NUMERICAL MODELLING AND ANALYSIS OF STONE COLUMN BENEATH A RIGID FOOTING AS A GROUND IMPROVEMENT TECHNIQUE FOR POOR SOIL USING FINITE ELEMENT BASED SOFTWARE: IN CASE OF BISHOFTU TOWN

By

Tewodros Eyasu

A Thesis Submitted to

The department of Civil Engineering,

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

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NUMERICAL MODELLING AND ANALYSIS OF STONE COLUMN BENEATH A RIGID FOOTING AS A GROUND IMPROVEMENT TECHNIQUE FOR WEAK SOIL USING FINITE ELEMENT BASED SOFTWARE: IN CASE OF BISHOFTU TOWN

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Approval of Board of Examiners

We, the undersigned, members of the Board of Examiners of the final open defense by Tewodros Eyasu Zima have read and evaluated his thesis entitled "Numerical Modeling and Analysis of Stone Column beneath a Rigid Footing as a Ground Improvement Technique for Poor Soil using Finite Element based Software: in case of Bishoftu town." and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirement of the Degree of Masters in Civil Engineering (specialization in Geotechnical Engineering).

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Declaration

I hereby declare that this MSc Thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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LIST OF ACRONYMS AND ABBREVATONS

A	Tributary area of soil per column in a large grid
A_C	Area of the stone column
A/A_C	Area ratio
B	Width of square footing
C_c	Compression indices
C_s	Swelling indices
E_{50}^{ref}	Secant stiffness in standard drained triaxial test
E_{oed}^{ref}	Tangent stiffness for primary oedometer loading
E_{ur}^{ref}	Unloading / reloading stiffness
F_d	Applied pressure
FEA	Finite Element Analysis
FEM	Finite Element Method
K_0	Coefficient of lateral earth pressure at rest
L_c	Column length
R_f	Failure ratio (Hardening Soil model)
R_{inter}	Strength reduction factor (Hardening Soil model)
S_e	Settlement
Uy	Vertical displacement
a	Column width
c	Cohesion
c_u	Undrained shear strength
d_c	Column diameter
e_o	Initial voids ratio
k_h	Horizontal permeability
k_v	Coefficients of vertical permeability
m	Power for stress dependency (Hardening Soil model)
n	Stress concentration ratio
p^{ref}	Reference pressure (Hardening Soil model)
s	Column spacing

t_f	Footing thickness
α_r	Area ratio
β	Settlement reduction factor
γ	Bulk unit weight
γ_{sat}	Saturated unit weight
υ	Poisson's ratio
ϕ	Angle of internal friction
ψ	Angle of dilatancy

ABSTRACT

Vibro stone columns, installed using the vibro replacement technique, are a cost-effective form of ground improvement for enhancing the bearing capacity and settlement characteristics of various soil types. Over the last ix decades stone column technique has become popular among other ground improvement techniques and it has been applied successfully around the world. However, in Ethiopia stone column which offers a cost-effective alternative to traditional piled solutions when supporting moderately loaded residential, commercial and industrial structures haven't applied yet. This study is intended to investigate and clarify the effectiveness and applicability of stone column as a ground improvement technique and to develop better understanding on the application in the case of Ethiopia. PLAXIS 3D Foundation v1.1 is used for this research in conjunction with the advanced Hardening Soil model, which is adopted for the parent soil and stone column. A set of material parameters and soil profile adopted for this study was collected from outskirt of Bishoftu town around Eastern industry zone and developed by correlations of SPT N values. The influence of key design parameters such as area ratio and column confinement, column length and foundation loading s upon the settlement performance of stone columns was investigated through a total of 123 numerical analyses including preliminary checks to ensure accurate numerical analysis. The modelling has shown that the area ratio and column length have a significant influence on the settlement performance of stone columns. In addition, foundation pressure has dictated that the stone column improvement is efficient at moderately loaded structures. The settlement of treated footing has been reduced to $^{1}/_{3}$ *of untreated settlement at high area of column, while at medium and low area ratios of column the settlement has been decreased to half of original untreated one. The effective combination of column length to area ratio is that of short column with high area ratio and long column with low area ratio. The result from PLAXIS 3D Foundation code was verified with a numerous case studies performed abroad. It was demonstrated that the output obtained from analysis fall with in very much acceptable range, although the column of low area ratio performed outskirt from the extent of practicable case studies. Based on the result high and mean area ratio is best suited to reinforce rigid footing, while the low area ratio was applicable efficiently for wide area loading such as embankments, which can tolerate larger settlement*

Key words: *Ground improvement*, *Stone column, Settlement, PLAXIS 3D Foundation*

CHAPTER ONE
INTRODUCTION

1.1. Background of the Study

In modern earth construction, there is an ever increasing need to utilize poorer soils as the availability of suitable construction sites decreases. In recent years, among the growing methods of weak ground treatment techniques, stone column foundations have been generally recognized as a useful technique to reinforce cohesive soils for structural foundations. The principal concept of stone columns reinforcement involves the replacement of a weak soil with coarse granular material such as crushed stone, or sometimes gravel, in the form of columns. Stone columns have four major improvement effects on soils: increasing the bearing capacity and slope stability of the soils, lowering the total and differential settlements, working as a drainage system to accelerate the consolidation settlement of cohesive soils and reducing the liquefaction potential of loose saturated cohesionless soils (Barksdale and Bachus, 1983). Thus, stone columns are generally applied on soft clays and silts, silty clays and liquefiable loose sands.

Over the last six decades stone column technique has become popular among other ground improvement techniques and it has been applied successfully around the world, mostly in Europe and USA. However, in Ethiopia stone column which offers a cost-effective alternative to traditional soil replacement and piled solutions when supporting moderately loaded residential, commercial and industrial structures haven't applied yet.

This study has developed a better understanding of the application and behavior of stone column beneath a rigid footing reinforcing a poor ground in the case of Bishoftu town. A soil data has been accessed from secondary source of data and the soil profile and material parameters has been correlated and used as an input. The investigation tool utilized was numerical modeling with PLAXIS 3D finite element based software. A series of 3D numerical analysis has been conducted to analysis the settlement performance of stone column. Investigation concerning the key parameters that mainly influence the performance of stone column has been addressed and discussed in depth by a researcher through an intensive literature review based on the analyses result.

1.2. Statement of Problem

As the need for land has increased worldwide, the land has become more valuable and the need to construct on unsuitable soils has become a major issue especially in urban areas. To use and construct on such type of soil, it needs to mitigate the problem. Some of the remedial measures available are replacing unstable soil, applying resistance structure (mat, pile foundation …) and improving the soil. The ground improvement techniques are preferable alternative to traditional piled solutions and replacing the soil. Among the ground improvement techniques stone column is a popular form of ground improvement which is used to enhance the settlement performance of weak soils.

Over the last sixty years stone column technique has become popular among other ground improvement techniques and it has been applied successfully around the world. However, in Ethiopia especially urban areas where the availability of weak soil is high (like Bishoftu, Addis Ababa, Dukem) either the soil replacement or foundation (mat, pile) have been applied mostly to overcome the problem. Stone column which offers a cost-effective alternative to traditional piled solutions when supporting moderately loaded residential, commercial and industrial structures haven't applied yet.

Thus, this necessitated verifying the effectiveness of the stone column over the recent applicable remedial measures (soil replacement and pile or mat foundation) through more sophisticated approaches eg. advanced numerical predictions. This research hence dealt with the numerical modeling and analysis of a stone column beneath a rigid footing as a ground improvement technique using PLAXIS 3D finite element based software for soft soil in the case Bishoftu town has developed the better understanding in the behavior and applicability of stone column to reinforce a weak ground.

1.3. Objectives of the Study

1.3.1 General Objective

The main aim of this thesis is to examine a better understanding on the behavior of stone column reinforced in weak soil, its effectiveness and applicability in Ethiopia, specifically in the case of Bishoftu**.**

1.3.2 Specific Objectives

- To analyze the performance of stone column as a ground improvement technique with respect to settlement reduction
- To determine the influence of key design parameters like area ratio, column length and the number of columns upon the settlement performance of stone columns
- To verify the effectiveness of stone column and to develop better understanding on the application

1.4. Methodology

A lot of tasks where performed to come up with a better finding that answer the research questions and complement the objective of study. A comprehensive literature review in the area of study was conducted to the better understanding of the stone column improved weak soil and intensive review has assessed to establish the factors that mainly affect the design and analysis of stone column. A set of material parameters and soil profile adopted for the study site was developed by correlations of SPT N values of field measurement accessed from Addis Geosystems Plc. and calibrated with published literatures of regarding soil properties.

A series of 3D FEA was proposed to investigate the settlement performance of stone columns in weak soil. PLAXIS 3D Foundation is a FE program which is specially developed for geotechnical engineering and is adopted for this thesis. The stress-strain behaviour of the soil and stone columns is simulated using advanced constitutive models i.e. Hardening Soil model.

1.5. Significance of the Study

It is expected that this work may contribute a lot of importance:

- Develop a better understanding of the behavior of stone columns in weak soils.
- Verify the previous analysis done abroad and enhance the confidence of the designers (esp. Geotechnical Engineers) to apply this preferable alternative in Ethiopia.
- Contribute as reference for future work.

1.6. Scope of the Study

The study mainly focused on the settlement performance of stone column, as the design of foundations on weak soils usually governed by settlement scenarios rather than bearing capacity criteria due to its high compressibility. The analysis is extended to study the influence of key design parameters such as area ratio, column length and foundation pressure. The result from PLAXIS 3D code was verified with a numerous case studies performed abroad.

1.7. Organization of Thesis

The content of this thesis is organized in to five chapters:

Chapter 1 provides general introduction with the research objectives, scope and significance of the study. Chapter 2 outlines a full detailed literature review of the stone column soil improvement technique, including historical development, construction methods and previous research works.

Chapter 3 presents study area description, material parameters soil profile adopted for modeling, preliminary checks for sensitivity analyses and generation of finite element analyses using PLAXIS 3D Foundation code.

In Chapter 4 analysis of results, validation of FEA and discussion is presented. Parametric study regarding the influence of key design parameters is also furnished.

The last chapter, Chapter Five, discusses the conclusions derived from this study and limitations in the analyses and recommendations for future work.

Figure 1.1 illustrates a brief research structure of this thesis.

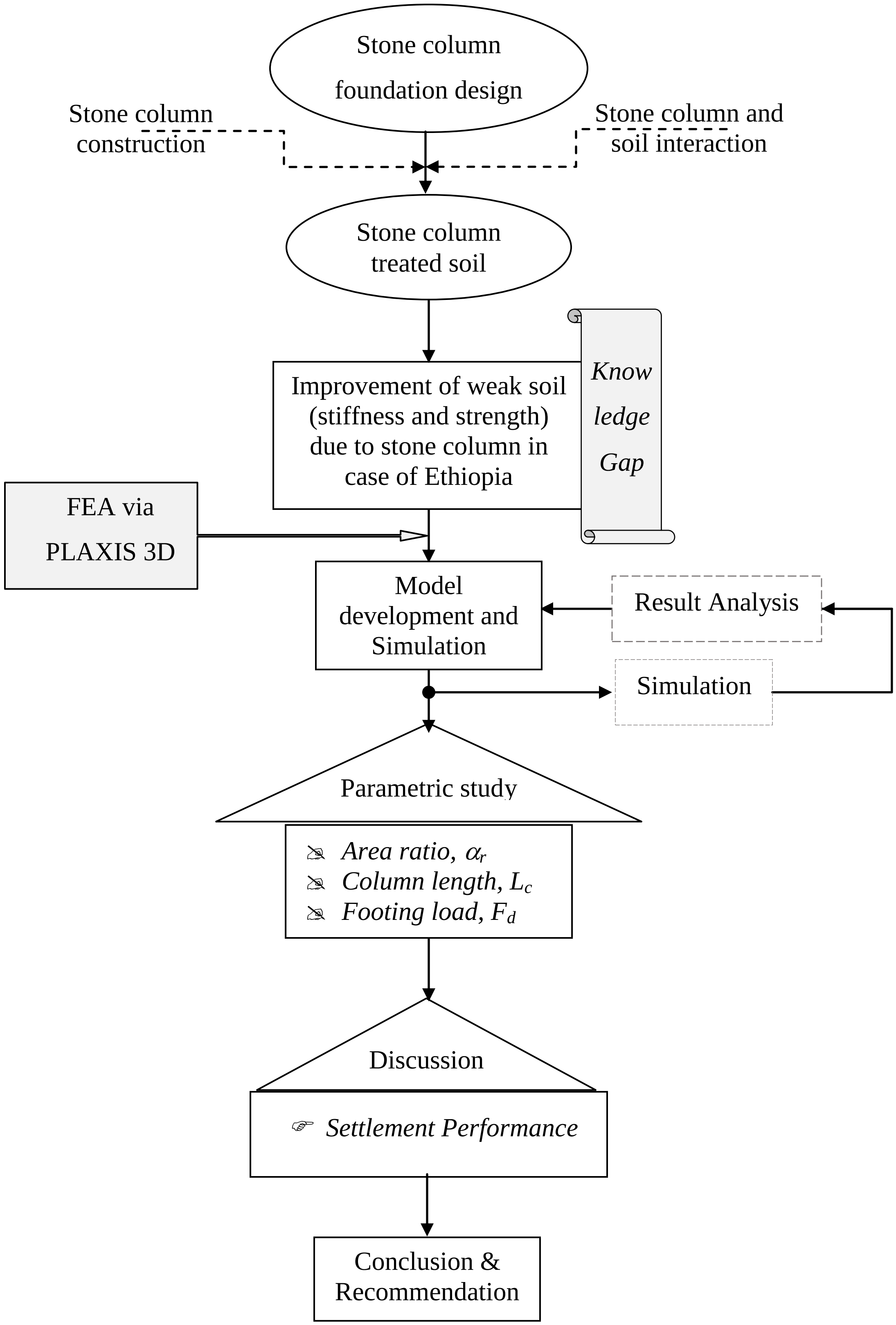


Figure 1. 1- Illustration of research structure

CHAPTER TWO

LITERATURE REVIEW

2.1. General

The inevitable construction over soft and weak soils in urbanized areas has increased the demands to develop new construction techniques that overcome the poor ground conditions of these areas. As a result, a wide range of ground improvement techniques have been developed to be more economical modern alternatives to the traditional methods of construction that are usually based on transferring the loads to bearing strata using deep concrete piles. The main concepts of these ground improvement options usually involve one or more of the following processes; densification, drainage, cementation and reinforcement (McKelvey, 2002). These ground improvement techniques have become of great importance in the last four decades, giving more applications in practice (Simpson and Tatsuoka, 2008).

2.1.1. Ground Improvement Techniques

Numerous ground improvement techniques are known involving mechanical and chemical stabilization, and hydraulic and electrical techniques. The selection of a suitable method is based on a number of factors. For example ground conditions, available material close to the site, and cost and effectiveness of the adopted method. Table 2.1 illustrates the classes of the known ground improvement techniques according to the improvement principle and the place of application.

2.1.2. Vibro-Stone Column

Deep vibratory techniques such as vibro compaction and vibro replacement are forms of ground improvement which are used to enhance the bearing capacity and settlement characteristics of weak soils. The vibro compaction technique was developed in 1936 by the Keller group and one of its first applications was to densify non-cohesive river-borne granular soils (McCabe and McNeill, 2007). The technique consists of lowering a vibrating poker into the ground, which imparts horizontal forces to the surrounding soil and causes the soil particles to re-arrange into a denser state. However, it was found that vibro compaction reaches its technical and economic limits in saturated sands with high silt contents, as fine particles attenuate the horizontal forces imparted by the vibrating poker (Sondermann and Wehr, 2004).

Table 2. 1: Classification of Ground Improvement Techniques

Classes of GIT	**Improvement Method**	**Place of Application**				**Principle of Technique**		
		Soil Mass	Soil Surface	Soft Pile	Tensioned Nail	Ground Reinforcement	Ground Improvement	Ground Treatment
Mechanical Techniques	Stone Columns			✓		✓		
	Deep Dynamic Compaction	✓					✓	
	Vibro-Concrete Columns			✓		✓		
	Surface Compaction		✓				✓	
	Compaction Grouting			✓			✓	
Chemical Techniques	Surface Mixing		✓					✓
	Deep Soil Mixing			✓		✓		
	Lime Columns			✓		✓		
	Jet Grouting	✓					✓	
	Permeation Grouting	✓					✓	
	Blasting	✓					✓	
Tension Techniques	Soil Nails				✓	✓		
	Geosynthetics		✓	✓		✓		
	Ground Anchors				✓	✓		
	Deep Soil Nailing				✓	✓		
Electrical Techniques	Electro-osmosis		✓		✓		✓	
Hydraulic Techniques	Preloading	✓					✓	
	Drainage/Surcharge		✓	✓			✓	
	Vertical Drains			✓		✓		
	Dewatering	✓						✓
Heating/Freezing Techniques	Freezing	✓						✓
	Heating	✓						✓

The limitation of vibro compaction in cohesive soils (i.e. soils composed of a high proportion of fine particles) was overcome in 1956 by the development of the vibro replacement technique. As with the vibro compaction technique, a vibrating poker penetrates the soil to form a borehole. The resulting borehole is then backfilled in stages with coarse aggregate, which is compacted by re-lowering the poker. This process results in stone columns which are tightly inter-locked with the surrounding soil. Figure 2.1 illustrates how the vibro replacement technique extends the range of soils types treatable by deep vibratory techniques.

Stone columns are one of the most widely used ground improvement techniques in soft clays, silts and loose silty sands for the last 60 years in Europe and 40 years in the U.S.A. The technique is also used as a remedy to liquefaction in loose liquefiable cohesionless soils. Stone columns are very effective to increase the bearing capacity and slope stability of embankments and slopes, decrease the total and differential settlements, increase the time rate of consolidation settlement by working as a drainage system in cohesive soils and reducing the liquefaction potential of liquefiable cohesionless soils (Barksdale and Bachus, 1983).

Alonso and Jimenez (2012)clearly explained the mechanism of these improvements as: bearing capacity due to shear strength increase, settlement reduction due to stiffness improvement, acceleration of the consolidation of cohesive soils and reduction of the susceptibility to liquefaction of cohesionless soils due to increase in the soil mass permeability.

Stone columns are type of rigid inclusions driven into the weak soil layer and they also change the properties of the native soil. The improvement degree in the bearing capacity of surrounding cohesive soil by those granular inclusions depends on the confinement of the soil around the column and the diameter and the compaction degree of the column (Greenwood, 1970). As the column is stiffer than the surrounding weak soil, when the load is applied on a footing, the load concentrates on the column and this concentration causes lateral expansion to the surrounding soil. Passive pressures resist the lateral expansion of the columns. This situation results in the column behaving as if it were in a triaxial chamber.

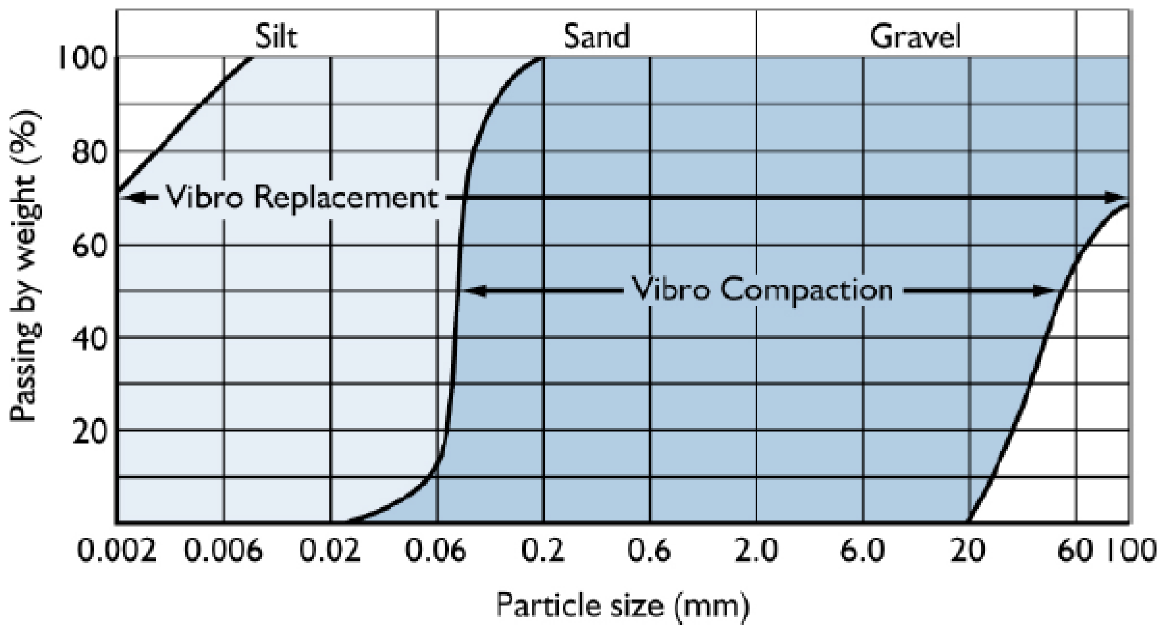


Figure 2. 1: Range of soil types treatable by vibro compaction and vibro replacement techniques (courtesy of Keller Ground Engineering)

2.1.3. Construction Process

Poker

The vibrating pokers used for the dry and wet top feed systems are depth vibrators suspended from a crane. The vibrating pokers are 300–500 mm in diameter and range in length from 2–5m. This length can be increased with extension tubes to reach depths of up to 26 m (Sondermann and Wehr, 2004). The poker consists of a cylindrical steel shell and the horizontal vibratory forces are generated from an eccentric weight located in the lower section. An elastic coupling is used at the top of the vibrator to prevent any vibratory forces being transferred to the extension tubes. Supply tubes for compressed air and water, which are used to aid the poker penetration, are also incorporated into the vibrator shell.

Dry top feed system

The poker is inserted into the ground which displaces the soil laterally. Compressed air is delivered to the cavity as the poker is withdrawn from the ground in order to prevent the cavity collapsing. A controlled volume of crushed stone is then tipped into the cavity from the ground surface and subsequently compacted by re-lowering the poker. This process is repeated until a stone column, which is tightly interlocked with the surrounding soil, is formed up to the ground surface. This construction method is limited to competent soils that can sustain an open borehole.

Wet top feed system

The poker penetration for the wet top feed system is aided by water jets at the poker tip. The jets support the cavity wall and dislodge any loose material during penetration. The flushed out material is carried upwards through the annulus surrounding the poker and treated in settlement lagoons. Field experience indicates that penetration is more effective when a larger volume of water is used, rather than a higher pressure (Sondermann and Wehr, 2004). Once the design depth is reached, the poker is raised slightly and crushed stone is added to the cavity through the annulus surrounding the poker. The poker is then re-lowered to compact the stone and this process is repeated until a stone column is formed up to the ground surface. This system is not commonly used due to the onerous task of treating the flushed out material and also as the water jets create a smear zone surrounding the cavity which reduces the drainage capacity of stone columns.

Dry bottom feed system

A specially developed machine called a 'vibrocat' is used for the dry bottom feed system. The vibrocat is different from traditional crane suspended vibrators as the vibrator is mounted on leaders, thus ensuring vertical columns. The vibrator can also generate pull down forces using a hydraulic winch to aid poker penetration. The supply tube for the crushed stone is attached to the vibrator and bends inwards at the tip. Therefore columns are formed without removing the poker, which allows the poker to case the borehole thereby preventing collapse. Once at the design depth the poker is raised by 100–500 mm, depending on soil conditions, and crushed stone flows into the cavity through the supply tube. The poker is then raised and lowered a number of times until satisfactory compaction of the stone backfill is achieved. This cycle is repeated until a column of compact stone is formed up to the ground surface.

2.1.4. Application of Stone Column

There are numerous benefits of using stone columns as an improvement technique for soft fine soils, including:

1. Reduction of foundation settlement,

2. Improvement of the bearing capacity of the soil,
3. Acceleration of the consolidation process,
4. Reduction of the risk of liquefaction due to seismic activity.

Settlement Control

Vibro replacement primarily enhances the settlement performance of treated soils due to the high stiffness of granular columns. If equal strain (i.e. uniform deformation) is assumed beneath the base of footings, it can be shown from equilibrium that a larger proportion of the applied load is taken by the stiffer stone columns (Figure 2.2). Therefore, stress concentration ratios develop in stone columns. Field experience indicates that stress concentration ratios typically range from 2.5–5.0 (Barksdale and Bachus, 1983). The stress concentrations which occur in columns reduce the vertical stress on the surrounding soil which, consequently, reduces the settlement.

A comprehensive review of the last twenty case studies by McCabe *et al.* (2009) highlights the effectiveness of vibro stone columns in enhancing the settlement performance of soft soils. In addition to reducing the magnitude of settlement, vibro stone columns also homogenise treated soil deposits and thus reduce differential settlement.

Vibro stone columns also act as vertical drains due to the high permeability of the stone backfill. This allows excess pore pressure to dissipate rapidly which decreases consolidation time. However, the drainage capacity of stone columns may be reduced slightly by the creation of a smear zone around the vibrating poker upon penetration.

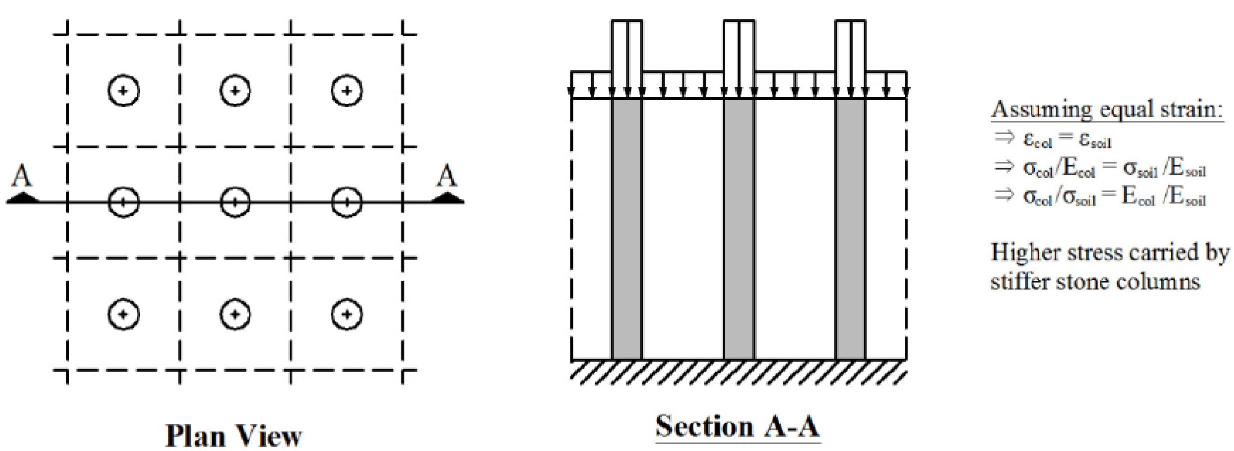


Figure 2. 2: Development of stress concentration ratios in stiffer stone columns

Bearing capacity and slope stability

The vibro replacement technique can be used to increase the stability of embankment side slopes if stone columns are installed deeper than the rotational failure surface. Stone columns have higher shear strength than the surrounding soil and therefore increase the shearing resistance along the failure surface. Moreover, stress concentrations ensure that a larger proportion of the applied load is carried by the stronger column material. The effectiveness of stone columns at increasing the stability of two full scale test embankments, founded in deep deposits of very soft cohesive soil, was investigated by (Munfakh *et al.*, 1984). The authors report that in addition to reducing the settlement and consolidation time, stone columns increased the load-carrying capacity of *in situ* soil by approximately 50%.

2.1.5. Limitations of Vibro Stone Column

Barksdale & Bachus (1983) stated that the vibro replacement technique is most effective in poor ground i.e. very soft to soft compressible clays/silts and in loose silty sands. However, vibro stone columns are highly dependent on the lateral support provided by the surrounding soil and consequently are not suitable for use in very weak soils. If the strength of the surrounding soil is insufficient, excessive column deformation will occur. Experience indicates that stone columns should not be installed in soils which contain layers of peat or decomposable organics greater than 1d–2d (d = column diameter) in thickness (Barksdale and Bachus, 1983). McCabe *et al.* (2009) state that considerable evidence exists to suggest that stone columns can be formed successfully in soils with undrained shear strengths (cu) well above 15–20 kPa, which is often quoted as the lower practical limit of the system. A small scale laboratory study conducted by Wehr (2006a) indicates that columns can be installed in soil with cu ≥ 4kPa.

The vibro replacement technique is also not appropriate in high sensitivity soils. The shear strength of high sensitivity soils dramatically reduces once the grain structure is disturbed. Therefore, the vibratory forces imparted from the poker during column installation may remould the soil and, thus, significantly reduce the soil strength.

2.1.6. Design Specification

Barksdale and Bachus (1983) summarized the design aspects of stone columns as the following:

i. The design load of stone columns is usually between 15 – 60 tons for each column.

ii. The effective treated soil layer thickness is between 6 to 9 meters for economic reasons. However, they also added that in Europe and America there are applications of stone columns as long as 21 meters.

iii. In cohesive soils of shear strength less than 7 kPa the design of stone columns is not recommended while from 7 to 19 kPa shear strength values are to be treated carefully. In soils having sensitivity greater than 5, the stone column construction is also not recommended.

iv. The soils including peat layers thicker than 1 meter are reported not to be suitable for the conventional stone column construction.

v. Stress concentration factor is generally used as 2 to 2.5 and the internal friction angle of the column material 38 to 45^{o} in theoretical analyses.

2.2. Performance of Stone Column

Weak soils, which have poor characteristics in terms of stiffness, strength and drainage, can be treated by vibro stone columns. As a result of these poor characteristics, foundations on such type of soils are subjected to large settlement even under relatively low loads. So, settlement is considered as the main governing criterion in the design of these stone columns foundations. Significant cost impact of the time of consolidation can be noticed in soft soils, stone column soil improvement technique can minimize the time of the consolidation. Consequently, this strengthens the soil more quickly and, therefore, less time would be required to complete construction projects such as embankments.

Before reviewing the literature of the research in this subject, it is important to define some concepts.

2.2.1. Basic Concepts of stone Column

(i) Stone Column Patterns

Balaam & Booker (1981) stated that mostly there are three types of arrangement for stone columns:

a) Triangular arrangement
b) Square arrangement
c) Hexagonal arrangement

a) Triangular arrangement b) Square arrangement c) Hexagonal arrangement

Figure 2. 3: Stone column patterns (after Mitchell, 1981).

(ii) Unit cell concept

Large arrays of columns are typically spaced on square or triangular grids to support wide loaded areas. Figure 2.4 illustrates that the shape of the zone of influence depends upon the column arrangement adopted. It follows from symmetry that the behavior of each stone column and its surrounding zone of influence within a large array will be identical. It also follows from symmetry that the sides of the zones of influence are free of radial displacements and shear stresses. Therefore the analysis may be simplified to one column and its surrounding zone of influence. The unit cell concept approximates the zone of influence by a circle of equivalent area. This concept applies to interior columns beneath wide loaded areas, the proportion of which increases with group size (e.g. embankments, large floor slabs). The unit cell concept is not valid for edge columns beneath wide loaded areas or small groups of columns beneath pad/strip footings due to a loss of lateral confinement.

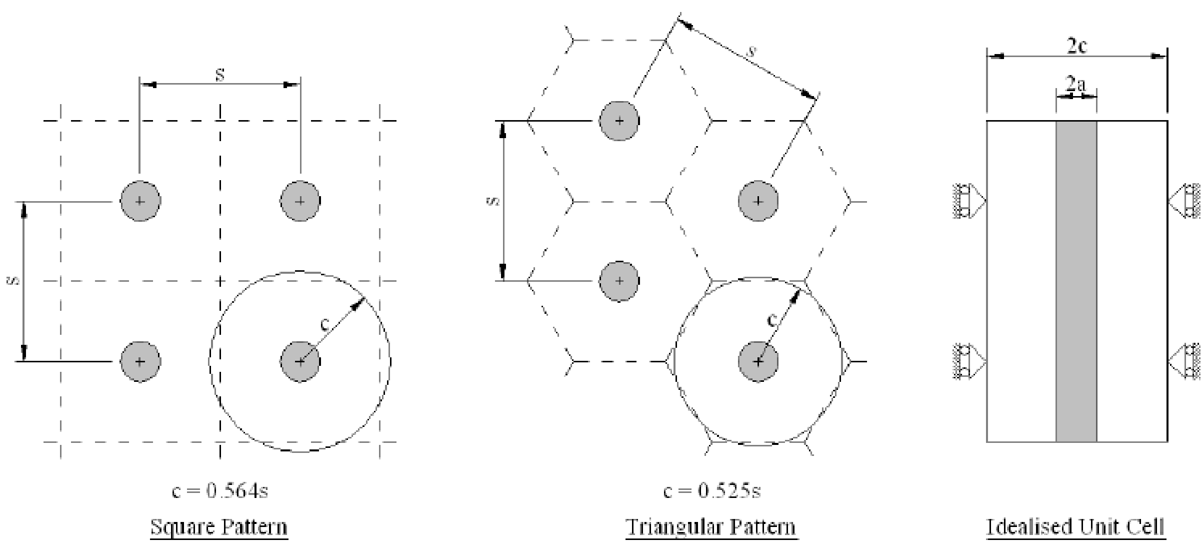


Figure 2. 4: Zones of influence for square and triangular arrangements of stone columns

Basically, stone column notion is described by unit cell theory (Indraratna and Redana, 1997). Unit cell includes two parts,

a) Stone column
b) The ambient soil within the region of effect of the stone column

(iii) Area Ratio (μs)

The area ratio is the ratio of the area of a stone column section to the area of its unit cell of soil, which is explained by the following equation (2-1):

$$\alpha_r = \frac{Ac}{A} = \frac{Ac}{Ac+As} \qquad [2.1]$$

Figure 2.5 shows both the area of stone column and its equivalent soil area in triangular pattern.

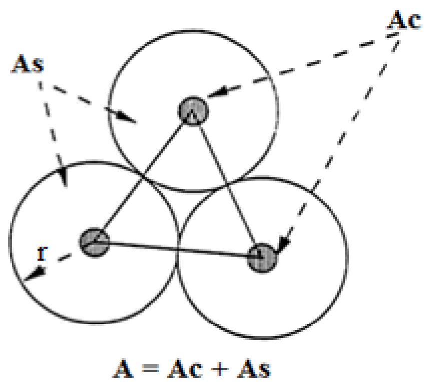


Figure 2. 5: Area definition of unit cell.

(iv) Stress Concentration Ratio (n)

The stress concentration ratio (n) is defined as the ratio of the uniform average vertical stress on a stone column (σ_c) to that applied on the surrounding soil (σ_s) within the unit cell (Hu, 1995), as illustrated in Figure 2.6 and by the following equation (2-2):

$$n = \frac{S}{St} \qquad [2.2]$$

Based on the concept of the unit cell, the relationship between the stress concentration ratio (n) and the ratios of stresses in both clay and stone column, a_s and a_r respectively, can be defined as follows:

$$\sigma = \sigma_s a_r + \sigma_c(1 - a_r) \qquad [2.3]$$

$$\sigma_c = \frac{\sigma}{1+(n-1)a_r} = \alpha_c \sigma \qquad [2.4]$$

$$\sigma_s = \frac{n\sigma}{1+(n-1)a_r} = \alpha_s\sigma \qquad [2.5]$$

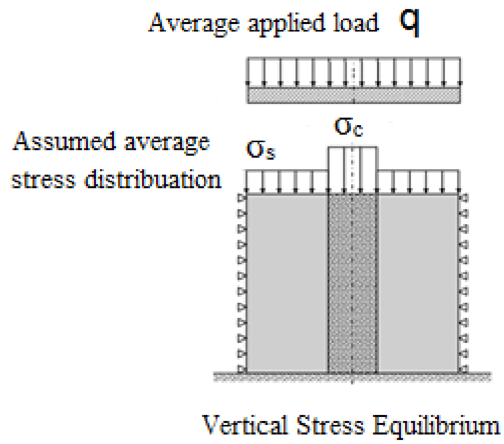


Figure 2. 6: Stress concentration ratio definitions (after Saadi, 1995).

Due to the discrepancy of the stiffness between the clay and the stone material, the column normally carries more load than the clay, especially in the initial stages of loading. This ratio is important to express the changes in stiffness and stress state of the treated clay during loading and after consolidation.

(v) Settlement Reduction Ratio β

The settlement reduction ratio for a given load level is defined as the ratio between the settlement of soil reinforced by stone columns at this load level and the corresponding settlement of the unreinforced soil ($\alpha_r = 0$). Many researchers use the term "Improvement Factor" to express this ratio. The value of this factor is ranged between 1 and 6.

$$\beta = \frac{S}{St} \qquad [2.6]$$

2.2.2. Settlement of Stone Column Reinforced Soft Soil

Forecasting of the settlement is one of the most important parameter which has strong and direct effect on the designing of the stone column. Thus, many researchers have been working on this parameter. The main part of assumption is based on the unit cell concept; in addition the loaded region is infinite.

Methods of Analysis

Priebe (1976) presented that stone column works similar to an incompressible granular column that is through elastic cylindrical without any change in lateral stresses by depth and no secondary shear. Owing to incompressibility of stone column, every change in volume of soil has straight relation to decrease in the height of stone column. Priebe (1976) proposed an improvement coefficient (R) for the settlement, that is the ratio of surface settlement of unimproved ground to improved ground and the replacement coefficient that is defined by the region of the unit cell divided by the region of the stone column. The main reason of its widely uses and popularity of Priebe's procedure is the easiness of applying an improvement coefficient to prevalent consolidation estimation. Nevertheless this procedure does not have consideration on the importance of the properties of the ambient soil.

Van Impe and De Beer (1983) proposed a procedure to assess the decreasing of settlement in the case of the stone column reinforcement within soft soil by considering changes of the coarse-grained strip at stable volume underneath limit of equilibrium states. In this analysis stone walls are used instead of stone columns with equal plan region.

Procedure of Equilibrium

Theory of Greenwood (1975)

Greenwood (1975) proposed empirical curves of settlement reduction owing to ground reinforcement by stone columns as a function of undrained soil strength and stone column spacing.

Greenwood method is based on the in-situ experience and broadly bordered by the data of stress concentration coefficient (n = 5 to 10) within the soft soil with shear strength of (20KN/m^2). Owing to decreasing of stress concentration (n) the ground's stiffness, that is dependent on the stiffness stone column, increases. As a consequently, for stress concentration coefficient (n) more than 15 ($n \geq 15$) large level of improvement is required.

Theory of Balaam *et al*., (1977)

Balaam *et al*., (1977) used finite element method at which examined the treatment of stone columns under a rigid footing by assuming that the stone columns and the soft soil keep the elastic situation in all over of ground which load is applied. They presented that, in the case of stone column's spacing when the spacing of stone column divided by diameter of them is less than 5 ($s/d < 5$), a considerable decreasing in settlement happened. Furthermore the stone column must be extended to the full depth of the clay layer.

Theory of Aboshi (1979)

Aboshi (1979) recommended an equation for the settlement reduction ratio β of improved ground by considering the unit cell theory:

$$\beta = (\frac{S_t}{S_0}) = \frac{m_v(a_c\sigma)H}{m_v\sigma H} = \frac{1}{1+(n-1)a_r} \quad [2.7]$$

where, $m_v =$ Coefficient of volume compressibility

For large amount of σ_o, the above equations which are presented by Barkslade and Aboshi for estimation of β are equal. Nevertheless Aboshi for prediction of settlement did not give any properties about in-situ soil.

Theory of Goughnour and Bayuk (1979)

Goughnour and Bayuk (1979) proposed the elasto-plastic examination (incremental procedure) for stone column. In this examination three assumptions were used:

- At the beginning, stone column is linearly elastic
- At failure, stone column is completely plastic
- At the plastic condition stone column is incompressible

They used the unit cell concept with an incremental method for solving the troubles. The application of incremental method is slow and difficult because this method needs to obtain the soil characteristics at high quality.

Theory of Barkslade & Bachus (1983)

Barkslade and Bachus (1983) and Aboshi *et al*., (1979) worked on the equilibrium procedure for estimating settlement of composite soil. In this procedure the stress concentration factor (*n*) is estimated based on empirical observations. Those parameters which are supposed for this procedure are:

- The unit cell theory is valid
- The settlement (vertical displacement) of stone column and soil are equal
- There is a uniform vertical stress owing to lateral loading within the stone column
- Equilibrium of force is retained through the unit cell

Barkslade and Bachus (1983) presented the curve of consolidation settlement of improved and unimproved ground.

Settlement of improved ground (S_t) is given by:

$$S_t = \left(\frac{C_c}{1+e_0}\right) H \log_{10}\left\{\frac{(\bar{\sigma}_0+\sigma_c)}{\sigma_0}\right\} \quad [2.8]$$

Settlement of unimproved ground (S_0) is given by:

$$S_o = \left(\frac{C_c}{1+e_0}\right) H \log_{10}\left\{\frac{(\bar{\sigma}_0+\sigma)}{\sigma_0}\right\} \quad [2.9]$$

Where, C_c = Compression index obtain from 1D consolidation test

e_o = Initial void ratio

H = Vertical height of stone column improved ground over which settlements are being calculated

$\bar{\sigma}_o$ = Average initial effective stress in the clay layer

σ_c = Change in stress in the clay layer owing to externally applied load

$$\sigma_c = \alpha_c.\sigma \quad [2.10]$$

σ = The average externally applied stress

α_c = The ratio of stresses in the clay

$$\alpha_c = \frac{1}{1+(n-1)a_r} \quad [2.11]$$

n = Stress concentration factor

a_r = Area replacement ratio

They presented that the settlement reduction ratio is defined by:

$$\beta = \left(\frac{S}{S_t}\right) = \frac{\log_{10}\left(\frac{\sigma_O+\alpha_C\sigma}{\sigma_0}\right)}{\log_{10}\left(\frac{\sigma_0+\sigma}{\sigma_0}\right)} \quad [2.12]$$

If assume stone column with long length that means σ_o is very large and very small applied stresses (σ), so the settlement reduction ratio is decreases:

$$\beta = \left(\frac{S_t}{S_0}\right) = \frac{1}{1+(n-1)a_r} = a_c \quad [2.13]$$

Barksdale (1983) mentioned that, the equilibrium procedure presented by equation (2.12) is more reliable to forecast settlement of composite ground, and the equation (2.13) gives a little bit unconservative assessment of anticipated ground improvement and is useful for preliminary assessments.

2.3. Background of Finite Element Approach

The requirements of compatibility, material behaviour, equilibrium and boundary conditions of displacement and forces should be satisfied in order to have an accurate solution to a geotechnical engineering problem (Potts and Zdravkovic, 1999). Importantly, Potts and Zdravkovic (1999) argued that these requirements are satisfied by numerical methods of analysis. When problems are encountered in geotechnical engineering, numerical methods of analysis are found to be a very flexible tool in finding a solution to the problems of complicated equations (Cundall and Strack, 1979; Ford, 1999; Potts and Zdravkovic, 1999).

2.3.1. Finite Element Approach

The finite element method is one of the methods that try to provide solutions to partial integral and differential equations. In a given domain, Desai and Abel (1972) states that the finite element method obtains solutions from problems and gives an approximate value of the variables at a number of points within the domain. The main principle here is that a given domain is divided into finite elements (Reddy, 1993). The key in finite element method is to solve a complex problem by finding an approximate rather than an accurate solution to a simpler alternative problem generated from the main complex problem (Rao, 2005). In fact, the finite element method has been a very common approach to solve geotechnical engineering problems.

The accuracy of the approximate solution can be developed in the finite element method by increasing the number of finite elements, which are identified in the domain. This is because the approximate solution converges to the actual solution since there is a tendency to infinity in the number of finite elements. As a result, the global error, which is the total finite element error, converges to zero (Reddy, 1993).

In solving geotechnical engineering problems, the FEM proves to be effective in dealing with problems of finite-boundary conditions, complex equations and behaviour as a continuum (Potts and Zdravkovic, 1999). Therefore, this method can solve problems of complex loading conditions, restraints and geometries. Because of the important advantages of FEM discussed above and the computer-supported calculation analysis in successive stages, this method is thought to be appropriate to stimulate the stone column in weak soil. Hence, the FEM was used in this study.

The Software of Finite Element

The design of many computer software programs which are used commercially or privately has involved the use of the finite element method principles (ABAQUS, SAP, PLAXIS etc...). The field of geotechnical engineering has observed the development of several specialized software. One of the specialized types of software that is widely used and proved effective in the geotechnical engineering problem analysis is PLAXIS. An important advantage of this software is that since it has been used for a long time (since 1987), it has been developed significantly (Brinkgrene et al., 2008). The wide use of PLAXIS finite element software program is attested in the large number of published studies in the field of geotechnical engineering which used Plaxis in the analysis of their results.

2.3.2. PLAXIS Software

The PLAXIS finite element programme has been developed to study the soil behaviour in geotechnical problems by using either plane strain or axisymmetric models and full 3D models. It was developed by researchers of the University of Delft, Holland. It is provided with full features that enable a realistic simulation for the generation of element meshes and also has refinement options for global and local meshes. The construction process can be simulated in this program by activating and deactivating clusters of elements (soils, plates, anchors and others), changing water tables and application loads and displacements. Boundary conditions are designed to cover most of the conditions of real soil problems. The analysis procedure allows for a realistic assessment of the stress and strain that results from the construction process. According to the geotechnical problem being studied, drained or undrained conditions can be adopted.

For current study, plaxis application was PLAXIS 3D Foundation, which was adopted to apply the results of the stone column model for the improvement of the soft soil. Using Plaxis 3D has overcome one of the most common shortcoming that many researchers could not avoid due to the unavailability of 3D finite element geotechnical software at their times, it is the homogenisation and dimensional changing processes to create a new distribution of stone columns that can be modelled as axisymmetric or plane strain using 2D finite element codes. PLAXIS 3D can efficiently capture the real dimensions of the stone columns foundations.

2.3.3. Material Model

Soil models, which use the finite element approach, have a group of mathematical equations that are integrated into the finite element software (PLAXIS) in order to generate output. These outputs would replicate the output which might be generated by the behaviour of soil. Mathematical equations in such model consider parameters, which, under certain conditions, might have an effect on the special behavior of soil in order to render anticipated results.

The behaviour of real soil is highly non-linear, with both strength and stiffness dependency on the stress and strain level (Potts and Zdravkovic, 1999). Furthermore, soil often shows time-independent behaviour and anisotropic tendencies. The behaviour of the soil may be approximated in order to render anticipated result by varying the degree of accuracy using material models. Nowadays, there are many complexes or simple kinds of material models. The material models available in PLAXIS 3D foundation may include: Linear Elastic model, Mohr Coulomb model, Hardening Soil model, Soft Soil model, Soft Creep model, Modified Cam Clay model and Hardening Soil model with small strain stiffness. In order to decide the suitability of a model, relevancy of the characteristics of soil type and the controlling parameters and features should be carefully taken into consideration. An overview of the material model used in the subsequent FEA is given below:

The Mohr Coulomb model is characterized by the simulation of elastic and perfect plastic behavior of soil. This simulation is with a certain yield value at which the soils show a perfect plastic behavior, but before this value, the behavior of soil is expected to be perfectly elastic. Mohr Coulomb model does not take into consideration irrecoverable soil deformation on loading under the yield stress value and the hardening of soft soil during plastic deformation (the stiffness response is considered to be constant for each soil). The model only assumes perfectly plastic straining at the yield stress value. The Mohr Coulomb model is consequently nit suitable in accurately modeling the soft soil properties around the stone column.

Although, the basic features of Soft Soil models are designed for soft soils, they have many limitations related to their tendency to over predict the range of elastic soil behavior. Consequently, over prediction of deformation in problems especially for normally consolidated soils. So, the behavior of the soft soil materials around the stone column may not be accurately modeled using the Soft Soil and Soft Creep model (PLAXIS, 2010a).

The Modified Cam-Clay model has shortcoming in analyzing the stone column reinforcement in soft soil. Beside the limitation of the Soft Soil models, it has the tendency to give a softening behavior, which might lead to mesh dependency and convergence problem of iterative procedures (PLAXIS, 2010a).

The Hardening Soil model with small strain stiffness have the same principle with Hardening Soil model. However, Hardening Soil model with small strain stiffness is enhanced to capture soil behavior at infinitesimal strains. The Hardening Soil model with small strain stiffness requires input resulting from very small strain value (usually in order of 0.001%). It cannot be applicable in this case with large deformation with respect to the reinforcement of stone column in the soft soil.

The Hardening Soil model as formulate by PLAXIS, within this study, is considered the best model for simulating the relevant features of soil behavior, originating from a combination of different soil types subjected to large deformation. Its ability of taking into account of stress dependency of stiffness moduli and accounting for the shear and volumetric hardening make it the most realistically model to capture the features soft soil combination. The Hardening Soil model is therefore, selected to represent the behavior of both soft soil and stone column material. In addition, a linear elastic model used to simulate the footing material.

The basic parameters for the PLAXIS Hardening Soil model include: cohesion, c in kN/m^2, angle of internal friction, ϕ in degrees, angle of dilatancy, ψ in degrees, secant stiffness in standard drained triaxial test, E_{50}^{ref} in kN/m^2, tangent stiffness for oedometer loading, E_{oed}^{ref} in kN/m^2, unloading and reloading stiffness, E_{ur}^{ref} in kN/m^2, power of stress level stiffness dependency, m. Alternative stiffness parameters include: compression index, C_c swelling index, C_s, initial void ratio, e_{int}.

2.4. Previous Related Works

2.4.1. Case Studies

Greenwood, (1970) examined a case study at Bremerhaven field test made a comparison between sand and stone columns. It was observed that the settlement reduction ratio was 0.6 for stone columns and 0.85 for sand columns after 15 months.

Baumann and Bauer (1974) conducted two case studies in which the dormitory foundation in Konstanz was reinforced with floating stone columns. Foundation soil was varved marine clay.

Raju (1997a) presented case studies from Malaysia in which the undrained shear strength of the soft soil was below 15kPa. This value is accepted as the lower bound for stone column reinforcement by most of the authorities. Soil profile for Kinrara interchange contained tin mine tailings (slimes) and Kebun Interchange contained marine clay. CPT soundings were conducted at Kebun and tip resistance values for top 11m are between 0.1 - 0.3MPa. Sand blanket was placed at the top to increase the consolidation rate and to transmit the loads to stone columns more uniformly. The embankment height at Kebun where the settlement gauge was placed was 2.6m (including 1m surcharge which was placed to reduce consolidation time). Stone column diameter was 1.1m with center to center spacing of 2.2m along a depth of 12m at Kebun. Settlement gauges were placed at the top of the stone columns and total settlement was read as 40cm. 1m settlements was observed for untreated ground under same circumstances. Using Taylor's square root of time, 90% of consolidation was estimated to take place in 225days in which the construction period was 45days.

Goughnour and Bayuk (1979) proposed the elasto-plastic examination (incremental procedure) for stone column. They used the unit cell concept with an incremental method for solving the troubles. The application of incremental method is slow and difficult because this method needs to obtain the soil characteristics at high quality.

Munfakh et al. (1983) examined a full scale field test of a reinforced earth embankment which has a reinforced foundation soil with stone column. Site was instrumented with inclinometers, subsurface settlement markers, earth pressure cells and pneumatic piezometers. As the installed column spacing was reduced, pore pressure increased. Jet water is thought to be the reason for this, which was approved by the dissipation of excess pore pressures overnight. Stone column usage reduced total settlements by 40%. Settlement rate was quite high than the unimproved area. Considerable saving was attained with respect to pile option. In addition to that, cost was also reduced by the construction of the test embankment.

Greenwood (1991) presented numerous case histories and lessons learned from these in detail as summarized below. Stone columns interact with the soil and are not just by-passing the soil, like piles. Stiffness of the stone column is stress dependent and the value is 2-20 times stiffer than soils unlike concrete piles which are 10000 times stiffer than soil. Author mentioned that the pressuremeter test is the best way to measure principal stress ratio and also mentioned that soundings may be useful for stiffness measurement but could not give stress dependent behavior of stone columns.

East Brent (Somerset) site: First of all, difference between silty clay and silty sand was not pointed out carefully. Central section slid after 92 days had same settlement with reinforced end in day 90. At the end of day 188, untreated end had less settlement than treated end. Linear excess pore water pressure increase in the treated depth explained the punching of the column end. Shearing displacement was occurred on the peat level at untreated end. Stone column was installed with insufficient depth causing these problems (Greenwood, 1991).

Canvey Island site: Soil under oil storage tank was treated. As loading reached the ultimate level, settlement was increased and stress concentration ratio was decreased (Greenwood, 1991).

Humber Bridge South Approach: Embankment on soft organic clays was treated. Chalk was used as fill material and compacted to a unit weight of much more than natural which supports that the material was turned into a plastic state. Stress measured on stone column increased progressively in second plateau of loading suggested bulging. Local direct stresses created due to the compaction of the chalk may be another logical alternative. Second evidence for this was the pore water dissipation and decrease in stress on columns after completion of the first plateau of loading. Stress ratio was unknown at the end as a result of destruction of the cells (Greenwood, 1991).

DeStephen et al. (1997) outlined a processing center at a nuclear power station founded on hydraulic fill reinforced with floating stone columns. Dry bottom feed technique was used to construct stone columns. Authors reflected the occurrence of heave at the surface with dry bottom feed method.

Cooper and Rose (1999) presented a case history, in which the roundabout embankment founded on soft alluvial deposits were reinforced with stone columns and vibrated concrete columns.

Watts et al. (2000) conducted full scale load tests to investigate the performance of strip foundation resting on treated and untreated variable fill. Dry top feed method was used to construct floating type stone columns. Settlement reduction ratio was measured as 62%. Authors indicated that bulging occurred at a depth three times the column diameter. Stress measurements indicated that stress concentration ratio is overestimated by current design methods.

Raju and Wegner (2004) examined 19 vibro replacement case histories in Asia between 1994 and 2004. In addition, wet top feed method and 3 different dry bottom feed methods were outlined.

Clemente and Parks (2005) investigated the performance of gas-fired power station site founded on placed fill reinforced with stone column constructed with dry bottom feed method. Large scale field load test program was conducted and settlements were monitored for 1.5 years which were showed good foundation performance.

Serridge and Sarsby (2009) conducted field trials of strip foundations on Bothkennar clay reinforced with floating stone columns constructed by dry bottom feed method. Untreated ground settlements were half the average of treated ground in a period of 44 weeks which was attributed to the dissipiation of pore pressures through stone columns. Significant stress transfer is measured underneath the columns with lengths shorter than critical length.

Mohamedzein and Al-Shibani (2011) presented a case study of an end-bearing reinforced embankment on soft soil with an undrained shear strength of min. 4 kPa.

2.4.2. Numerical Studies

Barksdale and Bachus (1983) have conducted finite element analyses to give non-linear solutions to settlement and stress concentration of stone columns reinforcing soft cohesive soils. In that study, the parameters of the stone column were selected as the angle of internal friction, $\phi_s = 42^o$ and a coefficient of at-rest earth pressure $K_o = 0.75$ for both the stone and soil. From the analysis results, they reported that soil shear strength did not influence the settlement since the soils which have modulus of elasticity greater than 1100 KN/m^2 did not fail by an interface or soil failure. The soil having smaller stiffness than 1100 kPa had been assumed to have a shear strength value of 19 kN/m^2. In non-linear analyses, the effect of interface was found to be very little as increasing the settlements; therefore, the effects of interface elements, having able to model the no slip, slip or separation using Mohr-Coulomb failure criteria, were ignored.

Elshazly et al. (2008) modelled stone column and improved soft clayey soil by the unit cell idealization in two-dimensional axisymmetric modelling in Plaxis software programme. They calibrated the load-settlement behaviour of the improved ground by stone column installation obtained by full-scale field load tests. The stress-strain behaviour of all soils was modelled by Hardening Soil model satisfying also Mohr-Coulomb's failure criterion. The area ratio of the calibrated model was equal to 0.29 and the column length was 10.8 end-bearing to harder stratum as in the field load tests. The modular ratio of the stone to soft clay was 8.5 and the loading was applied in three stages as 30 – 90 and 150 kPa.

Kuruoğlu (2008) modelled the plate loading tests previously completed by Özkeskin (2004) and calibrated the parameters of the soil. He has completed his modelling in PLAXIS 3D finite element software. After calibrating the untreated soil parameters by Mohr-Coulomb soil model, he carried out three rigid plate loading tests on rammed stone column improved soil. He conducted parametric study by using elastic composite soil model which matched the load-settlement behaviour with 3D stone column analysis. He analysed the silty clayey soil in undrained conditions and found the parameters from back analyses as; the undrained shear strength, c_u = 22 kN/m^2; modulus of elasticity, E_c = 4500 kPa and the Poisson's ratio, ν = 0.35. The stone columns were modelled with linear elastic material model and its modulus of elasticity was selected as 39 MPa, recommended by Özkeskin (2008) resulted from the back analysis of the single rammed stone column loading at the site.

Zahmatkesh and Choobbasti (2010) carried out finite element analyses to estimate the settlement of the treated ground with stone columns by using PLAXIS 2D software with plane strain modelling. Mohr Coulomb's failure criterion was utilized in drained analyses for all of the soil models including stones, soft clay and sand stress distribution layer. The selected parameters of the stone column and soft clay were reported as the modulus of elasticity was 4000 kPa for the soft clay while it was equal to 55000 kPa for the stone column material and 20000 kPa for sand layer. The Poisson's ratio was given as 0.35 for soft clay whilst 0.30 for stones and sand. The internal friction angle was defined for soft clay 23^o, for stone column material 43^o and sand layer

30^o. Angle of dilatancy was also defined for stones and sand as 10^o and 4^o, respectively. Drained cohesion was zero for all of the soil types defined. Finally, the interface strength values were given as 0.7 to soft clay, 0.9 to stone column material and 1.0 (rigid) for sand layer.

Table 2. 2 Summary of literature review in short format

Authors	**Material and geometric modelling**			**Finding**
	Foundation type	Improved soil type	Stone column α_r	Settlement reduction factor, β
Greenwood (1970)	Embankment	Clay, Peat	25%	0.61
Baumann & Bauer (1974)	Raft	Silt	47%	0.25
	Strip Footing	Sand	59%	0.25
Goughnour & Bayuk (1979b)	Embankment	Clay, Silt	34%	0.42
Munfakh et al. (1983)	Embankment	Clay	25%	0.59
Mitchell & Huber (1985)	Strip Footing	Silt, Clay, Sand	29%	0.65
Greenwood (1991)	Embankment	Clay, Peat	12%	0.60
	Storage tank	Clay	22%	0.44
	Embankment	Clay, Peat	11%	0.77
DeStephen et al. (1997)	Raft	Fill	39%	0.50
Raju (1997a)	Embankment	Silt, Fill	35%	0.25
	Embankment	Clay	20%	0.40
Cooper & Rose (1999)	Embankment	Clay, Silt, Peat	15%	0.39
Watts et al. (2000)	Strip Footing	Fill	21%	0.68
Raju et al. (2004)	Embankment	Silt, Fill	24%	0.38
Clemente et al. (2005)	Strip Footing	Sand	33%	0.29
Serridge & Sarsby (2009)	Strip Footing	Clay	20%	0.44
Ellouze et al. (2010)	Strip Footing	Sand	32%	0.50
	Strip Footing	Clay	15%	0.50
Mohamedzein & Al-Shibani (2011)	Embankment	Clay	15%	0.53

2.5. Summary of Literature Review

There are a lot of issues raised in this chapter. A general background and application of stone column have been discussed in detail. Vibro stone columns are a popular form of ground improvement, which enhance the settlement performance of treated soils. A review of the settlement performance of stone columns from field and numerical studies was also outlined in the previous sections. All points presented above reflect the performance of stone column that it has been applied successfully around the world for last sixty years.

However, in Ethiopia especially urban areas where the availability of weak soil is high stone column which offers an efficient alternative to traditional piled solutions when supporting moderately loaded structures haven't applied yet.

Thus this study is intended to investigate and clarify the effectiveness and applicability of stone column as a ground improvement technique and to develop better understanding on the application in the case of Ethiopia.

CHAPTER THREE

NUMERICAL MODELLING OF STONE COLUMN

3.1. Introduction

The scope of this chapter is to build and develop a numerical model to simulate a small group of stone column that support a rigid footing on soft soil. The description of study area was presented. Realistic boundary conditions including restraints, ground water table, applied loads, column installation methods are adopted. Material parameter for soil profile is calibrated from secondary data acquired from study area. Stone column material parameter is also presented. Constitutive models that represent the behavior of both stone column ad soft soil material are selected too.

In order to build a realistic model for a stone column that support a rigid footing on soft soil, the modelling process should involve a series of challenges; including the appropriate approach for simulation, dimension of the model, mesh geometry, boundary positions, selection of parameters used in the analysis and the right choices for the constitutive model that represent the studied soil. In addition, as the FEM is an appropriate technique, it is necessary to carry out a number of preliminary checks, such as mesh sensitivity and distance to the boundary, to ensure accurate numerical analysis.

3.2. Description of Study Area

3.2.1. Location

Bishoftu also known as Debre-zeit is a city in central Ethiopia. The city is located in the East Shewa zone of Oromia at $8^0$33’35”N – $8^0$36’46”N latitude and $39^0$11’57’’E – $39^0$21’15’’E longitude. It is situated a few dozens of miles (29.8 mil or 47.9 km) southeast of Addis Ababa and elevated at 1,920 m above mean sea level.

The climate here is mild, and generally warm and the temperate. In winter, there is much less rainfall in Bishoftu than summer. The average annual temperature is 18.7^0C and the average rainfall is 892mm.

The city has been developing greatly since the second half of the 20^{th} century, and its population number has been increasing greatly since 1990s. Based on the 2007 Census conducted by the

Central Statistical Agency of Ethiopia (CSA), this city has a total population of 99,928, of whom 47,860 are men and 52,068 are women. The area is famous with its lakes, including Lake Babogaya, Lake Bishoftu, Lake Cheleleka, Lake Hora, lake Kuriftu, Green Lake and others.

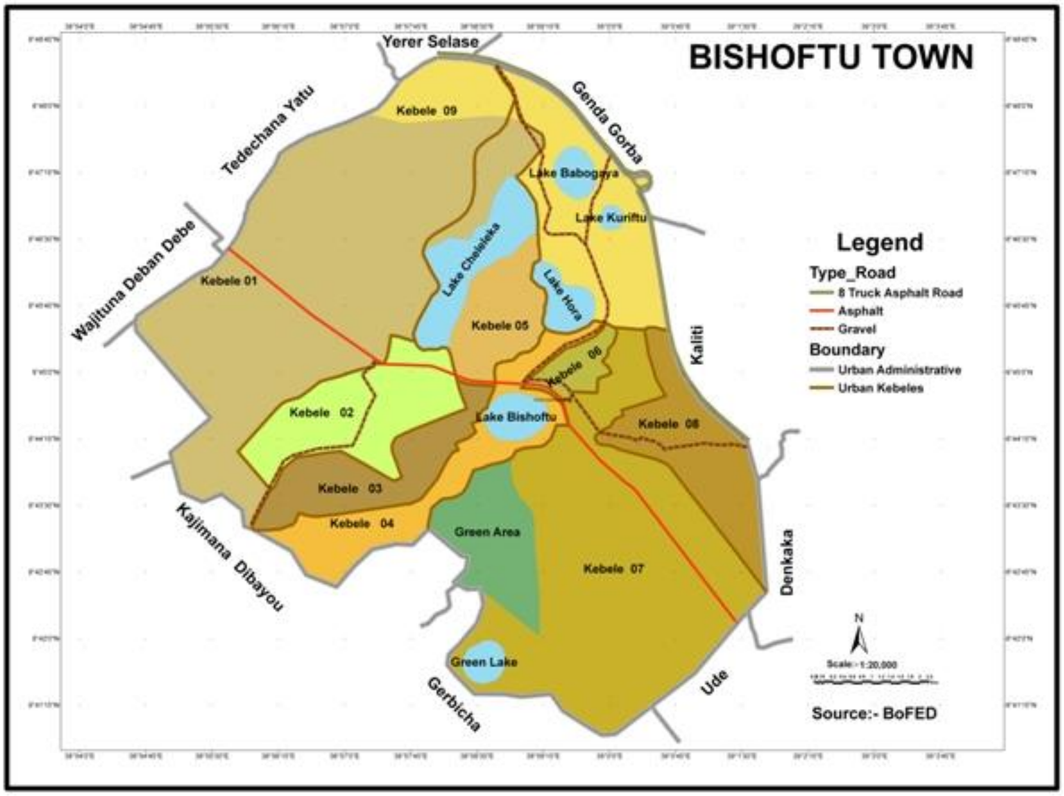


Figure 3. 1: Map of Bishoftu city (source: Google map)

The proposed site that the required secondary data acquired was selected reasonably at the outskirt Bishoftu at the vicinity Dukem of Eastern Industry Zone. The site is located on within the existing Eastern Industry zone compound towards side of the main asphalt road.

3.2.2. Geology and Subsurface Condition

Regional Geology

Debre Zeyt area lies within a transition zone between the main Ethiopian rift valley to the east and The Ethiopian Plateau to the west. The geology of the area containing the project site falls within the “Rift Axis Complex” termed by F. Mazzarini et.al. The Rift Axis Complex, according to this author, consist young central volcanoes, lacustrine deposits and basaltic cinder and spatter cones.

It can be thought that a violent expansion of magmatic gas or steam formed the maar deposit of Lake Babo Gaya as shallow, flat- floored creater filled with water to form the lake. An explosion caused by ground water coming into contact with hot lava or magma are known to form volcanic creater (Addis Geosystems Plc., 2016).

Site Geology and Subsurface Condition

Sub-Surface investigation has confirmed that the selected site and its surroundings covered with with CLAY (black cotton), sandy SILT and silty CLAY soil (Addis Geosystems PLC., 2016). Geological section showing lateral and vertical extent of these layers is shown in Appendix A.

Ground Water Observation

Measurement of ground water level was made on the days following completion of each borehole drilling operation and ground water was not recorded at any of boreholes within depth of 15m (Addis Geosystems PLC., 2016).

3.3. Development of Material Parameters

3.3.1. Development of Soil Material Parameters

The basic elements for modelling and analysis of stone column treated soil system are identifying parameters for base model, using in-situ test or correlations of parameters. Based on the geotechnical investigation made by (Addis Geosystems PLC., 2017) a representative value of standard penetration test (SPT) data are chosen and required soil parameter correlations are done. The soil types considered for this study are the stratified clay from Bishoftu in vicinity of Eastern Industry Zone. The engineering design parameters where extracted from SPT-N values of representative borehole log sheet, as shown in Appendix A, and considered to represent the soil in the study area. These parameters are used as input parameters for PLAXIS 3D Foundation.

The representative value of standard penetration test (SPT) data chosen for the soil parameter correlations has presented in the Appendix A-2. Empirical correlations between blow counts and consistency of soils as established by various are given below (Bowles, 1988; Carter and Bentley, 1999; G. Look, 2007). The SPT N value measured had corrected to N values corresponding to 60% of energy efficiency (N_{60}) (Skempton, 1986). The corrected N60 of each layer had used to correlate the required soil parameters (G. Look, 2007; Carter and Bentley, 1991; Kulhawy and Mayne, 1990).

Basic geotechnical properties

The general properties required for simulating the soil material as per Hardening Soil model mainly includes: bulk unit weight (γ, kN/m^3), saturated unit weight (γ_{sat}, kN/m^3), initial void ratio (e_0)…etc. The saturated unit weight (γ_{sat}, kN/m^3) of the cohesive soil (Eq. 3.6) has calculated by using the correlation recommended by (Carter and Bentley, 1991, 1991; Peck et. al. 1974; Bowles, 1977). In addition, the relation between initial void ratio and SPT N for fine grained cohesive soils is given below Eq. 3.7 (Cubrinovski and Ishihara, 1999).

$$\gamma_{sat} = 16.8 + 0.15N_{60} \quad (kN/m^3) \tag{3.1}$$

$$e_0 = 0.89N^{-0.12} \tag{3.2}$$

In-situ stress state

The in-situ state of stresses in soil is defined in terms of the current values of effective vertical stress (σ'_{vo}) and effective horizontal stress (σ'_{ho}). The stress ratio K_o, (the at-rest coefficient of horizontal soil stress, correlated from the effective friction angle (ϕ') as expressed below Eq.3.8 for normal consolidation.

$$K_o^{nc} = 1 - sin\phi' \tag{3.3}$$

Strength Characteristics

The strength characteristics required for simulating the soil material as per Hardening Soil model mainly includes: effective cohesion (c', kN/m^2), effective angle of internal friction (ϕ', ^{o}), initial angle of dilatancy (ψ)…etc.

According to the SPT N values, undrained shear strength parameter of the soil has calculated by using the correlations recommended by (Kulhawy FH and Mayne Paw, 1990;

$$c_u = 29N^{0.72} \quad (kN/m^2) \tag{3.4}$$

For the effective cohesion value, the method recommended by (Sorensen K.K and Okkels N, 2013) suggest that:

$$c' = 0.1c_u \quad (kN/m^2) \tag{3.5}$$

In order to determine the long-term soil parameters, effective angle of internal friction (ϕ'), had empirically correlated to effective cohesion value, as recommended by (G.Look, 2007) Table A.3 in the appendix.

Stiffness Characteristics

The compressibility resistance of the soil material is defined by its stiffness parameters. Empirical correlations for compression index (C_c) with soil parameters (i.e. initial void ratio, e_o) have done after Rendon-Herrero (1980) as expressed below (Eq. 3.6).

$$C_c = 0.3(e_o - 0..27) \quad [3.6]$$

The one-dimensional stiffness parameters are converted into three-dimensional parameters for the Hardening Soil model using the following expressions (Brinkgreve & Broere, 2006):

$$E_{oed}^{ref} = \frac{2.3(1+e_o)p^{ref}}{C_c} \quad [3.7]$$

$$E_{ur}^{ref} = \frac{2.3(1+e_o)(1+v)(1-2v)p^{ref}}{C_s(1-v)} = 3E_{oed}^{ref} \quad [3.8]$$

$$E_{50}^{ref} = E_{oed}^{ref} \quad [3.9]$$

Where: E_{50}^{ref} is Secant stiffness in standard drained triaxial test; E_{oed}^{ref} is Tangent stiffness for primary oedometer loading; E_{ur}^{ref} is Unloading / reloading stiffness.

The remaining advanced stiffness parameters (i.e. poisson's ratio, *v;* Power for stress-level dependency of stiffness, *m;* Failure ratio, R_f) are used as default value (Brinkgreve & Broere, 2006).

Permeability and consolidation coefficient

The value of the coefficient of permeability can vary over a wide range. The vertical hydraulic conductivities (kv0) are lower than the equivalent horizontal values and an anisotropy ratio (k_{h0}/k_{v0}) of 1.5–2.0 was found. (Killen, M. 2012).

A summary of the parameters developed for the soil is illustrated in Table 3.1.

3.3.2. Development of Stone Column Material Parameters

The stone column material chosen to be installed in the clay soil in order to improve its strength and stiffness is crushed stone used for construction purpose especially aggregate for concrete production. Its parameter was acquired from Project Profile for KASMA Engineering PLC Stone Crushing and Screening Plant (2004) and, its properties were calibrated with international published papers, which focused on stone column materials. A summary of the parameters developed for the stone column is presented in Table 3.1.

Table 3. 1: Soil and stone column parameters adopted for all subsequent finite element analysis

Soil Parameter	Stone Column	Layer I	Layer II	Layer III	Layer IV	Unit
Material model	HS	HS	HS	HS	HS	-
Depth	2.00 - 6.00	0.00 - 1.60	1.60 - 3.45	3.45 - 6.00	6.00 - 8.00	m
Type of material behavior	Drained	Drained	Drained	Drained	Drained	-
Bulk unit weight, γ	19.0	17.3	18.2	18.6	19.7	kN/m^3
Saturated unit weight, γ_{sat}	21.0	17.3	18.2	18.6	19.7	kN/m^3
Over-consolidation ratio, OCR	-	1	1	1	1	-
Permeability, K_h	1.7	8.6×10^{-5}	8.6×10^{-4}	8.6×10^{-4}	8.6×10^{-4}	m/day
Permeability, K_v	1..7	5.8E-05	7.2E-04	7.2E-04	7.2E-04	m/day
Poisson's ratio, ν	0.2	0.2	0.2	0.2	0.2	-
Cohesion, c'_{ref}	1*	9	19	23	29	kN/m^2
Friction angle, ϕ'	45	18	24	21	23	º
Dilatancy angle, ψ	15	0	0	0	0	º
Initial voids ratio, e_o	0.5	0.73	0.65	0.63	0.60	-
Compression index, C_C	-	0.14	0.11	0.11	0.10	-
Swelling index, C_S	-	0.04	0.03	0.03	0.03	-
Reference pressure, pref	100	100	100	100	100	kN/m^2
Lateral earth coefficient, K_0^{nc}	1.00	0.68	0.59	0.64	0.60	-
m	0.3	1.0	1.0	1.0	1.0	-
E_{50}^{ref}	70,000	2,866	3,340	3,478	3,675	kN/m^2
E_{oed}^{ref}	70,000	2,866	3,340	3,478	3,675	kN/m^2
E_{ur}^{ref}	210,000	8,599	10,019	10,433	11,024	kN/m^2

* A nominal value for c = 1kPa is adopted to ensure numerical stability (M. Killen, 2012).

3.3.3. Development of Structural Footing Material Parameters

The only structure model applied in this study, which is the last component of the full 3D stone column foundation model, is the concrete footing. Typical parameter values for the reinforced concrete material were assumed for the footing as per the general property of concrete structure as shown in Table 3.2. A linear elastic model was used to simulate the footing material.

Table 3. 2: Material properties of footing (EBCS, 1995)

Parameter	**Name**	**Value**	**Unit**
Material type	Type	Linear Elastic	-
Type of material behavior	-	Non Porous	-
Young's modulus	E	30×10^{6}	kPa
Unit weight	γ	24	kN/m^{3}
Poisson's ratio	ν	0.15	-

3.4. Preliminary checks to Ensure Accurate Numerical Analysis

The accuracy of the finite element model depends not only on the level of sophistication of the material model, but also on the boundary condition of the domain, on the number and type of the elements in to which the domain is discretized and other modeling issues

3.4.1. Influence of Distance to Boundary

External boundaries are an artificial representative of real forces, extensions and conditions that define the solution of a finite element model. It is not possible to include real natural extension of a mass of soil or some events applied on it, so finite element code (PLAXIS 3D Foundation) enables the user to substitute the reaction of these extensions and events as a restraint, displacements and forces at the boundary. Consequently, the user can quantify these effects and assign them to the studied model with minimum effect on the accuracy of this model. Positions of boundary restraints can significantly affect finite element simulation results. Since the generated reaction forces and displacement in these boundaries can influence the impact of the applied forces on the zones of interest. So to avoid any restriction that reflected on the accuracy of the finite element model, a user should select the location of this boundaries to be sufficient distant from any zone of interest, but at the same time the user should consider them not to be exceedingly far, costing more time in the analyzing process (Ammari, K. and Clarke, B. 2016).

Practically the sensitivity analysis was carried out on the model for only the side boundary. Bottom boundary is already considered as a natural boundary as it was stiff enough at 8m deep. The location of distance to the boundary from the footing center ranged from 2B-12B (Killen, 2012).

The sensitivity analysis was conducted on a selected 3m square footing. The vertical displacement (u_y) was measured at the points A, B and C, which are 0, 1 and 2m below the center of footing, respectively (Figure 3.3). The soil profile adopted for the subsequent FEA and sensitivity analysis has been described in detail previously in Section 3.4.

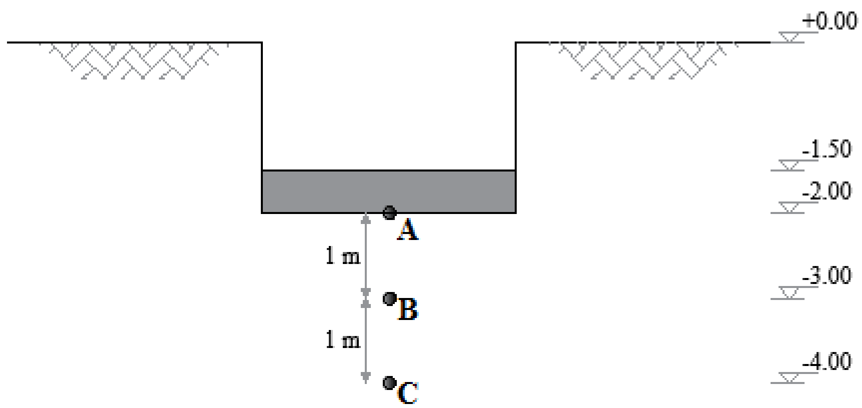


Figure 3. 2: Points A, B and C beneath pad footing

The accuracy of relevant boundary distance is determined by comparing the normalized error for vertical displacement (u_y) at a selected point where the side boundary is located at a distance xB against that of 12B.

Normalized error for vertical displacement, $u_y = \left(\frac{u_{y\,(xB)} - u_{y\,(12B)}}{u_{y\,(12B)}}\right)$

Where: $u_{y\,(xB)}$ - is vertical displacement at the selected point for the case of side boundary is located at (xB) m from the center of the footing pad.

$u_{y\,(12B)}$ - is vertical displacement at the selected point for the case of side boundary is located at (12B) m from the center of the footing pad.

The flowchart outlining the steps taken to ensure accurate numerical analysis for boundary distance is shown in Figure 3.4.

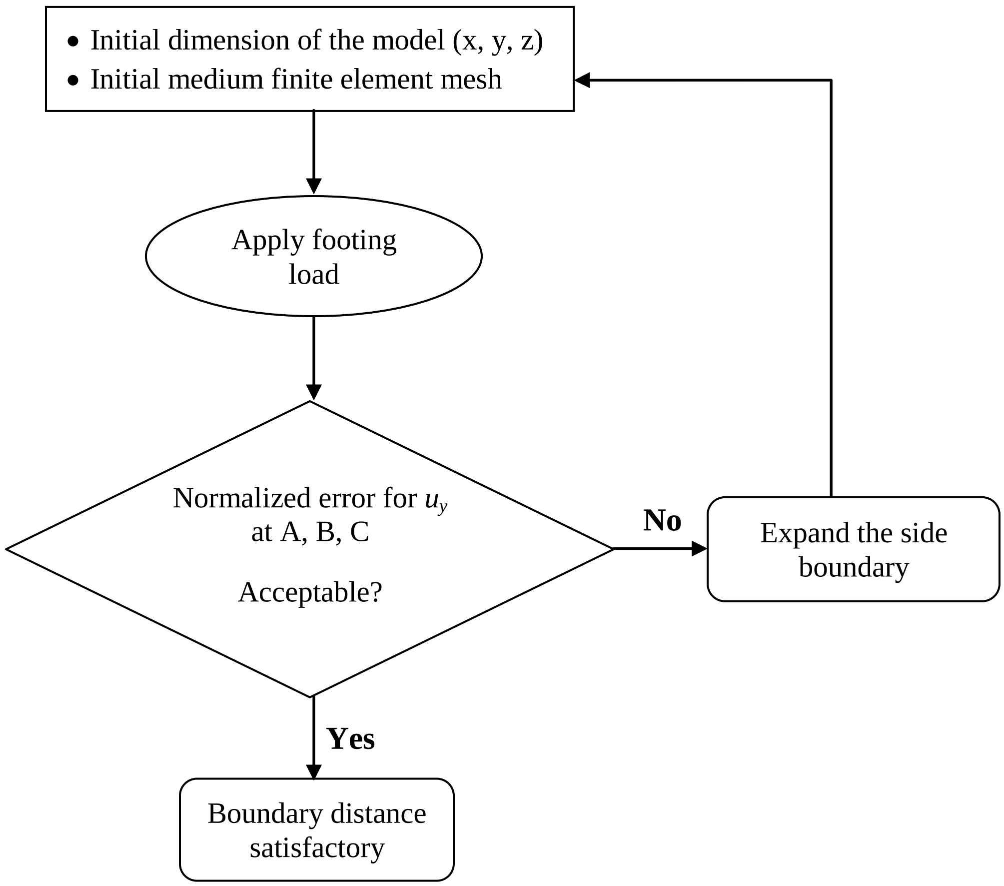


Figure 3. 3: Flowchart outlining preliminary analysis checks for boundary distance

The footing were loaded to 200kPa and the vertical displacement (u_y) and mean effective stress (σ'_{yy}) at the point X, Y and Z for different boundary distances are listed in Table 3.3.

Table 3. 3: Influence of the distance to boundary upon the settlement of a 3 m square footing

Distance to boundary (B = footing width)	**Vertical displacement, u_y (mm)**			**Normalized error (%)**		
	Point A	Point B	Point C	Point A	Point B	Point C
2B	141.24	125.54	67.72	1.7	4.0	13.2
4B	140.34	122.85	64.03	1.1	1.8	7.0
6B	139.28	121.29	60.25	0.3	0.5	0.7
8B	137.73	120.88	60.16	0.0	0.2	0.5
10B	137.19	120.02	59.62	0.0	0.0	0.0
12B	138.83	120.67	59.84	0.0	0.0	0.0

It appears that positioning the boundary closer than 6B to the center of the footing induces more settlement in the pad footing. The distance to the boundary from the center of the footing is conservatively chosen at 6B for all the subsequent FEA.

3.4.2. Mesh Sensitivity Analysis

The influence of the number of elements upon the accuracy of the finite element model was investigated by conducting a mesh sensitivity analysis. In the subsequent parametric studies, both the number of columns and column spacing are varied beneath pad footings, which results in various footing sizes. Mesh sensitivity analysis was conducted for a reasonably selected three different footing sizes where the accuracy of medium and fine meshes are compared against the very fine (v_f) meshes.

The vertical displacement (u_y) was examined for the mesh sensitivity analysis as the settlement performance of the stone column is the primary focus of this thesis. u_y was measured at the points A, B and C, which are 0, 1 and 2m below the center of footing, respectively (Figure 3.5).

The accuracy of medium and fine meshes is determined by comparing the normalized error for vertical displacement (u_y) against very fine meshes.

Normalized error for vertical displacement, $u_y = \left(\frac{u_{y,vf} - u_y}{u_{y,vf}}\right)$

Where: u_y - vertical displacement at the selected point for the medium and fine meshed elements

$u_{y,vf}$ - is vertical displacement at the selected point for very fine meshed elements

In addition to checking convergence of displacement with an increasing number of elements, the mesh is also checked for discontinuities at inter-element boundaries. Discontinuities typically occur in regions of rapid changes of stress and strain, i.e. beneath the edge of footings, and can be overcome by refining the mesh in these regions.

The flowchart outlining the steps taken to ensure accurate numerical analysis for mesh density is shown in Figure 3.5.

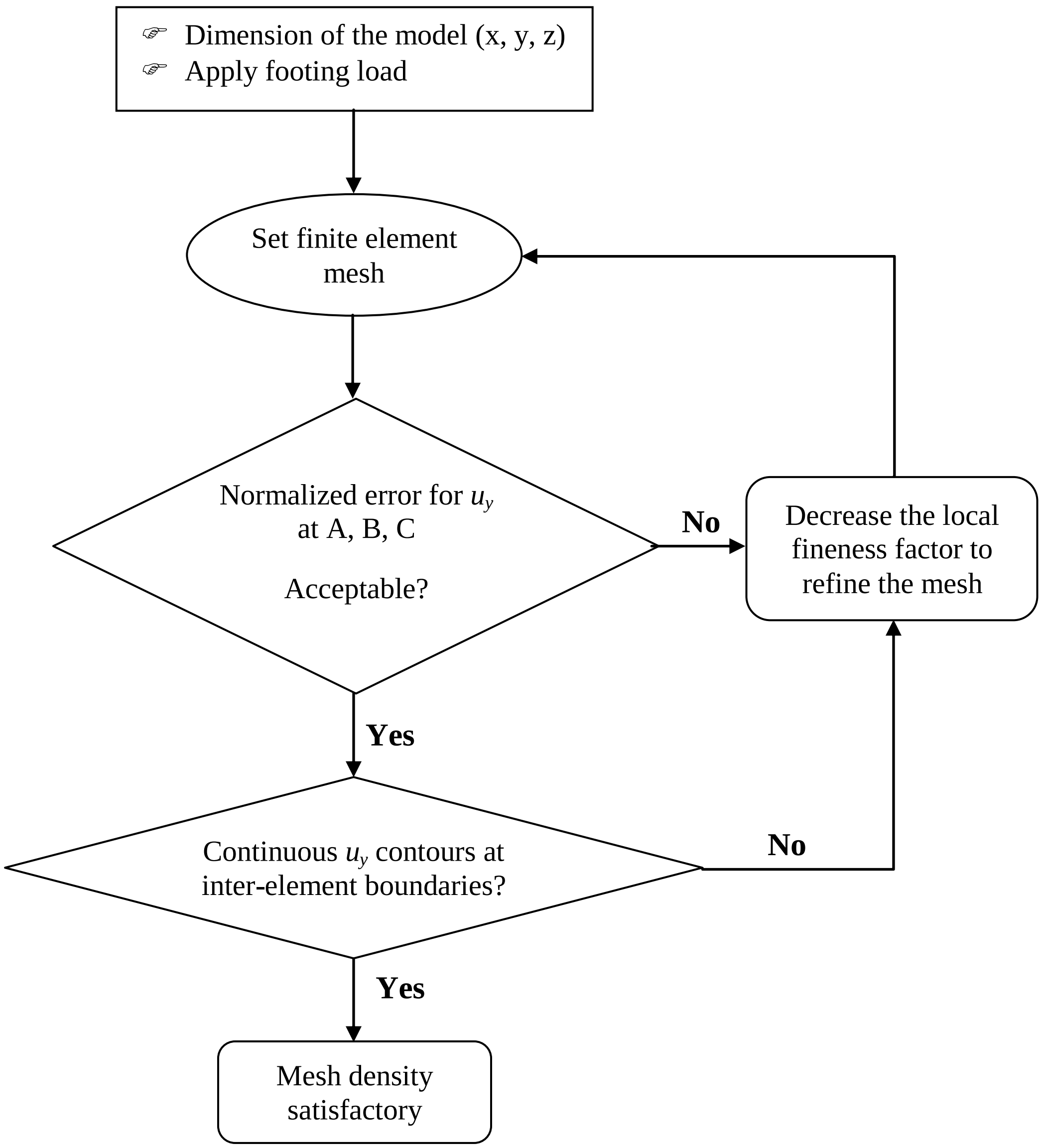


Figure 3. 4: Flowchart outlining preliminary analysis checks for mesh density

The number of elements used for each mesh and the normalized error for the vertical displacement is shown in Table 3.4. It is clear that the values for medium and fine meshes are converging towards the very fine mesh and also the normalized error between the fine and very fine meshes for the vertical displacement is quite low (maximum error is no grater than 5%).

Table 3. 4: Vertical displacement at points A, B and C

Footing width, B (m)	Mesh	No. of elements in 3D mesh	No. of nodes in 3D mesh	Vertical displ., u_y (mm)			Normalized error (%)		
				A	B	C	A	B	C
2	Medium	2192	6565	96.13	67.08	25.72	1.8	11.6	4.3
	Fine	3760	11085	95.19	62.30	25.28	0.8	3.7	2.5
	Very fine	10260	29003	94.45	60.09	24.67	0.0	0.0	0.0
3	Medium	1456	4541	140.59	120.34	66.54	2.4	14.7	7.0
	Fine	4736	13769	139.23	109.82	63.87	1.5	4.7	2.7
	Very fine	9632	27545	137.22	104.93	62.17	0.0	0.0	0.0
4	Medium	1776	5421	173.48	156.82	101.41	3.6	16.5	7.6
	Fine	4704	13681	170.91	140.84	96.67	2.1	4.6	2.6
	Very fine	9920	28337	167.40	134.66	94.24	0.0	0.0	0.0

3.4.3. Other Modeling Issues

3.4.3.1. Modeling of Soil Behavior

The long-term behavior of cohesive soils, which develop excess pore pressure during loading, can be modeled by two methods with PLAXIS 3D Foundation:

(i). Undrained loading followed by consolidation analysis

(ii). Drained analysis

Method (i) is a closer simulation of reality than method (ii), however it is far more time consuming. In method (i), the soil is specified to behave in an undrained manner during the application of footing loads. Initial undrained settlements are computed and long term settlements are then determined by conducting a consolidating analysis. The soil can be defined using either total or effective strength parameters for this method. However, it is not possible to capture the increase in in soil shear strength with consolidation or the stress dependency of soil stiffness when using total strength parameters and, consequently, effective strength parameters are used in all the subsequent FEA. The initial drained response of the soil is simulated by adding a very large bulk modulus to the pore water stiffness matrix, which ensures that all of the stresses generated from the footing loading will be taken by the pore water. The consolidation analysis is then conducted by removing the extra bulk modulus. The stresses in the pore water pressure are then transferred to the soil matrix and the resulting settlements for primary consolidation are computed.

Killen, M. (2012) conducted the consistency of Undrained and drained analysis, which shows the variation of settlement improvement factors with column length for three arrangements of stone column beneath a 3 m square footing. It appeared that both methods predict a similar variation of settlement improvement factor with column length for all configurations of columns (maximum normalised error is no greater than 7%)

Drained behavior in clays should be considered in evaluating long-term settlement of the ground. However, it may initially behave in an undrained mode in the period of column installation for short-term immediate settlement. Relying on the finding of Authors presented above and to save computational time, method (ii) is adopted for all subsequent FEA.

3.4.3.2. Modeling of Column-Soil Interface

Interface elements are available in PLAXIS 3D Foundation to model the interaction between smooth and rough surfaces i.e. between pile/basement walls and soil. Interface elements can simulate gap and slip displacements, which are normal and parallel to the interface, respectively. The loss of strength at the interface is modeled with a reduction factor (R_{inter}), which relates the interface strength to the soil strength through friction angle (ϕ) and cohesion (c):

$$c_i = R_{inter} c_{soil} \qquad [3.10]$$

$$tan\varphi_i = R_{inter} tan\varphi_{soil} \leq tan\varphi_{soil} \qquad [3.11]$$

Guetif *et al*. (2007) adopt rigid interface elements (i.e. R_{inter} = 1) on the basis that stone columns are tightly interlocked with the surrounding soil and the perfect bond exists along the column-soil interface. However, Gäb *et al.* (2008), Elahzy *et al.* (2008a), Domingues *et al.* (2007a) and many other authors model a perfect bond along the column-soil interface by omitting the interface elements.

Killen, M. (2012) conducted preliminary checks on the influence of interface elements upon the settlement performance of various configurations of columns beneath a 3m square footing. He had clearly showed that a similar variation of settlement improvement factors with column length was observed for column modelled with and without interface elements.

As columns are highly interlocked with the surrounding soil it was deemed appropriate to model the column-soil interface in the subsequent FEA by omitting interface elements, in keeping with the prior work of several other authors.

3.5. Generation of Finite Element Model

Numerical simulations were performed to provide a better understanding of the influence of the column arrangement beneath rigid footings on the load-settlement response.

3.5.1. Geometric Modelling

The various configurations of columns analysed are outlined in the following section. The soil profile, which was calibrated in the previous chapter, was adopted for all of the subsequent FEA. The footings are 0.6 m thick and founded 2 m below ground level. The column diameter is normally not a variable in design as the poker is of fixed diameter (430 mm is the most commonly used size) and the constructed column diameter depends on the properties of the surrounding soil. A diameter of 0.6 m is typical of stone columns constructed in soft soils using the bottom feed system. The column length was increased in 1 m increments (from 0 m, i.e. no columns, to 6 m, i.e. the top of the sandy silt profile adopted) to examine the effect of floating and end-bearing columns (Figure 3.7).

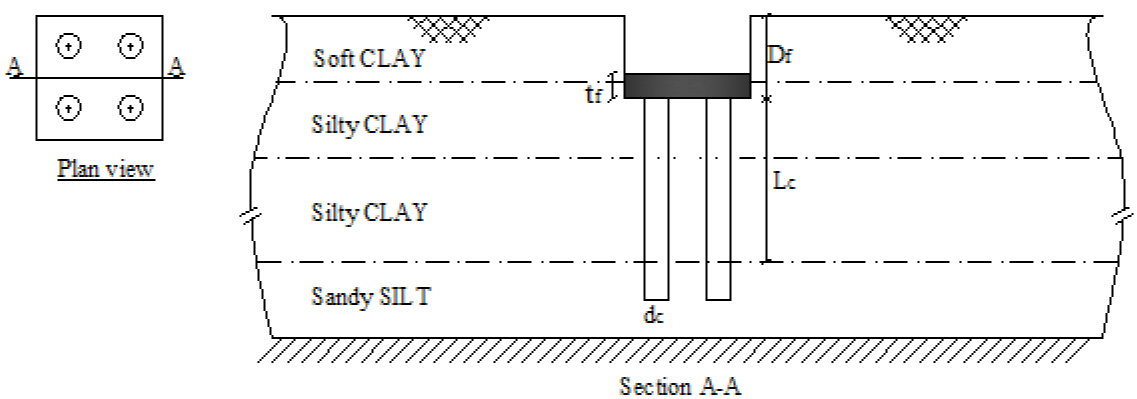


Figure 3. 5: Layout of columns in soil profile

3.5.2. The Theoretical Framework of the Numerical Simulations

The finite element method was used for the numerical simulations. The code PLAXIS 3D Foundation version 1.1 was used for the full 3D models. This code automatically generates the finite element mesh using a triangulation procedure, but global and local mesh refinements may be defined to ensure a good quality of the mesh (Brinkgrene *et al*., 2012).

The soft soil and the stone columns were modelled as continuum elements using 15-node wedge elements for the 3D cases. Perfect bonding between soil and columns at their interface was modelled because stone columns are tightly interlocked with the surrounding soil (Gäb *et al.*, 2008; Elahzy *et al.*, 2008a; Domingues *et al.*, 2007a). The rigid footing was assumed as perfectly rough and modelled as a very stiff plate that produces uniform settlements.

All of the subsequent FEA were performed using a small strain formulation and a staged construction process was modelled. Work planes are horizontal planes (x-z planes) at a certain vertical level (y-level) in which geometry points and lines and, in particular, structures and loads can be identified. Hence used to model the separation between the phases. Every model in PLAXIS 3D, before the beginning of any calculation phase, starts with the initial phase. The initial phase consists of the soil clusters only in which not all other structural parts are activated. Therefore, this stage calculates the stresses and deformations due to the soil clusters only by means of gravity loading. The undrained behavior is not considered for the initial phase because, in the this phase, the stresses from the self-weights are considered to have occurred for a long period of time making the development of excess pore water pressures irrelevant. Therefore in the initial phase, the option "ignore undrained behavior is selected". For the other loading phases, this option is not selected, as it is needed to consider the excess pore water pressures during construction and at the end of all loading phases l. Later, the footing and the column or columns were "wished-in-place", ignoring the changes in the natural soil due to column construction. Finally, the loading on the footing was simulated.

In any non-linear analysis where a finite number of calculation steps are used, there will be some drift from the exact solution. To limit this drift, tolerated error option is available in PLAXIS 3D. The tolerated error used in these runs for all phases except for few phases is 0.01. A tolerated error of 0.01 is suggested for most type of calculations. A tolerated error of 0.01 means the computed value differs from the exact solution by maximum of 1%. Therefore, this error is considered to be within safe and working limits. The maximum iterations that this version of PLAXIS 3D allows is 100. The default value is 50 iterations. Therefore, this value has worked and has resulted in convergence of the results of all phases.

Drained conditions were assumed for all the process, i.e. no excess pore pressures were generated. Consequently, the soil and column response was studied in effective stresses and vertical displacement (settlement) for all subsequent analyses.

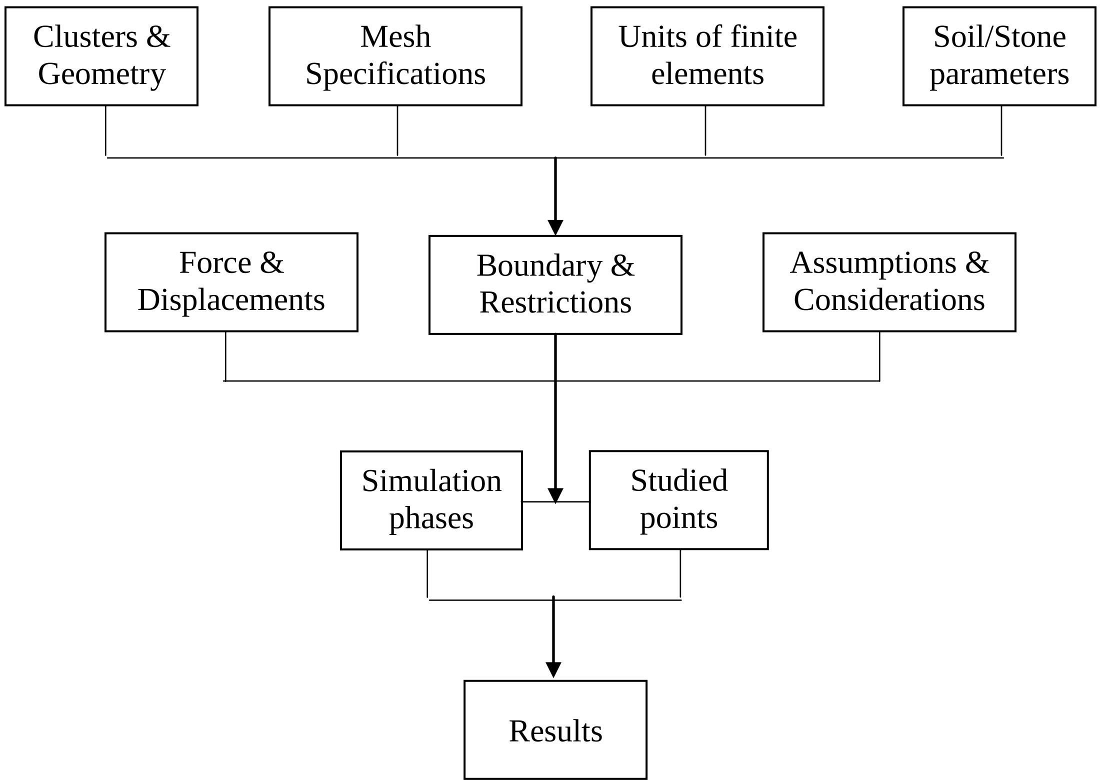


Figure 3. 6: Flowchart of theoretical framework of the numerical simulations

3.5.3. Subsequent FEA and Key Design Parameters

A series of FEA conducted using PLAXIS 3D Foundation code to examine the effect of key design parameters upon the settlement performance of stone columns. The key design parameters and considerations studied are illustrated in Table 3.5. In addition, the preliminary checks for sensitivity analysis have also dictated.

Table 3. 5: Format for subsequent FEA and key design parameters

FEA

Area ratio (ac)

Area ratio (ac)		28%		13%		7%	
Column group		Single	2×2	Single	2×2	Single	2×2
Length (m)	0	✓	✓	✓	✓	✓	✓
	1	✓	✓	✓	✓	✓	✓
	2	✓	✓	✓	✓	✓	✓
	3	✓	✓	✓	✓	✓	✓
	4	✓	✓	✓	✓	✓	✓
	6	✓	✓	✓	✓	✓	✓

Loading (Fd)

Loading (Fd)		50kPa			100kPa			150kPa			200kPa			250kPa			300kPa		
Area ratio (ac, %)		28	13	7	28	13	7	28	13	7	28	13	7	28	13	7	28	13	7
Length (m)	0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	2	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

Table 3. 6: Format for preliminary checks for sensitivity analysis

Preliminary check

Distance to boundary

Footing width	3m					
Factored width	2B	4B	6B	8B	10B	12B
Check	×	×	✓	×	×	×

Mesh Sensitivity

Footing width	2m			3m			4m		
Global mesh	Medium	Fine	Very fine	Medium	Fine	Very fine	Medium	Fine	Very fine
Check	×	✓	×	×	✓	×	×	✓	×

CHAPTER FOUR

RESULTS OF FEA AND DISCUSSION

4.1. Introduction

A series of 72 FEA were conducted using PLAXIS 3D Foundation to figure out the settlement performance of stone columns with various configurations. As described previously that the main focus of this study is to verify the effectiveness of stone columns and to develop better understanding of the behavior of stone column in weak clay soils. The parametric study mainly relied on the settlement performance of stone columns at working load levels. This is because of that the design of foundation on weak soil is usually governed by settlement rather than bearing capacity criteria, due to its high compressibility (Priebe, 1979).

This chapter scopes a detail result of analysis, influence of key design parameters, verification of the result of FEA and also the discussion of results.

4.2. Results of FEA and Parametric Study

4.2.1. Detail of FEA and Key Design Parameters

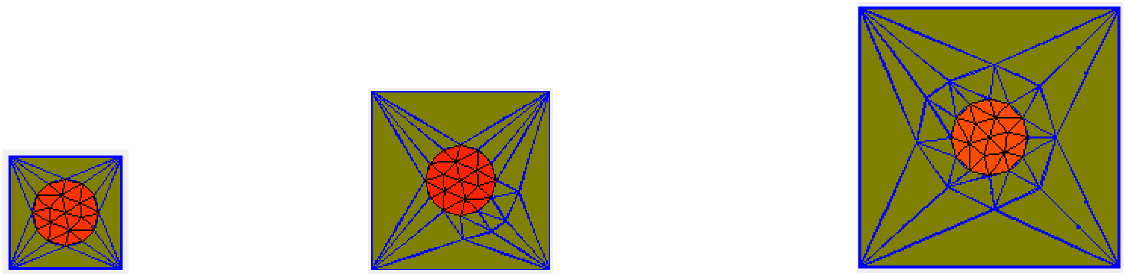
The subsequent FEA were conducted using PLAXIS 3D Foundation code to examine the effect of key design parameters upon the settlement performance of stone columns. The key design parameters and considerations studied are as follows.

(i) Area ratio and column

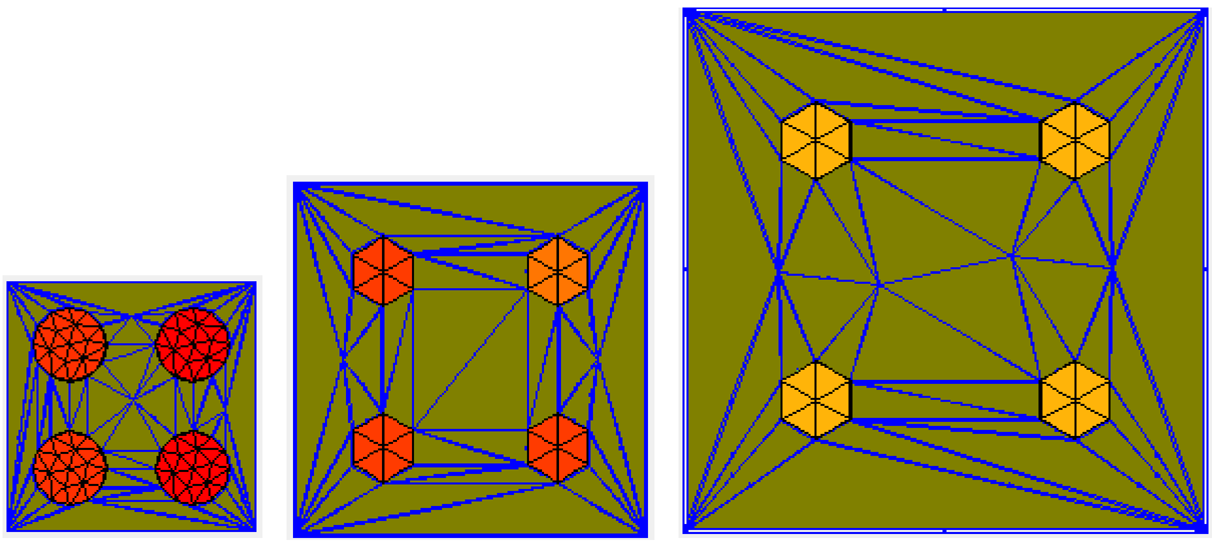
The area ratio for small loaded areas, columns at Ac/A > 28% would not practicable or economic to construct and for wide area loadings such as embankment, columns at Ac/Ac < 7% can tolerate larger settlements (McCabe *et al.*, 2009). Based on this the typical range of Ac/A chosen for parametric study is 28%, 13%, 7%, which corresponds to spacing of square grid at 1.0 m, 1.5 m and 2.0 m respectively.

The degree of confinement proved to individual stone column depends up on area ratio and number of columns within a group. The influence of column confinement was determined by analysing a single column and 2×2 group columns configuration.

The footing width has been decided based on the relationship between the area ratio, column diameter and column spacing as described above.



(a-i) (a-ii) (a-iii)



(b-i) (b-ii) (b-iii)

Figure 4. 1: Layout of (a) single column (b) 2×2 group columns for area ratio of (i) 28%, (ii) 13% and (iii) 7%

(ii) Foundation Load

The pressure applied on the model is based on the allowable bearing capacity for an untreated rigid footing. The allowable bearing capacity based on the SPT test according to Meyerhof (1976):

$$q_a = N_{60}\frac{K_d}{F_1} \quad \text{For } B \leq F_4 \qquad [4.1]$$

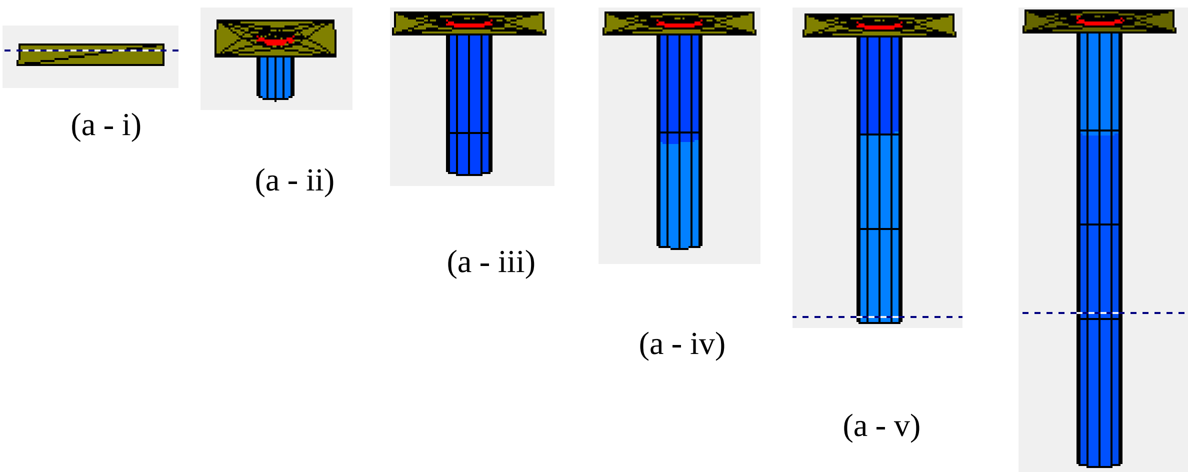
$$q_a = N_{60}\frac{K_d}{F_2}\left\{\frac{B+F_3}{B}\right\}^2 \quad \text{For } B > F_4 \qquad [4.2]$$

Where $K_d = 1+D/(3B) \leq 1.33$, $F_1 = 0.05$, $F_2 = 0.08$, $F_3 = 0.30$, $F_4 = 1.20$ and N_{60} is the average SPT blow counts from 0.5B above to 2B below the foundation level.

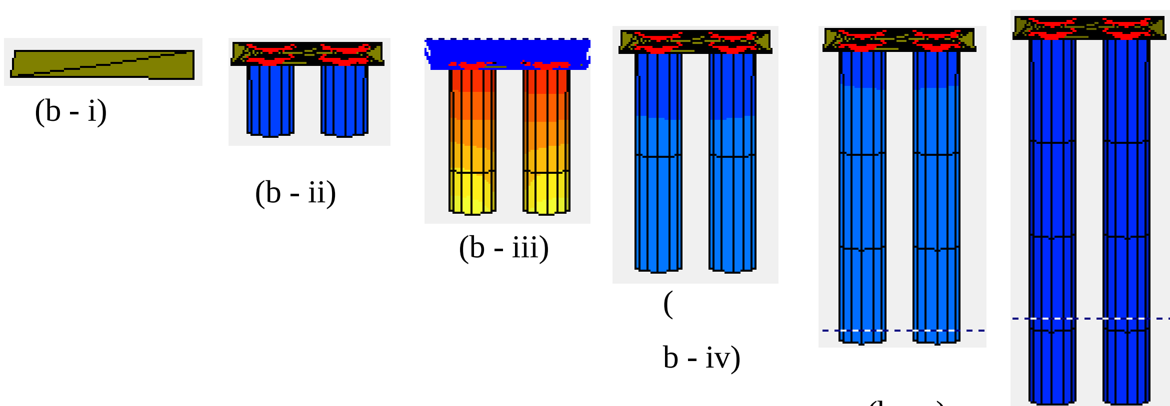
Hence, the foundation pressure applied on the base model is 200kN/m^2 for all subsequent FEA. In addition, to examine the influence of loading on the model the analysis was extended with loading of 50kN/m^2, 100kN/m^2, 150kN/m^2, and 250kN/m^2, 300kN/m^2.

(iii) Column length

The column length was increased in 1 m increments (from 0 m, i.e. no columns, to 6 m, i.e. the bottom of the sandy silt profile adopted) to examine the effect of floating and end-bearing columns.



(a - i) (a - ii) (a - iii) (a - iv) (a - v) (a - vi)



(b - i) (b - ii) (b - iii) (b - iv) (b - v) (b - vi)

Figure 4. 2: Layout of (a) single column (b) 2×2 group columns for length of (i) 0m, (ii) 1m (iii) 2m, (iv) 3m (v) 4m and (vi) 6m.

4.2.2. Result of FEA: Settlement Performance

The settlement performance was measured using a settlement reduction factor, β, which is defined as the ratio of the settlement of a treated footing to the settlement of an untreated footing. A various configuration of columns were modelled and analsed over a typical range of area ratios to examine the relationship of area ratio, foundation loading and column confinement with increment of column length.

As described previously that this study mainly focus on settlement performance, the extreme vertical deformation for each model with respect to the key design parameters had obtained a from the PLAXIS 3D Foundation output. Figure 4.3 illustrates the sample output of deformational behaviors of modelled soil profile and stone column.

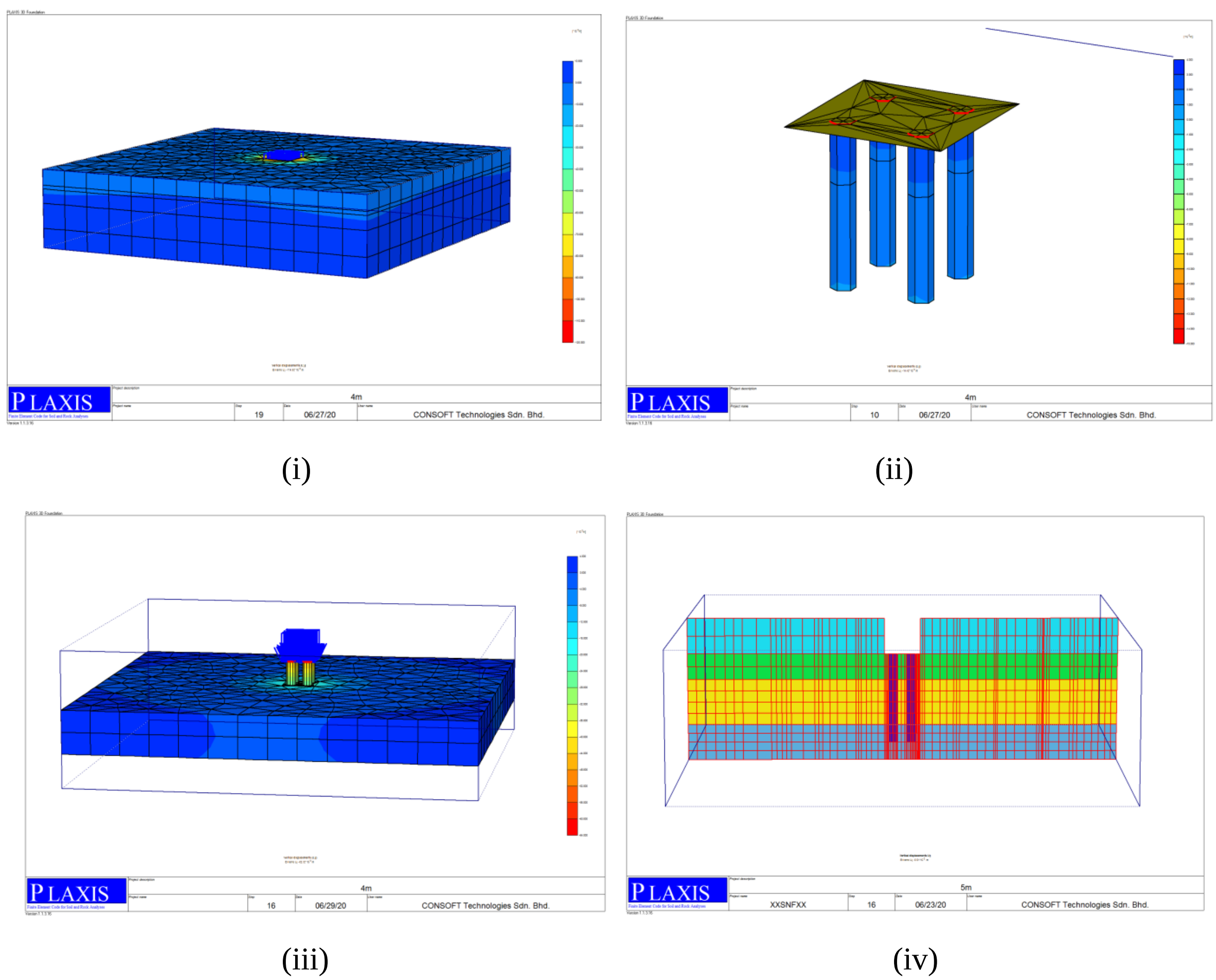


Figure 4. 3: Deformational shape view of (i) full 3D, (ii) stone column 3D, (iii) partial 3D and (iv) Section A-A

The settlement of untreated and treated rigid footing acquired from the simulation depending on the area ratio and column configuration had summarized and tabulated below.

Table 4. 1: Settlement output with column length at various area ratios

Settlement [mm]							
α_r		28%		13%		7%	
Column configuration		Single	2×2 group	Single	2×2 group	Single	2×2 group
Length [m]	0	39.54	76.44	56.37	122.32	75.14	162.71
	1	31.11	68.74	54.64	114.50	74.04	161.42
	2	27.15	65.54	50.74	113.90	72.99	157.72
	3	26.61	62.30	49.15	111.70	71.65	154.90
	4	26.31	56.56	48.55	105.84	63.76	140.45
	6	13.75	29.78	22.95	53.82	38.76	88.76

The settlement illustrated on Table 4.1 is the extreme vertical displacement resulted at each length of stone column due to imposed pressure of 200kPa upon the rigid footing. The result has presented in non-dimensional variable so called settlement reduction factor. Settlement reduction factor computed from the output is presented in Table 4.2.

$$Settlement\ reduction\ factor, \beta = \frac{Settlement\ of\ a\ treated\ footing}{Settlement\ of\ an\ untreated\ footing}$$

Table 4. 2: Settlement reduction factor with column length at various area ratios

Settlement reduction factor, β							
α_r		28%		13%		7%	
Column configuration		Single	2×2 group	Single	2×2 group	Single	2×2 group
Length [m]	0	1.00	1.00	1.00	1.00	1.00	1.00
	1	0.79	0.90	0.97	0.94	0.99	0.99
	2	0.69	0.86	0.90	0.93	0.97	0.97
	3	0.67	0.81	0.87	0.91	0.95	0.95
	4	0.67	0.74	0.86	0.87	0.85	0.86
	6	0.35	0.39	0.41	0.44	0.52	0.55

Also for the variability of foundation pressure applied on the rigid footing, the extreme vertical deformations (settlement) and settlement reduction factor are formatted in Table 4.3. The detail discussion and interpretation on the output demonstrated in the parametric study section 4.2.3.

Table 4. 3: Settlement (extreme vertical deformation, Uy) result with respect to foundation pressure

Settlement [mm]																			
F_d [kPa]		50kPa			100kPa			150kPa			200kPa			250kPa			300kPa		
α_r		28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%
Length [m]	0	10.33	15.51	19.94	15.48	26.06	34.42	24.05	38.01	56.29	39.54	56.37	75.14	57.57	79.78	105.31	76.86	107.65	138.45
	2	9.43	14.91	19.57	14.41	23.35	32.47	17.85	34.79	54.27	27.15	49.15	72.99	39.73	69.69	100.43	65.69	93.90	130.47
	4	7.74	14.26	18.58	13.16	21.87	31.64	16.69	31.73	52.64	26.31	48.55	71.65	32.89	69.24	89.97	55.46	90.78	126.05
	6	6.74	11.78	16.67	8.85	15.58	23.04	12.84	21.79	32.04	13.75	21.95	38.76	23.17	44.17	69.04	35.69	67.46	100.52

Table 4. 4 Settlement reduction factor with respect to foundation pressure

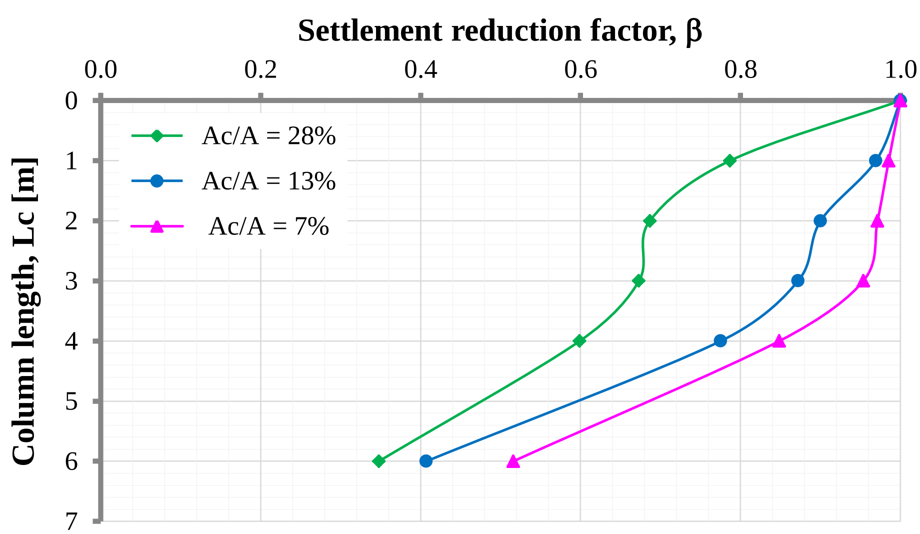
Settlement reduction factor, β																			
F_d [kPa]		50kPa			100kPa			150kPa			200kPa			250kPa			300kPa		
α_r		28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%	28%	13%	7%
Length [m]	0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	0.91	0.96	0.98	0.93	0.90	0.94	0.74	0.92	0.96	0.69	0.87	0.97	0.69	0.87	0.95	0.85	0.87	0.94
	4	0.75	0.92	0.93	0.85	0.84	0.92	0.69	0.83	0.94	0.67	0.86	0.95	0.57	0.87	0.85	0.72	0.84	0.91
	6	0.65	0.76	0.84	0.57	0.60	0.67	0.53	0.57	0.57	0.35	0.39	0.52	0.40	0.55	0.66	0.46	0.63	0.73

4.2.3. Parametric Study

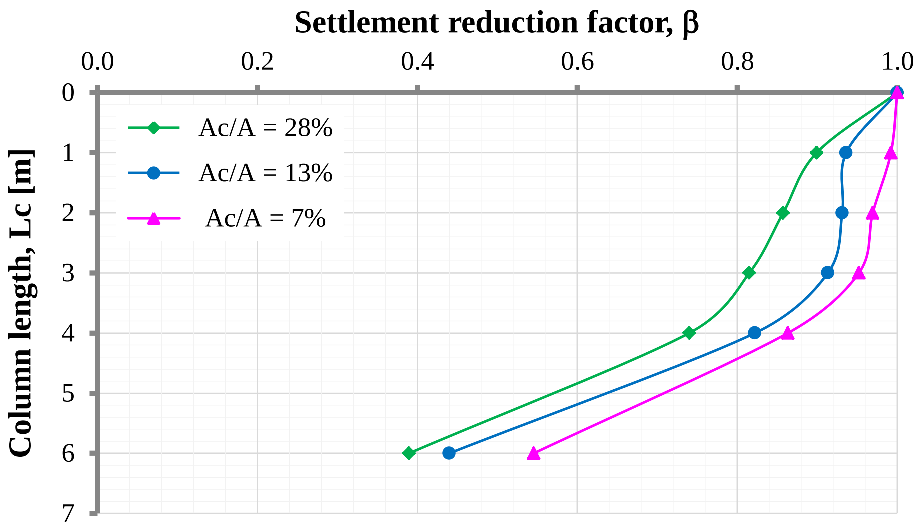
A various configuration of columns were modelled and analsed over a typical range of area ratios to examine the relationship of area ratio, foundation loading and column confinement with increment of column length. The influence these key design parameters has discussed below.

(i) Influence of area ratio and column confinement

As outlined above the extent of settlement performance, which is expressed by settlement reduction factor, of a weak soil is influenced by the amount of stone column installed in the surrounding soil. This effect is examined by the key design factor so called area ratio of stone column. The variation of settlement reduction factor with column length for various area ratios at column configurations of single and 2×2 groups of column is shown in Figure 4.4. The settlement of treated footing was reduced to one-third of untreated one (i.e. from 39.54mm → 13.75mm) for 28% of area ratio. In addition, it was decrease from 56.37mm → 22.95mm and 75.14mm → 38.76mm for α_r of 13% and 7% respectively. It can be seen that the settlement reduction factor decrease continuously with column length. The rate of decreasing in settlement reduction factor for high area ratio is more frequent than that of medium and low area ratio.



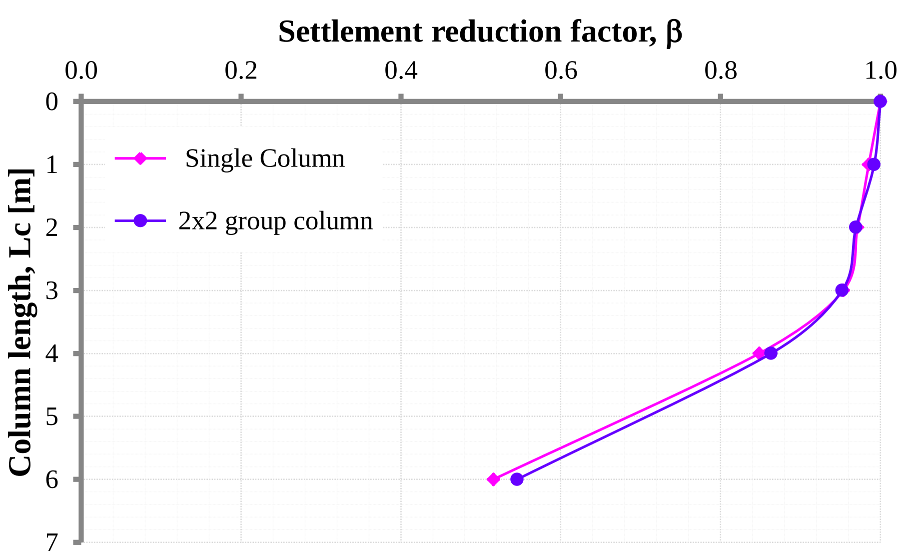
(a)



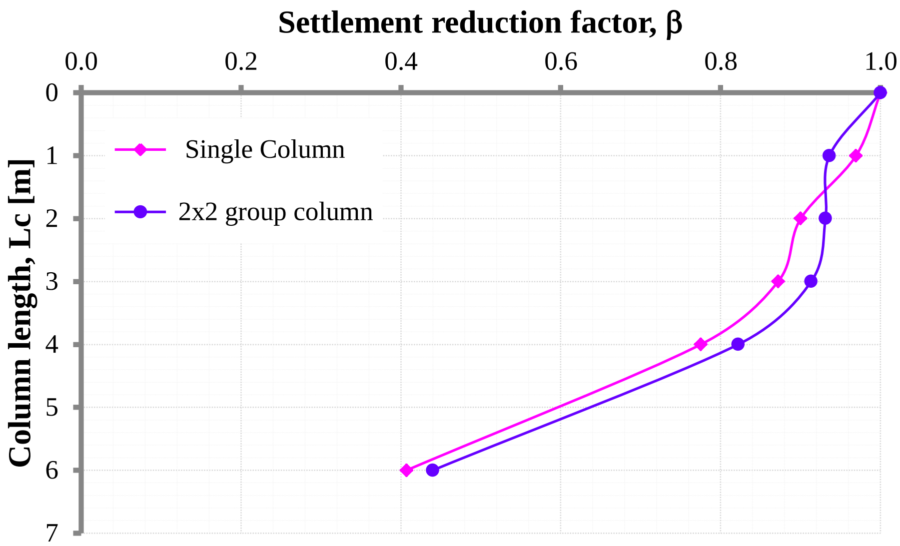
(b)

Figure 4. 4: Variation of settlement reduction factor with column length for (a) single column (b) 2×2 group columns at different area ratios

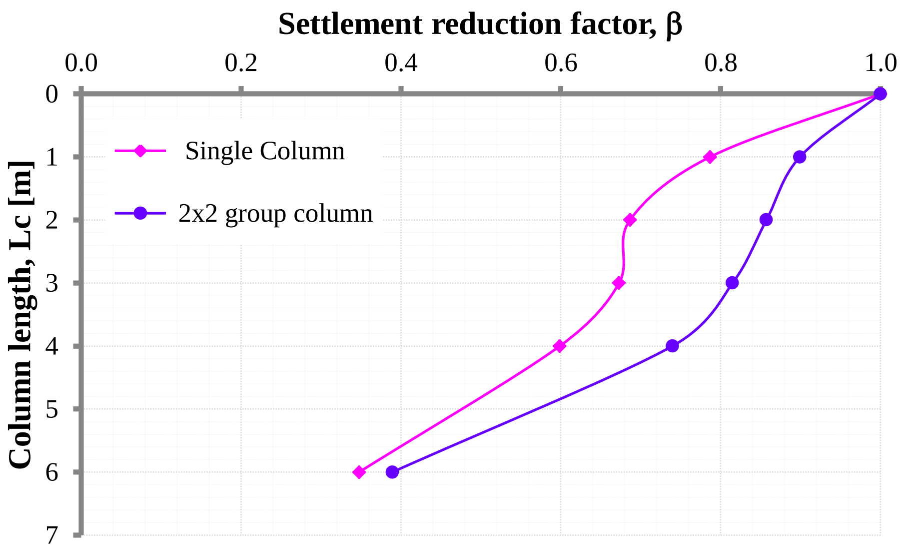
The influence of column confinement up on the settlement performance of stone column is shown in Figure 4.5(a), 4.5(b) and 4.5(c). It is clearly stated that there is comparatively similar rate of change in settlement reduction factor associated with lateral confinement. Hence, it appears that the variation of settlement reduction factor with column length is dictated by area ratio rather than the number of columns.



(a)



(b)



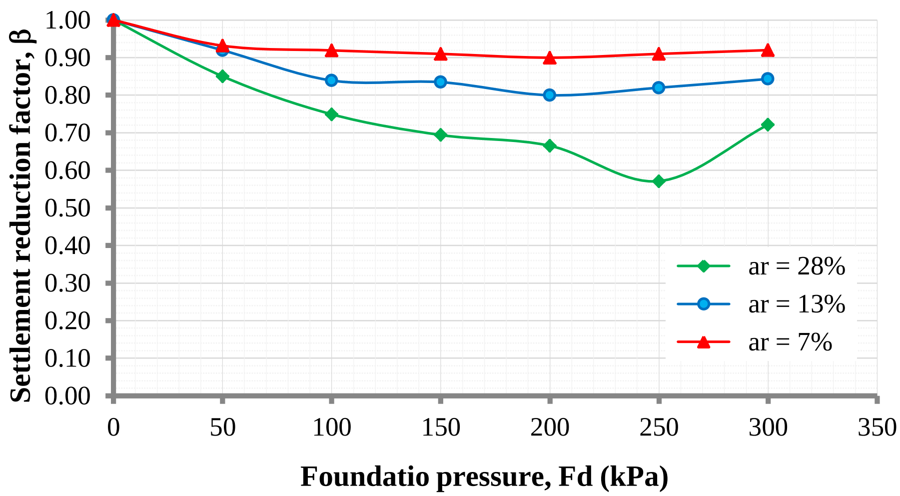
(c)

Figure 4. 5: Variation of settlement reduction factor with column length at area ratios, α_r of (a) 7%, (b) 13% and (c) 28% with respect to column confinement

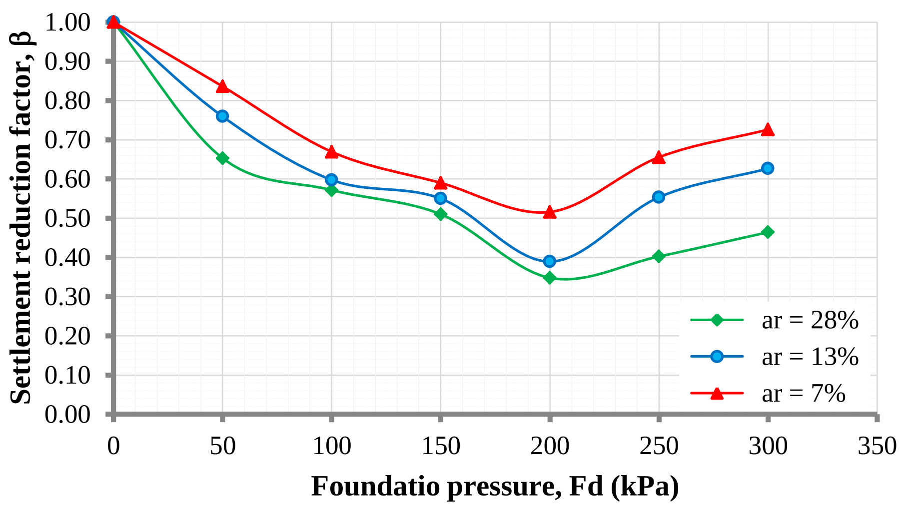
(ii) Influence foundation pressure

It was known that the behavior and magnitude of deformational response are directly proportional to the pressure acted upon the structure. The stone column improved ground has been affected by the surcharge imposed upon it in a given limit.

As shown in Figure 4.6 (i) and (ii) that the influence of structural load on the settlement reduction factor has increasing initially and decrease after critical load. The reason behind this effect is that the implication stone column to improve the weak soil is applicable more efficiently on moderate load. Beyond the moderate load the stone column fails to bear more loads with respect to the surrounding soil. This shows that over reinforcement at higher load has also negative effect on the improvement of weak ground. As described previously that 200kPa is a typical working load for the study site. The critical reduction factor was achieved at this load.



(i)

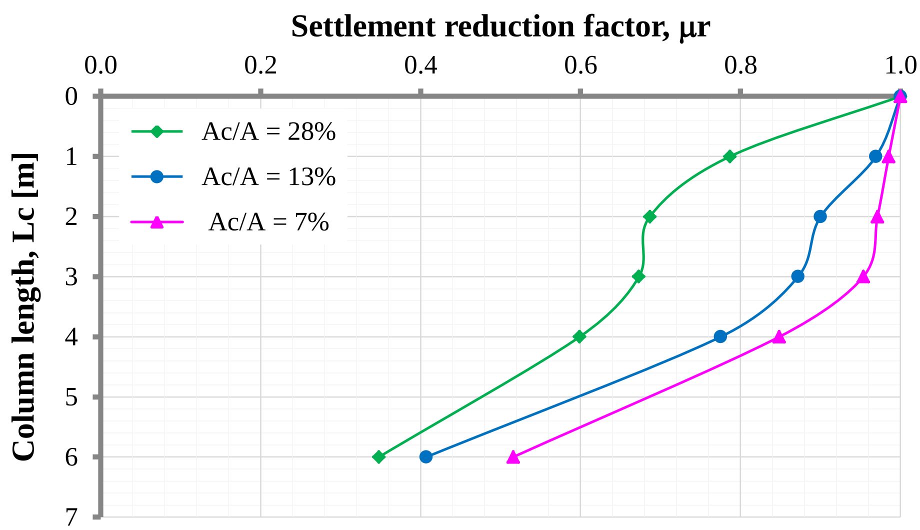


(ii)

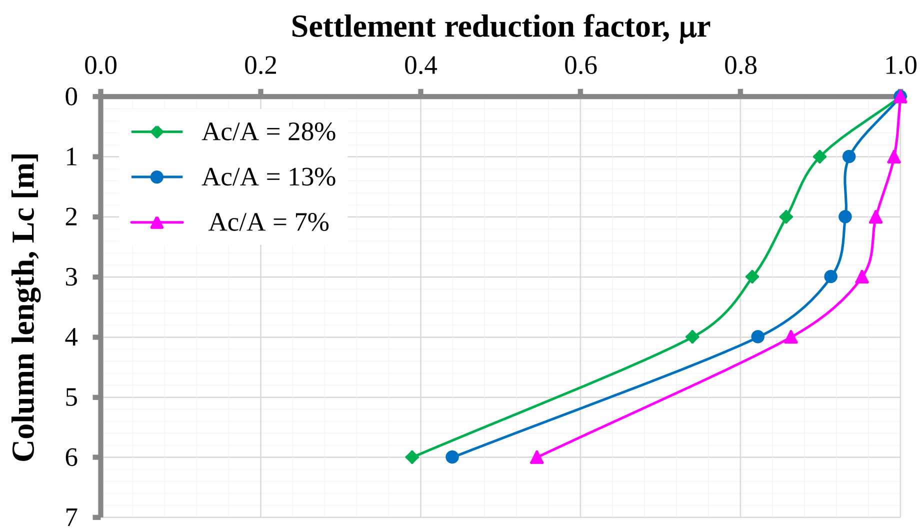
Figure 4. 6: Variation of settlement reduction factor with foundation pressure for (i) floating and (ii) end-bearing column type

(iii) Column length

It is clearly shown in Figure 4.7 (a) and (b) that the settlement reduction factor enhance continuously with column length. A *jump* in settlement reduction factors is observed when moving from floating to end-bearing column (i.e. $L_c = 4 \rightarrow 6$m). The improved settlement performance may be explained as the proportion of applied load transferred to the base of columns is supported by a rigid stratum in the case of end-bearing columns, rather than a compressible stratum in the case of floating columns.



(a)



(b)

Figure 4. 7: Variation of settlement reduction factor with column length for (a) single column (b) 2×2 group columns at different area ratios

It is illustrated in Figure 4.8 that the floating column for high area ratio has acceptable range of settlement reduction factor (i.e. β = 0.6) than that of medium and low are ratios. This suggests that the increase in settlement performance with column length is more pronounced at high area ratio. An end-bearing column, which is supported by a rigid stratum, transferred more load proportion to base and resulted in improved settlement performance. The arrangement of columns can be tailored to achieve a specific settlement performance, i.e. long columns at low area ratios or short columns at high area ratios. It appears that short columns at high area ratios are the most efficient configuration to achieve high settlement performance. Long columns (i.e. end-bearing column) are effective for both high and low area, but the volume of stone required for each configuration of columns necessarily considered for practical efficiency.

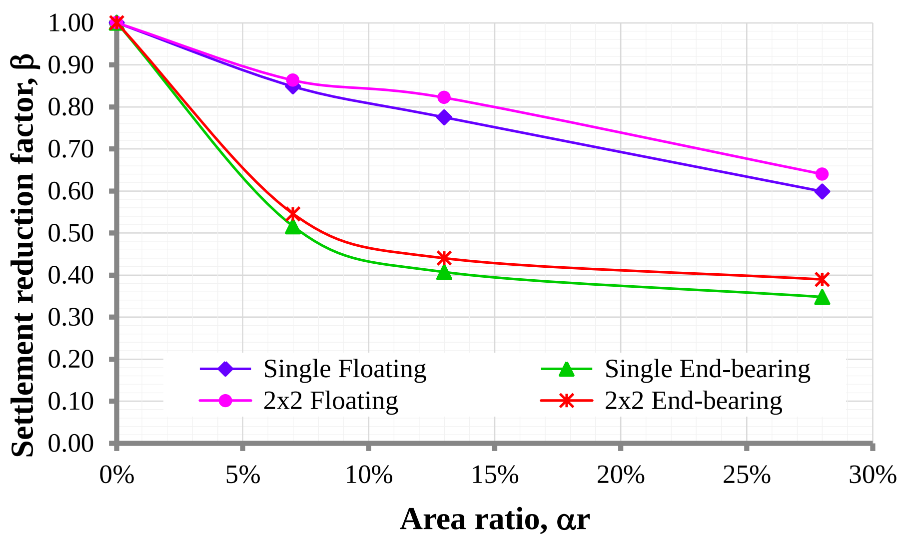


Figure 4. 8 Variation of settlement reduction factor with area ratio for floating and end-bearing columns

4.3. Validation of Results of FEA

The FEM is a powerful investigative tool which has many advantages over laboratory experiments and large scale field tests. However, the FEM is still an approximate technique which idealises real-life situations into a set of continuum components and adopts constitutive models to simulate soil behavior. As a result, it is necessary to validate the output of the FEM to ensure that the real-life situation is accurately modelled.

However, there is no field investigation and measurement held on the study area to validate the results of FEA. Hence, the ability of PLAXIS 3D Foundation to accurately capture the settlement performance of stone columns is validated with the help of existing case studies on stone column performance worldwide that are reported extensively in literature.

As it is stated previous that the extent of settlement performance is quantified by means of settlement reduction factor. Hence, the researcher tried to crosscheck the area ratio versus settlement reduction factor of this study with that of chosen case studies done abroad. The literatures of selected 20 case studies, which are briefly discussed on chapter 2, have used to verify the output of this study is drafted below in Table 4.9. To validate the FEA result with respect to previous practiced case studies, the settlement reduction factor for various area ratios and column confinement is formatted in Table 4.8.

As outlined in Table 4.9 that the area ratio and concurrent settlement factor for various case studies applied on a numerous countries worldwide reflect the effectiveness of stone column on various soil types with different area ratios. The Figure 4.9 demonstrates the extent of area ratio versus settlement reduction factor for different sites.

Table 4. 5: PLAXIS 3D Foundation output of settlement reduction factor with respect to area ratio for current study

Foundation type	**Improved soil type**	**Column type**	α_r	β	**Description**
Strip Footing	Soft clay, Silty clay, Sandy silt (Ash deposit)	End bearing	28%	0.35	αr = 28% single column
				0.39	αr = 28% 2×2 group column
			13%	0.41	αr = 13% single column
				0.44	αr = 13% 2×2 group column
			7%	0.52	αr = 7% single column
				0.55	αr = 7% 2×2 group column

Table 4. 6: Selected case studies worldwide on stone column performance to improve weak soil

Reference	**Study area**	**Foundation type**	**Improved soil type**	**Column type**	**α_r**	**β**
Greenwood (1970)	Bremerhaven, Germany	Embankment	Clay, Peat	End Bearing	25%	0.61
Baumann & Bauer (1974)	Konstanz, Germany	Raft	Silt	End Bearing	47%	0.25
	Quebec City, Canada	Strip Footing	Sand	End Bearing	59%	0.25
Goughnour & Bayuk (1979b)	Hampton, U.S.A.	Embankment	Clay, Silt	End Bearing	34%	0.42
Munfakh et al. (1983)	New Orleans, U.S.A.	Embankment	Clay	End Bearing	25%	0.59
Mitchell & Huber (1985)	California, U.S.A.	Strip Footing	Silt, Clay, Sand	End Bearing	29%	0.65
Greenwood (1991)	East Brent (Somerset), U.K.	Embankment	Clay, Peat	End Bearing	12%	0.60
	Canvey Island, U.K.	Storage tank	Clay	End Bearing	22%	0.44
	Humber Bridge S. Approach, U.K.	Embankment	Clay, Peat	End Bearing	11%	0.77
DeStephen et al. (1997)	New Jersey, U.S.A.	Raft	Fill	Floating	39%	0.50
Raju (1997a)	Kinrara, Malaysia	Embankment	Silt, Fill	End Bearing	35%	0.25
	Kebun, Malaysia	Embankment	Clay	End Bearing	20%	0.40
Cooper & Rose (1999)	Bristol, U.K.	Embankment	Clay, Silt, Peat	End Bearing	15%	0.39
Watts et al. (2000)	Bothkennar, U.K.	Strip Footing	Fill	End Bearing	21%	0.68
Raju et al. (2004)	Kajang, Malaysia	Embankment	Silt, Fill	End Bearing	24%	0.38
Clemente et al. (2005)	U.K.	Strip Footing	Sand	End Bearing	33%	0.29
Serridge & Sarsby (2009)	Bothkennar, U.K.	Strip Footing	Clay	Floating	20%	0.44
Ellouze et al. (2010)	Zarzis, Tunisia	Strip Footing	Sand	End Bearing	32%	0.50
	Damietta, Egypt	Strip Footing	Clay	End Bearing	15%	0.50
Mohamedzein & Al-Shibani (2011)	Omdurman, Sudan	Embankment	Clay	End Bearing	15%	0.53

As clearly depicted in Figure 4.9 that the result of subsequent FEA using PLAXIS 3D Foundation code ranged within extent of previously performed field measurement abroad. Stone column reinforcement with high area ratio (i.e. α_r = 28%) and moderate area ratio (i.e. α_r = 13%) is highly correspondent with the selected case studies. However, in the case of low area ratio (i.e. α_r = 7%) the result displays out of the range with respect to the practically verified limit. This is due to that as dictated earlier the α_r = 7% is the lower boundary of the proportion of stone column amount with in the surrounding soil that is practicable for wide area loadings such as embankment, which can tolerate larger settlements (McCabe *et al.*, 2009). Hence based on findings from Figure 4.9 it can be stated that the computed settlement performance from PLAXIS 3D FEA fall within very much acceptable range. Therefore output of the FEM approximately reflects the real-life situation of domain modelled.

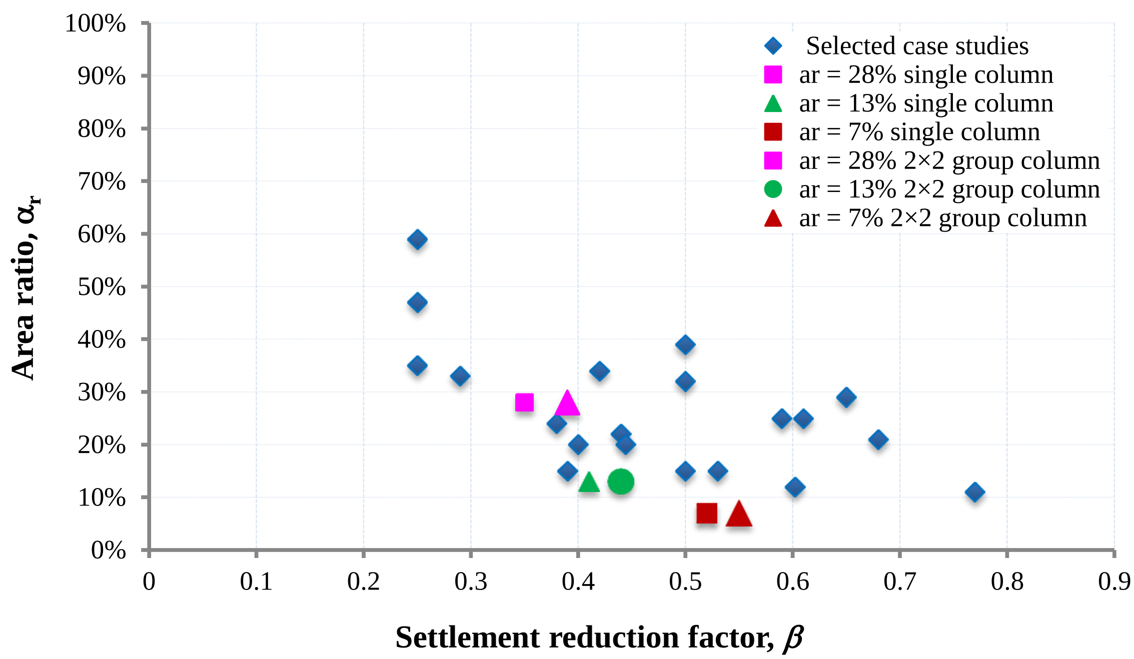


Figure 4. 9: Comparative display of existing case studies performed abroad versus current result of subsequent FEA using PLAXIS 3D Foundation code

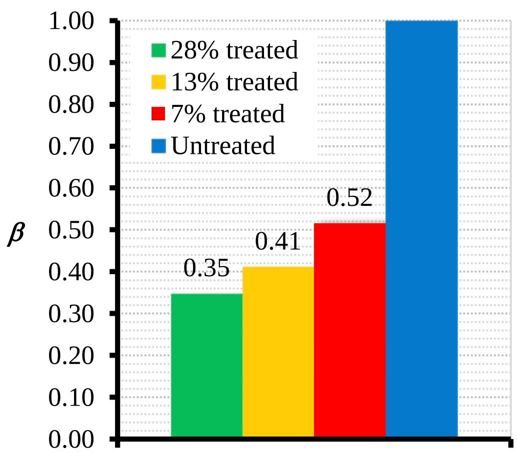
4.4. Discussion

A series of three-dimensional FEA were undertaken to examine the influence of key design parameters upon the settlement performance of stone columns. The stone column simulated in 3D FEA code with different area ratios, column confinement and foundation pressure with column length was analysed. The study mainly focused on the settlement performance of stone column, as the design of foundations on weak soils usually governed by settlement scenarios rather than bearing capacity criteria due to its high compressibility. Hence the settlement performance of stone column at working load levels of upmost importance.

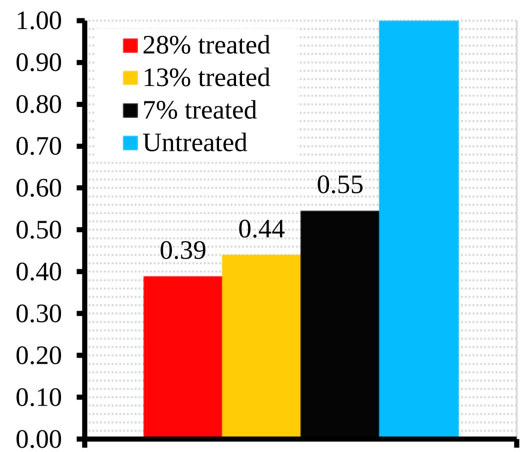
A various arrangement of columns were analysed over a typical range of area ratios to demonstrate the relationship between area ratio, column length and column confinement (i.e. increasing the number of columns). The settlement performance was also measured at 200kPa, which was deemed as a typical working load for the site of study area as per Meyerhof (1976). In addition, the influence of foundation pressure was conducted to come up with critical pressure at which the stone column can bear without tending to fail.

It was found that the area ratio, column length and column confinement all have positive influence upon the settlement performance of stone column.

As clearly illustrated in Figure 4.10 (i) and (ii) that the performance of stone column with respect to area ratio displayed that high area ratio pronounced a best settlement reduction factor than low area ratio. There was comparatively similar rate of change in settlement reduction with lateral confinement. Hence, it appears that the variation of settlement reduction factor with column length is dictated by area ratio rather than the number of columns.



(i)



(ii)

Figure 4. 10 Comparison of settlement reduction factor with respect to area ratios for (i) single column (ii) 2×2 group columns at different area ratios

It was demonstrated in Figure 4.11 (i), (ii) and (iii) that there is a significant effect of foundation pressure on the settlement performance of treated ground. As the stone column was designed to support a moderate load, the typical working load based on the bearing capacity of study site was used for subsequent FEA (i.e. F_d = 200kPa). In addition, the influence of foundation pressure was conducted with foundation pressure F_d of 50kPa -to- 300kPa with constant range of 50kPa based on the typical working load of the study site. In the parametric study section above the variation of settlement performance with foundation loading was analysed. The settlement was continuously reduced with increment of applied pressure up to 200kPa. Then after, it turns back when loaded with higher surcharge. It clarified that the stone column effectiveness when supporting moderately loaded residential, commercial and industrial structures.

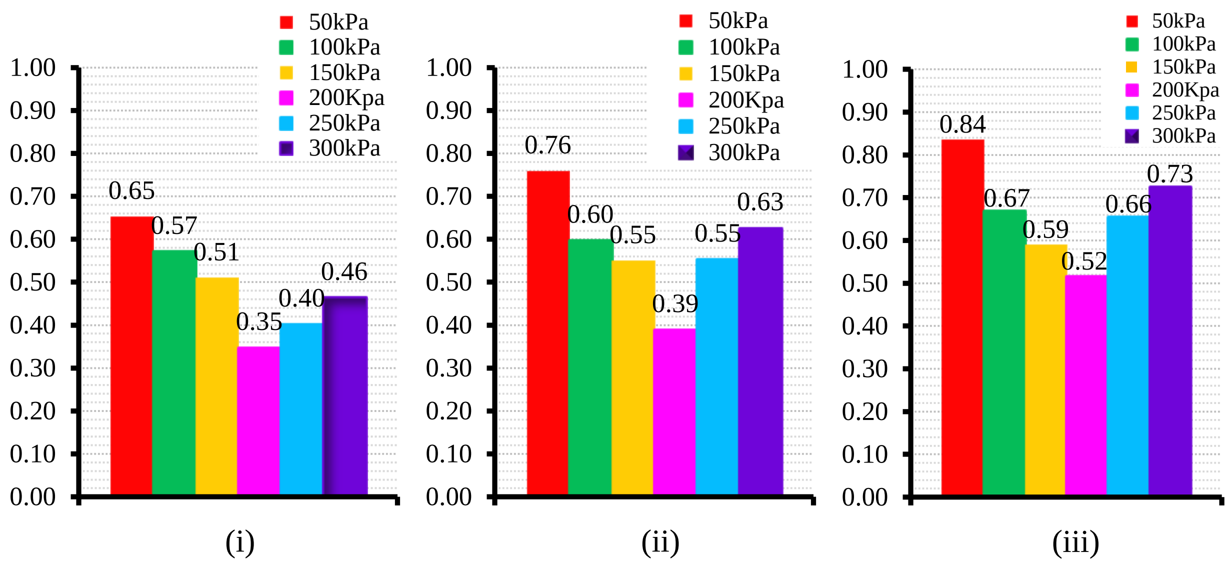


Figure 4. 11: Comparison of settlement reduction factor with respect to foundation pressure for (i) 28%, (ii) 13% and (iii) 7% area ratios

As shown in Figure 4.12 that the column length has direct influence on the deformational performance of stone column improved weak ground. It was discussed that the reduction factor enhance continuously with column length. A *jump* in settlement reduction factor has observed when moving down from floating to end-bearing column (i.e. 4→6m). This implies that the stone column can transfer a proportion of applied load to the rigid stratum at the base in the case of end-bearing columns. The comparatively acceptable reduction factor for floating column was achieve at high area ratio (i.e. β = 0.60 at L_c = 3m for α_r = 28%).

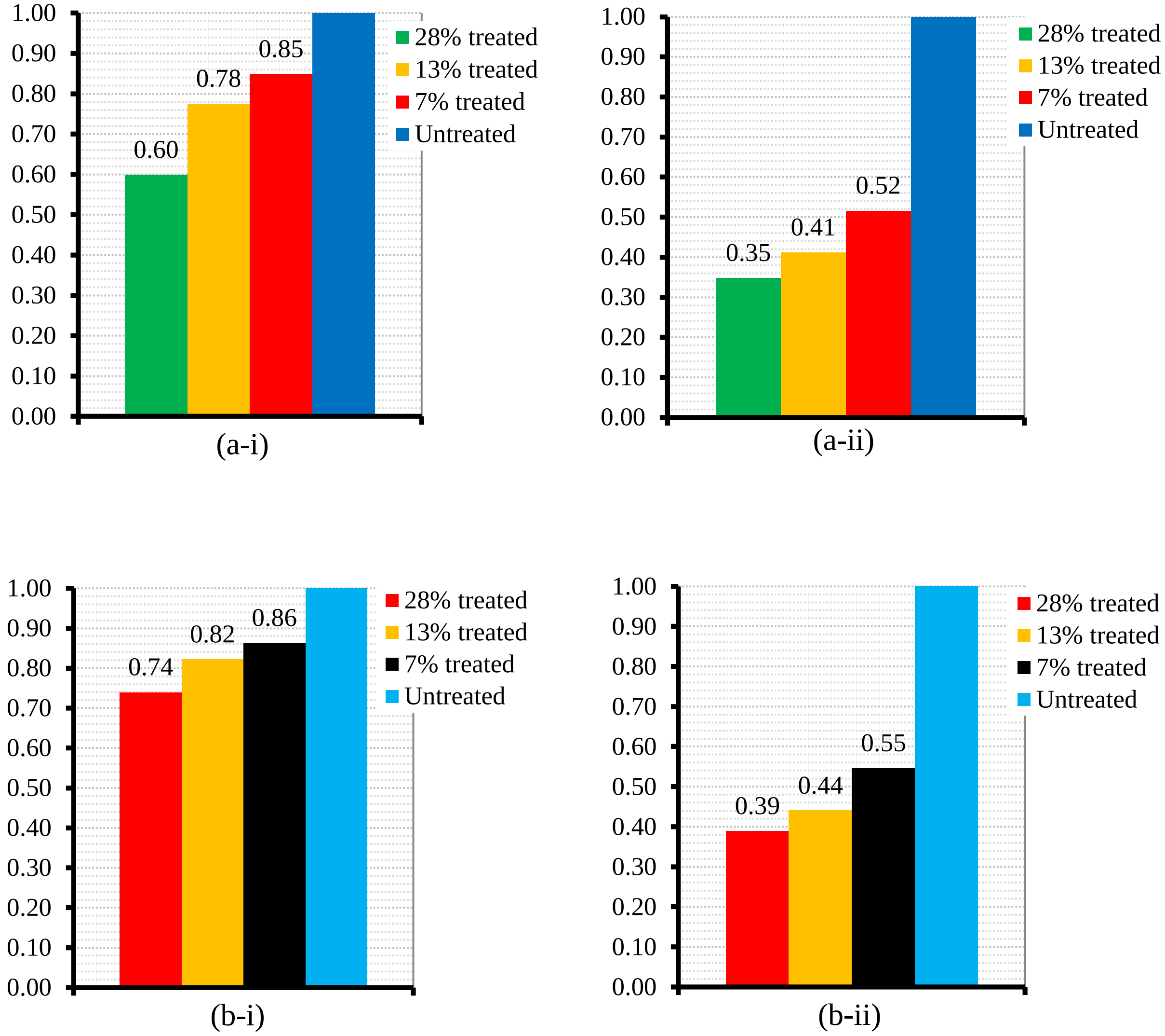


Figure 4. 12: Settlement reduction factor layout of (a) single column (b) 2×2 group columns with respect to column length of (i) floating and (ii) end-bearing column type

As the FEM is an approximate technique, it is necessary to validate the output of PLAXIS 3D FEM code result to ensure the real-life situation of the domain modelled. The result from PLAXIS 3D code was verified with a numerous case studies performed abroad. It was demonstrated that the output obtained from analysis fall with in very much acceptable range, although the column of low area ratio (i.e. α_r = 7%) performed outskirt from the extent of practicable case studies. Thus this low area ratio, as declared above, is applicable only for wide area loading such as embankment, which can tolerate larger settlement. Therefore the output obtained from PLAXIS 3D Foundation v1.1 computed to simulate the settlement performance of stone column installed beneath a rigid footing to reinforce a weak soil is comparatively reflect that the real-life situation is accurately modelled.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

In this study a serious of 15 preliminary checks for sensitivity analyses and 108 subsequent FEA were conducted and the extreme values of vertical displacement, U_y of each model was obtained to figure out the non-dimensional parameter so called settlement reduction factor which reflects the extent of settlement performance of stone column improved rigid footing with respect to that of untreated one. The key design parameters investigated were area ratio, column length, column confinement and foundation pressures that may influence the performance of stone column. A set of material parameters and soil profile adopted for the study site was developed by correlations of SPT N values of field measurement accessed from Addis Geosystems PLC. The study mainly focused on the settlement performance of stone column, as the design of foundations on weak soils usually governed by settlement scenarios rather than bearing capacity criteria due to its high compressibility. A various arrangement of columns were analysed over a typical range of area ratios to demonstrate the relationship between area ratio, column length and column confinement (i.e. increasing the number of columns). The settlement performance was also measured at 200kPa, which was deemed as a typical working load for the site of study area as per Meyerhof (1976). In addition, the influence of foundation pressure was conducted to come up with critical pressure at which the stone can bear without tending to fail.

Based on the study and results of FEA the following conclusions can be drawn:

- It was found that the area ratio, column length and column confinement all have positive influence upon the settlement performance of stone column.
- The performance of stone column with respect to area ratio displayed that high area ratio pronounced a best settlement reduction factor than low area ratio. The soil improved with area ratio of α_r = 28% reduced the settlement to $^1/_3$ of untreated one. The medium and low area ratio comparatively reduced $^1/_2$ of the settlement of weak soil. There was comparatively similar rate of change in settlement reduction with lateral confinement. Hence, it appears that the variation of settlement reduction factor with column length is

dictated by area ratio rather than the number of columns. Based on the result high and mean area ratio is best suited to reinforce rigid footing, while the low area ratio was applicable only for wide area loading such as embankments, which can tolerate larger settlement.

- The influence of foundation pressure was conducted with foundation pressure, F_d of 50kPa → 300kPa with constant range of 50kPa based on the typical working load of the study site. The result demonstrates that stone column performance at 200kPa is better than the other applied pressures. At 200kPa the settlement reduction factor was lowered to 35% rate (i.e. $\beta = 0.35$) for high area ratio. This clarifies the previously stated premise that the stone column is more efficient when supporting moderately loaded residential, commercial and industrial structures.
- The column length have direct influence on the deformational performance of stone column improve weak ground. It was discussed that the reduction factor enhance continuously with column length. A jump in settlement reduction factor has observed when moving down from floating to end-bearing column (i.e. 4→6m). This implies that the stone column can transfer a proportion of applied load to the rigid stratum at the base in the case of end-bearing columns. The comparatively acceptable reduction factor for floating column was achieved at high area ratio (i.e. $\beta = 0.6$ for $\alpha_r = 28\%$). The effective combination of column length to area ratio is that of short column with high area ratio and long column with low area ratio.
- The result from PLAXIS 3D code was verified with a numerous case studies performed abroad. It was demonstrated that the output obtained from analysis fall with in very much acceptable range, although the column of low area ratio (i.e. $\alpha_r = 7\%$) performed outskirt from the extent of practicable case studies.
- Thus and therefore based on the above raised points and verification with previous work done abroad the researcher come up with the conclusion that the performance of stone column can effectively reinforce the weak ground in Ethiopia, specifically in the case of Bishoftu, and added a better understanding on the behavior of stone column beneath a rigid footing.

5.2. Recommendations

A number of issues have been addressed in this study and various useful conclusions are drawn. But there are still some limitations in the analyses and unsolved problems worth further investigation. These limitations are presented here, together with some recommendations for future work.

- The material parameters and soil profile adopted for this study was accessed from secondary data source and correlated and calibrated standards. This may lead to either underestimation or overestimation of engineering properties to be used as input for FEA software code. Thus for future study the researcher has to investigate field measurements and laboratory tests to conduct more accurate analyses.
- This study mainly focused on the long term settlement performance of stone column. In addition to improving the settlement performance, stone column also act as vertical drains and enhance the drainage capacity of soils. However, this and other related point of views has not been investigated here. Thus it is recommended the behavior of stone column regarding drainage performance.
- The effects of some key design parameters have been addressed in detail. But there are still other influential points to be touched especially column installation effects, variability of material stiffness and strength characteristics, and effect of adjacent footing.

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Adama, Ethiopia

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APPENDICES

Appendix A: Data Collection and Calibration

A-1. Secondary data collected

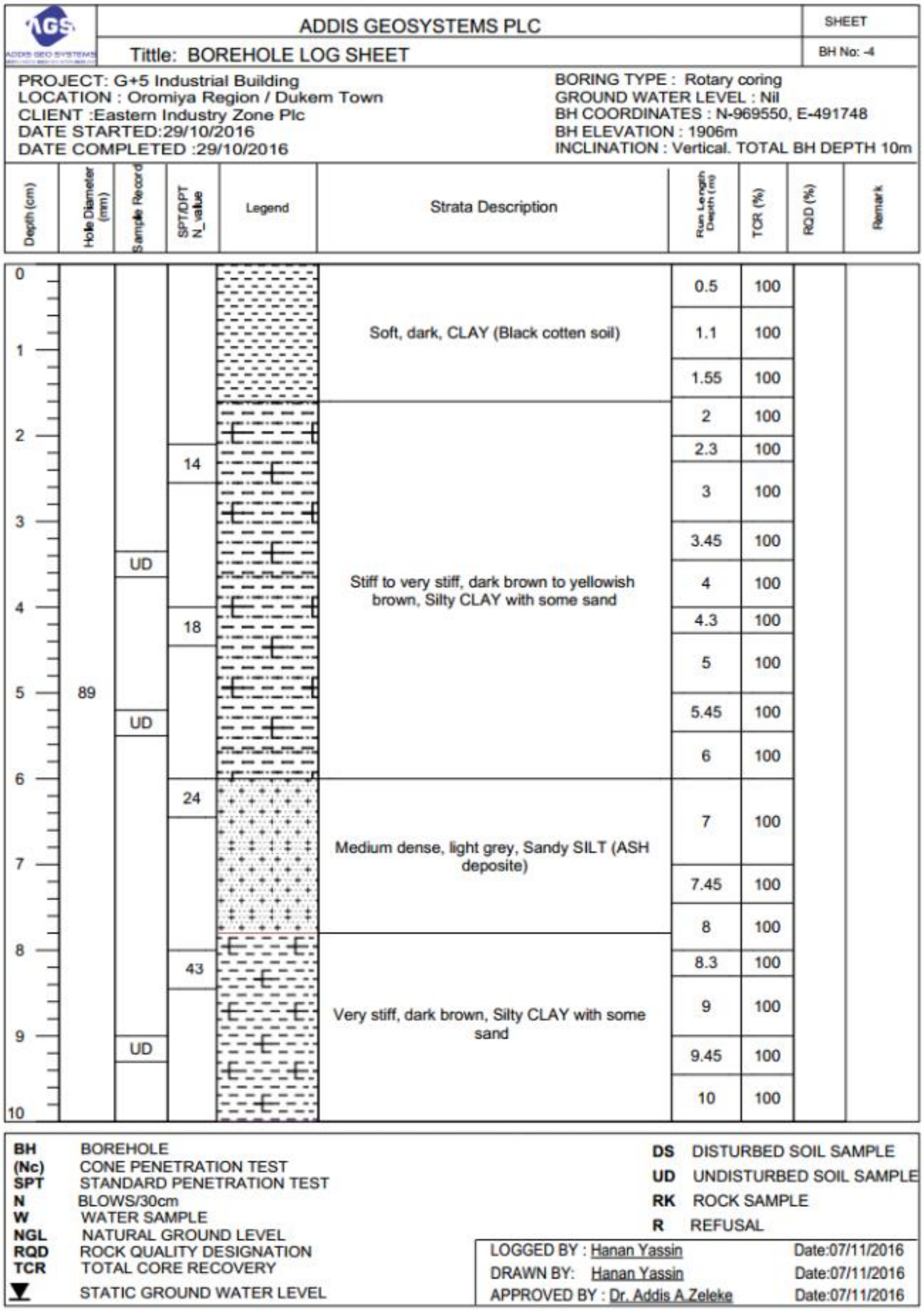

Figure A - 1: Representative bore log data accessed from Addis Geosystems PLC. (2016)

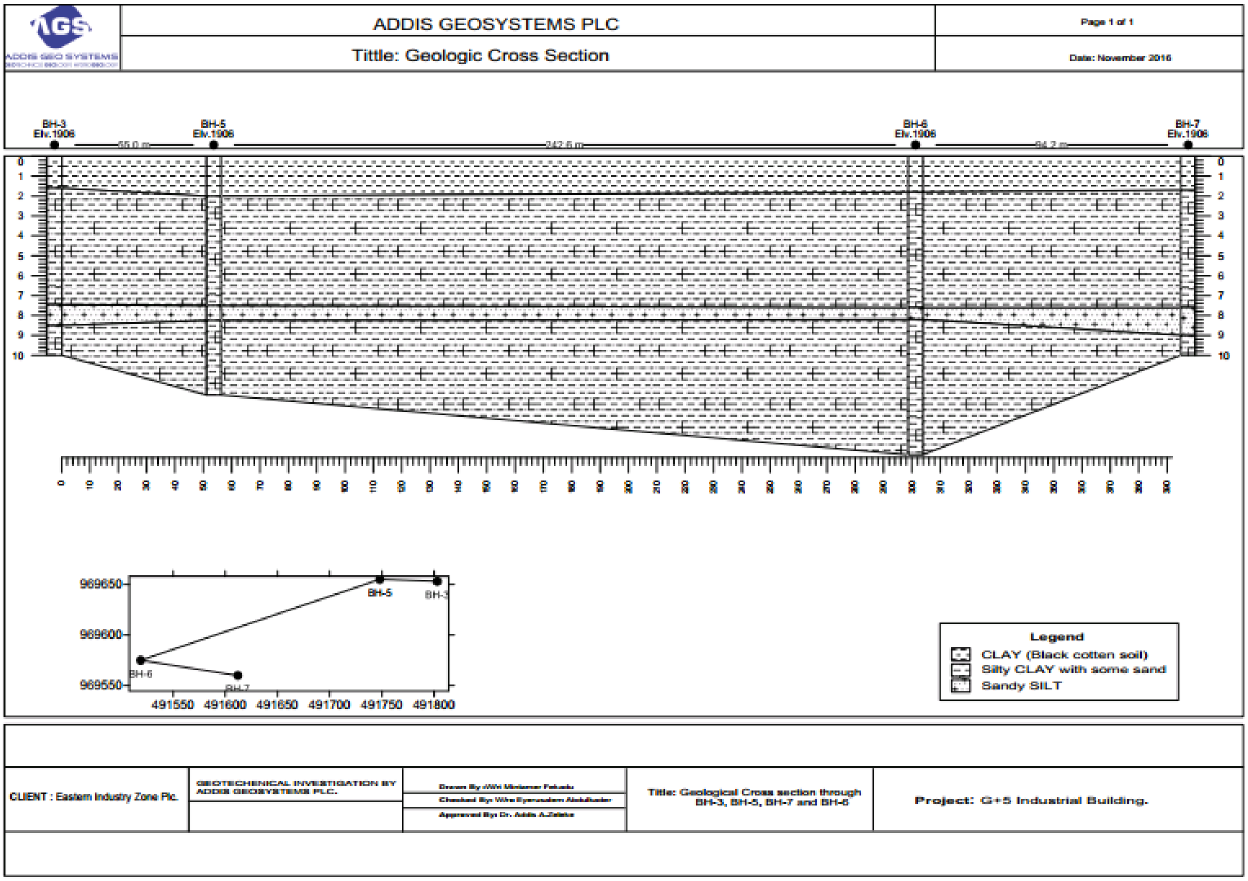

Figure A - 2: Geologic Cross-Section (Addis Geosystems Plc., 2016)

A-2. Data correlation

Table A - 1: Correlation of basic geotechnical properties

Layer	Depth	SPT N	C	SPT N (corrected)	γ_{unsat} (kN/m3)	γ_{sat} (kN/m3)
I	0.00 - 1.60	5*	0.6	3	17.25	17.25
II	1.60 - 3.45	14	0.62	9	18.15	18.15
III	3.45 - 6.00	18	0.66	12	18.60	18.60
IV	6.00 - 7.80	24	0.75	19	19.65	19.65

$\gamma_{sat} = 16.8 + 0.15N_{60} \quad (kN/m^3)$… Michael Carter & Stephen P. Bentley, 1991; Peck *et. al*. 1974

Table A - 2: Correlation of strength Characteristics

Layer	Depth	SPT N	C	SPT N (corrected)	c_u (kN/m2)	c' (kN/m2)	ϕ' (degrees)
I	0.00 - 1.60	5	0.6	3	92.40	9.24	18.48
II	1.60 - 3.45	14	0.62	9	193.91	19.39	24.39
III	3.45 - 6.00	18	0.66	12	232.38	23.24	21.08
IV	6.00 - 7.80	24	0.75	19	294.38	29.44	23.15

$c_u = 29N^{0.72} \quad (kN/m^2)$…………………………… Kulhawy and Mayne (1990)

$c' = 0.1c_u \quad (kN/m^2)$………………………………. Sorensen and Okkels (2013)

Effective cohesion (kPa)	*Friction angle (degrees)*
5–10	10–20
10–20	15–25
20–50	20–30
50–100	25–30

………………….G. Look (2007)

Table A - 3: Correlation of stiffness Characteristics

Layer	Depth	SPT N	C	SPT N (corr.)	K_o	e_o	C_c	C_s	ν_{ur}	E_{oed}^{ref}	E_{50}^{ref}	E_{ur}^{ref}
I	0.00 - 1.60	5	0.6	3	0.68	0.73	0.14	0.04	0.2	2,870	2,870	8,600
II	1.60 - 3.45	14	0.62	9	0.59	0.65	0.11	0.03	0.2	3,340	3,340	10,020
III	3.45 - 6.00	18	0.66	12	0.64	0.63	0.11	0.03	0.2	3,480	3,480	10,430
IV	6.00 - 7.80	24	0.75	19	0.61	0.60	0.10	0.03	0.2	3,670	3,670	11,020

$$K_o^{nc} = 1 - sin\phi'$$

$e_0 = 0.89N^{-0.12}$..Cubrinovski and Ishihara (1999)

$C_c = 0.3(e_o - 0..27)$ …………………………………Rendon-Herrero (1980)

$E_{oed}^{ref} = \frac{2.3(1+e_o)p^{ref}}{C_c}$……………………………….… Brinkgreve & Broere (2006)

$E_{ur}^{ref} = \frac{2.3(1+e_o)(1+v)(1-2v)p^{ref}}{C_c(1-v)} = 3E_{oed}^{ref}$ ……………. Brinkgreve & Broere (2006)

$E_{50}^{ref} = E_{oed}^{ref}$…………………………………………….. Brinkgreve & Broere (2006)

Appendix B: PLAXIS 3D Foundation Input

B-1. Structural footing material parameters

Table B - 1: Material properties of footing (EBCS, 2015)

Parameter	Name	Value	Unit
Material type	Type	Linear Elastic	-
Type of material behavior	-	Non Porous	-
Young's modulus	E	30×10^6	kPa
Unit weight	γ	24	kN/m^3
Poisson's ratio	ν	0.2	-
Thickness	t_f	0.6	m

B-2. Stone column and soil material parameters

Table B - 2: Soil and stone column parameters adopted for all subsequent finite element analysis

Soil Parameter	Stone Column	Layer I	Layer II	Layer III	Layer IV	Unit
Material model	HS	HS	HS	HS	HS	-
Depth	2.00 - 6.00	0.00 - 1.60	1.60 - 3.45	3.45 - 6.00	6.00 - 8.00	m
Type of material behavior	Drained	Drained	Drained	Drained	Drained	-
Bulk unit weight, γ	19.0	17.3	18.2	18.6	19.7	kN/m^3
Saturated unit weight, γ_{sat}	21.0	17.3	18.2	18.6	19.7	kN/m^3
Over-consolidation ratio, OCR	-	1	1	1	1	-
Permeability, K_h	1.7	8.6×10^{-5}	8.6×10^{-4}	8.6×10^{-4}	8.6×10^{-4}	m/day
Permeability, K_v	1..7	5.8E-05	7.2E-04	7.2E-04	7.2E-04	m/day
Poisson's ratio, ν	0.2	0.2	0.2	0.2	0.2	-
Cohesion, c'_{ref}	1	9	19	23	29	kN/m^2
Friction angle, ϕ'	45	18	24	21	23	º
Dilatancy angle, ψ	15	0	0	0	0	º
Initial voids ratio, e_o	0.5	0.73	0.65	0.63	0.60	-
Compression index, C_C	-	0.14	0.11	0.11	0.10	-
Swelling index, C_S	-	0.04	0.03	0.03	0.03	-
Reference pressure, pref	100	100	100	100	100	kN/m^2

Lateral earth coefficient, K_0^{nc}	1.00	0.68	0.59	0.64	0.60	-
M	0.3	1.0	1.0	1.0	1.0	-
E_{50}^{ref}	70,000	2,866	3,340	3,478	3,675	kN/m 2
E_{oed}^{ref}	70,000	2,866	3,340	3,478	3,675	kN/m 2
E_{ur}^{ref}	210,000	8,599	10,019	10,433	11,024	kN/m 2

Appendix C: PLAXIS 3D Foundation Output

C-1. Area ratio and column confinement

Table C - 1: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to area ratio of 28%

Area ratio α_r	Column Configuration	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
28%	Single	1	0	N/A	39.54	1.00
			1	Floating	31.11	0.79
			2	Floating	27.15	0.69
			3	Floating	26.61	0.67
			4	Floating	26.31	0.67
			6	End-bearing	13.75	0.35
	2×2	2	0	N/A	76.44	1.00
			1	Floating	68.74	0.90
			2	Floating	65.54	0.86
			3	Floating	62.30	0.81
			4	Floating	56.56	0.74
			6	End-bearing	29.78	0.39

Table C - 2: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to area ratio of 13%

Area ratio α_r	Column Configuration	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
13%	Single	1.5	0	N/A	56.37	1.00
			1	Floating	54.64	0.97
			2	Floating	50.74	0.90
			3	Floating	49.15	0.87
			4	Floating	48.55	0.86
			6	End-bearing	23.20	0.41
	2×2	3	0	N/A	117.60	1.00
			1	Floating	114.50	0.97
			2	Floating	113.90	0.97
			3	Floating	111.70	0.95
			4	Floating	105.84	0.90
			6	End-bearing	51.82	0.44

Table C - 3: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to area ratio of 7%

Area ratio, α_r	Column Configuration	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
7%	Single	2	0	N/A	75.14	1.00
			1	Floating	74.04	0.99
			2	Floating	72.99	0.97
			3	Floating	71.65	0.95
			4	Floating	63.76	0.85
			6	End-bearing	38.76	0.52
	2×2	4	0	N/A	161.71	1.00
			1	Floating	161.42	1.00
			2	Floating	157.72	0.98
			3	Floating	154.90	0.96
			4	Floating	140.45	0.87
			6	End-bearing	88.38	0.55

C-2. Foundation Load

Table C - 4: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 50kPa

Foundation Pressure	Area ratio	Footing width [m]	Length [m]	Column type	Se [mm]	β
50kN/m2	28%	1	0	N/A	10.33	1.00
			2	Floating	9.43	0.91
			4	Floating	7.74	0.75
			6	End-bearing	6.74	0.65
	13%	1.5	0	N/A	15.51	1.00
			2	Floating	14.91	0.96
			4	Floating	14.26	0.92
			6	End-bearing	11.78	0.76
	7%	2	0	Floating	19.94	1.00
			2	Floating	19.57	0.98
			4	Floating	18.58	0.93
			6	End-bearing	16.67	0.84

Table C - 5 Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 100kPa

Founndation Pressure	Area ratio	Footing width [m]	Length [m]	Column type	Se [mm]	β
100kN/m2	28%	1	0	N/A	15.48	1.00
			2	Floating	14.41	0.93
			4	Floating	13.16	0.85
			6	End-bearing	8.85	0.57
	13%	1.5	0	N/A	26.06	1.00
			2	Floating	23.35	0.90
			4	Floating	21.87	0.84
			6	End-bearing	15.58	0.60
	7%	2	0	Floating	34.42	1.00
			2	Floating	32.47	0.94
			4	Floating	31.64	0.92
			6	End-bearing	23.04	0.67

Table C - 6 Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 150kPa

Foundation Pressure	Area ratio, α_r	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
150kPa	28%	1	0	N/A	24.05	1.00
			2	Floating	17.85	0.74
			4	Floating	16.69	0.69
			6	End-bearing	12.84	0.53
	13%	1.5	0	N/A	38.01	1.00
			2	Floating	34.79	0.92
			4	Floating	31.73	0.83
			6	End-bearing	21.79	0.57
	7%	2	0	N/A	52.64	1.00
			2	Floating	56.29	1.07
			4	Floating	54.27	1.03
			6	End-bearing	32.04	0.61

Table C - 7: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 200 kPa

Foundation Pressure	Area ratio, α_r	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
200kPa	28%	1	0	N/A	39.54	1.00
			2	Floating	26.31	0.67
			4	Floating	27.15	0.69
			6	End-bearing	13.75	0.35
	13%	1.5	0	N/A	56.37	1.00
			2	Floating	48.55	0.86
			4	Floating	49.15	0.87
			6	End-bearing	21.95	0.39
	7%	2	0	N/A	75.14	1.00
			2	Floating	71.65	0.95
			4	Floating	72.99	0.97
			6	End-bearing	38.76	0.52

Table C - 8: Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 250kPa

Foundation Pressure	Area ratio, α_r	Footing width [m]	Column Length [m]	Column type	S_e [mm]	β
250kPa	28%	1	0	N/A	79.78	1.00
			2	Floating	69.69	0.87
			4	Floating	69.24	0.87
			6	End-bearing	31.71	0.40
	13%	1.5	0	N/A	57.57	1.00
			2	Floating	39.73	0.69
			4	Floating	32.89	0.57
			6	End-bearing	31.71	0.55
	7%	2	0	N/A	105.31	1.00
			2	Floating	100.43	0.95
			4	Floating	89.97	0.85
			6	End-bearing	69.04	0.66

Table C - 9 Settlement (extreme vertical deformation, Uy) and reduction factor with respect to foundation pressure of 300kPa

Founndation Pressure	Area ratio	Footing width [m]	Length [m]	Column type	Se [mm]	β
300kN/m2	28%	1	0	N/A	76.86	1.00
			2	Floating	65.69	0.85
			4	Floating	55.46	0.72
			6	End-bearing	35.69	0.46
	13%	1.5	0	N/A	107.65	1.00
			2	Floating	93.90	0.87
			4	Floating	90.78	0.84
			6	End-bearing	67.46	0.63
	7%	2	0	N/A	138.45	1.00
			2	Floating	130.47	0.94
			4	Floating	126.05	0.91
			6	End-bearing	100.52	0.73