SETTLEMENT ANALYSIS OF DRILLED SHAFT FOUNDATION ON NATURAL AND LIME-TREATED COMPRESSIBLE CLAYEY SOIL

By

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Presented to the Faculty of the Graduate School of
The University of Texas at Arlington in Partial Fulfillment
Of the Requirements
For the Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

THE UNIVERSITY OF TEXAS AT ARLINGTON

December 2014

Acknowledgements

I would like to express my sincere gratitude and appreciation to my supervising professor, Dr. Anand J. Puppala, for his support throughout the course of my research efforts. I would also like to thank Dr. Shih-Ho Chao and Dr. Xinbao Yu for serving on my thesis committee and giving me valuable comments, suggestions, and advice. Special thanks are extended to Dr. Aravind Pedarla, who guilded, encouraged, and supported me in carrying out this research. It was always a learning experience working with him, and I enjoyed in company on various occasions involving research discussions. I would like to thank Acharya Raju for his assistance in the final stages of preparation. I am also very grateful for my geotechnical lab colleagues for their advice and support for for my father and mother for their unwavering support. Last, but not least, I am appreciative of the encouragement of my family, especially my brothers and sisters, and for the inspiration of my friends, Kedir Wolde and Abdi Hassen, to whom all of my success is dedicated.

November 17, 2014

Abstract

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Drilled shaft foundations are primarily used to support structures such as bridge piers, buildings, and transmission towers. The main advantage of a drilled shaft foundation is that it transfers loads to the stronger subsoil layers or rock strata lying underneath, giving maximum bearing capacity to the overlying structure.

This goal of this research is to load bearing capacity of a drilled shaft on both treated and untreated compressible soil. In this study, both analytical and numerical studies were conducted to investigate the effect of lime treatment on the settlement of the drilled shaft. Extensive literature study was conducted on analysis of axially-loaded drill shafts and novel chemical ground improvement techniques for compressible clay. Basic laboratory tests were conducted, and engineering properties of both natural and lime treated soils were determined. The varying depth-settlement response of a vertically loaded drill shaft was analyzed, on both treated and non-treated clay soils.

Numerical analysis was performed, using PLAXIS 2D and the soil parameters obtained from laboratory tests. The main purpose of using the lime slurry pressure injection method is to reduce the project expenses by improving the capacity of the soil, as well as to increase the load bearing capacity of the foundations where bedrock is very deep .

During this study, three case scenarios were considered, involving varying depths of the drilled shaft, diameter of the drilled shaft, and the depth of the treated soil. The maximum settlement curves obtained analytically from the studies for the natural and treated soils were compared with the curves from the numerical results. Finally, recommendations and advantages of using lime treatment on compressible clays are highlighted.

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Chapter 1

INTRODUCTION

1.1 General

Due to their highly compressible nature and poor performance as a foundation subgrade, compressible or soft soils are usually replaced with borrowed material having good geotechnical characteristics. However In recent years, due to rapid growth in urban areas, construction of civil infrastructure foundations on compressible and problematic soils has become prevalent.

Foundations constructed on soft soils undergo large deformations and associated movements. This often results in costly repairs involving long construction delays and pollution. Working loads, evolving from structures built on compressible soil, raise several concerns, including factors like bearing capacity failures, differential settlements, and instability.

Geotechnical solutions to address the above mentioned concerns include excavation and soil replacement, ground improvement, physical stabilization with deep foundations, and other remediations. These ground improvement engineering techniques have been practiced for more than two decades (Han and Collin, 2005). Improvement of in situ soil strength at the site can be achieved by different stabilization techniques, among which chemical stabilization technique is most prominent.

In this research, an effort was made to study the impact of ground improvement with admixtures, followed by deep foundations on soft compressible clays. It also highlights the importance of deep foundations and chemical additives for ground improvement on soft soils.

Typical foundations on soft clays include bedrock-socketed drilled shafts and piles. However, the main concern arises when the depth of bedrock is very deep. Bridge abutments and piers transfer a lot of load to the underlying foundation subgrade and, in most instances, are socketed in bedrock. Attaining bedrock-embedded deep foundations in such instances is impossible; hence, this research targets novel methods of improving soft clay performance.

The effect of lime treatment on compressible clayey soil and the corresponding strength of the drill shaft foundation were investigated in this study. An effort was made to investigate lime pressure injection method and associated improvements in soil properties, as well as the performance of the drill shaft foundation under axial loading. Based on laboratory tests conducted on natural and treated clay soils, improvement in soil properties was documented and later utilized in 2D finite element analysis, using PLAXIS 8.0.

1.2 Research Objective

The main objective of the present research is to study the effects of lime-treated clay soils and corresponding drill shaft foundations on both untreated and treated compressible clay soil, where bedrock embedment is costly and unachievable. Laboratory testing of both natural and lime-treated soil samples was conducted, and strength parameters were identified. Later, the results were utilized for finite element foundation analysis for designing reinforced concrete drill shaft foundations of different lengths.

Finite element studies were conducted using PLAXIS software, which was developed specifically for the analysis of deformation and stability in geotechnical engineering projects. In order to understand the performance of piles in both natural and chemically-treated compressible clay, the following research objectives and specific tasks were developed:

- Comprehensive literature review on chemical-additive soil stabilization and its application technique to soft and compressible ground. Identification of key soil parameters influencing pile capacities.
- Selection of compressible clay soil and corresponding basic soil characterization studies. Selection of optimum lime dosages and corresponding characterization.
- Laboratory determination of strength parameters of both control and lime-treated soil using direct shear, unconfined compressive strength, and unconsolidated undrained tests .
- Analytical studies conducted on piles constructed on treated clays, and associated improvements in load-carrying capacity of deep foundations involved.
- Finite element studies, using PLAXIS, on shaft foundations on both untreated and treated ground
- Based on analysis of test data, design recommendations.

To summarize, the research tasks are presented in the form of a flow chart in Figure 1.1.

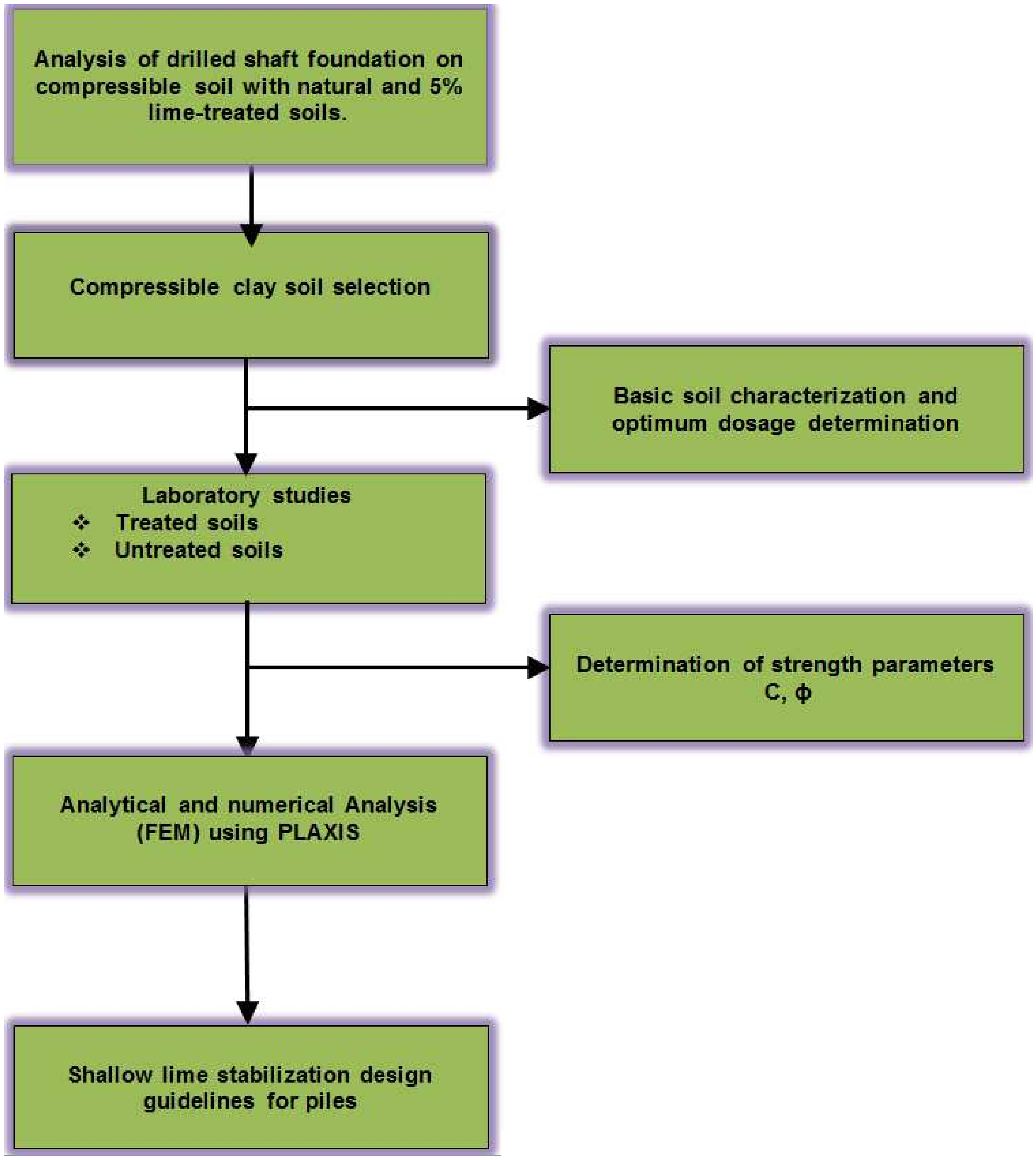

Figure 1.1 Flow chart representing the research task

1.3 Organization and Summary

This thesis consists of the following five chapters. The organization of each chapter is presented below:

Chapter 1 provides an introduction to the current research objective and the necessity of lime treatment on soft ground. Corresponding research objectives formulated for this study are also presented.

Chapter 2 presents a brief introduction to problems associated with soft compressible clays and remedial techniques. Studies related to lime stabilization of soil with the pressure injection method and the soil property interaction are discussed. The advances of finite element analysis in addressing drill shaft foundation are also studied.

Chapter 3 provides information on soil selection and the laboratory testing conducted on test soils. Soil strength property determination tests included direct shear, UU, and UCS tests. pH tests were also carried out for the determination of the optimum lime content of the treated clay soils.

Chapter 4 highlights results obtained from direct shear, unconsolidated undrained (UU), and the unconfined compression strength (UCS) tests. This chapter highlights the analytical determination of drill shaft foundation capacity, using the enhanced strength properties of lime-stabilized soil. Finally, finite element modeling (PLAXIS) analysis was used to assess the load deformation responses of pile foundations. Three scenarios are studied, with varying shaft sizes and treatment depths, under different circumstances

Chapter 5 presents all conclusions of the experimental results, design, as well as future recommendations

Chapter 2

LITERATURE REVIEW

2.1 Introduction

2.1.1 Compressible Clay

Compressible soil may contain layers of very soft materials like clay or peat. These layers undergo compression due to loading from overlying structures or from varying ground water levelsl. This compression eventually results in depression of the ground and disturbance to the foundations (British geological survey,1835).

Peat, alluvium, and laminated clays are common types of deposits associated with various degrees of compressibility. The deformation of the ground is usually a one-way process that occurs during or soon after construction.

If the ground is extremely compressible or is adjacent to structures that apply lesser or greater loads to the ground, the building may sink below the surface. If the compressible ground is not uniform, different parts of the building will sink at different rates or by different amounts (differential settlement). C.C. Swan(1988) conducted studies on the New Jersey Meadow Land complex constructed in the 1980 on the marshlands of the Hackensack River in central New Jersey which underwent differential settlements due to the presence of compressible clays. Due to the soft soil and the placement of compacted fill, significant settlements of around 12 cm were observed:

To mitigate the compressibility characteristics of soils, soil improvement techniques can be applied. The remediation techniques are dependent upon the function and availability of materials. This research focuses on implementing lime-treated soils and studying the performance of deep foundations by finite element analysis..

2.1.2 Soil Remediation Techniques

Chemical stabilization has been widely used in the past few decades. Among the different available additives, lime has proven to be the most successful stabilizer, both in ease of application and optimum performance (Little, 1990).

2.1.2.1 Lime Stabilization

Lime can be used to modify some of the physical properties and thereby improve the quality of soil, or to transfer the soil into a stabilized mass, which increases strength and durability. The amount of lime added depends upon the soil to be modified or stabilized. Generally, lime is suitable for clay soils with PI ≥ 20 % and > 35 % passing the No.200 sieve (0.074). Lime stabilization is applied in road construction to improve subbase and subgrades, for railroad and airport construction, for embankments, for canal linings, for improvement of soil beneath foundation slabs, and for lime piles (anon, 1985 & 1990). Lime stabilization includes the use of burned lime products, quicklime and hydrated lime (oxides and hydroxides, respectively), or lime by-products (codel,1998).

The improvement of the geotechnical properties of the soil and the chemical stabilization process using lime take place through two basic chemical reactions: 1) Short-term reactions including cat ion exchange and flocculation, where lime is a strong alkaline base which reacts chemically with clays, causing a base exchange. Calcium ions (divant) displace sodium, potassium, and hydrogen (monovalent) cations and change the electrical charge density around the clay particles. This results in an increase in the interparticle attraction, causing flocculation and aggregate, with a subsequent decrease in the plasticity of the soils. 2) Long-term reactions, including pozzolanic reaction, where calcium from the lime reacts with the soluble alumina and silica from the clay in the presence of water to produce stable calcium silicate hydrates (CSH), calcium aluminate hydrates (CAH), and calcium alumino-silicate hydrated (CASH) which generate long-term strength gain and improve the geotechnical properties of the soil. These hydrates were observed by many researchers (Diamond et al., 1964; Sloane, 1965; Ormsby & Kinter, 1973; and Choquette et al,1987). The use of lime for soil stabilization is either in the form of quicklime (CaO) or hydrated lime $CA(OH)_{2..}$ Agricultural lime or other forms of calcium carbonate, or carbonated lime will not provide the reactions necessary for improving the subgrade soils mixed with lime. The basic lime stabilization mechanism is shown in Figure 2.1.

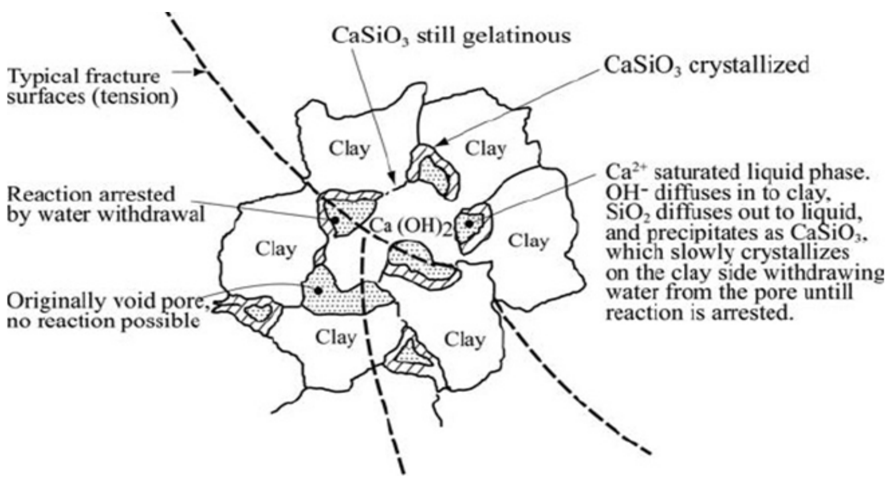

Figure 2.1 Basic lime stabilization mechanisms (Ingles & Metcalf, 1972)

2.1.2.2 Lime Slurry Pressure Injection

Lime slurry pressure injection is a stabilization operation that is used by the construction industry to improve the geotechnical performance of problematic compressible soils that persistently fail to meet requirements. Successful lime injection requires a matching of the application to local soils and conditions. A thorough investigation of the soil to be stabilized must be made. Hy-Rail drilling trucks capable of drilling 15-20 ft. or more in depth aid in securing samples for laboratory tests, obtaining a height of water table and other subsurface information. The reaction between various slurry compositions and the local soil must be established and laboratory tests conducted to determine the likelihood, if any, for a successful application. Just how lime injection stabilizes is not fully understood. Soil chemistry is an important factor. The degree of acidity or alkalinity, that is, the pH value, must be ascertained; iron and organic content must be at a minimum. Whether harm or good comes from injecting lime into a non-compatible soil is still an unanswered question.

Not all soils respond to lime injection. Lime slurries have shown merit in stabilizing clays with a high liquid limit and plasticity index. A 6% lime addition lowered the plastic indexes from 70 and 30 to 5 and 8 in two clay samples.

This method of soil improvement involves the use of advanced pressure injection equipment, which forces the lime to target depths in problematic subgrade soils. The high pressure fluids are injected into the soil at high velocities. They break up the soil and play an important role in foundation stability, particularly in the treatment of foundation ground under new and existing buildings Shibasaki (1996). Lime slurry agents are injected under a typical hydraulic pressure and cease when slurry is observed breaking out at the surface, or when a maximum pressure of 1,450 KPa is reached (Kayes et al. 2000).

The LPSI process involves injecting hydrated lime slurry under pressure to a depth of 1 to 3.1 m (3 to 10 feet), and occasionally 12.3 m(40 feet) or more. This procedure is repeated in staged intervals of typically 1–2 m to achieve a complex network of chemically-active slurry seams intersecting subgrade strata. The lime injection process is illustrated in Figure 2.2,(Thiele and Adamson 2006); Figure 2.3, high pressure injection (NIA Bulletin 331, (2006); and Figure 2.4, high pressure injection (Brinkgreve R.B.J (2006).

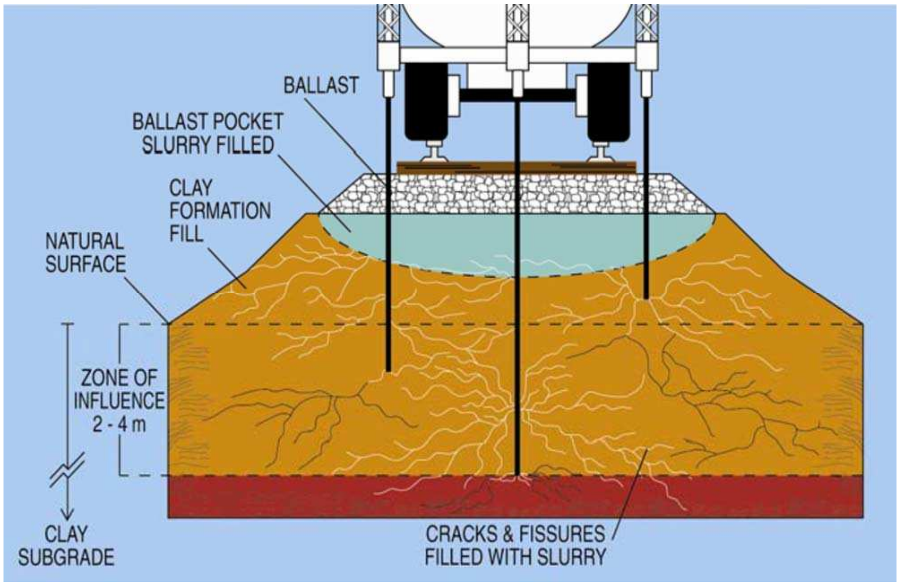

Figure 2.2 Lime slurry pressure injection method(Thiele and Adamson 2006)

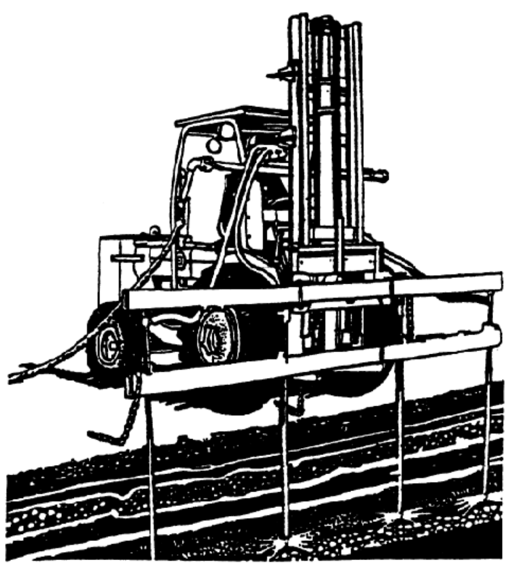

Figure 2.3 LSPI moves through the soil by following the paths of least resistance (NIA Bulletin 331, (2006).

Figure 2.4 High-pressure injections (Brinkgreve R.B.J (2006).

The amount of lime required for LSPI treatment can vary considerably, depending on soil properties, injection depth, permeability of the soil mass, and degree of stability required.

Although a general perception is that a soil mass should be dry and highly fractured and fissured to accommodate the flow of LSPI for stabilization, Bulletin 331 states that even when clays are wet, the fissures are still present due to the non-elastic nature of soil. However, when a "tighter," more-plastic and less-fissured clay is encountered, it is usually necessary to use closer spacing and more than one injection pass. Figure 2.5 illustrates a typical grid pattern of LSPI.

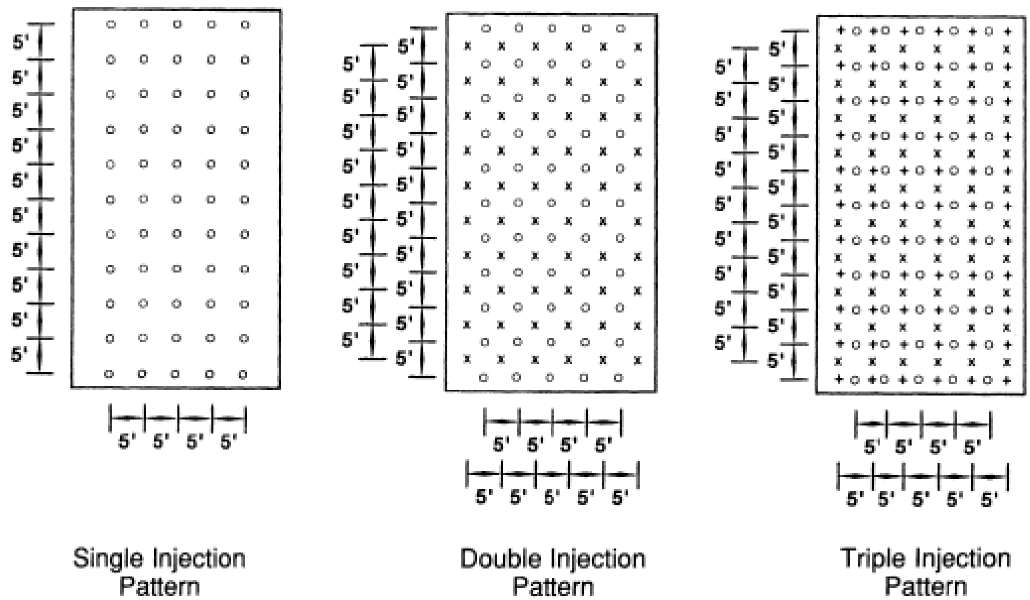

Figure 2.5 illustrates a typical grid pattern of LSPI (NIA BULLETIN 331, 2006).

2.1.2.3 Stabilized Compression Tests

As illustrated in Figure 2.6, the purpose of this test is to determine the additional strength gained by the action of the stabilizing mixture The stabilized samples are compared to unstabilized samples, prepared, and cured in a similar manner to treated samples.

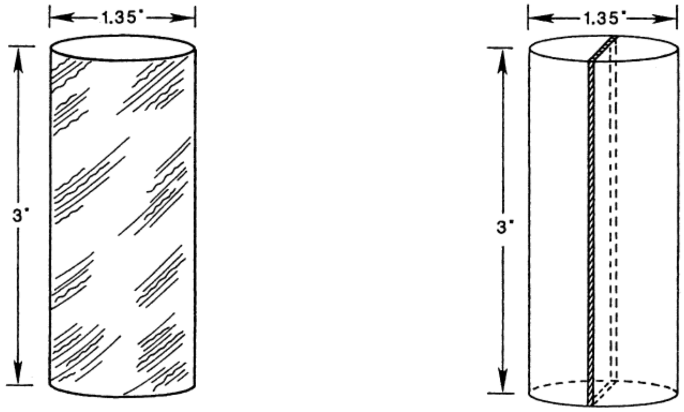

Figure 2.6 Glazed stabilized compression test (Blacklock, 1977).

The soil samples are prepared for both natural and 5% treated samples, and laboratory tests are carried out to investigate the parameters to be used for the analysis.

2.2 Deep Foundation

Drill shaft pile foundations are columnar elements in a foundation. They transfer loads from the super structure through weak compressible strata or water, onto stiffer or more compact and less compressible soils or rock. Drill shaft foundations used in marine structures are subjected to lateral loads from the impact of berthing ships and waves. Combinations of vertical and horizontal loads are carried where drill shaft foundations are used to support retaining walls, bridge piers and abutments, and machinery foundations.

Until recently, predictions of the settlement of drill shaft pile foundations, if made at all, have generally been based on empirical data or simplified one-dimensional consolidation approaches (Kawasaki et al, 1978). With the development of numerical techniques and the increased use of computers, increased efforts have been directed toward making more rational analyses of drill shaft pile foundation settlement behavior.

The currently available theoretical approaches may be classified broadly into three categories:

1) Methods based on the theory of elasticity, which employs the equations of Mindlin (1936) for subsurface loading within a semi-infinite mass;
2) Step-integration methods, which use measured relationships between pile resistance and pile movement at various points along the drill shaft pile foundation;
3) Numerical method and, in particular, the finite element method.

Methods in the first category have been described by several investigators, e.g., D'Appolonia and Romualdi (1963), Thurman and D'Appolonia (1965), Salas and Belzunce (1965), Poulos and Davis (1968), Mattes and Poulos (1969). All rely initially on the assumption of the soil as a linear elastic material, although more realistic soil behavior can be incorporated readily into the analyses in an approximate manner. Such methods also provide a relatively rapid means of carrying out parametric analyses of the effects of the drill shaft pile foundation and soil characteristics, and of preparing a series of solutions which can be used for design purposes. Moreover, the settlement of drill shaft pile foundation groups can be analyzed by a relatively simple extension of the single drill shaft pile foundation analysis.

The head load, P_0 is transferred to the surrounding soil by shear stresses (skin friction) along the lateral pile/soil interface and by end-bearing at the pile tip. The rate at which the head load is transferred to the soil along the drill shaft pile foundation and the overall deformation of the system are dependent on numerous factors. Among these are:

1) The cross section geometry, material, length, and, to a lesser extent, the surface roughness of the drill shaft pile foundation;

2) The type of soil (sand or clay) and its stress-strain characteristics;

3) The presence or absence of groundwater;

4) The method of installation of the pile; and,

5) The presence or absence of residual stresses as a result of installation.

An experience-based techniques approach has been followed to reduce the complex three-dimensional problem to a partially one-dimensional model, which is practicable for use in a design environment.

2.2.1 General Construction Methods

In principle, construction of drilled shafts is a very simple matter. A hole is drilled into the ground and concrete is placed into the hole. The practice is more complex:

1. The hole must be excavated, sometimes to great depths through very difficult and variable materials ranging from soft soils to hard rock.

2. The hole must then be kept open and stable, often at great depths in caving soils below the groundwater table, without adversely affecting the bearing stratum.

3. The reinforced concrete must be cast in the excavated hole in such a way as to ensure good bond and bearing into the founding stratum in order to transfer large axial loads to the founding stratum.

4. The completed drilled shaft must be a competent structural element that provides sufficient structural strength in compression, tension, and flexure to transfer the loads from the structure.

For a general discussion of construction methods, the approach to construction can be classified into three broad categories. These are: 1. The dry method: (a) Drill; (b) Complete and clean excavation, set reinforcement ; (c) Add concrete. 2. The casing method: (a) Drill with slurry; (b) Set casing and bail slurry; (c) Complete and clean excavation, set reinforcing; (d) Place concrete to head greater than external water pressure; (e) Pull. 3. The wet method: (a) Set starter casing; (b) Fill with slurry; (c) Complete and clean excavation, set reinforcing; (d) Place concrete through tremie; (e) Pull tremie while adding concrete.

In many cases, the installation will incorporate combinations of these three methods to appropriately address existing subsurface conditions. Because elements of the drilled shaft design can depend on the method of construction, it must be considered.

2.2.2 Design Consideration

2.2.2.1 General Considerations

Two factors must be considered in the design of drilling shafts. First, there must be an adequate factor of safety against bearing failure. Second, the settlement of drilled shafts at working load must be limited to a value that will not cause structural or esthetic damage to the bridge they support. The design criteria developed during this study incorporates these two factors.

In clay soils, embedment must be adequate to prevent excessive compression. In such soils, lower portions of the shaft may go into tension as the upper soils swell; hence, adequate reinforcement must be provided. Shafts may be anchored in expansive soils by belling into a stable, non-expansive stratum. In many areas of the Southwest, stable strata is not reached at reasonable depths, and heave will occur despite the best efforts of the designer and drilling contractor. Many clay shales present a problem in this respect. In such cases, the structure must be flexible enough to withstand differential movements, or an alternative foundation design needs to be employed. Since drilled shafts resist load through a combination of end bearing and skin friction, the capacity of a drilled shaft can be calculated either by employing preemptive values for end bearing and side friction based on a physical description of the soil (O'Neill and Reese, 1970), or by a rational limiting equilibrium procedure. The design procedure recommended herein employs the limiting equilibrium procedure, in which Eq. 2.1 is used

$$(QT)_{ult} = (QS)_{ullt} + (QB)_{ult} \tag{2.1}$$

Where, QT_{ULT} is the ultimate axial load capacity of the shaft, $(Q_S)_{Ult}$ is the ultimate capacity of the sides, QB_{ULT} is the ultimate capacity of the base.

Where e ultimate side and base capacities are calculated independently from the results of laboratory tests on representative soil samples or from subsurface penetrometer soundings.

The following expressions are used to calculate the ultimate side and base resistances in predominantly clay profiles:

$$QS_{ult} = Q'avg\ S \tag{2.2}$$

$$(QB)\ Ult = N_C\ C\ A_B \tag{2.3}$$

$\propto Avg$ is the ratio of the peak-mobilized shear stress to the shear strength of the soil averaged over the peripheral area of the stem; S= shear strength of the soil; NC= bearing capacity factor; C = average undrained cohesion of the soil for a depth of two base diameters beneath the base ("Shear strength" may be substituted for "cohesion" for soils having an undrained angle of internal friction of 10° or less); AS is the peripheral area of the stem; AB is the area of the base.

Many studies have been reported in which the values for α_{AVG} and N have been measured for driven piles. However, since the disturbance and stress changes in the soil due to the installation of a drilled shaft are not the same as for driven piles, it is not logical to assume that α_{AVG} and N are the same for drilled shafts and driven piles. Below are the reasons for initiating another research study.

1. Remolding of the borehole walls during drilling;

2. Opening of cracks or fissures in the soil during and after drilling;

3. Migration of excess water from the concrete into the soil, thereby softening (and weakening) the soil;

4. Shrinking of surface soils and mechanical interaction between the shaft and soil near the base (O'Neill and Reese, 1970);

5. Use of drilling mud during construction.

2.2.2.2 Shafts in Clays

The net load-carrying capacity at the base (that is, the gross load minus the weight of the pier) may be approximated as

$$Qp\ (net)\ =\ Ap\ (CNc*\ +q'\ (Nq*-q')\ =\ Ap\ [CNc*\ +q'\ (N)] \tag{2.4}$$

From Eq. (2.4), for saturated clays with Φ = o, $Nq*=\ 1$; hence, the net base resistance becomes

$$Q_{p\ (net)} = A_P\, C_u\, N_C* \tag{2.5}$$

Where C_u = undrained cohesion

The bearing capacity factor N_c^* is usually taken to be 9. When the L/D_b ratio is 4 or more, Nc^* = 9, which is the condition for most drilled piers. Experiments by Whitaker and Cooke (1966) showed that, for belled piers, the full value of $Nc*$ = 9 is realized with a base movement of about 10 -15 % of D_b. Similarly, for piers with straight shafts ($D_b\ =\ D_s$), the full value of Nc* = 9 is obtained with a base movement of about 20 % of D_b.

The expression for the skin resistance of piers in clays is similar to Eq.

$$Q_s = \sum_0^L L\ \alpha * Cu \tag{2.6}$$

Where p = perimeter of the pier cross section

The value of α* that can be used in Eq. (2.6) has not yet been fully established.

However, the field test results available at this time indicate that α* may vary between 1.0 to 0.3.

Kulhawy and Jackson (1989) reported the field test results of 106 straight-shafted piers - 65 in up lift and 41 in compression. The magnitude of α* obtained from these tests is shown in Figure (2.7). The best correlation obtained from these results is

$$\alpha^* = .21 + .25(Pa / C_u) \leq 1 \qquad (2.5)$$

Where Pa = atmospheric pressure = 1.058 ton/ft^2 (101.3 kpa)

So, conservatively, we may assume that

$$\alpha^* = 0.4 \qquad (2.6)$$

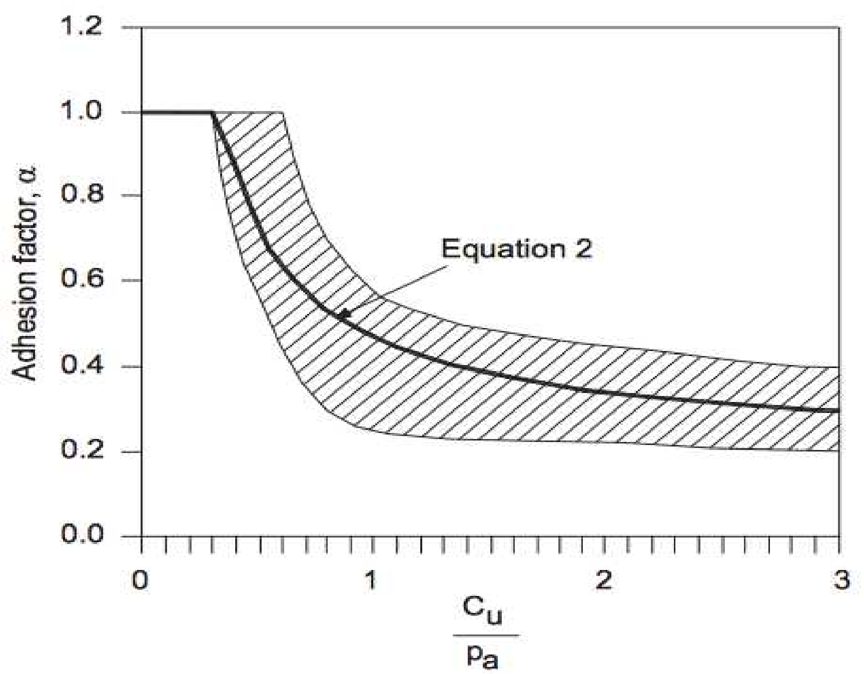

Figure 2.7 Variation of α* with Cu/Pa (Kulhawy and Jackson, 1989)

2.2.2.3 Settlement of Piers at Working Load

The settlement of drilled piers at working load is calculated in a manner similar to the settlement of piles under a vertical working load, $Qw,$ is caused by three factors:

$$S = S_1 + S_2 + S_3 \tag{2.7}$$

Where, S = total pile settlement, S_1 = settlement of drilled shaft, S_2 = settlement of drilled shaft caused by the load at the pile point, S_3 = settlement of drilled shaft caused by the load transmitted along the shaft

Determination of S_1

If the drilled shaft material is assumed to be elastic, the deformation of the drilled shaft can be evaluated using the fundamental principles of mechanics of materials:

$$S_1 = (Q_{WP} + \zeta Q_{WS})/A_P E_P \tag{2.8}$$

Where

Q_{WP} = load carried at the pile point under working load condition;

Q_{WS} = load carried by frictional (skin) resistance under working load condition;

A_P = area of pile cross section;

L = length of pile;

E_P = modulus of elasticity of the pile material;

The magnitude of ξ will depend on the nature of unit friction (skin) resistance distribution of f is uniform or parabolic, ξ = 0.5. However, for triangular distribution of f, the magnitude of ξ is about 0.67 (Vesic, 1977).

Determination of S_2

The settlement of a pile caused by the load carried at the pile point may be expressed in a form similar to that given for shallow foundation

$$S_2 = q_{WP}\, D\, (I - \mu s_2)\, I_{WP} / E_S \qquad (2.9)$$

Where D = width or diameter of pile

Q_{WP} = point load per unit area at the pile point = $\frac{Q_{WP}}{A_P}$;

E_S = modulus of elasticity of soil at or below the pile point;

μ_S = Poisson's ratio of soil;

I_{WP} = influence factor.

For all practical purposes, I_{WP} equals α_r. In the absence of experimental results, representative values of Poisson's ratio may be obtained from Table 2.3 (BRAJA M. DAS, 1983).

Vesic (1977) also proposed a semi empirical method to obtain the magnitude of the settlement, S_2 :

$$S_2 = Q_{wp} C_p / D_{qp} \qquad (2.10)$$

Where Q_P = ultimate point resistance of the drill shaft

C_p = an empirical coefficient

Representative values of Cp for various soils are given in Table 2.4.

Table 2.1 Typical values of Cp (source BRAJA M.DAS, 1983)

Soil type	**Bored Shaft**
Sand (dense to loose)	0.09-.18
Clay (stiff to soft)	0.03-0.06
Silt (dense to loose)	0.09-0.12

Determination of S_3

The settlement of a drilled shaft caused by the load carried by the shaft is given by a relation similar to Eq. 2.10, or

$$S_3 = (Q_{WS}/pL)\, D\, (1 - \mu s_2)\, I_{WS} E_s \qquad (2.11)$$

Where p = perimeter of the pile

L = embedded length of pile

I_{ws} = influence factor

Note that the term Q_{ws}/ PL in Eq.2.11 is the average value of f along the pile shaft. The influence factor, I_{WS}, has a simple empirical relation (Vesic, 1977):

$$I_{ws} = 2 + 0.35 \sqrt{L/D} \qquad (2.12)$$

Vesic (1977) also proposed a simple empirical relation similar to Eq.2.10 for obtaining S_3:

$$S_3 = Q_{ws} C_s \qquad (2.13)$$

Where C_s = an empirical constant = $(0.93 + 0.16 \sqrt{L/D})\, C_p$

The values of C_P for use in Eq.2.13 may be estimated from Table 2.4.

Sharma and Joshi (1988) used in all equations.

2.2.2.4 Reese and O'Neill

Based on a database of 41 loading tests, Reese and O'Neill (1989) proposed a method to calculate the load bearing capacity of drilled piers. The method is applicable to the following ranges.

1. Shaft diameter: D_s = 1.7 ft. to 3.93 ft. (0.52 m. to 1.2 m.)
2. Bell depth: L = 15.4 to 100 ft. (4.7 m to 30.5 m)
3. C_u = 600 lb. / ft^2 to 6000 lb. / ft^2 (29 kpa to 287 kpa)
4. Standard penetration resistance: N = 5 to 60
5. Over consolidation ratio = 2 to 15
6. Concrete slump 4 in. to 9 in. (100 mm to 225 mm)

Reese and O'Neill's procedure,

$$Qu = \Sigma\, N_{i=1}\, f_i\, p\, \Delta L_i + q_p A_b \qquad (2.14)$$

Where

f_i= ultimate unit shearing resistance in layers

P = perimeter of the shaft

Q_p= unit point resistance

A_b = area of the base

Cohesive soil

Based on eq. (2.14)

$$f_i = \propto_i{}^* c_u \quad (2.15)$$

The following values are recommended for α*: α* = 0 for top 5 ft. (1.5 m) and bottom, Ds, of the drilled shaft. (Note: If D_b > $D_{s,}$ then α* = 0 for 1 diameter above the top of the bell and for the peripheral area of the bell itself.)

α* = 0.55 elsewhere And

Q_p = 6C_{ub} (1+ 0.2 L/D_b) ≤ 9 C_{ub} ≤ 80 kip/ft^2 (3.83 MN/m^2) (2.16)

Where C_{ub} = average undrained cohesion within 2D_b below the base If D_b is large, excessive settlement will occur at the ultimate load per unit area, q_p, as given by Eq. (2.16). Thus, for $D_b > 75$ in (1.91), Q_p may be replaced by q_{pr}, or

$$Q_P = F_{qp} \quad (2.17)$$

Where

F_r = 2.5 / (ψ$_1$$D_b$ (in) + ψ$_2$ ≤ 1

ψ_1 = 0.0071 + 0.0021 (L/$D_{b)}$ ≤ 0.015

ψ_2 = 1.125(C_{ub}) $^{0.5}$ (0.5 ≤ Ψ ≤ 1.5)

To do so

1. Select a value of settlement, s.
2. Calculate $\Sigma N_{i=1}\, fi\ p\ \Delta Li\ and\ qpAb,$ as given in Eq. (2.6).

3. The calculated values in step 2 determine the side load and the end-bearing load.
4. The sum of the side load and the end-bearing load gives the total applied load.

.

2.3 Finite Element Studies

In the past decades, various numerical techniques have been developed and successfully applied to a wide range of geotechnical problems. In the full numerical analysis approach, attempts are made to satisfy all theoretical requirements, including realistic soil constitutive models and boundary conditions that realistically simulate field conditions. Approaches based on finite differences, boundary elements, and finite element methods are those most widely used. These methods essentially involve computer simulation of the history of the boundary value problem from green field conditions through construction and into the long term. Their ability to accurately reflect field conditions essentially depends on the ability of the constitutive model to represent real soil behavior and correctness of the boundary conditions imposed.

The finite element method has diverse applications in the field of geotechnical engineering. During the past decades, various numerical techniques have been developed and successfully applied to a wide range of geotechnical problems. In the full numerical analysis approach, attempts are made to satisfy all theoretical requirements, including soil constitutive models and boundary conditions that realistically simulate field conditions. Approaches based on finite differences, boundary elements, and finite element methods are those most widely used. These methods essentially involve computer simulation of the history of the boundary value problem of from green field conditions, through construction and in the long term. Their ability to accurately reflect field conditions essentially depends on the ability of the constitutive model to represent the real soil behavior and correctness of the boundary conditions imposed.

2.3.1 Full-Scale Single Shaft Under Vertical Load

In this research conducted by Brinkgreve (2004), a full-scale single shaft under vertical load was tested and analyzed. The same shaft was analyzed by PSI (Pile-Soil Interaction), which is a 3-D finite element, and the results were compared to the results obtained by using PLAXIS 2D, PLAXIS 3D , and by measured performance.

The shaft, with 1.3 m diameter and 9.5 m length, was constructed in over-consolidation clay. The parameters of the soil profile are shown in Table 2.2. The loading system included two hydraulic jacks, one reaction beam, and sixteen anchors supporting the reaction beam. In the PSI analysis, 20-node cubic elements were used. Because of the symmetry condition, only one-fourth of the pile-soil system was modeled and analyzed, as shown in Figure 2.8. One-fourth of the soil volume was 25-m by 25-m, and 16-m deep. The vertical load at the shaft top was modeled by the equivalent joint loads. The concrete shaft properties used in the linear elastic model were: Young's modulus $\mathrm{E} = 3 \times 10^7$ kpa, Poisson's ratio v =0.2 , and unit weight γ= 24 kn/m^3. Three coefficients of earth pressure values were considered: 1) $k_o = 1 - \sin\varphi$, 2) $K_o = \frac{\upsilon}{1-\upsilon} = 0.43$, and 3) $K_o = 0.80$ for over consolidation clay and other soil properties as shown in Table 2.2.

Table 2.2 Material parameter for soil data (Brinkgreve, 2004)

Parameter	**Value**	**Unit**
Material model	Mohr-Coulomb	-
Type of material behavior	Drained	-
Gravity ,γ_s	20	KN/M^3
Young's modulus, E_S	60000	Kpa
Poisson's ratio, v	0.3	-
Cohesion, c	20	Kpa
Friction angle, □	22.7	Deg.
Dilatency angle,Ψ	0	Deg.

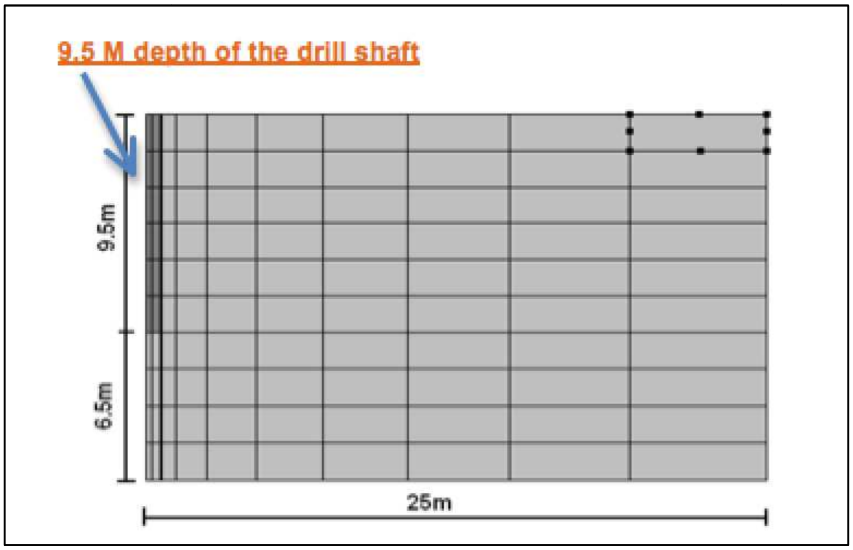

Figure 2.8 Side and 3d view of finite element mesh. (F.T Schuschnigg, and H. Schweiger)

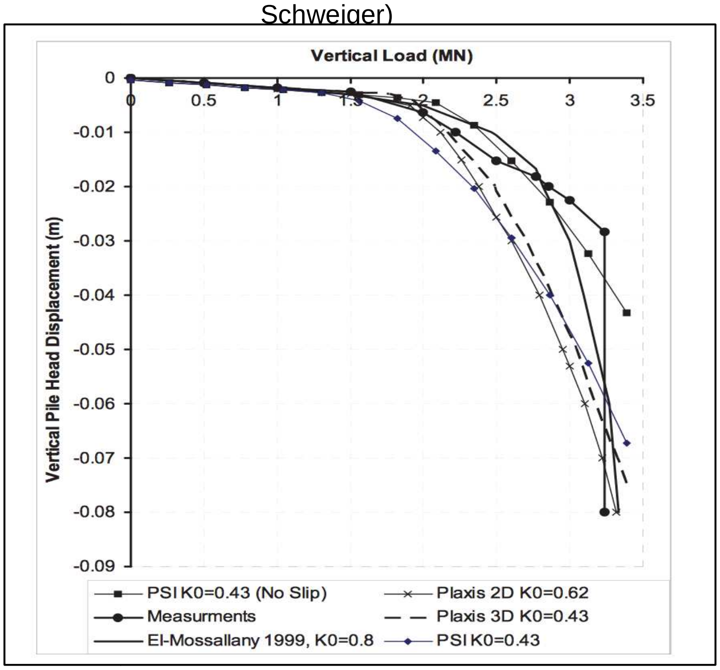

Figure 2.9 Comparison between PSI, PLAXIS, BEM, and test results. (F.Tschuchnigg and H. Schweiger, (2006)

Under a given load, as depicted in Figure 2.9, all the results of the numerical analysis agree that the initial stresses and PSI model are close to the PLAXIS 3D, but not the PLAXIS 2D at the same initial condition. The PSI analysis with no soil shaft interface slip gives the best agreement with the test results.

2.4 Summary

This chapter provides an introduction to the problems associated with compressive soils and correspoinding chemical improvement techniques. Previous studies conducted on the lime stabilization and pressure injection methods are detailed in this chapter. Studies were conducted on drilled shafts, and a case study was done of finite element analysis on vertically-loaded, single-loaded shafts. The next chapter details soil selection and basic soil characterization studies conducted. Details of the engineering test equipment and procedures are also presented.

Chapter 3

EXPERIMENTAL PROGRAM

3.1 Introduction

The experimental program for the current research involved basic soil characterization and assessment of strength improvement for 5% lime-treated soil. This chapter contains the procedural details. Basic soil characterization studies, direct shear tests, unconfined compressive strength or UCS, and unconsolidated-undrained or UU test procedures are presented in this chapter.

3.2 Site Selection

Soil from SH-183, which has been adopted for this research study, was collected from the SH-183 bridge project located between Decatur and Fort Worth, Texas. The 304-meters long bridge project includes two construction phases. During the first phase, the west lane was demolished, and the second phase will commence after the first lane is open to traffic. The work consisted of 70 pieces of 1.0 meter-diameter drilled shafts to depths ranging from 20 m to 40 m in depth.

 

a) Drill the hole b) Putting tremie after casing

c) Slurry after pouring concrete

Figure 3.1 SH-183 bridge project. a) Drill the hole; b) Putting tremie after casing ; c) Slurry after pouring concrete

The hole must then be kept open and stable, often at a great depths, in caving soils below the groundwater table, without adversely affecting the bearing stratum. This requires complete and clean excavation, and set reinforcement. The reinforced concrete must be cast in the excavated hole in such a way as to ensure good a bond and bearing into the founding stratum.

Figure 3.2 Site for SH-183 TXDOT PROJECT (Google map)

3.3 Soil Sampling and Laboratory Testing

The collected soil samples were subjected to laboratory tests, including specific gravity, sieve analysis, Atterberg limits, and standard proctor.

Figure 3.3 Manual soils sampling

3.3.1 Basic Soil Properties Tests

3.3.1.1 Specific Gravity

In soil mechanics, specific gravity of soil solids, represented by G_s, is an important parameter used to calculate the soil weight-volume relationship. ASTM D 854 provides the definition of the specific gravity of soil solids as the ratio of the density of soil solids to the density of an equal volume of water. In this study, the test to determine the specific gravity of soil solids was conducted as per ASTM D 854 standard test methods.

3.3.1.2 Atterberg Limits

Liquid limit (LL), plastic limit (PL) and shrinkage limit (SL), are necessary to correlate the consistency of the soils. The water content, as the boundaries of these states, are well known as shrinkage (SL), plastic (PL) and liquid (LL) limits, respectively (Lambe and Whiteman 2000). Therefore, the PL can be determined by the amount of water content at which the soil starts crumbling when rolled into a 1/8-inch diameter thread. In addition, LL is measured as the water content at which the soil flows. The difference between LL and PL values is called plasticity index (PI), which characterizes the plasticity nature of the soil. In this test, soil samples from intermediate depths are subjected to Atterberg limits tests to determine LL and PL as per Tex-104-E and Tex-105-E, respectively.

3.3.2 Basic Soil Properties

All representative soil samples used in this research were collected from SH-183 located in the city of Fort Worth, Texas, and soil samples were subjected to Atterberg limit tests to determine all basic soil properties. Table 3.1 presents a summary of various physical characteristics of these soils.

Table 3.1 Physical properties of the sample soil

Physical properties of the sample soil	
Percent Passing No. 200 Sieve	57 %
USCS Classification	CL
Liquid Limit, LL	46
Plastic Limit, PL	18
Plasticity Index, PI	28
Specific Gravity,Gs	2.70

3.3.3 Standard Proctor Compaction Test Results

Standard Proctor compaction tests were performed to establish compaction relationships of the soils as per ASTM D698 and AASHTO T99. In this experimental study, determined results are illustrated next sections.

The standard compaction curve obtained for this soil is shown in Figure 3.4. This clay attained a maximum dry density of 110.5pcf(17.35 kn/m^3) at 13.4 % moisture content.

Proctor Compaction Curve

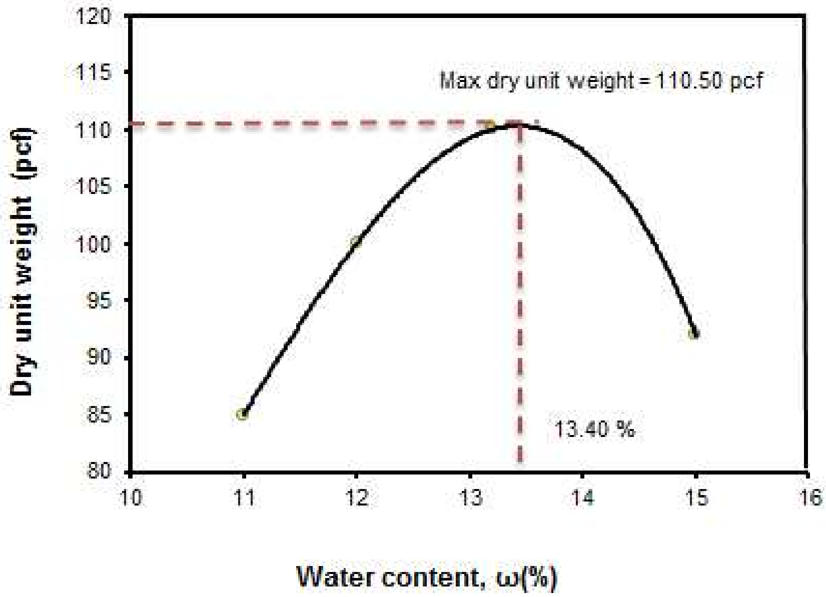

Figure 3.4 Proctor Compaction Curve

Table 3.2 Physical properties of the sample soil

Moisture content (%)	Wet OMC OMC Dry OMC	12.80 13.40 14.20
Dry Density (pcf)	Wet OMC OMC Dry OMC	104.95 110.50 104.95

3.4 Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization

This test identified the lime content required to satisfy immediate lime-soil reactions and still provide significant residual calcium and a high pH (about 12.4 at 24℃).

This was necessary to provide proper conditions for the long-term reaction that is responsible for strength and stiffness development.

This test method provided a means for estimating the soil-lime proportion requirement for stabilization of a soil. It was performed on soil passing through a 425-μm (No. 40) sieve. The optimum soil-lime proportion for soil stabilization was determined by tests of specific characteristics of stabilized soil, such as unconfined compressive strength.

Dry soil was screened through a No. 40 sieve. Lime dosages of 0, 2, 3, 4, 5 and 6 were tested in accordance with ASTM D 6276. Special attention was given to maintaining the room temperature at 25℃, as pH of lime- soil mixture is temperature dependent. The sample and test results are shown in Figure 3.5, 3.6, and 3.7 below .

Figure 3.5 Lime-treated soil Sample for pH Test

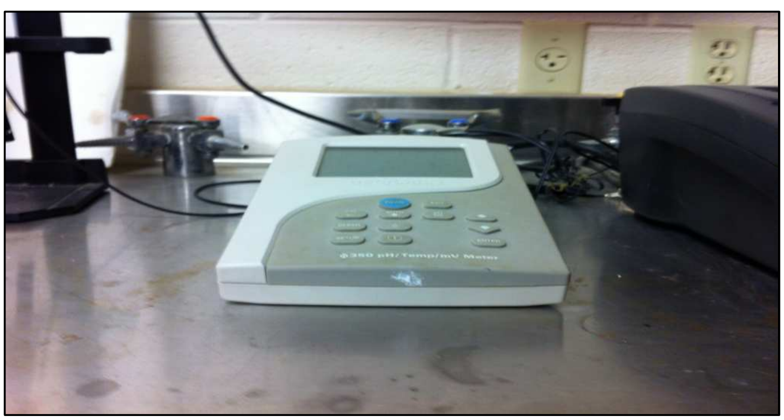

Figure 3.6 pH test apparatus

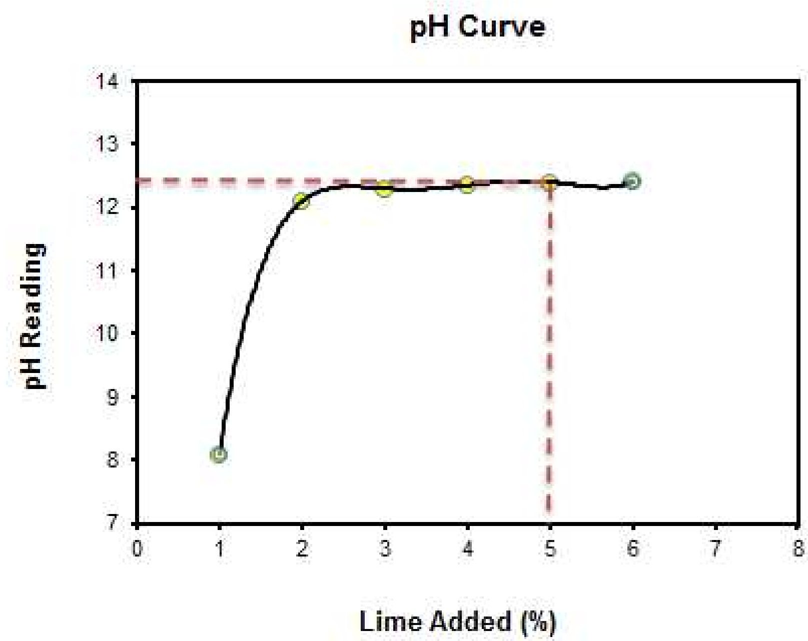

Figure 3.7 pH Test Curve

From the test results, it was observed that the maximum pH of 12.5 was achieved at 5% lime dosage. Hence, this dosage is considered optimum for the present study.

3.5 Unconfined Compressive Strength (UCS) Test

The unconfined compressive strength (UCS) test was performed in accordance with the ASTM D 2166 standard. The primary objective of the UCS test is to determine a compressive strength for soils which possess ideal cohesion The test procedure was initiated by placing the soil sample on the loading platform. A top cap was placed on top of the sample. The loading platform was raised slowly, until the top cap on the soil specimen touched the top plate of the triaxial setup. An external LVDT (Linear Variable Displacement Transducer) was connected so that its tip touched the top portion of the top plate of the triaxial setup. Once the setup was ready, the test progressed by inducing a lift to the soil specimen at a constant rate. As the specimen was raised, the LVDT measured the displacement, while the load cell measured the applied load. The test was stopped when the sample started to show signs of cracking. The load required to cause the sample to fail was noted, and the axial stress was thus calculated. The maximum axial stress obtained was the unconfined compressive strength of the soil. Figure 3.8 illustrates the equipment employed in this research to perform the unconfined compression test.

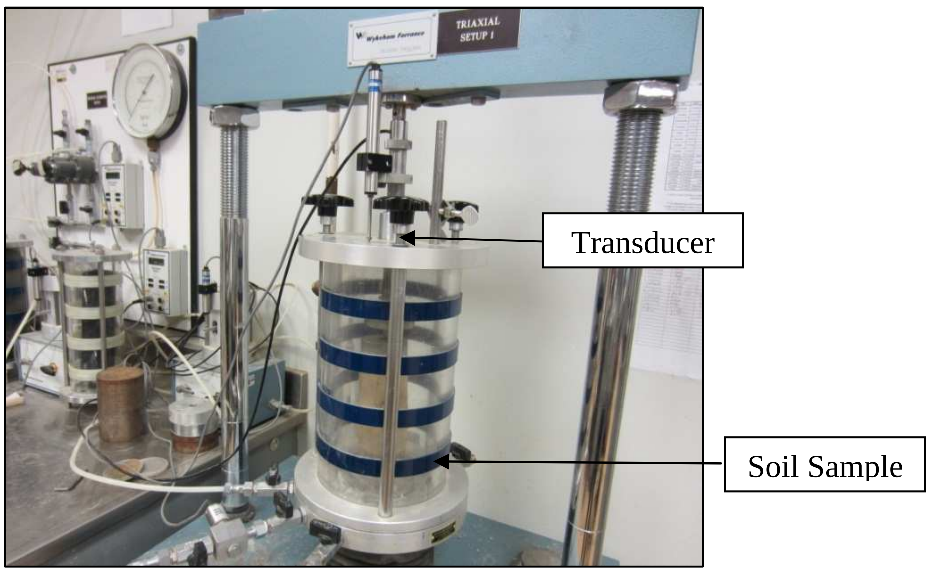

Figure 3.8 UCS test experimental setup

3.6 Unconsolidated- Untrained or UU Triaxial Tests

The unconsolidated-undrained test is also a measure of shear strength parameters (c and □). This test was performed using ASTM D 2859-95 (2003), titled “Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.” The test procedure required placing a cylindrical soil specimen, sealed by the rubber membrane, in a triaxial chamber and then applying a confining pressure by not allowing water to dissipate from the soil specimen. After that, the soil specimen was tested by applying deviatoric loading. The failed soil specimen is shown in the Figure 3.9 below.

Tests were performed under two different scenarios. The first one was related to natural case, and the second case was treated soil sample. The specimens were prepared at the maximum density and optimum moisture content. After the preparation process, the soil specimens were cured in the moisture room for 7 days to make the moisture content inside the soil specimen homogeneous and to allow the time necessary for reactions to take place.

Figure 3.9 Failed Soil Specimens

Both direct shear and UU triaxial tests were conducted to determine the shear strength and stress- strain relationships of the soils at optimum moisture content (OMC) states in the soil. Both direct shear and Triaxial UU tests were conducted for sample soils of natural and treated clay soils to determine cohesion, c, and angle of internal friction, □. These methods are summarized in the next chapter.

3.7 One-Dimensional Consolidation Test

The main objective of the one-dimension consolidation test is to determine the compressibility of saturated fine-grained soils, which is considered a time-dependent phenomenon. In this study, the tests were conducted in accordance with ASTM D-2435-96 standard procedure on the soil specimens prepared at OMC soil by using an automated consolidometer test setup, as shown in Figure 3.10. Porous stones were placed on both the top and bottom of the specimen to facilitate water dissipation from the soil. After that, the specimens with porous stones were placed in a consolidation ring and transferred into a consolidometer. Water was added into the consolidometer to keep the soil saturated. During the saturation process, normally 24 hours, the specimen was under a seating load of 100 psf in order to be certain that the specimens became saturated, with no swelling occurring prior to the loading. The load increments were programmed and specimen deformations were automatically recorded by the GeoJac system unit. At the end of the test, the specimen was carefully removed from the ring, and the weight of the specimen was recorded immediately. The weight of the specimen after oven-drying was also measured in order to calculate the moisture content of the saturated specimen. Finally, void ratios were calculated using the height-of-solids method and plotted with vertical stress to obtain the compression indexes of the specimens.

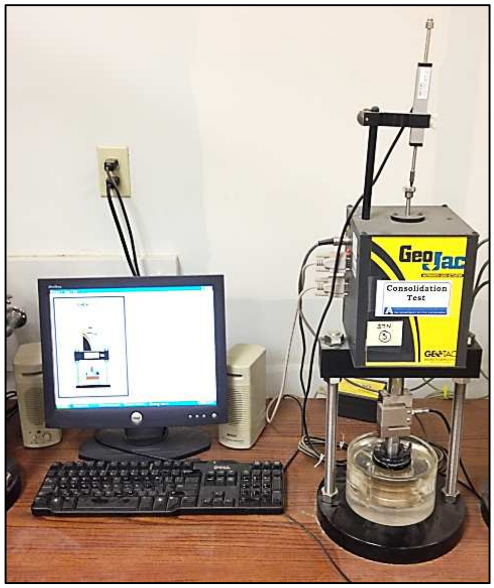

Figure 3.10 Automated consolidometer test setup

3.8 Direct Shear Test

Direct shear test is a simple method used to measure the friction angle, cohesion, and shear strength of soils. The testing method follows closely the ASTM D-3080-98 procedure for standard direct shear test. The direct shear machine used in UTA's geotechnical engineering laboratory is an improvement over electronic deformation devices, and automation of the data record system is an improvement over the traditional direct shear device with manual measurement. The soil specimen size of 2.5 in. diameter and 1.0 in. height were prepared at the OMC condition of the soil and at natural in-situ at 5% lime-treated condition for the soil. The soil specimen was placed in a shear box and installed in the direct shear testing machine. The specimen was then pre-consolidated under a water bath, with a load increment from the minimum applied at normal stress of 750 psf unit. During the consolidation stage, the upper and lower shear box halves were held in contact with each other with alignment screws.. The direct shear test setup used in this study is presented in Figure 3.11 below.

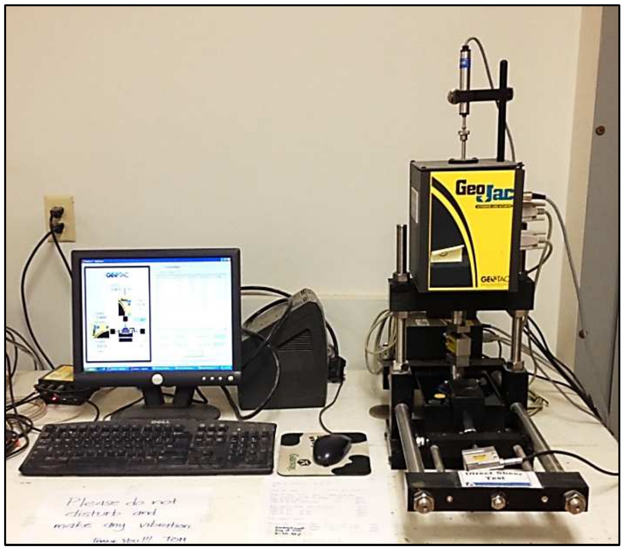

Figure 3.11 Direct shear test setup

3.9 Summary

This chapter presents the basic soil characterization studies and associated laboratory tests. Test procedures for the direct shear test, unconsolidated-undrained (UU) and unconfined compressive strength (UCS test) have been discussed step-by-step in accordance with the ASTM D 3080-98 , ASTM D 2859-95 and ASTM D 2166 standards, respectively. The next chapter will provide the analysis of these laboratory tests, and, based on these findings, the finite element model will be assessed with different cases on concrete pile for natural and lime-stabilized soils.

Chapter 4

LABORATORY INVESTIGATIONS AND ANALYSIS.

4.1 Introduction

The current chapter details the laboratory test results and corresponding finite element analysis. Direct shear test, unconfined compressive strength (UCS), and unconsolidated undrained (UU) tests were utilized to study the strength parameters for natural and lime-treated sample soils . This chapter details the strength and stiffness test results obtained from laboratory investigations. It also details the load deformation response analyses of shaft foundations that were conducted on treated and untreated ground, using finite element analysis and utilizing PLAXIS. Different, scenarios are considered to investigate the drilled shaft capacities in treated and natural clay soils.

4.2 Determination of Shear strength and Consolidation Parameters

4.2.1 Direct Shear Test

Direct shear tests were conducted on three samples of natural and three samples of lime-treated soil. The tests were conducted at the normal stresses of 5, 10, and 20 psi respectively. Test results of this soil are presented in Figures 4.1 and 4.2. From the results, it was observed that the friction angle of this soil sample was 22 degrees for the natural sample and 33 degrees for the lime-treated soil sample.

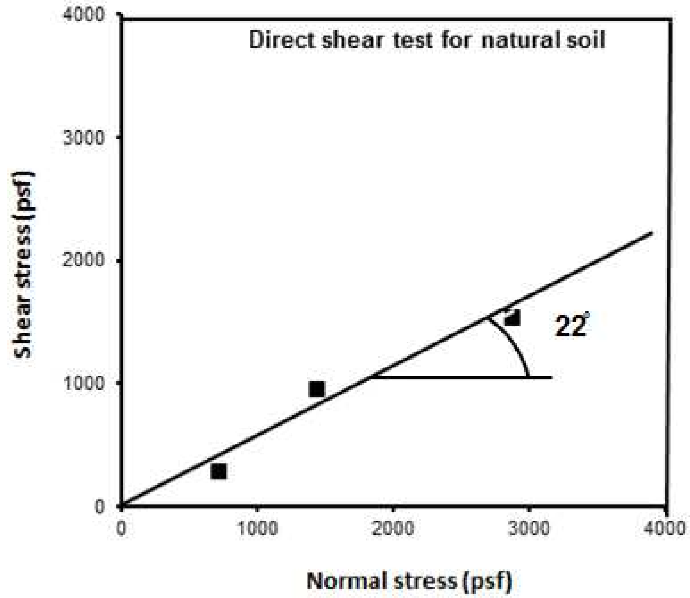

Figure 4.1 Shear strength versus effective normal stress for the natural soil

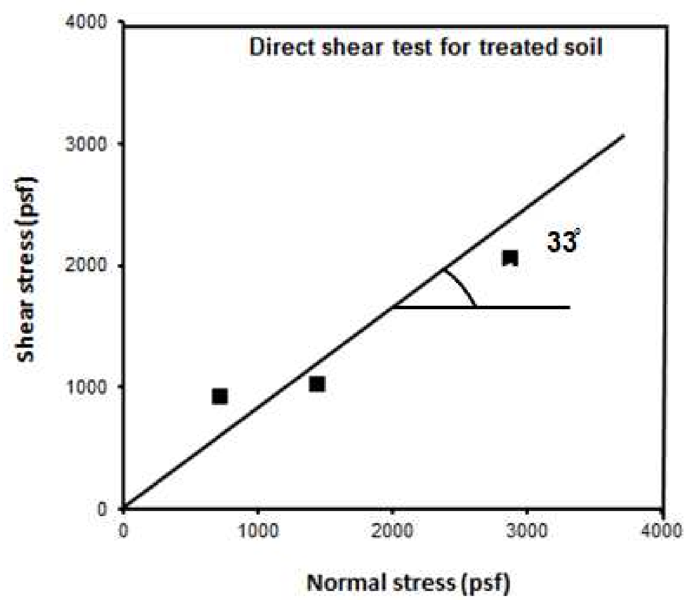

Figure 4.2 Shear strength versus effective normal stress for the lime-treated soil

From the test results, it was observed that with 5% lime treatment, the friction angle of the soil increased from 22^{o} to 33^{o}.

4.2.2 Unconsolidated-Undrained Triaxial Test Results

Unconsolidated-undrained (UU) tests were conducted on soils at both natural and 5% lime-treated conditions. Figures 4.3 to 4.6 present the results from UU tests. A summary of these results for natural and 5% lime-treated soils is presented in Table 4.1.

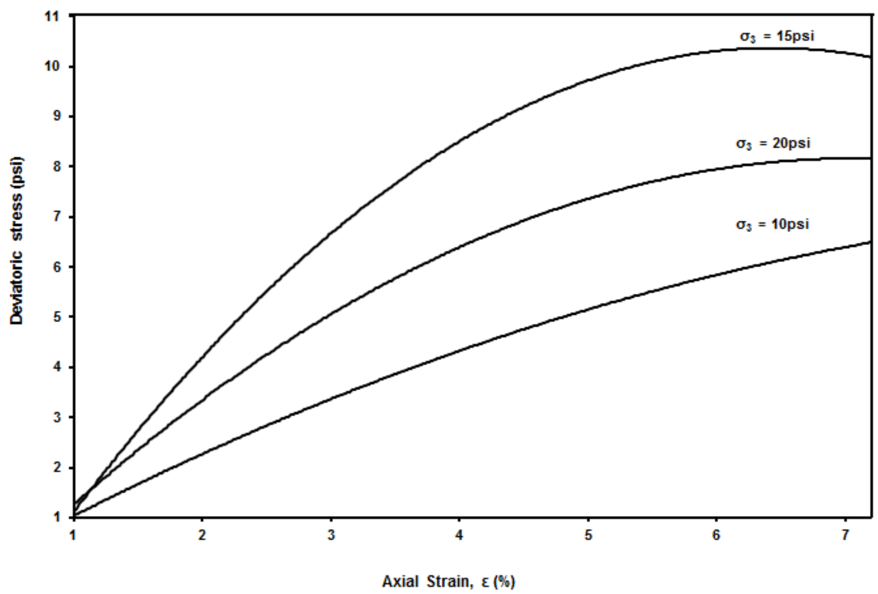

Figure 4.3 UU triaxial test results on treated soil at 10, 15, and 20 psi confining pressure

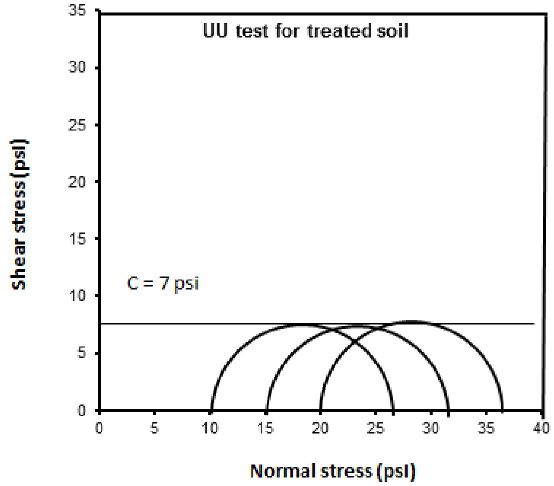

Figure 4.4 Mohr's circle at failure for 10, 15, and 20 psi confining pressure of treated soil sample

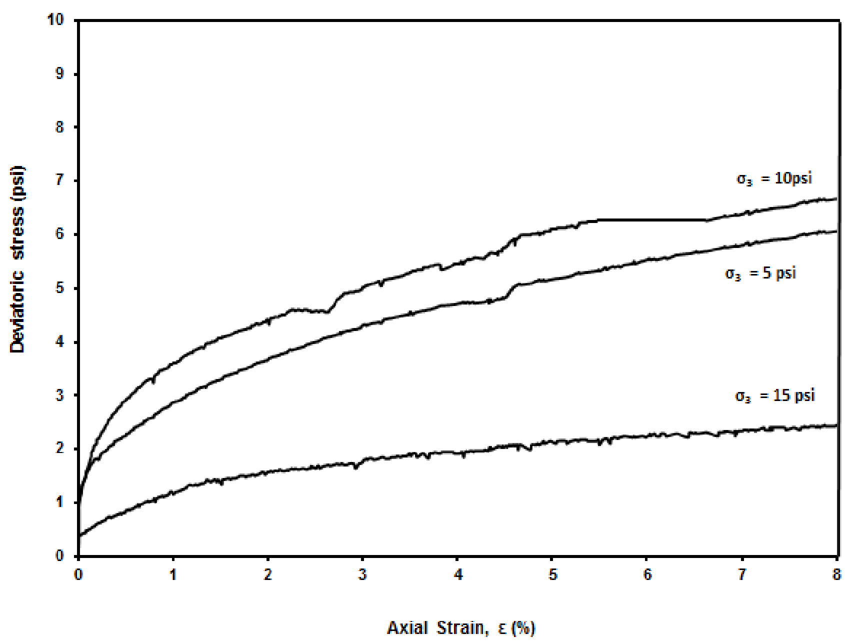

Figure 4.5 UU triaxial test results on natural soil at 10, 15, and 20 psi confining pressure

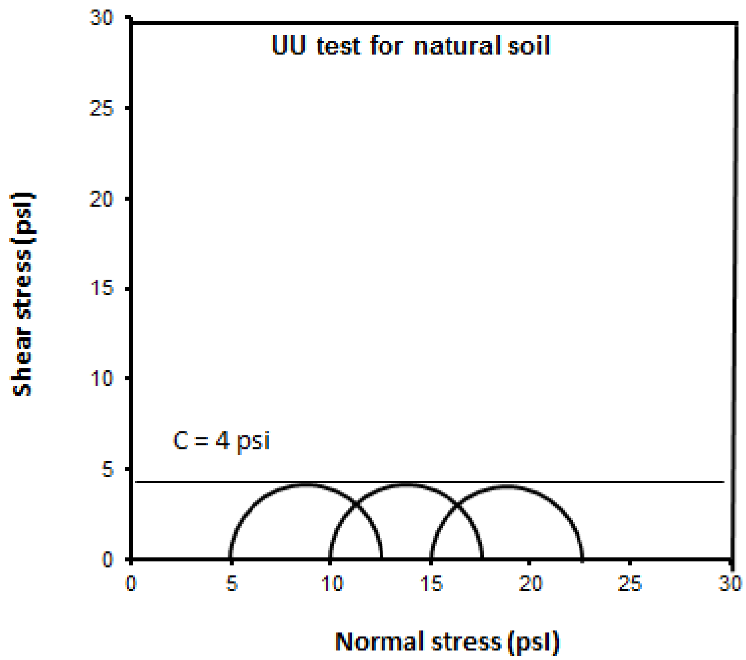

Figure 4.6 Mohr's circle at failure for 5, 10, and 15 psi confining pressure of natural soil

Table 4.1 Summary of direct shear and triaxial test results

Soil type	**Compressive strength (psi)**	**Internal friction angle(°)**
Natural soil	4	22
Treated soil	7	33

From the UU test results, as shown in Figures 4.3 to 4.6 and Table 4.1, it can be noted that the undrained shear strength of the 5% lime-treated soil increased to 7 psi from 4 psi. This increase in the strength was obtained after a curing period of 7 days.

4.2.3 Unconfined Compressive Strength Test

The objective behind performing the UCS tests was to determine the increase in the stiffness over time and calculate the Young's Elastic Modulus (E) for the natural and treated soils. UCS tests were conducted in the laboratory for lime-treated samples after a 7-day curing period. The soil samples were collected from the field in bags. The trial mixes were prepared for the pH test, as discussed in Chapter 3, and 5% lime was selected for the test. Implementation of 5% of lime proved to be a good selection, based on the results summarized in Table 4.1 for direct shear test and UU triaxial tests. The unconfined compression tests were conducted in accordance with the ASTM D 2166 standard. Table 4-2 provides a summary of the UCS test results.

Table 4.2 Summary of UCS test results

Time(Days)	Young's Elastic Modulus (E)		
	Sample 1	Sample 2	Average
	MPA	MPA	MPA
Natural soil	8	7	8
Treated soil	50	76	63

4.2.4 Consolidation Test

The consolidation test was conducted on the natural and 5% lime-treated soil for determination of the preconsolidation pressure, compressibility index, and swelling index. The findings from this test will be used later for determining the input parameters for the numerical analysis. Figure 4.7 shows the oedometer test results performed in the laboratory.

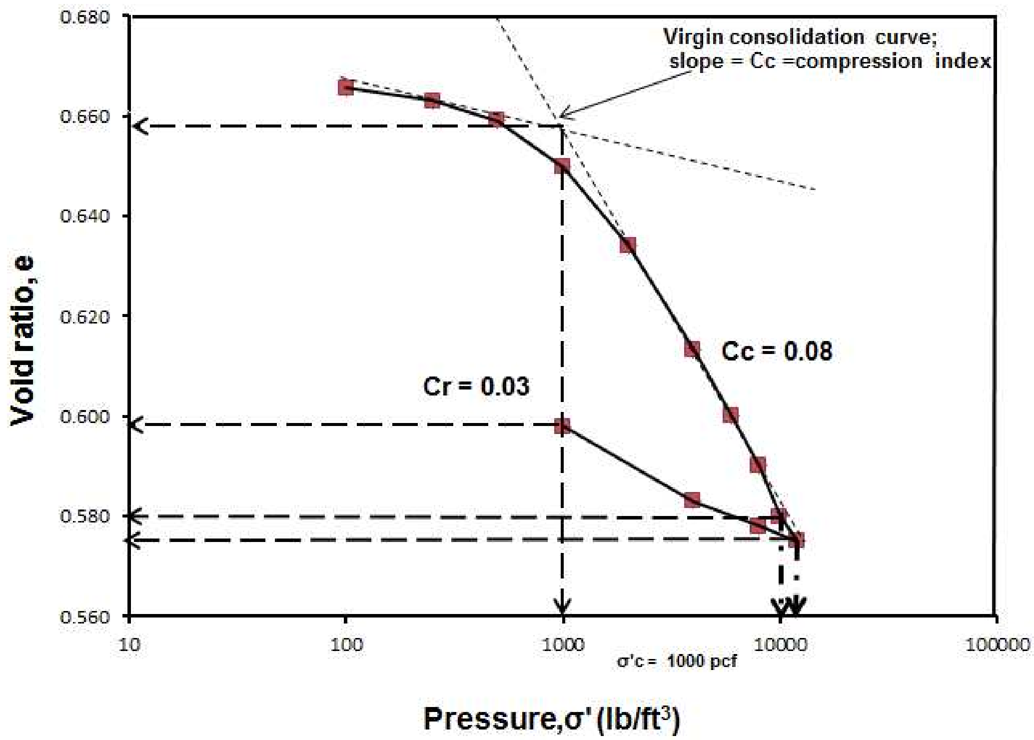

Figure 4.7 Consolidation test curve of 5% lime-treated soil

This test was carried out for the treated soil sample, within a period of one week, with a starting loading of 100 psf and incremental load up to 10,000 psf. The unloading test was performed to loading up to 1000 psf. Based on the past overburden experience and site conditions, the soil was normally consolidated. The compression and recompression indexes are shown on Table 4.3.

Table 4.3 Summary of consolidation test results

	(Compression index)CC	(Swelling index) CS	(Void ratio) e
Treated	0.08	0.03	0.68
Natural	0.21	0.021	0.88

4.3 Analytical and Numerical Analysis

4.3.1 Analytical Study of Single Axially-Loaded Concrete Drill Shaft Foundation.

Bearing Capacity Criteria.

Diameter of the drilled shaft = 1M and the initial trial length of the drilled shaft for this analysis is l = 5m under 175 kn load.

Qp = CuNc*Ap = (67)(9)(0.785) = 473 KN.

Qs =AsLCu α* = (0.785)(5)(67)(.73) = 191 KN.

The factor of safety is

$\frac{Qu}{Qw}$= $(473 + 191)$ /(175) = 3.79

Based on the assumption that the entire strata was a homogeneous clay layer without bedrock, the maximum settlement of the drilled shaft was calculated under a static load of 1000 kN. The diameter of the drilled shaft was kept constant at 1 m, and the embedment lengths were varied at 5, 10, 15, and 20 m. The maximum settlement was also calculated analytically for the 20m long drill shaft by varying the depth of the 5% lime-treated soil at 5, 10, 15, 20, and 30 m from the surface, as shown in Figure 4.8. Table 4.3 summarizes the results of the analytical study.

175 KN

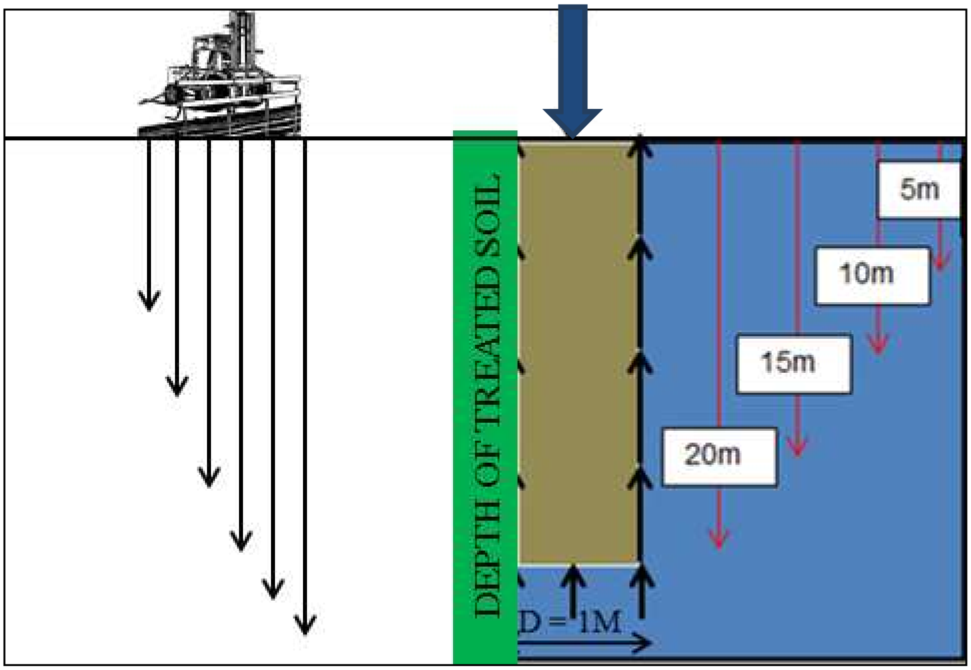

Figure 4.8 Drilled shaft foundation

The settlement of drilled shafts at working load was calculated in a manner similar to the settlement of piles under a vertical working load, as discussed in Chapter 2.

. $$S = S_1 + S_2 + S_3$$

Table 4.4 Analytical settlement results for natural and treated soil

	Unit	Natural soil				Treated soil				
Depth of shaft	M	5	10	15	20	5	10	15	20	30
Varying shaft length										
Settlement	cm	26	21	20	19	11	10.5	10.1	10	-
Varying Treated soil depth with 20 meter length of drilled shaft foundation										
Settlement	cm		-	-	-	13	12.5	12	2	1.9

From the analytical study, the maximum settlement for varying shaft depths observed for untreated soil was 26 cm and for the 5% lime-treated soil was 11cm. Thus, the maximum settlement of the soil decreased by 15cm, which is 57% less than that of the untreated soil. The maximum settlement for varying treated soil depths of the natural soil was 19 cm, and for the 5% lime-treated soil was 1.9 for 30-meters deep-treated soil. The maximum settlement for 5-meters deep-treated soil was 3 cm.

4.3.2 Finite Element Analysis (PLAXIS).

A soft soil model, also called the Cam-Clay model, which is especially meant for primary compression of near normally-consolidated clay-type soils, was used in this research.

4.3.3 Parameters Utilized in the Soft Soil Model

The parameters of the soft soil model coincided with those of the soft soil creep model. However, the soft soil model required the following material basic parameters -λ^*, κ^*, c, φ, ψ.

Apart from the isotropic compression test, the parameters κ^* and λ^* can be obtained from the one-dimensional compression test. Here, a relationship exists with the internationally recognized parameters for one-dimensional compression and recompression, C_c and C_r (assumed as equal to C_s). The void ratio, *e*, is assumed to be constant. In fact, *e* will change during a compression test, but this will give a relatively small difference in void ratio. For *e,* one can use the average void ratio that occurs during the test, or just the initial value.

Cohesion has the dimension of stresses. Any effective cohesion may be used, including a cohesion of zero.

The effective angle of internal friction represents the increase of shear strength with effective stress level. It is specified in degrees. Zero friction angle is not allowed. On the other hand, care should be taken with the use of high friction angles.

A dilatancy angle of zero degrees is the standard setting of the soft soil model.

Poisson's ratio value will usually be in the range between 0.1 and 0.2. If the standard setting for the soft soil model parameters is selected, then V_{ur} = 0.15 is automatically used.

4.3.4 Axially-loaded Single Drill Shaft Foundation

This chapter involves a reinforced concrete drill shaft foundation through a 30 m thick compressible clay layer, where bedrock is not available, The drill shaft foundation carried a load coming from the superstructure process that caused consolidation in the soil. Moreover, excess pore pressures were generated due to the stress increase around the drill shaft foundation.

In this thesis, focus is mainly on the deformations that occur with varying sizes of the drill shaft foundation, along with treatment depth. In this process, the behavior of the natural and 5% lime-treated soil parameters gathered from the laboratory experiments are listed in Table 4..

4.3.5 Assumptions considered for this study

The following assumptions were applied for the current research.

1. Compressible soil strata was assumed throughout, with no bedrock availability.
2. A typical vertical load of 175 KN(28 KIP) was applied from the overlying structure onto the shaft foundation. Most drilled shaft piers constructed for bridges in Texas are typically designed for 175 KN (28 kip).
3. Soil stabilization extended all the way to the boundary of the PLAXIS model for the treated soil calculation. However, the depth of treatment was entirely dependent upon the case scenario.

4. The active zone arising from expansive soil behavior was not considered in this analysis.
5. The lateral load effect was not considered in this analysis.
6. The dry construction method was adopted for drilled shaft construction in both treated and untreated clays.

The drill shaft foundation was comprised of reinforced concrete with a diameter of 1.0m; hence, a linear elastic material set was adopted. Soil parameters determined from the laboratory test results were utilized for analysis.

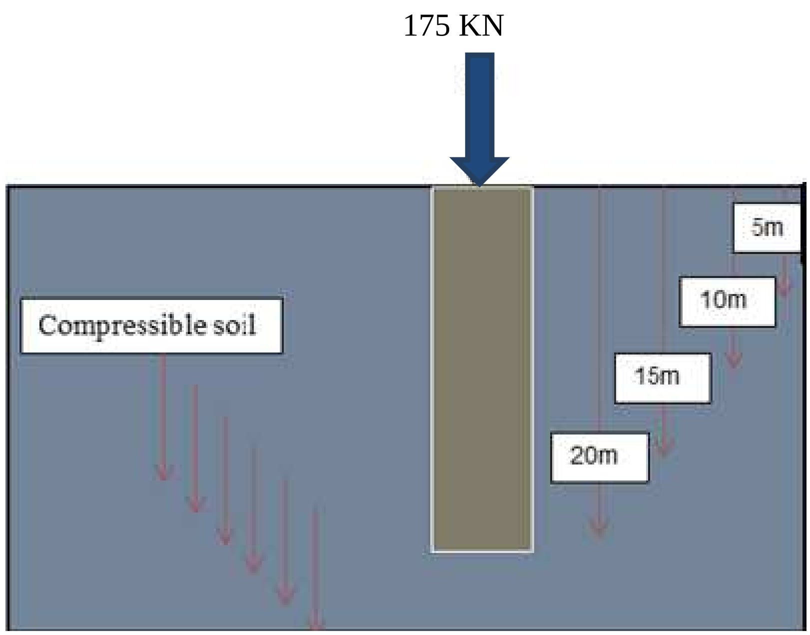

Figure 4.9 Concrete drill shaft foundations on compressible soil with different pile length

4.4 PLAXIS Model Input

4.4.1 Geometry Model

The geometry was simulated by means of an axisymmetric model, in which the drilled shaft positions along the axis of symmetry (Figure 4.10). In the general settings, the standard gravity acceleration was used (9.8 m/s^2). The unit of time should be set for days.

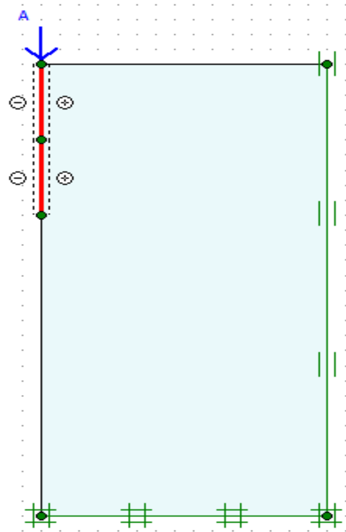

Figure 4.10 Axis of symmetry for PLAXIS analysis

Both the soil and the pile were modelled with 15-noded elements. The drill shaft foundation was generated using a 0.5 m width and the axisymmetric condition. The interface elements were placed around the drill shaft foundation to model the interaction between the drill shaft foundation and the soil. The interface was extended to about half a meter into the next clay layer. Note that the interface should be defined only at the side of the soil. A proper modelling of the pile-soil interaction was important to include the material interaction caused by the sliding of the soil along the drill shaft foundation during loading and to allow for sufficient flexibility around the drill shaft foundation tip.

The boundaries of the model were taken sufficiently far away to avoid the direct influence of the boundary conditions. Standard absorbent boundaries were used at the bottom and at the right hand boundary to avoid spurious reflections.

Table 4.5 Material properties of the subsoil and drill shaft foundation

Parameters	Symbol	Natural clay	Treated clay	UNIT
General				
Material model	Model	Soft clay	Soft clay	
Type of behavior	Type	Undrained (B)	Undrained(B)	
Unit weight above, p-line	$_{un}$	17	20	KN/m^3
Unit weight below p-line	$_{sat}$	19	23	KN/m^3
Parameters				
Poisson's ratio	V_{ur}			
Cohesion	C	0.15	0.15	- KN/m^2
Undrained shear strength	S_u	27.00	67.00	KN/m^2
Friction angle	□			
Dilatancy parameter	ψ	18	35	-
Lamda	λ*	0	0	-
Kappa	κ*	.05	0.02	- -
Compression index	Cc	.009	0.01	-
Swelling index	Cs	0.21	0.08	
Coefficient of permeability	k_x	.021	0.03	
	k_y	0.450	0.50	m/day
Interface				
Interface strength type	Type	Manual	Manual	-
Interface strength	R_{inter}	0.5	0.5	-
Initial				
Lateral earth pressure Coefficient	K_o	Automatic 0.5	Automatic 0.5	- -
Drill shaft foundation	EI	EA	W (weight)	
Unit	Kn m2/m	Kn/m	Kn/m/m	v_{ur}
-	4.31E6	17.27E6	5.88	0.1

4.4.2 Mesh Generation

The analysis was performed on the natural and treated soil, as shown in Figure 4.14, with a 15-node element selected for a finer mesh. The R_{inter} coefficient was 1. This factor related the interface strength (wall friction and adhesion) to the soil strength (friction angle and cohesion).

4.4.3.Calculation

Initial Phase

Initial effective stresses were generated by the k_o procedure, using the default value. Note that in the initial situation, the drill shaft foundation did not exist and that the clay properties were assigned to the corresponding cluster. The practice level was assumed to be the ground surface. Hydrostatic pore pressures were generated in the whole geometry according to this practice line.

Next Phase

A plastic option and staged construction option were selected by default selected and assigned the pile cluster and activated the load.

4.4.4 Case 1 : Effect of Drilled Shaft Depth on Maximum Settlement in both Treated and Untreated Soil

In this case, a 20-meter pile was embedded in both treated and untreated soil. Lime injected treatment was assumed to be 20 meters throughout the entire soil.

After generating the mesh for the natural soil, maximum settlement for different pile lengths was calculated from natural soil condition of PLAXIS (Figure 4.12). The same model was run for treated soil, and the different lengths, with maximum settlement curves, are shown in Figure 14.13 and Figure 14.14, respectively.

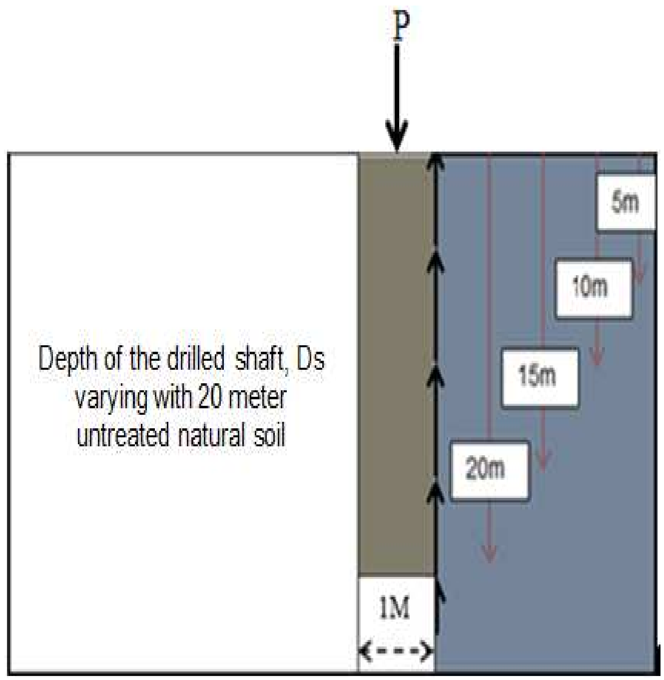

Figure 4.11 Maximum settlements for different lengths of drilled shaft diagram

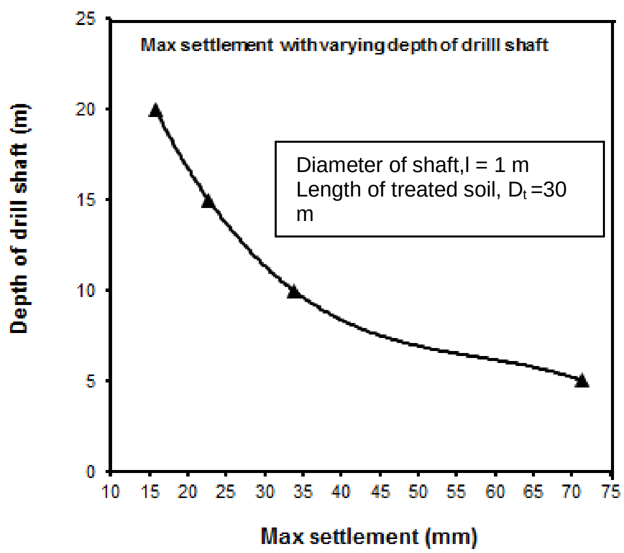

Figure 4.12 Numerical results for vertical deformation for different pile lengths calculated from natural soil condition

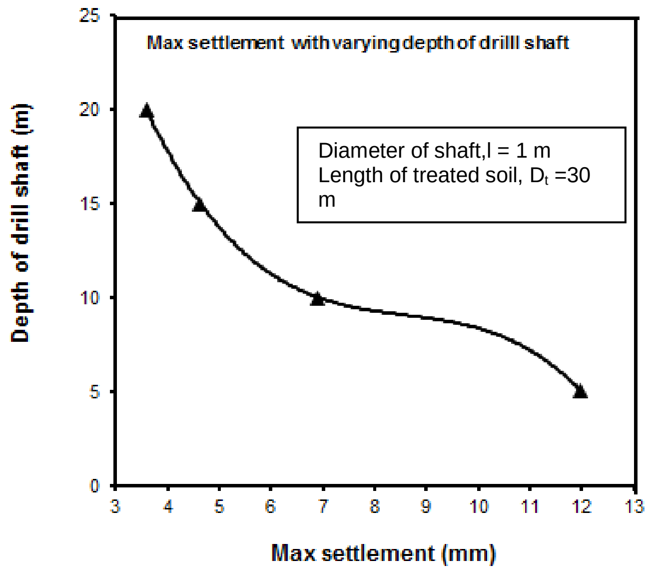

Figure 4.13 Numerical results for vertical deformation for different pile length calculated from 5% lime treated soil condition

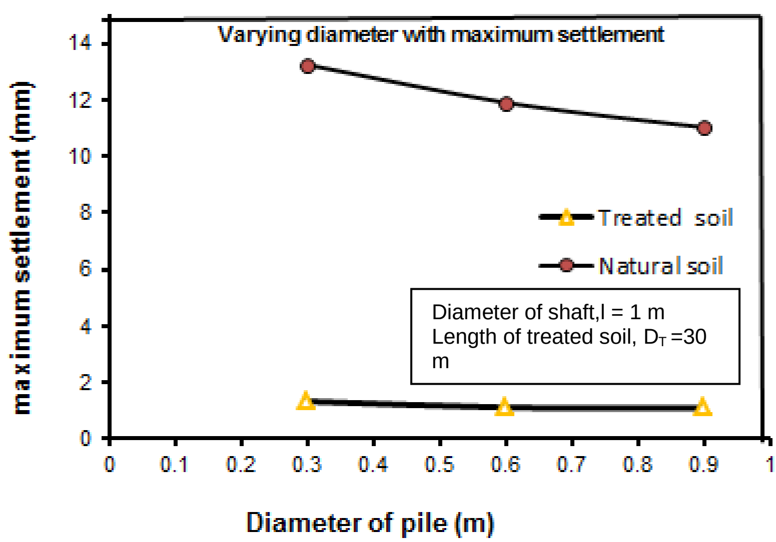

Figure 4.14 Comparison of maximum settlement with different drill shaft length from natural soil and 5% lime-treated soil

In this case, the results showed that the maximum settlement varied from 10 to 70 mm for natural and 0 to 15 mm for treated soil. For instance, in a 5m long drill shaft, there was a 15.71% settlement with lime-treated soil. In a 20m long shaft, there was a 7% reduction with natural soil. The range of 7 % to 16% less settlement was seen in treated soil than in the natural soil.

Table 4.6 Comparison results of natural and treated soils with varying depths

Type of soil	Shaft length(m)	Shaft Diameter(m)	Maximum settlement (mm)
Natural soil	5	1	70
Treated soil	5	1	15

The maximum settlement was about 70 mm for 20m length shaft with untreated soil, and minimum settlement was 3.5 mm for 20 m length shaft with treated soil.

4.4.5 Case 2 : Effect of Diameter of Drilled Shaft on Maximum Settlement

In this case, the shaft length was fixed at 20 meters and the effect of diameter was studied for treated and natural soils when loaded under 175 Kn (125 KIP). See Figure 4.16.

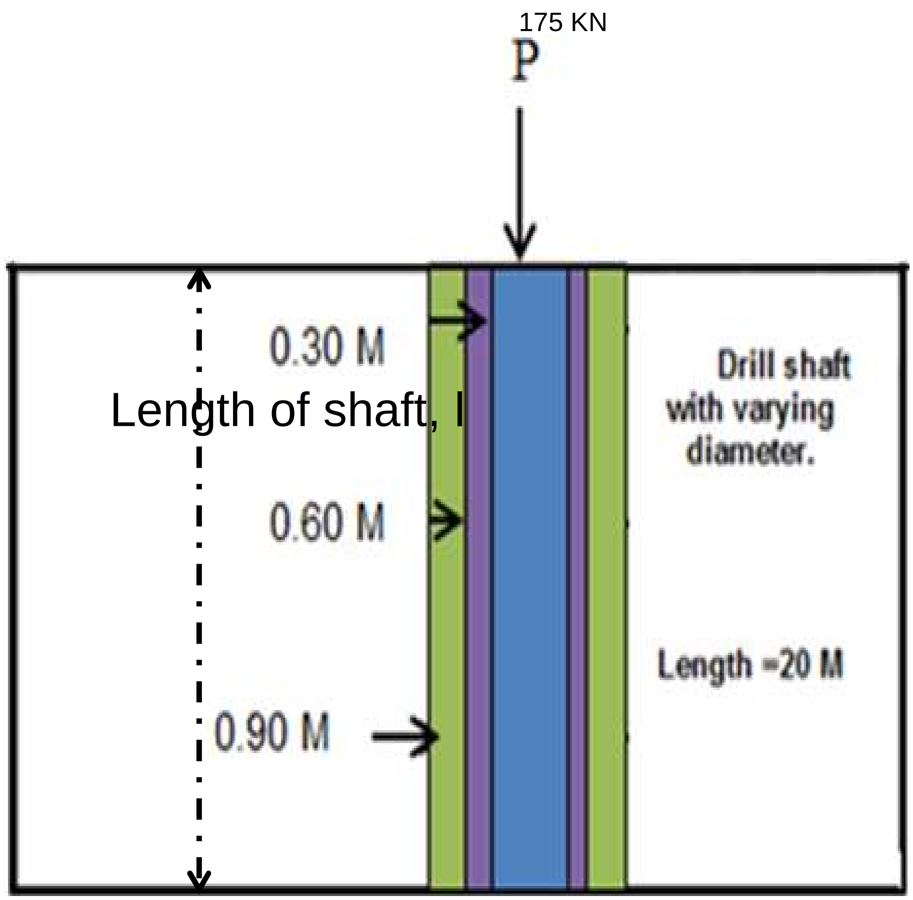

Figure 4.15 Numerical results for maximum settlement of varying diameters from natural and 5% lime-treated soil

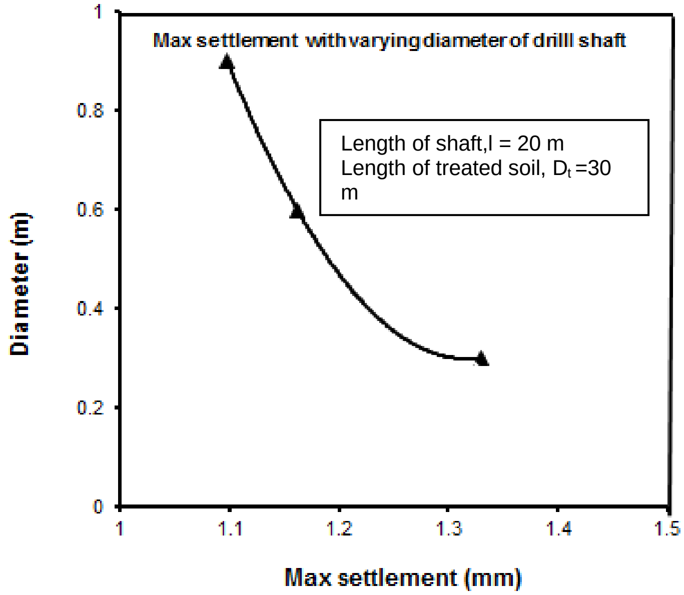

Figure 4.16 Numerical results for maximum settlement of varying diameters with 5% lime-treated soil

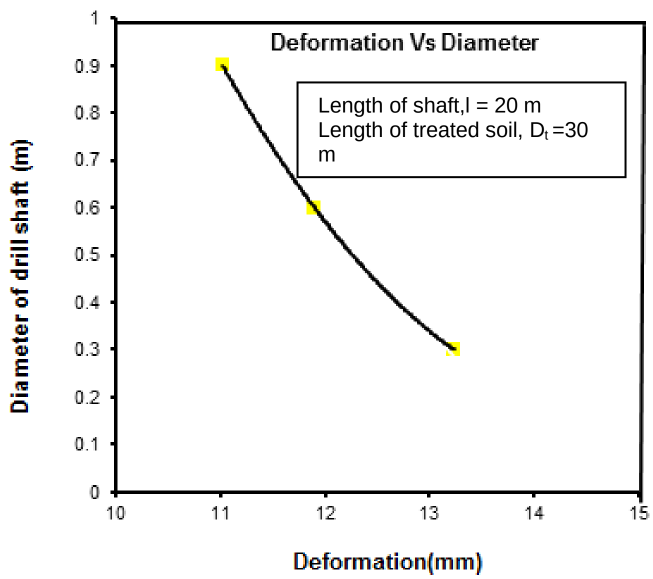

Figure 4.17 Numerical results for maximum settlement of varying diameters with natural soil

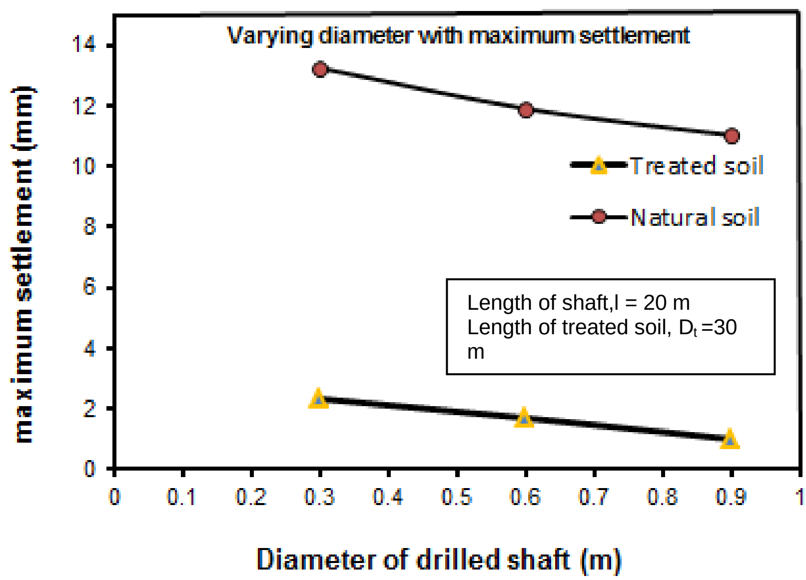

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Figure 4.18 Comparison of maximum settlement with a different drill shaft diameter with natural soil and 5% lime treated soil

Comparisons were made of varying diameters of the drilled shafts for natural and 5% lime-treated soils. In this case, we have seen that the maximum settlement varies from 11 to 13 mm for natural soil and 1 to 2.4 mm for 5% lime-treated soil.

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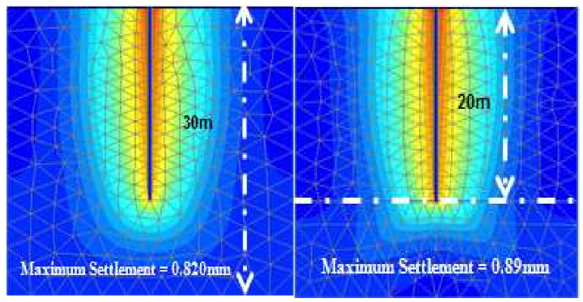
Table 4.7 Comparison results of natural and treated with varying diameter

Type of soil	Maximum settlement (mm)
Natural soil	13
Treated soil	2.4

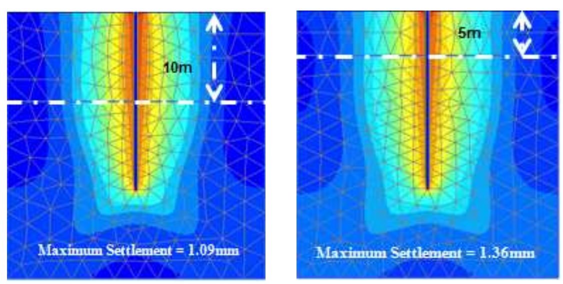
The maximum settlement was about 13.24 mm to 0.30m diameter shaft for untreated natural soil, and the minimum settlement was 1.098 mm for 0.90 m diameter drilled shaft for 5% lime treated soil.

4.4.6 Case 3 : Effect of Treatment Depth on the Maximum Settlement Observed

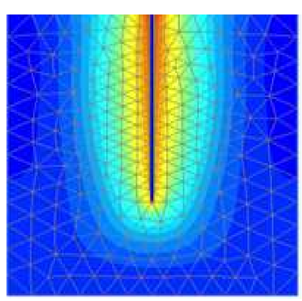
In this case, the length of pile was 20 meters; the diameter of pile 1.0m with a varying depth of treated soil 5mt., 10mt. ,20m., and 30 meters. The variation of settlement with load of 175 KN(28 KIP) was studied.



a) 30 Meter treated soil b) 20 Meter treated soil



c) 10 Meter treated soil d) 5 Meter treated soil



e) Natural soil

Figure 4.19 Effect of treatment depth (a) 30 m.; b) 20 m; c) 10 m; d) 5 m; e) natural soil

Numerical analysis was performed on 5% lime-treated soils with depths ranging from 0 to 30 meters. The maximum settlement was about 9.76 mm for natural soil, and the treated soil had a minimum settlement of 0.82mm for 30 meter depth of treated soil .

Table 4.8 Comparison results of natural and treated with varying lime-treated soil depth

Depth of treatment for 20 meters deep shaft with diameter of 1mt	Maximum settlement(mm)
No treatment	9.76
5 m	1.36
10 m	1.09
20 m	0.89
30 m	0.820

Finally, the comparison between analytical and numerical analysis for varying depths of the drill shaft and varying depths of treated soil with 20 meter length drilled shaft foundation and a constant load of 1000KN / 225 KIP are analyzed .

Table 4.9 Comparison of analytical and numerical results under 1000 KN /225 KIP load

Length of shaft/ Treated soil	Unit	5mt.	10mt.	15mt.	20mt.	30mt.
Max-settlement varying depth of the drill shaft foundation.						
Numerical	Cm	-	9	5	3.5	-
Calculated	Cm	26	21	20	19	-
Max-settlement Varying treated soil depth for 20 meters of drilled shaft foundation						
Numerical	Cm	11.7	10.9	9	7	3
Calculated	Cm	13	12.5	12	2	1.9

Initially, the calculations using the proposed numerical model were performed from the parameters obtained from the tests, which gave less settlements that varied by a factor of 2 to 5 than the analytical results in cases of varying lengths of shafts. It was seen that analytical and numerical calculated results show nearly the same settlements in cases of varying depths of the treated soil .

4.5 Summary

In this chapter, various lab experiments and analytical and numerical studies were conducted to investigate the effect of lime treatment on soil samples. Direct shear, unconsolidated undrained, unconfined compression strength and consolidation tests were conducted on untreated and 5% lime-treated soil samples. Furthermore, an analytical study was conducted to study the settlement behavior of a drilled shaft on treated and untreated soil. Finally, a numerical study was conducted in PLAXIS to investigate the effect of lime treatment on the settlement of the drilled shaft. The conclusions and summaries of those findings are in the next chapter.

Chapter 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In this study, various lab experiments and analytical and numerical studies were conducted to investigate the effect of lime treatment on a clayey soil collected from Fort Worth, Texas. After basic soil characterization studies, pH tests were conducted to obtain the optimum dosage of lime treatment for the soil. After the determination of the optimum lime content for the soil, direct shear, unconsolidated undrained, unconfined compression strength and consolidation tests were conducted on natural and lime-treated soils.

An analytical study was conducted to research the settlement behavior of a drilled shaft on treated and untreated soil. Finally, a numerical study was conducted in PLAXIS to investigate the effect of the lime treatment on the settlement of the drilled shaft. The following conclusions can be drawn based on this research:

1. The optimum dosage of the lime treatment of the soil for this research was determined using pH tests. The pH of the treated soil was determined at varying dosages, and a plot of pH versus lime content was developed. From the pH versus lime content plot, the optimum dosage of the lime content was determined to be 5% of the soil.

2. The internal friction angle of the natural and treated soils was determined by conducting direct shear tests at normal stress levels of 720, 1440 and 2160 psf. The drained friction angles of the natural and 5% lime-treated soils were found to be 22^0 and 33^0 , respectively. Thus, the friction angle of the treated soil increased by 50% at the optimum dosage of 5% lime treatment.

3. Unconsolidated undrained (UU) triaxial tests were conducted to assess the undrained shear strength parameter of the natural and the 5% lime-treated soil. The tests were conducted at the confining stresses of 5, 10, and 15 psi. The undrained shear strength of the untreated soil was 4 psi and that for the 5% lime-treated soil was 7 psi. Thus, the undrained shear strength of the soil was increased by 3 psi, which is a 75% increment from that of untreated soil. This result shows that the 5% lime treatment significantly increased the undrained shear strength of the soil.

4. The stiffness properties of the soils was determined using the unconfined compression strength test. Tests were conducted on the control and the 5% lime-treated soil using a displacement rate of 2.27 mm/min. The average elastic modulus of the control and the treated soils were 8 and 63 MPa. Test results indicated a significant increase in the elastic modulus of the 5% lime-treated soil. An almost 8 times increase in the elastic modulus suggests that the 5% lime determined from the pH test was appropriate. Thus, it can be concluded that the lime treatment is a suitable method to enhance the stiffness of the soil.

5. The one-dimensional consolidation test was conducted to determine the compressibility behavior of the lime-treated soil. The odometer test was conducted using a computerized data logging system in UTA lab. The compression index (Cc) and swelling index (Cs) of the lime-treated soil were determined to be 0.08 and 0.03, respectively.

6. An analytical study was conducted to investigate the improvement in the settlement of the drilled shaft foundation for treating soil over the natural soil. The length of the drilled shaft was varied at 5, 10, 15, and 20 m. The settlements of the drilled shaft in the case of treated soil were almost two times smaller than the settlement in the case of natural soil.

7. The maximum settlement was also calculated analytically for the 20 m length drill shaft by varying the depth of the 5% lime-treated soil at 5, 10, 15, 20, and 30 m from the surface. The settlements observed at 5, 10, and 15 m depths of treated soils were 13, 12.5, and 12 cm, respectively. This result indicated that the improvement in the skin friction resistance due to treated soil was not significant. However, as the depth of the treated soil reached a distance equal to the depth of the shaft, the maximum settlement dramatically reduced to 2 cm. This result shows that the there was a significant increase in the end bearing due to the lime treatment.

8. Finite element analysis in PLAXIS was conducted to investigate the effect of treated and natural soil in the maximum settlement. The depth of the drilled shaft varied from 5 to 20 m, with an increment of 5 m in between. The settlements calculated in the 5% lime treated soil were considerably lower than those of the natural soil. Although the settlement of the shaft in the natural soil decreased continuously with an increase in the shaft depth, the maximum settlement observed for the 20 m length shaft was still 5 times higher than the settlement of the shaft through lime-treated soil.

9. The second case study conducted in the PLAXIS was aimed at investigating the influence of the shaft diameter on the maximum settlement through natural and treated soil. While the depth of the shaft was fixed at 20 m, the shaft diameter varied from 0.3 m to 0.9 m, with an increment of 0.3 m in between. The numerical investigation revealed that the settlements in the case of lime-treated soil were almost 7 times smaller than that in the natural soil. However, the trend of the reduction in the settlement with an increase in the drilled shaft diameter was similar in both cases.

5.2 Recommendation

Though the finite element analysis provides efficient results, the programs need many input parameters and may be complicated to use. Analysis should be performed very carefully, as the parameters are very sensitive due to their small values.

For further study, more case studies could be searched for, selected, and studied extensively for the calculation. The use of advanced soil models and field study, like loading tests, will make it more practical. In addition, the soil behavior could be studied by using other constitutive models. More time and study are needed to compare results from different constitutive models.

References

ASTM Standard D4318-05.,2003. “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.” *Annual Book of ASTM Standards,* Soil and Rock (I), 4(8), ASTM International, West Conshohocken, PA.

ASTM Standard D3080-98,,2003, “Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.” *Annual Book of ASTM Standards,* 4(8), ASTM International, West Conshohocken, PA.

Anon.,1989a.” Dewatering myths. Ground Engineering,” 22 (2), 12–15.

Anon., 1989b.” The use of geotextiles and geocomposites in the United Kingdom.” Proc. American Society of Civil Engineers, Journal Construction Engineering and Management Division, 115 (2), 258–69.

Anon.,1990a. “Tropical Residual Soils,” Engineering Group Working Party Report. Quarterly Journal of Engineering Geology”, 23, 1–101.

Anon.,1990b.” Geotextile groundwork.” Ground Engineering, 23 (3), 14–22.

Anon.,1990c.” Cement grouts.” Ground Engineering, 23 (7), 18–21.

Anon.,1990d. “State-of-the-art Report on Soil Cement.” American Concrete Institute Materials Journal, 87 (4), 395–417.

Anon.,1991.” Jet set style.” Ground Engineering, 28 (8), 22–3.

Bell, F.G.,1988. “Lime stabilization of clay soils: Part 1, Basic principles.” Ground Engineering, 21 (1), 10–15.

Blacklock,.J. R., Joshi, R. C., and Wright, P.J., 1982. "Pressure Injection Grouting of Landfills Using Lime and Fly Ash," Proceedings, ASCE Specialty Conference, Grouting in Geotechnical Engineering, New Orleans National Lime Association. Lime Slurry Pressure Injection Bulletin, Bulletin 331.

Blacklock, J. R., and Lawson, C. H., 1977. "Handbook for Railroad Track Stabilization Using Lime Slurry Pressure Injection," Federal Railroad Administration Report FRA/ORD-77 /30.

Cambefort, H.,1967."*Injection des Sols,"* Eyrolles, Paris.

Dunnavant, T. W., and O'Neill, M. W.,1989. "Experiment p-y Model for Submerged, Stiff Clay." *Journal of Geotechnical Engineering", ASCE, 115(1)*, 95 – 114.

Day, R. W.,2006. *Foundation Engineering Handbook*. New York, NY: McGraw-Hill.

Eaglewood Cliffs, NJ: Prentice Hall.

Holtz, R. D., and Kovacs, W. D.,1981. "An Introduction to Geotechnical Engineering,"

Ingles, O.G. and Metcalf, J.B.,1972" *Soil Stabilization,"* Butterworths, Sydney.

Joshi, R.C., Natt, G.S. and Wright, P.J.,1981. "Soil improvement by lime-fly ash slurry injection." Proc. 10th International Conf. on Soil Mechanics and Foundation *Engineering", Stockholm,* Vol. 3, pp. 707–12.

National Lime Association, 1982. *Lime Stabilization Construction Manual.* 7th ed., Washington, D.C.

Matlock, H., and Reese, L. C.,1961. "Foundation Analysis of Offshore Pile-Supported Structures." Proceedings, Fifth International Conference, International Society of Soil.

Mechanics and Foundation Engineering, Paris, France, 2, 91-97.

McClelland, M.,1996. "History of Drilled Shaft Construction in Texas." Paper presented before the 75th Annual Meeting of the Transportation Research Board, Washington, D.C.

Rathmayer, H.G. and Saari, K.H.O. (eds),1983. "Improvement of ground." *Proc. 8th European Conf. on Soil Mechanics and Foundation Engineering, Helsinki,* Balkema, Rotterdam.

Winterhorn, H.F. and Schanid, W.E.,1971. "*Soil Stabilization Parameters.*" Technical Report No. AFWL-TR-70–35, Kirtland Air Force Base, New Mexico.

Biographical Information

Abdusemed Ali was born in Addis Ababa, Ethiopia. Coming from an Ethiopian family, Abdusemed is well versed in ceremonial and traditional ethics, and enjoys studying, soccer and other extra-curricular activities. His father was a merchant by profession, and his mother was a housewife. Abdusemed went to Awelia High School for his primary and secondary schooling. He earned his bachelor's degree at the Addis Ababa University in Addis Ababa. He worked on different projects in Ethiopia before moving to the USA.

Born in 1981, Abdusemed spent 29 years in Addis Ababa before moving to the USA in 2011 to live in the USA and in 2013 to pursue a Master's degree, with a specialization in geotechnical engineering at the University of Texas at Arlington. In the USA, he had an opportunity to work on road projects designed and inspected by the Texas Department of Transportation (TxDOT): FM-1709, city of West Lake; FM 407, city of Denton; and SH-183 bridge projects, city of Fort Worth.