Numerical Analysis of Soft Clay Reinforced with Stone Columns

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ABSTRACT

Soft clays usually exist in many coastal areas and they reveal high compressibility and poor strength. There are various methods for improving in situ conditions of soft clays. Among these methods, stone columns are considered to be the most advantageous and cost-effective ground modification techniques. Stone columns are widely used to reduce the settlement of soft clays, accelerate the rate of consolidation by reducing the length of the drainage path and increase the load bearing capacity. Among the other ground modification techniques, stone column is simple to apply. Stone column application replaces the soft soil by a stronger material and mitigates the potential for cyclic mobility by accelerating the dissipation of excess pore pressure during earthquakes. The aim of this study was to investigate the performance of stone columns in soft compressible clay. For this reason, finite element analyses were carried out to evaluate settlement of soft clay reinforced with stone and encased sand columns. Sand columns were considered to be encased using geotextiles. 15-noded triangular elements using Plaxis software were used for the analysis. A drained analysis for stones and sand columns was carried out using Mohr-Coulomb's criterion whereas for soft clay undrained analysis was performed. The modeling of sand and stone columns was designed by axisymmetric pattern in Plaxis. In the study, the excess pore water pressure, settlement and the consolidation rate of the natural, sand and stone column reinforced soft cohesive soil under static and dynamic loading were analyzed using different column diameters. The bulging failure and the vertical settlement of the stone and the sand columns in the presence and absence of dynamic loading were analyzed and discussed. Results of the analyses indicated that the main factor influencing the value of maximum bulging is the amount of the axial load applied on the stone columns. Excessive bulging of the stone and sand columns under the applied axial loading occurred in depth of 2 to 3 times of the diameter of the stone and sand columns. Thus, the depth of bulging was related to the diameter of stone and sand columns. In the study, the finite element analysis for three-dimensional full scale modeling was also studied. In full scale three-dimensional, 3D finite element analysis, the bulging failure, punching failure and the vertical settlement of stone columns were analyzed and results were discussed. The results of the analysis indicated that the stone columns closer to the axis of symmetry gave maximum value of vertical settlement. The punching failure of the stone columns increased with the increase in the applied loading. In three-dimensional analysis of full scale clay deposits reinforced by stone columns, the maximum value of bulging occurred at a depth of three times of the diameter of stone columns.

Keywords: Axisymmetric Unit Cell, Bulging Failure, Punching Failure, Stone Column, Sand Column, Static and Dynamic Loading.

ÖZ

Yumuşak killer genelde kıyı bölgelerinde bulunurlar ve yüksek sıkışabilirlik ve düşük mukavemet değerlerine sahiptirler. Yumuşak killeri iyileştirmek için değişik yöntemler mevcuttur. Bu yöntemler arasında en avantajlı ve ekonomik iyileştirme yönteminin taş kolonlar olduğu düşünülmektedir. Taş kolonlar yumuşak kildeki oturmaları azaltmak, drenaj yolunu kısaltarak konsolidasyon oranını hızlandırmak ve zeminin yük taşıma kapasitesini artırmak maksadı ile kullanılmaktadır. Diğer zemin iyileştirme yöntemleri arasında, taş kolonlar daha kolay uygulanabilmektedir. Taş kolon uygulaması yumuşak kilin daha güçlü bir malzeme ile yer değiştirerek deprem anında aşırı boşluk suyu basıncının çıkışını hızlandırarak zeminin sıvılaşma özelliğini iyileştirmektir. Bu çalışmanın amacı yumuşak sıkışabilir kilde taş kolon performansının araştırılmasıdır. Bu nedenle, sonsuz eleman analiz yöntemi kullanılarak, taş kolon ve kum kolonlarla güçlendirilmiş yumuşak kilin oturmaları değerlendirilmiştir. Kum kolonların geotekstil ile çevrelendirildikleri düşünülmüştür. 15-düğümlü üçgen elemanlarla Plaxis programı kullanılarak analizler yapılmıştır. Taş kolonlar ve kum kolonlar için Mohr-Coulomb kriterleri kullanılarak drenajlı bir analiz, yumuşak kil içinse drenajsız analiz gerçekleştirilmiştir. Kum ve taş kolon modellemesi Plaxis programında aksimetrik şablon kullanılarak tasarlanmıştır. Bu çalışmada, doğal, kum ve taş kolonla güçlendirilmiş yumuşak kohezyonlu zeminin aşırı boşluk suyu basıncı, oturma ve konsolidasyon oranı statik ve dinamik yükler altında değişik kolon çaplarında analiz edilmiştir. Taş ve kum kolonların kabarma göçmesi ve dikey yöndeki oturmaları statik ve dinamik yükler altında analiz edilip, tartışılmıştır. Analiz neticeleri göstermiştir ki

maksimum kabarma göçmesine etki eden en önemli faktör taş kolon üzerine uygulanan dikey yüklerdir. Taş ve kum kolonlarda dikey yükler altında meydana gelen aşırı kabarma taş ve kum kolonların çapının 2-3 katı derinlikte meydana gelmektedir. Böylelikle, kabarma derinliği taş ve kum kolonların çapı ile ilişkilendirilmiştir. Bu çalışmada sonlu eleman analizi üç boyutlu tam ölçek model için de çalışılmıştır. Tam ölçekli üç boyutlu 3D sonlu eleman analizinde, taş kolonların kabarma göçmesi, zımbalama göçmesi ve dikey yöndeki oturma analizleri yapılmış ve tartışılmıştır. Analiz neticeleri göstermiştir ki simetri eksenine yakın olan taş kolonlar maksimum dikey oturma değerleri vermişlerdir. Artan yük değerlerine bağlı olarak taş kolonların zımbalama göçme değerlerinde artış görülmüştür. Tam ölçekli üç boyutlu analizde taş kolon ile güçlendirilmiş kil katmanında maksimum kabarma taş kolonun çapının üç katı bir derinliğinde meydana gelmektedir.

Anahtar kelimeler: Aksimetrik Birim Hücre, Kabarma Göçmesi, Zımbalama Göçmesi, Kum Kolon, Statik ve Dinamik Yükleme, Taş Kolon.

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TABLE OF CONTENT

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

C	Cohesion
$C_r{}'$	Modified composite coefficient of consolidation in the radial direction
C_r	Coefficient of consolidation in the radial direction
$C_v{}'$	Modified composite coefficient of consolidation in the vertical direction
C_v	Coefficient of consolidation in the vertical direction
C_g	Geometry dependent constant
d_e	Diameter of the influence zone in the axisymmetric unit cell
d_c	Diameter of the stone column in the unit cell
E_c	Stiffness of the stone column in a unit cell
E_s	Stiffness of the surrounding soil in a unit cell
a_s	Area replacement ratio in axisymmetric unit cell
e	Void ratio
k_h	Permeability in horizontal direction
k_v	Permeability in vertical direction
m_v	Coefficient of volume compressibility
R	Diameter ratio
t	Time
$T_r{}'$	Modified time factor
U	Average overall rate of consolidation
U_r	Average rate of consolidation in the radial direction

U_v Average rate of consolidation in the vertical direction

ν Poisson's ratio

γ_w Unit weight of water

γ_{sat} Saturated unit weight of soil

γ_{unsat} Unsaturated unit weight of soil

ϕ Friction angle

ψ Dilatancy angle

β Settlement reduction factor

q Applied load

A_s Plane area of stone column

A_c Plan area of the ambient clayey soil through the unit cell

Chapter 1

INTRODUCTION

1.1 Problem Statement

The in-situ soils are not usually suitable for supporting the desired structures such as buildings, dams and bridges. Every day, it is becoming difficult to find good construction sites for the structures. The need for the modification of the problematic soils is increasing each day. Different types of ground modification techniques have been developed in the last fifty years. The type of modification technique that will be applied on soil depends on the type of the problematic soil existing on site. Stone column application has its advantage among the other modification techniques. It is much simpler than many other techniques and it is cost effective. The stone column technique is applied either by a replacement or a displacement method. Owing to improve the mechanical treatment of soft cohesive soils and to quicken construction, the acceleration of the consolidation rate is very important. Among many methods for ground reinforcement, stone column reinforcement is applied worldwide. Based on the stiffness of stone columns, they work similar to piles as good as vertical drains. The main aim of stone column reinforcement is to increase the carrying capacity of soft cohesive soil and also to accelerate the consolidation rate of the soft cohesive soil and hence decrease the duration of the settlement process.

On the basis of Barron (1948) drain well theory, improvement of soft cohesive soil due to stone column reinforcement was accurately analyzed by some researchers (Balaam et al., 1977; Bergado et al., 1994; Barkslade & Bachus, 1983; Priebe, 1995; etc).

In the present study, the consolidation behavior of untreated soft cohesive soil in comparison with the unit cell of stone column in axisymmetric pattern has been analyzed. The finite element analysis by static and dynamic loading on consolidation was applied and the results were discussed. The behavior of sand column and stone column within the unit cell idealization by axisymmetric pattern has been studied. The three-dimensional full scale ground reinforcement has been modeled, and the result of the finite element analysis within the consolidation analysis for each stone column has been discussed.

1.2 Scope

The real stone column behavior affects the load bearing capacity of soft clay owing to different stiffness of material. In the present study, the two-dimensional and three-dimensional finite element analyses for stone column reinforced ground were performed. The adaptation of axisymmetric pattern of stone column and sand column within the unit cell idealization was carried out. The adaptation plan was developed by supporting either stone, sand and soft cohesive soil as linear elastic. Because of the real field state which is three-dimensional, the stone column was also analyzed within the 3-dimensional modeling of the finite element analysis.

This thesis was about the study of adaptation plan of axisymmetric stone column and sand column and the parameters produced from this adaptation was considered and discussed. Furthermore, the stone columns behavior within the consolidation process by Plaxis 3D Foundation was studied.

1.3 Objectives of Study

The objectives of this study are:

- To study the behavior of single stone and encased sand column using the finite element method by axisymmetric modeling.
- To develop adaptation plan for getting the relevant parameters for two and three-dimensional modeling of stone column reinforced soft cohesive soil.
- To prove the adaptation procedure with comparisons of different diameter of the stone and encased sand column supported soft cohesive soil in numerical modeling.

1.4 Thesis Outline

This study includes the following chapters:

In Chapter 1, the background information and the scope and objectives of the thesis are outlined.

Chapter 2 presents the literature review of the subject. It describes the theoretical background and the application of the stone columns in soft cohesive soils.

Chapter 3 describes the numerical analysis of the stone columns. The results obtained from the finite element analysis for the natural and stone column reinforced soft cohesive soil were discussed.

In Chapter 4, the behavior of sand column and stone column within the axisymmetric unit cell was analyzed by finite element analysis and the results were discussed.

Chapter 5 discusses the behavior of stone column reinforced soft cohesive soil in full scale three-dimensional finite element analysis.

Chapter 6 provides the conclusions of this study based on the numerical analysis and finally, the list of references are given.

CHAPTER 2

LITERATURE REVIEW

2.1 Drain Theory

2.1.1 General Background

Since the drain theory for the analysis of stone column application is very important first the drain theory will be explained and reviewed, and the stone columns and the vertical drains will be discussed. Some differences and similarities between stone columns and vertical drains will be described as well. A brief review of finite element using 15-noded analysis with the software Plaxis will also be given.

As the compressibility of soft clays is very high, these soils need to be improved before construction. The selection of a soil improvement method is very important. The need for the improvement of soft cohesive soils and the selection of a suitable method are vital. There are different types of modification techniques which can be applied to such soil. Preloading is one of them. But in this method, since the dissipation of excess pore water pressure in vertical direction is very slow, its application is not very common.

Instead, the stone column application is usually preferred. In the stone column application, dissipation of excess pore water is enabled in horizontal direction and the drainage of the excess pore water pressure is accelerated due to the shortening of the drainage path. Horizontal flow into the drain wells supply the fast dissipation of excess pore water pressure that result in accelerated consolidation process.

The consolidation of soft soils modified by drain wells was examined by Barron (1948). In layered soils where there is higher permeability in horizontal direction than vertical direction, drain wells are much more suitable. The drain wells allow water to move from the ambient soil into the wells and this water is pumped out by vertical drainage system. The drain wells direct excess pore water into a drainage layer which is placed at top or bottom of the soil.

Vertical drains are widely used all over the world for land reclamation to accelerate consolidation process of clayey soil. Sand drains, prefabricated vertical drains, sand compaction piles, and stone columns are some types of improvement of soft soils with drain inclusions. During the improvement of soft soils two factors: bearing capacity and settlement of the soil are considered. Vertical drains decrease the path of drainage significantly in the radial direction and that the coefficient of consolidation in the horizontal direction becomes much higher than that in vertical direction. The vertical drains accelerate the consolidation process and consequently the strength of soft soils is improved.

Baron (1948) recommended his solution on the radial consolidation by vertical drain wells. For three dimensional consolidation of radial drainage (one- dimensional, 1D vertical compression, collectively with axisymmetric radial and vertical flow) Barron (1948) recommended:

$$\frac{\partial \bar{u}}{\partial t} = C_h \left(\frac{1}{r}\frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2}\right) + C_v(\frac{\partial^2 u}{\partial z^2}) \qquad (2.1)$$

However for radial flow only:

$$\frac{\partial \bar{u}}{\partial t} = C_h(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}) \qquad (2.2)$$

$C_{h=}$ Coefficient of consolidation in the radial direction

$C_{r=}$ Coefficient of consolidation in the vertical direction

$\bar{u}$ = Average excess pore water pressure at depth z in soil

u= Excess pore water pressure at radius r and at depth z

t= The time elapsed after the loading is applied.

z= vertical coordinate.

Carillo (1942) showed that the solution of Eq.2.1 can be obtained by combining separate solutions for vertical compressions by vertical flow and vertical compression by radial flow. The excess pore water pressure and degree of consolidation, at any time were found to be:

$$\acute{u}_{2r=}\frac{\acute{u}_z \quad \acute{u}_r}{\acute{u}_\iota} \qquad (2.3)$$

And $U = 1 - (1 - u_z)(1 - u_r)$ (2.4)

where,

U= Average overall rate of consolidation.

$u_{r\,=}$ Average rate of consolidation in radial direction

$u_{z=}$ Average rate of consolidation in vertical direction

For the vertical compression by vertical flow only, the Terzaghi one- dimensional (1D) consolidation gives:

$$\frac{\partial u_z}{\partial t} = C_v \frac{\partial^2 \quad u_z}{\partial z^2} \qquad (2.5)$$

Barron (1948) presented the strict solution of vertical drain based on free strain and equal strain. In the free strain theory, for each vertical drain, the load is supposed to be uniform over a circular area of effect, and the differential settlements happening over this area have no influence on the redistribution of stresses by arching of the load. The equal strain theory supposed that arching happens in top layer within the process of consolidation with no differential settlement in the layer of clay. The forecasted pore water pressures estimated by free strain and equal strain do not make a big difference. Consequently, the approximate solution based on the equal strain gives satisfactory result compared to the scrupulous free strain. The drain well that is suggested by Barron (1947) is shown in Figure 2.1.

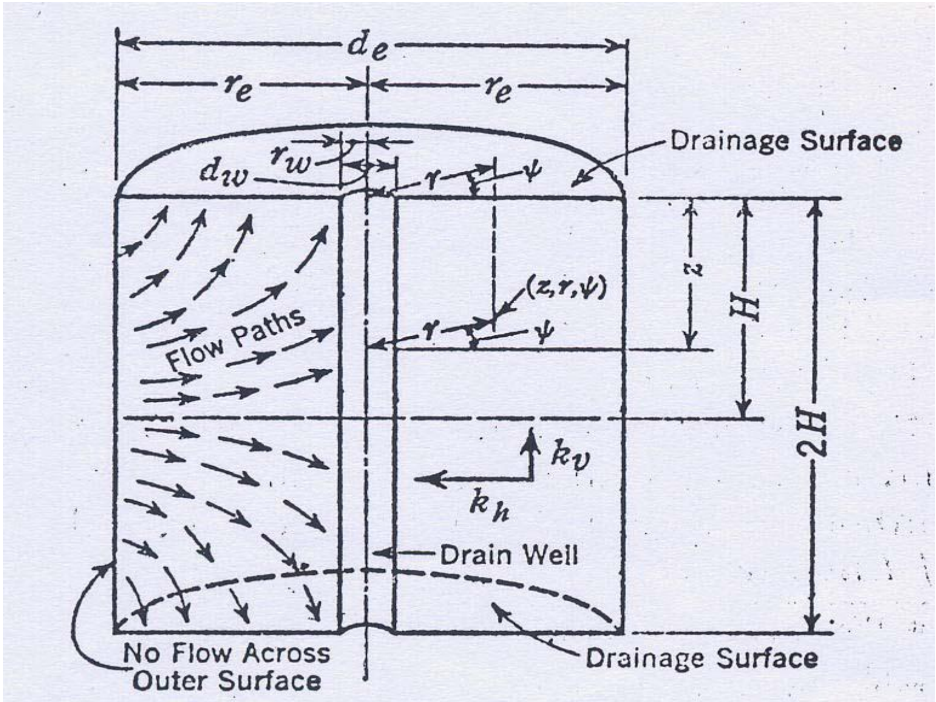
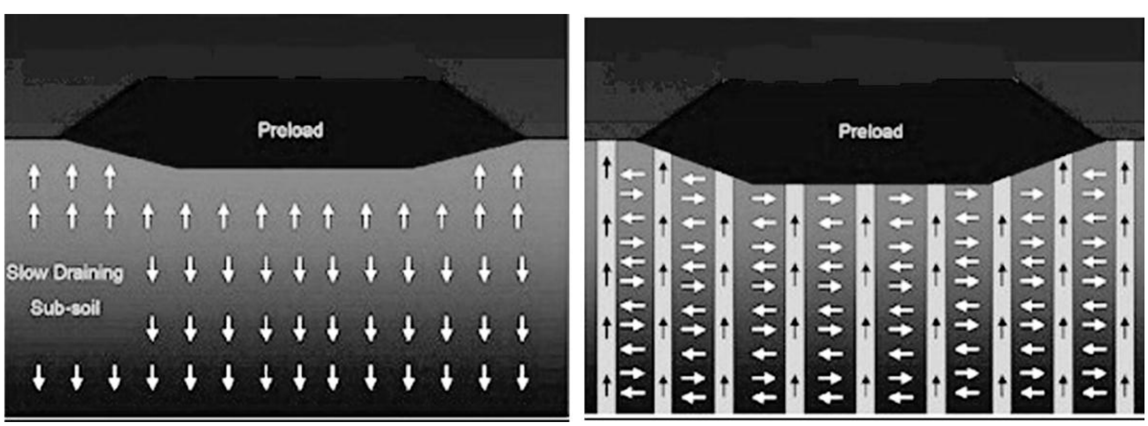


Figure 2. 1: Drain Well (Barron, 1947)

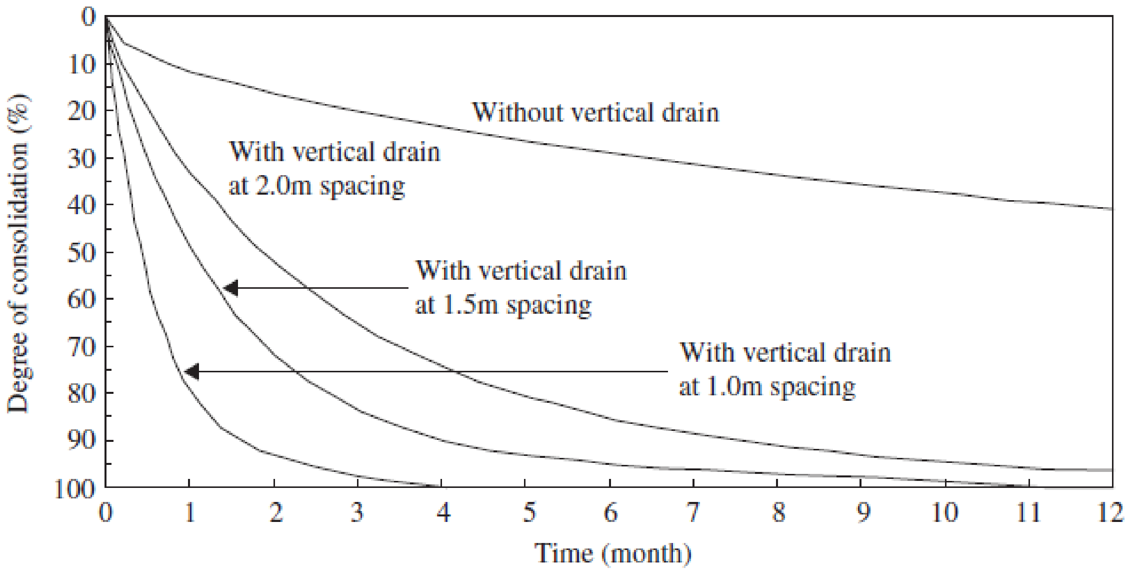
As shown in Figure 2.2 (a) and (b) total settlement of the clay layer can be reduced by preloading, but by applying the preloading technique. Application of preloading together with vertical drains accelerates the consolidation process significantly.



Without vertical drain

With vertical drain

(a)



(b)

Figure 2. 2: (a) and (b): Potential benefits of vertical drains (adapted from Lau et al., 2000)

The average degree of consolidation in the radial direction is presented by:

$$U_{h=1-exp-[\frac{8}{f(n)}]T_h} \qquad (2.6)$$

where:

$$\text{f (n)} = [N^2/(N^2-1)]\ln(N)-(3\,N^2-1)/(4N^2) \qquad (2.7)$$

$$\text{N}={}^{d_e}/_{d_w} \qquad (2.8)$$

d_e= Diameter of the influence zone in a cell.

d_w= Diameter of the drain well in a cell.

$$T_h={}^{C_h t}/_{de^2} \qquad (2.9)$$

where, T_h=Time factor in a radial flow

$$C_h=\frac{k_{h\ (1+e)}}{a_v\ r_w} \qquad (2.10)$$

where,

$k_{h=}$ Horizontal permeability of the soil

e= Void ratio

a_v =The coefficient of compressibility of the soil

γ_w = Until weight of water

2.2 Improvement of Soft Soils by Stone Columns

2.2.1 Introduction (A Brief Review)

Among various methods the improvement of bearing capacity of soft soils, by stone column is one of the most efficient methods in order to reach the desired bearing capacity of soft soil. In this method, some types of materials which are stiffer than the surrounding soil are applied into the boreholes which are already opened to constitute a layout of stone columns.

Those materials which are used in stone column method, have high permeability by comparison of the soft soils. Thus stone columns behave like drainholes and help to speed up the rate of consolidation process.

Stone column methods are based on friction among particles of materials used as columns. The materials used in this method have high strength than the ambient soil, and behave like piles. There are some differences between stone columns and drain holes. Stone column contents are stiffer than drain holes contents. Diameter of stone columns is smaller than drainholes Owing to stone column improvement, two great changes occur:

- Increase in carrying capacity
- Decrease in total settlement

Stone columns have round effect area which is described by unit cell concept. In Figure 2.3 the unit cell of stone column is shown.

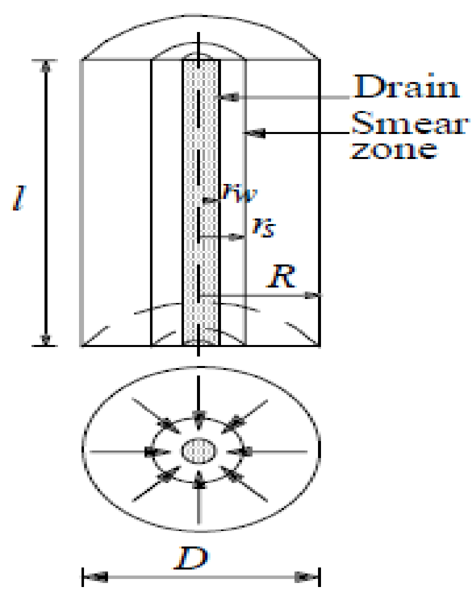


Figure 2. 3: Unit Cell of Stone Column (Indraratna & Redana, 1997)

2.2.2 Different Procedure for Applying Stone Column

Throughout the world, there are different methods to apply stone columns. The two main methods are a) vibro-replacement and b)vibro-displacement which can be applied for a wide range of soil types. In general, for cohesive soil, vibro-replacement procedure is useful and for granular soil (cohesionless soil) vibro-displacement method is more applicable.

2.2.2.1 Vibro-replacement

Vibro-replacement is a useful method for the improvement of soft soil at which columns were formed by coarse material in the soil by the aid of vibrators in certain depths. It is built with coarse material that replaces the natural soil (Priebe, 1995).

Stone column reinforcement by vibro-replacement method improves cohesive soils, which are very difficult to compact or sometimes they are categorised as non-compactable soils.

According to Priebe (1995) the following idealized conditions are assumed:

- The column is based on a rigid layer
- The column material is incompressible
- The bulk density of column and soil is neglected

Vibro-Replacement is a method for formation stone columns using a vibroflot. It is a combination of vibroflotation with a gravel backfill resulting in stone columns. In vibroflot method the vibroflot is penetrated to the required depth by the combined effect of vibroflot weight, vibration and jetting action. The vibroflot is then lifted out and from the ground surface, granular material is poured into the borehole in stages. There are two different types of application for vibro-replacement: wet method and dry method. Figure 2.4 and Figure 2.5 show the excavation procedure for wet method by vibro-replacement.

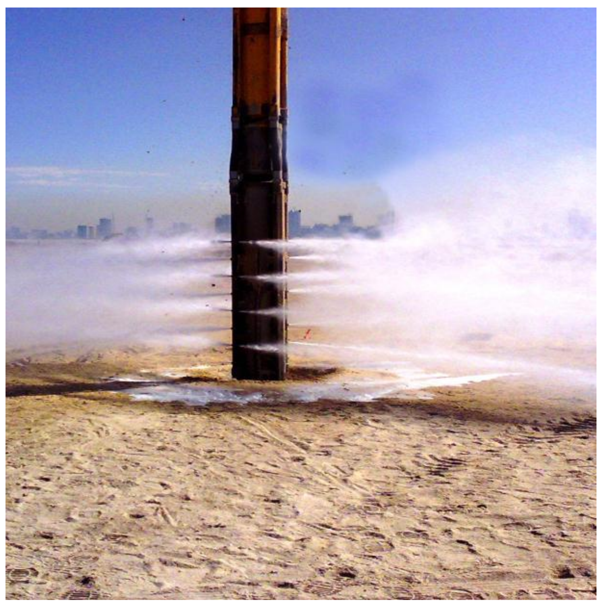


Figure 2. 4: Vibro-replacement in Wet Method (ICE, 2009)

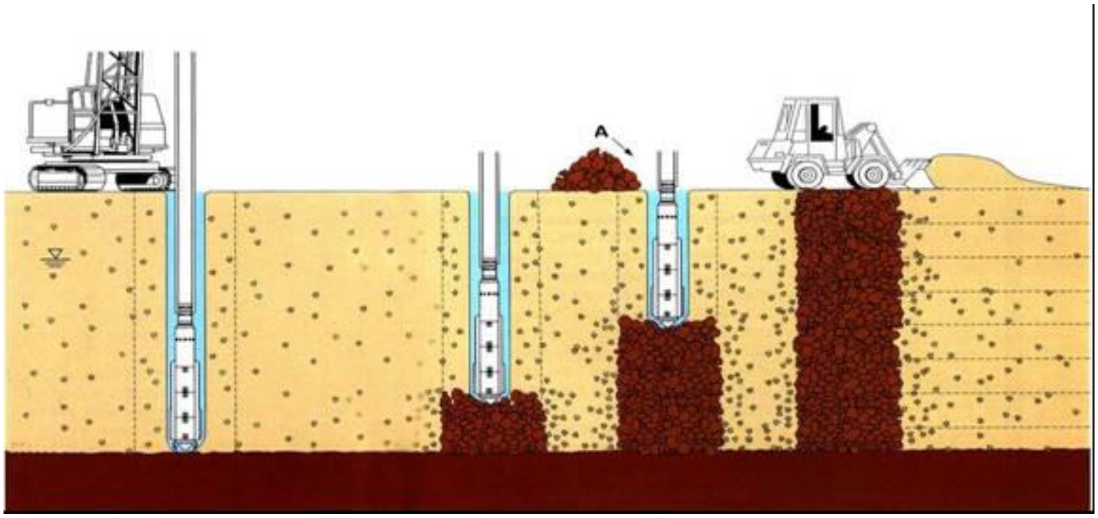


Figure 2. 5: The Application Vibro-replacement of Wet Method (Keller Far East, 2002)

In dry method, vibroflot penetrates the ground to the desired depth and then it is taken out. Stone column materials are poured into the hole in stages. The vibrator at the nose of vibroflot condenses stone column materials.

In vibro-replacement method, vibroflot by jetting water makes the hole, but in vibro-displacement method the hole is formed by forcing. Figure 2.6 shows the application of vibro-displacement dry method step by step. As shown in Figure 2.7, the vibro-replacement cab be applicable for fine-grained and coarse-grained material whereas vibro-displacemet is just applicable in coarse-grained soils.

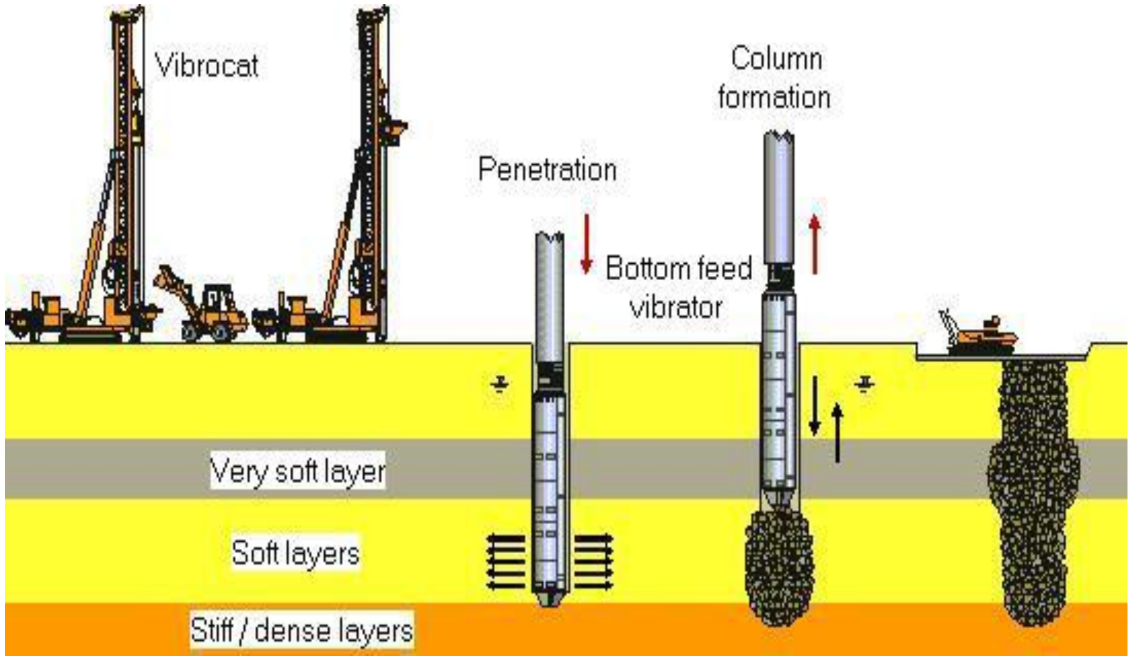


Figure 2. 6: The vibro-displacement Dry Method Application (Keller Far East, 2002)

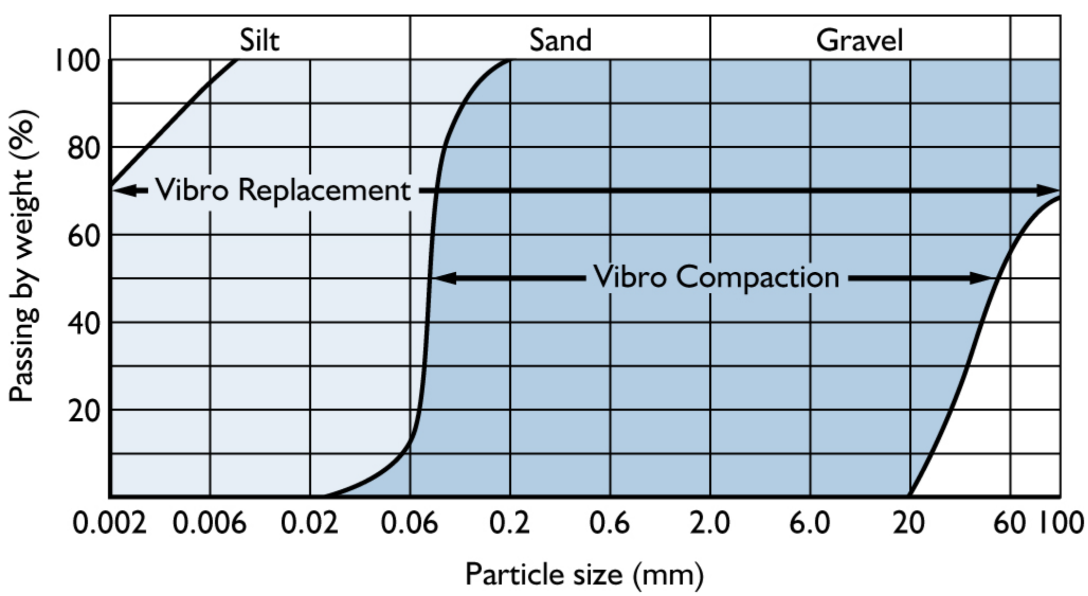


Figure 2. 7: Applicability of Vibro-Compaction and Vibro-Replacement
(Keller Far East, 2002)

2.2.3 Reinforcement of the Soil with Stone Columns

Reinforcement of the soil with stone column provides basically:

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

2.2.4 Analysis of Stone Column

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

A. Triangular arrangement

B. Square arrangement

C. Hexagonal arrangement

Figure 2.8 shows the three types of arrangements.

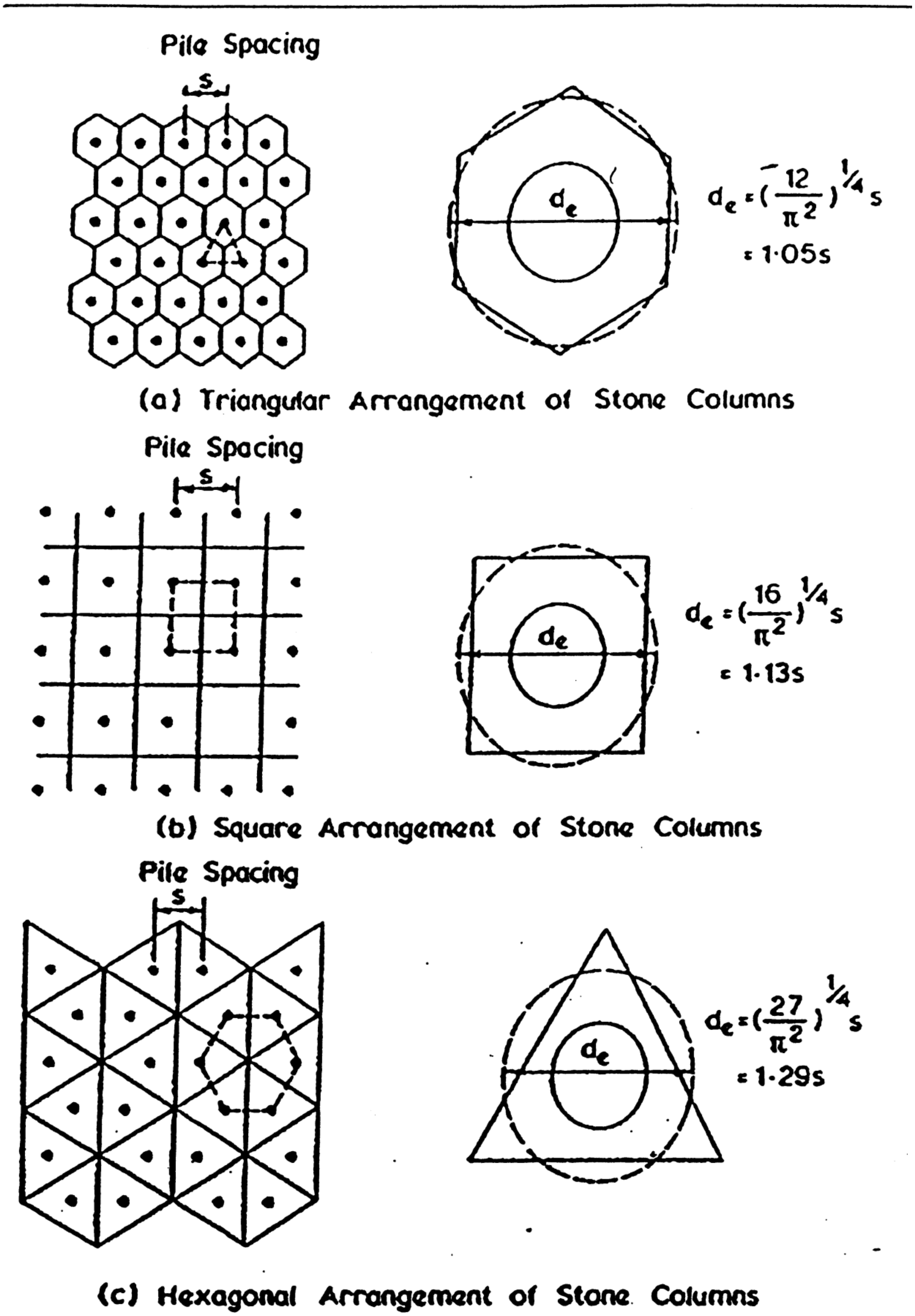


Figure 2. 8: Stone Column Arrangement (Balaam and Booker, 1981)

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

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

The relation between the spacing of stone columns and the diagonal of the unit cell is given by:

$$d_e = s.c_g \tag{2.11}$$

where,

$d_e =$ Diagonal of the unit cell

$s =$ Distance of the stone columns

c_g= Constant coefficient related to columns arrangement

In triangular type, $c_g = 1.05$, in square type, $c_g = 1.13$, and for hexagonal type, $c_g = 1.29$.

2.3 Failure Systems

2.3.1 Single Stone Column

Barksdale & Bachus (1983) proposed three types of failure styles for single stone columns. These are:

I. Bulging failure
II. Shear failure
III. Punching failure

In Fig 2.9 these three types of failures are shown.

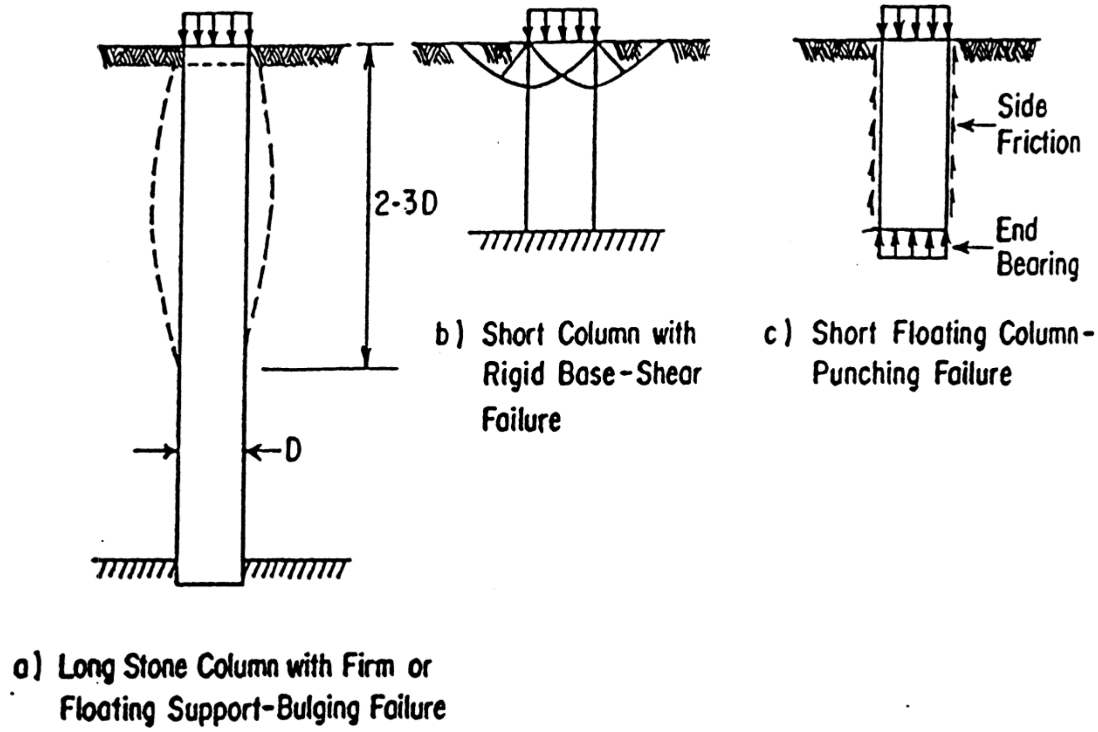


Figure 2. 9: Failure Styles of Single Stone Column (Barksdale & Bachus, 1983)

Bergado et.al (1991) proposed that the stone columns are basically built like end-bearing pile. Hughes Withers (1974) presented the diameter of floating columns within the length more than three times is failed, and these failures happened due to bulging at the top of columns.

2.3.2 Group Stone Columns

Barkslade & Bachus (1983) presented that, in position of rigid foundation supported by columns, the ultimate bearing capacity of stone columns per column is greater than an isolated single pile. They stated that, the nearby columns and the ambient soil are the reasons of the greater ultimate bearing capacity.

Vautrain (1977) presented that, underneath a broad flexible loading like an embankment, the settlement of stone columns and the soft soils are roughly equivalent. The failure styles of the group stone columns are illustrated in Fig 2.10. (Barkslade & Bachus, 1983)

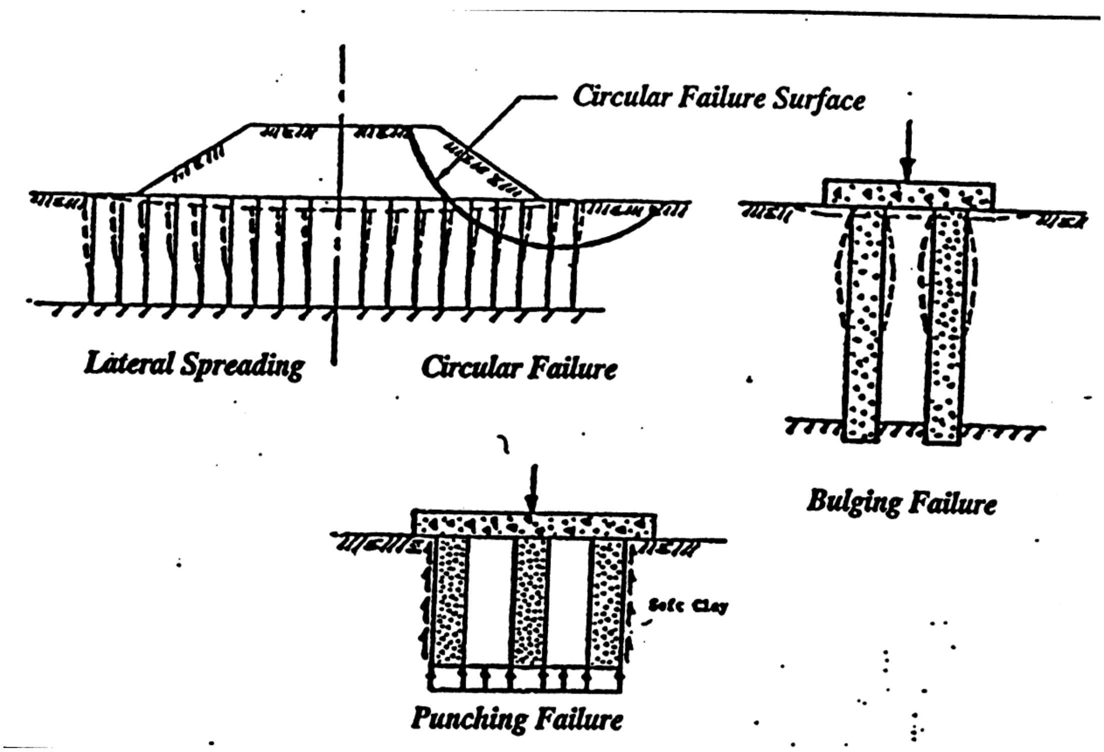


Figure 2. 10: Failure Styles of the Group Stone Columns (Barkslade & Bachus, 1983)

Greenwood (1970) presented that the lateral passive control on all sides of the columns which are away from the border of loaded region is greater owing to the equal side influence of applied load.

Unfortunately, there is no unique theory which completely explains the treatment of a ground reinforced by stone columns owing to complicated interactive treatment between the stone columns and the ambient soil. In this chapter, some of the widely applicable methods for stone column design, settlement calculations and failure will be described.

2.4 Designing Methods of the Ultimate Carrying Capacity

2.4.1 Ultimate Carrying Capacity of Single Stone Column (Bulging Failure)

I. Passive pressure method

Greenwood (1970) presented that in passive method the powerful part of the foundation soil is single stone column which is loaded by a strip footing as shown in Figure 2.11a. The single stone column swells up laterally and exerts lateral pressures on the ambient clay which are opposed by passive earth pressure.

Greenwood (1970) recommended an equation to assess ultimate carrying capacity of single stone column under consideration of the earth pressure method.

$$q_{\text{ult}} = \gamma z k_p + 2c_u\sqrt{k_p} \qquad (2.12)$$

where σ_1 and σ_3 are equal ($\sigma_1 = \sigma_3$) and,

$q_{\text{ult}} =$ The ultimate carrying capacity of stone column

$\gamma =$ The bulk density of soft cohesive soil (clay)

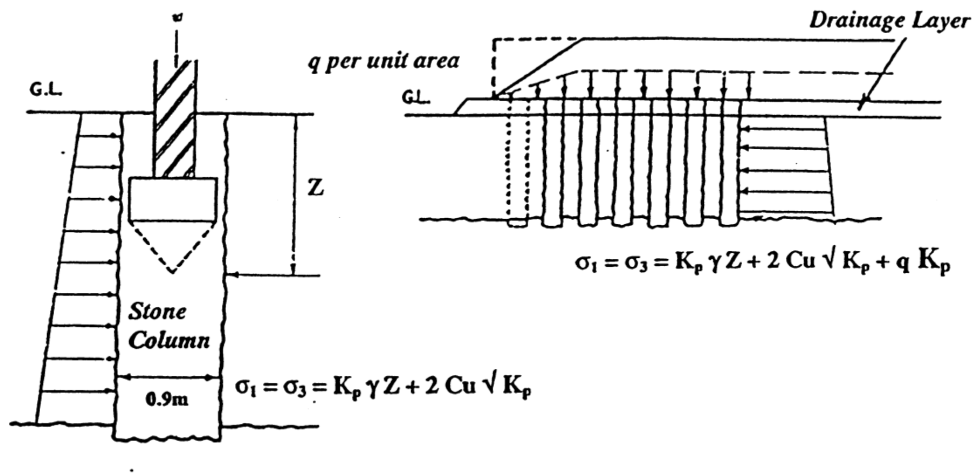
$c_u =$ The undrained shear strength

$z =$ Depth of stone column bulging + footing depth from the ground

$k_p =$ The coefficient of passive earth pressure = $\frac{(1+\sin\phi_s)}{(1-\sin\phi_s)}$ (2.13)

$\phi_s =$ Angle of internal friction of the stone column

In Figure 2.11b wide spread load is shown. In this situation, stone column far from the border of loaded region has much greater lateral passive restraint due to the equal surcharge pressure on all sides.



(a). Strip footing (b). Widespread loadings

Figure 2. 11: Stone Column under Strip Footing and Widespread Loading

Greenwood (1970) recommended an equation to assess the ultimate carrying capacity of the stone columns underneath the central region of the broad foundation which calculates for the development of passive pressure in the ambient clay underneath loading.

$$q_{\text{ult}} = \gamma z k_p + 2c_u\sqrt{k_p} + qk_p \tag{2.14}$$

where,

$q =$ Surcharge load per unit area

II. Pressuremeter method

Gibson & Anderson (1961) examined the state of a cylindrical swelling through an elasto-completely plastic soil. Due to changing plastically of the soil, the following equation is defined:

$$\sigma_{\mathrm{rl}} = \sigma_{\mathrm{r0}} + c_u[1 + \ln\{\frac{E_c}{2c_u(1+v_c)}\}] \tag{2.15}$$

σ_{rl} = The limiting lateral pressure

c_u= Undrained shear strength

σ_{r0} = The total initial lateral stress

E_c = Young's modulus for clay

v_c = Poisson's ratio for clay

Hansbo (1994) presented that E_c can be estimated by c_u into a certain interval at which$150c_u \leq E_c \leq 500c_u$. With considering of two parameters at which $150c_u \leq E_c \leq 500c_u$ and undrained condition ($v_c = 0.5$), the limiting lateral pressure (σ_{rl}) is defined by an interval that, $\sigma_{\mathrm{r0}} + 5c_u \leq \sigma_{\mathrm{rl}} \leq \sigma_{\mathrm{r0}} + 6c_u$, but Hansbo (1994) mentioned that σ_{rl} practically is assumed by the following equation:

$$\sigma_{\mathrm{rl}} = \sigma_{\mathrm{r0}} + 5c_u \tag{2.16}$$

Thus, the ultimate carrying capacity is defined by:

$$q_{\mathrm{ult}} = k_p(\sigma_{\mathrm{r0}} + 5c_u) \tag{2.17}$$

2.4.2 The Ultimate Carrying Capacity of Stone Column Group

Barkslade & Bachus (1983) proposed that the ultimate carrying capacity of stone column group can be resolved by estimating the failure surface with considering the direct failure lines as shown in Figure 2.12.

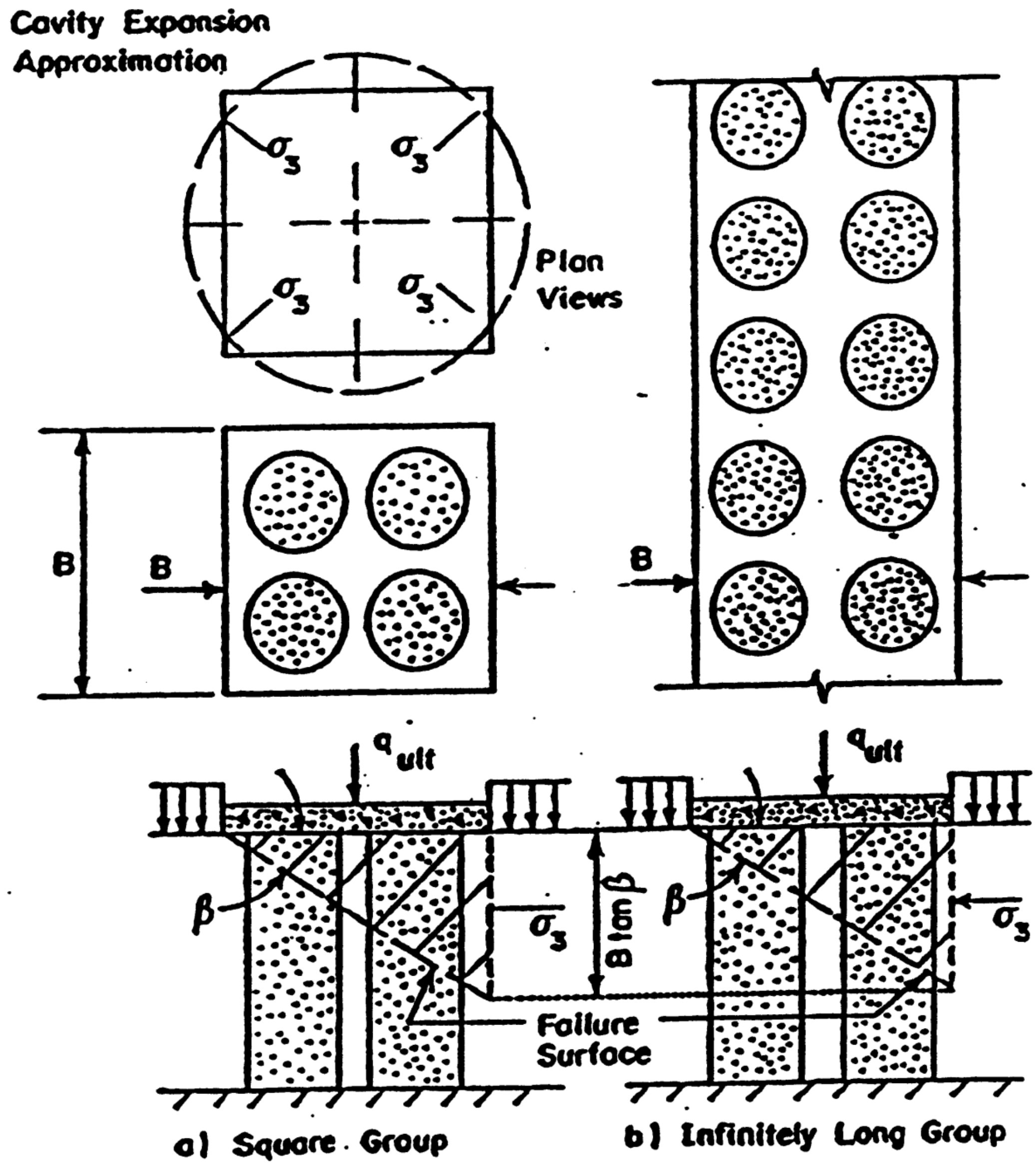


Figure 2. 12: Stone column group (Barkslade & Bachus, 1983)

The ultimate carrying capacity is defined by following equation:

(Barkslade & Bachus, 1983)

$q_{\mathrm{ult}} = \sigma_3 \tan^2\beta + 2c_{\mathrm{ave}} \tan\beta$ (2.18)

where,

q_{ult} = The ultimate carrying capacity

σ_3 = The ultimate lateral stress = $\frac{\gamma_c B \tan\beta}{2} + 2c$ (2.19)

Where, γ_c = the specific weight of the soft cohesive soil (saturated or wet)

B = Width of the foundation

β = Inclination of failure surface = $45 + \frac{\phi_{\mathrm{ave}}}{2}$

ϕ_{ave} = Internal friction angle of soil = $\tan^{-1}(\mu_s a_s \tan\phi_s)$

ϕ_s = Internal friction angle for coarse-grained soil

c_{ave} = Cohesion on the shear surface = $(1 - a_s)c_u$

a_s = Area replacement ratio = $\frac{A_s}{(A_s + A_c)}$

A_s = Plane area of stone column

A_c = Plan area of the ambient clayey soil through the unit cell

μ_s = Stress concentration factor of stone column

Where,

c_u = Undrained shear strength.

The evolution of the above method for the individual stone column is not applicable, because for the case of local bulging failure it does not take an appropriate consideration. Thus, for firm cohesive soils at which the undrain shear strength is more than 30 – 40kpa, this method is suitable and applicable. (Bergado et al., 1991)

For soft cohesive soils, the stone column capacity is predicted by using the capacity of single stone column placed within the stone column group and multiplied by the number of stone columns. (Barkslade & Bachus, 1983)

The ultimate carrying capacity of a single stone column which is isolated, in this concept is presented by:

$$q_{\mathrm{ult}} = C(N')_c \quad (2.20)$$

where,

$(N')_c =$ Composite carrying capacity coefficient (dimensionless carrying capacity factor) for the stone column which is 18 to 22.

$C =$ Cohesion of the soil

2.5 Methods of Designing: Settlement

Forecasting of the settlement is one of the most important parameter which has strong and direct effect on the designing of the stone column. Thus, many researchers have been working on this parameter (Aboshi et al., 1979; priebe, 1976; Balaam & Booker, 1981). The main part of assumption is based on the unit cell concept; in addition the loaded region is infinite. (Aboshi et al., 1979; Priebe, 1976; Balaam & Booker, 1981)

2.5.1 Methods of Analysis

I. Priebe (1976) presented that stone column works similar to an incompressible granular column that is through an elastic cylindrical without any change in lateral stresses by depth and no secondary shear. Owing to incompressibility of stone column, every change in volume of soil has straight relation to decrease in the height of stone column.

Priebe (1976) presented the following assumptions in the analysis:

a. The settlement of the ambient soft soil and stone column are equal,
b. Stresses in the two materials (stone column and soft soil) are uniform,
c. The condition of stresses of the ambient soil supposed to be isotropic,
d. Stone column carries on top of rigid layer.

Priebe (1976) proposed an improvement coefficient (R) for the settlement, that is the ratio of surface settlement of unimproved ground to improved ground and the replacement coefficient that is defined by the region of the unit cell divided by the region of the stone column. (Figure 2.13).

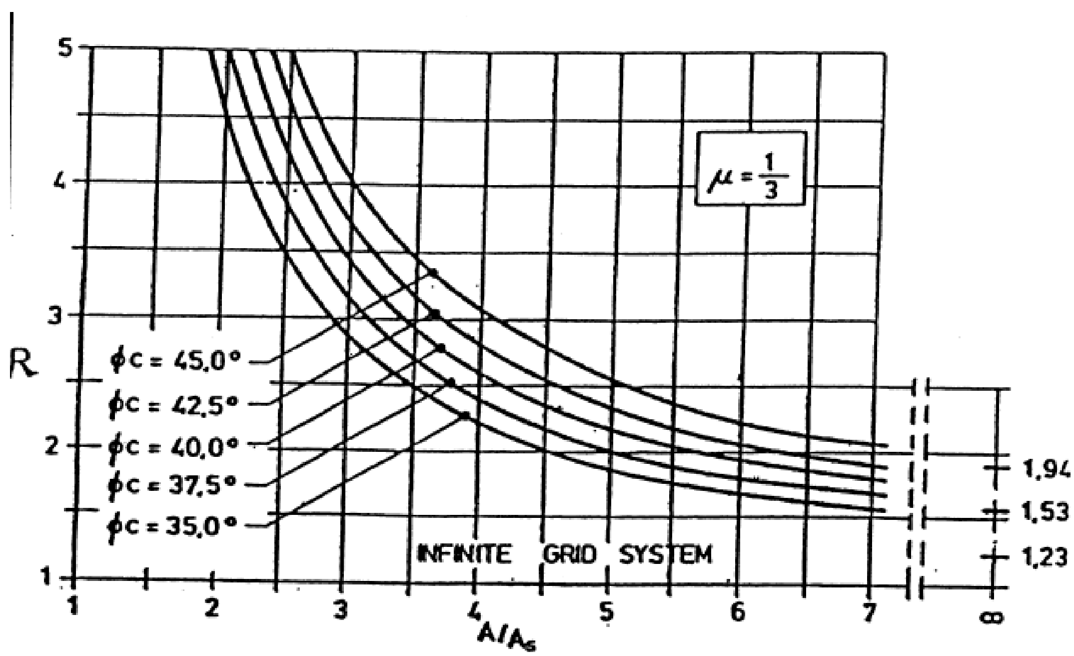


Figure 2. 13: Design Curves of Priebe (1995)

The main reason of its widely uses and popularity of Priebe's procedure is the easiness of applying an improvement coefficient to prevalent consolidation estimation. Nevertheless this procedure does not have consideration on the importance of the properties of the ambient soil.

II. Van Impe & De Beer (1983) proposed a procedure to assess the decreasing of settlement in the case of the stone column reinforcement within soft soil by considering changes of the coarse-grained strip at stable volume underneath limit of equilibrium states. In this analysis stone walls are used instead of stone columns with equal plan region as shown in Figure 2.14.

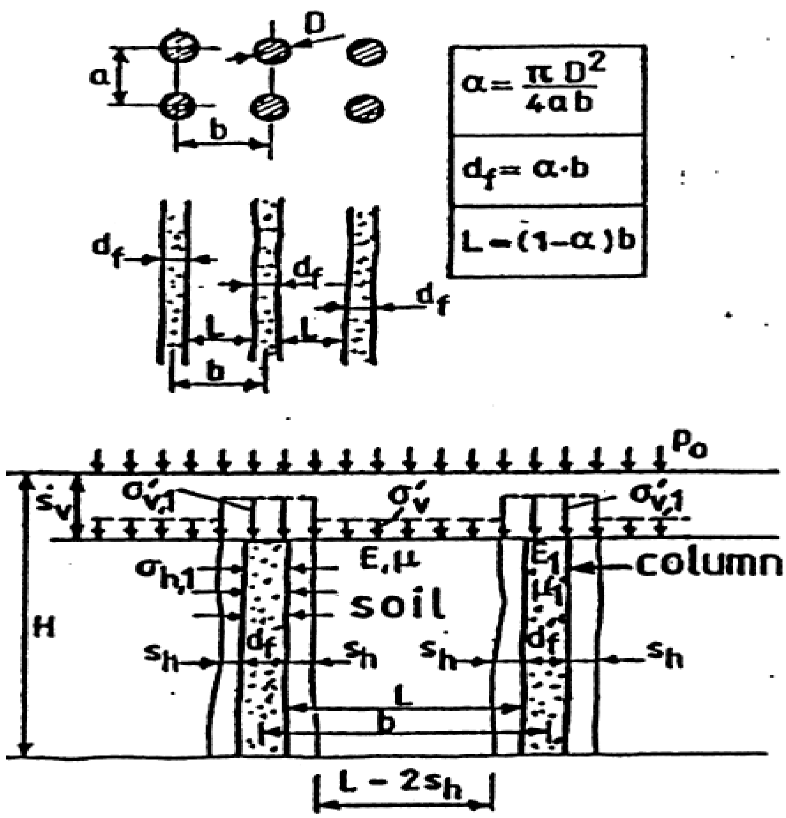


Figure 2. 14: Van Impe & De Beer Procedure (1983)

In Figure 2.15 the relationship between the settlement reduction coefficient (β) and the stress ratio of stone columns (m) versus area replacement ratio (a_s) is shown.

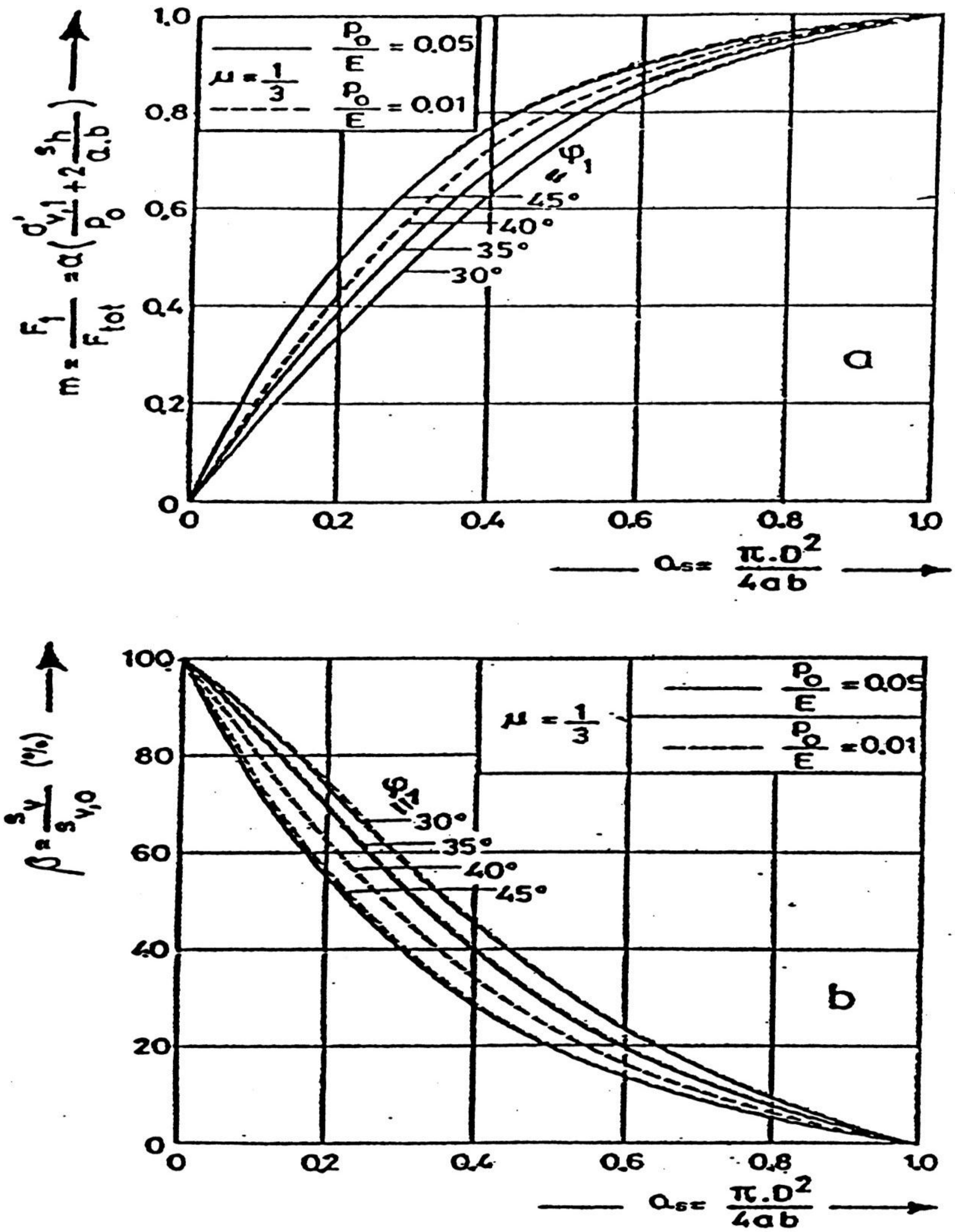


Figure 2. 15: Van Impe & De Beer Procedure (1983)

2.5.2 Procedure of Equilibrium

2.5.2.1 Theory of Barkslade & Bachus (1983)

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

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

Barkslade and Bachus (1983) presented the curve of consolidation settlement of improved and unimproved ground. Settlement of improved ground (S_t) is given by:

$$S_t = \left(\frac{C_c}{(1+e_0)}\right) H \log_{10}\{\frac{(\bar{\sigma}_0+\sigma_c)}{\sigma_0}\} \quad (2.21)$$

(Barkslade & Bachus, 1983) (Barkslade & Bachus, 1983)Settlement of unimproved ground (S_0) is given by:

$$S_0 = \left(\frac{C_c}{(1+e_0)}\right) H \log_{10}\{\frac{(\bar{\sigma}_0+\sigma)}{\sigma_0}\} \quad (2.22)$$

where,

C_c = Compression index obtain from 1D consolidation test

e_0 = Initial void ratio

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

$\bar{\sigma}_0 =$ Average initial effective stress in the clay layer

$\sigma_c =$ Change in stress in the clay layer owing to externally applied load

$$\sigma_c = \mu_c.\sigma \tag{2.23}$$

where,

$\sigma =$ The average externally applied stress

$\mu_c =$ The ratio of stresses in the clay

$$\mu_c = \frac{1}{1+(n-1)a_s} \tag{2.24}$$

$n =$ Stress concentration factor

$a_s =$ Area replacement ratio

They presented that the settlement reduction ratio is defined by:

$$\beta = (\frac{S_t}{S_0}) = \frac{\log_{10}(\frac{\sigma_0+\mu_c\sigma}{\sigma_0})}{\log_{10}(\frac{\sigma_0+\sigma}{\sigma_0})} \tag{2.25}$$

If assume stone column with long length that means σ_0 is very large and very small applied stresses (σ), so the settlement reduction ratio is decreases:

$$\beta = \left(\frac{S_t}{S_0}\right) = \frac{1}{1+(n-1)a_s} = \mu_c \tag{2.26}$$

Barlslade (1983) mentioned that, the equilibrium procedure presented by equation (2.25) is more reliable to forecast settlement of composite ground, and the equation (2.26) gives a little bit unconservative assessment of anticipated ground improvement and is useful for preliminary assessments.

2.5.2.2 Theory of Aboshi (1979)

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

$$\beta = \left(\frac{S_t}{S_0}\right) = \frac{m_v(\mu_c\sigma)H}{m_v\sigma H} = \frac{1}{1+(n-1)a_s} \tag{2.27}$$

where,

$m_v =$ Coefficient of volume compressibility

For large amount of σ_0, the above equations which are presented by Barkslade and Aboshi for estimation of β are equal. For suitability of this procedure, Aboshi (1979) presented a graph, at which the predicted settlement versus in-situ data of settlement is shown in figure 2.17. Nevertheless Aboshi for predition of settlement did not give any properties about in-situ soil.

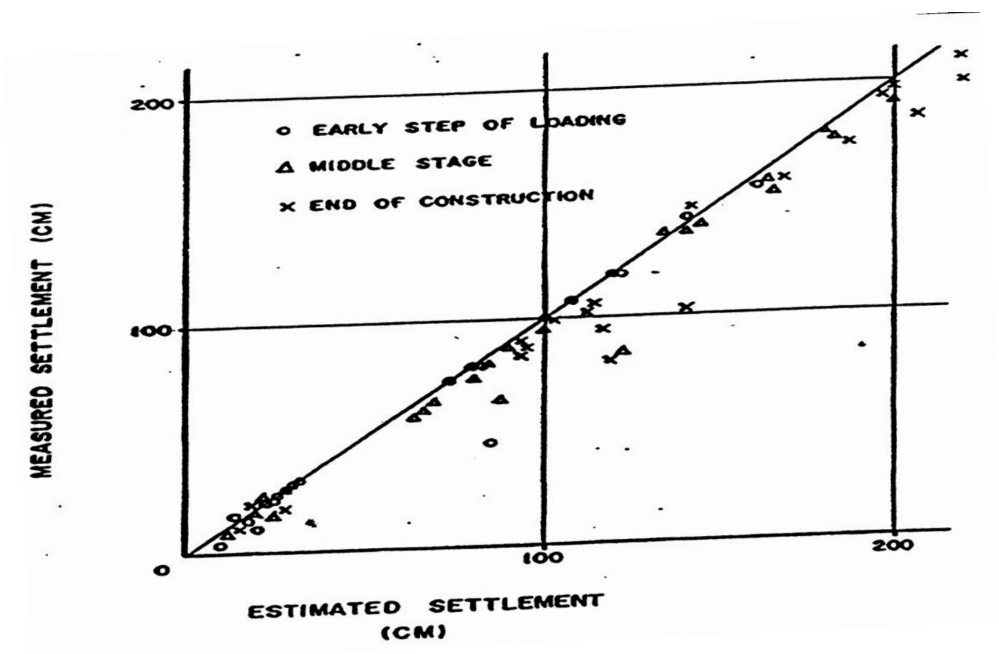


Figure 2. 16: Comparison of Predicted Settlement and Measured Settlement (After Aboshi et al., 1979)

2.5.2.3 Theory of Greenwood (1975)

Greenwood (1975) proposed empirical curves of settlement reduction owing to ground reinforcement by stone columns as a function of undrained soil strength and stone column spacing. Comparison of this method and equilibrium procedure (Barkslade, 1983) is shown in Figure 2.18 for different stress concentration coefficient (n). (n= 3,5,10 and 20)

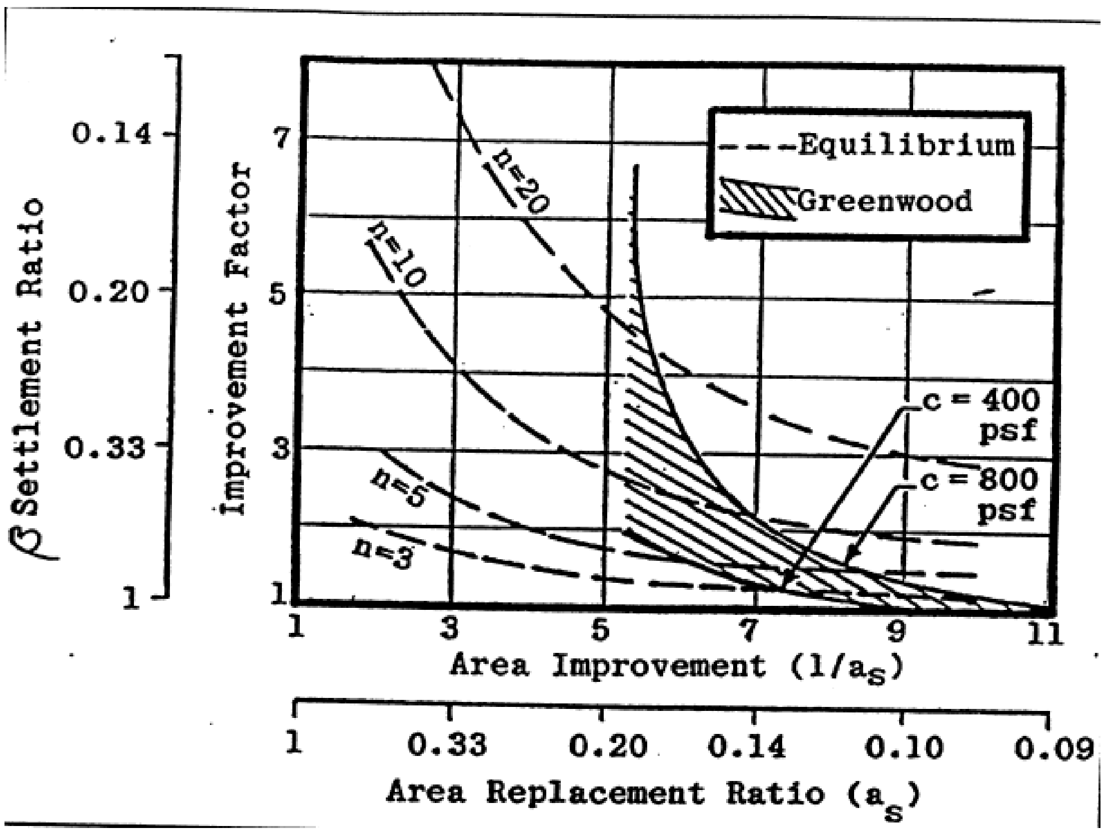


Figure 2. 17: Comparison of Greenwood method and equilibrium procedures (After Barkslade & Bachus, 1983)

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

2.5.2.4 Theory of Goughnour and Bayuk (1979)

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

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

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

2.5.2.5 Theory of Balaam et al., (1977)

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

2.6 Review of the Finite Element Procedure

2.6.1 Introduction

The main theory of finite element procedure is the idealization of the continuous sequence as accumulation of a finite numeral of distinct elements. Those elements are linked together at a finite digit of nodal points. Then, the treatment of the continuous sequence is estimated by supposing the treatment of the elements. Balanced equations are associated in expression of displacements of unknown nodal. On the basis of estimation made of this collection of equations establishes a solution for the finite element procedure. The performance of the following stages has a major role to reach accurate results;

- Analysis of continuous sequence
- Selection of fundamental approach
- Solving procedure of equation and calculation of the element stresses and strains for the displacement of nodal as a finite element examination.

2.6.2 Finite element detachment

For solving a problem, the detachment of the area into subareas is the first of a sequence of steps that must be executed. The detachment of the area brings in the decision as to the numeral, form and size of subareas used to display the proper body.

Basically for getting the valid results, the elements have to be small enough. The element size should be reduced in the regions where the coveted consequence is fairly constant.

2.7 PLAXIS software

2.7.1 Introduction

A finite element PC-software called Plaxis is applicable in geotechnical and hydraulical applications. In this program soil is modeled in order to simulate the treatment of soil. The Plaxis was developed by researchers of the University of Delft, Holland. Plaxis is planned to supply dominant software for theoretical geotechnical finite element analysis. The software uses defined interfaces that allow users to create a geometry prototype and finite element, three dimensional 3D and two-dimensional 2D, mesh. Prototype can be created by an axisymmetric or a plain strain model. Plaxis is powerful for estimating deformation, excess pore water pressure, consolidation settlement and other geotechnical parameters.

2.7.2 Types of PLAXIS element

For two dimensional, Plaxis 2D there are two triangular elements 1) 6-node and 2) 15-node, which can be defined in axisymmetric or plane strain analysis. For current study, 15-noded by triangular pattern was used.

Chapter 3

ANALYSIS OF SINGLE STONE COLUMN

3.1 Introduction

Finite element analysis is widely used to predict the behavior of ground under structures. In this study, an attempt was made to analyze the behavior of single stone column as a piece of natural soil reinforced by stone columns. Owing to the cross section of stone column which is circular, the area of influence of stone column is cylindrical. Thus, in this chapter the single stone column was modeled by axisymmetric pattern.

In-situ study of stone columns reinforcement shows the important role of stone columns to accelerate the consolidation rate of soft clay (Han and Ye, 1992). The important role of stone columns is to influence excess pore water pressure, once the load is applied, from the ambient soil and discharge it within the stone columns that lead to accelerate the consolidation rate. Hence, influential radial flow has to happen towards the single stone column when the unit cell is under loading with vertical compaction.

3.2 Unit Cell Concept

The behavior of single stone column reinforced clay and its influence to improve the geotechnical properties were analyzed via the unit cell estimation. Modeling of single column within cylindrical unit cell is designed by axisymmetric unit cell as illustrated in Figure 3.1

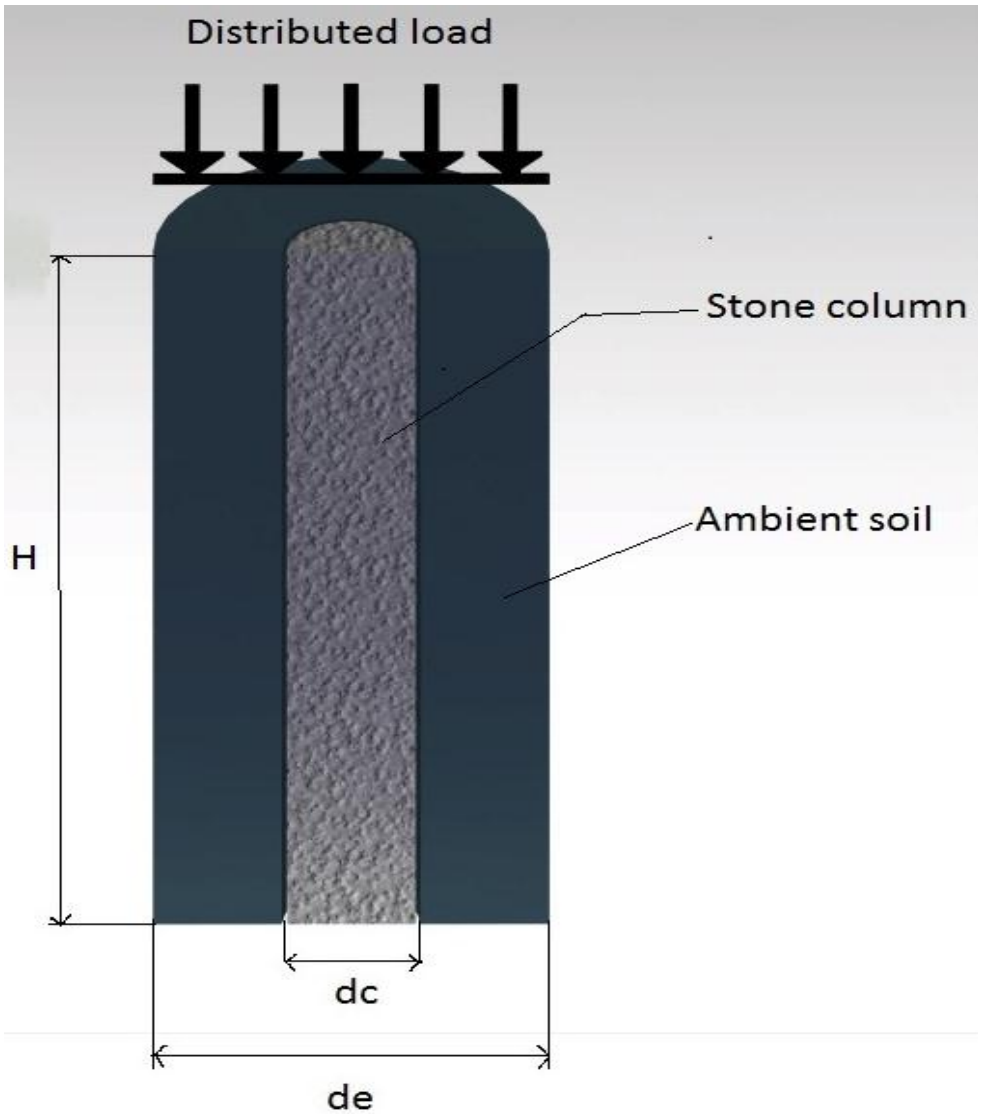


Figure 3. 1: Unit Cell of Stone Column (axisymmetric)

The main distinctions of stone columns in comparison with drain wells contain the consideration of stiffness distinction between the ambient soil and stone column. Han and Ye (2001) presented the modified consolidation coefficient for the patchy differential equation administrating the stone column consolidation within unit cell as:

$$c_r'(\frac{1}{r}\frac{\partial u}{\partial r}+\frac{\partial^2 u}{\partial r^2})+c_v'\frac{\partial^2 u}{\partial z^2}=\frac{\partial \bar{u}}{\partial t} \tag{3.1}$$

The radial and vertical consolidation coefficient is defined by:

$$C_r' = (\frac{k_r}{\gamma_w})\frac{[m_{v,c}(1-a_s)+m_{v,s}a_s]}{[m_{v,s}m_{v,c}(1-a_s)]} \tag{3.2}$$

$$C_v' = (\frac{k_v}{\gamma_w})\frac{[m_{v,c}(1-a_s)+m_{v,s}a_s]}{[m_{v,s}m_{v,c}(1-a_s)]} \tag{3.3}$$

The compressibility coefficient for stone column and the ambient soil is given by:

$$m_v = \frac{(1+\nu)(1-2\nu)}{E(1-\nu)} \tag{3.4}$$

Terzaghi's one-dimensional consolidation solution is applicable for the analysis of vertical flow, and Barron (1947) drain well equations are applicable for the analysis of radial flow. The one-dimensional consolidation solution owing to vertical flow, which is described in the books of soil mechanics, is applicable for stone column reinforced ground by application of consolidation coefficient owing to vertical flow (C_v') rather than (c_v). The rate of consolidation in Terzaghi's one-dimensional theory is given by:

$$U = 1-\frac{u_z}{u_0} \tag{3.5}$$

where,

u_z= Excess pore water pressure at time (t)

u_0= Initial excess pore water pressure

3.3 Finite Element Analysis (Consolidation Analysis)

Plaxis 2D, finite element analysis was carried out for natural clay and for the same clay modified by single stone column (unit cell) under static and dynamic loading for a period of 560-days. The modeling of single stone column is designed by axisymmetric pattern in Plaxis. For consolidation analysis, coupled consolidation concept was assumed. The following assumptions in consolidation analysis were used:

- Strains are small,
- Stone column is full-depth saturated,
- The soil thickness within the consolidation procedure is constant.

The different diameters of stone column were applied for the analysis and the results were compared. The axisymmetric unit cell was analyzed. During consolidation analysis, the loading applied was assumed to be uniform and it was assumed that it was applied immediately through the sand layer. During the consolidation analysis, the distributed load was assumed to remain constant. The stone column behaves like drain wells within the unit cell. The soil model was defined as linear elastic model. The consolidation computation was continued until the minimum rate of dissipation of pore water pressure was accomplished. The results of finite element analysis for treated clay by single stone column and untreated clay were compared. The properties of clay, stone column material and sand are given in Table 3.1. The geometry data of different diameters of stone column is given in Table 3.2, and the data of distributed dynamic and static loading is given in Table 3.3.

Table 3. 1: The properties of clay, stone column material and sand used in Plaxis

	Model	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_h (m/day)	k_v (m/day)	E' (kPa)	ν'	K_0
Clay	Mohr-coulomb	Undrained	13.5	13.5	0.001	0.001	2700	0.33	0.7
Stone column	Mohr-coulomb	Drained	19	20	5	5	30000	0.3	0.7
Sand	Mohr-coulomb	Drained	16	20	1	1	3000	0.3	0.5

Table 3. 2: The geometry data of different diameters of stone column

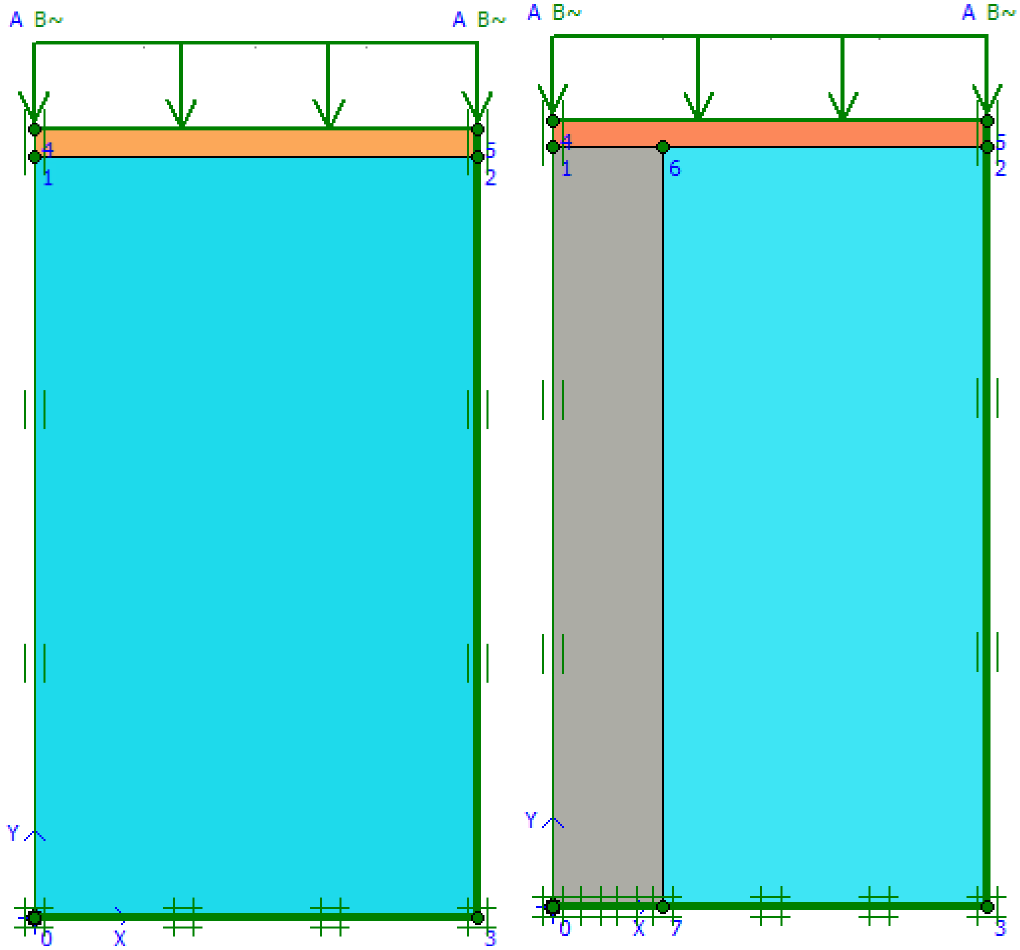
	Height (m)	de (m)	dc (m)	R
Model 1	7	4	1	4
Model 2	7	4	0.75	5.33
Model 3	7	4	0.5	8
Model 4	7	4	0.25	16

Table 3. 3: The data of static and dynamic loading

<table>
<tr><td rowspan="3">Load</td><td rowspan="2">Dynamic</td><td>Frequency(Hz)</td><td>20</td></tr>
<tr><td>Amplitude(kPa)</td><td>20</td></tr>
<tr><td colspan="2">Static(kPa)</td><td>120</td></tr>
</table>

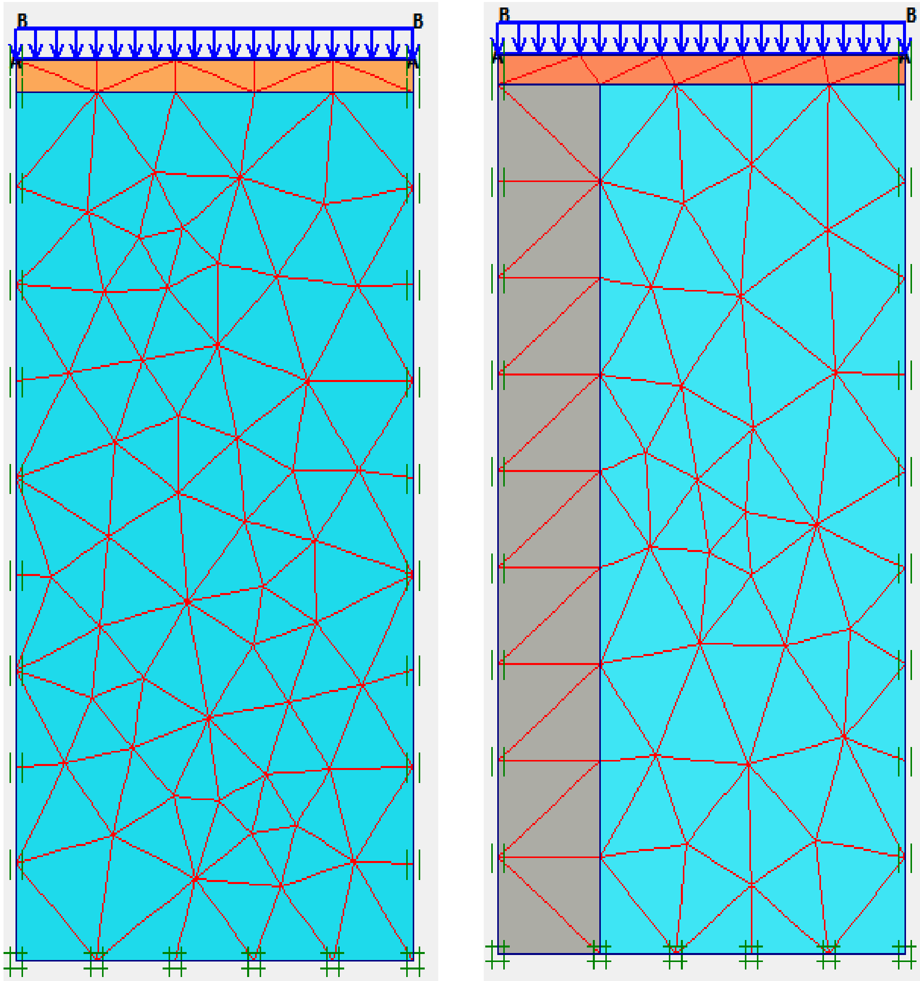
3.3 Procedure of analysis

The stone column (unit cell) used to reinforce the soft cohesive soil was modeled in the finite element analysis, Plaxis 2D, program as shown in Figure 3.2 (a-b). The finite element mesh for those two models is shown in Figure 3.3 (a-b).



(a) (b)

Figure 3. 2: (a) Untreated Soil Model, (b) Unit Cell Stone Column (Axisymmetric)



(a) (b)

Figure 3. 3: (a) Untreated Soil Mesh, (b) Unit Cell Stone Column Mesh

3.4 Discussion of the results

The stone column unit cell simulated in finite element analysis in comparison with the untreated clay model for different ratios of R ($\frac{de}{dc}$) was analyzed. Firstly, the settlement behavior of the untreated clay was compared with the settlement behavior of different diameters of stone column. Owing to stone column reinforcement, the settlement behavior of clay is improved based on ratio of R. Thus by decreasing the ratio of R, with consideration of the full-depth of the stone column, the settlement of the soft clay as shown in Figure 3.4- 3.5- 3.6 is decreased.

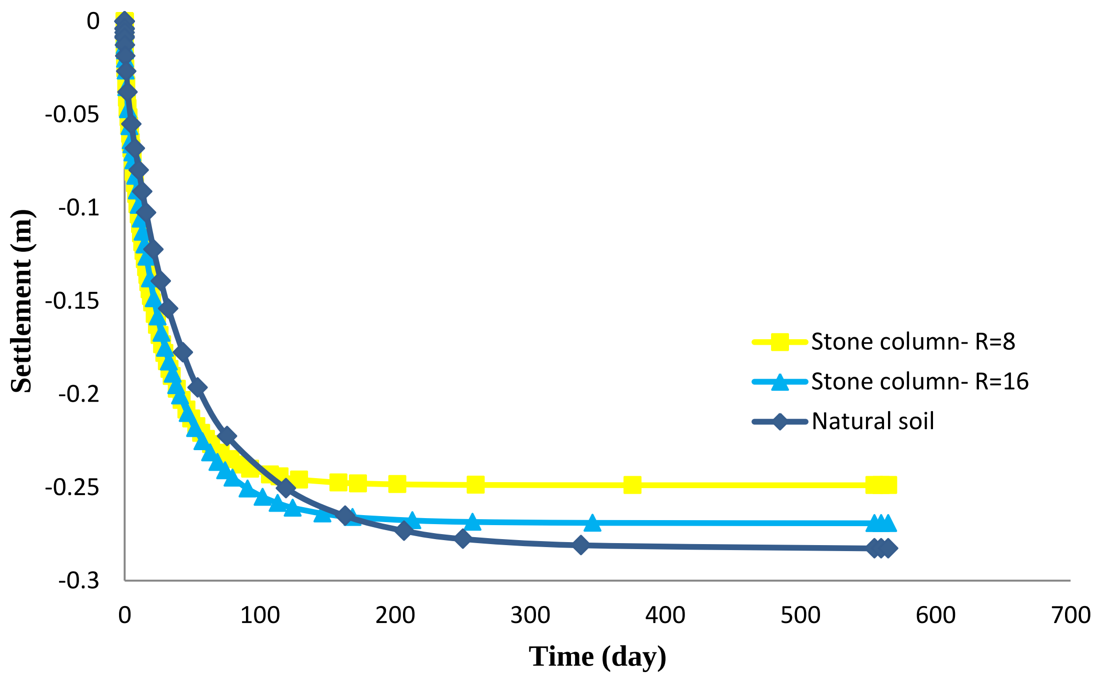


Figure 3. 4: Influence of the ratio of R on the settlement behavior of clay in case of H=7, de/dc=16 and de/dc=8

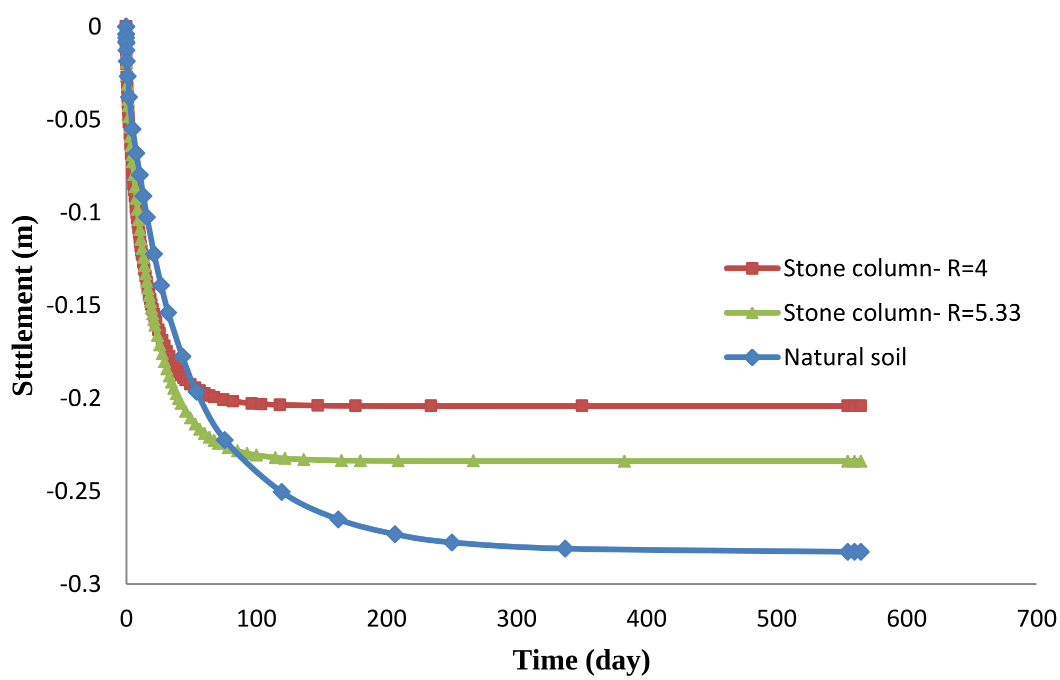


Figure 3. 5: Influence of the ratio of R on the settlement behavior of clay in case of H=7, de/dc=5.33 and de/dc=4

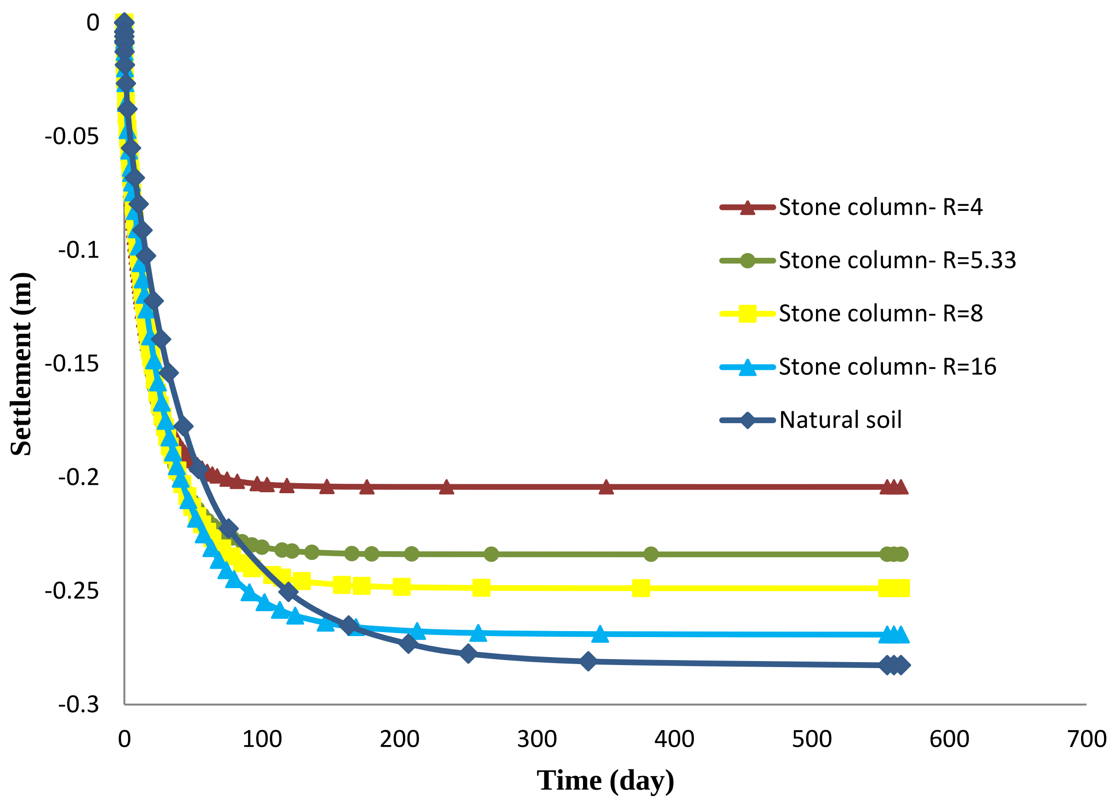


Figure 3. 6: The influence of different diameter of stone column on the settlement behavior

The results obtained from the finite element analysis also clearly presented the important role of stone column to accelerate the consolidation process. Stone column plays two influential roles in the soft cohesive soil, 1) as a part of soil, it improves the settlement behavior of the soft soil, 2) stone column behaves like drain wells and accelerates the consolidation process. Figure 3.7- 3.8- 3.9; show the stone column with different ratio of R and the untreated clay soil. The figures indicated that with larger diameter of stone column application, the consolidation behavior of soil was improved and the consolidation process was accelerated. Within the constant depth of the stone column, less time was needed for the average rate of consolidation of the soil to be achieved.

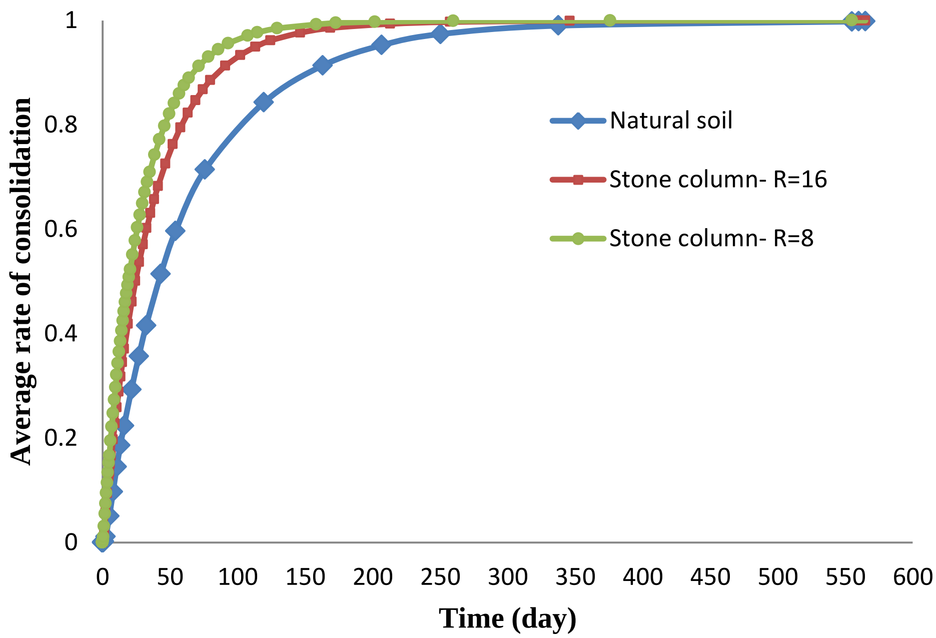


Figure 3. 7: The comparison of the rate of consolidation of the untreated clay and the stone column unit cell with different diameters: H=7m, R=16 and R=8

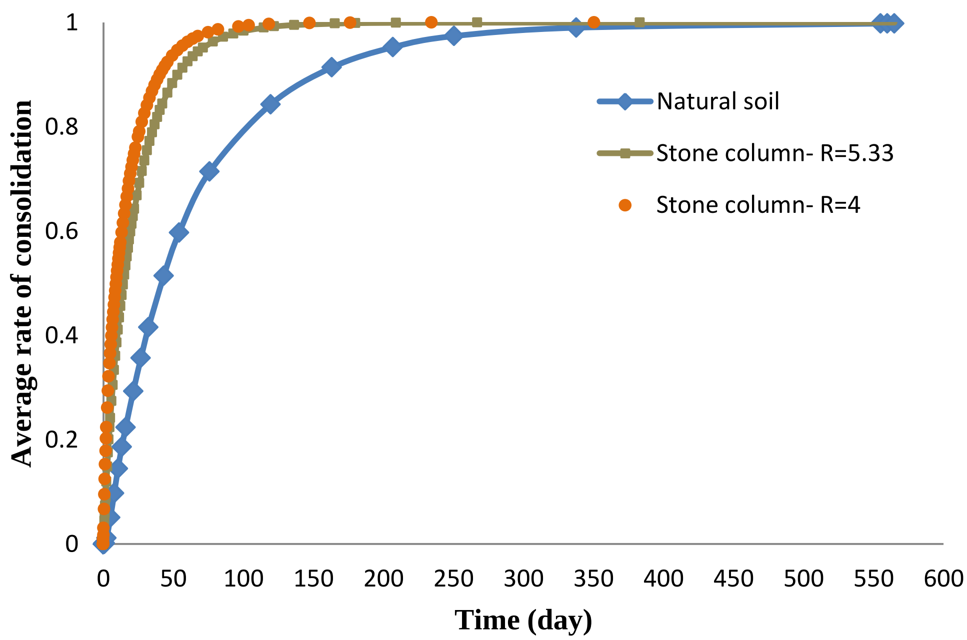


Figure 3. 8: The comparison of the rate of consolidation of the untreated clay and the stone column unit cell with different diameters: H=7m, R=5.33 and R=4

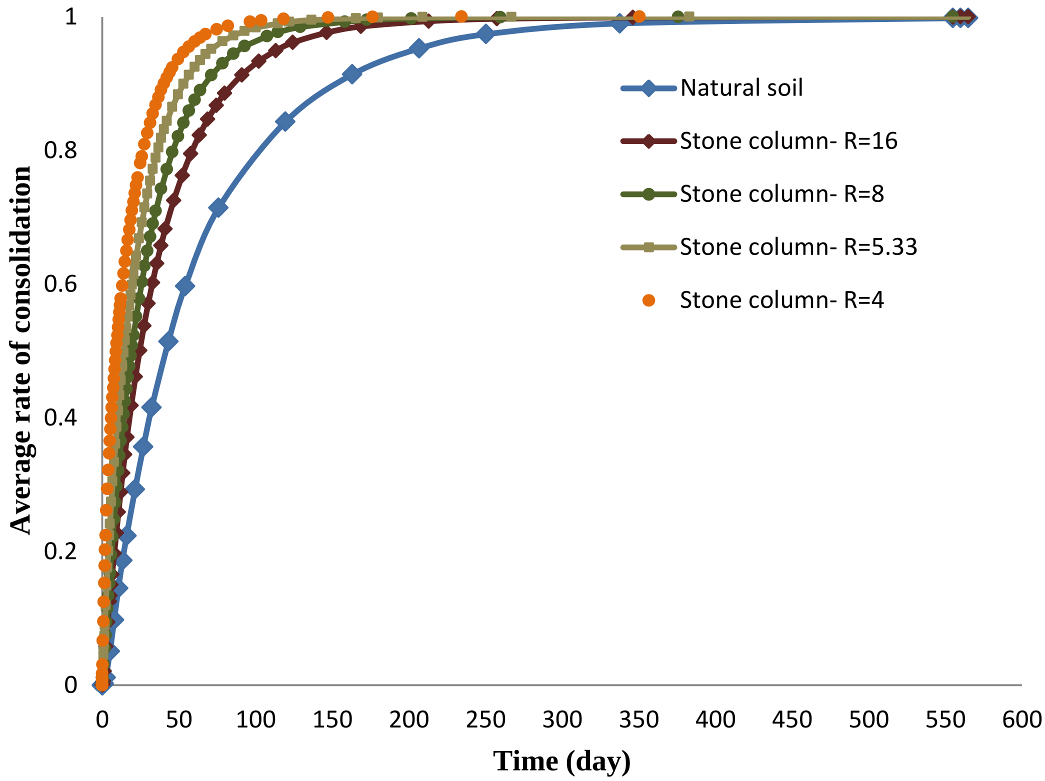


Figure 3. 9: The comparison of the rate of consolidation of the untreated clay and the stone column unit cell with different diameters: H=7m, R=5.33, R=4, R=8 and R=16

The results of the finite element analysis for the axisymmetric modeling of stone column unit cell and the untreated clay were compared and the results indicated that, the rate of dissipation of excess pore water pressure was accelerated with the larger diameter of the stone column. Figure 3.10, Figure 3.11 and Figure 3.12 indicated that the complete consolidation of the clay layer with larger stone column diameter was achieved at much shorter period of time than the untreated clay. The excess pore water pressure is decreased with a reduction in the R ratio. Thus in the smaller ratio of R, the dissipation rate of excess pore water pressure is much quicker than the untreated clay.

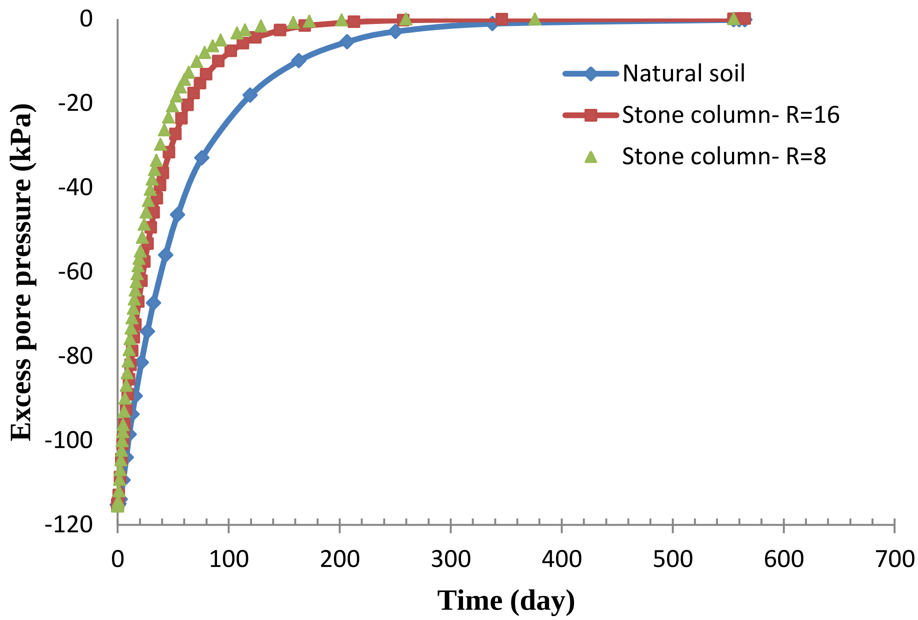


Figure 3. 10: Dissipation of excess pore water pressure for the untreated clay and the clay with the stone column unit cell with different diameter: H=7m, R=16 and R=8

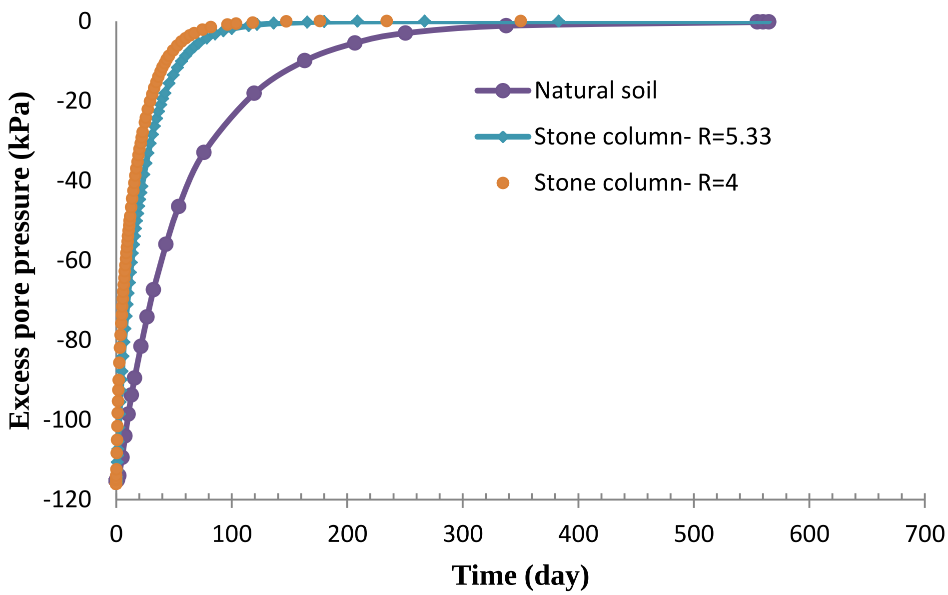


Figure 3. 11: Dissipation of excess pore water pressure for the untreated clay and the clay with the stone column unit cell with different diameter: H=7m, R=5.33 and R=4

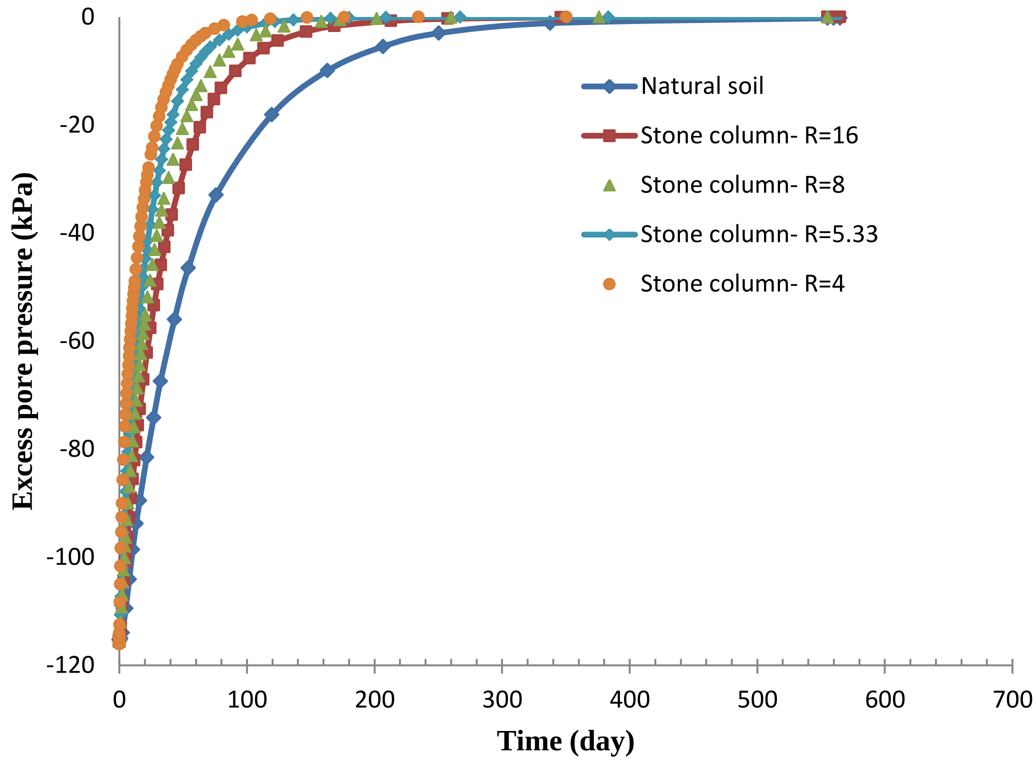


Figure 3. 12: Dissipation of excess pore water pressure for the untreated clay and the clay with the stone column unit cell with different diameter: H=7m, R=5.33, R=4, R=8 and R=16

Chapter 4

COMPARISON OF STONE COLUMN AND SAND COLUMN IN AXISYMMETRIC MODEL

4.1 Introduction

In general, structures (buildings, roads, bridges, etc.) are subjected to static and dynamic loading. Thus, in the case of the improvement of soft soil by stone column reinforcement, static and dynamic loading must be considered. Dynamic loads can be generated by vehicles movement, earthquake, train travel, etc. In geotechnical field, the settlement behavior of the soil which can be subjected to two types of loading (static & dynamic) should be improved for these different types of loadings. In this chapter the behavior of stone column and sand column under static and dynamic loading will be analyzed.

4.2 Analysis of Stone Column and Sand Column (Unit Cell)

4.2.1 Bulging failure

Axial load on the stone column and the sand column may cause a big bulge in depth of 2 to 3 times of the diameter of stone column under the ground surface. Owing to this bulge, the lateral stress through the cohesive soil increases in order to supplies extra confinement for the stone column. A balance condition is finally attained resulting in decreased vertical displacement in comparison with the untreated soil. (Barkslade & Bachus, 1983)

Hughes & Withers (1974) performed tests on sand column in the diameter of 12.5 to 38 mm and the length of 150 mm. They showed the bulging failure of sand column as shown in Figure 4.1.

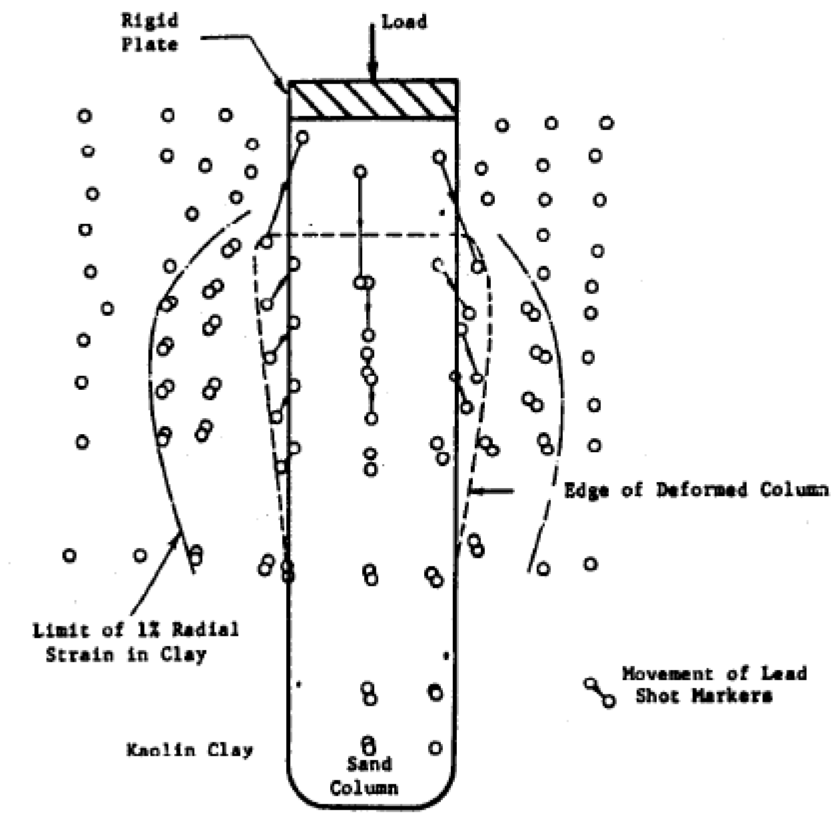


Figure 4. 1: The bulging failure of stone column (Hughes & Withers, 1974)

4.2.2 Vertical displacement

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

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

Barkslade and Bachus (1983) presented the curve of consolidation settlement of improved and unimproved ground. Settlement of improved ground (S_t) is given by:

$$S_t = \left(\frac{C_c}{(1+e_0)}\right) H \log_{10}\{\frac{(\bar{\sigma}_0+\sigma_c)}{\sigma_0}\} \tag{4.1}$$

Settlement of unimproved ground (S_0) is given by:

$$S_0 = \left(\frac{C_c}{(1+e_0)}\right) H \log_{10}\{\frac{(\bar{\sigma}_0+\sigma)}{\sigma_0}\} \tag{4.2}$$

where,

$C_c =$ Compression index obtain from 1D consolidation test

$e_0 =$ Initial void ratio

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

$\bar{\sigma}_0$ = Average initial effective stress in the clay layer

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

$$\sigma_c = \mu_c . \sigma \tag{4.3}$$

where,

σ = The average externally applied stress

μ_c = The ratio of stresses in the clay

$$\mu_c = \frac{1}{1+(n-1)a_s} \tag{4.4}$$

n = Stress concentration factor

a_s = Area replacement ratio

Barkslade & Bachus (1983) presented that the vertical settlement of stone column can be calculated by the following equation:

$$S_s = \frac{\sigma_s L}{D_s} \tag{4.5}$$

where,

S_s= Vertical displacement of stone column

σ_s= Average stress in stone column

L= Length of the stone column

D_s= constrained modulus of the stone column (for upper bound young's modulus, Es can be used)

4.3 Unit Cell of Sand Column and Stone Column

4.3.1 Comparison of axisymmetric modeling for sand column and stone column

Plaxis 2D, finite element analysis was carried out on sand column reinforced clay and on the same clay modified by single stone column (unit cell) under static and dynamic loading for a period of 560-days. The modeling of sand column and stone column is designed by axisymmetric pattern in Plaxis. For consolidation analysis, coupled consolidation concept was supposed. The following assumptions in consolidation analysis were used:

- Strains are small,
- Stone column and sand column are full-depth saturated,
- The soil thickness within the consolidation procedure is constant

The different diameters of stone column and sand column under various static loading were chosen and analyzed by using the axisymmetric unit cell pattern. During consolidation and dynamic analysis, the loading applied is assumed to be uniform and applied immediately at the top of stone and sand columns. During the consolidation analysis, the distributed load assumed to remain constant. The soil model was defined as linear elastic model. The consolidation computation was continued until the minimum rate of pore water pressure was accomplished. The results of finite element analysis for the treated clay by stone column and sand column were compared. The properties of stone column unit cell and sand column unit cell are given in Table 4.1 and Table 4.2, respectively. The geometry data of different diameters of stone column and sand column are given in Table 4.3 and Table 4.4, respectively. The data of distributed dynamic and static loading for stone column and sand column is given in Table 4.5 and Table 4.6, respectively.

Table 4. 1: The properties of stone column unit cell

	Model	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_h (m/day)	k_v (m/day)	E' (kPa)	ν'	K_0
Clay	Mohr-coulomb	Undrained	13.5	13.5	0.001	0.001	2700	0.33	0.7
Stone column	Mohr-coulomb	Drained	19	20	5	5	30000	0.3	0.7

Table 4. 2: The peropertis of sand column unit cell and the geotextile

	Model	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_h (m/day)	k_v (m/day)	E' (kPa)	ν'	K_0
Clay	Mohr-coulomb	Undrained	13.5	13.5	0.001	0.001	2700	0.33	0.7
Sand	Mohr-coulomb	Drained	16	20	1	1	20000	0.3	0.5
Geotextile	Elastic	$\text{EA} = 1.9\frac{\text{KN}}{m}$							

Table 4. 3: The geometry data of different diameters of stone column

	Height (m)	de (m)	dc (m)	R
Model 1	10	6	1.5	$R_1 = 4$
Model 2	10	6	1.2	$R_2 = 5$
Model 3	10	6	1	$R_3 = 6$

Table 4. 4: The geometry data of different diameters of sand column

	Height (m)	de (m)	dc (m)	R
Model 1	10	6	1.5	$R_1 = 4$
Model 2	10	6	1.2	$R_2 = 5$
Model 3	10	6	1	$R_3 = 6$

Table 4. 5: The dynamic and static loading data for stone column

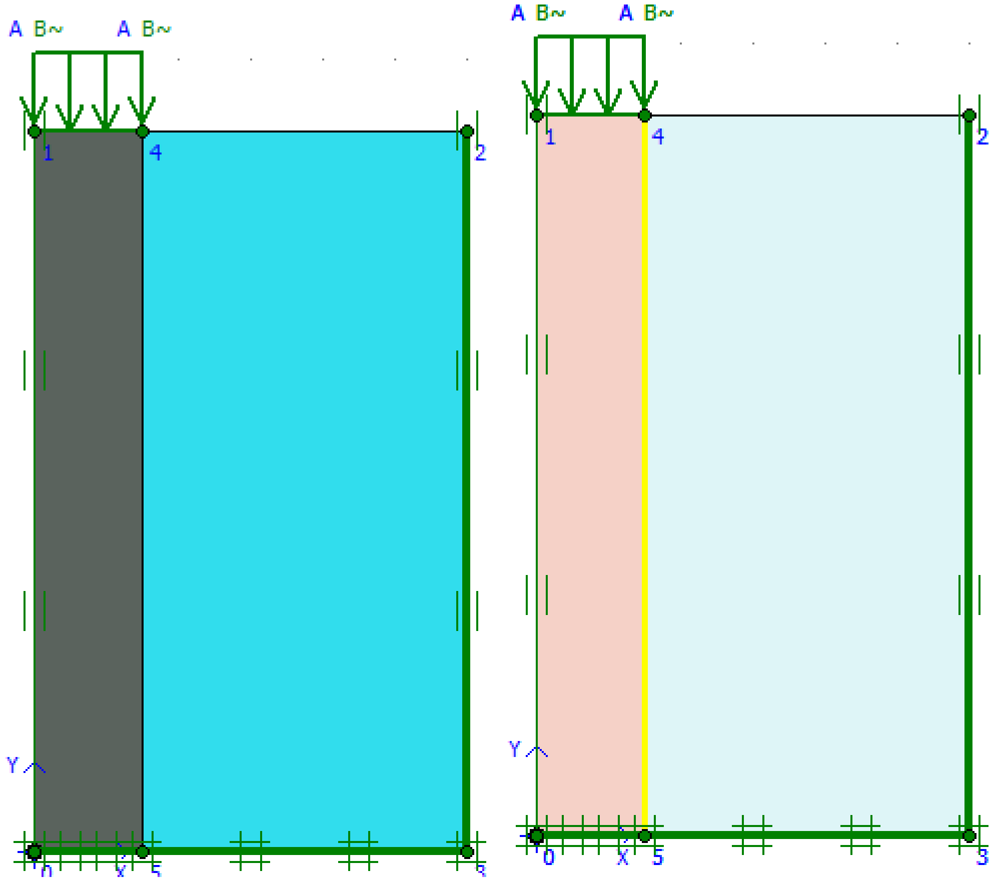
Load	Dynamic	Frequency(Hz)	20
		Amplitude(kpa)	20
	Static	q_1(kPa)	80
		q_2(kPa)	110
		q_3(kPa)	130

Table 4. 6: The dynamic and static loading data for sand column

Load	Dynamic	Frequency(Hz)	20
		Amplitude(kpa)	20
	Static	q_1(kpa)	80
		q_2(kpa)	110
		q_3(kpa)	120

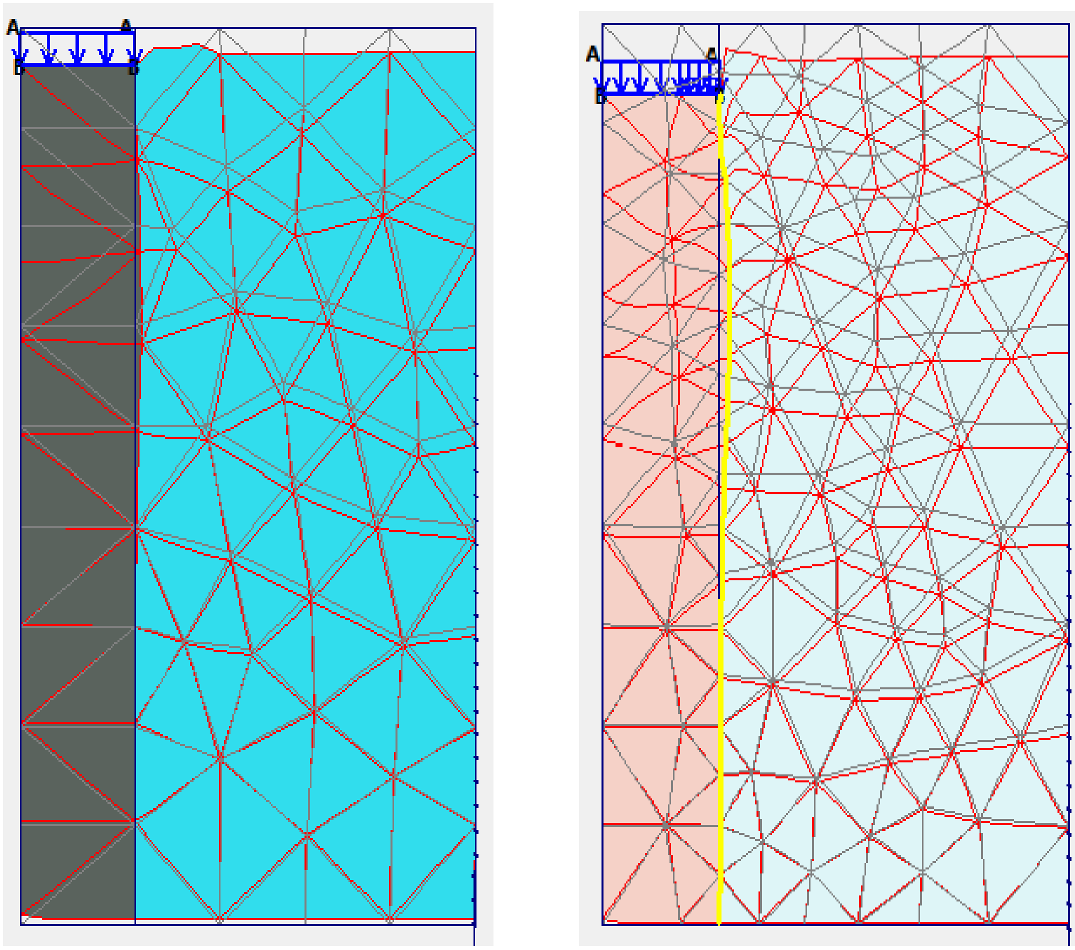
4.3.2 Analysis of Stone Column and Sand Column

The stone column and sand column coated with geotextile were modeled in the finite element analysis, Plaxis 2D, program as shown in Figure 4.2 (a-b). The deformed mesh of stone column and sand column for R= 4, q_1 load and dynamic load at the end of analysis is shown in Figure 4.3 (a-b).



(a) (b)

Figure 4. 2: (a) Stone column unit cell (axisymmetric), (b) Sand column coated with geotextile (axisymmetric)



(a) (b)

Figure 4. 3: (a) The stone column deformed mesh, (b) The sand column deformed mesh (R= 4 and q1= 80kPa)

4.4 Discussion of the Results

The bulging of stone column and sand column in different ratios of R ($\frac{d_e}{d_c}$) and under various amounts of loading (q_1, q_2, q_3) was analyzed. As shown in Figure 4.4- 4.5- 4.6 - 4.7, the bulging of stone column and sand column at the specific ratio of R occurred in the same depth. In other word, the depth of bulging is related to the diameter of stone and sand column. Thus by decreasing the diameter of stone and sand column, the depth of the maximum value of bulging will be decreased. Through keeping accurate observation on the maximum value of bulging in stone column and sand column, at depth of two times of the diameter of stone and sand column, the maximum value of bulging occurred. As illustrated in Figure 4.4- 4.5- 4.6- 4.7, the main influence of different amounts of the axial load on the stone and sand column is the value of maximum bulging. By increasing the axial load at top of the stone and sand column the value of maximum bulging will be increased.

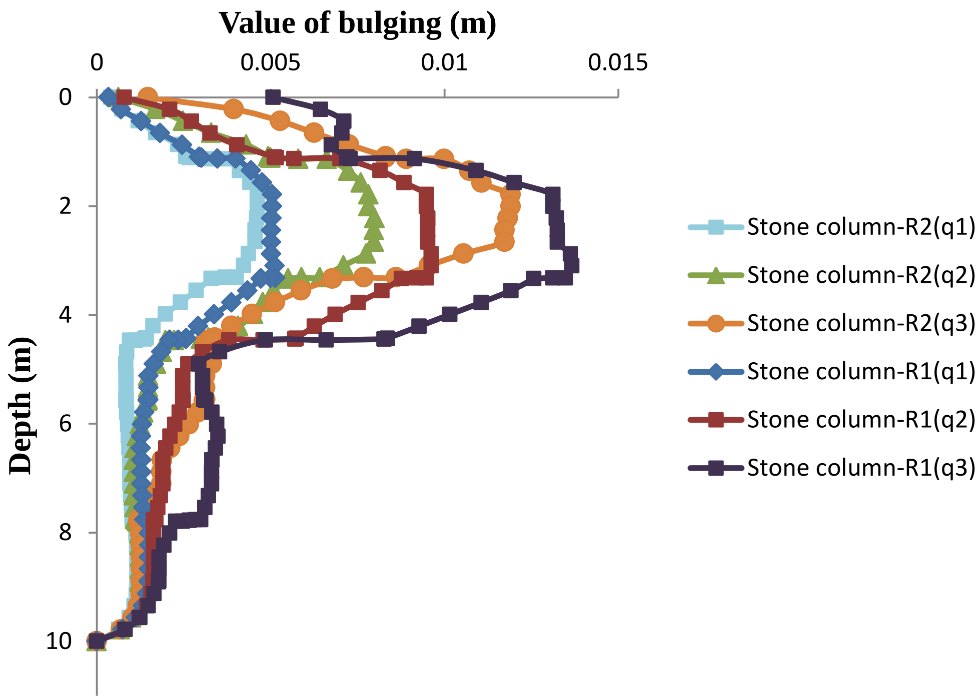


Figure 4. 4: The value of bulging of stone column with depth for the ratios of R1 & R2 and under various loading (q1,q2,q3)

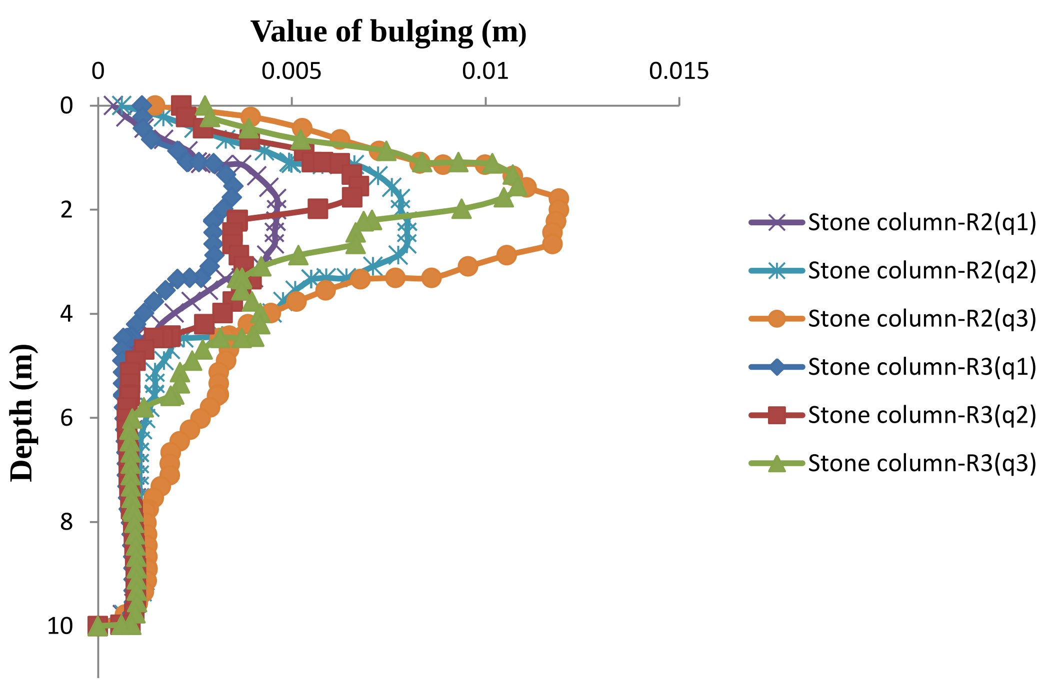


Figure 4. 5: The value of bulging of stone column with depth for the ratios of R2 & R3 and under various loading (q1, q2, q3)

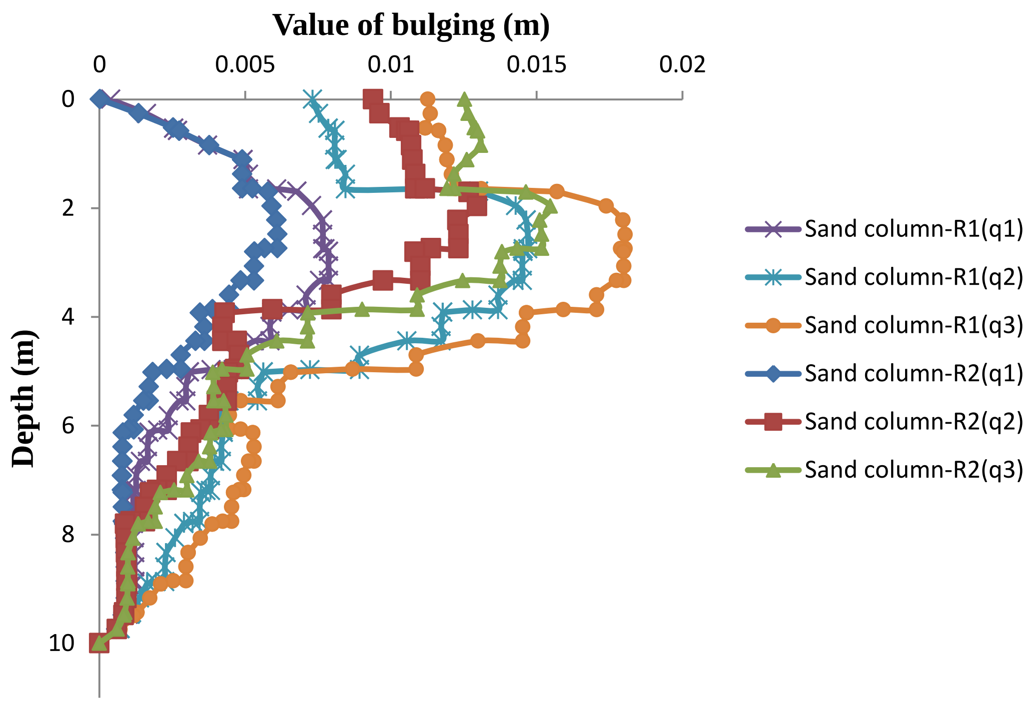


Figure 4. 6: The value of bulging of sand column with depth for the ratios of R1 & R2 and under various loading (q1, q2, q3)

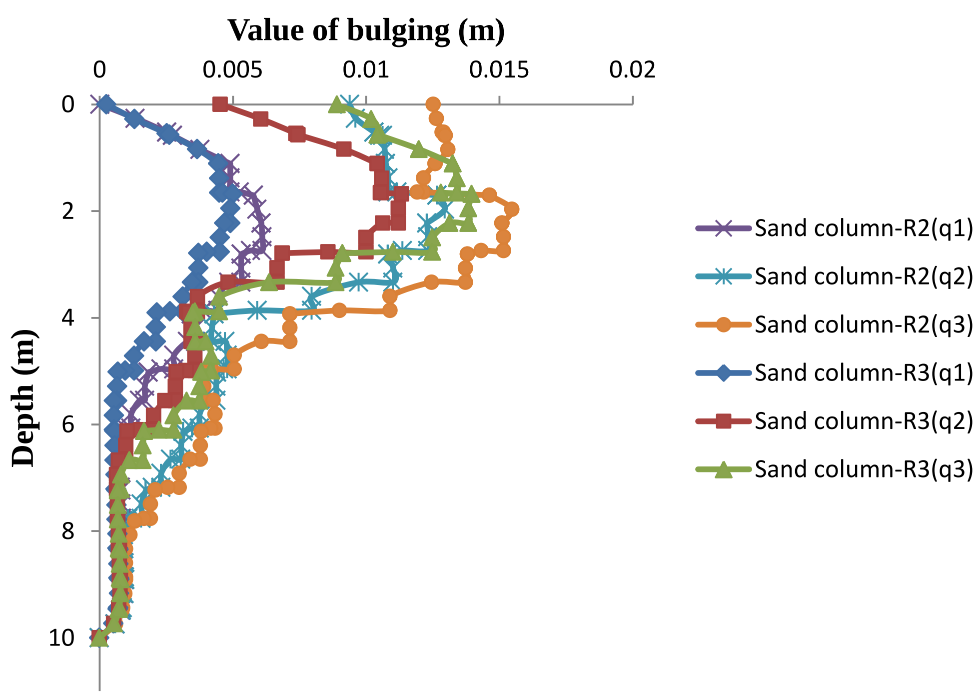


Figure 4. 7: The value of bulging of sand column with depth for the ratios of R2 & R3 and under various loading (q1, q2, q3)

At the specific rate of R ratio, the bulging behavior of stone column and sand column was analyzed. The value of bulging for stone column and sand column from the ground surface was gradually started to increase until the depth of two times of the diameter of stone and sand column. Below this depth, the value of bulging was gradually decreased and at the bottom of stone column became zero. As shown in Figure 4.8 and Figure 4.9, at the same R ratio and at the same loading the maximum value of bulging for stone column in comparison with sand column was smaller. On the other hand, the depth of the maximum value of bulging for stone column and sand column were close to each other indicating a relationship between the diameter of columns and the depth of the maximum value of bulging.

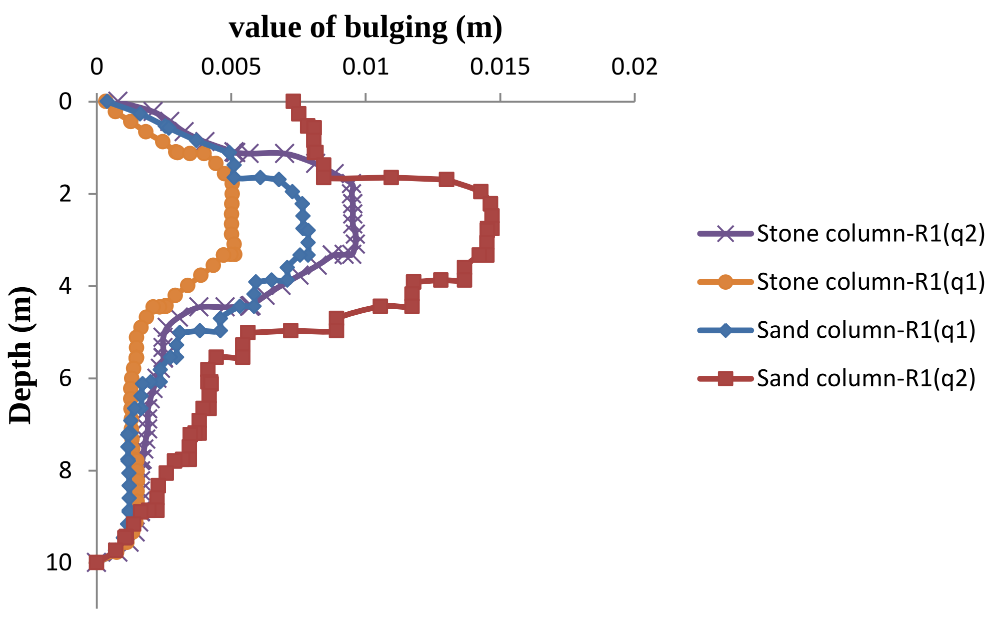


Figure 4. 8: Comparison of the maximum value of bulging for stone column and sand column with depth in the ratio of R1

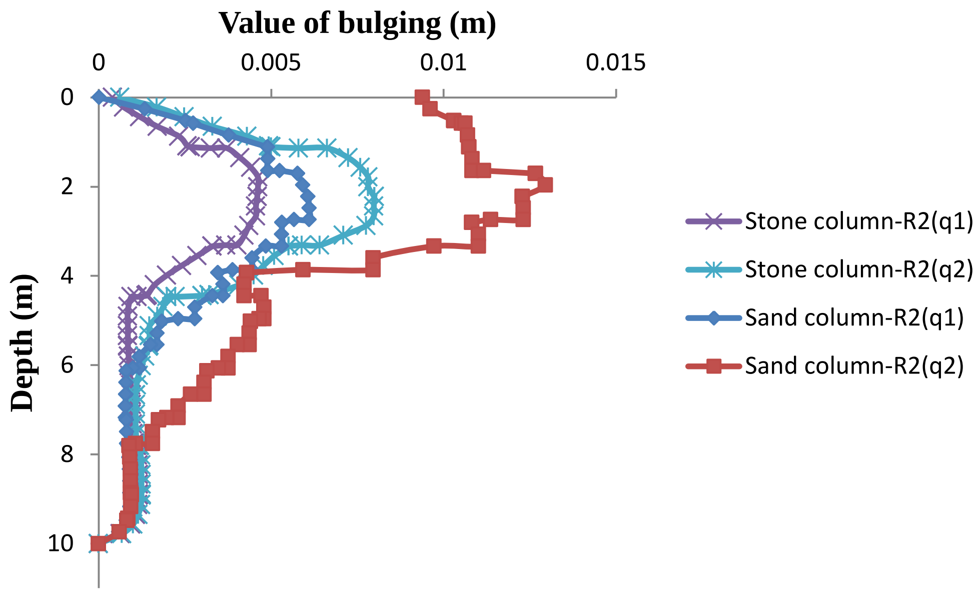


Figure 4. 9: Comparison of the maximum value of bulging for stone column and sand column with depth in the ratio of R2

The bulging behavior of stone column and sand column under static and dynamic loading was analyzed and the results were compared in Figure 4.10-4.12. As shown in Figure 4.10 and Figure 4.11, under static plus dynamic loading, the maximum value of bulging for stone column and sand column, decreased considerably compared with the static loading only. Combination of static and dynamic loading at the top of the stone column and sand column did not have much influence on the depth of maximum value of bulging as illustrated in Figure 4.12.

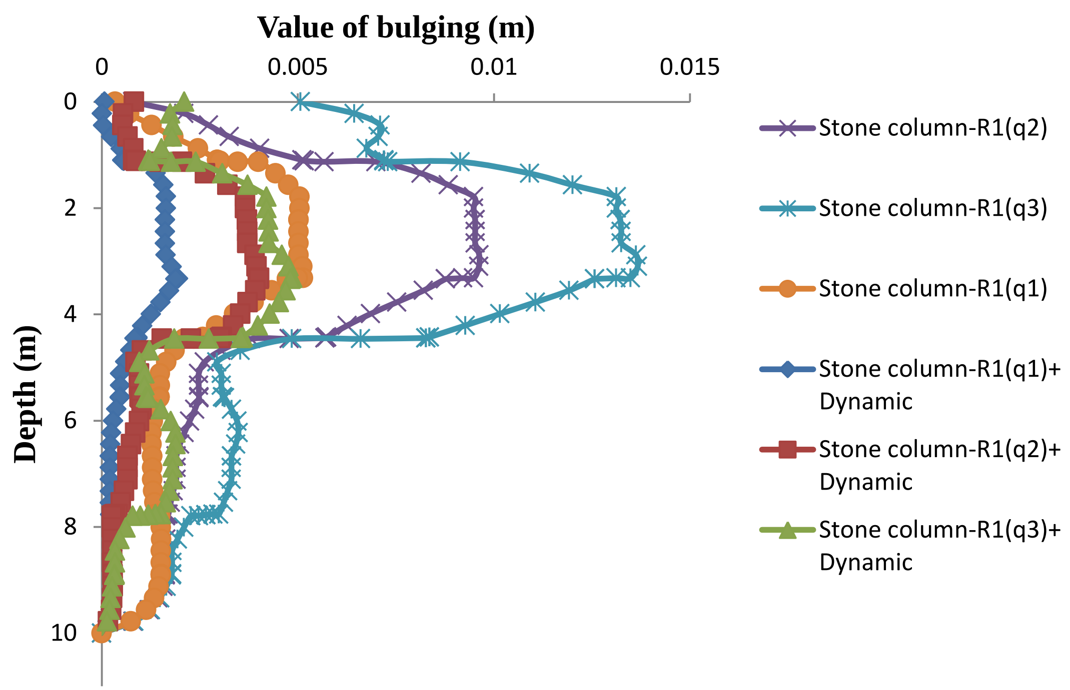


Figure 4. 10: Comparison of the maximum value of bulging for stone column under static and dynamic loading with depth in the ratio of R1

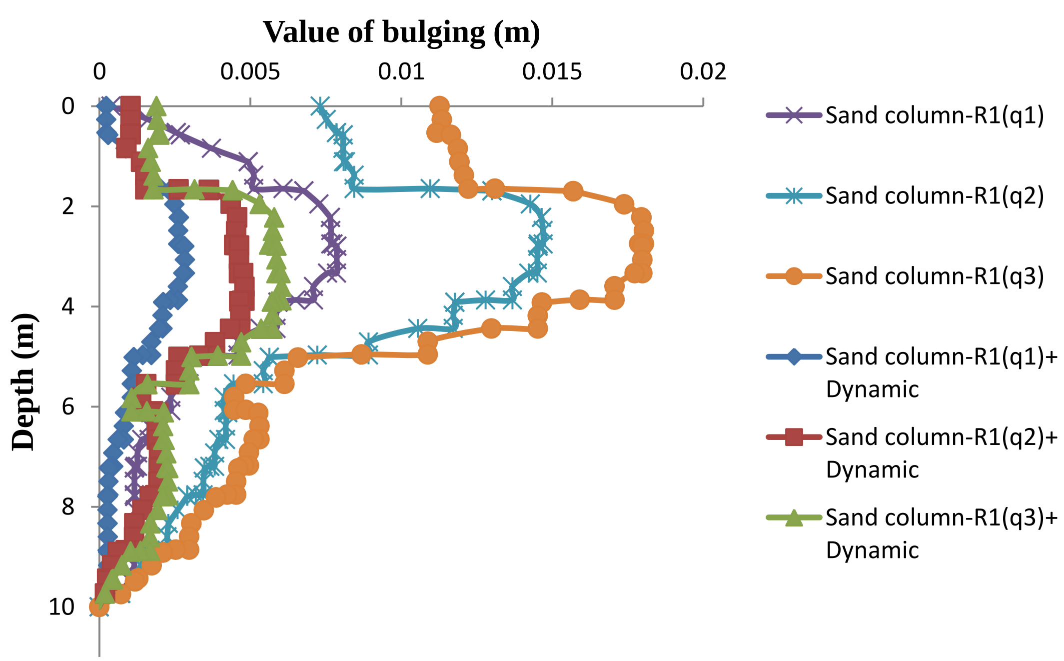


Figure 4. 11: Comparison of the maximum value of bulging for sand column under static and dynamic loading with depth in the ratio of R1

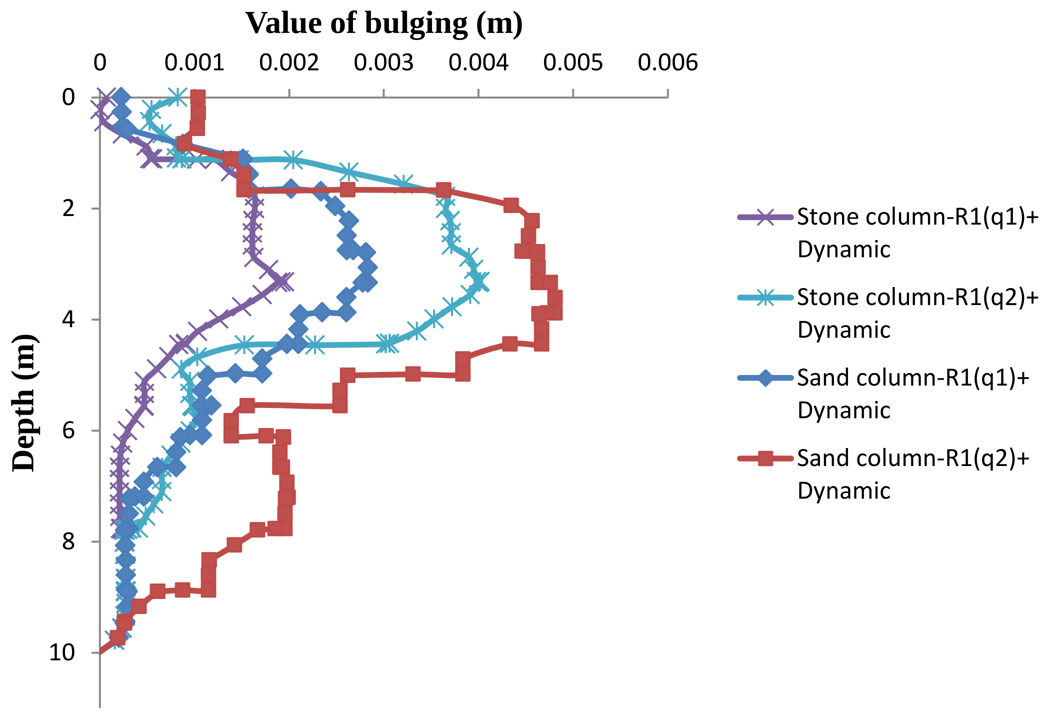


Figure 4. 12: Comparison of the maximum value of bulging between stone column and sand column under combination of static and dynamic loading with depth in the ratio of R1

The value of bulging for stone column and sand column from the ground surface was gradually started to increase until the depth of two times of the diameter of stone and sand column. Below this depth, the value of bulging gradually decreased and at the bottom of stone column became zero.

The value of vertical displacement of sand column and stone column under static loading and also combination of static plus dynamic loading were analyzed. At the same R ratio and the same amount of static loading, The value of vertical displacement of sand column in comparison with stone column was much greater in the upper half of the sand column, as shown in Figure 4.13. Figure 4.14 and Figure 4.15 indicated the results of finite element analysis for the value of vertical displacement of stone and sand column in existence and absence of dynamic loading. The figures indicate that, the dynamic loading affected the value of vertical displacement and caused a decrease in the displacement values of stone and sand columns. Under the static loading only, the value of vertical displacement in the stone and sand columns was greater than the vertical displacement of stone and sand columns in existence of dynamic loading. At the same R ratio and the same type of loading, the Figure 4.16 indicates the results of finite element analysis for stone column and sand column under static plus dynamic loading. Figure 4.16, clearly indicates that the value of the vertical displacement at the upper half of sand column is much greater than stone column.

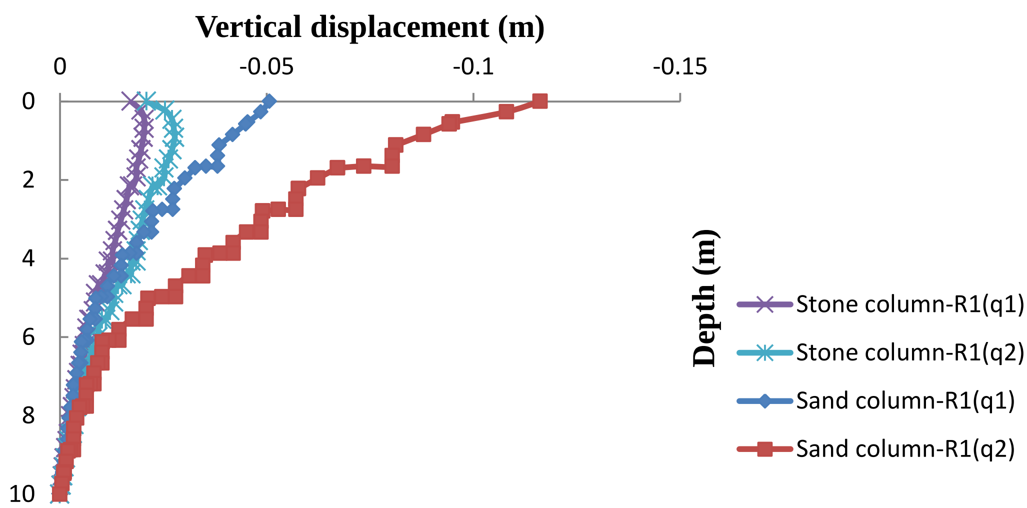


Figure 4. 13: Comparison of the value of vertical displacement with depth for stone column and sand column in the ratio of R1

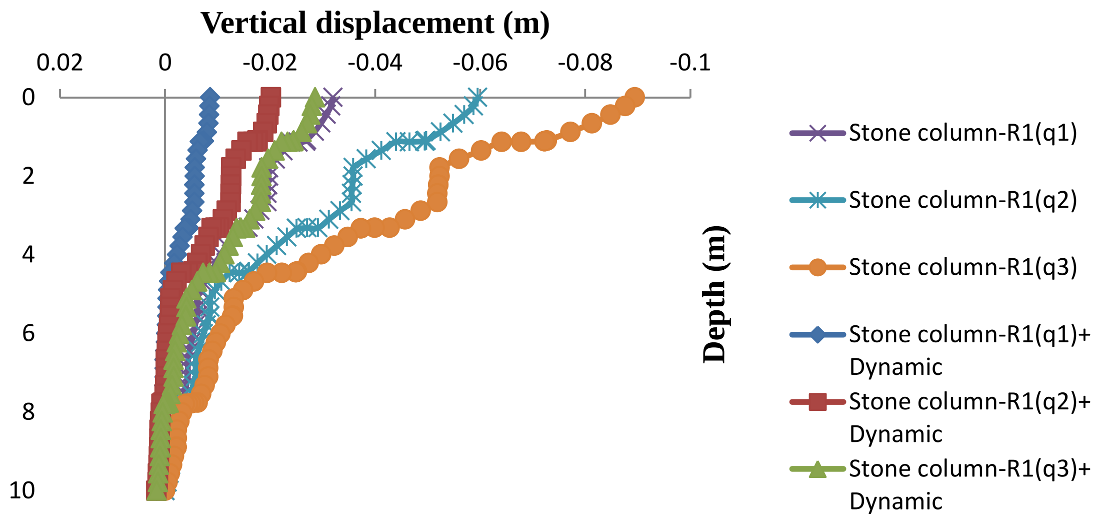


Figure 4. 14: Comparison of the value of vertical displacement with depth for stone column in the ratio of R1

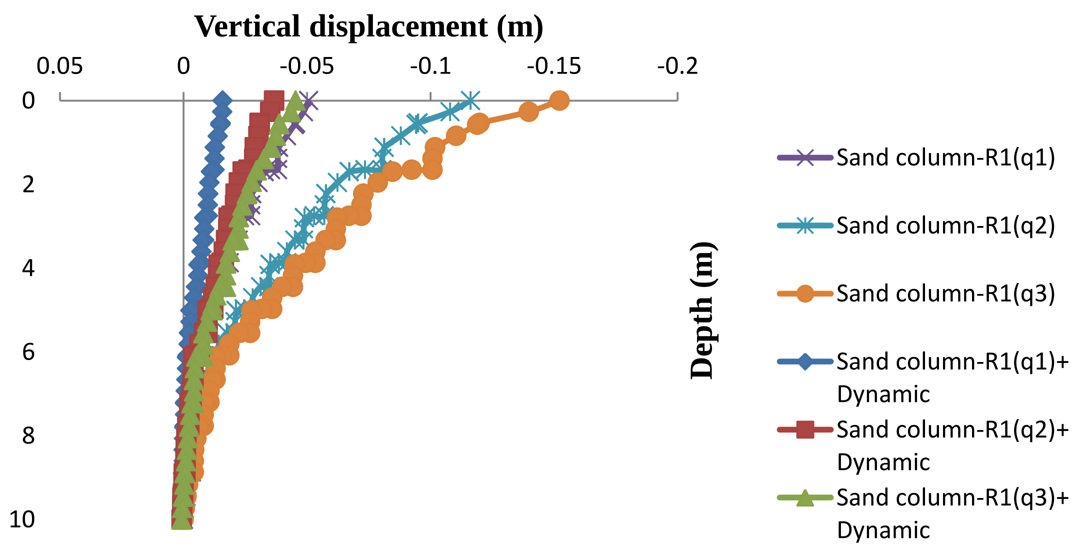


Figure 4. 15: Comparison of the value of vertical displacement with depth for sand column in the ratio of R1

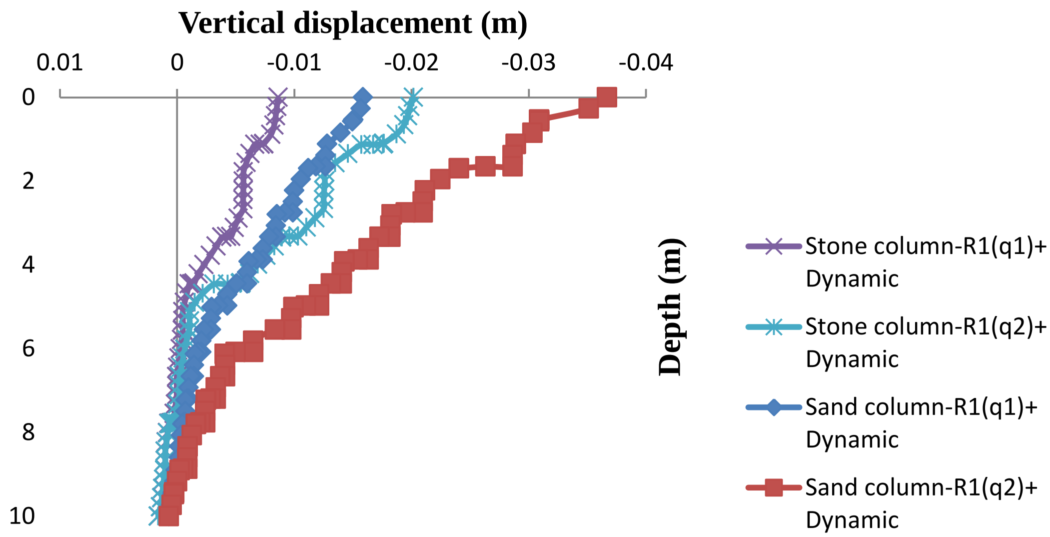


Figure 4. 16: Comparison of the value of vertical displacement with depth for sand column and stone column in the ratio of R1

Chapter 5

3-DIMENSIONAL ANLYSIS OF FULL SCALE GROUND IMPROVEMENT

5.1 Introduction

Clay deposits typically pose stability and settlement problems for building projects. In soft cohesive soils, the ground improvement by stone column reinforcement provides safe constructions with considering the economic efficiency. The in-situ conditions need soil reinforcement to restrain instability and settlements of constructions. Stone column reinforcement was selected as the cost effective and the most proper method to reinforce the soft cohesive soil for this project. The proposed modeling in the study encountered the ground which included two different layers of clay deposits.

Stone column was used in order to modify the soft cohesive soil and decrease the settlement and increase the ultimate load carrying capacity of the ground underneath constructions. In this chapter, the analysis of the ground reinforcement by stone column was carried out with the footing load on stone columns. The finite element analysis program, Plaxis 3D Foundation, was used to analyze the consolidation and settlement of the stone column reinforced soft compressible clay.

5.2 3-Dimensional modeling of stone column

The full scale modeling of ground improvement by stone column reinforcement with the distributed load on the stone columns is shown in Figure 5.1 FEM was used in the analysis. The stone columns were applied by square arrangement. The stone columns were continued through the first clay layer up to a depth of 10 m as shown in Figure 5.2. The clay layer was divided by geometry lines as shown in Figure 5.3 and used the refine cluster option to reach a dense mesh around stone columns so that more accurate results would be obtained.

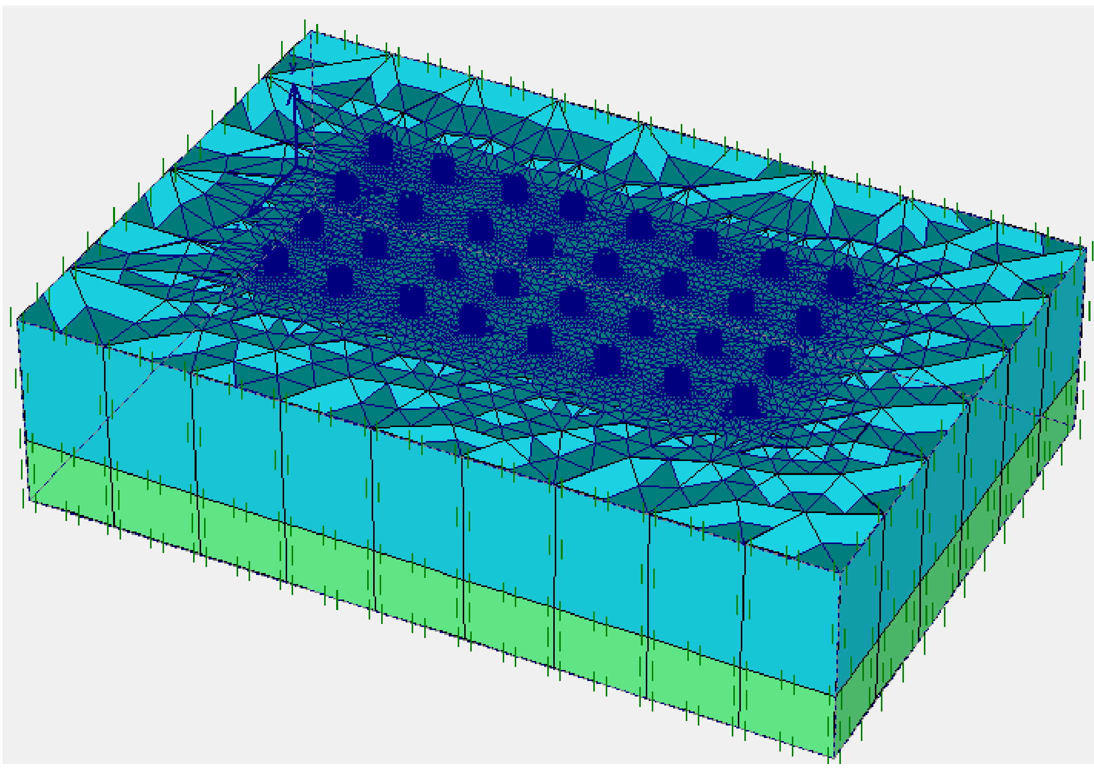


Figure 5. 1: The full scale ground with distributed loading on stone columns

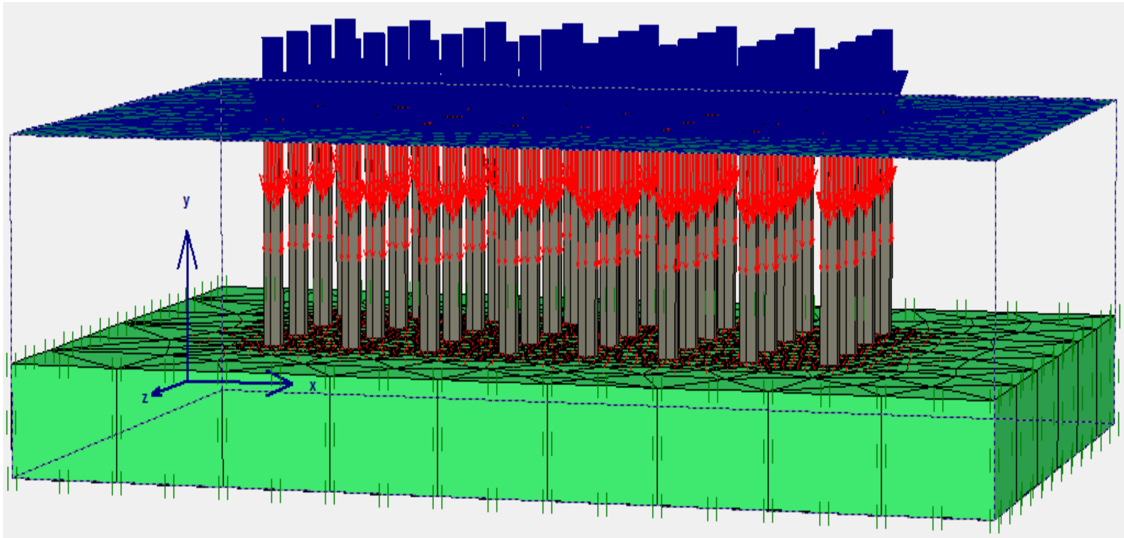


Figure 5. 2: Position of stone columns

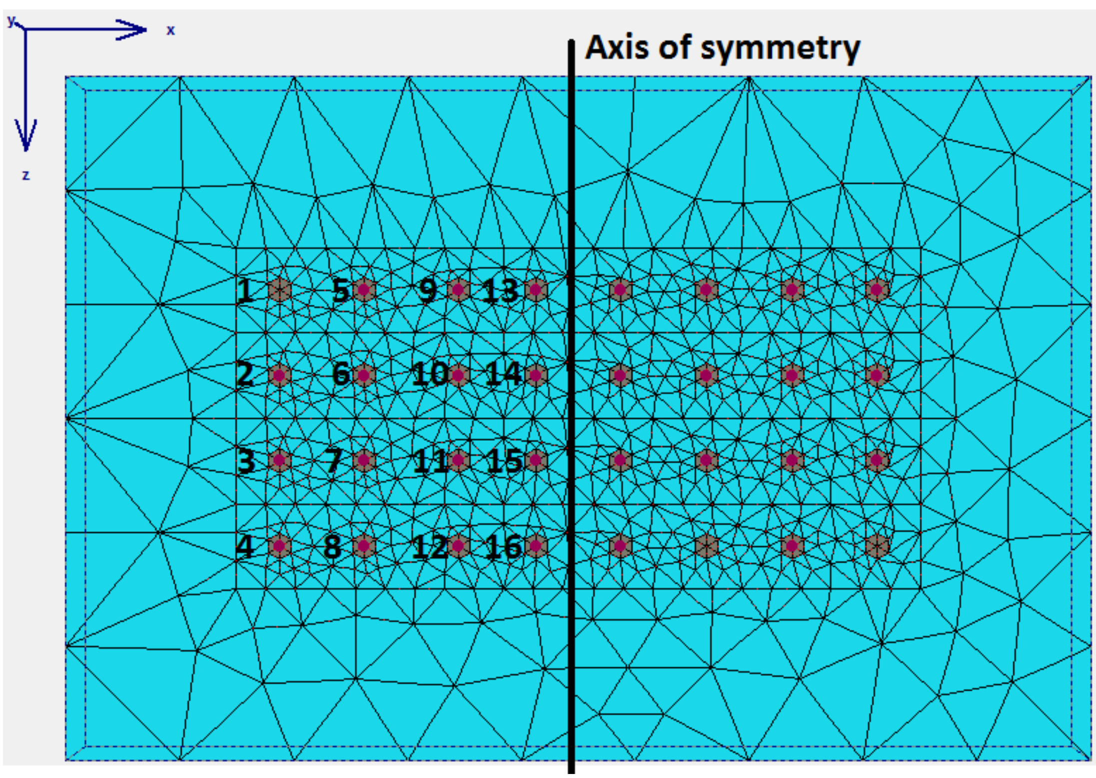


Figure 5. 3: The plan of dense mesh around stone columns

In the study, Plaxis 3D, finite element analysis of stone columns reinforced clay under distributed loading was studied for a 560-days period. Two different diameters under various loading in the same situation were chosen and the analysis was carried out. For consolidation analysis, coupled consolidation concept was assumed. The following assumptions in consolidation analysis were used:

- Strains are small,
- Stone columns are full-depth saturated,
- The soil thickness within the consolidation procedure is constant

The plane is symmetrical. During consolidation analysis, the loading applied is assumed to be uniform and the load is applied immediately at the top of stone column. During the consolidation analysis, the distributed load is assumed to remain constant. The soil model was defined as linear elastic model. The consolidation computation was continued until the minimum rate of pore water pressure was accomplished. The results of finite element analysis for treated clay by two different diameters of stone columns were compared. The properties of stone columns and the clays are given in Table 5.1. The geometry data of different diameters of stone columns are given in Table 5.2. The data of distributed static loading for stone columns is given in Table 5.3.

Table 5. 1: Properties of the stone columns and the clay layers

	Model	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_x (m/day)	k_y (m/day)	k_z (m/day)	E' (kPa)	ν'	K_{0x}	K_{0y}
Clay	Mohr-coulomb	Undrained	16	18	0.001	0.001	0.001	9500	0.32	0.59	0.59
Stiff clay	Mohr-coulomb	Undrained	18	20	0.001	0.001	0.001	40000	0.3	0.5	0.5
Stone column material	Mohr-coulomb	Drained	19	20	8	8	8	34000	0.3	0.7	0.7

Table 5. 2: The geometry data of different diameters of stone column

	Height (m)	de (m)	dc (m)	R=de/dc
Model 1	10	5	1.5	$R_1 = 3.33$
Model 2	10	5	1.2	$R_2 = 4.16$

Table 5. 3: The data of distributed static loading for stone columns

Distributed pressure (kPa)	Static	$q_1 = 130$
		$q_2 = 150$

5.3 Discussion of the results

Because of symmetric, the maximum bulging of stone columns at the left half of the plan in Figure 5.3 X and Z-direction with different ratio of R ($\frac{d_e}{d_c}$) and under various amount of loading (q_1, q_2) was analyzed. As shown in Tables 5.4-5.7, the maximum bulging of stone columns at the specific ratio of R occurred at the same depth. In the other word, the depth of bulging is related to the diameter of stone columns. Thus, by decreasing the diameter of stone columns, the depth of the maximum value of bulging decreased. Through keeping accurate observation on the maximum value of bulging in stone columns, the maximum value of bulging occurred at a depth of three times of the diameter of stone columns. The main factor influencing the value of maximum bulging is the amount of the axial load applied on the stone columns. The results of the analysis indicated that by increasing the axial load on top of the stone columns, the value of maximum bulging was increased. In addition, it was observed that the maximum value of bulging failure in X-direction in comparison with Z-direction is totally different. In X-direction, by getting closer to the axis of symmetry the value of maximum bulging decreased and conversely in Z-direction the maximum value of bulging increased.

In this study, the vertical settlement of each column at the left half of the plan was also measured. The maximum value of vertical settlement was obtained for those stone columns which were in the middle of the plane. That means by getting closer to the axis of symmetry, the maximum value of vertical settlement increased, showing that the maximum value of the consolidation settlement was obtained in stone column number 14 and 15.

The punching failure of each stone column was also measured. The results of the finite element analysis indicated that by increasing the amount of loading at specific R ratio the value of punching failure increased.

Table 5. 4: The results of FEM for Stone columns with loading= 150 kPa: dc=1.5 m and R1

<table>
<tr><th rowspan="2"></th><th rowspan="2">Stone column NO.</th><th rowspan="2">Settlement (m)</th><th rowspan="2">Punching failure(m)</th><th colspan="3">Max bulging failure</th></tr>
<tr><th>Value in X-direction(m)</th><th>Value in Z-direction(m)</th><th>Depth(m)</th></tr>
<tr><td rowspan="16">Group Stone column dc = 1.5m - Loading = 150 Kpa, R1= 3.33</td><td>1</td><td>1.06E-02</td><td>5.80E-03</td><td rowspan="4">1.34E-03</td><td rowspan="4">1.04E-03</td><td rowspan="4">4.5</td></tr>
<tr><td>2</td><td>1.15E-02</td><td>6.40E-03</td></tr>
<tr><td>3</td><td>1.15E-02</td><td>6.40E-03</td></tr>
<tr><td>4</td><td>1.06E-02</td><td>5.80E-03</td></tr>
<tr><td>5</td><td>1.16E-02</td><td>6.00E-03</td><td rowspan="4">7.38E-04</td><td rowspan="4">1.34E-03</td><td rowspan="4">4.5</td></tr>
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<tr><td>7</td><td>1.26E-02</td><td>6.60E-03</td></tr>
<tr><td>8</td><td>1.16E-02</td><td>6.00E-03</td></tr>
<tr><td>9</td><td>1.15E-02</td><td>5.70E-03</td><td rowspan="4">6.23E-04</td><td rowspan="4">1.42E-03</td><td rowspan="4">4.5</td></tr>
<tr><td>10</td><td>1.27E-02</td><td>6.50E-03</td></tr>
<tr><td>11</td><td>1.27E-02</td><td>6.50E-03</td></tr>
<tr><td>12</td><td>1.15E-02</td><td>5.70E-03</td></tr>
<tr><td>13</td><td>1.19E-02</td><td>5.70E-03</td><td rowspan="4">3.15E-04</td><td rowspan="4">1.53E-03</td><td rowspan="4">4.5</td></tr>
<tr><td>14</td><td>1.30E-02</td><td>6.40E-03</td></tr>
<tr><td>15</td><td>1.30E-02</td><td>6.40E-03</td></tr>
<tr><td>16</td><td>1.19E-02</td><td>5.70E-03</td></tr>
</table>

Table 5. 5: The results of FEM for Stone columns with loading= 130 kPa: dc=1.5 m and R1

	Stone column NO.	Settlement (m)	Punching failure(m)	Max bulging failure		
				Value in X-direction(m)	Value in Z-direction(m)	Depth(m)
Group Stone column dc = 1.5m - Loading = 130 Kpa, R1= 3.33	1	9.20E-03	5.00E-03	1.15E-03	9.00E-04	4.5
	2	1.00E-02	5.50E-03			
	3	1.00E-02	5.50E-03			
	4	9.20E-03	5.00E-03			
	5	9.90E-03	4.90E-03	6.21E-04	1.16E-03	4.5
	6	1.09E-02	5.60E-03			
	7	1.09E-02	5.60E-03			
	8	9.90E-03	4.90E-03			
	9	1.02E-02	5.00E-03	5.30E-04	1.22E-03	4.5
	10	1.12E-02	5.70E-03			
	11	1.12E-02	5.70E-03			
	12	1.02E-02	5.00E-03			
	13	1.05E-02	4.70E-03	2.65E-04	1.32E-03	4.5
	14	1.14E-02	5.40E-03			
	15	1.14E-02	5.40E-03			
	16	1.05E-02	4.70E-03			

Table 5. 6: The results of FEM for Stone columns with loading= 150 kPa: dc=1.2 m and R2

Group Stone column dc = 1.2m - Loading = 150 Kpa, R2 =4.16	Stone column NO.	Settlement (m)	Punching failure(m)	Max bulging failure		
				Value in X-direction(m)	value in Z-direction(m)	Depth(m)
	1	8.20E-03	5.00E-03	9.04E-04	6.81E-04	3.5
	2	8.80E-03	5.40E-03			
	3	8.80E-03	5.40E-03			
	4	8.20E-03	5.00E-03			
	5	8.70E-03	4.80E-03	5.01E-04	9.12E-04	3.5
	6	9.50E-03	5.40E-03			
	7	9.50E-03	5.40E-03			
	8	8.70E-03	4.80E-03			
	9	8.80E-03	4.80E-03	3.15E-04	9.56E-04	3.5
	10	9.54E-03	5.30E-03			
	11	9.54E-03	5.30E-03			
	12	8.80E-03	4.80E-03			
	13	8.90E-03	4.60E-03	2.25E-04	9.82E-04	3.5
	14	9.59E-03	5.10E-03			
	15	9.59E-03	5.10E-03			
	16	8.90E-03	4.60E-03			

Table 5. 7: The results of FEM for Stone columns with loading= 130 kPa: dc=1.2 m and R2

	Stone column NO.	Settlement (m)	Punching failure(m)	Max bulging failure		
				Value in X-direction(m)	Value in Z-direction(m)	Depth(m)
Group Stone column dc = 1.2m - Loading = 130 Kpa, R2=4.16	1	7.10E-03	4.30E-03	7.90E-04	5.94E-04	3.5
	2	7.60E-03	4.60E-03			
	3	7.60E-03	4.60E-03			
	4	7.10E-03	4.30E-03			
	5	7.60E-03	4.20E-03	4.37E-04	7.96E-04	3.5
	6	8.30E-03	4.70E-03			
	7	8.30E-03	4.70E-03			
	8	7.60E-03	4.20E-03			
	9	7.70E-03	4.20E-03	2.75E-04	8.35E-04	3.5
	10	8.32E-03	4.60E-03			
	11	8.32E-03	4.60E-03			
	12	7.70E-03	4.20E-03			
	13	7.80E-03	4.20E-03	1.95E-04	8.57E-04	3.5
	14	8.34E-04	4.60E-03			
	15	8.34E-04	4.60E-03			
	16	7.80E-03	4.20E-03			

The situation of ground surface and the surface of stiff clay for dc= 1.5m, R1 and distributed load= 150 (kPa) with considering the consolidation settlement for a period of 560-days are shown in Fgure 5.4 and Figure 5.5 respectively.

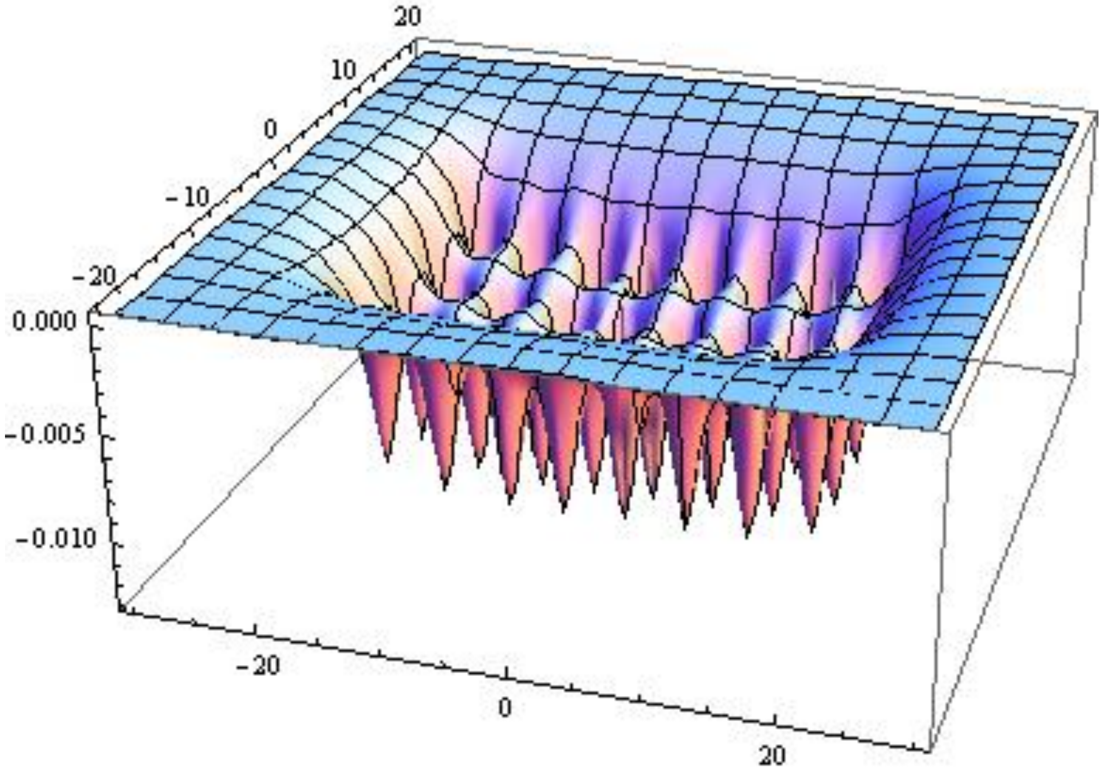

Figure 5. 4: The Situation of ground surface (dc=1.5 m, R1, loading= 150 kPa)

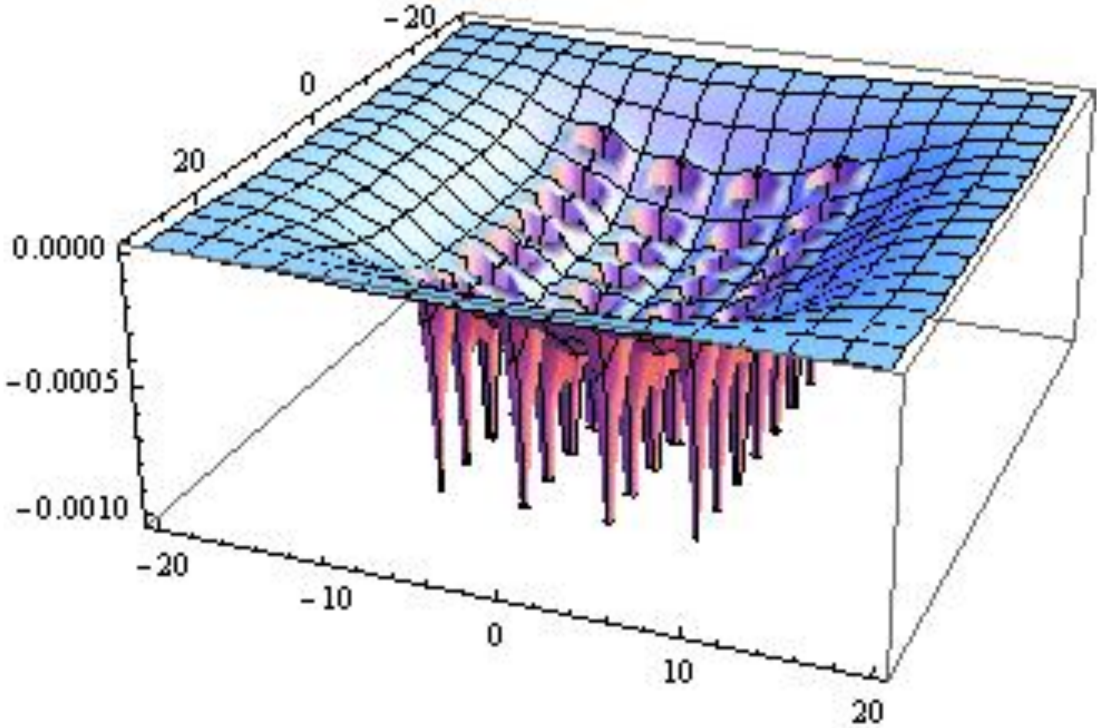

Figure 5. 5: The situation of stiff clay's surface (dc=1.5 m, R1, loading= 150 kPa)

Chapter 6

CONCLUSIONS AND RECOMMONDATIONS

6.1 Conclusions

In the present study, the matching plan for adaptation of axisymmetric stone column unit cell and sand column unit cell was recommended. The recommended matching plan was analyzed for applicability of linear elastic model of soft cohesive soil. The finite element analysis for three-dimensional full scale modeling was also studied. The following conclusions are obtained:

1. Owing to stone column reinforcement, the settlement behavior of clay was improved based on ratio of R ($\frac{\mathrm{de}}{\mathrm{dc}}$). By decreasing the ratio of R, with consideration of the full-depth of the stone column, the settlement of the soft clay was decreased.
2. Stone column plays two influential roles in the soft cohesive soil, a) as a part of soil, it improves the settlement behavior of the soft soil, b) stone column behaves like drain wells and accelerates the consolidation process.
3. The analysis of stone column with different ratio of R and the untreated clay indicated that with larger diameter of stone column application, the consolidation behavior of soil was improved and the consolidation process was accelerated.

Within the constant depth of the stone column, less time was needed for the average rate of consolidation of the soil to be achieved.

4. The results of the finite element analysis for the axisymmetric modeling of stone column unit cell and the untreated clay indicated that, the rate of dissipation of excess pore water pressure was accelerated with the larger diameter of the stone column. The results also indicated that the complete consolidation of the clay layer with larger stone column diameter was achieved at much shorter period of time than the untreated clay. The excess pore water pressure decreased with a reduction in the R ratio. Thus in the smaller ratio of R, the dissipation rate of excess pore water pressure is much quicker than the untreated clay.
5. The bulging of stone column and sand column at the specific ratio of R were occurred in the same depth. In other words, the depth of bulging is related to the diameter of stone and sand column. Thus, by decreasing the diameter of stone and sand column, the depth of the maximum value of bulging will be decreased.
6. The main influence of different amount of the axial load on the stone and sand column is the value of maximum bulging. By increasing the axial load at top of the stone and sand column the value of maximum bulging will be increased.
7. The value of bulging for stone column and sand column from the ground surface was gradually started to increase until the depth of two times of the diameter of stone and sand column. Below this depth, the value of bulging was gradually decreased and at the bottom of stone column became zero.
8. At the same R ratio and at the same loading, the maximum value of bulging for stone column in comparison with sand column was smaller. On the other hand, the depth of the maximum value of bulging for stone column and sand column

were close to each other indicating a relationship between the diameter of columns and the depth of the maximum value of bulging.

9. Under static plus dynamic loading, the maximum value of bulging for stone column and sand column, decreased considerably compared with the static loading only. Combination of static and dynamic loading at the top of the stone column and sand column did not have much influence on the depth of maximum value of bulging.
10. At the same R ratio and the same amount of static loading, the value of vertical displacement of sand column in comparison with stone column was much greater in the upper half of the sand column.
11. The results of finite element analysis for the value of vertical displacement of stone and sand column in existence and absence of dynamic loading indicated that, the dynamic loading affected the value of vertical displacement and caused a decrease in the displacement values of stone and sand columns. Under the static loading only, the value of vertical displacement in the stone and sand columns was greater than the vertical displacement of stone and sand columns in existence of dynamic loading.
12. At the same R ratio and the same type of loading, the results of finite element analysis for stone column and sand column under static plus dynamic loading indicated that the value of the vertical displacement at the upper half of sand column is much greater than stone column.
13. In three-dimensional analysis of full scale clay deposits reinforced by stone columns, the maximum value of bulging occurred at a depth of three times of the diameter of stone columns.

14. In three-dimensional analysis of full scale clay deposits indicated that the maximum value of bulging failure in X-direction in comparison with Z-direction is totally different. In X-direction, by getting closer to the axis of symmetry the value of maximum bulging decreased and conversely in Z-direction the maximum value of bulging increased.
15. The maximum value of vertical settlement was obtained for those stone columns which were in the middle of the plane. Therefore, by getting closer to the axis of symmetry, the maximum value of vertical settlement increased.

6.2 Recommendations

1. Due to the non-linear behavior of soil, the performance of stone column by different types of non-linear options of soil in Plaxis 2D can be analyzed.
2. The performance of stone column encased with different types of geogrid in full depth and half depth of coating can be analyzed.
3. The full scale analysis of stone column performance in three-dimensional and two-dimensional analyses for comparison of the results within the consolidation process can be studied.

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