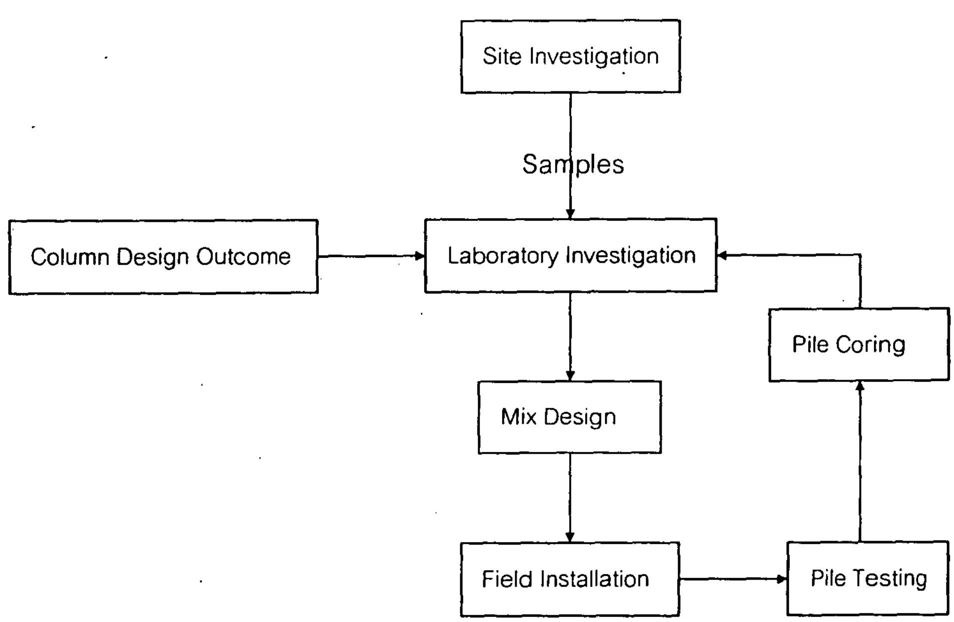
-Laboratory trials and mix design

-Field Installation of columns

-Pile Testing

-Pile coring and testing

-Review of the process



4.1.1 Geotechnical characteristics of soft Bangkok clay

The soft Bangkok clay in the lower Chao Phraya Plain extends for 200-250km in the East-West direction and 250-300km in the North-South direction. The thickness of the soft to medium stiff clay in the upper layer varies from 12 to 20m while that of the total clay layer including the lower stiff clay is about 15 to 30m as can be seen in Figure 4.1 and 4.2. Thicker deposits are found close to the Gulf of Thailand and the thickness decreases towards the north (Balasubramaniam, 1991).

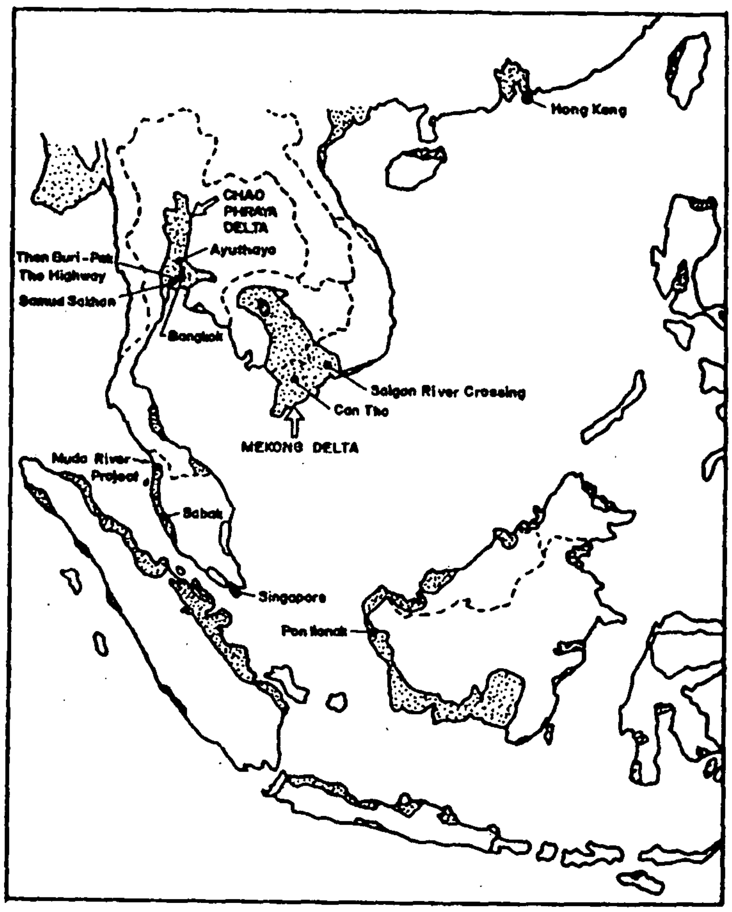


Figure 4.1 Distribution of recent clays in Southeast Asia (Balasubramaniam, 1991)

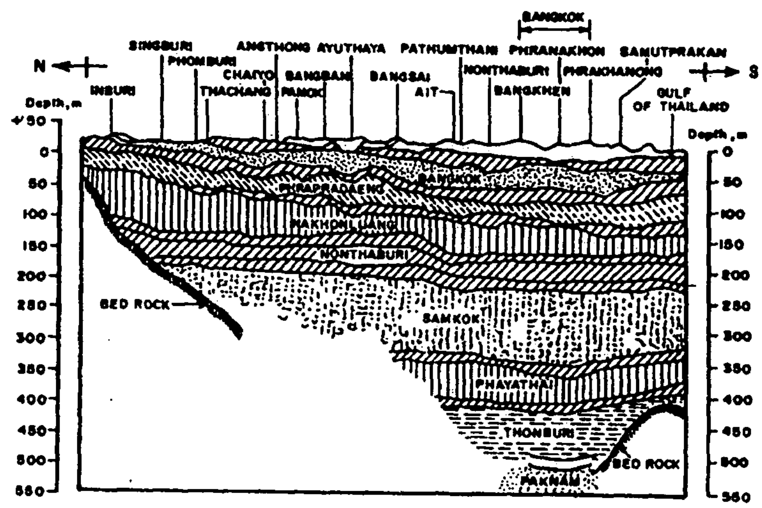


Figure 4.2 System of aquifers under the Chao Phraya plain (Balasubramaniam, 1991)

4.1.2 Physical and chemical properties of soft Bangkok clay

The untreated properties of soft Bangkok clay are shown in Tables 4.1 and 4.2 in terms of physical properties and chemical properties respectively which are derived from Kasem and Pinit. This base is very important for comparison of physical and chemical properties in different sites throughout Bangkok and the surrounding regions.

Table 4.1 Characteristics values of the physical properties of soft Bangkok clay (Kasem and Pinit, 1998)

Properties	Characteristics values
Liquid limit, LL, (%)	103
Plastic limit, PL, (%)	43
Plasticity index, PI, (%)	60
Water content, w, (%)	76-84
Liquidity index, LI	0.62
Grain size distribution	
Clay (%)	69
Silt (%)	28
Sand (%)	3

Total unit weight, γ_t (kN/m^3)	14.3
Dry unit weight, γ_d (kN/m^3)	7.73
Initial void Ratio, e	Dark gray
Activity	0,87
Sensitivity	7.4

Table 4.2 Characteristics values of the chemical properties of base clay (Kasem and Pinit, 1998)

Properties	Characteristics values
Soil pH (Soil: Water Ratio = 1:1)	6.1
Cation exchange capacity, (meq/100g oven dry soil)	28.2
Exchangeable cations	
Na^+, (meq/100g)	3.26
K^+, (meq/100g)	1.99
Ca^+, (meq/100g)	6.78
Mg^+,(meq/100g)	6.2
Total soluble salt content (meq/l)	8.7
Organic carbon, (%)	2.87
Organic matter, (%)	5.6
Cation in pore water	
Na^+, (meq/l)	3.22
K^+, (meq/l)	0.34
Ca^+, (meq/l)	6.98
Mg^+, (meq/l)	10.05
Electrical conductivity, (mohm/cm)	2.29

4.2 Field Site Location and Investigation.

The test site selected for the field trials is located along the Thon Buri – Pak Tho Highway, Bangkok, Thailand. This is the main road that connects Bangkok to the southern part of Thailand (Figure 4.3). Figure 4.4 shows the plan of the site. At the site, one borehole 30m deep was carried out to sample the soil in the site. Unit weight, water content (ASTM D 2216), soil classification (ASTM D 2487), specific gravity (ASTM D 854), atterberg limit (ASTM D 4318) and grain size distribution (ASTM D 422) were undertaken for each soil layer to evaluate the physical properties of the soft Bangkok clay at the site.

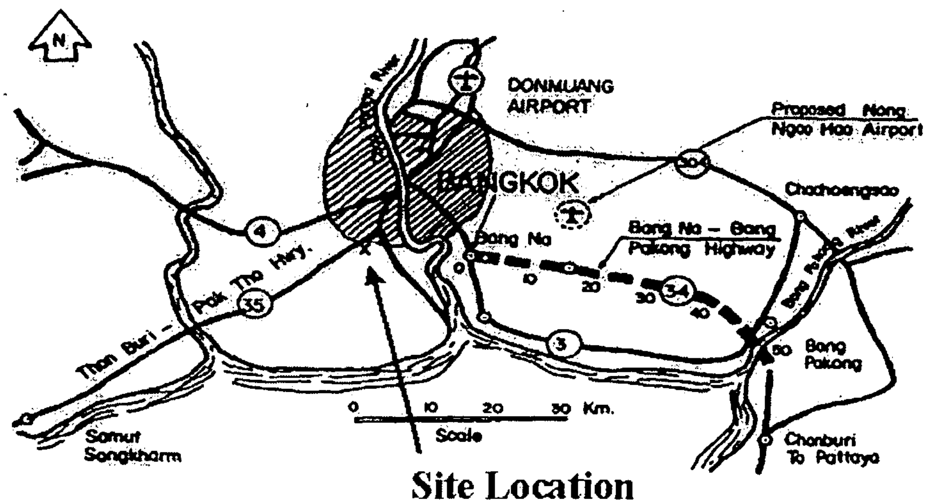


Figure 4.3 Site Location

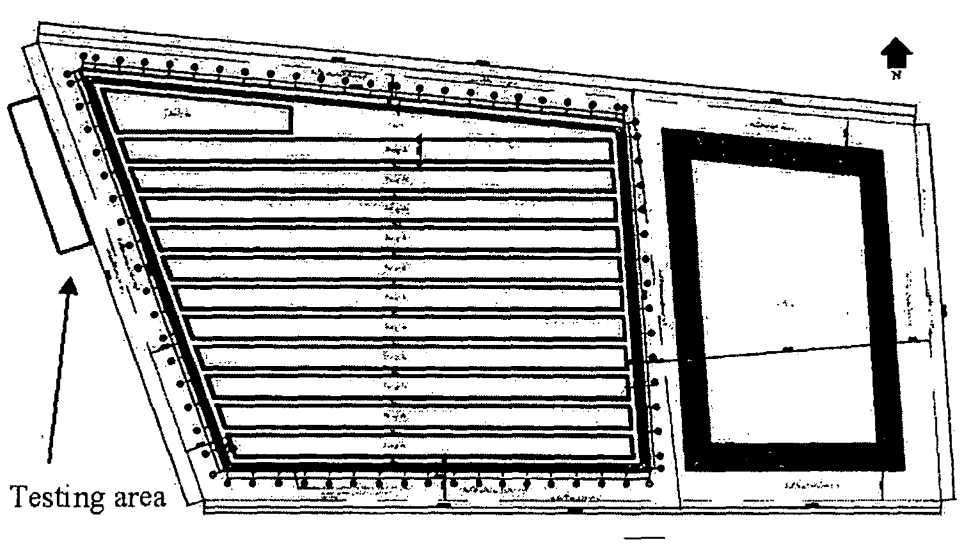


Figure 4.4 Plan of the site

4.2.1 Sampling Procedure of soil.

The 30m borehole was carried out by wash boring. Figure 4.5 shows the schematic layout and a site photograph of the wash-boring operation. The soil was sampled from the surface to a depth of 17m using a thin-walled piston as shown in the Figure 4.5b. The soil was carefully removed from the borehole to avoid disturbance of the soil and. the thin-walled piston pressed into the soil at the bottom of the boring. The soil-filled tube was then removed, sealed to prevent moisture loss and labelled for laboratory testing. When the soil was too hard to sample by thin-walled piston, a split-barrel sampler as shown in the Figure 4.5a was used to obtain a sample. The Standard Penetration Test (SPT) was also carried out to obtain data on in situ soil strength.

The numbers of blows for each 6in (150mm) of penetration were recorded and the first 150mm was considered to be a seating drive. The sum of the number of blows required for the second and third 150mm of penetration was taken as the "standard penetration resistance", or the "N-value".

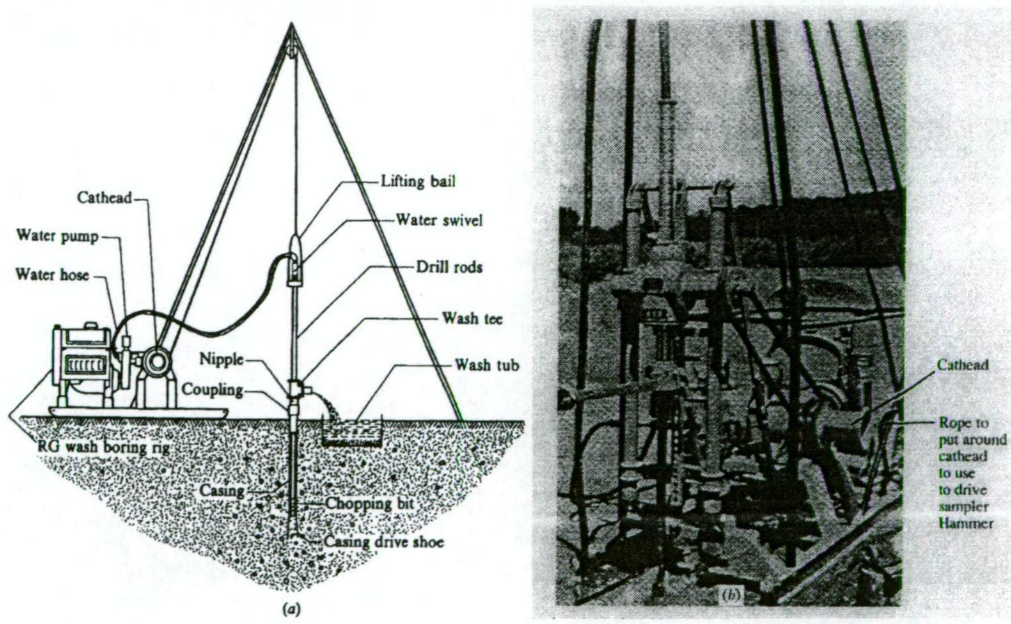


Figure 4.5 (a) Schematic of wash-boring operations; (b) photograph of wash-boring operation. (Braja, 1995)

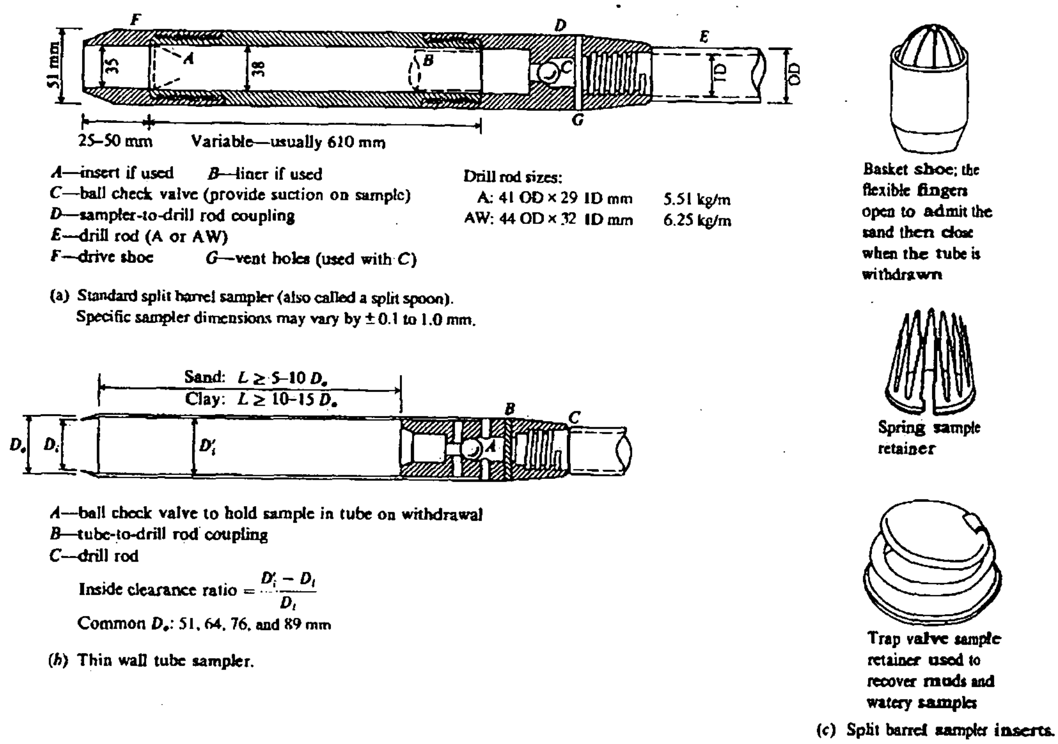


Figure 4.6 Commonly used in situ testing and sample recovery equipment, for both split barrel and thin wall tube (ASTM D 1586 and D 1587)

4.2.2 Sampling Procedure of cored samples of cement columns.

After static pile load test was carried out to each cement column (see Section 4.12), each column was cored to extract samples for unconfined compressive strength testing. Figure 4.7 shows an example of coring samples in the field. Each column was cored in the middle, in 1200mm consecutive lengths through along the column. The cored samples were wrapped with plastic sheet to protect against losing or gaining moisture. Each cored sample was identified with column number, depth and direction. After wrapping the samples were stored in a moisture controlled room.

Figure 4.7 Coring samples

4.3 Pile Installation

4.3.1 Type of Cement used

"Elephant Brand" Type One Portland Cement, which was produced by the Siam Cement Public Company (Thailand) was used in this study. The properties of Elephant Brand Type One Portland cement are shown in table 4.3. A cement truck delivered the cement to a cement silo at the site.

Table 4.3 Properties of Elephant Brand Type One Portland Cement

Chemical Composition	%(by weight)
Silicon dioxide (SiO_2)	21.63
Aluminum oxide (Al_2O_3)	5.09
Ferric oxide (Fe_2O_3)	2.92
Magnesium oxide (MgO)	0.91
Sulphur trioxide (SO_3)	1.68

Loss of ignition	0.82
Insoluble residue	0.11
Tricalcium silicate ($3Ca,SiO_2$)	58.00
Tricalcium aluminate ($3CaO,Al_2O_3$)	8.60
Fineness, specific surface (Blaine)	3000 cm^2/g

4.3.2 Installation of Cement Columns for Testing

Twelve cement columns of six different column lengths were installed. There were two columns each of 5m, 7m, 9m, 11m, 13m and 15m column length. A cement content of 200kg/m^3 and a 1.1/1 w/c ratio was used to produce the columns. The pressure of pre-jet with water was about 15000kPa and the pressure to jet the cement slurry was 20000kPa. The data of water and cement slurry flow rate, jetting pressure and cement volume were recorded with a computer and are shown in Appendix 1.

4.4 Unconfined Compression Tests

4.4.1 Test Apparatus

4.4.1.1 **Compression Device** – The compression device is a hydraulic loading device, which manufactured by Wykeham farrance, England. It has the maximum load capacity of 100kPa and control to provide the rate of loading.

4.4.1.2 **Deformation Indicator** – The deformation indicator is a dial indicator graduated to 0.03mm and having a travel range of at least 20% of the length of the test specimen

4.4.1.3 **Dial Comparator** – for measuring the physical dimensions of the specimen to within 0.1% of the measured dimension.

4.4.1.4 **Timer** – A timing device indicating the elapsed testing time to the nearest second for establishing the rate of strain application.

4.4.1.5 **Balance** – The balance used to weigh specimens to determine the mass of the specimen to within 0.1% of its total mass.

4.4.1 Specimen Preparation and Set up

4.4.2.1 Soil specimen – the required size of the specimens were 70mm diameter and 140mm height. The specimens were trimmed to the required length, diameter and weighed.

4.4.2.2 Cored specimen of the cement columns – the required size of the specimens was 45mm diameter and 90mm height. After trimming the specimens to the required length, they were soaked in water at least for a week before testing.

4.4.2 Testing Procedures

An axial strain of 1%/mm was used for unconfined testing. Figure 4.8 shows the sample placed on the loading device before testing. Load and deformation values were recorded at sufficient intervals to define the shape of the stress-strain curve. Loading was continued until the load values decreased with increasing strain or until 15% strain was reached. A sketch of each test specimen at failure was also made for future reference. This procedure was used with both the soil specimens and the cored column specimens.

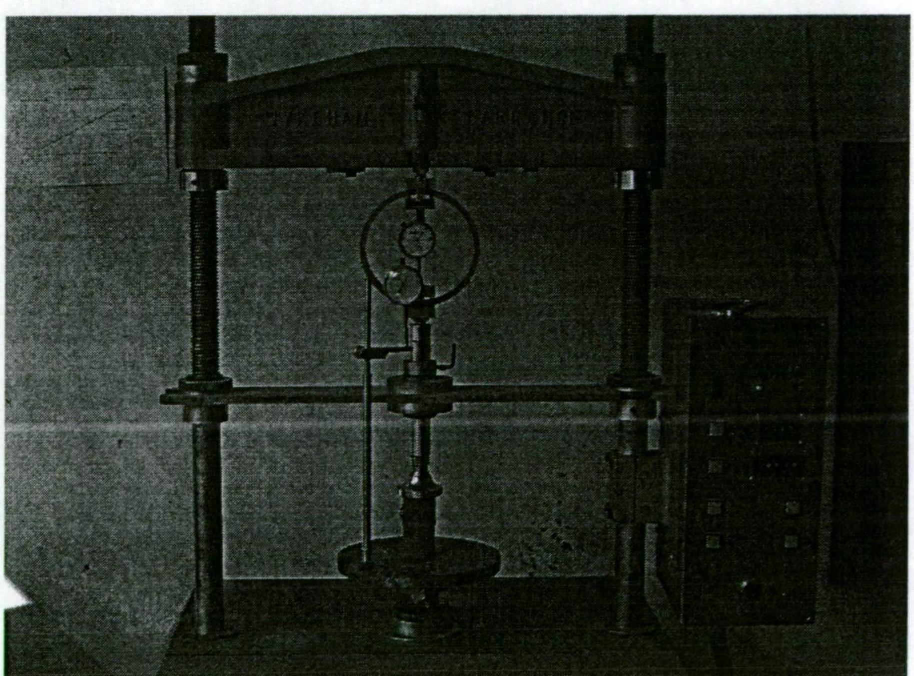


Figure 4.8 the loading device.

4.5 Oedometer Tests

Oedometer testing was carried out to determine the load deformation characteristics of the in situ material. In this case a lever arm type of oedometer was used to determine the consolidation behavior of the soil.

4.5.1 Apparatus

The lever arm type of oedometer used in the consolidation tests of this study is shown in Figure 4.9 the lever arm ratio used was 1:11, and the maximum axial pressure was 1600kPa.

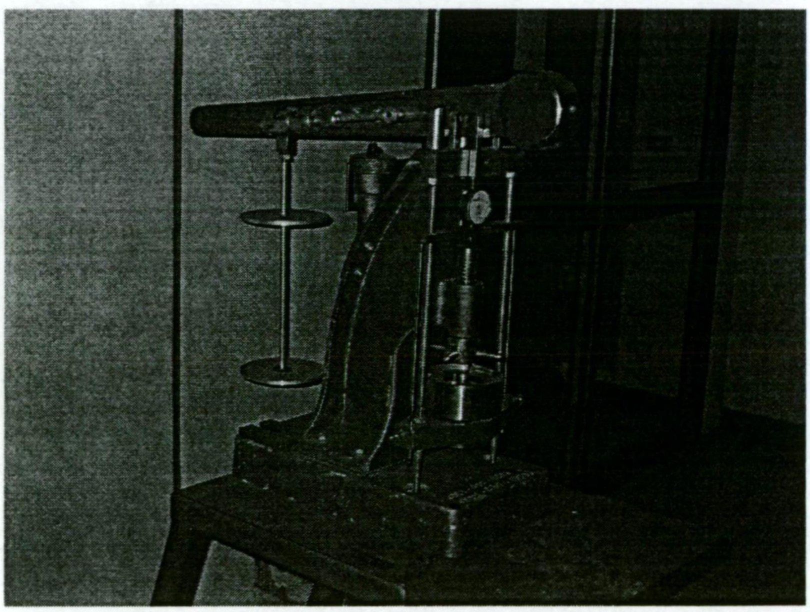


Figure 4.9 oedometer device

4.5.2 Specimen set-up

Porous stones were used with the oedometer, which were boiled in water beforehand. When the preparation of the specimen was completed, the sample ring containing the specimen was placed on a porous stone of the oedometer. Pieces of cut filter paper saturated with water were placed at the top and bottom ends of the specimen, to prevent the clogging of the porous stone during consolidation. The container of the oedometer was filled with de-aired distilled water before the prepared specimen was submerged to saturate the specimen.

4.5.2 Compression and rebound tests

The steps involved in the compression and rebound tests were summarized as follow:

a) The scale loads were applied in steps, so that the specific vertical compression pressures of 12.5, 25, 50, 100, 200, 400, 800 and 1600kPa were applied over a 24 hour duration for each loading.
b) Reading of the strain dial gauge was recorded at fixed time intervals until 90% consolidation was reached.
c) The above two steps (a) and (b) were repeated until the maximum load was applied.
d) After the load had been increased to its maximum load, it was reduced, again in steps, to allow specific rebound pressures of the magnitudes of 800, 400, 200 and 100kPa over 12 hour duration. Recording of the initial and final rebound dial readings were made.
e) At the end of the test, after the final reading of the last rebound pressure, the specimen was weighed with the sample ring, and then, they were placed in the oven for further weighting for the process of determining the water content in the specimen.

4.6 Triaxial Tests

4.6.1 Testing Equipment

The triaxial test arrangement used is shown in Figure 4.10. The equipment consists of a triaxial cell, load cell and readout units. Pressure transducers having a capacity of 1000kPa were used to monitor pore pressure. Cell pressure is applied by self-compensating mercury pots. The vertical load is applied by using a loading frame and measured by the load cell. A displacement transducer fixed on the cell is used for measuring the axial displacement. All equipment was carefully calibrated before use in the testing.

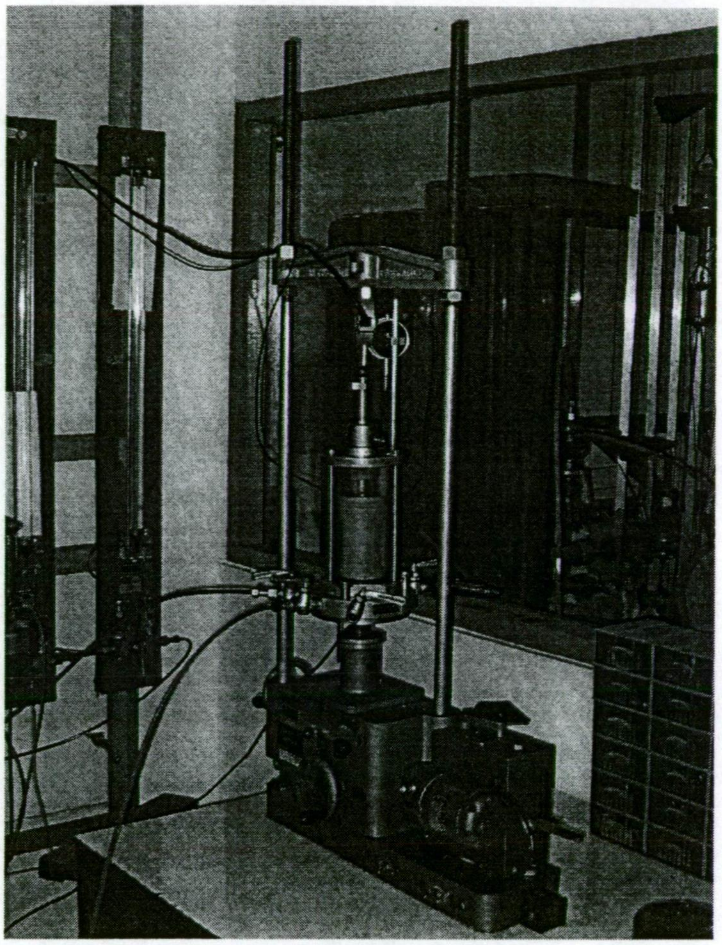


Figure 4.10 triaxial equipment

4.6.2 Sample Preparation

Soil samples obtained by 72mm diameter long thin piston sampler from 6.00 to 6.50m depth were used as "undisturbed" samples.

After extruding the samples from the tubes, each was cut and trimmed into sections (100mm length and 50mm diameter), covered with paraffin wax and stored in the humidity-controlled room. During the testing phase each sample stored in the moist room was taken out, wax carefully removed and trimmed to the final required shape and size. Two porous stones with filter paper were placed along the circumference of the samples. The sample was then enclosed within the rubber membrane sealed by "O" rings at both ends and the chamber filled with water.

4.6.3 Testing Procedures

4.6.3.1 Saturation of the Specimens

Even though the specimen itself was fully saturated, there was always a very small amount of air entrapped between the rubber membrane and the sample, and between porous stones and filter papers. A back pressure of 200kPa was applied to ensure a fully saturated condition. During the application of pressures, the cell pressure was always maintained higher than the back pressure by 10 kPa. When the back pressure reached 200 kPa, the specimen was left for at least 24 hours to achieve the fully saturated condition. At the end of 24 hours saturation, the saturation state was checked by closing the drainage system and increasing the cell pressure to estimate the increasing of pore pressure. For most of the specimens, B value could be achieved to the value of 1 in one minute. This defined full saturation.

4.6.3.2 Consolidation of the specimens

After fully saturating the sample, the maximum back pressure was held constant and the chamber pressure increased until the difference between the chamber pressure and the back pressure equaled the desired effective consolidation pressure.

4.6.3.3 Testing of the specimens

After consolidation, every specimen was left for 24 hours to ensure the full dissipation of excess pore pressure. In the drained tests, all samples were tested at a strain rate of 0.0033mm/min to allow the soil to drain without developing excess pore pressure. The drainage valves were kept opened and the cell pressure was kept constant.

4.6.4 Calculation of Stresses and Strains

In triaxial stress conditions, the stress parameters p and q, which are also a function of the invariant of the stress tension, are defined as follows:

$$P = \frac{1}{3}(\bar{\sigma}_1+2\bar{\sigma}_3) \qquad \text{----------4.1}$$

$$q = (\bar{\sigma}_1-\bar{\sigma}_3) \qquad \text{----------5.2}$$

Where p and q are mean normal stress and deviator stress respectively. The incremental strain parameters for triaxial stress conditions are follows:

$$d\varepsilon_s = \frac{2}{3}(d\varepsilon_1-d\varepsilon_3) \qquad \text{----------4.3}$$

$$d\varepsilon_v = (d\varepsilon_1+2d\varepsilon_3) \qquad \text{----------4.4}$$

$$d\varepsilon^e_v = \frac{K}{1+e}\,\frac{dp}{p} \qquad \text{----------4.5}$$

$$d\varepsilon^p_v = d\varepsilon_v-d\varepsilon^e_v \qquad \text{----------4.6}$$

$$d\varepsilon^e_s = 0 \qquad \text{----------4.7}$$

$$d\varepsilon^p_s = d\varepsilon_s \qquad \text{----------4.8}$$

Where $d\varepsilon_s$ and $d\varepsilon_v$ are incremental shear strain and incremental volumetric strain respectively.

Natural strain values used in the analysis are calculated as follows:

$$\varepsilon_l = \ln(L_0/L) \qquad \text{----------4.9}$$

$$\varepsilon_v = \ln(V_0/v) \qquad \text{---------4.10}$$

Where ε_l and ε_v are axial and volumetric strains respectively.

4.6.5 Correction on the results

The following errors and corrections were taken into account in the interpretation of triaxial test results.

4.6.5.1 Correction for Cross-Sectional Area

Henkel (1958) suggested that the calculation of stress at any stage of test should be based on the area of the specimen at that instant. Therefore, deviator stress is calculated assuming that the samples remain cylindrical throughout the loading process. The expression for area correction is,

$$A_c = A_0 \left(\frac{1-\Delta V/V_0}{1-\Delta L/L_0} \right) \qquad \text{---------4.11}$$

Where,

A_c : the corrected cross-section area of the specimen

A_0 : the initial cross-sectional area after consolidation

V_0, L_0 : the initial length and volume of the specimen

ΔL, ΔV: the change in length and volume of specimen at specimen at certain stage of shearing, respectively.

4.6.5.2 Correction for Rubber Membrane

Generally, in any type of triaxial test, it is necessary to put filter paper around the specimen and enclose the specimen in a rubber membrane. The following membrane correction formula (suggested by Henkel, 1958) was applied on the calculated deviator stress at each strain level.

$$\sigma_{am} = \pi D M_r \varepsilon_a \frac{(1-\varepsilon_a)}{A_0} \qquad \text{---------4.12}$$

Where,

σ_{am} : the axial stress taken by the membrane

ε_a : axial strain

D : the initial diameter of sample

M_r : the compression modulus of membrane per unit width

A_o : the initial cross-section area of sample

The thickness of membrane and modulus used in this study were 0.2mm and 60 kPa, respectively.

4.7 Static Load Tests

4.7.1 Testing Program

Twelve cement columns of six different column lengths were installed. There were two columns each of 5m, 7m, 9m, 11m, 13m and 15m column length. After 28 days of installation, two piles were chosen to carry out static load tests. It took a maximum of five days to complete the testing of a pair of piles. Sometime it took less than five days for long columns as the cement column crushed early during testing. Six pairs of testing piles were completed over a month. Figure 4.11 shows the static load tests at the site.

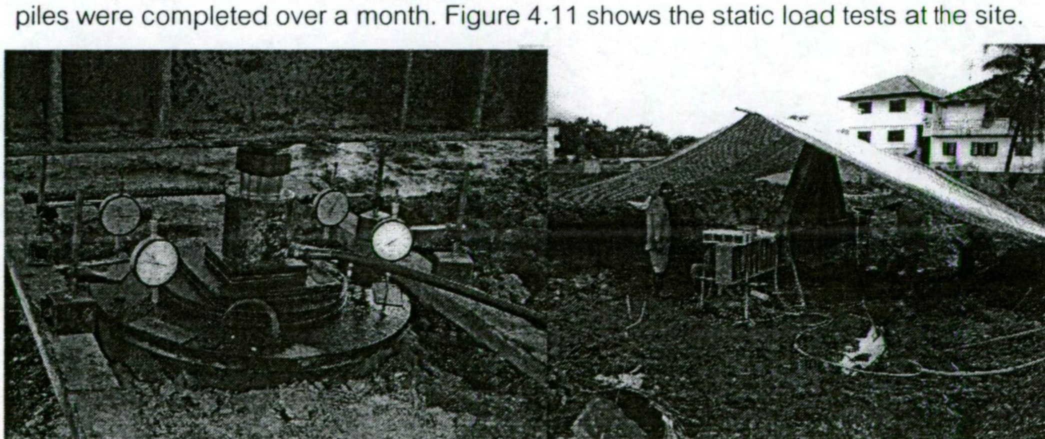


Figure 4.11 static load test at the site

4.7.2 Apparatus and Instrumentation

4.7.2.1 **The hydraulic jacking system** for applying the vertical load included the hydraulic jack, hydraulic pump and pressure. Two different sets of the jacking system were used in this test being 30tons hydraulic jack capacity and 50tons hydraulic jack capacity respectively.

4.7.2.2 **Steel bearing plates** of sufficient thickness to prevent bending under the loads involved were placed at the pile top.

4.7.2.3 **Ball bearing** was centered between the top of the hydraulic jack and the bottom of the test beam. It ensured that the load was at the centre of the column while the load test was in the process.

4.7.2.4 **A reaction test beam**, which had sufficient size and strength to avoid excessive deflection under load.

4.7.2.5 A sufficient number of **anchor piles** to provide adequate reactive capacity of the column were. In this test, two-anchor piles were used for each pile test for lengths between 5m and 11m. Four anchor piles were used for each pile test where the columns were between 13m and 15m in length. The anchor piles were 0.5m diameter and 6m length.

4.7.2.6 **Reference beams** were independently supported with supports sufficiently stiff to support dial gages.

4.7.2.7 **Dial gages** were mounted on the reference beams approximately equidistant from the center of and on opposite sides of the test pile with stems parallel to the longitudinal axis of the pile and bearing on lugs firmly attached to the test plate.

4.7.3 Testing Procedure

Unless failure occurs first, the pile is loaded in three sequences. In the first sequence, load the pile to 100% of the anticipated pile design load by applying the load in increment of 25% of the individual pile design load. Maintain each load increment until the rate of settlement is not greater than 0.25mm/hour but not longer than 2 hours. Provided that the test pile has not failed, allow the total load to remain on the pile for 24 hours. After the required holding time, remove the test load in decrements of 50% of the anticipated pile design load with 1 hour between decrements. The second sequence and the third sequence follow the same process as the first but they are loaded to 200% and failure occurred representatively. A schematic representation of the plie testing system is shown below in Figure 4.12.

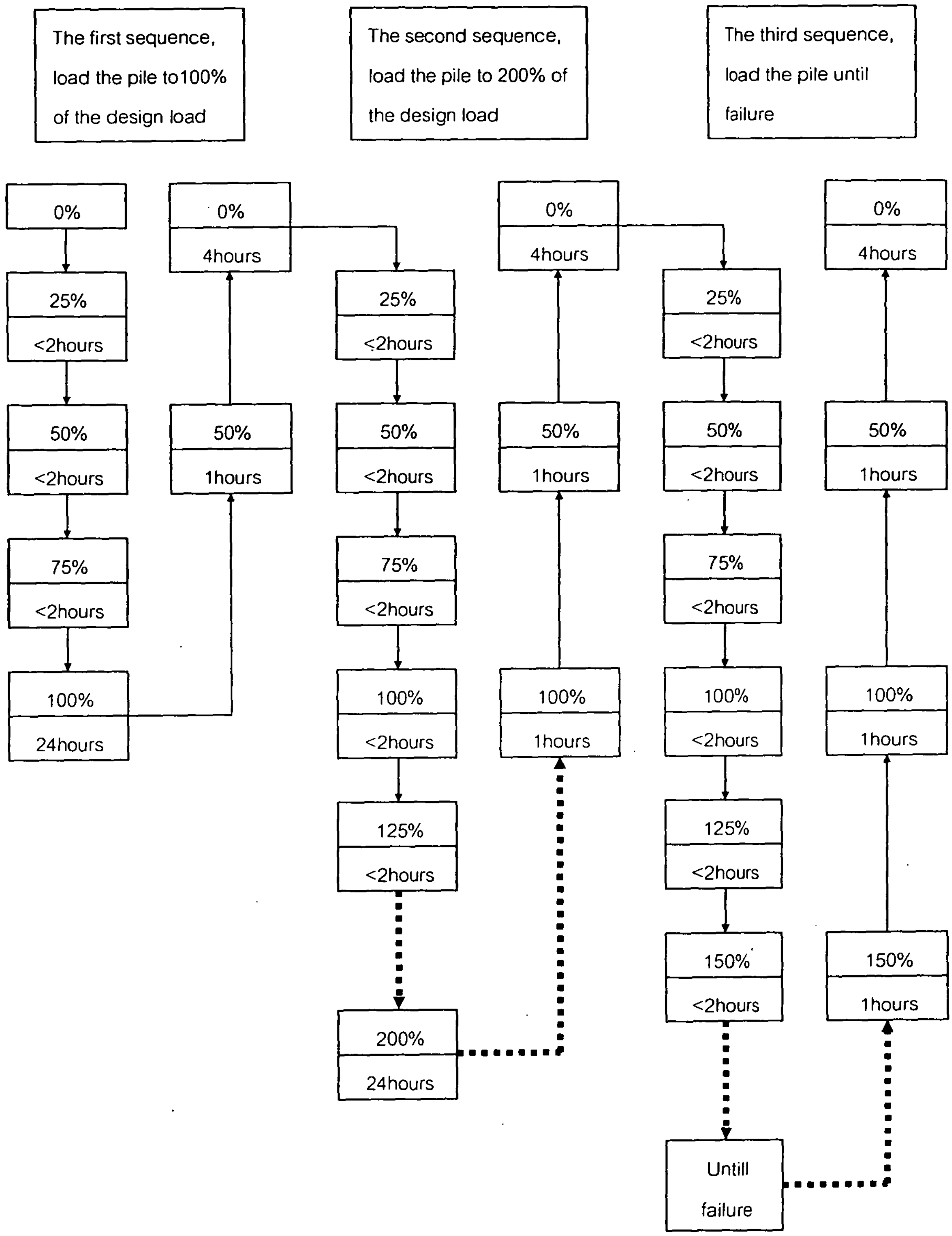


Figure 4.12 A schematic representation of the plie testing system

Chapter 5

EXPERIMENTAL RESULTS AND DISCUSSIONS

5.1 General

This chapter contains the results obtained from soil field tests, laboratory tests on extracted soil specimens and field observations on pile testing.

5.2 Results from the laboratory and the field.

5.2.1 Physical Properties of the in-situ soil

The soil was sampled from one bore hole (BH-1) as describe in Chapter 4 and four field vane shear tests (FVST 1-4). Index tests were performed on the subsoil samples. The soil profile and properties were plotted down to 30m depth and are summarized in Table 5.1 and Figure 5.1. They show Shear Strength, soil classification, N-value, Unit Weight, Atterberg Limit, Specific Gravity and Field Vane Shear Test.

The Specific Gravity varies from 2.67-2.80 in the soft clay. It is about 2.69, 2.63 and 2.75 in the stiff clay, dense sand and hard clay respectively. The highest liquid limit of 105% was recorded at 6.00-6.50m depths. The water content ranges from 40% to 115% in the soft clay. The results show that soft Bangkok clay has a very high liquid limit and water content.

The field vane shear test is a means to evaluate the in-situ undrained strength of clays. Figure 5.1 shows the uncorrected undrained shear strength plotted with depth. The results indicate variation in the upper 1.0 to 3.0m and it could be attributed to the fact that the soil in this zone experienced desiccation and weathering process. At deeper depths, the data shows a more consistent trend. It can be seen that the lowest strength was recorded at 6.00m depth.

Unconfined compressive testing was undertaken to determine undrained shear strength, Su, of the top clay layer. The results were in the range of 6 to 7kPa, while undrained moduli, E_u, were in the range of 900 to 1100kPa. The undrained modulus can also be given in terms of the uncorrected field vane shear strength, S_{uv}, as $E_u = \alpha S_{uv}$. The value of α for Bangkok clay lies between 70 and 250 (Balasubramaniam and Brenner, 1981). Bergado (1990) reported that a α value of 150 is the best estimate for the settlement prediction of Bangna- Bangpakong Highway (The highway from Bangkok to Pataya).

From the testing of soil properties, which are shown in Table 5.1. The clay layer can be classified as below. This classification also used to model the clay layer in the PLAXIS program in Chapter 6.

Weathered Clay is the hard and dry top clay layer. This is verified by field vane shear test, unconfined compressive testing, unit weight and water content. Strength and unit weight of the Weathered Clay is higher than the Soft Clay and as expected the water content is lower. The depth of the layer was about 2m.

Soft Clay is the low strength and high water content clay layer. This is verified by field vane shear test, unconfined compressive testing and water content. Sometimes the water content of soft Bangkok clay is higher than 100% and in this study, this occurred from 2m to 14m depth.

Stiff Clay is the lower high strength clay layer. The simple way to separate this clay from the soft clay is to note that the vane can not be turned because the clay is too hard. Thus the Standard Penetration Test (SPT) is undertaken to obtain the strength. In this study, this clay layer is from 14m to 20m depth.

Table 5.1 The summary of the soil properties from the laboratory and the site.

Sample No.	Depth. (m) From.	To.	Unconfind. ton/m^2	SPT N,Correct	Su(correct) From FVS, t/m^2	Natural Water Content	Liquid Limit	Plastic Limit	Plastic Index	Specific Gravity Gs	Unit Weigth ton/m^3	Overconsolidated Ratio(OCR)
ST-1	1.50	2.00	2.22		5.17	45.3	67	27	40		1.71	5.20
ST-2	3.00	3.50	0.86		3.09	85.0	81	31	50		1.52	
ST-3	4.50	5.00	0.37		1.08	115.7	94	35	59		1.44	1.13
ST-4	6.00	6.50	0.34		0.93	106.1	105	40	66		1.39	
ST-5	7.50	8.00	1.06		1.58	90.9	96	35	62		1.49	1.53
ST-6	9.00	9.50	1.23		1.75	89.2	96	44	51	2.80	1.56	
ST-7	10.50	11.00	1.83		2.53	55.3	73	32	41	2.74	1.65	
ST-8	12.00	12.50	1.37		3.19	40.4	66	28	38	2.67	1.75	
ST-10	15.00	15.50	4.02		4.96	54.6	59	26	33		1.75	
ST-11	16.50	17.00	4.46			39.7	62	31	31		1.88	
SS-1	18.00	18.45	-	8.61		30.4	NL.	NP.	-	2.69		
SS-2	19.50	19.95	-	7.93		18.6	34	18	16			
SS-3	21.00	21.45	-	7.69		17.4	NL.	NP.	-			
SS-4	22.50	22.95	-	7.67		16.2	NL.	NP.	-	2.63		
SS-5	24.00	24.45	-	8.03		14.6	NL.	NP.	-			
SS-6	25.50	25.95	-	8.12		14.6	NL.	NP.	-			
SS-7	27.00	27.45	15.93	8.42		23.5	51	14	37		1.93	
SS-8	28.50	28.95	2.02	8.19		24.7	56	21	35		2.02	
SS-9	30.00	30.45	5.58	9.51		28.3	44	24	20	2.72	2.01	

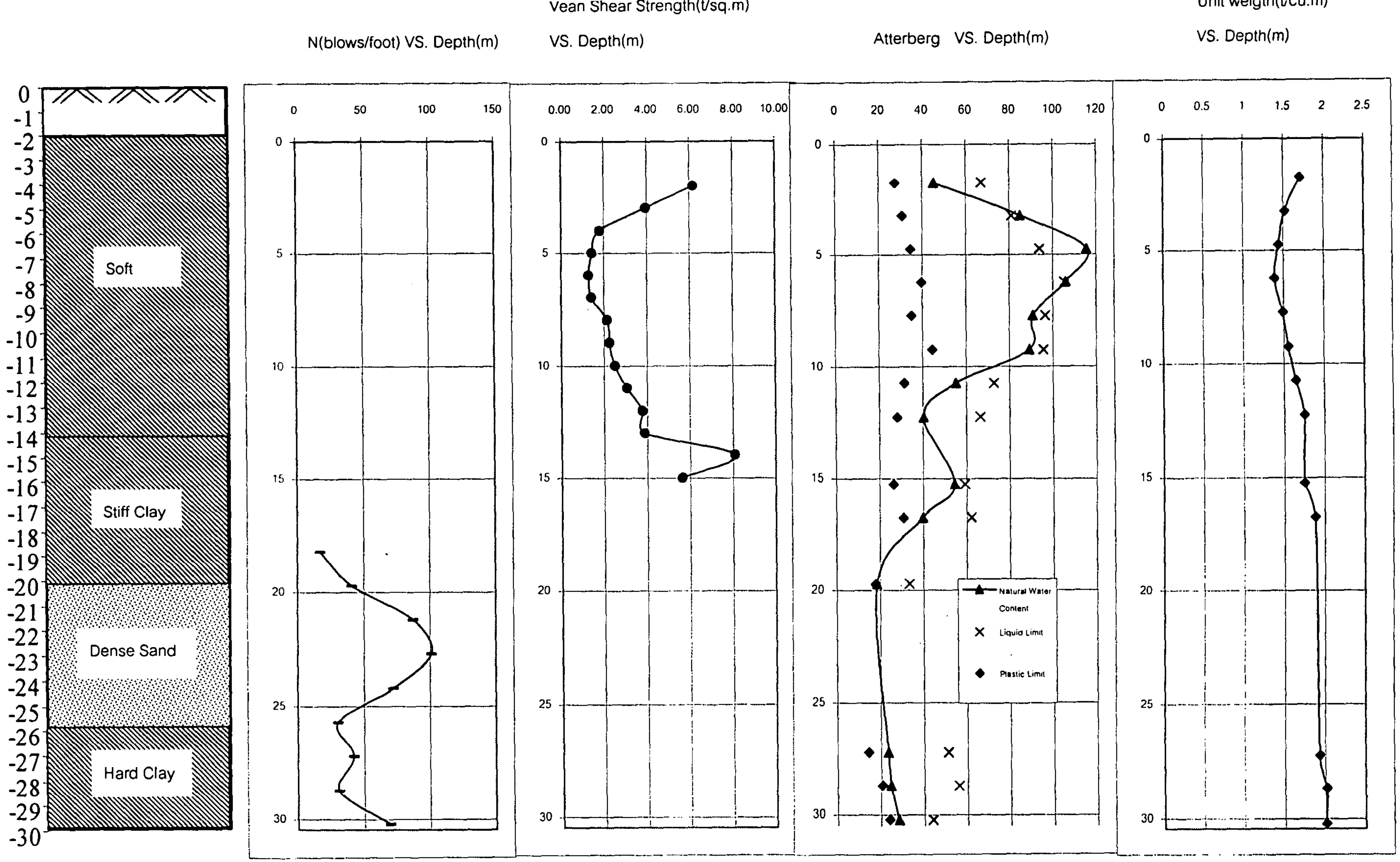


Figure 5.1 The soil profile and the summary of soil properties.

5.2.2 Oedometer test data.

The soil sample extracted from 7.50-8.00m depth was used to carry out oedometer testing to determine the Compressibility Index (C_c), the Compressibility Ratio (CR), Recompressibility Index (C_r), Recompressibility Ratio (RR), Initial Voil Ratio (C_o), Overcosolidated Ratio (OCR), Effective Overburden Pressure (P'_o) and Maximum past pressure (P'_c) of the soft clay layer. These parameters as shown below, they are used to representative the parameters of soft clay in the PLAXIS model in Chapter 6.

$C_c = 1.87$	$CR = 0.53$
$C_r = 0.15$	$RR = 0.042$
$e_0 = 2.55$	$OCR = 1.59$
$P_o' = 4.41\ t/m^2$	$P_c' = 7.0\ t/m^2$
C_v(Log Time) = 0.99 cm^2/sec.	$C_v(\sqrt{Time})$ = 1.10 cm^2/sec.
k_v(Log Time) = 0.20 m/day	$k_v(\sqrt{Time})$ = 0.23 m/day
$m_v = 0.2375\ cm^2/kg$.	

5.2.3 Consolidation Drained (CD) triaxial test data

The soil sample extracted from 6.00-6.50m depth in the soft clay was taken to be representative of the soft clay to carry out the CD triaxial test. Figure 5.2 shows the Stress Path data obtained for the soil samples. It can be seen from Figure 5.2 that the estimated strength parameters are C = 0.8ksc and **Ø** = 14. These parameters are used in the PLAXIS model in Chapter 6 to predict the long-term settlement.

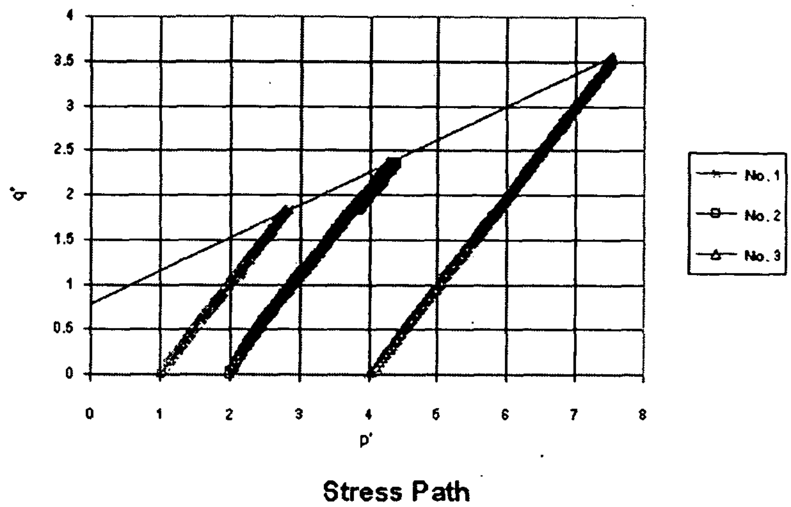


Figure 5.2 The Stress Path of soil samples

5.2.4 Unconfined compressive strength of cored samples

A total of 120 cored samples from 12 cement columns were subjected to unconfined compressive testing. The data is summarized in Figure 5.3 and the average line of the shear strength (Su). The average value is about 650kPa with a standard deviation of 174kPa and a coefficient of variation (cv) of 25%. This average value was adopted as the representative cement column strength in the analytical model in Chapter 6. However this average value may not be appropriate for use because of scatter of data caused by the difficulties of mix control in the field installation. This is discussed later in the thesis.

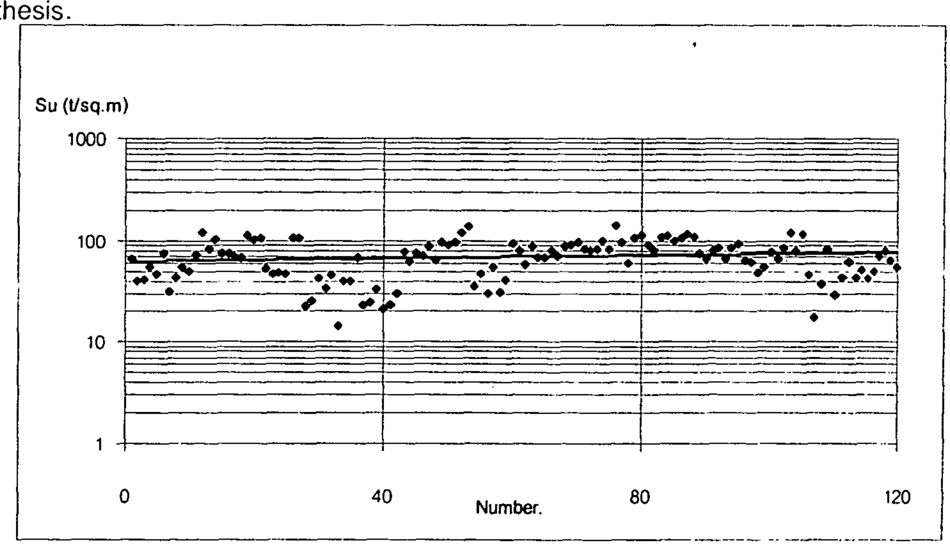


Figure 5.3 Unconfined Compressive Strength of cored samples

5.2.5 Ultimate Soil Design Load

From equation 3.1 to 3.3, the ultimate soil design load of each column length can be determined and summarised in Table 5.2. The details of calculation for each column length are shown in Appendix 2.

5.2.6 Static pile load testing

The field data from the static pile load test are illustrated in Figure 5.4 to Figure 5.9. The data was used to determine the ultimate load for each column using Tomlinson's method, De Beer and Fellinius' method and Davidson's method. All methods predict ultimate load by failure at the column to soil interface. A comparison of the calculated ultimate strength, according to the different criterion, is provided in Table 5.2. The test procedure for the static pile load tested was described in Chapter 4.

Table 5.2 The result of the static pile load test

Column No.	Length of cement columns(m)	Ultimate soil design load(tons)	Ultimate load (tons)			
			Tomlinson	De Beer and Fellinius	Davisson	average
1	5	8.20	11.00	10.00	11.00	10.67
2	5	8.20	11.10	10.00	10.50	10.53
3	7	10.80	10.00	8.00	10.00	9.33
4	7	10.80	12.50	11.25	-	11.88
5	9	13.50	13.50	9.50	-	11.50
6	9	13.50	12.25	9.00	-	10.63
7	11	19.00	9.13	7.00	9.20	8.44
8	11	19.00	11.13	9.00	11.25	10.46
9	13	22.20	9.75	9.00	9.80	9.52
10	13	22.20	7.20	8.00	7.00	7.40
11	15	30.60	17.00	16.00	17.30	16.77
12	15	30.60	9.20	9.20	9.20	9.20

5.2.7 Comments on test data

If the data in Table 5.2, and that represented in Figures 5.4 to 5.9, is considered it becomes obvious that the overall test outcomes reflect two types of failure criteria. The first type, for 5m and 7m cement columns (Figure 5.4, 5.5) relate to failure of the surrounding soil as reflected in the relative low stiffnesses seen in Figures 5.4 and 5.5. The second type, for 9m, 11m, 13m and 15m long cement columns relate to crushing failure within the cement columns themselves. The stiffnesses seen in Figures 5.6 to 5.9 then reflect that of the local material within cement columns. The variability of the cemented treated material produced in the field also becomes evident (Figure 5.8).

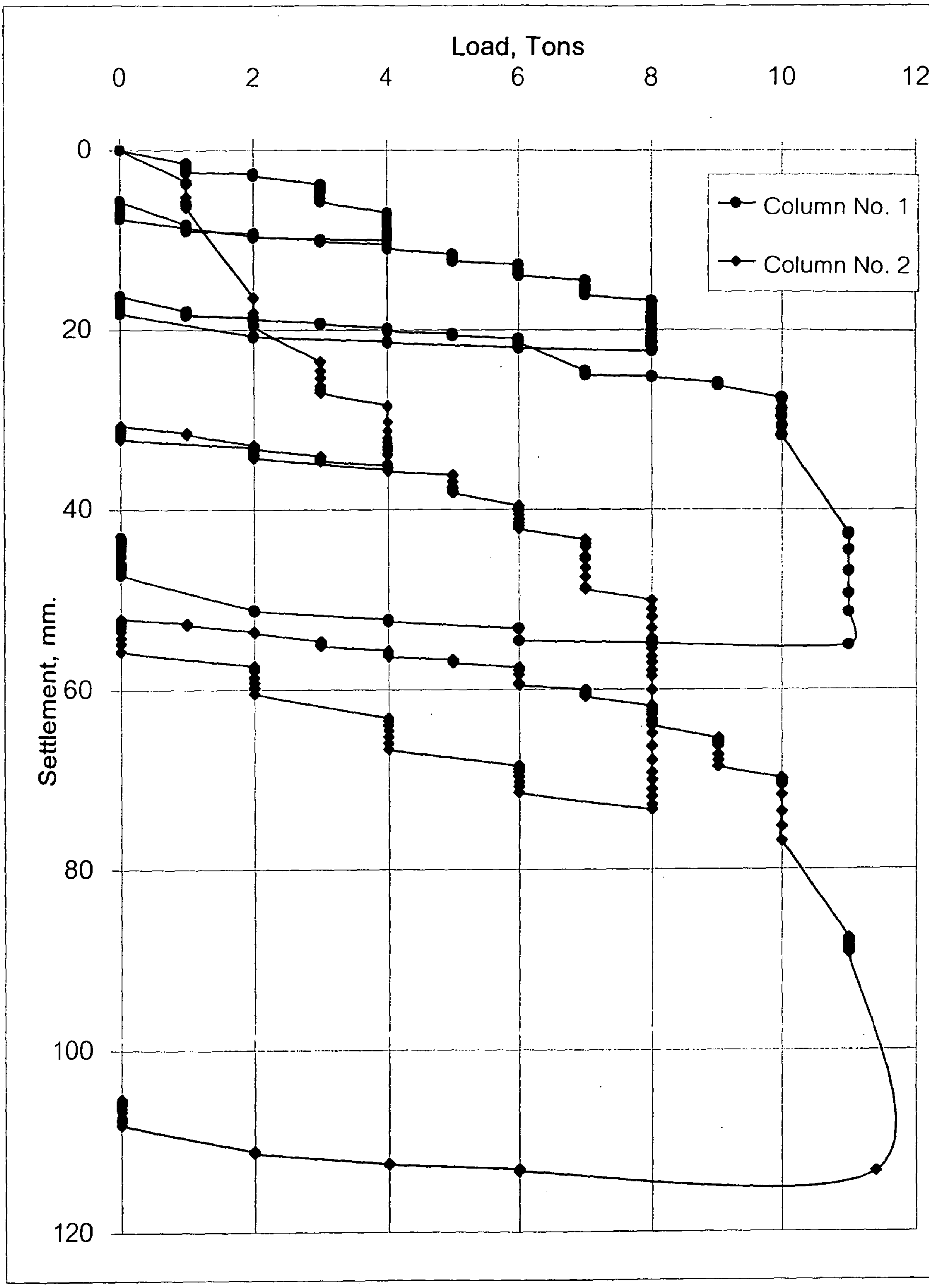


Figure 5.4 The field data of the static pile load test 5m length.

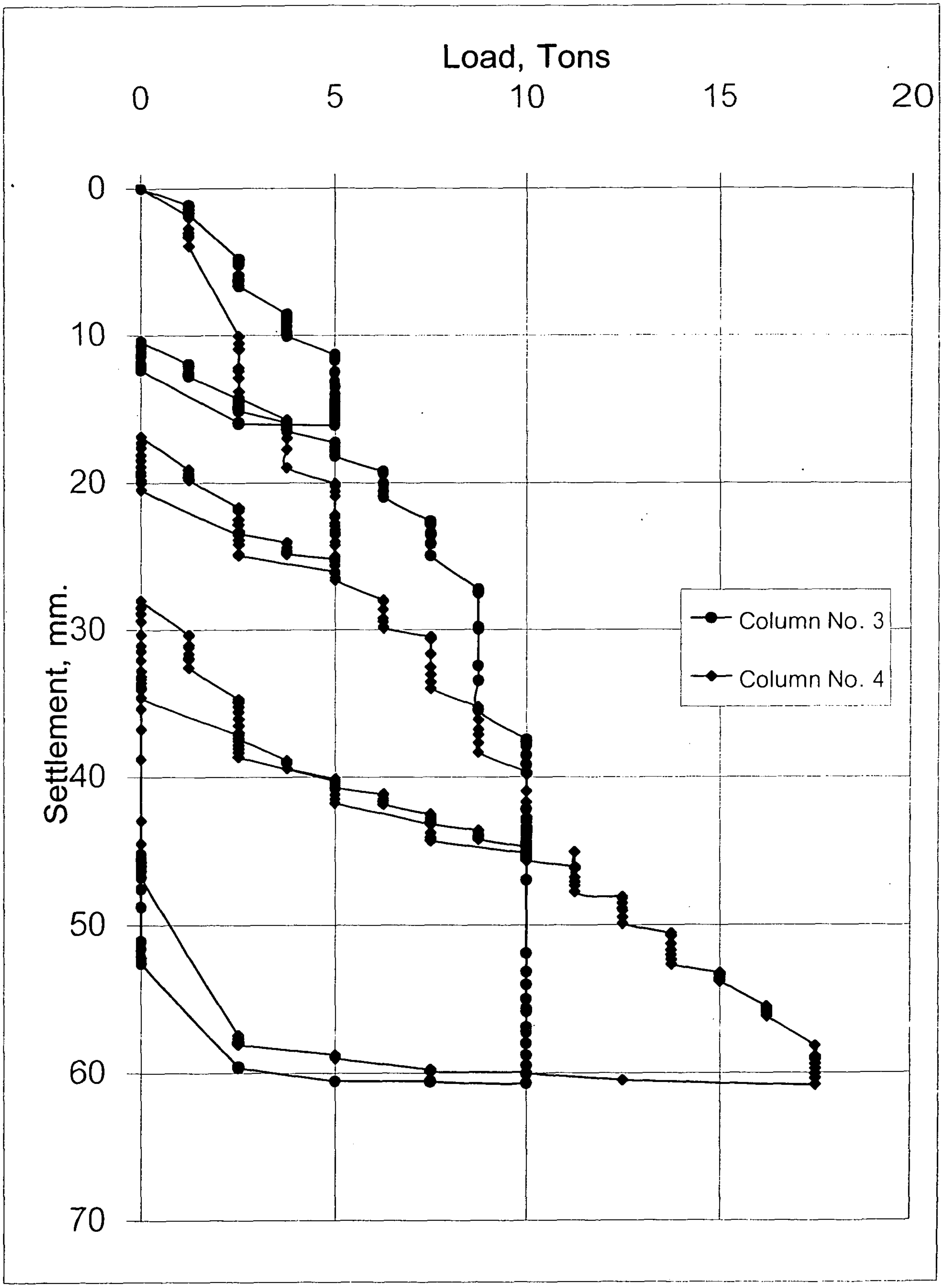


Figure 5.5 The field data of the static pile load test 7m length

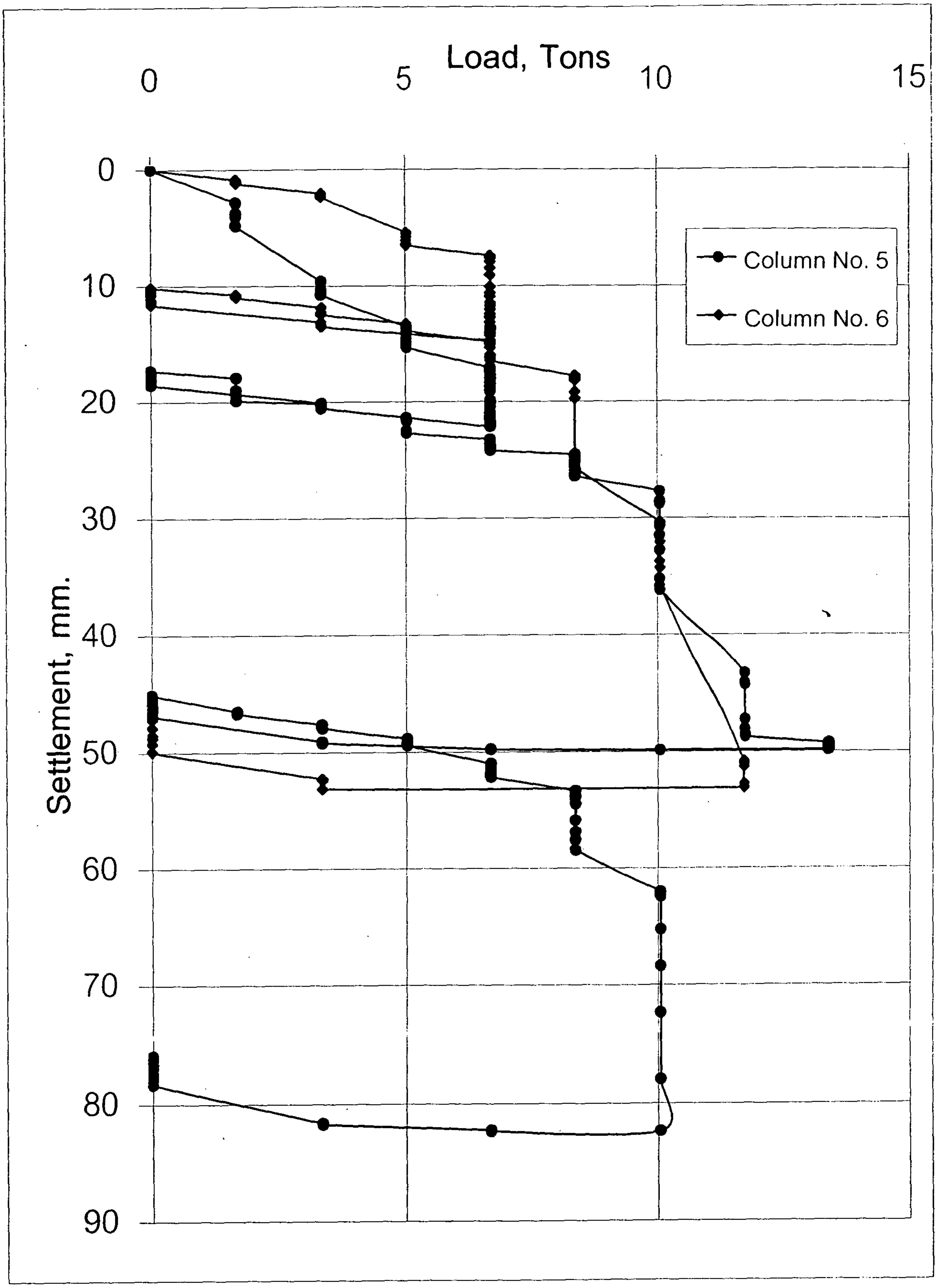


Figure 5.6 The field data of the static pile load test 9m length.

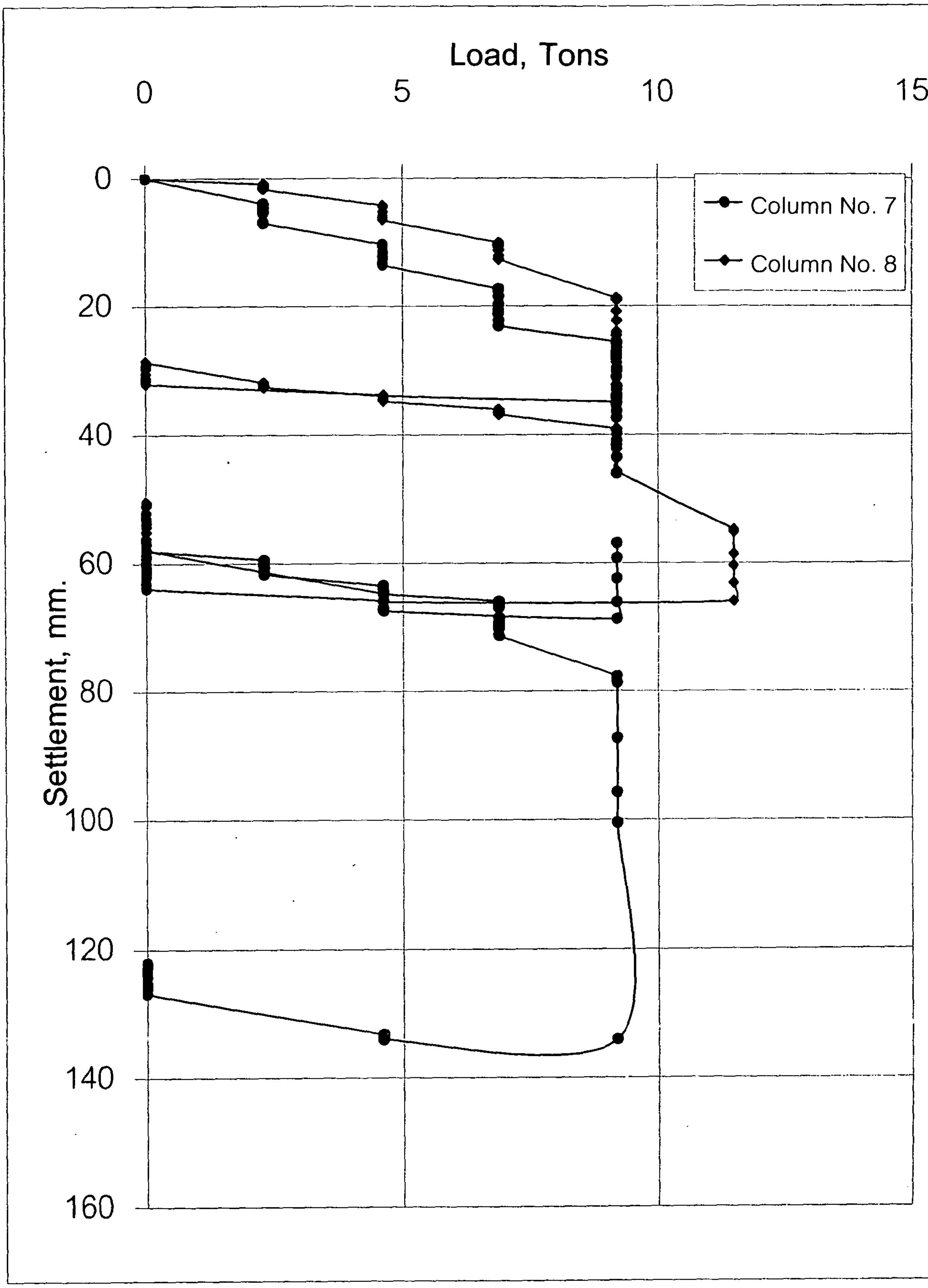


Figure 5.7 The field data of the static pile load test 11m length.

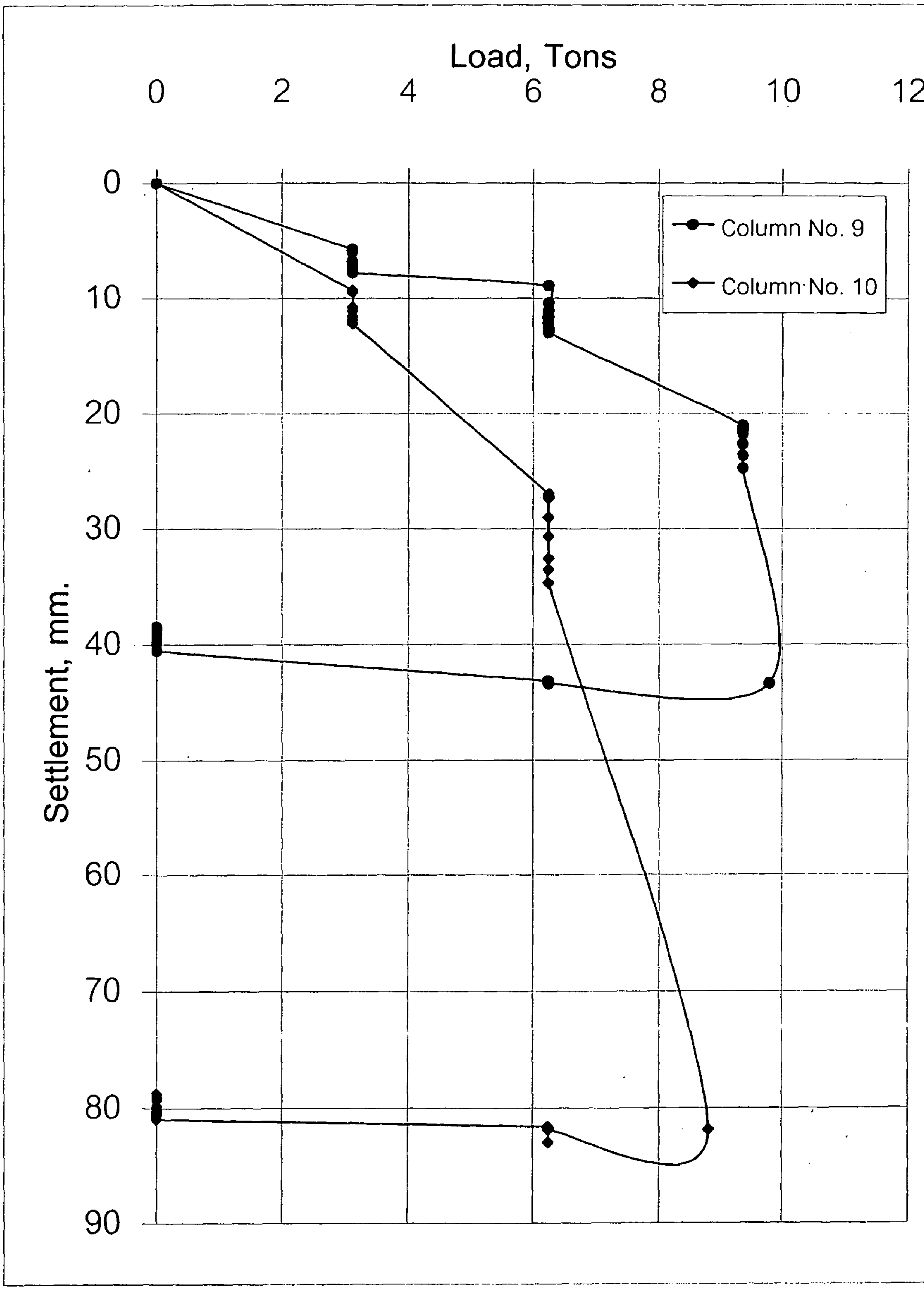


Figure 5.8 The field data of the static pile load test 13m length.

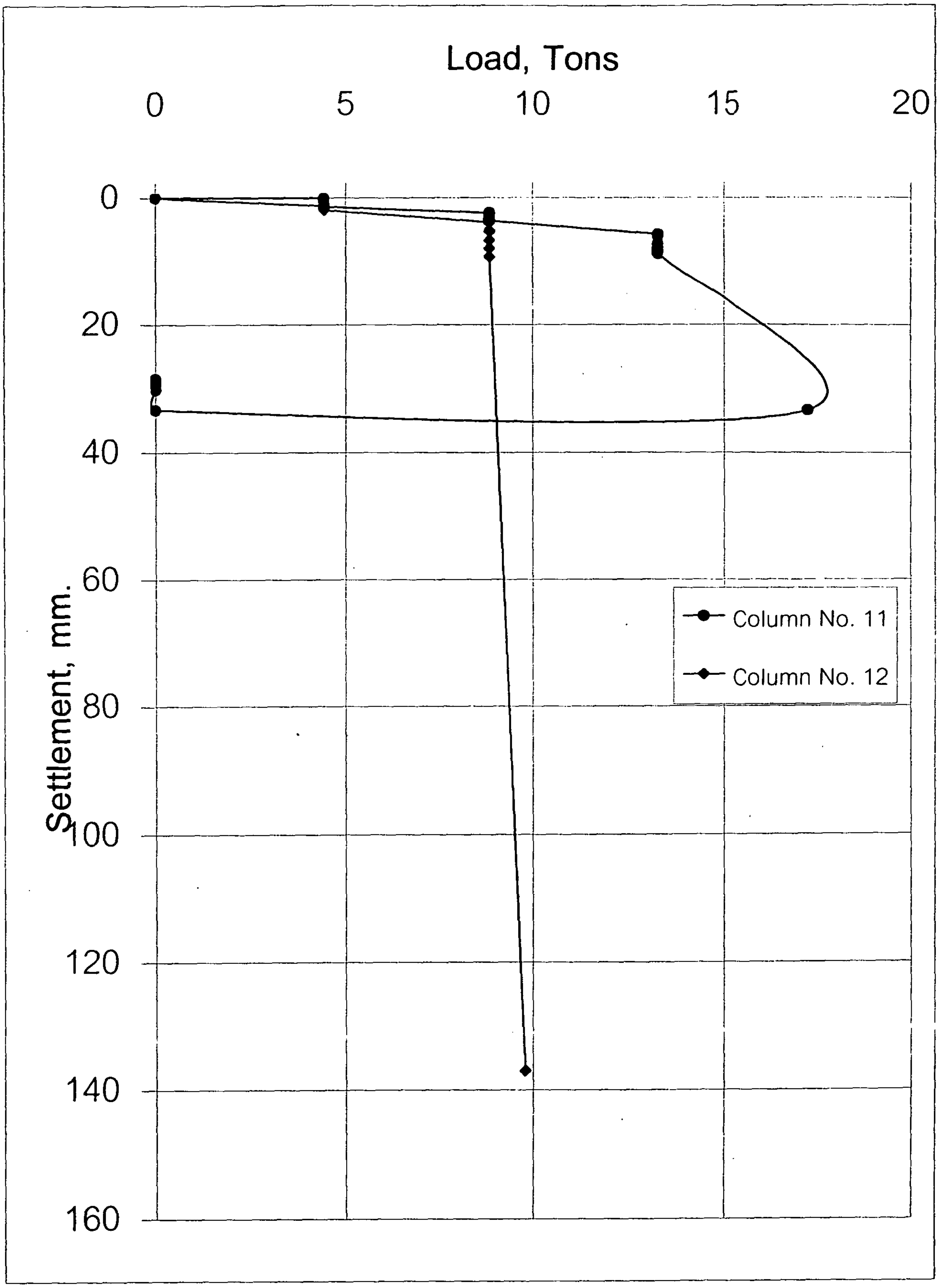


Figure 5.9 The field data of the static pile load test 15m length.

Chapter 6

MODELING FIELD BEHAVIOR

6.1 Introduction to PLAXIS Program

6.1.1 Brief Description of PLAXIS

PLAXIS (Brinkgreve and Vermeer, 1998) is a finite element package specially intended for the analysis of deformation and stability in geotechnical engineering projects. Geotechnical applications usually require advanced constitutive models for the simulation of the non-linear and time-dependent behaviour of soil. In addition, since soil is a multi-phase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. Although the modelling of the soil is an important issue, many geotechnical-engineering projects involve the modelling of structures and the interaction between the structures and the soil. PLAXIS is equipped with special features to deal with the numerous aspects of complex geotechnical structures (Brinkgreve and Vermeer, 1998).

To help understand how PLAXIS software operates, a flow chart on the overall process involved during running the program is shown in Figure 6.1.

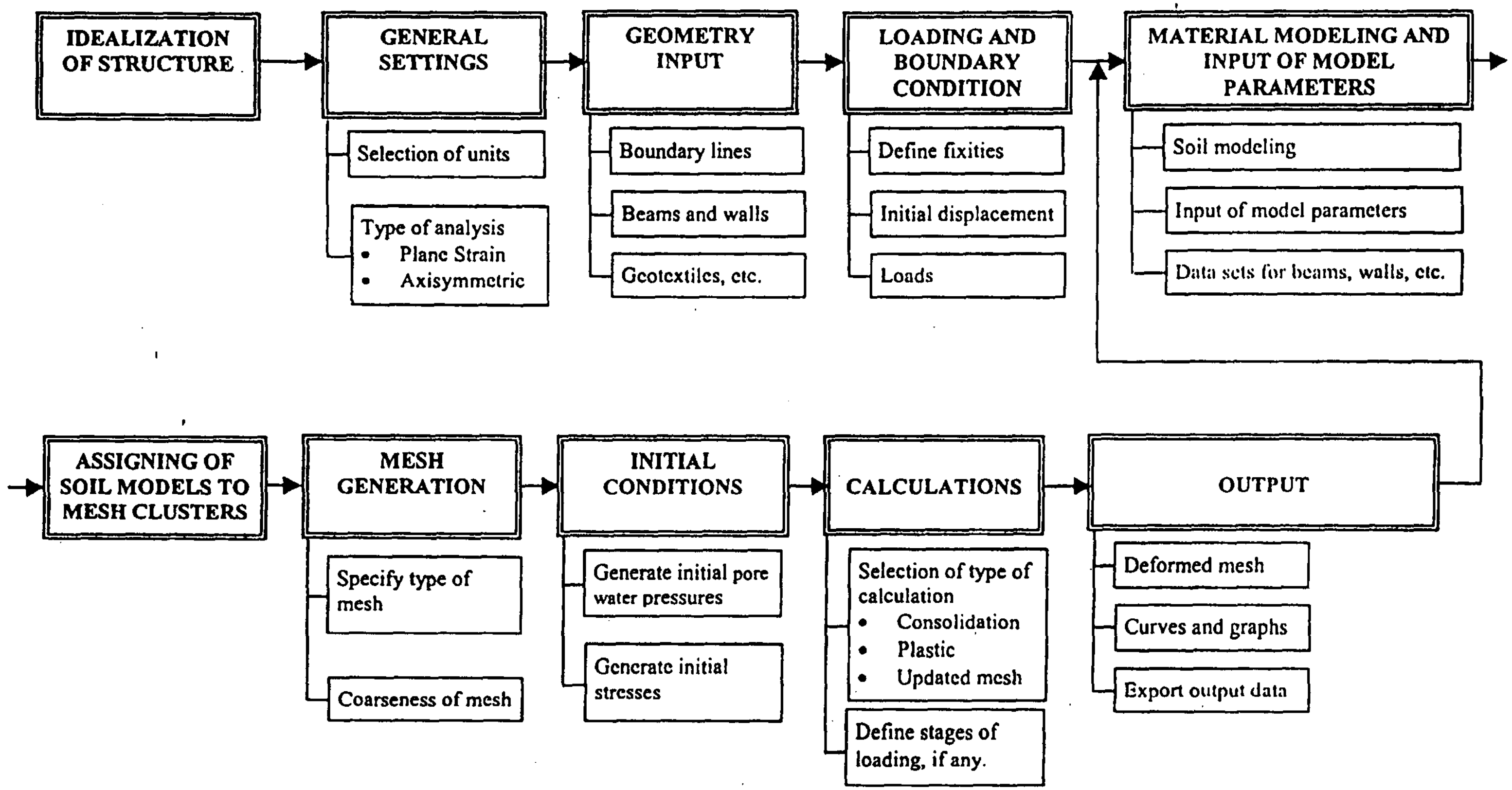


Figure 6.1 Flow Chart of the Overall Process Involved in Running PLAXIS Program
(Brinkgreve and Vermeer, 1998)

6.1.2 Brief summary of the Features of the Program

The information in the following paragraphs are brief, and the detailed discussion of the different topics can be obtained in PLAXIS Manual, Version 7 (Brinkgreve and Vermeer, 1998).

Graphical input of geometry models. The input of soil layers, structures, construction stages, loads and boundary condition is based on convenient drawing procedures (CAD), which allows a detail and accurate modelling of real situations to be achieved. From this geometry model a finite element mesh is automatically generated.

Automatic mesh generation. PLAXIS allows for fully automatic generation of unstructured finite element meshes with options for global and local mesh refinement. The mesh generator is a special version of the Triangle generator, which was developed by Sepra (Brinkgreve and Vermeer, 1998).

High-order elements. High order elements are available to enable a smooth distribution of stresses in the soil and accurate prediction of failure loads. In addition to the quadratic 6-node triangular elements, 15-node cubic strain triangles are available which perform extremely well in axis-symmetric analyses.

Beams. Special beam elements are used to model the bending of retaining walls, tunnel linings and other slender structures, The behaviour of the elements is defined using its flexural rigidity, normal stiffness and ultimate bending moment. A plastic hinge may develop for elasto-plastic beams, as soon as the ultimate moment is mobilized. Beams may be used together with interfaces to perform highly realistic analyses of a large range of geotechnical structures.

Interfaces. These joint elements are needed for calculation involving soil-structure interaction. They may be used to simulate the thin zone of intensely shearing material at contact of footings, piles, geotextiles, retaining walls, etc. Values of interface friction angle and adhesion, which are not necessarily the same as the friction angle and cohesion of the surrounding soil, may be assigned to these elements.

Geotextiles. Geotextiles and geogrids are often used in practice for the construction of reinforced embankments or retaining soil structures. They can be simulated in PLAXIS by the use of special tension elements. It is often convenient to combine these elements with interfaces to model the interaction with the surrounding soil.

Tunnels. PLAXIS offer a convenient option to create circular and non-circular tunnels composed of arcs. Beams and interfaces may be added to model the tunnel lining and the interaction with the surrounding soil. Fully iso-parametric elements are used to model the curved boundaries within the mesh. Different practical methods are implemented to analyze the deformations that occur due to the construction of the tunnel.

Mohr-Coulomb model. This robust and simple non-linear model is based on soil parameters that are known in most practical situations. Not all non-linear features of soil behaviour are included in this model, however, it may be used to compute realistic ultimate loads for circular footings, short piles, etc. It may also be used to calculate a safety factor using a 'phi-c reduction' approach. This model was used in the analysis phase of the thesis.

Advanced soil models. PLAXIS offers a variety of soil models in addition to the Mohr-Coulomb model. To accurately analyze the logarithmic compression behaviour of normally consolidated soft soil, a Cam-Clay type model is available. This is referred to as the Soft Soil Model. An improved version of this model also includes the modelling of secondary compression (creep). For stiffer soils, such as Overcosolidated clays and sand, an elasto-plastic type of hyperbolic model is available, which is called the Hardening Soil Model.

Steady state pore pressure. Two alternative approaches exist for the generation of steady-state pore pressures. Complex pore pressure distributions may be generated on the basis of two-dimensional ground water flow analysis. As an alternative for simpler situation, multi-linear pore pressure distributions can be directly generated on the basis of phreatic lines.

Excess pore pressures. PLAXIS distinguishes between drained and undrained soils to model permeable sand as well as almost impermeable clays. Excess pore pressure is computed during plastic calculations when undrained soil layers are subjected to loads. Undrained loading situation is often decisive for stability of geotechnical structures. In cases of insufficient stability, intermediate consolidation periods have to be introduced to reduce the excess pore pressures.

Automatic load stepping. PLAXIS can be run in automatic step-size and automatic selection mode. This avoids the need for users to select suitable load increments for plastic calculations by themselves and it guarantees an efficient and robust calculation process.

Arc-length control. This feature enables accurate computations of collapsed loads and failure mechanisms to be carried out. In conventional load-controlled calculations the iterative procedure breaks down as soon as the load is increased beyond the peak load. With are length control, the applied load is scaled down to capture the peak loads and any residual loads.

Stage construction. It is possible to simulate construction and excavation processes by activating and deactivating clusters of elements. This procedure allows for a realistic assessment of stresses and displacements as cause, for example, by the construction of an earth dam or an excavation of a deep basement.

Updated Lagrangian analysis. Using this option the finite element mesh is continuously updated during the calculation. For some situations, a conventional small strain analysis may show a significant change of geometry. In these situation it is advisable to perform a more accurate Updated Lagrangian calculation, which is called the Updated Mesh Analysis in PLAXIS.

Consolidation. The decay of excess pore pressures with time can be computed in a consolidation analysis. A consolidation analysis requires the input of permeability coefficients in the various soil layers. Automatic time stepping procedures make the analysis robust and easy to use.

Safety factors. The factor of safety is usually defined as the ratio of the failure load to the working load. This definition is suitable for foundation of structures, but not for embankment and sheet piled walls. For this latter type of structure, it is more appropriate to use the soil mechanics definition of a safety factor, which is the ratio of the available shear strength to the minimum shear strength needed for equilibrium. PLAXIS can be used to compute this factor of safety using a' phi-c reduction' procedure.

Presentation of results. The PLAXIS postprocessor has enhanced graphical features for displaying computational results. Exact values of displacements, stresses and structural forces can be obtained from the output tables. Plots and tables can be sent to output devices or to the Windows clipboard to export them to other software.

Stress Paths. A special toll is available for drawing load displacement curves, stress paths and stress-strain diagrams. Particularly the visualization of stress paths provides a valuable insight into local soil behaviour and enables a detailed analysis of the results of a PLAXIS calculation.

6.1.3 Theories and Numerical Background in the Development of PLAXIS

Following are the brief discussion of the background theories and numerical tool on which PLAXIS is based. These contain deformation theory, ground water flow theory and consolidation theory. The corresponding finite element formulations are also discussed.

Deformation theory

The basic equations for the static deformation of a soil body are formulated within the framework of continuum mechanics. A restriction is made in the sense that deformations are considered to be small. This enables a formulation with reference to the original undeformed geometry. The continuum description is discreted according to the finite element method. The static equilibrium of a continuum can be formulated as:

$$L^T\sigma + P = 0 \qquad 6.1$$

This equation relates the spatial derivatives of the six stress components, assembled in vector σ, to the three components of the body forces, assembled in vector p. L^T is the transpose of a differential operator, defined as:

$$L^T = \begin{vmatrix} \frac{\partial}{\partial x} & 0 & 0 & \frac{\partial}{\partial y} & 0 & \frac{\partial}{\partial z} \\ 0 & \frac{\partial}{\partial y} & 0 & \frac{\partial}{\partial x} & \frac{\partial}{\partial z} & 0 \\ 0 & 0 & \frac{\partial}{\partial z} & 0 & \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{vmatrix} \qquad 6.2$$

In addition to the equilibrium, the kinematics relation can be formulated as:

$$\varepsilon = L^T u \qquad 6.3$$

This equation expresses the six strain components, assembled in vector ε, as the spatial derivatives of the three displacements, assembled in vector u, using the previously defined differential operator L^T. The link between Eq. (6.1) and (6.3) is formed by a constitutive relation (relation between rate if stress and strain) representing the material behaviour. The basic constitutive relation is as follows:

$$\sigma = M \ \varepsilon \qquad 6.4$$

The combination of Eq. (6.1), (6.3) and (6.4) would lead to a second-order partial differential equation in the displacements u. However, instead of a direct combination, the equilibrium equation is reformulated in a weak form according to Galerkin's variation principle:

$$\int \delta u^T (L^T \sigma + p)\, dV = 0 \qquad 6.5$$

In this formulation δu represents a kinematically admissible variation of displacements.

Groundwater

Flow in a porous medium can be described by Darcy's law. Considering flow in a vertical x-y plane the following equation apply:

$$q_x = -k_x \frac{\partial \varphi}{\partial x} \qquad 6.6$$

$$q_y = -k_y \frac{\partial \varphi}{\partial y} \qquad 6.7$$

The equations express that the specific discharges, q, follows from the permeability, k, and the gradient of the groundwater head. The head, φ, is defined as follows:

$$\varphi = y - \frac{p}{\gamma_w} \qquad 6.8$$

where y is the vertical position, p is the stress in the pore fluid (negative for pressure) and γ_w is the unit weight of the pore fluid. For steady flow the continuity condition applies:

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \qquad 6.9$$

Consolidation theory

The governing equation of consolidation which are used in PLAXIS follow the Biot's theory (Biot, 1956). Darcy's law for fluid flow and elastic behavior of the soil skeleton are also assumed. The formulation is based on small strain theory. According to Terzaghi's principle, stresses are divided into effective stresses and pore pressures:

$$\sigma = \sigma' + m\,(p_{steady} + p_{excess}) \qquad 6.10$$

where

$$\sigma \quad = (\sigma_{xx}\sigma_{yy}\sigma_{zz}\sigma_{xy}\sigma_{yz}\sigma_{zx})^{T} \quad \text{———} \quad 6.11$$

$$\sigma \quad = (111000)^{T} \quad \text{———} \quad 6.12$$

and σ is the vector with total stresses, σ' in the vector containing the effective stresses, p_{excess} is the excess pore pressure and m is containing unity terms for normal stress components and zero terms for the shear stress components. The steady state solution at the end of consolidation process is denoted as p_{steady}. Within PLAXIS, p_{steady} is defined as:

$$p_{steady} \quad = \sum - M_{weight}\, p_{input} \quad \text{———} \quad 6.13$$

where : p_{input} is the pore pressure generated in the input program based phreatic lines or on ground water flow calculation. Note that within PLAXIS compressive stresses are considered to be negative; this applies to effective stresses as well as to pore pressures. In fact it would be more appropriate to refer to p_{excess} and p_{steady} as pore stresses, rather than pressure. However, the term pore pressure is retained, although it is positive for tension.

6.1.4 Soil Models and Model Parameters

The soil models and the corresponding soil parameters used in PLAXIS program are discussed in the following paragraphs. For quick evaluation of the required model parameters, Table 6.1 tabulates the soil models and their required parameters.

Linear Elastic Model. This model represents Hook's law of isotropic linear elasticity. The model involves two elastic stiffness parameters, namely Young's modulus, E, and Poisson's ratio.

Mohr-Coulomb Model (Perfect-Plasticity). Plasticity is associated with the development of irreversible strains. In order to evaluate whether or not plasticity occurs in a calculation, a field function, f, is introduced as a function of stress and strain, A yield function can often be presented as a surface in principal stress space. A yield model is a constitutive model with a fixed yield surface, i.e., and a yield surface that is fully defined by model parameters and not effected by (plastic) straining. For stress states represented by points within the yield surface, the behaviour is purely elastic and all strains are reversible.

This well-known model is used as a first approximation of soil behaviour in general. The models involves five parameters namely Young's modulus (E), Poisson's ratio (ν), the cohesion (c), the friction angle (Ø), and the dilatancy angle (ψ)

Soft Soil Model (SSM). This is a cam-clay type model, which can be used to simulate the behaviour of soft soil like normally consolidated clays and peat. This was proposed by Vermeer and Brinkgreve (1998). The model performs best in situations of primary compression. The SSM requires the following material constants:

λ^* = The modified compression index taken from (ε_v-ln p) plot. This constant is not the same with the intrinsic compression index as defined by BURLAND (1990)

κ^* = Modified swelling index (slope of swelling line of above-mentioned (ε_v - ln p) plot.

C = Cohesion

Ø = Friction angle

Ψ = Dilatancy angle

Soft Soil Creep Model. This is a second order model formulated on the framework of visco-plasticity. The model can be used to simulate the time-dependent behaviour of soft soils. In addition to the five basic material constants use in Soft Soil Model, μ, which is called modified creep index, is added to account for creep effect

Hardening soil Model. This is an elasto-plastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. This second order model can be used to simulate the behaviour of stiffer soils, such as Overcosolidated clays, sand and gravel. Some parameters used in this are the same from those used in Mohr-Coulomb Model.

Table 6.1 Soil Models and Model Parameters Used in PLAXIS Program

Catergory	Index/Physical		Strength			Compressibility and Consolidation						Permeability	
	γ_{wet}	γ_{dry}	c'	ϕ'	ψ	E'_{ref}	ν'	λ^*	κ^*	OCR	m	k_v	k_h
Name of Symbol	Wet unit weight	Dry unit weight	Soil cohesion (CID test)	Friction angle (CID test)	Dilatance angle	Drained Modulus (CID,50% cut)	Poisson's ratio	See legend	see legend	Overcon-solidation ratio	Power for stress lev depen-dency	Coefficient of vertical permea-bility	Coefficient of holizontal permea-bility
Unit	kN/m^3	kN/m^3	kPa	Degrees	Degrees	kPa						m/day	m/day
Model Symbol													
SSM	R	R	R	R	R	R	R	R	R	R		R	R
HSM	R	R	R	R	R	R	R			R		R	R
MCM	R	R	R	R	R	R	R			R	R	R	R
SSCM+	R	R	R	R	R	R	R	R	R	R		R	R
EM	R	R				R	R					R	R

Legend: SSM=Soft soil Model

HSM=Hardening Soil Model

MCM=Mohr Coulomb Model

SSCM=Soft Soil Creep Model

EM=Elastic Model

λ^*=CR/2.303

κ^*=RR/2.303

R=Required

(+)=Additional parameter is μ^* ,which is called the modified creep index

These are failure parameters, c, Ø and ψ. Other parameters for soil stiffness are as follows:

E^{ref}_{50} = Secant stiffness in standard drained triaxial test. (The secant cuts at q of 50% q_{ult}).

E^{ref}_{oed} = Tangent stiffness for primary oedometer loading.

M = Power for stress-level dependency of stiffness. M= 1.0 for soft clay, and 0.5 for sand and silts (Janbu, 1963).

6.2 General

This chapter contains the numerical predictions obtained using the finite element program to model field behaviour. Six models of different lengths of cement column were simulated using the program. There were 5m, 7m, 9m, 11m, 13m, and 15m length, which were the same lengths of the pile tests in the field. Figure 6.2 shows an example of the model set up in the PLAXIS program. The parameters of the soil needed for running the program are obtained from the laboratory test result in Chapter 5 and also from back calculation until the calculated ultimate pile load agreed with the full scale test data.

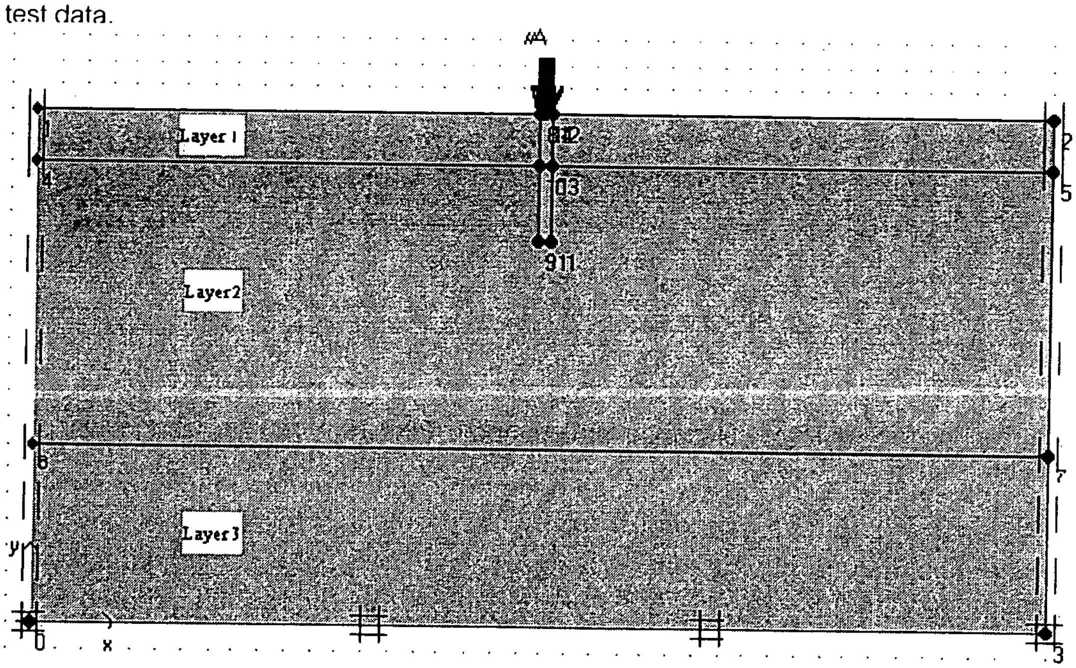


Figure 6.2 an example of the model in PLAXIS program

The mesh used in the model is shown in Figure 6.3. Load and settlement curves for each pile length were derived using the program to calculate the ultimate pile load for comparison with the ultimate load of the piles obtained from the field-testing.

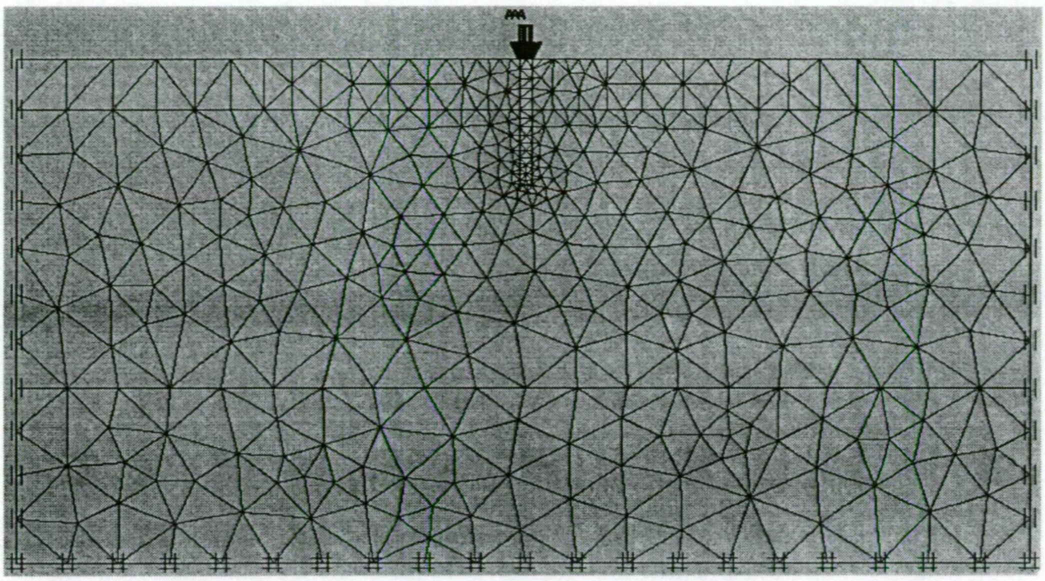


Figure 6.3 an example of division into multi small mesh

6.3 Material behaviour models

In general, the stress strain relationship of finite element formulation is expressed as follows:

$$\delta\sigma_{ij} = C\delta\varepsilon_{ij} \qquad \text{----------6.14}$$

Where $\delta\sigma_{ij}$ and $\delta\varepsilon_{ij}$ are stress and strain increment tensors, respectively, and C is constitutive matrix that depends on the model used in the analysis.

6.4 Weathered clay

At the construction site, the upper layer 0m to 2m depth consists of the weathered crust, which is heavily overconsolidated. Elastic, perfectly plastic model with constant value of

Poisson's ratio has been used for this soil (Chai 1992). In this study, elastic perfectly plastic Mohr-Coulomb model was also used for this type of soil. The strength parameters were obtained from the existing test data on Bangkok clay (Balasubramaniam et al, 1978). The modulus of elasticity was obtained from finite element back-analysis by assuming the Poisson's ratio of 0.25 (Chai, 1992). The value of the modulus of elasticity was found by back-calculation until the ultimate load is consistent with the field test data. The modulus of elasticity and the strength parameters are tabulated in Table 6.2.

6.5 Soft clay

The Cam Clay model has been widely used for representing stress-strain relationship of the soft Bangkok clay, which is normally consolidated and lightly overconsolidated. In PLAXIS program, Cam Clay type model, which is modified Cam Clay Model and Soft Soil model, is available. For slightly overconsolidated clay (OCR < 2), the Soft Soil model has been suggested to use rather than modified Cam Clay model (Brinkgreve and Vermeer, 1998). Moreover, the Mohr-Coulomb failure criteria can represent the behavior of lightly overconsolidated clay that has apparent cohesion intercept. In this study, the Soft Soil Model is used for predicting behavior of soft clay layer as it provided more reasonable solutions than the other models.

The elastic soil model is used for predicting the behavior of the medium stiff clay layer. There were no direct measurements of the moduli of the soils at the site, however, the undrained modulus, E_u, can be given in terms of the uncorrected field vane shear strength, $C_{u(VST)}$, as $E_u = \alpha\ C_{u(VST)}$. The value of α for Bangkok clay lies between 70 and 250 (Balasubramaniam and Brenner, 1981). Bergado (1990) reported that α of 150 is the best fitted for the settlement prediction of Bangna-Bangpakong Highway

For the purposes of numerical modeling, the foundation soil has been divided into 3 layers. The first layer is 0.0m to 2.0m weathered clay, the second one is 2.0m to 14.0m soft clay and the last one is 14.0m to 20.0m stiff clay. The Morh-Coulomb Model parameters were determined based on the one-dimensional consolidation tests. The friction angle of the soft clay layer was found from CD Triaxial test to be 18degree. The apparent cohesion intercept in this study is obtained from back calculation.

The permeability of clays is one of most difficult parameters to determine. The representative vertical permeability used in this study was based on the oedometer test results, as shown in Chapter 5.

6.6 Soil-cement piles

The Mohr-Coulomb model was selected to simulate the behavior of the cement column. The coefficient of permeability ratio k_{pile}/k_{soil} = 30 was used based on previous study recommendations (Lorenzo, 2001). The elastic modulus E_{col} was obtained from back calculation until the ultimate pile load was found to be in agreement with the full-scale test data.

6.7 Parameters in the model

There are many parameters that influence the ultimate load of cement columns including Young's modulus (E), Cohesion (C), Internal friction angle (ϕ'), Coefficient of permeability (K) and Poisson's ratio (ν). In this study, Mohr-Coulomb model (MC Model) was chosen to represent the Bangkok clay. The most influential parameter for the MC Model is Young's Modulus and internal friction angle.

Three different values of Young's Modulus were used to represent the variation of the Bangkok clay with depth. The values adapted were $100C_{soil}$, $200C_{soil}$ and $250C_{soil}$ (C_{soil} is the shear strength of soil from the laboratory). For the three different values of Young's Modulus of cement column are from $10C_{col}$, $20C_{col}$ and $30C_{col}$ (C_{col} is the unconfined compressive strength from the laboratory). These are show in Table 6.2. Also short term and long term conditions are considered. Table 6.2 shows the parameters, which were used as input into the program. All data is derived from the field data test provided earlier in chapter 5.

6.8 The comparison between the field result and the PLAXIS analysis

The analytical results from the PLAXIS program are illustrated in Figure 6.4 to Figure 6.15. The summarised comparison of the field data and the analytical results are provided in Table 6.3. It is apparent that the average ultimate load of cement columns in the field of the column length 5m, 7m, and 9m is very similar with the ultimate load in the program in the same length which using long term condition and low Young's modulus. The ultimate load of column 11m length in the field is similar with using medium Young's modulus in short term condition. However if we see in each column of 11m length, it can be seen that the ultimate load of the first column (9.20tons) is fitting with using low Young's modulus in short term condition. While the ultimate load of the second column (11.50tons) is similar with using high Young's modulus in short term condition. This result can tell that the soil is non-homogenous. The soil parameters even in the same area, sometime they are very different. Also another reason is when cement columns are constructed in the field, it is impossible to control Young's modulus and cross section area to be the same for entire column. It is different from modeling in the program, which can be simulated in the same cross section area and Young's modulus for entire column. In the same reason, the ultimate load of column 13m length is quite different from the field and the program. In some parts of the column in the field were very weak, it made the field ultimate load quite low. The most similar ultimate load between from the field and the program is using low Young's modulus in short term condition. Also in column 15m length, using low Young's modulus in short term conditions results the most fitting ultimate load.

From the analysis, the parameters can be concluded in table 6.4. The most suitable parameters for Bangkok soil is using low Young's modulus in the PLAXIS program.

Using long term condition in the program for column length 5m, 7m and 9m, and using short term condition for column length 11m, 13m and 15m.

Table 6.2 Soil parameters used in the model

Depth	Type of Model	C'	ϕ'		E' (ton/sq.m)			Kv	γ_{wet}	γ_{dry}	λ	κ
(m)		ton/sq.m	short term	long term	low E'	medium E'	high E'	10^{-5} m/hour	ton/cu.m	ton/cu.m		
0-2	MC	1.86	0	13	186	372	465	2.08	1.71	1.177	-	-
2-13	SSM	0.8	0	14	80	160	200	0.208	1.54	0.84	0.17	0.034
13-20	MC	3.5	0	12	350	700	875	2.08	1.815	1.233	-	-
Cement Columns	MC	65	0	28	650	1300	1950	6.25	1.8	1.4	-	-

MC = Mohr-Coulomb Model

SSM = Soft soil Model

Table 6.3 Comparing the field ultimate load result with the PLAXIS analysis result

Column length (m)	Ultimate load (tons)						
	Low Young's modulus		Medium Young's modulus		High Young's modulus		In the field.
	Short term condition	Long term condition	Short term condition	Long term condition	Short term condition	Long term condition	
5	6.00	11.00	6.00	11.00	6.00	11.00	11.00
7	7.00	11.80	7.80	13.50	7.80	14.00	11.25
9	8.00	13.60	8.53	15.40	8.53	17.10	12.88
11	9.37	15.30	10.60	19.50	11.00	21.10	10.35
13	11.70	15.00	15.60	22.20	15.90	23.00	8.48
15	12.00	15.00	18.00	22.20	22.50	28.00	13.10

Table 6.4 The most suitable parameters of Bangkok clay

Deapth (m)	Type of Model	C' ton/sq.m	ϕ'		low E' ton/sq.m	Kv 10^{-5} m/hour	γ_{wet}	γ_{dry}	λ	κ
			short term	long term						
0-2	MC	1.86	0	13	186	2.08	1.71	1.177	-	-
2-13	SSM	0.8	0	14	80	0.208	1.54	0.84	0.17	0.034
13-20	MC	3.5	0	12	350	2.08	1.815	1.233	-	-
Cement Columns	MC	65	0	28	650	6.25	1.8	1.4	-	-

MC = Mohr-Coulomb Model

SSM = Soft soil Model

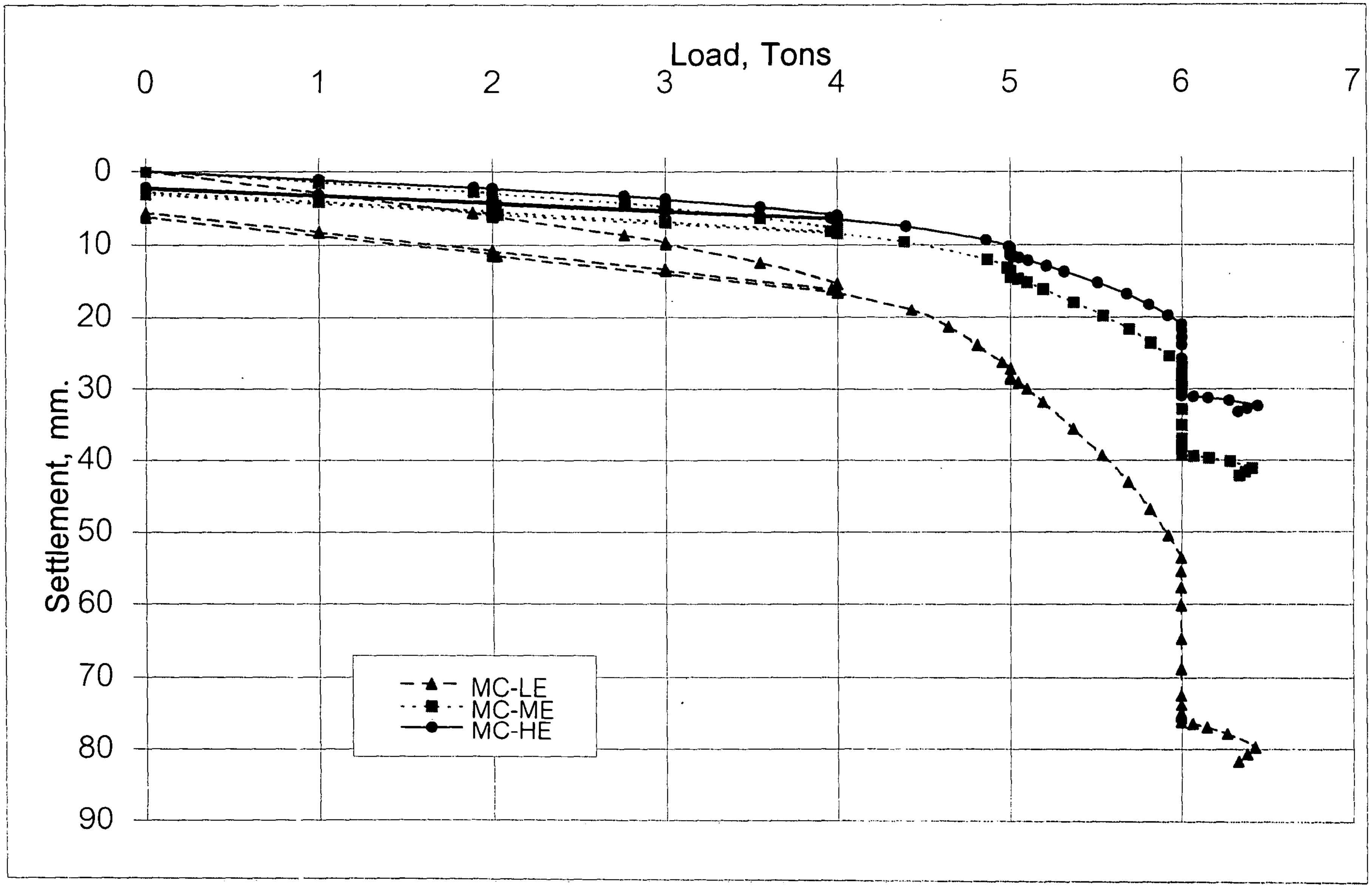


Figure 6.4 The comparison of three different E with short term condition (phi=0) in the model of column 5m length

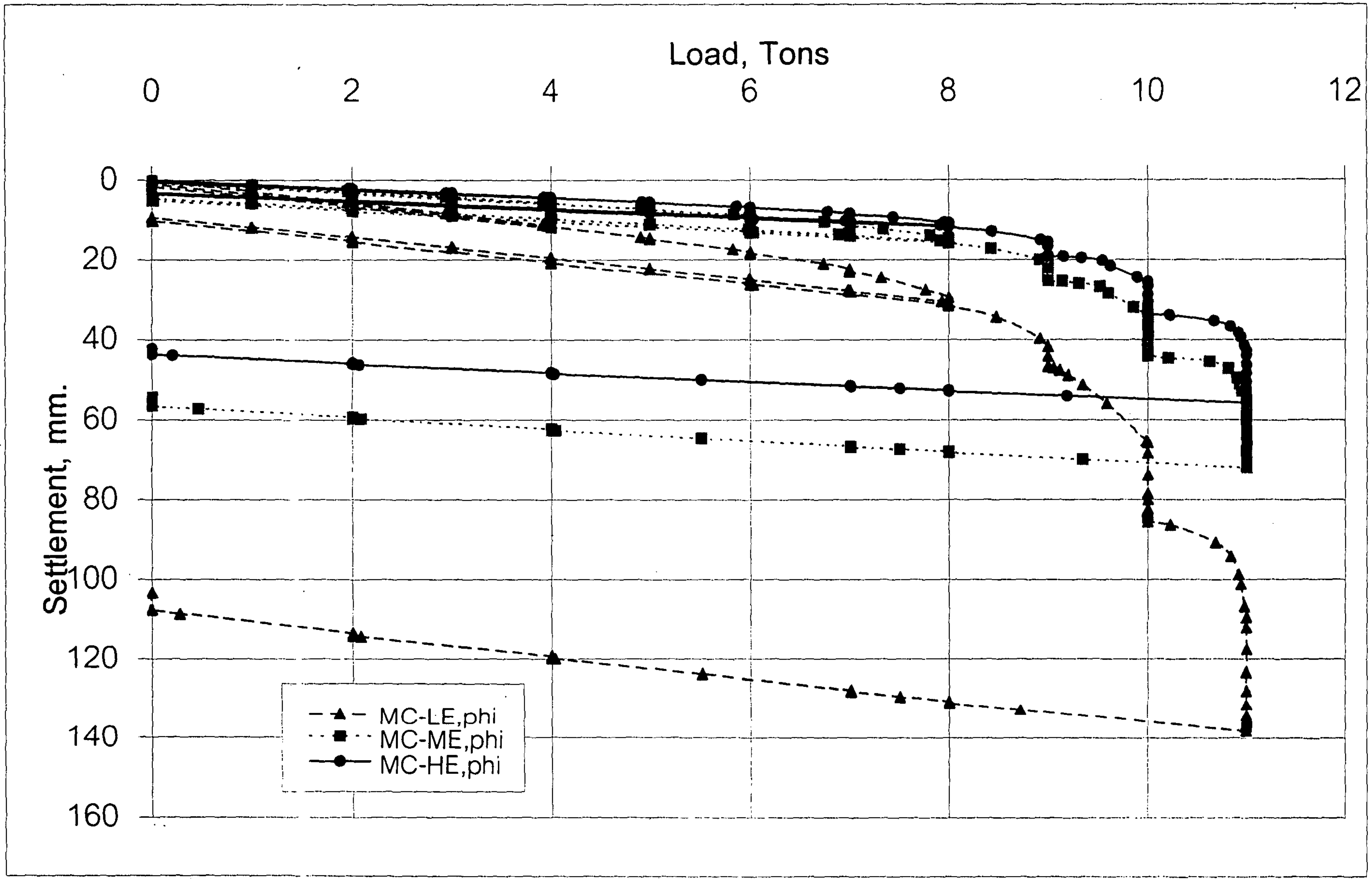


Figure 6.5 The comparison of three different E with long term condition in the model of column 5m length

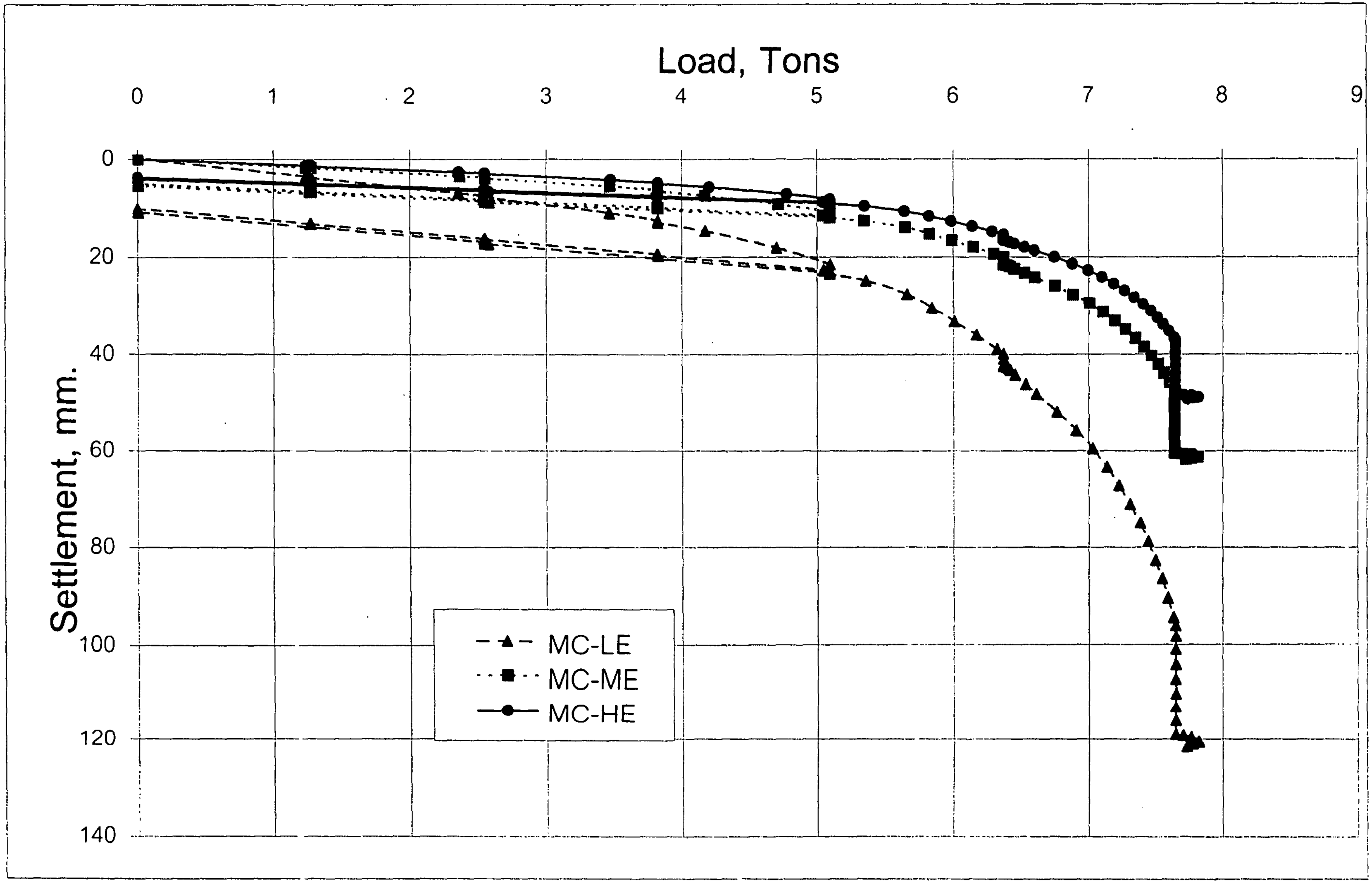


Figure 6.6 The comparison of three different E with shot term condition (phi=0) in the model of column 7m length

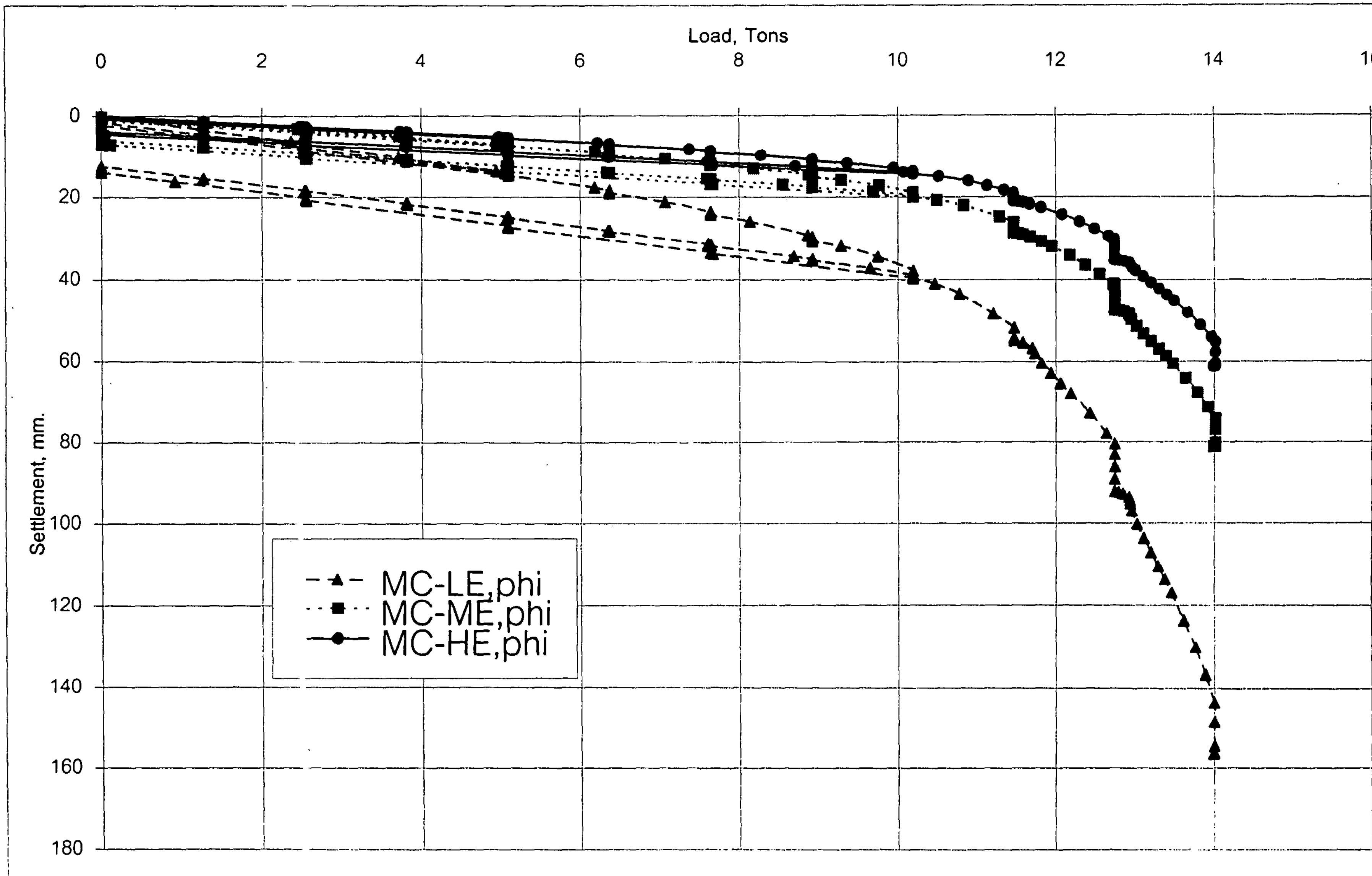


Figure 6.7 The comparison of three different E with long term condition in the model of column 7m length

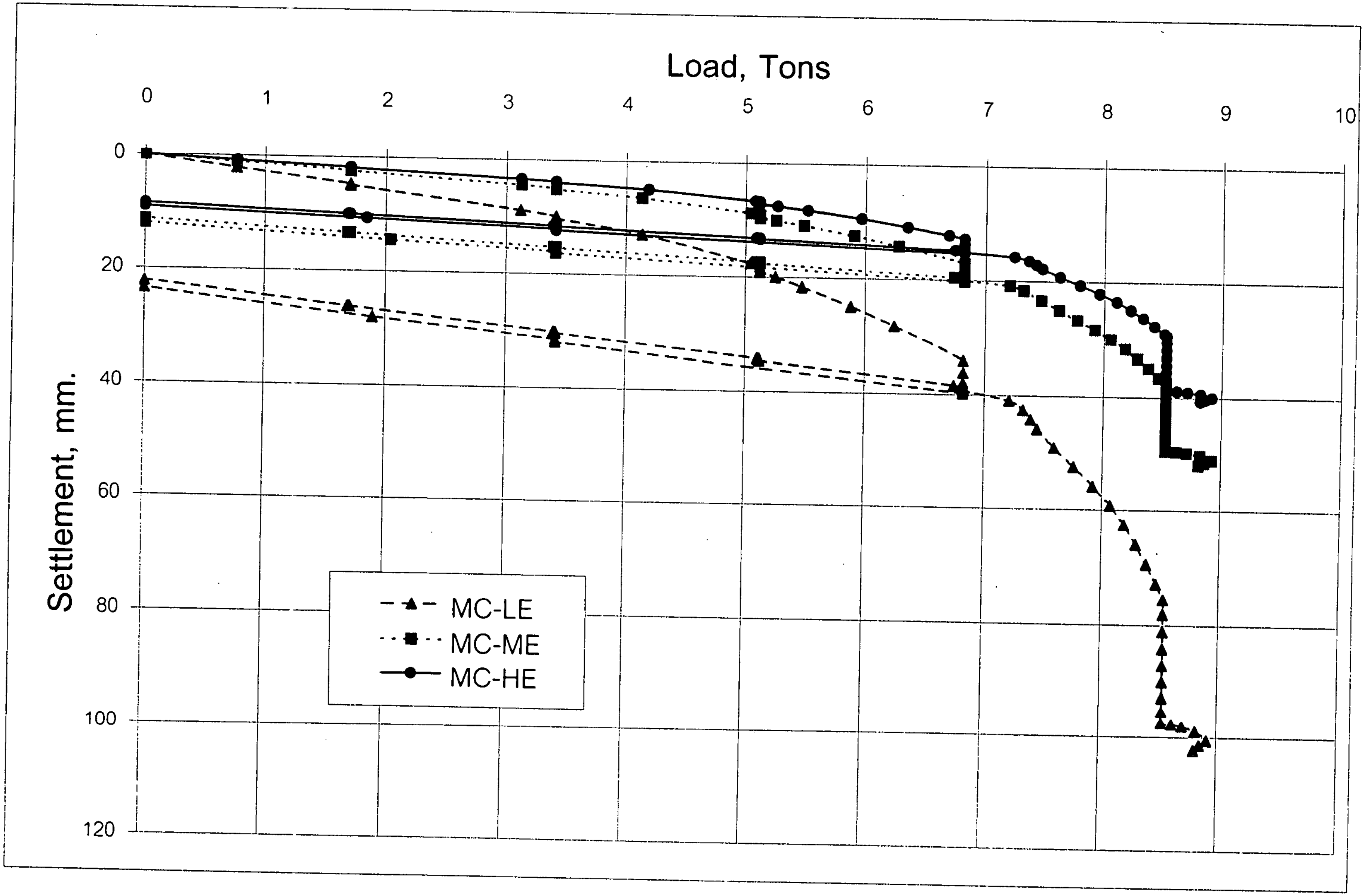


Figure 6.8 The comparison of three different E with shot term condition (phi=0) in the model of 9m length

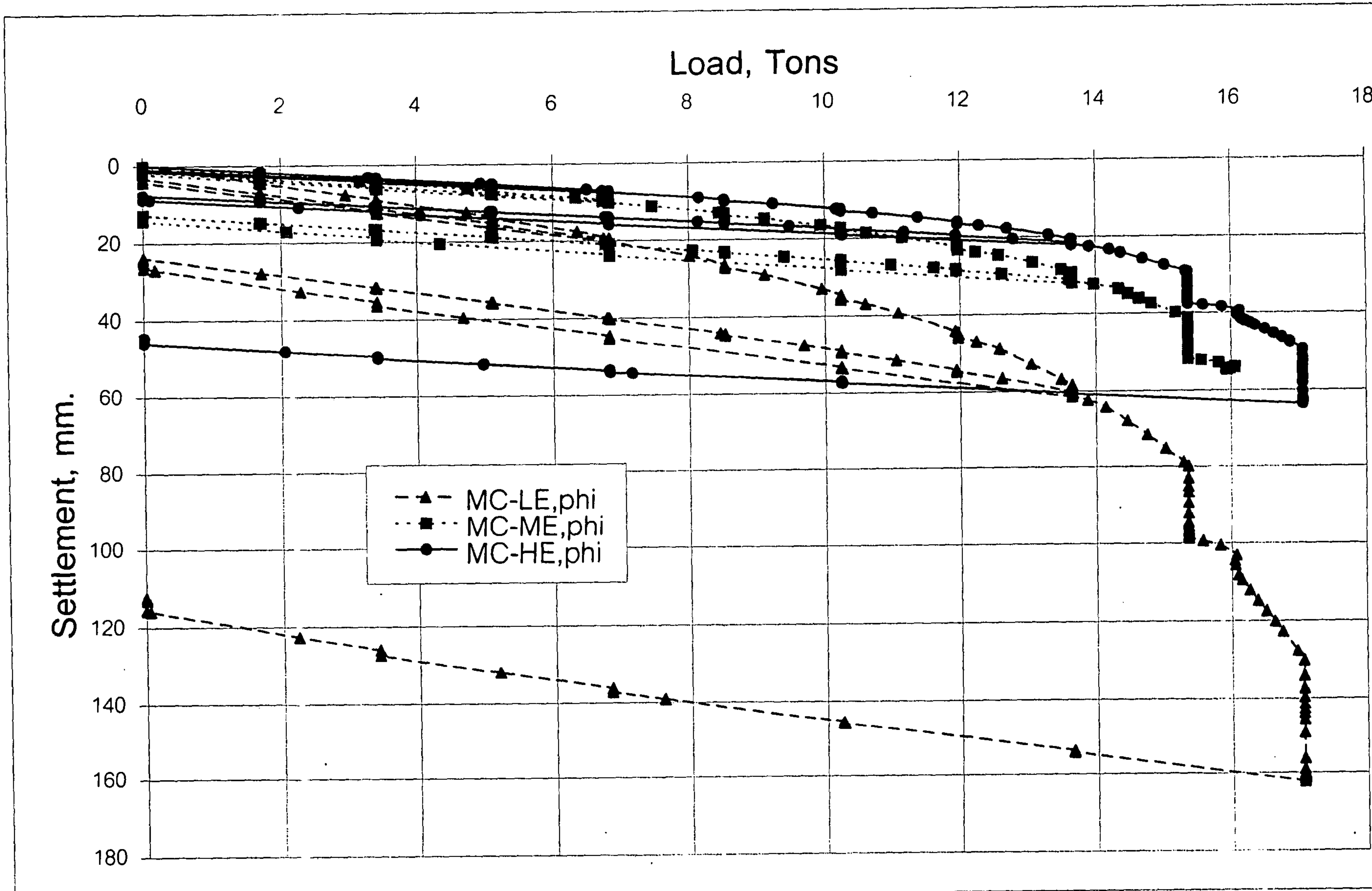


Figure 6.9 The comparison of three different E with long term condition in the model of column 9m length

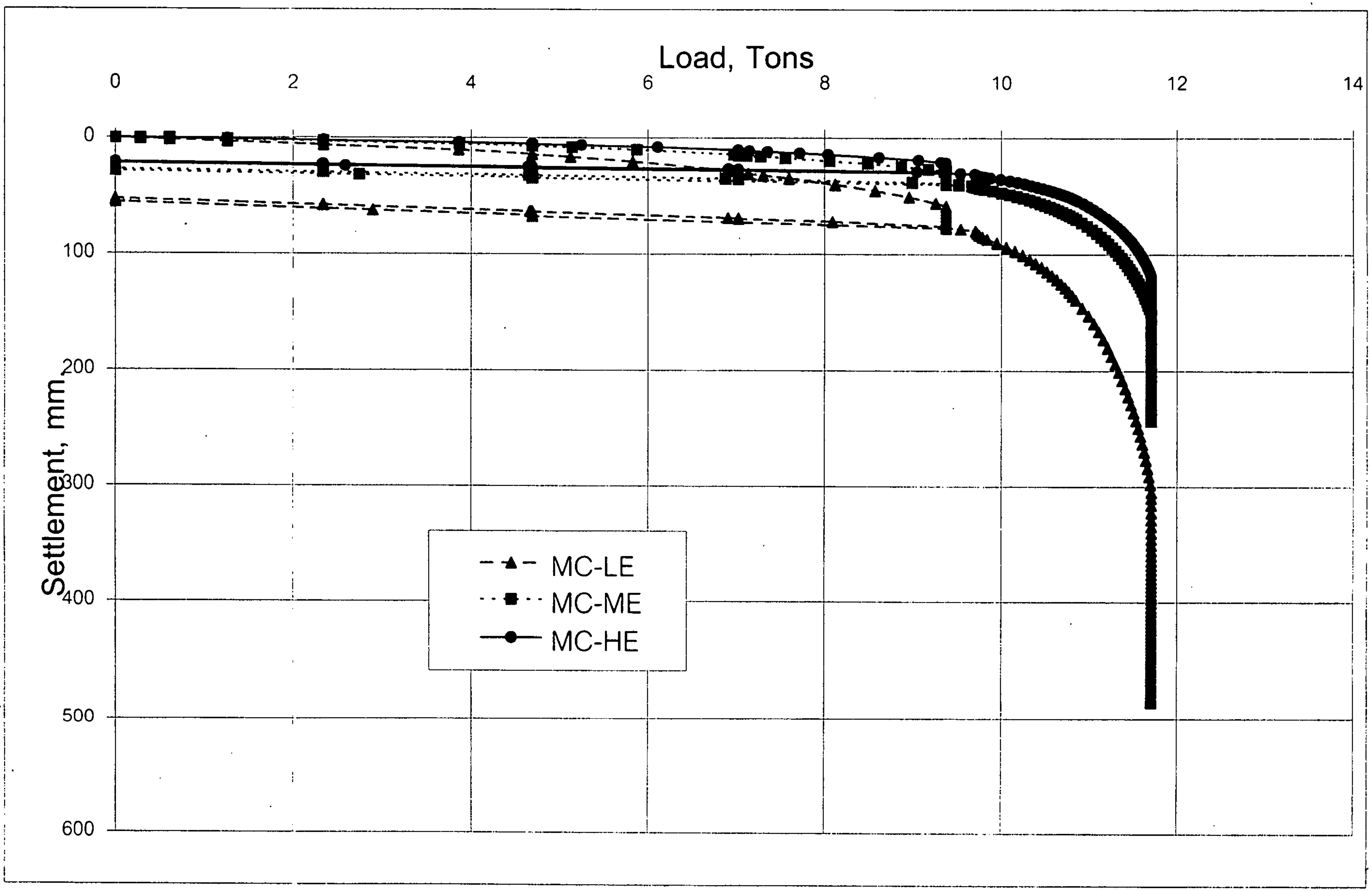


Figure 6.10 The comparison of three different E with shot term condition (phi=0) in the model of column 11m length

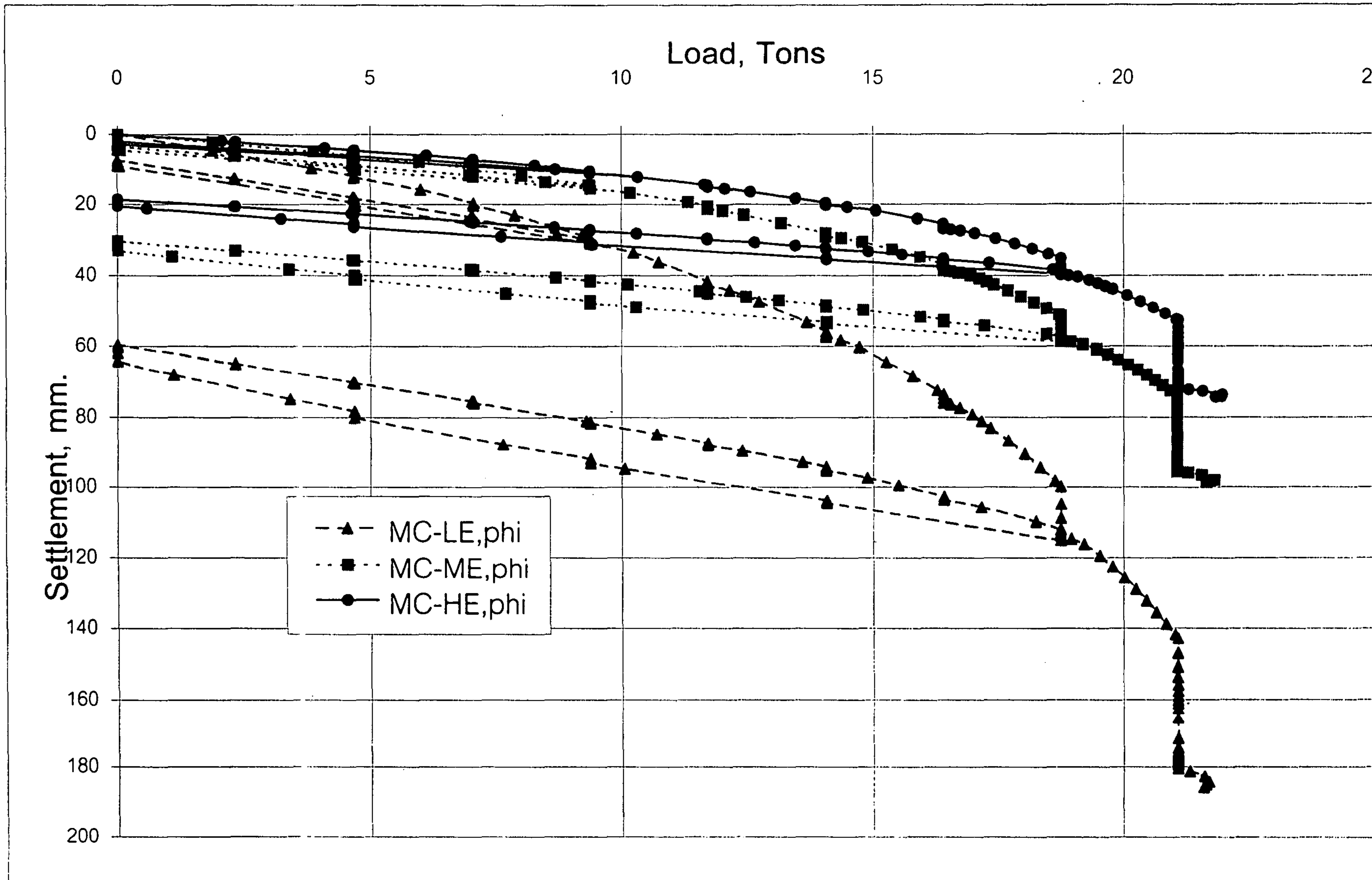


Figure 6.11 The comparison of three different E wiht long term condition in the model of column 11m length

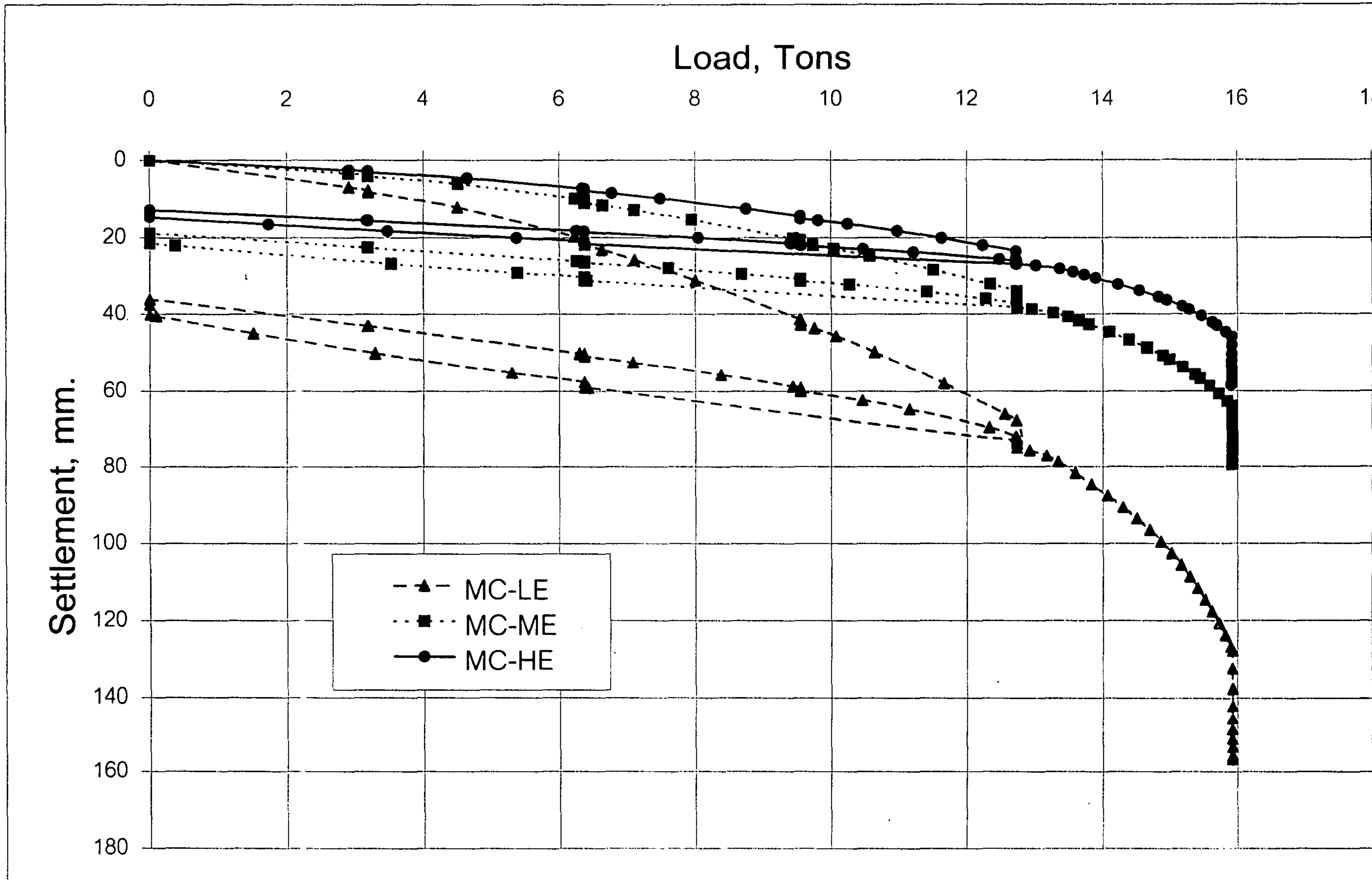


Figure 6.12 The comparison of three different E with shot term condition (phi=0) in the model of column 13m length

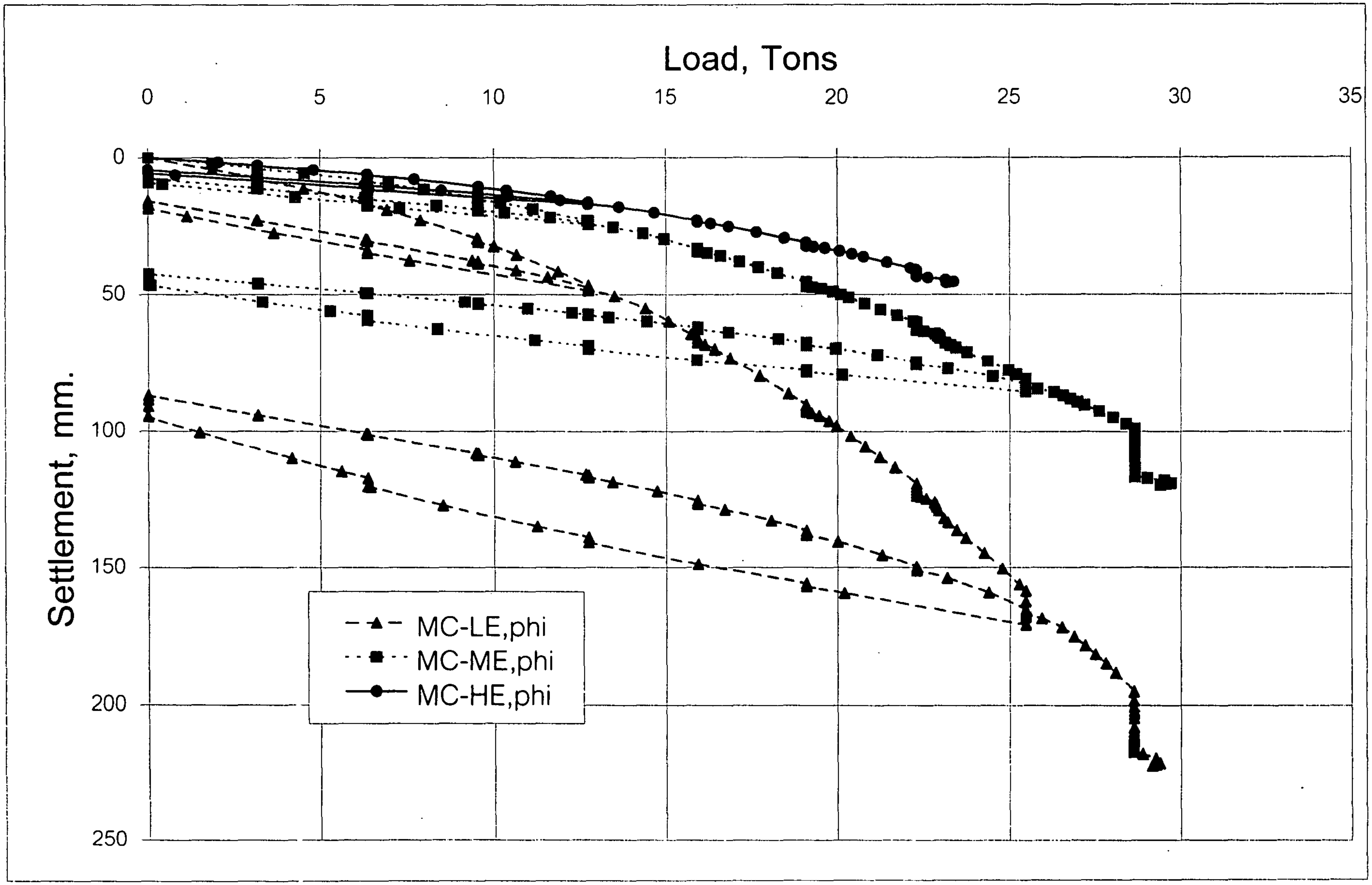


Figure 6.13 The comparison of three different E with long term condition in the model of column 13m length

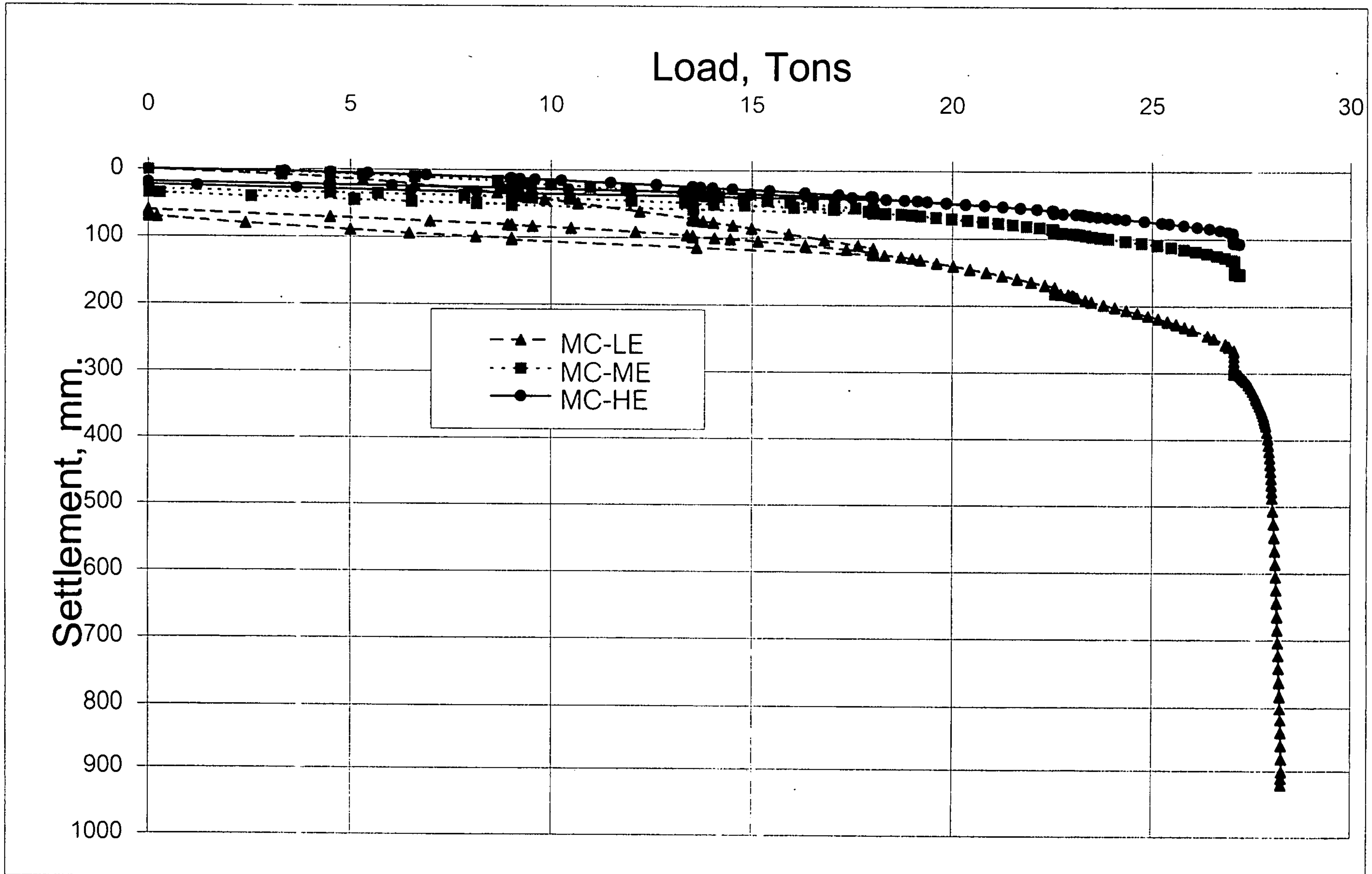


Figure 6.14 The comparison of three different E with shot term condition (phi=0) in the model of column 15m length

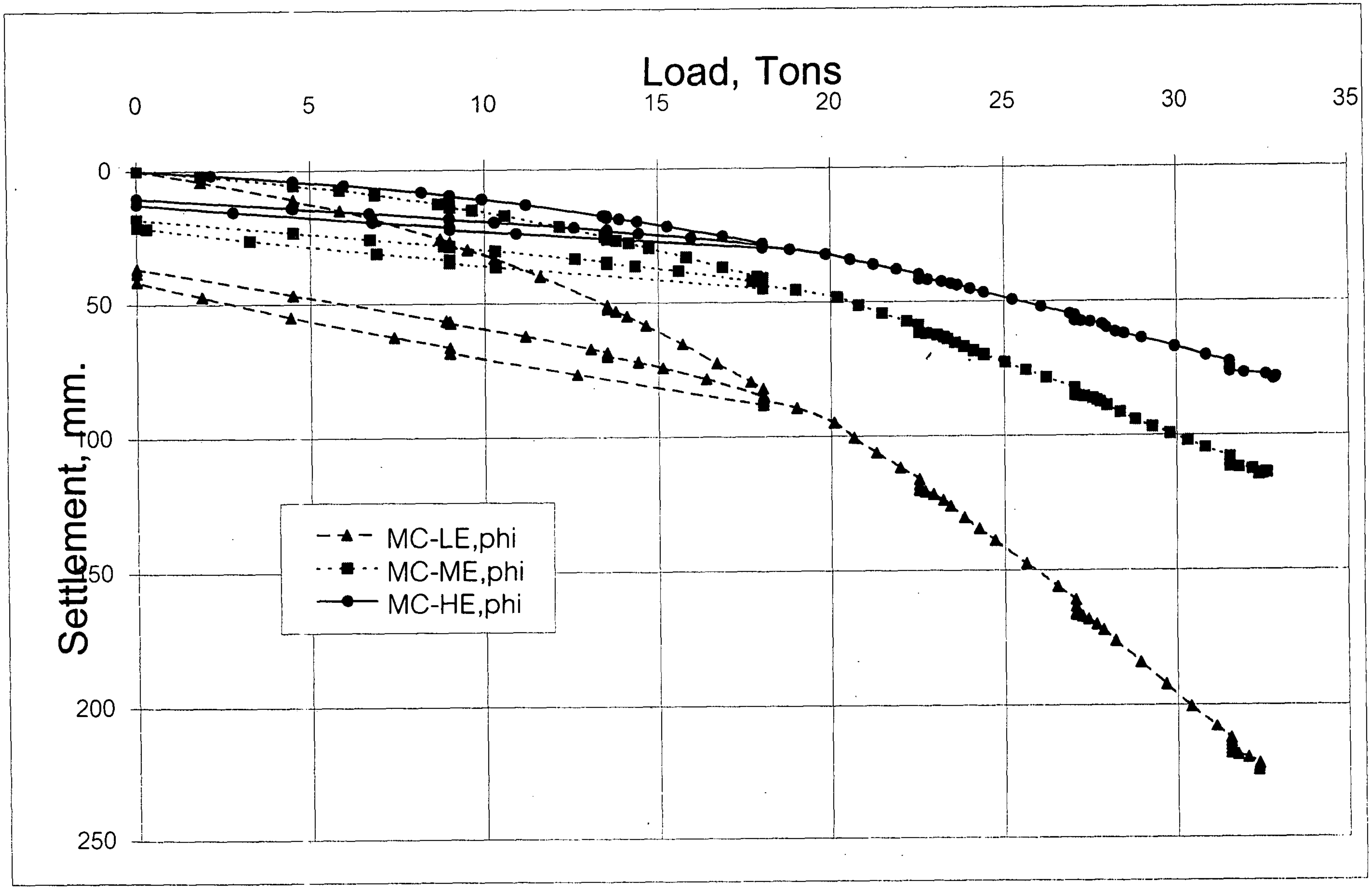


Figure 6.15 The comparison of three different E with long term condition in the model of column 15m length

Chapter 7

EFFECTS OF DESIGN PARAMETER

7.1 Undrained Shear Strength of an In Situ Soil (C_u^{Soil})

It is very difficult to evaluate a representative of an Undrained Shear Strength of an in situ soil in each soil layer because soils are non-homogenous. Also sometime soils are disturbed by samplers when they are excavated. These effect to precisely estimate the undrained shear strength.

7.2 The Change of Thickness of Soil Stratum

Generally the soil stratum thickness in the same area is very similar. However sometime ever it is in the same area, the soil stratum thickness is very different. If in this case occurs, it causes a mistake to evaluate a pile capacity. For example, at a site is showing 15m thick of soft clay from a bore hole. A designer designs the pile length of 15m to site the pile tip on a stiff clay layer. However at the position of installation, which far from the bore hole about 10m, has a 20m thick of soft clay layer as show in Figure 7.1. Thus the design value is not equal to the pile capacity. The pile capacity is less than the preliminary pile design load. Because if the pile tip sits on stiff clay, the end baring value will be greater than the pile tip sits on soft clay.

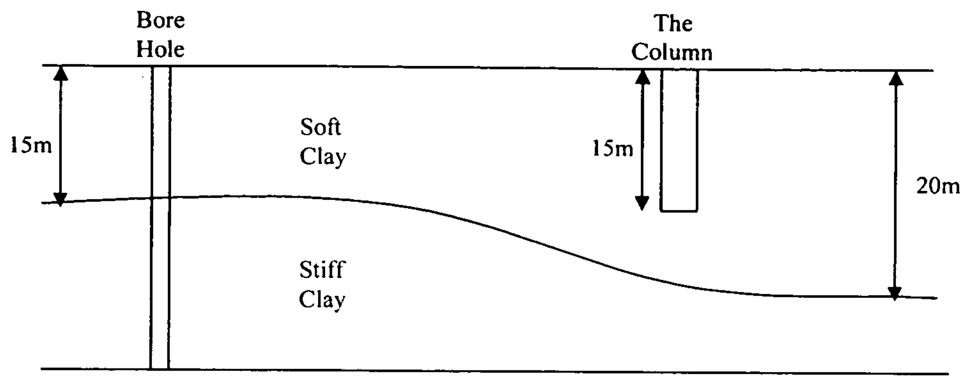


Figure 7.1 The change of thickness of soil stratum

7.3 Cross Section Area and Surface Area of Columns

These parameters are very difficult to control to be the same with every column. Even in the same column, cross section area is sometime different between column body. Sometime pile top bigger than pile tip. Sometime in the middle is the biggest. These depend on injection pressure. This pressure is usually difficult to control to be stable at preliminary design injection pressure all the time. Sometime it swings up and down. Resulting cross section area of the cement column is less or bigger than the design cross section area. Another reason is a non-homogeneous of soil. Even using the same pressure injects slurry through different soil layers, the cross section area is still different. It is because, different soil layers have different soil properties. Even in the same layer, just different depth, the soil has different properties. It results different cross section area in the same column.

7.4 Undrained Shear Strength of the Cement Column (C_u^{Col})

This is another parameter that can not control to be the same for the whole column. The reason is non-homogeneous of soil. Different soil physical properties cause different undrained shear strength of the cement column. Another reason is the difficulties of mix control in the field installation. The installation machine that available in Thailand now cannot provide the well mix columns. Resulting column strength is not equal for the whole column.

To determine the undrained shear strength of the cement column generally the average of unconfined compressive strength of cored sample is the representative for whole column. From this study, however, the average of unconfined compressive strength of cored samples is not suitable to be the representative cement column strength. From the result, the ultimate load of static pile load testing focus only on clashing columns. All most all of them are lower than the average of the cored sample strength. It means, the average of cored sample strength is not suitable to be the representative. This may cause a mistake in design. Thus it is better to use the 90% lower value of the cored sample strength to be the representative. The 90% lower value of the cored sample strength in this study is 456.60kPa, which it comes from the equation 7.1. It is shown below.

$$X_{90} = \overline{X} - 1.3 * SD \qquad \text{------------}7.1$$

Where X_{90} = the 90% lower value of the cored sample strength

$\overline{X}$ = the average of cored sample strength

SD = a standard deviation

Chapter 8

DESIGN PROCESS

This chapter provides the inexperienced designer with a step-by-step description of the major processes involved in the design of cement columns. The processes are summarised in Figure 8.1 and a brief description of each section is provided in this chapter.

Site Investigation

The proposed site is typically explored by wash-boring. Samples are obtained using thin walled piston samplers in soft clays. When the soil is too hard to sample by thin-walled piston, a split-barrel sampler is used to obtain samples. The Standard Penetration Test (SPT) is also carried out to obtain data on in situ soil strength simultaneously when sampling the soil by a split barrel sampler. Vane shear tests are also carried out to determine the shear strength in soft clay at shallow depth. These techniques are usually adequate to determine the soil profile. The full details of sampling procedures of soil and determining the soil profile are shown in Chapter 4 and Chapter 5 respectively.

Laboratory Investigation

The soil speciments which are sampled from the site are brought to the laboratory. These samples are undertaken for each soil layer to better evaluate the physical properties of the soft (Bangkok) clay at the site. The typical methods of testing are shown in Chapter 4 and include physical properties testing, oedometer testing and consolidation drained triaxial testing.

Mix design

Soil from the site is mixed in the laboratory with various proportions of cement using 1.1/1 w/c ratio. This w/c ratio is based on empiric to provide a mix that can be pumped. The various soil-cement mixes are cast into containers and cured up to 28days before testing in unconfined compression to determine the optimum mix design.

For soil-cement samples which are mixed with various proportions of cement from mix design are carried out by Unconfined compressive test to determine the strength and comparing with the preliminary design. These for determine the suitable proportion of cement in field installation. The strength of various proportions of cement in this study are shown in the table 8.1. These information are supplied by industry who installed the testing piles in the site. This thesis used the cement content of 200kg/m^3 which the average value was 1253.75kPa.

Table 8.1 The strength of various proportions of cement

Depth (m)	Curing (day)	Unconfined Compressive Strength (kPa)			
		Cement 125 (kg/m^3)	Cement 150 (kg/m^3)	Cement 200 (kg/m^3)	Cement 250 (kg/m^3)
3	28	884	1013	1240	1423
6	28	753	873	1030	1154
12	28	730	1026	1411	1654
15	28	880	1087	1334	1522
Average		811.75	999.75	1253.75	1438.25

Column Design

The failure mechanisms associated with cement column are essentially the same as these for concrete piles. They can be termed surrounding soil failure via skin friction being exceeded from Eq. 3.2-3.4 and column failure via crushed of soil-cement from Eq. 3.5. The soil properties for surrounding soil capacity and the material strength for column load capacity are from laboratory investigation. These two load capacity must be similar in magnitude for economical reasons. The details of cement column design methods are shown in Chapter 3.

Installation Test

Different pressure and rising speeds are tested to find out which pressure and rising speeds can create similar cement content and diameter with the preliminary design. This study used a rising speed of 9centimeters in 10seconds. The pressure of pre-jet with water was about 15,000kPa and the pressure to jet the cement slurry was 20,000kPa, which are shown in Chapter 4.

Pile Testing

Static pile load testing is carried out to determine pile capacity. If it is less than the preliminary design, the pile is cored and the cored samples returned to the laboratory to determine the representative strength. Selecting a new cement content and follow the chart through iteration until the pile capacity is good as or better than the preliminary design.

To predict new cement content is comparing between the unconfined compressive strength in the laboratory of the used cement content and the 90% lower value of the cement column strength in the field. This comparison provides the ratio to predict the 90% lower value of the column strength in the field of other cement content. For example, the cement content of 200kg/m^3 provided 1253.75kPa of the unconfined compressive strength in the laboratory and 456.58kPa of the 90% lower value of the column strength in the field. If using the cement content of 250kg/m^3, which provide 1438.25kPa of the unconfined compressive strength in the laboratory, it would provide 456.58*1438.25/1253.75 = 523.77kPa of the 90% lower value of the column strength in the field. From table 5.2, the ultimate soil design load of the columns7m length is 524.60kPa. It can be seen the similar value between the 90% lower value and the ultimate soil design load. This could tell that the cement content of 250kg/m^3 can use for creating 50cm diameter of a 7m length of cement column in this site.

Field Installation

After cement content, jet pressure and rising speed are determined to create a cement column with good as or better than the preliminary design. The cement columns in the project may be installed.

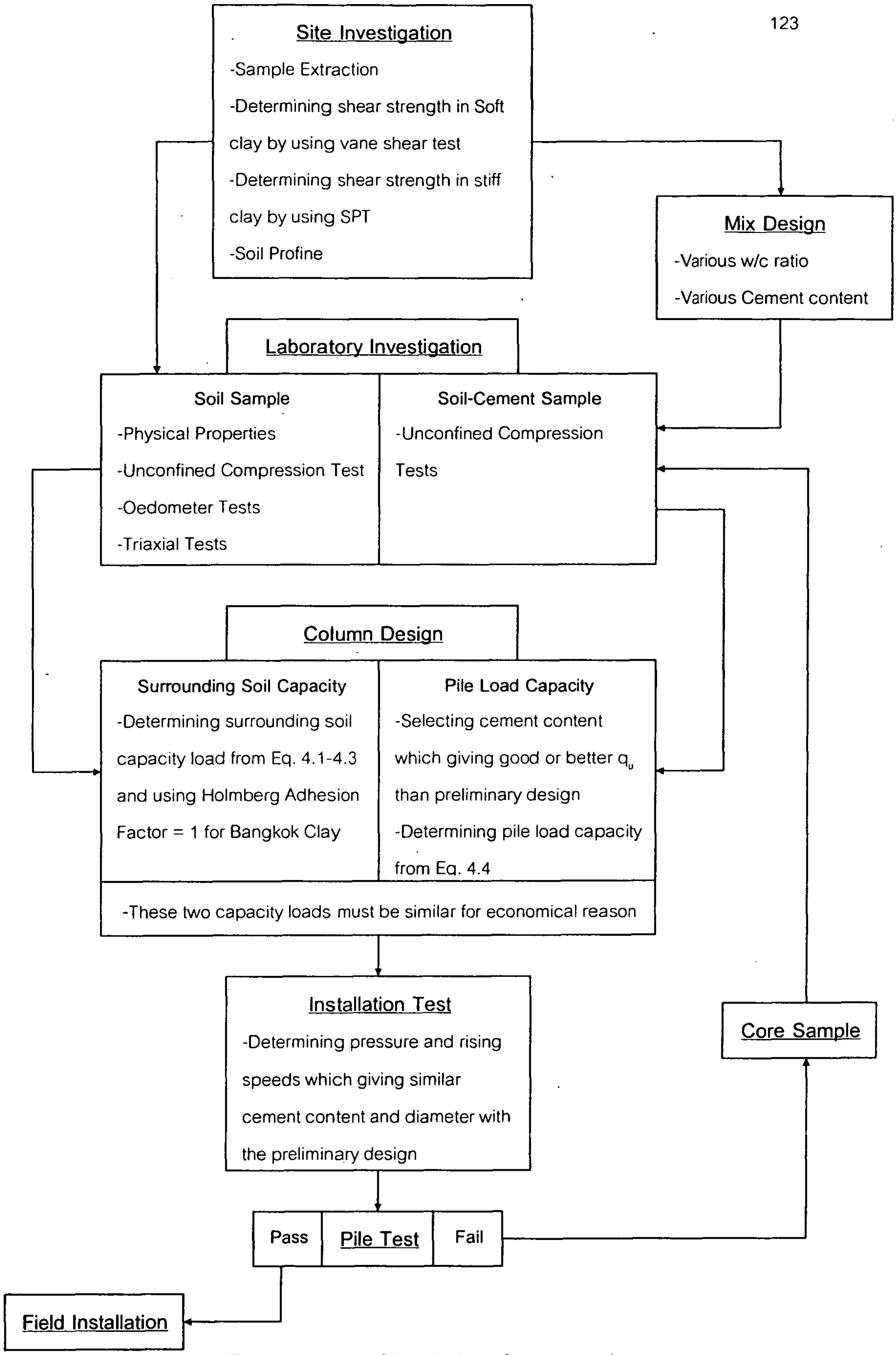


Figure 8.1 The processes of the design of cement columns

Chapter 9

CONCLUSIONS AND RECOMMEDATIONS

9.1 Conclusion

Field testing

An extensive field sampling program (one bore hole and four field vane shear testings) was undertaken to determine the in situ soil properties. The testing includes shear strength, soil classification, N-value, unit weight, Atterberg limit, specific gravity, field vane shear test, consolidation and consolidated drained triaxial test. Coring of the installed cement columns was undertaken to determine the unconfined compressive strength of the soil cement.

It was determined that the test site consisted of five soil layers from the ground down to 30m depth. The first soil is weathered clay, which it is about 2m thick. The second layer is a soft clay, extending from around 2m to 14m depth. The third layer is a stiff clay extending from about 14m to 20m depth. The fourth and the fifth layers are dense sand and hard clay representatively. The dense sand layer is about 6m thick extending from 20m to 26m depths and below that is the hard clay layer.

Laboratory testing.

Soil samples were excavated from the bore hole and returned to the laboratory to determine physical soil properties. The topsoil is weathered clay. Its shear strength is about 19kPa, natural water content is around 45% and its unit weight is about 17kN/m^3. The second layer is soft clay. It has shear strength about 8kPa, natural water content is 83% and its unit weight is about 15kN/m^3. The stiff clay is the third layer, which has shear strength about 38kPa. Its natural water content is about 36% and unit weight is about 18kN/m^3. For the dense sand and hard clay were not considered in this study because the deepest cement column was only 15m.

Coring testing

A total of 120 cored samples from 12 cement columns were subjected to unconfined compressive testing. The average shear strength which assume to be the representative of the column strength was found to be 650kPa, with a standard deviation of 174.91kPa and a coefficient of variation of 26%. However this average value should not be appropriate for use. It is because of scatter of data cause by the difficulties of mix control in the field installation. This may cause a mistake in design. Thus it is better to use the 90% lower value of the columns strength to be the representative.

Pile testing

A total of 12 cement columns in 6 different lengths, in pairs of 5m, 7m, 9m, 11m, 13m, and 15m, were installed. After a minimum of 28days curing cement columns were subjected to static pile load testing to determine the ultimate load for each column.

The design approve adapted was to use end bearing and skin friction with an adhesion faction of 1 for soft Bangkok clay. This was able to predict capacity to a column depth of 7m. Beyond a depth of 7m the crushing strength of the soil-cement (127.60kN for this study) was found to be the governing criteria.

Analysis of the data for ultimate load of the 5m length (about 110kN) and the 7m length (about 120kN) of cement columns show that these columns underwent soil failure because the ultimate load of these columns were lower than crushing strength of the cement column material. The other longer of cement columns failed by crushing of the soil-cement material.

It was found that the 9m length of cement column was the best economic maximum depth as at this depth in this study crushing strength (127.60kN) and the ultimate soil design load (150kN) become very similar. This was achieved at a cement content of 200kg of cement for 1m^3 of soil and a water and cement ratio of 1.1/1. If cement columns greater than 9m in length are required for bearing capacity, it is necessary to increase the cement content. However this will both increase cost and increase installation difficulties making the use of cement columns ineffective use in the clay studied.

Computer Modelling

The finite element package PLAXIS specially intended for the analysis of deformation and stability in geotechnical engineering projects was used to model the columns. Six models of six different lengths of cement column, which were the same length with the actual field-testing, were simulated in the program.

There are many parameters, which influence the ultimate load of cement columns. The most influential would also depend on the soil model selected. In this study, the Mohr-Coulomb Model (MC Model) was chosen to be representative of Weathering clay and Stiff clay. For Soft clay, the Soft Soil Model (SSM) was chosen to be representative. The most influential parameters for the MC Model and SSM are Young's Modulus (E) and internal friction angle (Ø).

The program used three different value of Young's Modulus to put into the program. To determine which one is the most appropriate for use with Bangkok clay. The three different value of Young's Modulus used to model Bangkok clay were $100C_{soil}$, $200C_{soil}$ and $250C_{soil}$ and the three different values used to model of the cement column were $10C_{col}$, $20C_{col}$ and $30C_{col}$ for low E, middle E and high E representatively. Also short term and long term conditions were considered.

The parametric study, using the lowest Young's Modulus in the program, gave the similar ultimate load result with the actual field pile load test of cement column. Therefore PLAXIS program can be used to design and analyse the ultimate load of cement columns and using the low E is the most appropriate for Bangkok clay.

9.2 Recommendations for Further Study

This study investigated the behaviour of different lengths of cement columns in soft Bangkok clay loaded to ultimate capacity. The FE PLAXIS program was then used to model the field behavior.

Both construction economics and laboratory work indicated that 200kg/cu.m of cement was most appropriate for column construction. However higher levels of cement to an upper limit of around 250kg/cu.m should be investigated to further validate the model for bearing capacity for longer columns or stiffer soils.

In cut-off walls permeability, not column crushing strength is the governing criteria. So the 200kg/cu.m still allow the column to perform satisfactorily at the site in their role of cut-off walls for soft intrusion.

It would provide a higher strength in the same cement content if using a new installing technic. It combines between a high pressure jetting and a rotary mixed. This would reduce a gap between the strength of mix design in the laboratory and the strength of cored samples in the field. This technic would give a easier and more accurate predication of the column strength from mix design and less cost of construction.

It would also be useful to carry out further research higher strength cement columns (1500-3000kPa) with different clays (softer) to further extend the model to greater depths of column installation.

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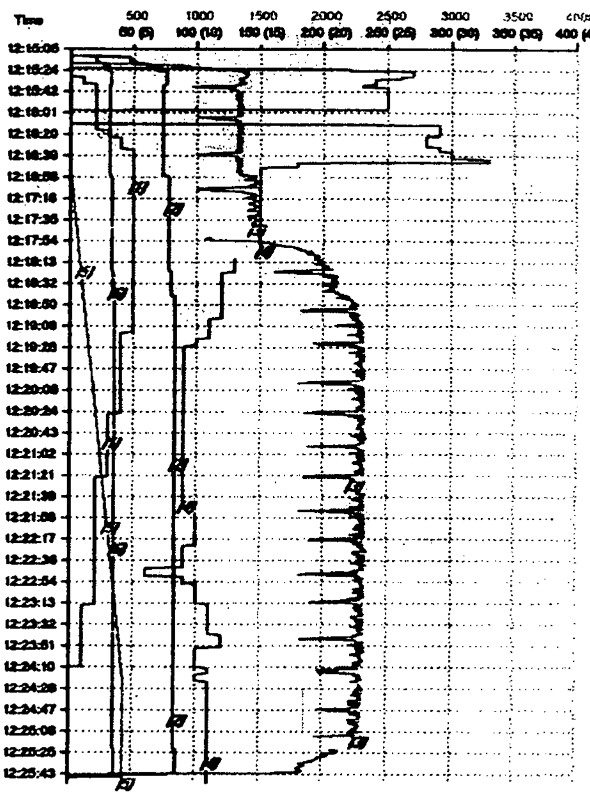
APPENDICES 1

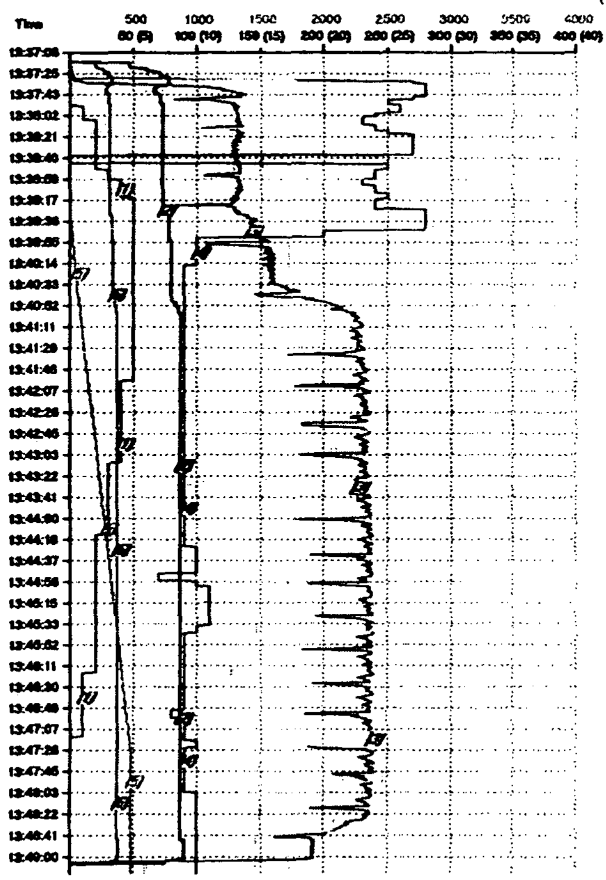
INSTALLATION DATA

Project	Protecting salted water by cut-off walls.
Owner	King Mongkut's University of Technology Thonburi
Contactor	Puga Company
Dath	Sep/10/02
Column No.	1
Cement content	200kg/m^3
w/c ratio	1.1/1



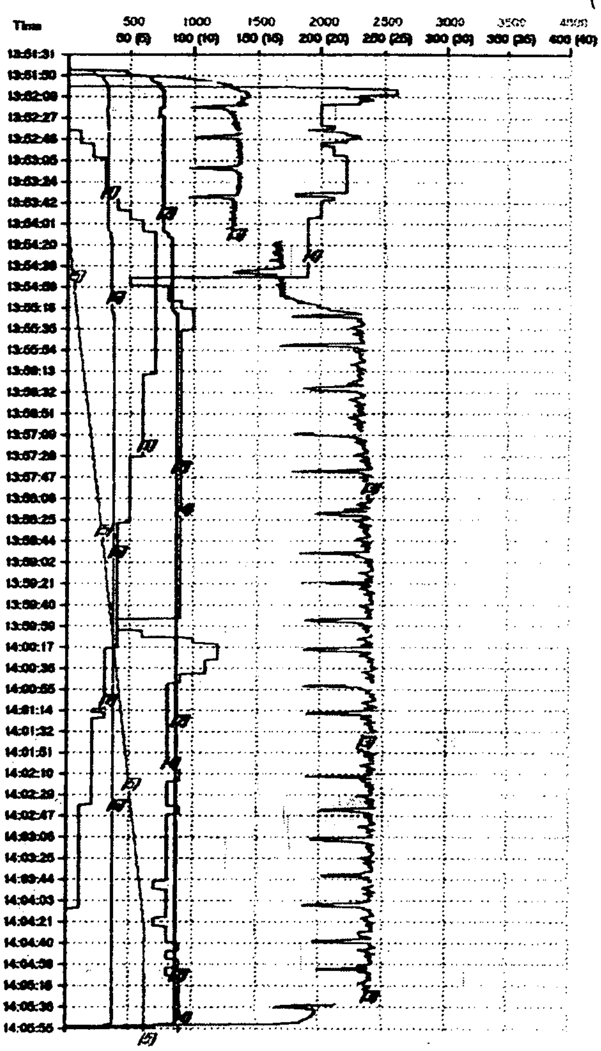
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.2	75.5	135.6	26.8	0.0	31.9
1.6	73.0	135.2	24.8	0.0	30.9
1.8	73.1	134.2	0.0	0.0	30.9
3.2	73.1	132.3	28.6	0.0	30.9
5.2	73.1	131.1	30.3	0.0	30.9
5.2	77.7	146.2	15.1	10.0	32.9
5.2	77.9	146.5	15.1	28.1	32.9
5.2	77.9	146.8	15.1	45.3	32.9
5.2	77.9	151.8	15.2	62.5	32.9
5.2	77.9	194.2	13.3	79.6	32.9
5.2	80.3	200.3	12.3	97.1	33.9
4.9	82.7	222.4	12.2	115.0	35.0
4.6	82.7	229.9	10.7	133.2	35.0
4.3	82.7	227.5	9.4	151.4	35.0
4.0	82.7	232.6	9.3	169.6	35.0
3.7	82.7	231.8	9.4	187.9	35.0
3.5	82.7	186.2	9.3	206.0	35.0
3.1	82.7	232.8	9.2	224.2	35.0
2.8	82.6	231.8	9.2	242.4	34.9
2.6	82.7	186.3	9.3	260.5	35.0
2.2	82.7	230.6	9.4	278.7	35.0
2.0	82.7	232.6	9.5	296.8	35.0
2.0	82.7	190.4	9.6	315.0	35.0
2.0	82.7	231.3	8.9	333.2	35.0
1.8	82.7	229.6	9.4	352.3	35.0
1.5	82.6	226.5	10.7	369.5	34.9
1.2	82.6	230.6	11.4	387.7	34.9
0.8	82.6	232.8	11.7	405.9	34.9
0.4	82.6	228.2	10.6	424.1	34.9
0.2	82.7	231.6	10.8	431.2	35.0
0.0	82.7	224.8	11.1	431.2	35.0
0.0	82.7	221.2	11.1	431.2	35.0
0.0	85.1	200.7	11.1	431.2	36.0
0.0	42.3	0.0	11.2	431.2	17.9

Project	Protecting salted water by cut-off walls.
Owner	King Mongkut's University of Technology Thonburi
Contactor	Puga Company
Dath	Sep/10/02
Column No.	2
Cement content	200kg/m^3
w/c ratio	1.1/1



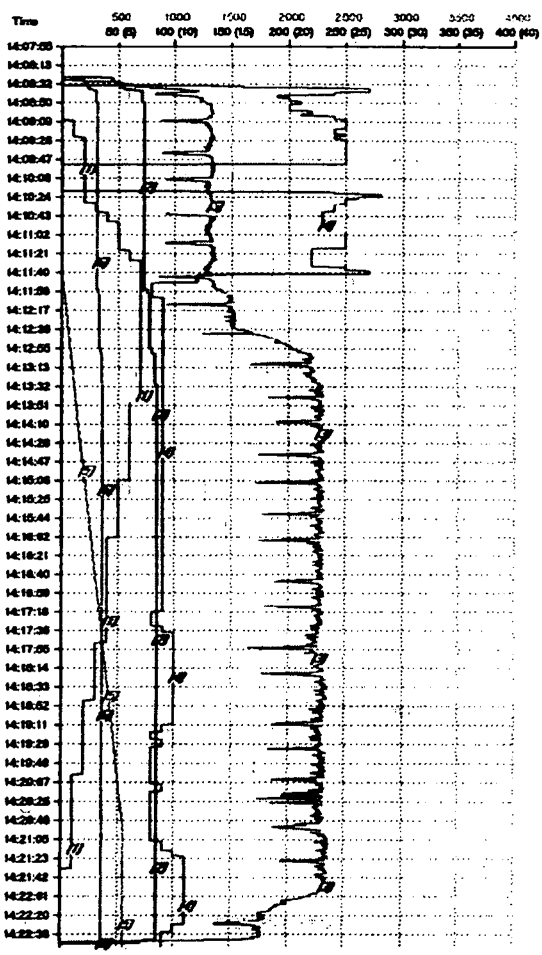
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.2	0.0	0.0	0.0
0.0	70.7	1.5	0.0	0.0	29.9
0.0	70.8	131.5	28.2	0.0	29.9
1.2	73.1	132.3	23.3	0.0	30.9
2.2	73.0	132.6	26.9	0.0	30.9
2.2	73.1	130.9	0.0	0.0	30.9
3.5	73.1	134.0	23.4	0.0	30.9
4.8	73.1	129.7	24.5	0.0	30.9
5.3	77.9	141.4	28.2	10.1	32.9
5.2	80.3	113.0	10.1	27.4	33.9
5.2	80.3	152.8	10.1	45.2	33.9
5.2	80.3	166.8	9.3	83.0	33.9
5.2	84.9	214.5	9.1	88.9	35.9
5.0	87.4	225.0	9.1	100.0	37.0
4.8	87.4	231.8	9.1	118.4	37.0
4.4	87.4	234.0	9.2	138.6	37.0
4.2	87.4	235.6	8.9	157.8	37.0
3.9	87.4	236.1	9.0	177.1	37.0
3.5	87.4	232.3	9.3	196.3	37.0
3.3	87.4	202.9	9.2	215.6	37.0
3.0	87.3	232.8	9.3	234.8	37.0
2.7	87.4	229.4	9.2	254.1	37.0
2.5	87.4	179.1	9.3	273.2	37.0
2.5	87.4	233.0	9.4	282.5	36.9
2.5	87.4	233.7	9.5	311.7	37.0
2.4	87.4	109.4	10.4	330.3	37.0
2.1	87.4	239.0	10.8	350.1	37.0
1.8	87.4	232.8	10.7	369.3	36.9
1.5	87.3	237.6	9.4	388.8	37.0
1.2	87.4	237.9	9.2	407.8	37.0
1.0	87.4	236.2	8.9	427.0	37.0
0.8	87.4	234.9	8.7	445.2	37.0
0.4	87.4	235.4	9.1	465.2	37.0
0.4	87.4	237.1	9.4	484.8	37.0
0.2	87.4	239.0	9.4	485.2	37.0
0.2	87.4	237.4	9.6	485.2	37.0
0.1	87.3	231.5	9.5	485.2	36.9
0.1	87.4	161.4	9.8	485.2	37.0
0.0	89.9	191.6	9.8	485.2	38.0

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/10/02
Column No. 3
Cement content 200kg/m^3
w/c ratio 1.1/1



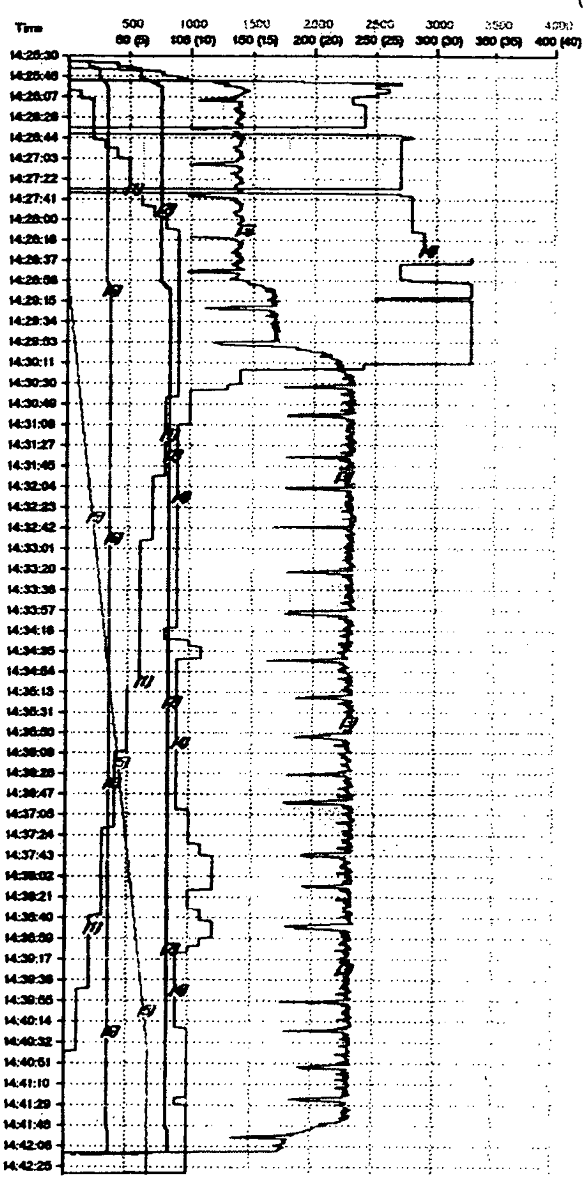
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	66.0	85.3	0.0	0.0	27.9
0.2	75.5	140.5	22.9	0.0	31.9
0.4	75.5	132.6	20.5	0.0	31.9
1.2	75.4	134.9	22.2	0.0	31.9
2.8	75.5	135.7	21.6	0.0	31.9
3.2	75.5	135.2	21.8	0.0	31.9
4.0	75.5	129.7	19.6	0.0	31.9
5.9	75.5	133.2	19.2	0.9	31.9
6.7	82.6	164.1	19.3	18.5	33.9
7.2	82.6	170.4	19.2	36.8	33.9
7.2	82.7	166.7	8.3	55.0	33.9
7.2	82.7	207.0	9.5	73.3	35.9
7.1	87.4	229.9	9.4	92.4	37.0
6.8	87.4	228.4	9.4	111.5	37.0
6.6	87.4	232.8	9.0	130.8	37.0
6.2	87.4	231.6	8.9	169.5	37.0
5.9	87.3	234.4	8.8	188.5	37.0
5.7	87.4	220.5	8.9	208.1	37.0
5.4	87.4	234.2	8.7	227.3	37.0
5.1	87.3	235.6	8.7	248.8	37.0
4.8	87.4	239.5	8.8	266.0	37.0
4.6	87.4	228.4	8.9	285.3	37.0
4.2	87.4	234.2	8.9	304.4	36.9
4.0	87.4	239.7	8.9	323.7	37.0
3.7	87.4	230.1	9.0	342.9	37.0
3.7	87.4	231.5	8.9	362.2	37.0
3.7	87.4	238.1	4.4	381.3	36.9
3.4	87.3	190.4	12.3	400.5	37.0
3.2	87.4	243.2	10.6	410.7	37.0
2.8	87.4	240.5	8.3	438.8	37.0
2.5	87.4	222.6	8.4	453.1	37.0
2.3	87.4	242.9	8.4	477.3	37.0
2.1	87.4	244.8	8.3	496.5	37.0
1.8	87.4	239.3	8.6	515.7	37.0
1.5	87.4	230.4	8.4	534.9	37.0
1.2	87.3	237.3	8.5	554.1	36.9
1.0	87.4	193.7	8.0	573.4	37.0
0.7	87.4	239.5	8.4	592.6	37.0
0.6	87.4	242.4	8.4	611.8	37.0
0.6	87.4	237.4	8.0	631.0	37.0
0.4	87.4	243.8	7.1	635.3	37.0
0.2	87.4	242.2	8.7	635.3	37.0
0.0	87.4	232.5	8.6	635.3	36.9
0.0	87.4	241.4	8.7	635.3	37.0
0.0	87.4	187.2	8.7	635.3	37.0
0.0	0.1	0.0	8.7	635.3	0.0

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/10/02
Column No. 4
Cement content 200kg/m^3
w/c ratio 1.1/1



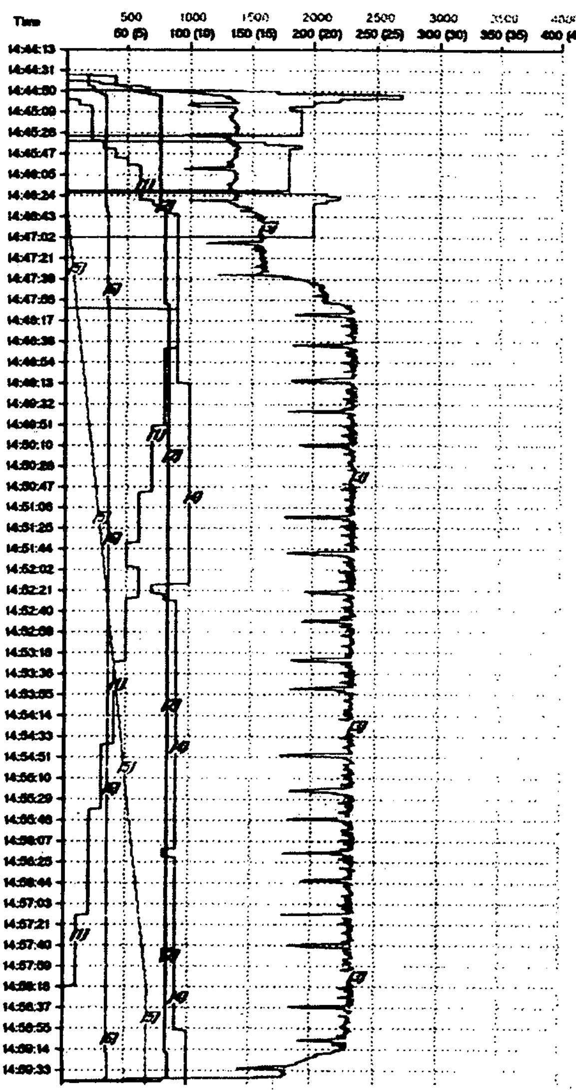
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	0.1	0.0	0.0	0.0	0.0
0.0	50.8	62.4	0.0	0.0	21.5
0.1	73.0	129.7	21.4	0.0	30.9
0.7	73.1	126.7	25.0	0.0	30.9
1.7	73.0	131.6	25.1	0.0	30.9
1.9	73.0	132.6	25.0	0.0	30.9
1.9	73.1	92.6	0.0	0.0	30.9
2.1	73.1	132.0	26.4	0.0	30.9
4.0	73.1	134.2	23.2	0.0	30.9
5.2	73.1	130.6	25.1	0.0	30.9
6.0	73.1	132.5	22.4	0.0	30.9
7.0	73.1	130.4	27.3	12.9	30.9
7.1	75.5	142.6	7.7	29.1	31.9
7.1	77.7	145.8	9.1	46.1	32.9
7.1	77.9	162.6	9.0	63.2	32.9
7.2	80.4	206.7	8.9	88.4	34.0
7.0	82.7	220.2	8.7	98.5	35.0
6.8	85.0	230.4	9.1	116.5	35.9
6.5	85.1	229.7	9.0	135.5	36.0
6.2	85.1	219.5	9.0	154.1	36.0
6.0	85.0	219.5	9.0	172.9	35.9
5.7	85.1	227.0	9.0	198.6	36.0
5.3	85.0	188.2	8.9	210.3	35.9
5.1	85.0	229.1	9.0	229.0	35.9
4.0	85.1	228.0	8.9	247.7	36.0
4.2	85.0	227.9	9.0	265.4	36.0
4.0	85.0	228.2	9.1	285.2	35.9
4.0	85.1	229.9	9.0	304.6	35.9
3.9	85.1	226.5	7.6	322.7	35.9
3.7	85.0	229.9	9.3	341.4	35.9
3.3	85.0	226.2	10.0	360.2	36.0
3.0	85.0	222.2	10.4	372.0	36.0
2.7	85.0	228.9	10.1	397.7	35.9
2.4	85.0	235.2	10.3	418.5	35.9
1.8	85.1	232.8	9.3	435.4	35.9
1.6	85.1	229.6	8.6	454.1	36.0
1.3	85.1	226.5	8.5	472.7	36.0
1.0	85.1	221.5	8.3	491.5	36.0
0.7	85.0	232.5	8.4	510.1	36.0
0.6	85.0	185.6	8.4	529.6	35.9
0.6	85.0	230.3	8.5	548.9	35.9
0.4	85.0	236.2	10.9	558.3	35.9
0.3	85.0	234.4	10.9	558.3	35.9
0.0	85.0	213.2	11.0	558.3	35.9
0.0	85.0	175.7	11.2	558.3	35.9
0.0	85.0	177.9	9.4	558.3	35.9

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/10/02
Column No. 5
Cement content 200kg/m^3
w/c ratio 1.1/1



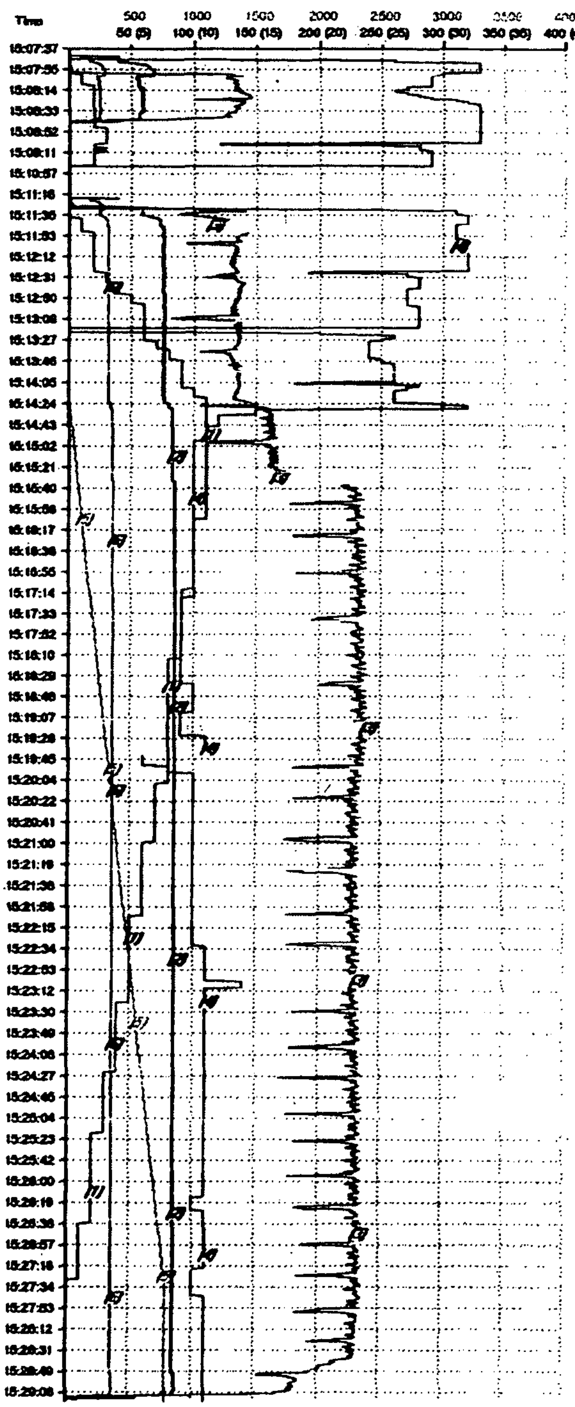
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	58.9	88.7	0.0	0.0	24.9
1.3	75.5	135.2	23.0	0.0	31.9
2.0	75.5	139.0	24.3	0.0	31.9
2.5	75.5	142.2	27.3	0.0	31.9
4.7	75.5	136.6	26.8	0.0	31.9
5.4	75.5	130.0	26.8	0.0	31.9
6.1	75.5	134.4	28.4	0.0	31.9
7.7	75.5	141.0	28.4	0.0	31.9
8.7	75.5	137.1	28.8	0.0	31.9
8.9	75.5	139.5	32.9	0.0	31.9
8.9	75.5	146.2	32.4	6.8	31.9
8.9	82.6	164.8	32.9	23.9	34.9
8.9	82.6	170.9	33.2	42.1	34.9
9.0	82.7	124.3	32.9	60.4	35.0
8.7	82.7	217.8	24.3	78.8	35.0
8.4	82.7	232.0	13.0	96.8	35.0
8.1	82.7	228.5	10.1	115.0	35.0
7.8	82.7	228.4	9.2	133.2	35.0
7.5	82.7	227.9	9.0	151.5	35.0
7.3	82.7	232.8	8.9	169.7	35.0
7.0	82.7	178.5	8.8	187.8	35.0
6.8	82.7	232.5	8.8	206.0	34.9
6.4	82.7	229.6	8.8	224.3	35.0
6.2	82.7	232.8	8.8	242.6	35.0
5.9	82.7	232.0	8.8	262.3	35.0
5.7	82.7	229.1	8.9	278.1	35.0
5.6	82.7	181.7	8.9	297.3	35.0
5.6	82.7	228.5	7.7	315.7	35.0
5.6	82.7	220.5	11.3	334.0	34.9
5.3	82.7	228.7	8.6	352.2	35.0
5.1	82.6	231.8	8.7	370.4	35.0
4.7	82.7	209.4	8.7	388.9	35.0
4.4	82.7	222.9	8.8	406.9	35.0
4.2	82.7	180.2	8.8	425.2	35.0
3.8	82.7	233.7	9.2	443.3	35.0
3.8	82.7	231.5	9.2	481.4	35.0
3.3	82.7	229.7	9.5	479.7	35.0
3.0	82.7	220.9	9.6	498.0	34.9
2.7	82.6	231.6	11.6	516.4	35.0
2.9	82.7	229.7	11.7	534.5	34.9
2.5	82.7	231.5	9.7	552.7	35.0
2.1	82.7	231.5	11.1	570.8	34.9
1.9	82.7	231.3	11.0	589.2	34.9
1.5	82.6	231.3	9.4	607.5	35.0
1.3	82.6	175.4	9.2	625.7	35.0
1.0	82.7	230.6	8.9	643.8	35.0
0.7	82.7	232.3	9.3	662.0	35.0
0.4	82.7	231.8	9.8	680.1	35.0
0.2	82.6	230.4	9.6	686.7	34.9
0.0	82.7	228.0	9.7	686.7	35.0
0.0	82.7	203.4	10.1	686.7	35.0
0.0	85.0	178.6	10.2	686.7	35.9
0.0	0.1	0.0	10.2	686.7	0.0

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/10/02
Column No. 6
Cement content 200kg/m^3
w/c ratio 1.1/1



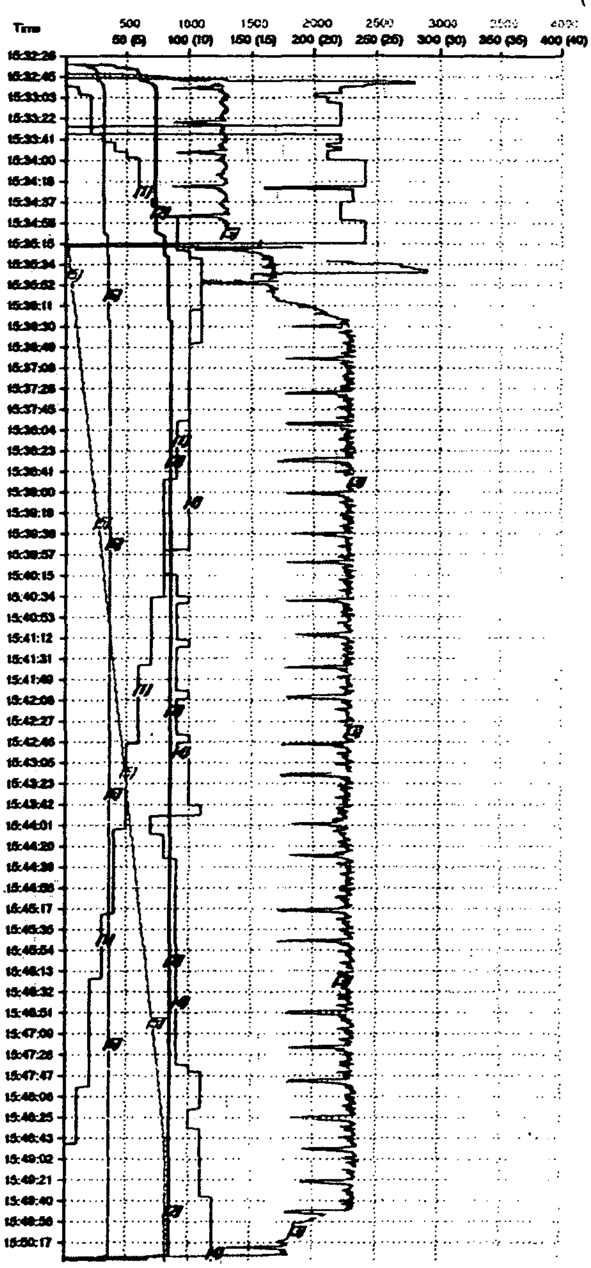
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.2	0.1	0.0	0.0	0.0	0.0
0.2	0.1	0.0	0.0	0.0	0.0
0.2	61.2	91.3	17.0	0.0	25.9
2.4	75.4	134.4	18.7	0.0	31.9
2.5	75.4	135.7	19.1	0.0	31.9
4.2	75.5	135.6	18.0	0.0	31.9
5.6	75.4	136.2	18.4	0.0	31.9
6.3	75.5	138.5	22.3	0.0	31.9
8.6	80.3	159.7	19.6	14.7	33.9
8.6	80.3	157.9	0.0	32.4	33.9
8.7	80.3	160.3	0.0	50.0	33.9
8.8	80.3	194.9	0.0	67.7	33.9
9.0	80.3	210.1	0.0	85.4	33.9
8.8	82.7	228.9	9.1	103.5	35.0
8.8	82.7	227.7	9.3	121.7	35.0
8.4	82.7	231.1	9.4	139.8	35.0
8.0	82.7	231.3	9.5	158.0	35.0
7.8	82.7	233.5	9.6	176.2	35.0
7.4	82.7	224.1	9.8	194.4	35.0
7.2	82.7	228.5	9.5	212.6	35.0
6.8	82.7	231.3	9.5	230.9	34.9
6.8	82.7	232.6	9.6	249.1	34.9
6.0	82.7	229.1	9.6	267.2	34.9
5.8	82.7	228.5	9.7	285.4	34.9
5.8	82.7	231.8	9.8	303.7	34.9
5.5	82.7	232.0	6.8	321.9	35.0
5.2	82.7	194.9	8.9	340.1	35.0
5.0	82.7	232.3	9.0	356.3	34.9
4.6	82.7	232.8	9.0	378.5	34.9
4.4	82.7	227.9	8.9	394.7	34.9
3.8	82.7	233.8	8.8	412.9	35.0
3.5	82.6	231.8	8.8	431.0	34.9
3.3	82.7	233.2	8.7	442.3	35.0
3.0	82.7	223.4	8.8	467.4	35.0
2.6	82.7	227.0	8.9	485.6	34.9
2.4	82.7	232.5	9.0	503.8	34.9
2.3	82.7	233.7	9.0	522.9	35.0
2.2	82.7	225.8	9.1	540.1	34.9
1.9	82.7	234.0	8.6	558.3	35.9
1.8	82.6	232.6	9.0	575.5	35.0
1.3	82.7	233.1	9.0	594.6	34.9
1.1	82.7	233.8	9	612.8	35.0
0.8	82.7	196.6	9.0	638.8	34.9
0.4	82.7	233.7	8.9	649.1	34.9
0.2	82.6	223.4	9.3	667.0	35.0
0.0	82.6	227.7	9.4	679.5	35.0
0.0	82.7	229.1	9.6	679.5	35.0
0.0	82.7	220.3	9.7	679.5	35.0
0.0	82.7	177.6	9.8	679.5	35.0

Project	Protecting salted water by cut-off walls.
Owner	King Mongkut's University of Technology Thonburi
Contactor	Puga Company
Dath	Sep/10/02
Column No.	7
Cement content	200kg/m^3
w/c ratio	1.1/1



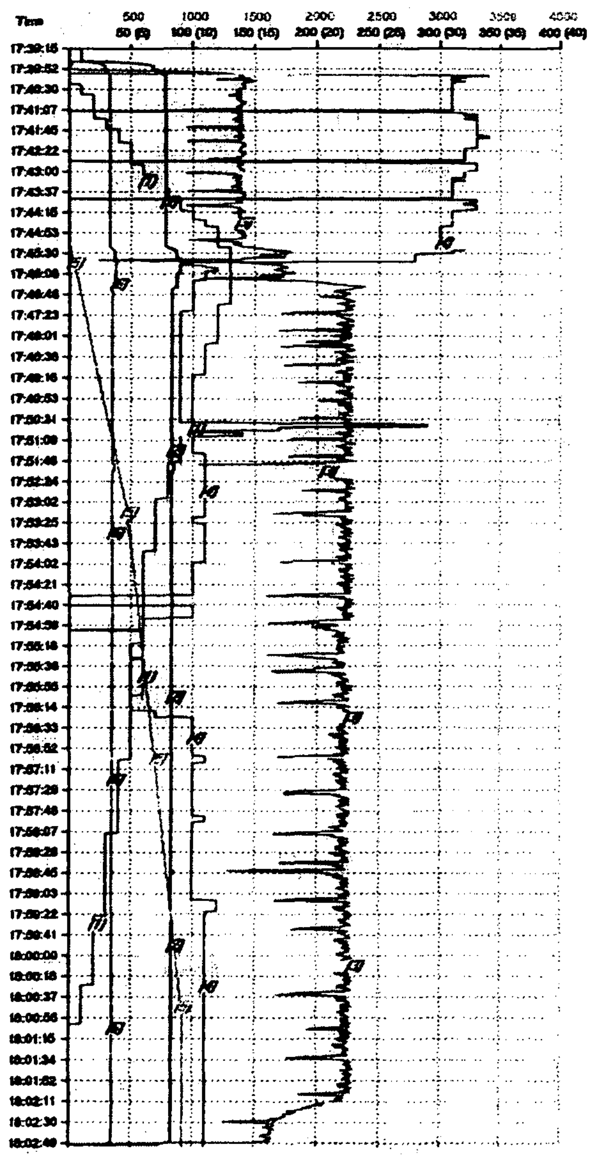
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.3	68.4	1.9	33.2	0.0	28.9
2.0	58.9	139.0	26.4	0.0	24.9
2.5	59.0	131.6	33.2	0.0	25.0
2.6	0.1	0.0	33.2	0.0	0.0
2.6	0.1	0.0	28.8	0.0	0.0
0.2	0.1	0.0	0.0	0.0	0.0
0.5	0.1	0.2	0.0	0.0	0.0
0.4	63.6	102.1	31.6	0.0	26.9
1.8	75.5	134.0	31.4	0.0	31.9
2.5	75.4	131.5	31.6	0.0	31.9
3.1	75.4	131.6	28.2	0.0	31.9
5.1	75.5	134.7	27.1	0.0	31.9
5.6	75.5	131.3	27.8	0.0	31.9
6.8	75.5	136.4	24.4	0.0	31.9
8.9	75.5	133.3	25.0	0.0	31.9
9.1	75.4	135.9	17.7	0.0	31.9
11.0	80.3	150.9	29.6	11.2	33.9
11.0	82.6	163.6	11.7	294.0	34.9
11.0	82.6	162.2	10.3	47.8	34.9
11.0	82.6	166.0	10.2	55.9	34.9
11.0	85.0	224.4	9.5	84.5	35.9
10.7	85.0	232.0	9.8	103.2	35.9
10.4	85.0	226.7	9.5	122.0	35.9
10.1	85.1	236.9	9.5	140.7	36.0
9.8	85.0	101.5	9.8	153.4	35.9
9.5	85.1	232.5	9.4	178.2	36.0
9.2	85.1	233.8	9.3	197.1	36.0
8.9	85.0	228.4	9.3	215.8	35.9
8.6	85.1	229.1	9.4	234.6	36.0
8.3	85.0	227.9	9.4	253.6	35.9
8.0	85.0	237.1	9.4	272.5	35.9
7.9	85.0	238.8	9.5	291.3	35.9
7.9	85.1	234.9	9.3	310.1	36.0
7.7	85.1	229.6	11.2	328.8	36.0
7.5	85.0	234.2	6.1	347.5	35.9
7.1	85.0	224.4	10.4	366.2	36.0
6.8	85.0	232.6	10.4	385.0	36.0
6.5	85.0	227.9	10.4	403.8	35.9
6.2	85.0	224.4	10.3	422.4	35.9
5.9	85.0	226.8	10.3	441.2	35.9
5.5	85.0	233.2	10.3	460.0	35.9
5.2	85.1	230.1	10.5	478.7	36.0
4.8	85.0	229.7	10.5	487.4	35.9
4.7	85.1	227.2	10.6	526.1	36.0
4.7	85.1	228.0	10.7	534.8	36.0
4.4	85.0	227.0	10.9	563.5	35.9
4.0	85.0	227.2	11.0	572.2	35.9
3.7	85.0	230.9	11.0	590.0	35.9
3.4	85.0	169.2	10.9	603.7	35.9
3.0	85.0	234.4	10.9	626.3	35.9
2.7	85.1	231.1	11.0	647.0	35.9
2.5	85.1	180.7	11.2	666.7	36.0
2.1	85.1	230.4	11.2	684.3	36.0
1.8	85.0	229.1	11.2	703.0	36.0
1.7	85.0	235.4	10.2	721.8	35.9
1.4	85.1	233.3	10.8	748.0	35.9
1.1	85.1	224.4	10.7	759.2	36.0
0.8	85.0	232.6	10.6	777.9	36.0
0.4	85.0	231.6	10.1	793.5	35.9
0.2	85.0	203.8	10.8	793.5	35.9
0.0	85.1	231.1	10.7	793.5	35.9
0.0	85.1	225.8	11.1	793.5	36.0
0.0	85.0	192.3	11.1	793.5	36.0
0.0	85.0	166.7	11.2	793.5	35.9

Project	Protecting salted water by cut-off walls.
Owner	King Mongkut's University of Technology Thonburi
Contactor	Puga Company
Dath	Sep/10/02
Column No.	8
Cement content	200kg/m^3
w/c ratio	1.1/1



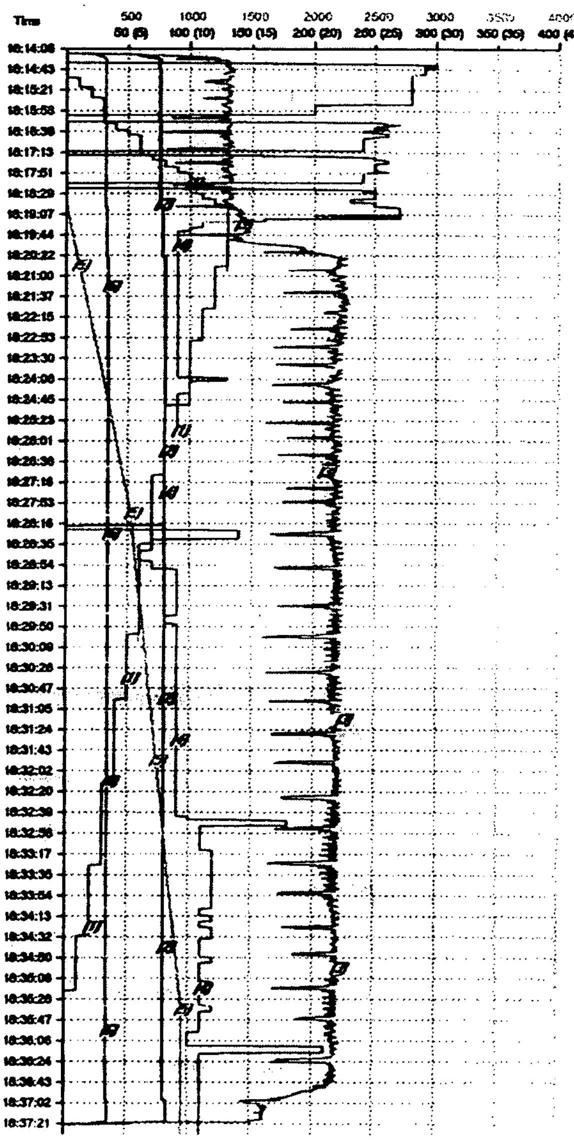
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	61.2	83.8	0.0	0.0	25.9
2.1	73.0	126.5	20.6	0.0	30.9
2.5	73.0	128.0	22.3	0.0	30.9
3.2	73.0	127.5	22.2	0.0	30.9
5.6	73.1	127.4	23.7	0.0	30.9
5.8	73.1	130.6	23.7	0.0	30.9
7.2	73.1	124.1	21.6	0.0	30.9
8.8	73.1	131.8	23.7	1.8	30.9
8.8	80.3	156.1	0.0	18.7	33.9
11.0	82.7	168.4	27.3	36.6	35.0
11.0	82.6	166.3	11.4	54.8	34.9
11.0	82.6	188.5	10.7	73.1	35.9
10.7	85.0	221.4	10.1	91.5	35.9
10.5	85.0	228.0	9.6	110.2	35.9
10.1	85.1	227.9	9.5	128.9	35.9
9.9	85.1	223.6	9.5	147.6	35.9
9.8	85.0	224.8	9.5	166.3	35.9
9.3	85.0	230.6	9.7	185.1	35.9
9.0	85.1	225.6	9.6	203.8	36.0
8.7	85.0	225.5	9.6	222.5	35.9
8.4	85.0	179.3	9.7	241.3	35.9
8.1	85.0	224.3	9.8	268.0	36.0
7.8	85.1	201.5	9.8	278.7	35.9
7.8	85.1	229.6	8.2	287.4	35.9
7.4	85.1	230.0	9.4	316.1	36.0
7.2	85.0	227.2	9.5	334.9	36.0
6.9	85.0	226.7	9.3	353.7	35.9
6.6	85.1	226.8	9.4	372.3	35.9
6.3	85.0	224.8	9.3	391.1	35.9
6.0	85.1	226.5	9.3	409.7	35.9
5.7	85.0	229.4	9.3	428.4	35.9
5.5	85.0	225.8	9.4	447.3	35.9
5.1	85.1	219.5	9.4	468.0	35.9
4.8	85.1	217.9	9.6	484.7	35.9
4.8	85.0	224.8	9.5	503.8	36.0
4.8	85.0	229.4	11.1	522.3	36.0
4.4	85.0	219.1	6.8	541.1	35.9
4.0	85.0	227.2	8.4	559.8	35.9
3.6	85.0	226.7	8.6	578.5	36.0
3.5	85.0	231.5	8.7	597.4	35.9
3.3	85.0	173.2	8.9	616.2	36.0
2.8	85.1	224.3	8.9	634.9	35.9
2.7	85.0	219.5	9.0	653.7	35.9
2.4	85.1	229.0	9.1	672.4	35.9
2.1	85.1	230.4	9.3	691.1	35.9
1.8	85.0	186.3	9.3	709.8	35.9
1.8	85.0	229.6	9.3	728.6	35.9
1.6	85.0	229.1	11.2	747.2	35.9
1.2	85.0	227.7	11.0	766.0	36.0
1.0	85.0	233.3	10.3	784.7	35.9
0.6	85.1	181.9	10.6	803.4	35.9
0.2	85.1	233.3	11.3	820.6	35.9
0.0	85.1	235.7	11.2	820.6	36.0
0.0	85.0	190.6	11.5	820.6	36.0
0.0	85.0	229.6	11.7	820.6	36.0
0.0	85.1	192.8	11.6	820.6	35.9
0.0	85.1	179.3	11.5	820.6	35.9

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/10/02
Column No. 9
Cement content 200kg/m^3
w/c ratio 1.1/1



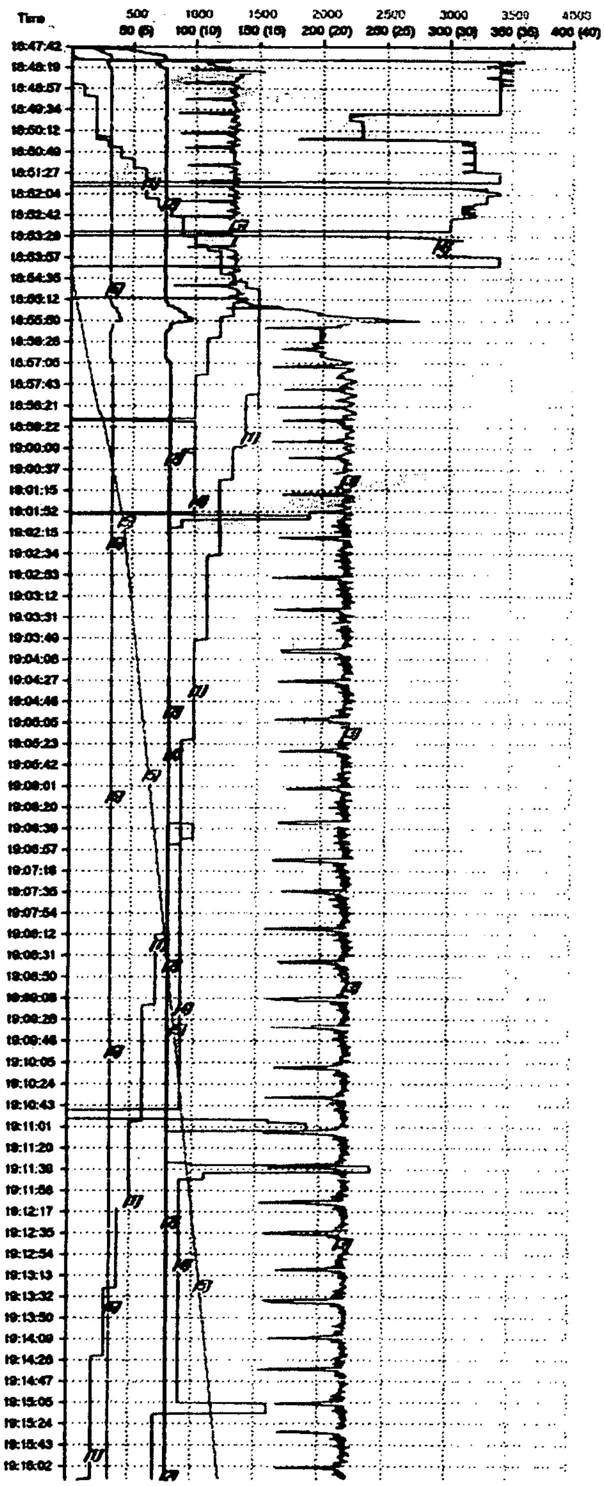
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	73.0	83.8	0.0	0.0	30.9
0.0	77.9	126.5	31.2	0.0	32.9
1.9	77.9	128.0	0.0	0.0	32.9
3.7	77.9	127.5	32.9	0.0	32.9
5.2	77.9	127.4	32.4	0.0	32.9
6.3	77.9	130.6	31.6	0.0	32.9
8.4	77.9	124.1	30.9	0.0	32.9
9.8	77.9	131.8	31.4	0.0	32.9
11.7	77.9	156.1	30.3	0.0	32.9
13.0	87.3	168.4	28.4	24.0	36.9
13.0	87.3	166.3	11.3	63.5	34.9
12.8	82.7	188.5	10.3	101.2	36.9
12.2	82.6	221.4	9.1	174.0	35.0
11.6	82.7	228.0	9.3	210.4	35.0
11.0	82.7	227.9	9.3	245.9	35.0
10.5	82.7	223.6	9.3	283.2	35.0
9.9	82.7	224.8	9.3	319.6	34.9
9.6	82.6	230.6	9.1	356.2	35.0
9.2	82.7	225.6	9.9	392.8	35.0
8.8	82.7	225.5	11.2	423.4	35.0
8.0	82.7	179.3	10.8	485.6	35.0
7.3	82.7	224.3	10.7	488.1	34.9
6.9	82.7	201.5	10.7	505.4	35.0
6.6	82.7	229.6	10.7	524.7	35.0
6.3	82.7	230.0	9.0	542.8	35.0
6.2	82.7	227.2	9.9	567.0	35.0
6.2	82.7	226.7	9.8	579.2	34.9
6.0	82.7	226.8	6.0	587.5	35.0
5.8	82.6	224.8	5.2	615.7	35.0
5.6	82.7	226.5	5.1	633.9	35.0
5.5	82.7	229.4	4.8	652.2	35.0
5.4	82.7	225.8	4.7	670.5	35.0
5.0	82.7	219.5	10.3	683.8	35.0
4.8	82.7	217.9	10.3	707.1	35.0
4.4	82.7	224.8	10.3	725.3	35.0
4.1	82.7	229.4	10.4	743.5	35.0
3.8	82.7	219.1	10.5	761.7	35.0
3.5	82.7	227.2	10.5	779.9	35.0
3.2	82.7	226.7	10.5	798.2	35.0
2.8	82.7	231.5	10.5	816.4	35.0
2.8	82.7	173.2	10.5	834.5	35.0
2.6	82.7	224.3	11.4	862.7	35.0
2.3	82.6	219.5	11.2	870.3	35.0
1.8	82.7	229.0	11.1	883.0	34.9
1.6	82.7	230.4	11.1	907.3	35.0
1.3	82.7	186.3	11.0	925.4	35.0
0.6	82.7	229.6	11.0	926.0	35.0
0.3	82.7	229.1	10.9	926.0	35.0
0.0	82.7	227.7	10.9	926.0	35.0
0.0	82.7	233.3	11.1	926.0	35.0
0.0	82.6	181.9	11.4	926.0	35.0
0.0	82.6	233.3	11.4	926.0	34.9
0.0	73.0	235.7	11.4	926.0	34.9

Project	Protecting salted water by cut-off walls.
Owner	King Mongkut's University of Technology Thonburi
Contactor	Puga Company
Dath	Sep/12/02
Column No.	10
Cement content	200kg/m^3
w/c ratio	1.1/1



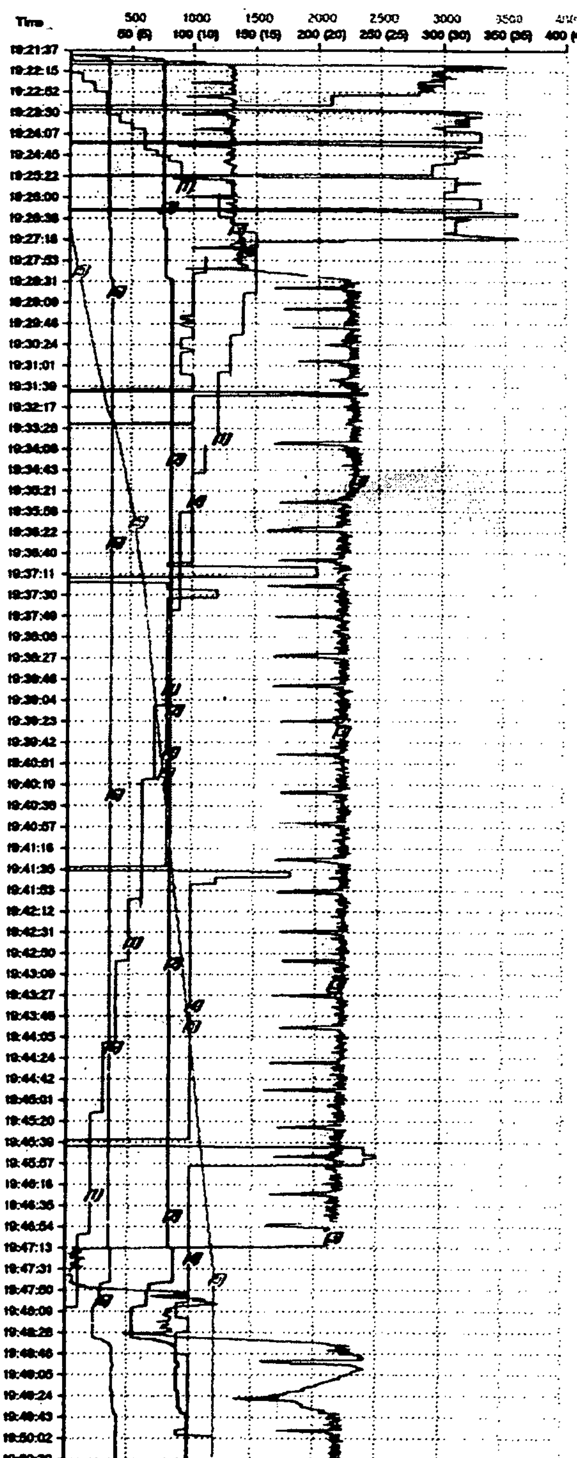
(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	75.5	133.2	29.2	0.0	31.9
1.9	75.5	134.0	28.0	0.0	31.9
3.2	75.5	127.5	20.3	0.0	31.9
5.2	75.5	129.1	25.1	0.0	31.9
5.6	75.5	129.2	0.0	0.0	31.9
8.5	75.5	135.7	24.4	0.0	31.9
10.8	75.5	132.6	25.3	0.0	31.9
12.9	77.9	140.3	19.9	26.0	32.9
13.0	77.9	137.9	9.3	60.4	32.9
12.8	77.9	216.2	9.2	84.7	32.9
12.2	80.3	221.9	9.1	130.2	33.9
11.8	80.3	222.9	9.1	185.7	33.9
11.2	80.4	222.1	9.1	201.1	34.0
10.7	80.3	221.5	9.1	238.4	33.9
10.1	80.3	222.4	9.2	271.8	33.9
9.8	80.3	217.3	13.1	307.2	33.9
9.2	80.3	222.2	10.1	342.6	33.9
8.7	80.3	215.6	8.0	378.5	33.9
8.3	80.3	220.7	7.9	413.3	34.0
7.6	80.4	217.9	7.9	448.8	33.9
7.3	80.4	217.6	8.0	484.0	33.9
6.8	80.3	218.3	8.1	519.3	33.9
6.7	80.4	221.2	7.4	544.9	34.0
6.4	80.4	214.9	8.7	576.3	34.0
6.0	80.3	216.9	8.7	594.2	33.9
5.9	80.3	223.9	8.6	611.7	34.0
5.8	80.4	217.1	8.8	629.9	33.9
5.7	80.4	209.7	8.9	647.0	34.0
5.5	80.3	214.9	9.1	584.7	34.0
5.2	80.3	212.6	9.2	692.4	33.9
5.0	80.3	218.3	9.1	700.0	33.9
4.7	80.3	215.6	9.0	717.8	34.0
4.4	80.4	218.5	9.0	736.3	34.0
4.2	80.4	218.3	9.0	753.0	33.9
3.8	80.4	217.3	9.0	770.8	33.9
3.7	80.2	212.6	9.0	763.3	34.0
3.3	80.3	218.8	11.2	808.0	34.0
3.1	80.4	208.4	11.5	823.6	34.0
3.0	80.4	219.7	11.5	841.3	34.0
2.7	80.4	217.3	11.5	859.0	34.0
2.3	80.4	219.0	11.5	876.6	33.9
2.0	80.4	216.6	11.5	884.3	33.9
1.7	80.3	220.2	11.5	911.5	34.0
1.4	80.3	212.5	11.4	929.6	33.9
1.0	80.4	217.8	11.4	947.2	34.0
0.6	80.3	218.3	11.3	951.2	33.9
0.4	80.4	219.3	10.2	951.2	33.9
0.2	80.3	212.0	11.1	951.2	33.9
0.2	80.3	212.8	10.8	951.2	33.9
0.0	82.6	161.9	11.0	951.2	34.9
0.0	0.1	0.0	11.0	951.2	0.0

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/12/02
Column No. 11
Cement content 200kg/m^3
w/c ratio 1.1/1



(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.0	75.5	115.2	34.6	0.0	31.9
1.0	75.5	131.1	34.0	0.0	31.9
2.5	75.5	133.5	34.5	0.0	31.9
2.5	75.5	131.6	22.6	0.0	31.9
4.1	75.5	129.9	31.9	0.0	31.9
5.0	75.5	134.4	33.7	0.0	31.9
6.3	75.5	126.5	32.7	0.0	31.9
8.4	75.5	86.3	31.6	0.0	32.9
9.5	75.5	131.5	28.4	0.0	32.9
12.2	75.5	130.1	34.0	0.0	32.9
13.2	75.5	131.1	0.0	12.0	33.9
15.0	75.5	133.5	13.5	45.3	33.9
15.0	80.4	244.3	11.6	83.8	34.0
15.0	80.3	195.2	11.4	118.4	33.9
15.0	80.3	217.9	11.4	152.0	33.9
14.7	80.3	214.0	9.7	167.5	33.9
14.2	80.3	165.0	9.5	222.9	33.9
13.8	80.3	215.7	9.7	280.8	33.9
13.4	80.3	219.0	9.4	316.6	34.0
12.8	80.4	217.8	9.5	351.5	33.9
12.3	80.4	217.6	9.6	388.6	33.9
12.1	80.3	219.3	0.0	422.1	33.9
11.8	80.4	215.6	8.1	443.8	34.0
11.5	80.4	211.3	8.2	451.4	34.0
11.3	80.3	222.6	8.1	473.1	33.9
11.0	80.3	216.8	8.1	496.8	34.0
10.9	80.4	218.5	7.9	514.5	33.9
10.5	80.4	221.9	7.8	532.3	34.0
10.4	80.3	215.6	7.8	549.9	34.0
10.1	80.3	166.5	7.7	567.6	33.9
9.9	80.3	223.1	7.7	585.2	33.9
9.8	80.3	215.4	7.5	603.8	34.0
9.4	80.4	219.5	7.6	620.6	34.0
9.2	80.4	223.6	7.6	638.3	33.9
8.9	80.4	190.4	7.5	655.8	33.9
8.8	80.2	221.2	7.5	673.7	34.0
8.7	80.3	214.2	10.5	681.4	34.0
8.5	80.4	217.6	9.1	708.1	34.0
8.2	80.4	221.5	9.0	726.7	34.0
8.0	80.4	207.0	8.9	744.4	34.0
7.8	80.4	217.6	8.9	762.8	33.9
7.4	80.4	221.9	8.9	778.7	33.9
7.1	80.3	215.7	8.9	787.4	34.0
6.9	80.3	217.0	8.8	815.1	33.9
6.7	80.4	164.4	8.9	832.7	34.0
6.4	80.3	218.1	8.9	850.4	33.9
6.2	80.4	217.1	9.0	868.0	33.9
5.9	80.3	218.3	8.9	885.8	33.9
5.8	80.3	220.7	9.0	903.4	33.9
5.6	80.3	219.7	9.2	921.1	34.0
5.4	80.3	219.5	18.7	938.7	34.0
5.2	80.3	219.7	8.5	956.4	34.0
4.8	80.3	218.1	24.1	974.1	34.0
4.6	80.4	218.1	8.6	981.8	34.0
4.3	80.3	221.9	8.6	1003.4	34.0
4.1	80.3	183.8	8.7	1027.1	34.0
3.9	80.3	219.1	8.9	1044.7	34.0
3.8	80.3	222.6	8.9	1062.4	34.0
3.3	80.4	220.0	9.1	1088.1	34.0
3.0	80.4	220.3	9.1	1097.8	34.0
2.8	80.4	218.1	9.2	1115.4	34.0
2.5	80.4	219.1	9.3	1133.1	34.0
2.3	80.4	217.6	15.6	1150.7	34.0
2.2	80.3	167.5	6.9	1168.4	33.9
2.0	80.3	219.7	6.7	1186.8	33.9
1.8	80.3	214.4	6.7	1203.7	33.9

Project Protecting salted water by cut-off walls.
Owner King Mongkut's University of Technology Thonburi
Contactor Puga Company
Dath Sep/12/02
Column No. 12
Cement content 200kg/m^3
w/c ratio 1.1/1



(1)Depth (m)	(2)Flow (L/Min)	(3)Pressure (Bar)	(4)Drill (Rpm)	(5)Cement (kg)	(6)Stroke (Rpm)
0.0	0.1	0.0	0.0	0.0	0.0
0.5	75.5	131.1	29.8	0.0	31.9
2.7	75.5	130.8	28.4	0.0	31.9
3.3	75.5	91.1	31.2	0.0	31.9
5.3	75.5	130.1	33.2	0.0	31.9
7.8	75.4	130.1	31.9	0.0	31.9
9.8	75.5	97.1	30.7	0.0	31.9
12.3	75.5	129.2	33.4	0.0	31.9
12.9	75.5	132.3	31.9	0.0	31.9
15.0	77.9	140.2	22.4	28.8	32.9
15.0	77.9	136.4	11.0	63.0	32.9
14.8	82.6	231.6	9.6	97.3	34.9
14.2	82.6	228.2	9.5	134.2	34.9
13.7	82.7	220.2	9.4	170.7	35.0
13.2	82.7	221.0	9.4	207.1	34.9
12.7	82.7	225.1	9.5	243.8	35.0
12.3	82.7	222.6	10.2	280.8	35.0
11.6	82.7	229.4	10.3	316.4	35.0
11.7	82.7	228.2	10.3	355.3	34.9
11.3	82.7	227.5	10.2	422.8	34.9
10.6	82.7	229.9	10.3	458.5	35.0
10.0	82.6	231.3	10.4	494.9	35.0
9.4	82.7	220.3	10.5	537.4	35.0
9.1	82.7	218.5	10.4	571.8	35.0
9.0	82.7	220.0	20.4	601.2	35.0
8.8	82.7	221.0	11.5	620.4	35.0
8.7	82.6	216.1	8.1	636.6	35.0
8.5	82.7	167.7	8.2	656.8	35.0
8.2	82.7	226.0	8.1	675.0	34.9
8.0	82.7	219.1	8.2	693.2	35.0
7.8	82.7	223.8	8.2	711.4	35.0
7.5	82.7	215.4	8.2	729.8	35.0
7.3	82.7	220.9	8.2	747.8	35.0
7.0	82.7	220.7	8.2	786.2	34.9
6.8	82.6	221.0	8.3	784.2	35.0
6.5	82.7	214.2	8.3	802.4	35.0
6.3	82.7	224.6	8.3	820.6	35.0
6.1	82.7	221.2	8.3	838.7	35.0
5.8	82.6	226.5	0.0	857.0	35.0
5.5	82.7	217.8	10.0	875.1	35.0
5.2	82.7	170.3	10.1	893.3	35.0
5.0	82.7	220.7	10.1	911.6	34.9
4.6	82.7	212.8	10.2	929.7	35.0
4.4	82.7	222.6	10.1	947.8	35.0
4.1	82.7	223.8	10.1	966.1	35.0
3.8	82.6	193.8	10.2	984.3	34.9
3.5	82.7	219.7	10.1	1002.6	35.0
3.2	82.7	225.5	10.2	1202.7	35.0
2.9	82.7	219.3	10.2	1038.8	35.0
2.7	82.7	225.0	10.2	1057.8	35.0
2.8	82.7	220.7	9.9	1075.2	35.0
2.5	82.7	221.4	0.0	1093.4	35.0
2.4	82.6	211.6	24.4	1111.5	34.9
2.2	82.7	221.5	9.6	1129.7	35.0
1.8	82.6	222.9	9.6	1147.9	34.9
1.6	82.6	210.4	9.7	1168.2	34.9
1.3	87.3	211.8	9.6	1184.4	36.9
1.0	87.3	18.8	9.6	1203.6	36.9
0.7	66.0	3.8	9.7	1204.7	27.9
0.5	54.2	81.7	9.4	1204.7	22.9
0.2	54.2	83.9	9.4	1204.7	22.9
0.0	89.9	48.4	9.6	1204.7	38.0
0.0	89.9	236.1	9.5	1204.7	38.0
0.0	92.1	223.4	9.5	1204.7	38.9
0.0	99.2	172.5	9.7	1204.7	42.0
0.0	99.1	212.0	11.7	1204.7	41.9
0.0	99.1	214.5	11.6	1204.7	41.9

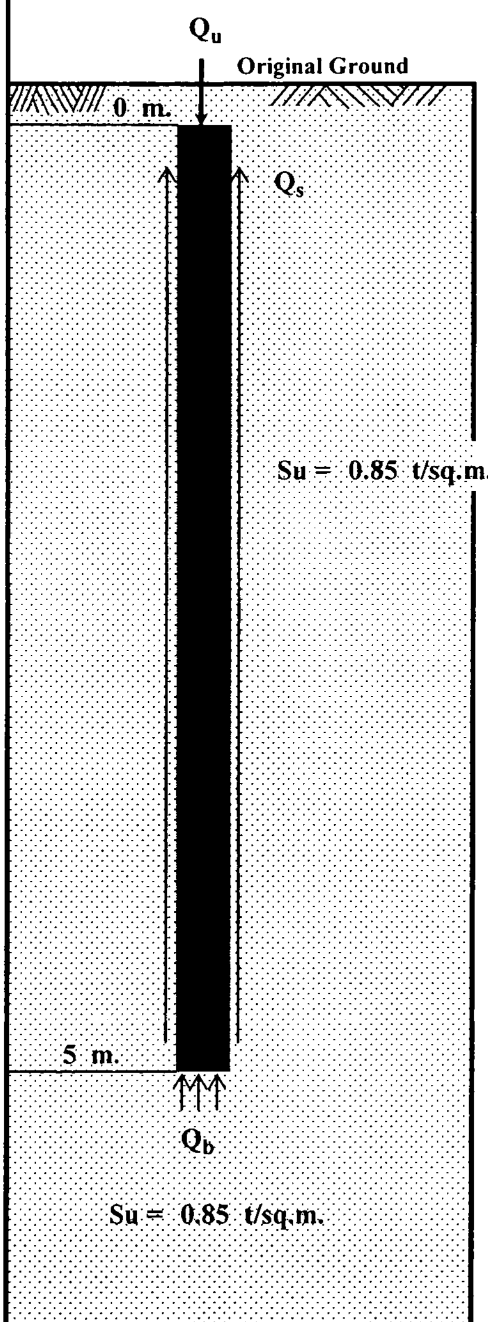
APPENDICES 2

THE DETAILS OF CALCULATION OF ULTIMATE SOIL DESIGN LOAD

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	:	**Cement Column**
Pile Size	:	**Diameter 0.50 m.**
Pile Length	:	**5 m.**
Cross Section Area	:	**0.196 m.^{2}**
Perimeter	:	**1.571 m.**
Depth of Pile Top	:	**0 m. below ground surface**
Depth of Pile Tip	:	**5 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s =$ 1 x 0.85 x 7.9 Tons

$Q_s =$ 6.7 Tons

End Bearing in clay layer

$Q_b =$ 9 x 0.85 x 0.196 Tons

$Q_b =$ 1.5 Tons

$Q_u = Q_s + Q_b$

$Q_u =$ 8.2 Tons

Factor of Safety = 2.5

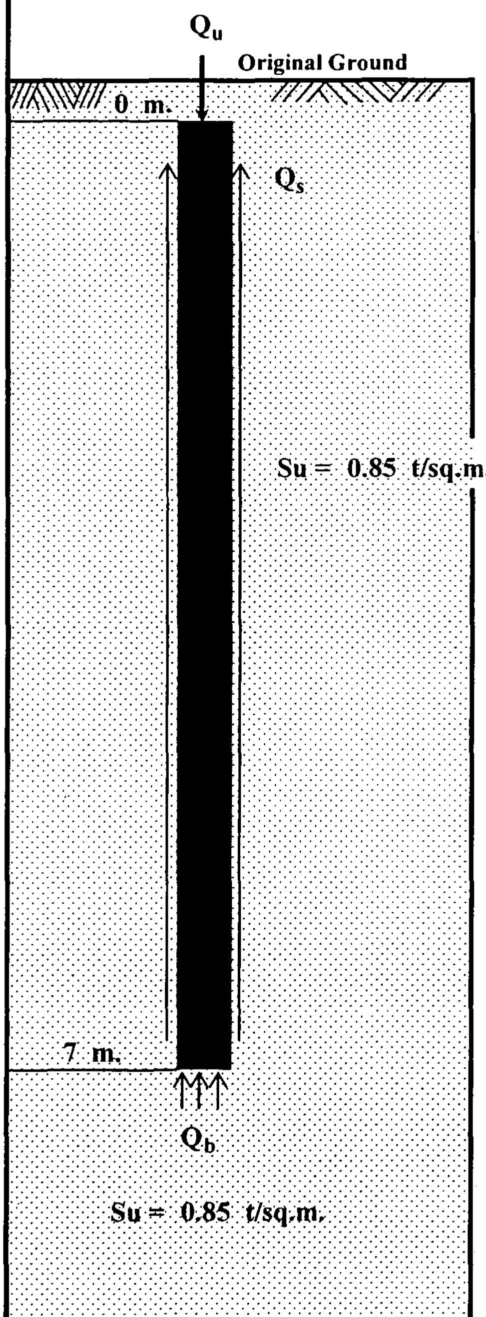
$Q_{safe} =$ 3.27 Tons

Say $Q_{safe} =$ 3 Tons

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	:	**Cement Column**
Pile Size	:	**Diameter 0.50 m.**
Pile Length	:	**7 m.**
Cross Section Area	:	**0.196 $m.^2$**
Perimeter	:	**1.571 m.**
Depth of Pile Top	:	**0 m. below ground surface**
Depth of Pile Tip	:	**7 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s =$ 1 x 0.85 x 11.0 Tons

$Q_s =$ 9.3 Tons

End Bearing in clay layer

$Q_b =$ 9 x 0.85 x 0.196 Tons

$Q_b =$ 1.5 Tons

$Q_u = Q_s + Q_b$

$Q_u =$ 10.8 Tons

Factor of Safety = 2.5

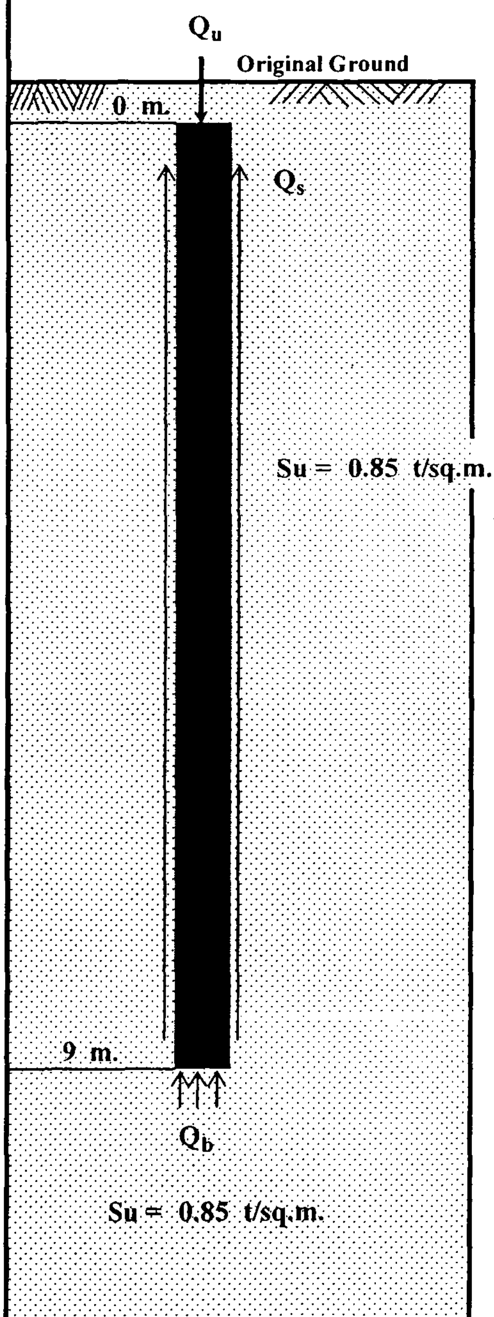
$Q_{safe} =$ 4.34 Tons

Say $Q_{safe} =$ 4 Tons

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	:	**Cement Column**
Pile Size	:	**Diameter 0.50 m.**
Pile Length	:	**9 m.**
Cross Section Area	:	**0.196 m.^{2}**
Perimeter	:	**1.571 m.**
Depth of Pile Top	:	**0 m. below ground surface**
Depth of Pile Tip	:	**9 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s = 1 \times 0.85 \times 14.1$ Tons

$Q_s =$ 12.0 Tons

End Bearing in clay layer

$Q_b = 9 \times 0.85 \times 0.196$ Tons

$Q_b =$ 1.5 Tons

$Q_u = Q_s + Q_b$

$Q_u =$ 13.5 Tons

Factor of Safety = 2.5

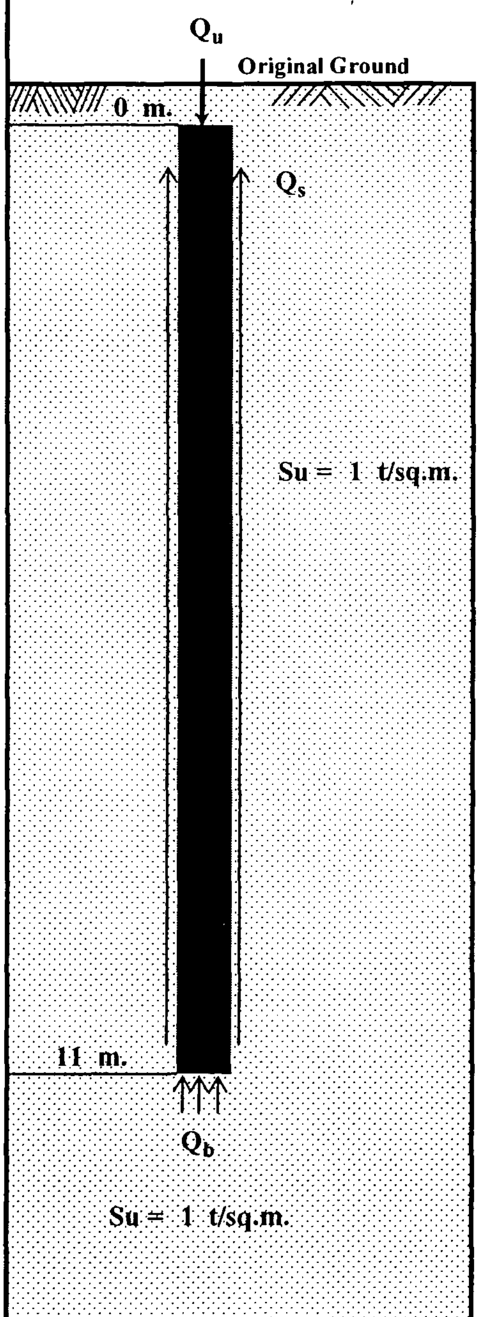
$Q_{safe} =$ 5.41 Tons

Say $Q_{safe} =$ 5 Tons

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	**:**	**Cement Column**
Pile Size	**:**	**Diameter 0.50 m.**
Pile Length	**:**	**11 m.**
Cross Section Area	**:**	**0.196 $m.^2$**
Perimeter	**:**	**1.571 m.**
Depth of Pile Top	**:**	**0 m. below ground surface**
Depth of Pile Tip	**:**	**11 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s =$ 1 x 1 x 17.3 Tons

$Q_s =$ 17.3 Tons

End Bearing in clay layer

$Q_b =$ 9 x 1 x 0.196 Tons

$Q_b =$ 1.8 Tons

$Q_u = Q_s + Q_b$

$Q_u =$ 19.0 Tons

Factor of Safety = 2.5

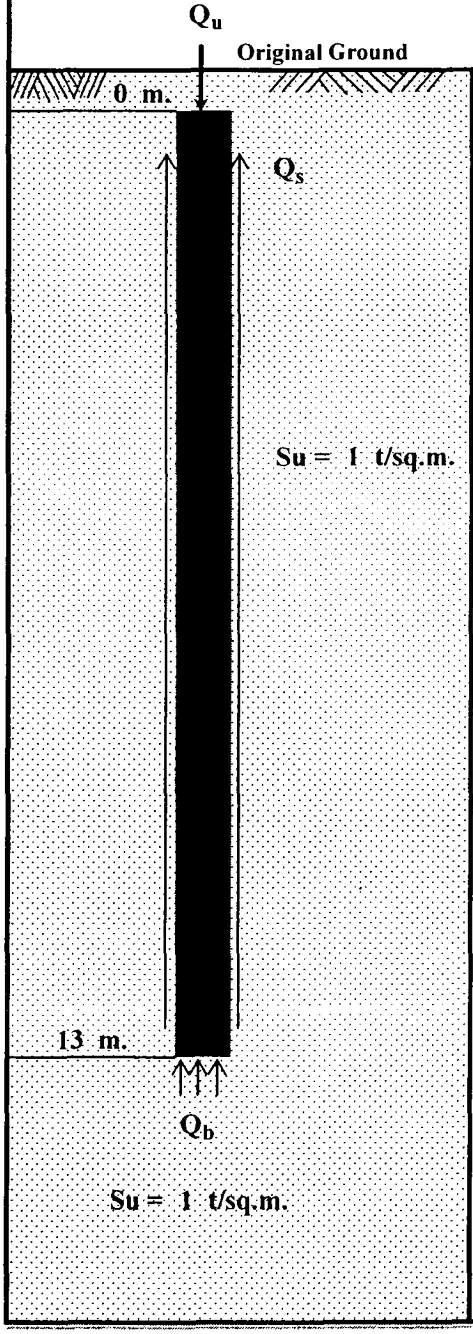
$Q_{safe} =$ 7.62 Tons

Say $Q_{safe} =$ 7 Tons

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	**:**	**Cement Column**
Pile Size	**:**	**Diameter 0.50 m.**
Pile Length	**:**	**13 m.**
Cross Section Area	**:**	**0.196 m.^{2}**
Perimeter	**:**	**1.571 m.**
Depth of Pile Top	**:**	**0 m. below ground surface**
Depth of Pile Tip	**:**	**13 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s = 1 \times 1 \times 20.4$ Tons

$Q_s = 20.4$ Tons

End Bearing in clay layer

$Q_b = 9 \times 1 \times 0.196$ Tons

$Q_b = 1.8$ Tons

$Q_u = Q_s + Q_b$

$Q_u = 22.2$ Tons

Factor of Safety = 2.5

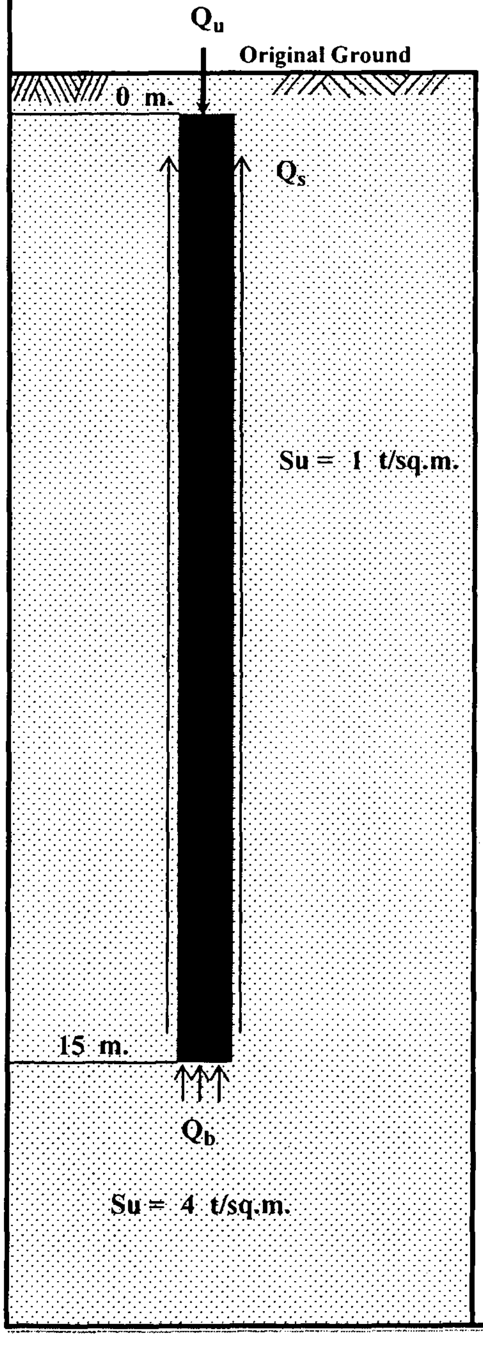
$Q_{safe} =$ 8.87 Tons

Say $Q_{safe} =$ 8 Tons

DESIGN EXAMPLE FOR PILE FOUNDATION

PROJECT : Thesis

LOCATION : Rama II Rd., Bangkok



Type of Pile	:	**Cement Column**
Pile Size	:	**Diameter 0.50 m.**
Pile Length	:	**15 m.**
Cross Section Area	:	**0.196 $m.^2$**
Perimeter	:	**1.571 m.**
Depth of Pile Top	:	**0 m. below ground surface**
Depth of Pile Tip	:	**15 m. below ground surface**

$Q_u = Q_s + Q_b$

$Q_s = K_s \times P'_o \times \tan\delta \times A_s$ (For SAND)

$Q_s = \alpha \times S_u \times A_s$ (For CLAY)

$Q_b = P'_o \times N_q \times A_b$ (For SAND)

$Q_b = N_c \times S_u \times A_b$ (For CLAY)

Skin friction in clay layer

$Q_s = 1 \times 1 \times 23.6$ Tons

$Q_s =$ 23.6 Tons

End Bearing in clay layer

$Q_b = 9 \times 4 \times 0.196$ Tons

$Q_b =$ 7.1 Tons

$Q_u = Q_s + Q_b$

$Q_u =$ 30.6 Tons

Factor of Safety = 2.5

$Q_{safe} =$ 12.3 Tons

Say $Q_{safe} =$ 12 Tons