IMPROVEMENT OF SOFT SOIL CHARACTERISTICS USING STONE COLUMNS

Thesis of Master of Science

By

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<u>THESIS ACCEPTANCE CERTIFICATE</u>

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Date:_________________________________

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ABSTRACT

Due to rapid increase in construction activities over the past few decades, the importance of ground improvement techniques has increased manifold. Ground improvement techniques are numerous but the basic aim is to improve the engineering parameters of problematic soils so as to achieve desired properties. Stone columns reinforce soft soil owing to their high stiffness, strength and increase in drainage path. Different models have been used to predict load-settlement behavior of soil treated by stone columns. The load carrying capacity of stone columns is dependent on various parameters which provide an insight into their load-sharing mechanism. This research aims at investigating the effects of floating stone columns in clayey soil with silty deposits by developing small-scale laboratory models. A comparison is made between untreated and single stone column treated soil by varying the undrained shear strength of soil and L/D ratio of stone column. Effect of group columns is investigated in terms of varying spacing among them. Based on rigorous testing conducted in laboratory, soil parameters have been identified to develop accurate numerical models in PLAXIS 2D software. It is inferred from comparison between the two that results of small-scale laboratory testing are in close agreement with results from FEM. The undrained shear strength of soil affects the applicability of stone columns as a ground improvement technique and the stone columns are effective in the range of low to medium shear strength soil (S_u<54KPa). There exists a critical length of column beyond which the load carrying capacity does not significantly increase. (L/D= 4 – 5.5). Group effect of columns is more pronounced at closer spacing and is found negligible beyond S/D=3. Bulging failure is observed in single stone column loaded in compression and the effect of bulging is prominent at 1.5–2D from the top. However group columns do not fail by bulging. The small scale testing is effective to predict load-settlement behavior of stone columns in field however controlled environment should be

available for conducting testing. Further research is needed to address the effects of curing time, encasing of columns and varying diameter of stone columns.

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Chapter 1

INTRODUCTION

1.1 GENERAL

Soil is a naturally occurring material formed due to mechanical weathering of rocks or decomposition of organic materials or existing as transported sediments. Studying the engineering behavior of soil is essential in determining its performance pertaining to various intended functions. Due to ever growing population and fast tracked urbanization, infrastructure development has expanded to areas which are otherwise deemed unsuitable for supporting structures like bridges, buildings, reservoirs, storage tanks, etc. Such areas incapable of meeting the minimum requirements, set by geotechnical engineers, need to be thoroughly investigated. Proper measures should be devised in order to make these areas suitable for their intended purpose.

Ground improvement techniques are numerous subject to site specific material, loading conditions, design specifications and nature/sensitivity of structure. It is imperative that all techniques are aimed at improving the properties of problematic soils within economic constraints. Stone columns are used as ground improvement technique especially in soft soils which are unable to withstand large loads coming from overlying structures. Stone columns increase the bearing capacity of soft soils due to the installation of stiffer material than the soil and help increase the permeability of ground owing to the high permeability of granular material in the form of columns. The granular material helps in increased consolidation rate thereby reducing the risk of excessive settlement after application of load.

In literature, reports suggest that stone columns are used to distribute the loads between themselves and transferring the rest to the surrounding soil hence reducing the load on soft soils. They can be used to replace almost 10 to 35 percent of soil. Ground improvement by stone columns is a better method as compared to other procedures in terms of cost and applicability. It is a simple technique requiring no special efforts for construction and can be put to immediate use after construction. It is harmless to environment and can be constructed in any weather conditions.

1.2 PROBLEM STATEMENT

Soft soils are most often known for their low shear strength, high moisture content, low permeability and compressibility. Though, in some cases it might be their swelling nature that causes problems. Soils such as soft clays have the tendency to consolidate under the application of load leading to excessive settlement of the structure that it supports hence compromising the serviceability of structures. Due to low permeability, increasing loads cause surge in pore water pressure which causes reduction in effective stresses hence loss of shear strength and eventually triggering failure. Such soils generally require longer duration for the dissipation of pore water pressure and when not given proper time can settle quickly under loads.

Due to low shear strength, soft soils have less bearing capacity and cannot accommodate the service loads coming from overlying structures. Many a time differential settlement has been reported to be the cause of damage to structures due to underlying soft soils. To cope up with such problems different ground modification techniques have been developed in the past and modified conforming to advancements in the field of geotechnical engineering. The type of modification technique is subject to geology and condition of the site as well as economic constraints.

Stone columns are similar to piles in stiffness as well as acting like vertical drains but less costly. A stone column is a vertical cylindrical hole excavated in soft soils and filled with stones. The construction method may either be replacement method or displacement method. Research has been carried out to analyze the performance of stone columns and different methods have been adopted by different researchers such as numerical models by finite element analysis or small scale laboratory models to compare and predict the performance of full scale models in field.

Current design practice is based on unit cell concept in which the loads are considered to be acting upon equivalent area of stone columns. The unit cell approach ignores the interaction between multiple columns and also the group efficiency of columns. Hence in a unit cell approach the ultimate capacity of a group of stone columns is calculated to be the load carrying capacity of all the single columns. While this holds true for interior columns, the outer columns on the periphery show quite variations. The failure mode is also observed to be bulging failure for single columns ([1]; [5]) however this does not happen so in group of stone columns where general shear failure is observed [6] or sliding of entire surrounding soil [8].

1.3 RESEARCH SCOPE

Stone columns interact with the surrounding clay, increasing its stiffness by laterally restraining the soil, being a stiffer material when they are loaded. The increase in bearing capacity and reduction in settlement is considered to vary with type of soil, spacing and diameter of stone columns and stiffness of granular material. Stone column-clay interaction has been deemed complex to properly understand however many researchers have adopted different techniques to analyze their behavior. In this thesis, an attempt is made to understand the behavior of stone columns in soft clay. To study the effects of the improvement technique on settlement and bearing capacity of soil, the testing requires the preparation of laboratory models to simulate the column-soil interaction on a small scale. The testing procedure involves soil identification/classification tests, strength tests, compaction tests, load bearing tests with and without improvement. The unit cell concept was followed to analyze the behavior of single column while axisymmetric columns were installed to study the group behavior. The laboratory models were also replicated as numerical models and tested using finite element analysis software PLAXIS 2D.

1.4 RESEARCH OBJECTIVES

Although improving the characteristics of soft soil using stone columns has been used in the past all over the world but research in this field is still not adequate to address the complex nature of interaction between the soft soil and the granular material. Many researchers have adopted techniques of their own choosing like developing analytical models/numerical models or opting for full scale testing to assess the nature of the columns in ground however the variables involved in this procedure are too many, rendering the research inadequate. This thesis is aimed at achieving the following objectives;

a. To study the behavior of single stone column in soft soil and understand its interaction with the surrounding soil so as to understand settlement behavior of a footing loaded in compression.

b. To study the group effect of multiple stone columns in soft soil and analyzing group efficiency while comparing it to single stone column.

c. To perform a parametric study to analyze the effects of different parameters and variables like shear strength, length to diameter ratio, spacing to diameter ratio on the performance of stone columns in soft soil.

d. To analyze the failure mode of single and group of stone columns with varying parameters.

e. To validate the results obtained from experimental models by comparing it with field conditions simulated by numerical analysis using PLAXIS 2D.

1.5 ORGANIZATION OF THESIS

This thesis is presented in the following format and order;

a. Chapter 1 covers the introduction, scope and objectives of the research.

b. Chapter 2 presents the historical development and theoretical background of stone columns and the techniques and concepts necessary to understand their interaction. Furthermore different design methods and analytical procedures by other researchers have also been discussed.

c. In Chapter 3, the experimental matrix and detail work plan is discussed and involves the laboratory testing and preparation of numerical models to study the composite material.

d. Chapter 4 outlines the results obtained from rigorous laboratory testing as well as numerical simulation using PLAXIS 2D and drawing out comparison between the two methods.

e. In Chapter 5, the conclusions drawn from detailed experimental work along with the recommendation are presented and the shortcoming of the experimental procedure is discussed. Finally a brief suggestion about future prospects of the project is presented along with list of references.

Chapter 2

LITERATURE REVIEW

2.1 GENERAL

Ground improvement by use of stone columns has been considered as an effective technique among various other methods to enhance strength characteristics of soft soils. In this method, granular materials having greater stiffness and permeability than the surrounding soil are inserted into boreholes made in the soil. Moreau et al [1] first used stone columns in the late 1830s for bearing loads coming from ironworks in Bayonne, France. Stone columns have been used in other countries since then with the introduction of new modifications to existing methods or new methods of installation. They are primarily intended to;

a. Increase the bearing capacity of underlying weak soil

b. Increase the degree of consolidation hence increasing time rate of settlement

c. Reducing total and differential settlement

d. Increasing the stiffness of soil and hence providing stability

e. Reducing the liquefaction potential of soils prone to it [2]

Stone columns act almost similar to piles in load bearing mechanism. Depending upon the soil characteristics, they can be installed as floating columns, resisting overlying loads by the skin friction developed along the length owing to the granular material. They can also be installed as end bearing columns to transfer the load coming from footings to underlying stronger strata through a soft layer of soil. Stone columns also act as vertical drains by increasing permeability and incorporating radial flow in the ambient soil although they are stiffer then vertical drains. They have been successfully used to increase the bearing capacity and to reduce the settlement of foundation of structures like liquid storage tanks, earthen embankments, raft foundations, etc., where relatively large settlement would not cause much damage. They are used for the stability of slopes of embankments constructed on soft soil. Although stone columns have been widely used all over the world but despite of improvement in construction methods/ equipment, present design methods are limited to empirical methods and detailed information is not available on the design of stone columns in codes/textbooks [28].

2.2 STONE COLUMN CONSTRUCTION METHODS

Stone column construction involves the partial replacement of soil with a relatively stiffer stone material which penetrates into the weaker strata sharing the loads from overlying structures. Depending upon the nature of soil and site conditions [11], Hayward Baker Inc. 1996 generally stone columns are installed either by;

a. Replacement (wet) method

b. Displacement (dry) method

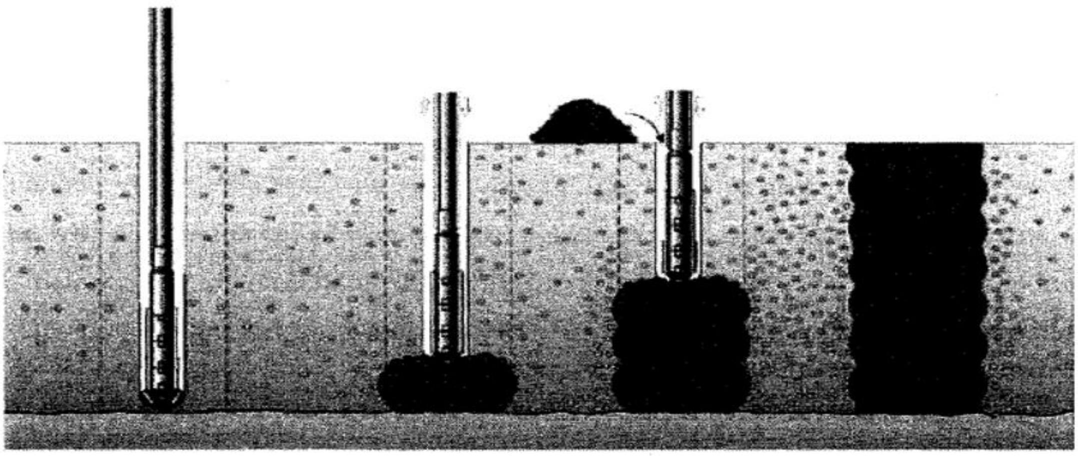


Figure 2. 1: Construction of aggregate piers by vibro-replacement (after Hayward baker, Inc., 2006).

The replacement method (wet method) is used for soft soils whereby water is inserted into the hole inside the ground which softens the soil and forms soil slurry. The soil slurry is removed by the force of water and filled with stone material. The stone material is either fed from the ground surface which falls under gravity known as top feed process (figure 2.3) or it can be delivered to the nose of the vibratory probe and compacted through bottom feed process as shown in figure 2.2. The stones are put in lifts of 0.4-0.8m, and compacted with the help of vibratory probe, thus helping to densify the soil in lateral as well as in the vertical direction [9].

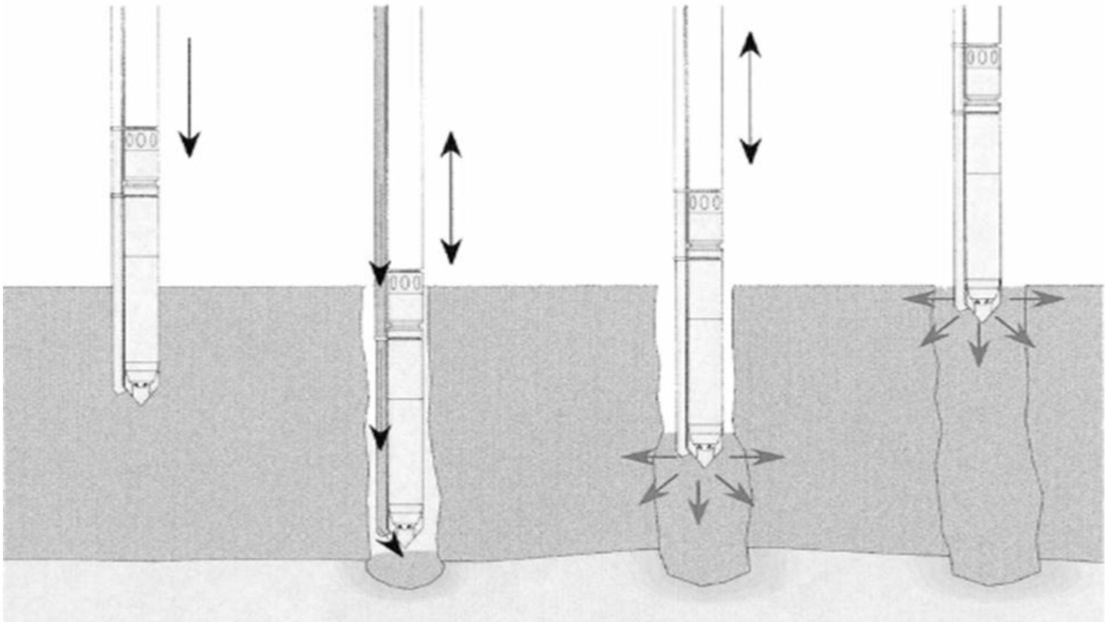


Figure 2. 2: Dry Method (Bottom fed process)

In the displacement method air jetting is used as a means to remove the soil material from the hole which is then backfilled with crushed stone material with the help of a vibrating probe either through top or bottom fed technique [21]; [10]. Figure 2.4 shows the applicability of Vibro Compaction and Vibro Replacement techniques in relation to the particle size of soil. Vibro Replacement method is suitable for finer soils while Vibro Compaction method is suitable for coarse grained soils.

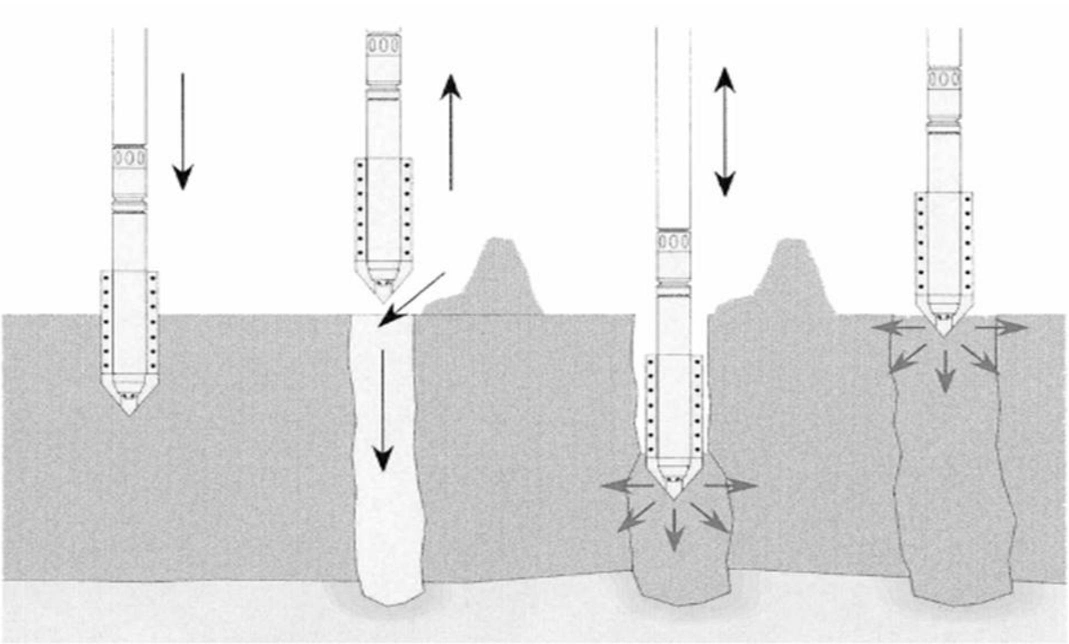


Figure 2. 3: Dry Method (Top Fed Process)

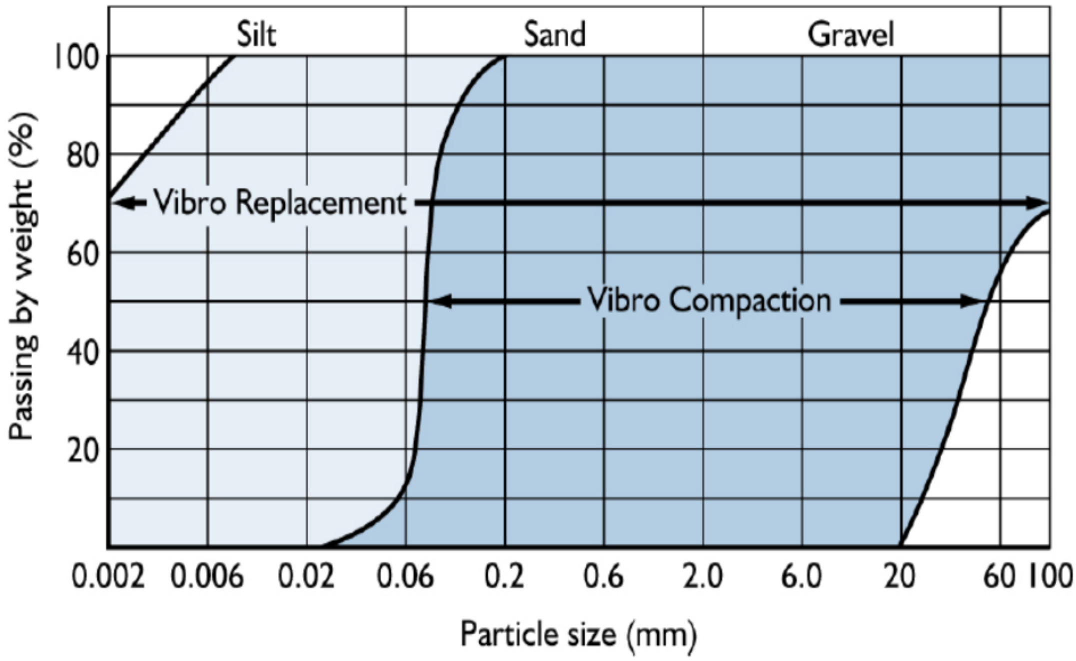


Figure 2. 4: Applicability of Vibro-Compaction and Vibro-Replacement[39]

Another method recently developed involves excavation of borehole by means of auger before constructing stone column. The augered hole is then filled with stone material and compacted and vibrated at the same time with the help of vibratory probe as shown in figure 2.5. The columns constructed by this method are known as “vibropiers” and are suitable in cohesive soils with sufficient strength to avoid borehole collapse after the auger is removed.

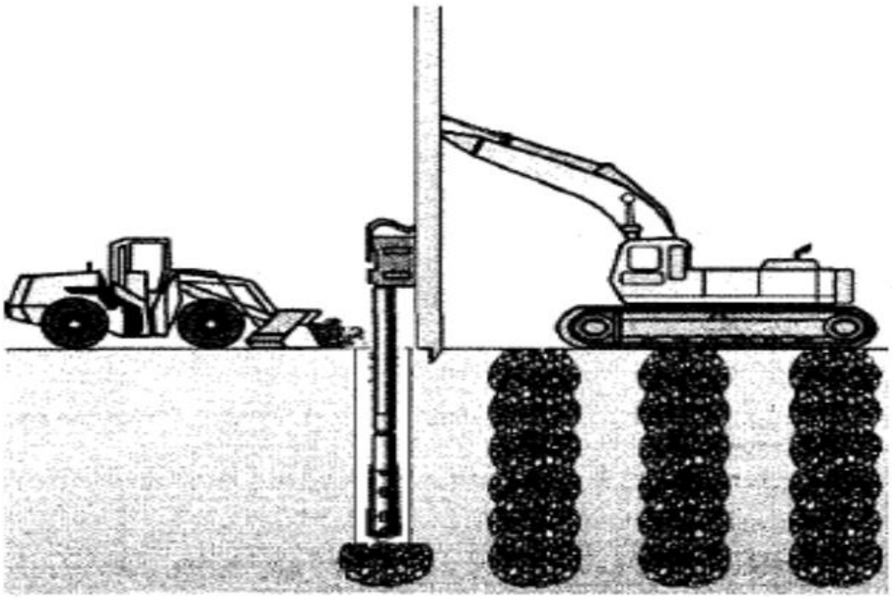


Figure 2. 5: Construction of vibropiers (after Hayward Baker, Inc., 2006).

The Geopier method is another technique of constructing stone columns which includes excavation of the borehole of diameter typically 0.61 to 0.76m with the help of an auger. This is followed by pouring in clean, well graded crushed stone (without fines) with maximum size up to 50mm or 2in. The stones are placed and compacted in lifts typically of 0.3m in thickness. A specially designed dropping hammer is used with a beveled head at 45 degrees which compact the soil efficiently, gaining increased stiffness in lateral and vertical directions. A detailed installation procedure is shown in figure 2.6. The ramming increases the nominal diameter of the stone column by 3in or 76mm beyond its initial diameter thus restraining the adjacent soil and increasing its stiffness which in return confines the column. The bottom length of stone column is increased by one diameter of the column due to impact loading from top during densification process. In other words, the soil is pre-stressed and pre-strained which enhances its bearing capacity and reduces settlement. The uneven side of the Geopier helps in increased side friction which increases the load carrying capacity of the column [14].

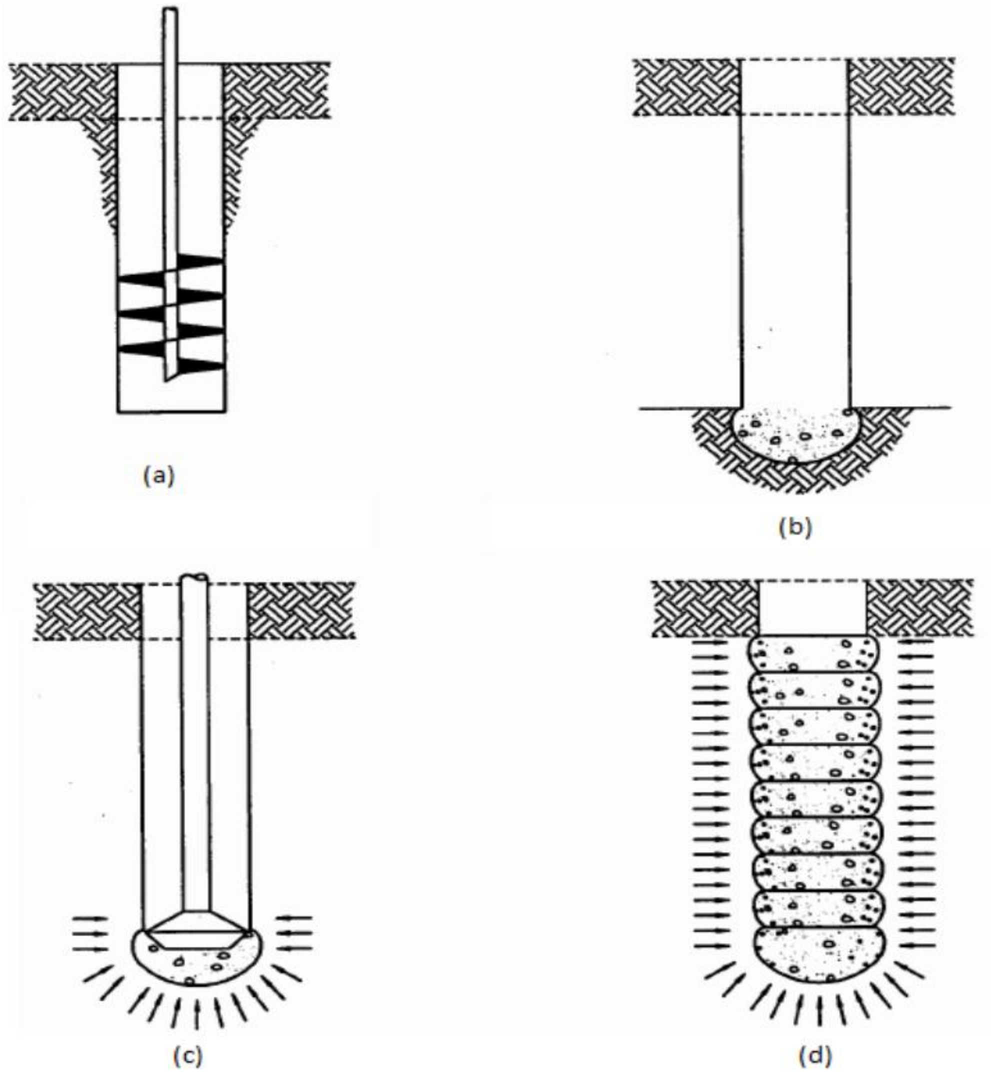


Figure 2. 6: Typical construction of Geopier (a) Making cavity (b) Creating the bottom bulb (c) Ramming with undulating layers (d) Installation complete [45]

2.3. FAILURE MECHANISM

Before 1974, research on stone column was limited to field experimental work. In

1974, Hughes and Withers described in their paper the bulging of columns in the upper part approximately about four times of its diameter from top of the column. Furthermore, it explained the bulging behavior of group of columns following the unit cell concept. The unit cell concept considers the individual behavior of columns. Each column in a unit cell acts independently in taking the load coming from the footing and bears no interaction with adjacent column. The load is transferred to the surrounding soil symmetrically around the column known as the equivalent diameter. Lawton [12] considers three possible failure behaviors of single columns loaded under axial load. These are mentioned as under:

a. Bulging failure
b. General or local shear failure
c. Punching failure

Most often, the failure mechanism is dependent on the confining capability of the soil surrounding the column as well as the soil present beneath the layer, which supports the stone column. Bulging failure occurs in the stone material either, due to short length of the column, when the length is less than 2 to 3 times the diameter of the stone column, or if the stiffness of the stone column is not much more than the surrounding soil. If the soil is layered, the column will bulge in the weaker layers where the induced horizontal stresses are more than the lateral confinement provided by the parent soil. Local or general shear failure occurs in the same way as shallow footing fails in unimproved soil. Similarly, punching failure occurs, when the load supported by the column is much more than the sleeve friction that is developed at the periphery of the composite layer [13]. [2] presented the failure mechanism of stone column in uniform homogenous soil as in figure 2.7.

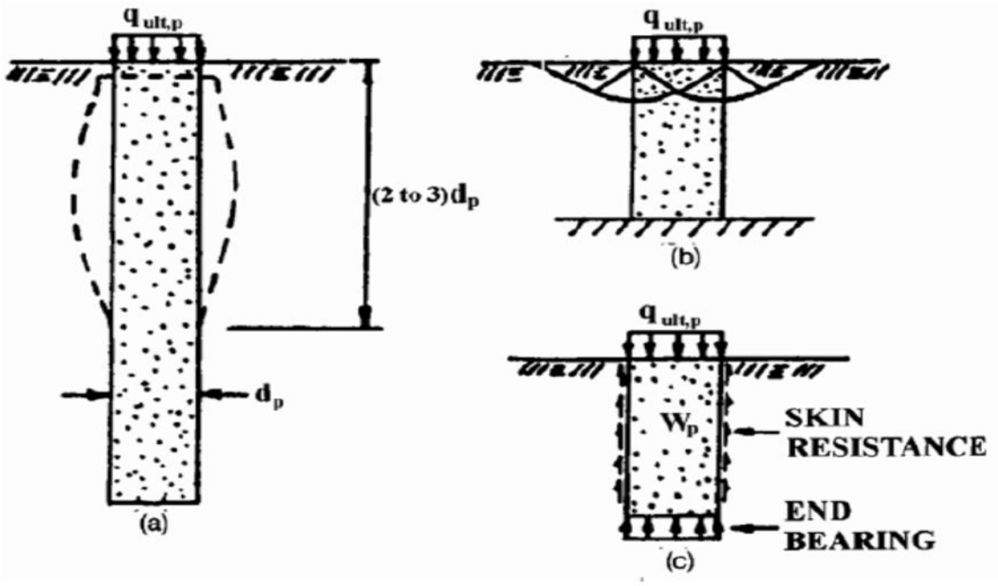


Figure 2. 7: Failure mechanism for single column in homogenous soil: (a) Bulging, (b) General or Local shear, (c) Punching ([2]).

2.3.1. Bulging Failure of Stone Columns

Mitchell [9] describes the phenomenon of bulging failure of single stone columns in soft soils owing to the difference between the relative stiffness of the two materials. When load is applied to the columns, shear strength is mobilized along the sides of the column as well as within the column material. The columns tend to bulge outward when the shear stresses increase within the column. This bulging is restrained by the lateral earth pressure that increases with the depth of soil due to overburden pressure. Hence, it is accurate to say that the bulging will be more at the top of the column due to less resistance of soil matrix.

Gibson and Anderson [15] analyzed the cylindrical swelling of columns through elasto-completely plastic soil. Hughes and Withers [1] related the bulging of column to the swelling of the cylindrical cavity in clay as in the pressure meter test.

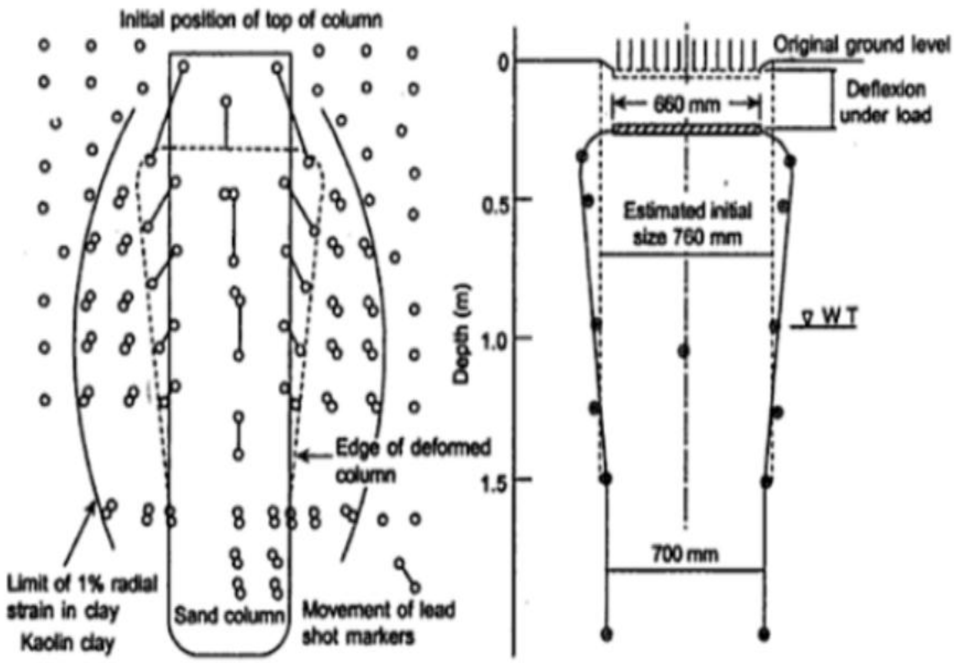


Figure 2. 8: Deformation of stone column [1]

They constructed a model in normally consolidated clay (Kaolinite) with low shear strength (19.1kPa). The bulging happened at a distance of 2 to 3 times diameter of the column from the top. In this model, sand columns of 150mm length with diameter varying between 12.5mm to 38mm were used. The main objective of this study was to observe the deformation behavior of the laboratory as well field tests, which were similar as shown in figure 2.8. The load coming from the top caused the bulging as well as vertical movement of the column, which in turn pressurized the adjacent soil.

2.3.2. General Shear Failure of Stone Columns

Many researchers found out that stone columns also failed at localized spots as well as due to shearing at the composite region. To study the bearing capacity of improved ground the general shear failure approach, was adopted. Madhav [6] considered the stone column as a granular trench stipulating the plain strain effect. An analytical approach was adopted to help understand this stipulation. The work equation is developed using upper bound analysis by equating the work done which is a result of; (i) weight of soil (ii) applied load and (iii) soil surcharge, to the internal energy in the soil continuum which behaves as a plastic region, for which Coulomb's yield criteria holds valid.

The general shear failure mechanism is analyzed for two scenarios;

a. The width of the trench is equal to or less than the width of footing, which rests on top of the trench soil system, ($A/B \leq 1$) and

b. The width of the trench is equal to or greater than the width of the footing resting on top of trench soil system ($A/B \geq 1$).

The Different zones of plastic equilibrium under the area AGCDEF are mentioned as under;

a. An elastic zone AGC with wedge angle ξ (Active Rankine Zone)

b. A transition zone GCD in radial shear state with included angle $\theta 1$ bounded by log spiral and angle of internal friction, $\Phi 1$ of trench material.

c. A transition zone GDE in radial shear state with internal angle $\theta 2$ bounded by log spiral and angle of internal friction, $\Phi 2$ of the weak clay.

d. Zone GEF in passive Rankine state with wedge angle η

When load is applied from the top and is transferred to the soil in the elastic zone it would cause it to spread but it is negated by friction and adhesion between the soil and the footing. The wedge AGC is laterally restrained and acts as if it were part of the footing hence the soil is in state of elastic equilibrium. This wedge moves downwards with the same initial velocity V_F of the footing under the superimposed load. The downward movement of the footing and the wedge AGC is accompanied by the sideways movement of the surrounding soil. The central angle θ_1 and θ_2 change as the wedge angle ξ and η

change. The ratio of width of the trench to strip width and the angle of internal friction Φ_1 of the trench material also affects the central angle. The properties for the trench material are cohesion C_1, angle of internal friction, Φ_1 and density, γ_1. This theory is generally valid for C- Φ- γ soils. The properties of natural soil are cohesion C_2, angle of internal friction Φ_2 and density γ_2.

The velocity component of the zones AGC, GCD, GDE and GEF act in the same direction to that of the force V_F, while that of surcharge acts in the opposite direction. This convention is related to the work done in the direction of force, V_F or against the direction of V_F. The equation for the work done by ultimate load, q_{ult} is as under;

$$q_{ult} = C_2N_c + (\gamma_2 B/2)\, N_\gamma + \gamma_2 D_f N_q \qquad \text{(Eq. 2.1.)}$$

Where $N_c = [C_1/C_2]\, N_{c1} + N_{c2}$

And $N_\gamma = [\gamma_1/\gamma_2]\, N_{y1} + N_{\gamma 2}$

N_{c1}, N_{c2}, $N_{\gamma 1}$, $N_{\gamma 2}$ and N_q are dimensionless factors, depending upon the properties of trench material, surrounding soil and ratio of trench width to strip footing width.

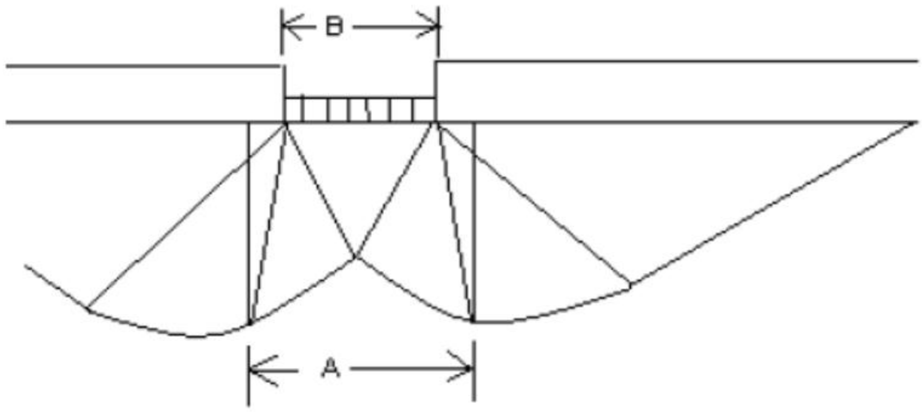


Figure 2. 9: Mechanism of general shear failure (A/B≥1) by Madhav and Vitkar [6]

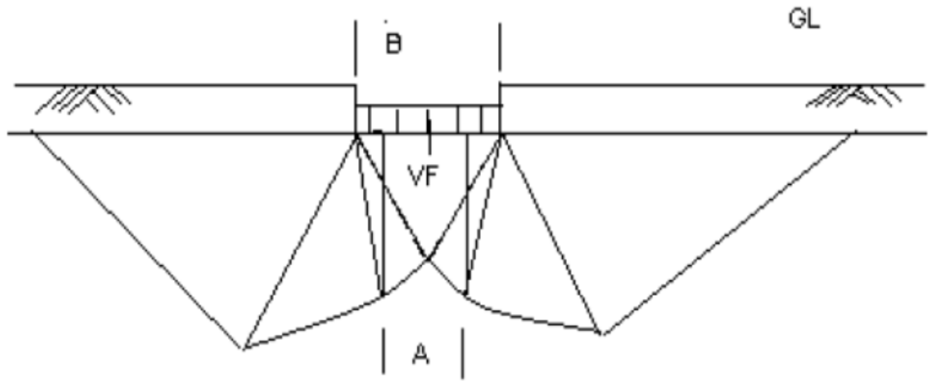


Figure 2. 10: Mechanism of general shear failure (A/B≤1) by Madhav and Vitkar [6]

2.4. UNIT CELL CONCEPT FOR STONE COLUMNS

In order to simplify the assumption and representation of composite materials present under the footing and their behavior under the applied load the unit cell concept was proposed by different researchers including [1], [17], [28], [18], [19], [20], [22]). In 1974, Hughes and Withers [1] reported that column bulges in the top part at approximate distance of four times diameter from top of column. Furthermore, it is reported that stone columns in a group also fail by bulging following unit cell concept. The unit cell concept is based on the assumption that the column and the surrounding soil acts as a single unit deforming at same strain (Figure 2.11). For that to happen two conditions are necessary; rigid loading and a loading area larger than the thickness of the reinforced zone [22].

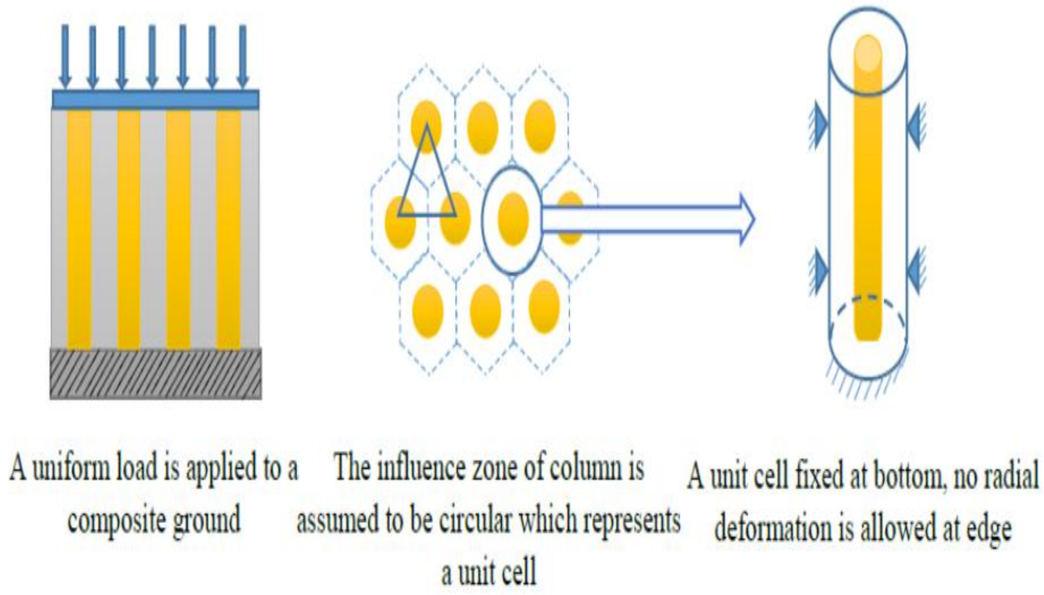


Figure 2. 11: Unit cell concept [18]

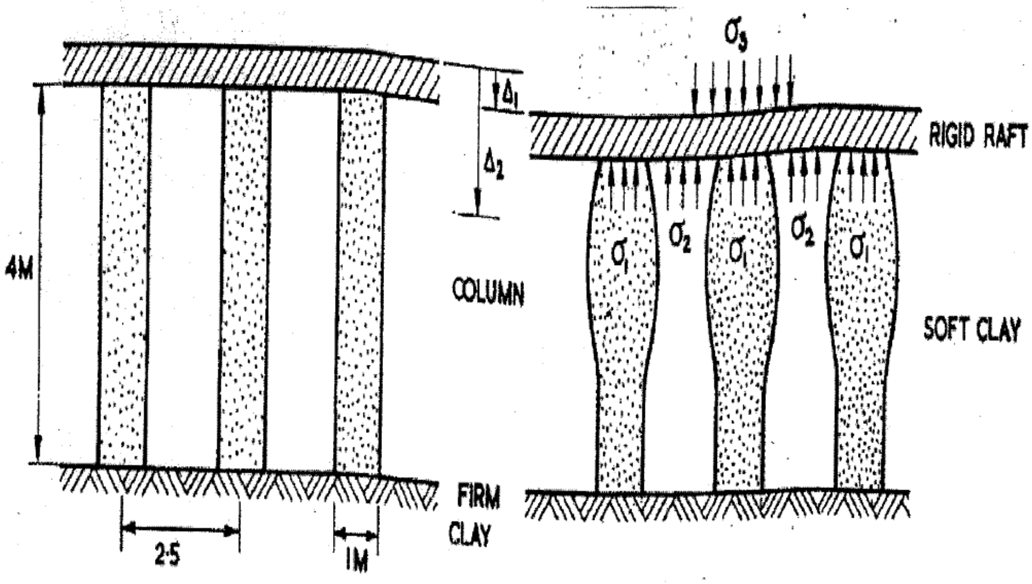
Meanwhile in a group, each stone column acts independently from adjacent columns and there will be no interaction among the columns (Figure 2.12). The capacity of the group is taken as the combined capacity of all the single columns in a group. According to unit cell idealization the following assumptions are made;

a. The unit cells are infinitely represented by each column in the lateral direction

b. The load applied from top on the unit cell remains within the periphery of the unit cell

c. The lateral deformation at the periphery of the unit cell, resulting from loading, does not cross the outer boundary of the unit cell due to symmetric loading and geometry (one-dimensional loading)

d. At the boundary of the unit cell no shear stresses exist



(a) Before failure (b) After failure

Figure 2. 12: Stone column group under a large raft [1]

2.5. GROUP INTERACTION OF MULTIPLE STONE COLUMNS

[23] Terashi first reported on group interaction as well as group efficiency. They reported that unit cell concept did not hold valid for group of stone columns. This observation was based on results from centrifuge setup. However, Hughes and Withers [1] indicated that group columns act as independent columns if the center-to-center distance between them is more than 2.50d. Different failure mechanisms are bulging of columns, general and local shearing of columns and punching of individual stone columns.

In 1995, Hu [24] performed laboratory tests on group of stone columns shown in figure 2.13. He indicated that group interaction played an important role in understanding the failure behavior of group of columns. A number of failure criteria were identified, such as bulging, bending, punching and shearing. However, due to close interaction of the columns bulging happened in the lower portion of interior columns while, the exterior columns bulged in the top regions.

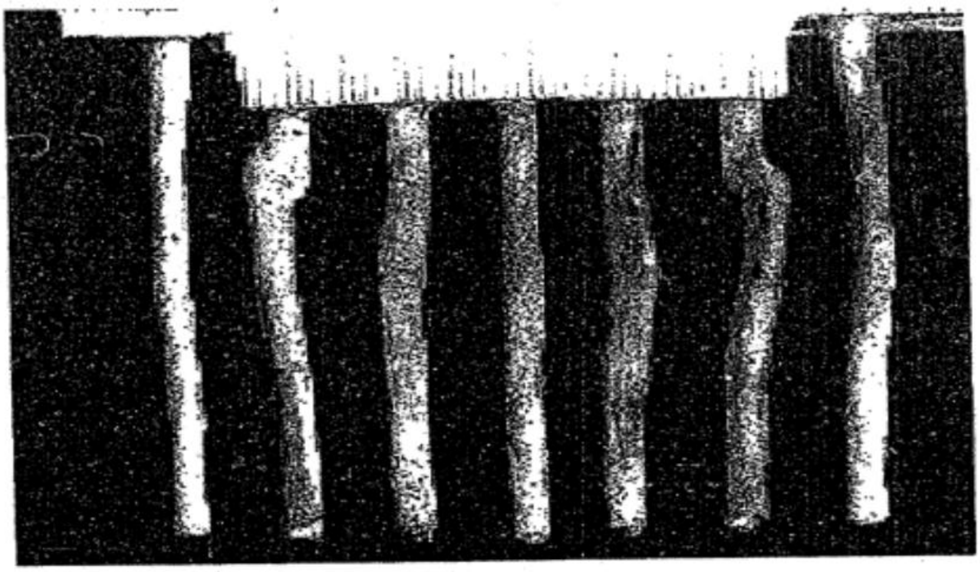


Figure 2. 13: Deformed shape of group of stone columns after [24]

General shear failure was mostly associated with group columns. [3] Mckelvey carried out experimental tests on a group of five stone columns and found out that the bulging of the interior column was uniform, whereas the columns on the periphery bulged away from adjacent interior columns.

Ambily and Gandhi [28] carried out experimental work as small scale laboratory testing and replicated the results numerically as well using finite element software. Their work included testing of isolated as well group of stone columns. They found out that mode of failure of column alone loaded was by bulging but the same was not true for entire area loaded. Due to confinement effect of the surrounding soil the column did not fail even for a high settlement value of 10-mm. This behavior was comparable to the behavior of group of columns as the interior columns did not fail by bulging. These tests were conducted for an area replacement ratio of up to 10% below which group interaction was not significant.

2.6. DESIGN PARAMETER

Frikha [26] states that numerous factors are responsible for affecting the load transfer mechanism of stone columns in soil. Experimental studies reveal the following parameters that affect the behavior of vertically loaded stone column in soft soil; stone column diameter (d_s), stone column spacing (s), area replacement ratio (a_s), shear strength of surrounding soil (C_u), strength of column material, slenderness ratio of column (L/D), drainage conditions, grain size of column material, rate of deformation.

2.6.1. Stone Column Diameter

Stone columns inclusion in soft soil is a process that compensates itself by lateral expansion. The extent of the expansion depends upon the strength of the surrounding soil. Softer the soil around the stone column more will be the diameter of the stone column. Besancon [27] developed a graphical relation between diameter of stone column and undrained shear strength of soil.

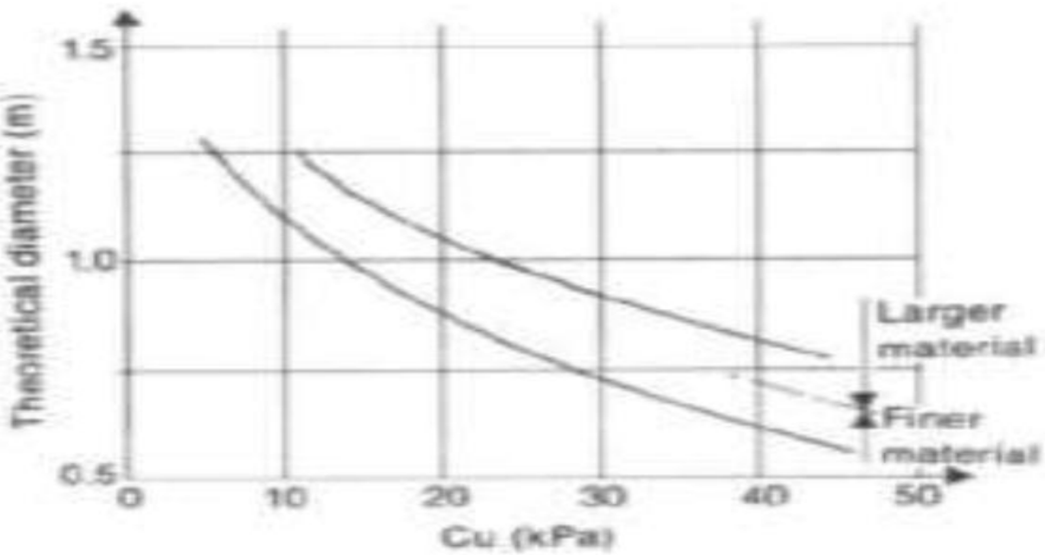


Figure 2. 14: Effect of soil strength on theoretical column diameter [27]

The installation procedure also affects the diameter of the stone column. Due to vibrations, the completed diameter is always greater than the original hole excavated. The diameter resulting from Vibrofloat procedure (diameter 300-500mm) is from 0.6m in stiff clays to 1.1m in soft clays. The diameter of stone column constructed by dry method is less than that by wet method [29]

2.6.2. Stone Column Pattern

Stone columns can be installed in different patterns to give different densities. According to [30], stone columns can be installed in three arrangements as shown in figure 2.15. They are;

a. Triangular pattern

b. Square pattern

c. Hexagonal pattern

Mostly equilateral triangle pattern is used for installation of stone columns because it gives the densest packing of all.

2.6.3. Spacing of Stone Columns

The spacing of stone columns has no specific guidelines and is subject to site-specific conditions and the purpose to be served by the improved area. The minimum and maximum spacing cannot be determined from guidelines and are rather found out by the maximum allowable settlements and the maximum control area to be covered from one column such that no overlapping of improved zones occurs [31]. The spacing of stone columns can be decided upon many factors such as the foundation of the structure for e.g. closer spacing is provided under isolated footing than raft footing [31]. The loading pattern, shear strength of soil, column material and installation techniques are various factors that decide the spacing of columns.

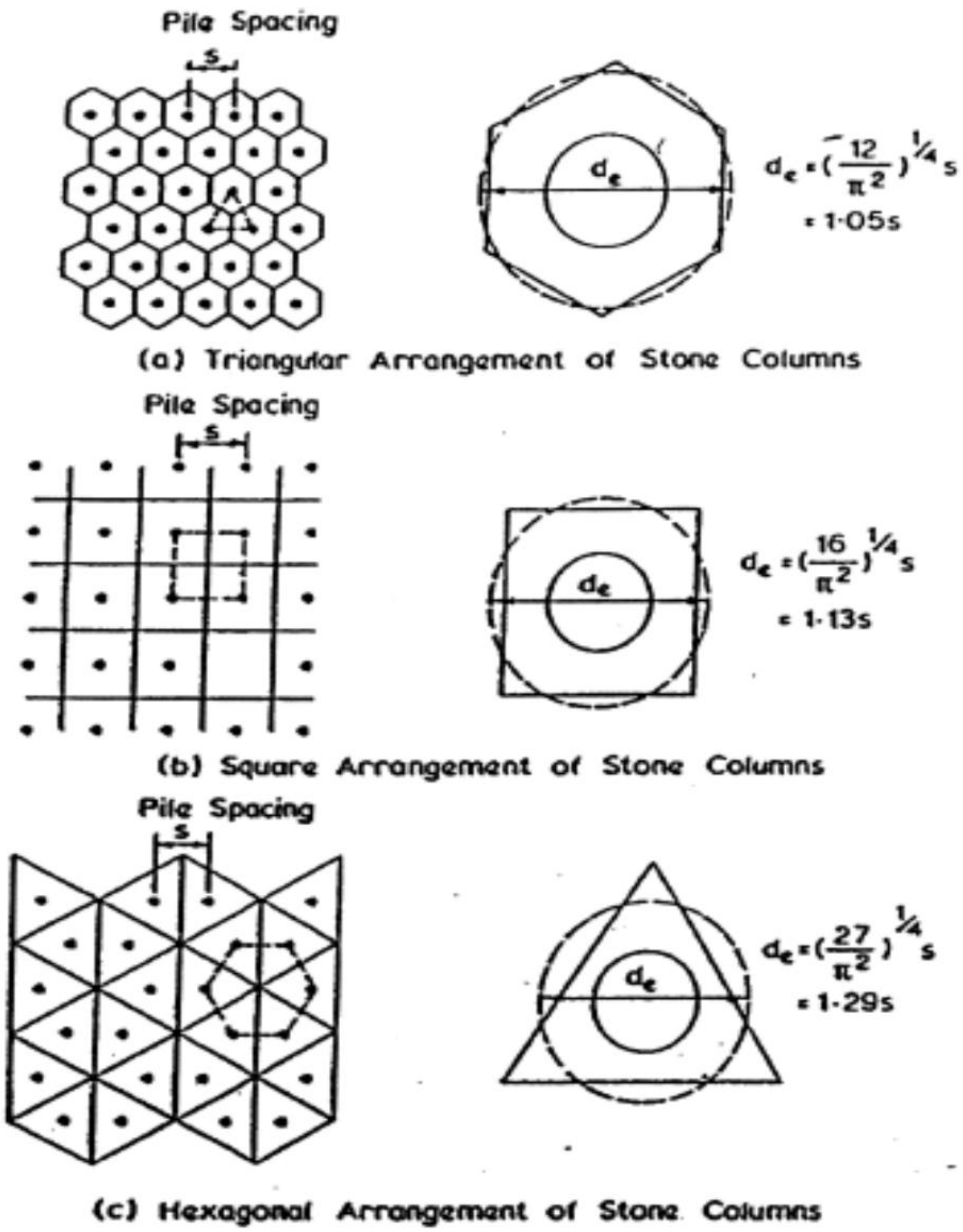


Figure 2. 15: Different stone column patterns [30]

An important factor considered while determining spacing of the columns is the tributary area of the column. The tributary area is considered as an equivalent circle around the column having same total area. The equivalent circle has an effective diameter, which is related to the spacing of columns by the equation;

$D_e = S. C_g$ (Eq. 2.2)

Where;

D_e= equivalent diameter of unit cell

S= spacing of columns

C_g= constant related to columns pattern (for triangular pattern, C_g= 1.05, square pattern, C_g=1.13, hexagonal pattern, C_g= 1.29

2.6.4. Area Replacement Ratio

The amount of soil replaced by the stone column is expressed in terms of area replacement ratio, a_s which is defined as the area of stone column to the area of the soil within the unit cell after compaction of the soil. The ground reinforcement response by granular columns is improved by increasing the area replacement ratio up to certain extent [32].

$$a_s = A_s/A \quad \text{(Eq.2.3)}$$

Where: a_s = area replacement ratio

A_s= area of stone column

A= area of soil

Area replacement ratio can also be expressed in terms of spacing of columns by the equation,

$$a_s = C_1 (D/S)^2 \quad \text{(Eq.2.4)}$$

Where: D = diameter of compacted stone column

S = center to center spacing of columns

C_1 = constant dependent of the pattern of stone columns (for triangular pattern, it is equal to $\pi/(2\sqrt{3})$ and for square pattern it is $\pi/4$)

2.6.5. Stress Concentration Factor

Stress concentration factor, n is defined as the ratio of the stress in columns to the stress in the surrounding clay.

$$n = \sigma_s/\sigma_c \quad \text{(Eq. 2.5)}$$

When load is applied on ground treated with stone columns, the stress concentration increases in the stone column while it is accompanied by reduction of stresses in the surrounding clay. This is due to mutual load sharing between the two and due to the stone column being the stiffer material as well. According to Ambily and Gandhi [28], the stress concentration factor "n" increases with the increase in modular ratio and decreases when the shear strength of the surrounding soil increases. The stress concentration factor "n" increases with increase in time of consolidation, ([33], [34]) and decreases along the length of column from top to bottom ([2]). The stress concentration factor "n" is found out be more for end bearing columns as compared to floating columns [35].

2.7. ANALYSIS OF STONE COLUMNS

2.7.1. Small Scale Models

Ambily and Gandhi, [28] carried out experimental studies on single as well a group of seven columns in a cylindrical tank by varying different parameters like spacing between columns, shear strength of clay, stiffness of column material and loading conditions. The results were compared with numerical models prepared by PLAXIS software, which showed a good agreement. Spacing more than 3 times the diameter of columns does not show any significant improvement while a single column represents the behavior of interior columns among a group of columns. Furthermore it was found out that reducing the area replacement ratios affects the group interaction of columns and area ratios less than 10% have no significant effect on improvement of soil.

Malekpoor and Poorebrahim performed tests on both floating and end-bearing compacted lime-soil rigid stone columns [36]. Laboratory tests were performed to analyze the influence of varying diameter of column (D), length to diameter ratio (L/D) and area replacement ratio (A_r). Scaling effects become negligible when diameter of column is increased up to 100mm.

Christoulas [41] carried out laboratory tests on the stone column improved ground by installing pressure cells and electronic piezometers to calculate the lateral stresses as well pore water pressures generated in the composite model. It was confirmed that length of bulging failure was about 2.5 to 3 times the diameter of column as found out by [1].

Shivashankar performed plate load tests on small laboratory samples of stone columns in layered soil [37]. The stiffness of improved ground was determined by loading the entire area and the columns were separately loaded to find the axial capacity of the columns. The top weak soil was found to have more influence on the strength, bearing capacity of soil as well bulging of column.

2.7.2. Field Tests (Large Scale Testing):

Meyerhof [38] compared data collected from various field tests with that of simple analysis equations developed by changing different parameters like spacing between the columns, length to diameter (L/D) ratio of columns, properties of the weak soil medium, properties of the column material, stresses developed due to installation technique and magnitude of overlying load. The correlation between the analyses as well field results is satisfactory due to number of uncertainties involved in estimation of various parameters. However, the strength, stiffness of composite soil and dilation is mainly influenced by the spacing between the columns and degree of compaction of column material.

2.7.3. Numerical Models

S.K. Tan [40] performed two-dimensional (2D) finite element analysis on floating stone columns in soft soil using finite element software PLAXIS -2D ver 9.0. Undrained analysis is conducted along with consolidation analysis to study the settlement and consolidation characteristics of the improved soil system. Influence of various parameters like area replacement ratio, angle of internal friction of column material, loading rate and earth pressure after installation are analyzed. These influencing factors are confirmed as important factors in design of floating stone columns.

Guetif [44] proposed a method for estimating the improvement of Young's modulus of soft clay by performing numerical analysis considering a composite model in PLAXIS software. The vibro-compaction method is simulated using a Mohr-Coulomb perfect plastic behavior model. The improvement of Young's modulus of soft clay is estimated from the results obtained and the zone of influenced determined based upon the results.

Chapter 3

EXPERIMENTAL PROGRAM AND NUMERICAL MODELS

3.1 GENERAL

Stone column as a ground improvement technique needs a borehole to be made in the soft soil which is then filled with granular material (crushed stone, gravel etc.). The granular is compacted to a desired density to help achieve the strength to sustain overlying loads. The process of improvement is a function of two processes taking place respectively:

- The granular material reinforces the soil owing to their high strength, being a stiffer material they sustain most of the load applied to the soil. The also introduce the confining effect by having greater skin friction along the depth and the bulging provides the necessary passive pressure.

- Due to having higher permeability than the original soil present, columns act as vertical drains thereby increasing the rate of consolidation and reducing consolidation settlement after the loads are applied.

To study the load sharing mechanism between the native soil and the stone columns it is pertinent to rely on large scale field testing but due to limited resources certain laboratory tests need to be introduced which help understand their behavior as a ground improvement technique. The laboratory tests help correlate the results with either large scale testing or numerical modeling and thus be validated. To study the behavior of stone columns as a ground improvement technique and analyze their effects laboratory testing was performed on the untreated soil sample as well as on the improved soil samples at Geotechnical Engineering laboratory, NUST Institute of Civil Engineering (NICE). All tests were performed in accordance with the standards set by ASTM.

3.2 MATERIALS

The materials used for preparation of the models and to carry out fundamental tests are discussed below.

3.2.1 Soil

The soil samples were retrieved from Jehangira area of Sawabi (33.971091^{o}, 72.218583^{o}) (Figure 3.1). The locals were questioned regarding properties of soil and the area visited afterwards to retrieve the samples. They samples collected were first air-dried for 24hrs and then subjected to pulverization. After pulverization the soil sample was again oven dried for 24hrs at temperature of $105^{\circ}C$ as shown in figure 3.2 (a) and (b).

Figure 3. 1: Soil sample collection point at Jahangira area Sawabi (Archives Google Maps)

(a) (b)

Figure 3. 2: (a). Pulverizing sun dried soil sample (b). Oven drying soil sample at 105 degree Celsius

3.2.2 Sand

Clean sand was procured from Lawrencepur and air-dried before subjected to its intended use. The sand is passed from 4.75mm sieve before its use and direct shear test is performed to find its angle of internal friction as well as shear strength.

3.2.3 Stones

Crushed marble stones were collected from Rashakai area of Mardan. The crushed stones were thoroughly washed in Geotechnical Engineering laboratory at NICE and then air dried for 24hrs. The air dry stones were passed through sieve # 3/8" and retained on sieve # 10. The sample selected comprised of stones with sizes from 10mm to 2mm.

3.3 SOIL CHARACTERIZATION

Basic tests were performed in NICE Geo-technical engineering laboratory to determine the characteristics of soil while conforming to the standards provided by ASTM. Following tests were performed:

3.3.1 Grain Size Distribution Test

Hydrometer analysis test was performed as per ASTM D 422-63. Oven dried sample was weighed to 500gm after pulverizing. The samples were kept in graduated cylinders for 24hrs and readings noted. Care was exercised while noting the readings on the hydrometer. Figure 3.3 shows the test performed in laboratory.

3.3.2 Atterberg Limits

The consistency limit of soil was found out as per ASTM standards. Soil sample was passed through Sieve No. 40 oven dried for 24hrs to find out the liquid limit (ASTM D423-54T), plastic limit (ASTM D424-54T) and shrinkage limit of soil (ASTM D 427-39) (see Figure 3.4). The Atterberg limits are useful in classifying the soil according to plasticity. Soil was classified using Unified Soil Classification System (USCS) and AASHTO classification system.

Figure 3. 3: Hydrometer Analysis of Soil sample

Figure 3. 4: Casagrande's apparatus for finding Liquid Limit of Soil

3.3.3 Specific Gravity Test

Specific gravity of soil was found out using Water Pycnometer method as per ASTM standard D854-14. Soil was passed through sieve no.4 and 200 gram sample was weighed. Care was taken while replacing the soil with equal volume of water. Figure 3.5 shows the test being performed in laboratory.

Figure 3. 5: Performing Specific Gravity test using (a) Pycnometer and (b)Volumetric flasks

3.3.4 Compaction Test

Standard Proctor as well as Modified Proctor Compaction test was conducted to establish the relationship between moisture content and dry density of soil sample through which the maximum dry density and optimum moisture contents were determined as per ASTM D 1557-02. The soil was compacted in 5 layers with application of 25 blows to each layer, constituting a compaction effort of 12,375 ft-lb/ft^3 and 56,250 ft-lb/ft^3 for each method. Compaction was done with the specified hammers weighing 5.5 lbs and 10 lbs and dropping from a height of 12 in and 18 in in a mold of 4 in diameter.

3.3.5 Unconfined Compressive Strength Test

The unconfined compressive strength tests were performed on test samples of 40 mm diameter and 80 mm height conforming to ASTM standard D 5102-96. Soil sample was passed through sieve no. 40 and UCS test was performed to establish the relationship between undrained shear strength of soil (S_u) and moisture content. The soil samples were carefully prepared at respective moisture contents and the trimmed to the specified ratio. The samples were tested in digital UCS machine as shown in figure 3.6.

3.3.6 One-Dimensional Consolidation Test

According to ASTM D2435, one dimensional consolidation test was conducted at moisture contents of 31%, 35% and 39% to determine the coefficient of vertical consolidation (C_v) and coefficient of volumetric compressibility (m_v) of soil. The Elastic Modulus (E_s) of the soil was calculated from these results.

3.3.7 Tri-Axial Compression Test

Consolidated undrained (CU) test also known as consolidated quick test was

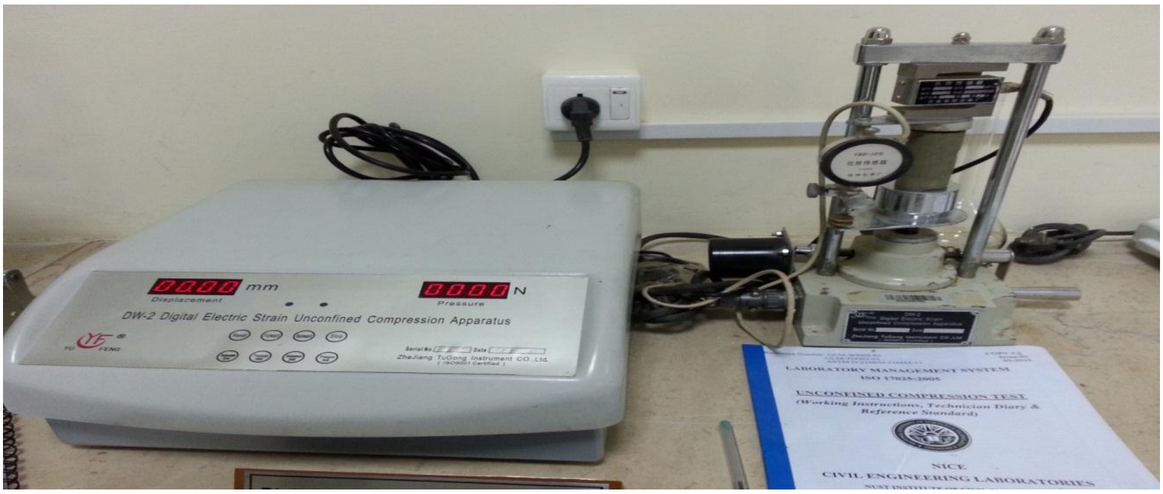


Figure 3. 6: Unconfined Compression Strength test using digital electronic strain UC apparatus

performed as per ASTM Standard D 4767 for finding out ultimate, laterally confined, compressive strength, angle of internal friction of soil, Cohesion, shear strength, Modulus of Elasticity, pore water pressure and Poisson's ratio of soil. Different trials were performed at different moisture content to get stress-strain relationships of loaded soils. Triax-50 apparatus at NICE Geo-technical engineering laboratory was used for performing the tri-axial compression test as shown in figure 3.7. Care was taken while noting down readings of load at different increments of time and pore pressure as shown in figure 3.8.

Figure 3. 7: Control Unit of TRIAX50 Tri-Axial Compression Test machine

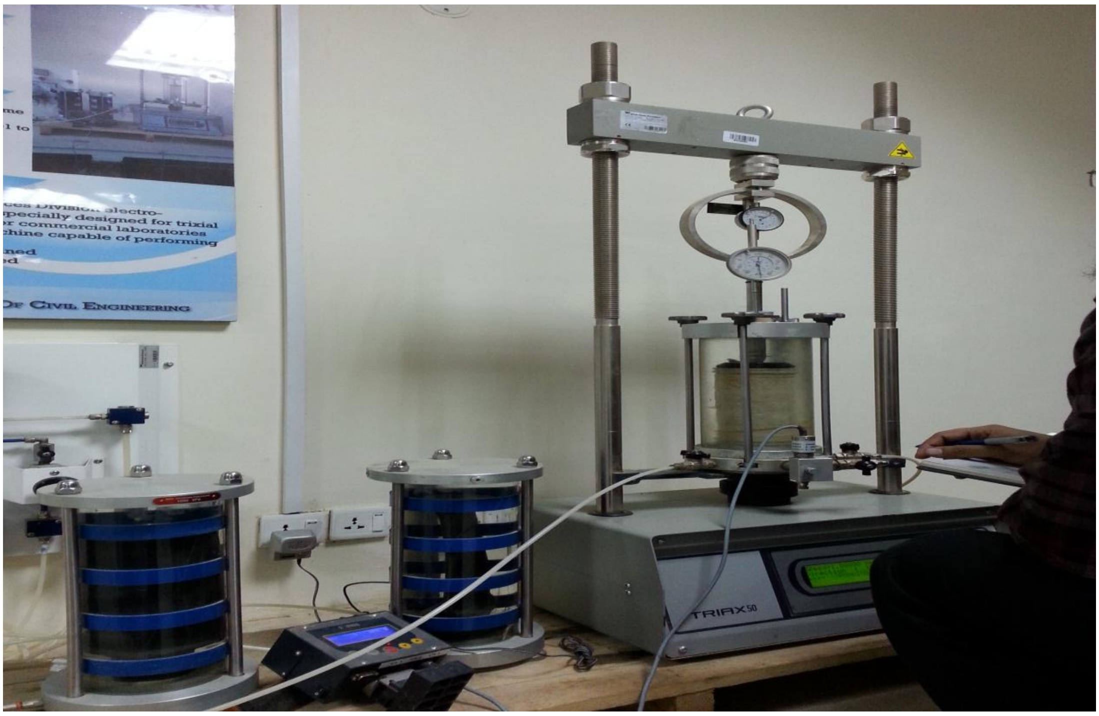


Figure 3. 8: Performing Tri-Axial compression test (CU)

3.4 LABORATORY TESTING OF SOIL SPECIMENS (LOAD VS. SETTLEMENT TEST)

3.4.1 Mold Description

Special steel mold were fabricated for testing the soil specimens in compression chamber of UTM. The specifications of the mold are:

a. Inner diameter = 300 mm

b. Height = 350 mm

c. Wall thickness = 6 mm

d. Weight ~ 29kg

Before loading the mold with soil thin coat of grease oil was applied to the inner walls for reducing friction while loading. The mold is shown in figure 3.9.

Figure 3. 9: Special steel mold for Load testing of soil (Not to Scale)

3.4.2 Preparation of Soft Clay Bed

For each trial, 28 Kg of soil was weighed and water taken in graduated cylinders depending upon the shear strength of soil. All the tests were performed on normally consolidated clay bed at three different shear strengths of 54 kPa, 32 kPa and 14 kPa. Each sample was first subjected to unconfined compressive strength test to find the undrained shear strength of soil at moisture content of 31%, 35% and 39%. For preparation of clay bed, oven dried clay sample was taken and 1446 ml, 1634 ml and 1820 ml water was added to each layer to obtain desired shear strength values of 54 kPa, 32 kPa and 14 kPa. Water was thoroughly mixed with the soil for uniform moisture throughout the soil. (See figure 3.10a and 3.10b).

(a) (b)

Figure 3. 10: (a) Calculating amount of soil and water (b) Mixing of soil and water for clay bed

Prior to filling the steel mold with soil sample a thin coat of oil was applied along the inner surface to reduce friction between clay and wall. The soil was filled in the mold in 6 layers, each layer 50 mm thick after compaction. Each layer was given 70 blows with a 10 kg tamper dropped from a height of 300 mm. The compaction energy provided to each layer was equal to 12,375 ft - lb/ft^3 as in the case of Standard Proctor Test. The mold was filled with soil up to a total height of 300 mm. Care was taken while compacting the soil with the tamper to ensure that no significant air voids are left after compaction (See Figure 3.11a and 3.11b).

Moisture content of the soil was calculated after the clay bed was prepared to ensure that it exhibited the required shear strength. The mold were covered with polythene sheet for 4 days and kept at room temperature to ensure uniformity of moisture (See Figure 3.12). The molds were loaded vertically in a compression chamber (UTM) to study the load-displacement behavior of the untreated soil.

(a) (b)

Figure 3. 11: (a) Compacting soil bed with 10 Kg Tamper (b) Soil bed after compaction

Figure 3. 12: Compacted soil bed covered in polythene sheet for uniform transfer of moisture

3.4.3 Installation of Single Stone Column

Columns were constructed by replacement method after preparation of soil bed. A thin open-ended seamless steel pipe of 37 mm outer diameter with wall thickness 1 mm was inserted into the clay at the center of clay bed up to the desired depth. Slight grease was applied on the outer surface of the pipe for easy penetration and withdrawal of the pipe without significant disturbance to the surrounding soil. To mitigate suction effects, a maximum height of 50 mm of soil was removed at a time.

After removing the soil crushed stones were poured into the hole from top in layers of 50 mm each. For achieving uniform density light compaction effort was adopted and it was ensured that it did not create any disturbance in the surrounding soft clay by bulging of the columns. A steel rod weighing 1.25 kg was used as a tamper and 15 blows provided to each layer from a height of 100 mm. Stone columns were installed to a depth of 150 mm, 200 mm and 250 mm according to length to diameter ratio of 4, 5.5 and 7. After the installation of column, the steel mold was covered in polythene sheet to ensure uniform transfer of moisture. After 4 days the treated soil was loaded in compression chamber for load-displacement behavior. Figure 3.13a, 3.13b, 3.13c and 3.13d shows different steps involved in installing single stone column in the soil.

3.4.4 Installation of Group Columns

Group efficiency of stone columns was found out by installing 3 stone columns at spacing of 3 in and 4.5 in i.e. varying S/D ratio of 2 and 3 respectively. The shear strength of soil was kept constant at 32 kPa while the L/D ratio was kept constant at L/D=5.5. Triangular pattern was selected for installation of columns as it gives the densest packing of all. Spacing was properly calculated locating center of one column from two points using method of arcs. Soil was removed through replacement method using seamless

Figure 3. 13: (a) Removal of soil by replacement method (b) Pouring of crushed stone from top (c) Compaction of crushed stone to required density (d) Stone column in soil sample

open-ended pipe. After removal of soil crushed stone was poured from the top in equal layers in each hole simultaneously. After installation of columns, the molds were covered with polythene sheet for 4 days so that uniform transfer of moisture took place prior to loading in compression (Figure 3.14a, 3.14b).

3.4.5 Loading Sequence

To study the load-displacement behavior of untreated soil and soil treated with stone columns, the molds were loaded in the compression chamber of UTM (figure 3.15). The soil samples were loaded in two different sequences:

a. Entire area loaded

b. Column alone loaded

(a) (b)

Figure 3. 14: (a) laying out triangular arrangement for stone columns (b) 3 stone columns in triangluar arrangement

Figure 3. 15: Compression loading machine for testing of untreated and treated soil sample

The entire area of untreated and stone column treated soil was loaded with a 100 mm thick steel plate having 200 mm diameter to study the load-displacement behavior. The plate was placed in the center of steel mold with no side touching the walls of the mold. Strain gauges were placed at the top surface to constantly monitor the vertical displacement. Load was applied at a constant loading rate of 0.025 MN/s. The readings were noted down till 10% strain was reached. For column loaded condition, a special steel plate with outer diameter of 40 mm was fabricated. Load was applied on top of the column to find the limiting axial capacity of the stone column. A schematic diagram of the loading setup is shown in Figure 3.16a and 3.16b. After each trial, samples were extracted from the mold and soil surrounding the column was removed carefully to observe the failure pattern of the column and recorded (See Figure 3.17a and 3.17b).

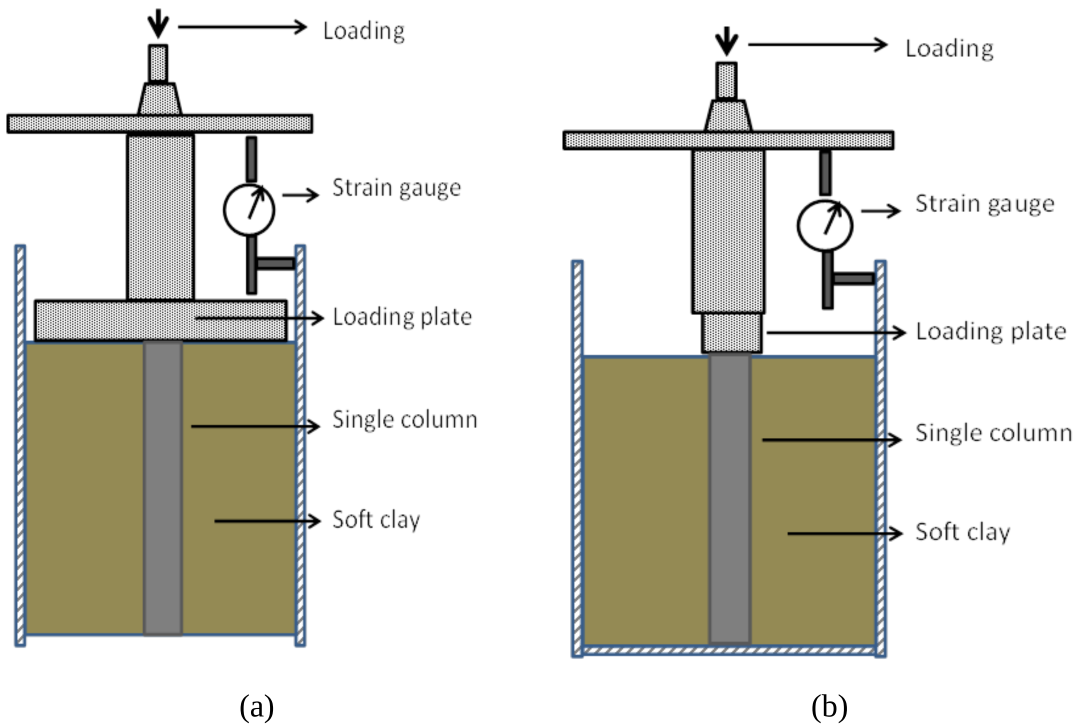


Figure 3. 16: (a) Schematic diagram of application of load over entire area (b) Schematic diagram of load over column only

(a) (b)

Figure 3. 17: (a) Extracted soil sample for analyzing its behavior after loading
(b) Section of stone column for analyzing failure mode

3.5 NUMERICAL MODELING USING FEM SOFTWARE

Finite element analysis is widely used to predict the settlement behavior of soil subjected to various loadings. The experimental work carried out in laboratory was validated by modeling the same conditions in FEM software PLAXIS 2D and tested to understand the variations in both methods and the factors causing it. The behavior of single stone column as a soil reinforcing material is depicted by unit cell concept where cylindrical shape is assumed for load sharing between stone column and soil.

3.5.1 PLAXIS 2D Software

The soil was modeled using FEM based software PLAXIS 2D which gives accurate results and is easy to operate. To model the unit cell axisymmetric loading was considered. Input parameters were defined and coordinates marked to define the cluster. Soil parameters were allocated to the cluster and mesh was generated using 15-noded elements. The 15-noded elements give more stress points and accurate calculations. Boundary conditions were defined and stages selected for calculation. The calculations were run and output recorded (See Figure 3.18a and 3.18b).

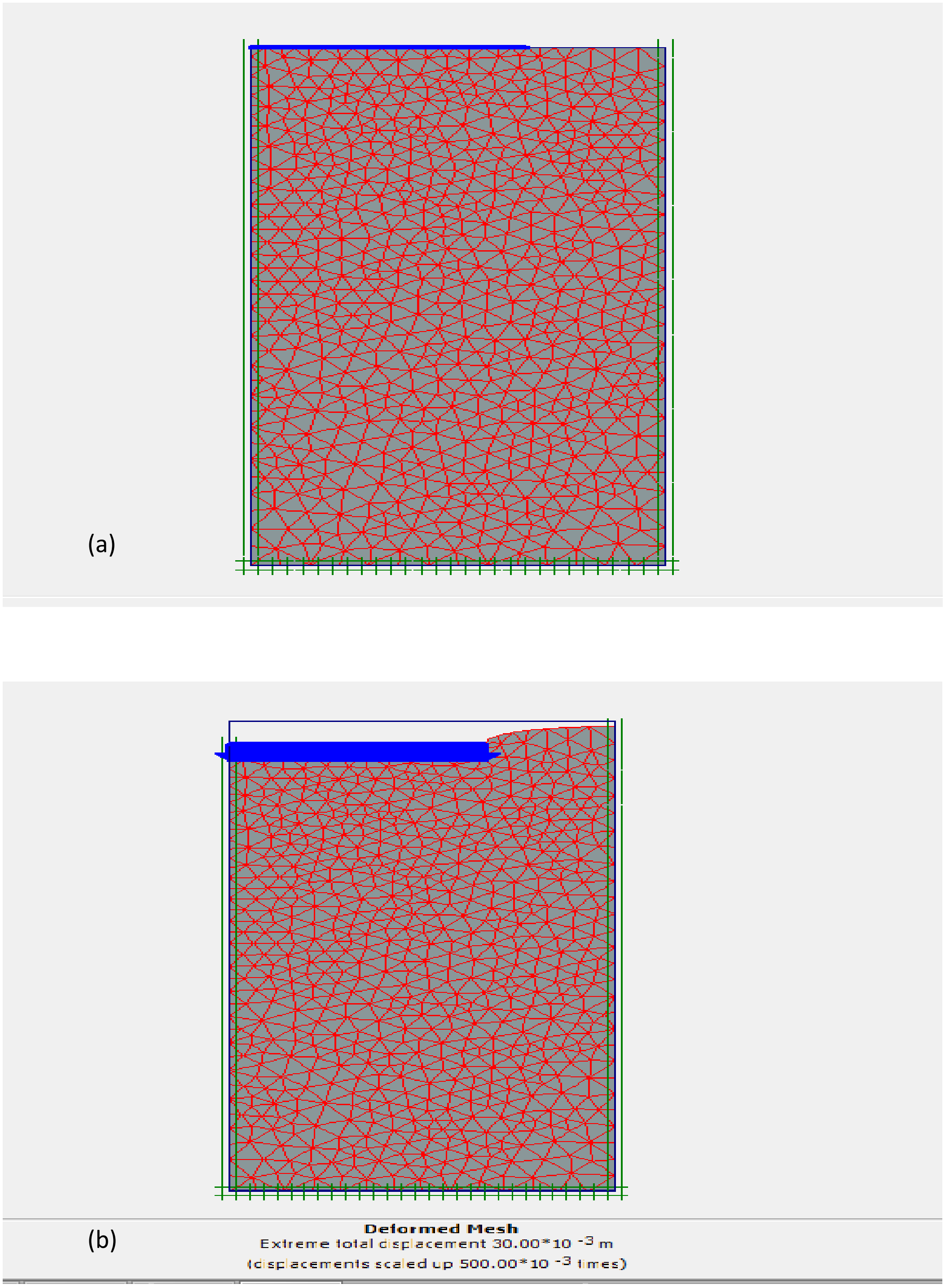


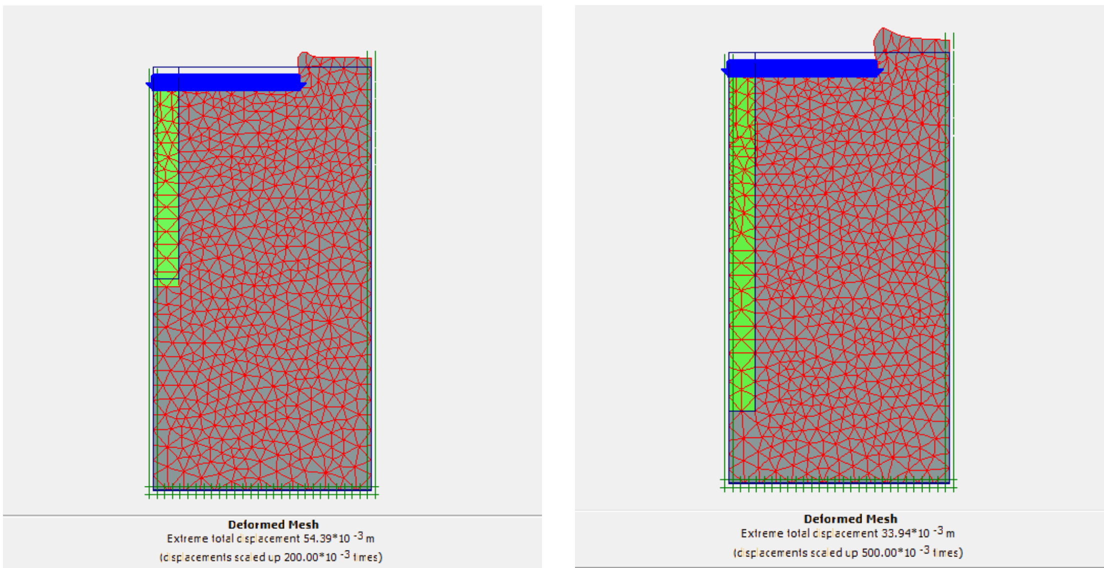
Figure 3. 18: (a) Untreated soil model with loading plate at its top half model
(b) Deformed mesh for untreated soil after running model

The input parameters for both soil and column material are given in table 3.1. The soil was modeled as Mohr-Coulomb model with undrained loading criteria designated for clay and drained loading criteria for column material.

Table 3. 1: Input parameters used in PLAXIS 2D Software

Material	w %	S_u KPa	γ_{sat} KN/m3	γ_{unsat} KN/m3	m_v m^2/kN	E KPa	v	Ø
Soil	31	54	18.87	14.4	0.000217	4610.39	0.4	0
	35	32	17.64	13.06	0.000395	2531.64	0.42	0
	39	14	17.02	12.72	0.00074	1351.35	0.44	0
Stone	-	-		16.643max 13.982min	-	45,000	0.33	41˚

Modulus of elasticity of soil used was found out from one-dimensional consolidation test. Poisson's ratio, $_v$ for both soil and column material as well as modulus of elasticity, E and angle of internal friction, Ø of crushed stone were suitable values selected from literature (Table 2.7 [42]). The models for single stone column treated soil before and after testing are shown in Figure 3.19a and 3.19b.



(a) (b)

Figure 3. 19: (a) Stone column treated soil, Su=32KPa, L/D=4 (b) Stone column treated soil, Su=32KPa, L/D=7

Chapter 4

RESULTS AND DISCUSSIONS

4.1 DETERMINATION OF THE ENGINEERING PROPERTIES OF SOIL

Laboratory tests were conducted according to the standards set by ASTM to determine the engineering properties of soil and other material used in experimental program.

4.1.1 Classification of Soil

Hydrometer analysis was conducted as per ASTM standard 422-63. The clay size fraction is greater than 60% (i.e. < 0.005mm) while remaining 40% is silt size fraction. The Gradation curve is shown in Figure 4.1.

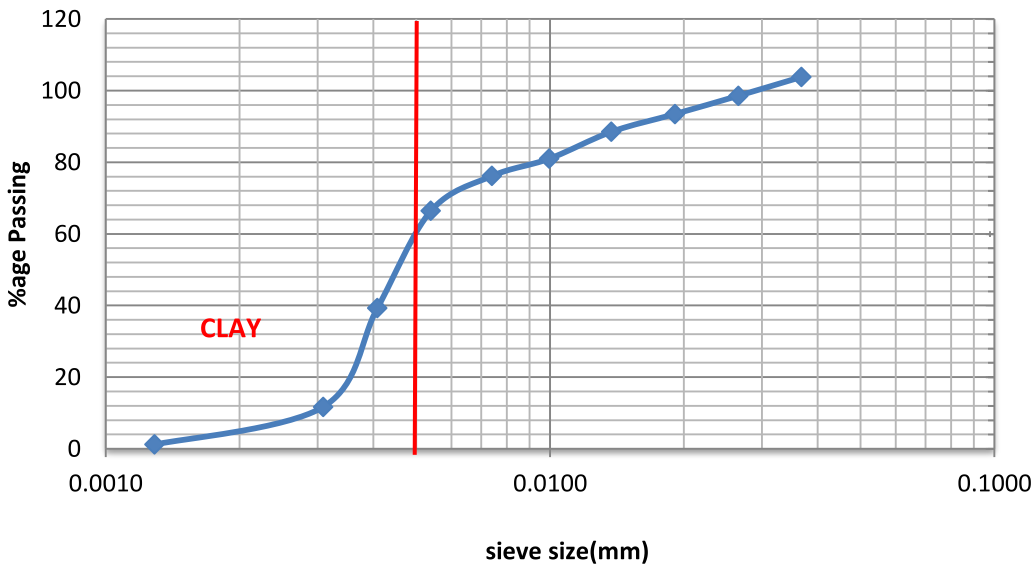


Figure 4. 1: Gradation curve of soil sample

The classification of soil is done after calculating Atterberg limits as well as the gradation curve. Liquid limit and plastic limit values were found out by conducting different trials and taking average values to minimize the percentage of error. Figure 4.2 presents the liquid limit curve. Soil sample has been classified using both Unified Soil Classification System (USCS) as well AASHTO system. The soil is classified as CH and A-7-6(26) in terms of USCS and AASHTO classification system. Figure 4.3 and Figure 4.4 shows classification of Soil based on USCS classification and AASHTO classification. The values used for classification of soil are given in table 4.1a and 4.1b.

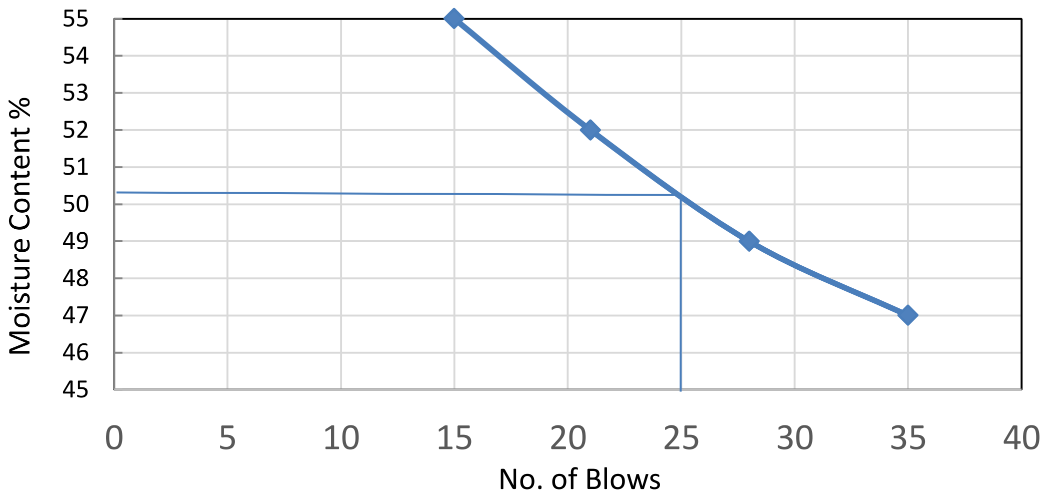


Figure 4. 2: Liquid Limit Test

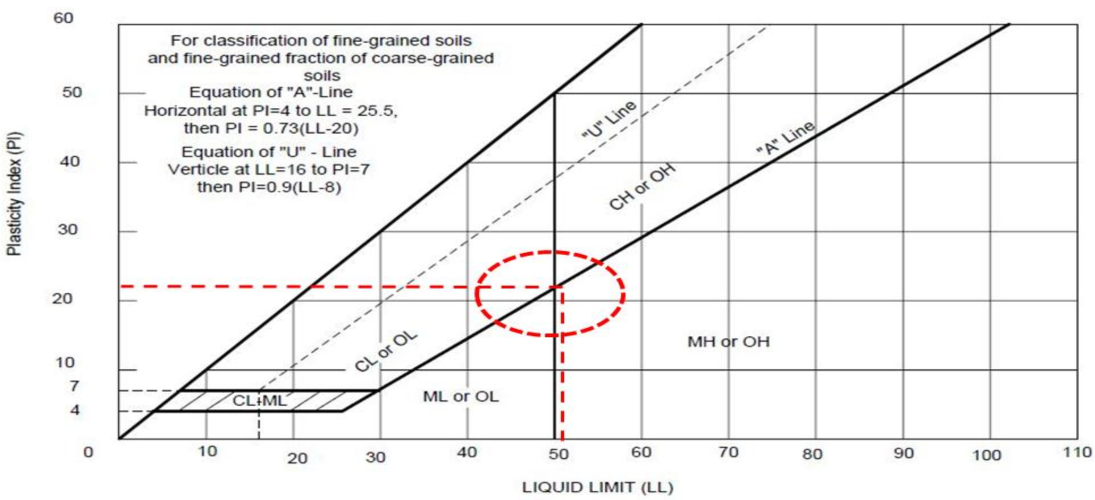


Figure 4. 3: USCS classification A-Line chart

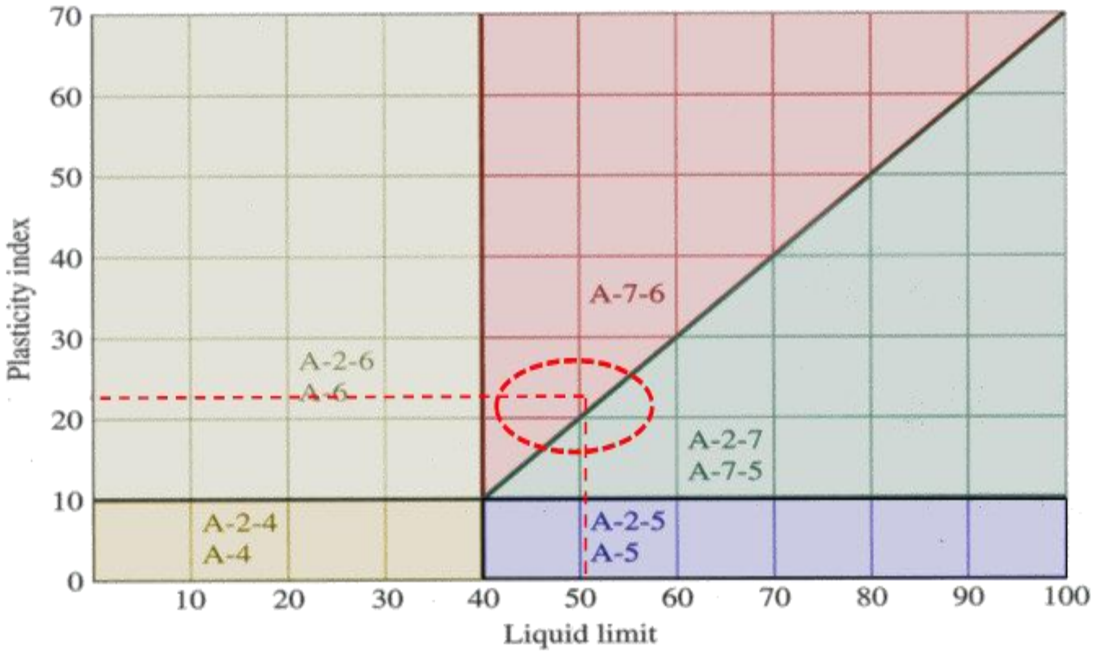


Figure 4. 4: AASHTO classification of soil sample

Table 4. 1: (a) Unified Soil Classification System values (b) AASHTO classification values

(a)

USCS SOIL CLASSIFICATION	
P200	100%
Liquid limit	50.4
Plastic limit	28
Shrinkage limit	26.43
Plasticity index	22.4
USCS	CH
Designation	Fat clay

(b)

AASHTO SOIL CLASSIFICATION	
P200	100%
Liquid limit	50.4
Plastic limit	28
Shrinkage limit	26.43
Plasticity index	22.4
AASHTO	A-7-6(26)
Designation	Clayey soil

Clay size limit is taken at 0.005 mm and less as specified by ASTM. In 1938 the USDA Bureau of soils changed the 0.005 mm size limit to 0.002 mm however engineers still favor the original 0.005mm limit.

4.1.2 Soil Compaction Test

Soil compaction tests were conducted both by Standard Proctor method as well Modified Proctor method to determine the soil moisture-density relationship of soil intended for testing. The results of Standard Proctor Compaction test were later on used in testing as Standard Proctor method is best suited for fine soils. The Optimum moisture content is noted here to be equal to 33% and the maximum dry density equals 99 lb/ft^3. The compaction curve was useful in determining dry densities at different moisture contents which were utilized in unconfined compression test for undrained shear strength-Moisture relationship. The moisture-dry density graph is given in Figure 4.5.

4.1.3 Unconfined Compression Strength Test

Unconfined compression test was performed on the soil sample to find out the change in undrained shear strength of soil with varying moisture contents on either dry or wet side of optimum. The changes were recorded and a curve was plotted and based on this curve, shear strength values were selected for preparation of ground for installation of stone columns. The values of undrained shear strength, S_u were recorded to be 54kPa, 32kPa and 14kPa for moisture contents of 31%, 35% and 39%. These values of moisture content were selected on the basis of optimum value as one value lies on dry side of the optimum and two values on the wet side of optimum. The relationship of moisture content-undrained shear strength is shown in figure 4.6.

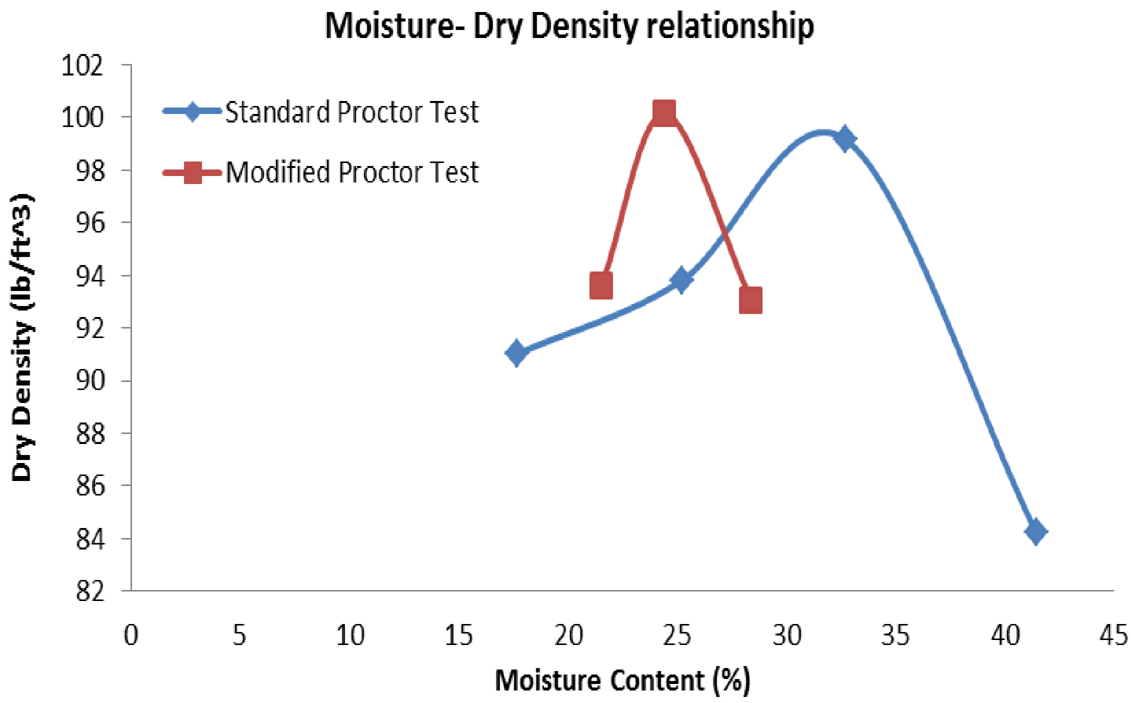


Figure 4. 5: Soil moisture-Dry density relationship

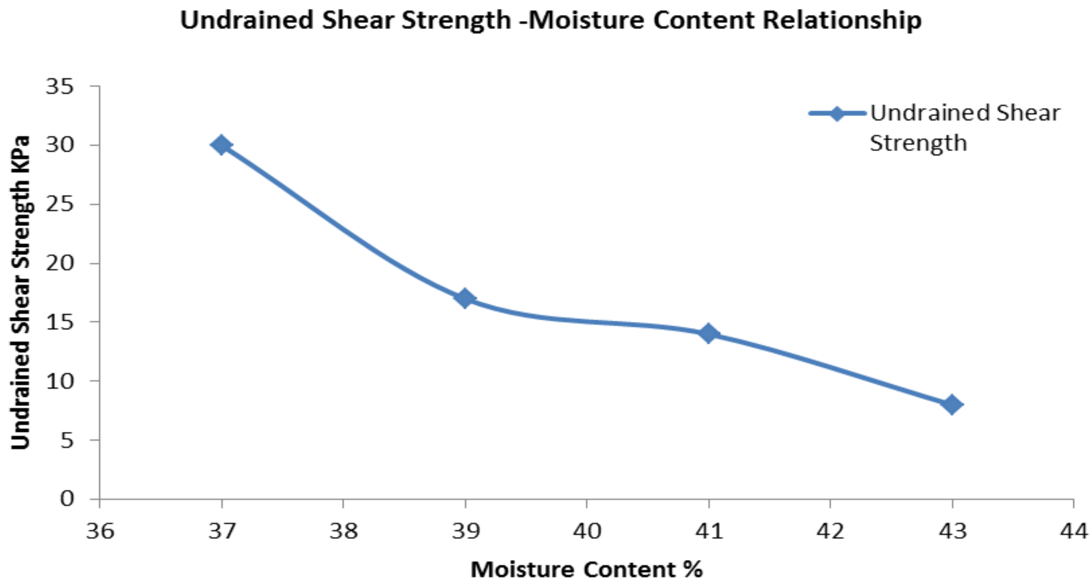


Figure 4. 6: Unconfined compression strength test on soil sample

Table 4.2 shows the engineering properties of the clay soil used for testing after gathering results from the basic laboratory tests conducted.

Table 4. 2: Engineering properties of soil determined from lab testing

Property	Results
Optimum Moisture Content (%)	33
Liquid Limit (%)	50.4
Plastic Limit (%)	28
Plasticity Index	22.4
Shrinkage Limits (%)	26.43
Specific Gravity	2.56
Passing No. 200 Sieve (%)	100
Silt Contents (%)	40
Clay Contents (%)	60
Maximum Dry Density (lb/ft^3)	100
Classification according to USCS	CH

4.2 COMPARISON OF TREATED AND UNTREATED SOIL

4.2.1 Effect of Shear Strength

The aim of the research is to develop small scale models that can be used for laboratory testing to study the effects of treatment of soft soil deposits with floating stone columns and compare the results with untreated soil. These columns are installed in weak soil of varying shear strengths (S_u= 54 kPa, 32 kPa, 14 kPa) to understand the type of soil most responsive to treatment by stone columns.

Crushed marble stones ($CaCO_3$) were collected from Rashakai, District Mardan, KPK. The stones were washed and passed through 10mm sieve and retained on 2 mm sieve. The maximum and minimum dry density of the crushed stones is 105.94 lb/ft^3 and 86 lb/ft^3. A light compacting effort was adopted to achieve a density of 95.5 lb/ft^3 as suggested by Ambily and Gandhi [28]. The properties of the column material are given in table 4.3.

Figure 4.7, 4.8 and 4.9 shows the comparison of the stone column treated soil with that of the untreated soil at undrained shear strength of 54 kPa, 32 kPa and 14 kPa respectively. It can be inferred from the results that adequate increase in strength of

Table 4. 3: Properties of crushed stone used as column material

S. No.	**Properties**	**Stone ($CaCO_3$)**
1.	Size limit	10 mm-2 mm
2.	Specific gravity, G_s	2.94
3.	Angle of internal friction, Φs	41°
4.	Maximum dry density $_{\gamma max}$	105.94 lb/ft^3
5.	Minimum dry density $_{\gamma min}$	89 lb/ft^3
6.	Fineness Modulus	-
7.	Water absorption	0.23%

soil is noted when it is reinforced with stone columns. At low moisture content i.e. 31% (Su=54 kPa), load-carrying capacity is increased up to 33% at full prescribed strain of 10%. At maximum value of load sustained by the untreated ground i.e. 27 kN, the settlement of the soil is reduced by 43% when improved with stone column. Similar behavior is noticed while treating soil of Su= 32 kPa with stone columns. The percentage increment in strength is 30% of the untreated ground at full strain of 10%. While, the percentage reduction in settlement in treated soil is 39.3%. The values are calculated at maximum value of load sustained by untreated ground.

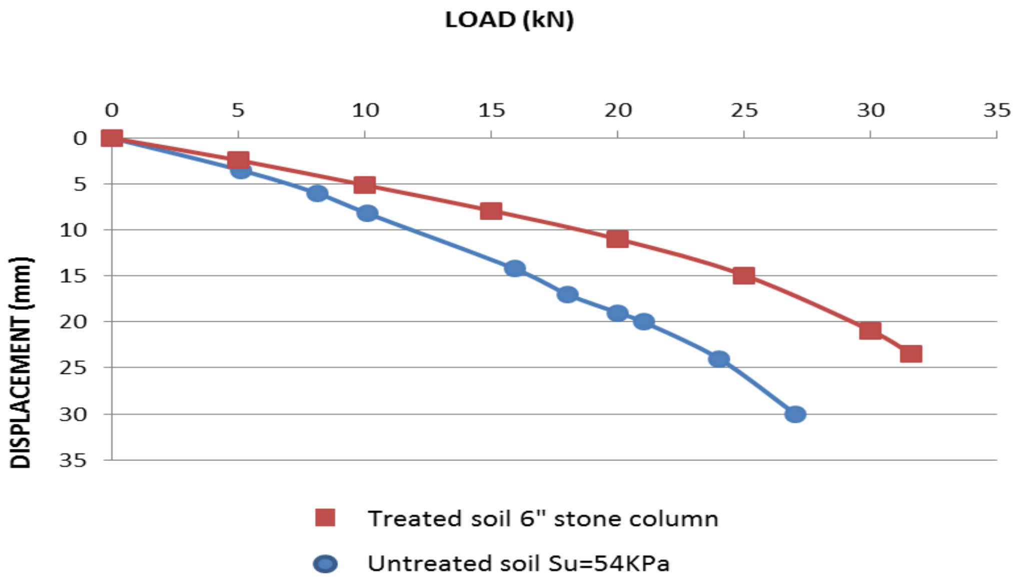


Figure 4. 7: Comparison of untreated soil and soil treated with stone columns at Su=54 kPa

It is worthwhile to note that at low load values the settlement is almost same but as the load increases the settlement undergone by untreated soil is more than that of treated soil. This trend can be seen in Figure 4.8.

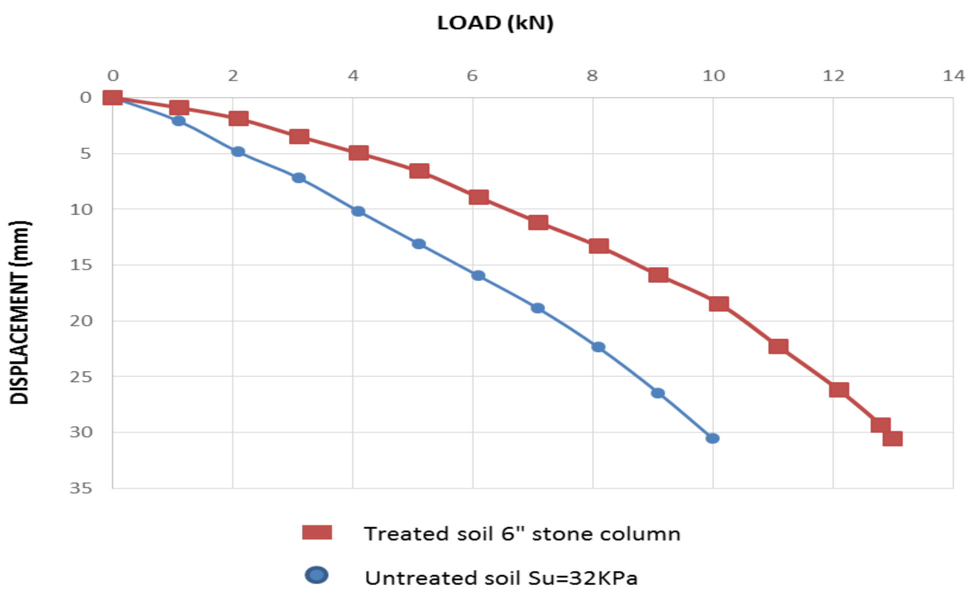


Figure 4. 8: Comparison of untreated and soil treated with stone column at Su=32 kPa

Figure 4.9 shows the comparison of untreated soil at S_u= 14 kPa i.e. 39% moisture content with stone column treated soil. The strength increment is found out to be 9.52% compared to the untreated soil. The settlement percentage at maximum load value of untreated is reduced by 16.67% when treated with stone column.

While this improvement is notable it can be deduced from the relationship that the percentage increment in strength keeps on reducing when we go beyond the optimum moisture content on the wet side. The downward trend in percentage increment is not much when the shear strength is varied from S_u= 54 kPa to S_u=32 kPa. But it is more pronounced when the shear strength is further reduced from S_u=32 kPa to S_u=14 kPa. Based on this observation, a range has been observed for performance of stone columns. It is important to note here that stone columns are effective for soils whose moisture content is varied up to 10% on dry side and 20% on wet side. Beyond this range the stone columns are not effective.

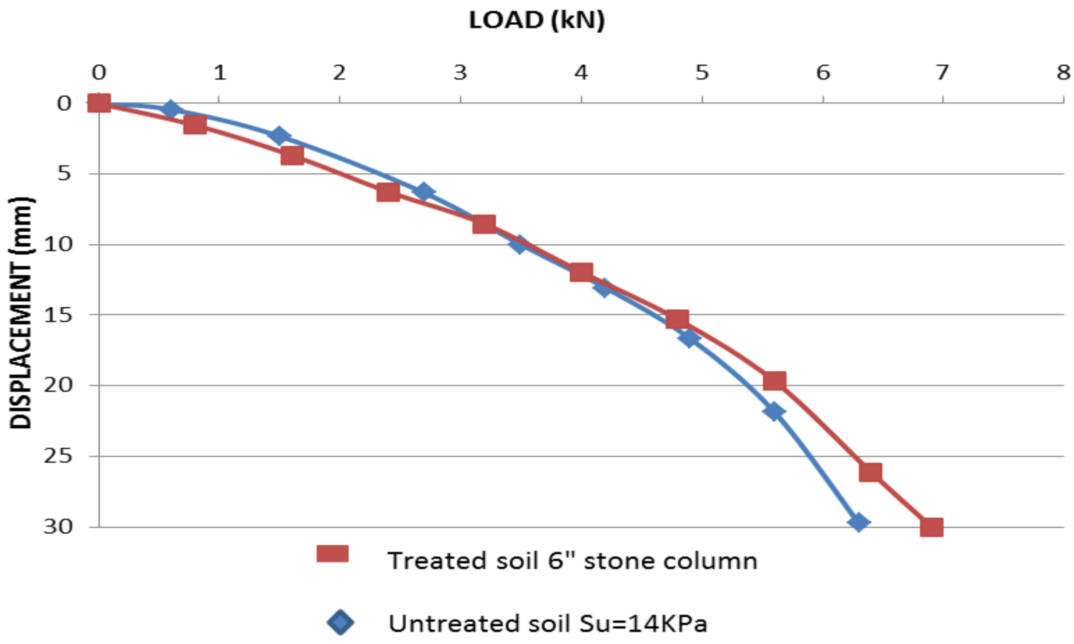


Figure 4. 9: Comparison of untreated soil and stone column treated soil at Su= 14 kPa

4.2.2 Effect of Length to Diameter (L/D) Ratio

Figure 4.10 illustrates the relationship between loading intensity and the corresponding settlement of stone column treated ground at different length to diameter ratios i.e. L/D= 4, 5.5 and 7. The columns were installed to a depth of 6”, 8” and 10” respectively. It is illustrated here that as the slenderness ratio of column increases the load carrying capacity decreases. The load carrying capacity is higher at L/D=4 (the column is short) as compared to L/D=7. The load carrying capacity depends upon the bulging of column. The surrounding soil provides the confining effect, when the column bulges. Short columns bulge less as compared to long columns and hence they can sustain greater loads.

Research shows that column length beyond critical limit does not show any increase in load carrying capacity due to failure by bulging in the top region [1]. The results obtained during lab testing have been compared to results by previous researchers and close agreement can be found as L/D ratio of 4.1, 4.5 and 4 to 5 can sustain maximum load ([1], [7], [4].

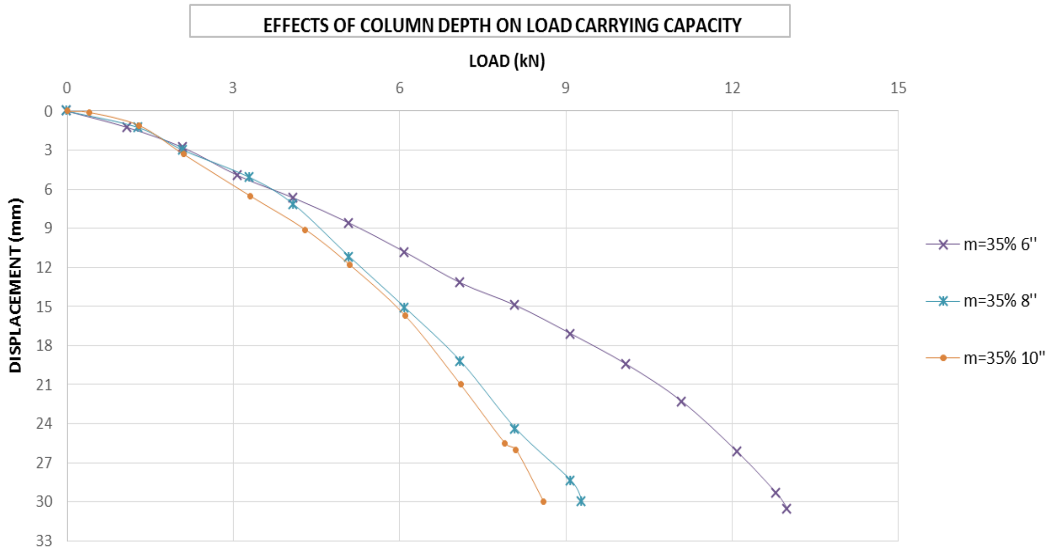


Figure 4. 10: Effect of L/D on load carrying capacity of stone columns

4.2.3 Group Efficiency of Stone Columns

Tests were carried out on group of stone columns to study the group efficiency of stone columns. The columns were installed at spacing of 2 to 3 times their diameter. The L/D ratio was selected 5.5 and soil was prepared at undrained shear strength of S_u= 32 kPa. The triangular arrangement was selected for installation of columns and spacing was adjusted with help of scale. Figure 4.11 shows the comparison of group column spacing effects. The group effect enhances the load sustainability. As can be seen in figure 4.11 when spacing is increased from S/D=2 to S/D=3, the load carrying capacity is decreased by 43%. The group effect is negligible when columns are spaced far apart. Average stress is calculated by method presented by [8] for calculation of average stress for unit cell at any depth i.e.

$$\sigma_{avg} = \sigma_{col} * R_a + \sigma_{soil}(1-R_a) \qquad \text{(Eq. 4.1)}$$

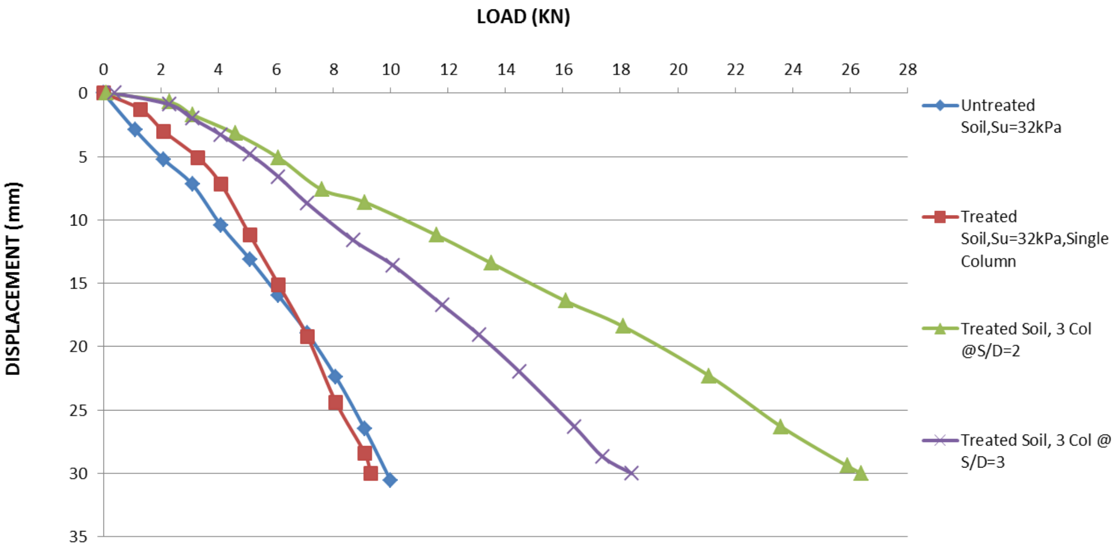


Figure 4. 11: Group effect of stone columns at S/D=2 and S/D=3 (L/D=5.5)

It is found out that average stress in single stone column at Su=32 kPa and L/D=5.5 is 1861.05 kPa while average stress on single column in group of 3 columns at S/D=2 is equal to 1733.7 kPa. It can be inferred that the values of average stress on stone columns are almost same due to the fact that the single column depicts the results of column situated in the middle of multiple columns in field. Had it been near the periphery the situation would have been different.

4.2.4 Failure of Stone Columns

Figure 4.12 (a) and (b) shows section of column after being subjected to loading. The failure mode of stone columns was analyzed by carefully removing the soil from the mold. The soil was brushed off from the column an observation was made by calculating the width of column section at different depths. Bulging failure was seen in most of the columns at top most layers. As studied in previous research by [1], bulging was observed at depth of 1.5D-2D. This depth varied with varying shear strength. The bulging failure mode was not observed when the whole soil area was loaded along with the stone column. However it could be easily seen in those tests where only stone columns were loaded. The stone columns were loaded with a small specially fabricated steel plate. The Undrained shear strength was varied as well as the L/D ratio of stone columns. Table 4.4 shows the values of maximum load and stress developed in stone columns.

Figure 4. 12: (a) and (b) Bulging of stone column in topmost layer

Table 4. 4: Column alone loaded at varying shear strength and L/D

Column Loaded					
Limiting Stress (KPa)					
	L/D	**Moisture Contents**	**Shear Strength**	**Experiment**	
		%	KPa	Load (KN)	Stress (KPa)
Single Column	4	31	55	3.6	3349.88
	4	35	32	1.6	1488.84
	4	39	17	-	
	5.5	-	-	-	
	5.5	35	32	2.0	1861.05
	5.5	-	-	-	
	7	-	-	-	
	7	35	32	1.6	1488.84
	7	-	-	-	

4.3 NUMERICAL MODELING BY PLAXIS 2D

4.3.1 Comparison of FEM results with experimental values at varying shear strength

Numerical modeling was carried out using PLAXIS 2D software for validating the experimental results carried out in laboratory. The soil was modeled with the same parameters that were used in laboratory. Soil medium was defined and lateral movement restricted along sides of mold only allowing vertical movement. At the bottom both horizontal and vertical restraints were added. The model was run at the designated undrained shear strengths to calculate the load-displacement behavior of untreated soil sample. Figure 4.13 shows the comparison of the load-displacement behavior of untreated soil sample obtained both through laboratory testing and PLAXIS 2D software. The results vary slightly due to highly controlled conditions for PLAXIS calculation as compared to experimental results.

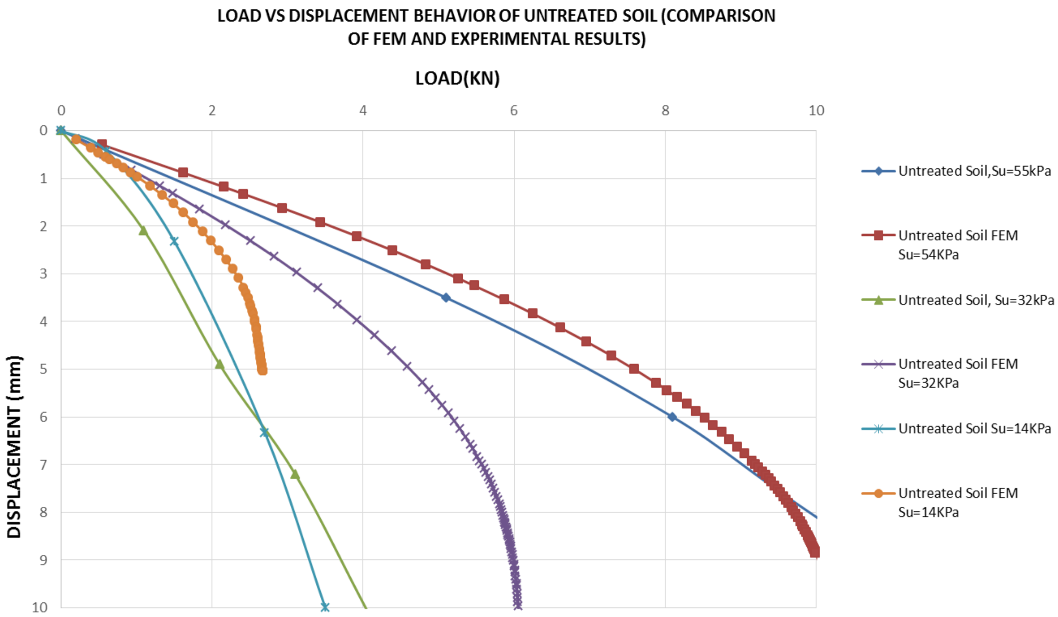


Figure 4. 12: Comparison of FEM vs Laboratory results for untreated soil sample at varying Undrained Shear Strength

4.3.2 Comparison of Treated and Untreated Soil at Varying Shear Strength using FEM

Stone columns were installed in soil models by assigning properties of crushed stone to the model. The results were found in close agreement to the experimental results as the percentage increase in load carrying capacity is nearly the same. Figure 4.14 shows comparison of untreated soil and treated soil by varying undrained shear strength from 54 kPa to 14 kPa. It is found from figure 4.14 that at shear strength Su=54 kPa the increase in strength is up to 21.4% at settlement value of 6mm beyond which sheer drop in load carrying capacity is observed. Similarly at Su=32 kPa, 15.4% increase in load carrying capacity is noted at 6 mm settlement value beyond which the load carrying capacity suddenly drops. As was observed in laboratory testing the effect of treatment was not much at Su= 14 kPa, same is observed with FEM the effect of treatment with stone columns cannot be observed. The comparison of experimental results and scaled graph is FEM result is made in figure 4.15. Both results can be correlated as similar trend is observed at till settlement values of 10 mm 8 mm and 6 mm for Su=54 kPa, 32 kPa and 14 kPa respectively. A sheer drop in load carrying capacity as determined through FEM after these values while no such drop is observed in experimental testing.

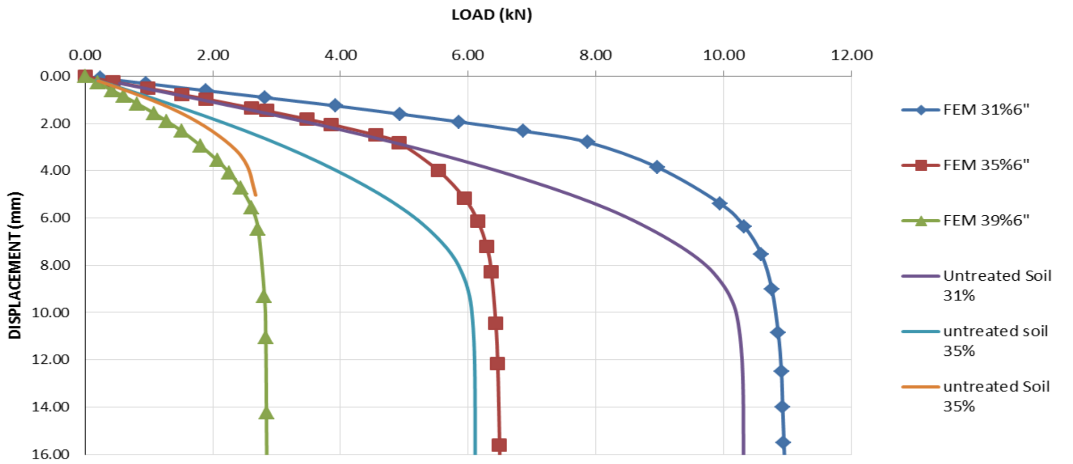


Figure 4. 13: Comparison of untreated and treated soil by PLAXIS 2D

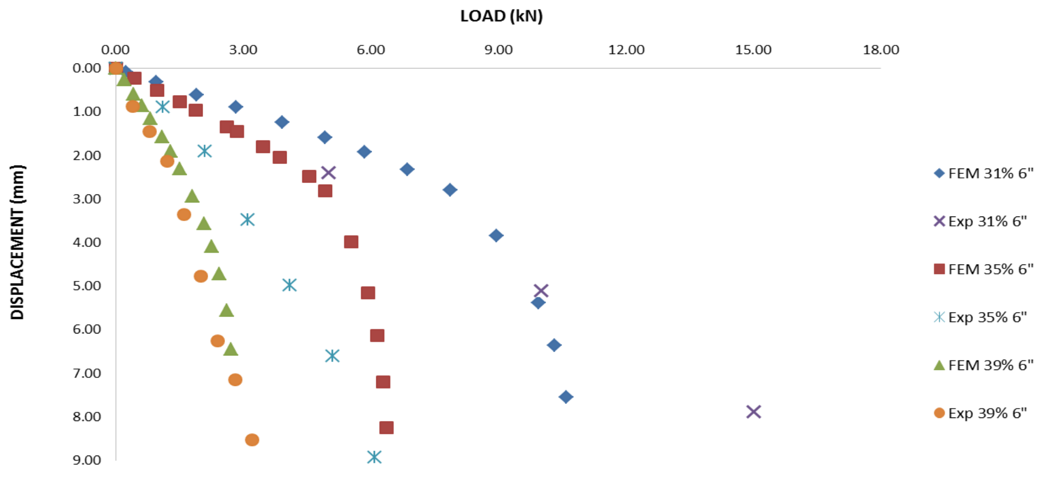


Figure 4. 14: Treated soil experimental results Vs FEM results (Scaled graph)

4.3.3 Effect of L/D of Stone Column on Load Carrying Capacity of Treated Soil

Length to diameter ratio of stone columns was increased to study their effects on load carrying capacity of improved ground. The experimental results and the FEM results were in close agreement to each other as the load carrying capacity was found maximum at L/D 4 to 5.5 beyond which it reduced again as the slenderness ratio was increased. The comparison of increasing L/D of stone column is illustrated in figure 4.16.

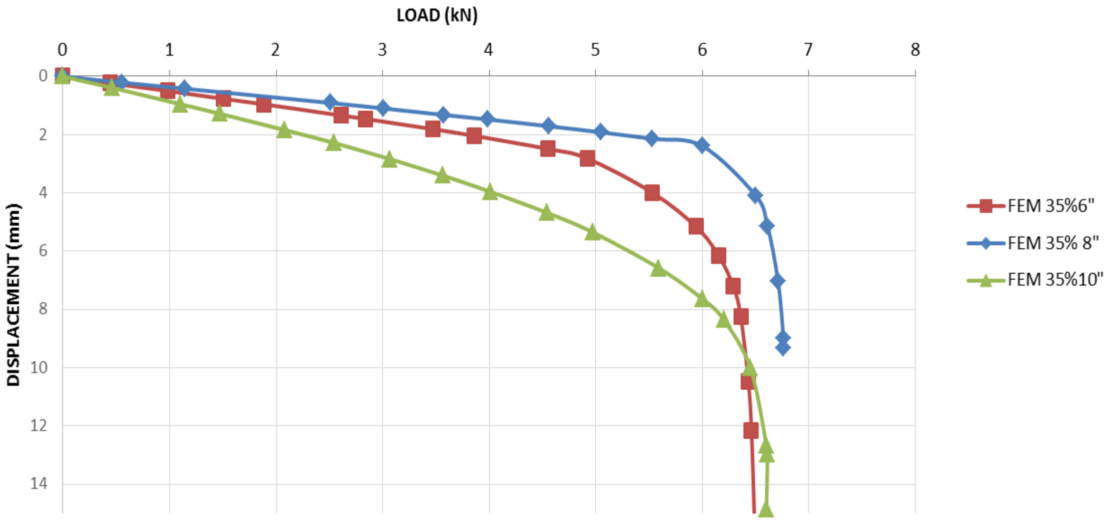


Figure 4. 15: Effect of varying L/D ratio of stone column on load carrying capacity of treated soil

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

The contents of this research can be summarized as;

1. Design parameters like shear strength of soil, length of stone column, spacing of stone column, slenderness ratio, etc. are reviewed from literature review and its effects on load carrying capacity is evaluated.

2. In order to correctly calculate the relevant parameters of the soil under investigation, a number of laboratory tests are carried out and correlated for optimum results.

3. Load test carried out on single stone column to observe the load-displacement response of single column under vertical loading by varying the parameters evaluated from research.

4. Load test carried out on group of 3 stone columns arranged in triangular pattern to observe the load-displacement behavior by varying the spacing between columns.

5. Comparison between single stone column and group of stone columns is made to analyze the group efficiency.

6. Numerical models are in PLAXIS 2D to validate the testing program carried out in laboratory. The laboratory tests are simulated by FEM method.

5.2 CONCLUSIONS

Based on this research following conclusions can be made after carrying out rigorous laboratory tests and also running numerical simulations by using FEM software PLAXIS 2D;

1. Soil of varying shear strength is improved by reinforcing the soil with stone columns. The improvement is dependent on the undrained shear strength of soil and at low shear strengths no such improvement can be observed. When the undrained shear strength is increased from Su=14 kPa to Su=32 kPa, the load carrying capacity increases from 10% to 30% but it only increases from 30% to 33% when Su is increased from 32 kPa to 54 kPa. Hence the effective range for stone columns can be worked out as Su= 10 kPa-50 kPa.

2. Settlement of the treated soil is found to be less than the untreated soil at same loads. A comparison was made at maximum loads carried by the untreated soil and a similar observation can be made as the settlement is reduced by 43% at maximum load sustained at Su=54 kPa. The settlement is reduced by 39% when Su is decreased from 54 kPa to 32 kPa. However there is a marginal drop in settlement reduction i.e. it becomes 16.67% when Su is reduced from 32 kPa to 14 kPa.

3. The effect of L/D ratio of stone columns on their load carrying capacity is studied by increasing the length of the stone columns from 6” to 8” and then 10”. The load carrying capacity is increased up to a critical limit of L/D 4-5.5 beyond which it decreases. This critical limit has been ascertained by both experimental and FEM results.

4. The group efficiency is observed to be greater at closer spacing while it becomes negligible at S/D>2. The load carrying capacity of group columns also validate that single

stone column at center of mold with same area ratio simulates the behavior of a column at the center of multiple columns.

5. Experimental testing gives relatively over estimated load carrying capacity as compared to FEM results due to many parameters that are hard to control when carrying experimental testing. However the method adopted for carrying out load testing of mold is in close agreement to testing by FEM which validates the results obtained in laboratory testing.

6. Visual observation of column section after loading was made and failure mode of single stone column is observed to by bulging. The bulging was found out to be more pronounced in the top part of the column from 1.5 to 2D and it became negligible with depth. However, bulging failure was not found in whole area loaded as well as in group columns.

7. Based on visual observation it was inferred that stone columns increased the drainage of water as more moisture was found on the surface after 4 days curing period of the mold.

8. Stiffness improvement factor is nearly constant for different shear strengths.

5.3 RECOMMENDATIONS

1. The laboratory tests provide good substitute for large scale testing as long as the parameters are found in controlled environment. Moisture content should be controlled while preparing clay bed for stone columns otherwise results may differ.

2. The single stone column can be used to simulate the behavior of center columns in multiple columns in field.

3. Group columns were investigated at constant shear strength values and spacing varied. Research needs to be done to find a relationship between improvement factor and spacing of columns by varying shear strength of soil.

4. The molds were tested by stress controlled compression machine. The results should be compared with strain controlled machine to draw out comparison when time duration of loading is changed.

5. The molds were covered for 4 days for moisture to spread out uniformly. However the effects of different curing periods also need to be investigated to develop a relationship between load carrying capacity and settlement at different curing periods.

6. Design parameters of varying shear strength of soil, length of stone columns and spacing of stone columns were investigated in this research. However this research should be extended to find out the effects of varying column diameter, varying area ratio, and varying mold size.

7. This research can be extended to find the effects of encasing stone columns with different materials on load carrying capacity.

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