الجمهورية الجزائرية الديمقراطية الشعبية
People's Democratic Republic of Algeria
وزارة التعليم العالي والبحث العلمي
Ministry of Higher Education and Scientific Research

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كـلية العلـــوم و التكـــنولوجـيا
قسم: الهندسة المدنية و الري
المرجع : ....

Thesis presented with a view to obtaining

LMD Doctorate in Civil Engineering

Option: Numerical modeling in civil engineering

Contribution à l'analyse numérique des remblais sur sols mous

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Presented by:

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Publicly supported on June 20, 2021

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To my dear parents and to all my family

ACKNOWLEDGEMENTS

First of all, I would like to thank Allah owner of many graces for enabling me to execute this research and complete this work.

I would like to thank my supervisor Dr. REMADNA Mohamed Saddek from the University of Biskra for his guidance and support throughout the preparation of this thesis. His availability, his experience, and his rigor allowed this thesis to succeed. It was also a great pleasure for me to work under his direction,

I would also like to thank Dr. BENMEBAREK Sadok, Professor at the University of Biskra for having done me a great honor by accepting to chair the jury of this thesis. My thanks also go to Dr. DEMAGH Rafik (Professor at the University of Batna 2) and Dr. BAHLOUL Ouassila (Lecturer at the University of Batna 2), first of all for doing me honor by agreeing to participate in the jury of this thesis, and also for the time and the interest that they brought to this research by agreeing to examine this work.

I thank also my co-supervisor Dr. Ahmad Safuan A Rashid (Associate Professor) Universiti Teknologi Malaysia, Johor Bahru 81310, Malaysia.

All my thanks and gratitude to all the teachers and doctoral students of the MN2I2S laboratory at the University of Biskra for their encouragement. I wish to express my sincere thanks to the Algerian Ministry of Higher Education and Scientific Research for funding his PhD program.

I thank my family very much for their support during all these years of preparing this letter, my mother and brothers. I also extend my sincere thanks to my friends for their moral encouragement and support.

Abstract

The present research work is concerned with the contribution to the numerical analysis of embankments on soft ground called Sabkha. Vast expanses of arid, saline soils (sabkha) that occur Middle East and North Africa and elsewhere possess a very low density and strength that necessitate improvement before any actual construction takes place. This soil is not only soft and very humid during the flooding seasons but also has frequent small areas of very soft soil which is here called locally weak zones (LWZ). LWZ is characterized by low strength and high compressibility. This work presents the results of two-dimensional axisymmetric numerical analyze that were carried out using PLAXIS 2D 2017, for the modeling of an embankment supported by stone columns on sabkha soil. The study focuses on the evaluation of the maximum bulging of the stone column and on the settlement of the embankment. It has been demonstrated that ordinary stone columns (OSC) were ineffective due to excessive bulging (221.16 mm) caused by the lack of lateral pressure. On the other hand, the encased stone columns (ESC) showed good behavior, namely a much reduced bulging (42.09 mm) and a reasonable settlement (0.962 m vs. 1.560 m for an OSC) so that it is possible to build safe very high embankments. The numerical analysis also shows that the length of the encasement should just be greater than the depth of the LWZ. Besides, an extensive parametric study was conducted to investigate the effects of the variations of embankment height, stiffness of geosynthetic, the depth of the locally weak zone, area replacement ratio (ARR), and the stone column friction angle, on the performance of the (ESC) - embankment composite in (LWZ).

Secondly, 2D numerical simulations were performed to investigate the effects the reinforced stone columns with external reinforcement and internal reinforcement called as vertical encasement and horizontal strips (VESC+HRSC) which are one of the best improvement methods of locally weak zones (LWZ), especially to increase the stability of high embankments, namely a much reduced bulging and a reasonable settlement, (Indeed, numerical results showed for a (VESC+HRSC) combination, a vertical settlement of 0.74 m and a lateral deformation of 20.02 mm vs. 1.56 m and 221.16 mm for an OSC). Besides, an extensive parametric study is conducted to investigate the effect of the spacing of the horizontal reinforcing strips and of the column reinforced length. The influence of stone column diameter, depth of locally weak zone, and the effective stiffness of the geosynthetic, on the performance of the (RSC) - embankment composite are also investigated.

Keywords: Reinforced-Stone Columns; Geosynthetic; Finite Element Modelling; Locally Weak Zone; Sabkha Soil.

Résumé

Le présent travail de recherche s'intéresse à la contribution à l'analyse numérique des remblais sur sol mou appelé Sabkha. De vastes étendues de sols arides et salins (sabkha) qui se trouvent au Moyen-Orient et en Afrique du Nord et ailleurs possèdent une densité et une résistance très faibles qui nécessitent une amélioration avant toute construction réelle. Ce sol est non seulement mou et très humide pendant les saisons de crue mais comporte également de fréquentes petites zones de sol très mou que l'on appelle ici zones localement faibles (LWZ). LWZ se caractérise par une faible résistance et une haute compressibilité. Ce présent travail présente les résultats d'une analyse numérique axisymétrique bidimensionnelle réalisée à l'aide de PLAXIS 2D 2017, pour la modélisation d'un remblai soutenu par des colonnes ballastées sur un sol de sabkha. L'étude porte sur l'évaluation du renflement maximal de la colonne ballastée et sur le tassement du remblai.

Il a été démontré que les colonnes ballastées ordinaires (OSC) étaient inefficaces en raison d'un renflement excessif (221,16 mm) causé par le manque de pression latérale. Par contre, les Colonnes ballastées enveloppées (ESC) ont montré un bon comportement, à savoir un renflement très réduit (42,09 mm) et un tassement raisonnable (0,962 m contre 1,560 m pour un (OSC) de sorte qu'il est possible de construire des remblais très hauts et sûrs. L'analyse numérique montre également que la longueur de l'enrobage doit être juste supérieure à la profondeur de la (LWZ). En outre, une étude paramétrique approfondie a été menée pour étudier les effets des variations de la hauteur du remblai, de la rigidité du géosynthétique, de la profondeur de la zone localement faible, du taux de remplacement de la zone et de l'angle de frottement de la colonne ballastée, sur les performances de la (ESC) - remblai composite en (LWZ).

Deuxièmement, des simulations numériques 2D ont été effectuées pour étudier les effets que les colonnes ballastées renforcées avec armature externe et armature interne appelées enrobage vertical et bandes horizontales qui sont l'une des meilleures méthodes d'amélioration des zones localement faibles (LWZ), en particulier pour augmenter la stabilité du remblai sur l'autoroute, à savoir un renflement très réduit et un tassement raisonnable, (En effet, les résultats numériques ont montré pour une combinaison (VESC +HRSC), un tassement vertical de 0,74 m et une déformation latérale de 20,02 mm vs 1,56 m et 221,16 mm pour un OSC). En outre, une étude paramétrique approfondie est menée pour étudier l'effet de l'espacement des bandes de renforcement horizontales et de la longueur de la colonne renforcée. L'influence du diamètre de la colonne, de la profondeur de la zone localement faible et de la rigidité effective du géosynthétique, sur la performance du composite (RSC) - remblai sont également étudiées.

Mots-clés : colonnes ballastées renforcées; géosynthétique ; Modélisation par éléments finis ; Zone de faiblesse locale ; Sol de Sabkha.

الملخص

يهتم العمل البحثي الحالي بالمساهمة في التحليل العددي للسدود على أرضية ناعمة تسمى السبخة. مساحات شاسعة من التربة القاحلة والمالحة (السبخة) التي تحدث في منطقة الشرق الأوسط وشمال إفريقيا وأماكن أخرى تمتلك كثافة وقوة منخفضة جدًا تتطلب التحسين قبل حدوث أي بناء فعلي. هذه التربة ليست ناعمة ورطبة جدًا خلال مواسم الفيضانات فحسب، بل تحتوي أيضًا على مساحات صغيرة متكررة من التربة الرخوة جدًا والتي تسمى هنا المناطق الضعيفة محليًا. تتميز هذه المناطق بقوة منخفضة وانضغاطية عالية. يعرض البحث نتائج التحليل العددي ثنائي الأبعاد المحوري الذي تم إجراؤه باستخدام برنامج البلاكسيس 2017 لنمذجة جسر مدعوم بأعمدة حجرية على تربة السبخة. تركز الدراسة على تقييم الانتفاخ الأقصى للعمود الحجري وهبوط الردم.

ثبت أن الأعمدة الحجرية العادية كانت غير فعالة بسبب الانتفاخ المفرط (221.16 ملم) الناجم عن نقص الضغط الجانبي. من ناحية أخرى، أظهرت الأعمدة الحجرية المغلفة سلوكًا جيدًا، أي انتفاخ أقل بكثير (42.09 مم) وهبوط معقولاً (0.962 م مقابل 1.560 م لـلأعمدة الحجرية العادية بحيث يمكن بناء ردميات آمنة ومرتفعة جدًا. يوضح التحليل العددي أيضًا أن طول الغلاف يجب أن يكون أكبر من عمق المنطقة الضعيفة محليا. بالإضافة إلى ذلك، تم إجراء دراسة بارامترية موسعة لاستقصاء آثار التغيرات في ارتفاع الردم، وصلابة الجيوسنتيتيك، وعمق المنطقة الضعيفة محليًا، ونسبة استبدال المنطقة وزاوية احتكاك العمود الحجري، على أداء الاعمدة الحجرية المغلفة مركبة مع الردم في وجود المنطقة الضعيفة محليا.

ثانيًا، تم إجراء عمليات محاكاة عددية ثنائية الأبعاد للتحقق من تأثيرات الأعمدة الحجرية المقواة مع التعزيز الخارجي والتعزيز الداخلي المسماة بالتغليف العمودي والشرائط الأفقية وهي واحدة من أفضل طرق التحسين للمناطق الضعيفة محليًا، خاصةً زيادة ثبات الردم على الطريق السريع، وهو انتفاخ أقل بكثير وتسوية معقولة، (في الواقع، أظهرت النتائج العددية لتركيبة التعزيز الخارجي والتعزيز الداخلي للعمود، تسوية رأسية تبلغ 0.74 متر وتشوه جانبي 20.02 ملم مقابل 1.56 م و221.16 مم لـلأعمدة الحجرية العادية). إلى جانب ذلك، تم إجراء دراسة موسعة للتحقق من تأثير التباعد بين شرائح التسليح الأفقية وطول العمود المقوى. تأثير قطر العمود الحجري وعمق المنطقة الضعيفة محليًا، وصلابة الجيوسنتيتيك، على أداء الاعمدة الحجرية المغلفة مركبة مع الردم في وجود المنطقة الضعيفة محليا.

كلمات مفتاحية: أعمدة من الحجر المقوى؛ جيوسينثيتيك. نمذجة العناصر المحدودة؛ منطقة ضعيفة محليا؛ تربة السبخة.

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LIST OF ABBREVIATIONS

LWZ**:**	Locally weak zone
OSC**:**	Ordinary stone columns
ESC**:**	Encased stone columns
VESC+HRSC**:**	Vertical encasement stone column and horizontal layers horizontally reinforced stone column
RSC**:**	stone column
VESC**:**	Vertical encasement stone column
HRSC**:**	horizontal layers horizontally reinforced stone column
LL**:**	Liquid limit
PL**:**	Plastic limit
PI**:**	Plasticity Index
Cc**:**	Indicators of Compressibility
Cs**:**	Indicators
OCR**:**	Over consolidation ratios
ρmax**:**	Maximum dry density
Wopt**:**	optimum moisture content
FEM**:**	Finite element methods
PVC**:**	Polyvinyl chloride
PM **:**	Polyamides
PS**:**	Polyesters
PE**:**	polyethylene
PP**:**	polypropylene
SC**:**	Stone column
de**:**	Diameter of the unit cell
s**:**	Distance between adjacent columns
cg**:**	Constant coefficient related to columns arrangement

ARR:	Area replacement ratio
cu:	undrained shear strength
NS:	number of the reinforcement strips
Sv:	spacing between layers
Φ:	The angle of shearing resistance of the stone column
SSC:	single sand columns
HL:	Horizontal layers
n:	Numbers of geogrid
s:	geogrid spacing
USC:	unreinforced stone column
CSE:	Columns supported embankments
J:	Geotextile tensile stiffness
Hc:	height of sand column
Hs:	height of specimen
GEC:	Geosynthetic encased column
GESC:	Geosynthetic encased stone column
RAP:	Without Reinforced
RCDW:	Recycled construction and demolition waste,
n*:	Stress concentration ratio
Es:	Oedometric modulus of soft soil
qs:	Average vertical stress acting on the soft soil
E*si:	change in oedometer modulus of the soft soil estimated in i-th layer
M si:	oedometer modulus of deformation of the soft soil estimated in i-th layer
K0s:	Coefficient of lateral earth pressure at rest for soft soil
Ms:	Oedometer modulus of deformation of the soft soil
hci:	Incremental horizontal stress in i-th layer; and
vs:	Poisson"softhes of clay
e:	Equivalent wall with thickness
r:	Radius of granular column
yn,0:	Unit weight of embankment fill
H:	Height of embankment
d:	Thickness of platform below the embankment
B:	columns center to center spacing
ɤ.1:	Submerged unit weight of granular column; and
Q1:	Friction angle of granular column.
SSM:	Soft Soil Model
Hemb:	high of embankment
re:	The radius of the influence area of the column
rc:	The radius of the column within the unit cell
DEP:	Sabkha layer thickness
B:	The locally weak zone width

HS:	Hardening Soil
M-C:	Mohr–Coulomb model
ɤ sat:	Soil unit weight below phreatic level
ψ:	Dilation angle
c:	Effective cohesion
m:	The slope of the critical state line
E50 ref:	Secant stiffness in standard drained triaxial test
Eoed ref:	Tangent stiffness for primary oedometer loading
Eurref:	Unloading/ reloading stiffness power for stress-level dependency of stiff-ness
AM:	Analytical methods
PEA:	Pulko et al
R&K:	Raithel & Kempfert
Z&Z:	Zhang & Zhao
zb:	The bulging depth
β:	The settlement ratio
SESC:	The settlement of encased stone column
SOSC:	The settlement of ordinary stone column
W:	The locally weak zone width
Δdc:	The lateral deformation to column
I.F.V:	Improvement factor value
Vosc:	The value without reinforcement
Vwr:	The value with reinforcement
Sv:	The horizontal reinforcing strips
Y:	Column reinforced length
D:	the stone column diameter
SFEL:	The settlement value of a stone column at the end of the loading stage
SFC:	The final settlement value of a stone column at the end of the consolidation
Δs:	The variation in the change the settlement
S.I.F:	settlement improvement factor
RSC:	Reinforced stone column
SFOSC:	The settlement value of ordinary stone column
SFRSC:	The settlement value of reinforced stone column

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General introduction

General introduction

This study is concerned with the contribution to the numerical analysis of embankments on soft ground. Interest is particularly focused in this study, on the soil called sabkha. It will be discussed in detail the improvement of the soil by reinforced stone columns. Sabkha soil is not only soft and very humid during the flooding seasons but also has frequent small areas of very soft soil which is here called locally weak zones (LWZ). LWZ is characterized by low strength and high compressibility (the coefficient of compressibility averages a value of 6). Sabkha is a salty flat soil that can be found all over the world and is especially prevalent in hot and arid countries. It can be encountered in coastal areas. Sabkha soil is known by different names. Embankment construction on these problematic soils is always a challenge for geotechnical engineers. However, there are several soil improvement techniques to overcome these challenges. Stone columns are one of the popular techniques for supporting embankments. Structures such as dams, road embankments and storage tanks, frequently have many problems with irregular, excessive settlements or overall stability due to geological situation and weak soil. Stone columns (likewise known as granular piles) are increasingly used as soft soil reinforcement to support a variety of structures, in other words, these are soft soil improvement techniques which are commonly and successfully used to reduce settlement, reduce the liquefaction potential, and to speed up the consolidation of soft soils. When the stone columns (OSCs) are installed in extremely soft soils ($c_u < 15$ kPa) such as peat soils, and marine clays, etc., the lateral confinement presented by the surrounding soil may not be sufficient to form the stone column. This may lead to the excessive bulging of stone columns, especially in the upper portion of the columns, which can significantly reduce their capacity due to low bearing capacity and high compressibility.

Since the performance of ordinary stone columns is highly dependent on the lateral confinement provided by surrounding soil, when it comes to very soft soils ($c_u < 15$ kPa) the application of this solution may not be feasible, different techniques have been proposed to reinforce the performance of ordinary stone columns (OSCs). Aboshi et al. (1979) reinforced the top portion of the column with a steel skirt. Rao and Bhandari (1980) used concrete plugs to prevent lateral bulging of the stone columns. As Juran and Riccobono (1991) suggested mixing the granular material that is placed at the top of each column with cement.

Horizontal layers of geogrid in the top portion of the column were adopted by Sharma et al. (2004) Debbabi et al (2021). In addition, Murtaza and Samadhiya (2016) reinforced the stone columns with horizontal geogrid strips. The deep mixing method was used by Rashid et al. (2017).

Another method that can be used to provide the required lateral confining pressure to increase the bearing capacity of granular columns is to encase the column with a suitable geosynthetic. The columns may be encased with geosynthetics which are the main materials used to increase the strength and stability of geotechnical structures. The idea of encased stone columns was first proposed by Van Impe (1986). This technique has been successfully used in different projects Kempfert et al (2002), Montez and Brasil (2008). The main advantage of geosynthetic encased stone columns (ESCs) over ordinary stone columns (OSCs) is the higher stiffness resulting from the hoop force in the geosynthetic, which ascent the load capacity. In addition, the encasement prevents the lateral intermix of the granular material with the surrounding soft soil and thus does not influence the drainage capacity of the stone columns.

It must be said that, numerous researches carried out during the last two decades, have examined and characterized the behavior of sabkha soils. Different approaches have been proposed to stabilize sabkha soils, in particular the use of chemical and mechanical processes Akili et al. (1981) Abduljauwad and Al-Amoudi (1995). However, no research till now has taken into account the reinforcement of the soil of Sabkha by geosynthetic-reinforced stone columns. In this context, this research work is presented as a contribution in the form of a numerical modeling that studies the behavior of road embankments built on sabkha soil reinforced by encased stone columns and we were lucky to have in hand a recent study from Benmebarek et al. (2015) which provided us with the real geotechnical data for (LWZ) of the sabkha site of Chott El Hodna (Algeria). Hence, the advantage of having relatively reliable soil data and of testing the sensitivity of the targeted results only by varying the properties of the encased stone column.

Thesis organization: This study includes the following chapters:

The first chapter: is a bibliographic summary on the generality of compressible soil. General information on compressible soils is presented in this chapter and in particular the sabkha soils: Categories of soft soils and their technical characteristics, types of Sabkha; Characteristics of sabkha soils; Problems related to sabkha soil; Foundation problems in the soil of sabkha; Problems posed by the construction of embankments on sabkha soils.

The second chapter: In this chapter, it is presented the soil treatment techniques most used at present, in particular the method of soil reinforcement by geosyhnetic layers and the method of soil reinforcement by the encased stone columns. A bibliographical synthesis on previous studies and historical cases are briefly presented in this chapter.

The third chapter: represents our first contribution in this thesis, namely the numerical investigation of the improvement of the embankment response through the use of encased stone column on locally weak zone (sabkha soils). An intense parametric study is carried out to determine the sensitivity of the targeted results (i.e. lateral deformation of the column and vertical settlement) with regard to the variation of the principal parameters, namely, the height of the embankment, the rigidity of the geosynthetic, the length of the envelope, the area replacement ratio (ARR), the thickness of the Sabkha layer, and the angle of friction of the granular material constituting the stone column. All the results are discussed as the study is progressing.

The fourth chapter: represents our second contribution in this thesis, namely the numerical investigation of the improvement that is realized in the response of comparison of (VESC), (HRSC), and (VESC + HRSC), an (OSC) is first investigated. Then, the effect of the spacing of the horizontal reinforcing strips, the effect of the column reinforced length, the influence of stone column diameter, depth of locally weak zone, and the stiffness of the geosynthetic are investigated as well in this study.

A list of reference is given at the end of the thesis.

Background and Problem Statement

Field explorations in Chott el Hodna in Algeria have shown that locally weak zones are mostly circular in shape Benmebarek et al. (2015) (see Figure 1,2). The nature of sabkha soils both chemically and physically causes some problems during construction with stone columns. These problems were identified by several researchers Akili et al. (1981) Abduljauwad and Al-Amoudi (1995). The most common problems are the compressibility of sabkha soil which varies from one point to another and can lead to large differential settlement. Sabkha deposit can withstand high pressures in dry conditions but when wet it exhibits high deformation and low shear strength and hence poses great challenges to the engineers. Khan and Hasnain (1981), as massive reported severe damage to a large number of buildings and roads constructed on sabkha soils in Libya, and Saudi Arabia. In this study, interesting solutions are suggested to solve the problems of Sabkha soil (LWZ), especially during the rainy seasons.

Figure 1. Example of the locally weak zone: Chott El Hodna in Algeria (Benmebarek et al. 2015),

First Part

First Part: Bibliographic Study. Summary of Embankments over Soft Ground

Chapter 1

General information on compressible soils

1.1. Introduction

Soft soil deposits typically show excessive settlement characteristics and have a low bearing capacity. Weak soils are fairly extensive all over the world and the places of them are in important cities. There are two main problems encountered with civil constructions in weak soil deposits, excessive settlement and low shear strength. In this chapter, we present the geological and geotechnical nature of compressible soils.

Compressible soils are characterized by:

- a mostly clayey nature with more or less organic matter less important but rarely negligible;
- a very high water content, and a low apparent specific weight (these soils are very usually saturated);
- very low shear strength;
- high compressibility resulting, even under low load, in amplitudes of significant settlement, the rate of settlement decreasing with time, but not canceling out a few years.

These soils, generally of recent formation (a few thousand years) contain almost always, in greater or lesser proportion, of organic matters, they can be divided into three categories: peat; vases and soft clays; sabkha soils.

1.2. Soft soil

1.2.1. Peat

Peat is a very compressible organic natural deposit, with high contents of matter organic, with very high water contents and very high degrees of saturation, the content of which decomposed vegetable fibers constitutes an anisotropic structure which influences the mechanical resistances.

The settlement of peat generally does not follow the classic laws of the consolidation of clays:

- the preconsolidation pressure is generally difficult to determine, although it is likely normally consolidated soil;
- the consolidation phase is generally very short and difficult to define; secondary compression is often predominant.

The compression indices determined by the oedometer are very strong (greater than 1). The permeability generally has a much stronger horizontal component than the vertical component. This permeability decreases notably during the compaction.

1.2.2. Vases and soft clays

From a geological point of view, the vases are deposits formed in fresh or salt water, made up of generally very fine grains (less than 200 μ with a large percentage of particles less than 2 μ) of variable mineralogical nature, arranged in flakes (structure called "nests" bees "). The proportion of water retained is quite high, the particles adhering to each other’s, not according to the arrangement giving the greatest compactness, but according to the directions in which they came into contact.

The mud generally contains a certain proportion of organic matter (most often less than 10%). It can be peaty if the presence of certain microorganisms promotes the formation of peat. In coastal areas, the presence of sodium chloride prevents the proliferation of these microorganisms, and therefore, the deposited mud is not peat.

By consolidating, the mud loses part of its water, the structure is destroyed, and it turns into a clay or marl, the less soft as the consolidation is more important.

In fact, from a geotechnical point of view, mud and soft clay are often confused. For the geotechnical, these soils are characterized by:

- a water content is generally close to the liquidity limit, and a low dry specific weight γ_d (often less than 10 KN / m^3);
- an organic matter content of approximately 2 to 10%;
- weak undrained Cu cohesion (Less 15 KPa);
- high compressibility giving rise to significant secondary settlements;
- low permeability;
- a normally consolidated state (with over consolidation on the surface).

1.2.3. Sabkha soils

Sabkha soils is a salty flat soil that can be found all over the world and is especially prevalent in hot and arid countries. While sabkha soils is known by different names, for engineering purposes Fookes and Collis (1975) limit these names to: sabkha (coastal salt marsh), playa (a salty surface playa) and salina (relative deep area with high salt ground water table which creates a salt crust on the surface of ground due rise in water table). Where three main features characterize sabkha soils: (a) high content of salty minerals; (b) shallow ground water table; and (c) relatively hard shell (Ghazali et al. 1985). Cutis et al. (1963) first reported on sabkha reported in the Arabian Gulf. Subsequently, it was documented to exist at other parts of the world, e.g. along the cost of Mexico

(Shearman, 1970), the cost of Australia (Akpokodje, 1985). It is usually found between 15◦ and 45◦ north and south of the equator (Abduljauwad and Al-Amoudi, 1995b), and at locations where precipitation rate is less than evaporation rate El-Naggar (1988). Figure 1-1 shows distribution of sabkha soils around the world Ahmed Mohamed Alnuaim (2010).

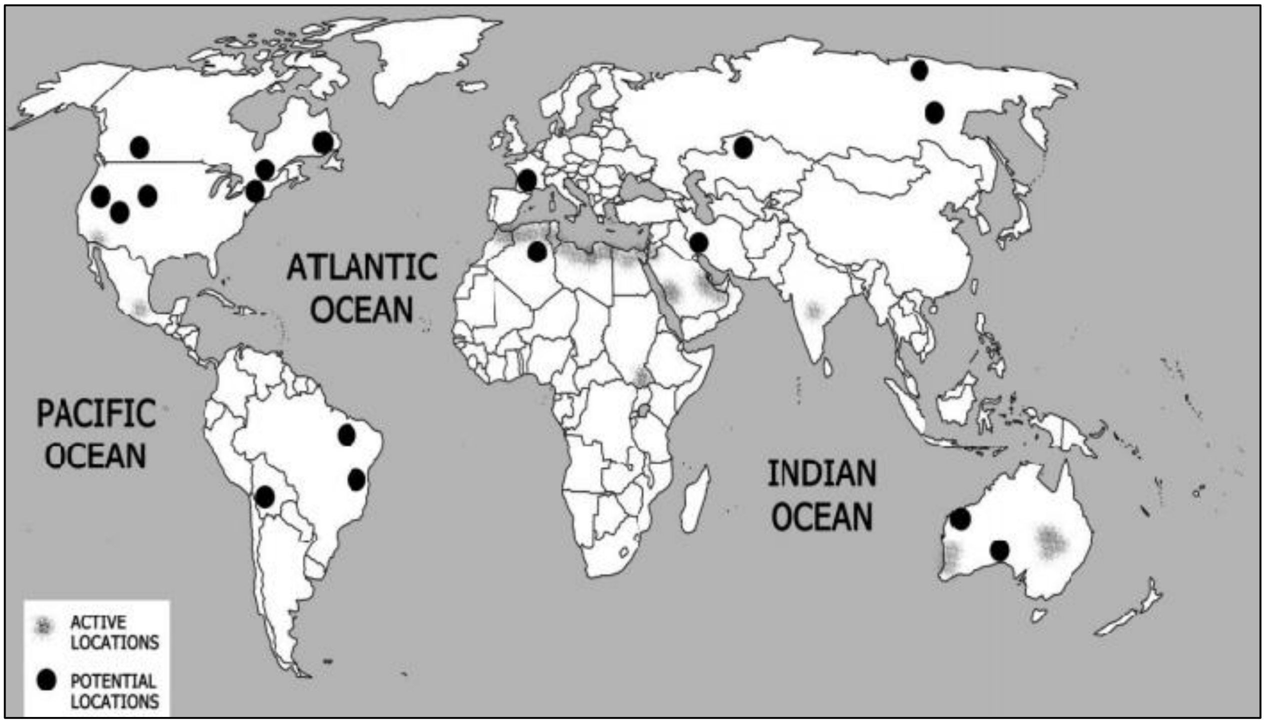


Figure 1-1. Distribution of sabkha soils around the world (after Al -Amoudi, 1994a).

1.2.3.1. Types of Sabkha soils

Several studies have been conducted on the sabkhas of the Kingdom of Saudi Arabia (Kinsman, 1969; Akili, 1981; Abdul-Jawad et al. 1994; etc.), there are two main types of sabkhas soils:

a) Coastal sabkha soils

These sabkha soils are the result of deposition of marine sediments, at least in their parts towards the sea. Most coastal sabkha soils are supratidal surfaces, which were developed following a sedimentation order which seems to have started thousands of years ago by sea water breaking over sand dunes. Figure 1-2 shows the possible sabkha soils process in coastal areas according to Akili (2004), and Figure 1-3 shows the distribution of sebkha soils along the coasts of Saudi Arabia.

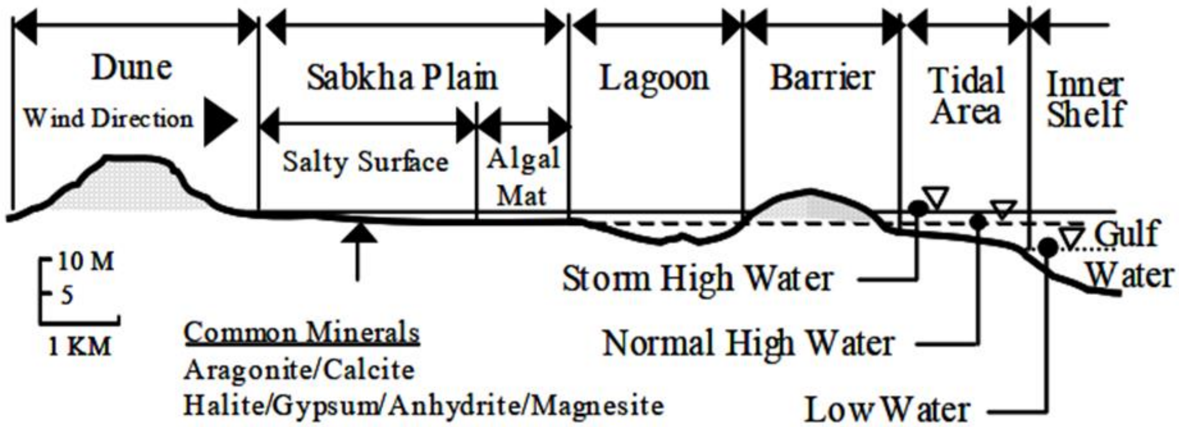


Figure 1-2. Generalized cross section across a typical coastal sabkha with typical surface features. (Akili, 2004).

b) Continental or inland Sabkha (CS)

They usually develop as surfaces, from which the wind has removed dry sediment particles, parallel to the water table, at levels that are controlled by the humidity of the sediment (Johnson et al, 1978), (Figure 1-4)

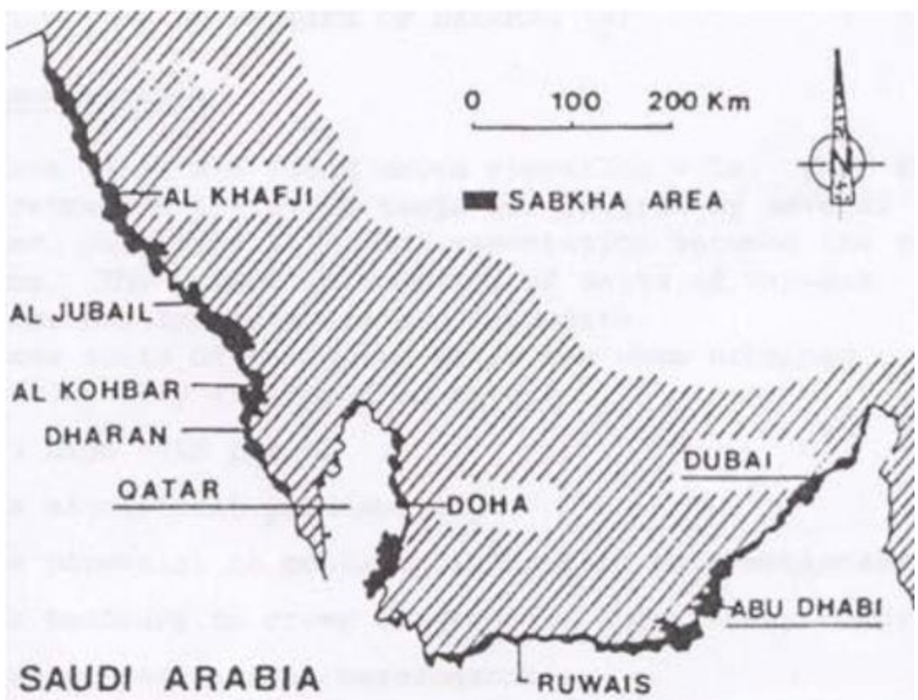


Figure 1-3. Distribution of sabkha along the coasts of Saudi Arabia.

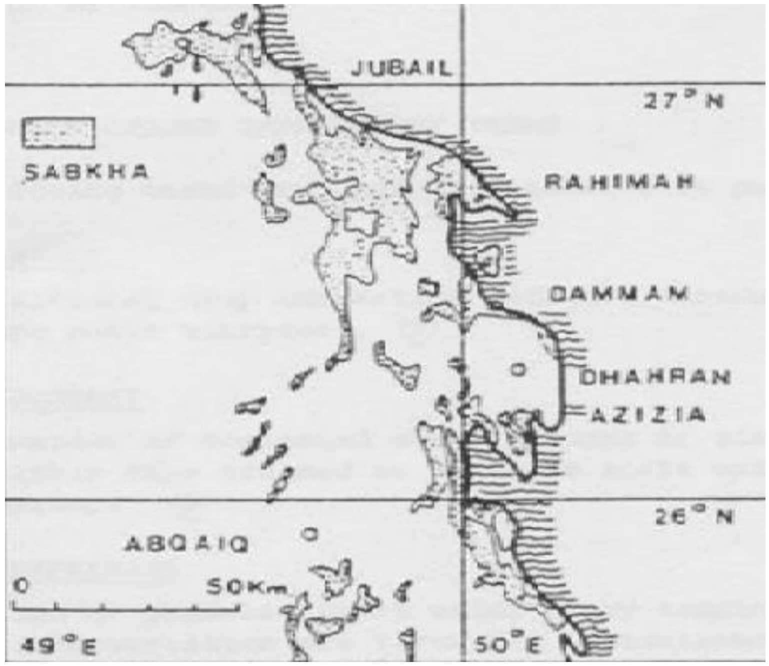


Figure 1-4. Inland sabkha.

1.2.3.2. Characteristics of sabkha soils

The sabkha soils are characterized by the presence of diagenetic salts of different composition and texture at different depths. The precipitation of salts below the water table is attributed to the increase in the salt concentration above its saturation limit (Al-amoudi, 1992; 1995). The deposition of salt in the surface layers is attributed to the evaporation of moisture which has been sucked up from the upper layers by capillary action. The sebkhas still exist in the form of alternating cemented and non-cemented layers, as well as pieces of quartz and / or sand carbonate. In cement layers, the main cementing materials are aragonite and calcite ($CaCO_3$), gypsum ($CaSO_4.2H_2O$), anhydrite ($CaSO_4$), and halite (NaCl). The upper layers of sabkha may exhibit firm and stiff characteristics in its dry state. However, when moistened with water, the resistance is greatly reduced, since the cementing salts are susceptible to leaching and dissolving or softening which leads to loss of resistance in wet conditions. In addition, sabkha soils are characterized by the volumetric change due to the alternative hydration and dehydration of the unstable gypsum Al-amoudi (1992; 1995), Berrabah (2015).

Although sundry papers have been published about the sabkha characteristics, a rough distinction between muddy and sandy sabkha soils can be made (Juillie Y and Sherwood D.E, 1983).

- Muddy sabkha soils: These sabkha soils are relatively young. sabkha soils are generally found between +2m and -6m related to present sea level and are all near the coast.

- Sandy sabkha soils: Sandy sabkhas are often sandy layers interbedded with sandy mud. Table 1.1 highlights the physical characteristics of both sabkha types. Clearly, the muddy sabkhas are the worst to construct road embankment on Juillie and Sherwood (1983)

Table 1-1. Typical soil properties of muddy and sandy sabkhas (Juillie Y and Sherwood D.E, 1983)

Properties	Muddy Sabkhas	Sandy Sabkhas
Salt content (%)	2 to 18	2 to 15
Static cone resistance(MN/m2)	0.2 to 2	1 to 6
Water content (%)	25 to 90	4 to 40
Internal friction	00 to 220	200 to 350
Bearing capacity (kN/m2)	15 to 30	30 to 60
Percentage of Ca CO3 (%)	20 to 90	> 30
Cohesion (kN/m2)	0 to 55	Zero
In-situ density	1.0 to 1.35	1.3 to 1.85
Plasticity index	0 to 40	Non plastic
Compression index	0.4 to 0.95	Zero
S.P.T. values (blows)	0 to 4	2 to 10
Percentage Fines	25 to 95	5 to 25

1.2.3.3. Factors affecting the sabkha soils formation

There are a number of factors affecting the formation of sabkha soil in the Arabian Gulf. These factors can be divided into five groups as follows according to Al-Amoudi (1992) Ahmed Mohamed Alnuaim (2010): Geochemical, Biological, Geomorphological, Hydrological, Climatic.

1.2.3.4. Geotechnical Properties

This section is focussed on the geotechnical properties of sabkha. It presents a summary of findings from several investigations on sabkha soils. This includes: grain size distribution, permeability, consolidation, standard Proctor test results, Atterberg limits, etc. Most of the samples were collected from Ras Al-Ghar, which is a small part of the Al-Riyas sabkha Ras Al-Ghar is considered to be representative of sabkha soil Al-Amoudi et al (1992), Ahmed Mohamed Alnuaim (2010). This section is focussed on the geotechnical properties of sabkha.

a) Atterberg Limits

Al-Shayea et al. (2002) evaluated, the plasticity index (PI) the plastic limit (PL), and the liquid limit (LL) for the sabkha soil as 5.4%, 22.9% and 28.3%, respectively. Ahmed Mohamed Alnuaim (2010).

b) Water content

Usually sabkha soils contains high level of moisture, which is known as sabkha brine. Water content is determined using ASTM D 4643 and was found to be about 25% for sabkha soil in Arabian Gulf (Al-Shayea et al., 2002). Ahmed Mohamed Alnuaim (2010).

c) Unconfined Compression Strength

Sabkha soils usually have a low level strength especially under wet conditions. Al-Amoudi et al. (1992) evaluated the average of unconfined compression strength of undisturbed sabkha to be about 19 kPa. Ahmed Mohamed Alnuaim (2010).

d) Grain-Size Distribution

Al-Amoudi et al. (1992) determined the grain-size for sabkha soils by using the ASTM D 422 test. The results of the sieve analyses for sabkha using dry sieving, distilled water, sabkha brine and methylene chloride are shown in Figure 1-5. The results showed that distilled water most effectively dissolves the salt and cementation materials in sabkha soils. On the other hand, the sabkha brine and methylene chloride had the same results. Figure 1-5 shows the results of the grain-size distribution. Al- Amoudi et al. (1992) believe that the results obtained by using sabkha brine and methylene chloride are more accurate than ASTM D 422 for sabkha soil Ahmed Mohamed Alnuaim (2010).

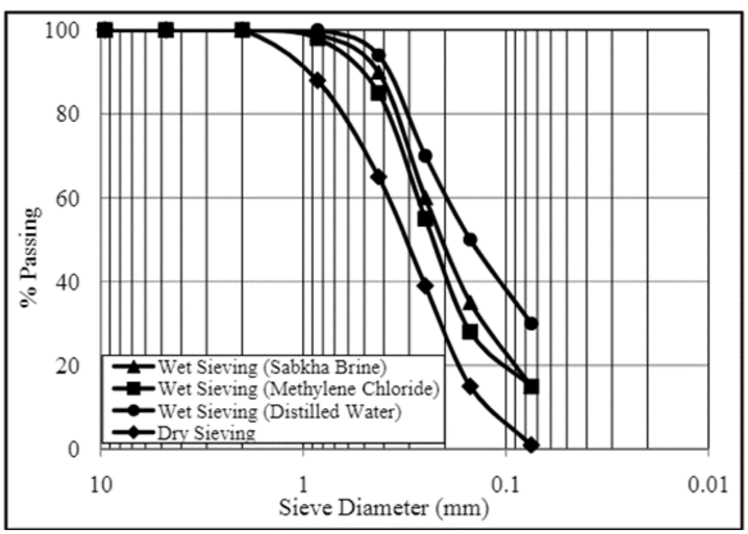


Figure 1-5. Grain-size distribution for sabkha (after Al-Amoudi et al. 1992).

e) Compressibility

Sabkha soils experiences significant reduction in its void ratio when subjected to flooding and leaching. However, the conventional oedometer is unable to predict the behaviour of sabkha because it is not capable of leaching the specimens. To address this issue, Al-Amoudi and Abduljauwad (1994a) modified the conventional oedometer by boring two holes below the porous stone from which percolating water could be collected. over consolidation ratios Ahmed Mohamed Alnuaim (2010). Abduljauwad and Al-Amoudi (1995) tested compressibility of sabkha soils by using the modified oedometer; the samples were soaked and leached using both distilled water and sabkha brine. The sabkha experienced significant reduction of void ratio for both distilled water and sabkha brine, with the reduction being greater for leaching with distilled water. However, the compression (Cc) and swelling (Cs) indices remain the same in both the soaking and leaching of distilled water and sabkha brine. Table 1.2 shows the results for both soaking and leaching with both distilled water and sabkha brine. The average over consolidation ratios (OCR) for sabkha is 19 and 20 for distilled water and sabkha brine, respectively AL-Amoudi et al (1992) Ahmed Mohamed Alnuaim (2010).

Table 1-2. Oedometer results for soaked and leached using distilled water and sabkha brine (Abduljauwad and Al-Amoudi, 1995).

The characteristics	Distilled Water	Sabkha Brine
Initial void ratio	0.93	0.94
Void ratio at soaking condition	0.925	0.924
Void ratio at leaching condition	0.9	0.912
Final void ratio	0.526	0.635
Compression (Cc)	0.18	0.18
Swelling (Cs)	0.016	0.016

1.2.3.5. Density and Compaction parameters

Al-Amoudi et al. (1992) studied the effects of distilled water, sabkha brine and oven temperature on moisture-density relationship of sabkha soil. The results showed that there was no effect on moisture-density curves using either distilled water or sabkha brine. The optimum water content for sabkha soil was found to be 10%. Al-Shayea et al. (2002) evaluated the maximum dry density as 2.022 g/cm^3 (19.83 kN/m3) (ρmax) and optimum moisture content (Wopt) as 13.55%. Ahmed Mohamed Alnuaim (2010).

1.2.3.6. Previous experience with chemical stabilization

The loose, low density, low strength, bulky structural arrangement and metastable fabric of sabkha particles are the key elements needing stabilization. These characteristics are further accentuated by the heterogeneity of sabkha and its concentrated groundwater Al-Amoudi (1994).

Some studies have recently been reported on chemical stabilization of sabkha. Farwana and Majidzadeh (1988) used emulsified asphalt and observed some improvement in strength and stability together with an ability to withstand wet-dry cycles. They claimed that bitumen would serve as a protection against water intrusion to the saline soil and the soil-bitumen mixture would be a substitute for the cohesive strength that many sabkhas normally lack. However, the reported significant improvement in dynamic modulus is not attributed to the emulsion stabilization per se, but to the method of curing whereby oven-drying was used. This has been confirmed more recently by the author A1-Amoudi and Asi (1991); the emulsified asphalt tended to reduce the maximum dry density and increase the optimum moisture content. Furthermore, the unconfined compressive strength was also reduced by the addition of emulsion Al-Amoudi (1994).

Stipho (1989) stabilized two simulated-saline sabkha soils using lime for the fine-grained and cement for the coarser-grained samples. Although various tests were employed and some improvements were recorded, it is very difficult to state whether the same behavior could be obtained for real sabkha soils or not. Further, the artificial method of creating the cementation and the cementing agents themselves in no way resembles the field conditions Akili and Torrance, 1981; A1-Sanad et al., 1989). Moreover, sabkha soil usually consists of silty sands, sandy silts and/or clayey sands, and the separation of fine-grained and coarse-grained soils does not represent actual conditions Al-Amoudi (1994).

AI-Amoudi et al. (1974) have conducted an extensive stabilization program on the effect of inert materials (i.e., nonreactive, including crusher fines, marl) and chemical stabilizers (cement, lime and emulsified asphalt) at five additions (0, 2.5, 5, 7.5 and 10%) on the unconfined compressive strength of an eastern Saudi sabkha. The 7-day cured specimens were prepared at moisture contents either lower or around the optimum moisture content obtained from the standard Proctor tests. The results indicate that significant improvements were only observed for the cement and lime stabilizers. The average strength was improved from 70.1kPa for the control (untreated) specimens to 271 to 1391 kPa and 246 to 1600 kPa for the 2.5 to 10%ocement and lime specimens, respectively. Such improvements ranged from about 250% to 2200% compared with the control. What is important and relevant to the present investigation is the fact that the optimum moisture content from strength perspectives was around 10.7% and 8.5% for the cement- and lime stabilized sabkha mixtures compared to about 12.5% for the compaction tests Al-Amoudi (1994).

1.2.3.7. Problems of sabkha as foundation soil

According to Al-Amoudi (1992) the problems can be divided into the following:

- A potential variation in compressibility of sabkha sediments will lead to excessive differential settlements. This is attributed to the fact that sabkha deposits, generally are known to vary from loose or very loose conditions to dense conditions with a relatively short distance of five to ten meters. As an order, sabkha has a high potential for collapse mainly due to the dissolution of sodium chloride, the leaching
  of calcium ions and the adjustment of soil grain (Al-Amoudi and Abduljawad, 1995).
- Resistance decrease significantly in the surface layers of sabkha due to precipitation, floods, or simply due to the absorption of water from humid environments.
- The alternative volumetric change due to hydration and alternative dehydration of the unstable gypsum will damage the above-ground construction of sabkha (Akili, 1981).
- The interaction of sabkha with fresh water could dissolve some of the cementing materials and decrease the strength (Al-Amoudi, 1992).

1.3. Problems posed by the construction of embankments on compressible soils

The construction of embankments on compressible soils often poses difficult problems. First of all, avoid breaking the load-bearing soil, which can compromise the rest of the construction and create significant damage to the surrounding structures. Then there is the problem of subsidence, with slower but equally harmful effects. When these settlements are significant and they were not taken into account from the start of construction, they

- To cause a deformation of the embankment rendering it unfit for its initial use;
- Cause parasitic thrusts on nearby buried structures (piles, sheet piles, etc.) until their break.

1.3.1. Definition of an Embankment

The embankment is a process consisting of bringing a group of soil or inert materials onto the ground to create a platform or fill a vacuum, dams, road embankments and storage tanks.

1.3.1.1. Types of embankment

- Fill
- Cut

The geometry of track's embankment depends upon the ground topography, alignment of the track, highest flood level, number of tracks, gauge of the track, side slopes, overhauling loading, bearing capacity of the soils and the future extension plans of the government.

1.3.2. Soil constituting the embankment

A bibliographical study was carried out on the characterization of the materials that can constitute the embankment. The characterization should allow the identification of parameters for the use of behavior models such as gravel alluvial embankment described by Valle (2001) and coarse soil backfill described by Fragaszy et al. (1992).

1.3.3. Loads generated by the embankment

The embankment brings loads to the foundation soils which are firstly proportional to its average density (γ) whose estimation is therefore necessary for any load assessment. The measurement of (γ) is often made difficult in earthworks by the speed with which the embankment is placed. In addition, this density can change by increasing or decreasing the water content of the embankment material (rain, etc.) Yasmina Akou (1995).

When the backfill is very wide compared to its height H, it is normal to assume that in the central part the pressure distribution at the backfill base is vertical and uniform:

$$\sigma_v = \gamma . H \tag{1.1}$$

Although it is no longer the same at the edges, it is recognized that the stresses are vertical everywhere and proportional to the embankment height above the point considered (Figure 1-6). This hypothesis is considered to be of lower quality because the width at the head (a) of the embankment decreases in importance compared to its height (H) Schlosser. F (1973).

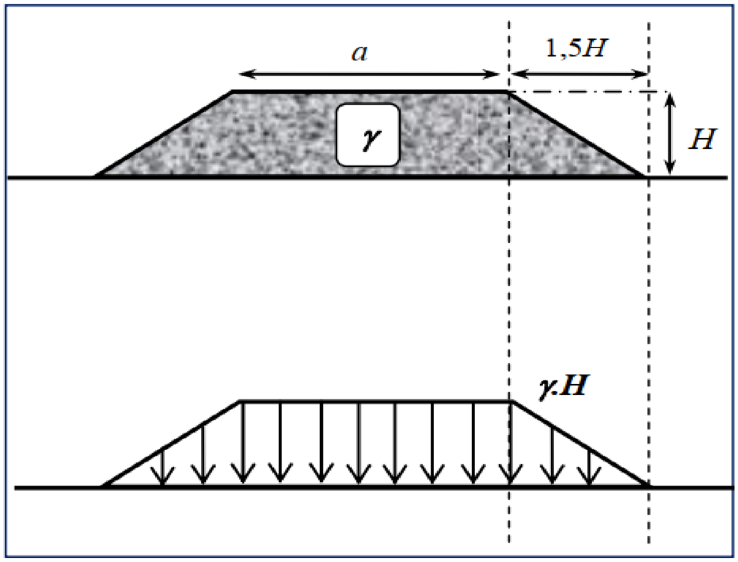


Figure 1-6. Distribution of stresses at the base of an embankment.

1.3.4. Stability issues

Instability of an embankment over soft soil may result from local failure, superficial failure, toe slope failure or deep-seated slope failure as shown in Figure 1-7. The problem of column supported embankments constructed on soft foundations has been significantly addressed by several researchers using numerical methods (Abusharar and Han 2011; Zhang et al. 2014). Han et al. (2004) stated that the deep seated slope (global slope failure) problem is considered the major concern when constructing embankments over soft soils. Thus, one of the ground improvement techniques that has proven to be effective for solving deep seated slope stability problems is the reinforced of stone columns with geosynthetic to support embankments over soft soils (Zhang et al., 2014). Han (2012) summarized the potential modes of failure of columns under embankments into six major types: sliding, rotation, bending, horizontal shear, circular shear and combined failure. He observed that these failure modes are basically dependent on the column's strength, rigidity, length, diameter, location and end bearing, the strength and stiffness of soft soil, and the slope angle and height of the embankment Shaymaa Kadhim (2016).

For slope stability analysis, Bishop's modified method can be considered the most commonly used limit equilibrium method (LEM) for analysis of the stability of embankments over soft soils. A numerical analysis to investigate the stability of an embankment supported with deep mixed columns proposed by Han et al. (2004) showed that the critical slip surface was not circular as Bishop's modified method. Also, they concluded that Bishop's modified method overestimated the stability factor of safety for embankments supported by deep mixed columns over soft soil Shaymaa Kadhim (2016).

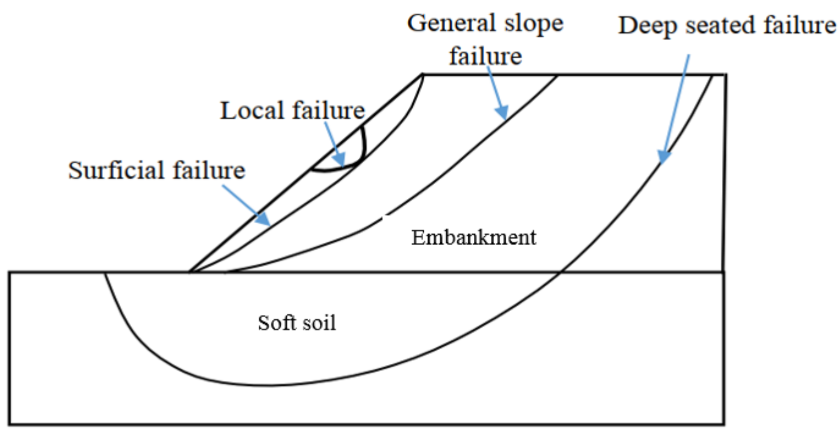


Figure 1-7. Potential slope stability failures (after Han et al 2004).

1.3.4.1. Settlement problems

In contrast to failure due to lack of stability, settlement is a slow deformation of the soil under the weight of the embankment which results (Figure 1-8) in the center of the embankment by a vertical depression;

- under the influence of the embankment, by a vertical depression combined with a lateral displacement foundation soil;
- outside the fill of the embankment, by a lateral displacement of the foundation soil up to a distance depending on the thickness of compressible soil.

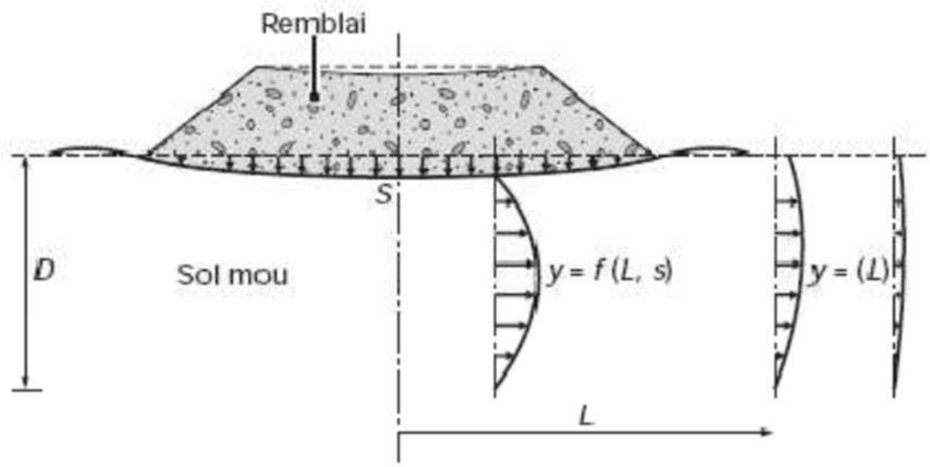


Figure 1-8. Diagram of settlement and lateral displacement of the foundation soil (Pilot et al 1988).

Vertical movements commonly have an amplitude of several tens of centimeters. For very soft or very thick layers, this amplitude can reach several meters. These displacements are more important in the axis of the embankment than towards the ridges of embankments, which causes a deformation of the platform. The horizontal displacements are generally smaller than the vertical displacements, the ratio between the two being in particular a function of the safety coefficient, the geometry of the embankment and the thickness of the soft soils. However, horizontal displacements of several tens of centimeters have been observed. The speed of settlement is variable, depending on the nature of the compressible soils, their thickness and the presence of the draining layers Benmebarek et al. (2015).

The use of reinforcement, geosynthetic-reinforced stone columns on is increasing in the construction of embankments overlying soft soils (sabkha soils). The analysis of these problems is far from straight forward and traditional techniques are not always sufficiently accurate to be a reliable method for design. Numerical analysis using the finite element method overcomes several of the disadvantages of the traditional methods and produces coupled predictions for focuses on the evaluation of the maximum bulging of the stone column and on the settlement of the embankment in sabkha soil.

1.3.4.2. Water flow problems

Compactable soil is often found at the bottom of the valley. Creating a embankment across or along a river valley disrupts the flow of water in times of floods. Flood flows can erode the toe of the embankment, which must then be protected. It also often requires openings through the embankment to allow water to pass through. The deformation of compressible soil can affect the flow of water in the ground water table Hounlelou and Ghislain S. D (2018).

1.3.4.3. Problems related to embankment-structure reactions

Deformations of compactable soil extend under the weight of the embankment beyond the boundaries of the bearing area on the surface. For this reason, embankment construction can cause settlements under existing adjacent structures (railways, other paths, pathway in which the embankment forms expansion, surface foundations of buildings or engineering structures, etc.).

Vertical and horizontal deformations of compressible soils can also lead to very large forces on fixed structures or those that cannot follow surrounding soil movements (eg deep foundations for structures, buildings, or sidewalks. Example).

This interaction between dams and existing or planned structures can have significant consequences for the structures' operating conditions. It must be taken into consideration carefully during the project development and construction work stages Hounlelou and Ghislain S. D (2018).

1.4. Conclusion

Compressible soils are characterized by low shear strength which increases with consolidation, high compressibility and delayed behavior under loading over time. Any

embankment construction on this type of soil generally poses two types of problems related to stability and settlement.

Chapter 2

Methods for Enhancement Embankment over Soft Ground

2. Introduction

With the development of modern society, the demand for transport increases exponentially year after year. Numerous embankments have been built to support roads. Inevitably, soft clay (for example peat and sabkha soils) and other soft compressive soils, which were technically considered not suitable for construction, could be used with specific techniques. The unfavorable characteristics, such as low shear strength, high compressibility, etc., challenge the geotechnical occupation (Han 1999). They limited the design and construction of floor structures, such as the maximum size of bridges and the maximum construction rate. To break these limits, geotechnical engineers are constantly looking for better technical and economic means. In recent decades, many innovative methods have been used to control post-construction establishments of problematic dams based on embankment. To solve these problems, different approaches have been applied to modify the weak structures of soft soil as follows:

- Geosynthetic reinforcement,
- Encased stone columns,
- Stone column, sand column,
- Piled raft
- Vacuum preloading process,
- piles,
- Preloading with vertical drain.

Table 2-1. A comparison of techniques used to control embankment settlements (Modified from Magnan 1994).

Techniques used	Advantage	Disadvantage
Geosynthetic reinforcement	Installation is very simple. Aside from reinforcement, it could also serve as separation between embankment fill and foundation soil to avoid the penetration of granular materials into soft soil.	It is not an effective method if is not combined with other stiffer inclusions such as piles, stone columns.
Encased stone columns	used for increasing the strength and stability of geotechnical structures and in very soft soil situation, use of encased stone columns very important (sabkha soils)	It could take time and they are expensive

Stone column, sand column	Fast	Hazardous vibration could be generated during the construction. Besides, they could not be used in very soft soil situation, since stones and sand need some confinement to sustain their strength.
Piled raft	functional, and reliable	It is almost the most expensive technique among those discussed. It is typically used for bridge approach embankment.
Vacuum preloading process	it saves time and cost on transporting preloading weights.	Limited experiences are available.
Piles	Fast and effective	Expensive
Preloading with vertical drain	Easy to practice	Since the discharge capacity of fabricated vertical drain is hard to estimate precisely, monitoring may be needed to determine the degree of consolidation

In this chapter, we present the geological and geotechnical nature of compressible soils, and we generally present the soil treatment techniques most used at present, in particular the method of soil reinforcement by geosynthetic layers and the method of soil reinforcement by the encased stone columns.

2.1. Reinforcement of Compressible Soils by Geosynthetics

2.1.1. Geosynthetic Performance

One of the techniques for building embankments on compressible soils consists of a reinforcement solution with geosynthetic layers placed at the base of the embankment. This technique represents an economically and technically interesting alternative. Holtz (2001) reports that in 1970 there were only five or six types of geosynthetics available, while today more than 600 different geosynthetics are sold worldwide. The annual global consumption of geosynthetics is nearly 1 billion m^2. In less than 40 years, geosynthetics have revolutionized many aspects of our practice, and in a few applications they have completely replaced the traditional building material. In many cases, the use of a geosynthetic allows significantly increasing the safety factor, improving performance, and reducing costs compared to a conventional design and an alternative construction.

2.1.2. Definitions and Types of Geosynthetics

The name "geosynthetic" designates the synthetic layers used in contact with the ground. Applied for thirty years in the field of civil engineering, geosynthetics are mainly used in the form of sheets to ensure a drain, filter, separation or reinforcement role as well. They allow, when incorporated into the structure, reduce the quantity of materials used, use quality materials limited, to prevent the interpenetration of two layers, to reinforce in great deformation an embankment on soft ground Geotextiles are woven, non-woven, or knitted, permeable products made from polymer and used in the fields of geotechnics and civil engineering. The function of the geotextile in the soil can be separation, filtration, and also reinforcement.

Geogrids are geosynthetics whose function is to strengthen soils. A geogrid is a flat structure based on polymer, constituted by an open and regular network of elements resistant to traction and which can be assembled by extrusion, by gluing or by interlacing, whose openings have dimensions larger than those of the constituents and allowing the containment of the soil. The size of the mesh must be sufficient to allow the penetration of large elements of the soil, and the creation of an effect of nesting of these constituents in the geogrid.

Geocomposites result when two or more materials are combined in the manufacturing process of geosynthetics. They can be compounds of geotextiles-geonets, geotextiles-geogrids, geotextiles-geomembranes, geomembranes-géonets, geotextile-polymer nuclei, and the same three-dimensional polymer structures of cells. There are almost no limits to the variety of geocomposites that are possible and useful. They can be used, either in geotechnics (separation and reinforcement functions), or for wearing courses (reinforcement function, particularly in repair). Geosynthetics consist mainly of synthetic fibers for reasons of cost and resistance to the chemical and biological actions of the soil.

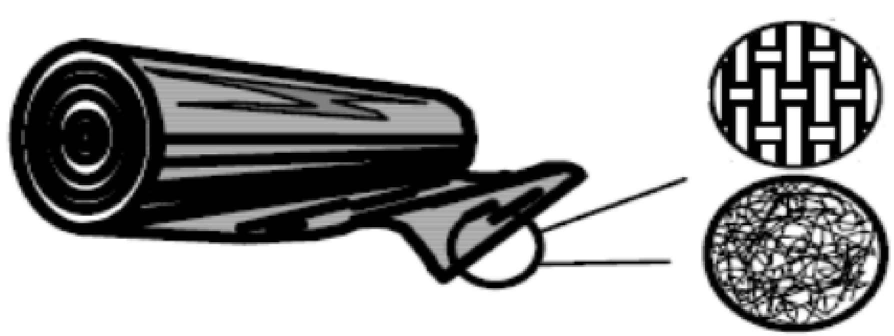
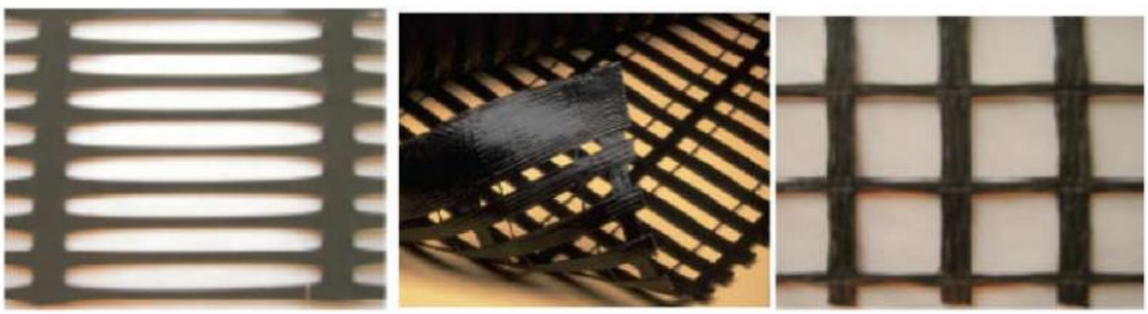


Figure 2-1. Geotextiles (Bathurst 2007a).

Figure 2-2. Geomembranes (Bathurst 2007a).



Extruded geogrids **uniaxial geogrids** **biaxial geogrids**

Figure 2-3. Various types of geogrids (Bathurst 2007a).

Figure 2-4. Geocomposites (Geotextiles-Geomembranes) (Bathurst 2007a).

These fibers are obtained by spinning and then drawing of macromolecular structures also called polymers. The polymers most commonly encountered in geosynthetic layers are the following:

- Vinyls, such as polyvinyl chloride (PVC), very resistant to water and to the attack of many chemicals or micro-organisms; they are widely used as drains.

- Acrylics are used in geosynthetics in the form of resin or emulsion to consolidate them
- Polyamides (PM ex: Nylon) are very sensitive to water and therefore little used.
- Polyesters (PS) have high mechanical properties and inertness to acids and microorganisms which make them very interesting. They are however attacked on the surface by the bases.
- Polyole fins such as polyethylene (PE) or polypropylene (PP) are also widely used due to their high chemical insensitivity and good mechanical properties and their low cost.

2.1.3. Functions of Geosynthetics

The incorporation of geosynthetics into the soil improves its mechanical and hydraulic behavior. The main roles are as follows (Figure 2-5):

- **Reinforcement:** use of the resistance capacity of a geotextile or a product related to geotextiles in order to improve the mechanical properties of soils.
- **Protection:** function consisting in preventing the localized damages concerning a given material by using the geotextile, in general a geomembrane.
- **Filtration:** maintenance of the soil or other particles subjected to hydraulic forces by allowing the passage of fluids through or in a geotextile.
- **Drainage:** collection and transport of rainwater, groundwater or other liquids in the plane of a geotextile or a product related to geotextiles.
- **Separation:** prevention against mixing of two materials of different natures by the use of a geotextile.

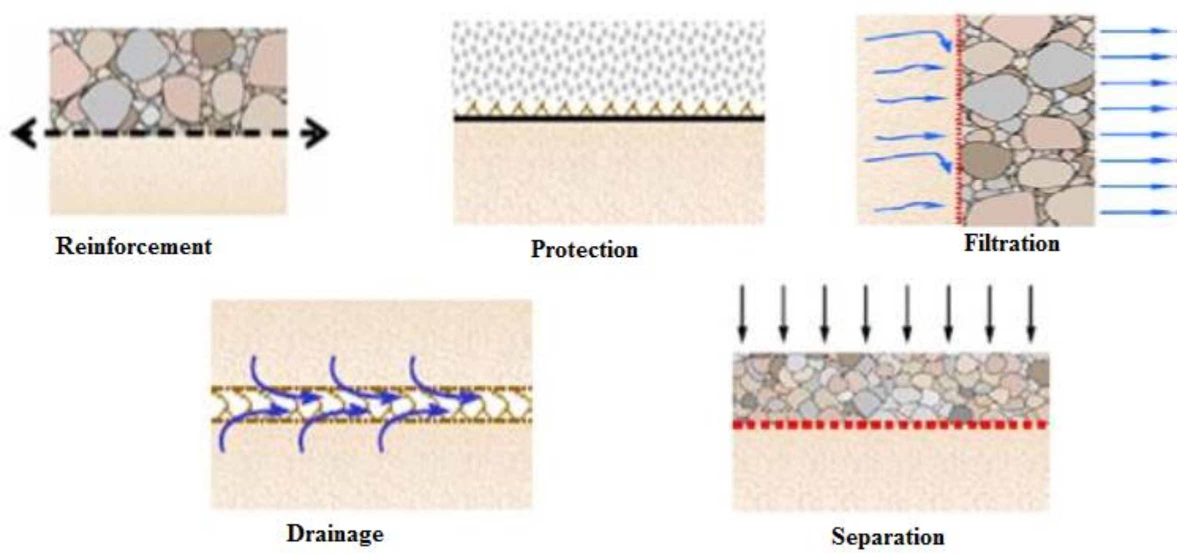


Figure 2-5. Main roles of geosynthetic materials.

Geosynthetics are generally defined by their main function. In a number of applications, in addition to the primary function, geosynthetics generally performs one or more secondary functions. It is important to consider the main and secondary functions in the calculations and design features.

The geomembrane finds its place in many areas of construction. It is used in the hydraulic environment for the creation of canals or basins, whether for irrigation, drinking water supply or wastewater lagooning. This field extends to all applications concerning water such as watertight masks for dikes and dams or navigable canals. The geomembrane is also used for sealing household or industrial waste storage, whether it is of animal, vegetable or chemical origin. Indeed, the lack of a naturally watertight site and the current regulations encourage the use of geomembranes.

The use of geotextiles and geotextile-related products depends on the needs of the work in which they are placed. Geotextiles offer a wide range of tensile strength and stiffness; they can be used in soil reinforcement as in walls or encased stone columns (ESC).

The road was the first field of employment where geotextiles were used in large quantities; geotextiles are used for roads and tracks as separators, reinforcements, filters, drains and to fight against slope erosion.

2.1.4. Previous Reinforced Embankments by geosynthetics on Soft Soil

Volman et al, (1977). In this historic case, two test embankments were made on 4.2 m of peat and clay. One of the embankments was unreinforced and the other was reinforced with a woven geotextile sheet. The reinforcement has a tensile strength of 61 KN / m, the deformation at break equal to 20% and the stiffness of average tension equal to 258 KN / m. The authors report that the embankment without reinforcement failed for a height of 3.5 m, while the embankment with reinforcement reached 4.5 m without rupture.

Rowe and Soderman (1984, 1985) present a study of the stability analysis of reinforced embankments combining the limit equilibrium method and the finite element method. The test embankment was carried out on 3.8 m of organic clay. The tensile strength of the reinforcement and its stiffness (215 KN / m and J = 2000 KN / m respectively). The height at break was 1.75 m without reinforcement and 2.75 m for an embankment reinforced at the base with geosynthetics.

The limit balance analysis of these embankments executed by the authors predicted heights at break equal to 1.7 m and 2.55 m for the unreinforced and reinforced embankment, respectively. These forecasts are compared with the heights observed at break in both cases.

Gnanendran et al (2015) study the behaviors of test embankment constructed on an alluvium deposit in Moncton, and the reinforced test embankment constructed on a soft compressible soil in Sackville, the soils at both these sites have the same geological depositional history, where a high-strength polyester woven geotextile was used as basal reinforcement are discussed in this chapter. Performance monitoring included the instrumentation of the foundation soil with inclinometers, pneumatic piezometers, settlement plates, settlement augers.

Fouad Berrabah et al (2020) carried out three-dimensional finite element analyses to simulate the behavior of Geosynthetic-Reinforced Embankment over Locally Weak Zone (sabkha soils), Comprehensive numerical analyses were performed to study the influence of the stiffness of the geosynthetic, the diameter of the locally weak zone and the friction angle of the embankment fill is also analyzed. Results of 2D and 3D numerical analyses were also compared.

Figure 2-6. Installation of a geogrid (Berrabah et al 2020).

2.2. Reinforcement of Compressible Soils by Encased Stone Columns

2.2.1. Introduction

Stone column reinforcement (SCR) method consists of partial replacement of loose and/or soft soil with vertical columns composed of compacted stone or granular material. Stone columns (SCs) have been extensively used under large raft foundations and embankments, this technique was further developed in Germany by employing vibration (Baumann and Bauer, 1974) and Greenwood and Kirsch (1984). SCs reinforce the soft soil by reducing compressibility and increasing bearing capacity, SC installation can reduce the settlements up to 50% compared to untreated case for both cohesive and cohesion less soils under large areas (Bachus & Barksdale, 1989). Other advantages of SCs can be classified as; allow faster consolidation in cohesive soils, mitigate liquefaction and improve stability. Another method that can be used to provide the required lateral confining pressure to increase the bearing capacity of granular columns is to encase the column with a suitable geosynthetic. The columns that can be encased with geosynthetics are the main materials used for increasing the strength and stability of geotechnical structures. The idea of encased stone columns was first proposed by Van Impe (1989). This technique has been successfully used in different projects. The main advantage of geosynthetic encased stone columns (ESCs) over ordinary stone columns (OSCs) is the higher stiffness resulting from the hoop force in the geosynthetic, which ascent the load capacity. In addition, the encasement prevents the lateral intermix of the granular material into the surrounding soft soil. Installation of the ordinary stone columns (OSCs)

2.2.1.1. Basic design parameters

The parameters used such as spacing (S), diameter (D), and arrangement of columns (triangular, squared or hexagonal pattern) to improve the bearing capacity and to reduce the settlement

2.2.1.2. Stone column spacing (S)

Column design must be site specific and precise guidelines cannot be provided on maximum and minimum column spacing.

2.2.1.2.1. Stone column diameter (D)

Installation of stone columns in soft cohesive soils is basically a self-compensating process, i.e. the softer the soil, the bigger is the diameter of the stone column formed. Due to lateral displacement of stones during vibrations/ramming, the completed diameter of the hole is always greater than the initial diameter of the probe or the casing. The column diameter installed by vibro flot (diameter 300-500 mm) varies between 0.6 m in case of stiff clays to 1.1 m in very soft cohesive soils (Ranjan 1989).

2.2.1.2.2. Stone column arrangement

Stone columns should be installed preferably in an equilateral triangular pattern which gives the densest packing although a square pattern and hexagonal pattern may also be used. A typical layout in an equilateral triangular square pattern and Hexagonal are shown in Figure 2-7

The relation between the column spacing and the unit cell diameter is given by:

$$de = s.cg \tag{2.1}$$

where, de = Diameter of the unit cell, s = Distance between adjacent columns, cg = Constant coefficient related to columns arrangement. In triangular arrangement, cg = 1.05, in square arrangement, cg = 1.13, and for hexagonal arrangement, cg = 1.29.

2.2.1.2.3. Incorporation rate

The rate of incorporation or the coefficient of substitution (α) is the ratio of the area of the column (Ac) to the area of the domain of influence of the column (A):

$$(a) = Ac/A \tag{2.2}$$

2.2.1.2.4. Stress concentration ratio

When a mass of soil treated with stone columns is subjected to a uniformly distributed stress ($\Delta\sigma v_0$). Stone columns have strengths and stiffnesses greater than those of treated soil, which leads to a vertical stress concentration on the noted columns ($\Delta\sigma v_{,c}$) and a reduction in the stress on the noted soil (Δσv,s) see Figure 2-8. The stress concentration ratio noted n defined as the ratio between the vertical stress on the column to that on the ground:

$$n = \frac{\Delta\sigma v,c}{\Delta\sigma v,s} \tag{2.3}$$

The ratio n depends on the relative stiffness of the column and the surrounding soil. The value of n is generally between 2 and 6 (Aboshi 1979).

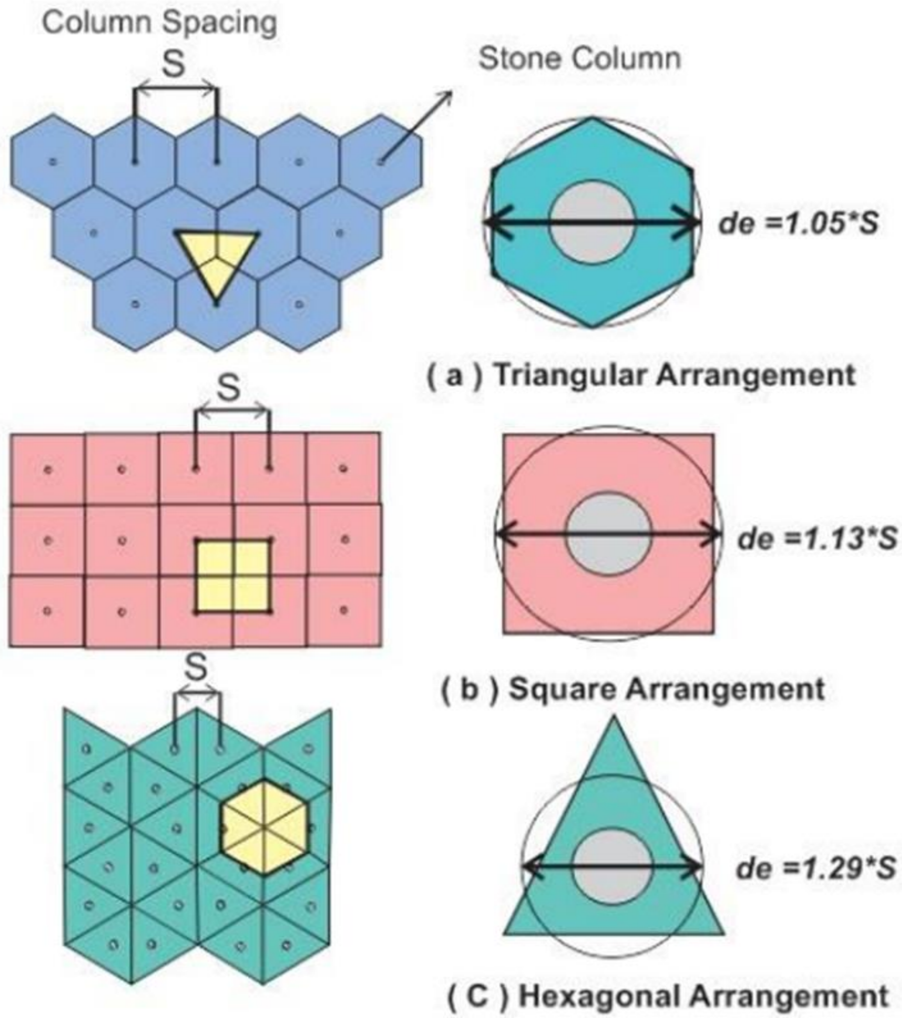


Figure 2-7. Arrangement of column.

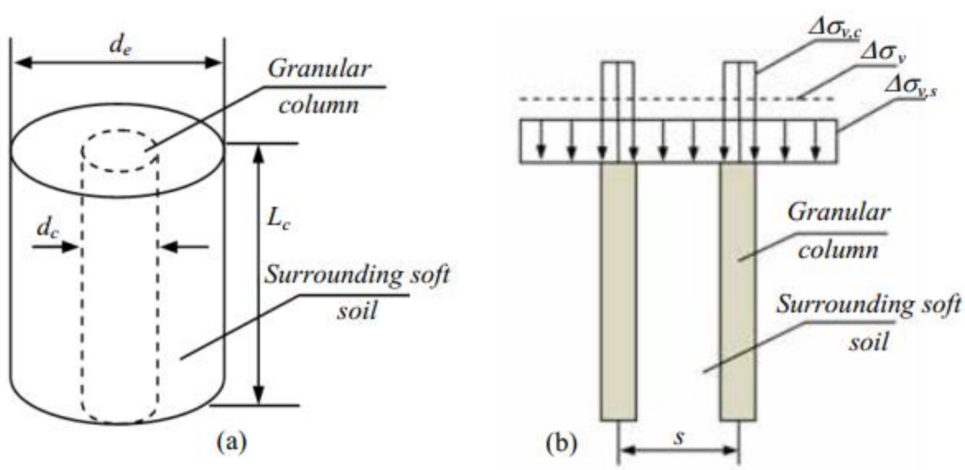


Figure 2-8. (a) Unit cell scheme and (b) stress distribution.

2.2.1.3. Installation methods

The following methods are commonly used to install stone columns; replacement method involves replacing in-situ soil with stone column materials (Figure 2-9 and 2-10). A vibratory probe (vibro flot), accompanied by a water jet, is used to create the holes for the columns. This technique is suitable when the ground water level is high and the in situ soil is relatively soft.

Displacement method is utilized when the water table is low and the in-situ soil is firm. It involves using a vibratory probe, which uses compressed air, to displace the natural soil laterally. Figure 2-9 depicts the different construction stages of installation.

The grain size of the stone column material is one of the main controlling parameters in the design of the stone columns. Hence, the influence of column material in the performance of stone column was studied through laboratory experiments on model stone columns installed in clay by Dipty and Girish (2009). Five reinforcement materials were studied: stones, gravel, river sand, sea sand and quarry dust. It was found that stones are the most effective material and gravel is the most efficient.

It is common knowledge that bulging and subsequent failure of granular piles mainly occur due to high stress concentration near the top of the granular pile. Stresses near the top of the treated ground are significantly influenced by the presence of granular mat as well as the cross-sectional area of the granular piles. After installing stone columns, a blanket of sand or gravel of 0.3 m or more in thickness is usually placed over the top. This blanket works both as a drainage layer and also to distribute uniform stresses under the structure. As shown in Figure 2-11, the vibro-replacement method is applicable for fine-grained and coarse-grained material whereas vibro-displacemet is just applicable in coarse-grained soils.

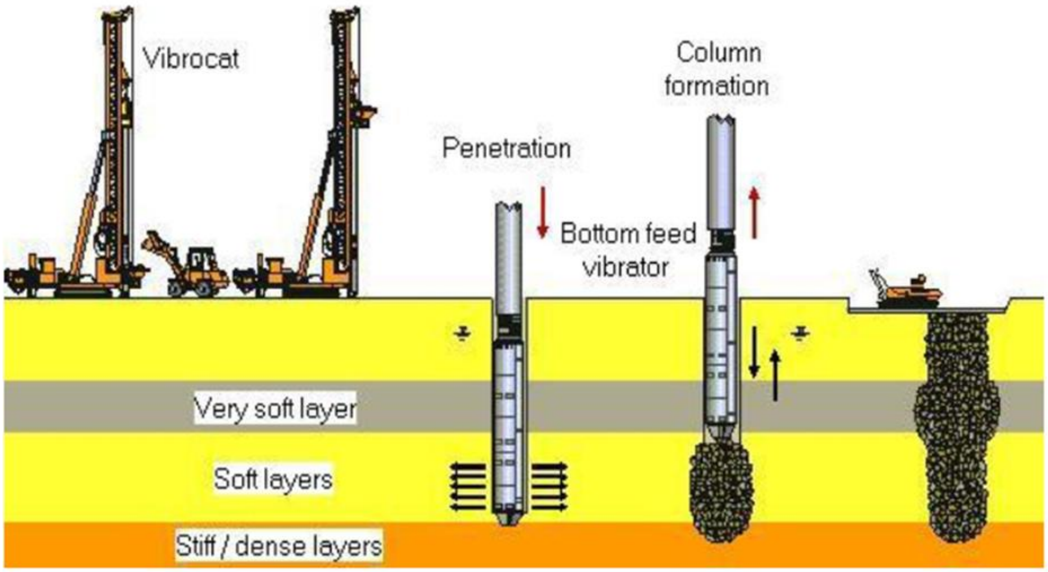


Figure 2-9. Dry bottom feed installation method (Keller, 2002).

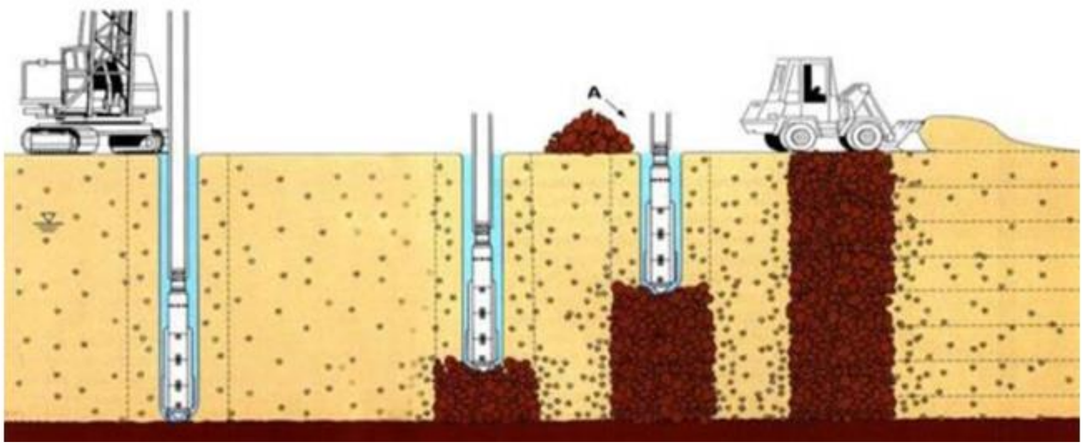


Figure 2-10. The Application Vibro-replacement of Wet Method (Keller Far East, 2002).

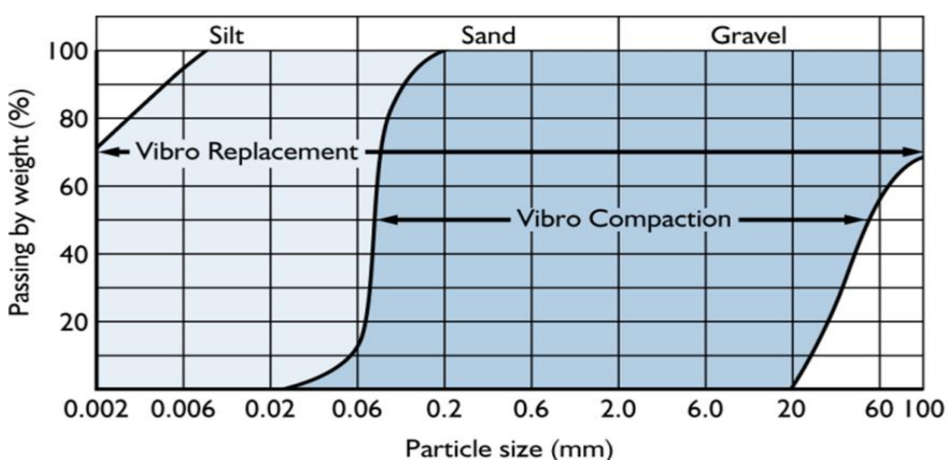


Figure 2-11. Applicability of Vibro-Compaction and Vibro-Replacement (Keller Far East, 2002).

2.2.1.4. Improvement of Soft Clay Characteristics Using Stone Column technique

Reinforcement of the soil with stone column provides basically:

- Reduce the total and differential settlement of soft cohesive ground due to the applied load,
- Improvement of the bearing capacity of the soil,
- Acceleration of the consolidation process,
- Density cohesion less material and protect it from potential liquefaction under seismic loading.

2.2.1.5. Investigation of Stone Column

2.2.1.5.1. Basic Concept (Unit Cell)

The unit cell concept was first applied to sand drain analysis on radial consolidation (Barron, 1948). Most of designs developed by researchers (Murugesan and Rajagopal 2006; Ambily and Gandhi 2007; Raithel and Kempfert, 2005; Alkhorshid et al 2018) are based on the unit cell concept (see Figure 2-12), which is based on the assumption that the column and the surrounding soil are going to deform together at the same strain. Correspondingly, in order to accomplish the equal strain, two conditions should be met: rigid loading and a loading area larger than the thickness of the reinforced zone (Han, 2012).

It was hypothesized that there was no lateral deformation of the soil at the edge of the unit cell (Ambily and Gandhi, 2007). The stress concentration ratio of a unit cell is thereby the ratio of constrained modulus of the column to that of the surrounding soil at an equal strain condition (Han, 2015). The most common term used in unit cell concept is the area replacement ratio (ARR), that is, the cross sectional area of the column divided by the total cross sectional area of the unit cell.

The assumptions of this concept are therefore:

- Rigid, impermeable side boundaries
- One dimensional deformation (i.e. vertical)
- Large loading area
- Zero shear stresses at all boundaries

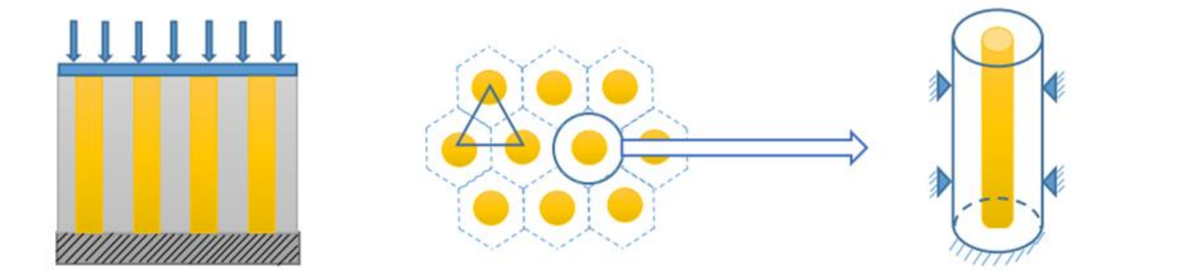


Figure 2-12. Unit cell concept (after Gniel and Bouazza, 2009).

2.2.1.6. Failure Systems

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

Bulging failure: Stable ductile deformation which occurs when lateral resistance is less than axial load. Lateral deformation occur in columns near the edge of footing, more obvious in slender columns.

Shear failure: Developed when column is subjected to high stress ratio and low confinement.

Punching failure: Generally observed in short column due to insufficient skin friction developed along its length and when stress at the column toe is high. In Figure 2-13, 2-14 these three types of failures are shown.

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.

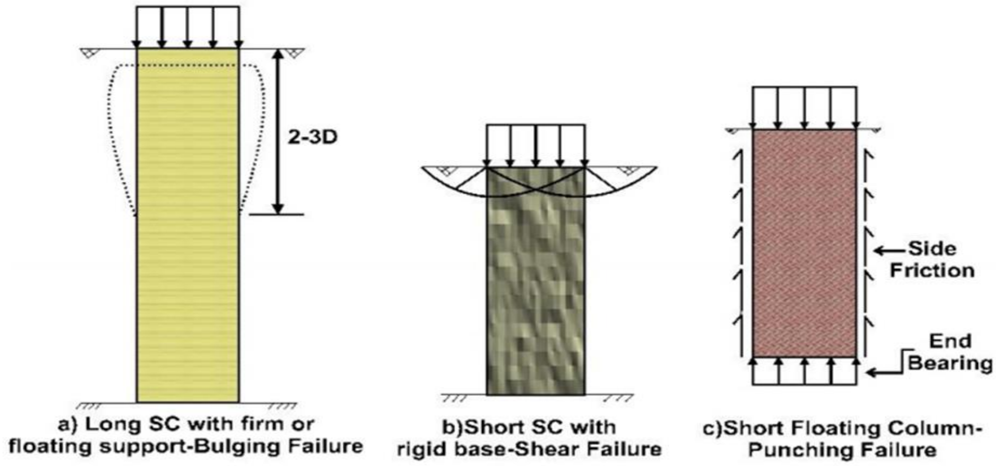


Figure 2-13. Failure Styles of Single Stone Column (Barksdale & Bachus, 1983).

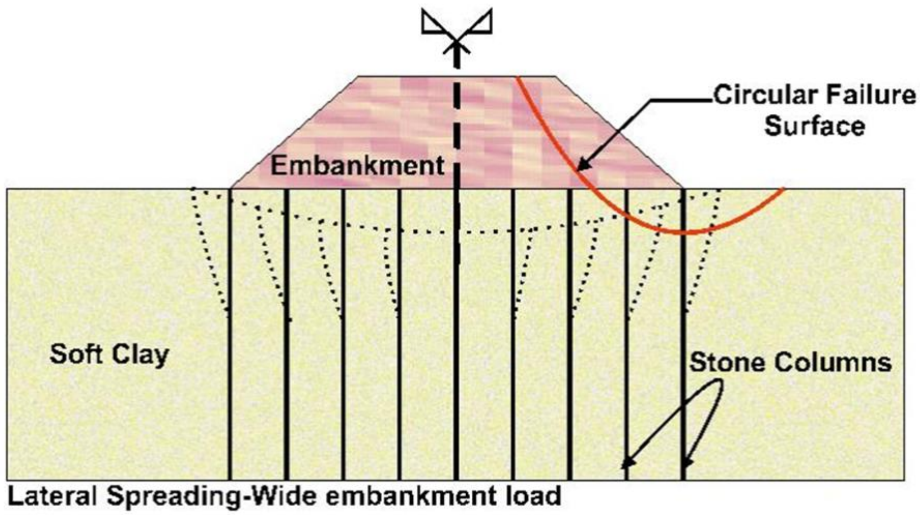


Figure 2-14. Failure types of stone columns under embankment (Barksdale & Bachus, 1983).

2.2.1.7. Settlement of stone columns

Whilst the installation of Stone columns is increasingly used as soft soils reinforcement to support a variety of structures, a significant settlement reduction is also achieved as compared to untreated soft ground. Tallapragada et al. (2011) reported a considerable reduction in the settlement of encased stone columns (ESCs) compared with ordinary stone columns(OSCs). This reduction was even greater for larger diameters (D) and longer lengths of columns (Lc).

2.2.2. Embankment on Geosynthetic -Reinforced Stone Columns (EGRC)

These are soft soil improvement techniques whose mechanical properties of marginal soils were well established. However, the use of stone columns is usually associated with excessive deformation due to the lack of lateral support from the surrounding soil. The lateral support is expressed by means of the undrained shear strength. According to German regulations, stone columns can be applied, if soft soils have undrained shear strength of cu = (15 – 25) kN/m^2. In contrast to conventional techniques, encased stone columns (ESCs) can be used as a ground improvement and bearing system in very soft soils, for example peat or sludge with undrained shear strengths cu < 2 kN/m^2, (Kempfert, 2003).

The lack of lateral support causes lateral deformation in the upper part in the stone column and excessive settlements which lead to failure by bulging. When an embankment is constructed on the soft ground reinforced with ordinary stone columns (OSCs), lateral spreading of ground occurs beneath the embankment. The lateral spreading reduces the confinement of the stone column. Therefore, further developments of the stone column technique include the reinforcement of the column using either horizontal layers (HL) of reinforcement or encasing individual stone column by geosynthetics (ESCs).

2.2.2.1. Reinforced stone column with horizontal layers of geosynthetic (HL)

For Madhav et al. (1994), based on numerical analysis, the degree of decreasing lateral deformation and increasing bearing capacity depends on vertical spacing between horizontal layers (Sv) which has a predominant effect on stone columns. Also the internal friction angle of the stone column material (Φ) has a great importance.

Sharma et al. (2004) used a series of laboratory tests on layered stone columns reinforced with horizontal layers of geogrid (Figure 2-15). The tests were performed on single sand columns (SSC) of 300 mm in length and 60 mm in diameter. The number of geogrid layers and the distances between adjacent layers were different in every model test. The results confirmed the influence of horizontal layers (HL) of geogrid on the amelioration of bearing capacity and the decrease of lateral deformation.

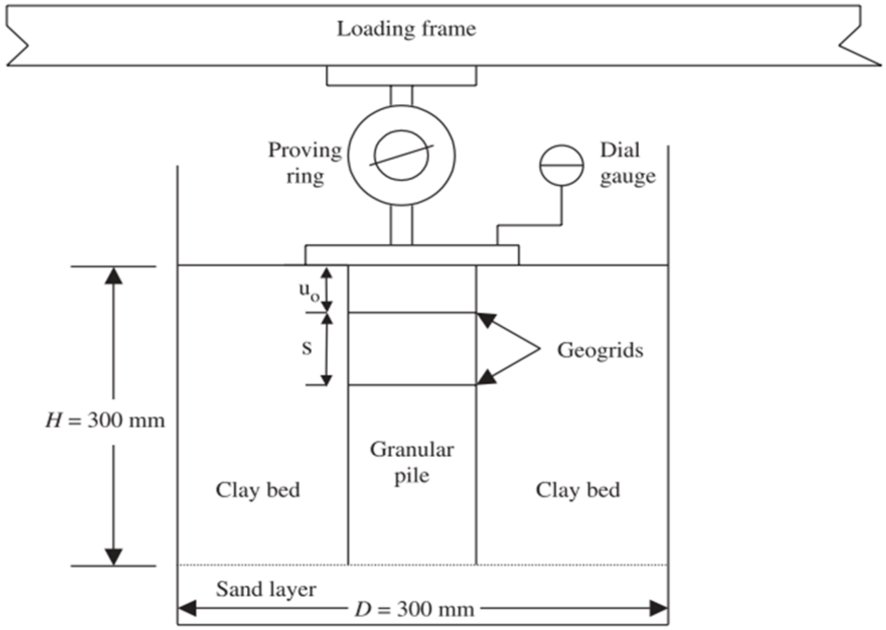


Figure 2-15. Experimental setup by (Sharma et al. 2004).

Two series of load tests were conducted. First, the load tests were conducted by loading the stone column alone using a diameter (60 mm) bearing plate which had a same size with the stone column diameter. Then, the load tests were conducted by loading the entire area by using a diameter (120 mm) bearing plate. The load was applied in increments of 45 to 275 N. The diameter of the lateral deformation was measured at different depths from the top of the stone column and the settlement was recorded with a dial gauge.

As shown in Figure 2-16-a, the stress required for a given settlement increased when the clay bed was reinforced with a granular pile, as granular material offered higher resistance to deformation by virtue of its higher friction angle and accelerated drainage by virtue of its high permeability compared with that of a clay bed. The stress increased further when the pile alone was loaded, as the granular material in a pile resisted load better than the soft clay bed. It was also observed that the geogrid effectively improved the load carrying capacity of the granular column (Figure 2-16-b). The amelioration factors increased with the increase of numbers of geogrid (n) and decrease of geogrid spacing (s). Based on Figure 216-b, the stress to induce a settlement of 3 mm increased 80% comparing to the unreinforced stone column (USC). It was also observed that for 5 numbers of geogrid layers with a spacing of 10 mm, the bulge was negligible equal to around 4% of the column diameter. Meanwhile, the bulge length was 1.33 times of the column diameter Hosseinpour I (2015).

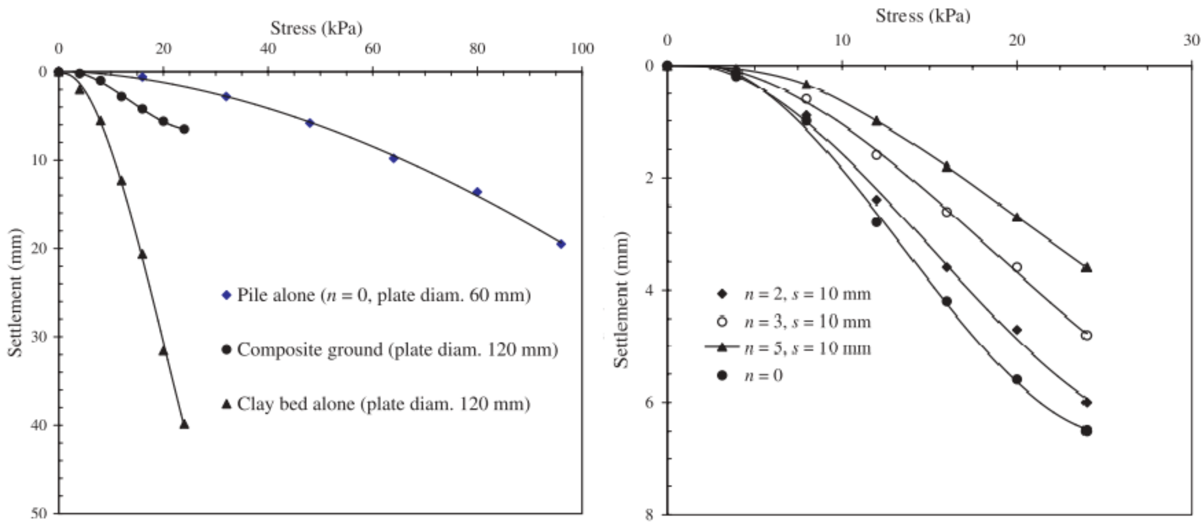


Figure 2-16. (a) Stress–settlement curves for a clay bed alone, a stone column alone (n= 0), and composite ground and (b) Effect of number of geogrids (n) on the stress–settlement response of composite ground (Sharma et al. 2004).

Mahmoud Ghazavi et al. (2018) implemented series of laboratory tests and numerical simulation by using vertical encasement or horizontal layers (Figure 2-17). Some large body laboratory tests have been performed on horizontally reinforced stone columns with diameters of 60, 80, and 100 mm and groups of stone columns with 60 mm diameter. Results show that the bearing capacity of stone columns increases by using horizontally reinforcing layers. Also, they reduce lateral bulging of stone columns by their frictional and interlocking effects with stone column aggregates. Numerical analysis results showed that the bearing capacity increases considerably with increasing the number of horizontal layers and decreasing space between layers.

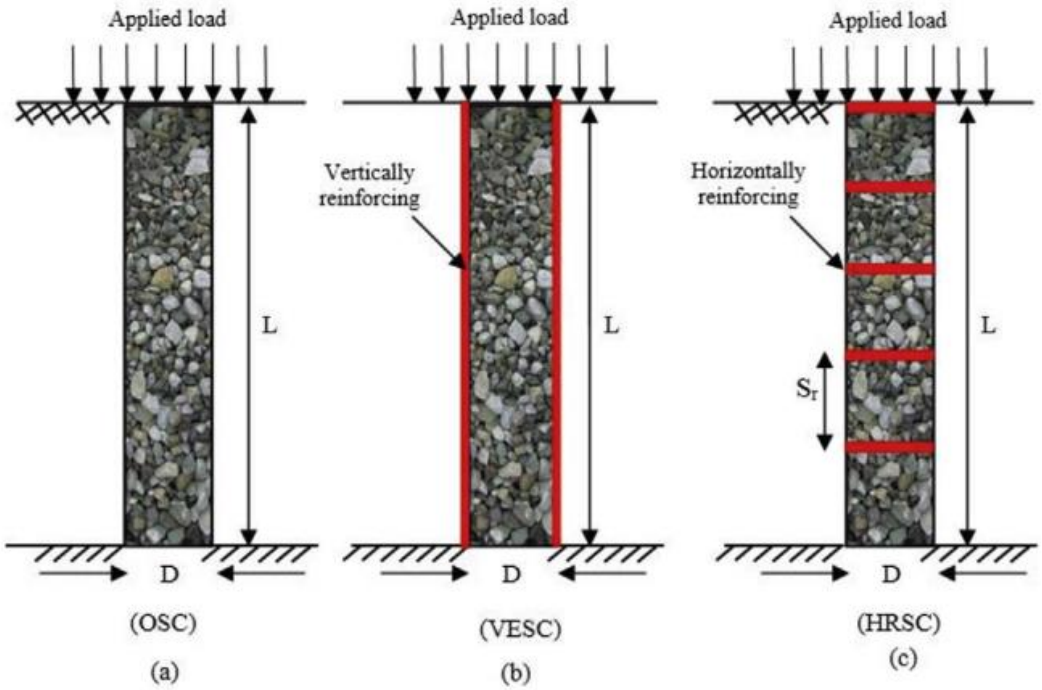


Figure 2-17. Schematics of: (a) OSC; (b) VESC; (c) HRSC (Mahmoud Ghazavi et al 2018).

Ahad Ehsaniyamchi et al. (2019) reported the numerical performance of the stone columns are often used to improve the load-carrying characteristics of weak soils. In very soft soils, however, the bearing capacity of stone columns may not significantly improve the load-carrying characteristics due to the very low confinement of the surrounding soil. In such cases, encased stone columns (ESCs) or horizontally reinforced stone columns (HRSCs) may be used (Figure 2-18). The results show that with proper reinforcing stone columns, in addition to a considerable reduction in settlement, the consolidation time can be greatly decreased and most of the settlement will occur during the loading period. Also, the consolidation settlement rate may be increased by using a smaller column diameter and a larger area replacement ratio for the unit cell, stiffer geosynthetic reinforcements, and greater values for the internal friction angle of the stone column materials.

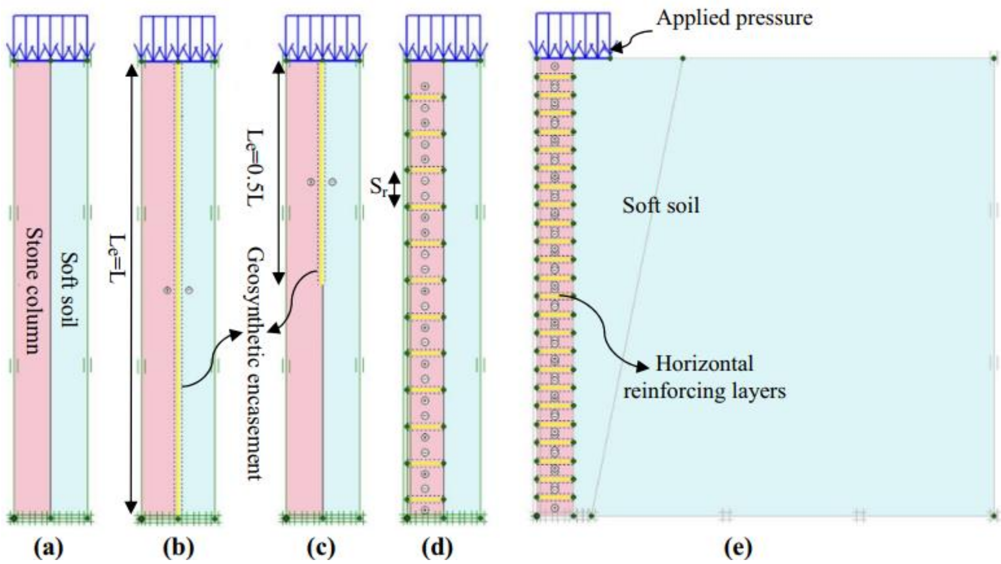


Figure 2-18. Examples of various models used in numerical analyses. Unit cell models of: (a) OSC, (b) full-length ESC, (c) half-length ESC, (d) HRSC and (e) single HRSC with Sr = 0.25D (Ahad Ehsaniyamchi et al 2019).

2.2.2.2. Encasing stone column with geosynthetic materials

Columns supported embankments (CSEs) are constructed on soft soil to accelerate construction, improve embankment stability, control total and differential settlements, and protect adjacent facilities. They are selected to be stiffer and stronger than the surrounding soft soil, and if properly designed, they can prevent excessive movement of the embankment (Almeida and Marques, 2013). The columns are installed at a spacing determined by the design engineer, with lower costs for construction if the columns are properly spaced. A geosynthetic reinforced bridging layer, also known as a load-transfer platform or a load-carrying geosynthetic layer, is often used to transfer embankment and surcharge loads to the columns and to prevent settlements between them. The bridging layer consists of compacted sand or gravel, which may or may not include geosynthetic reinforcement. When included, the geosynthetic reinforcement consists of one or more layers of planar polymeric material, which may be a woven geotextile or, more often, a geogrid Hosseinpour I (2015).

When the granular columns are installed in very soft soils, they may not derive significant load capacity owing to low lateral confinement provided by the surrounding soil. McKenna et al. (1975) reported cases where the granular columns were not restrained by the surrounding soft clay which led to failure due to excessive bulging, and also the soft clay squeezed into the voids of the aggregate. The squeezing of clay into the stone aggregate ultimately reduces the load bearing and also drainage capacity of the granular column. The problem can be solved by wrapping the compacted sand or gravel column with an appropriate-stiffness geosynthetic encasement (Raithel et al. 2002; Alexiew et al. 2005; Murugesan and Rajagopal, 2009). Additional confinement provided by geosynthetic encasement leads the granular columns to become stiffer and thus the load carrying capacity improves (see Figure 2-19). This is particularly more important when the objective is reducing the vertical stress on surrounding soil, leading to reduction in horizontal stress acting on the foundations of adjacent structures. Additional and recent application of the encased granular columns is preventing the residual foundation soils to collapse (Araujo et al. 2009). Encasement also prevents intermixing of the surrounding soft clay into the column aggregate and thus the drainage capacity of the granular column remains intact Almeida et al. (2015) Hosseinpour I (2015).

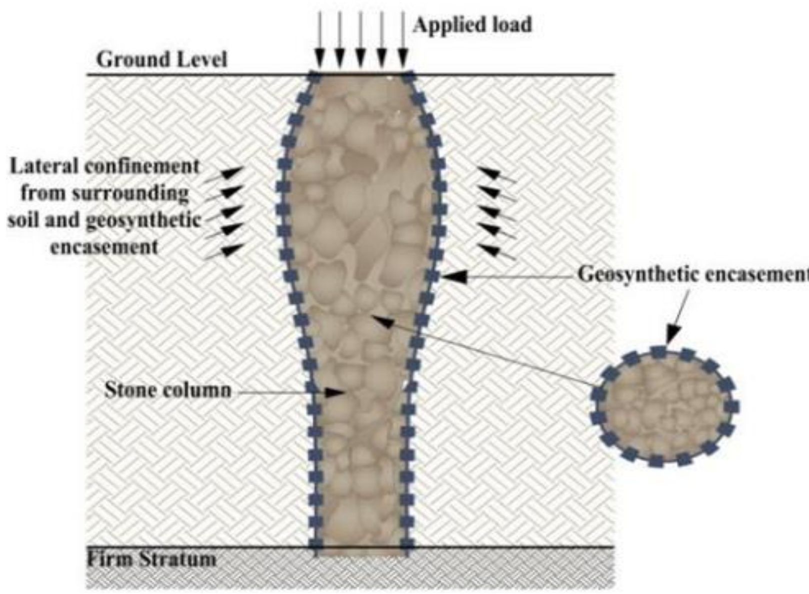


Figure 2-19. encased stone column (Murugesan & Rajagopal, 2009).

Ghionna and Jamiolkowski (1981) and Van Impe and Silence (1986) were the first to recognize that stone columns could be encased by geosynthetics material. They introduced an analytical design technique that was used to assess the required geosynthetic tensile stiffness, and details on this technique were provided by Kempfert et al. (1997). Later, Raithel and Kempfert (2000) proposed an analytical solution for computation of settlement based on the geotextile stiffness and area replacement ratio. This analytical method gives satisfactory estimation of the settlement, vertical stresses on the top of the encased column and soft soil, and geosynthetic hoop force, as well. Alexiew et al. (2005) and Raithel et al. (2005) reported the successful use of ESC in some projects in Europe. Mello et al. (2008) also reported its first use in South America in Sao Jose dos Campos. The general scheme of geosynthetic encased columns supporting road embankment is shown in Figure 2-20 Hosseinpour I (2015).

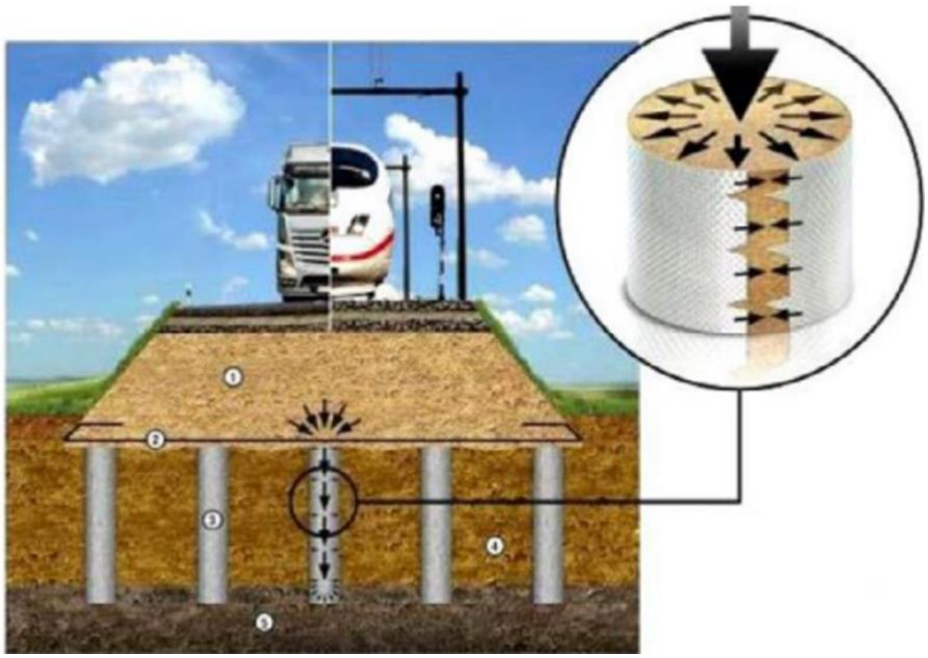


Figure 2-20. Embankment with geosynthetic on GEC (Raithel 1999, 2000 and EBGEO 2011).

2.2.2.3. Installation method of encasing stone column

2.2.2.3.1. Displacement method

Encased stone columns can be executed with or without lateral displacement of the soft soil thus two different methods are generally available with regards to the construction technology. The first technique is the displacement method where a closed-tip steel pipe is driven down into the soft soil followed by the insertion of the circular weave geotextile. The geotextile casing is then filled up with sand or crushed stone aggregate. The tip then opens and the pipe is pulled upwards under optimized vibration designed to compact the column material. The sequence of the displacement method is shown in Figure 2-21. The displacement method is commonly used for extremely soft soils. Encased columns with the displacement method usually have a diameter of approximately 0.80 m and the diameter of the geotextile is ideally equal to the diameter of the internal tube (Alexiew et al. 2005). The column spacing is typically between 1.5 m and 2.5 m and the tensile stiffness modulus of the geotextile (J) generally varies between 1500 kN/m and 4000 kN/m (Kempfert et al. 2002). Figure 2-21 also shows the sequence of encased column installation commonly adopted in Brazil Hosseinpour I (2015).

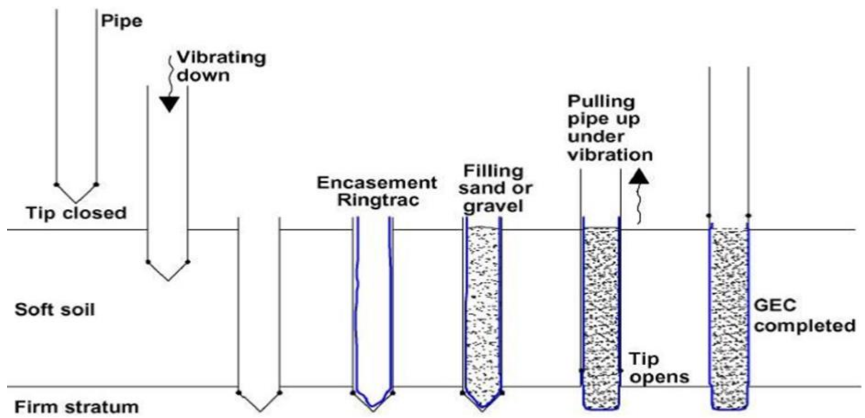


Figure 2-21. Displacement method for GEC installation (Alexiew et al. 2005).

2.2.2.3.2. Replacement method

The replacement method uses an open-bottom PVC/steel pipe (casing) equal in diameter to the GEC. The casing is pushed down to the underlying rigid layer. The soil within the pipe is withdrawn using a helical auger to form a cavity. The geosynthetic is then placed inside the pipe and the cavity is filled with the granular material. Granular column content broadly consists of either sand or gravel. When the cavity is totally filled, the pipe is pulled out. To achieve the desired relative density of the column material, the column material can be compacted by vibrating the pipe as long as the pipe is dragged out. The replacement method installation is shown in Figure 2-22 Alkhorshid, N. R. (2017).

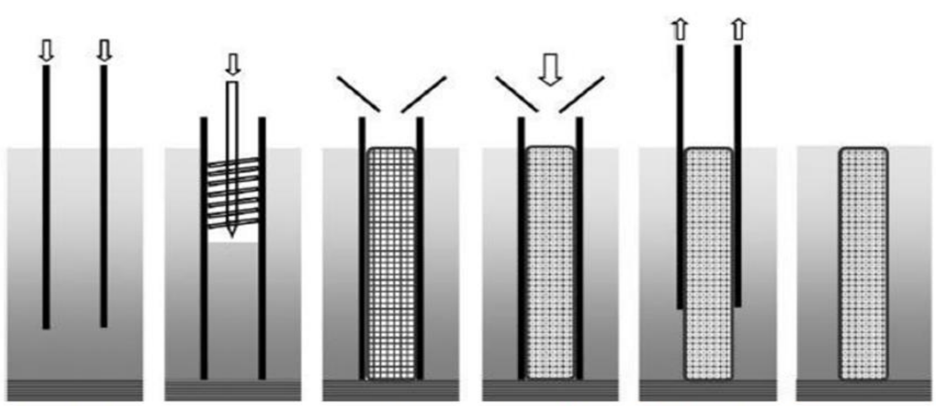


Figure 2-22. Replacement method stages for encased column installation (Gniel and Bouazza, 2010).

2.3. Review of previous researches on encased stone columns

Ancient researchers, especially in the last two decennia have attempted to investigate the behaviour of the soft soils when they are stabilized with single of (ESCs). In general, these studies showed that the encased stone columns (ESCs) showed good behavior, namely a much reduced bulging and a reasonable settlement for an (OSC) so that it is possible to build safe very high embankments. The following sections provide a comprehensive review with emphasis on the most important experimental tests, numerical analyses, analytical methods.

2.3.1. Experimental tests

numerous studies have experimentally studied the effects of geosynthetic reinforcement on the bearing and deformation characteristics of composite soil. Geosynthetic reinforcement was used in two modes: encased element and circular discs placed at regular intervals over a partial or total length of the stone column. Most of these studies were carried out by means of laboratory tests with small-scale modeling of one or more groups of granular or sand columns reinforced by geotextile or geogrid. In some of the experimental surveys, a reinforced stone column was loaded directly on top to represent direct “footing loading”.

Rajagopal et al (1999) studied the influence of geocell confinement on the strength and stiffness behaviour of encased granular soils. A large number of triaxial compression tests were performed on granular soil encased in single and multiple geocells. The different configurations used in tests program and configuration with four interconnected cells are shown in Figures 2-23-a and 2-23-b, respectively. The geocells were fabricated by hand using different woven and nonwoven geotextiles and soft mesh to investigate the effect of the stiffness of the geocell on the overall performance of geocell-soil composite. In general, it was observed that the granular soil develops a large amount of apparent cohesive strength due to the confinement by the geocell. The magnitude of this cohesive strength was observed to be dependent on the properties of the geosynthetic used to fabricate the geocell. The stiffness of the composite was also found to increase with the provision of geocell reinforcement. The results have shown that using three interconnected cells in the testing programme is adequate to simulate the performance of geocell reinforcement layer consisting of many interconnected cells. A simple methodology has been presented in the paper to estimate the magnitude of the apparent cohesive strength developed by the granular soil as a function of the geometric and material properties of the geocell.

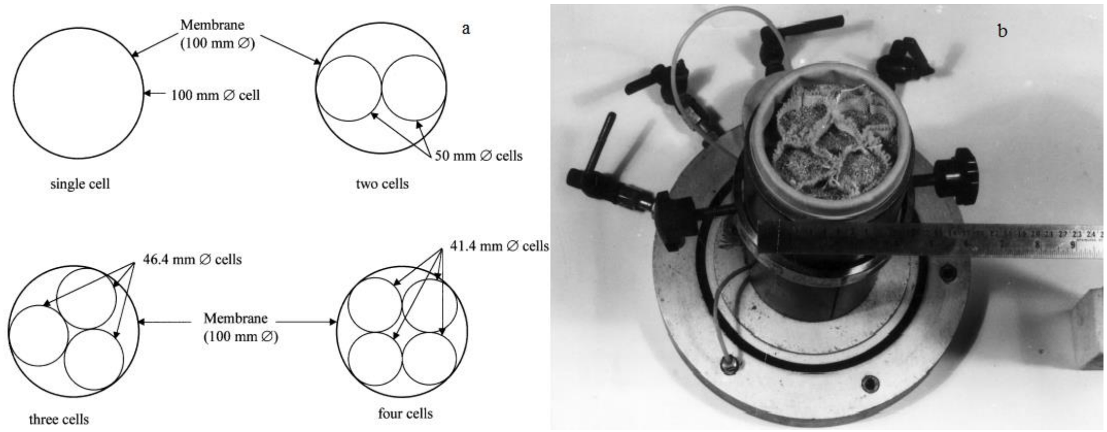


Figure 2-23. (a) Different configurations of cells used in triaxial tests and (b) triaxial test sample with four encasements (Rajagopal et al. 1999).

In general, it was observed that the sand columns showed a large amount of apparent cohesive strength due to the confinement provided by the geocell (Figure 2-24-a). The magnitude of this cohesive strength was observed to be dependent on the properties of the geosynthetic used to fabricate the geocell encasement. Also, encased sand columns showed a higher peak stress compared with un-reinforced sand columns (Figure 2-24-b). In addition to the increase in the strength of sand columns, there was a corresponding increase in the stiffness of the column, which was indicated by steeper stress-strain curves in Figure 2-24-b. Because of the additional confining pressure on the column due to the membrane stresses, the peak stresses occurred at larger strains. This was similar to the unreinforced soils developing peak stress at higher strains at higher confining pressures. It was concluded that the use of three encasement cells in model tests was adequate to represent the stiffness behavior of geocells with many interconnected cells Hosseinpour I (2015).

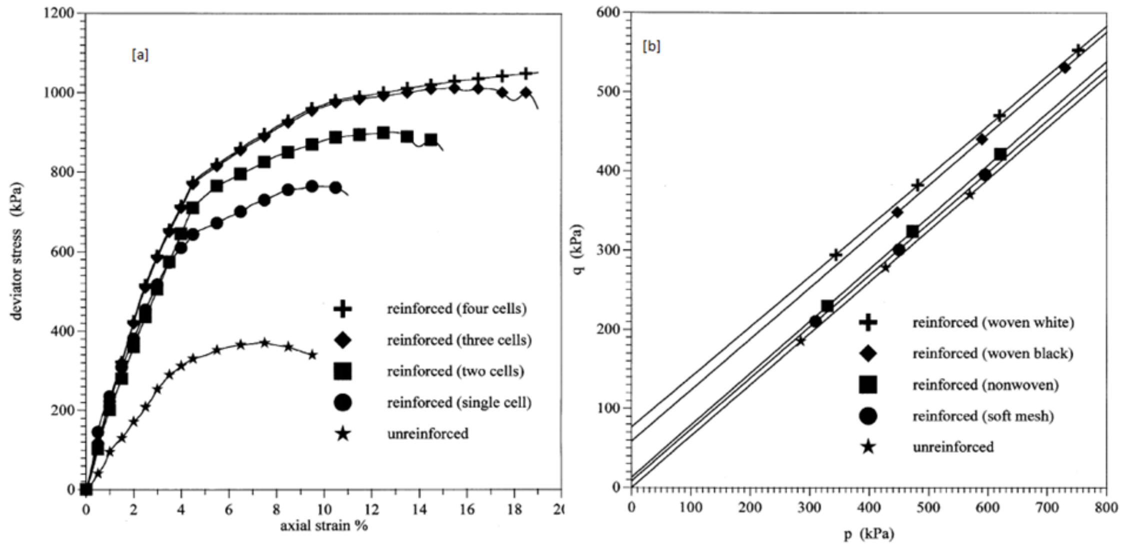


Figure 2-24. (a) stress-strain curves for sand column with different configurations of geocells (b) p-q curves for sand column samples with geocells (Rajagopal et al. 1999).

Ayadat and Hanna (2005) conducted a series of laboratory tests on the geofabric encapsulated stone column to investigate its performance in a collapsible soil. The load carrying capacity and the deformation characteristics of the composite mass were studied. The collapsible fill was kaolin clay which filled in a stress-controlled cylindrical chamber of 390 mm inside diameter, 520 mm depth and 17.5 mm wall thickness. The coarse, uniformly graded sand with the particle size range from 1.18 to 2.36 mm were used as the backfill material of the stone column. The columns formed were 250 mm diameter with 250 mm, 300 mm and 410 mm length. Four non-woven geofabrics were tested in this investigation (Terram 700, 1000, 1500 and 2000). The sand columns were loaded axially using a strain controlled loading system until the failure point. LVDTs were used to measure the settlement of the specimen and then the settlement-load bearing curves were compared. From the investigation, it was found that the geofabric encapsulated sand column has prevented the premature failure of the column in the collapsible soil. The load carrying capacity of the encapsulated sand columns increased with the increase of geofabric material stiffness (igure 2.25). Also, the increase of column rigidity (use of stiffer geosynthetic) and column length increased the load carrying capacity of the collapsible soil Hosseinpour I (2015).

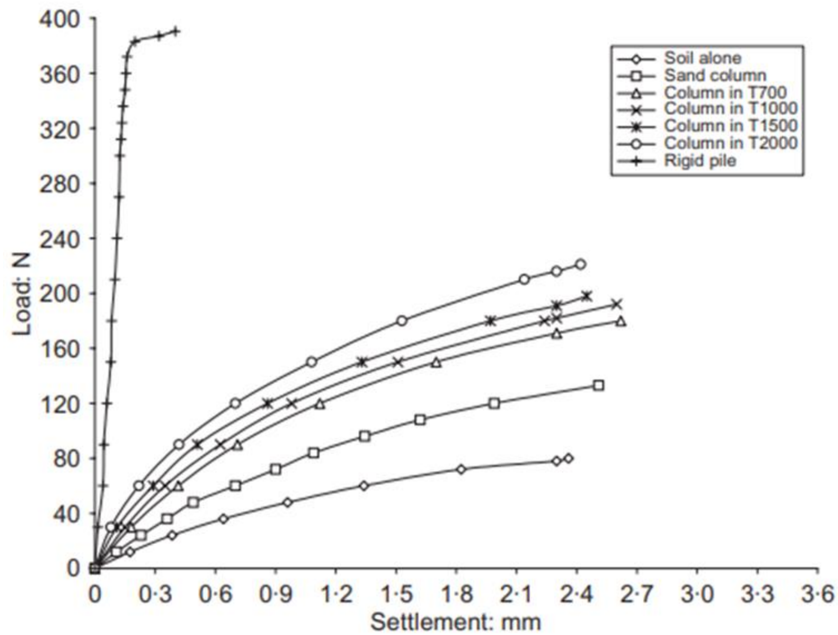


Figure 2-25. Load–settlement curves for various foundation supports (Ayadat and Hanna, 2005).

Black et al (2007) investigates the performance of stone columns in a weak deposit such as peat. It evaluates the effects of reinforcing stone columns by jacketing with a tubular wire mesh and bridging reinforcement with a metal rod and a concrete plug. A series of plate loading tests was conducted on isolated stone columns installed in a soil bed consisting of a peat layer sandwiched between two layers of sand. The work has shown that the settlement characteristics of the soil can be improved by installing stone columns and that a significant enhancement in the load settlement response is achieved when the columns are reinforced by the various methods.

Results of the study are presented in the form of load– displacement graphs. In practice, it is usual to consider a foundation penetration of 6% of the foundation diameter as the failure load for routine purposes in the United Kingdom. Therefore, the results are evaluated at a settlement of approximately 9 mm. The results are also analyzed in terms of initial stiffness, referred to as the modulus of subgrade reaction Ks, which is generally used for settlement predictions.

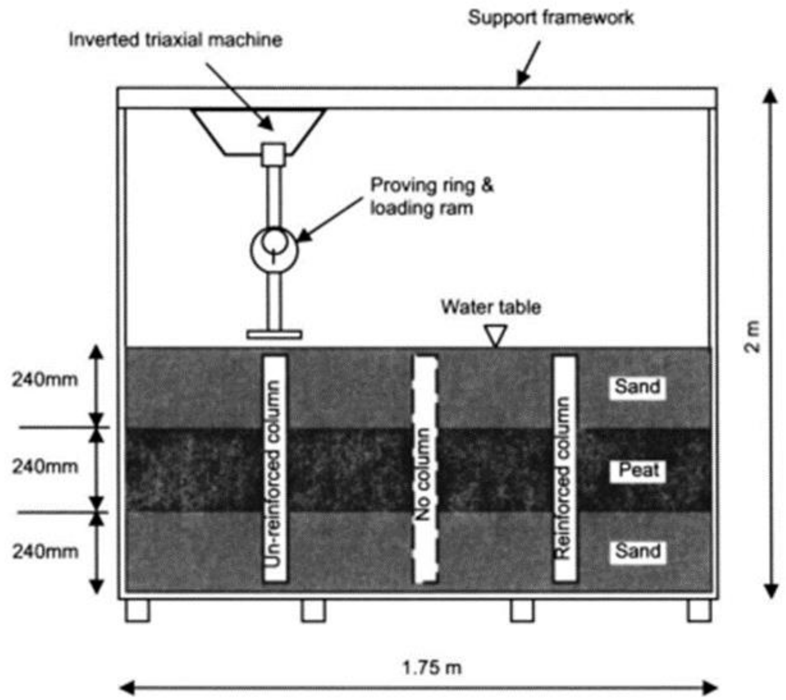


Figure 2-26. Testing box setup (Black et al. 2007).

The load–displacement characteristics of footings supported by stone columns were investigated by applying load to a circular plate supported on: untreated soil; soil treated with stone columns; and soil treated with stone columns encased by wire mesh. The results showed that the settlement characteristics of the soft soil can be improved by installing sand columns and that a significant enhancement in the load-settlement response was achieved when the sand columns were encased by wire mesh (igure 2.27). Furthermore, it was observed that using sand column caused the stiffness of the composite system increased and the stiffness improved significantly as the stone column was reinforced by wire mesh casing (stiffer casing) Hosseinpour I (2015).

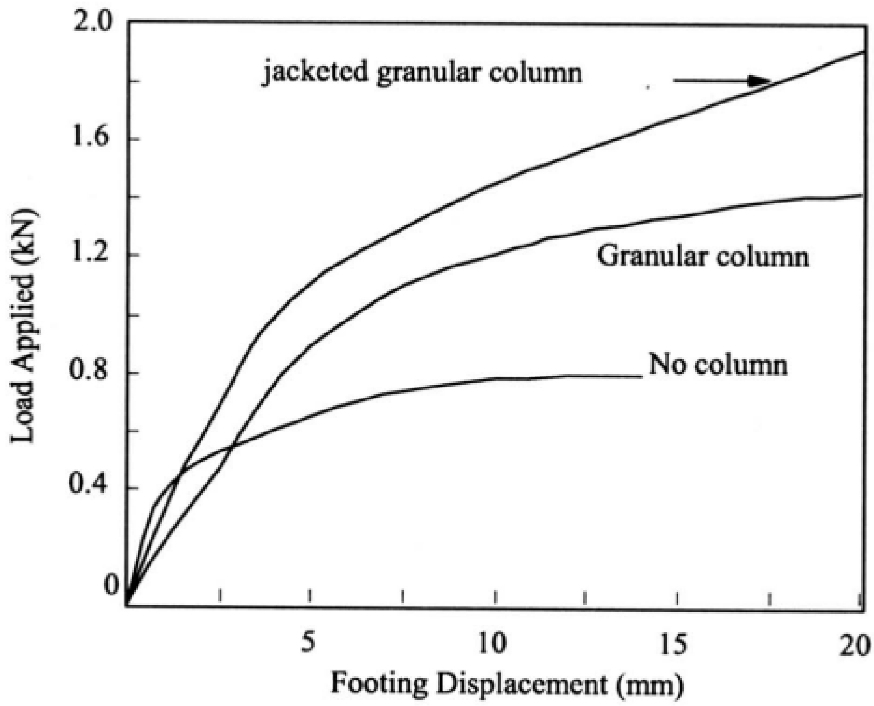


Figure 2-27. Load-settlement curves of treated and untreated ground (Black et al. 2007).

Najjar et al (2010) evaluate the degree of improvement in the mechanical properties of soft clays in practical applications involving the use of sand drains or sand columns in clayey soils. For this purpose, 32 isotropically consolidated undrained triaxial tests were performed on normally consolidated kaolin specimens. The parameters that were varied were the diameter of the sand columns (de), the height of the columns (Hc), the type of columns geotextile encased versus non-encased, and the effective confining pressure. Test results indicated that sand columns improved the undrained strength significantly even for area replacement ratios that were less than 18%. The increase in undrained strength was accompanied by a decrease in pore pressure generation during shear and an increase in Young’s modulus. The drained shear strength parameters were found to be relatively unaffected by the sand column reinforcement, except for fully penetrating columns with high area replacement ratios (Ar).

Figure 2-28. Installation of encased sand (Najjar et al. 2010).

It was observed that the encased sand-columns resulted in substantially higher undrained strengths for the composite mass, when compared to the effect of non-encased columns. For fully penetrating columns and for area replacement ratios of 7.9% and 17.8%, the increase in undrained shear strength over the unreinforced clay ranged from 29 to 61% and from 88 to 100%, respectively. These increases were substantial given the relatively small area ratios used (Figure 2-29). The concept of the “critical column length” established for non-encased sand columns at about six column diameters, beyond which strength gain becomes negligible, appeared to be invalid for encased columns with area replacement ratios of 7.9%. It was also found that the degree of improvement in the undrained shear strength for clays reinforced with encased sand columns appeared to decrease at higher effective confining pressures Hosseinpour I (2015).

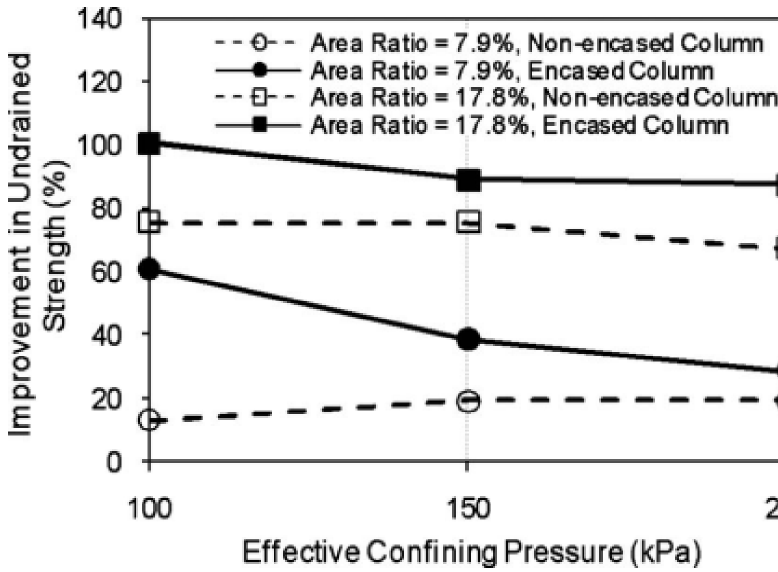


Figure 2-29. Variation of improvement of undrained shear strength with pressure (Najjar et al. 2010).

Yoo and Lee (2012) presented the results of an investigation on improvement in load-carrying capacity and settlement reduction of a GESC using field-scale load tests. Also, the effect of the geogrid encasement length and column strain is investigated. In addition, isolated GESC behaviour was compared to rammed-aggregate pier (RAP) and conventional stone column (CSC) behaviour. A geogrid-encased stone column was constructed using crushed stones classified as GP which the minimum and maximum grain size were 1 to 25 mm, GESC stages installation see (Figures 2-30). The results show that additional confinement provided by the geogrid encasement increased the stiffness of the stone column and reduced the settlement of the soft ground. Also, bulging of the GESC was observed to occur directly beneath the base of the geogrid encasement. The improvement in the performance of GESC was found to be significant, even with partial encasement.

The results showed that using the GESC system in soft soils, the lateral bulging is considerably decreased due primarily to the added confinement by the geogrid encasement, thus improving its load carrying effect by reducing settlement and preventing rapid column failure (Figure 2-31). In the case of GEC, maximum deflection within the geogrid-encased region was around 5 mm, indicating a reduction of approximately 3 times of the lateral deflection compared with the conventional stone column (CSC). To optimize the reinforcement effect of the geogrid, it was recommended that the column be encased to at least 4D from the top thus covering the region where bulging failure may occur. It was also observed that geogrid hoop strain reached its maximum value within a depth of 1D from the top of the encased column, and decreased at greater depth. By measuring hoop strain in these tests, it was seen that the critical encasement length of geogrid was 2 to 3D.

Figure 2-30. GESC installation: (a) auger is used to remove soil; (b) insertion of aggregate by funnel; (c) compaction of aggregate; (d) geogrid sleeve is placed; (e) insertion of aggregate into geogrid sleeve; (f) GESC installation is completed (Yoo and Lee, 2012).

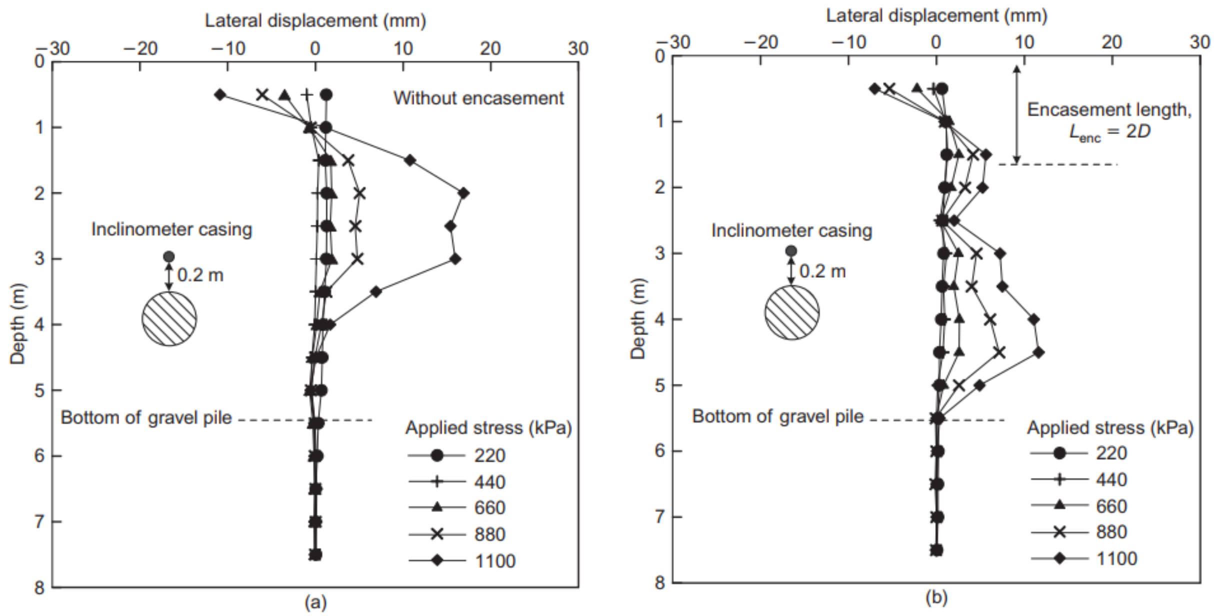


Figure 2-31. Lateral deflection at Gimhae site: (a) conventional stone column; (b) GESC (Yoo and Lee, 2012).

Alkhorshid et al (2019) carried out a series of laboratory of the behavior of geotextile encased and conventional granular columns placed in very soft soils was evaluated in this study through load capacity tests conducted on large-scale laboratory model columns (Figure 2-32). where used different types of encasement (three woven geotextiles with different values of tensile stiffness) and different column fill materials (sand, gravel and recycled construction and demolition waste, RCDW). The results of load capacity tests conducted on large-scale models constructed to simulate the different types of GECs indicate that the displacement method adopted during column installation can lead to an enhanced shear strength in the smear zone that develops within the very soft soil. In addition, breakage of the column fill material was found to affect the load-settlement response of gravel and RCDW columns. Furthermore, the excess pore water pressure generated in the surrounding soil during installation, was found to remain limited to radial distances smaller than three times the GEC diameter.

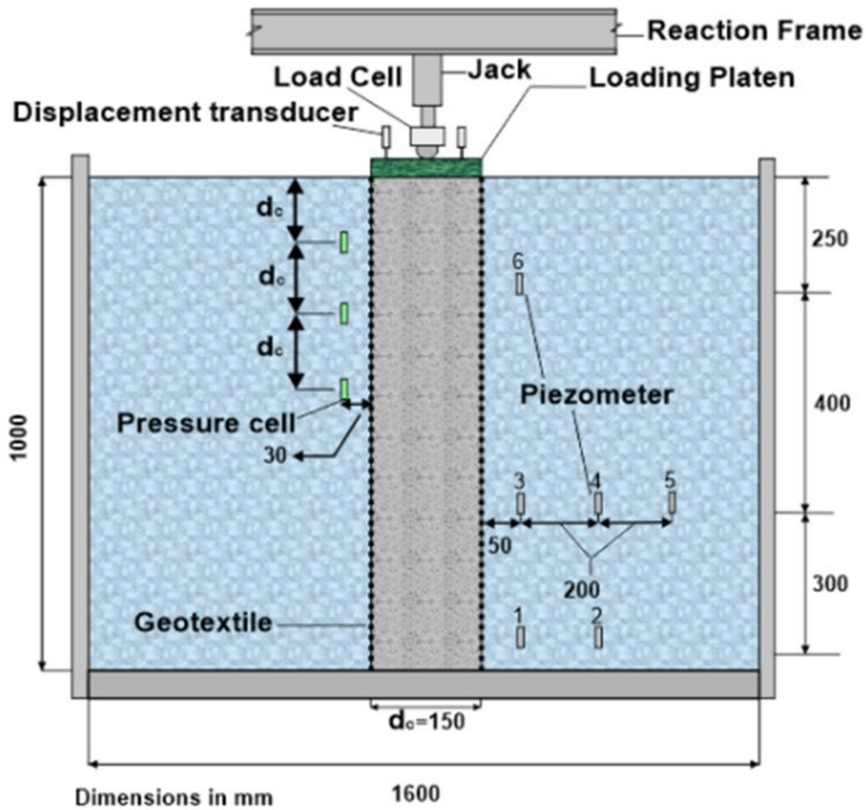


Figure 2-32. Schematic of the load test on ESC. (Alkhorshid et al. 2019).

2.3.2. Numerical tools applied to ESC

Many successful numerical studies of encased granular columns are available in the literature improved soft soils. They can reasonably simulate the interaction mechanisms between the soft soil and geosynthetic by adopting the stress-strain coupled formulation. The numerical analysis, allows a more fundamental understanding of GEC behaviour by supporting parametric studies to investigate the influence of the input parameters which were mostly verified with experimental investigations. Several two and three dimensional finite element analyses were performed to study the influence of the critical parameters such as area replacement ratio (Ar), soft clay thickness, embankment height, Encasement stiffness (J), Encasement length, Friction Angle of Stone-Column Materials.

Murugesan and Rajagopal (2006) investigated the qualitative and quantitative improvement in load capacity of the stone column by encasement through a comprehensive parametric study using the finite element (FE) analysis. It is found from the analyses that the encased stone columns have much higher load carrying capacities and undergo lesser compressions and lesser lateral bulging as compared to conventional stone columns. The results have shown that the lateral confining stresses developed in the stone columns are higher with encasement. The encasement at the top portion of the stone column up to twice the diameter of the column is found to be adequate in improving its load carrying capacity. As the stiffness of the encasement increases, the lateral stresses transferred to the surrounding soil are found to decrease. This phenomenon makes the load capacity of encased columns less dependent on the strength of the surrounding soil as compared to the ordinary stone columns.

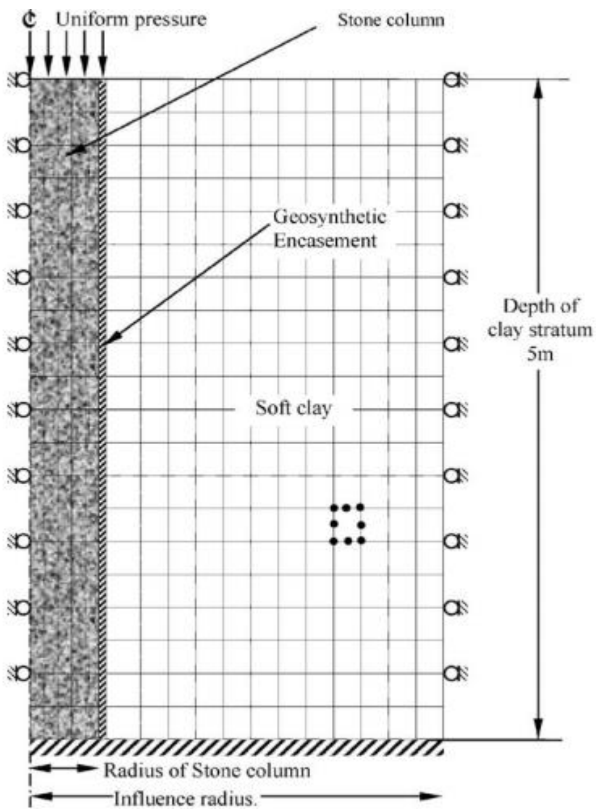


Figure 2-33. Typical finite element mesh used in the analyses (Murugesan and Rajagopal, 2006).

The improvement in the performance of the stone column due to encasement was studied by applying pressure only over the stone column area. By encasing, it was found that the stone columns were confined and the severe lateral bulging has significantly reduced. The lateral bulging observed in the stone columns of two sizes (0.6 and 1 m diameters) with and without encasement was compared (Figure 2-34-a). It was observed that in ordinary stone columns (OSCs), there was severe bulging near the ground surface up to a depth equal to twice the diameter of the stone column. On the other hand, the encased stone columns were undergone much lesser lateral expansion near the ground surface. The encased columns were undergone slightly higher lateral expansions at deeper depths as compared to the OSCs. This could have happened because the applied surface load was transmitted deeper into the column due to encasement effects.

concerning to lateral confining stresses mobilized along the column, it was observed that the lateral stresses were higher in the encased column as compared to the corresponding lateral stresses in OSCs (Figure 2-34-b). The increase in confining pressure was seen over the full height of the stone column, which led to mobilization of higher vertical load capacity in the encased columns. The lateral stresses mobilized in the OSCs without geosynthetic encasement were found to be the same for both diameters of the stone columns (0.6 and 1 m). On the other hand, the lateral stresses mobilized in encased stone columns were higher for smaller diameter columns.

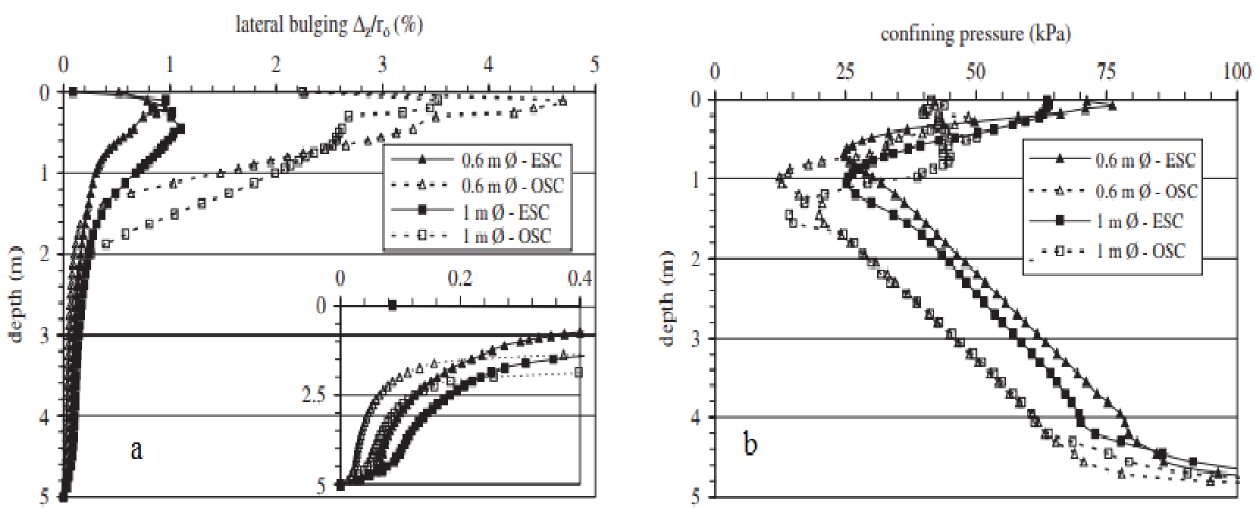


Figure 2-34. (a) Lateral bulging observed in stone columns and (b) confining pressure along the column length (Murugesan and Rajagopal, 2006).

Ambily and Gandhi (2006) studied the actual stress intensity on the stone column and soil using Finite Element Analysis (FEA) (Figure 2-35). Sand pad is provided at the surface to drainage and the impact of thickness of sand pad on load sharing between stone column and soil is analyzed by the analysis for both rigid and flexible loading condition.

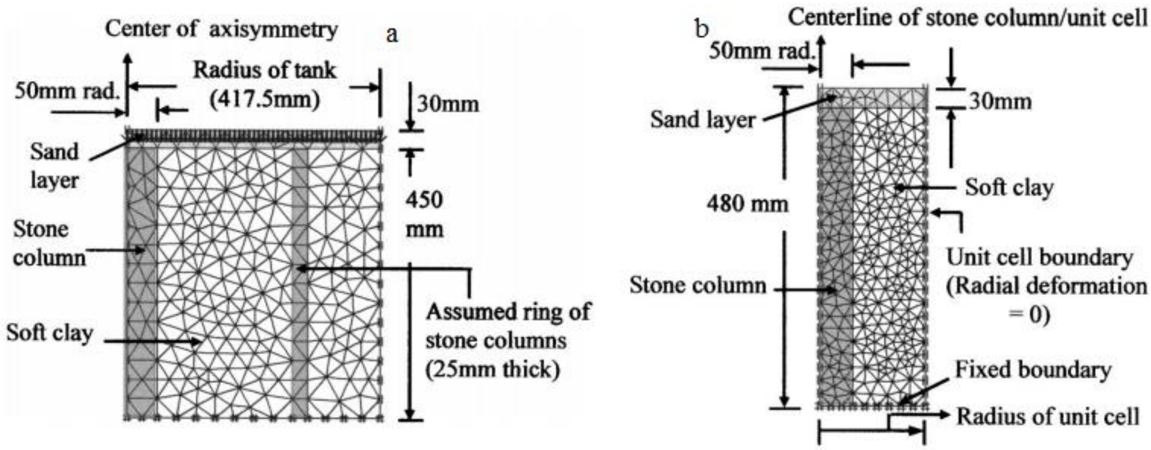


Figure 2-35. (a) Finite-element discretization for group test (b) Finite-element discretization for single column, Ambily and Gandhi (2006).

Khabbazian et al. (2010) carried out three-dimensional finite element analyses to simulate the behavior of a single granular column with and without encasement in very soft clay using the computer program ABAQUS. Comprehensive numerical analyses were performed to study the influence of the geosynthetic stiffness, friction and dilation angle of the column material, length of geosynthetic encasement, diameter of the column, length of the column, and the coefficient of in situ lateral earth pressure. The lateral extent of the soft soil around the column was selected such that the numerical model results were not affected by the imposed conditions along the circumferential boundary of the soft soil (Figure 2-36). Model results show that the stress–settlement behaviour of granular columns can be significantly improved by encasing them. The stiffness of the encasement was found to have a major effect on the stress–settlement response of encased columns and their associated load-carrying capacity. For partially encased columns, the optimum length of encasement was found to be a function of the stress that is applied to the column.

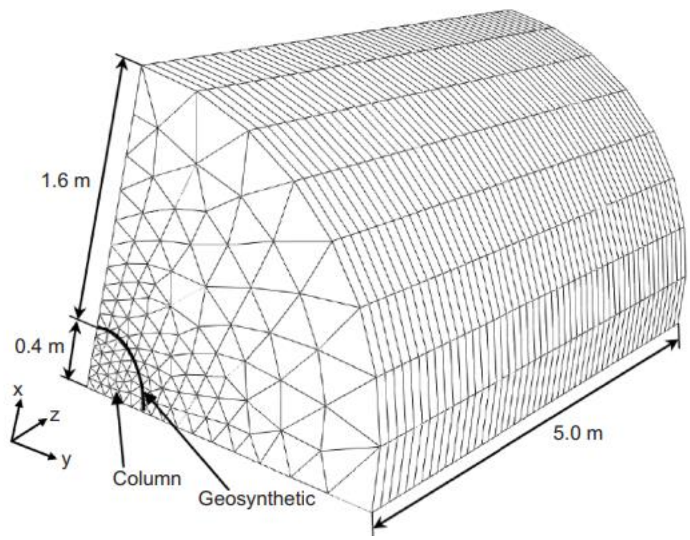


Figure 2-36. Typical finite element mesh used in the analyses (Khabbazian et al. 2010).

Results showed that the stress–settlement response of granular columns significantly improved by encasing them. The stiffness of the encasement was found to have a major effect on the stress–settlement response of encased columns and their associated load carrying capacity (Figure 2-37-a). The maximum value of lateral displacement of a GEC was much less than that of a conventional granular column for the same vertical settlement (Figure 2-37-b). This was due to the fact that the increased stiffness of a GEC allowed larger loads to be transmitted to greater depths, which in turn caused the lateral displacements to be more evenly distributed over the length of the column than what was observed in a granular column.

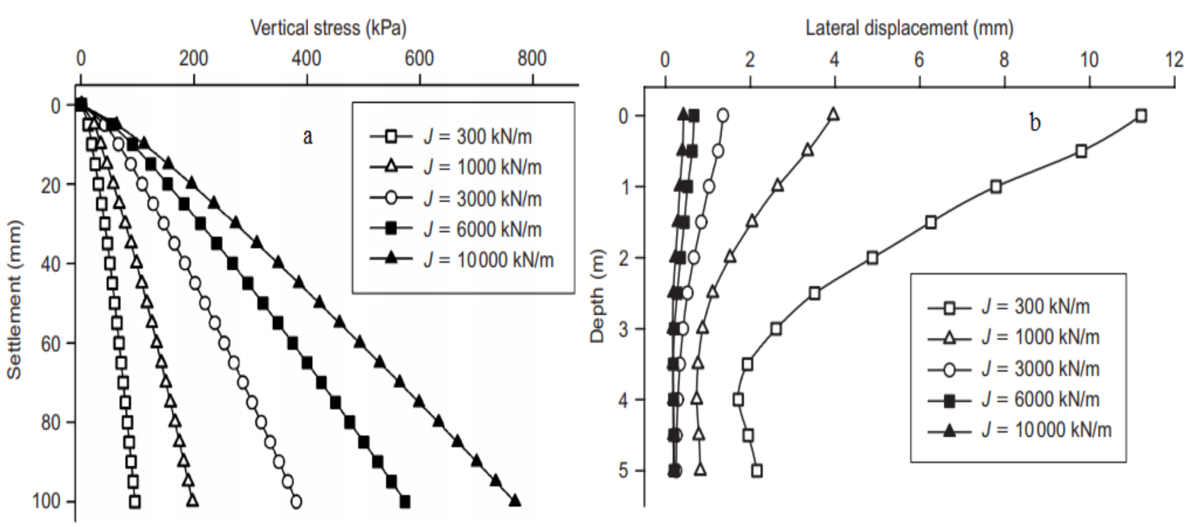


Figure 2-37. (a) Effect of encasement stiffness on the stress–settlement behavior of a ESC and (b) influence of encasement stiffness on column bulging (Khabbazian et al. 2010).

Results indicated that improving the strength characteristics of granular column materials (friction and dilation angle) increased the load-carrying capacity of a given column. However, in many cases, it was more efficient to select encasement with a higher stiffness. Decreasing the diameter of an encased granular column improved its stress–settlement response. It was also observed that lateral displacements increased with the diameter of column (Figure 2-38).

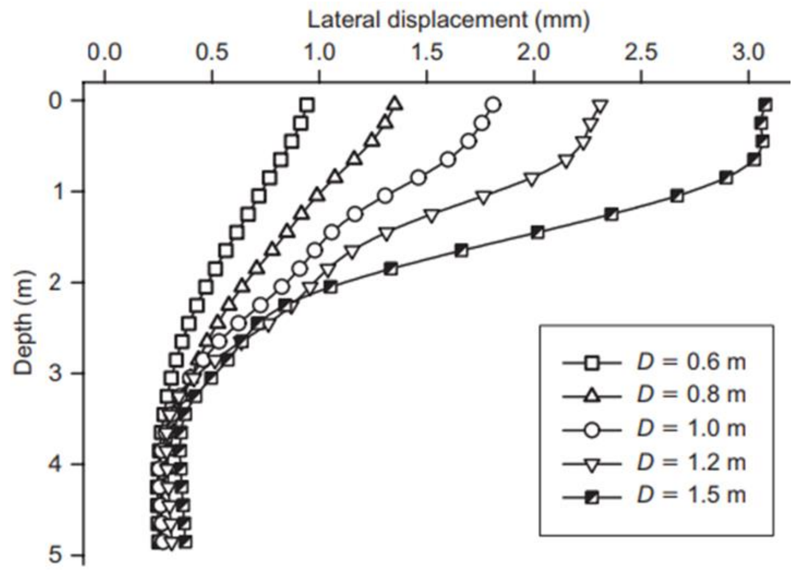


Figure 2-38. Lateral displacement vs. depth for a GEC with varying column diameter (Khabbazian et al. 2010).

Elsawy (2013) studied embankment was constructed to a height of 5.0 m in two 2.5 m layers over a period of 21 days. Construction over Bremerhaven clay using full scale unreinforced and reinforced with ordinary and geogrid-encased granular columns by means of numerical analyses using PLAXIS software. The consolidation behavior of this system was investigated to study the improvement in the reinforced soil during and after consolidation. The development of stress concentrations in conventional and encased columns during the consolidation process, and its role in reducing total settlement, were also studied. Furthermore, the influence of stress concentrations in conventional and encased stone columns on the consolidation process was investigated. The conventional and encased stone columns had a diameter (de) of 1.0 m and a spacing/diameter ratio (S/d) of 3.0. The stone columns were installed in a square pattern, which produced an equivalent unit cell with a diameter 3.39 m (Figure 2-39).

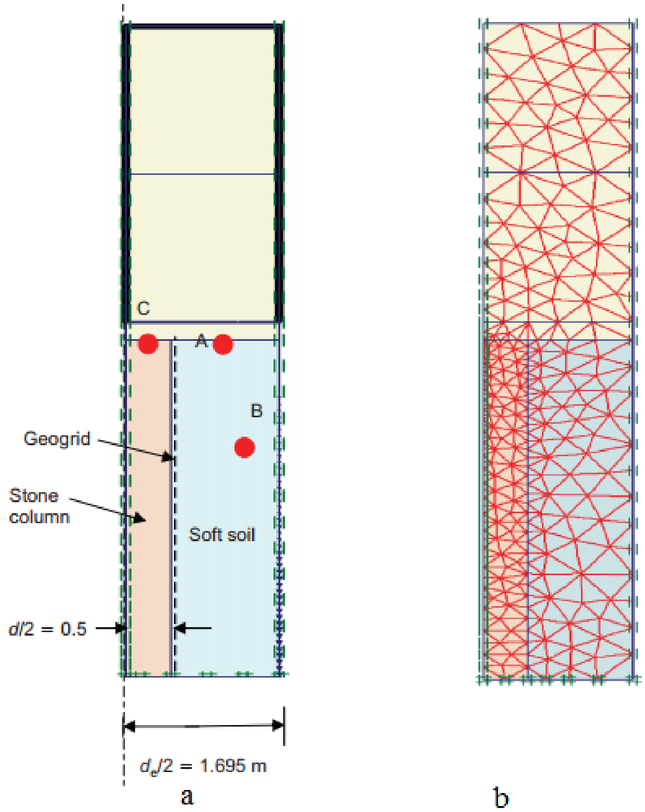


Figure 2-39. (a) Model geometry and (b) Mesh generation of GEC unit cell (Elsawy 2013).

The granular columns were simulated with a diameter of 1.0 m and length of 6.0 m. The results for long-term consolidation analyses indicated that granular column increases bearing capacity of the clay and accelerate the dissipation of excess pore water pressure. As expected, once the granular column is encased (ESCs), relevant improvement occurs in the performance of granular column. Results of FE analysis showed that the soft soil reinforced with encased stone columns had a smaller settlement and a shorter consolidation time than the soft soil reinforced with conventional stone columns (CSCs). The reduction in settlement was more remarkable with increasing consolidation time, and with increasing embankment load (Figure 2-40).

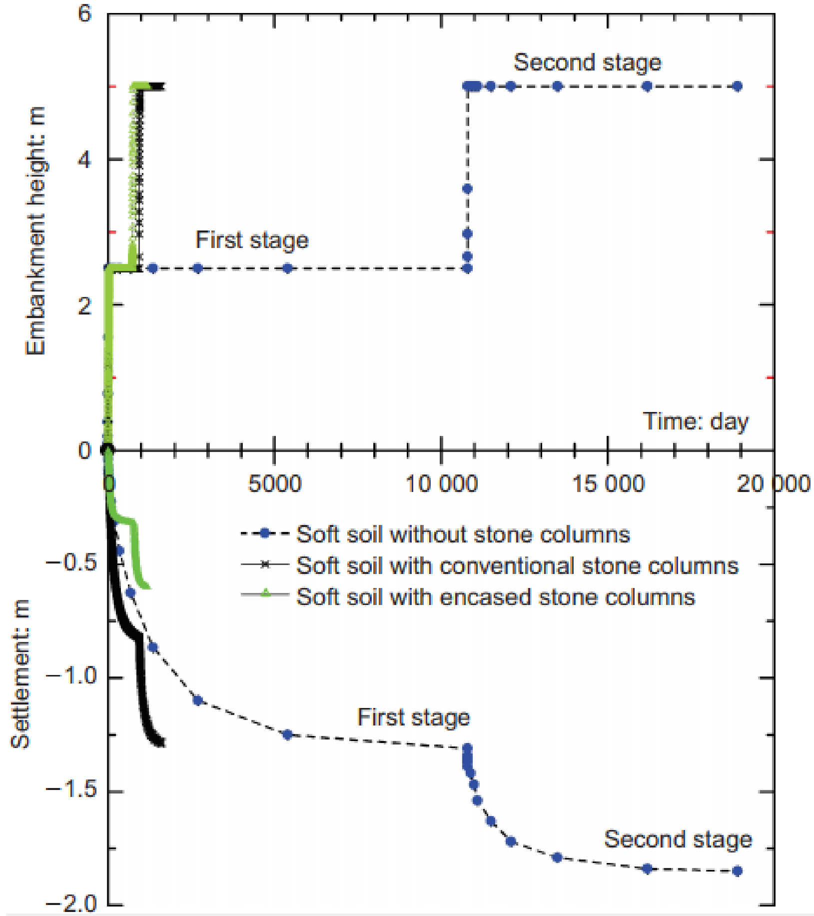


Figure 2-40. Settlement of unreinforced and reinforced soft soil with conventional and encased columns at point A (Elsawy 2013).

Zhang and Zhao (2014) presented an analytical solution based on the unit-cell concept, to predict deformation behaviors of geotextile-encased stone columns at any depth below the top plane of the columns (Figure 2-41). Under vertical loads at the tops of the stone columns, an axial compression deformation occurred that was accompanied by a lateral expansion near the top. This deformation characteristic of stone columns was incorporated directly into the proposed analytical method. The shear stress between the encased stone column and the surrounding soil in the vertical direction also was taken into account. In this method, the confining pressure provided by the soil was analyzed based on an analogy with passive earth pressure. The method was verified via comparison with two other analytical solutions. Where parametric studies were conducted to investigate the effects of geotextile encasement, vertical applied stress, and column spacing and diameter on the deformation behaviors of columns.

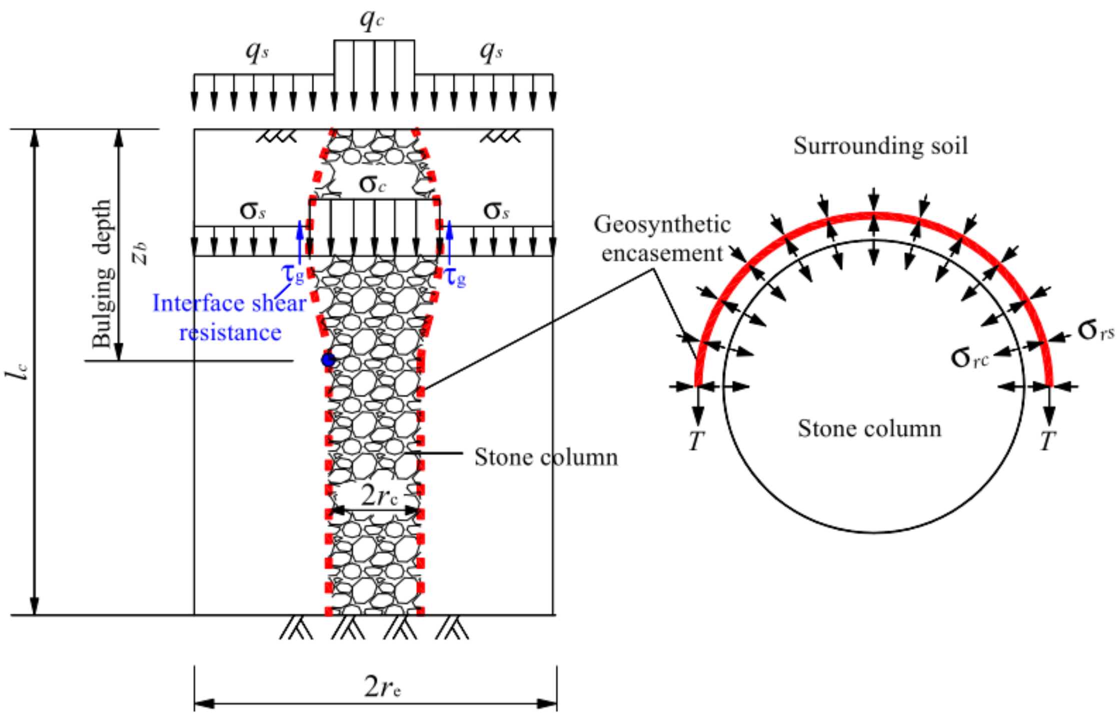


Figure 2-41. Calculation model of geotextile-encased column (Zhang and Zhao 2014).

To investigate the influence of column spacing (S), column diameter (de) and geotextile encasement (J) was varied from 0 to 2,000 to 4,000 kN = m. on deformation behaviors of geotextile encased stone columns (GESCs), a series of parametric analyses was performed. As compared with non-encased stone columns, where column bulging was decreased significantly because of the additional lateral confinement from the geotextile encasement of the column, which implied that geotextile encased stone columns were better supported laterally than non-encased stone columns and therefore can provide more bearing capacity (Figure 2-42-a). geotextile encasement had a reduction effect on the settlement of stone columns, and this reduction was more effective for encasements with higher stiffness values than for encasements with lower stiffness valued (Figure 2-42-b). Column spacing (S) and column diameter (de) were also found to have a dominating effect on settlement reduction. Increasing column diameter and decreasing the spacing between them, and thereby increasing the area replacement ratio (Ar), caused to a significant reduction in settlement. Hence the selection of encasement stiffness for the encased stone column should be made based on column diameter and column spacing.

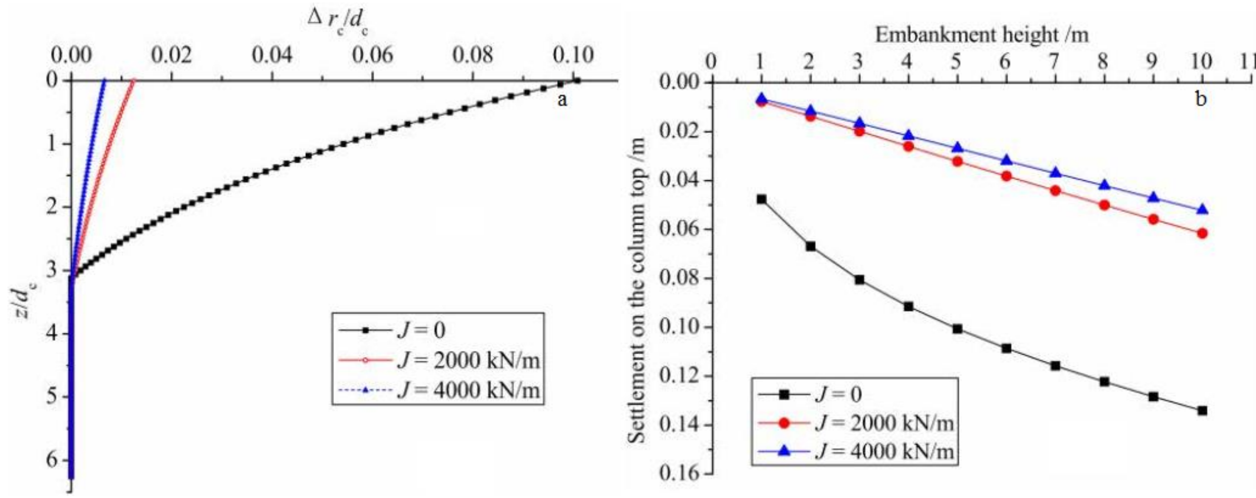


Figure 2-42. (a) Bulging depths of stone column and (b) Settlement at top of stone column (Zhang and Zhao 2014).

Chen et al (2015) presented numerical simulations and laboratory tests of an embankment reinforced with geosynthetic-encased stone column (GECs) (Figure 2-43, 2-44) The results of the study showed that the encased stone column failure was caused by the columns bending. The stability of the embankment was evaluated by 2D and 3D simulations. Based on the obtained results, they came to a conclusion that 3D simulations provided closer estimations to the laboratory tests than 2D simulations, which agrees with the bending failure mechanism of the GECs. It is suggested that one more row of such columns may be required to provide higher lateral resistance in the soils in front of the toe to improve the stability of the embankment.

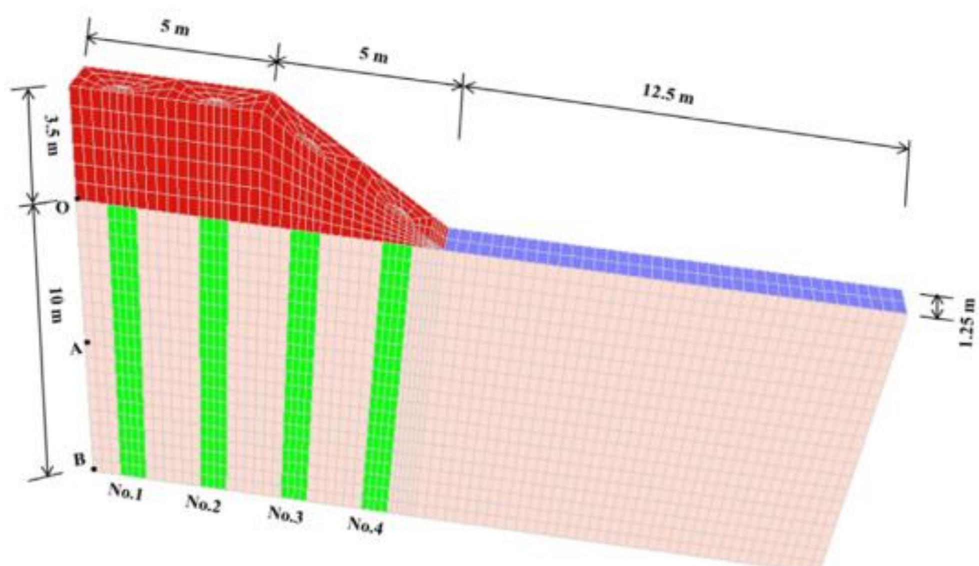


Figure 2-43. The numerical model of the GECs-supported embankment (Chen et al. 2015).

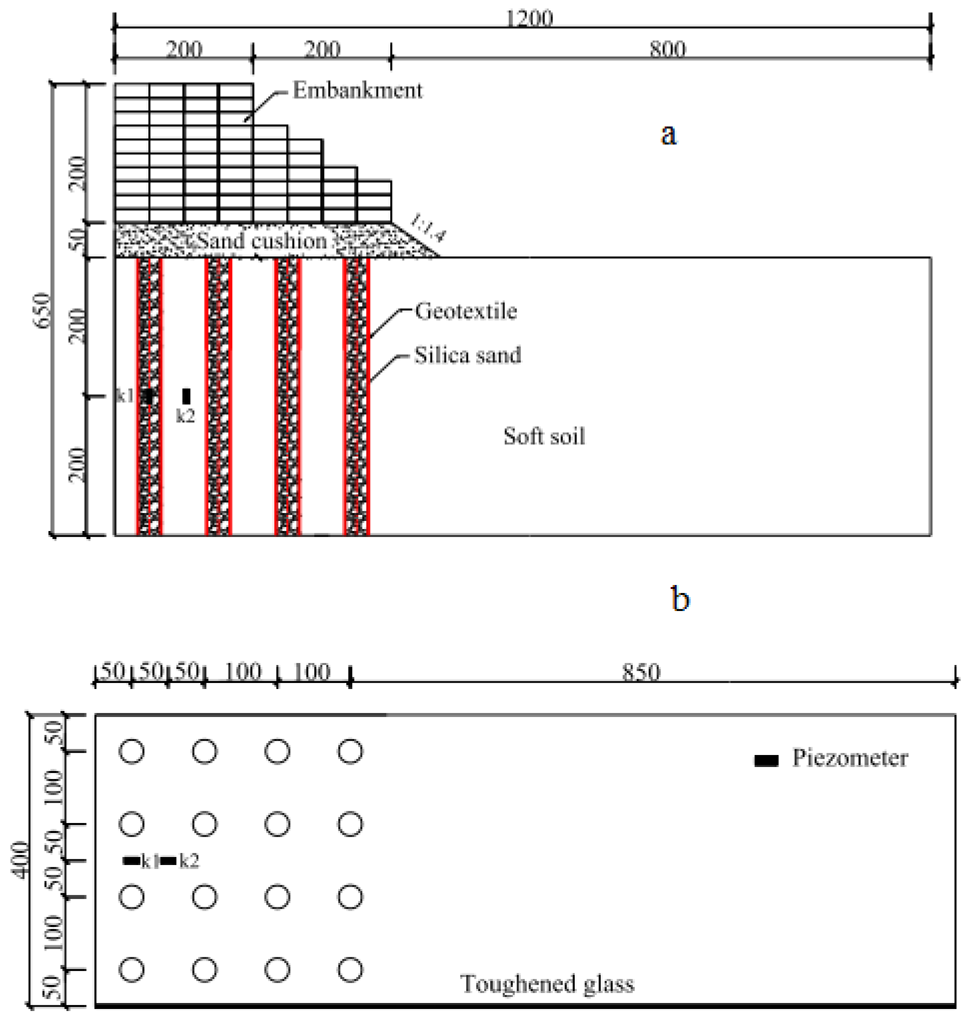


Figure 2-44. Dimensions of the laboratory model embankment on GECs reinforced soft soils (units are in mm): (a) section view; (b) plan view (Chen et al. 2015).

2.3.3. Analytical methods

For the case of stone columns used in very soft soils (such as sabkha soils), in the last years it has developed the encased stone column technique (ESC). For the design of this improvement technique, various analytical methods have been developed recently that will be mentioned in this section. Regarding analytical and semi-analytical solutions, two methods were introduced until the present time as named "Belgian Method", proposed by Van Impe (1986) and the "German Method" proposed by Raithel and Kempfert (2000). The confinement provided by geosynthetic in the Geosynthetic Encased Columns (GEC) is considerably greater than the confinement provided by the surrounding soil and consequently the (GEC) supports a greater load than the ordinary stone columns (OSCs).

Raithel and Kempfert (2000) or in other words (German method) developed a numerical and an analytical calculation model for the design of the geotextile coated sand columns foundation system (GEC) design development considers the theory of elasticity and identical settlements for both granular column and surrounding soil, this method predicts the behavior of unit cell for long-period drained condition when maximum value of bulging and settlement are obtained. Generally, an analytical, axial symmetric model is used for calculating and designing a geotextile encased column foundation. Raithel and Kempfert (2000) presented a closed form analytical solution to obtain stresses and deformations on column (system material of column and geosynthetic) and soil (normally soft or very soft soil). As illustrated in Figure 2-45, there is an equilibrium between the loading on unit cell ($\Delta\sigma 0$) and vertical stresses shared by the column ($\Delta\sigma v,c$) and surrounding soil ($\Delta\sigma v,s$). In this method, the hoop tensile force can be calculated by:

$$\mathrm{Fr} \ = \mathrm{J}.\frac{\Delta \mathrm{r_{geo}}}{\mathrm{r_{geo}}} \tag{2.4}$$

where, (Δrgeo) and (rgeo) are lateral bulging and initial radius of geotextile, respectively.

The model was developed based on the conventional calculation models used for granular columns, which are completed by the effect of geotextile encasement and uses an iterative process, by means of the Equations 2.5, 2.6 and 2.7.

$$\left\{\frac{\Delta\sigma_{v,s}}{E_{oed,s}} - \frac{2}{E^*}.\frac{V_s}{1-V_s}\left[K_{a,c}.\left(\frac{1}{a_E}.\Delta\sigma - \frac{1-a_E}{a_E}.\Delta\sigma_{v,s} + \sigma_{v,0,s}\right) - K_{0,s}.\Delta\sigma_{v,s} - K_{0,s}.\sigma_{v,0,s} + \frac{(r_{geo}-r_c).J}{{r_{geo}}^2} - \frac{\Delta r_{c.}}{{r_{geo}}^2}.J\right]\right\}.h = \left[1 - \frac{r_c^2}{(r_c+\Delta r_c)^2}\right].h \tag{2.5}$$

Where the horizontal deformation of the column can be determined through:

$$\Delta r_c = \frac{K_{a,c}.\left(\frac{1}{a_E}.\Delta\sigma - \frac{1-a_E}{a_E}.\Delta\sigma_{v,s} + \sigma_{v,0,s}\right) - K_{0,s}.\Delta\sigma_{v,s} - K_{0,s}.\sigma_{v,0,s} + \frac{(r_{geo}-r_c).J}{{r_{geo}}^2}}{\frac{E^*}{\left(\frac{1}{a_E-1}\right).r_c} + \frac{J}{{r_{geo}}^2}} \tag{2.6}$$

$$E^* = \left(\frac{1}{1-V_s} + \frac{1}{1-V_s}.\frac{1}{a_E}\right)\frac{(1+V_s).(1-2V_s)}{(1-V_s)}.E_{oed,s} \tag{2.7}$$

Where:

$\Delta\sigma 0$: Applied stress at the top of unit cell

$\Delta\sigma v,c$: Increase of vertical stress on stone column

$\Delta\sigma v,c$: Increase of vertical stress on stone column

Ka, c: Coefficient of active earth pressure of column

h: Column length

This equation can be solved by an iteration process. The oedometric modulus (Eoed,s) of the soil should be introduced stress dependent. More details are shown in Raithel and Kempfert (2000).

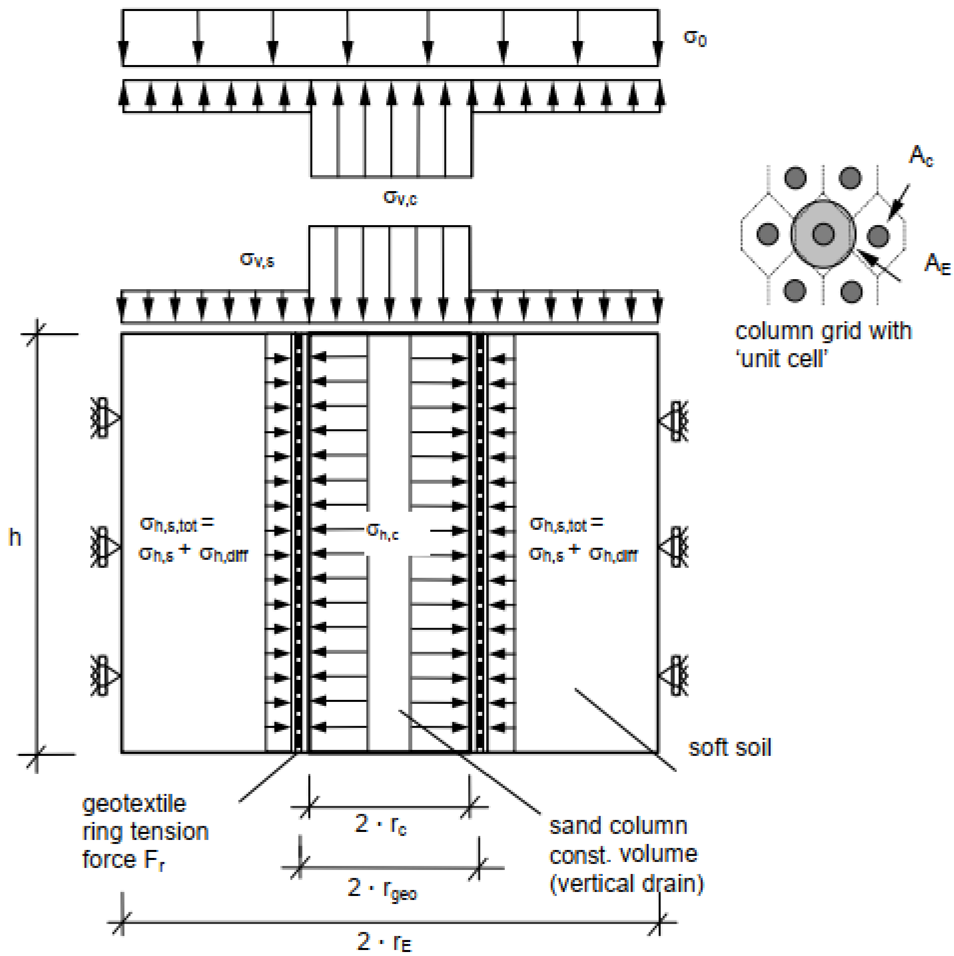


Figure 2-45. Analytical model for Geotextile Encased Columns, simplified picture after (Raithel & Kempfert 2000).

Where the results showed that Raithel and Kempfert (2000) get large settlements in the geotextile when using analytical model compared to the numerical analyses, especially directly after loading, as illustrated in Figure. 2-46-a. It can also be shown that the hoop tension forces (FR) and the settlement efinitely depend on the stiffness of the geotextile and the area ratio (Ar) of the column grid, as shown in Figure. 2-46-b.

We conclude from the study of Raithel and Kempfert (2000) for calculation and design of the foundation system ‘geotextile coated sand columns’ was reflected. The most important advantage of the new foundation system is the possibility to use this system in very soft soils like peat or sabkha soils, because the radial support is guaranteed through the composite between the coating and the surrounding soil. On soft organic soils underlain by bearing layers in reachable depths the new foundation system provides the possibility to build safe and flexible foundations with low settlements due to enormous settlement reduction, acceleration of settlements and increase of the shear strength, especially during the rainy seasons.

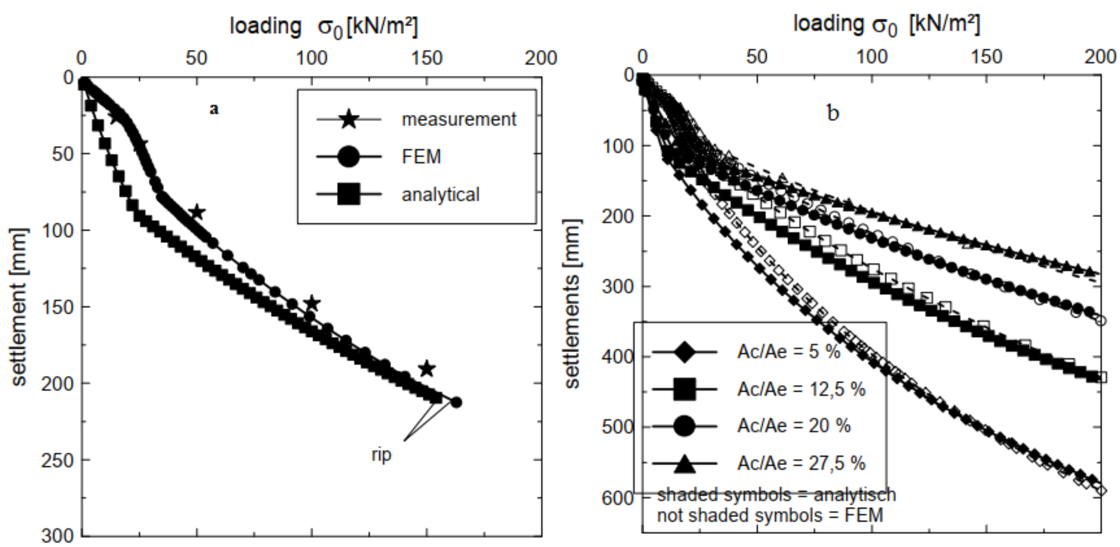


Figure 2-46. Load-settlement curves of (a) Comparative calculation-large scale model test and, (b) parametric study: variation of area ratio Ac/Ae for J = 1000 kN/m (Raithel and Kempfert, 2000).

Pulko et al. (2010) presented design method for non-encased and encased stone columns. where the developed analytical closed-form solution is based on previous solutions, initially developed for non-encased columns and for non-dilating rigid plastic column material. In the present method, the initial stresses in the soil/column were taken into account, with the column considered as an elasto-plastic material with constant dilatancy, the soil as an elastic material and the geosynthetic encasement as a linear-elastic material. To check the validity of the assumptions and the ability of the method to give reasonable predictions of settlements, stresses and encasement forces, comparative elasto plastic finite element analyses have been performed. The agreement between the two methods is very good, which was the reason that the new method was used to generate a parametric study in order to investigate various parameters, such as soil/column parameters, replacement ratio (Ar), load level and geosynthetic encasement stiffness (J) on the behaviour of the improved ground. Where the results showed the influence of key parameters and provide a basis for the rational predictions of settlement response for various encasement stiffnesses, column arrangements and load levels. The practical use of the method is illustrated through the design chart, which enables preliminary selection of column spacing (S) and encasement stiffness (J) to achieve the desired settlement reduction for the selected set of the soil/column parameters.

If stone columns are regularly distributed, a regularly shaped area around the stone-column may be considered as a “unit cell”, consisting of stone-column and the surrounding soft soil in a zone of influence (Figure 2-47).

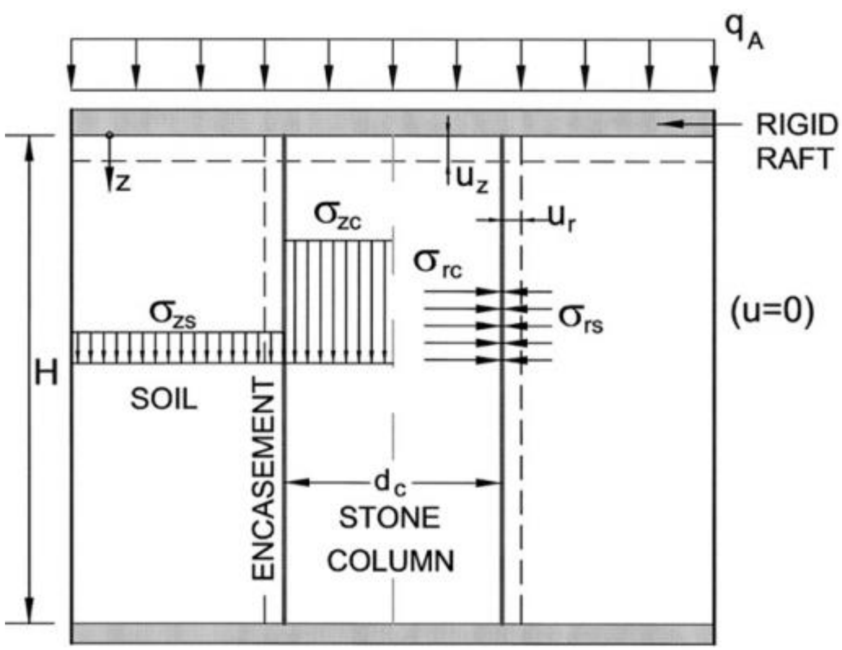


Figure 2-47. Basic features of the model based on regular patterns of stone columns (Pulko et al. 2010).

Castro and Sagaseta (2011) developed an analytical method based on the unit cell concept. that is, the final bearing column and the surrounding soil, are modeled in axial symmetry under a uniform rigid load. The loose soil is treated as elastic material and column as elastic plastic material using Mohr Coulomb crop standard and unbound flow base, with fixed expansion angle. The behavior of flexible packaging plastics is also seen by means of limited tensile strength. The solution is presented in closed form and can be used directly in a spreadsheet. Parametric studies to reduce stability, stress concentration and consolidation time show the efficiency of the column encapsulation, which is mainly governed by the casing stiffness compared to the soil hardness. Column encapsulation is equally beneficial for joint area replacement ratios but columns with smaller diameters have better enclosure. Moreover, the applied load must be limited to prevent the casing from reaching the tensile strength limit. A simplified formulation of the solution has been developed assuming a state of depletion. The results are consistent with the numerical analyzes.

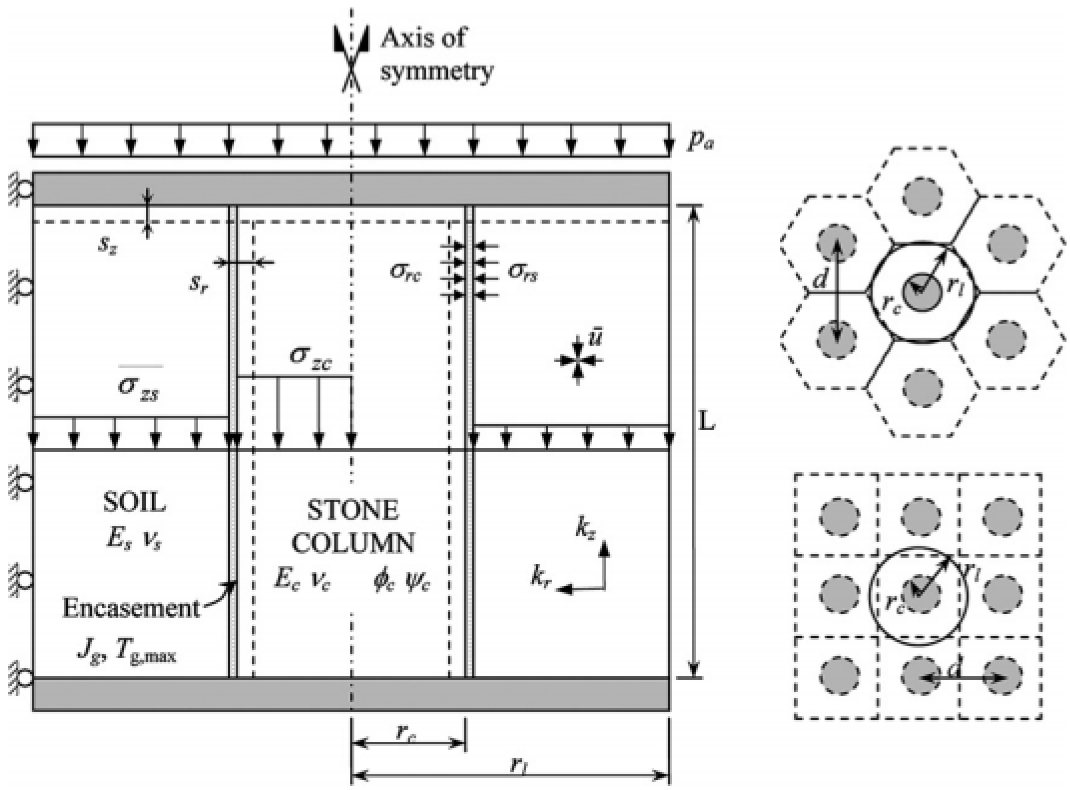


Figure 2-48. unit cell (Castro and Sagaseta 2011).

Yang Zhou and Gangqiang Kong (2019), this study presents an analytical solution for predicting the vertical settlement behavior of geosynthetic-encased stone columns (GESCs) and ESC-supported embankments, taking into account the lateral deformation of GESC. The analysis is performed based on the deformation pattern of a GESCs-reinforced foundation. The solution is obtained by enforcing affinity between the vertical settlements of the column and the soil for each element of the GESC-reinforced foundation. This method has been verified via a comparison with data from the literature. Parametric studies were conducted to investigate the influence of embankment fill height (He), dilatancy angle (ψ) and encasement stiffness (J), on the behaviors of the GESC. Theoretical studies for Yang Zhou and Gangqiang Kong (2019) confirmed the influence of the bulging mechanism of GESCs on the settlement of GESC-reinforced foundations were the depth of radial bulging of GESCs is up to three times the diameter of the columns, the results indicate that the theoretical approaches proposed by Yang Zhou and Gangqiang Kong (2019) are suitable for predicting the deformation of GESC-supported embankments.

The unit cell concept can be separated from that of an embankment supported by a GESC-reinforced foundation Figure 2-49.

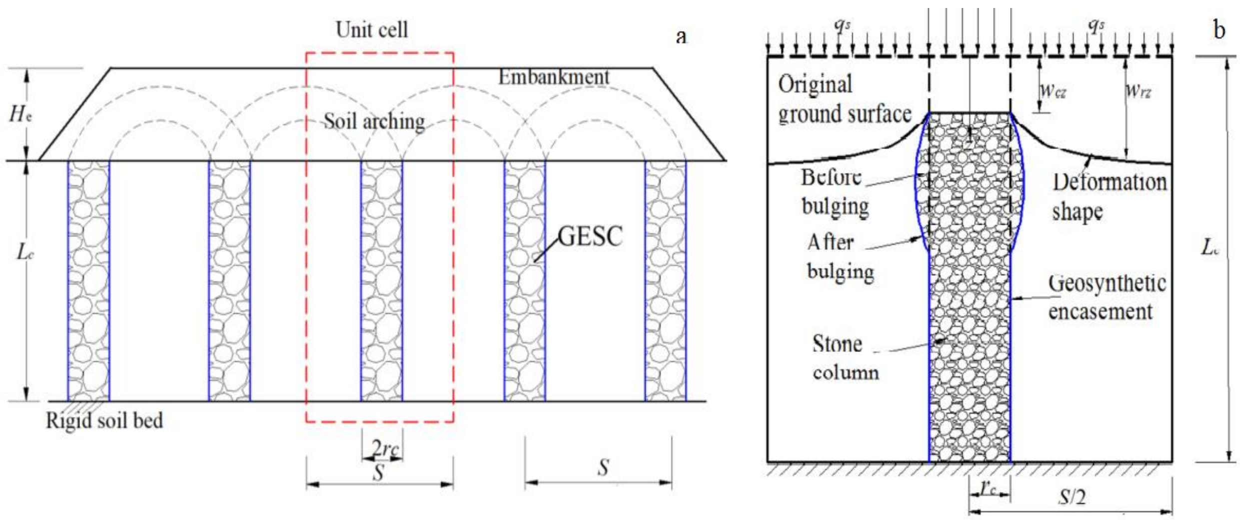


Figure 2-49. Diagram of GESC-reinforced foundation: (a) GESC-supported embankment; and (b) assumed unit cell for vertical deformation analysis (Yang Zhou and Gangqiang Kong. 2019).

Additional studies on this topic include those of Malarvizhi and Ilamparthi (2008), Wu et al. (2009), Murugesan and Rajagopal (2010), Zhang et al. (2011, Yoo (2015), discuss the various analytical approaches for the design of encased stone column, However, additional studies are necessary.

2.3.4. Conclusion

We have exposed in this chapter the techniques most used in practice for soil improvement, reinforcement with geosynthetic layers represents an economically and technically interesting alternative.

Among all these methods, the use of a geosynthetic proves to be a cost-effective alternative solution in terms of saving natural resources, time and integrating sustainability and environmental protection. Geosynthetics perform various functions, namely, filter, separation, drainage, waterproofing, protection, and reinforcement. It can be seen that geosynthetics can significantly increase the safety factor and the height of the embankment. There is also an increase in performance due to uniform settlements after embankment construction and reduced displacement during construction which reduces the amount of material.

Through literary study (Experimental, Numerical, Analytical methods), geosynthetic - reinforced stone columns have proven to be an ideal solution to improve the performance of columns in soft soils especially sabkha soil. The columns may be encased with geosynthetics which are the main materials used to increase the strength and stability of geotechnical structures.

Second Part: Numerical Modeling

Chapter 3.

Numerical modeling of encased stone columns supporting embankments on sabkha soils

3.1. Introduction

The present study investigates the behavior of geosynthetic-encased stone columns supporting embankment on locally weak zones by using numerical modelling. The investigation work is concerned with the construction of road embankments on a specific soil called Sabkha in Algeria. This soil is not only soft and very humid during the flooding seasons but also has frequent small areas of very soft soil which was called earlier by Benmebarek et al. (2015), locally weak zones (LWZ). LWZ is characterized by low strength and high compressibility.

This study is presented in the following sequences. First, the adopted model (FEM) is verified by analytical methods Raithel et al (2000), Pulko et al (2011), and then the results are compared with an already published numerical study. All the relating to the geometry and geotechnical characteristics, are of course, identical to the studies of comparison. Subsequently, the discussion and validation of our numerical model, the improvement of the embankment response through the use of encasement was investigated. Then an intense parametric study is carried out to determine the sensitivity of the targeted results (i.e. lateral deformation of the column and vertical settlement) with regard to the variation of the principal parameters, namely, the height of the embankment, the rigidity of the geosynthetic, the length of the envelope, the area replacement ratio (ARR), the thickness of the Sabkha layer, and the angle of friction of the granular material constituting the stone column. All the results are discussed as the study is progressing and finally, a conclusion was drawn at the end of the study. Figure 3.1 shows a flowchart of the research methodology.

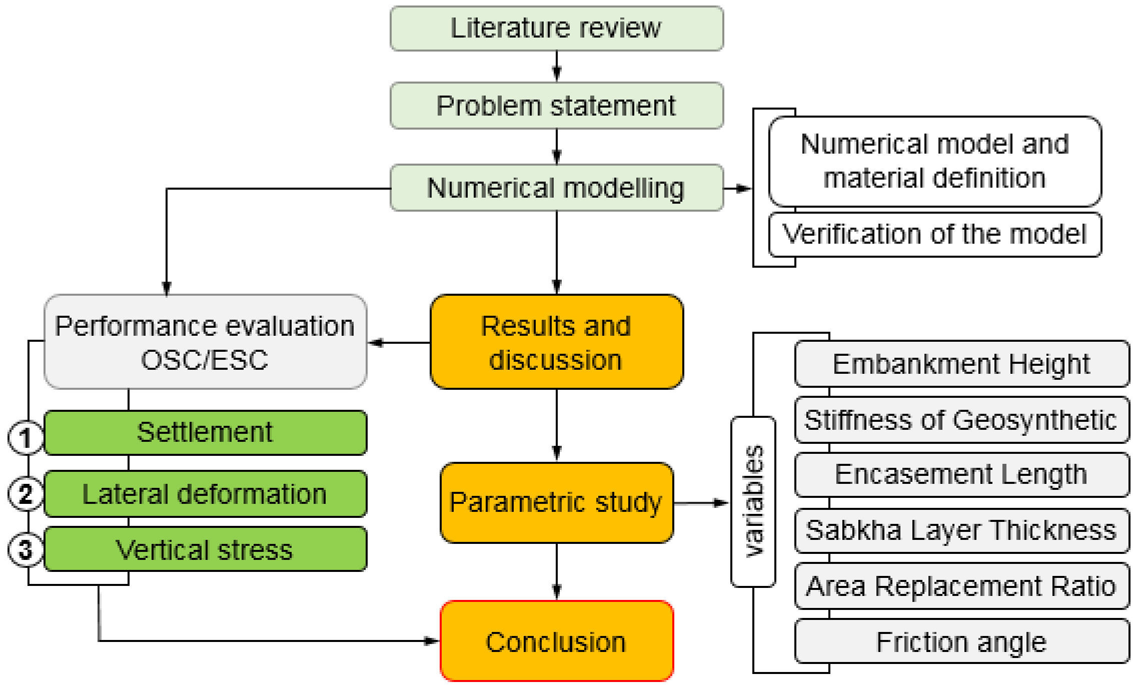


Figure 3-1. Research methodology flowchart.

3.2. Behavior models and the numerical tool used

The analysis of geotechnical projects is possible thanks to numerous finite element codes. The engineer with experience in this field knows that the weight of the assumptions allowing the passage from reality to the model is difficult to assess. He knows that the jargon finite elements is sometimes off-putting - he would like not to have to intervene on the numbering of the nodes, of the elements, on certain choices reserved for the numerical. He would like to have the code on the PC managing his daily office and technique, in order to make a parametric study of delicate problems. Above all, he demands that his days should not be cluttered with laborious data entry and file interpretations. Designed by numerical geotechnicians, the Plaxis finite element code certainly represents a current optimum in scientific and practical terms in 2D pseudo-static analysis. Scientifically, it is a non-linear analysis tool in non-standard elastoplasticity (5 parameters), with taking into account pore pressures (and even linear consolidation), endowed with robust, proven resolution methods and algorithms, as well as automatic selection procedures avoiding delicate

choices for the operator with little knowledge. Although very reliable numerically, the code uses high-precision elements (triangles at 15 knots), as well as recent resolution control processes (arc length method). From a practical point of view, the tree menu system on the screen makes it flexible and pleasant to use, because the operator does not burden the mind excessively. As the use of textbooks becomes rare, they are of reduced volumes, easy to consult. All the simplified options (initialization of the constraints, pore pressures) make it possible to get to the point (predict the behavior of a structure), even if it means carrying out later, with the same code and the same data, a refined calculation.

Plaxis program integrates many foundational models adaptable to a large number of materials, we can cite the flexible model, the Mohr Coulomb model, the model of broken rocks, the soil model with hardening (HS), the model of soft soils (SSM), the model of soft soil with creep. The program allows the user to enter a new code of conduct,

3.2.1. Definition of models used in this study

3.2.1.1. The Mohr Coulomb model (M-C)

This popular model is generally used as a first approximation of soil behavior. It has five coefficients: Young's modulus E, Poisson's ratio ν, cohesion C, angle of friction φ, and expansion angle Ψ.

3.2.1.2. The Hardening Soil Model (HS)

The HSM model aims to improve the Mohr Coulomb model on various points, it is essentially:

- to take into account the evolution of the deformation modulus when the stress increases: the oedometric curves in stress-deformation are not straight lines;
- Distinguish between charge and discharge;
- to take into account the nonlinear evolution of the modulus when the shear modulus increases: the modulus E50 is not realistic: there is a curvature of the stress-strain curves before reaching plasticity.

3.2.1.3. The Soft Soil Model (SSM)

This model (abbreviated SSM) is a model derived from Cam-Clay. Historically, the Cam Clay model was developed in Cambridge in the 1960s by Roscoe, Schoffield et al. The basic idea of this model is to take into account the effect of stress stiffness induced by medium stress on the clay. Under the influence of medium pressure, the water content decreases and the clay becomes more resistant. It is a flexible plastic model with a loading surface. Below the bearing surface, the material remains elastic, whereas if the point representing the effective stress condition reaches the bearing surface, the plastic deformations appear with irreversible behavior. The linked surface of plasticity limits the distance between acceptable and unacceptable states.

Concerning the constitutive models in this study, the soft clay was simulated using the Hardening Soil (HS) model, which is stress-dependent. An elastoplastic Mohr–Coulomb model was adopted for both the granular column and the embankment material, and locally weak zone behavior was represented by the soft soil model (SSM). The behavior of the geosynthetic was simulated using line elements with two translational degrees of freedom at each node. Geosynthetic can sustain only tensile forces and be modelled as a linear elastic material with tensile stiffness J. The geosynthetic encasement used in this study was geotextile type branded as Ringtrac. Ringtrac is a registered trademark of HUESKER Synthetic GmbH.

3.3. Numerical modeling

3.3.1. Presentation of the finite element model and material parameters

In order to simulate the unit cell, an axisymmetric model was undertaken using the finite element code PLAXIS 2D V2017 program available commercially to analyse deformation and stability for a variety of geotechnical problems. In this numerical analysis, a very fine mesh was used because stresses and displacements are very high in this problem. The problem of using a stone column to support a large embankment over locally weak zone (Sabkha soil) was studied.

Appropriate choices of material properties are necessary to have an accurate simulation of the reinforcement system in numerical modelling. The properties of the embankment, soft clay,

stone columns can be found in the literature (Alkhorshid et al 2018). The columns were installed in a square grid with spacing, s = 2.5 m supporting an embankment of 10 m high (Hemb).

The thickness of the clay soil and the length of the stone column are assumed to be 10 m underlain by a rigid, hard stratum. the radius of the column within the unit cell rc, was equal to 0.4 m, radius of the influence area of the column re = 1.4 m, area replacement ratio (Arr = r^2c/r^2e) equal to 8.16 %, and geosynthetic tensile stiffness (J) equal to 2000 kN/m. The dimensions and properties of the locally weak zone (Sabkha soil) were chosen to match values stated by (Benmebarek et al 2015) as B = 0.6 m and DEP = 3 m respectively width and depth of the LWZ as shown schematically in figure 3.2c. The groundwater level is assumed at the ground surface. The vertical or horizontal displacements were restrained at the bottom boundaries of the unit cell, but vertical displacements were allowed at the lateral borders. Table 3.1 shows the parameters used in the FEM.

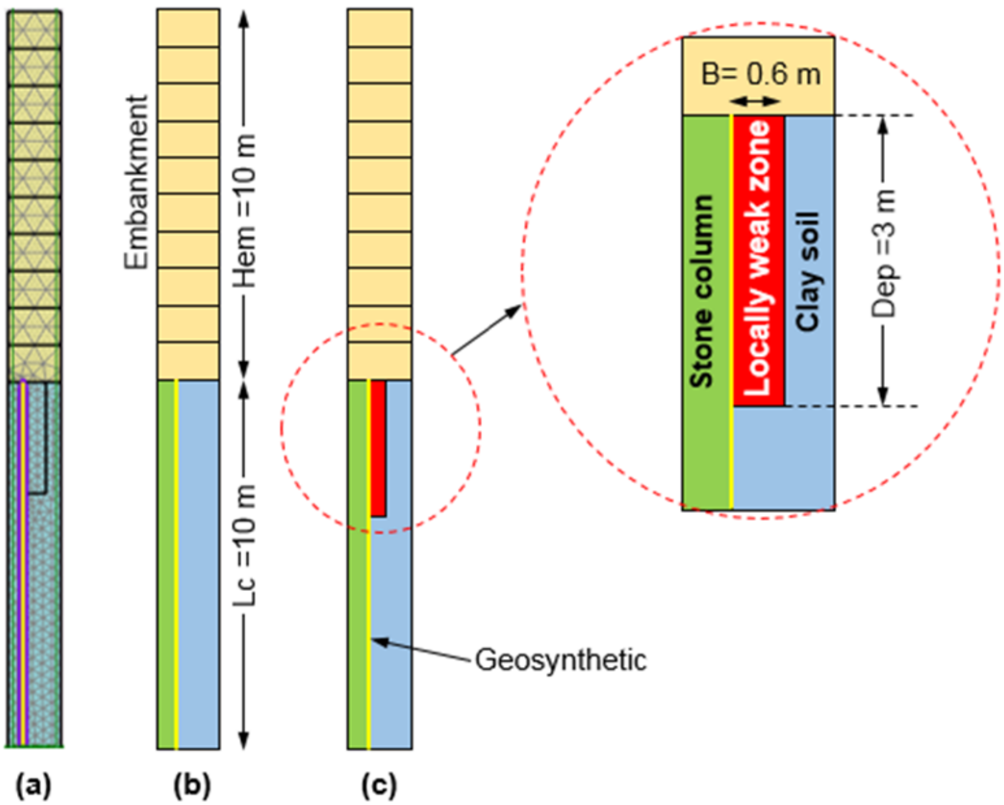


Figure 3-2. Finite-element axisymmetric simulation of the geosynthetic-encased column in the unit cell concept. (a) Boundary condition and finite-element mesh, (b) Scheme of ESC adopted in numerical analyses without the locally weak zone, (c) Scheme of ESC adopted.

Table 3-1. Parameters of materials used in the numerical analysis

Material properties	Soft clay	Stone column	Embankment	Locally weak zone
Material model	HS	M-C	M-C	SSM
γ_{SAT} (KN/m^3)	16	19	22	18
E' (Kpa)	-	45000	42000	-
φ' (°)	23	39	35	5
ψ (°)	0	5	0	0
C' (Kpa)	7	0	6	5
ν'	0.2	0.3	0.33	-
E_{50}^{REF} (Kpa)	2313	-	-	-
E_{OED}^{REF} (Kpa)	1850	-	-	-
E_{UR}^{REF} (Kpa)	6938	-	-	-

C_C	-	-	-	6
C_S	-	-	-	0.6
E	-	-	-	3
P^{REF} (Kpa)	100	-	-	-
OCR	1	-	-	-
K_0	0.6	0.37	0.43	0.91
M (POWER)	1	-	-	-

3.3.2. Verification of the finite element model

The numerical model of the finite-element was verified by (PLAXIS) software with analytical methods (AM) and (FEM). The variation in settlement and radius (of the column) with embankment height, are shown in figure 3.3 (a and b) with (AM), and 5 (a and b) with (FEM).

Figure 3.3a shows the comparison between the results of analytical methods and the present finite element study. The comparison was made through the relation between variations in the settlement on the top of the encased stone column plotted against embankment height (Hemb). Therefore, the settlements estimated by (Pulko et al 2011) (PEA) are in good agreement with the once in the current study. The maximum values of radius variation under different embankment heights estimated by (Raithel & Kempfert 2000) (R&K), figure 3.3b shows good agreement with that of the current study. On the other hand, (Pulko et al 2011 and Zhang & Zhao 2014) (Z&Z) led to an underestimation and an overestimation of the radius, respectively. The maximum radius variation values of (Zhang & Zhao 2014) up to an embankment height of 2 m, are closer to those of the current study. This indicates that the present study confirms the hypothesis of (Pulko et al 2011) in the settlement and (Raithel & Kempfert 2011) in the variation of the radius.

In this section, the obtained values using finite element analysis of maximum radius variation, and settlement at the top of the encased stone columns, were compared with the FEM obtained from (Alkhorshid et al 2018). The comparison is plotted in Figure 3.4 and it is clear that the current study results are in good agreement with the maximum values of radius variation, and vertical settlement of the study carried out by (Alkhorshid et al 2018). The agreement between the current study and (Alkhorshid et al 2018), is satisfactory for values of embankment heights up to 6 m, as shown in figure 3.4b. Therefore, the adopted numerical analysis methods can be used to ascertain further the behavior of the stone column on locally weak zone.

3.3.3. Results and discussions

The numerical analyses were conducted to simulate the construction of embankment on ordinary stone column (OSC) and encased stone column (ESC), the behavior improvement is determined based on the decrease in stone column settlement and decrease in lateral deformation of the stone column with and without locally weak zone (Sabkha soil).

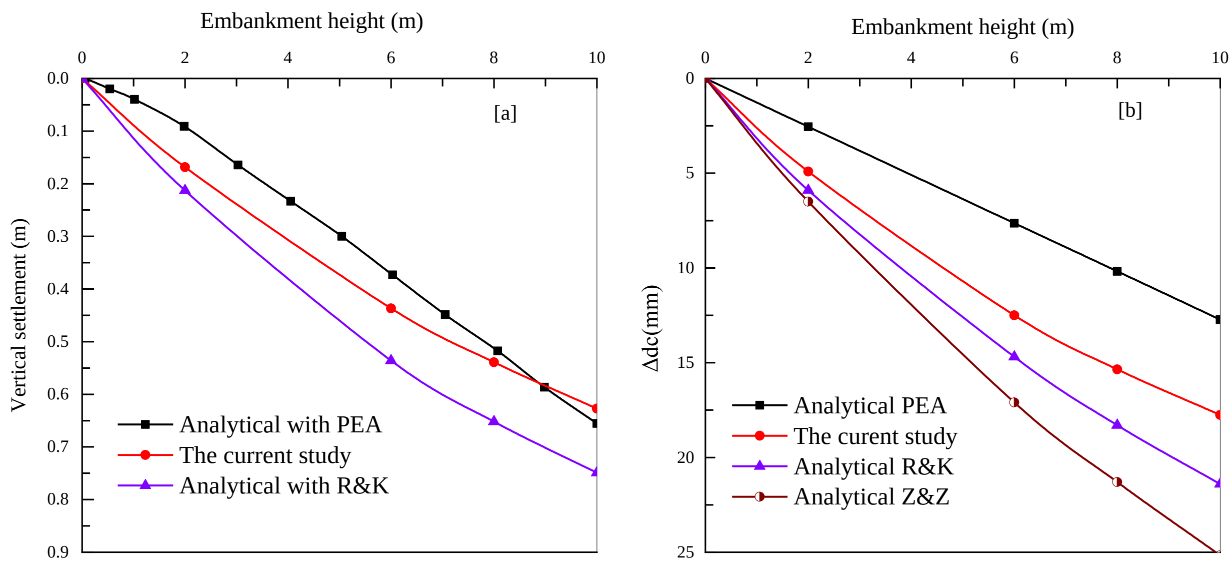


Figure 3-3. Settlement at the tops of the encased stone column, (b) Maximum radius variation.

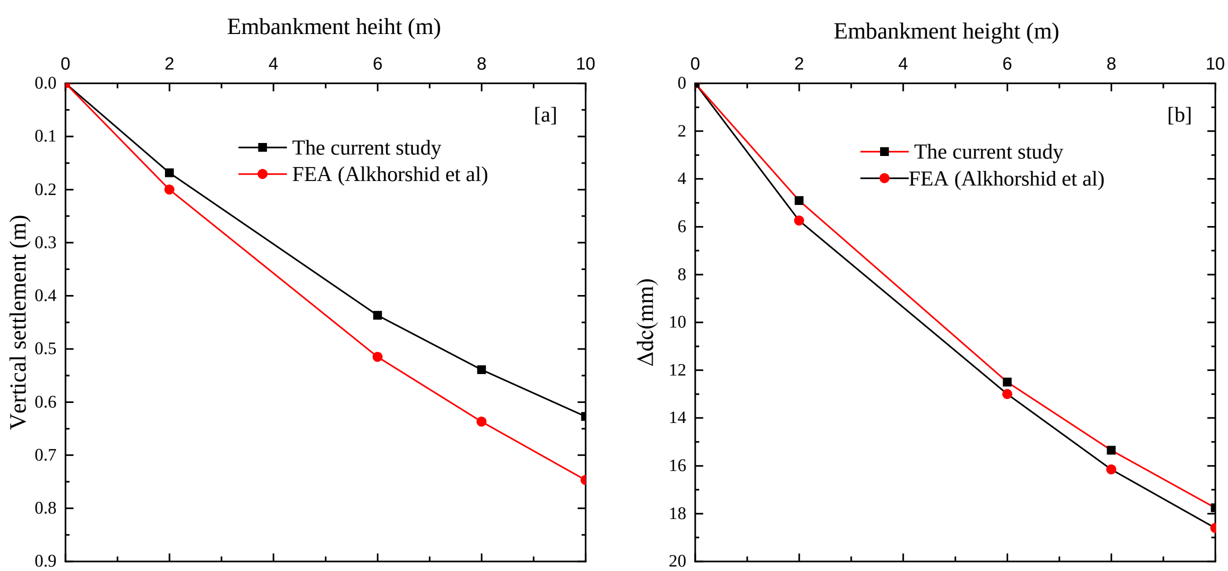


Figure 3-4. (a) Settlement at the tops of the encased stone column, (b) Maximum radius variation.

3.3.3.1. Settlement and Lateral Deformation

Figure 3.5 shows the variation of the settlement on the top of the column plotted against embankment height (Hemb) for the two cases (with and without locally weak zone). Figure 3.5a shows the difference in the settlement between the locally weak zone (LWZ) and non-locally weak zone for ordinary stone column. The settlement values are 1.56 m and 0.736 m respectively. In comparison with the encased stone column, the settlement of the encased stone column decreased by 0.962 m in the locally weak zone and 0.627 m in the case of the non- locally weak zone as shown in Figure 3.5b. Predicted values of settlement variation are significantly influenced by the embankment height. Furthermore, the large difference between the values of settlements are due to the decrease in the shear strength of the locally weak zone (Sabkha soil).

Figure 3.6 shows the variation of the lateral deformation observed along the length of the stone column for with and without locally weak zone. Figure 3.6a shows that the large lateral deformation at the top portion of the column in the two cases of the ordinary stone column is 221.16mm and 31.43 mm, respectively. Similarly, Figure 3.6b shows reduced lateral deformation for the encased stone column with and without locally weak zone to be 42.09 mm and 17.76 mm respectively. This explains that Sabkha soil is one of the biggest problems with column installation due to the low shear strength. The lack of lateral support causes large lateral deformation (bulging) in the upper part at locally weak zone (Sabkha soil).

From Figure 3.5 and 3.6 it can be concluded that as compared with a stone column without geotextile encasement (OSC), the use of (OSC) in the locally weak zone (Sabkha soil) can be problematic due to the lack of adequate lateral confining pressure, particularly in the upper portion of the column. This typically serves as the prime motivation for using the (ESC).

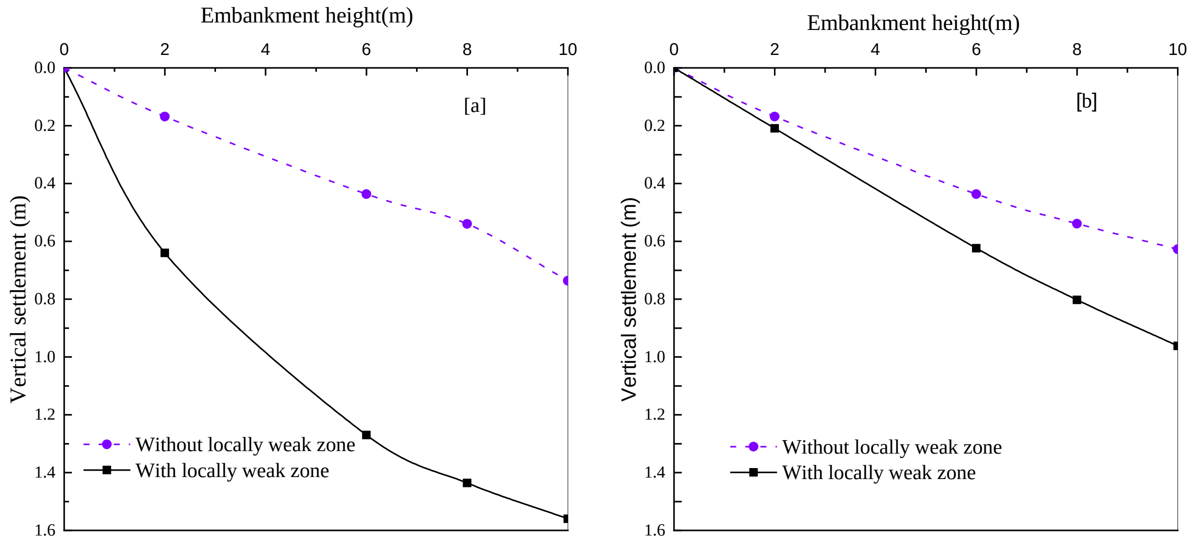


Figure 3-5. Vertical settlement. (a) Ordinary stone column (OSC), (b) Encased stone column (ESC).

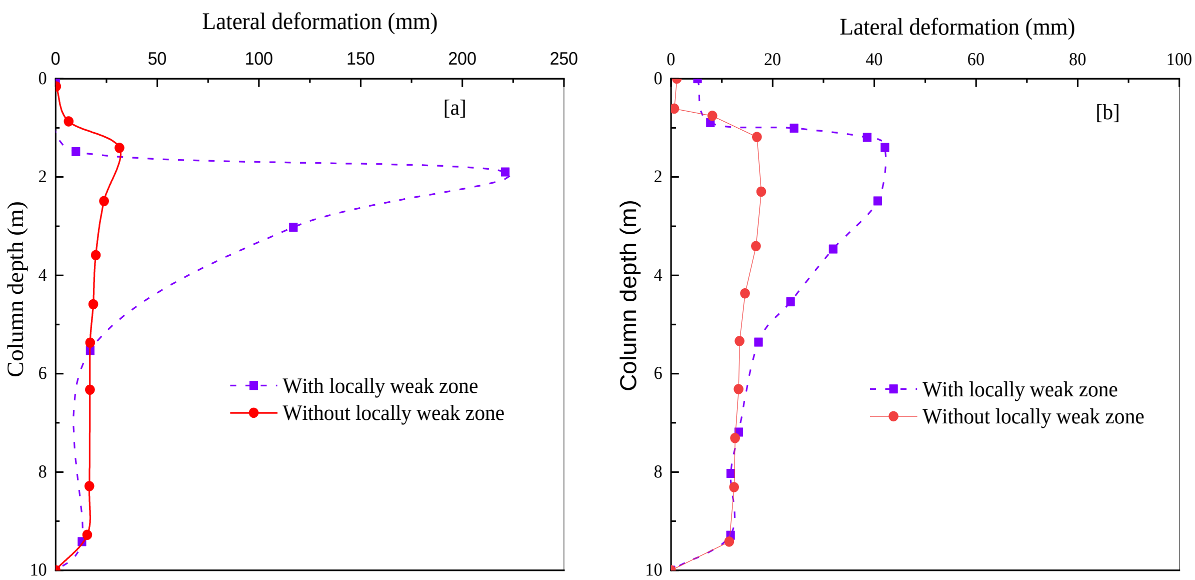


Figure 3-6. Radius variation. (a) Ordinary stone column (OSC), (b) Encased stone column (ESC).

3.3.3.2. Parametric study

In order to investigate the influence of a number of the input parameters on the behavior of the geosynthetic encased stone column with the locally weak zone (Sabkha soil), a series of parametric analyses were performed. In these parameter analyses, basic parameters listed in Table 3.2 were adopted.

Table 3-2. Cases Considered

Category	Description/range	Base values
Embankment height, Hemb (m)	2, 6, 8, 10	10
Encasement stiffness, J (kN/m)	500, 1000, 2000, 4000	2000
Encasement length (m), Lenc	4, 6, 8, full encasement	10
Influence of sabkha layer thickness Dep	1.5, 3, 6	3
Area replacement ratio, ARR %	8.16, 12.75, 18.36	8.16
Friction angle of stone-column materials ø	30, 39, 45	39

3.3.3.2.1. Effect of embankment height

Figure 3.7a shows the lateral deformation of the column as a function of the depth for different values of the height of the embankment (Hemb). The results show an increase in the lateral deformation consequent with the increase in the height of the embankment. Increasing the height of the embankment increases the vertical stress above the columns and the compressible soil (Sabkha soil). The consequence is an increase in the horizontal stress exerted on the encased stone columns where for embankment heights of 10, 8, 6, and 2 m, the lateral deformations are 42.09, 34.34, 25.90, and 7.27 mm, respectively.

This explains that increase of the height of the embankment increases the lateral deformation of the column which is consistent with the findings of Alkhorshid et al 2018 and Raithel et al 2000. As we could note, the bulging zone moves downward, as the embankment height increases, where the value of the bulging depth (z_b) at a height of 2 m with the maximum bulging occurring 1D below the top of the encased stone column. At a height of 6 m, the maximum bulging was 1.12 m which is equivalent to 1.4 of the diameter of the stone column (D), similarly at height of 8 m maximum bulging was 1.27 m which is equivalent to 1.58D. Furthermore, at 10 m height, 1.40 m equal 1.75D was obtained as the maximum bulging. A similar phenomenon was observed (Raithel et al 2000).

Figure 3.7b shows the vertical settlement distributed at the surface for a distance from the stone column centerline to the outer edge of the unit cell as a function of the height of the embankment (Hemb), where Hemb was varied between 2 and 10 m. It should be noted that the effect of embankment height is very important for the stability of the embankment, thus, increasing the height of the embankment increases the load applied to the compressible soil (Sabkha soil).

The value of the settlement at the height of 2 and 6 m is estimated to be 0.26 and 0.65 m, respectively. As for the height, 10 and 8 m the estimated settlement are 0.99 and 0.83 m, respectively. The settlement increases with increasing height of the embankment.

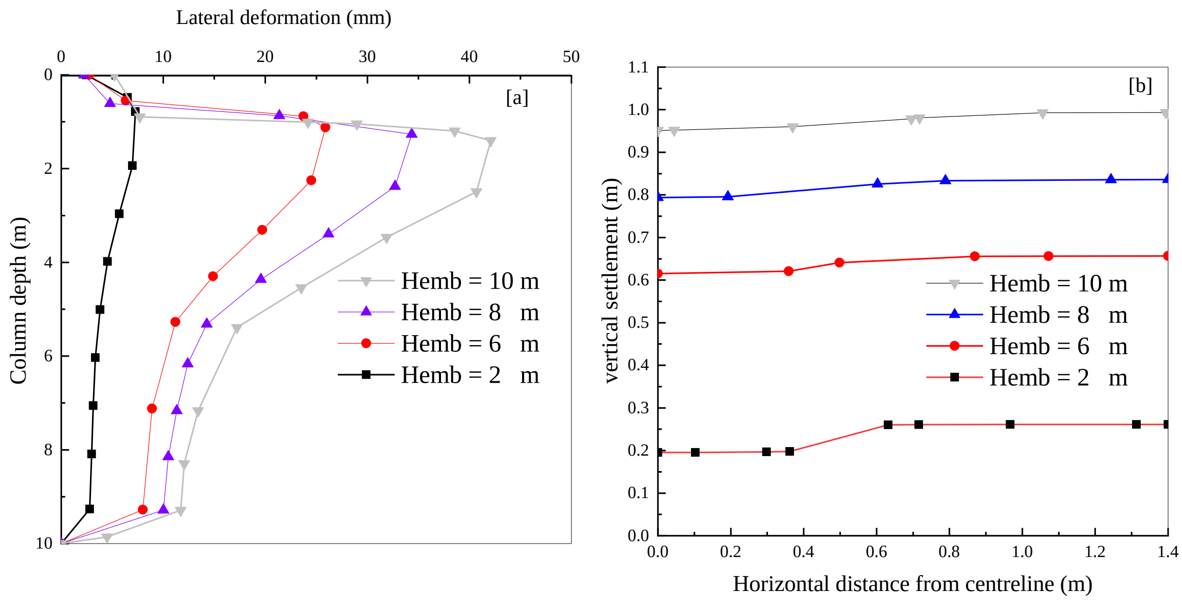


Figure 3-7. (a) Lateral deformation of the column as a function of the depth for different values of the height of the embankment (Hemb), (b) Vertical settlement distributed at the surface for a distance from the stone column centerline to the outer edge of the unit cell as a function of the height of the embankment (Hemb).

3.3.3.2.2. Influence of the stiffness of geosynthetic encasement

The influence of the tensile stiffness of the geosynthetic used for encasement on the performance of the stone column has been investigated numerically (Alkhorshid et al 2018, Elsawy 2013, Yoo et al 2015). In this present study, the effect of the stiffness of an encased stone column was examined by choosing four different values of tensile strength different from 500, 1000, 2000, and 4000 kN/m. As compared with a stone column without geotextile encasement (OSC), the use of (OSC) in the locally weak zone (Sabkha soil) can be problematic, as it was mentioned previously, due to the lack of adequate lateral confining pressure, particularly in the upper portion of the column. This typically serves as the prime motivation for using (ESC).

The effect of geosynthetic encasement is clearly illustrated in Figure 3.8a, as the (OSC) exhibited considerable lateral bulging, as much as 221.16 mm, the lateral deformation at the top of the column is reduced by 60.73, 68.83, 80.96 and 89.60% when the column is encased in geotextile with the stiffness of J=500, 1000, 2000, and 4000 kN/m, respectively. However, it should be noted that this large difference between the values of lateral deformation of ordinary stone columns (OSC) and geotextile encased stone columns are due to the low shear strength in the locally weak zone.

Since when installing the stone column, it does not find any lateral force in this zone, as we observe in Figure 3.8b, the beneficial effect of geotextile encasement on the reduction of maximum lateral bulging is also evident in Figure 3.8c. This confirms that the encased stone columns are very effective in very soft soil (Almeida et al 2015). The encasement, besides increasing strength and stiffness of the stone column, prevents a lateral deformation of stones when the column is installed even in extremely soft soils, thus enabling quicker and more economical installation. Encasement material also prevents the mixing of fine-grained soil with stone material, which has a negative effect on the stone column drainage efficiency during the consolidation process.

The hoop tension force is a property of geotextile material. Figure 3.8d shows the relationship between geotextile stiffness and hoop tension force. It can be seen that, by increasing geotextile stiffness, the value of the hoop tension force increased. The hoop tensions obtained are 109.52, 173.97, 210.80, and 230.03 kN/m for geotextile Ringtract 500, 1000, 2000, and 4000 kN/m, respectively. The same was observed by Murugesan and Rajagopal 2006 and it may be observed that hoop force in geotextile follows a variable pattern with depth.

When the encased stone column reinforced ground is loaded, concentration of stress occurs in the stone column, and an accompanying reduction in vertical stress occurs in the surrounding less stiff soil (Sabkha soil), figure 3.9 shows the difference in vertical stress on top of the encased column and the soft soil. It was observed that with stiffness of J= 0, 500, 1000, 2000, and 4000 kN/m vertical stress in the encased stone column was 171.32, 261.51, 503.52, 933, and 1826.36 kPa, respectively. However, in surrounding soil the vertical stress was 169.81, 152.20, 128.32, 90.13, 48.45 kPa. This is further illustrated in Figure 3.9 (b & c).

Figure 3.9 (b & c) shows the effective stress distributions as cross marks. When it comes to the ordinary stone columns (OSC), these cross marks in the surrounding soil are visibly greater than those for the encased column, which means that a higher share of vertical stress goes to surrounding soil, this is consistent with the findings of Alkhorshid et al (2018).

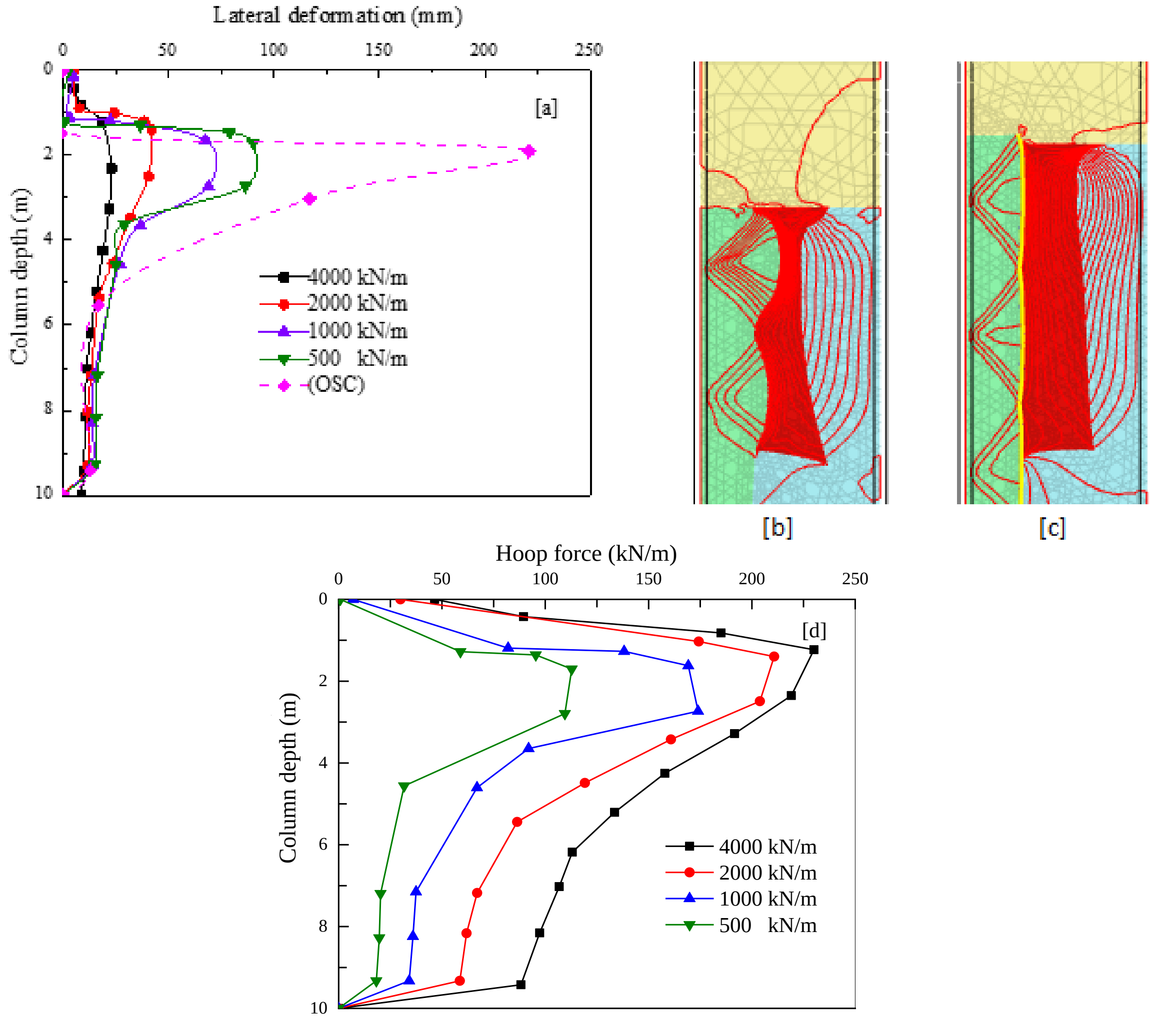


Figure 3-8. (a) Lateral deformation vs. depth, (b) Deformed for (OSC) modeled by FEM. (c) Deformed for (ESC) modeled by FEM, (d) Hoop force vs depth with different tensile stiffnesses.

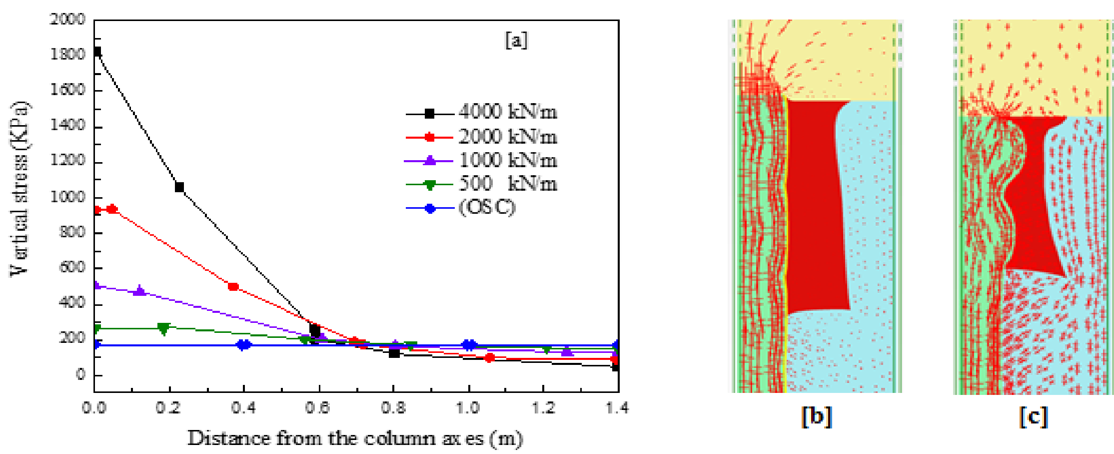


Figure 3-9. (a) Vertical stress on top of the encased column and the soft soil, Effective stress distribution: (b) Encased stone column (j= 2000 kN/m), (c) Ordinary stone column.

3.3.3.2.3. Effect of encasement length

Figure 3.10 a shows the distribution of the lateral bulging of the encased stone column through its depth using different encasement depths from 4 to 10 m (fully encased), tensile strength (J= 2000 kN/m). When the stone column is partially encased at a certain depth (ideal depth), it's bulging in the encased zone is slightly smaller than that of the full-encased column case. However, the non-encased zone has higher values of the column bulging (encasement depths less than 6 m). The non-encased zone in the column starts with a maximum value that generates a largely differentially lateral bulging at the endpoint of the encasement in the encasement depths 4 m. Below that, the lateral bulging values decrease gradually with depth until it reaches zero at the column base, as shown in Figure 3.10. The shallower the encasement depth is, the higher the lateral bulging values in the non-encased zone of the stone column are. Hence, this analysis shows that the length required for the encasement to limit both the settlement and especially the bulging depends on the depth of the locally weak layer. The encasement should just go beyond the weak area. As illustrated in figure 3.10a, at the length of the encasement 4, 6, 8, 10 m (fully encased), the bulging value is 58.13, 41.24, 41.99, and 42.09 mm, respectively.

Settlement ratio β, the ratio of the settlement of ESC to that without encasement (OSC), is defined as β= (SESC/SOSC). In Figure 3.10b, the settlement ratios β are plotted against the encasement length. When increasing the length of the encasement, we notice a decrease in the settlement ratios. For length of the encasement 4, 6, 8, and 10 m (fully encased), the ratio of settlement values are 0.75, 0.70, 0.68, and 0.66, respectively. This shows that the encasement length is important in decreasing the settlement.

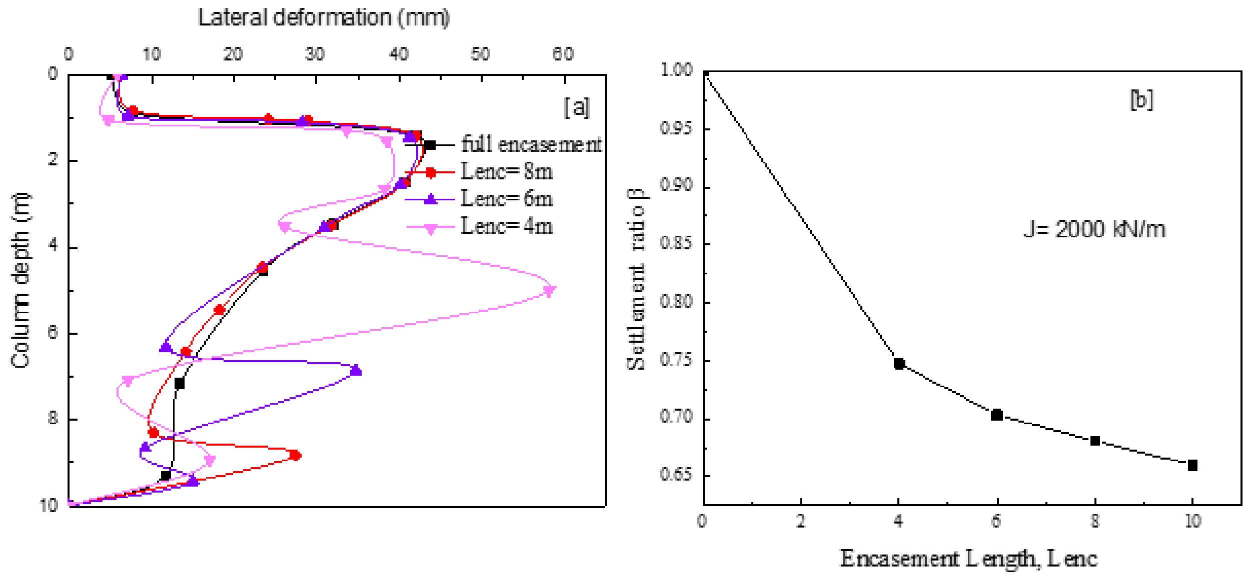


Figure 3-10. (a) Distribution of the lateral bulging of the encased stone column through its depth, (b) Ratio of settlement vs. Encasement length (Lenc).

3.3.3.2.4. Influence of sabkha layer thickness

Figure 3.11 shows the effect of the Sabkha layer thickness on the encased stone column for the vertical settlement. There is an increase in the vertical settlement consequent with the increase in the Sabkha layer thickness, as we notice in figure 3.11, vertical settlements for Sabkha depths of 1.5, 3, and 6 m are 0.89, 0.99, 1.18 m, respectively. It can be concluded from here that the depth of the Sabkha layer has a major impact on the instability of the embankment.

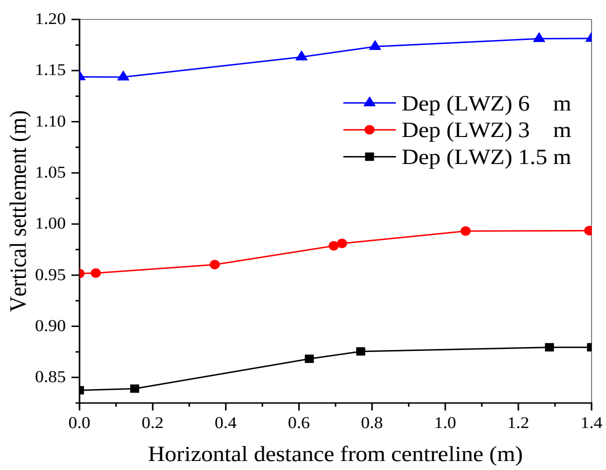


Figure 3-11. Influence of the depth of locally weak zone on the settlement at the embankment base.

3.3.3.2.5. Influence of area replacement ratio (ARR) on the performance of ESC

The effect of area replacement ratio on settlement ratio is better illustrated with the degree of improvement in the Sabkha soil that is realized through the use of any type of ground improvement technique, a parameter called the settlement ratio (β) is commonly used. The settlement ratio is defined, as mentioned above, as the ratio of the settlement of the encased stone column (ESC) to the settlement ordinary stone column (OSC). The lower the value of (β), the better the performance realized due to ground improvement. (Collin 2004) stated that the ARR should be selected to be between 10 and 20 % for the preliminary design of (ESC). To investigate the effect of variations in the ARR have on (ESC) response, three (ESC) were modelled with ARR values equal to 8.16, 12.75, and 18.36 %. During these analyses, all other parameters were maintained at their base values.

The variation of settlement ratios β versus area replacement ratio with varying encasement stiffness is shown in figure 3.13. It was observed that with increasing area replacement ratio, the settlement ratio decreases. For example, for an area replacement ratio varying from 8.16% to 12.75%, the settlement ratio (β) decreases by 14.50%. Similarly, between 12.75% and 18.36% the settlement ratios (β) decreases by 17.72% for the encasement stiffness of 4000 kN/m. On the other hand, when the encasement stiffness is 2000 kN/m, for an area replacement ratio of 8.16% to 12.75%, the settlement ratio (β) decreases by 13.72% and, for 12.75% to 18.36% the settlement ratio (β) decreases by 17.72%. However, the same case for the encasement stiffness 1000 kN/m, for an area replacement ratio of 8.16% to 12.75%, the settlement ratio (β) decreases by 7.81%, and for 12.75% to 18.36% the settlement ratio (β) decreases by 11.10%. Similarly, for a small value of the encasement stiffness 500 kN/m, for the ARR of 8.16% to 12.75%, the settlement ratio (β) decreases by 8.77%, and for 12.75% to 18.36 the settlement ratio (β) decreases by 4.91%. It can be concluded from this study that with an increase in area replacement ratio associated with an increase in encasement stiffness, the settlement ratio β decreases better. Similar observations have been reported by Yoo 2015. This is what is needed in the presence of the locally weak zone (Sabkha soil).

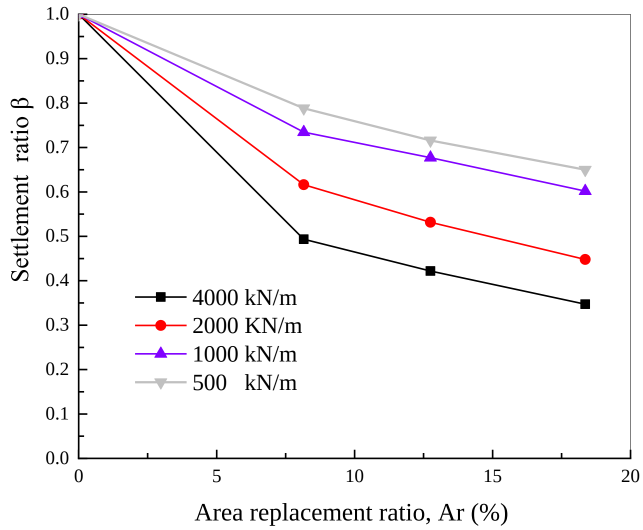


Figure 3-12. Settlement ratio vs. Replacement ratio.

Figure 3.14 shows the effect of variations in area replacement ratio on the lateral deformation of (ESC) with different values of encasement stiffness (500, 1000, 2000, and 4000 kN/m). For all the analyzed cases, the bulging increases with the increase of the area replacement ratio. For the three area replacement ratios of 8.16%, 12.75%, and 18.36%, considered in this study, and with an encasement stiffness of 500 kN/m, the maximum bulging is estimated at 90.48, 107.7, and 133.33 mm, respectively as shown in figure 3.14a. On the other hand, in the case of 1000 kN/m encasement stiffness, lateral deformation is reduced by values 68.93, 70.89, and 99.026 mm shown in figure 3.14b. However, in the cases of encasement stiffness of 2000 kN/m and 4000 kN/m the bulging reduces progressively by values 42.09, 48.52, and 37.73 mm for the stiffness 2000 kN/m, and 21.87, 23, and 24.77 mm for the stiffness 4000 kN/m, the results are shown in figures 3.14c and 3.14d.

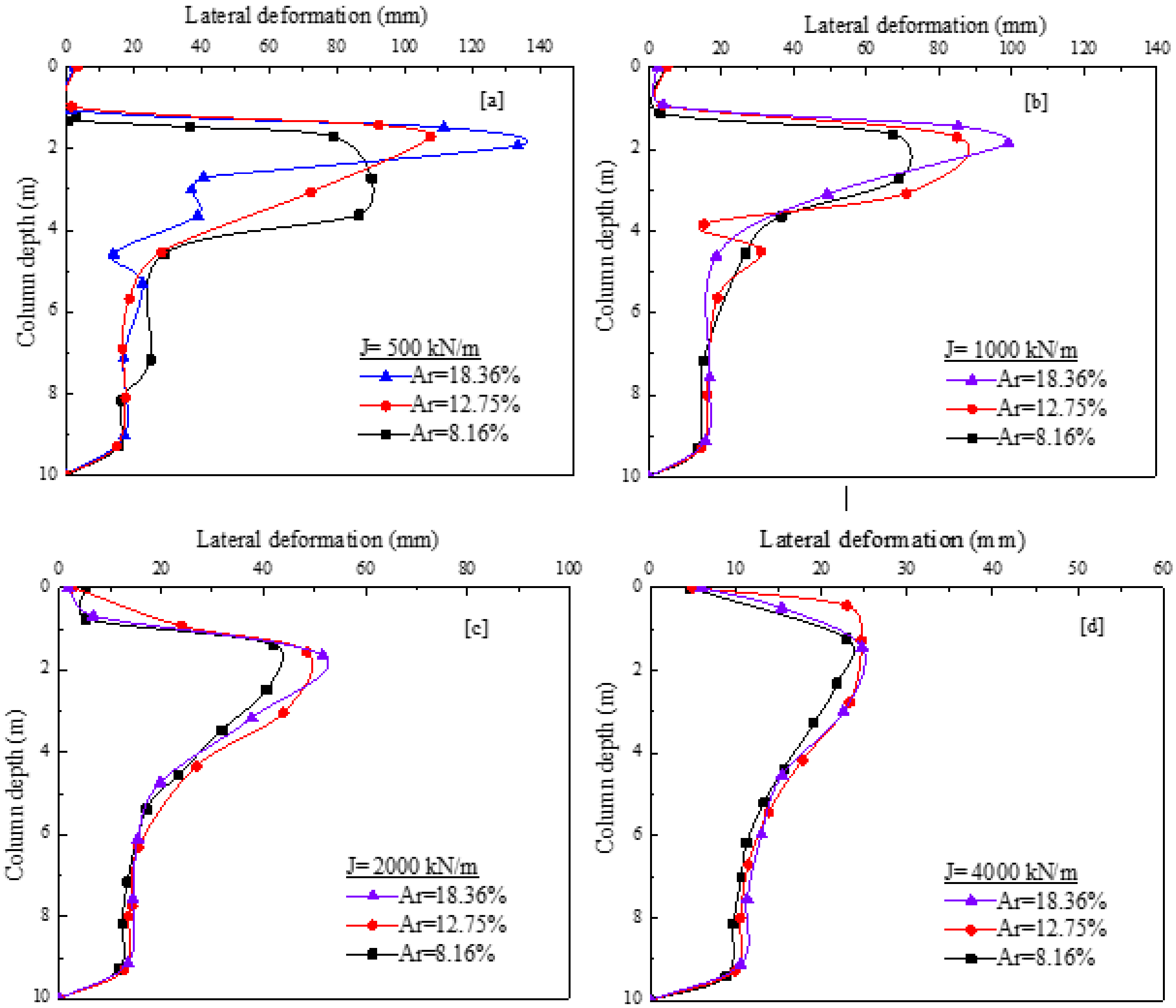


Figure 3-13. Lateral deformation vs. Depth. (a) j=500 kN/m, (b) j=1000 kN/m, (c) j=2000 kN/m, (d) j=4000 kN/m.

3.3.3.2.6. Influence of friction angle of stone column materials

In order to study the effect of the friction angle of stone-column materials on the lateral deformation and the vertical settlement of the encased stone column, we performed analyses with a series of three friction angles (30°, 39°, and 45°).

Figure 3.15a shows the lateral deformation of the column as a function of the depth for different angles of the friction of the stone column material. It can be seen that the higher the friction angle value, the lesser the lateral deformation. The difference between the lateral deformation of angle 30° and angle 39° is estimated to 7.6 mm, and this difference between angle (39°, 45°) is 4.82 mm. This shows that the friction angle of the stone column has an important role in reducing lateral deformation.

The results indicated that the efficiency of ESC is better if the column material is compacted well to achieve a high friction angle. This is consistent with the findings presented by (Alkhorshid et al 2018). As explained in the previous section, the friction angle of (ESC) is also critical to enhancing the settlement response of the column and the soil. Figure 3.15b shows the time as a function of settlement, for different friction angles of the column, where we can note that the increase in the angle of friction of the column will decrease the settlement. For the friction angle of 30^o, 39^o and 45^o, the estimated settlement is 1.10 m, 0.99 m and 0.91 m respectively. The angle of friction for the stone column is important in reducing settlement especially in the presence of the locally weak zone.

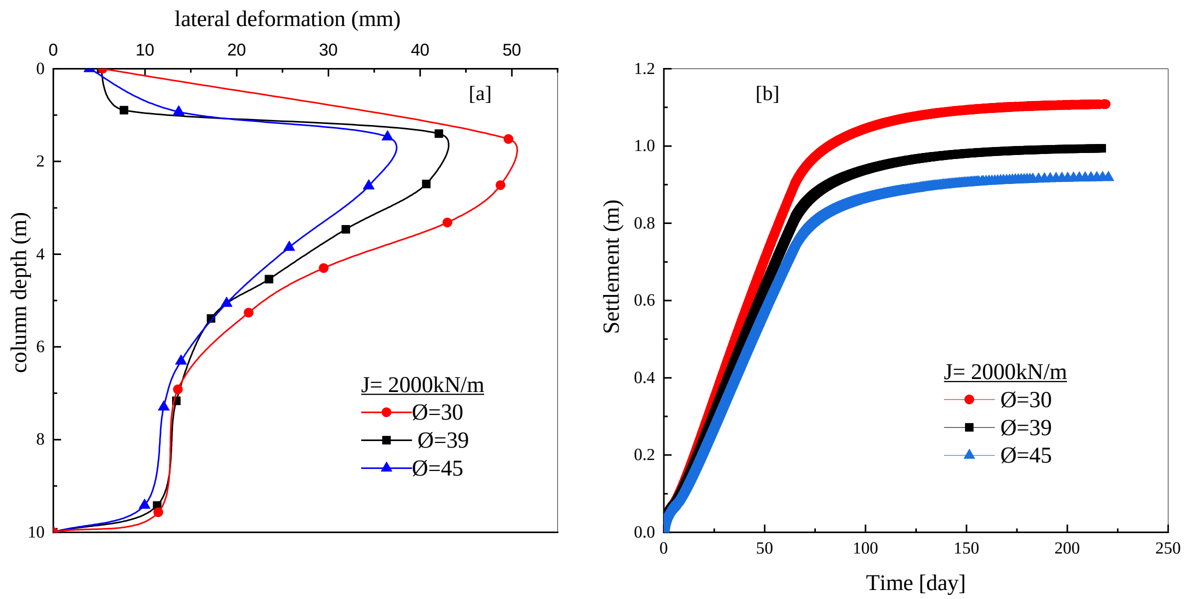


Figure 3-14. Response of stone column with varying friction Angle of encased stone column. (a) Maximum lateral deformation of stone column, (b) Settlement at top of stone column and surrounding soil.

3.4. Conclusions

The present study presents the results of two-dimensional axisymmetric numerical analyzes that were carried out using PLAXIS 2D 2017, for the modeling of an embankment supported by stone columns on sabkha soil with locally weak zones. The study focuses on the evaluation of the maximum bulging of the stone column and on the settlement of the embankment. Besides, an extensive parametric study was conducted to investigate the effects of the variations of embankment height, stiffness of geosynthetic, the depth of the locally weak zone, area replacement ratio (ARR), and the stone column friction angle, on the performance of the (ESC) - embankment composite in (LWZ). Some important guidelines for selecting the ideal encased stone column (ESC) to support embankments on over locally weak zone were established through this numerical study.

According to the results obtained from the present study, the following conclusions are made:

- It has been demonstrated that ordinary stone columns (OSC) were ineffective to support the embankment due to excessive bulging (221.16 mm) caused by the lack of lateral pressure. On the other hand, the encased stone columns (ESC) showed good behavior, namely a much reduced bulging (42.09 mm) and a reasonable settlement (0.962 m vs. 1.560 m for an OSC) so that it is possible to build safe very high embankments.
- The numerical analysis also shows that the length of the encasement should just be greater than the depth of the LWZ. This means that the encasement length is not required to a depth that equals the depth of the stone column. The value of the lateral deformation is 41.24 mm for an encasement length of 6 m, 41.99 mm for an encasement length of 8 m, and 42.09 mm for a full encasement. For instance, the ideal encasement depth for the present case study is 6 m (60% from full encasement).
- The area replacement ratio (ARR) leads to two opposite effects. On the one hand, it decreases the value of settlement ratio (β), but on the other hand, it decreases the effectiveness of the encasement by increasing the lateral deformations.
- Furthermore, this numerical analysis has shown that the increase in the internal friction angle of the stone column material leads to an increase in the resistance of the column against failure and, consequently, the lateral deformations and settlements of the column decrease in (LWZ). For example, for the friction angle of 30^o,39^o and 45^o, the estimated settlement is 1.10 m, 0.99 m and 0.91 m respectively.
- The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results in increases in the settlement of the column.
- Increasing the height of the embankment increases the vertical stress above the column and the compressible soil (Sabkha soil). The consequence is an increase in

the horizontal stress exerted on the encased stone column. On the other hand, the depth of radial bulge is affected by the height of the embankment, as it increases with the height of the embankment increases.

- Increase in the stiffness of the geosynthetic encasement of stone columns leads to increases in the column stiffness, the hoop tension force mobilized in the encasement, and the lateral confinement provided to the stone column. Where the hoop strains in the geosynthetic encasement are at a maximum near the top surface and decrease with depth. The stone column encasement causes reducing the total stress in the soft soil along with consolidation.

Chapter 4.

Numerical modeling of horizontally layered geosynthetic reinforced encased stone columns supporting embankment on sabkha soils

4.1. Introduction

The present research work is concerned with the construction of road embankments on a specific soil called Sabkha in Algeria. This soil is not only soft and very humid during the flooding seasons but also has frequent small areas of very soft soil which are called locally weak zones (LWZ) in the context of this study (see figure 4.1a). LWZ are characterized by low strength and high compressibility. Two-dimensional axisymmetric analyses were carried out using PLAXIS 2D 2017. The study demonstrates that ordinary stone columns (OSC) are ineffective given the nature of these soils due to the excessive bulging caused by the lack of lateral pressure. On the other hand, the reinforced stone columns with external reinforcement and internal reinforcement (VESC+HRSC) are one of the best improvement methods of locally weak zones (LWZ), especially to increase the stability of embankment on the highway, namely a much reduced bulging and a reasonable settlement, so that it is possible to build safe and very high embankments. Besides, an extensive parametric study is conducted to investigate the effect of the spacing of the horizontal reinforcing strips and of the column reinforced length. The influence of stone column diameter, depth of locally weak zone, and the effect stiffness of the geosynthetic, on the performance of the (ESCs) - embankment composite are also investigated.

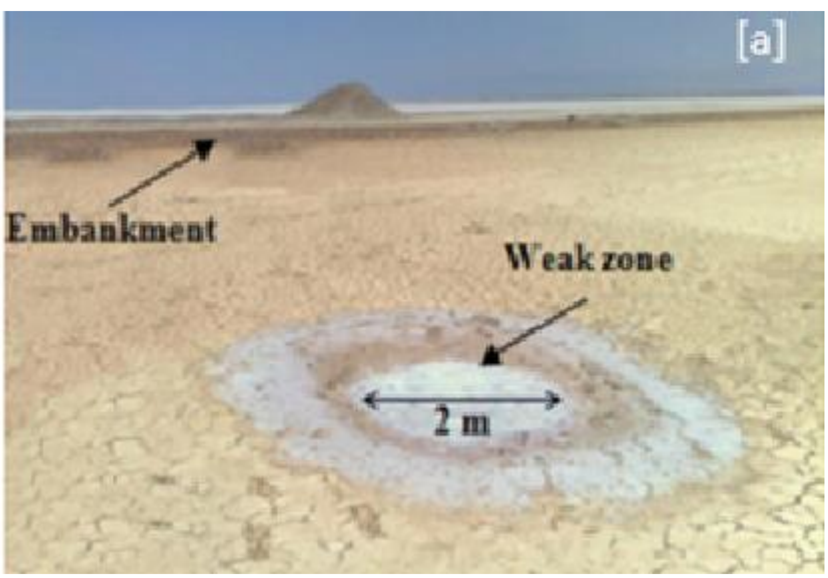


Figure 4-1. Example of the locally weak zone (Benmebarek et al. 2015).

4.2. Numerical modeling

4.2.1.1. Presentation of the finite element model and material parameters

In order to simulate the unit cell, an axisymmetric model is undertaken using the finite element code PLAXIS 2D V2017 program available commercially to analyse deformation and stability for a variety of geotechnical problems. In this numerical analysis, a very fine mesh is used because stresses and displacements are very high in this problem. The problem of using a stone column to support a large embankment over locally weak zone (Sabkha soil) is studied.

Appropriate choices of material properties are necessary to have an accurate simulation of the reinforcement system in numerical modelling. The properties of the embankment, soft clay and stone columns can be found in the literature Alkhorshid et al (2018). The columns are installed in a square grid with spacing, s = 2.5 m supporting an embankment of 10 m high (Hemb). The thickness of the clay soil and the length of the stone column are assumed to be 10 m underlain by a rigid, hard stratum. the radius of the column within the unit cell rc, is equal to 0.4 m, radius of the influence area of the column re = 1.4 m, area replacement ratio (Arr = r^2c/r^2e) equal to 8.16 %, and geosynthetic tensile stiffness (J) equal to 2000 kN/m, spacing of the horizontal reinforcing strips Sv = 0.25 m. The dimensions and properties of the locally weak zone (Sabkha soil) are chosen to match values stated by Benmebarek et al (2015) as B = 0.6 m and DEP = 3 m respectively width and depth of the LWZ as shown schematically in figure 4.2. The groundwater level is assumed at the ground surface. The vertical or horizontal displacements are restrained at the bottom boundaries of the unit cell, but vertical displacements are allowed at the lateral borders. Table 3.1 shows the parameters used in the FEM.

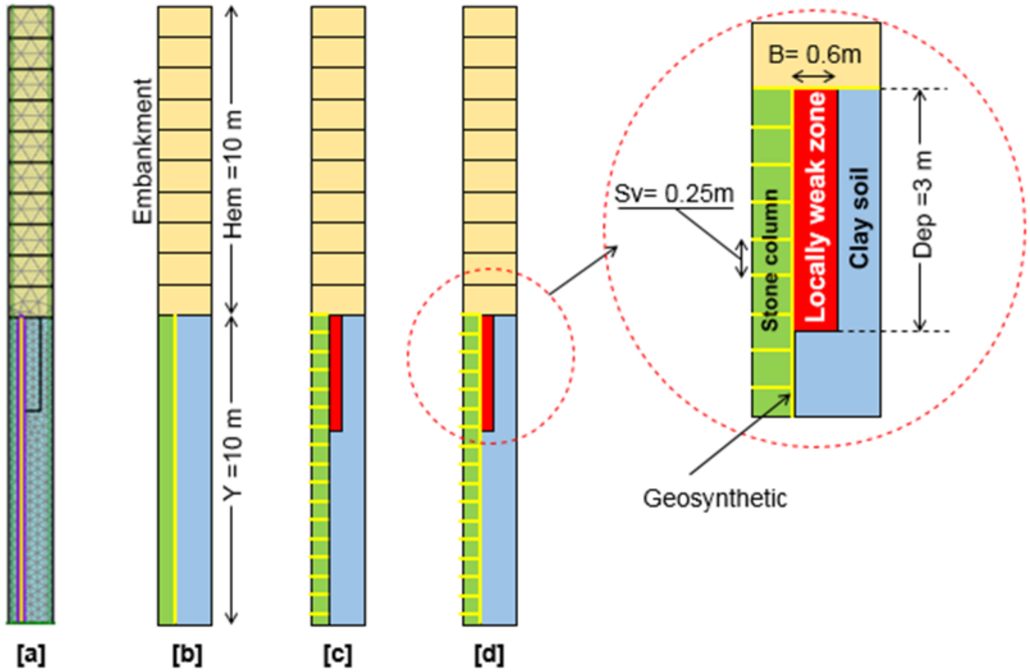


Figure 4-2. Finite-element axisymmetric simulation of the geosynthetic- reinforced column in the unit cell concept: (a) Boundary condition and finite-element mesh; (b) (VESC); (c) (HRSC); (d) (VESC + HRSC).

4.2.1.2. Results and discussion

4.2.1.3. Settlement and lateral deformation

Figure 4.3 shows the variation in the settlement on the top of the column plotted against embankment height (Hemb) and radius variation with and without the locally weak zone. Figure 4.3a shows that differences in the settlement between the locally weak zone (LWZ) and non-locally weak zone for ordinary stone columns are 1.56 m and 0.736 m, respectively, Figure 4.3b shows that the large lateral deformation at the top portion of the column in the two cases of the ordinary stone column is 221.16 mm and 31.43 mm. This explains that Sabkha soil or (LWZ) is one of the biggest problems with column installation due to the low shear strength. The lack of lateral support causes large lateral deformation (bulging) in the upper part at locally weak zone (LWZ).

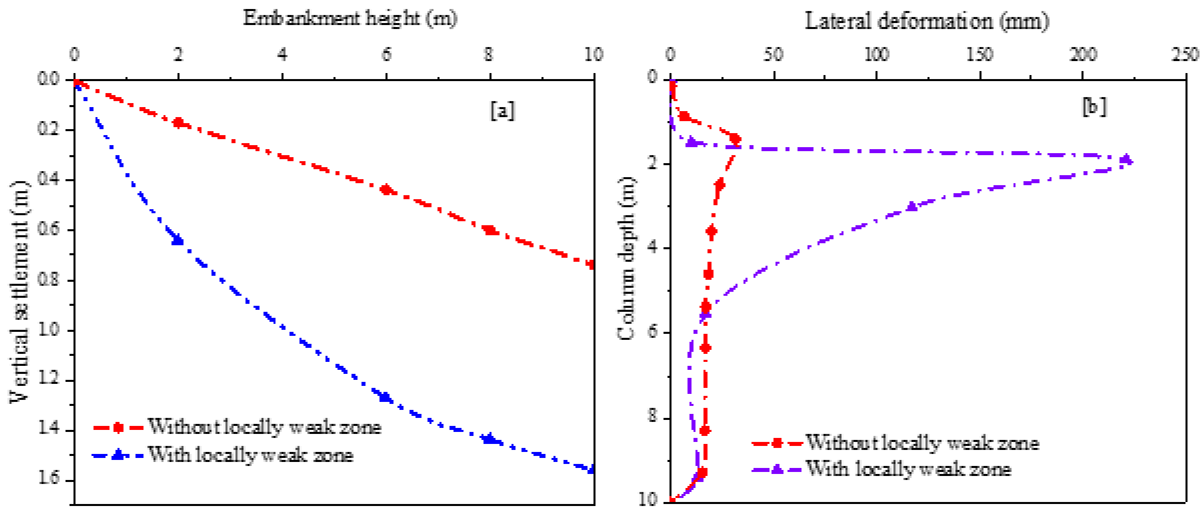


Figure 4-3. Ordinary stone column: (a) Vertical settlement; (b) Radius variation.

4.2.1.4. Comparison of (HRSC), (VESC), and (VESC + HRSC)

When stone columns are installed in exceptionally soft soils, the lateral confinement offered by the surrounding soil may not be adequate to form the stone column, and the bulging of the stone column augments and leads lead to larger surface settlements. This is the major limitation of the stone column technique, especially in this case of sabkha soil (LWZ). The efficiency of stone column installed in soft soil can be improved by surrounding the stone column with a suitable geosynthetic in a tubular form (VESC), horizontally reinforced stone column (HRSC), the reinforced stone column with vertical encasement and horizontal layers (VESC+HRSC). As shown in figure 4.4.

Figure 4.4 illustrates vertical settlement distribution at the surface for a distance from the stone column centerline to the outer edge of the unit cell as a function of the height of the embankment Figure 4.4a. Similarly, variation of the lateral deformation are observed along the length of the stone column figure 4.4b, with different types of reinforcement. As shown, stone columns with vertical and horizontal reinforcement (VESC+HRSC) give excellent bearing capacity if compared to (HRSC) and (VESC).

To illustrate the effect of reinforcement on the deformation of stone column, a new parameter is defined as improvement factor value (I.F.V), the ratio of the value of without reinforcement (OSC) to that value with reinforcement, is defined as I.F.V = (Vosc/Vwr) the results have been summarised in Table 4-1.

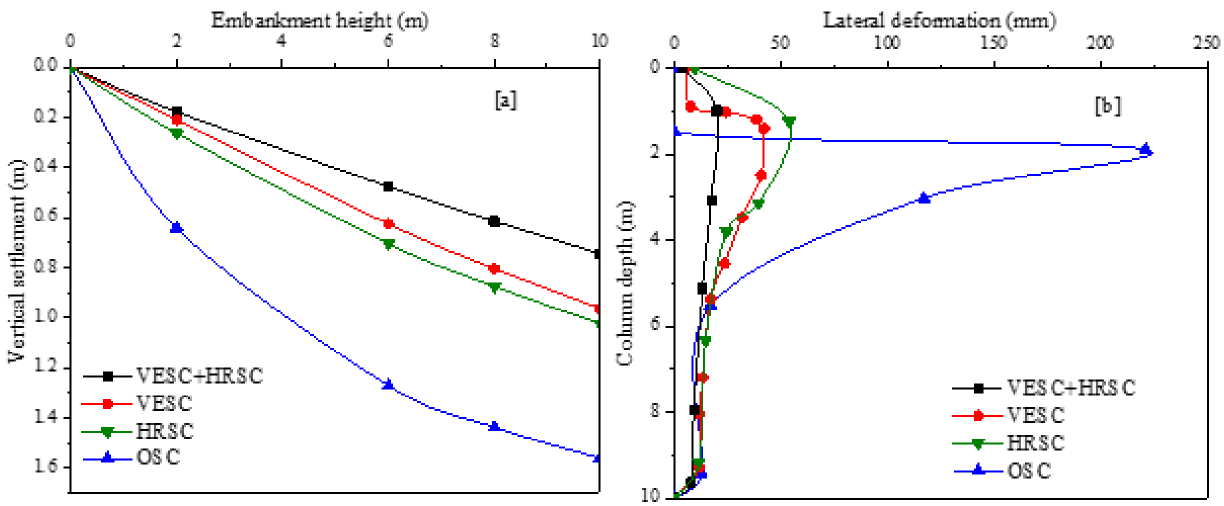


Figure 4-4. (a) vertical settlement versus embankment height; (b) Lateral deformation vs. depth. (H_{em} = 10 m, Y = 10m, Sv = 0.25m, j =2000 kN/m).

Table 4-1. Results of FEM analysis

Types of reinforcement	Vertical settlement (m)	Improvement factor value (I.F.V)	Lateral deformation (mm)	Improvement factor value (I.F.V)
(OSC)	1.56	-	221.16	-
(HRSC)	1.02	1.529	53.94	4.099
(VESC)	0.96	1.625	42.09	5.253
(VESC+HRSC)	0.74	2.108	20.02	11.044

As illustrated in Figure 4.5 (a & b). In contrast, shows the effective stress distributions as cross marks. Where the reinforced stone column with vertical encasement and horizontal layers **(VESC + HRSC)** causes reduction in the effective stress in the surrounding soft soil.

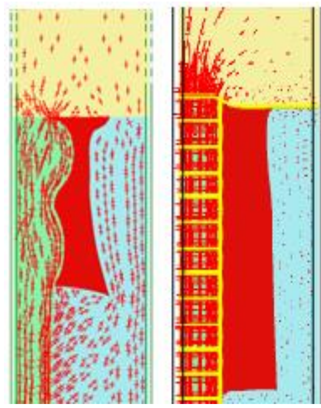


Figure 4-5. Effective stress distribution: (a) Ordinary stone column (OSC); (b) (VESC + HRSC).

4.2.1.5. Parametric study

Results of the parametric analyses on column reinforced by (VESC + HRSC) are presented as follows: Effect spacing of the horizontal reinforcing strips (Sv) and column reinforced length (Y), the diameter of the stone column (D), Influence of depth of locally weak zone (Dep), reinforcement stiffness (kN/m) Table 4-2 shows the parameters of the problem analyzed for the parametric analyses. Cases Considered

Table 4-2. Cases Considered

Category	Description/Range	Base Values
Spacing of the horizontal Reinforcing strips Sv (m)	0.25, 0.5, 0.75	0.25
Column reinforced length (Y)	5, 7.5, 10	10
Diameter of stone column (D)	0.6, 0.80, 1.1	0.8
Depth of the locally weak zone (Dep)	1.5, 3, 6	3
Reinforcement stiffness (kN/m)	500, 2000, 3000, 5000	2000

4.2.1.5.1. Effect spacing of the horizontal reinforcing strips (Sv)

The advantageous effect of HRSC mainly depends on the vertical spacing between the horizontal reinforcing strips and that the bearing capacity of HRSC increases with a decrease in the spacing between the reinforcing layers. The results of FEM showed that increasing the spacing of the horizontal reinforcing strips increase the lateral deformation and reduces settlement improvement of the reinforced column. Figure 4.6a shows the lateral deformation of the column as a function of the depth for different values of the vertical spacing between the horizontal reinforcing strips (Sv): 0.25, 0.5, and 0.75 m, the lateral deformations are 20.02, 30.34, and 33.4 mm, respectively. The spacing of the horizontal reinforcing strips can be reduced due to the lack of adequate lateral confining pressure, particularly in the upper portion of the column. This typically serves as the prime motivation for using the (VESC+HRSC) in this case (LWZ).

Figure 4.6b shows the vertical settlement distributed at the surface for a distance from the stone column centerline to the outer edge of the unit cell as a function of the values of the vertical spacing between the horizontal reinforcing strips. It should be noted that the effect of vertical spacing is significant to the stability of the embankment; thus, increasing the vertical spacing between the horizontal reinforcing strips increases the load applied to the compressible soil (Sabkha soil). The value of the settlement at the vertical spacing of 0.25, 0.5, and 0.75 m is estimated to be 0.74, 0.86, 0.90 m, respectively.

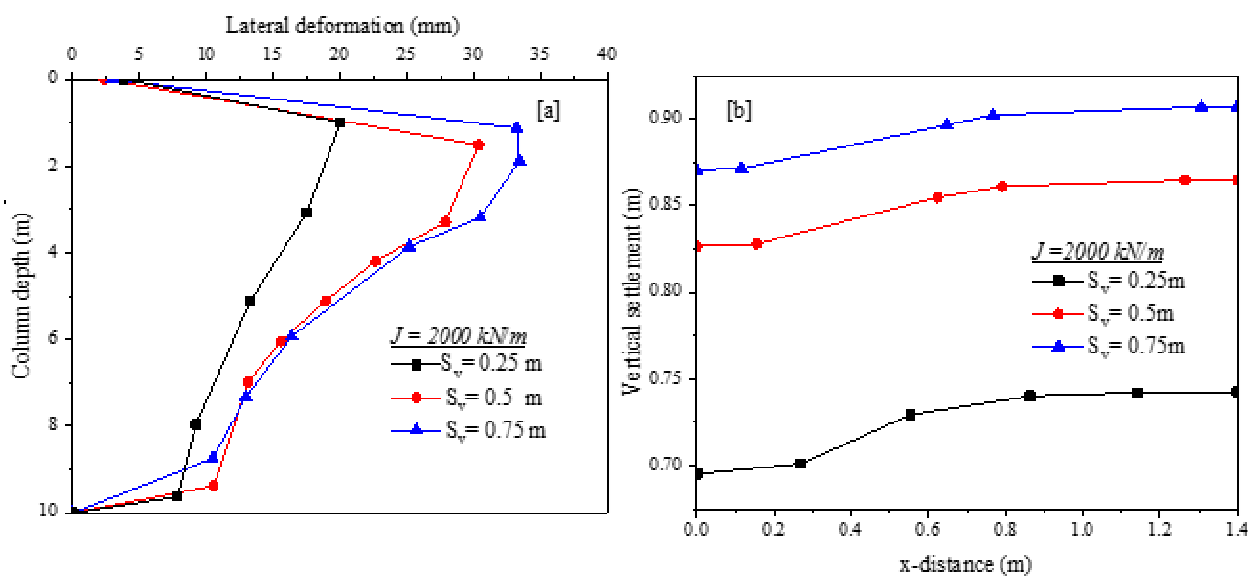


Figure 4-6. (a) Effect of spacing on bulge profiles; (b) Settlement of granular piles reinforced.

4.2.1.5.2. Effect column reinforced length (Y)

To investigate the influence that the column reinforcement length (Y) has on the performance of stone columns supporting embankment on locally weak zones, finite element analyses are performed with the column reinforcement length varying from 5 m (half-length reinforced), 7.5 m and 10 m (fully reinforced Y = Lc). In these analyses, all other parameters are maintained at their \base" values. Figure 4.7a illustrates the distribution of the lateral bulging of the stone column along the depth with the column reinforcement length (Y/Lc): 0.5, 0.75, and fully reinforced (Y/Lc = 1). Considering the results presented in Figure 4.7a, we can conclude that the column bulging significantly decreased because of the fully reinforcement, at the length of the reinforcement 5, 7.5, and 10 m with a bulging value of 61.41, 35.27, and 20.02 mm, respectively. It has also been noticed that when the stone column is partially reinforced, it's bulging in the reinforced zone is less than that non-reinforced zone. However, the non-reinforced zone has higher values of the column bulging. The non-reinforced zone in the column starts with a maximum value that generates a largely differential lateral bulging at the endpoint of the reinforcement in the encasement depths 5 m, and 7.5 m. Besides, the bulging lateral values decrease gradually in depth until they reach zero at the column base.

Settlement ratio β, the ratio of the settlement of (VESC+HRSC) to that without reinforcement (OSC), is defined as β= (SHRSC+VESC/SOSC). In Figure 4.7a, the settlement ratios β are plotted against the column reinforcement length (Y/Lc). When increasing the length of the reinforcement, a decrease in the settlement ratios is noticed. For a length of the reinforcement (Y): 5, 7.5 and 10 m fully reinforced (Y = Lc), the ratio of settlement values are 0.75, 0.70, and 0.66, respectively. This shows that the reinforcement length is important in decreasing the settlement in the locally weak zone (LWZ). That full-length reinforcement is more efficient than half-length reinforcement or 75% of column length.

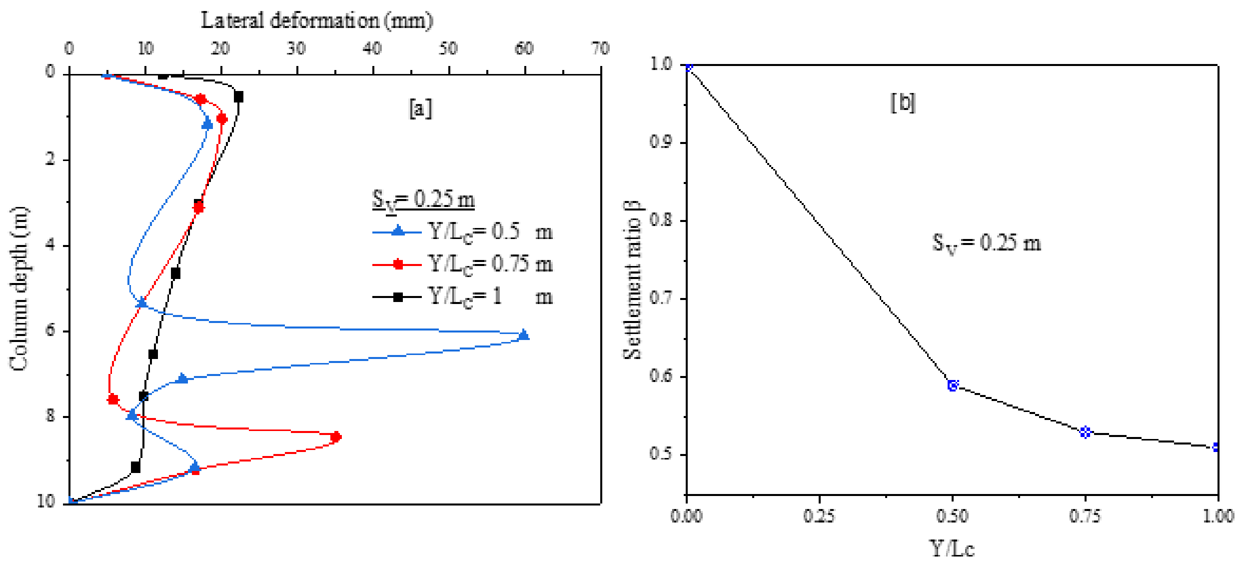


Figure 4-7. Distribution of the lateral bulging through its depth, (b) Ratio of settlement vs. reinforcement length of the (VESC + HRSC).

4.2.1.5.3. Influence of the stone column diameter (D)

Figure 4.8 shows the variation of the lateral deformation observed along the length of the stone column of various diameters of 0.60, 0.80, and 1.10 m, results of numerical studies show that the advantage of vertical and horizontal reinforcing increases with the increase of column diameter, the lateral deformations obtained are 21, 20.02, and 18.35 mm, respectively. as illustrated in the Figure 4.8a. Due to providing greater interactive shear mobilization at the top and bottom surfaces of reinforcing layers with aggregate materials in columns with larger diameters. However, for the VESC, Murugesan and Rajagopal 2006. reported that the benefit of vertical encasement decreases with the increase in the diameter of the VESC, this is illustrated in figure 4.8b, the bulging value is 30.56, 42.09, 48.52 mm, respectively. The load transfer mechanism from the soft soil to the reinforcing layers in the (VESC+HRSC) occurs by frictional surfaces from the column center to its periphery and may be amplified by interactive passive effects of reinforcement ribs in geosynthetic type reinforcing strips. Therefore, with increasing the column diameter, the area of the top and bottom surfaces between the reinforcement and column grains increases, leads to an increase in the effectiveness of the horizontal reinforcing layers with increasing the column diameter. However, the primary function of the encasement in the VESC is the hoop tension that decreases due to increasing the column diameter.

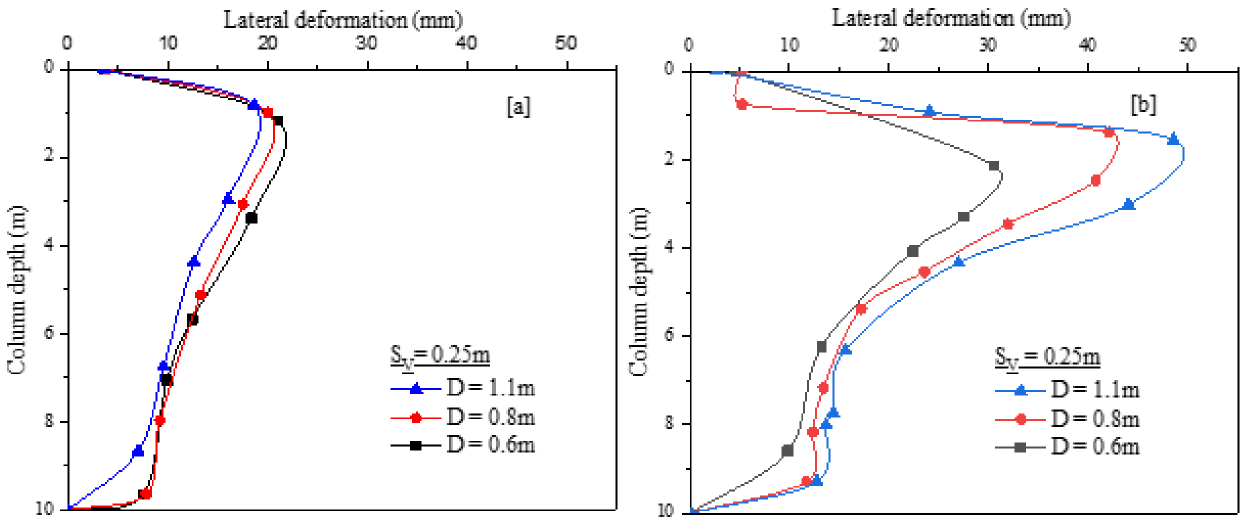


Figure 4-8. Lateral deformation vs. depth: (a) (VESC + HRSC), (b) (VESC).

4.2.1.5.4. Influence of depth of locally weak zone (LWZ)

Figure 4.9 shows the effect of the locally weak zone thickness for the settlement at the top of the stone column and surrounding soil. The findings of the present study suggest that there is an increase in the vertical settlement consequent with the increase in the depth of locally weak zone, as illustrated in figure 4.9, the vertical settlement for without the locally weak zone is 0.55 m, and with (LWZ) in depths of 1.5, 3, and 6 m are 0.69, 0.74, and 0.82 m respectively. The conclusion that Can be drawn is that the depth of the (LWZ) layer has a major impact on the instability of the embankment.

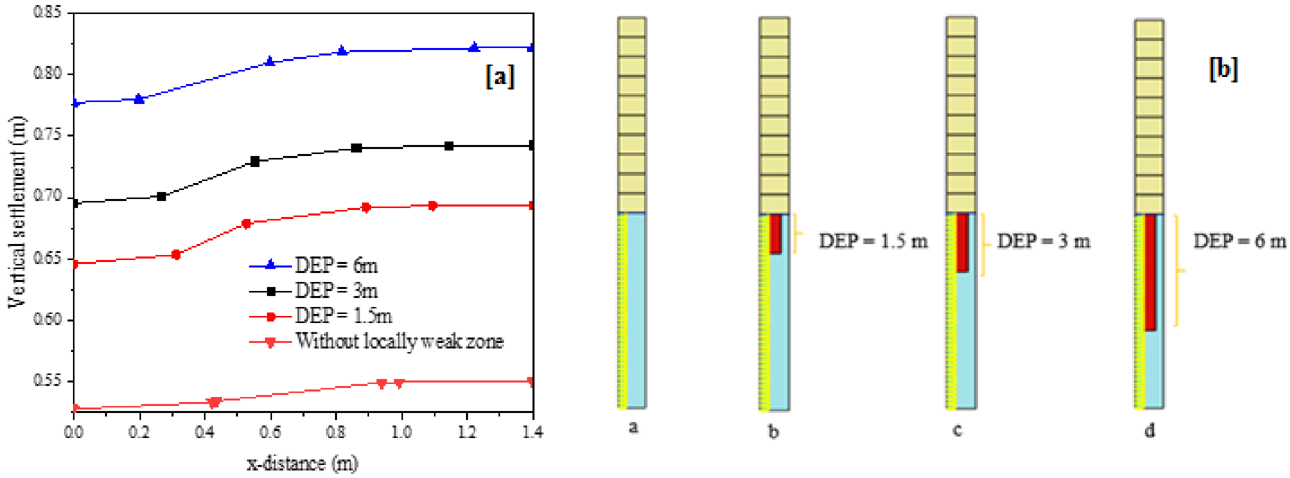


Figure 4-9. (a) Vertical settlement as a function of the depth of the locally weak zone (DEP); (b) Various models used in numerical analyses.

4.2.1.5.5. Influence of reinforcement stiffness (J)

The influence of the tensile stiffness of the geosynthetic used for reinforcement on the performance of the stone column has been investigated experimentally and numerically (Malarvizhi & Ilamparuthi 2004, Ayadat & Hanna 2005, Araujo et al 2009, Debbabi et al 2020). In the present study, the effect of the stiffness of a reinforced stone column is examined by choosing four values of tensile strength from 500, 2000, 3000 and 5000 kN/m, figure 4.10a shows the time-settlement behavior of stone columns with reinforcement stiffness. As seen in the figure 4.10b, the variation in the change of the settlement (Δs) at the end of the loading stage and final settlement value of a stone column at the end of the consolidation (long-term settlement). While parameter (SFEL) is defined as the settlement value of a stone column at the end of the loading stage (65 day), while parameter SFC is defined as the final settlement value of a stone column at the end of the consolidation. It is observed that when the tensile strength increases, the change in the settlement decreases; this is further illustrated in Figure 4.10b, the variation in the change the settlement (Δs) at tensile strength 500, 2000, 3000 and 5000 KN/m are 0.19, 0.08, 0.05 and 0.03, respectively. We conclude from these results the consolidation time can greatly decrease with tensile strength increase, especially in this case in the presence of a locally weak zone (LWZ) where the merging process ends before the rainy season.

Effect on settlement reduction ratio to better illustrate the degree of improvement in the soft soil is realized through the use of any type of ground improvement technique, a parameter called the settlement improvement factor (S.I.F.) defined as the ratio of the settlement caused by an ordinary stone column (OSC) to the settlement caused by a reinforced stone column (RSC) with the same conditions (long-term settlement) S.I.F = (SFOSC/ SFRSC), the more the value of (S.I.F) increases, the better the performance that has been realized due to ground improvement, and that's because with increasing tensile strength. For the tensile strength 500, 2000, 3000, and 5000 kN/m the settlement improvement factor value is 1.53, 2.36, 2.73, and 3.25 mm, respectively. Besides increasing strength and stiffness of the stone column, it prevents a lateral deformation of stones when the column is installed in extremely soft soils (LWZ).

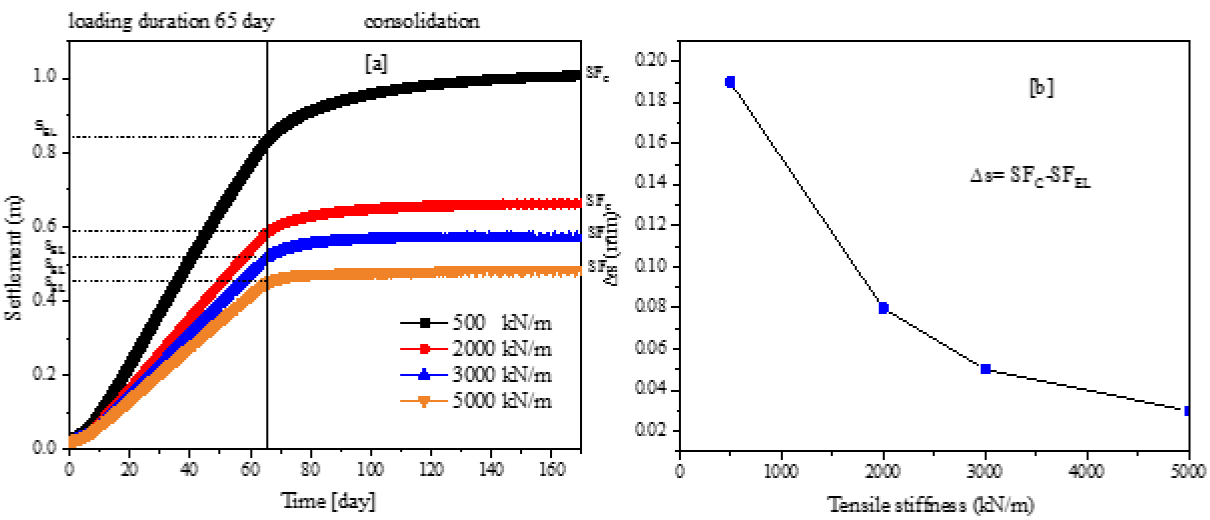


Figure 4-10. (a) Time-settlement behavior of various stone columns with different tensile stiffnesses; (b) Variation in Δs with tensile stiffness.

4.3. Conclusions

This study provides information on the behaviour of embankments supported by a combination of external reinforcement and internal reinforcement (VESC+HRSC) stone column in the locally weak zones. Several numerical simulations were performed to analyse the impact of (LWZ) on stone column with respect to lateral deformation and settlement behaviour. The following points emerged from the present investigation:

- The use of embankments supported by ordinary stone columns in the locally weak zones (Sabkha soil) is one of the biggest geotechnical problems, were considerable lateral bulging to much as 221.16 mm and vertical settlement by 1.56 m.
- The results of this study indicate that the use of the combination of external reinforcement and inner reinforcement (VESC+HRSC) in embankments supported by stone columns on locally weak areas (Sabkha soil) allows to obtain acceptable bulging by reducing the lateral deformation at the top of the column by 90.94%, it also reduces the vertical settlement of the stone column by 52.56%. Indeed, numerical results showed for a (VESC+HRSC) combination, a vertical settlement of 0.74 m and a lateral deformation of 20.02 mm vs. 1.56 m and 221.16 mm for an OSC.
- The combination of external reinforcement (VESC) and internal reinforcement (HRSC) are more effective in soft soils rather than the peripheral sheath or horizontal circular strips on its own.
- In (HRSC+VESC), with increasing the length of reinforced part of the columns, and decreasing interval spaces between reinforcement layers, the ultimate capacity and stiffness of stone columns increases. Vertical spacing between horizontal layers (Sv) has a dominant effect of stone columns.
- The benefit of encasement decreases with increasing the stone column diameter in (VESC), while it increases with increasing the diameter column in (VESC+HRSC).
- The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results an increase in the settlement at top of the stone column and surrounding soil. In the upcoming studies, these problems deserve further inspection.
- The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results in increases in the settlement of the stone column. The conclusion that Can be drawn is that the depth of the (LWZ) layer has a major impact on the instability of the embankment

GENERAL CONCLUSION

The contribution to the numerical analysis of embankments on soft ground as Sabkha with Geosynthetic -Reinforced Stone Columns (GRC) geomaterials demonstrate the undeniable interest of this reinforcement solution for compressible soils of low bearing capacity, where soft ground strengthening by Geosynthetic -Reinforced Stone Columns is a very effective technology, less expensive and more environmentally friendly, that is, it has less environmental impact compared to other techniques of soil strengthening. It is generally intended to improve the mechanical properties of soft soils. A treatment that thoroughly improves the loose soil in order to make it more likely to support light foundations for lightweight constructions such as embankment, oil tanks, engineering structures, etc.

This thesis consists of two main parts:

The first part is devoted to a bibliographic summary containing two chapters, the first generality of compressible soils and secondly methods for enhancement embankment over soft ground.

The second part consists of two chapters related to the numerical modeling of encased stone columns supporting embankments on sabkha soil on one side, and Numerical Modeling of Horizontally Layered Geosynthetic Reinforced Encased Stone Columns Supporting Embankment on Sabkha Soil (LWZ) on the other hand.

First, the generality of compressible soils were presented. In the second chapter, we presented a review of the important previous work published in the literature on improving the performance of foundations treated with geosynthetic reinforcement, stone columns, geosynthetic -reinforced stone columns (vertical encasement (VESC) and horizontal layers (HRSC)) in which special attention was paid to improving the performance of settlement and the lateral deformation. This bibliographical summary allowed us to draw the following conclusions:

Among all these methods, the use of a geosynthetic proves to be a cost-effective alternative solution in terms of saving natural resources, time and integrating sustainability and environmental protection. Geosynthetics perform various functions, namely, filter, separation, drainage, waterproofing, protection, and reinforcement. We have seen that geosynthetics can significantly increase the safety factor and the height of the embankment. It is also an increase in performance due to uniform settlements after embankment construction and reduced displacement during construction and this reduces the amount of material.

Through literary study (Experimental, Numerical, Analytical methods), geosynthetic - reinforced stone columns have proven to be an ideal solution to improve the performance of columns in soft soils especially sabkha soil. The columns may be encased with geosynthetics which are the main materials used to increase the strength and stability of geotechnical structures. In recent years there have been various studied on the performance of the encased columns considering column length, column arrangement and influence of encasement. However, additional studies are necessary.

Secondly, two different numerical studies were carried out using the PLAXIS 2D 2017 computer code to contribute to the issues presented. The first study exposes a two-dimensional axial symmetry analysis intended to evaluate the effect for the modeling of an embankment supported by encased stone columns (ESC) on soft soil structure on the one hand, and to verify the effectiveness of these effects on leveling the soft ground on the other hand. Based on the results obtained, the following conclusions were drawn:

- The study has focused on the evaluation of the maximum bulging of the stone column and on the settlement of the embankment. It has been demonstrated that ordinary stone columns (OSC) were ineffective due to excessive bulging (221.16 mm) caused by the lack of lateral pressure. On the other hand, the encased stone columns (ESC) showed good behavior, namely a much reduced bulging (42.09 mm) and a reasonable settlement (0.962 m vs. 1.560 m for an OSC). So that it is possible to build safe and very high embankments.

- The area replacement ratio (ARR) leads to two opposite effects. On the one hand, it decreases the value of settlement ratio (β), but on the other hand, it decreases the effectiveness of the encasement by increasing the lateral deformations. Furthermore, this numerical analysis has shown that the increase in the internal friction angle of the stone column material leads to an increase in the resistance of the column against failure and, consequently, the lateral deformations and settlements of the column decrease in (LWZ). The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results in increases in the settlement of the column. Increasing the height of the embankment increases the vertical stress above the column and the compressible soil (Sabkha soil).

- The consequence is an increase in the horizontal stress exerted on the encased stone column. On the other hand, the depth of radial bulge is affected by the height of the embankment, as it increases with the height of the embankment increases. Increase in the stiffness of the geosynthetic encasement of stone columns leads to increases in the column stiffness, the hoop tension force mobilized in the encasement, and the lateral confinement provided to the stone column. Where the hoop strains in the geosynthetic encasement are at a maximum near the top surface and decrease with depth. The stone column encasement causes reducing the total stress in the soft soil along with consolidation.

The second numerical study carried out, provides information on the behaviour of embankments supported by a combination of external reinforcement and internal reinforcement (VESC+HRSC) stone column in the locally weak zones. Several numerical simulations were performed to analyse the impact of (LWZ) on stone column with respect to lateral deformation and settlement behaviour. The following points emerged from the present investigation:

The results of this study indicate that the use of the combination of external reinforcement and inner reinforcement (VESC+HRSC) in embankments supported by stone columns on locally weak areas (Sabkha soil) allows to obtain acceptable bulging by reducing the lateral deformation at the top of the column by 90.94%, it also reduces the vertical settlement of the stone column by 52.56%. The combination of external reinforcement (VESC) and internal reinforcement (HRSC) are more effective in soft soils rather than the peripheral sheath or horizontal circular strips on its own.

- In (HRSC+VESC), with increasing the length of reinforced part of the columns, and decreasing interval spaces between reinforcement layers, the ultimate capacity and stiffness of stone columns increases. Vertical spacing between horizontal layers (Sv) has a dominant effect of stone columns.

The benefit of encasement decreases with increasing the stone column diameter in (VESC), while it increases with increasing the diameter column in (VESC+HRSC). The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results an increase in the settlement at top of the stone column and surrounding soil. In the upcoming studies, these problems deserve further inspection. The reduction in differential settlements is sensitive to the geometry of the locally weak zone, increasing in the depth of (LWZ) results in increases in the settlement of the stone column. The conclusion that Can be drawn is that the depth of the (LWZ) layer has a major impact on the instability of the embankment.

Although the conclusions reached in this study cannot necessarily be generalized to all cases with different geometries and soil/geosynthetic properties, they do provide a useful indication of general trends in behavior of embankments supported by encased stone columns in weak zones so that it is possible to build safe very and high embankments. Future experimental research is needed in this area to validate the simulation-based observations that are made herein and to better understand the behavior of embankments supported by encased stone columns in soft ground.

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