Behaviour of Single and Group of Geotextile Encased Stone Columns in Single-Layered and Layered soil

Rowad Esameldin Farah

Submitted to the
Institute of Graduate Studies and Research
in partial fulfillment of the requirements for the degree of

Doctor of Philosophy
in
Civil Engineering

Eastern Mediterranean University
July 2020
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Ali Hakan Ulusoy
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Doctor of Philosophy in Civil Engineering.

Prof. Dr. Umut Türker
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Doctor of Philosophy in Civil Engineering.

Prof. Dr. Zalihe Nalbantoğlu Sezai
Supervisor

Examining Committee

1. Prof. Dr. Yeliz Yükselen Aksoy
2. Prof. Dr. Huriye Bilsel
3. Prof. Dr. Zalihe Nalbantoğlu Sezai
4. Prof. Dr. Abdulazim Yıldız
5. Asst. Prof. Dr. Eriş Uygar

ABSTRACT

Stone columns (SC) have become a widely-utilized technique of enhancing the bearing capacity of soft soils. Although the SCs in single-layered soil have been studied extensively, SCs constructed in a ground consisting of varying soil layers are not fully understood. In this study, to investigate the behavior of single and group of SCs in different soil layering systems, small scale laboratory tests were conducted on non-encased (NEC) and encased (EC) floating and end bearing SCs installed in single-layered and layered soil consisting of loose sand overlying the soft soil. Different area replacement ratios (ARRs), which is the SC area to unit cell area ratio, 1.56% and 6.25% and the length to diameter ratios (L/D) of 5, 7.5, 10 and 15 were selected to evaluate the effect of these factors on the vertical stress-settlement behavior, bearing improvement ratio, subgrade modulus and bulging failure of NEC and EC-SCs in single-layered and layered soils. Test results indicated that bearing capacity of single-layered and layered soils was improved in all cases of single SC applications. In both soil layering systems, the single NEC floating SC (FSC) with smaller area replacement ratio (1.56%) and L/D of 10 gave much better results than all other NEC-SCs applications. The inclusion of geotextile encasement resulted in further improvement of both soil layering systems. The single encased FSC with smaller ARR (1.56%) had superior improvement among all single SC applications. However, in both soil layering systems, in case of single EC end bearing SC (EBSC) with higher ARR of 6.25% and L/D of 7.5, reduction in bearing capacity and subgrade modulus were obtained. For single SC applications, in single-layered soft soil, with smaller ARR of 1.56%, the inclusion of geotextile encasement resulted in a reduction in bulging diameter and increased the bulging depth of FSC which resulted

in an increase in SC bearing capacity. While for single SC applications, in single-layered soft soil, in case of encased floating and end bearing SCs with higher ARR of 6.25 % there was no bulging confronted. The bearing capacity and subgrade modulus of non-encased central column among group of SCs in both soil layering systems, were higher than the other SCs in the group. In addition to these, the bearing capacity and subgrade modulus of soils were significantly improved with reducing the spacing to diameter ratio of SCs. Moreover, the geotextile encasement provided an additional improvement to the SCs. In single-layered soil, non-encased floating central SC with spacing to diameter ratio of 2.5, bulging failure occurred and with decreasing the spacing to diameter ratio to 1.5, the bulging in the central SC was reduced. Whereas, with the geotextile encased floating group of SCs in single-layered soil, with smaller spacing to diameter ratio of 1.5, no bulging failure happened.

Keywords: Area replacement ratio, bulging, end bearing stone column, floating stone column, geotextile, layered soil, single-layered soil, soft soil, stone columns.

ÖZ

Taş kolonlar (SC), yumuşak toprakların taşıma kapasitesini arttırmak için yaygın olarak kullanılan bir teknik haline gelmiştir. Tek katmanlı topraklardaki SC'ler kapsamlı bir şekilde incelenmiş olsa da, değişen toprak katmanlarından oluşan bir zeminde inşa edilen SC'ler tam olarak anlaşılamamıştır. Bu çalışmada, tek ve gurup SC'lerin farklı toprak katmanlı sistemlerindeki davranışlarını araştırmak için, yumuşak toprağın üzerinde gevşek kumdan oluşan tek katmanlı ve katmanlı olmak üzere, sarmalanmamış (NEC) ve sarmalanmış (EC) yüzer ve uç taşımalı SC'lerde küçük ölçekli laboratuvar testleri yapılmıştır. SC alanı - birim hücre alanı oranı olan farklı alan değiştirme oranları (ARR'ler), % 1.56 ve % 6.25 ve 5, 7.5, 10 ve 15'in uzunluk/çap oranları (L/D) seçilerek, bu faktörlerin NEC ve EC-SC'lerin tek katmanlı ve katmanlı zeminlerde düşey gerilme-oturma davranışı, taşıma iyileştirme oranı, alt modülüs ve şişkinlik göçmesi üzerindeki etkisi değerlendirilmiştir. Test sonuçları, tek katmanlı ve katmanlı zeminlerin taşıma kapasitesinin tüm tekli SC uygulamalarında iyileştirildiğini göstermiştir. Her iki toprak katmanlama sisteminde, daha küçük alan değiştirme oranına (% 1.56) ve 10 L / D'ye sahip tek NEC yüzen SC (FSC), diğer tüm NEC-SCs uygulamalarından çok daha iyi sonuçlar verdi. Jeotekstil kaplamanın dahil edilmesi, her iki zemin katmanlama sisteminin daha da iyileştirilmesine neden olmuştur. Daha küçük ARR'ye sahip tekli FSC (% 1.56), tüm tek SC uygulamaları arasında üstün iyileşme gösterdi. Bununla birlikte, her iki zemin katmanlama sisteminde,% 6.25 daha yüksek ARR'ye ve 7.5 / L / D'ye sahip tek EC uç taşımalı SC (EBSC) durumunda, taşıma kapasitesinde ve alt modül modülünde azalma elde edilmiştir. Tek katmanlı uygulamalar için, tek katmanlı yumuşak toprakta,% 1.56 daha küçük ARR ile, jeotekstil kaplamanın eklenmesi şişkinlik

çapının azalmasına ve FSC'nin şişirme derinliğinin artmasına neden olarak SC taşıma kapasitesinde bir artışa neden olmuştur. Tek SC uygulamaları için, tek katmanlı yumuşak toprakta,% 6.25 daha yüksek ARR değerine sahip kapalı yüzer ve uç taşımalı SC'ler durumunda, herhangi bir şişkinlik ile karşılaşılmadı. Tek katmanlı uygulamalar için, tek katmanlı yumuşak toprakta,% 1.56 daha küçük ARR ile, jeotekstil kaplamanın eklenmesi şişkinlik çapının azalmasına ve FSC'nin şişirme derinliğinin artmasına neden olarak SC taşıma kapasitesinde bir artışa neden olmuştur. Tek SC uygulamaları için, tek katmanlı yumuşak zeminde,% 6.25 daha yüksek ARR değerine sahip kapalı yüzer ve uç taşımalı SC'ler durumunda, herhangi bir şişkinlik ile karşılaşılmadı. Her iki toprak katmanlama sistemindeki SC grupları arasında çevrili olmayan merkezi kolonun taşıma kapasitesi ve alt zemin modülü, gruptaki diğer SC'lerden daha yüksekti. Bunlara ek olarak, zeminlerin taşıma kapasitesi ve alt zemin modülü de SC'lerin boşluk / çap oranının azaltılmasıyla önemli ölçüde geliştirildi. Ayrıca, jeotekstil kaplama SC'lere ilave bir gelişme sağlamıştır. Tek katmanlı toprakta, boşluk çapı 2.5 olan, kabarma göçmesi meydana gelen ve boşluk / çap oranını 1.5'e düşüren, sarmalanmış olmayan yüzer merkezi SC'deki şişkinlik azaltıldı. Oysa, jeotekstil tek katmanlı zeminde yüzen küçük SC grubu ile, daha küçük boşluk / çap oranı 1.5 olan, şişkinlik göçmesi meydana gelmedi.

Anahtar kelimeler: Alan değiştirme oranı, şişkinlik, uç taşımalı taş kolon, yüzer taş kolon, jeotekstil, katmanlı zemin, tek katmanlı zemin, yumuşak zemin, taş kolonlar.

I dedicate this thesis for my small and big families in Sudan and North Cyprus (Yousra Osman, Farah Rowad, my parents and siblings).

ACKNOWLEDGEMENT

My respects and appreciations for all of you.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS AND ABBREVATIONS

A	Unit cell area
ARRs	Area replacement ratios
A_c	Stone column area
A_s	Stone column tributary area of soil
BIR	Bearing improvement ratio
c	Cohesion of the surrounding soil
C_c	Compression index
C_c	Coefficient of curvature
C_r	Rebound index
c_u	Undrained cohesion
C_u	Uniformity coefficient
D	Diameter of stone column
D_0	Original diameter of the stone column
D_b	Bulging diameter of stone column
d_e	Equivalent diameter of the unit cell
d_p	Prototype stone column diameter
d_s	Particle size of the aggregates
D_{50}	Mean Diameter
E	Elastic modulus of soil
EBSC	End bearing stone column
EC	Encased
F_c	Unit less cavity expansion factors
FSC	Floating stone column

F_q	Unit less cavity expansion factors
G	Shear module of the reinforced soil
Gs	Specific gravity
I_r	Rigidity index
k	Subgrade modulus
L	Stone column length
L_c	Critical length
L/D	Length to diameter ratio
LL	Liquid limit
LVDTs	Linear variable differential transformers
M	Ratio of depth to column diameter
n	Stress concentration ratio
NEC	Non-encased
N_p	Bearing capacity factor
q_r	Vertical stress of reinforced soil at a given settlement
q_u	Vertical stress of unreinforced soil at the same settlement
q_{ult}	Ultimate bearing capacity
P'	Average effective stress at the failure depth
p_c	Additional confinement provided by encasement material
PI	Plasticity index
PL	Plastic limit
SC	Stone column
s	Spacing of the columns
S_r	Settlement of reinforced soil
S_u	Settlement of the unreinforced soil

T	Tensile strength of the encasement material
u	Pore water pressure
UCS	Unconfined compressive strength
USCS	Unified Soil Classification System
ν	Poisson's ratio of soil
Z	Total length of the stone column
σ_c	Average vertical stress on a stone column
σ_s	Average vertical stress applied on the surrounding soil
μ_c	Ratio of stresses in the stone column
μ_s	Ratio of stresses in the surrounding soil
σ_{vz}	Vertical stress at a depth
σ_v	Ultimate vertical stress of stone column
ρ	Density of the stone column material
ϕ°	Internal friction angle
σ_{r0}	Initial radial total stress
σ_3	Ultimate cavity resistance
σ_{rl}	Ultimate lateral stress of soil
ρ	In-situ bulk density
$\rho_{d(max)}$	Maximum dry density
$\rho_{d(min)}$	Minimum dry density

Chapter 1

INTRODUCTION

1.1 Aim of the study

The structures built on problematic soils encountered several problems such as total and differential settlements, liquefaction, long-term stability and durability (McKelvey et al., 2004; Murugesan and Rajagopal, 2009; Shivashankar et al., 2010; Mohanty and Samanta, 2015; Chenari et al., 2017). Among several techniques used for ground improvement, the stone column (SC) application is a widely used technique due to being the versatile and cost-effective ground improvement technique (Hughes and Withers, 1974; Alamgir et al., 1996; Madhav, 2000; Andreou et al., 2008; Babu et al., 2013; Prasad et al., 2017; Mehrannia et al., 2018). SCs bearing capacity is dependent on the confinement of the surrounding soils. However, SCs in very soft soils experience extreme bulging due to inadequate lateral confinement of the surrounding soils. Especially the soil confinement provided by soft soils with an undrained cohesion around 15 kPa or less (Alexiew et al., 2005) may not be sufficient to develop the required bearing capacity of the SCs (McKenna et al., 1975; Alexiew et al., 2005; Murugesan and Rajagopal, 2009). The confinement of the SC can be enriched by encasing the SC with high-modulus geosynthetic materials, such as geotextiles (Raithel and Kempfert, 2000). The increase in the confinement of the SC is directly proportional to bearing capacity improvement of the SC and the reduction in the lateral bulging (Debnath and Dey, 2017).

For the SC enhancement of weak soils where the depth of the hard layers lies at deeper layers, the choice of floating SCs (FSC) may be one of the best options (Dash and Bora, 2013). Literature review indicated that most of the conducted researches on the SCs performance were mostly on the condition of end bearing SCs, EBSCs, (McCabe et al., 2009; Chenari et al., 2017; Ng, 2017). There are very limited investigations performed on the vertical stress-settlement behavior of FSCs (McCabe et al., 2009; Chenari et al., 2017; Ng, 2017). Hence, in this study, small scale laboratory tests have been conducted to examine the performance of both FSCs and EBSCs with and without geotextile encasement in single-layered and layered soils.

Studies in the literature (Murugesan and Rajagopal, 2009; Shivashankar et al., 2010; Dash and Bora, 2013) have examined the SCs performance in the settlement and bearing capacity of soft single-layered soils. There have been very limited researches conducted on the SCs behaviour in layered soils (Shivashankar et al., 2011; Das and Pal, 2013; Mohanty and Samanta, 2015; Prasad et al., 2017). However, in reality, footings are usually set in multi-layered soils and comprehending the SCs behaviour in layered soils is very important for civil engineers (Barksdale and Bachus, 1983; Shivashankar et al., 2011; Killeen, 2012; Das and Pal, 2013; Mohanty and Samanta, 2015). In the literature, the condition of layered soils was generally considered to be a soft soil overlaying a very soft soil, each of varying thicknesses (Shivashankar et al., 2011; Mohanty and Samanta, 2015; Prasad et al., 2017). Some studies considered the SCs performance in layered soils comprising of soft soils overlaying a stronger silty soil (Shivashankar et al., 2011; Prasad et al., 2017). After an extensive literature review, it was noticed that the performance of SCs in loose sand overlaying a soft soil has not been considered. Therefore, in this study, the behaviour of SCs in layered soil consisting of loose sand overlaying a soft soil was studied in a small-scale laboratory model test tank with the single and group of SCs.

In the present study, physical and engineering property tests have been conducted on soft soil, sandy soils and crushed stone aggregates. After soil samples preparation in the model test tank was completed, load-settlement tests on non-reinforced and reinforced soils with single and group of SCs were conducted. The effect of area replacement ratios (ARRs) on the ultimate vertical stress of non-encased and encased SCs was documented in the literature (Murugesan and Rajagopal, 2009; Tandel et al., 2017). The effect of ARR value on the behaviour of the SC was considered alone and it has been observed that an increase in ARR value was directly proportional in improving SC bearing capacity (Murugesan and Rajagopal, 2009; Black et al., 2011; Tandel et al., 2017). However, in the literature, the effect of ARR value together with changing length to diameter ratio, L/D has not been considered. This point still needs to be further investigated so that a better understanding of the effect of ARR value with different L/D ratios on the vertical stress-settlement behaviour of SCs could be achieved. In the present study, the effect of both ARRs and L/D ratios of SCs on the vertical stress-settlement behaviour, bearing improvement ratio, subgrade modulus and bulging failure of NEC and EC SCs in soft soils was examined and the obtained results were discussed in detail. In the laboratory test tank, the SCs were constructed by using two different area replacement ratios, ARRs: 1.56% and 6.25%. The studied L/D ratios of the SCs were 5, 7.5, 10 and 15.

1.2 Research Outlines

This study involves of five chapters. The first chapter presents the study aim (research problem and research objectives). The second chapter delivers a comprehensive literature review. The materials and methods that have been used and followed in the experimental work were elaborated in Chapter 3. The results and discussions of the experimental work were analysed and discussed extensively in Chapter 4. The last chapter summarized all the findings of the research with conclusions and recommendations for future works and studies.

Chapter 2

LITERATURE REVIEW

2.1 Background

2.1.1 Introduction

In this study, the behavior of single and group of EC-SCs in soft and layered soils will be studied. This chapter illustrates the basic knowledge of previous studies on SCs behavior.

2.1.2 Problematic soils and ground improvement

The unavoidable construction over soft soils in costal and urban areas which leads to different problems such as total and differential settlements, liquefaction, and instability and poor durability over the long term (McKelvey et al., 2004; Murugesan and Rajagopal, 2009; Shivashankar et al., 2010). Those problems increased the demand to come up with new techniques of construction to solve the soft ground condition problems of these areas. Consequently, a vast choice of ground improvement techniques which are more economical contemporary alternatives to the old construction techniques that were depended on loads transferring to bearing layers by utilizing concrete piles were established (McKelvey, 2004). The major ideas of these ground improvement alternatives generally include at least one of the subsequent processes drainage, densification, cementation and reinforcement (McKelvey, 2004).

The techniques of vibro compaction and vibro replacement are utilized to amend the weak soil's bearing capacity and settlement (McCabe et al., 2009). Densification of cohesionless soil by vibro compaction was initially introduced and applied in 1963 by Keller group (McCabe et al., 2009). The vibro compaction is a method of applying horizontal forces to the surrounding soil through a vibrating poker. These horizontal forces raise a rearrangement of the surrounding soil particles which lead to soil densification.

Because of the restriction of vibro compaction method in cohesive soils, the vibro replacement (column) method has been established in 1956 to deal with cohesive soils (Mc Kelvey., 2004). The vibro SC method depends on a vibrating poker, which is used to open a borehole. The borehole is then filled with aggregate materials and by pushing down the poker; the aggregates are compacted and interlocked with the surrounding soil. Then, the SC is formed. Figure 2.1 shows the vibro compaction and replacement techniques suitability.

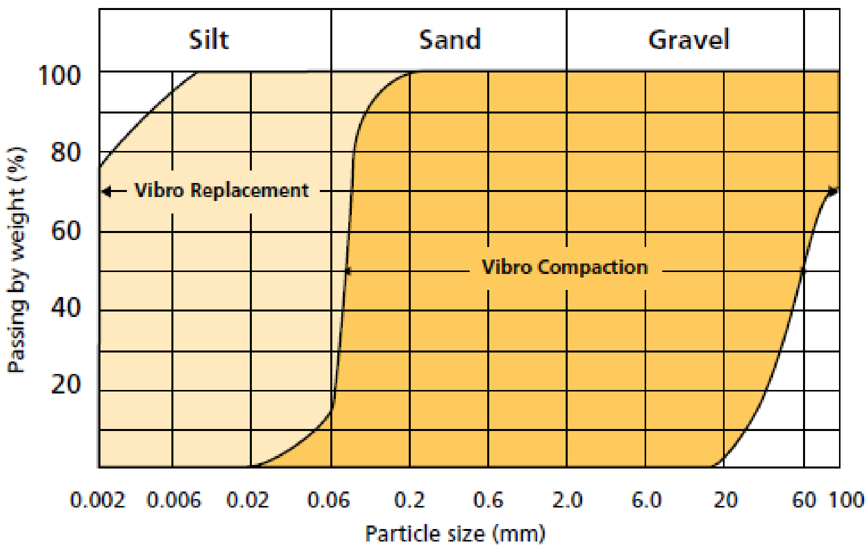


Figure 2.1: Vibro compaction and replacement techniques Suitability (URL 1)

2.1.3 Stone column construction

The poker (vibratory probe) is the fundamental apparatus used for vibro SC. The poker diameter and length ranges between 0.3-0.5 m and 2.0-5.0 m, respectively. The length can be extended with extension tubes to reach up to 26 m depth (Sondermann and Wehr, 2004). Horizontal forces are applied through the vibratory poker from its eccentric weight, which is placed in the lower part of the poker. The transferring of vibration to the extension tube is prevented by the top part of the poker (elastic coupling). The compressed air and water are applied through supplying tubes which help penetration of the poker. The construction of vibro SC has two methods: vibro replacement and vibro displacement methods (Sondermann and Wehr, 2004).

2.1.4 Vibro replacement stone column applications

The vibro replacement technique is implemented in different conditions such as in the case of liquefaction susceptibility, excessive settlement and when the case of not stable sloping embankment sides (Barksdale and Bachus, 1983; Vekli et al., 2012).

2.1.5 Stone columns application configurations

2.1.5.1 Stone column loading configurations and patterns

For experimental field and laboratory studies, the SCs loading configurations are presented in Figure 2.2 (Najjar, 2013).

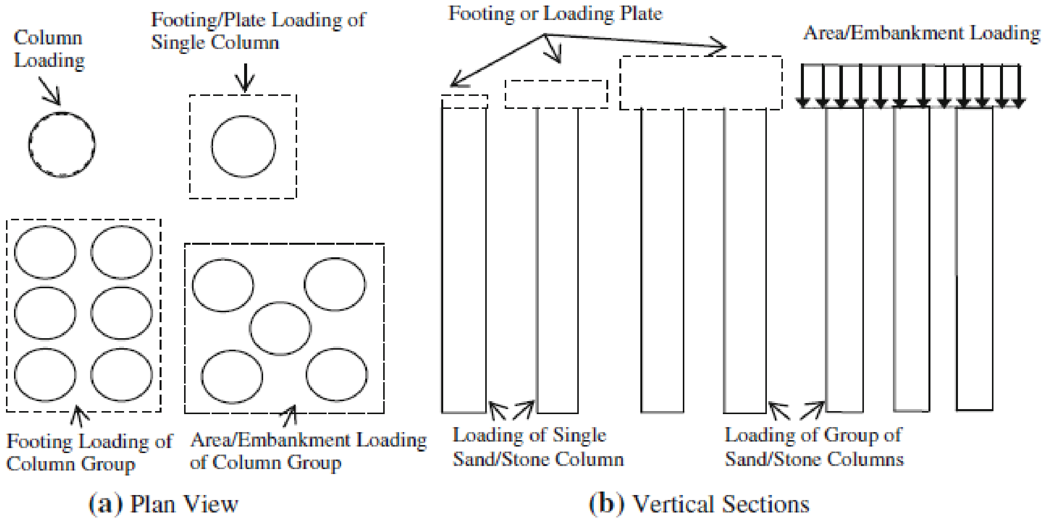


Figure 2.2: Typical loading configurations for SCs (Najjar, 2013)

2.1.5.2 Unit cell

According to Balaam and Poulos (1978) the unit cell of a reinforced foundation ground by SC can be seen in Figure 2.3. The unit cell's equivalent diameter, d_e for three different types of arrangements used is shown in Figure 2.3. s is the spacing of the columns in the figure.

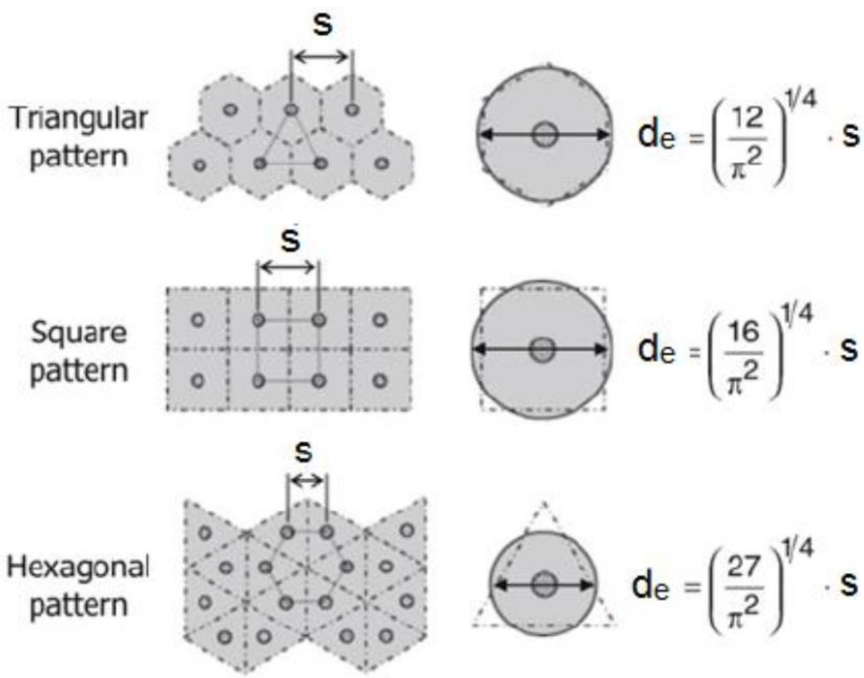


Figure 2.3: Different SC arrangements (Balaam and Poulos, 1978)

Balaam and Poulos (1978) have approximated the unit cell area to be as same as circle. According to Hu (1995) the unit cell of a reinforced foundation ground by SC can be described in the following equation:

$$A=A_c+A_s \tag{2.1}$$

where:

A: Unit cell area.

A_c: SC area.

A_s: SC tributary area of soil.

The using of unit cell concept in calculation and designing the SC reinforced foundations will be discussed in the later section.

2.1.5.3 Area Ratio (AR) or Area Replacement Ratio (ARR)

Area ratio is the amount of soil replaced by stones. It is determined as the ratio of SC area to unit cell (Shivashankar et al., 2010). AR is defined in the following equation:

$$\mathrm{AR} = \frac{A_c}{A} \tag{2.2}$$

where:

AR: Area ratio.

A_c: SC area.

A: Unit cell area.

2.1.5.4 Stress Concentration Ratio (n)

When an embankment or foundation rests on a ground that is reinforced with SC, concentration of stresses develops in the SC and the stresses carried by the surrounding soil is shown in Figure 2.4 (Barksdale and Bachus, 1983; Saadi, 1995). When SC and surrounding soil have nearly same vertical settlement (Vautrain, 1978) and since the SC stiffness is higher, the concentrated stress in SC is higher than the surrounding soils (Vautrain, 1978).

The distribution of vertical stress within a unit cell can be explicit as stress concentration ratio, n which can be calculated by this equation (Saadi, 1995):

$$n = \frac{\sigma_c}{\sigma_s} \tag{2.3}$$

where:

σ_c: Vertical stress on the SC.

σ_s: Vertical stress on the surrounding soil.

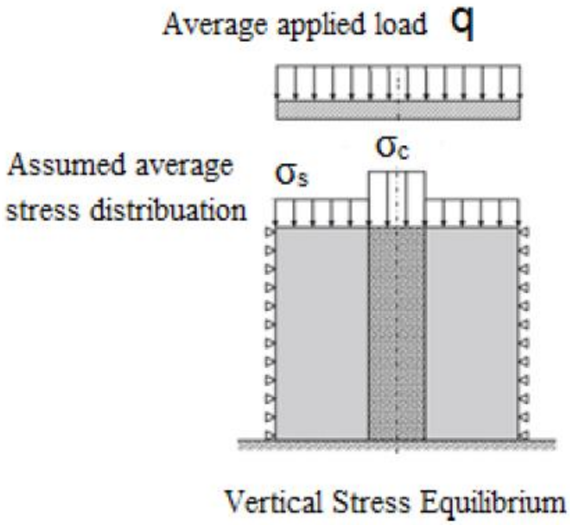


Figure 2.4: Stress concentration of column (Saadi, 1995)

The average applied stress q which is applied over the unit cell area can be presented in the following equation (Barksdale and Bachus, 1983):

$$q=\sigma_c.AR+\sigma_s(1-AR) \tag{2.4}$$

By rearranging Equation 2.4, the stress on surrounding soil and SC can be written in the following equations (Barksdale and Bachus, 1983):

$$\sigma_c = \frac{n.q}{1+(n-1).AR} = \mu_c.q \tag{2.5}$$

$$\sigma_s = \frac{q}{1+(n-1).AR} = \mu_s.q \tag{2.6}$$

Where, μ_c is the stress ratio in SC and μ_s is stresses ratio in surrounding soil.

2.1.5.5 Settlement Reduction Ratio β and Bearing Improvement Ratio, BIR

According to Al Ammari (2016) the ratio of settlement reduction, for a particular load level can be defined as the ratio of reinforced soil settlement at the specified load level, S_r to the corresponding unreinforced soil settlement, S_u.

$$\beta = \frac{S_r}{S_u} \tag{2.7}$$

where:

β: Settlement Reduction Ratio.

S_r: Settlement of reinforced soil at the specified load level.

S_u: Settlement of the unreinforced soil at the same load level.

The settlement reduction ratio, is also known as bearing improvement ratio, BIR which is ranging between 1 and 6 (Al Ammari, 2016).

The BIR represents the ratio of vertical stress of reinforced soil to vertical stress of unreinforced soil at the same settlements (q_r/q_u).

$$BIR = \frac{q_r}{q_u} \tag{2.8}$$

where:

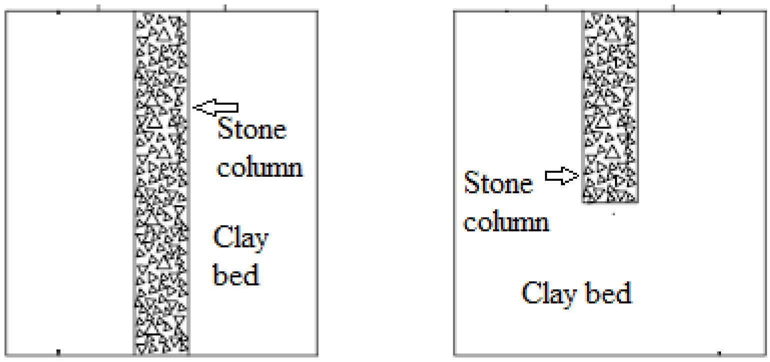
BIR: Bearing Improvement Ratio.

q_r: Vertical stress of reinforced soil.

q_u: Vertical stress of unreinforced soil.

2.2 Stone columns types

Some researchers (Malarvizhi and Ilamparuthi, 2004; Dash and Bora, 2013) have classified SCs into two types: floating SC (FSC) which is suspended in the soil bed and end bearing SC (EBSC) which is resting on the base of the soil bed on testing tank or hard stratum as shown in Figure 2.5.



(a) (b)

Figure 2.5: SC types (a) EBSC and (b) FSC (Malarvizhi and Ilamparuthi, 2004)

2.3 Mode of failures of stone columns

Muir Wood et al. (2000) stated SCs deformation modes: bulging, shearing, punching, and bending. Single SCs tend to bulge or punch, relying on column length, whereas SCs group exhibit bulging, punching, shearing, and bending (Muir Wood et al., 2000). As Nazariafshar et al. (2017) stated in single SC application, the stresses due to loading around the single SC are uniform due to the location of SC in the centre of the loading plate. However, According to Nazariafshar et al. (2017) in columns group since the columns are not positioned at the loading plate centre, the induced stresses around the SCs are not uniform and the deformations generated are the incorporation of bulging and bending in the columns group (Nazariafshar et al., 2017). Bulging failure commonly happens when the base of the SC is floating in soft soil or lying on a hard layer. In a long SC with L/D ratio more than 5, failure occurs due to bulging (Christoulas et al., 2000; Shahu et al., 2000; Ghazavi and Afshar, 2013; Chen et al., 2015; Hong el at., 2016). In a FSC, with L/D ratio less than 3 (a short SC), the ultimate vertical stress is controlled by the punching failure (Aboshi et al., 1979; Nazariafshar et al., 2017). In short SCs, the general shear failure is encountered when the SC rests on hard layers (Madhav and Vitkar, 1978). The failure mechanism can be seen in Figure 2.6.

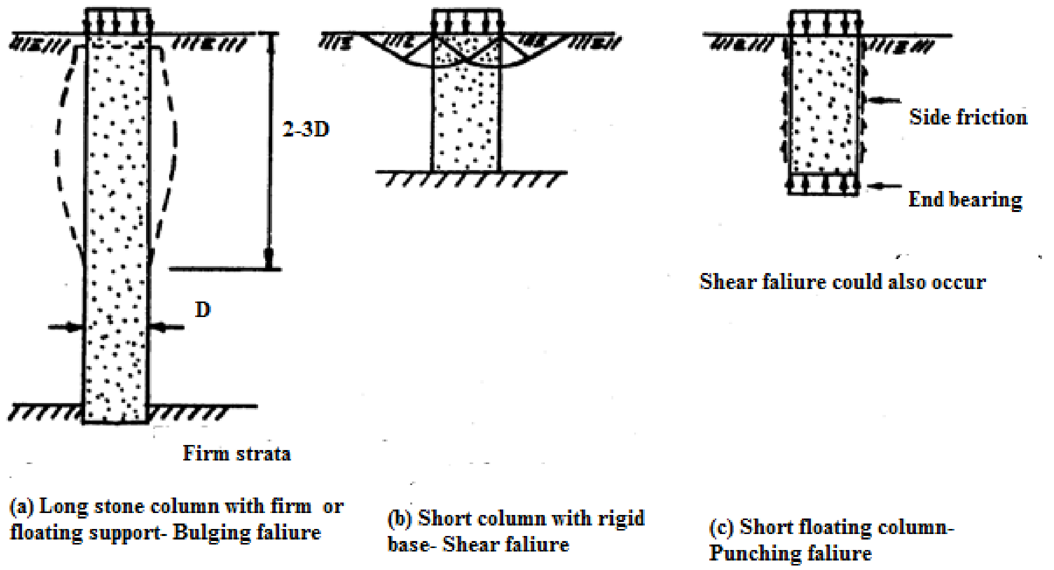


Figure 2.6: Failure modes in single-layered soft soil (Barksdale and Bachus, 1983)

2.4 Theoretical analysis and design methods

The SCs foundation composite design requires considering the both materials responses: SC aggregates and the surrounding soil. This consideration makes the analysis and interpretation of the material behaviour complicated unless some assumptions for these materials have been made (Hu, 1995).

Some researchers have indicated that the granular material behaviour was similar to elastic material and the clay behaviour was similar to elasto plastic material (Baumann and Bauer, 1974; Hughes et al., 1975; Priebe, 1976), whereas the contemporary SC design approaches are relied on theory of plasticity (Al Ammari, 2016).

Some theories have been adopted in the literature: cylindrical cavity expansion theory, ultimate vertical stress of SC and modelling considerations which are discussed in the following sections.

2.4.1 Cylindrical cavity expansion theory

The generation of passive resistance by the surrounding soil was the first estimation to be well modelled as a boundlessly long cylinder which extends around the axis of symmetry until the surrounding soil ultimate passive resistance is developed (Vesic, 1972). The expanding cylindrical cavity nearly simulates the lateral bulging of the column into the surrounding soil which resembles pressuremeter test in which a cylinder is extended towards a borehole side (Barksdale and Bachus, 1983).

Vesic (1972) has improved the cavity expansion theory for cohessionless and cohesive soils by considering the concept of unit cell and the mode of bulging failure with a view to determine the ultimate cavity resistance (ultimate bearing capacity of one column) of the reinforced soil. Equation 2.9 gives the determination of the ultimate cavity resistance:

$$\sigma_3 = cF_c + qF_q \tag{2.9}$$

Where σ_3 is the ultimate cavity resitance F_c and F_q are unitless cavity expansion factors, which are related to angle of internal friction of reinforced soil and the index of rigidity (I_r). Where q is the stress at the failure depth and c is the soil cohesion. The rigidity index (I_r) is given in Equation 2.10 and also Figure 2.7 shows the determination of F_c and F_q:

$$I_r = \frac{G}{c_u + P' \tan''\phi'} \tag{2.10}$$

Where G is the shear module of the reinforced soil; c_u is the soil's undrained cohesion; P' is the mean effective stress at the failure depth; and ϕ' is the angle of internal friction of the reinforced soil.

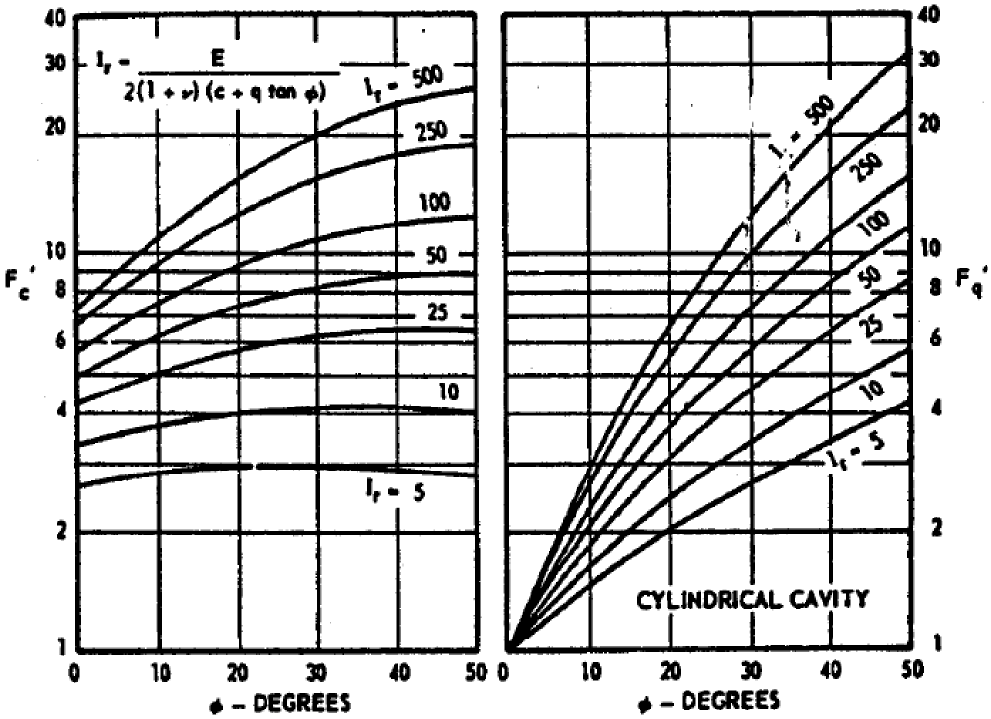


Figure 2.7: F_c and F_q determination (Vesic, 1972)

Hughes and Withers (1974) used the cylindrical cavity expansion method to calculate the lateral ultimate stress by using Equation 2.11:

$$\sigma_{rl} = \sigma_{r0} + c[1 + \log \frac{E}{2c(1+\nu)}] \tag{2.11}$$

Hughes and Withers (1974) approximated Equation 2.12 as:

$$\sigma_{rl} = \sigma_{r0} + 4c_u \text{ or } \sigma_{rl} = \sigma_{r0}' + 4c_u + u \tag{2.12}$$

Where σ_{rl}, σ_{r0}, E, ν and c_u are the ultimate lateral stress, total initial in-situ lateral stress, modulus of elasticity, Poisson's ratio and soil's undrained cohesion, respectively.

Accordingly, the ultimate vertical stress of single SC in cohesive soil can be calculated by the following equation (Hughes and Withers, 1974):

$$q_{ult}=\frac{1+\sin\phi'}{1-\sin\phi'}(\sigma_{r0}+4c_u-u) \tag{2.13}$$

Where ϕ' is SC's internal friction angle; σ_{r0} total in situ lateral stress; c_u undrained cohesion; and u pore water pressure.

2.4.2 Ultimate bearing capacity of stone column

Some researchers have proposed some derived formulas for determine the ultimate vertical stress of NEC-SC in cohesive soil when the SCs are controlled by bulging failure (Hughes and Withers, 1974; Mitchell, 1981; Christouls et al., 2000). The derived formulas are presented in Table 2.1.

Table 2.1: Gives the ultimate vertical stress value of NEC-SCs with different derived formulas in the literature

Derived formula	Reference
$q_{ult}=\frac{1+\sin\phi_s'}{1-\sin\phi_s'}(\sigma_r'+4c_u)$	Hughes and Withers (1974)
$q_{ult}=c_u x\ N_p$	Mitchell (1981)
$q_{ult}=\frac{\pi D\times L\times c_u}{A_s}$	Christouls et al. (2000)

ϕ_s'=Internal friction angle of stone column
σ_r'=Effective radial stress
c_u=Undrained cohesion
q_{ult}=Ultimate vertical stress of stone column
N_p=Bearing capacity factor
D=Stone column diameter
L=Stone column length
A_s=Stone column area

Murugesan and Rajagopal (2008) have proposed derived formula for estimating the ultimate vertical stress of EC SC in cohesive soil which is shown in Equation 2.14.

$$q_{ult} = \frac{1+\sin\phi'}{1-\sin\phi'}(\sigma_{r0}+4c_u-u) + p_c \qquad (2.14)$$

Where ϕ' is SC's internal friction angle; σ_{r0} total in situ lateral stress; c_u undrained cohesion; u pore water pressure; and p_c is additional confinement provided by encasement material which can be calculated by Equation 2.15:

$$p_c = \frac{2T}{D} \qquad (2.15)$$

Where T is the tensile strength of the encasement material; and D is the SC diameter.

2.4.3 Modelling considerations (scaling effect)

In laboratory conditions, it is quite expensive and time-consuming to test full-scale SC-reinforced soils. For these reasons, experiments performed in laboratory are usually limited to observation of the behavior of small models which simulate the actual foundations at a predefined ratio (Altaee and Fellenius, 1994). The main difficulty in laboratory tests is the scaling effect. Allersma (1995) stated that four types of physical models can be recognized according to the scale of the model. These types are the full and small scales field tests, small-scales physical laboratory tests (1g) and centrifuge tests. Altaee and Fellenius (1994) also indicated that the void ratio of soil in the small-scale model must be no looser than the prototype soil and it must not be denser than prototype soil.

Debnath and Dey (2017) defined a similitude ratio as the ratio of any prototype linear dimension to the equivalent dimension of the small-scale model. Debnath and Dey (2017) stated that typically the prototype SCs have a diameter (d_p) ranging from 0.6 m to 1.0 m and the column length to diameter ratio (L/d_p) ranging between 5-20

(Shahu and Reddy, 2011). Muir Wood et al. (2000) specified that the particle size of the aggregates (d_s) used in the prototype SCs range from 25 mm to 50 mm and the d_p/d_s ratio ranging between 12-40.

According to Barksdale and Bachus (1983), the depth of SC bulging was nearly 2-3 times SC diameter. Researchers indicated that the wedge of failure in foundation bed extended from 2 to 2.5 times the width of footing away from its center (Selig and McKee 1961; Chummer 1972). Meyerhof and Sastry (1978) also stated that the zone of failure under rigid piles reached to a depth two times their diameter.

Considering all these aforementioned criteria, the dimensions of the model test tank to be used within the scope of this study have been decided.

Chapter 3

MATERIALS AND METHODS

3.1 Introduction

In this chapter the materials and methods used to study the behaviour of single and group of floating SCs (FSCs) and end bearing SCs (EBSCs) with and without encasement in single-layered soft soil and layered soil are presented.

The laboratory tests for index and engineering properties of soft soil, sand and crushed stone aggregates were conducted according to American standards, ASTM.

3.2 Experimental investigation

3.2.1 Properties of materials used

For the model test, soft soil, sand, crushed stone aggregates, and geotextiles were utilized as materials.

3.2.1.1 Soft soil

The soft soil used in the study was taken from around 1 m depth from the ground surface in Famagusta, North Cyprus. The approximate location of the particular area from which the soft soil was taken is shown in Figure 3.1and the soft soil properties were presented in Table 3.1.

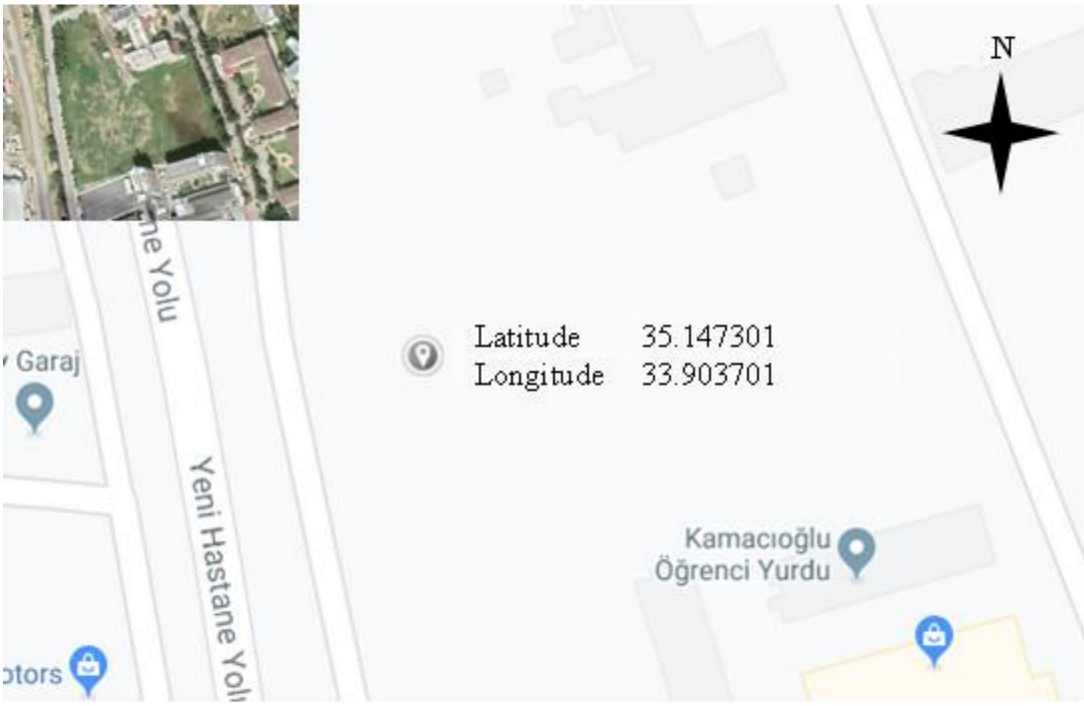


Figure 3.1: Geographical location of soft soil (URL 2)

Table 3.1: Physical and index properties of the soft soil

Index properties	Values
Fraction of clay size (<2µm) [a] (%)	52.0
Fraction of silt size (2µm-74µm) [a] (%)	48.0
In-situ bulk density, ρ [b] (g/cm^3)	1.77
In-situ moisture content [b] (%)	30.0
Specific gravity [c], (Gs)	2.65
Liquid limit [d], LL (%)	58.0
Plastic limit [d], PL (%)	30.0
Plasticity index [d], PI	28.0
Compression index, C_c	0.20
Rebound index, C_r	0.21
Activity[d]	0.58
Soil classification [e]	CH

a According to ASTM D 422-98
b According to ASTM D 2937-17
c According to ASTM D 854-06
d According to ASTM D 4318
e According to ASTM D 2487-00 (Unified Soil Classification System)

The compaction test was conducted to obtain the optimum water content as 25% and the maximum dry density as 1.53 g/cm^3. To analyse the worst soil condition in the laboratory, soil specimens at different water contents were prepared and unconfined compressive strength, UCS tests were carried out on those specimens. The variation of UCS with increasing water content values was shown in Table 3.2. According to Das (2008), the consistency of the soil at 33 kPa was described to be soft. The values in Table 3.2 indicated that the water content corresponding to 33 kPa was found to be 33%. Throughout this study, the soil water content in the model test tank has been preserved at 33% and all the tests in the model tank were performed at this water content. The bulk density of the soil at the same water content was 1.81 g/cm^3.

Table 3.2: UCS of soil corresponding to different water content values

Water content (%)	Unconfined compressive strength (kPa)	Consistency (Das, 2008)
16	580	Hard
20	554	Hard
26	284	Very Stiff
32	48	Soft
33	33	Soft

3.2.1.2 Sand

The sand utilized in this study was taken from Bedis Beach in Famagusta, North Cyprus. The approximate location of the specified area from which the sand was taken is presented in Figure 3.2 and the properties of the Bedis sand are presented in Table 3.3. According to the Unified Soil Classification System, the Bedis sand was classified as poorly graded sand (SP).

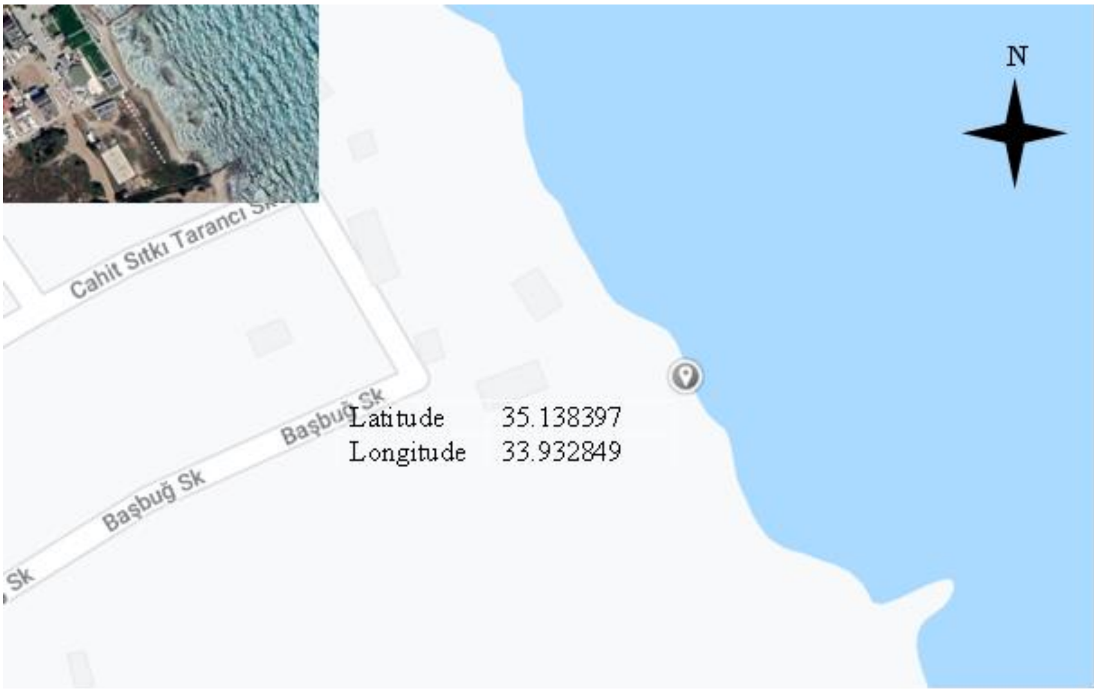


Figure 3.2: Geographical location of sandy soil (URL 3)

Table 3.3: Properties of Bedis sand

Parameters	Values
Specific gravity[b]	2.65
Maximum dry density [a], $\rho_{d(max)}$ (g/cm^3)	1.55
Minimum dry density [a], $\rho_{d(min)}$ (g/cm^3)	1.46
Internal friction angle, ϕ° at loose state	31.0
Uniformity coefficient (C_u)[c]	1.29
Coefficient of curvature (C_c)[c]	1.06
Mean Diameter, D_{50} (mm)	0.22
Unified Soil Classification System, USCS[c]	SP

a According to Impact method, Bowles 1992
b According to ASTM D 854-06
c According to ASTM D 2487-00 (Unified Soil Classification System)

3.2.1.3 Crushed stone aggregate

The crushed stone aggregate used in the SC's construction were collected from a local quarry, lime stone, which had sizes of 1-5 mm. The properties of the crushed stone aggregate are shown in Table 3.4.

Table 3.4: Properties of the crushed stone aggregates

Parameters	Values
Maximum dry density [a], $\rho_{d(max)}$ (g/cm^3)	1.61
Minimum dry density [a], $\rho_{d(min)}$ (g/cm^3)	1.49
Specific gravity[b]	2.48
Internal friction angle, ϕ° (at 70% relative density) (^{o})	46.0
Bulk density (at 70% relative density), (g/cm^3)	1.57
Uniformity coefficient (C_u)[c]	1.67
Coefficient of curvature (C_c)[c]	0.99
Mean Diameter, D_{50} (mm)	3.00
Unified Soil Classification System, USCS[d]	SP

a According to Impact method, Joseph E Bowles 1992
b According to ASTM D 854-06
c According to ASTM D 2487-00 (Unified Soil Classification System)

The particle size distribution of the crushed stone aggregates, soft soil, and sand are given in Figure 3.3.

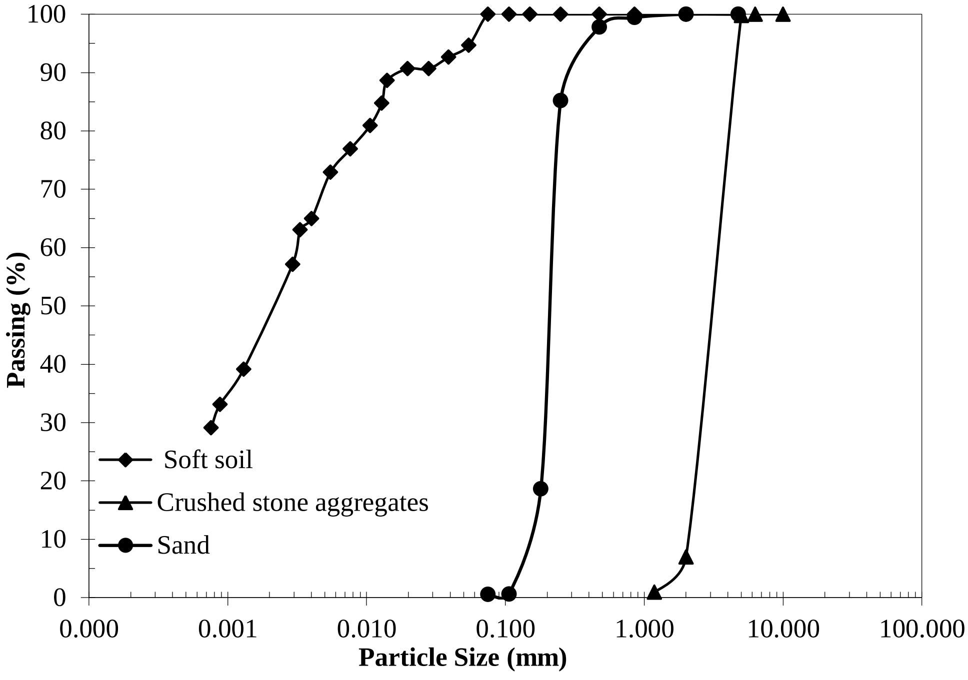


Figure 3.3: Particle size distribution of soft soil, sand and crushed stone aggregates used in the study

3.2.1.4 Geotextile

Non-woven geotextile was utilized to encase and increase the confinement of material in the SC. Practically, the tensile strength of geosynthetics materials utilized in granular piles is maintained at 400 kN/m whereas in laboratory model tests, it was within the range of 1.5 to 20 kN/m (Murugesan and Rajagopal, 2009; Ali et al., 2012; Ghazavi and Afshar, 2013). In this study, the geotextile material had 7 kN/m tensile strength and the non-woven geotextile properties were presented in Table 3.5 and shown in Figure 3.4.

Table 3.5: Properties of the non -woven geotextile

Test	Standard	Value	Unit
Weight	EN ISO 9864	500	g/m^2
Thickness (2kPa)	EN ISO 9863-1	2.7	mm
Tensile strength (longitudinal transverse)	EN ISO 10319	7	kN/m
Break extension (longitudinal-cross)	EN ISO 10319	min. 60	%
Static puncture	EN ISO 12236	2040	N
Dynamic puncture	EN ISO 13433	10.04	mm
Water permeability, VH50	EN ISO 11058	0.034	$l/s^* m^2$
Visible pore size, O90	EN ISO 12956	0.070	mm

Figure 3.4: Non-woven geotextile

3.2.2 Experimental set-up

3.2.2.1 Model design

Two types of single SCs were considered in the study, FSC with L= 50 cm and EBSC with L= 75 cm which extended up to the base of model test tank. The FSCs and EBSCs have been constructed in two different diameters: 5 cm and 10 cm; the diameter and the length of the SC in the model test set-up were designed along the lines exemplified by Debnath and Dey (2017) as discussed in section 2.5.3.

In case of group of SCs, preliminary tests were performed and accordingly only floating type of SC with L=50 cm, diameter of 5 cm and spacing of 7.5 cm and 12.5 cm was considered. The spacing to SC diameter ratios (s/D) were used to be 1.5 and 2.5.

As aforementioned, in the present study, the diameter (D) and the length (L) of the SC in the model tests were designed along similar lines as Debnath and Dey (2017). The diameter (D) and the lengths (L) of the FSCs and EBSCs in the model test were 5 cm, 50 cm and 75 cm, respectively. Thus, the L/D ratios in FSCs and EBSCs were 10 and 15, respectively, placing them in the 5 to 20 range as suggested by Debnath and Dey (2017). Also, different diameter (D) and the same lengths (L) of the FSCs and EBSCs in the model test tank were used as 10 cm diameter, 50 cm and 75 cm lengths, respectively. Thus, the L/D ratios in FSCs and EBSCs were 5 and 7.5, respectively placing them in the 5 to 20 range as suggested by Debnath and Dey (2017). The particle size of the SC aggregates (d_s) used in the study ranged between 1 mm and 5 mm and the mean particle size diameter (D_{50}) was 3 mm resulting in D/D_{50} ratios of 17 and 33 of 5 cm and 10 cm diameters SCs which were in the 12 to 40 ranges suggested by Muir Wood et al. (2000). The selected dimensions of the model test in this study fit the suggested similitude ratio of Debnath and Dey (2017) and Muir Wood et al. (2000), discussed in Section 2.5.3.

In the model test, the layered soil consists of loose sand overlaying soft soil. Altaee and Fellenius (1994) stated that physical modelling in sand depends mainly on the initial average effective stress and the initial void ratio of the prototype soil. Since there was no specific prototype in mind and the maximum void ratio of loose sand used in this study was known, the maximum void ratio of the loose sand was assumed to be the initial void ratio in the model test.

As aforementioned in Section 2.5.3, by considering the findings of the previous researchers for bulging depth of SC, failure wedge in foundation bed and the failure zone under rigid piles (Selig and McKee 1961; Chummer 1972; Meyerhof and Sastry, 1978; Barksdale and Bachus, 1983), a circular steel tank of 40 cm diameter and 80 cm height was utilized in this study so that the test tank sides are not interfering with the failure wedges. The thickness of the soil bed in the test tank was 70 cm.

3.2.2.2 Test set-up and procedure

For single SC construction, 5 cm and 10 cm steel augers were utilized to drill the circular hole in the soft soil bed in the test tank. A steel rod of 2 cm diameter was utilized for the compaction of the SC material to achieve the required uniform density in the SC. The application of the load on the SC area was through 5 and 10 cm diameter steel circular plates which were the same as SC area. Two variable differential transformers, LVDTs have been used to measure the settlement of the loading plate.

For testing the layered soil in the model test tank, 35 cm of soft soil was placed in the tank and then 35 cm of loose sand was spread on top of it. Further explanation of this soil sample preparation is given in Section 3.2.2.4.

For group of SCs construction, preliminary tests have been performed and accordingly floating type of SC was selected for the load-settlement tests and the same steps as used for single FSCs were then followed.

Finally, the load-settlement tests were performed sequentially with first the single-layered soft soil and then the layered soil. The load-settlement behavior of the natural (non-reinforced) soft soil bed was tested in the model test tank and then the load-settlement behavior of SC-reinforced soft bed with and without geotextile was examined.

3.2.2.3 Preparation of single-layered soft soil in the test tank

In the study, the single-layered soft soil bed was placed in the testing tank at 33% water content and 1.81 g/cm^3 bulk unit weight. Before placing the soft soil in the testing tank, lubricating oil has been applied on the walls of the testing tank and a nylon sheet was placed in order to ease the removal of the soil sample after testing. A sand bed of 5 cm thickness was placed in the bottom of the testing tank for drainage purpose. To achieve identical and homogeneous soil samples in all tests, the required amount of soft soil to fill the testing tank was calculated and divided into seven equal layers of 10 cm thickness and then compacted and placed in the testing tank. After placing the soft soil in the testing tank, a circular plate with the same diameter of the test tank was placed on top of the soft soil bed surface. To simulate the situation in the field, a surcharge pressure of 9.05 kPa was applied to the circular plate as in-situ overburden pressure to consolidate the soft soil in the test tank. Two dial gauges with an accuracy of 0.002 mm were attached to the circular plate to measure the settlement. The measurement of the settlement was continued until 0.04 mm/day was reached. The soft soil consolidation was completed in five days.

3.2.2.4 Preparation of layered soil in the test tank

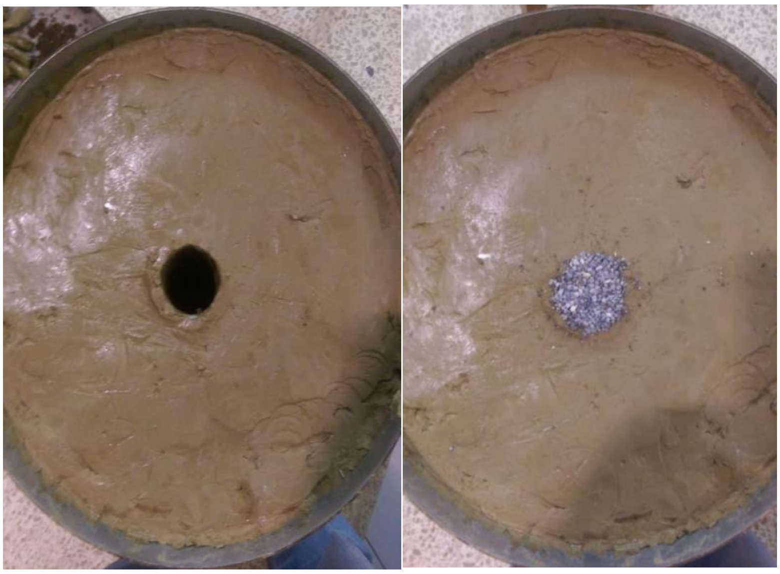
For the preparation of layered soil, the same procedure was followed as for the single-layered soft soil. The soft soil and loose sand thicknesses were 35 cm each. After placement and complete soft soil layer consolidation, the sand layer was spread over it. For the placement density of the sand layer, the minimum index density of 1.46 g/cm^3 was utilized, as given in Table 3.3.

3.2.2.5 Single stone column construction

A steel auger was used for drilling the required diameter of a cylindrical hole at the centre of soft soil bed in the test tank. Then the crushed stone aggregates were placed in the hole with the help of a hollow steel pipe which was vertically settled and pushed into the drilled hole in the soft soil bed. The steel pipe has been coated internally and externally with oil to avoid the friction between the pipe and the surrounding soil. The verticality of the pipe was neatly checked by a level. Known quantities of crushed stone aggregates were used to charge the hole, which were charged in layers. Each layer of crushed stone aggregate was subjected to uniform compaction to achieve a density of 1.57 g/cm^3 which was corresponding to 70% relative density which was considered as a dense state. For the FSC, the required amount of crushed stone aggregates was partitioned into five equal batches, and each batch was poured into the hole through the pipe. After that, the pipe was pulled out and compaction was performed on each batch to attain the specific height of 50 cm. For avoiding lateral distortion of the surrounding soil during the construction of SC, each layer was lightly compacted by using a 1.83 kg tamping rod with 25 blows of

10 cm drop. For the EBSC construction which was sitting at the base of the test tank, the calculated amount of crushed stone aggregates were partitioned into seven batches, and each batch was placed into the hole in the similar manner as in the construction of the FSC.

For encasement of the SC with geotextile, the required area of geotextile was calculated and cut according to the volume of the hole plus 2 cm for gluing. The geotextile was formed into a cylindrical shape with the two edges overlapping by 2 cm. Then, the fabric of the areas of overlap was bonded together with epoxy which was allowed to set-up for 24 hours. For the EC-SCs, a tubular steel pipe with a 4 cm outer diameter was utilized for vertical insertion of the geotextile into the bored hole. The same series of procedure and conditions were applied in layered soil as were applied in single-layered soft soils in case of both floating and EBSCs. Figures 3.5 and 3.6 show the construction of single SC the single-layered and layered soils in the test tank.



(a) (b)

Figure 3.5: SC construction in single-layered soil (a) Before discharging crushed stone aggregates and (b) After discharging crushed stone aggregates

(a) (b)

Figure 3.6: SC construction in layered soil (a) Discharging crushed stone aggregates and (b) After discharging crushed stone aggregates

Tables 3.6-3.9 show the details of the single SCs constructed in the single-layered and layered soils in the test tank.

Table 3.6: Details of the 5 cm diameter SCs constructed in the single-layered soft soil

Sample no	Thickness of soft soil (cm)	Stone column diameter, D (cm)	Stone column length, L (cm)	Geotextile used
1	70	--	--	No
2	70	5	50	No
3	70	5	50	Yes
4	70	5	75	No

Table 3.7: Details of the 10 cm diameter SCs constructed in the single-layered soft soil

Sample no	Thickness of soft soil (cm)	Stone column diameter, D (cm)	Stone column length, L (cm)	Geotextile used
1	70	--	--	No
2	70	10	50	No
3	70	10	50	Yes
4	70	10	75	No
5	70	10	75	Yes

Table 3.8: Details of the 5 cm diameter SCs constructed in the layered soil

Sample no	Thickness of layered soils (cm)	Layers formation*: A and B A@top B@bottom	Stone column diameter, D (cm)	Stone column length, L (cm)	Geotexti -le used
1	70	A+B	--	--	No
2	70	A+B	5	50	No
3	70	A+B	5	50	Yes
4	70	A+B	5	75	No

*A= Sand (35 cm in thickness) (thickness= 7 times stone column diameter)
B= Soft soil (35 cm in thickness) (thickness= 7 times stone column diameter)

Table 3.9: Details of the 10 cm diameter SCs constructed in the layered soil

Sample no	Thickness of layered soils (cm)	Layers formation*: A and B A@top B@bottom	Stone column diameter, D (cm)	Stone column length, L (cm)	Geotextile used
1	70	A+B	--	--	No
2	70	A+B	10	50	No
3	70	A+B	10	50	Yes
4	70	A+B	10	75	No
5	70	A+B	10	75	Yes

*A= Sand (35 cm in thickness) (thickness= 3.5 times stone column diameter)
B= Soft soil (35 cm in thickness) (thickness= 3.5 times stone column diameter)

3.2.2.6 Group of floating stone columns construction

For the group of SCs construction, preliminary tests were performed and accordingly floating type of SC was selected for the load-settlement tests and the same steps as used for single FSCs were then followed in case of single-layered and layered soils. For the group of NEC- FSCs (5cm diameter column), two spacing were used which were 2.5 and 1.5 times the diameter of SC. The encasement was only applied for the selected case from preliminary tests (1.5xD) for the group of NEC-FSCs. Tables 3.10 and 3.11 show the details of the group of SCs constructed in single-layered soft soil and layered soils in the test tank. Figure 3.7 shows the arrangement of group of SCs in the test tank.

Table 3.10: Details of the group of 5 cm diameter FSCs constructed in single-layered soft soil

Sample no	Thickness of soft soil (cm)	Stone column diameter, D (cm)	Stone column length, L (cm)	Spacing, s (cm)	Geotextile used
1	70	5	50	2.5xD=12.5	No
2	70	5	50	1.5xD=7.5	No
3	70	5	50	1.5xD=7.5	Yes

Table 3.11: Details of the group of 5 cm diameter FSCs constructed in layered soil

Sample no	Thickness of layered soils (cm)	Layers formation*: A and B A@top B@bottom	Stone column diameter D (cm)	Stone column length, L (cm)	Spacing, s (cm)	Geo-textile used
1	70	A+B	5	50	2.5xD=12.5	No
2	70	A+B	5	50	1.5xD=7.5	No
3	70	A+B	5	50	1.5xD=7.5	Yes

*A= Sand (35 cm in thickness)
B= Soft soil (35 cm in thickness)

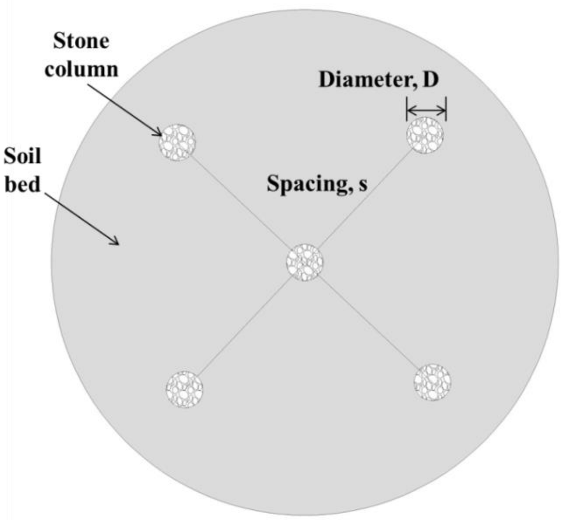


Figure 3.7: Triangular arrangement of group of SCs in test tank

3.2.3 Test procedure of load application and extraction of deformed shapes

The bearing capacity and settlement performance of a single SC are considerably affected by the method of vertical load application over the SC (Shivashankar et al., 2010). Barksdale and Bachus (1983) stated that by applying the load on the soil–SC composite, the bearing capacity of the composite increases to a value more than the bearing capacity of the SC alone. In this study, to evaluate the SCs effectiveness in soft soil improvement, the worst condition was tried to be simulated in the vertical load application so that bearing capacity of only SC could be evaluated. Therefore, a footing diameter of 5 and 10 cm were used as the same SCs diameter and the application of the load was vertically applied over the SC area.

After the soil and column specimens' preparation in the test tank, the load application over the SC area was vertically applied at a constant rate of 1.2 mm/min up to 30 mm vertical settlement of the footing. The loading period was kept short to simulate undrained condition during construction. Figures 3.8 to 3.11 present the schematic diagram of the single floating and EBSCs in the single-layered and layered soils used in this study, respectively.

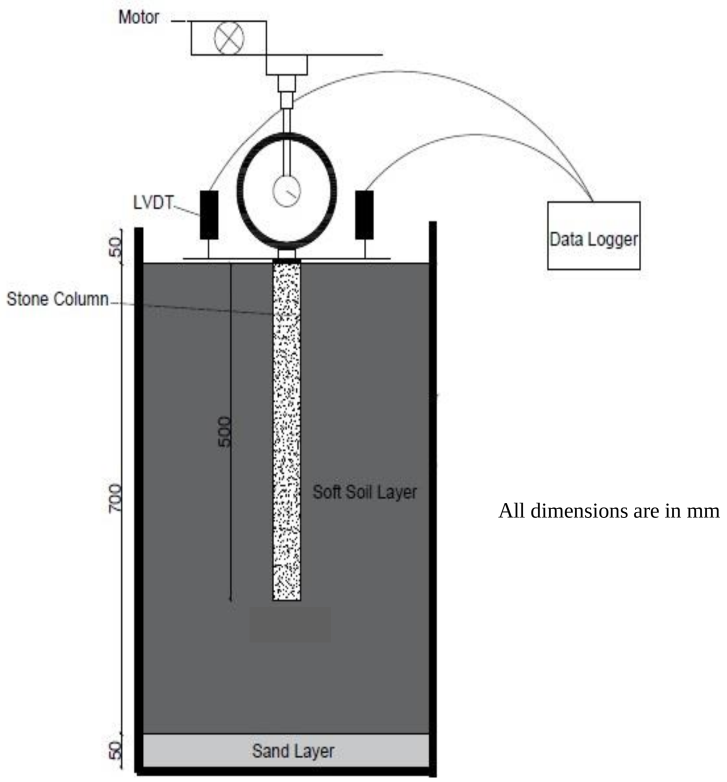


Figure 3.8: The schematic diagram for single FSC in single-layered soft soil

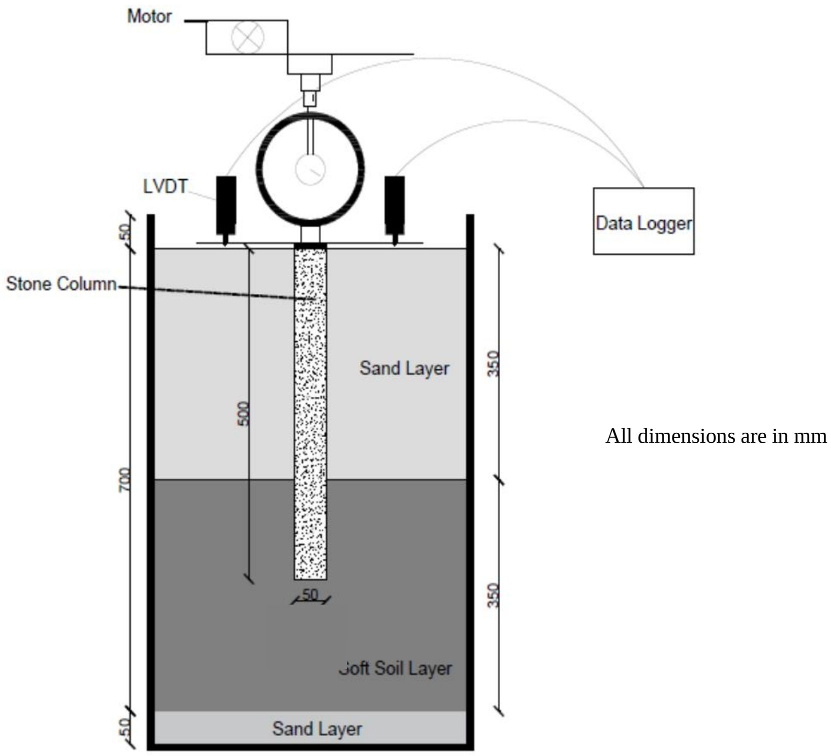


Figure 3.9: The schematic diagram for single FSC in layered soil

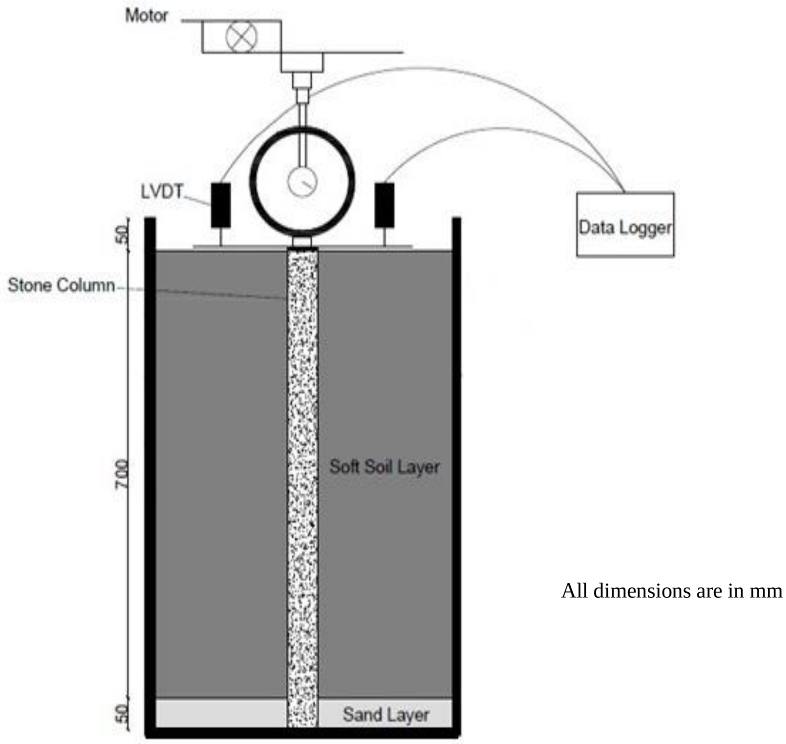


Figure 3.10: The schematic diagram for single EBSC in single-layered soft soil

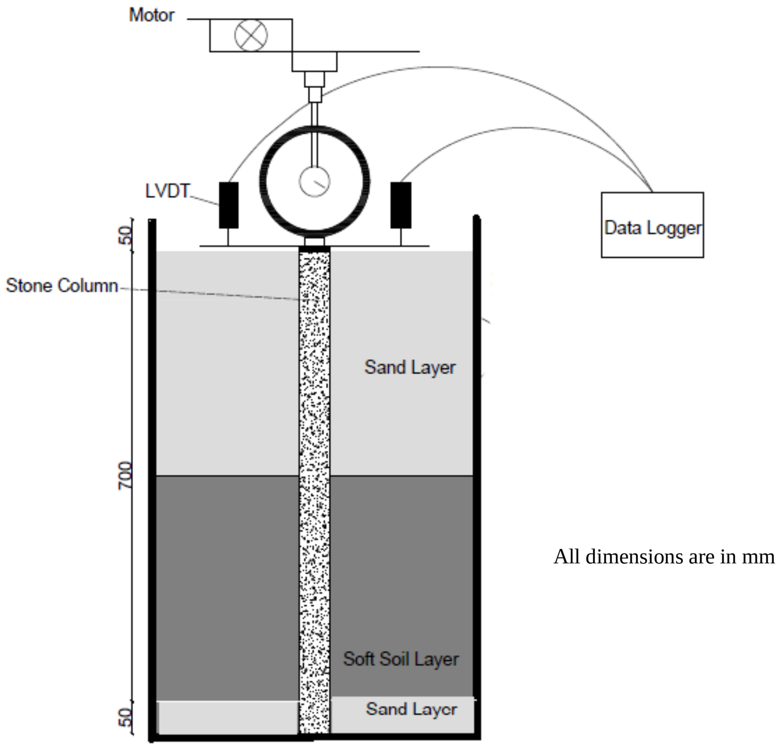


Figure 3.11: The schematic diagram for single EBSC in layered soil

In case of single-layered soft soil after completion of each test, the crushed stone aggregate in the SC was carefully scooped out and then the hole was filled with cement paste to maintain the shape of bulging formed during loading. After hardening of the cement paste, the surrounding soil was removed neatly and then the deformed shape of the SC was extracted. The bulged shape diameter of SC was then measured.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Introduction

All the experimental work performed in this study was according to the methods explained in detail in Chapter 3. This Chapter presents and discusses the experimental results in two parts. The first part investigates the behaviour of the single NEC and EC-SCs in single-layered and layered soils, and the second part discusses the behaviour of the group of NEC and EC-SCs in single-layered and layered soils.

4.2 Behaviour of single non-encased and encased floating and end bearing stone columns in single-layered and layered soils

In this section, the vertical stress-settlement behaviour of single SC, the bearing improvement ratio (BIR), subgrade modulus (k), and bulging failure of single SC in single-layered and layered soils will be discussed. The behaviour of NEC and EC FSC and EBSC-SCs in single-layered and layered soils will be studied with different area replacement ratios (ARRs). The ARRs of 1.56% (5 cm diameter column) and 6.25% (10 cm diameter column) will be considered in the present study.

4.2.1 Vertical stress-settlement behaviour of single stone columns

In the small-scale laboratory model tests reported in the literature, different settlement values were considered in the determination of the ultimate vertical stress of the SC. Some researchers (Tandel et al., 2012; Hasan and Samadhiya, 2017) considered the ultimate vertical stress corresponding to 30 mm settlement.

Malarvizhi and Ilamparuthi (2004) considered the ultimate vertical stress at the settlement value corresponding to 10% of the of the SC diameter whereas Deb et al. (2011) and Debnath and Dey (2017) determined the ultimate vertical stress corresponded to a settlement of 20% of the footing diameter. A comprehensive literature review has shown that there is no specific standard on this. Hughes and Withers (1974) stated that the SC's bearing capacity has been achieved at a settlement of 58% of the diameter of SC whereas Al-Mosawe et al. (1985) found that the ultimate bearing capacity of the SC was obtained at vertical settlement of 60% of SC diameter. In this study, for comparison purposes and also in order to reach the clear ultimate load in the load-settlement curves of each SC application, the loading of the SC was continued until 30 mm settlement and the vertical stress corresponding to this settlement value was considered to be the ultimate vertical stress of the SC.

4.2.1.1 Vertical stress-settlement behaviour of single stone columns in single-layered soft soil

4.2.1.1.1 Behaviour of floating stone columns

Figure 4.1 presents the vertical stress-settlement curves of single SCs with and without encasement for both FSC and EBSCs. Only the behaviour of the NEC and EC floating SCs, FSCs, will be discussed under this section. The ARR of the SCs in the model test tank was 1.56%. Figure 4.1 indicated that the NEC-FSC resulted in a significant amendment in the settlement behaviour of soft soil. The ultimate vertical stress value of the NEC- FSC reached 650.4 kPa at 30 mm settlement. On the other hand, the EC-FSC resulted in higher settlement values than the NEC-FSC and resulted in an ultimate vertical stress value of 692.5 kPa at 30 mm settlement. However, a further increase in the vertical stress value of the EC-FSC was obtained beyond 30 mm settlement. The higher settlement value of the EC-FSC could be explained because of the high tensile strength (7.0 kN/m) of the geotextile material used in this study. At initial loading stages of the SC, due to this high tensile strength, the geotextile material could not stretch and it did not provide good interaction between SC material and surrounding soil. Consequently, no adequate lateral transfer of loads to the surrounding soil was achieved and the column loads were transferred to deeper layers of the EC-SC and resulted in higher values of settlement up to a vertical stress value of 640.0 kPa. Beyond this point, as the geotextile material expanded under the increased applied loading, better interaction between the geotextile and the surrounding soil was attained and under the same applied vertical stresses, lesser values of the settlement were obtained in the EC-SC compared with the NEC-SC.

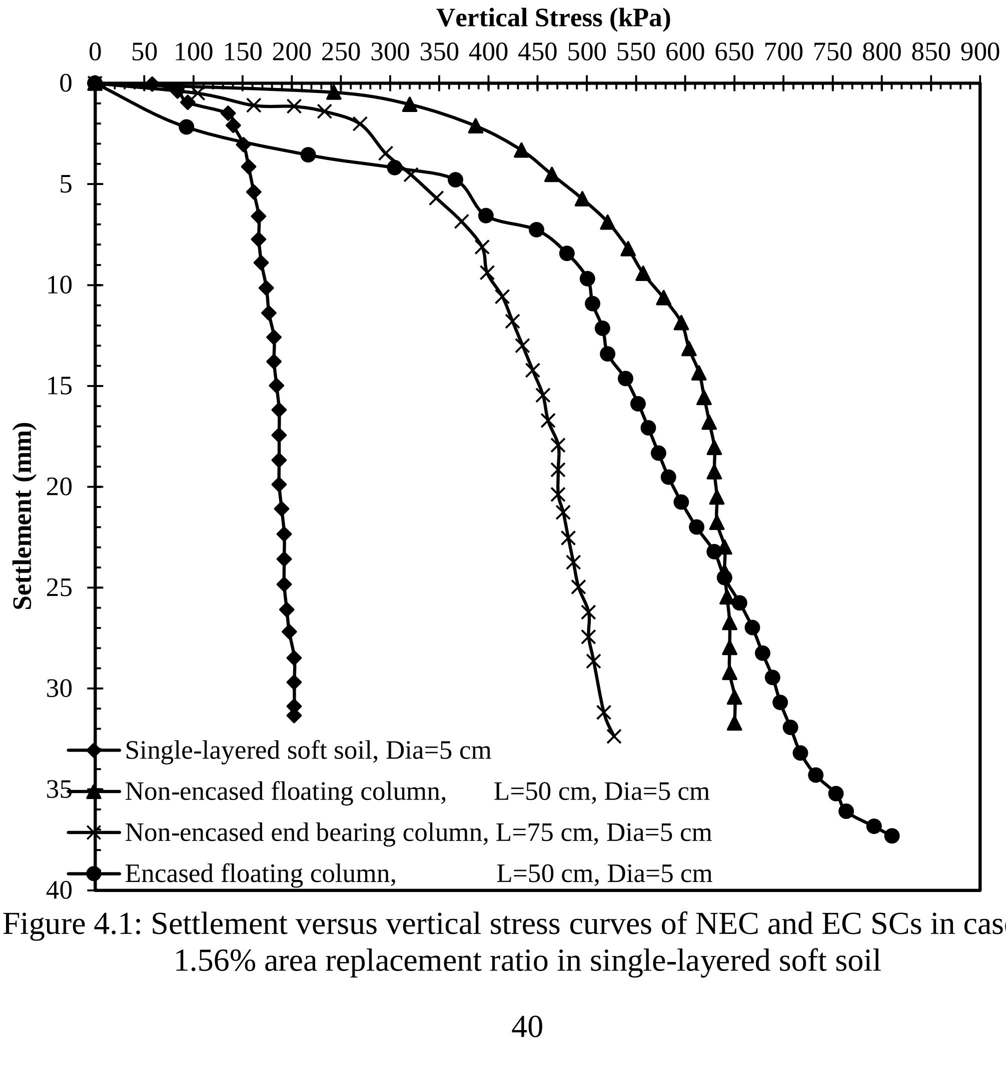


Figure 4.1: Settlement versus vertical stress curves of NEC and EC SCs in case of 1.56% area replacement ratio in single-layered soft soil

Figure 4.2 presents the vertical stress-settlement curves of single FSCs with and without encasement with an ARR of 6.25%. The figure indicated that the inclusion of NEC-FSC in soft soil increased the ultimate vertical stress of the soil. The ultimate vertical stress of the NEC-FSC was 328.1 kPa at 30 mm settlement. At initial loading stages, the EC-FSCs resulted in higher settlement values than the NEC-FSC. This behaviour was the same as in the behaviour of EC-FSC with an ARR of 1.56%. As aforementioned, this behaviour was due to the high tensile strength of the geotextile material used in the study. Under the applied lower vertical stress values, the geotextile did not show any lateral expansion and lateral load transfer of the SC to the surrounding soil did not occur. After reaching a vertical stress of 294.0 kPa, further improvement of the EC-FSC was gained due to the lateral interaction of the geotextile material with the surrounding soil. Figure 4.2 indicated that higher resistance to settlement of the EC-FSC was attained after reaching 21 mm settlement at a vertical stress value of 294.0 kPa.

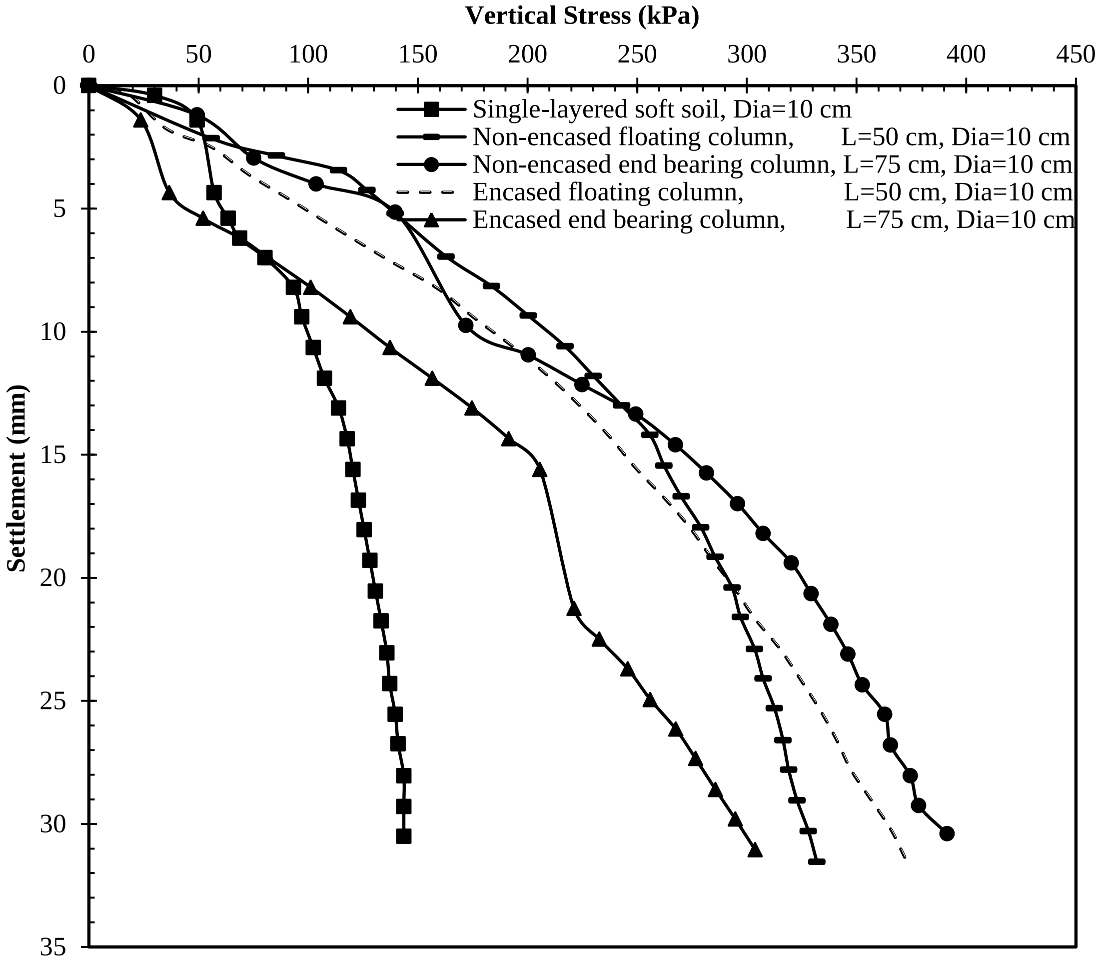


Figure 4.2: Settlement versus vertical stress curves of NEC and EC-SCs in case of 6.25% area replacement ratio in single-layered soft soil

The comparison of Figures 4.1 and 4.2 indicated that the NEC-FSC with ARR of 1.56% gave much better results than the NEC-FSC with ARR of 6.25%. Similar behavior was also obtained for the EC-FSCs. With a smaller ARR value of 1.56%, better improvement in the bearing capacity of the SC was gained (Murugesan and Rajagopal, 2009). This finding was in good harmony with the findings of Murugesan and Rajagopal (2009), who stated that the encasement effect of SC was inversely proportional to the SC's diameter.

4.2.1.1.2 Behaviour of end bearing stone columns, EBSCs

Figure 4.1 showed the vertical stress-settlement curve of the NEC-EBSC with an ARR value of 1.56%. When the vertical stress-settlement curve of the NEC-EBSC was compared with the single-layered soft soil, the figure indicated that the NEC-EBSC resulted in a significant improvement in the settlement behaviour of single-layered soft soil. The 30 mm settlement of the NEC-EBSC was obtained under the applied vertical stress value of 512.6 kPa whereas, for the single-layered soft soil, 30 mm settlement was obtained under the vertical stress value of approximately 202.6 kPa.

The vertical stress-settlement curves of the EC and NEC-EBSC with an ARR value of 6.25% were shown in Figure 4.2. The ultimate vertical stress of the NEC-EBSC was found to be around 386.8 kPa at 30 mm settlement. The EC- EBSC exhibited lower bearing capacity than the NEC-EBSC. The ultimate vertical stress value of the EC-EBSC was about 296.2 kPa at 30 mm settlement. This behaviour of the SC will be discussed in detail in Section 4.2.1.1.3.

The comparison of the values in Figures 4.1 and 4.2 indicated that the NEC-EBSC with ARR of 1.56% (L/D=15) resulted in much better improvement than the NEC-EBSC with ARR of 6.25% (L/D=7.5). These findings indicated that neither the effect of ARR, nor the effect of L/D on the SC bearing capacity should be considered separately. The two factors must be considered together so that the effect of diameter and length of the SC on soil improvement could be better evaluated. Considering only the L/D ratio of the SC, McKelvey et al. (2004) stated that SCs with L/D ratio in the range of 6.0-10.0 resulted in better soil improvement. In the present study, the NEC-EBSC with ARR of 6.25% and L/D=7.5, which is in the range of 6-10, resulted in less soil improvement compared to NEC-EBSC with ARR of 1.56% and L/D=15.0. Although the ARR value was lower, the higher L/D ratio of 15 resulted in better soil improvement.

4.2.1.1.3 Comparison of the behaviour of floating and end bearing stone columns with an ARR values of 1.56% and 6.25%

From Figure 4.1, it can also be seen that with the same diameter (5 cm), the NEC-FSC with length 50 cm (L/D=10) gave much better results than the NEC-EBSC with length 75 cm (L/D=15). SCs with shorter length resulted in better bearing capacity improvement. This finding was in good harmony with the findings of McKelvey et al. (2004), who stated that the recommended ratio of SC L/D should be between 6-10. The SCs with a length longer than 10 times the SC diameter would not provide any additional improvement in SC's bearing capacity (McKelvey et al., 2004).

Figure 4.2 indicated that the NEC-EBSC had a L/D ratio of 7.5, whereas the L/D ratio of the NEC-FSC was 5.0 (both with the same diameter of 10 cm). An increase in the L/D ratio of the NEC-EBSC resulted in an increase in the bearing capacity of the single-layered soft soil. The ultimate vertical stress of the NEC-EBSC was around 386.8 kPa at 30 mm settlement, whereas for the NEC-FSC, this value was around 328.0 kPa. These findings were in harmony with the findings of McKelvey et al. (2004) and Ali et al. (2010).

However, the ultimate vertical stress of EC-EBSCs with L/D ratio of 7.5 resulted in a lower ultimate vertical stress value than the EC- FSC with L/D ratio of 5.0, both with the same diameter of 10 cm (Figure 4.2). Under the same applied loading, the comparison of the improvement values obtained for the short and long columns indicated that lesser time was needed for the densification of the SC materials in shorter column before the lateral expansion of the geotextile took place. After the densification of the SC material in the shorter column, expansion of the geotextile material occurred as a result of which, some of the applied loads on the SC were transferred laterally to the surrounding soil and caused an increase in the soil-column interface shear resistance of the SC. This contributed to the SC bearing capacity and resulted in higher bearing resistance. While, in the EC longer SC, more time was needed for the densification of SC materials and because of this longer time, insufficient soil-column interface shear resistance was attained and smaller load-bearing resistance of the EC longer SC was achieved.

Figure 4.3 summarizes the results of the ultimate vertical stress of the NEC and EC-SCs with different ARRs values (different diameters).

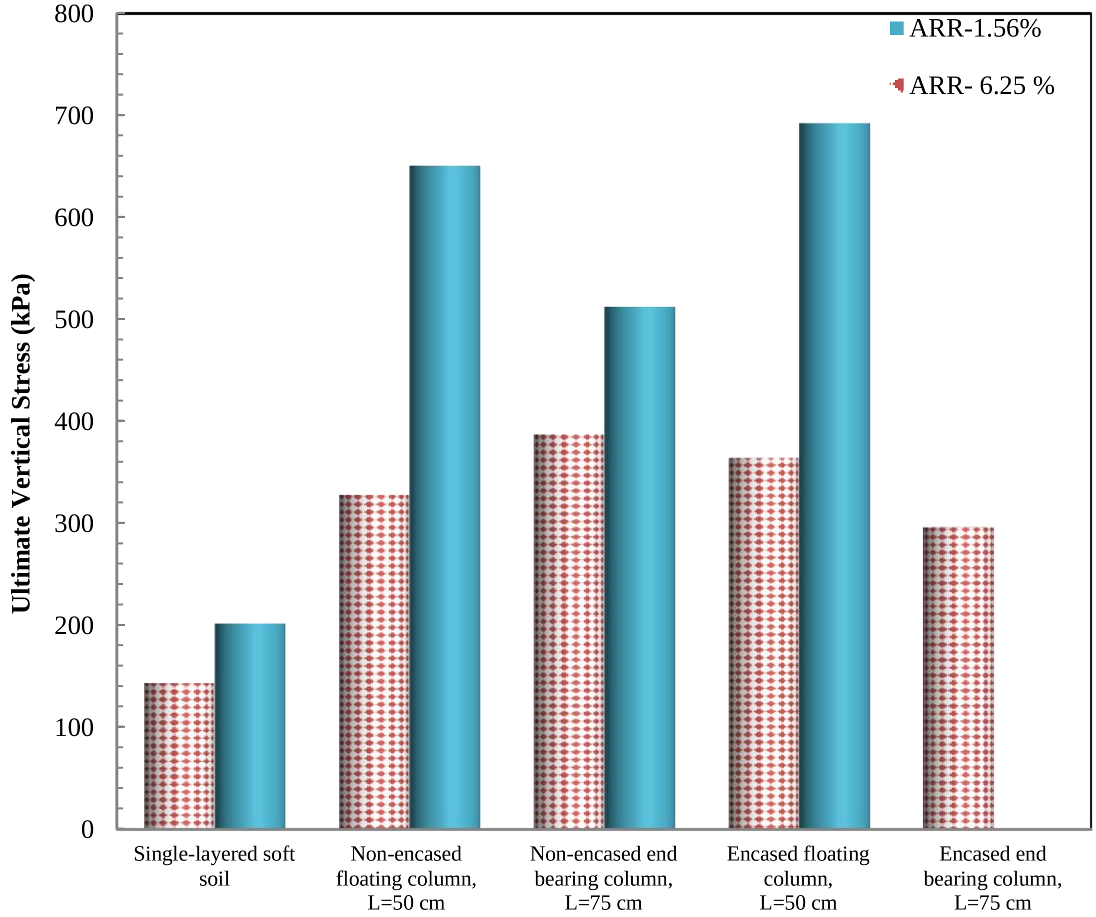


Figure 4.3: Ultimate vertical stress of NEC and EC-SCs with different ARRs in single-layered soft soil

The calculated ultimate vertical stress, σ_v, of the NEC-SCs from Equation 2.13 (Hughes and Withers, 1974) given in page 16 was compared with the measured ultimate bearing capacity of the NEC-SCs in single-layered soft soil. Table 4.1 shows the predicted and the measured ultimate vertical stress of NEC SCs with regards to the critical length of the SCs in single-layered soft soil.

Table 4.1: Comparison of the predicted and measured ultimate vertical stress values of NEC-SCs in single-layered soft soil with 5 and 10 cm diameter SCs (ARR: 1.56% and 6.25%)

Stone column		The measured ultimate vertical stress (kPa)	The calculated ultimate vertical stress, σ_v* (kPa)
Non-encased floating column	L=50 cm, Dia=5 cm	650.4	606.0
Non-encased end bearing column	L=75 cm, Dia=5 cm	512.6	606.0
Non-encased floating column	L=50 cm, Dia=10 cm	328.1	606.0
Non-encased end bearing column	L=75 cm, Dia=10 cm	386.8	606.0

*by Hughes and Withers (1974)

The SC critical length was defined as the length beyond which no further increase in the ultimate vertical stress could be achieved. From Equation 2.13, the critical length of the SC with 5 cm diameter was 50 cm and with 10 cm diameter, it was 100 cm. Figures 4.4, 4.5 and Table 4.1 exhibited the effect of critical length on the ultimate vertical stress of NEC-SCs in single-layered soft soil. From the figures, it could be seen that when the SC length has been increased close to the critical length, the ultimate vertical stress of the SCs increased which was in good harmony with the findings of Hughes and Withers (1974).

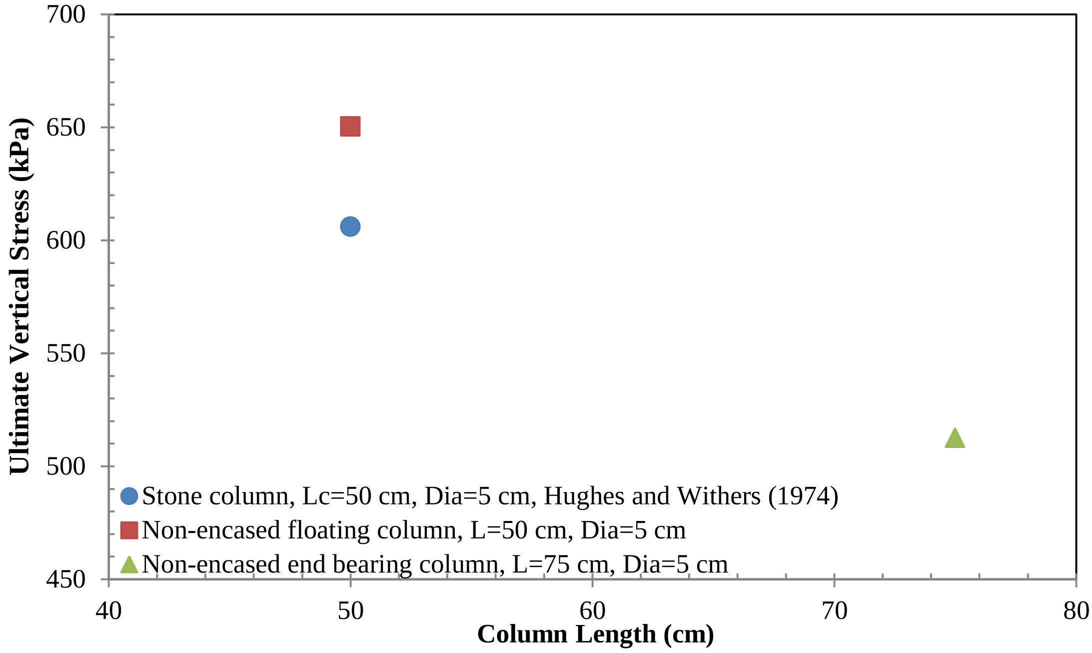


Figure 4.4: Effect of critical length on the ultimate vertical stress of NEC-SCs in single-layered soft soil in case of 5 cm diameter SC (1.56% ARR)

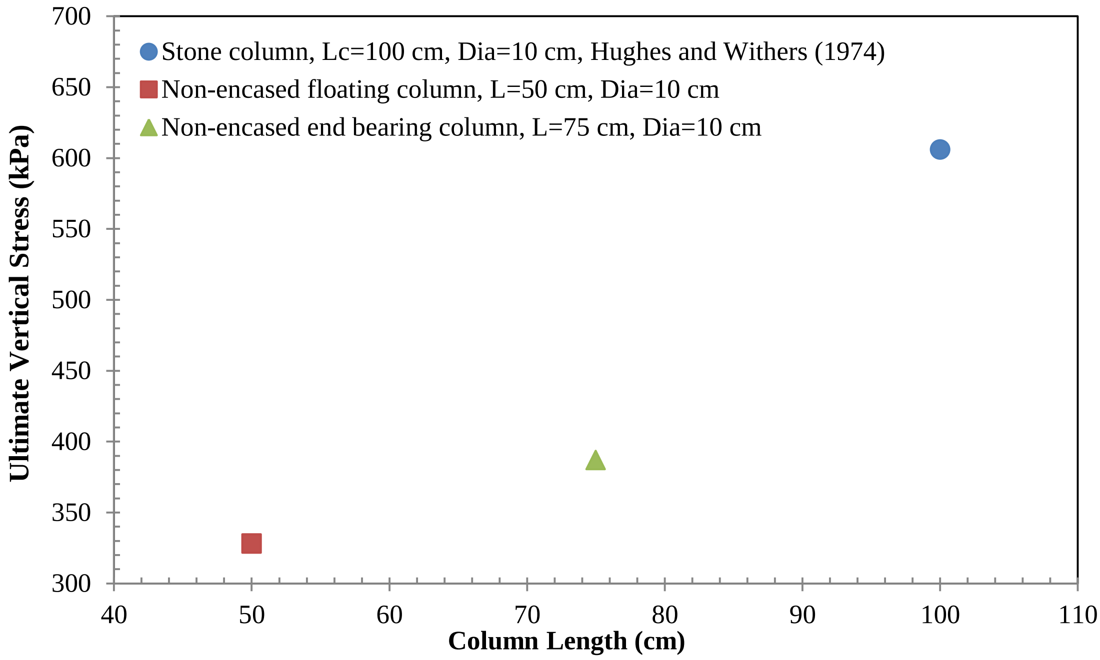


Figure 4.5: Effect of critical length on the ultimate vertical stress of NEC-SCs in single-layered soft soil in case of 10 cm diameter SC (6.25% ARR)

4.2.1.2 Vertical stress-settlement behaviour of single stone columns in layered soil

4.2.1.2.1 Behaviour of floating stone columns

Figure 4.6 presents the vertical stress-settlement curves of single SCs with and without encasement for both FSCs and EBSCs. The behaviour of the NEC and EC-FSCs will be discussed under this section. The ARR of the SCs in the model test tank was 1.56%. Figure 4.6 indicated that the NEC-FSC resulted in a significant amendment in the settlement behaviour of layered soil.

The ultimate vertical stress value of the NEC-FSC was about 186.4 kPa at 30 mm settlement. The EC-FSC produced a higher vertical stress value than the NEC-FSC after reaching 16 mm settlement. The vertical stress value of this SC at 16 mm settlement was 189 kPa and then due to the further expansion of the geotextile material under the applied stresses, this value increased and reached 367.5 kPa at 30 mm settlement.

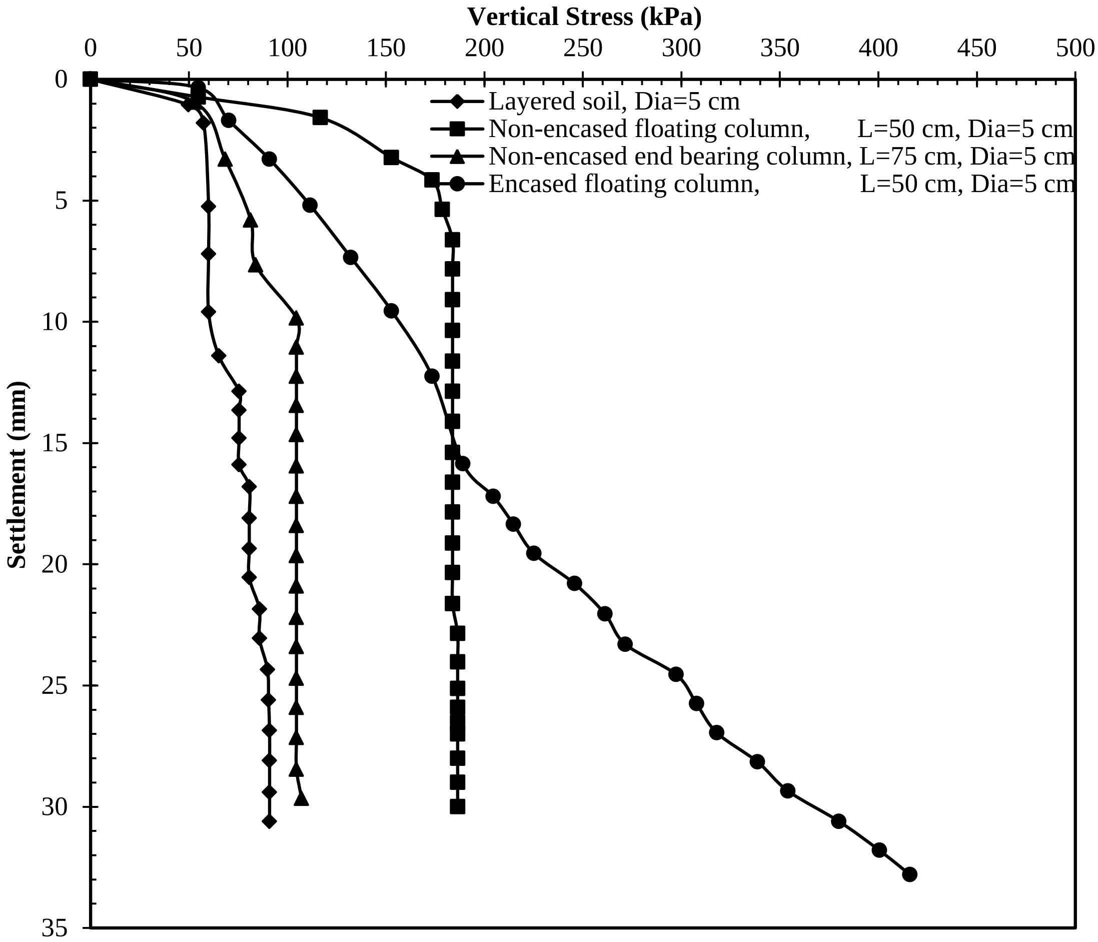


Figure 4.6: Settlement versus vertical stress curves of NEC and EC-SCs in case of 1.56% area replacement ratio in layered soil

Figure 4.7 presents the vertical stress-settlement curves of single FSCs with and without geotextile encasement with an ARR of 6.25%. The figure indicated that the inclusion of NEC-FSC in layered soil increased the ultimate vertical stress of the soil. The ultimate vertical stress of the NEC-FSC was 124.7 kPa at 30 mm settlement. The EC-FSCs resulted in higher ultimate vertical stress value than the NEC-FSC. The ultimate vertical stress value of the EC- FSC was nearly 240.5 kPa.

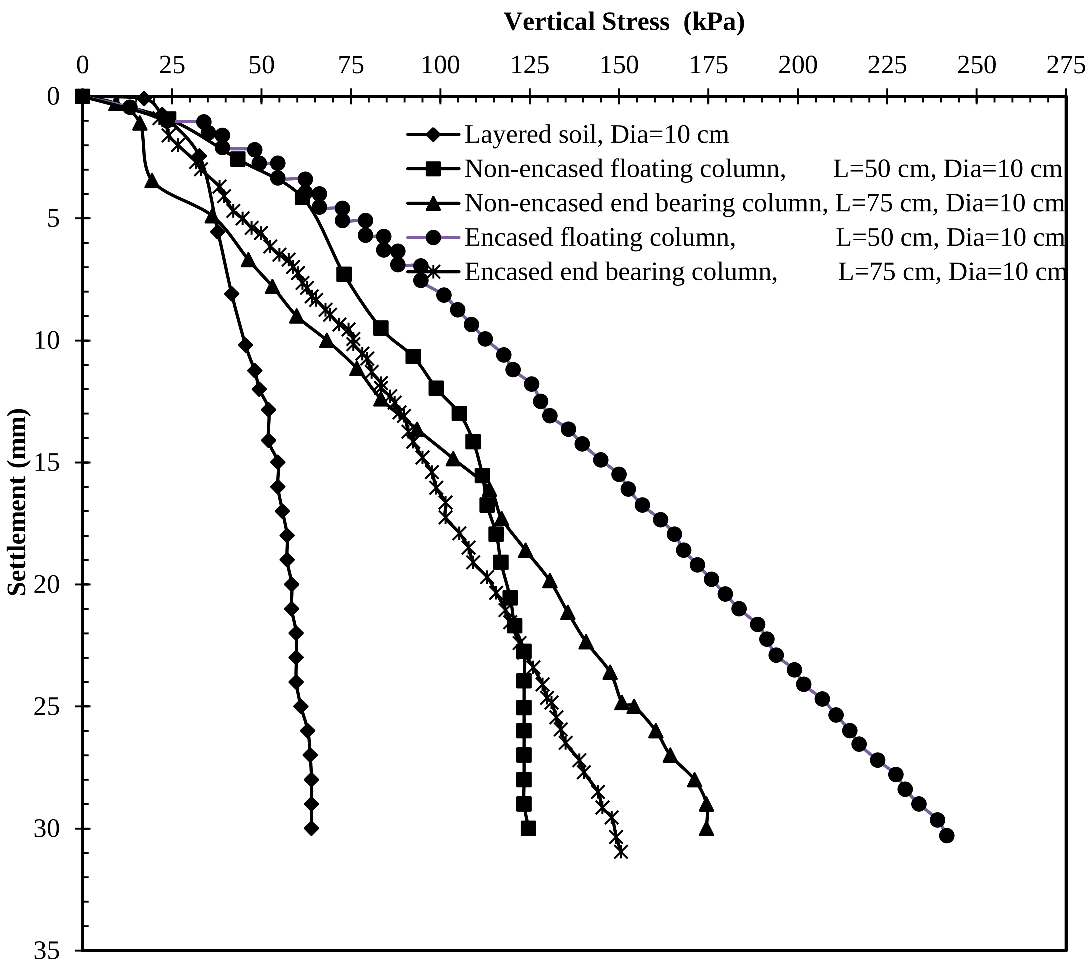


Figure 4.7: Settlement versus vertical stress curves of NEC and EC-SCs in case of 6.25% area replacement ratio in layered soil

The comparison of the findings in Figures 4.6 and 4.7 indicated that the NEC-FSC with ARR of 1.56% gave better bearing capacity amendment than the NEC-FSC with ARR of 6.25%. Similar behavior was also obtained for the EC- FSCs. With a smaller ARR value of 1.56%, better improvement in the SC's bearing capacity was obtained.

4.2.1.2.2 Behaviour of end bearing stone columns

Figure 4.6 presented the vertical stress-settlement curve of the NEC-EBSC with an ARR value of 1.56%. When the vertical stress-settlement curve of the NEC-EBSC was compared with the layered soil, the figure indicated that the NEC-EBSC resulted in a significant amendment in the settlement behaviour of layered soil. The 30 mm settlement of the NEC-EBSC was obtained under the applied vertical stress value of

109.3 kPa whereas, for the layered soil, 30 mm settlement was achieved under the stress value of nearly 90.9 kPa.

The vertical stress-settlement curves of the EC and NEC-EBSC with an ARR value of 6.25% were shown in Figure 4.7. The ultimate vertical stress of the NEC-EBSC was found to be around 174.5 kPa at 30 mm settlement. The EC-EBSC revealed lesser bearing capacity than the NEC-EBSC. The ultimate vertical stress value of the EC-EBSC was about 150.5 kPa at 30 mm settlement. Once again, that behaviour was explained due to further lateral expansion of the geotextile material under the increased loading.

As it can be seen from Figures 4.6 and 4.7, the NEC-EBSCs with ARRs of 1.56% (L/D=15) and of 6.25% (L/D=7.5) resulted in a considerable bearing capacity improvement of the layered soils.

4.2.1.2.3 Comparison of the behaviour of floating and end bearing stone columns with an ARR values of 1.56% and 6.25%

Figure 4.6 presented that the NEC-FSC with length 50 cm (L/D=10) gave better soil improvement than the NEC-EBSC with the length 75 cm (L/D=15). With the same diameter (5 cm), SCs with shorter length resulted in better bearing capacity improvement. This finding in layered soil was in good harmony with the findings of McKelvey et al. (2004).

Figure 4.7 indicated that the NEC-EBSC had a L/D ratio of 7.5, whereas the L/D ratio of the NEC-FSC was 5.0 (both with the same diameter of 10 cm). The ultimate vertical stress of the NEC-EBSC was around 174.5 kPa at 30 mm settlement, whereas for the NEC-FSC, this value was around 124.7 kPa. These findings were also in harmony with the findings of McKelvey et al. (2004) and Ali et al. (2010), even though the soil layering conditions were different.

Nevertheless, the ultimate vertical stress of EC-EBSCs with L/D ratio of 7.5 resulted in a lower ultimate vertical stress than the EC-FSC with L/D ratio of 5.0, both with the same diameter of 10 cm (Figure 4.7). The reason for such findings was aforementioned in detail in Section 4.2.1.1.3.

Figure 4.8 summarizes the results of the ultimate vertical stress of NEC and EC-SCs with different ARRs values in layered soil.

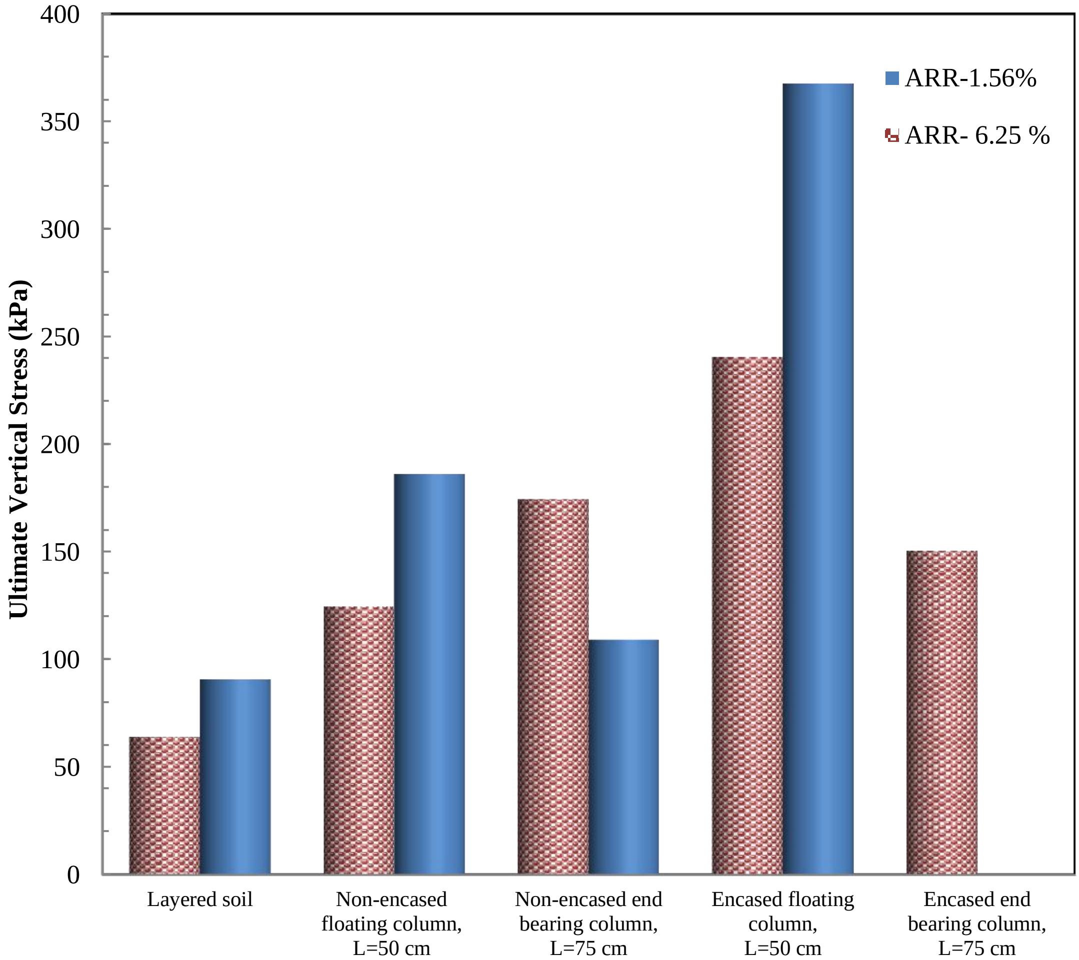


Figure 4.8: Ultimate vertical stress of NEC and EC-SCs with different ARRs in layered soil

4.2.1.3 Comparison of vertical stress-settlement behaviour of single SCs in single-layered and layered soils

The shape of the curves obtained in Figure 4.6 for the NEC floating, EC floating and NEC end bearing SCs in layered soil was very similar to the one obtained in Figure 4.1 for the single-layered soft soil. However, comparison of the values of the ultimate vertical stresses in single-layered and layered soils from Table 4.1 and 4.2 pointed out that there was a clear reduction in the ultimate vertical stress values obtained for the layered soil. That result could be explained due to the existence of loose sand overlaying the soft soil. Very little confinement provided by the surrounding loose sand caused a reduction in the layered soil bearing capacity, whereas, in single-layered soil, higher confinement of surrounding soft soil caused an increase in the ultimate vertical stress values of the SC.

Similarly, the comparison of the values of the ultimate vertical stress of SCs in single-layered and layered soils obtained in Figures 4.2 and 4.7 and also in Tables 4.2 and 4.3 pointed out the significant reduction in the ultimate vertical stress values obtained for the layered soil.

Table 4.2: Ultimate vertical stress of NEC and EC-SCs in case of single-layered soft soil with respect to ARR and column length

Stone column		ARR (%)	Ultimate vertical stress (kPa)
Single-layered soft soil	Dia=5 cm		202.6
Non-encased floating column	L=50 cm, Dia=5 cm		650.4
Non-encased end bearing column	L=75 cm, Dia=5 cm	1.56	512.6
Encased floating column	L=50 cm, Dia=5 cm		692.5
Single-layered soft soil	Dia=10 cm		143.6
Non-encased floating column	L=50 cm, Dia=10 cm		328.1
Non-encased end bearing column	L=75 cm, Dia=10 cm	6.25	386.8
Encased floating column	L=50 cm, Dia=10 cm		364.0
Encased end bearing column	L=75 cm, Dia=10 cm		296.2

Table 4.3: Ultimate vertical stress of NEC and EC SCs in case of layered soil with respect to ARR and column length

Stone column		ARR (%)	Ultimate vertical stress (kPa)
Layered soil	Dia=5 cm		90.9
Non-encased floating column	L=50 cm, Dia=5 cm		186.4
Non-encased end bearing column	L=75 cm, Dia=5 cm	1.56	109.3
Encased floating column	L=50 cm, Dia=5 cm		367.5
Layered soil	Dia=10 cm		64.0
Non-encased floating column	L=50 cm, Dia=10 cm		124.7
Non-encased end bearing column	L=75 cm, Dia=10 cm	6.25	174.5
Encased floating column	L=50 cm, Dia=10 cm		240.5
Encased end bearing column	L=75 cm, Dia=10 cm		150.5

4.2.2 Bearing improvement ratio of single stone column

To evaluate the SCs efficiency on soft soils bearing capacity, the bearing improvement ratio (BIR) was presented. The BIR represents the ratio of vertical stress of reinforced soil at a given settlement to vertical stress of unreinforced soil at the same settlement (q_r/q_u).

4.2.2.1 Bearing improvement ratio of single stone column in single-layered soft soil

Figure 4.9 and Table 4.4 illustrate the BIR of SCs in single-layered soft soil with and without encasement in case of 1.56% ARR. In the figure, the corresponding settlement values (s) to bearing improvement ratio were normalized by footing diameter (D) and the values were given as s/D in percentages.

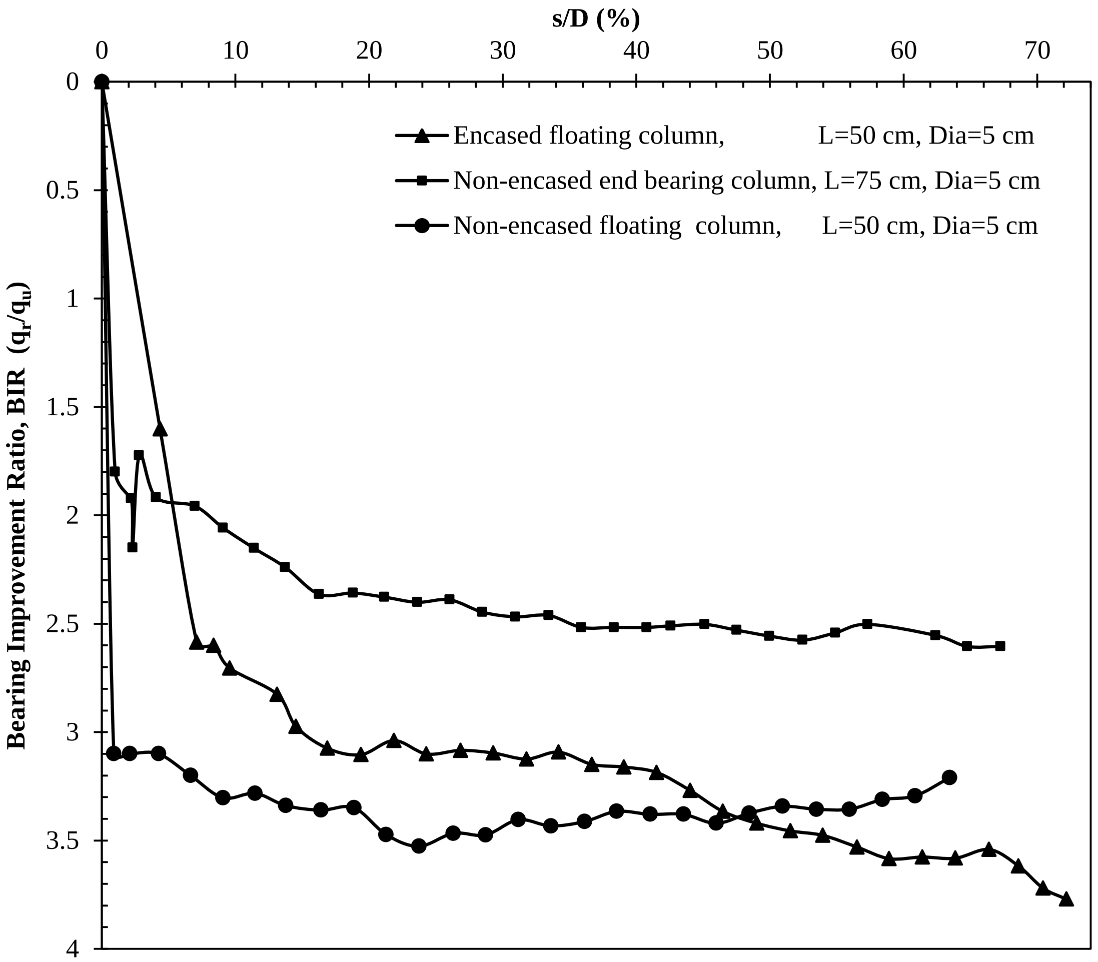


Figure 4.9: BIR of SCs in single-layered soft soil with and without encasement in case of 1.56% ARR

Table 4.4 gives the calculated BIRs corresponding to 30 mm settlement. From the table, it was clear that when the SC was EC with geotextile the BIR value increased. Compared with the BIR value of the NEC-FSC in single-layered soft soil, the BIR value of the EC- FSC in single-layered soft soil increased by 1.04 fold.

Table 4.4: BIR of SCs in case of single-layered soft soil with and without encasement in case of 1.56% ARR

Stone column		BIR (corresponding to 30 mm settlement)
Non-encased floating column	L=50 cm, Dia=5 cm	3.29
Encased floating column	L=50 cm, Dia=5 cm	3.42
Non-encased end bearing column	L=75 cm, Dia=5 cm	2.53

Figure 4.10 shows the BIR versus the normalized settlement curves of SCs in single-layered soft soil with and without encasement in the case of 6.25% ARR.

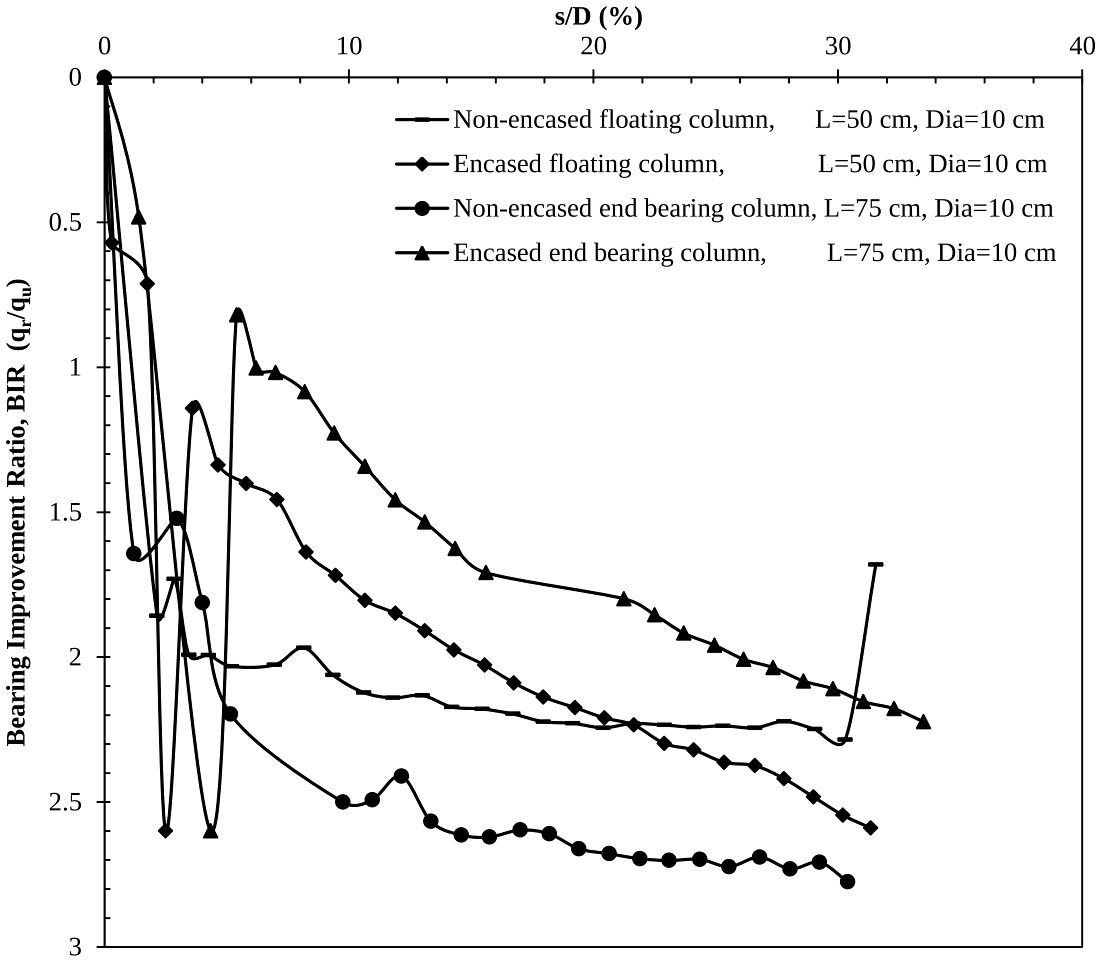


Figure 4.10: BIR of SCs in single-layered soft soil with and without encasement in case of 6.25% ARR

The BIR values corresponding to 30 mm settlement for NEC and EC-SCs were given in Table 4.5. With geotextile enhancement, the BIR value of the EC-FSC in single-layered soft soil increased by 1.11 times the BIR value of the NEC-FSC in single-layered soft soil. While in EC-EBSC case, the BIR value dropped around 1.29 folds compared to the NEC-EBSC in single-layered soft soil.

Table 4.5: BIR of SCs in case of single-layered soft soil with and without encasement in case of 6.25% ARR

Stone column		BIR (corresponding to 30 mm settlement)
Non-encased floating column	L=50 cm, Dia=10 cm	2.28
Encased floating column	L=50 cm, Dia=10 cm	2.54
Non-encased end bearing column	L=75 cm, Dia=10 cm	2.75
Encased end bearing column	L=75 cm, Dia=10 cm	2.12

From Tables 4.4 and 4.5, it can be seen that among the NEC SCs, the NEC-FSC with L=50 cm and Dia=5 cm (1.56% ARR) had the best bearing capacity improvement. Whereas, for the EC-SCs, the EC-FSC with L=50 cm and Dia=5 cm (1.56% ARR) had the superior improvement to the bearing capacity among all the EC-SCs in single-layered soft soil.

4.2.2.2 Bearing improvement ratio of single stone column in layered soil

Figure 4.11 presents the BIR versus the normalized settlement curves of SCs in layered soil with and without encasement in case of 1.56% ARR.

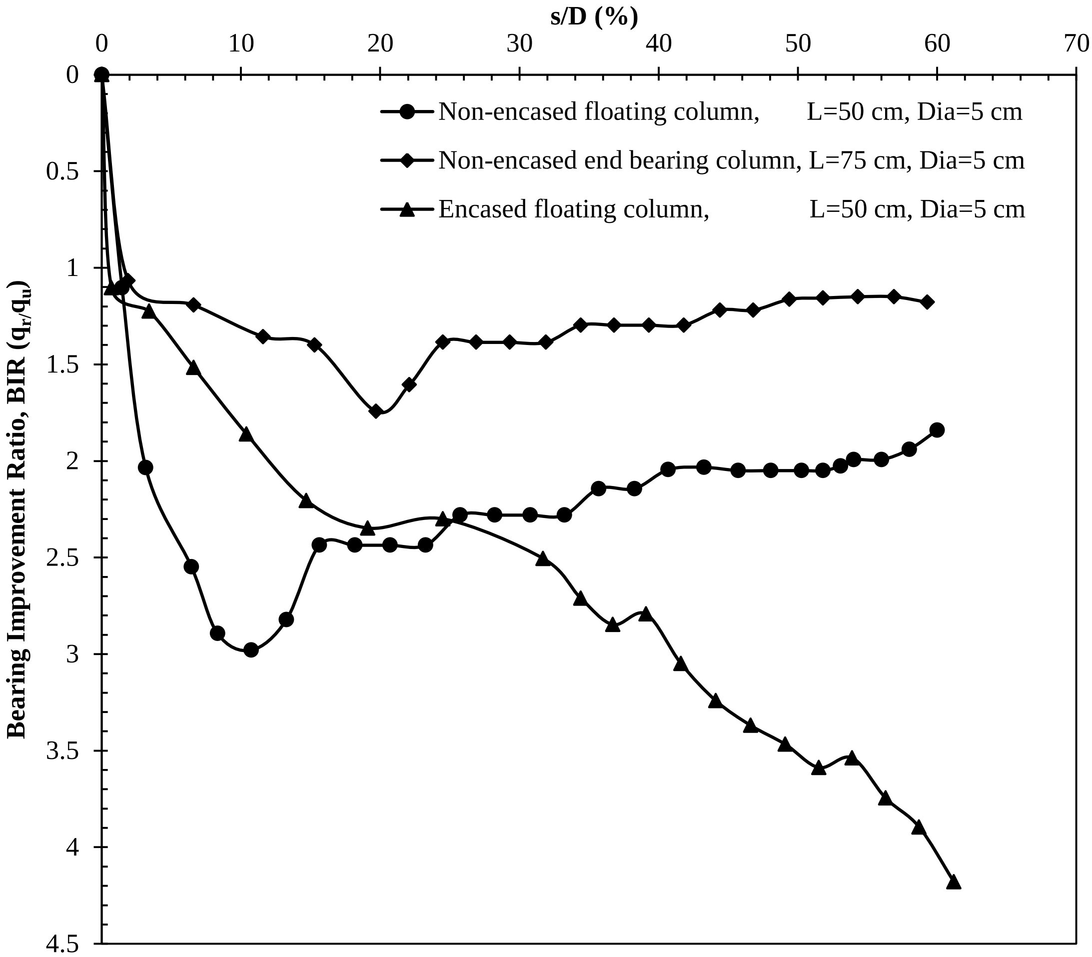


Figure 4.11: BIR of SCs in layered soil with and without encasement in case of 1.56% ARR

Table 4.6 presents the calculated BIRs values corresponding to 30 mm settlement. From the table, it can be seen that the BIR value improved with geotextile encasement of the SC. With geotextile enhancement, the BIR value of the EC- FSC in layered soil increased by 1.97 times the BIR value of the NEC-FSC in layered soil.

Table 4.6: BIR of SCs in case of layered soil with and without encasement in case of 1.56% ARR

Stone column		BIR (corresponding to 30 mm settlement)
Non-encased floating column	L=50 cm, Dia=5 cm	2.05
Encased floating column	L=50 cm, Dia=5 cm	4.04
Non-encased end bearing column	L=75 cm, Dia=5 cm	1.20

Figure 4.12 shows the BIR versus the normalized settlement curves of SCs in layered soil with and without encasement in case of 6.25% ARR.

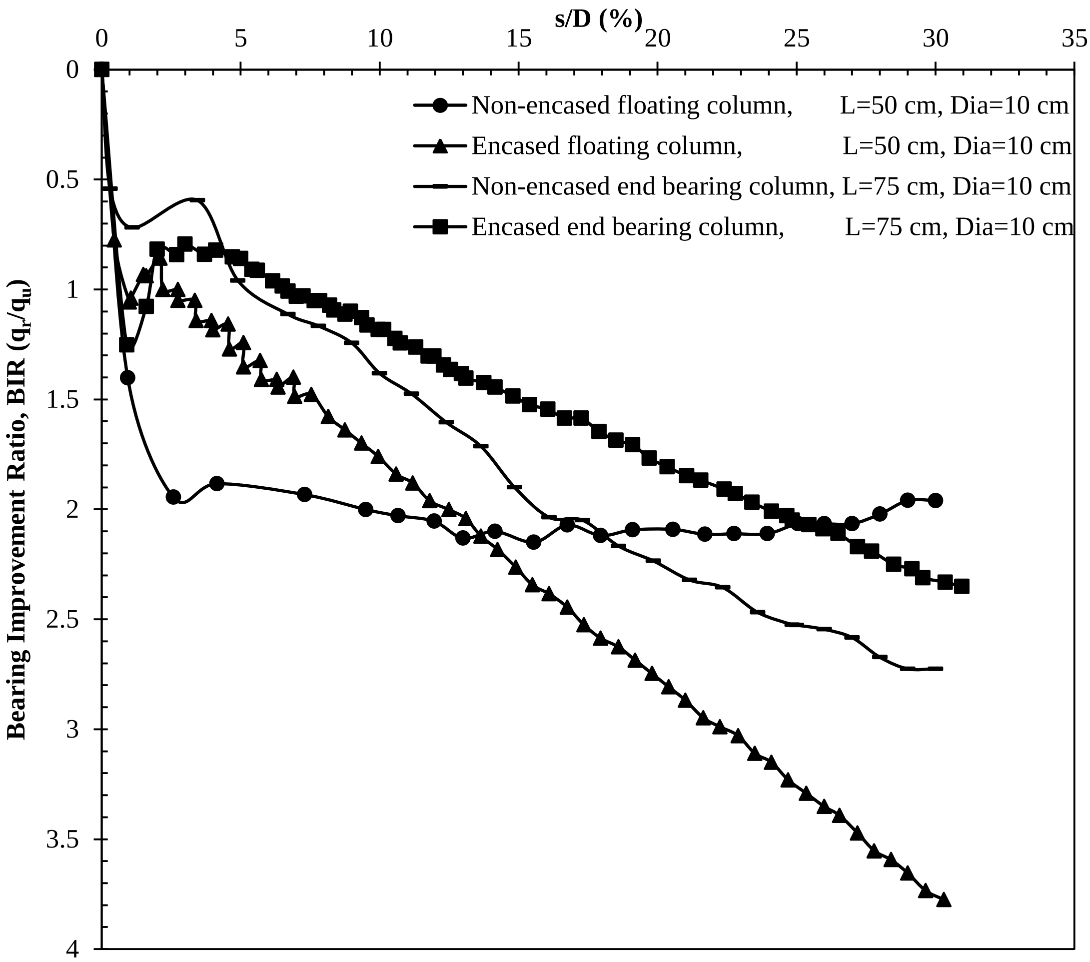


Figure 4.12: BIR of SCs in layered soil with and without encasement in case of 6.25% ARR

The BIR values corresponding to 30 mm settlement for NEC and EC-SCs were given in Table 4.7. With geotextile enhancement, the BIR value of the EC- FSC in layered soil increased by 1.93 times the BIR value of the NEC-FSC in layered soil. While in the case of EC-EBSC, the BIR value dropped about 1.16 folds compared to NEC-EBSC in layered soil.

Table 4.7: BIR of SCs in case of layered soil with and without encasement in case of 6.25% ARR

Stone column		BIR (corresponding to 30 mm settlement)
Non-encased floating column	L=50 cm, Dia=10 cm	1.95
Encased floating column	L=50 cm, Dia=10 cm	3.76
Non-encased end bearing column	L=75 cm, Dia=10 cm	2.73
Encased end bearing column	L=75 cm, Dia=10 cm	2.35

4.2.2.3 Comparison of bearing improvement ratio of single stone columns in single-layered and layered soils

From Tables 4.4 and 4.6, it can be seen that in both cases of soil layering conditions, the BIR values increased with geotextile encasement of the SCs with 5 cm diameters. The BIR value of the NEC-FSC in single-layered soft soil was approximately 1.60 times higher than the BIR value of the NEC-FSC in layered soil. Also, the BIR value of the NEC-EBSC in single-layered soft soil was approximately 2.10 times higher than the BIR value of the NEC-EBSC in layered soil. This was explained due to the inadequate lateral confinement provided by the loose sand, which caused a reduction in the BIR value of the NEC-SCs in layered soil.

From Tables 4.5 and 4.7, it can be seen that in both cases of soil layering conditions, the BIR values increased with geotextile encasement of the FSCs with 10 cm diameters. The BIR value of the NEC-FSC in single-layered soft soil was approximately 1.17 times higher than the BIR value of the NEC-FSC in layered soil. Also, the BIR value of the NEC-EBSC in single-layered soft soil was approximately similar to the BIR value of the NEC-EBSC in layered soil.

From Tables 4.4-4.7, the findings indicated the effectiveness of geotextile in the improvement of bearing capacity of SCs. The inclusion of geotextile in the FSCs increased the bearing capacity of both single-layered and layered soils and resulted in higher BIR values compared to the BIR values of the NEC-FSCs in case of 5 cm and 10 cm diameters.

4.2.3 Subgrade modulus of single stone column, *k*

Subgrade modulus is defined as a soil settlement under specific stress. Therefore, subgrade reaction modulus k is given by q_{ult}/s, where q_{ult} is the ultimate vertical of soil and s is the corresponding settlement at that point.

4.2.3.1 Subgrade modulus of single stone column in single-layered soft soil

Table 4.8 presents the subgrade modulus of SCs in case of single-layered soft soil with and without encasement in case of 1.56% and 6.25% ARR.

From Tables 4.8, it can be seen that in all SCs applications, a remarkable amendment of the subgrade modulus values of the single-layered soft soil was achieved. In the case of NEC SCs, the superior amendment to the single-layered soft soil was achieved with the application of NEC-FSC with ARR of 1.56%. Whereas, the EC-FSC with ARR 1.56 % had the most significant improvement of subgrade modulus of single-layered soft soil among all SC applications.

Table 4.8: Subgrade modulus of NEC and EC-SCs in case of single-layered soft soil with respect to ARR and column length

Stone column		ARR (%)	Subgrade modulus, k (kN/m^3)
Single-layered soft soil	Dia=5 cm		6753.3
Non-encased floating column	L=50 cm, Dia=5 cm		21680.0
Non-encased end bearing column	L=75 cm, Dia=5 cm	1.56	17086.7
Encased floating column	L=50 cm, Dia=5 cm		23083.3
Single-layered soft soil	Dia=10 cm		4786.7
Non-encased floating column	L=50 cm, Dia=10 cm		10936.7
Non-encased end bearing column	L=75 cm, Dia=10 cm	6.25	12893.3
Encased floating column	L=50 cm, Dia=10 cm		12133.3
Encased end bearing column	L=75 cm, Dia=10 cm		9873.3

4.2.3.2 Subgrade modulus of single stone column in layered soil

Table 4.9 presents the subgrade modulus of SCs in case of layered soil with and without encasement in case of 1.56% and 6.25% ARR.

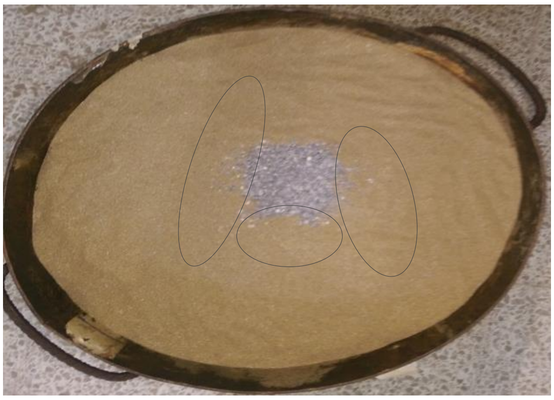
From Tables 4.9, it can be seen that in all SCs applications, a significant improvement of the subgrade modulus values of the layered soil was reached. In the case of NEC SCs, the greater amendment to the layered soil was reached with the application of NEC-FSC with ARR of 1.56%. Whereas, the EC- FSC with ARR 1.56% had the most considerable improvement of subgrade modulus of layered soil among all SC applications.

Table 4.9: Subgrade modulus of NEC and EC-SC in case of layered soil with respect to ARR and column length

Stone column		ARR (%)	Subgrade modulus, k (kN/m^3)
Layered soil	Dia=5 cm		3030.0
Non-encased floating column	L=50 cm, Dia=5 cm		6213.3
Non-encased end bearing column	L=75 cm, Dia=5 cm	1.56	3643.3
Encased floating column	L=50 cm, Dia=5 cm		12250.0
Layered soil	Dia=10 cm		2133.3
Non-encased floating column	L=50 cm, Dia=10 cm		4156.7
Non-encased end bearing column	L=75 cm, Dia=10 cm	6.25	5816.7
Encased floating column	L=50 cm, Dia=10 cm		8016.7
Encased end bearing column	L=75 cm, Dia=10 cm		5016.7

4.2.4 Bulging failure of single stone columns

Figure 4.13 shows the picture of the SCs with and without geotextile encasement in layered soil after loading. In this study, in the case of SC in layered soil, the bulging failure of the SC after loading could not be investigated because of inadequate lateral confinement provided by the loose sand surrounding the SC. Under the applied loading, the SC materials spread out unevenly toward the loose sand and did not show typical bulging failure. For this reason, the deformed shapes of the SC from the hardened cement paste in layered soil could not be obtained. Uneven spreading of the column materials into the surrounding loose sand in layered soil without encasement can be seen in Figure 4.13 a.



(a)

(b)

Figure 4.13: SCs in layered soil after loading a) without encasement b) with encasement

In the present study, the L/D ratios of the EC and NEC-SCs were 10, 15, 5, 7.5 which were equal or greater than 5. According to many researchers (Barksdale and Bachus, 1983; Christoulas et al., 2000; Shahu et al., 2000; Ghazavi and Afshar, 2013; Chen et al., 2015; Hong el at., 2016), the bearing capacity of the EC and NEC SCs was controlled by bulging failure. Depicted images of the hardened deformed shapes of the SCs were shown in Figure 4.14. The SCs deformed shapes were measured and the SCs bulging were calculated. The calculated SCs bulging were given in Figures 4.15-4.18 in which all the curves were presented in the form of non-dimensional bulging profiles.

(a) (b)

Figure 4.14: Hardened deformed shapes of the SCs a) NEC-EBSC with 5cm diameter b) NEC-EBSC with 10 cm diameter

Figure 4.15 shows the non-dimensional bulging profile for NEC and EC- FSCs in single-layered soft soil in case of 1.56% ARR. In the figure, bulging was expressed as a percentage of the diameter of the SC (D_b-D_0)/D_0 (%), where D_b is the bulging diameter of SC and D_0 is the original SC diameter. The profile showed the change of the non-dimensionalised bulging with depth normalization (Z/D_0), where Z is the SC total length. According to McKelvey et al. (2004), the axial load transfer from the foundation to the SC was higher near SC's top part whereas, in the lower part, the SC carried little loads or no load transfer. In the present study, for the NEC-FSC, the maximum bulging was found to occur at a depth of 1.0 times the SC original diameter from the top of the SC (5.0 cm) whereas, for EC-FSC, the maximum bulging was found to occur at a depth of 3.0 times the original diameter of the SC from the top (15.0 cm). The maximum bulging for NEC and EC- FSCs were 16.5% and 11.4%, respectively. It became clear that whenever the SC was EC in geotextile, the maximum bulging depth increased whereas the maximum bulging diameter was reduced by about 30% compared to the NEC-FSC. In consequence to the additional SC confinement provided by the geotextile, excessive bulging into the surrounding soft soil was prevented and the applied axial load was transferred to deeper layers, resulting in higher amendment of the EC-SC.

The total length of the NEC-FSC experiencing bulging was found to be within the depth of 0.0-17.5 cm ($3.5D_0$) of the SC whereas, in the EC- FSC, the bulging total length was within the depth of 2.5 cm to 27.5 cm ($0.5D_0$-$5.5D_0$) of the SC.

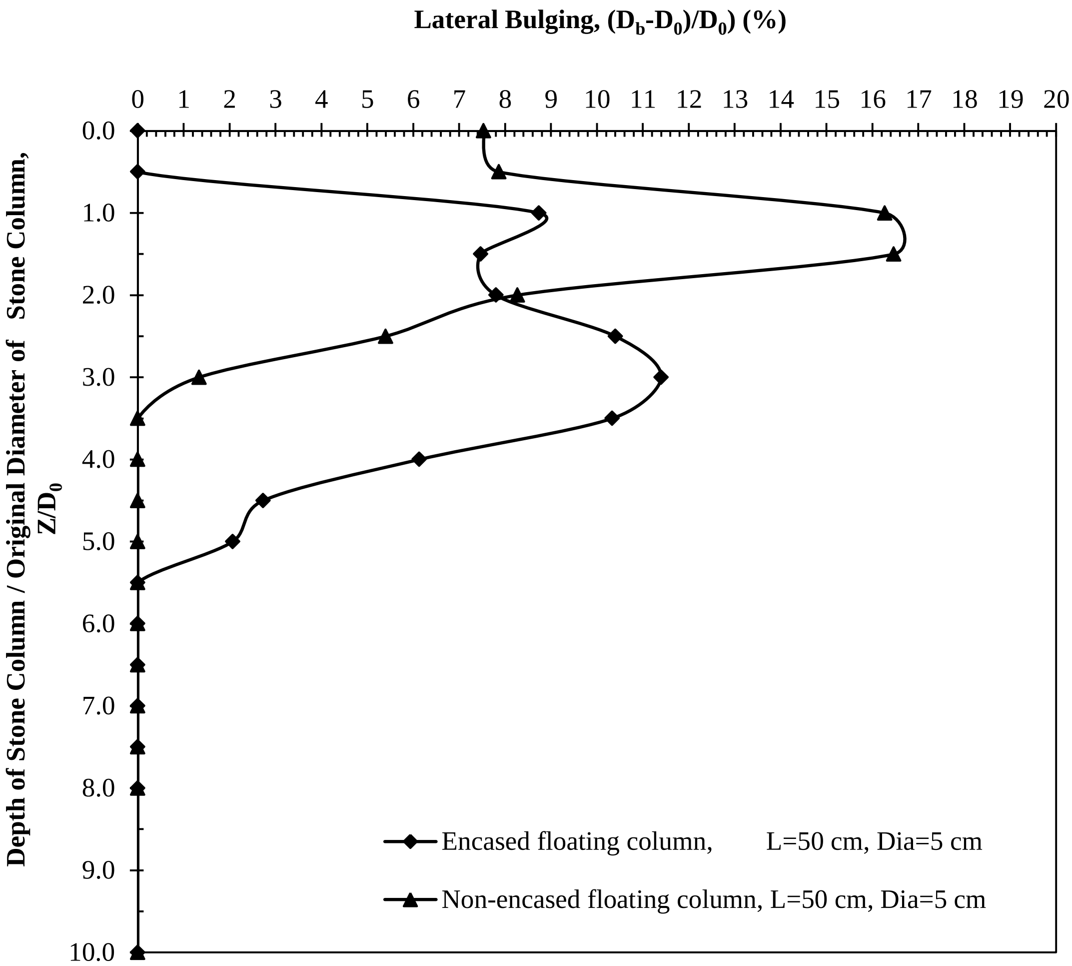


Figure 4.15: Change of the non-dimensionalised bulging profile with depth normalization for NEC and EC-FSCs in single-layered soft soil in case of 1.56% ARR

Figure 4.16 shows the non-dimensional bulging profile for NEC-EBSCs in single-layered soft soil in case of 1.56% ARR. In the figure, the maximum bulging occurred at a depth of 1.5 times the original diameter of the SC from top (7.5 cm). The maximum bulging for the NEC-EBSC was 23.0%.

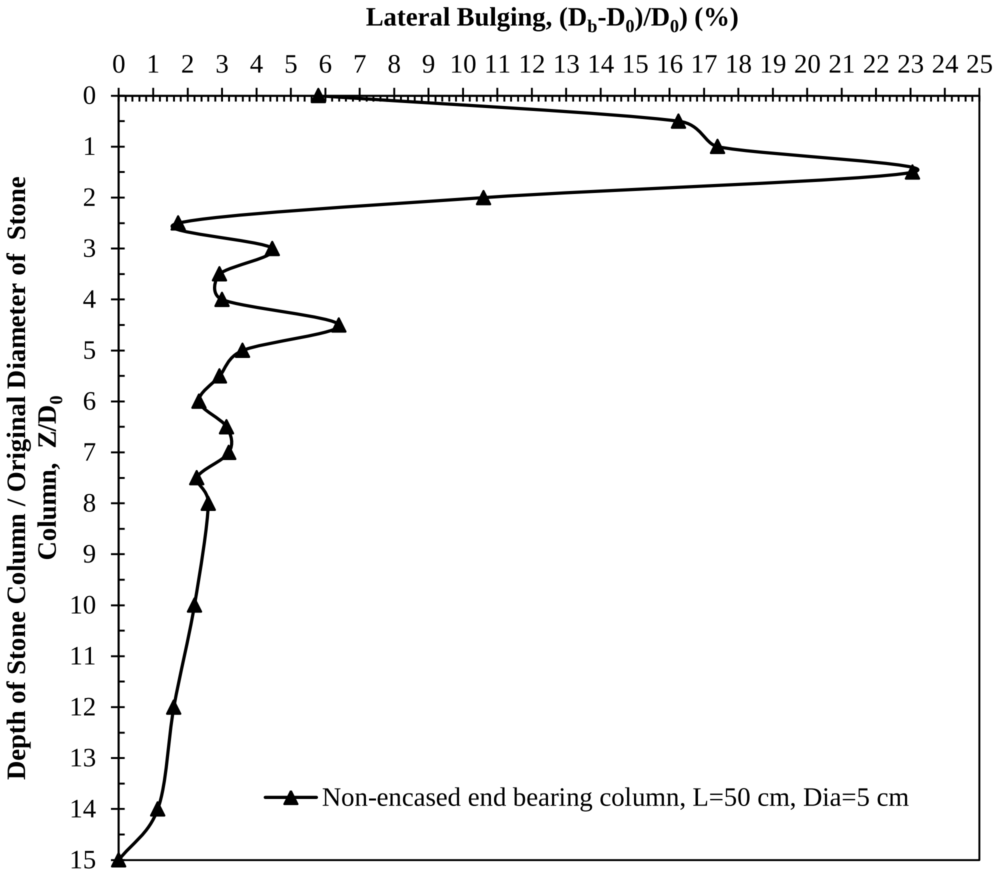


Figure 4.16: Change of the non-dimensionalised bulging profile with depth normalization for NEC-EBSC in single-layered soft soil in case of 1.56% ARR

From Figures 4.15 and 4.16, it can be seen that the diameter and the depth of maximum bulging of NEC-EBSC with L/D=15 ratio were higher than the non-EC-FSC with L/D=10. The significant deformation of the NEC-EBSC due to bulging resulted in little or no load transferred to SC lower parts. As a result, significant reduction in the ultimate vertical stress of NEC-EBSC was obtained, as can be seen in Figure 4.3. These findings were in harmony with the findings of McKelvey et al. (2004) and Ali et al. (2010).

Table 4.10 presented the comparison of the bulging failure for NEC floating, EC floating and NEC-EBSCs in single-layered soft soil with 1.56% ARR with the existing findings in the literature. The comparison of these findings indicated that the obtained results of the present study were in harmony with the findings of Deb et al., 2011; Debnath and Dey, 2017; Hasan and Samadhiya, 2017.

Table 4.10: Bulging failure for NEC floating, EC floating and NEC end bearing SCs in single-layered soft soil with 1.56% ARR (5 cm diameter)

Stone column	Bulging depth and diameter	Present study	Hasan and Samadhiya (2017)	Deb et al. (2011)	Debnath and Dey (2017)
Non-encased floating stone column	Max bulging depth	5.0 cm from the surface ($1.0^* D_0$)	$(1\text{-}1.6)^* D_0$	$1.20^* D_0$	--
	Max bulging diameter	5.82 cm ($1.16^* D_0$)	--	$1.24^* D_0$	--
Encased floating stone column	Max bulging depth	15 cm from the surface ($3^* D_0$)	--	--	$2.84^* D_0$
	Max bulging diameter	5.57 cm ($1.114^* D_0$)	--	--	$1.08^* D_0$
Non-encased end bearing stone column	Max bulging depth	7.5 cm from the surface ($1.5^* D_0$)	$(1\text{-}1.6)^* D_0$	$1.20^* D_0$	--
	Max bulging diameter	6.15 cm ($1.23^* D_0$)	--	$1.24^* D_0$	--

D_0= Original diameter of the SC.

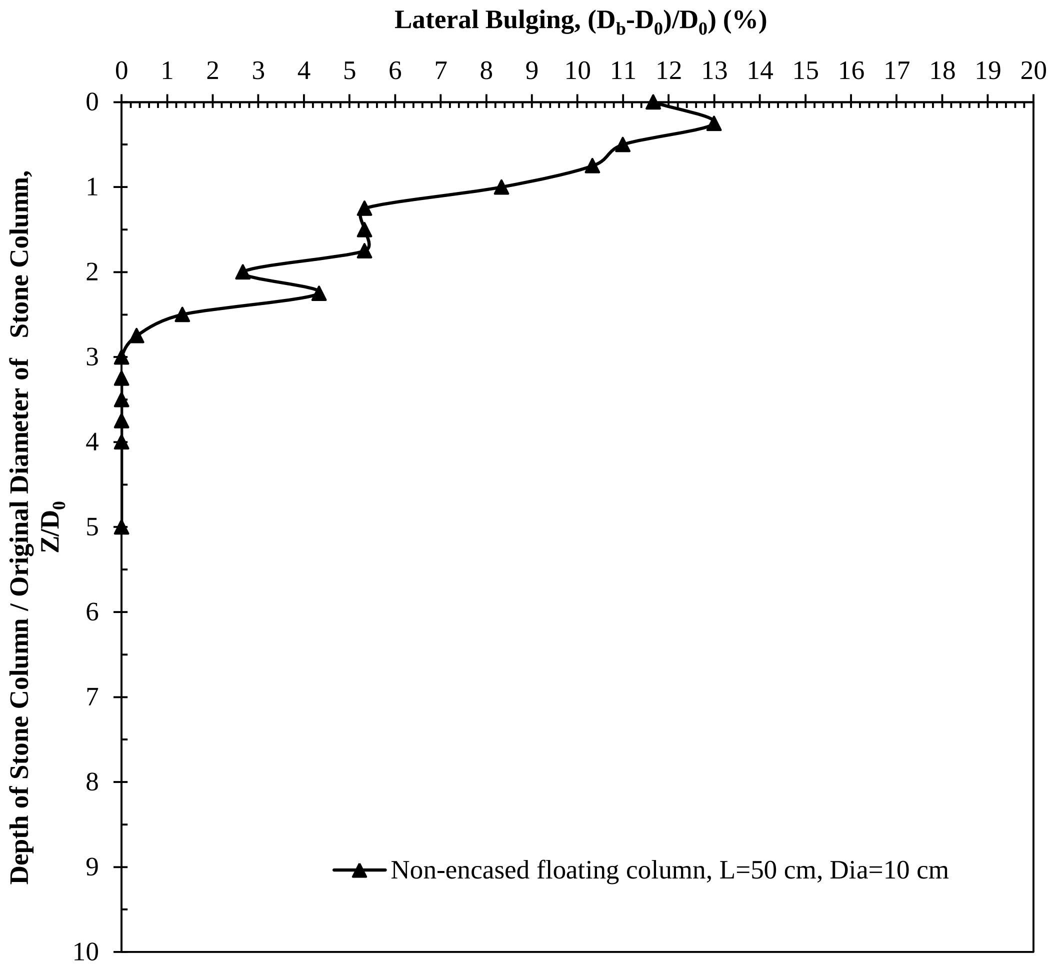


Figure 4.17: Change of the non-dimensionalised bulging profile with depth normalization for NEC-FSC in single-layered soft soil in case of 6.25% ARR

Figure 4.17 shows the non-dimensional bulging profile for NEC-FSC in single-layered soft soil in case of 6.25% ARR. In the figure, the maximum bulging occurred at a depth of 0.25 times the original diameter of the SC from top (2.5 cm). The maximum bulging for NEC-FSC was 13.0%. The total length of the NEC-FSC experiencing bulging was found to be within the depth of 0.0-30.0 cm ($3.0D_0$) of the SC. While in the case of the EC- FSC with 6.25% ARR, the bulging failure was not confronted.

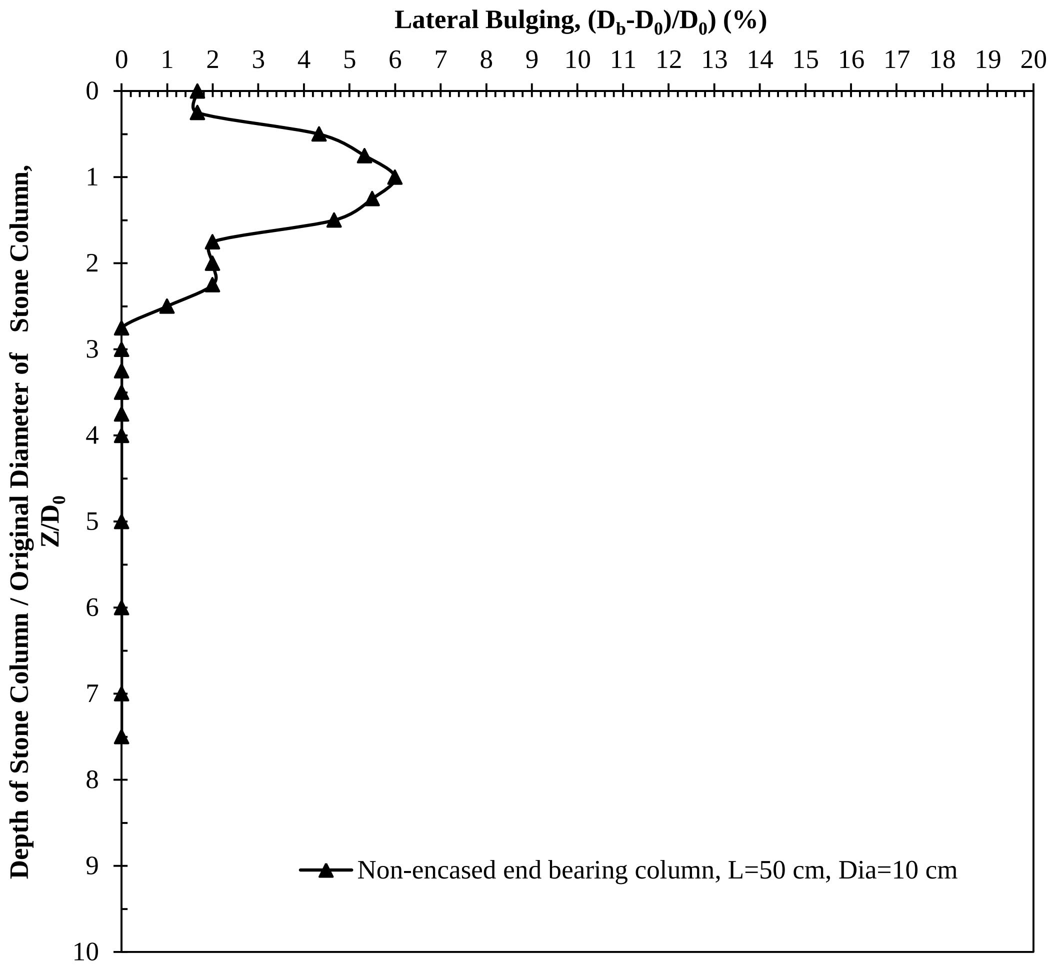


Figure 4.18: Change of the non-dimensionalised bulging profile with depth normalization for NEC-EBSC in single-layered soft soil in case of 6.25% ARR

Figure 4.18 shows the non-dimensional bulging profile for NEC-EBSC in single-layered soft soil in case of 6.25% ARR. In the figure, the maximum bulging was found to occur at a depth of 1.0 times the original diameter of the SC from top (10 cm). The maximum bulging for NEC-EBSC was 6.0%. The total length of the NEC-FSC experiencing bulging was found to be within the depth of 2.5-25.0 cm (0.25-2.5D_0) of the SC. While in the case of EC-EBSC with 6.25% ARR, the bulging failure was not confronted.

From Figures 4.17, 4.18 and Table 4.11, it can be seen that the diameter of maximum bulging of NEC-EBSC with L/D=7.5 ratio was lower than the NEC-FSC with

L/D=5.0. The significant deformation of the NEC-FSC due to bulging resulted in little or no load transferred SC lower parts. As a result, a significant drop in the ultimate vertical stress of the NEC-FSC was obtained, as it can be seen in Figure 4.3.

Table 4.11: Bulging failure of NEC-FSC and NEC-EBSCs in single-layered soft soil with 6.25% ARR (10 cm diameter column)

Stone column	Bulging depth and diameter	Values
Non-encased floating stone column	Max bulging depth	2.5 cm from the surface ($0.25^* D_0$)
	Max bulging diameter	11.3 cm ($1.13^* D_0$)
Non-encased end bearing stone column	Max bulging depth	10.0 cm from the surface ($1.0^* D_0$)
	Max bulging diameter	10.6 cm ($1.06^* D_0$)

4.3 Behaviour of group of floating stone columns in single-layered and layered soils (considering the central column)

In this section, the vertical stress-settlement behaviour of SCs group, subgrade modulus, k, and bulging failure of SCs in single-layered and layered soils will be discussed. For the group of NEC-FSCs with 5 cm diameter, two spacing to diameter ratios, s/D were used which were 2.5 and 1.5. The encasement was only applied for the best performance of group of NEC-SCs which was the group of NEC-FSCs with 1.5xD.

4.3.1 Vertical stress-settlement behaviour of group of columns

In this section, the vertical stress-settlement behaviour of the central SC surrounded by SCs will be discussed.

4.3.1.1 Vertical stress-settlement behaviour of central columns in single-layered soft soil

Figure 4.19 presents the vertical stress-settlement curves of the central column among group of SCs in case of single-layered soft soil with and without encasement.

As it can be seen from Figure 4.19, the application of NEC floating central column among group of SCs with s/D ratio of 2.5 improved the bearing capacity of single-layered soft soil by 1.42 times. Whereas for the NEC floating central column among group of SCs with s/D ratio of 1.5, the increase in the bearing capacity of the single-layered soft soil was found to be 3.6 times and with geotextile encasement, the bearing capacity of the single-layered soft soil increased by 3.88.

It is clear from Figure 4.19 that the ultimate vertical stress of the central NEC floating column was increased with reducing the spacing. This behaviour could be explained due to better confinement for the central column provided by smaller spacing of the SCs, which enhanced the bearing capacity of the column. This finding was also in good harmony with the finding of Das and Dey (2018).

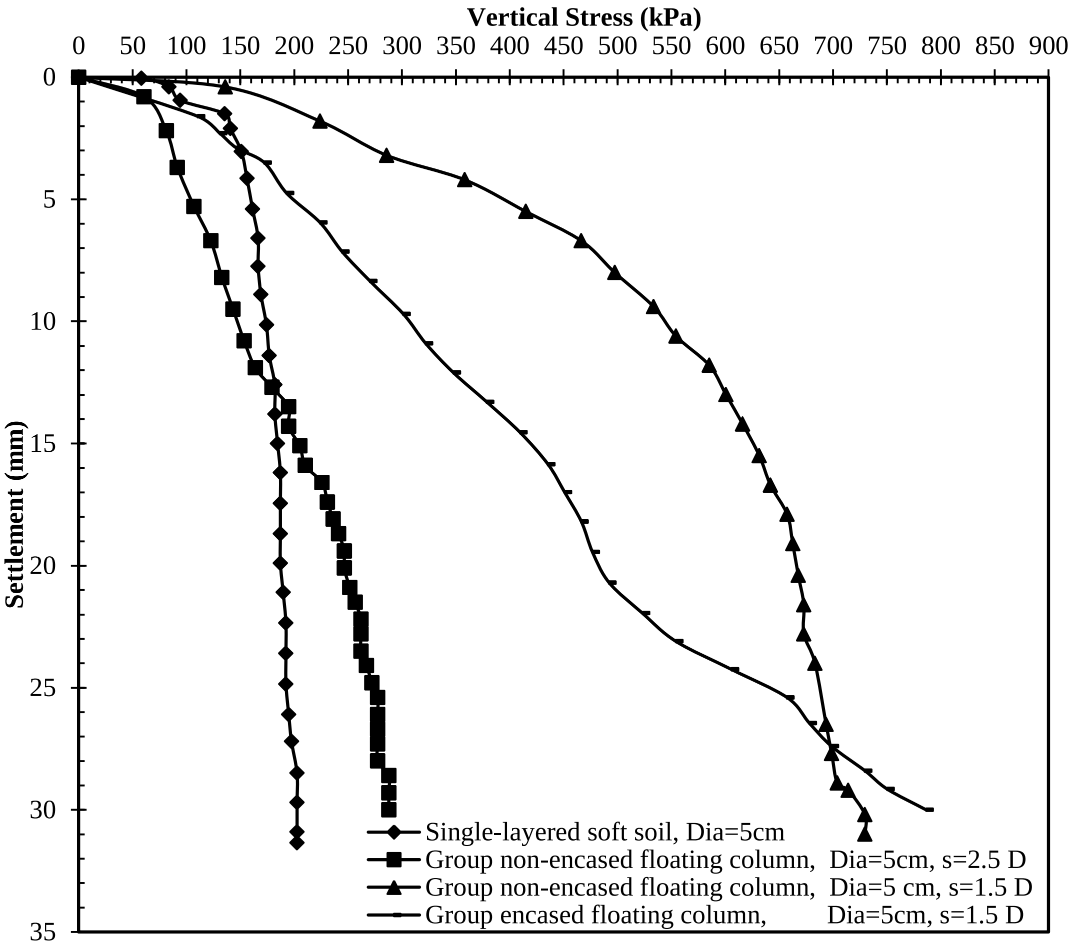


Figure 4.19: Settlement versus vertical stress curves of group of NEC and EC-FSCs in single-layered soft soil with two different spacings, s

4.3.1.2 Vertical stress-settlement behaviour of central columns in layered soil

Figure 4.20 shows the vertical stress-settlement curves of the central floating column among group of SCs in case of layered soil with and without encasement.

As it can be seen from Figure 4.20, the application of NEC central floating column among group of SCs with s/D ratio of 2.5 slightly enhanced the bearing capacity of layered soil. Whereas for the NEC central floating column among group of SCs with s/D ratio of 1.5, the bearing capacity of the layered soil increased by 2.68 times and with geotextile encasement, the bearing capacity of the SC in layered soil increased by 4.41.

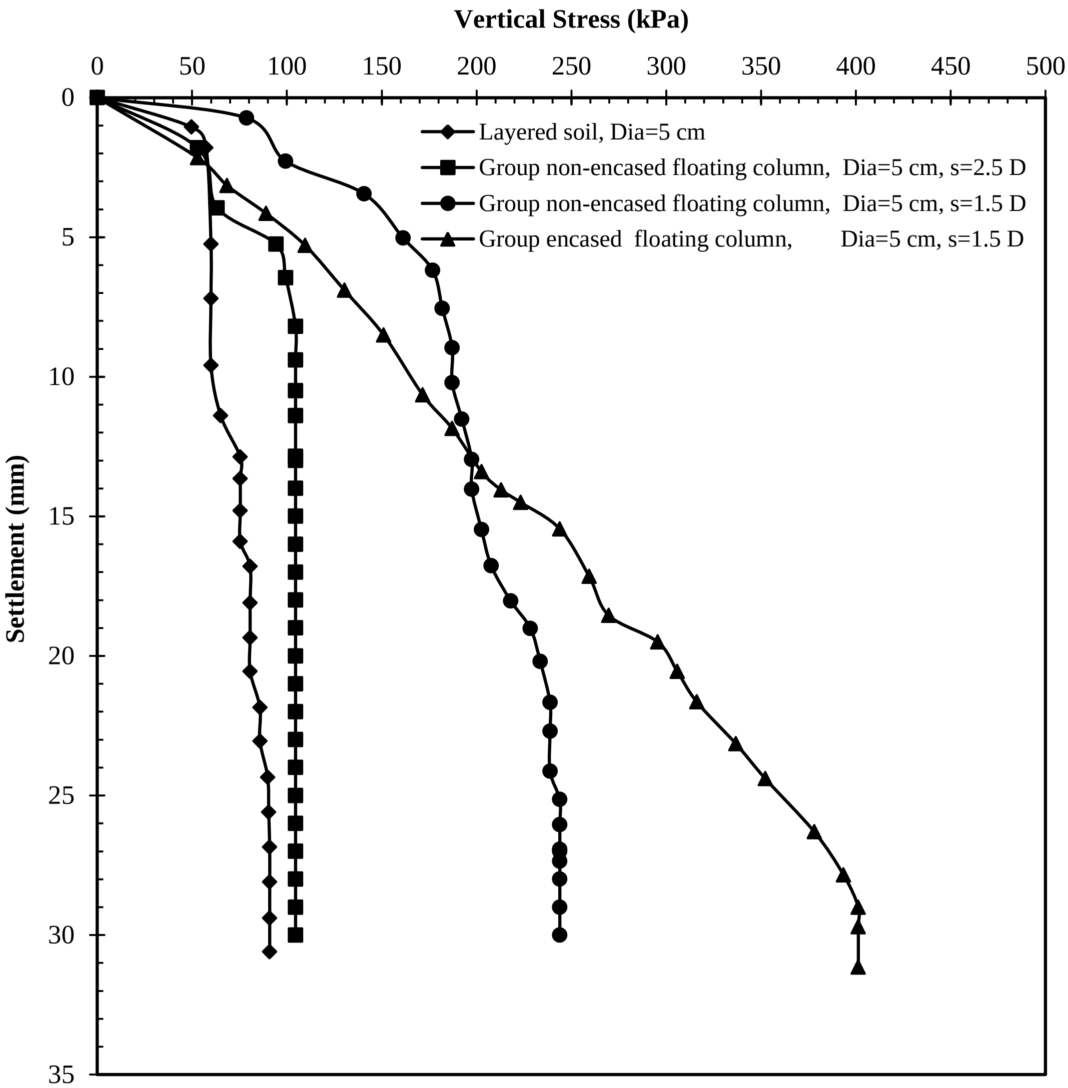


Figure 4.20: Settlement versus vertical stress curves of group of NEC and EC-FSC in layered soil with two different spacings, s

From Figure 4.20, it can be seen that the ultimate vertical stress of the central NEC floating column was significantly improved with a reduction in the spacing of the SCs. This finding was in good harmony with the findings of Dash and Bora (2013) even though the soil layering systems were different. As aforementioned, this behaviour could be explained due to better confinement for the central column provided by smaller spacing of the SCs, which enhanced the bearing capacity of the column. Whereas when column spacing increased, the columns behaved as individual column and less confinement was provided to the central column, as a result of which less bearing capacity improvement of SC was achieved (Dash and Bora, 2013).

4.3.1.3 Comparison of vertical stress-settlement behaviour of group of stone columns in single-layered and layered soils

From Figures 4.19 and 4.20, it can be seen that the application of group of NEC-SCs has enhanced the bearing capacity of single-layered and layered soils. In both cases, although the soil layering systems were different, the smaller spacing to diameter ratio in both cases resulted in better improvement than the bigger spacing to diameter ratio. In both cases, the geotextile encasement provided an additional enhancement to the SCs.

Table 4.12 summarized the ultimate vertical stress of group of SC applications in single-layered and layered soils. Comparison of the values of the ultimate vertical stresses of group of SCs in single-layered and layered soils from Table 4.11 showed that the ultimate vertical stress values obtained for the layered soil were much smaller than the values in the case of single-layered group of SCs. That phenomenon could be explained due to the existence of loose sand overlaying the soft soil in the layered soil condition. Very little confinement provided by the surrounding loose sand caused a reduction in the bearing capacity of the layered soil, whereas, in single-layered soil, higher confinement of surrounding soft soil caused an improvement in the ultimate vertical stress values of the SC.

Table 4.12: Ultimate vertical stress of group of SCs in single-layered and layered soils

Stone column		Ultimate vertical stress (kPa)
Single-layered soft soil	Dia=5 cm	202.6
Group non-encased floating columns	Dia=5 cm, s=2.5 D	287.7
Group non-encased floating columns	Dia=5 cm, s=1.5 D	729.7
Group encased floating columns	Dia=5 cm, s=1.5 D	786.4
Layered soil	Dia=5 cm	90.9
Group non-encased floating columns	Dia=5 cm, s=2.5 D	104.0
Group non-encased floating columns	Dia=5 cm, s=1.5 D	243.9
Group encased floating columns	Dia=5 cm, s=1.5 D	401.3

4.3.2 Subgrade modulus, *k* of central columns

In this section, the subgrade modulus reaction of the centred SC surrounded by group of SCs is discussed.

4.3.2.1 Subgrade modulus, *k* of central columns in single-layered soft soil

Figure 4.21 shows the subgrade modulus values of the central floating column among group of SCs in case of single-layered soft soil with and without encasement.

From Figure 4.21, it can be seen that in all group of SCs applications a remarkable amendment of the subgrade modulus values of the single-layered soft soil was reached and it was also observed that with decreasing the s/D ratio of columns, the subgrade modulus increased.

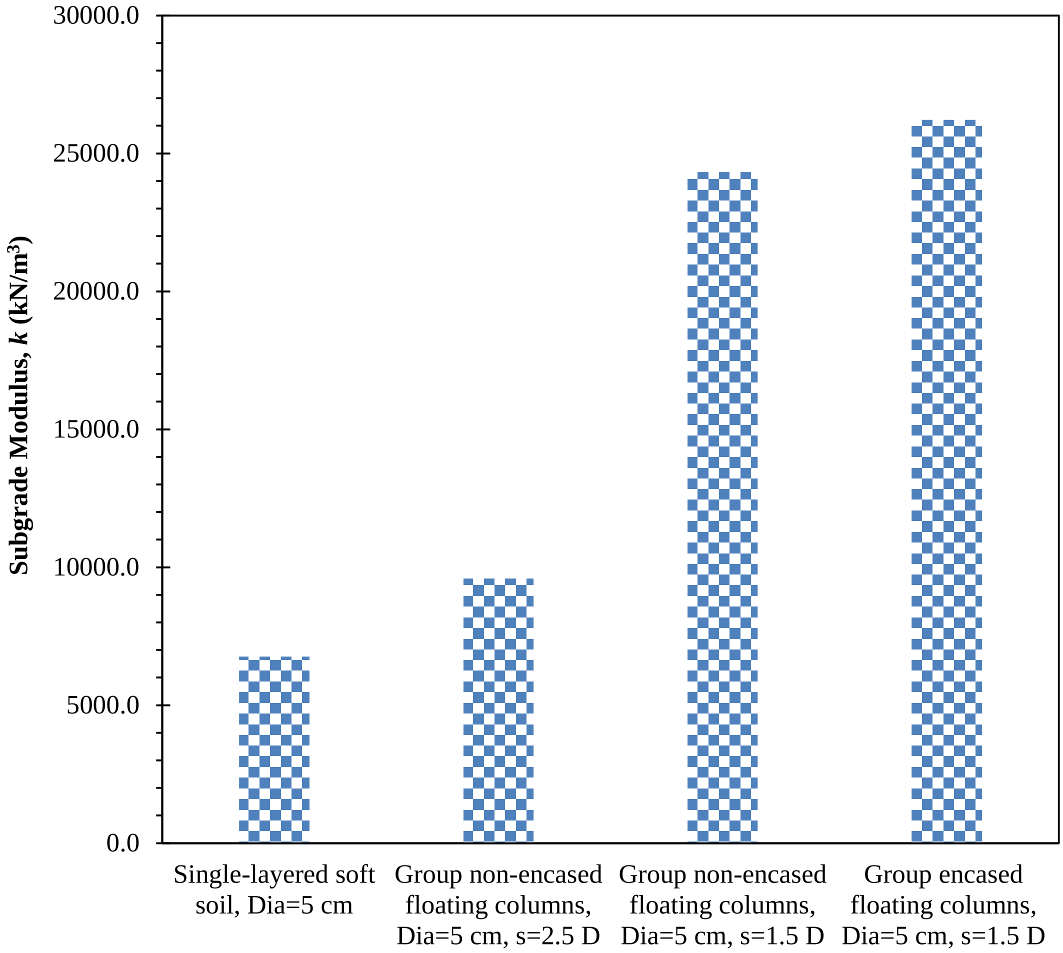


Figure 4.21: Subgrade modulus values of group of NEC and EC- FSC with two different spacing in single-layered soft soil

4.3.2.2 Subgrade modulus, *k* of central columns in layered soil

Figure 4.22 shows the subgrade modulus values of the central column among group of SCs in case of layered soil with and without encasement.

From Figure 4.22, it can be seen that in all group of SCs applications a notable improvement of the subgrade modulus of the layered soil was achieved and also decreasing the s/D ratio resulted in an improvement in the subgrade modulus.

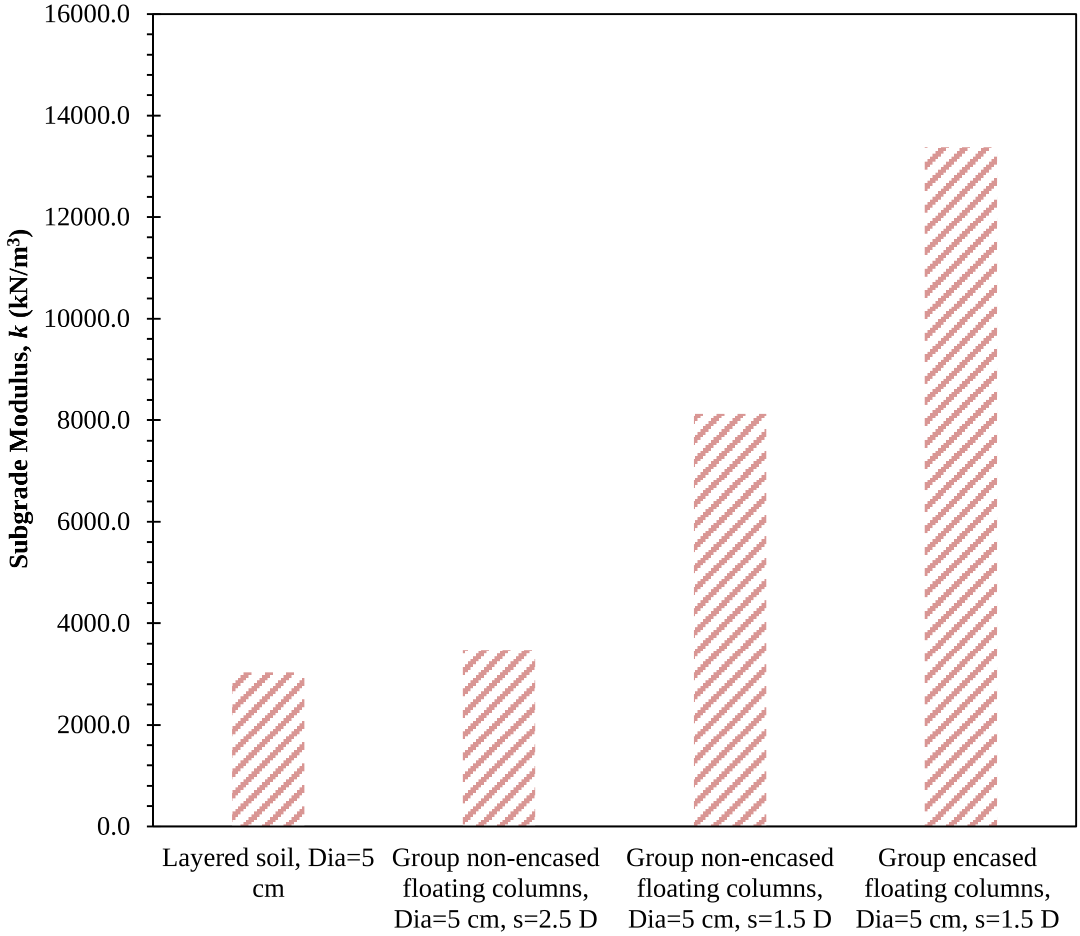


Figure 4.22: Subgrade modulus values of group of NEC and EC-FSC with two different spacing in layered soil.

4.3.2.3 Comparison of subgrade modulus of central column in single-layered and layered soils

Figures 4.21 and 4.22 have presented that the subgrade modulus values were increased when the single-layered and layered soils were reinforced with group of SCs and remarkable amendment of the soil was achieved with spacing to diameter ratio of 1.5.

4.3.3 Bulging failure of group of SCs

As in the case of the single SC application in layered soil, the bulging failure of the group of SCs after loading could not be investigated. The bulging failure of NEC central floating columns among group of columns with two different spacing was measured and shown in Figure 4.23. The results were also summarized in Table 4.12.

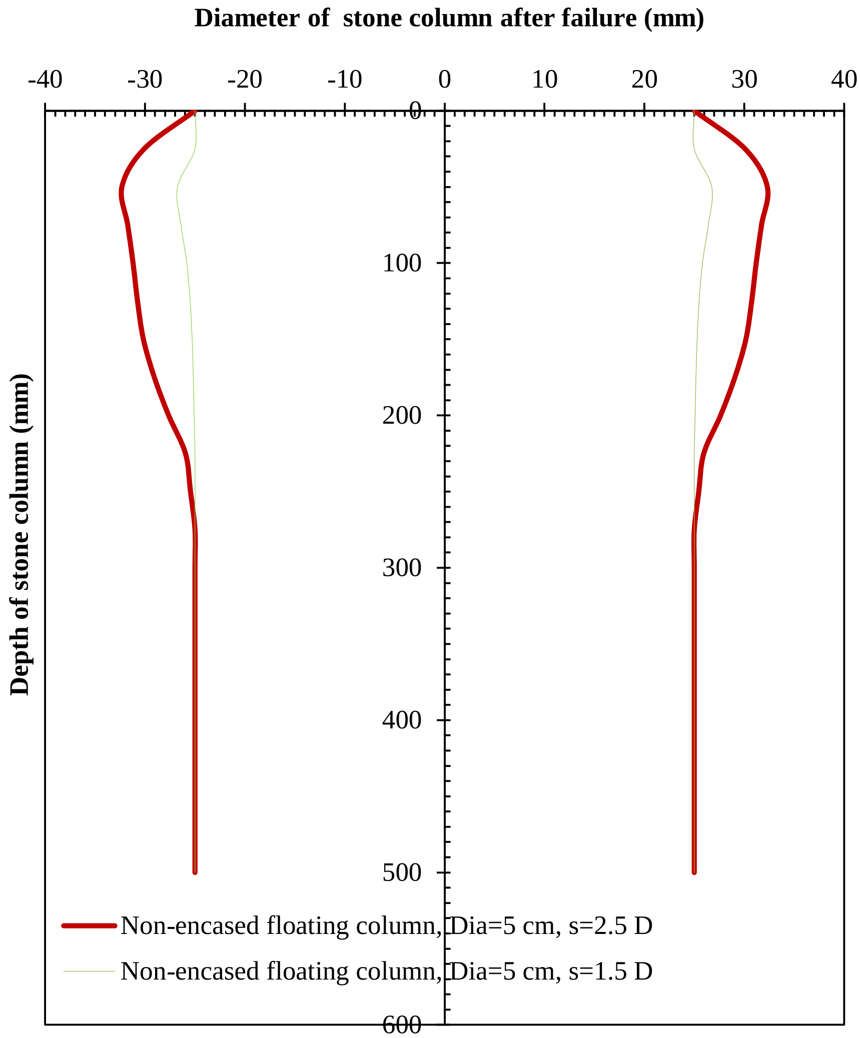


Figure 4.23: Bulging failure of NEC central floating columns among group of columns with two different spacing in single-layered soft soil

Bulging failure in NEC central FSC with s/D ratio of 2.5 was presented in Table 4.13. During testing, it was observed that the columns away from the centre were slightly bent.

Bulging failure of NEC central FSC with s/D ratio of 1.5 was also presented in Table 4.13. It was observed that the columns away from the centre were not deformed (not bulged nor bent). For EC central FSC among group of columns with s/D ratio of 1.5, there was no bulging encountered. Also, the columns away from the centre were neither bulged, nor bent.

Table 4.13: Bulging failure of NEC central FSC in single-layered soft soil with two different spacing to diameter ratios.

Stone column	Bulging depth and diameter	Values
Non-encased floating stone column, s=2.5xD	Max bulging depth	5.0 cm from the surface (1.0* D_0)
	Max bulging diameter	6.40 cm (1.28* D_0)
Non-encased floating stone column, s=1.5xD	Max bulging depth	5.0 cm from the surface (1.0* D_0)
	Max bulging diameter	5.35 cm (1.07* D_0)

D_0= Original diameter of the SC.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the small-scale laboratory test results, the following conclusions can be drawn from the study:

In case of single SC applications:

- In both soil layering systems (single layered and layered), in all EC and NEC-FSC and EBSC applications, the SCs with a smaller ARR value of 1.56% and L/D ratios of 10 and 15 gave better performance than the SCs with an ARR value of 6.25% and L/D ratios of 5 and 7.5. These findings indicated that neither the effect of ARR nor the effect of L/D on the SC bearing capacity should be considered alone. The two factors must be considered together so that the effect of diameter and length of the SC on soil improvement could be better evaluated.
- In both soil layering systems, in all SC applications, an increase in the L/D ratio within the range of 6-10 increased the ultimate vertical stress of the SCs. With ARR of 1.56%, the NEC-FSC with a length of 50 cm (L/D=10) gave much better results than the NEC-EBSC with a length of 75 cm (L/D=15).
- In both soil layering systems, with ARR of 6.25%, the NEC-EBSC with a length of 75 cm (L/D=7.5) gave much better ultimate vertical stress values than the NEC-FSC with a length of 50 cm (L/D=5). However, the ultimate vertical stress of EC- EBSCs with L/D ratio of 7.5 resulted in a lower

ultimate vertical stress than the EC- FSC with L/D ratio of 5.0 (both EC-EBSC and the EC-FSC with the same diameter of 10 cm). Under the same applied loading, the comparison of the values obtained for the short (50 cm) and long columns (75 cm) indicated that lesser time was needed for the densification of the materials of SC in the shorter column before the lateral expansion of the geotextile took place. As a result, an increase in the soil-column interface shear resistance of the SC was achieved, and this participated in the bearing capacity of the SC and resulted in a higher bearing resistance.

- By comparing the values of the ultimate vertical stresses in single-layered and layered soils, test results indicated that there was a considerable reduction in the ultimate vertical stress values obtained for the layered soil because of the presence of loose sand overlaying the soft soil, which provided inadequate confinement to the SCs.
- In both soil layering systems, all SC applications enhanced the bearing capacity of the soil and this increase in soil bearing capacity was presented as the bearing improvement ratio, BIR.
- In both soil layering systems, in case of NEC-SCs, the superior amendment of subgrade modulus was achieved with the application of NEC-FSC with an ARR value of 1.56% (5cm diameter column). Whereas, the EC-FSC with ARR 1.56% (5cm diameter column) had the most significant improvement of subgrade modulus among all SC applications.
- In single-layered soft soil, the smaller bulging diameter obtained in the SC applications resulted in a higher bearing capacity of the SCs. The presence of geotextile in the EC-FSC with ARR 1.56% reduced the bulging diameter and

increased the depth of bulging as a result of which higher bearing capacity was attained compared to the NEC-FSC with the same ARR value. In NEC-FSC with L/D=10 and ARR value of 1.56%, there was less bulging than the NEC-EBSC with L/D=15. As a result, higher bearing capacity was achieved in the NEC-FSC.

- In single-layered soft soil, in the case of NEC-FSC with L/D=5 and ARR value of 6.25%, more bulging failure was observed; as a result, a lower bearing capacity was attained in the NEC- FSC than the NEC- EBSC with L/D=7.5 and same ARR value.

In case of group of SC applications (central column among group of SCs):

- In both soil layering systems, the application of NEC central column among group of SCs improved the bearing capacity of soils. While the bearing capacity of soils was significantly improved with reducing the spacing to diameter ratio. This was due to the smaller spacing between the columns which provided better confinement for the central column and resulted in better improvement in the SC bearing capacity. In addition, the geotextile encasement provided an additional improvement to the SC.
- By comparing the values of the ultimate vertical stresses of SC groups in single-layered and layered soils, it can be stated that there was a significant reduction in the ultimate vertical stress values obtained for the layered soil compared to the single-layered soft soil. This behaviour was similar to the findings in the case of single SC application.
- All cases of group of SC applications had a remarkable amendment of the subgrade modulus values in single-layered and layered soils. It was observed

that decreasing the spacing to diameter ratio of columns resulted in an improvement in the subgrade modulus of the columns.

- Bulging failure occurred in single-layered soil, NEC floating central SC among group of columns with s/D ratio of 2.5, whereas in the columns away from the centre, slight bending took place. Furthermore, with decreasing the spacing to diameter ratio of SCs, the bulging in the central SC was reduced and no deformation confronted in the columns away from the centre. With the geotextile EC group of SCs with smaller spacing to diameter ratio no deformation (neither bulging, nor bending) confronted in the SCs.

5.2 Recommendations

The literature review indicates that very limited research was conducted on the performance of SCs in layered soils especially, the performance of SCs in loose sand overlaying a soft soil has not been considered.

In this study, only two layering systems were tested to study the performance of single and group of SCs in these soil layering systems. For further studies, different soil layering conditions with changing layer thickness should be implemented.

In the present study, two SC diameters and lengths were chosen to investigate the performance of single SCs in single-layered and layered soils. For further studies, different SC's diameters and length could be studied to see the effect of SC diameter and length on the performance of the SCs.

In this study, in the group of SCs application, two different spacing to diameter ratios have been studied. For further studies, different spacing to diameter ratios of SC could be investigated.

In the present study, one type of non-woven geotextile material with specific tensile strength was used to encase the SCs. In future studies, the same geotextile material but with different tensile strength could be selected. In addition, different types of geotextile materials could be utilized to examine the behaviour of SCs with these encasement materials.

In this study, the behavior of stone columns in short term was studied. For Further studies, the behavior of stone columns in long term could be investigated.

In this study, the behavior of stone columns was examined by conducting small scale laboratory tests. For Further studies, the obtained experimental results could be verified by using numerical modelling software.

REFERENCES

Aboshi, H.; Ichimoto, E.; Enoki, M.; Harada K. (1979). The compozer—a method to improve characteristics of soft clays by inclusion of large diameter sand columns. In: *Proceedings of the International Conference on Soil Reinforcement, ENPC*, pp. 211–216.

Al Ammari, K. (2016). *Predicting stiffness and stress variation of saturated clay improved with vibro stone columns and evaluating its effect on improving reinforced foundations* (Doctoral dissertation, University of Leeds).

Alamgir, M., Miura, N., Poorooshasb, H. B., & Madhav, M. R. (1996). Deformation analysis of soft ground reinforced by columnar inclusions. *Computers and Geotechnics*, *18*(4), 267-290.

Alexiew, D., Brokemper, D., & Lothspeich, S. (2005) Geotextile encased columns (GEC): load capacity, geotextile selection and pre-design graphs. *In Contemporary Issues in Foundation Engineering*, 1-14.

Ali, K., Shahu, J. T., & Sharma, K. G. (2010). Behaviour of reinforced stone columns in soft soils: an experimental study. In *Indian geotechnical conference* (pp. 620-628).

Ali, K., Shahu, J. T., & Sharma, K. G. (2012). Model tests on geosynthetic-reinforced stone columns: a comparative study. *Geosynthetics International*, *19*(4), 292-305.

Allersma, H. B. G. (1995). Simulation of subsidence in soil layers in a geotechnical centrifuge. *IAHS Publications-Series of Proceedings and Reports-Intern Assoc Hydrological Sciences*, *234*, 117-126.

Al-Mosawe, M. J., Abbass, A. J., & Majieed, A. H. (1985). Prediction of ultimate capacity of a single and groups of stone columns. In *Iraqi conference on Engineering ICE* (Vol. 85, pp. 61-68).

Altaee, A., & Fellenius, B. H. (1994). Physical modeling in sand. *Canadian Geotechnical Journal*, *31*(3), 420-431.

Andreou, P., Frikha, W., Frank, R., Canou, J., Papadopoulos, V., & Dupla, J. C. (2008). Experimental study on sand and gravel columns in clay. *Proceedings of the Institution of Civil Engineers-Ground Improvement*, *161*(4), 189-198.

ASTM, D. 422-98 (1998). Standard test method for particle-size analysis of soils. *Annual Book of ASTM Standards*, *4*, 08.

ASTM, D. 2937–17 (2017). Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method. *Annual book of ASTM standards*, 7.

ASTM, D. 854-06 (2006). Standard test method for specific gravity of soil solids by.

ASTM, D. 4318 (2005). Standard test methods for liquid limit, plastic limit, and plasticity index of soils.

ASTM, D. 2487-00. (2000). Standard Practice for Classification of Soils for Engineering Purpose (Unified Soil Classification System).

Babu, M. D., Nayak, S., & Shivashankar, R. (2013). A critical review of construction, analysis and behaviour of stone columns. *Geotechnical and Geological Engineering*, *31*(1), 1-22.

Balaam, N. P., & Poulos, H. G. (1978). Methods of analysis of single stone columns (No. Research Rpt. R335 Monograph).

Barksdale, R. D. & Bachus, R. C. (1983). *Design and construction of stone columns volume II, appendixes*. Federal Highway Administration, Washington, *DC*.

Baumann, V., & Bauer, G. E. A. (1974). The performance of foundations on various soils stabilized by the vibro-compaction method. *Canadian Geotechnical Journal*, *11*(4), 509-530.

Black, J. A., Sivakumar, V., & Bell, A. (2011). The settlement performance of stone column foundations. *Géotechnique*, *61*(11), 909-922.

Bowles, J. E. (1992). *Engineering properties of soils and their measurement*. McGraw-Hill, Inc.

Chen, J. F., Li, L. Y., Xue, J. F., & Feng, S. Z. (2015). Failure mechanism of geosynthetic-encased stone columns in soft soils under embankment. *Geotextiles and Geomembranes*, *43*(5), 424-431.

Chenari, R. J., Fard, M. K., Chenari, M. J., & Sosahab, J. S. (2017). Physical and numerical modeling of stone column behavior in loose sand. *International Journal of Civil Engineering*, *17*(2), 231-244.

Christoulas, S.; Bouckovalas, G.; Giannaros, C. (2000). An experimental study on model stone columns. *Soils Found*. 40(6), 11–22.

Chummer, A. V. (1972). Bearing capacity theory from experimental results. *Journal of the Soil Mechanics and Foundations Division, 98(12)*, 1311–1324.

Das, B.M. (2008). *Advanced Soil Mechanics*. Taylor and Francis, London and New York.

Das, M., & Dey, A. K. (2018). Prediction of Bearing Capacity of Stone Columns Placed in Soft Clay Using SVR Model. *Arabian Journal for Science and Engineering*, 1-11.

Das, P., & Pal, S. K. (2013). A study of the behavior of stone column in local soft and loose layered soil. *EJGE*, *18*, 1777-17786.

Dash, S. K., & Bora, M. C. (2013). Improved performance of soft clay foundations using stone columns and geocell-sand mattress. *Geotextiles and Geomembranes*, *41*, 26-35.

Deb, K., Samadhiya, N. K., & Namdeo, J. B. (2011). Laboratory model studies on unreinforced and geogrid-reinforced sand bed over stone column-improved soft clay. *Geotextiles and Geomembranes*, *29*(2), 190-196.

Debnath, P., & Dey, A. K. (2017). Bearing capacity of geogrid reinforced sand over encased stone columns in soft clay. *Geotextiles and Geomembranes*, *45*(6), 653-664.

Ghazavi, M., & Afshar, J. N. (2013). Bearing capacity of geosynthetic encased stone columns. *Geotextiles and Geomembranes*, *38*, 26-36.

Hasan, M., & Samadhiya, N. K. (2017). Performance of geosynthetic-reinforced granular piles in soft clays: Model tests and numerical analysis. *Computers and Geotechnics*, *87*, 178-187.v.

Hong, Y. S., Wu, C. S., & Yu, Y. S. (2016). Model tests on geotextile-encased granular columns under 1-g and undrained conditions. *Geotextiles and Geomembranes*, *44*(1), 13-27.

Hu, W. (1995). Physical modelling of group behaviour of stone column foundations (Doctoral dissertation, University of Glasgow).

Hughes, J. M. O., & Withers, N. J. (1974). Reinforcing of soft cohesive soils with stone columns. *Ground engineering*, *7*(3).

Hughes, J. M. O., Withers, N. J., & Greenwood, D. A. (1975). A field trial of the reinforcing effect of a stone column in soil. *Geotechnique*, *25*(1), 31-44.

ISO, B. 9864: 2005. *Geosynthetics. Test Method for the Determination of Mass per Unit Area of Geotextiles and Geotextile-Related Products*.

ISO, E. (2005). 9863-1 Geosynthetics–Determination of thickness at specified pressures–Part 1: Single layers. *CEN, Brusel*.

ISO, S. (2008). 10319: 2008:"Geosynthetics. *Wide strip tensile test "(in Romanian)*.

ISO, B. 12236, 1996,". *Geotextiles and Geotextile Related Products. Static Puncture Test (CBR-Test)*.

ISO, E. (2007). 13433: 2006. *Geokunststoffe—Dynamischer Durchschlagversuch (Kegelfallversuch)(Geosynthetics—Dynamic perforation test (cone drop test)), Deutsches Institut für Normung eV, Berlin*.

ISO, E. 11058: 1999–Geotextiles and geotextile-related products–determination of water permeability characteristics normal to the plane, without load. European Committee for Standardization, Brussels. *International Organization for Standardization, Geneva, Switzerland*.

ISO, E. (2010). 12956. *Geotextiles and geotextiles–related products–Determination of the characteristic opening size. CEN, Brussels*.

Killeen, M. (2012). Numerical modelling of small groups of stone columns. *Géotechnique*, *42*(2), 289-301.

Killeen M (2012) Numerical modelling of small groups of stone columns. Doctoral dissertation, National University of Ireland, Galway.

Madhav, M. R. (2000). Granular Piles-Recent Contributions. *A short term course on Ground Improvement and Deep foundations held at IIT Madras*.

Madhav, M. R., & Vitkar, P. P. (1978). Strip footing on weak clay stabilized with a granular trench or pile. *Canadian Geotechnical Journal*, *15*(4), 605-609.

Malarvizhi, S. N., & Ilamparuthi, K. (2004). Load versus settlement of clay bed stabilized with stone and reinforced stone columns. In *3rd Asian Regional Conference on Geosynthetics* (pp. 322-329).

McCabe, B. A., Nimmons, G. J., & Egan, D. (2009). A review of field performance of stone columns in soft soils. *Proceedings of the Institution of Civil Engineers-Geotechnical engineering*, *162*(6), 323-334.

McKelvey, D., Sivakumar, V., Bell, A., & Graham, J. (2004). Modelling vibrated stone columns in soft clay. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, *157*(3), 137-149.

McKenna, J. M., Eyre, W. A., & Wolstenholme, D. R. (1975). Performance of an embankment supported by stone columns in soft ground. *Geotechnique*, *25*(1), 51-59.

Mehrannia, N., Kalantary, F., & Ganjian, N. (2018). Experimental study on soil improvement with stone columns and granular blankets. *Journal of Central South University*, *25*(4), 866-878.

Meyerhof, G. G., & Sastry, V.V.R.N. (1978). Bearing capacity of piles in layered soils. Part 2. Sand overlying clay. *Canadian Geotechnical Journal*, *15(2*), 183–189.

Mitchell, J. K. (1981). Soil improvement: state-of-the-art, Department of Civil Engineering, University of California.

Mohanty, P., & Samanta, M. (2015). Experimental and numerical studies on response of the stone column in layered soil. *International Journal of Geosynthetics and Ground Engineering*, *1*(3), 27.

Muir Wood, D., Hu, W., & Nash, D. F. (2000). Group effects in stone column foundations: model tests. *Geotechnique*, *50*(6), 689-698.

Murugesan, S., & Rajagopal, K. (2006). Geosynthetic-encased stone columns: numerical evaluation. *Geotextiles and Geomembranes*, *24*(6), 349-358.

Murugesan, S., & Rajagopal, K. (2008). Performance of encased stone columns and design guidelines for construction on soft clay soils. In *Geosynthetics in Civil and Environmental Engineering* (pp. 729-734). Springer, Berlin, Heidelberg.

Murugesan, S., & Rajagopal, K. (2009). Studies on the behavior of single and group of geosynthetic encased stone columns. *Journal of Geotechnical and Geoenvironmental Engineering*, *136*(1), 129-139.

Najjar, S. S. (2013). A state-of-the-art review of stone/sand-column reinforced clay systems. *Geotechnical and Geological Engineering*, *31*(2), 355-386.

Nazariafshar, J., Mehrannia, N., Kalantary, F., & Ganjian, N. (2017). Bearing capacity of group of stone columns with granular blankets. *International Journal of Civil Engineering*, *17*(2), 253-263.

Ng, K. S. (2017). Settlement ratio of floating stone columns for small and large loaded areas. *Journal of GeoEngineering*, *12*(2), 89-96.

Prasad, S., Vasavi, C., & Sai, K. (2017). Behaviour of stone column in layered soils using geotextile reinforcement. *International Journal of Civil Engineering and Technology (IJCIET) Volume*, *8*, 453-462.

Priebe, H. J. (1976). Evaluation of the settlement reduction of a foundation improved by Vibro-Replacement. *Bautechnik*, *2*, 160-162.

Raithel, M., & Kempfert, H. G. (2000). Calculation models for dam foundations with geotextile coated sand columns. In *ISRM International Symposium*. International Society for Rock Mechanics and Rock Engineering.

Saadi, A. (1995). *The behaviour of strip footings on stone columns* (Doctoral dissertation, South Bank University).

Selig, E. T., & McKee, K. E. (1961). Static and dynamic behavior of small footings. *Journal of the Soil Mechanics and Foundations Division*, *87*(6), 29-50.

Shahu, J. T., Madhav, M. R., & Hayashi, S. (2000). Analysis of soft ground-granular pile-granular mat system. *Computers and Geotechnics*, *27*(1), 45-62.

Shahu, J. T., & Reddy, Y. R. (2011). Clayey soil reinforced with stone column group: model tests and analyses. *Journal of Geotechnical and Geoenvironmental Engineering*, *137*(12), 1265-1274.

Shivashankar, R., Babu, M. D., Nayak, S., & Manjunath, R. (2010). Stone columns with vertical circumferential nails: laboratory model study. *Geotechnical and Geological engineering*, *28*(5), 695-706.

Shivashankar, R., Babu, M. D., Nayak, S., & Rajathkumar, V. (2011). Experimental studies on behaviour of stone columns in layered soils. *Geotechnical and Geological Engineering*, *29*(5), 749.

Tandel, Y. K., Solanki, C. H., & Desai, A. K. (2012). Reinforced stone column: remedial of ordinary stone column. *International Journal of Advances in Engineering & Technology*, *3*(2), 340.

Tandel, Y., Jamal, M., Solanki, C., Desai, A., & Patel, J. (2017). Performance of small group of geosynthetic-reinforced granular piles. *Marine Georesources & Geotechnology*, *35*(4), 504-511.

Vautrain, J. (1978). Mur en terre armee sur colonnes ballastees. *BULL LIAISON LAB PONTS CHAUSS*, (SPEC VI-E).

Vekli, M., Aytekin, M., Ikizler, S. B., & Çalik, Ü. (2012). Experimental and numerical investigation of slope stabilization by stone columns. *Natural hazards*, *64*(1), 797-820.

Vesic, A. S. (1972). Expansion of cavities in infinite soil mass. *Journal of Soil Mechanics & Foundations Div*, *98*(sm3).

Sondermann, W., & Wehr, W. (2004). Deep vibro techniques. *Ground Improvement*, *2*, 57-92.

URL 1: http://kellerfoundations.co.uk/technique/vibro-stone-columns.

URL 2: https://www.google.com/maps/place/35%C2%B008'50.3%22N+33%C2%B054'13.3%22E/@35.147301,33.9025503,18z/data=!4m5!3m4!1s0x0:0x0!8m2!3d35.147301!4d33.903701.

URL 3: https://www.google.com/maps/place/35%C2%B008'18.2%22N+33%C2%B055'58.3%22E/@35.138397,33.9306603,17z/data=!3m1!4b1!4m5!3m4!1s0x0:0x0!8m2!3d35.138397!4d33.932849.