GROUND IMPROVEMENT USING ENCASED STONE COLUMNS

Thesis submitted in fulfillment of the requirements for the Degree of

DOCTOR OF PHILOSOPHY

By

ANKIT THAKUR

Department of Civil Engineering

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, DISTRICT SOLAN, H.P., INDIA

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………DEDICATED TO

MY PARENTS………

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DECLARATION BY THE SCHOLAR

I hereby declare that the work reported in the Ph.D. thesis entitled **"Ground Improvement Using Encased Stone Columns"** submitted at **Jaypee University of Information Technology, Waknaghat, Himachal Pradesh, India,** is an authentic record of my work carried out under the supervision of **Prof. Ashok Kumar Gupta& Dr. Saurabh Rawat.** I have not submitted this work elsewhere for any degree or diploma. I am fully responsible for the contents of my Ph.D. Thesis.

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SUPERVISOR'S CERTIFICATE

This is to certify that the work reported in the Ph.D. thesis entitled **"Ground Improvement Using Encased Stone Columns",** submitted by **Ankit Thakur** at **Jaypee University of Information Technology, Waknaghat, Himachal Pradesh, India,** is a bonafide record of his original work carried out under my supervision. This work has not been submitted elsewhere for any other degree or diploma.

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(Ankit Thakur)

ABSTRACT

The problem of soft soils along coastal regions, weak subsoil conditions and poor fill soils renders construction difficulties for geotechnical engineers. To conquer these obstacles, a variety of soil improvement strategies are available. One of the most common methods for supporting construction in such site conditions is the use of granular stone columns. However, due to the low lateral confinement offered by the surrounding weak soil, stone columns undergo failure due to squeezing of stone column material into the adjacent soil. This diminishes the structural integrity of the employed technique and renders a compromised load capacity and settlement behaviour. Thus, encasement of granular columns has been utilized for rectifying the aforementioned concern.

The present study investigates a typical soil condition readily encountered by the site and Geotechnical engineers consisting of weak cohesionless soil over a comparatively stiffer underlying soil layer. Such soil profiles have been reported in the literature and are a common condition found along the Indian coastal region, parts of the mainland and abroad. The use of stone columns in soft cohesive soils for improving load capacity is comprehensively investigated and reported in the available literature. However, amongst the very few studies of reinforcing poor cohesionless soil using rammed stone columns, the outcomes from the present study will contribute significantly in precisely deciphering and validating the load capacity and failure mode depicted by conventional floating stone columns when installed in cohesionless soil condition. The present study further adds to the existing literature by investigation of effect of vertical and horizontal encasing/reinforcing the stone columns for mitigation of bulging failure suffered by conventional floating stone columns under compressive load in cohesionless medium.

In the present study, model testing of a single unreinforced stone column (diameter 40 mm and length 300 mm) treated as a representation of a conventional stone column has been conducted for examination of its load bearing capacity, consequent settlement and failure undergone when subjected to a gradual increasing compressive load. The study further investigates the variation in load – settlement characteristics and failure pattern as the unreinforced stone columns are converted into reinforced stone columns by applying two

distinctive geotextile reinforcing orientations: vertical (encasement) along the full length of columns and horizontal (circular disc) placed at regular spacing within the stone column length.

In addition to this model testing of unit cell, the present study also deals with the assessment of group of unreinforced and reinforced stone columns under two different configurations namely triangular and square. The two arrangements of three and four stone columns are also subjected to model testing under compressive lading and evaluated for load – settlement and failure mechanism. For more intensive analysis, load ratio and stress concentration ratios are also investigated.

The model testing results depicted that the load carrying capacity increase with geotextile reinforcement as compared to unreinforced columns. The horizontal reinforcement renders a higher load capacity and smaller settlement with horizontal reinforcement than vertical encasement of floating stone columns. The pattern is unanimous for both single and group of three and four stone column configurations. The failure mechanism depicted arrested bulging for reinforced stone columns than unreinforced stone columns. Exhumation of both vertical and horizontal stone columns showed restrained bulging near the top of stone column whereas for unreinforced aggregates of stone columns were found to be squeezed into the surrounding cohesionless soil. It is also discovered that both the load ratio and stress concentration ratios are related to the failure mechanism observed.

The validation and further investigation of single columns, numerical modelling using axisymmetric simulation in PLAXIS 2D software has been carried out. Likewise, for accurate modelling of both unreinforced and reinforced stone column groups, three – dimensional finite element (FE) analysis has also been adopted through Plaxis 3D. According to the FE results, restrained bulging with higher load bearing capacity is observed for horizontally reinforced stone columns than vertically encased floating columns. Similar to model testing, bulging is observed during failure. However, the results from FE analysis are found to be on a higher side than the testing results due to the incompetency of the model in modelling the installation effect of stone columns. Within the allowed spectrum of variance, the experimental and FEM findings are considered to be in strong agreement.

The model testing results are also validated using codal provision and theories for recorded load capacity and settlement observed. It is observed that for permissible settlement of 30 mm, results from available codal provisions renders upper boundary results than observed experimental results for all the cases of unreinforced and reinforced single and group stone columns. Moreover, theoretical and empirical relationship for horizontal reinforced stone columns is virtually non – existent in literature and hence observations made from the present study add significantly to the database for future investigators. The results from model testing and theoretical analysis depicted variance within a range of 15 – 25%.

Based on the results from model testing, FE axisymmetric and three – dimensional numerical modelling and theoretical codal predictions, it can be concluded that weak cohesionless soil can be improved for bearing capacity and minimization of settlement using stone columns. However, inclusion of reinforcement in the form of circular discs renders higher load carrying capacity than unreinforced and vertical encased floating columns. For reinforced stone columns, vertical and horizontal reinforcement render equal effect in load carrying behavior. However, FE analysis result depicted higher load capacity for horizontally reinforced than vertical reinforced stone columns with three and four stone columns, respectively. Furthermore, it can be concluded that stress concentration ratios differ significantly between unreinforced and reinforced, being highest for horizontally reinforced columns. The variation can be accounted based greater contribution of mobilized shearing stress at aggregate – Geotextile – aggregate interface than at vertical aggregate – Geotextile – soil interface during column settlement. Moreover, it can also be concluded that both FE analysis without installation modelling and reported codal provisions yielded conservative results varying about 25% from the actual testing results.

Keywords: Stone columns; encasement; horizontally reinforced; load – settlement; finite element; codal provisions; stress concentration ratio

LIST OF ACRONYMS AND ABBREVIATIONS

Abbreviations

ESC	Encased stone columns
OSC	Ordinary stone columns
SC	Stone columns
SCF	Stress concentration factor or ratio
SP	Settlement plate

LIST OF SYMBOLS

Symbol	Dimension	Meaning
Latina Symbols		
A_c	[L^2]	Cross-sectional area of the column
A_e	[L^2]	Cross-sectional area of the unit cell
A_s	[L^2]	Cross-sectional area of the surrounding soil
a_s	[1]	Area replacement ratio
B	[L]	Half width of the plain strain unit cell
b_c	[L]	Half width of the wall in the plain strain unit cell
C	[1]	Material constant
c	[F/L^2]	Cohesion
$c`_r$	[L^2/T]	Modified coefficient of radial consolidation
$c`_v$	[L^2/T]	Modified coefficient of vertical consolidation
C_c	[1]	Compression index
c_r	[L^2/T]	Coefficient of radial consolidation
C_s	[1]	Swelling index
c_u	[F/L^2]	Undrained shear strength
c_v	[L^2/T]	Coefficient of vertical consolidation
C_α	[1]	Secondary compression index
d	[L]	Diameter of the stone column
d_e	[L]	Diameter of the unit cell
E	[F/L^2]	Elasticity modulus
e	[1]	Void ratio
E_0	[F/L^2]	Initial Elasticity modulus
E_{50}	[F/L^2]	Secant modulus at 50% strength
E_c	[F/L^2]	Elasticity modulus of the stone column
E_s	[F/L^2]	Elasticity modulus of the soil
E_{ur}	[F/L^2]	Unloading and reloading Elasticity modulus

Symbol	Dimension	Meaning
H	[L]	Thickness of the soil
h	[L]	Encasement depth
J	[F/L]	Stiffness of the geosynthetic materials
k_h	[L/T]	Horizontal permeability
K_o	[1]	Coefficient of earth pressure at rest
k_v	[L/T]	Vertical permeability
L	[L]	Length of the stone column
m_{VC}	[L²/F]	Coefficient of compressibility of the stone material
m_{VS}	[L²/F]	Coefficient of compressibility of the surrounding soil
N	[1]	Diameter ratio
n	[1]	Number of geosynthetic layers
n_s	[1]	Steady stress concentration ratio
P	[F/L²]	Applied pressure
p'	[F/L²]	Effective mean stress
p_o	[F/L²]	Initial effective mean stress
q	[F/L²]	Applied load
r_c	[L]	Radius of the stone column or the drain well
r_e	[L]	Radius of the unit cell
R_s	[1]	Ratio between the total settlement of the reinforced clay with ordinary stone columns and the non-reinforced clay
S	[L]	Spacing distance
t	[T]	Time
T	[F/L]	Hoop tension force using geosynthetic materials
u_o	[F/L²]	Original pore water pressure
U_v	[1]	Degree of vertical consolidation
v	[L]	Vertical displacement
x	[L]	Horizontal distance from the column centerline
y	[L]	Depth under the top of the column
z	[L]	Depth in soil

Symbol	Dimension	Meaning
Greek Symbols		
α	[1]	Rate of the increase of the modulus of elasticity of the granular pile with depth
γ	$[F/L^3]$	Soil unit weight
γ_d	$[F/L^3]$	Dry soil unit weight
γ_{wet}	$[F/L^3]$	Wet soil unit weight
ε_1	[1]	Vertical strain
ε_a	[1]	Axial strain
ε_v	[1]	Volumetric strain
κ^*	[1]	Modified swelling index
λ^*	[1]	Modified compression index
μ^*	[1]	Modified secondary compression index
ξ	[1]	Poisson ratio factor
σ	$[F/L^2]$	Applied average stress
σ'_r	$[F/L^2]$	Effective radial stress
σ'_{ro}	$[F/L^2]$	Initial effective radial stress
σ'_v	$[F/L^2]$	Effective vertical stress
σ'_{vo}	$[F/L^2]$	Initial effective vertical stress
σ_1	$[F/L^2]$	Major stress, (maximum vertical stress in triaxial test)
σ_3	$[F/L^2]$	Minor stress,(confining pressure in triaxial test)
σ_h	$[F/L^2]$	Horizontal stress
σ_s	$[F/L^2]$	Stress in soft clay ground
σ_v	$[F/L^2]$	Total vertical stress
φ	[1]	Friction angle
ψ	[1]	Dilatancy angle
ν	[1]	Poisson ratio
ν_c	[1]	Poisson ratio of the stone

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CHAPTER-1
INTRODUCTION

1. 1 GENERAL

The chapter deals with the motivation and need of study for undertaking the present research work. It highlights the available ground improvement techniques with focus on the improvement method using stone columns. The chapter also cover the primary parameters used for the design of stone columns. Enhancement of unreinforced stone column method to reinforced stone columns based on its mechanism has also been reported. The chapter – wise organization of the thesis has also been presented.

1.2 MOTIVATION

Due to urban/industrial civilization, availability of construction sites with required soil properties are readily falling short. Therefore field engineers are constrained by the need to employ sites with weak strata and complicated behavior due to presence of problematic soil layers and their varied engineering properties. Such abandoned sites, often comprises of in – situ soft clay yielding significantly low shear strength, high compressibility accompanied by excessive settlement or weak cohesionless soil with poor bearing capacity, sites prone to liquefaction, landslides or fill areas associated with global/local stability concerns. Hence to maintain the structural integrity and serviceability, thorough site preparation before undertaking any construction activity becomes mandatory. However, among the prevalent ground improvement/modification techniques, precise selection and implication of an effective solution should be in accordance to the economical viability and design specification [1].

One of the remedial measure which has been majorly employed in the last decade involves insertion of granular material within the weak soil deposit in a cylindrical form (column) using the process of rammed compaction or vibration compaction. This process of soil reinforcement through columns of granular material (stone columns) is found to perform satisfactorily in loose sandy, sandy silts to alluvial silty – clays and soft cohesive soils (Ketkar & Telang, 1994; Manish Kumar, et al., 2002). With its inception in 1970, the concept of stone column has been used to improve/ modify/ rectify/ renovate underlying soil and foundations of various types of structures such as high – rise buildings, towers, industrial plants, petroleum storage tanks etc.

Ultra High Voltage Direct Current (**UHVDC**) transmission project was executed by Keller Ground Engineering India Pvt Ltd. for the construction of India's first Ultra High Voltage Direct Current (UHVDC) of 6,000MW capacity energy highway project undertaking of Powergrid Corporation of India Ltd (PGCIL) at Biswanath Chairali, Assam [2]. An area of over 10,00,000 m^2 has been modified to facilitate the installation and construction of service and other heavy building structures, valve control hall, transformers, and tower foundations. During the site investigation, it was revealed that the subsoil consisted of poorly graded loose to medium fine sand underlain by medium to stiff silt layer followed by dense silt layer. The site lying in seismic Zone V depicted a very low bearing capacity in addition to liquefaction susceptibility. The site was successfully rectified by installation of stone columns of diameter 0.8 – 1.1 m using top feed vibro – stone column technique. The efficacy of the improved ground was also validated using stone column load tests. The executing firm Keller solutions also claimed that the stone column method also yielded lower carbon footprint than the other alternative solutions.

Likewise, Keller also executed installation of 0.9 m diameter of stone columns for improving the low bearing capacity and long – term settlement problem of approx. 1,00,000 m^2 area needed for stacking of 4 – 5 containers along with beams as a part of an expansion project of existing terminal facilities at Pipavav, Gujarat by Pipavav Port Ltd (GPPL), a group company of A.P. Moller – Maersk [3]. Similarly the foundation for 49 steel storage tanks for storage of crude oil, polishing units, demineralized water units and product storage was also remediated using vibro stone columns. [2]. A significantly large stretch of 6,00,000 m was delivered with stone column foundation with length of 10 m and diameter 0.8 m for allowing controlled settlements and increased bearing capacity needed for working of over 10 rigs. The load tests on single and group of stone columns depicted satisfactory performance deliverables for the improved ground.

Other such praiseworthy projects utilizing stone column improvement technique have also been carried out by Soil and foundation Experts. It includes construction of a temple in

Rohtak, Haryana over granular piles having diameter of 12 m and rammed stone columns with 500 mm diameter. Similarly, rammed stone columns of diameter 400 mm with granular piles of 7 m length have also been employed for restoration of about 1000 pillars of Kalyana Mandapa situated at Hanuma Konda, Warangal. Likewise, picture tube plant at Karzan, Vadodara has also been constructed over gravel piles having diameter of 300 – 400 mm. Foundation support for Fire Water tank in LPG Bottling Plant under Indian Oil Corporation Ltd. at Madanpur Khadar has also been carried out using group of gravel columns 3 × 22 m [4]. Recently, the construction committee for foundation construction of the famous Ram Temple at Ayodhya has also recommended the use of vibro – stone columns for supporting rafts foundation [5]. Thus, it can be seen that stone column construction has been widely executed under various superstructures, soil conditions and a viable solution to improve weak ground conditions.

1.3 NEED OF STUDY

The Indian sub-continent has a significantly large coastline extending about 6000 kms from west to east, covering the entire southern region of the subcontinent. Due to the rapid development of coastal areas for ports and industries, stabilization of soil for supporting these heavy structures is a common need. As per the prevalent soil profiling in such areas, loose/weak sand deposit in the top subsoil layer is not uncommon. The soil profile of off – shore Hoogli delta, Kolkata, India is identified with loose fine sand and silty sand upto a depth of approximately 10 m [6] Similarly, obtaining loose sandy soil deposit as top subsoil layer upto significant depth is also found in land, primarily in areas around Rajasthan, parts of Punjab and Gujarat [7]. For example, during the construction of thermal power plant at Goindwal Sahib, Punjab, India, the subsoil was found to be loose sand with a low relative density of less than 40%. The loose sandy soil comprised of only 4% - 6% of fine content, inducing a liquefaction potential to the subsoil [8]. Soil profile depicting sand fill layer (1 m) above clean sand layer of 4 m with very loose to very high density has also been encountered during the construction of a 10-story apartment building in Vila Velha, Espirito Santo, Brazil. Similarly, soil profile with the uppermost layer of 6 m consisting of loose to medium dense sand soil over 7 m of dense sand has also been modified at Espirito Santo, Brazil during the construction of a 6-story apartment building [9]. Hence obtaining loose sand soil over a hard stratum of dense sand /stiff clay is common for site engineers. Since a many reported studies of modifying cohesionless soil using stone columns [10] are available, a comprehensive investigation regarding the reinforcing action on shallow cohesionless soil layer becomes necessary. It also adds up to the contribution of providing design approach without detailed subsurface investigation.

Bulging is found to be the most common mode of failure for all types of stone column configurations (end bearing and floating) in homogeneous/layered soil system. Therefore, finding a viable solution for rectification of bulging failure without compromising the integrity of the modification technique becomes paramount. Encasement of stone columns in cohesive soils has been reported by many researchers in the past [10-19], however encasement of stone columns within surrounding cohesionless soil is addressed by only a few researchers [10]. Similarly, variation in the orientation of reinforcement also is an alternative which requires subsequent comprehension of mechanism prior to field execution. Hence based on the aforementioned reasons, the present study for investigation of conventional and reinforced (vertical and horizontal) stone columns in cohesionless soil has been undertaken.

1.4 GROUND IMPROVEMENT METHODS

The practice of improving the ground and making the soil subsurface ready for a geotechnical project has been followed since ancient period. However in the last few decades, there is a significant development in this research area with the invention of more robust methods and better understanding of soil performance. Mitchell et al. [20] have provided brief definition of ground stabilization as "the controlled alteration of the state, nature or mass behavior of ground materials in order to achieve an intended satisfactory response to existing or projected environmental and engineering actions".

Chu et al. [21] have broadly categorized the techniques for the soil improvement into four classes. (1) The first class comprises of soil enhancement methods without admixtures such as soil replacement, preloading, sand drains etc. [22-26]. The poor soil can be improved by replacing it with some other capable materials such as sand, gravel or crushed rock and any other soil of same type. This method of replacement of soil is the oldest and the simplest method for improvement of soil. The replacement of soil can effectively reduce the compressibility of the soil, thereby increasing its bearing capacity.

Alternatively, preloading can be used for soil stabilization in which the soil is covered at the top with a surcharge fill, which accelerates settlement before the construction. Once the required consolidation takes place, the layer of the fill is removed and the soil is ready for construction. The technique shown in Figure 1.1 is suitable for clayey soil where sand or vertical drains may be used to speed up the settlement process by reduction in drainage path.

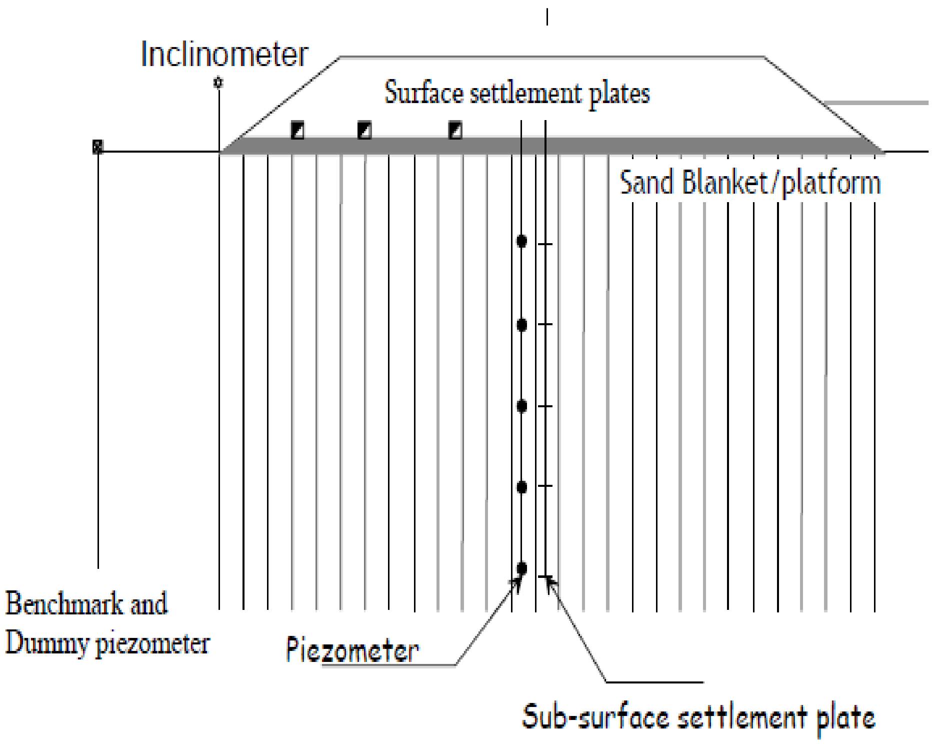


Figure 1.1 Vertical Drain system with preloading [23]

(2) The modification using admixtures or insertions (e.g. stone columns, compaction piles) form the second category of the soil improvement methods [27-29]. This category of modification is also referred to as “in-situ” densification since it enhances the density of the soil under consideration. Another method of installation of stone column is done by vibro – replacement method as shown in Figure 1.2.

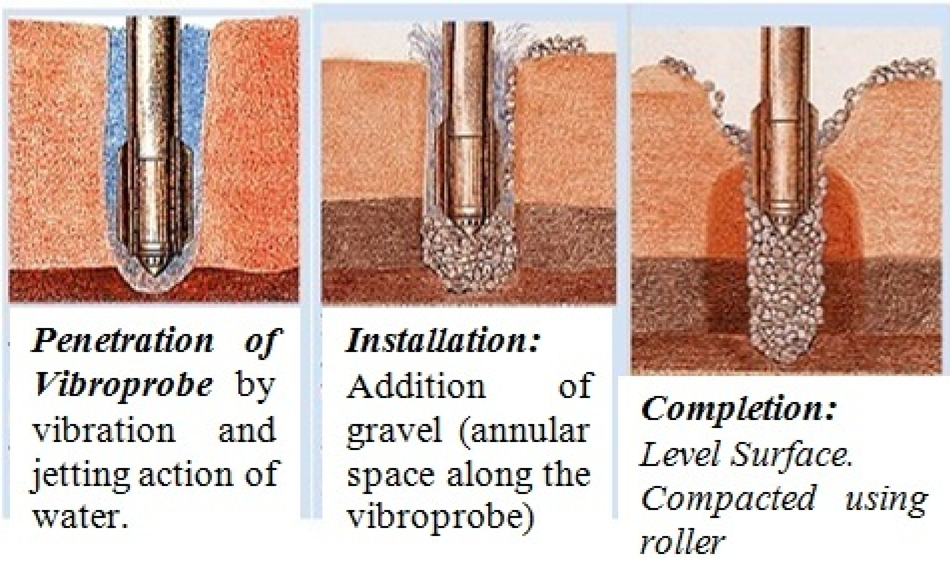


Figure 1.2 Installation of stone column by vibro – replacement method [30]

(3) The third group includes the stabilization of ground using additives and the common methods used are chemical stabilization, jet grouting and deep mixing [31-35]. The common methods for chemical stabilization include the usage of cement, lime, and fly-ash. Out of these, fly ash is proven to be efficient in modification processes due to its cementitious characteristics. The formation of deep mixing columns also stabilizes the soil at large depths. The method involves the injection of binder (lime or cement) and blending it with a soil by mechanical mixing to form columns. Jet grouting is another way of improving the properties of soil. As shown in Fig. 1.3, the method of jet grouting involves the removal of sand layer with a stream of dry air along with the insertion of cement slurry with lower jet.

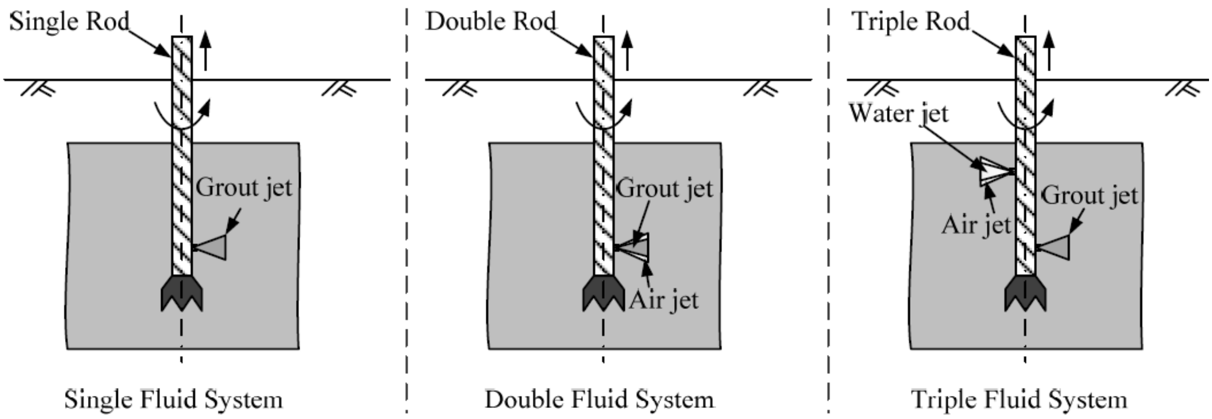


Figure 1.3 Jet grouting [35]

(4) Lately, thermal stabilization methods like ground heating and ground freezing are also used for improvement of unstable soil [36]. Thus, differentiating on the basis of methodology and nature of work, the available ground improvement methods can be summarized as given in Table 1.1.

Table 1.1 Different types of ground improvement techniques based on methodology

S. No.	METHODOLOGY	PROCESS
1.	Replacement/weight reduction method	Removal of unsuitable soil, Use of light material such as fly ash, slag, geofoam, tyre chips, wood chips etc.
2.	Pre – compression or Pre – loading	Preload the in – situ weak/poor soil prior to construction
3.	Consolidation	Use of wick drains, vacuum consolidation
4.	Densification	Static compaction, vibrofloatation, dynamic compaction, ***stone columns***, blasting and grouting
5.	Soil reinforcement	Use of geosynthetics (geotextiles, geogrids etc.), soil nailing, micropiles, anchors and tiebacks
6.	Stage construction	Construction of embankments in parts/stages
7.	Chemical soil treatment	Grouting, use of admixtures, deep mixing
8.	Biotechnical stabilization	Use of flora or vegetation (brush layering, brush matting etc)
9.	Thermal stabilization	Artificial ground freezing (AGF), Use of energy piles

1.5 GROUND IMPROVEMENT USING STONE COLUMNS

Among various modification techniques, the stone columns are preferred for supporting a range of structures in weak soil owing to the rigidity provided by the columns. Stone columns are widely used for upgrading the load carrying capacity of poor soils and making it possible to construct a stable foundation on the soil. They also enhance the stiffness, shear strength and drainage of the soil. The technique of stone columns has been widely used to support the geotechnical structures ranging from river embankment, small footings, and roadway embankment building foundations, oil storage tanks and bridges. The ground modification through stone columns results in increase in load carrying capacity of soil and settlement reduction. The relatively established technique of stone column construction which has been evolving since 1960 is executed either on a firm ground as end bearing columns or within the weak soil layer, in the form of floating columns. The ability of stone columns usually depends upon the length of columns, properties of the surrounding soil, method of construction and the properties of the stone fill. Columns that are formed on the stiffer section of soil are generally more usable owing to their better load – settlement behaviour than the floating stone columns.

During loading, there is downward force on column that causes shear stress at column interface and the nearby ground. Thus, the load gets dissipated along column length with the upper part of the stone column bearing the major fraction of applied load. However, if the lateral pressure due to confinement and the rigidity of untreated soil fails to support the load, aggregates of the stone column squeezes into the surrounding soil leading to bulging of the top of the stone column. However, this bulging does not necessarily end up with column failure. The development of these radial stresses under the applied axial load leads to the mobilization of shear stresses within the stone column material. Thus, columns constructed of material have significant angle of friction is preferred as it not only provided large shearing resistance but also better drainage potential.

However, the behaviour of group of stone columns is quite different from that of the isolated stone column. When group of columns is installed in soil, stone column reinforced soil is loaded using a relatively rigid foundation which results in both the stone columns and surrounding soil experiencing equal amount of settlement under load. The load transfer usually depends upon the rigorousness, columns spacing and the size of the granular particles comprising the columns. The greater compressibility of the stone columns makes them expand radially into the nearby soil resulting into the axial shortening of the columns. This effect forces the soil to change its volume. The radial expansion of the adjacent columns provides additional confinement, still there is larger expansion in upper section of stone column since confining pressure and stiffness of soil is less there. The vertical stress gets transferred between the column and the nearby soil depending upon their relative rigidity when load is applied. The resulting consolidation increases confinement of the column and the hardness of the soil until there is equilibrium between the column and the nearby soil.

Therefore, the modes of failure for isolated stone column (SC) are found to be governed by three main criteria's:

[a] The length of stone column (end bearing or floating)
[b] The surrounding soil condition (Homogeneous or layered)
[c] The loading area

For homogeneous soil condition as shown in Fig. 1.4 with loading area equal to the stone column diameter, both end bearing and floating SCs are found to fail due to expansion when the column length exceeds its critical length. For the columns of length less than the critical length, shear failure occurs in the columns with rigid base while in floating columns, both shear and punching failure is encountered.

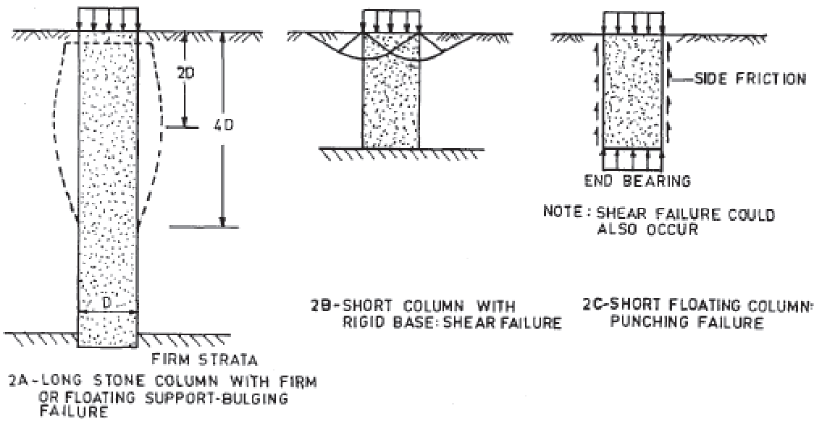


Figure 1.4 Modes of failure for stone columns in homogeneous soils [37]

However, as the load is applied on an area greater than that of the column diameter (Fig. 1.5), the bulging of the column reduces ultimately which results in greater load bearing efficiency of SC. Likewise, greater load distribution efficiency between the column and nearby soil due on their relative stiffness also results in less settlement.

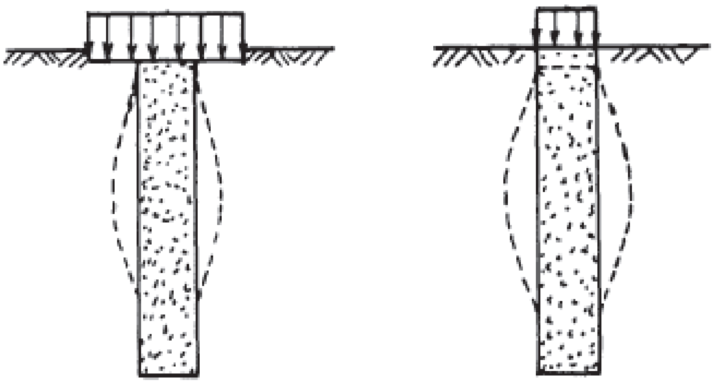


Figure 1.5 Load applied to the stone columns [37]

For SC installed in non – homogeneous (layered soil strata), the failure mode of stone columns depends upon the H/D ratio, where D is the diameter of the SC and H is the thickness of the soft soil strata. In general bulging or shear failure is encountered in SC region lying in soil strata having the least stiffness amongst the different soil layers. For SC lying in soil strata with H/D ratio less than 1, only restricted local bulging is found. However, for soil condition with H/D ratio greater than 2, it is found that significant large bulging occurs along the SC length resting within the soft layer. The failure modes of SC encountered in non – homogeneous soil condition are shown in Fig. 1.6.

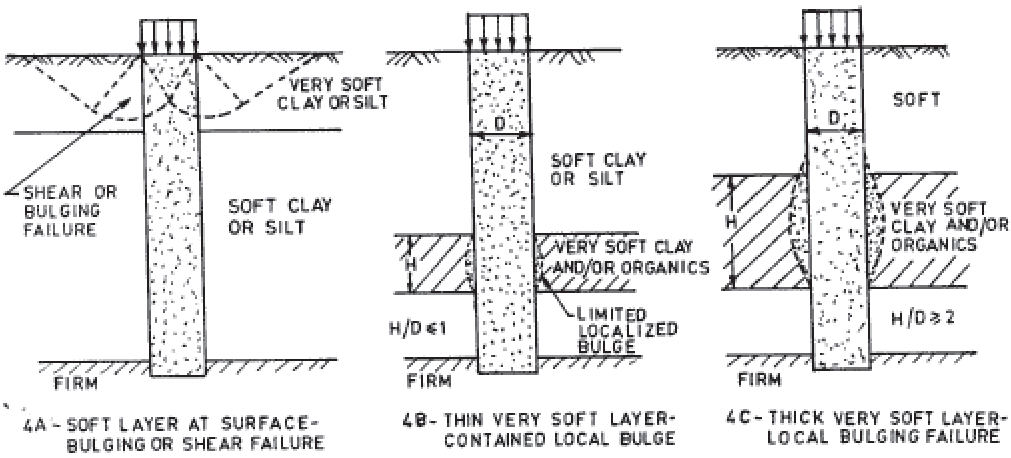


Figure 1.6 Different modes of column failures in non-homogeneous cohesive soil [37]

1.6 INSTALLATION OF STONE COLUMNS

The method of stone column installation is also known as vibro – replacement technique. In this method, a borehole is formed in the soil with the help of a vibrating poker and is filled with coarse aggregates of gravel and crushed rock. These aggregates are then densified with the help of poker resulting in tightly interlocked stone columns. The compactness of the aggregates depends upon the soil conditions and stone columns installation methods as well. Commonly used aggregates for building stone columns comprises of typical rocks which are crushed and uniformly grinded to form small particles of the size ranging between 10 mm – 75 mm. The size for the aggregates depends upon the method of installation. The installation method for SC can be split into two methods: top – feed method and bottom feed method. In case of top – feed method, the aggregates are fed to the borehole through the space to the vibrator tip from the ground level which is created by the lifting of vibrator to a few millimetres above. The borehole is created using a vibrator that is penetrated to the design depth using the weight and vibrations from the vibrator. In addition to the vibrator, the borehole execution is accompanied by air jets which are located at the vibrator tip. The vibrator is then lowered, which displace and consequently further densifies the underlying aggregates. During top feed operation, the size of the aggregates is usually kept large so that they can easily reach the base of deeper bore holes, whereas, in bottom feed methods, the aggregates are fed to the borehole using an attached feeder pipe. In case of dense strata, the design depth is reached by the vibrator only after pre – drilling of the dense layer. In case of bottom – feed method, the maximum particle size depends upon feeder tube size and lies between 10 mm to 40mm. The aggregates are compacted to about 60-100% of their relative density depending upon the compaction forces applied to them. Sometimes the fine gravel or sand particles are used to prevent voids in the columns and therefore prevent them to act as vertical drains.

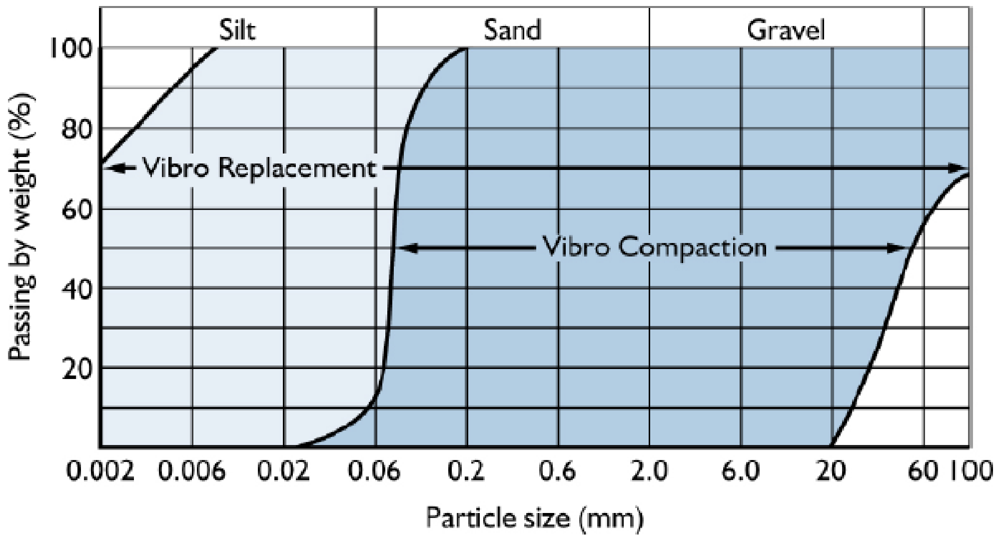


Figure 1.7 Installation methods for different types of soil types

Installation of SCs using vibro – replacement and vibro – compaction method depends upon the efficiency of the method which is a function of the in – situ soil type as shown in Fig. 1.7. It is observed that as the particle size of soil increases from silt to sand, construction of stone columns using vibro – compaction is preferred over vibro – replacement method. Thus, for enhancement of properties of soft cohesive soil, stone columns are usually constructed using vibro – replacement method. In case of non – cohesive soils (35% of sand and less than 15% silt and clay) which are accompanied by high liquefaction susceptibility concerns are found to render limited applications for geotechnical purposes. Stone columns constructed using vibro – compaction technique has been reported in effectively mitigating the liquefaction potential of sandy soil, thereby stabilizing the site against seismic events.

1.7 DESIGN OF STONE COLUMNS

Stone column's performance particularly depends upon soil surface type, column installation method, depth of the column, area under treatment and stone column type whether floating columns or end bearing piles. In addition, behaviour of stone columns is significantly influenced by its basic design parameters:

(1) **Stone column diameter, D:** Stone column diameter varies with the compressibility of the surrounding soil. Diameter of stone column formed is larger when soil is softer. Due to the lateral displacement of the stones during installation, diameter of column gets increased compared to the initial size of the borehole and final size depends upon soil

type, its un drained shear strength, installation method, vibrator type and the size of the aggregates of which the column is formed.

(2) **Pattern of stone column**: Pattern in which columns are installed during the construction is another factor that decides the performance of stone column. The three patterns for installation include triangular, square and hexagonal, out of these the equilateral triangular configuration forms the compact packing of the columns.

(3) **Spacing between the columns:** It is important to optimize the distance between the columns using field trials for big projects to obtain the required load bearing ability of the soil and settlement of the structure.

(4) **Replacement ratio (a_s):** In most of the settlement analyses, the ground under observation is represented as a unit cell containing surrounding ground and stone column. The soil replaced by the stone column is quantified in terms of area replacement ratio, a_s.

$$a_s = \frac{A_s}{A_s + A_c} \tag{1.1}$$

where A_c = cross sectional area of stone column; A_S = Area of soil around the column, within the unit cell

(5) **Stress concentration factor (SCF):** Ratio of average stress in the column due to applied load to the stress in the soil within the unit cell. Value of SCF usually lies in the range 2.5-5.0 at the surface of the ground. The value of SCF increase with the consolidation time and decease with the column length.

1.8 REINFORCED STONE COLUMNS

Stone columns are widely used globally due to their versatility and relatively broad applicability in different soil and foundation situations. They are inexpensive and easy to construct. They essentially work by reinforcing the ground to increase the bearing capacity, control the rate of settlement, reduce total and differential settlement, improve slope stability and increase resistance to liquefaction.

1.9 GEOSYNTHETICS REINFORCEMENT MATERIALS

The term "geosynthetics" includes all fabricated synthetic (usually polymeric) materials which find application in the fields of geotechnology. As per the definition of American Society for Testing and Materials (ASTM), a **geosynthetic material** is a planar product manufactured from a polymeric material that is used with soil, rock, earth, or other geotechnical related material as an integral part of a civil engineering project, structure, or system [38].Their polymeric nature makes them suitable particularly for ground surfaces which requires high degrees of durability pre-construction and thus these synthetic products are widely used for stabilizing the territory. Geosynthetic material was first utilized in a dam in 1959 in Contrada Sabetta Italy. Since then, these materials are broadly used and about more than 600 different geosynthetic materials are available these days for civil engineering purposes.

1.9.1 TYPES OF GEOSYNTHETIC MATERIALS

The characteristic feature of geosynthetics depends upon their manufacturing methods and the types or amount of polymeric material used for their preparation. Thus various kinds of reinforced materials are available as given in Table 1.2. The most important of geosynthetic materials are discussed below with their specific applications:

(i) ***Geotextiles*** are porous geosynthetic materials composed exclusively of textiles and are widely used for both critical and non-critical engineering functions of filtration, separation, reinforcement, drainage, reinforcement and protection of geo membranes. The basic polymer groups used for the manufacturing of geotextile fibers include polypropylene (PP) (92% of geotextiles), polyester (PET), polyethylene (PE) and polyamide (Nylon). The intrinsic flexibility of these materials makes them support dynamic loads.

(ii) ***Geogrids*** are usually single/multi-layered materials, made up from extruding and elongated high density polyethylene or polypropylene having grid structure with apertures. Stiffness and high tensile strength of the materials makes them suitable for soil and aggregate reinforcement.

(iii) ***Geonets*** are usually formed by the insertion of molten polyethylene polymer in dies rotating in opposite directions resulting in the formation of "net" of closely spaced criss-crossing polymer strands.

(iv) ***Geocomposite*** Variable geosynthetic materials such as geotextiles fused to geonets, together forms Geocomposite where each component has specific function in the hybrid structure.

(v) ***Geomembranes*** are Lining and as liquid or vapors barrier in the form of thin sheets of rubber or plastic material known as Geomembranes.

(vi) ***Geosynthetic Clay Liners (GCLs)*** comprise of mono layer of bentonite clay which is finely grounded. The GCL serves as effective water barrier as the clay gets swollen on wetting. GCLs are manufactured either by placing bentonite in between or by making a layer of it on geotextiles and/or Geomembranes and layers are fixed by stitching and /or adhesives.

(vii) ***Geopipes*** are plastic pipes, usually made up of high density polyethylene (HDPE), polyvinyl chloride (PVC), polypropylene (PP), acrylonitrile butadiene styrene (ABS), polybutylene (PB) and cellulose acetate butyrate (CAB). These geosynthetic materials have widely used in the construction of highway and railway edge drains, interceptor drains and leachate removal systems.

(viii) ***Geofoam*** It is any type of foam material used for geotechnical applications. Polystrene is the common polymer used for preparing geofoam which are commonly used within soil embankments, under roads, airfield pavements and railway tracks.

(ix) ***Geocells*** These are also known as Cellular Confinement Systems where 3-dimensional structures filled with soil, rock or concrete. These are made up of polymer strips which are pulled in such a way to form a honeycomb-like mat. The geocells provide physical strength to the soil and help in transferring load.

Table 1.2 Classification of the reinforcement materials

<table>
<tr><td rowspan="18">Basis of Classification</td><td rowspan="11">Type</td><td colspan="2">Natural material like bamboo, straw, tree trunks</td></tr>
<tr><td colspan="2">Metallic material such as galvanized stainless and aluminum sheets</td></tr>
<tr><td rowspan="9">Geosynthetic materials</td><td>Geotextile</td></tr>
<tr><td>Geogrid</td></tr>
<tr><td>Geonet</td></tr>
<tr><td>Geocomposite</td></tr>
<tr><td>Geomembrane</td></tr>
<tr><td>Geosynthetic Clay Liner (GCL)</td></tr>
<tr><td>Geopipe</td></tr>
<tr><td>Geofoam</td></tr>
<tr><td>Geocell</td></tr>
<tr><td rowspan="4">Form</td><td colspan="2">Sheets</td></tr>
<tr><td colspan="2">Grids/nets</td></tr>
<tr><td colspan="2">Anchors</td></tr>
<tr><td colspan="2">Small pieces</td></tr>
<tr><td rowspan="3">Arrangement</td><td colspan="2">Regular (micro) - reinforcement in layers</td></tr>
<tr><td colspan="2">Random (macro) - small pieces are mixed with soil</td></tr>
<tr><td colspan="2">Composite (both regular and random)</td></tr>
</table>

Out of these, geotextiles are typical penetrable synthetic constituents made of textiles like polyethylene, polyester, polypropylene or combination of these. Sometimes, the additives like antioxidants, thermal stabilizers and UV inhibitors are used to enhance the characteristics of the geotextile. These are extensively applicable in road and railway construction for different purposes including drainage, separation, filtration, soil modification; for reducing erosion in canals and coastal areas; in dams, retaining walls for drainage; construction of sports areas and for soil monitoring in agriculture. On the basis of their synthetic procedures, geotextiles can be woven, non-woven, knitted or composite geotextiles (**Figure 1.8)**. Another important classification is based upon the primary functions performed by these materials and hence are categorized as separation, filtration and reinforcement geotextiles. Functional classifications also include protection, surface erosion control, containment and subgrade stabilization geotextiles [39].

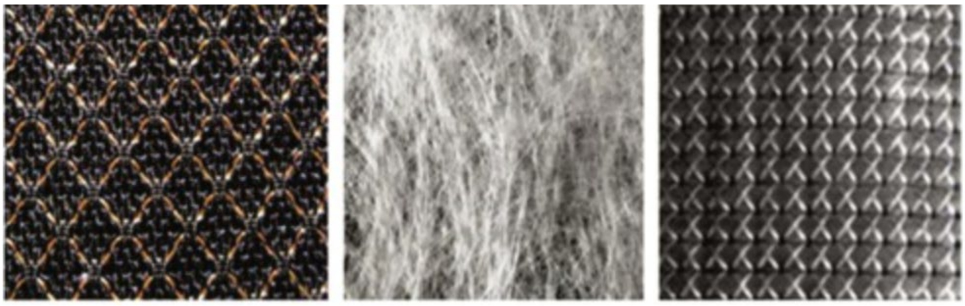


Figure 1.8 Woven, nonwoven, and knitted geotextiles

The choice of a particular geoetextile material also depends upon technical properties which represent the capability of the material for the design and engineering application. These properties include physical, hydraulic, mechanical, interfacial and durability properties. The physical properties usually comprises of unit mass, thickness, length and breadth of the material. The significant hydraulic properties of the geotextile materials are the size of the opening and permeability to the geotextile plane. The mechanical properties include the geotextile puncture resistance, tensile strength (**Figure 1.9**), rigidness of the reinforcement material. The durability and UV degradation of the geotextile material is also important in deciding its applicability for a particular function.

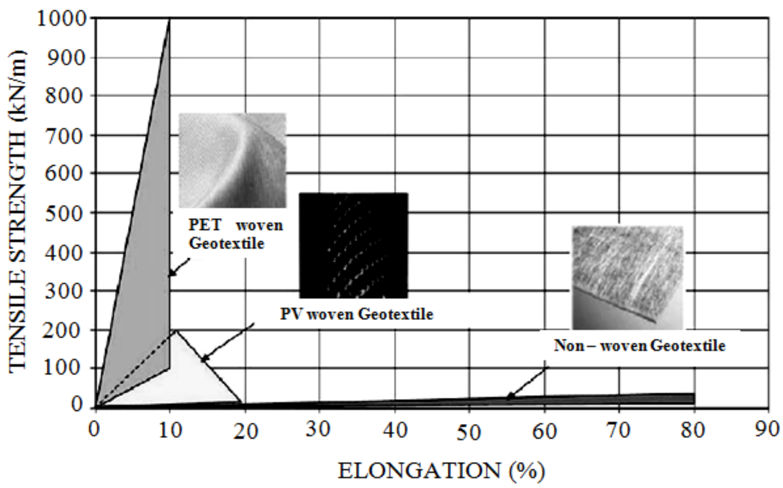
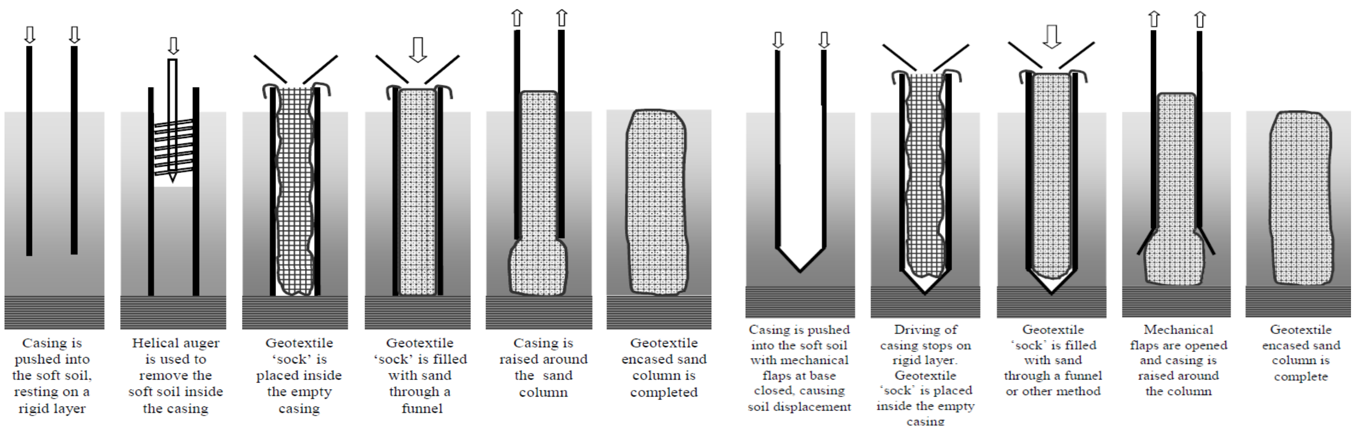


Figure 1.9 Tensile strength vs. elongation curve for woven and nonwoven geotextiles [39]

1.10 VERTICALLY REINFORCED/ENCASED STONE COLUMNS

To compensate for the weak response against bulging as obtained by surrounding soft soil, circumferential layering of columns (particularly of sand columns) using geotextile has been attempted as a solution. Similarly, stone column encasement using layer of geogrids (geogrid encased stone columns) [40-43] has also been utilized. The columns thus formed are known as geotextile/geogrid encased columns (GECs). The basic principle of working of GECs is to provide additional confinement against bulging and hence increasing the stiffness of the composite system. The stiffness of the encasement material defines the stiffness of the stone column – soil composite system. As the axial loading acts on the stone column, axial shortening of the columns due to radial expansion occurs. The stiffness of the encasement material contributes to the resistance against the column bulging. This resistance increases the stiffness of the column material and hence renders strength increase in the nearby soil [40, 41]. The installation of GECs can be done with replacement or displacement method as shown in Fig. 1.10.



(a) Replacement method [42] **(b)** Displacement method [42]

Figure. 1.10 Construction of GECs

1.11 HORIZONTALLY REINFORCED STONE COLUMNS

Horizontal layering of synthetic materials in the upper part of the column is done to monitor expansion (**Figure 1.11**). As the columns are loaded, there is a probability of lateral expansion of the columns. When a column expands laterally, it provides additional confinement to the adjacent columns. The magnitude of this expansion is dependent upon the physical properties of column material and soil and also on whether the column used is isolated or a part of group of columns. In case of single column, bulging usually occurs in upper part of the column where lateral confining pressure as well as stiffness of the surrounding soil is poor. The placement of geosynthetic material in horizontal layers within this bulge prone section of the column results in increasing the shearing between the column material – geosynthetics interface and restricts the lateral deformation (i.e bulging) [44].

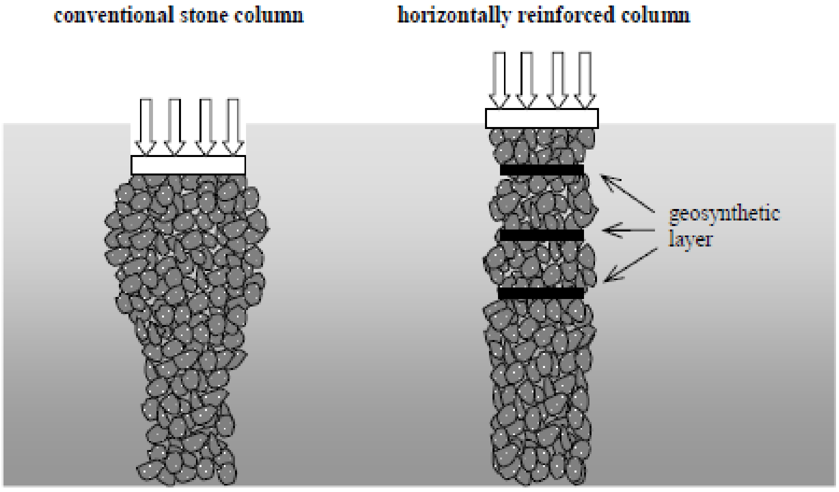


Figure 1.11 Geosynthetic layering of ordinary stone columns

1.12 ORGANIZATION OF THE THESIS

Complete thesis has been arranged into five chapters and a brief description of each chapter is given below:

Chapter 1 includes the briefing of various ground improvement methods with focus on stone columns. The chapter also brings out the mechanism, failure modes, design parameters and installation methods for stone columns. The chapter also covers the development of vertically/encased stone columns and horizontally reinforced stone columns.

Chapter 2 comprises of description of modification techniques for natural soil, ordinary stone columns, geosynthetic material and its advantages and applications of geosynthetics, installation of stone columns (installation methods and installation effects), behavior of treated soil, review of literature (numerical, theoretical and experimental investigations) emphasizing on the load transfer and stress concentration in reinforced soil, stone columns mechanism and performance, consolidation behavior of the soil, geosynthetic reinforcement in modifying the capability of stone columns.

Chapter 3 includes the details of materials used, tests performed, model tests and testing procedure employed to study the behavior of unreinforced and reinforced (horizontally and vertically) stone columns in sandy soil. The chapter also presents the results of various tests and their discussion in the light of literature.

Chapter 4 includes the results from the experimental investigations have been discussed in this chapter.

Chapter 5 includes the results from the finite element modeling using Plaxis 3-D. Validation of Plaxis 3-D results from experimental results have also been discussed in this chapter.

Chapter 6 summarises the key conclusions from this study, and also makes some recommendations for future research.

CHAPTER-2
LITERATURE SURVEY

2.1 GENERAL

This chapter deals with the review of literature on conventional stone columns as well as reinforced stone columns. The chapter covers all the three aspects of experimental testing, numerical and theoretically modelling for conventional/ordinary stone columns, vertically reinforced/encased stone columns and horizontally reinforced stone columns. The reported results from various researchers have analyzed and further summarized in the chapter. The chapter also presents the objectives and scope of the present research work.

2.2 GROUND IMPROVEMENT TECHNIQUES

Various techniques are usually employed to induce the parameters of saturated clayey soil such as its in-situ performance in terms of load bearing, settlement, consolidation, drainage rate which depends upon installation of these methods and their applicability to different types of soil [45-48]. Most of the available methods for the improvement of soil surface have emphasized on increased load bearing capacity, modified shear strength and reduction in consolidation settlement of foundation surface such as replacement of soil, preloading with the help of vertical drains, stone columns, stabilization using additive and thermal methods [49-53]. Most of the modification techniques involve the improvement with/without admixtures, stabilization by additives and thermal methods [54-58]. Some of the trivial methods involve construction of deep foundation through unsuitable soils, replacement of soft soils with soils with suitable geotechnical parameters, usage of additives to stabilize the soft soils, construction in phases and wait until the occurrence of natural consolidation and dewatering of soft soil. In addition to these methods, the new improvement techniques have also been utilized these days viz. reinforcement by geosynthetics, fibre reinforcement, vacuum preloading method,, preloading with/without vertical drains, lime columns installation, micro- or mini-pile construction, dynamic compaction and fitting stone columns.

2.3 CONVENTIONAL STONE COLUMNS (SC) EXPERIMENTAL STUDIES

Because of their flexibility and large range of uses in various forms of soils and engineering systems, ordinary/traditional stone columns (SC) have been widely used around the world. The method of construction of SC is quite simple and of low cost as well [59]. This works in supporting the ground surface by regulating the settlement rate, decreasing the total and differential settlements [60, 61] and increment in liquefaction resistance of soil thereby protecting the structure against earthy trembles [62-64]. The insertion in stone columns in weak soil efficiently strengthens the soil by forming composite mass, hence it increases the consolidation and bearing capacity of weak soil. [65, 66].In addition, the utilization of stone columns has now found to be satisfactory from environmental point of view [64]. Shadi S. Najjar [67] provides a current state-of-the-art overview of reviewed academic papers as shown in Table 2.1 and publications on the modelling, measuring, and study of soft clays reinforced by sand/stone columns in terms of bearing capability and settlement requirements. The study is organised chronologically to show how this area of science has evolved over the past 40 years.

Table 2.1 Stone/sand-column reinforced soils database of scientific studies

References	**Un-reinforced (U) / Reinforced (R)**	**Design/ analysis**	**Experiment**	**Type of experiment (*S*=single, G = group)**	**Type of loading**	**Size of Sapmle**	***U*=undrained, *D*=drained, *PD*=partially**	**Sample size (cm*cm*cm)**
Greenwood (1970)	U/R	-	-	-	-	-	-	-
Bauman and Bauer (1974)	U	✓	✓	G	F	135	D	
Hughes and Withers(1974)	R	✓	✓	S	C	-	D	22.5*16*15

Hughes etal. (1975)	U/R	-	✓	S	C	66	U	-
Greenwood (1975)	U/R	-	✓	S	F	91	D	-
Brauns (1978)	U/R	✓	-	-	-	-	-	-
Mitchell (1981)	U/R	✓	-	-	-	-	-	-
Charles and Watts (1983)	R	-	✓	S	A	-	D	H=60, D=100
Bachus and Barksdale (1983)	U/R	✓	-	-	-	-	-	-
Bergado etal. (1987a)	U/R	-	✓	S	C & F	30 - 120	D	-
Bergado and Lam (1987)	U/R	-	✓	S	C	-	D	-
Juran and Guermazi (1988)	U/R	-	✓	S	A	-	PD, U	H=20, D=10
NarasimhaRaoetal.(1992)	U	-	✓	S	F	1.5D	-	100*80*100
Hanand Ye (1991)	U/R	-	✓	S	C & F	125	D	-
Priebe (1995)	U/R	✓	-	-	-	-	-	-
Rajagopal etal. (1999)	R	✓	✓	S	C	-	0.62%	Sand

Watts etal.(2000)	U/R	✓	✓	S	F	75	82-123 kPA	-
Raithel and Kempfert (2000)	U/R	✓	-	-	-	-	-	-
Muir Wood etal. (2000)	U	-	✓	G	F	10	D	D=30
Malarvizhi and Ilamparuthi (2004)	U	-	✓	S	F	7	-	D=30
Sivakumar etal. (2004)	U/R	-	✓	S	A & F	4	U	D =10
Lillis etal.(2004)	U/R	-	✓	S	C	-	D	-
McKelvey etal. (2004)	R	-	✓	G	F	9*9	0.0060 mm/Minute	H=50, D=41.30
Ayadat and Hanna (2005)	U/R	✓	✓	S	F	4	Controlled Strain	H=52/ D=39
Kim and Lee (2005)	U/R	-	✓	G	F	10*6	1 kgf	25*10*25
Murugesan and Rajagopal (2006)	R	-	-	FEM	-	-	-	-
Black etal.(2006)	U/R	-	✓	S	F	6	D	D=30, H=40
White	U/R	-	✓	S	C &	230	D	-

etal.(2007)					F			
Black etal.(2007)	U/R	-	✓	S	-	-	U & D	D=10, H=20
Ambily and Gandhi (2007)	U/R	✓	✓	S	A & C	-	0.062 mm/minute	D=21-83, H=45
Murugesan and Rajagopal (2008)	U/R	✓	✓	S	C	-	U	D=21, H=50
Andreou etal. (2008)	U/R	-	✓	S	-	-	U & D	D=10, H=20
Elshazly etal. (2008a)	R	-	✓	S	F	200	PD	-
Elshazlyetal. (2008b)	R	-	-	FEM	-	-	-	-
Chenetal.(2009)	R	-	✓	S	C	-	D	-
Wu and Hong (2009)	U/R	✓	✓	S	C	-	0.3percent/ minute	Sand
Gniel and Bouazza (2009)	U/R	-	✓	S	C & A	-	PD	H=31, D=15
Lo etal.(2010)	R	-	-	FEM	-	-	-	-
Murugesan and Rajagopal (2010)	U/R	✓	✓	S/G	F	2D	U	120*120*0.6
Najjar etal.(2010)	U/R	-	✓	S	A	-	U	D=7, H=14

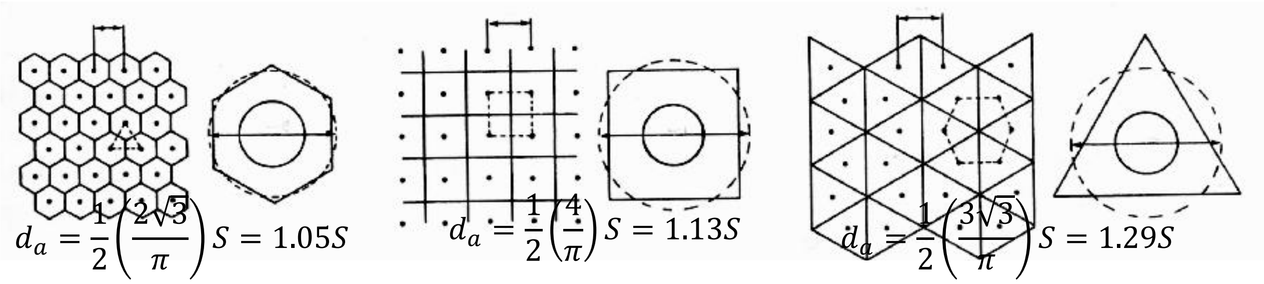
Pulko etal.(2011)	U/R	✓	-	-	-	-	-	-
Castro and Sagaseta (2011)	U/R	✓	-	-	-	-	-	-
Black etal.(2011)	U/R	-	✓	S	F	6	D	D=30, H=40
Siva kumar etal. (2011)	U/R	-	✓	S	F	6	D	D=30, H=40
Cimentada etal. (2011)	U/R	-	✓	S	A	-	100kPA	D=25, H=15
Shahu and Reddy (2011)	R	-	✓	G	F	10	D	D & H=30
Fattah etal.(2011)	U/R	-	✓	S	F	22	2.5 minute	110*100*40
Stuedlein and Holtz (2012)	U/R	-	✓	S	C & F	2.75 m	D	-
M. S. S. Almeida (2013)	U/R	✓	✓	S	F	-	D	-
M. Ghazavi (2013)	U/R	-	✓	G	F	-	D	-
S. K. Dash (2013)	U/R	-	✓	S	C & F	6	D	Sand
L. Zhang (2013)	U/R	✓	✓	G	F	-	D	-
Y. Jiang (2013)	U/R	✓	-	G	F	-	-	-
A. J. Choobbasti	U	✓	-	G	F		D	-

(2014)								
M. Khabbazian (2014)	U	-	✓	G	F	10	D	-
M. Ghazavi (2014)	U	✓	-	G	F	-	D	-
P. Andreou (2014)	U	-	✓	S	C	-	D	-
M. R. Shirazi (2015)	U/R	-	✓	G	C & F	2D	D	-
M. S. S. Almeida (2015)	U/R	✓	✓	G	F	-	D	-
H. D. Golakiya (2015)	U/R	✓	✓	S/G	F	2D	U	-
M. Qu (2016)	U/R	✓	✓	S	-	-	U & D	D=10, H=25
L. Sinyakov (2016)	U/R	-	✓	S	A	-	U	D=7
P. Yuvaraj (2016)	U/R	-	✓	S	C	66	U	-
H. Canakci (2017)	U/R	✓	✓	G	F	-	D	-
A. S. A. Rashid (2017)	R	✓	✓	S	C	-	0.070 mm/minute	-
S. Chandrawanshi (2017)	U/R	✓	-	-	-	-	-	-

G. Ye (2017)	U/R	-	✓	S	C & F	30 - 120	D	-
W. Liu (2018)	U/R	✓	✓	G	F	135	D	
K. S. Ng (2018)	U/R	-	✓	S	A	-	PD, U	H=20, D=10
A. K. Ahmed Naseem (2018)	U/R	-	✓	G	F	2D	D	-
S. H. Lajevardi (2019)	U/R	-	✓	G	F	-	D	-
I. Hosseinpour (2019)	R	-	-	FEM	-	-	U & D	-
M. J. Shabani (2019)	U/R	-	✓	S	A	-	U	D=10, H=18
X. Tan (2020)	U/R	-	✓	G	C & F	2D	D	-
A. K. Dey (2020)	U/R							
S. S. Roy (2020)	U/R	✓	✓	S	F	-	D	-

2.3.1 STONE COLUMNS INSTALLATION

For stone column, the rock is crushed into the particles whose size is less than 1/7th of stone column diameter. Stone columns are basically cylindrical columns constructed below the ground level consisting of granular material of size in the range 25-100 mm. The common patterns for the installation of these columns can be triangular, square or hexagonal (Figure 2.1).



(a) Triangular Orientation (b) Square Orientation (c) Hexagonal Orientation

Figure 2.1 Pattern for the Installation of Stone Columns [68]

2.3.1.1 INSTALLATION METHODS

The stone columns are constructed by adopting one of the following methods [68-70]:

(a) Replacement method, which is also known as vibrofloat method involve the displacement of in-situ soil by creating a hole in the weak soil mass using a stream of water from a vibrofloat unit. The crushed stones are filled into their hole to build stone columns. This technique is suitable for the relatively soft soil and the soft where the underground water is at high level.

(b) Displacement method involves the use of compressed air from vibrofloat for moving the natural soil. This method is widely utilized for the firm soil and soils with low underground water table.

(c) Non-displacement method (Case-Borehole or rammed columns method) involve the forced addition of stone granules into a pre-bored hole by intensive ramming.

2.3.1.2 INSTALLATION EFFECTS

The technique of stone column has been extensively used for modifying the surface of soft clays, silty soil, and loose grain sandy soils making them ready for the construction activities [71-73]. Particularly there is change in the soil properties during displacement. The system used to mount stone columns has also been shown to have a significant impact on the initial stress stage of the surrounding soil and column, which eventually contributes to the column's performance/failure process [74-75]. This mainly depends on the adjacent stone columns spacing, type of soil, equipment used for installation as well as procedure adopted. Installation of stone columns influences the compactness, density and permeability of the improved soil around the stone column ending up with sufficient amount of settlement reduction [76-78]. The stone column provides additional drainage path, thus enhancing of shear strength parameters of the soil and primary consolidation [79-81]. However, smear and disturbing effects like clogging, arching affect the hydraulic nature of the soil, ultimately leads to destruction of horizontal permeability in comparison to the ideal drainage conditions and hence found to reduce the consolidation of the soil [82-85].

Guetif et al. [86] have evaluated the improvement in characteristics of soft clay after reinforcement with stone columns, utilizing the method of numerical analysis with the help of Plaxis software and assuming Mohr-Coulomb's behavior for the constituents of the composite soil. It has been found that, after eleven methods of stone columns installation, coefficient of lateral earth pressure, effective mean stress and Young's modulus of the clay is increased significantly along with reduction in the settlement.

By varying the parameters viz. shear strength of the soft clay, distance between the columns, and loading conditions Ambily et al. [87] having observed the actions of the single column as well as a set of seven. To determine the condition, lab experiments were carried out the rigidity of upgraded ground and columns restricted axial capacity. The column's axial strength has been identified to be decreased, and settlement is increased with an for a rise in spacing of up to 3 s/d, further increase makes negligible change. 15-noded triangular components with the programme tool PLAXIS was utilized for finite element analyses. The numerical outcomes from FEM were in good correlation with experimental results as shown in Figure 2.2.

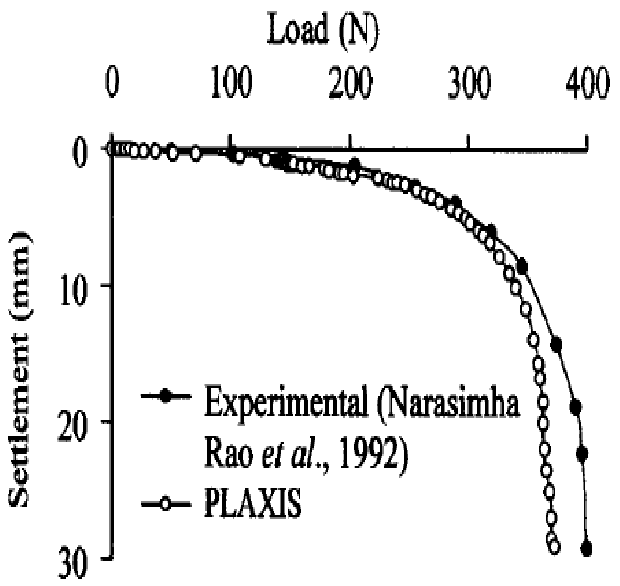


Figure 2.2 (a) Validation of PLAXIS [87]

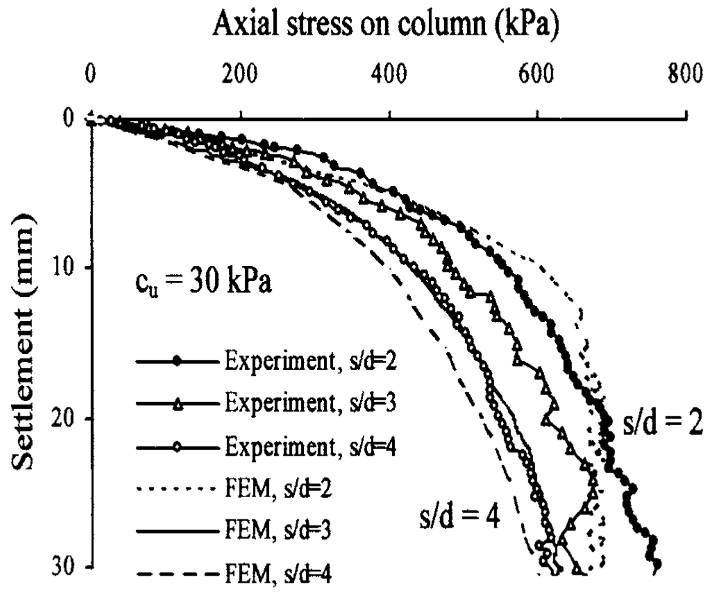


Figure 2.2 (b) Effect of *s/d* on behavior of stone column [87]

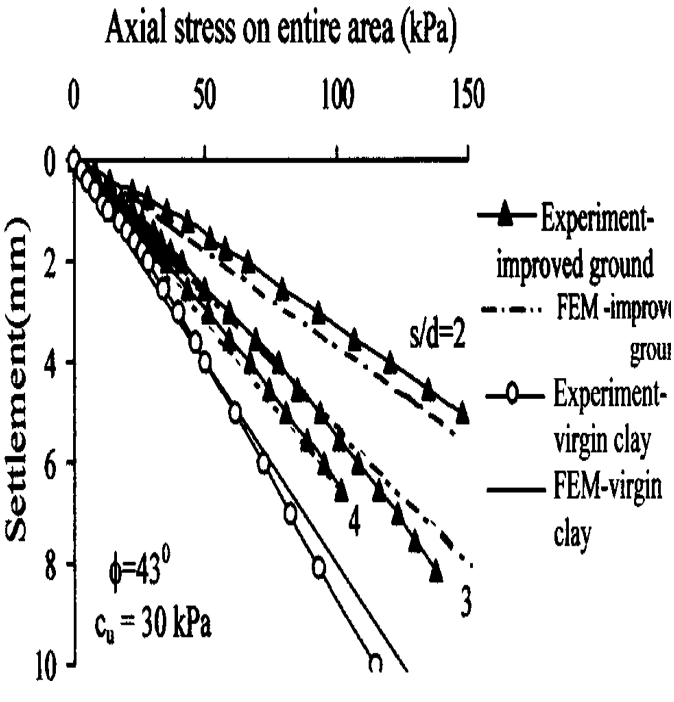


Figure 2.2 (c) s/d effect on stress /settlement behavior entire area loaded [87]

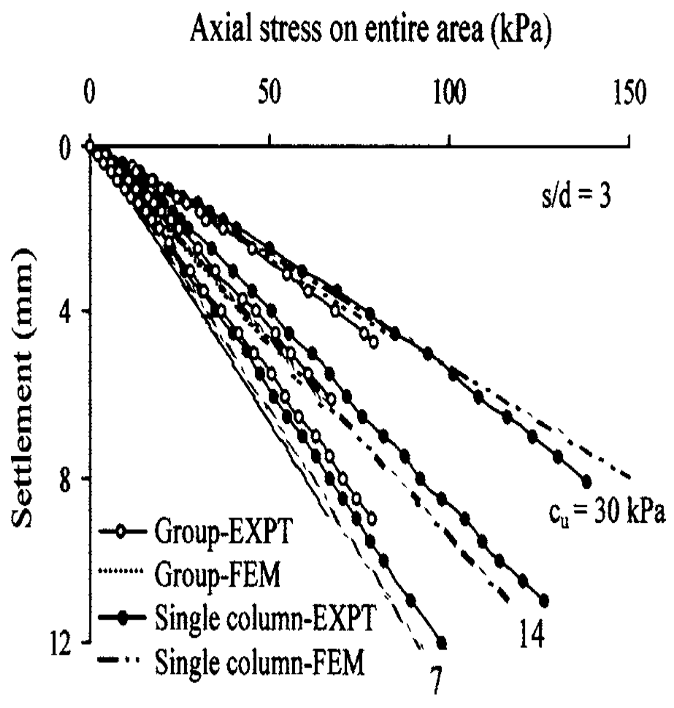
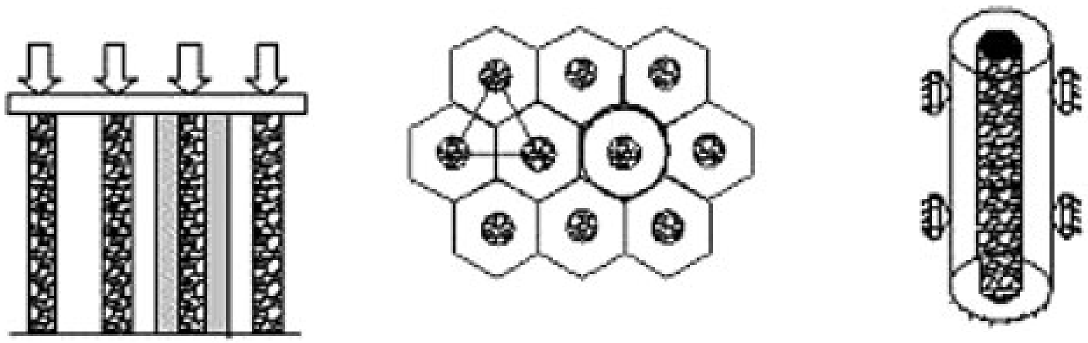


Figure 2.2 (d) Comparison btw group and single column test [87]

2.3.2 BEHAVIOR OF AMALGAMATED GROUND

2.3.2.1 UNIT CELL TECHNIQUE

Effect of installation pattern of the stone columns i.e. triangular, squared and hexagonal can be determined by considering the different arrangements of stone columns as unit cell. A unit cell is a cylinder made up of one stone column and its surroundings, with an impact zone diameter (de) as seen in Figure 2.3.Figure 2.3 also illustrates the separation of composite ground into unit cells [88].



(a) Uniform load applied on composite ground

(b) Influence zone of each column (Unit cell concept)

(c) Radial boundaries of unit cell

Figure 2.3 Concept of unit cell

Bergado et al. [89] have modulated a theory depending on the stress concentration to evaluate stability of stone column reinforced ground surface, where each column is isolated into its own unit cell (Figure 2.4).

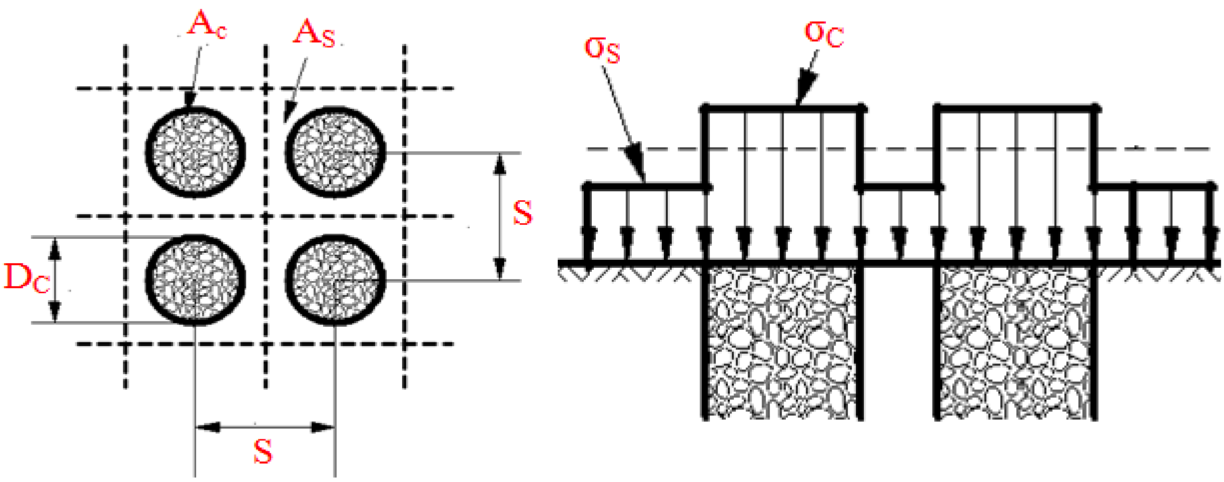


Figure 2.4 Diagram of composite ground [89]

The ratio of granular pile area (Ac) to the entire area of the corresponding unit cell is used in this case to express the area replacement ratio.

$$\sigma = \sigma_c a_s + \sigma_s (1 - a_s) \tag{2.1}$$

Equations (2.2) and (2.3) give the area replacement ratio for square and triangular patterned columns in terms of stone column diameter (d) and spacing (S).

$$a_S = \left(\frac{\pi}{4}\right) \times \left(\frac{d}{S}\right)^2 \tag{2.2}$$

$$a_S = \left(\frac{\pi}{2\sqrt{3}}\right) \times \left(\frac{d}{S}\right)^2 \tag{2.3}$$

Several studies have been conducted to investigate the settlement and loading behaviour of soil reinforced with stone columns by varying design parameters such as area ratio, width and depth of treated region, and column distribution (uniformity and concentration at edges or at centre) [90-93]. The installation of column using deep mixing method, where a stabilizing agent is injected into the soil before column, reduces the settlement of the soil and hence increases the bearing capacity, column failure mechanism and the deformation of ground surface of soft soil [94-97]. Investigations also shown that when earth is reinforced with stone columns, stress concentrations in the columns develop, followed by a decrease in stress in the surrounding soil. The fact that the vertical settlement of the stone column and surrounding soil is almost identical after loading is a likely explanation; the stone column's greater rigidity in contrast to the local soil causes tension accumulation in the column [98]. Figure 2.4 (b) shows the stress distribution in the column (σ_c), soft clay field (σ_s), and overall stress (σ). The stress concentration factor (SCF) represents the distribution of vertical stress within the unit cell and is defined as the ratio of stress in the column to stress developed in the surrounding soil.

$$SCF = {\sigma_C}/{\sigma_S} \tag{2.4}$$

Average stress (σ) over unit cell is given by

$$\sigma = \sigma_C a_S + \sigma_S (1 - a_S) \tag{2.5}$$

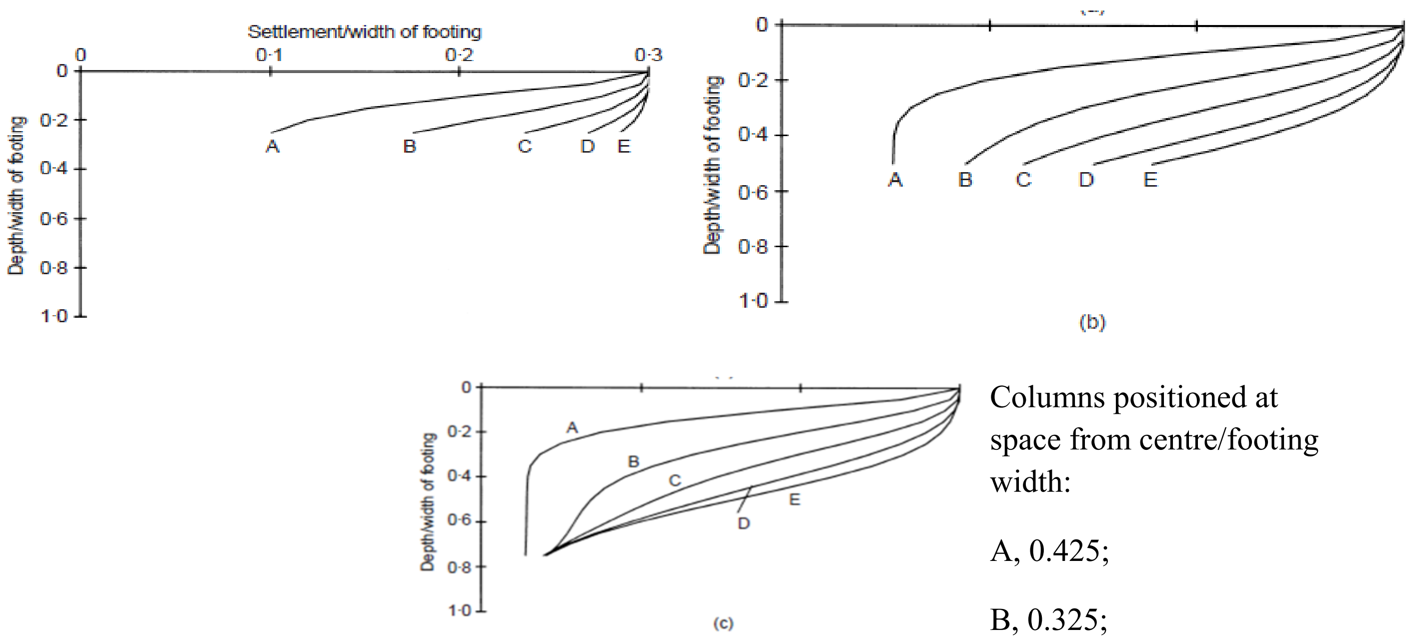
Stress in stone column (σ_c) and in surrounding clayey ground (σ_S) is then given as follows:

$$\sigma_C = {(SCF \cdot \sigma)}/{\left[1 + (SCF - 1) \cdot a_S\right]} = \mu_C \sigma \tag{2.6}$$

$$\sigma_S = \sigma \Big/ \left[1 + (SCF - 1) \cdot a_S\right] = \mu_S \sigma \tag{2.7}$$

Najjar [99] has presented a review that compiles various field and laboratory tests conducted on clayey soil reinforced partially or fully penetrating with ordinary or encased stone/sand columns that have been used as single or group of columns. Some numerical methods are based on finite element models whereas some assumes analytical models for composite system.

Wood et al have performed test on clay beds reinforced with stone column. [100] by employing model tests that were subjected to surface footing and have been explored by varying the specifications of stone columns viz. the diameter, length and spacing of the columns. To find out the deformed shape of columns and to study how columns transfer load to surrounding clay either by bulging or by forming a failure plane an exhumation technique has been utilized. Laboratory experiments have shown that there is a major association between the footing and individual stone columns within the community, resulting in separate load settlements in different positions under the footing, as shown in Figure 2.5.The numerically analyzed results have been qualitatively correlated with the findings from the experimentally determined model tests.



Columns positioned at space from centre/footing width:

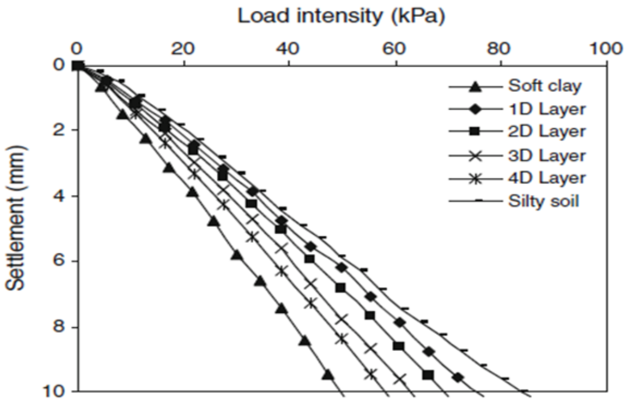
A, 0.425;

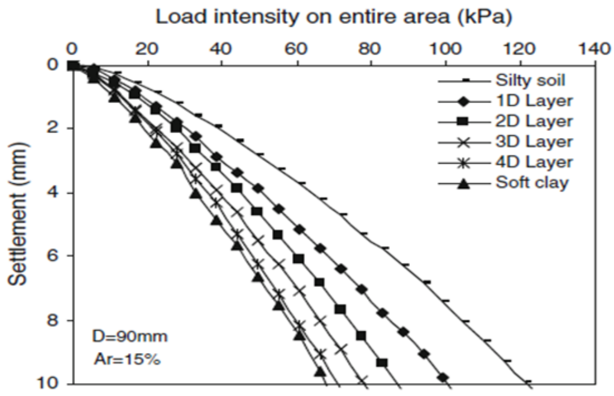
B, 0.325;

C, 0.225;

Figure 2.5 Mathematical research of columns in terms of settlement profiles. Column length/footing width: (a) 0.250; (b) 0.500; (c) 0.750.

Shiva Shankar et al carried out a series of laboratory plate load testing in unit cell tanks for finding the improvement in rigidity, load bearing strength, and bulging resistance of stone columns set in soft soils. [88, 101]. They also proposed a new scheme for fitting stone columns with vertical nails around the diameter of the column, which will increase the column's efficiency. (a) The entire unit cell tank was loaded to estimate the increased surface rigidity, and (b) only the column was loaded to approximate the limiting axial power (Figure 2.6).Results have indicated better lading carrying capacity, slighter compression and slighter lateral bulging for such stone columns even at lower area ratio as compared to the conventional ones. With rise in number, diameter, and depth of embedment of nails this enhancement has been found to be increased. Presence of stone columns in clay results in distribution of pore pressure rate and decrease in vertical stress. This leads to transfer of load to the column due to tits stiffness resulting in the increment in the stress concentration factors [102].





(a) **(b)**

Figure 2.6 (a) Load-settlement action of untreated layered soils and **(b)** The thickness of the upper weak layer has an influence on the load settlement action of composite land. [101].

Rangeard et al. [103] have studied the effect of compaction force produced during installation of stone columns on hydro mechanical behavior of reinforced soil by comparing the response of sand columns installed by replacement and displacement methods with /without compaction. Chandra wanshi et al. [104] has conducted the small scale model tested on soft clays with stone columns constructed by both replacement and displacement methods. (Figure 2.7). The testing parameters have been calculated by using idealized unit cell. Consolidation pressure of 150 kPa for 24 hours is given to plain and soft clay surfaces reinforced with columns of stones to observe the settlements. The study has concluded that the rate and ultimate settlement value mainly depend upon three factors: s/d ratio, method of stone column construction and applied compactive efforts. A reduced final settlement has been obtained by using smaller s/d ratio and higher compactive efforts in replacement method for building the stone columns. The impact of high capacity micropiles on the load carrying behavior of soft clay has been investigated by Borthakur et al. [105], whereas Sadaoui et al. [106] have conducted a case study to monitor the settlement of structures formed on stone column reinforced soft soil using field measurement and back calculations.

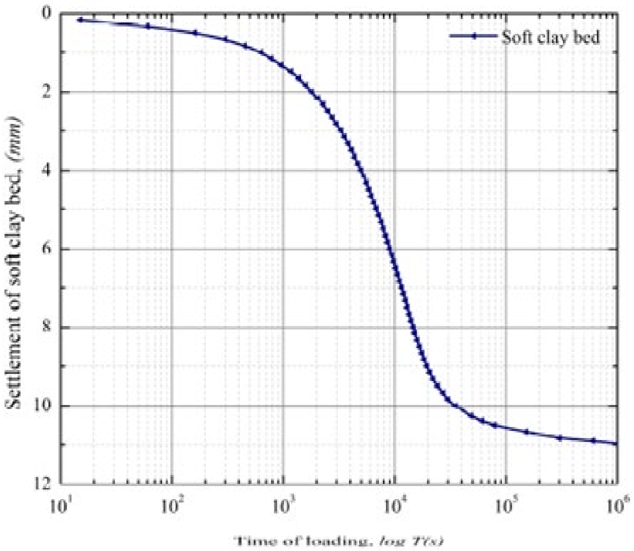


Figure 2.7 (a) Settlement vs. Loading Time for Soft Clay Different Bed [104]

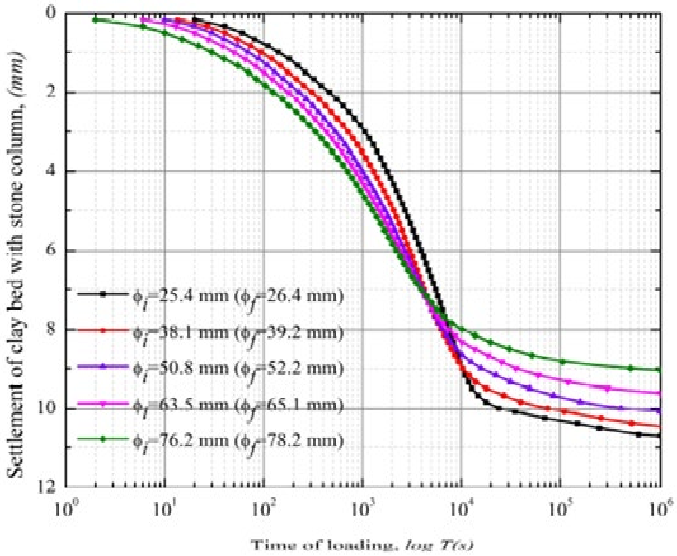


Figure 2.7 (b) Settlement vs. Loading Time for 5 dia. of Stone Columns (RP-MC Method) [104]

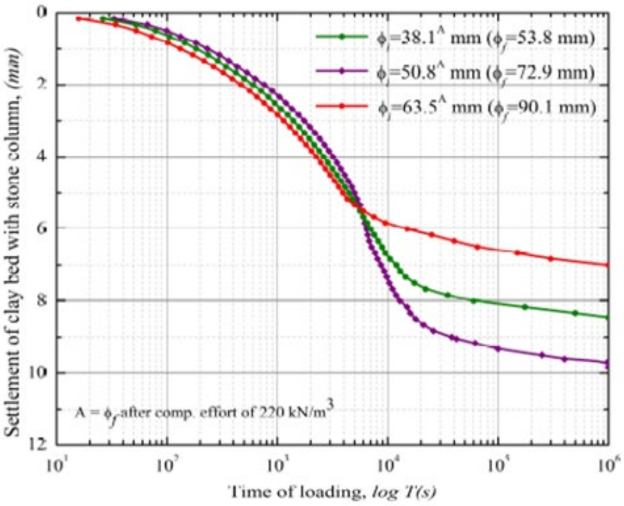


Figure 2.7 (c) Settlement vs. Loading Time for 38.1, 50.8, 63.5 mm dia. of Stone Columns (RP-HC Method) [104]

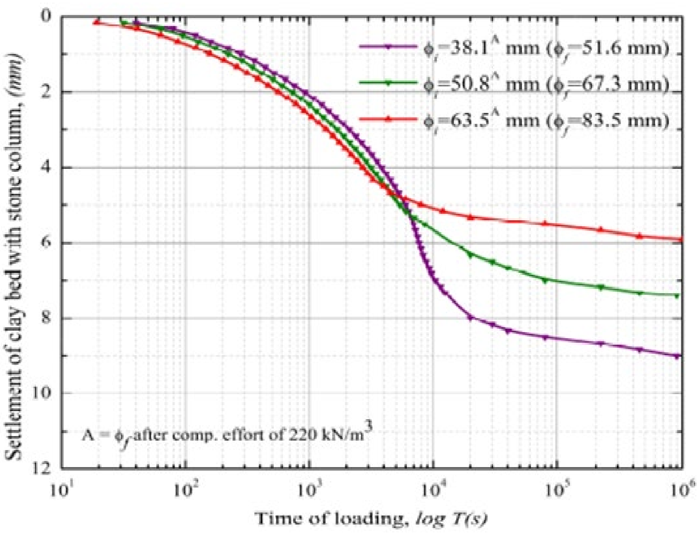


Figure 2.7 (d) Settlement vs. Loading Time for 38.1, 50.8, 63.5 mm dia. of Stone Columns (DP-HC Method) [104]

Hughes et al. analysed the action of single stone column [107] occurrence of deformations in and near the column loaded with soil containing chamber of cylindrical shape has been predicted by using laboratory radiography. Cho et al. has performed the experiments [108] to compute the reduction in settlement of a gravel compaction pile (SCP) and a sand compaction pile (GCP) using finite element method. Lee et al. have reported effects of model sand compaction piles (SCPs) setup in soft clay [109] using the frozen pile technique, a high-g displacement technique, as well as a 1-g displacement method. Both displacement methods have been found to provide extra increased capacity to ground enhancement, which was not given by frozen pile models.

Christoulas et al. [110] have carried out two equipped axial loading experiments on model column stone using kaolin clay and reported the bulging of superior part of the column along the diameter of column (Figure 2.8). Achievement of stone columns in improving the load carrying strength of soil as well as settlement reduction of foundation is controlled by various factors including the tensile strength of material used for the fabrication of the column, method of installation of column, type of the soil under consideration, geometric parameters of the column.

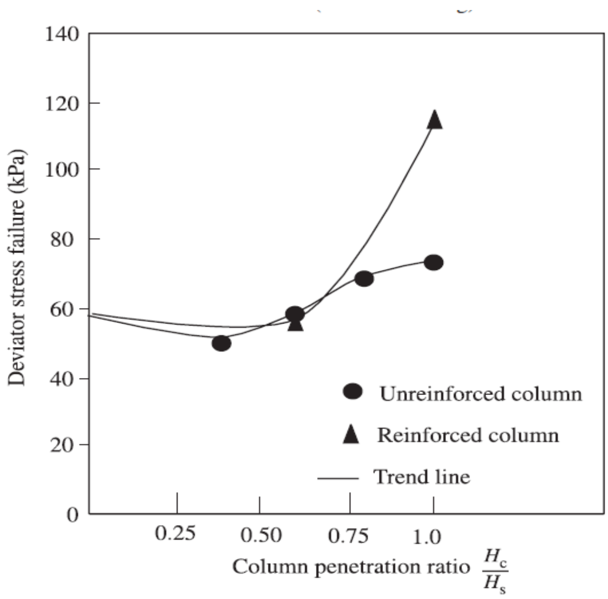


Figure 2.8 (a) Deviator stress at collapse: contrast intermediate to reinforced and unreinforced columns (At consistent loading) [110]

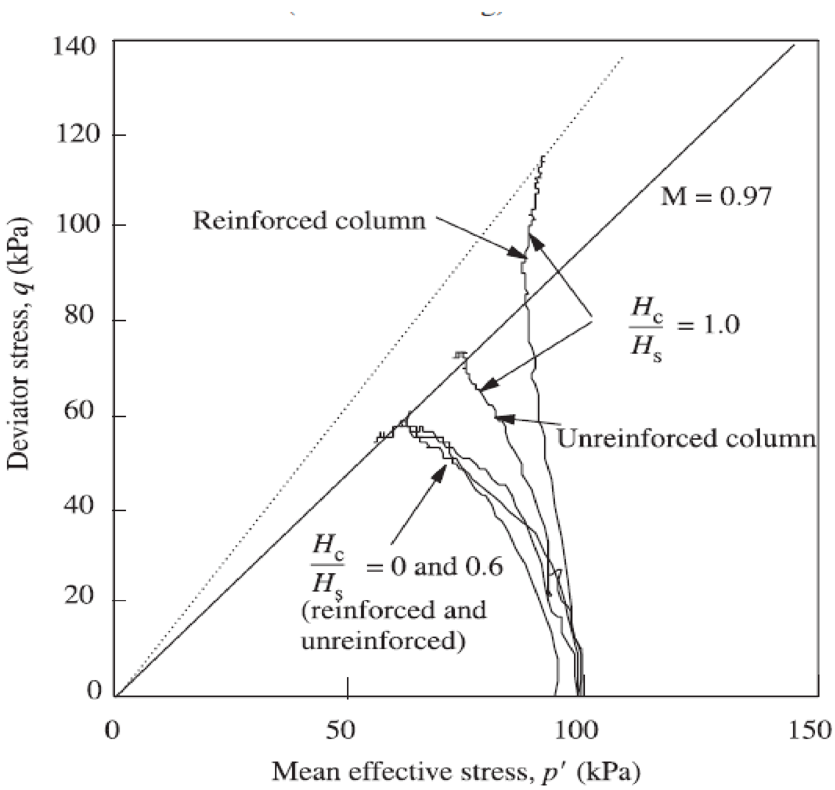


Figure 2.8 (b) Stress path: differentiation intermediate to reinforced and unreinforced columns (At consistent loading) [110]

Siva kumar et al. [111] has performed two set of tests on kaolin specimen under varying conditions. The specimens installed with the full depth column under the conditions of uniform loading have found to be of more tensile strength when compared to the untreated ones under same set of conditions. Soil's bearing capacity is improved when length of column gets increased.

Mohanty et al. [112] have studied effects of soil layering on the behaviour of stone columns and surface installed with stone columns by performing a numerical study and a number of small-scale experimental studies. For the analysis, they found various types of layering structures on soft clay overlying stiff clay and (ii) vice versa. Experiments were conducted on 88 mm diameter stone columns placed in a two-layered soil, using the unit cell principle to research the behaviour of a single column within an infinite number of columns. Performance of the total improved surface and that of the stone column have been evaluated in terms of stress versus settlement response. Detailed parametric study is done by employing finite element-based software model.

Raee et al. [113] studied the bearing capability of a rigid strip footed mounted on a sand slope of slope-stabilizing stone columns has been investigated using experimental and numerical methods. The rigidity of the stone columns, as well as their spacing across each section, has been used to measure a wide range of conditions. Different types of columns like concrete piles, ordinary columns of stones, encased columns have been considered for this purpose, while particle image velocimetry method has been used to obtain soil displacement fields. Relationships between bearing strength and other geotechnical parameters have been figured out by exploring the results.

Frikha et al. [114] have described the laboratory test to understand the manner in which remoulded kaolin clay affects the properties of stone columns. Plots have been shown in (Figure 2.9 & 2.10). Construction of stone column was stimulated by expanding the hollow cylindrical remoulded kaolin specimens laterally at different rates; the specimens were initially exposed to K_o consolidation way. Then they were placed into typical undrained consolidated triaxial checks; in addition to this the excess pore pressure was also recorded.

Various experiments have been performed for computing the consequences of consolidated stress and stone column on the undrained Young's modulus and shear strength of the kaolin clay. Outcome has shown the enhancement in the Young's modulus on increase in expansion in cavity ration and consolidation stress. Also the undrained shear strength of enhanced clay upsurges with the reduction in consolidation stress. The study has also suggested that consolidation shear stress reduces the proportion of a undrained Young's modulus to un drained conditioned of shear strength. In this way, a model has been developed for designing stone columns.

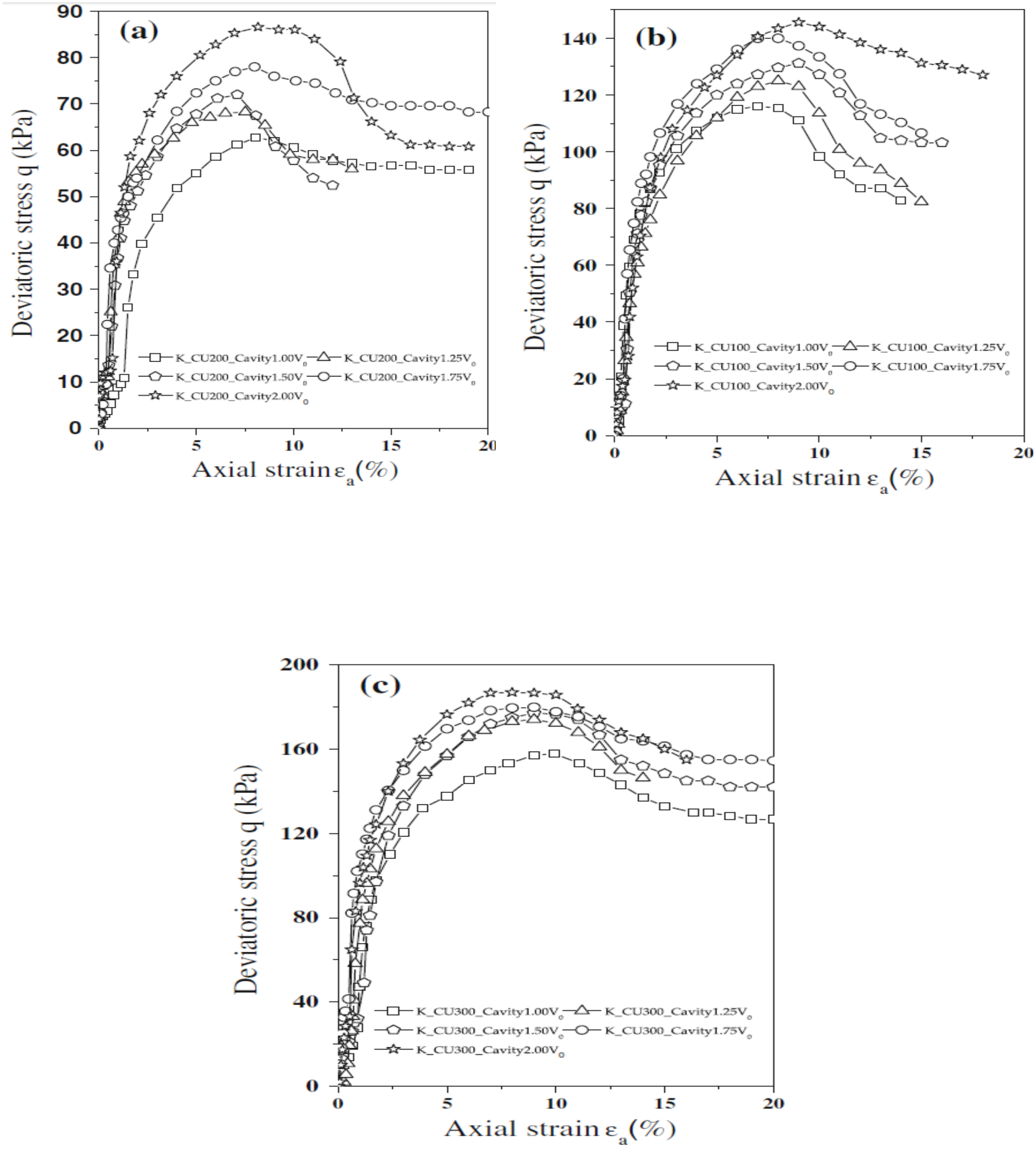


Figure 2.9 (a) During tri axial shear loading axial shear-strain curves = 100 k Pa, **(b)** During triaxial shear loading axial stress strain curves =200 k Pa and **(c)** During tri axial shear loading axial stress–strain curves = 300 k Pa [114]

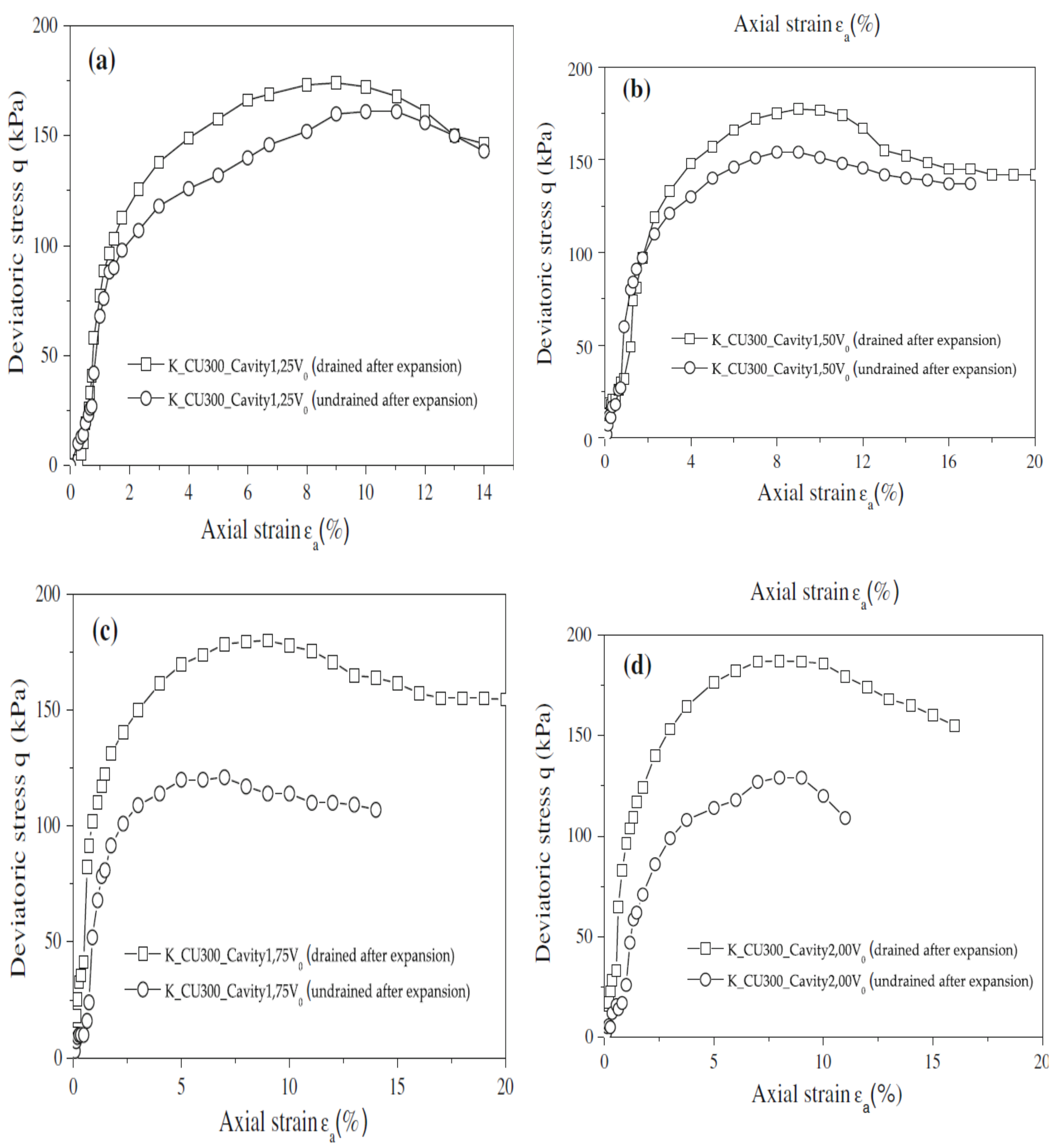


Figure 2.10 Axial stress–strain relationship (triaxial shear loading = 300.0 kPa) for samples of drained along with un drained ;Distinct cavity expansion proportion V/V_0 post cavity expansion **(a)** 1.250, **(b)** 1.50, **(c)** 1.750 and **(d)** 2.0 [114]

2.4 CONVENTIONAL STONE COLUMNS (SC) NUMERICAL STUDIES

Since from last 30 years, the technique of stone columns has been emerged as one of the best reinforcement methods for improvement of weak soil owing to its verified performance, constructability, durability, limited period routine, and reduces prices. However, bearing capacity of the stone columns is related to high uncertainties probably due to (i) the generality of existing formulas used for its estimation and (ii) non- consideration of (a) Variety of stone column, (b) implementation method, (c) length of column/diameter (L/d) and (d) other factors affecting load carrying capacity of the stone columns. Load applied to stone column reinforced soil is distributed between the nearby soil and stone column, the ratio depending upon their rigidity. They carry most of the applied load due to more stiffness of stone column. Modular ratio expresses stiffness ratio, outlined as proportion of elasticity modulus of stone column to surrounding soil (E_C/E_S). Researchers have studied load behavior and stress concentration of ground surface modified using stone columns by using elastic – plastic laws where effect of stone columns has been considered to be distributed equally and homogenized throughout reinforcement area [115-117].

Another empirical method commonly used for the embankment study, load using soft soil installed with stone columns of variety of geometric parameters involves the utilization of support vector regression (SVR) as well as artificial neural network (ANN) systems with the settlement prediction based on document prior knowledge [118- 120]. The ANN modeling method has found to be sensitive and more accurate with the prediction value falls within 95% prediction value [121], but the statistical performance of SVR model dominates that of ANN [122]. Along with un drained shear strength, bearing capacity columns made up of stones and cavity expansion factors are being discovered to be changes inversely and ultimately along with confining pressure available in columns at failure in statistical methods concern to footings spread on aggregate pier stabilized soil were analyzed. [123].

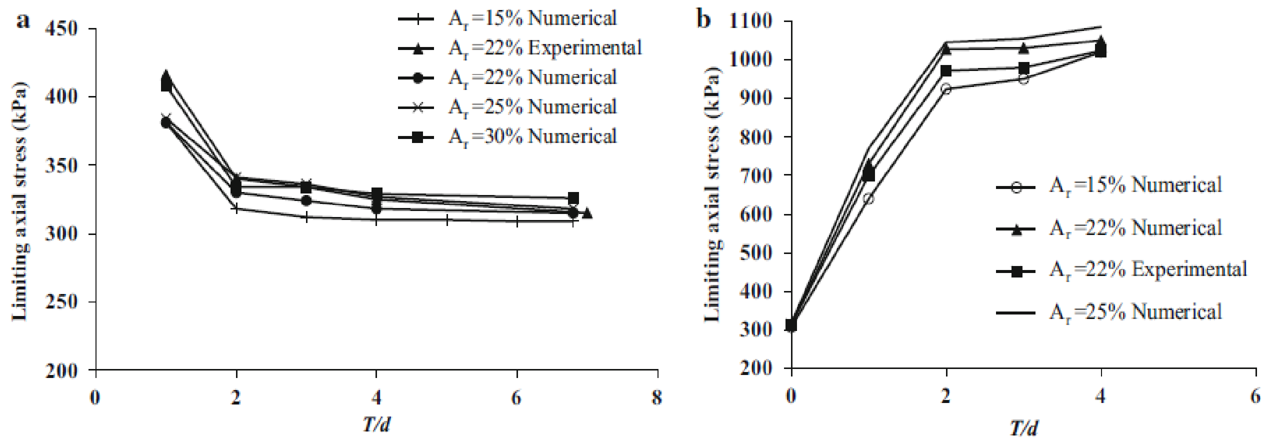


Figure 2.11 Limiting axial-stress of stone columns vs Top clay thickness of single stone column loading (a) L_1 where L_1= Soft clay overlapping over rigid clay layering arrangement (b) L_2 where L_2=Layering system rigid clay overlying loose clay [112]

Mohanty et al. [112] studied a software model based on finite elements has been developed for detailed systematic study. In the numerical analysis the soil and stone columns failure criterion was based on elastic-perfectly plastic Mohr Coulomb model with the drained conditions has been used. Results have indicated that for both layering systems, a variation in limiting axial stress of stone column with thickness of superior most clay layer has been obtained only up to the value when the thickness is further than two times the size of the stone column, it stays constant, while in the existence of a surface layer surface for depth up to four times the diameter of the column of stone whole ground surface experiences this change. (Figure 2.11).

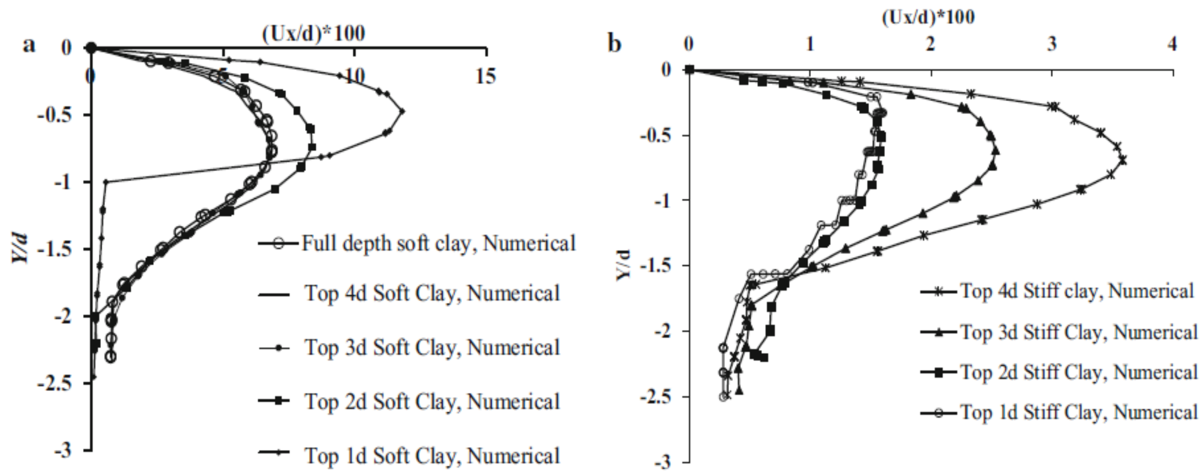


Figure 2.11 Lateral displacement along the length of stone column for stone column area loading (c) L_1 (d) L_2 [112]

Raee et al. [113] have also carried out series of finite-element analyses on prototype slope that have been found to compliment the results laboratory model tests in terms of optimum parameters and load-settlement behavior. Results shown in Figure 2.12 have concluded that increase in rigidity and decrease in stone columns spacing due to which the soil's bearing capacity is enhanced. Moreover, there has been significant improvement in load-settlement response of the rigid footing.

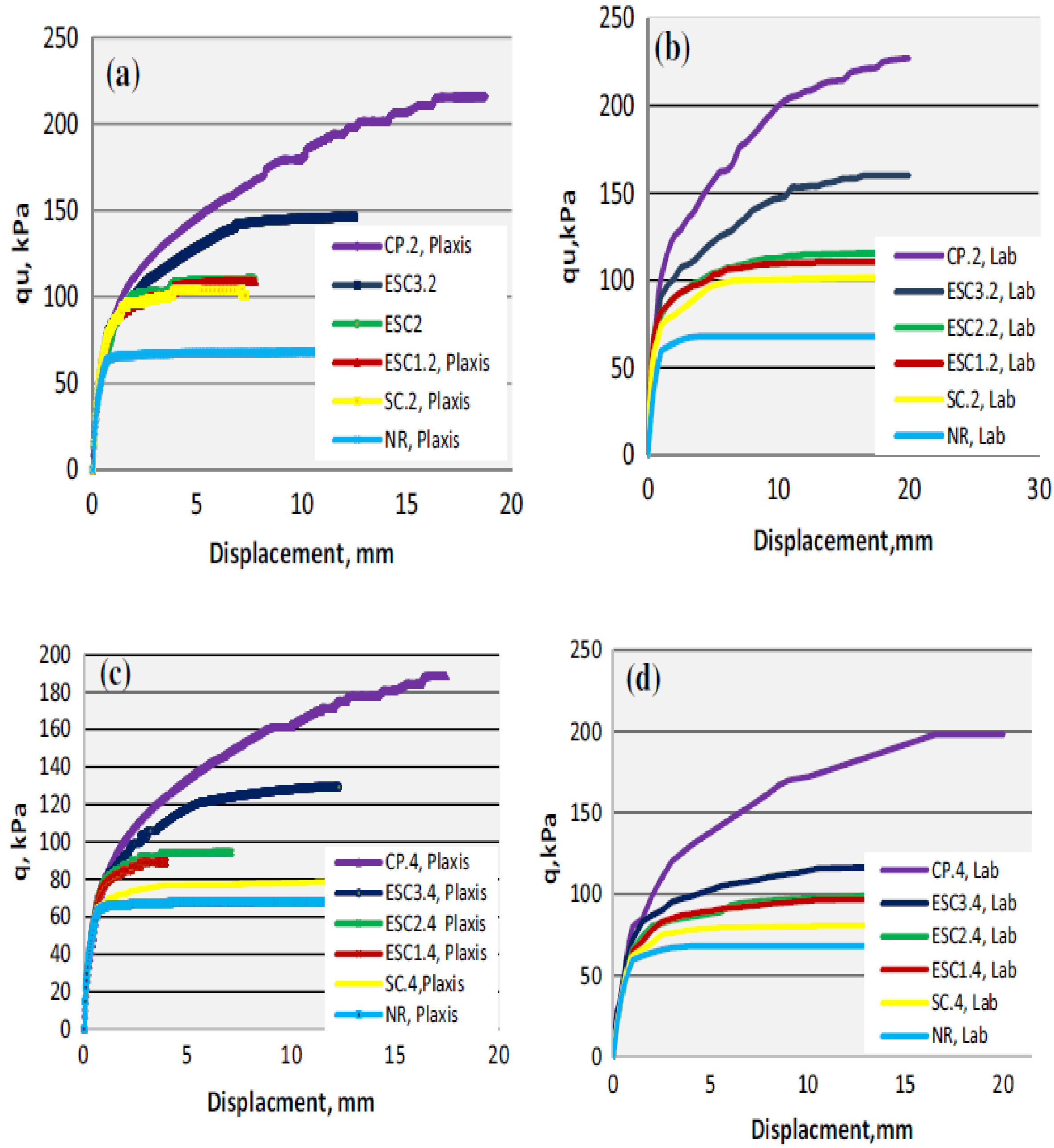


Figure 2.12 Rigidity effects on displacement due to load variation of multiple positioning of column a, b s/D= 2.0 also for c, d s/D= 4.0 [113]

The effect of granular bed on the stone columns has been studied by Nassaji et al. [124]. The numerical testing has considered Mohr Coulomb lost criterion for all the products and the accuracy of model has been found to be in agreement with results from laboratory test data. Findings have indicated transfer of stress towards the depth of column on construction of granular beds, thereby reducing the concentration of stress in columns upper part resulting in lesser lateral bulging.

It has also been observed the load carrying capacity has significantly enhanced as well as decreased settlement of the ground on using a granular bed on stone columns, with increase in the thickness of granular layer the effect becomes greater.

Castro [125] has considered a new approximate solution for rigid footing settlement resting on soft soil that is modified along with insertion of stone columns groups. The finite element method has been accustomed to confirm validation a few assumptions like load distribution with depth, simplified geometric model and boundary conditions. For making the problem axially symmetric set of columns have been modified to single columns of the very same cross-sectional area. Using Mohr-Coulomb model soft soil has assumed as linear whereas columns as plastic strains, value of dilatancy angle is kept constant by a non associated flow rule.

Division of soil profile is assumed to be independent on horizontal slices acc to method as well as compatibility- stress equilibriums of deformations were enforced in vertical and horizontal slices. Solution has been represented in closed form that can be used in a spreadsheet. There has been a good correlation between proposed solution and numerical outcomes as shown in Figure 2.13 & 2.14 within all range of common values, justifying validation of hypothesis solution. Further, solution has been found to be comparable to tiny level lab experiments cited in the literature.

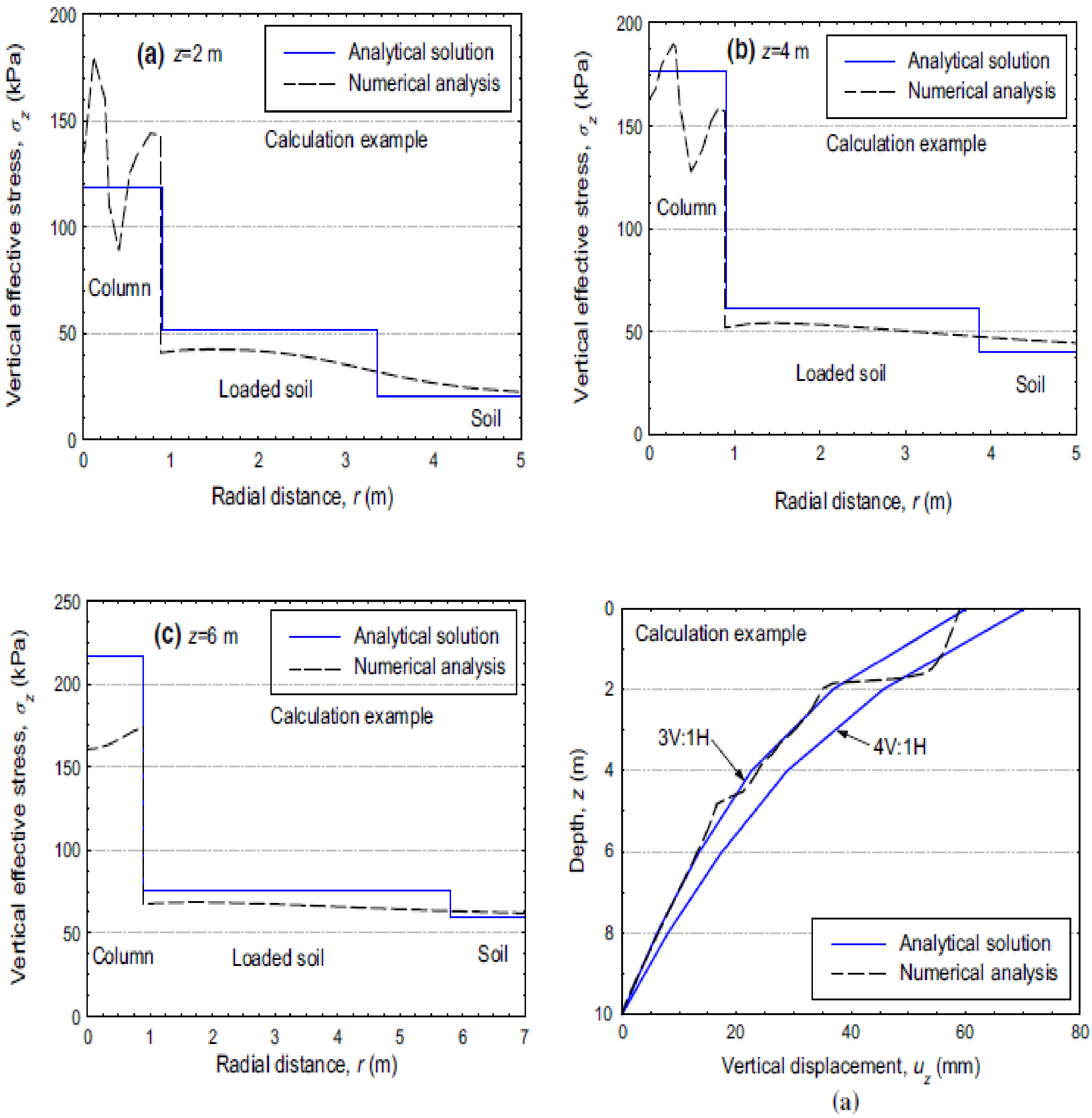


Figure 2.13 Vertical stresses at several depths [125]

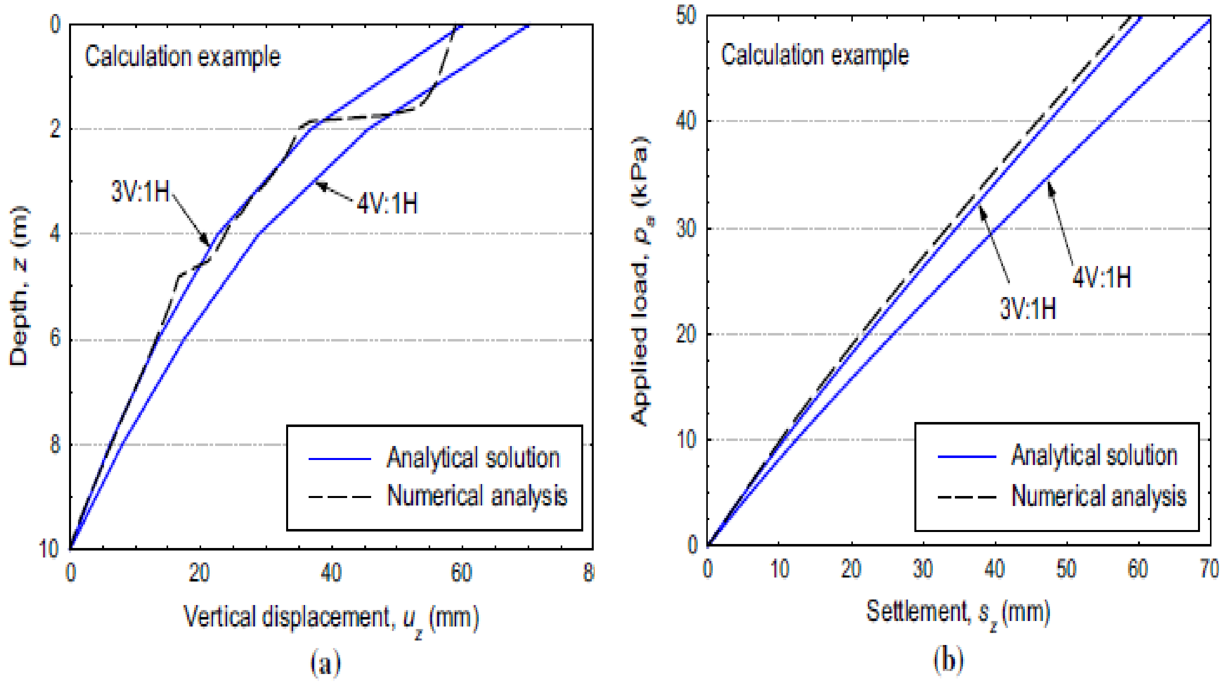


Figure 2.14 Various load grouping: (a) vertical displacement along with depth; (b) load–settlement curve [125]

Sexton et al. [126, 127] have employed elasto-viscoplastic Creep-SCLAY1S model along PLAXIS 2D is used explore influence of stone columns in creep-prone clay. (I) Isotropy, (II) Anisotropy, and (III) Anisotropy and bonding are three different cases which are considered. Ratio of total (creep plus primary) to primary settlement factors has found to be same for isotropic and anisotropic cases. However, for the combined case, smaller ratio has been observed. As soil creeps, there has been transference of vertical stress from soil to column resulting in additional yielding of column and reduction in radial stresses. These 2D and 3-D finite element methods have also been employed to examine the nature of mechanical characteristics of stone column, area replacement ratio, thickness of column and columns spacing on the load sharing and responded to the settlement of the soil [128-131].

Maheshwari et al. [132] have analysed a concentrated load travelling at a steady speed to which a infinite beam is exposed to and resting on stone column-reinforced earth beds, where Pasternak shear layer as granular filling layer, while Kelvin–Voigt model idealizes soft soil and Winkler springs stone column. Hyperbolic constitutive laws have been employed to depict the nonlinearity among granular fill, soft soil and stone column. Insertion of stone columns in soft soil reduces its liquefaction, resulting in the stability of the slope, protecting the structure during earthy vibrations. Also the group of columns performs better than the single column in the reduction of settlement [133-135].

Etezad et al. [136] have constituted a model utilizing limit equilibrium model and by assuming reinforced material as composite to analyze ultimate bearing capacity of installed shear collapse mechanism of a set of columns in soft soil. Stone columns performance of settlement is based upon the length of column, area ratio, number of column and low area ratios is found to be dominated by the column length [137, 138].

Kadhim et al. [139] have employed two dimensional finite difference model used by FLAC/SLOPE 7.0 software to assess the potential settlement of the road embankment built on soft soil modified by reinforcement with stone columns. The various engineering parameters like the column length, column's diameter, soil properties, height and the friction angle of the embankment fill have significant effect on embankment fill stability. Ng et al. [140] have studied the effect of parameters like frictional angle of column, undrained shear strength of surroundings and modular ratio on the bearing capacity of single stone column by employing 3D numerical review. Results have concluded bulging and bulging and punching in combination as two important failure modes for the system. Settlement ratio of floating stone columns in little along with huge loaded areas by 2D finite element analyses by Ng et al. [141]. Chenari et al. [142] have performed numerical simulation that was used to assess the bearing prediction ratio as well as settlement reduction factor to stone columns positioned in loose sands.

Effect of stone columns installation on improvement of soft ground has been studied by Shehata et al. [143] using finite element model (FEM). The analytical findings have reported increase in coefficient of lateral earth pressure, bearing capacity and settlement improvement factor and as a result given in Figure 2.15, the final loading stage stress increases and concentration ratio decreases. Numerical instabilities have been removed by combined use of 2-D and 3-D numerical analysis.

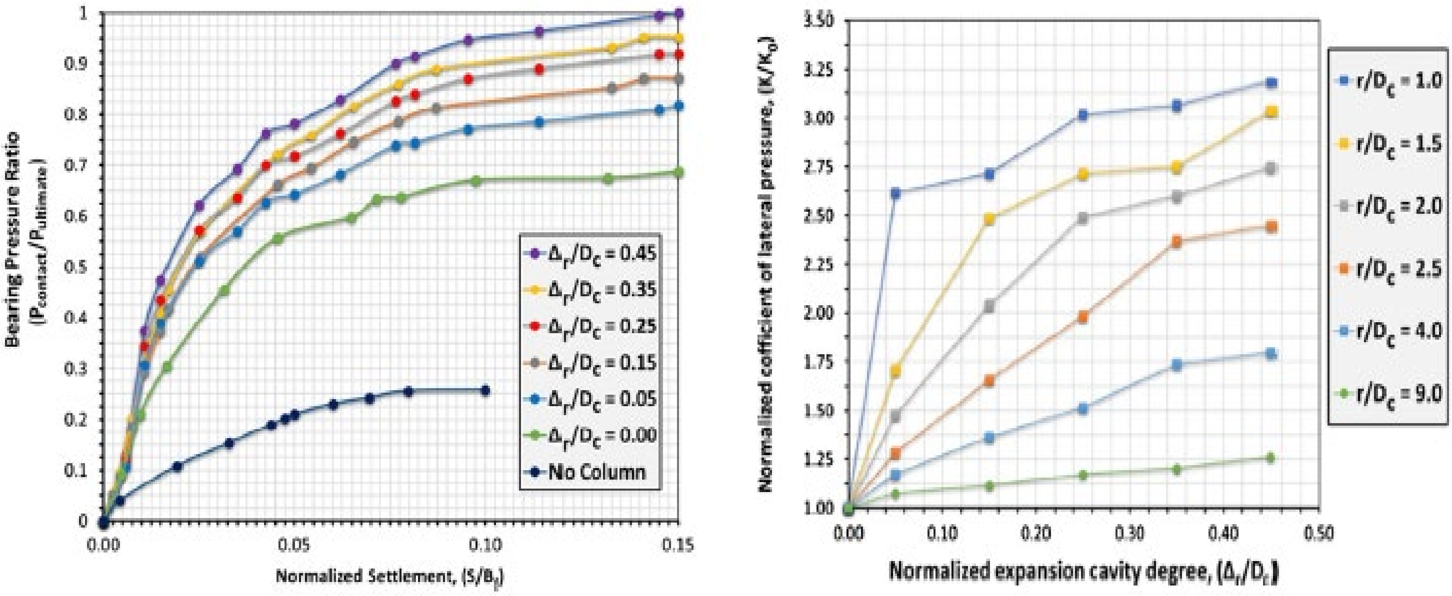


Figure 2.15 (a) Linked contact pressure (P) and Normalized Settlement relationship **(b)** For inconsistent expansion ratios (Δ_r /D_c) change in lateral pressure parameter (K/K_o) along radial distance (r/D_c [143]

The effect of deformation of column has also been studied by employing numerical modeling [144-146]. Castro et al. [147] studied effects of elastic strains of encased stone columns by employing an analytical solution on their plastic deformation as well as consolidation nearby the columns which is a technique for ground improvement in soft soils. Soil has been considered as an elastic material by utilizing a unacquainted flow law with consistent dilatancy angle and also the Mohr Coulomb yield criteria, and column as elastic perfect plastic material. Results have been presented in the form of spreadsheet. The researchers have compared the approach with loading process in drained conditions. The analytical solution has indicated more effectiveness of surrounding columns in its upper section that changes by depth before column appears elastic. Critical length of the encasement has been found to be higher in comparison to column length.

Ng et al. [148]; have investigated consolidation as well as settlement of a batter foundation, for this purpose 2D finite element analysis was conducted on the floating stone columns by use unit cell method. Both undrained and consolidated investigations have been conducted in the study. For various field substitution ratios, computerized settlements for excess pore pressure distribution have been compared over time (Figure 2.16).

A simple approximation method, based on these coupled consolidation experiments, has been developed for the extrapolation of consolidated degree for floating stone columns. This proposed model has been found to be much sensible than that of another design technique for yielding specifications and impact of salient parameters assumed in the research work. The frictional angle of column material, area replacement ratio, loading rate, as well as post-installation earth pressure are significant design considerations for floating stone columns. There has been a close agreement between suggested model and firmly developed Priebe's or α-β technique (Figure 2.17).

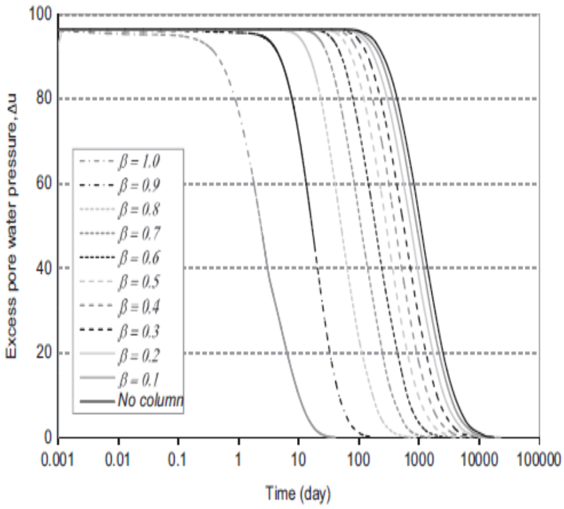


Figure 2.16(a) Excess pore water pressure distribution for β=0.0 to -1 [148]

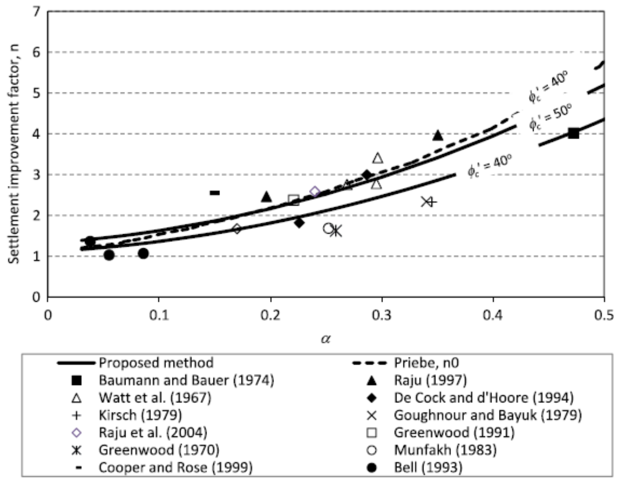


Figure 2.16(b) Comparison of End-bearing results [148]

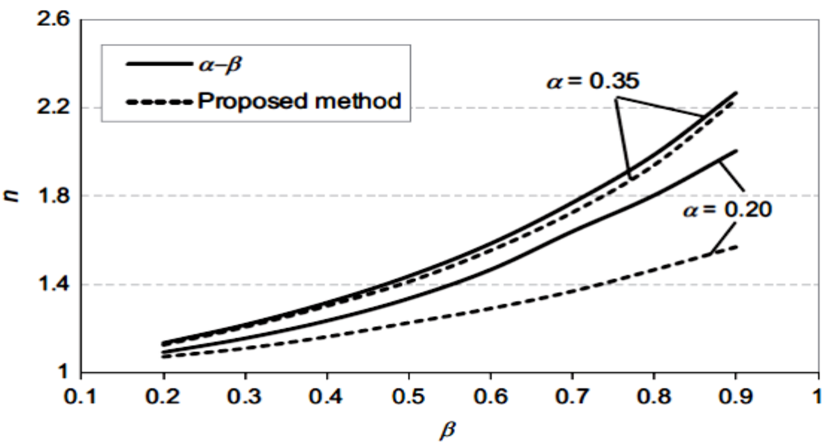


Figure 2.17 Comparison between Proposed techniques with the α–β technique [148]

The consolidation and settlement rate also depends upon the pressure distribution along the stone columns and clogging due to the migration of particles [149-150], mechanical parameters of the soil like shear strength [151], on the fact whether the single column or group of columns is utilized to modify the ground surface [152] and penetrating of stone columns by deep mixing method [153]. Hanna et al. [154] to examine the effect of various factors including elasticity modulus of material including clay content of stone columns, as well as column diameter, spacing, as angle of shearing resistance of column material on failure mode of the column numerical analysis have been conducted.

Andreou et al. [155] have conducted experiments to make a comparison three-dimensional calculation method for the soil reinforced with stone columns with the results from axisymmetric analysis in conjunction with in-situ measurements. The 3D method has been found to be effective particularly for the foundation soil with lower tensile parameters. The reduction in settlement has been predicted by applying unit cell theory and it has also been found that the even under high pressures, the failure of composite system does not take place. The results from 3D methods for the failure mechanism have shown a good correlation with those proposed by analytical method of the cylindrical cone.

2.5 CONVENTIONAL STONE COLUMNS (SC) THEORETICAL STUDIES

In the past a number of theoretical investigations have been performed for finding the behaviors of the stone columns. This result in improvement of various empirical, analytical, and numerical techniques for assessment of stone columns and often utilize the design models. Most of the studies based on stone columns make the use of axisymmetric finite elemental calculations instead of three dimensional computations due to the tedious nature of the latter. Furthermore the analytical method using Priebe's theory also attracts the attention of civil engineers as a method to estimate factor of reduction of settlement soil using a single stone column.

The design curves for determining the reduction on the settlement of conventional stone column reinforced soil were introduced by Greenwood. These empirical curves represent the variation of settlement reduction with columns spacing and undrained shear strength of natural soil. Later there was modification in the curves [156] where the settlement has been plotted against the area ratio (Figure 2.18). However, these curves are hardly utilized these days.

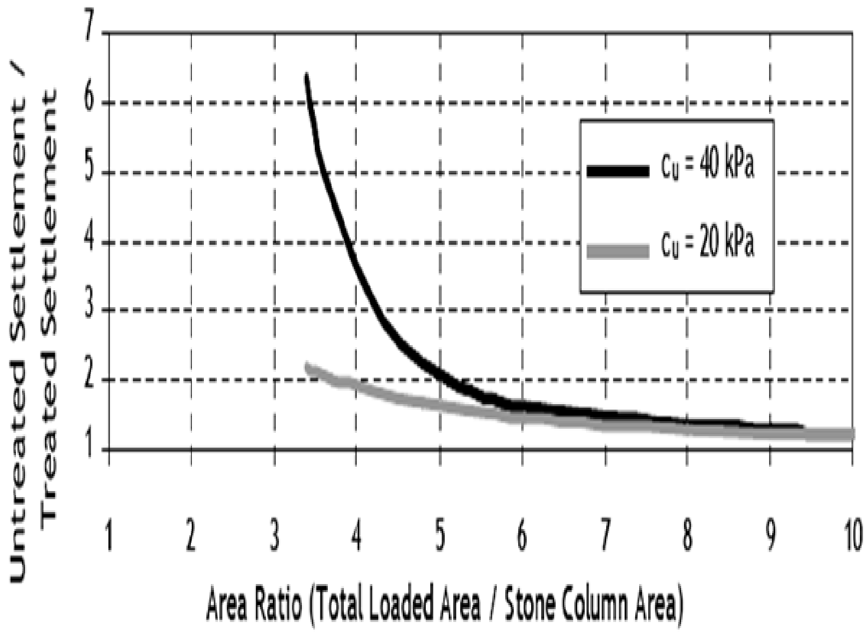


Figure 2.18 Modified Greenwood curves (plot of settlement vs. area ratio) [156]

Another theoretical method for the evaluating the settlement reduction has been proposed by Priebe [157]. The method involved the examination of unit cell which was the part of an unlimited load area on an unlimited column grid. Stone column were supposed to be encircled by elastic material and rigid. When confining pressure in nearby soil is lesser than lateral pressure in column, soil has started to settle. The design curves given by Priebe were the plots of settlement improvement factors as function of area proportions for a number of granular components (Figure 2.19). Technique has been modified and included modular ratio of column, column compressibility, as well as soil confinement from modelling stone in calculations.

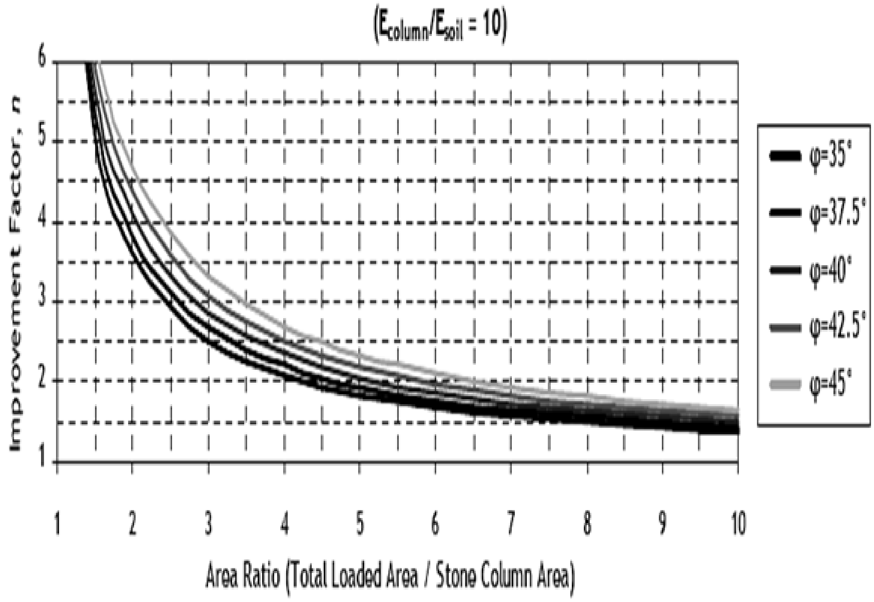


Figure 2.19 Modified Priebe design curves [157]

Pivarc et al. [158] has employed Priebe's theory for improvement of soft and loose fine grained soil reinforced with stone columns made up of crushed stones is stabilized by making vibrator in use . Improvement factors from this analytical method used have been compared with those from mathematical and experimental versions of stone columns and agreement between three methods has been found to be satisfactory. The three models in the sample have been created using the vibro-replacement process, which involves scraping soil from some kind of hole rather than flattening it to the sides as in the vibro-displacement approach.

Gueguin et al. [98] have analyzed the bearing capability of soft clayey soils which were installed with stone columns using yield design theory. An alternative homogenization technique to study the geotechnical structures has been adopted by the researchers as strong heterogeneity of clay makes it difficult to study such structures by means of direct methods. For establishing the macroscopic strength of the stone column reinforced soil with convex ellipsoidal sets procedure has involved an easy approximation of numerical lower and upper boundary conditions, followed by utilization of both static and kinematic approaches in an excepted finite element method. A geotechnical problem is subjected to this procedure to compute ultimate bearing capacity of foundations installed with stone columns. Shear strength of structure and efficiency of proposed numerical method has been highlighted.

Fatteh et al. [159] used mathematical analysis and the SPPS (Statistical version for the Social Sciences) software to create a general equation. The bearing capacity of clustered floating stone columns installed in clays of variable undrained shear strength between (4.0-25.0) kPa, as well as exclusive L/d ratios and diameter built by cased bore technique was determined using the equation. The far more guiding variable in predicting the load bearing potential of stone columns, according to research, is the area replacement ratio.

Han et al. [160, 161] have proposed a common theoretical approach for calculating consolidation rate for stone columns reinforced in soft clay by assuming (i) free drainage, (ii) high drained elastic modulus and (iii) deformation of stone columns. Findings shown in Figure 2.20 suggest acceleration of consolidation rate by enhancing modular ratio and minimizing diameter ratio. The solution also demonstrated that during consolidation , the stress shifting and the excess pore water pressure dissipation take place probably because of drainage and vertical stress reduction in the process. Lorenzo et al. [162] have described an equation demonstrating the interdependence of pile consolidation and its surrounds soil by studying the consolidation of a typical unit cell of soil-cement stone column upgrade ground. It has been stated that the soil-cement pile efficiently regulates the radial drainage of water into nearby soil as well as entrance of pore water from nearby soil during consolidation.

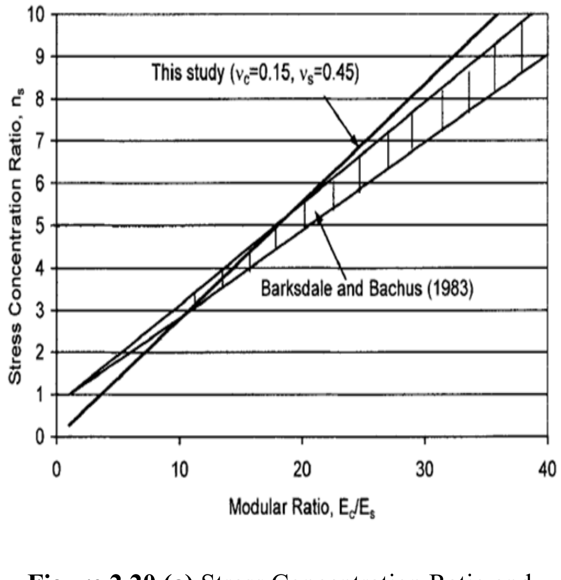


Figure 2.20 (a) Stress Concentration Ratio and Modular Ratio relationship [160]

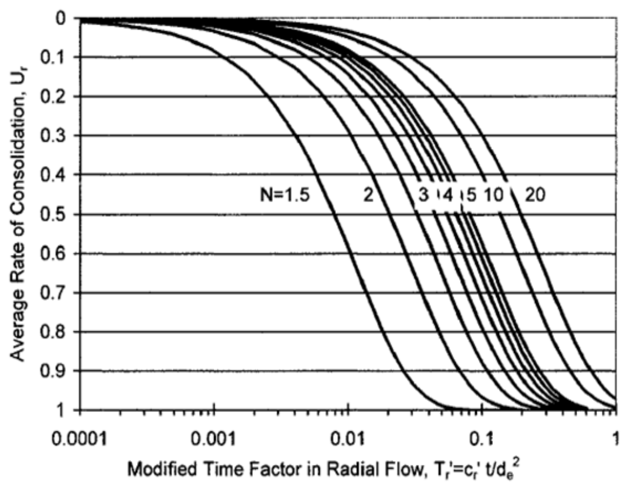


Figure 2.20(b) Consolidation rate in Radial Flow [160].

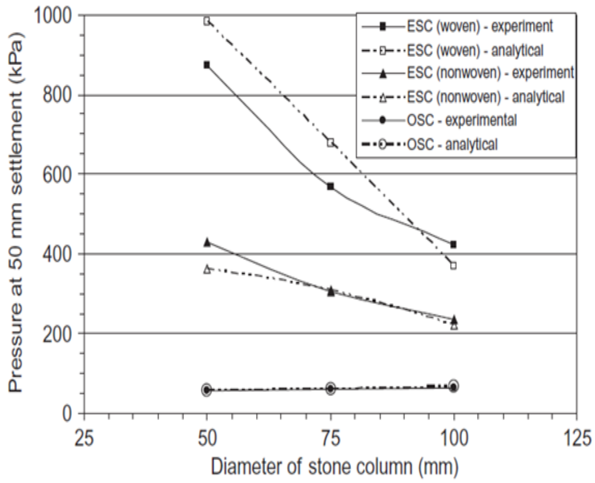
Number of consolidation theories has been generalized by different researchers by considering vertical and radial flows within column and effect of parameters of improved soil [163-165], whereas some of the consolidation investigations consider the influence of static and dynamic loads as well as lateral deformation on the improved soil [166-168].

2.6 ENCASEMENT MATERIALS USED IN LITERATURE

The successful confining stress exerted by surrounding soil, which varies with the construction process, determines the tensile strength of the stone column. During the placement of the fill, an axial deformation of the stone column occurs, followed by lateral expansion of the underlying rock, resulting in increased carrying potential and mobilisation of additional confining tension on the stones. In extremely soft soils, stone columns installation reduces the efficacy of columns owing to inadequate lateral confinement due to the soft soil. To cope up with this effect, the stone columns reinforced along with encasement materials have been commonly employed, providing satisfactory results [169-171]. Encasing stone columns boosts carrying capability as well as decreases composite base settling by avoiding undue enlargement and grinding of the stone into in the earth. [172-173]. Also safety factor of the structure increases because stabilization of slope is been described in presence of reinforced stone columns, [174]. The possibility of failure of the stone columns due to bulging gets delayed in the encased stone columns [175-176]. The literatures have suggested the usage of number of sustainable materials including geopolymers, fly ash, areca fiber, calcium carbide residue as stone column fillers [177-181].

2.7 ENCASED (BOTH VERTICALLY AND HORIZONTALLY) SC EXPERIMENTAL STUDIES

Murugesan et al. [182-184] investigated the quantitative and qualitative enhancement of stone column load potential by conducting lab model studies on stone columns placed in clay surfaces rendered under organized conditions at a large scale in an experimental tank. Experiments were accomplished with a set of stone columns given in Figure 2.21 without as well as with the geosynthetic encasements. The results have shown that the axial load capacity is directly proportional to modulus of encasing material and column diameter. Stress concentration is increased on stone columns in contrast to unreinforced columns has showed that these columns behave as semi rigid piles. Results were further used to design a model for geosynthetic encasement for present settlement and load in Figure 2.22.



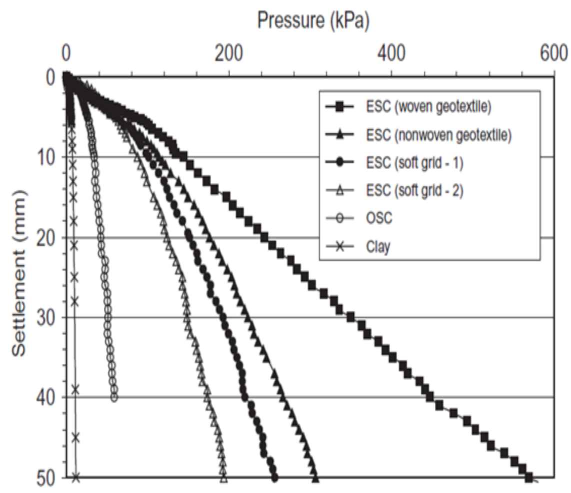
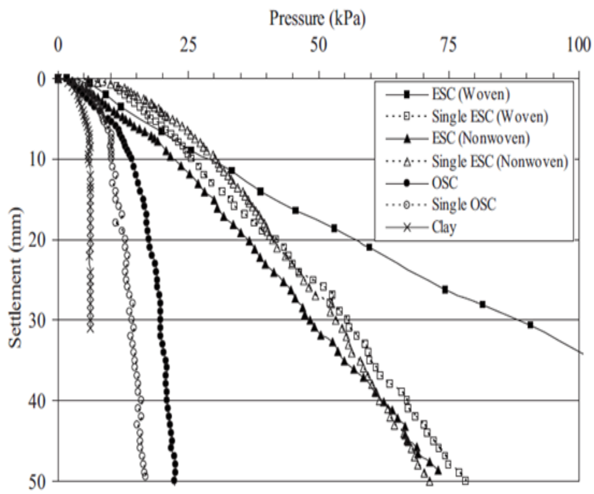


Figure 2.21 (a) changes described in pressure correlated to 50.0 mm settlement along with diameter of stone column [182]

Figure 2.21 (b) Pressure–settlement curves for 75.0 mm diameter columns encased in various kinds of geosynthetics [182]



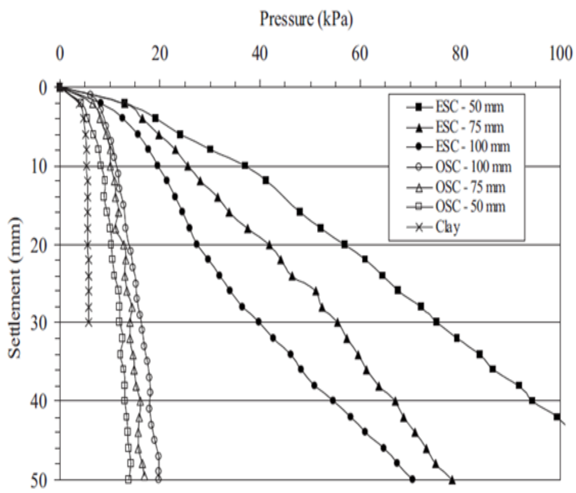


Figure 2.22 (a) Comparison between correlated responds from tests on isolated columns pressure-settlement respond to set of stone columns [184]

Figure 2.22 (b) Pressure-settlement responds of solo OSCs and ESCs (woven) [184]

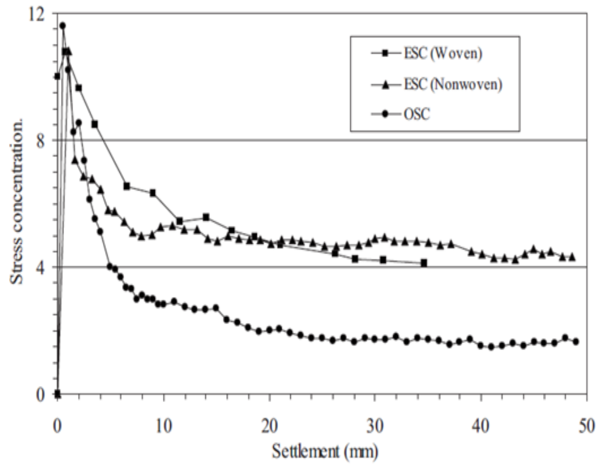


Figure 2.22 (c) Effect of stress concentration columns made up of stones with settlement (load test on set of columns-75.0 mm diameter) [184]

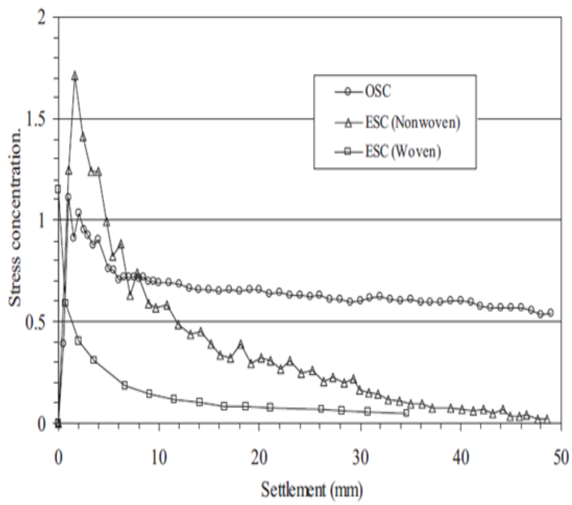


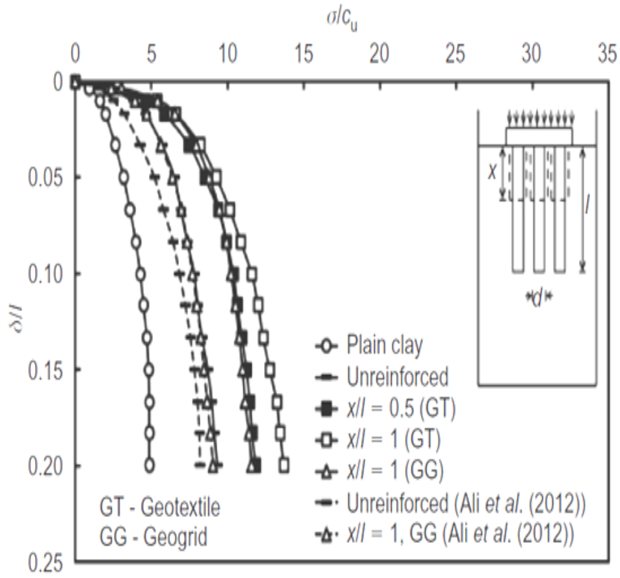
Figure 2.22 (d) Concentration of stress on clay surface with settlement [184]

Due to insufficient lateral resistance offered by very soft soil in the case of bulging of columns on loading stone columns have been found to experience excessive settlement; this bloating can be dissipated to some degree by encasing the columns with geosynthetics or by inserting horizontal circular geosynthetic discs within columns at uniform gap[185-188]. By assorting numerous variables for instance column diameter, columns spacing etc. in discrete types of stones impact of geosynthetically reinforced stone columns has been studied [189-192].

By reinforcing soil vertically or horizontally and by use of encasement of iron or by mixing fibre behavior of encased stone columns has also been considered. The lateral confinement provided by geosynthetic material in case of encasement reduces the lateral bulging to 3/4th of the bulging in ordinary stone columns [193-197].

Ali et al. [10] have carried out model tests to measure comparative improvement in composite ground breakdown tension due to separate reinforcement in long floating as well as end bearing single as well as sets of columns with/without the assistance. With the help of exhumed deformed column shapes failure mechanism for various kinds but also the arrangement of these supports has been studied. For end bearing columns geogrid has been observed to be the most suitable geosynthetic encasement, whereas both geotextile and geogrid was similarly successful for horizontal circular discs and encasement configurations for floating columns Figure 2.23.

Other important findings from studies (a) Geosynthetic encasement improve bulging by providing lateral imprisonment to columns through mobilization of hoop stresses, while horizontal circular discs do the same by friction mobilization; and (b) comparative behavior of floating and end-bearing group columns on bear capacity of improved ground for various kind and arrangement of stone columns has been observed similar to perform of unsettled and end-bearing single columns.



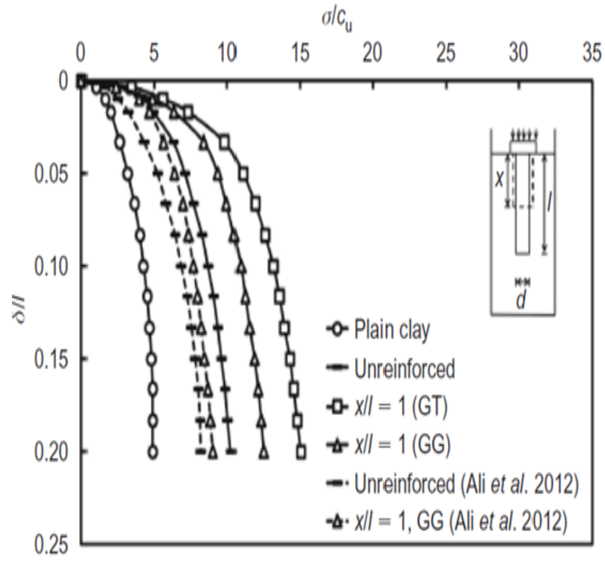


Figure 2.23 (a) The effect of geosynthetic form with encasement length over composite ground failure stress was enhanced with even a set of encased floating columns of stones (d=30.0 mm, l = 300.0 mm, Dr =50.0 %, Ar =25.0 %). [10]

Figure 2.23 (b) With solo fully encased floating columns made up of stones (d=30.0 mm, l= 300.0 mm, Dr =50.0 percent, Ar =25.0 percent), the effect of different forms of geosynthetic and encasement duration on collapse stress of the synthesized ground was increased. [10]

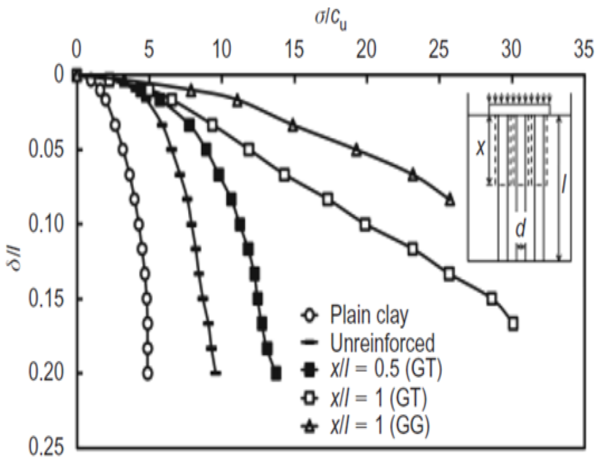


Figure 2.23 (c) Consequences of encasement in case of synthesized ground upgraded with set of end-bearing columns (d=30.0 mm, l=300.0 mm, Dr =50.0 Percent, Ar =25.0 Percent) [10]

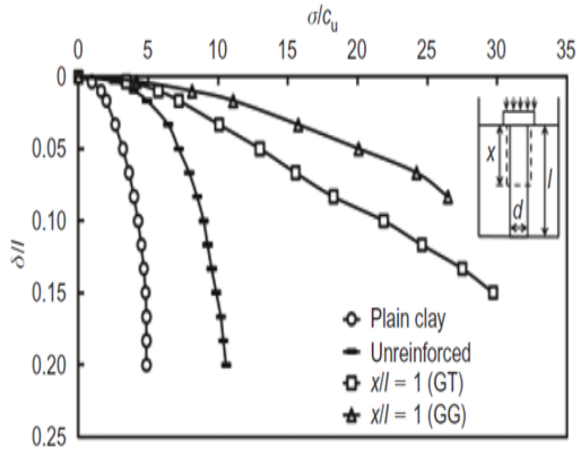


Figure 2.23 (d) Consequences of encasement in case of synthesized ground upgraded with only end-bearing columns (d=30.0 mm, l=300.0 mm, Dr =50.0 %, Ar =25.0%) [10]

Procedure of casing outer walls of granular piles with geotextile or geogrid of certain tensile strength has been explored by Yoo et al. [198]. In case of bearing and settlement ability due to their improved strength, such confined piles have been found to have more advantageous than conventional sand compaction piles (SCP). As compare to SCP or Gravel compaction pile (GCP) methods lesser aggregation has been found in this method. Various mechanical characteristics like load carrying strength as well as concentration of stress ratio of a composite ground enhanced using SCPs strengthened with geotextile encased sand piles (GESP) has been depicted by conducting the tests. On artificial sediment clay surfaces reinforced several loading tests have been performed with three varying tensile strengths of geotextiles with different replacement ratios using conventional SCPs and GESPs. Results shown in Figure 2.24 have indicated that larger bearing capacity of soft clay is received when settled with GESPs as compare to capacity when settlement is done with SCPs and failure mechanism is different in both cases; in former case it is due to buckling whereas it is bulging in latter one. However, in buckling failure of GESPs tensile strength merely affects load carrying capacity.

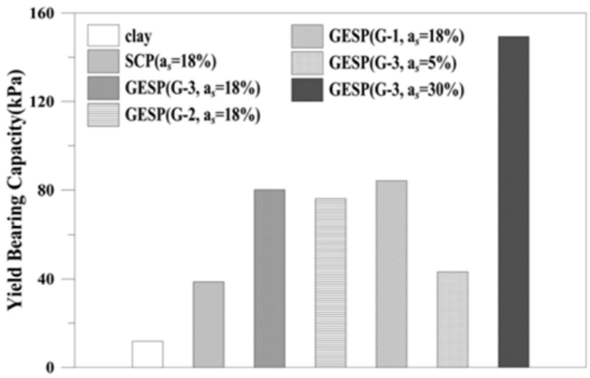


Figure 2.24 (a) Comparison of Ultimate Bearing Capacities of Test Cases [198]

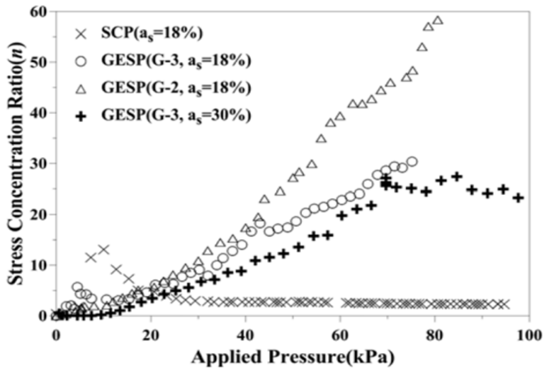


Figure 2.24 (b) differentiation of Stress Concentration Ratio from Experimental Findings [198]

Harish C et al. [199] have investigated the improvement of clay using sand columns which have been using for ground modulations where flexible structures are there including oil storage tanks, road embankments etc. Tensile strength of surrounding black cotton soil is main factor on which load carrying capacity of these columns depends. In addition, sand columns reinforced with geosynthetics have also been suggested for improving the load bearing ability of columns thereby ensuring their easy formation in weak soils. Study has also emphasized on effects of diameters and evaluation of load responses with inequality of column encasement length. Important outcomes of investigation are (i) combination of 15% sand and 15% lime in sand columns with the 30 mm diameter of the column in five columns group give better strength of 1.5 kN; (ii) 30 mm diameter column in five columns group encased with 100% geogrids provides higher strength of 2.234 kN; and (iii) homogeneity of soil is affected by more number of closely spaced layering and reduction in improvement rate of cotton soil results due to formation of band between soil and encasement.

Hasan et al. [200] have conducted a sequence of lab model experiments and numerical analytical calculations on granular piles reinforced with geo synthetics under a small loading using unit cell idea. Experimental tests have been performed on unreinforced, horizontal and vertical reinforced and combined vertical &horizontal reinforced granular piles. Either the whole cylindrical tank or piles area is subjected to loading. Encasement, its stiffness, tensile power of clay, dimensions of the granular piles in the construction of foundation are various geotechnical factors whose effects has been analyzed by this study. Vertical load intensity-settlement relationships represent laboratory results and have been compared with PLAXIS 3D results. Experimental findings have concluded that significant alteration in ultimate load carrying capacity of piles and ultimate bearing capability of the modified surface has been made by encasement. Fattah et al. [201] conducted various tests by varying the column spacing, two length-to-diameter (L/d) ratios of columns made up of stones and distinct embankment heights the performance of embankment models constructed on loose soil reinforced with standard and encased stone columns have been compared. For this purpose, 39 model experiments have been conducted on soil with undrained shear strength of approximately 10 kPa.

To measure vertical stress on a column, earth pressure cells are used in both models, and for reinforced soft soil, the same has been computed by putting another cell at the embankment base between two columns. For embankment models on soft clay reinforced with ESCs (either floating or end bearing), encasement by geo grids are well efficient in an improvement of bearing ratio of reinforced soil by approximately 1.290, 1.390 and 1.630 times & 1.40, 1.570 and 1.830 times that of untreated soil for 200.0, 250.0 & 300.0 mm embankment heights with L/d =5.0 as well as 8.0 sequentially& spacing s=2.50d shown in Figure 2.25. It has also been found that bearing capacity of the treated soil or untreated soil rises with decrease in stone columns spacing for a present embankment.

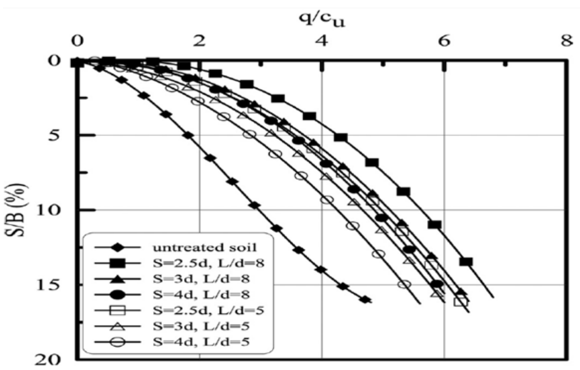


Figure 2.25 (a) For a 200.0 mm high embankment model residing on soft soil reinforced to ESCs [201] bearing ratio Vs settlement ratio plot

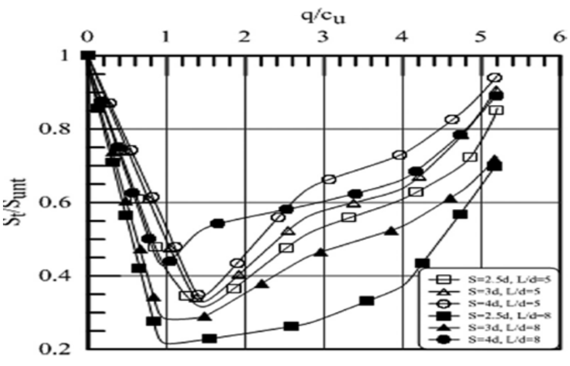


Figure 2.25 (b) Bearing ratio for a 200.0 mm large embankment lying on loose soil reinforced on ESCs versus Settlement improvement ratio [201] representation

Investigation on encasement stiffness and strength on behavior of independent granular columns encased with geotextile and reinforcing soft soil using model tests has been conducted by Hong et al. [202]. The displacement Vs Pressure curve has been shown in Figure 2.26. To ensure comparable response of prototype-scale and model-scale geotextile encased columns, similarity tests have been first performed to compute appropriate parameters of constituents used in model tests. Experimental outcomes have established in improving all the modeled sand columns bearing capacity on encasement, as well as on its rupture. Further, for sand columns encased in medium to high-density geotextiles rigidity there has been noticeable up gradation. The radial strain of the columns has also been found to be controlled significantly on encasement. There has been prevailed expansion of sand columns encased with small stiffness geotextile in the upper 2.50 D depth of columns, however, a rough steady lateral deformation with the height of column have been noticed in case of column encased with relatively high rigidity.

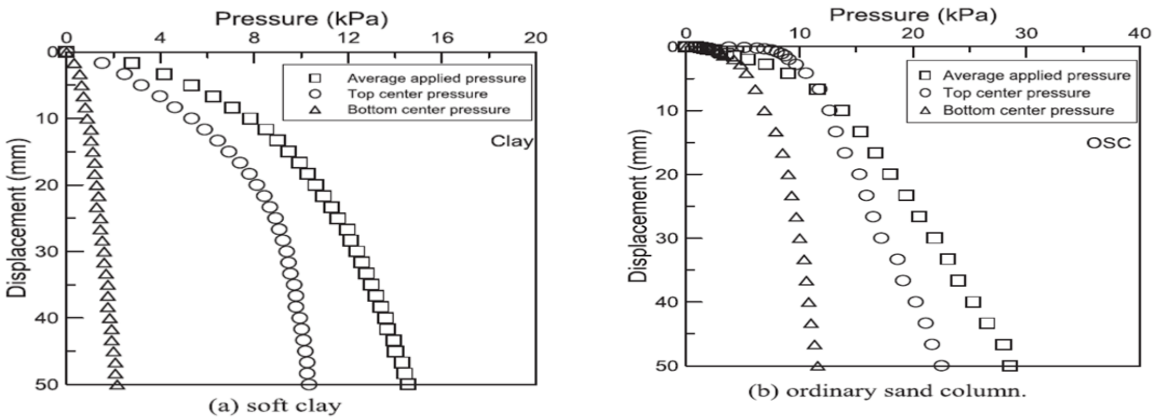


Figure 2.26 Representation of vertical pressure-displacement association with ordinary sand column as well as soft: **(a)** soft clay; **(b)** ordinary sand column [202]

The field response of soft clay foundation with geosynthetic encased stone columns has been assessed in two test embankments by Hosseinpour et al. [203]. They have showed reduction in maximum horizontal displacement of soil by three times for about 2.5 times high load and settlement of soil by improvement factor of about 5 for equal load application in matter of geotextile encased granular columns (Figure 2.27). Using the geosynthetics as reinforcement, there has been significant decrease in extra pore water pressure which gives rise to more stabilization of ground.

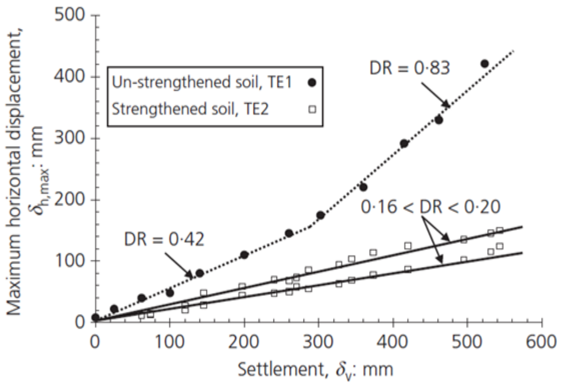


Figure 2.27 (a) Graph showing Co-relation between soil horizontal deformation for TE1 and TE2 and settlement [203]

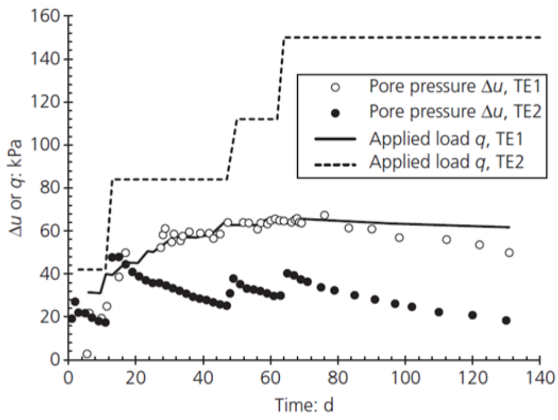


Figure 2.27 (b) Discrepancy of extra pore pressure estimated in middle of soft clay with time for TE1 and TE2 [203]

Miranda et al. [204] have considered response of stone columns utilizing consolidated drain triaxial tests executed on encased and non-encased stone columns. For that, they have tested two stone columns of variable densities with two different geotextiles. A change has been observed in the volume of encased specimens while defrosting process and isotropic consolidation stage, making encasement diameter slightly higher than that of stone columns and resulting in development of certain axial stress in the sample which leads to noticeable efficacy of the geo textiles.

The study has also highlighted the outcome that enhancement in confining pressure provided by mobilized friction angle of the stone columns and geotextiles, resulting in better tensile strength of encased versus non-encased ones. There has been significant improvement in the results when encasement of the stone columns was done with geotextiles; the effect being stronger at low confining pressure.

Miranda et al. [205] have also conducted a laboratory study to observe the influence of two different geotextile encasements on behavior of soft soils installed with full stifling encased columns in terms of pore pressure, soil column stress dispersal as well as soil deformation while the method of consolidation. As results shown in Figure 2.28 small scale laboratory tests have been performed to analyze a horizontal slice of a representative "unit cell" and these tests were performed for a big instrumented Rowe-Barden odometric cell.

Observations have indicated that encased columns have been capable of sustaining about 1.7 times more vertical stress in comparison to non-encased columns. Also stress concentration factor for non-encased columns has been found to be lesser than encased columns. Behavior of two variable geotextile encasements has been found to be quite similar, both showed comparable stiffness at low radial strains. However, there have been large differences at the end of tests, probably resulting from partial breakage of longitudinal joint in one geotextile due to bad adhesion of geotextile fabric.

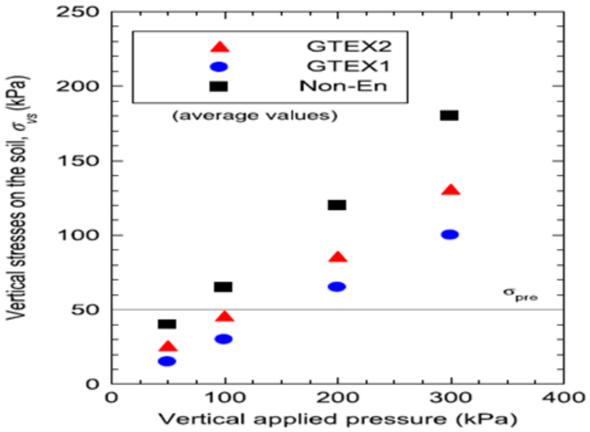


Figure 2.28 (a) Representation of vertical stresses on soil under drained conditions [205]

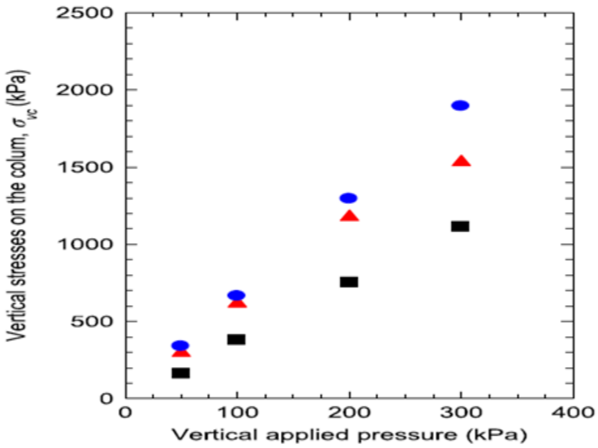


Figure 2.28 (b) Representation of vertical stresses on column under drained conditions [205]

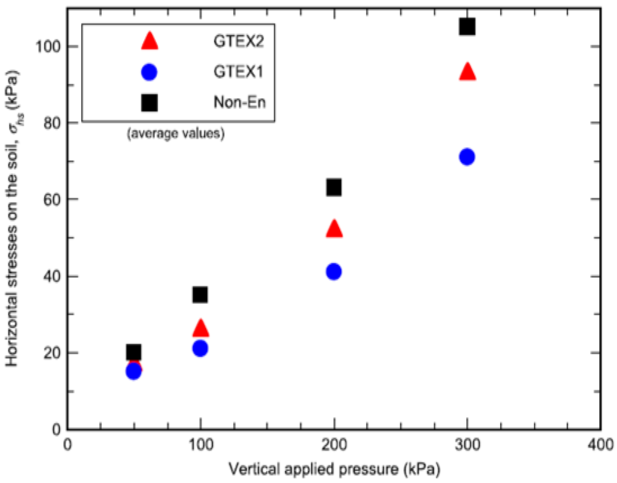


Figure 2.28 (c) Soil having horizontal stresses under drained conditions [205]

Akhitha et al. [206] have used the shredded tyre chips and aggregates as material for stone columns. To find out load settlement response of end bearing and floating stone columns with/without geotextile encasement have been prepared and plate load test has been carried out on these columns. Results given in Figure 2.29 confirmed greater load carrying capacity in case of geotextile encased columns containing 70% stone and 30% tyre chips. Often the end bearing columns have showed outstanding performance that the floating ones. The usage of waste shredded tyre chips has appeared as low cost, efficient and eco friendly method.

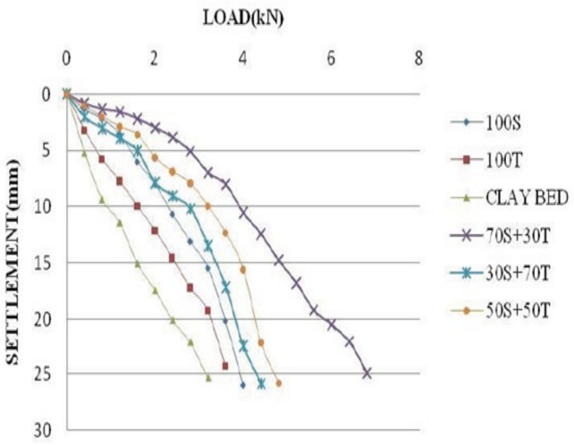


Figure 2.29 (a) Test results for floating columns [206]

Figure 2.29 (b) Comparison of floating columns with and without encasement [206]

Dutta et al. [207] have discovered innovative use of plastic water bottles as new kind of encasement for fly ash columns penetrating completely in loose clay. Columns of ash have been covered with geo cell-reinforced fly-ash beds besides jute geotextile separator, with cellular mattresses made of plastic water bottles. In loose clay, systematic model testing on encased fly-ash columns, geo cell composite systems, and encased fly-ash column-geo cell composite systems were conducted; these updated systems were found to achieve increased footing capability by 5.0, 8.50, and 12.0 folds, respectively, as compared to the untreated clay bed. With enhancement in mattress height over column, encased column has contributed less accompanying by an increased been quested from geo cell mattress in overall footing capacity in composite system. Further, tests have been carried out on sets of three as well as four end bearing encased fly-ash columns in triangular as well as square shape correspondingly to compute efficiency of suggested plastic bottle encasement in the sets of columns; where the sets of four columns showing preferable footing capacity besides that over individual column and set of three-columns Figure 2.30.

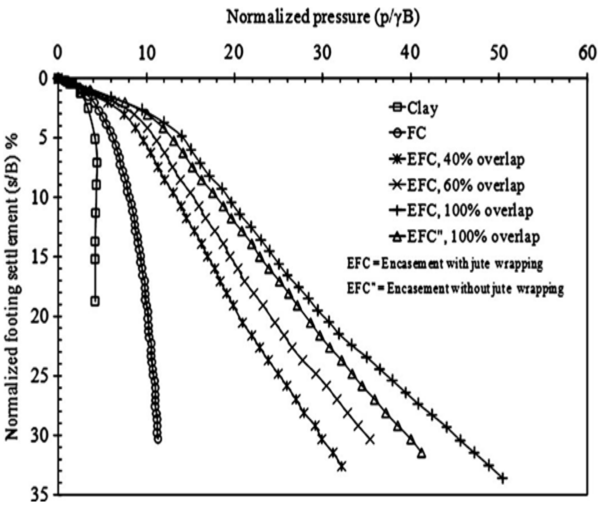


Figure 2.30 (a) Footing depicting pressure-settlement responses across fly-ash columns of full-length encasements formulated along differing lap over percentages (.40.0, 60.0, & 100.0 %) throughout plastic bottle cells. [207]

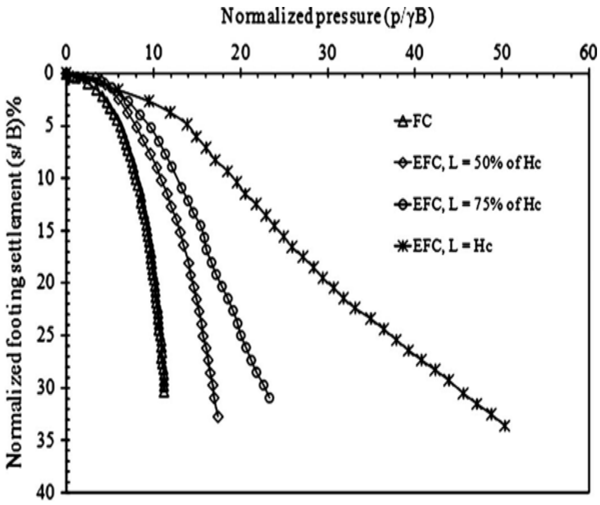


Figure 2.30 (b) Footing showing pressure-settlement responds in column of encased fly-ash columns having different lengths (100.0 % lap over) [207]

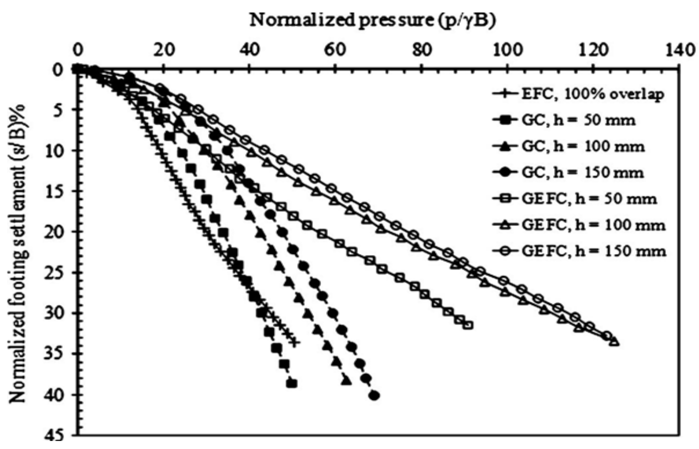


Figure 2.30 (c) Footing showing Pressure-settlement responds throughout various composite systems (GC, EFC, and GEFC) in soft clay [207]

Sarvaiya et al. [208] have considered response of reinforced floating stone columns rested on soft soil. Different types of geosynthetic materials RSC- Monofilament, RSC TF-41, RSC TF-422 and for RSC TF-52J have been tested. The load settlement results using these materials have been compared with untreated clay bed and with soil installed with ordinary stone columns to understand about the effects of various forms of reinforcement on load settlement. It has been found that if shear strength of the encasing material is enhanced, load bearing capacity of treated surface has been enhanced. Also, the half-length of the encasement has performed equally well in carrying load when compared to fully encased columns in case of geo synthetically reinforced stone columns.

A descriptive review on the behaviours of standard stone columns (OSC) as well as rigid stone columns (RSC) in sandy earthen slopes has been conducted by Hajiazizi et al. [209]. For this purpose, they have constructed an embankment sandy slope and then saturated it with rain and finally loaded the increment. The Observations from this lab modeling have been verified through 3D finite difference method.

Kumar [210] has combined the geo synthetic encased columns and vacuum application method to examined the development of exceptionally soft clay soils. Small scale unit cell tests were carried out to determine the efficacy of the encased stone column, under which the columns were exposed to vacuum loading under various pressures. The results from these studies have been matched with those of ordinary surcharge preloading conditions shown in Figure 2.31. Applying vacuum to clay soil by encased stone columns uniformly spread the vacuum pressure, resulting in instant settlement. The rate of consolidation and undrained shear strength of the soil have been found to be significantly improved when vacuum is applied. Thus the vacuum application method through encased stone columns can be used as feasible method for the improvement of ground that would accelerate the construction activity making it economically beneficial.

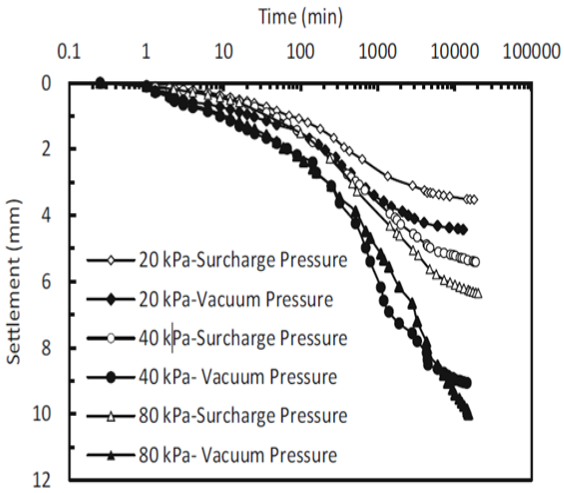


Figure 2.31 (a) Vacuum and surcharge differentiation preloading time-settlement graph [210]

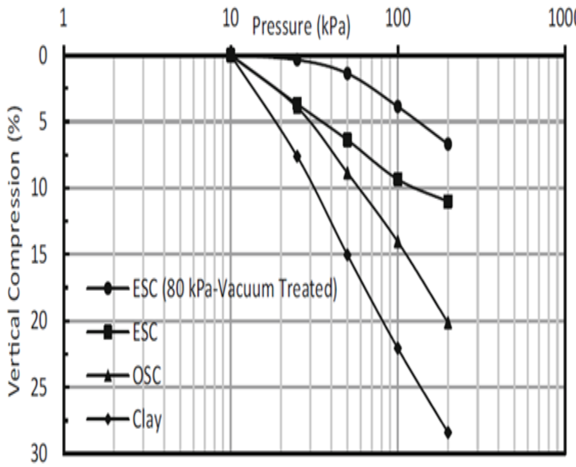


Figure 2.31 (b) Pressure Response – Vertical Compression comparison chart for various systems [210]

Mehrannia et al. [211] have carried out experiments for bearing capacity of a individual stone column, a single granular blanket as well as the combination of both, using large scaled physical methods and found an enhancement in all the three cases. The granular blanket, reinforced with geogrid, has been examined in two variable thicknesses whereas the stone columns have been installed with geotextile. The results shown in Figure 2.32 suggested that in case of combination of stone column and granular blanket, there has been development in loading bearing capacity of soft soils with advantages of horizontal and vertical drainage. The encasement of geo grid and geotextile in granular blanket and stone column respectively, has improved the effectiveness of method by increasing rigidity of soil. Installation of 75.0 mm reinforced granular blanket over a standard or reinforced stone column has found to partially increase the maximum load ratio, thus improving the effectiveness of individual constituents. Punching mode has been observed as failure mode for the stone columns, when combined with granular blanket. In all the cases, deformations in stone column have been symmetrical with no lateral distortion.

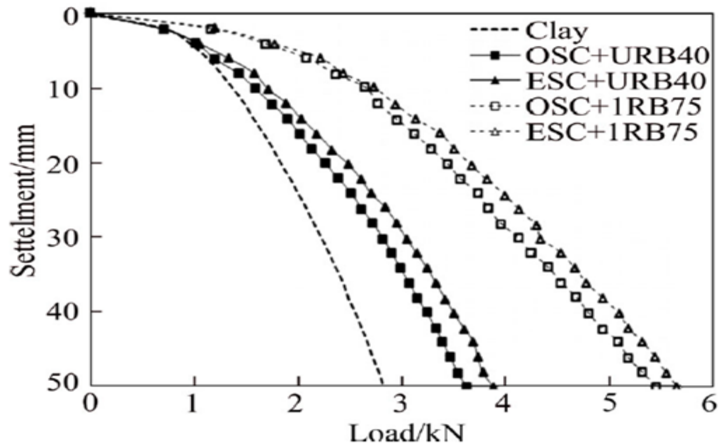


Figure 2.32 Variation of load settlement in combined form of granular blankets and stone columns [211]

Rathi et al. [212] have studied the development of soft soil after construction of stone column by comparing the results for columns reinforced soil with those from unreinforced as shown in soil (Figure 2.33). It has been found that when we use geogrid stone columns settlement of soil has been decreased from 17 mm to 8 mm, probably because of higher stiffness and more density of soil on construction of stone columns. However, when the stone columns made of geotextiles have been used, this settlement has been reduced to 7.5 mm. the reason for this decrease can be greater rigidity of the soil and reduction in the failure of the columns. It has been studied that the loading of stone columns with horizontal reinforcement leads to the penetration of some stones into the soil bed, which has been reduced in case of geotextile encased stone columns, provided the path for the drainage.

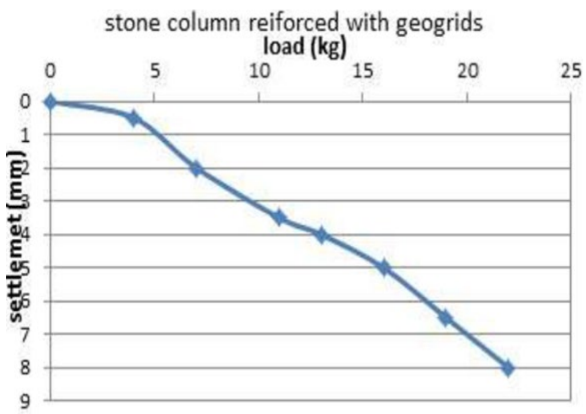


Figure 2.33 (a) Soil with stone column reinforced with Geogrids showing load settlement curve [212]

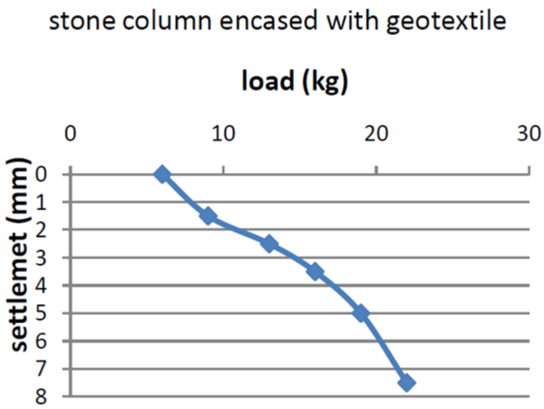


Figure 2.33 (b) Representation of load settlement curve of soil bed with stone column encased with geotextile [212]

Lajevardi et al. [213] have performed experimental investigations on individual and set of encased stone columns and compared results with unreinforced stone columns. They resulted that by using the geotextile encasement bearing capacity of the individual as well as set of columns has been increased. Also the square grid formation of stone columns has found to be more effective in enhancing the load bearing ability when compared to the triangle formation of columns in group.

Das et al. [214, 215] conducted experimental experiments with a rough soil-cement bed over the column to predict load carrying ability and bearing capability of stone-based columns. Person and collection of stone columns were subjected to experiments without and with a sheet of soil-cement bed. This bed has significantly reduced the bulging and vertical settlement and hence there is the reduced stress on the composite foundation. Therefore, overall settlement reduction of foundation results that due to the increase in the load carrying capacity and bearing capacity of stone columns. Authors have also examined the thickness effects of this layer, columns length and columns spacing on development of performance of stone columns.

Gniel et al. [216, 217] have performed small scale tests by considering the overlapping of encasement and interlock among stone aggregate as well as section of overlap to analyses the actions of geogrid-encased columns of stones. The reinforcement of geosynthetics can be done horizontally as well as vertically. However, bearing capacity of stone columns gets improved by utilizing horizontally reinforced film as [218, 219].

The effect of reinforcement pattern has been analyzed by Hasan et al. [220] by comparing reinforcement of geosynthetic in form of horizontal strips, vertical encasement and vertical-horizontal combination by model tests and numerical analysis. By varying the parameters like encasement length and rigidity, geotechnical parameters of the column materials, columns types, and the columns diameter evaluation of the monotonic and cyclic behavior of stone columns encased with geotextile has been carried out [221].

The results have shown that by increasing the stiffness of encasement an increase in columns cyclic behavior is obtained. Hataf et al. [222] have tested the effect of encasement length and aggregate type on bearing capacity of singular stone column in both dry sand and clay bed.

The half encased stone columns have been found less effective in their performance than fully encased ones. Also the smaller aggregates have shown better improvement than the coarser materials. The experimental studies have been corroborated with numerical modeling.

2.8 ENCASED (BOTH VERTICALLY AND HORIZONTALLY) SC NUMERICAL STUDIES

Hasan et al. [200] have also carried out numerical analytical calculations from PLAXIS 3D on granular piles reinforced with geosynthetics under a short term loading using the concept of unit cell. Incorporation of geosynthetics in the encased granular piles has helped in controlling the lateral expansion of the piles (Figure 2.34).

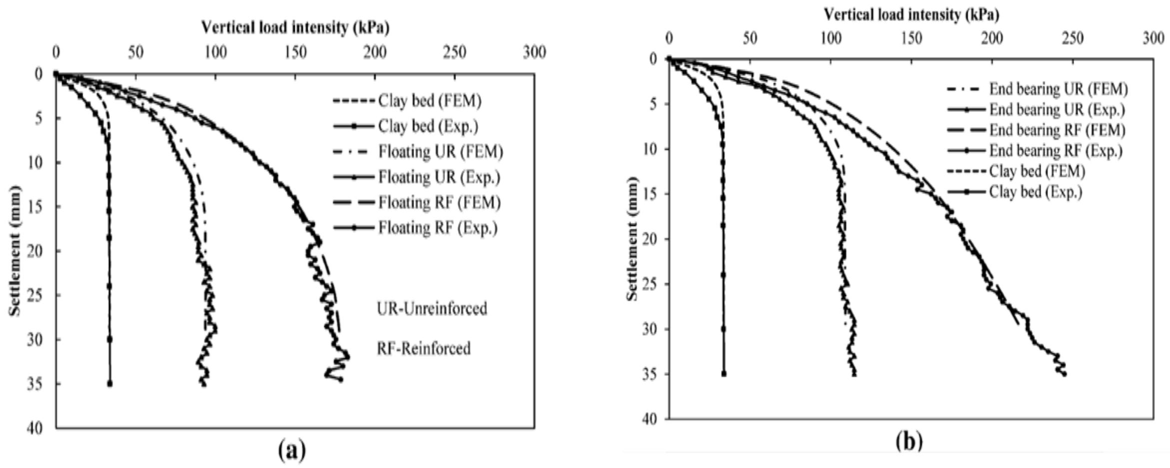


Figure 2.34 Granular piles' vertical load intensity settlement reactions; (a) Floating, (b) End bearing [200]

The behaviours of rigid stone columns (RSC) and ordinary stone columns (OSC) in sandy earthen slopes has also been conducted through 3D finite difference method by Hajiazizi et al. [209]. The optimal location for the placement stone columns has been found to be in the middle of the slope due to maximum displacements in that part.

Both the lab experiments as well as mathematical results have indicated that in the event of RSC construction between the center of the slope, there was an increase in sandy slope stability of up to 1.360 times while comparison to a slope with OSC. Also shear strength of the soil has been increased with RSC up to 1.41 times the OSC. In case of filling of cement grouts inside the stone pillar, Failure mechanism in case of RSC has been found to occur as bends while shear failure has been found to exist in an OSC.

Das et al. [214, 215] performed numerical studies to predict improvements in load carrying ability and bearing capacity in person and groupings of stone columns without and with the use of a rough soil-cement bed over the column. The Load settlement curve is shown in Figure 2.35 below.

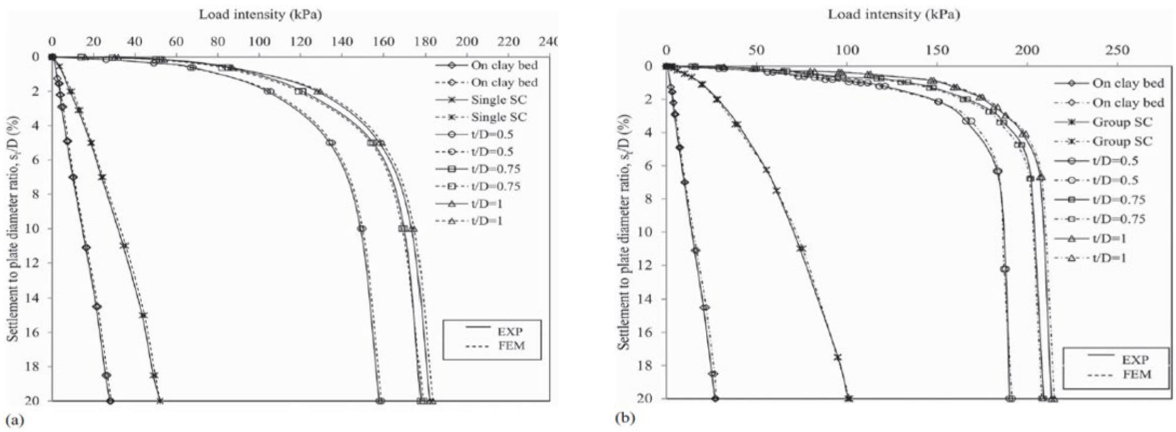


Figure 2.35 Load settlement graph of **(a)** Individual Column; and **(b)** set stone columns for various t/D ratio [215]

Murugesean et al. [223] have conducted complete parametric study using finite element analysis and established higher load bearing ability and lesser compressions and lateral bulging of encased stone columns in comparison to ordinary stone columns. Extra confinement because of encasement due to geosynthetics reduces the degree of embankment load transfer to ground thereby decreasing the settlement of the foundation [224-226]. Castro et al. [227, 228] has studied investigated of sets of encased stone columns beneath a stiff footing using systematic 2-D and 3-D finite element evaluation. Analyses have shown that column positioning (both columns number and position of column) merely affects the settlement reduction obtained with treatment under the conditions of constant values of area replacement ratio (i.e. columns area over footing area) encasement rigidity to diameter of column. For high encasement firmnesses, the placement of column near the footing edges has found to reduce the settlement, making them more beneficial, however, there has been higher maximum hoop force.

Using finite element method mounted on vertically encased floating stone columns community inserted in soft clay, Debnath et. al. [229] performed three-dimensional computational simulations on an unreinforced (USB) and a geogrid-reinforced sand bed (GRSB). Elasto-plastic materials were used to model the encasement materials. Results have indicted as shown in Figure 2.36 that increased bearing capacity with reinforcement as contrast to unreinforced clay bed. There has been a rise in stress concentration ratio and improvement factor of columns with GRSB with an increase in the settlement.

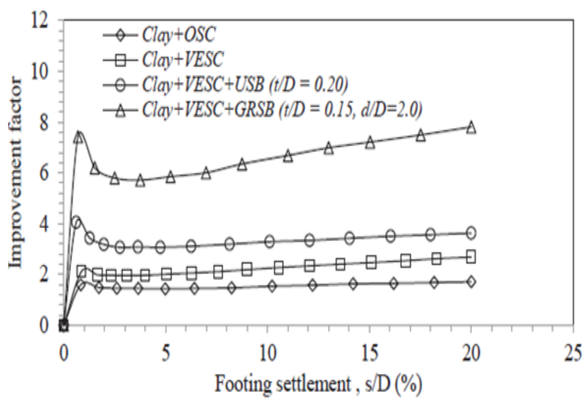


Figure 2.36 (a) Change of improvement factor with different combinations of reinforcement [229]

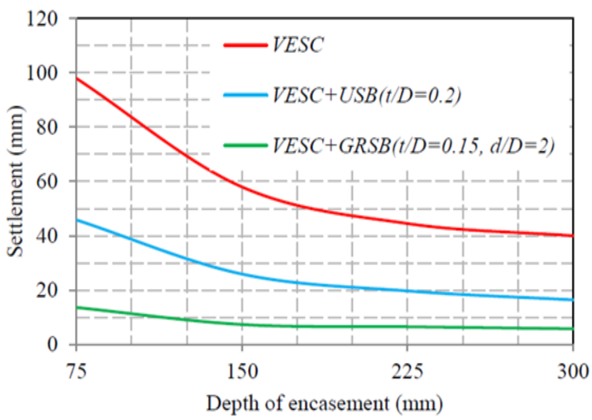


Figure 2.36 (b) Alteration of the settlement versus the depth of encasement [229]

Various 2D, 3D, plain strain and axisymmetric techniques have been used to generalize performance of reinforced stone columns neglecting membrane result of encasement material. Encasement of stone columns with geosynthetics imparts extra lateral confinement making the columns stronger and stiffer. Moreover the encasement also prevents vertical drainage of stone column and hence reduces the settlement. [219, 220,222, 230-233]. Fahmi et al. [234] have employed finite element model using the software PLAXIS 3D and field load tests to examine the character of encases columns made up of stones under cyclic loads. Gu et al. [235, 236] have used three dimensional discrete element methods (DEM) investigation of solo geogrid encased stone columns under unconfined compression.

Hosseinpour et al. [237] have differentiated findings of 3D analysis on embankment on geosynthetic encased stone columns with 2D axisymmtric and plain strain scrutiny and established the better utility of 3D method in evaluating the effects of upgraded soil, where as Shabani et al. [238] have depicted impact of geo synthetic reinforcement in the soil with low un drained shear strength by performing numerical studies. In recent times, the strength of soft marine clay has been increased by the usage of fly ash and calcium carbide residue based polymer, affording a green method for the stabilization of the soil [239]. The geosynthetic encasement has also been tested under oil storage tanks where they have shown reduction in long-term settlement and lateral deformation by providing more lateral confinement and transference of load from the storage tanks on to the encased stone columns [240]. In one ofthe most recent studies, Sun et al. [241] have applied mechanistic –empirical analysis to geosynthetic-stabilized pavements using the layered elastic theory and the solution derived from the method may help in analyzing the design of three-layer flexible pavements.

The behaviour of foundations in liquefiable soils upgraded by deep soil mixing columns towards earthy vibrations has been studied by Hasheminezhad et al. [242]. For this purpose, they have conducted numerical analysis through 3D finite difference model using FLAC software. The bearing capacity and settlement of the foundation has been evaluated in terms of diameter and length of the columns as well as spacing between the columns in case of group of columns. Low settlements and considerable seismic bearing capacity has been found to prevail in the foundations in liquefiable soils in the presence of deep soil mixing columns.

The three dimensional elemental simulation of a geotextile encased stone column has been mathematically modeled by Tan et al. [243]. The encasement has been modeled using geogrid elements in FLAC 3D to forecast the stress-strain nature of encased stone column under uniaxial compression. The numerical analysis can help in extrapolation of mechanical relation between the gravel and encasement. Both numerical and experimental testing has indicated that the parameters of geotextile play critical function in controlling the bearing capacity of the stone columns.

2.9 ENCASED (BOTH VERTICALLY AND HORIZONTALLY) SC THEORETICAL STUDIES

Murugesan et. al. [182, 184] have also conducted theoretical predictions along with laboratory tests to get knowledge about presentation of encased stone columns and has observed elimination of penetration of the soft clay into the stone aggregates which results into higher return of columns of stone given in Figure 2.37. They have computed the hoop strain, ε_c in the geosynthetic from axial strain ε_a assuming deformation of the stone aggregate without volume changes using the relation (2.8)

$$\varepsilon_C = \frac{1-\sqrt{1-\varepsilon_a}}{\sqrt{1-\varepsilon_a}} \tag{2.8}$$

The hoop strains decreases with increase in the column diameter and depth in all the cases, probably due to reduction in column strain at greater depths. The execution of partially encased columns of stones was found to nearby to that of fully encased columns of stones.

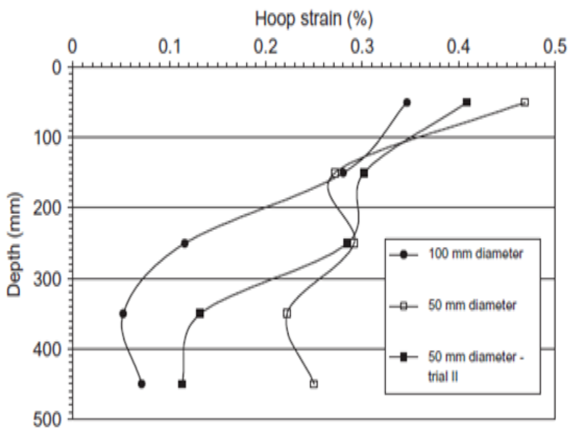


Figure 2.37 (a) Variation in geosynthetic encasement hoop strain (nonwoven geotextile)[182]

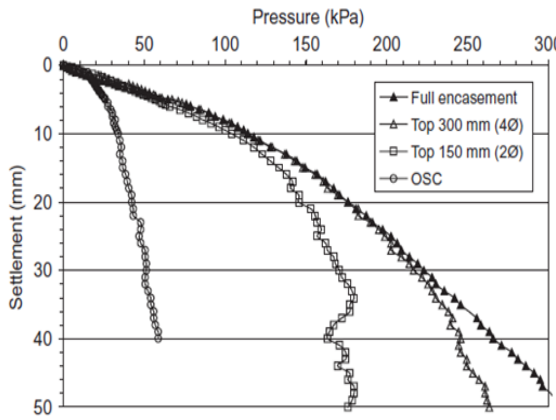


Figure 2.37 (b) Quality of partially encased stone columns (75.0 mm diameter encased for nonwoven geotextile)[182]

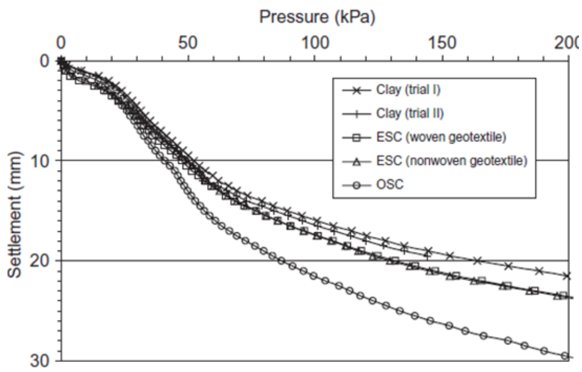


Figure 2.37 (c) Response of experiments with full area-unit cell loading resulting in pressure settlement cell [182]

Bearing stress values obtained for the encased columns using laboratory tests by Hong et. al. [202] has shown good correlation with the values estimated by an analytic expression utilizing cavity expansion theory shown in Figure 2.38.

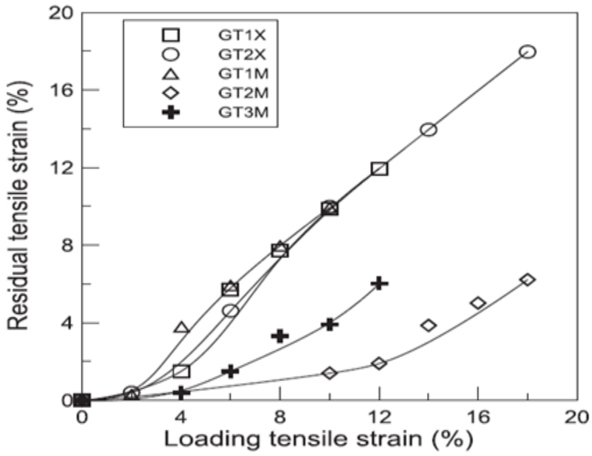


Figure 2.38 (a) Relationship between loading and residual tensile strains for all measurements Geotextiles [202]

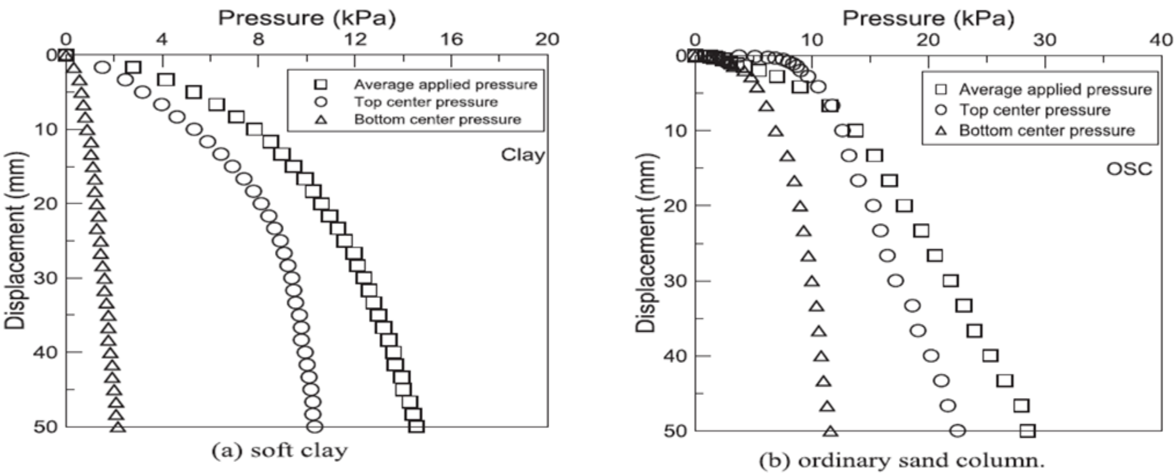


Figure 2.38 (b) Relationship between vertical pressure-displacement for soft clay and the ordinary sand column: (a) soft clay; (b) ordinary sand column [202]

Castro et al. [227] have also corroborated numerical simulations with theoretical studies The study has involved the conversion of all group column in just one central column with an equivalent area and encasement rigidity under footing, thus simplifying the model. The studies have established that there has been a critical column length of around 2-3 times of footing width for settlement reduction in fully encased columns reinforcing the homogeneous soil (Figure 2.39).

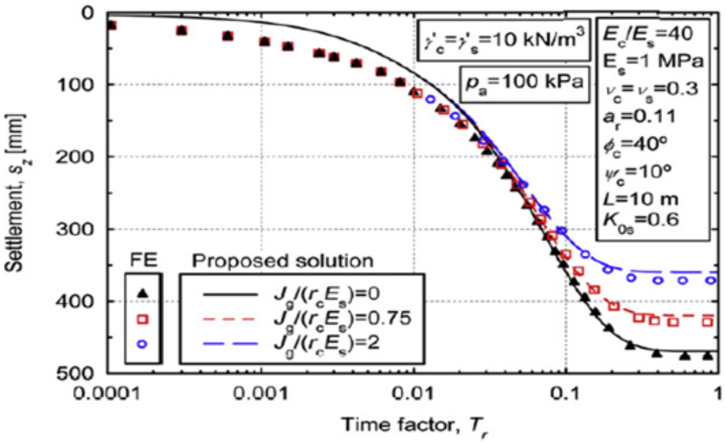
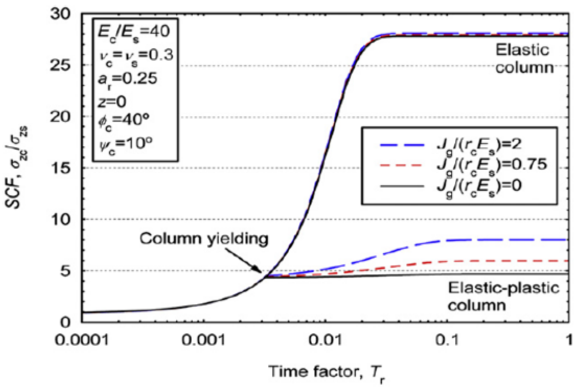


Figure 2.39 (a) for Common encasement stiffness's with time-settlement curves [227]



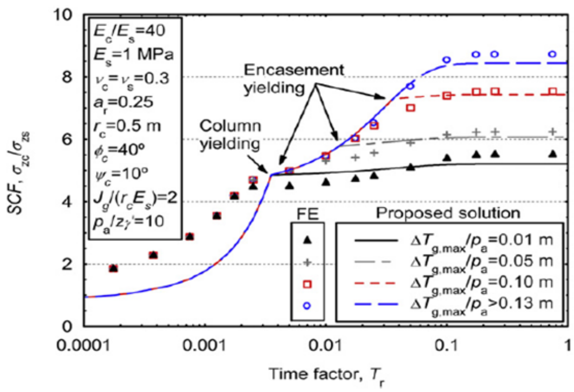


Figure 2.39 (b) Time Influence of the encasement stiffness with stress concentration factor [227]

Figure 2.39 (c) Time Plastic columns Influence of the tensile strength of the encasement with Stress concentration factor [227].

Wu et al. [244, 245] have conducted triaxial stress tests and proposed an analytical model depending on cavity expansion theory to formulate axial stress-strain relation for encapsulated stone columns given in Figure 2.40. An equation has been confirmed between axial strain and volumetric chang of soil so that behavior of granular material can be analyzed under a continuous increment in lateral pressure. The encapsulating reinforcement generated enhanced deviatoric stress, reduced volumetric and radial strains, and boosted confining pressure, which were all gathered and analyzed.

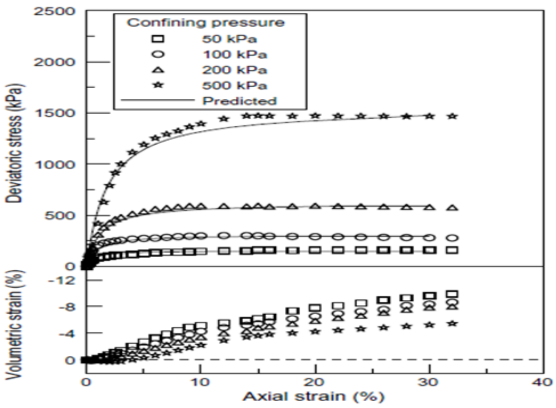


Figure 2.40 (a) un-reinforced sand specimens tri axial test results [244]

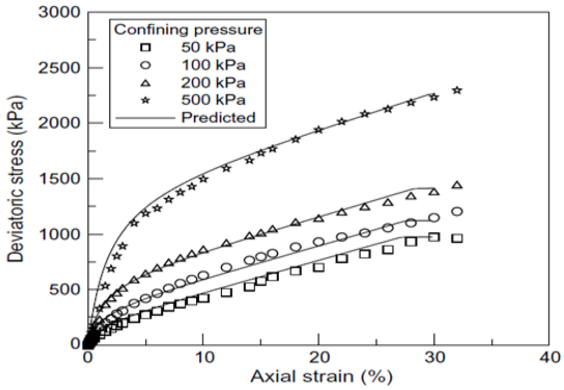


Figure 2.40 (b) Reinforced sand specimens experimental and analytical results [244]

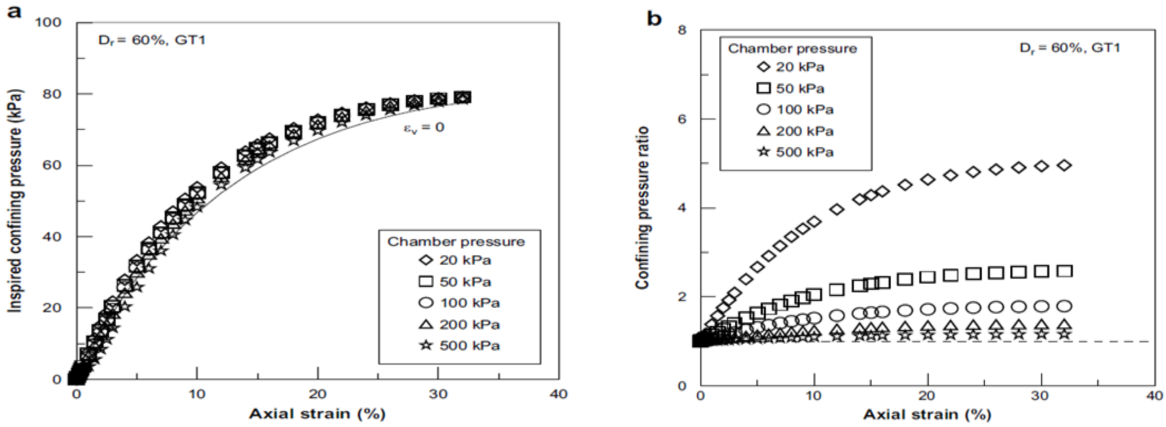


Figure 2.40(c) Inspired confining pressure for reinforced samples with variable chamber pressures (D_r= 60.0 %, GT1 geotextile). (a) Inspired confining pressure, (b) confining pressure ratio [245]

Deb et al. [246, 247] have studied the response of loose soil that has already been internalized by a Kelvin–Voigt model and the stone columns by Winkler springs while the multilayer reinforced stone columns by treating the granular filling and geosynthetic reinforcement as Pasternak shear layer. For prediction of uniaxial compressive strength of fibre reinforced polymer confined concrete, support vector machine regression methods [248-251] have also been employed. The studies have established that the encasement of stone columns with geosynthetics imparts extra lateral confinement making the columns stronger and stiffer. Moreover the encasement also prevents the vertical drainage of stone column and reduces the settlement [252].

Dey et al. [253] have used empirical approach using support vector regression, SVR to estimate bearing capacity of sand bed resulting over vertically encased floating stone columns in soft clay (Figure 2.41). Total two hundred forty five experimental observations has been obtained and three SVR model based on different kernel functions have been developed, which have been further used to derive an empirical design chart for the estimation of bearing capacity.

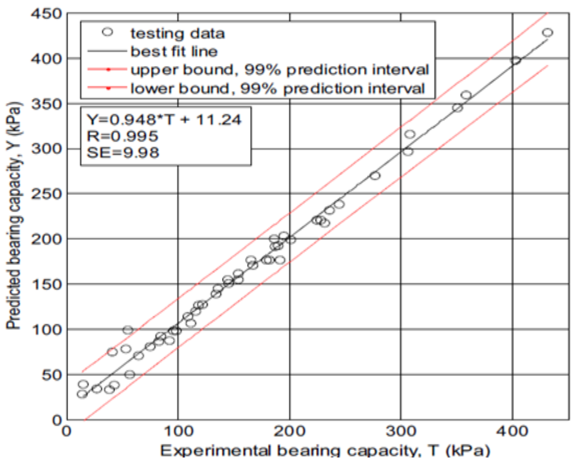


Figure 2.41 (a) SVR-ERBF model graph of experimental bearing capacity vs. predicted bearing capacity for research collection [253]

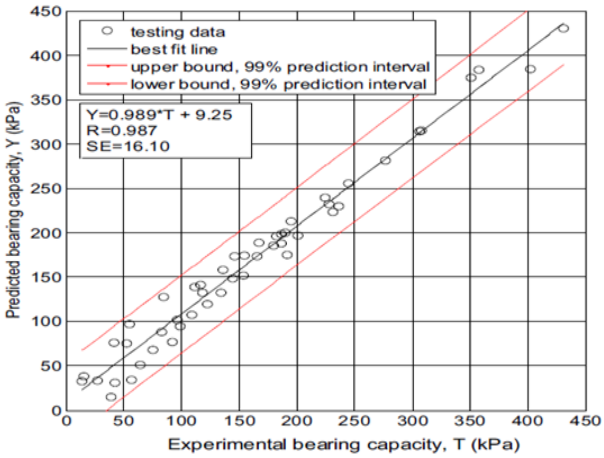


Figure 2.41 (b) Graph of experimental bearing capacity against predicted bearing capacity for experimental set: SVR-RBF model [253]

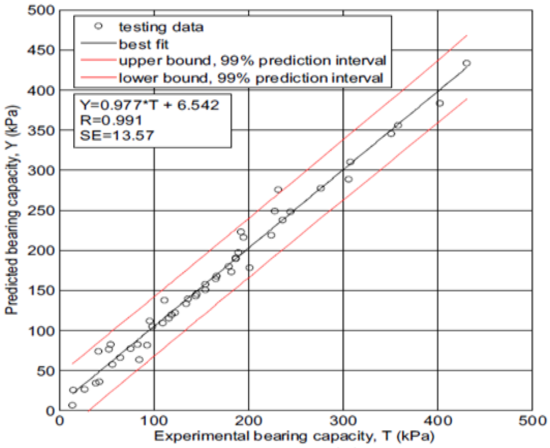


Figure 2.41 (c) Graph of experimental bearing capacity against predicted bearing capacity for experimental set: ANFIS model [253]

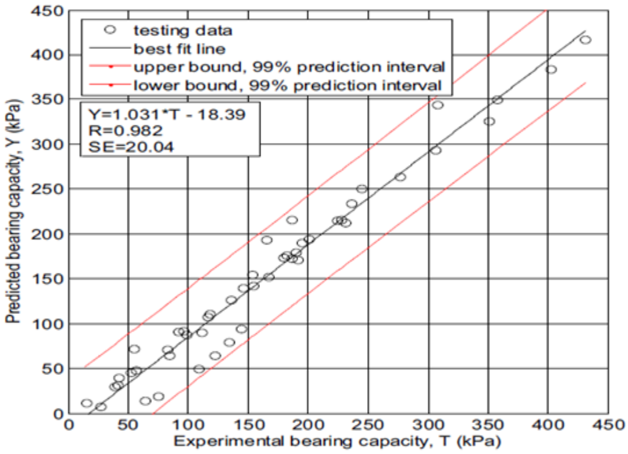


Figure 2.41 (d) Graph of experimental bearing capacity against predicted bearing capacity for observation set: SVR-POLY model [253]

2.10 SUMMARY OF LITERATURE REVIEW

The shortage of suitable building sites has led to new methods for upgrading ground surface, according to the type of the soil and specifically suitable for a particular construction design. The utilization of stone columns of particular arrangement has emerged as one of the most commonly used and low cost techniques preferred for the treatment of soft soils having low bearing ability and large settlements. The technique of stone column has been first employed in 1960s. Since then the method has fascinated the research aptitude of number of civil engineering and therefore, detailed data has been available for the behavior of columns made up of stones modified soil in terms of its settlement, bearing capacity, load transfer, failure pattern. There exists numerous simplified geometrical methods for both ordinary stone columns and the columns encased with geosynthetics whose suitability depends upon parameters to be analyzed, for example the settlement or bearing capacity; and analysis types like numerical or analytical method in 2D or 3D. For the embankment of a linear structure, 3D slice of columns model is recommended because there is no need to transform the problem parameters. However, calibration and turning of parameters using unit cell has been found suitable for gravel trenches or homogenization. The behavior of isolated column differs from the column in a group under distributed load. The isolated column with load just on top of it has proven to be useful in field tests. Further, for the groups of columns, important parameter is the critical column length which is dependent on the loading area. For non-encased columns, critical length is about two times of the footing width and for encased columns it is higher [254]. The studies have also included the variation of these geotechnical parameters with the changing geometric properties of the columns like column's diameter, spacing between the columns, installation method etc. and the hydro-mechanical properties of the soil like excess pore water pressure, stiffness that usually vary with the type of the soil. All these factors affect the lateral deformation of the soil therefore affecting its bearing capacity. The investigations related to the response of soil reinforced with stone columns involve experimental tests that include field and small scale laboratory tests to mimic the large construction sites. These days, analytical analyses and mathematical models also been employed for finding the characteristics of soil under reinforcement. Most of the finite element model analyses and empirical solutions assumes the elastic behavior for soil and material used for fabrication of column, still the response studies of stone columns have established elastoplastic model for the composite structure.

Performance of stone columns is usually depicted in terms of its lateral bulging and consolidation because of confinement. Stress concentration ratio is an important parameter in the interpretation of foundation behavior with stone columns. This property of composite system is dependent on the progressive consolidation which changes with time. Therefore, the researchers have conducted numerous studies to map the load settlement and stress concentration of the soil modified using stone columns. However, there has been wide range of stress concentration values and the uncertainty related to their computation. The literature reports have also suggested that lateral deformation due to the confinement leads to the expansion of the columns and ultimately ends up with the failure of the method. This problem has been overcome by reinforcement of stone columns with geosynthetic materials. A range of geosynthetic materials have been tested as stone column fillers under varying conditions and installation patterns of the reinforcement viz. vertical encasing, horizontal striping and the combination of both using experimental techniques and numerical models. The use of reinforcement has resulted in reduced settlement as well as enhanced bearing capacity of composite foundation. There has been acceleration in extraction of excess pore water pressure, probably because of consequential decrease in vertical stress on surrounding soil and increase in hoop tensile force. Still there is lack of understanding of degree of relationship among the stiffness of casing, bearing capacity of geosynthetic encased stone columns, the depth of encasement, settlement of the soil, load transfer, consolidation of the soil in case of geo synthetically having soil installed reinforced columns made up of stones. The knowledge of all these parameters is essentially required to control the performance of the encasement in the stabilization of the poor ground surface and hence there is significant requirement to conduct numerical, field and experimental investigations to explore these aspects.

2.11 KEY GAPS IN EXISTING LITERATURE

Few field studies evaluating the load-settlement properties of geosynthetically reinforced columns have been performed previously. Despite the fact that they fixed the scale effect in addition to laboratory testing, they were concerned with problems in controlling the way ground water was raised and added to the composite cell of the soil and the column. As a result, judging based on these tests may be deceptive and misrepresent the real circumstance. These experiments are often fairly expensive and time-consuming.

- Stone columns have been used to improve the action of soft cohesive soils. However, in real field conditions, the presence of thin cohesion-less soil up to a few metres deep over a stiffer stratum is not unprecedented, necessitating further study.
- Evaluation of reinforced/unreinforced stone column efficiency (load settlement characteristics) in cohesionless soil is needed for the development of a design method without a thorough subsurface investigation.
- Effect of vertical encasement for floating stone columns in cohesive soils depicted increase in bearing capacity due to rapid dissipation of pore water pressure. However, the effect and load settlement mechanism of vertical encasement in cohesionless soils in rendering enhanced stone column behavior needs detailed investigations.
- Likewise effect and load settlement mechanism of horizontally reinforcement of stone columns in cohesionless soils also requires detailed investigation.

- Efficacy of modeling unreinforced and reinforced stone columns in 2-D and 3-D Finite Element condition needs to be examined.
- Limited studies on group behavior of reinforced stone columns under different arrangements leaves a gap for further and in-depth comprehension of stone column behavior in cohessionless soils.

2.12 OBJECTIVES OF THE STUDY

Thorough analysis of studies related to soil stabilization reveal that number of research groups has focused in practical aspects of stone column behavior in modifying the properties of available soil and making it ready for construction of geotechnical structures. The technique has been practiced for decades in different kinds of soils and emerged as promising method for the purpose. However the studies have also emphasized on the lack of installation of very soft soils including sandy soils probably owing to its failure due to lateral expansion in such types of soils. This tendency of lateral disintegrate of stone columns can be measured by the installation of geosynthetic materials in columns. By improving the permeability and hardness of the treated soil, these materials increase the compactness of the composite framework and provide added rigidity to the structure. This stops the ordinary stone columns from bulging out. Thus, the use of geosynthetic constituents (both vertical encasement and horizontal layering) improves the efficiency of stone columns and expands their application spectrum.

An extensive review of the literature shows the need for a thorough analysis highlighting key problems that could affect the overall efficiency of the stone column, as well as effectively monitoring the invisible subsurface during loading using appropriate model tests and computational tools. Keeping all these points in mind, this research proposal mainly focuses on

(1) To study how stone column strengthened sand behaved in terms of load carrying ability and soil settlement.
(2) To study the effect of geotextile placed horizontally in the form of discs on the achievement of stone columns in the construction purpose.

(3) To analyze the impact of vertical encasement of stone columns in modifying the technical properties of sand.
(4) To make a comparison of response of unreinforced stone columns with that of geotextile reinforced stone columns in terms of load transfer and settlement reduction.
(5) To assess the difference in the performance mechanism of individual and set of stone columns in all the cases.

For achieving this purpose, the finite element method (FEM) results using Plaxis 3D software has been validated with model testing. The results of present study would help in understanding the mechanism of enhancement in the execution of reinforced stone columns in sand.

2.13 SCOPE OF THE STUDY

Based on the engineering and research experience described above, ordinary and encased stone columns with geogrid materials are used for strengthening soft soil in both undrained and drained environments. This research examines the stone columns-soft soil base structure to see how the behaviour of the reinforced soft earth has changed. The effect of geosynthetic encasement on the stone column's stiffness, bearing capability, and bulging is also studied.

CHAPTER-3
METHODOLOGY

3.1 GENERAL

The present chapter focuses on the analysis of behaviour of stone columns reinforced sandy soil by performing laboratory tests on the models of conventional and geosynthetic covered stone columns on the sandy surface. Investigation on single and group of three and four columns have been explored and the influence of various factors like stiffness of geosynthetic material, geotechnical parameters of stone columns such as length, diameter, distance between the columns and load area on the performance of stone columns has been investigated. Two different experimental series were performed. The first series comprised of single stone columns with varying reinforcement arrangement and the second series included the experimental investigations on group of three and four columns under different conditions. Unreinforced as well as reinforced (horizontal as well as vertical) stone columns were tested in both the cases. For checking the reproducibility and accuracy of the results, around 20% of the tests in each series were repeated. The numerical modeling using software 3D PLAXIS results has been validated by experimental testing results.

The detailed description about the ground surface preparation, construction of stone columns and experiments conducted has been presented in this chapter.

3.2 MATERIALS USED

In this work, three different materials were used; soil sample, aggregates for stone column and polymeric geotextile for encasement. Properties of these materials have been presented below:

3.2.1 SOIL SAMPLE USED

A thorough investigation of literature revealed that most of the tests include soft cohesive soil being improved by stone column. However, in common practice, even on a rigid foundation, there exists weak non-cohesive soil up to a few meters in depth. Therefore, it is requisite to check the performance of stone column in cohesionless soil i.e. sand for prediction of load carrying capacity and settlement to find out the designed approach to be employed with thorough investigation of soil subsurface. According to FHWA [255], in transition zone soil particle are in the size ranges between 0.02-0.6 mm, having better response to vibro compaction than the vibro floatation technique. In the present days, using compaction method stone columns have been installed, thus sandy soil Shown in Figure 3.1 was investigated in this research that was obtained from Jaypee University of Information Technology. The particle size distribution curve of soil is shown in Figure 3.2.

Figure 3.1 Sample of soil used in investigation

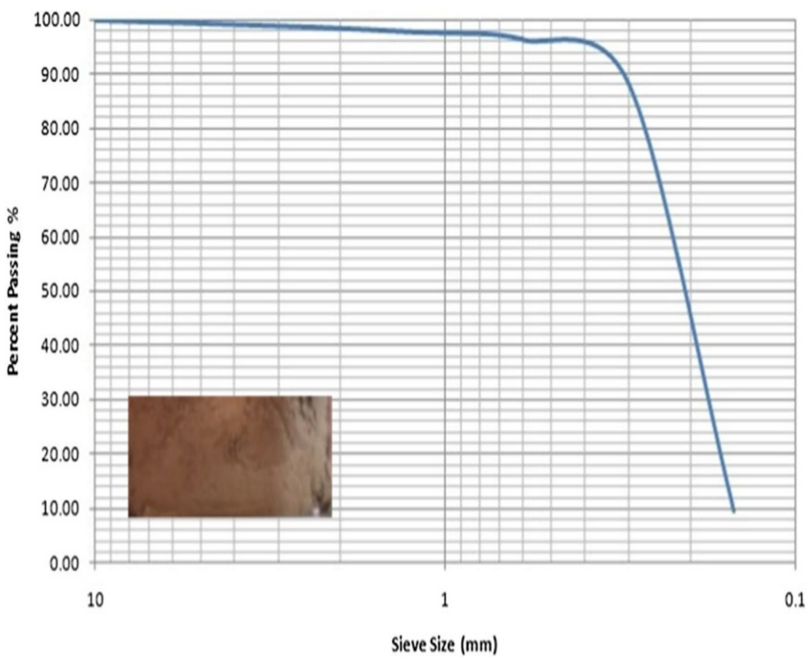


Figure 3.2 Particle-size distributions for soil

Table 3.1 summarises the different characteristics of sandy soil.

Table 3.1 Properties of soil sample

Properties of the sample	**Value**	**Remarks**
Soil	Sand	-
Classification of Soil	SP	-
Angle of Friction (ϕ)	20°	IS 2720 – 13 (BIS 1986 (a))
Cohesion (c)	1.96 kN/m^2	
Specific Gravity	2.65	Pycnometer Method Clause 8.3 BS 1377 : Part 2 (1990)
Water Content	3.44%	-
Optimum Moisture Content	8.40%	BS 1377 : Part 4 (1990)
Dry Density	0.0016 kg/cm^3	
Bulk Unit Weight ($\gamma_{unsat.}$)	19.66 kN/m^3	
Saturated Unit Weight ($\gamma_{sat.}$)	21.75 kN/m^3	
Poisson's ratio	0.30	IS 9221-1979
Modulus of elasticity	20,000 kN/m^2	

3.2.2 MATERIAL FOR STONE COLUMNS

The size ranges of crushed stones to construct stone column is very important factor that should be decided carefully. According to Ali et al. [10], the stone aggregates must be in the size range of 6 mm - 40 mm so that the ratio d/D used for the prototypes can be satisfied. In practice, stone columns of diameter (d) in the range (0.6-1) m are commonly constructed by using crushed stones with size (D) in between 25-50 mm. Thus for all practical applications, the ratio d/D is taken in the range of 12-40 [100, 10]. In the present work, aggregates of size between 2- 10mm were chosen (Figure 3.3) so d/D ratio lies in the range 4-20 for the modelled stone columns.

Figure 3.3 Stone Columns are made out of crushed stones.

As a result, Panipat was used to extract 10 mm aggregates (passing). On the 4.75 mm sieve, 63% of aggregates were preserved, while 25% of aggregates were recollected on the 10 mm sieve, which has a particle size distribution curve seen in Figure 3.4.The aggregate sizes (D_{10}), (D_{30}) and (D_{60}) were 0.20, 0.47 and 0.65, respectively. According to IS Classification as C_u and C_c values were 3.25 and 1.69 respectively, hence stone column falls in the category of well graded gravel (GW). The dry unit weight γ_d = 22.78 kN/m^3, $(\gamma_d)_{min.}$ = 19 kN/m^3 and $(\gamma_d)_{max.}$= 25.54 kN/m^3 was also found by the procedure mentioned in literature [169, 256-259]. Internal friction for aggregates was measured using a direct shear test at 1.25 mm per minute under normal stresses of 300 kPa, 200 kPa, 150 kPa, and 100 kPa, having internal friction angle of 42^o. Table 3.2 lists the geotechnical parameters of stone aggregates.

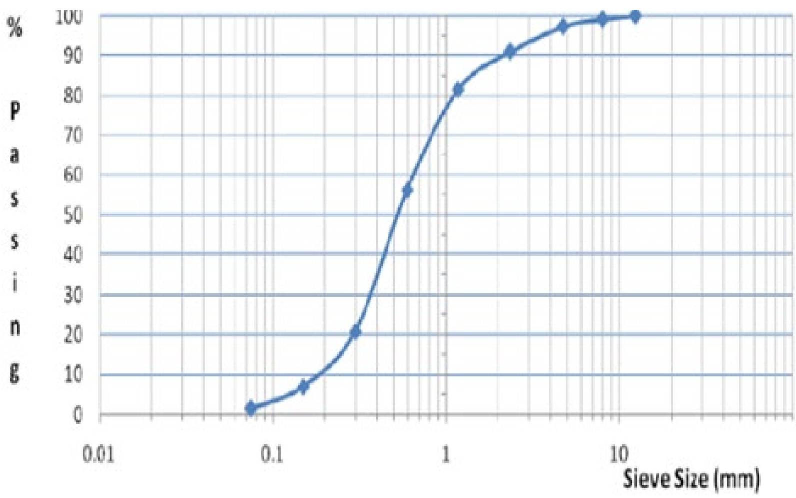


Figure 3.4 Distribution curve for the particle size of the aggregates

Table 3.2 Characteristics of soil and stone column material used.

Parameter	Aggregates	Remarks
Classification	GW	-
Friction angle (ϕ)	42°	IS 2720 – 13 (BIS 1986 (a))
Cohesion (c)	0.10 kN/m^2	
Bulk Unit Weight ($\gamma_{unsat.}$)	22.78kN/m^3	BS 1377 : Part 4 (1990)
Saturated Unit Weight ($\gamma_{sat.}$)	23.25 kN/m^3	
Poisson's ratio	0.30	IS 9221-1979
Modulus of Elasticity	55,000kN/m^2	

3.2.3 REINFORCEMENT MATERIAL: GEOTEXTILE

Polymeric reinforcement materials are part of civil engineering called geosynthetics. Geosynthetic, according to ASTM, is a polymeric planar substance used for earth, dirt, and geotechnical-related materials as a vital part of the civil engineering industry or structure.

Geotextiles were used to reinforce conventional stone columns. In this study, the geotextile used to reinforce the stone column was Woven Polypropylene geotextile. The properties of geotextile have been summarized in Table 3.3.

Table 3.3 Properties of Woven polypropylene Geotextile (as provided by manufacturer)

Property	Particulars	Units	Test Method	Value
Tensile Strength	WARP	kN per m	IS-1969	45.0
	WEFT	kN per m	IS-1969	34.0
Elongation at Break	WARP	Percent	IS-1969	30.0
	WEFT	Percent	IS-1969	28.0

3.3 EQUIPMENTS & APPARATUS

3.3.1 TANKS

Two model tanks with dimensions of 300 mm * 300 mm * 550 mm were made to model the stone columns. There are three sides of iron and one side of acrylic sheet. The model tank and its proportions are depicted in Figure 3.5. The tank sides were thick enough to avoid lateral deformation that could occur during filling. Before the test bed was mounted, the interior surface of the tank was coated with epoxy and blended. The interior surfaces of the tank were covered with epoxy, which reduced pressure on the tank's side walls.

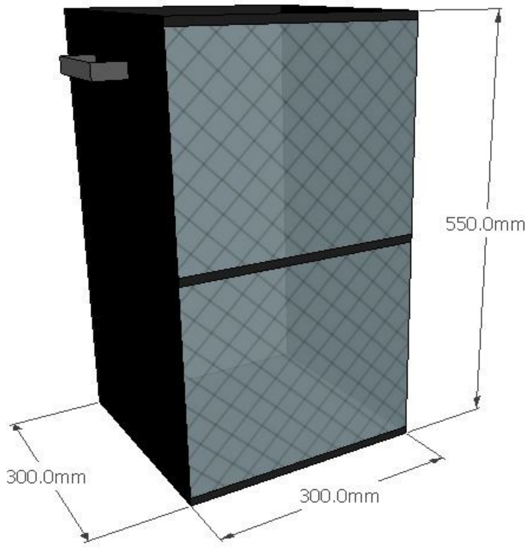


Figure 3.5 (a) Model tank (front view)

Figure 3.5 (b) Model tank (side view)

3.3.2 LOADING SYSTEM

The Universal Testing Machine was used to apply the load (UTM). The UTM is a computer that measures material tensile and compressive power. A load frame, a load cell, a cross head, an output device, and some test fixtures make up the system. When the specimen is mounted between the cross heads and the unit is turned on, the UTM applies a uniformly growing load to the specimen. On the output device, the load and the resulting settlement are shown. As seen in Figure 3.6, the load was applied at a rate of 1kN/min until the appropriate settlement was achieved.

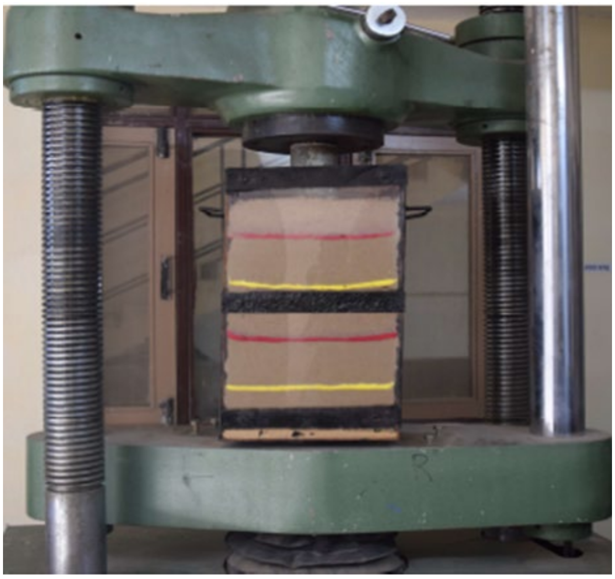


Figure 3.6 Testing of stone columns.

3.3.3 IRON CYLINDER FORTESTING

An iron cylinder of diameter 80 mm and height 20 mm is used for testing as shown in Figure 3.7. The testing cylinder's diameter was chosen such that the dimension of the tank is 3-5 times the diameter of the loaded area. This is done to ensure that the tank walls do not exert any forces on the column.

Figure 3.7 Iron cylinder used for testing

3.3.4 IRON SHEET FOR TESTING

Iron sheet of 200 mm * 200 mm was used for loading the surface of the tank. The thickness of the plate is 10 mm. The picture of steel sheet used is shown in Figure 3.8 below.

Figure 3.8 Iron sheet used for testing

3.3.5 COMPACTION TOOLS

The soil was compressed in 100 mm sections with a 2.6 kg hammer. A steel rod weighing 1.316 kg and measuring 15.5 mm in diameter and 100 mm in length was used to compress the stone fill.

3.4 MODELING CONSIDERATIONS

3.4.1 SCALING OF MODEL TANKS FOR SINGLE STONE COLUMN

Two model tanks with dimensions of 300 mm * 300 mm * 550 mm were made to model the stone columns. There were three sides of iron and one side of acrylic sheet. The model tank and its proportions are depicted in Figure 3.9. According to Ali et al [10], the tank dimensions were selected. Boundary effects and geometric similitude ratio, l/d and are considered. Then modeling of test tanks dimensions and stone columns parameters were determined. The diameter of prototype stone columns is usually kept between 0.6-1.0 m, with a length of about 5-20 m [100]. Moreover, minimum diameter of stone column should have a value of 13 mm so as to install with complete integrity. However, the stone columns used in this analysis were 40 mm in diameter, resulting in a similitude ratio (dmodel/dprototype) of (0.04 – 0.06).

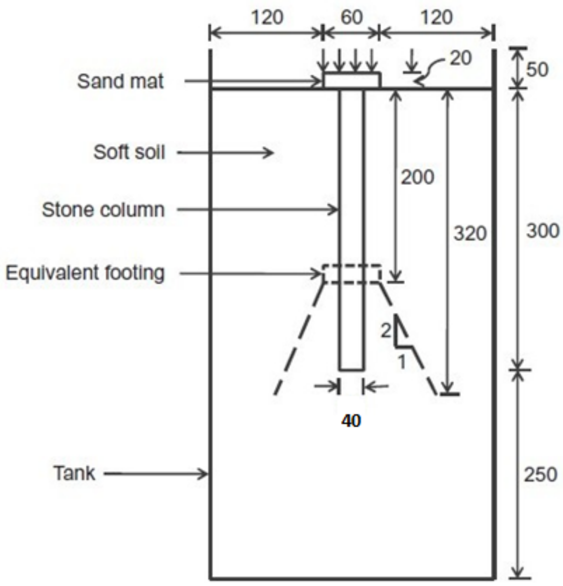


Figure 3.9 Schematic views of modeled Single stone columns

3.4.2 SCALING OF MODEL TANKS FOR GROUP STONE COLUMN (3 & 4 STONE COLUMNS)

Boundary effects and geometric similitude ratio l/d are considered. Then modeling test tanks and stone columns parameters were determined. Moreover, minimum diameter of stone column should have a value of 13 mm so as to install with complete integrity. However, the stone columns used in this analysis were 40 mm in diameter, resulting in a similitude ratio (dmodel/dprototype) of (0.04 – 0.06). Similarly, for prototypes, the geometric similitude ratio should be between 5 and 20 [93], and the l/d ratio in this analysis was held at 8.The most significant criterion, induced stresses, becomes negligible with in the tank boundaries while modeling the tank dimensions. This shows that the boundaries should be far in order to produce constrain so that overestimation of results are easy to check. As a result, a conceptual footing with a diameter of 120 mm resting on a 300 mm long stone column was considered, and a comparable footing at 2/3 column length was selected to follow the full model column size (i.e. at depth of 200 mm from the surface of ground).

2:1 dispersion method is used to calculate the effect of vertical stress from a footing settling on columns made up of stones at the earth's surface at a depth of 2.B (i.e. 240.0mm underneath the location of equal footing), and we found that only 16% of stresses formed within the tank edges. Generated stresses become negligible for tank boundaries with a width and depth of 300 mm or more, as seen in Figure 3.10. As a result, evaluation of a model tank with three iron sides and one plastic sheet side, with dimensions of 300 mm in length, 300 mm in width, and 550 mm in height (depth). Every sheet of sand with a thickness of 10 cm was packed using the rainfall process.

With a bulk density of 19.7 kN/m^3, which was weighed continuously while filling was done, the unit weight of each soil layer was kept stable, so a known volume mould was used at three different positions inside the soil layer. Five layers of sand were used to achieve the final height of 50 cm. The inner walls of the model tank were greased to reduce friction between the model tank as well as the sand. To achieve a true thickness and surface in both samples, the top surface of the sand bed was flattened and cutting was performed. In both of the samples, the sand bed was prepared in the same way. The relative density can remain constant during the test if strict caution is maintained.

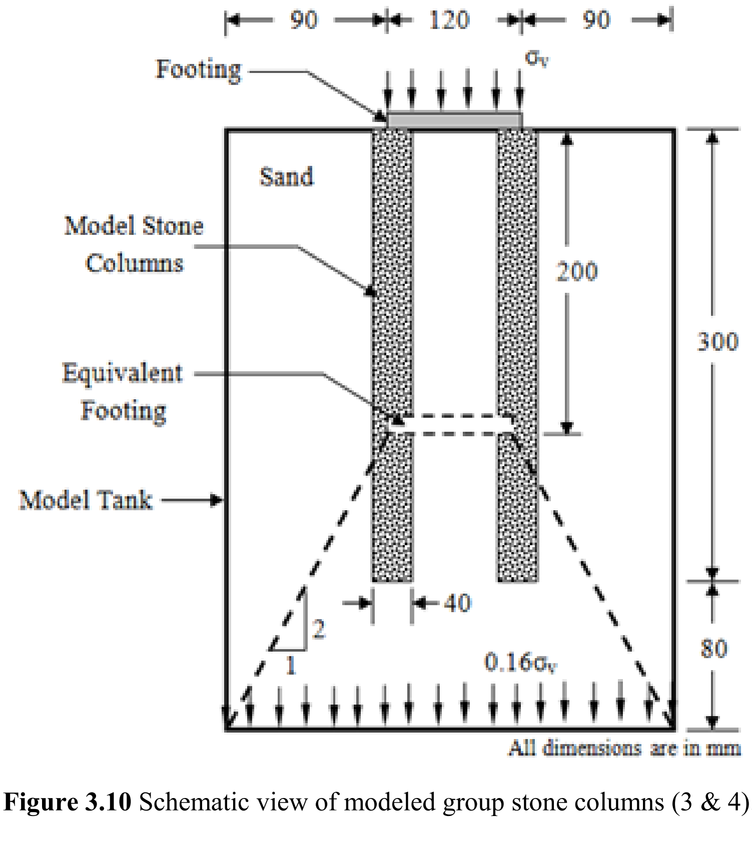


Figure 3.10 Schematic view of modeled group stone columns (3 & 4).

3.5 STONE COLUMNS CONSTRUCTION FOR UNREINFORCED COLUMNS

When deciding the stone column parameters [length (l) and diameter (d), different factors such as boundary effects and geometric similitude ratio, l/d] were taken into account, and thus in our analysis, we used a diameter of 40 mm and l/d ratio of 8. Stone columns were constructed using open-finished, slim-walled pipes with a thickness of 2 mm and an internal diameter of 40 mm. In all samples, the inner and outer surfaces of the pipes were greased with a thin layer of oil to prevent a significant unsettling effect in the underlying soil.

3.5.1 SINGLE STONE COLUMN

The procedure for casting the column is as follows:

1. **Filling the soil**: The outer surface of the tank was marked with ten-centimeter-thick layers. Compaction was done by using a rammer to deliver a series of 15 blows per 10 cm. The tracer was then mounted and the next layer was filled in, as seen in Figure 3.11. The tracer was a red and yellow powder dye that was used to mark every 10 cm layer in order to easily identify deformation patterns from the stone column analysis. The soil was then filled to a height of 50 cm above the tank's bottom, and the stone columns were cast simultaneously.

Figure 3.11 (a) Filling of soil in ten-centimeter layers.

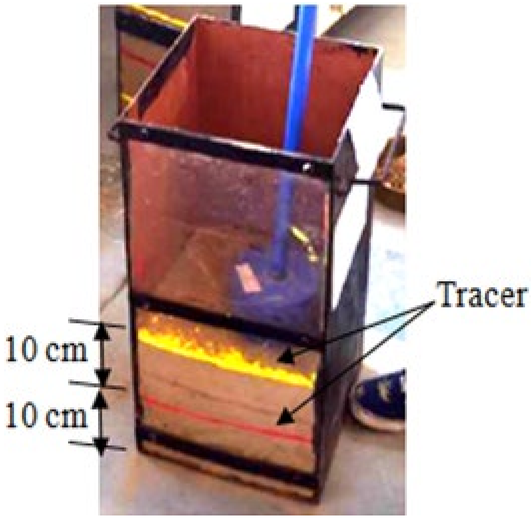


Figure 3.11 (b) Tracers are used to mark 10cm layers.

2. **Hollow pipe casing:** To complete the casting of stone columns, the soil replacement process is used. The method has been used for small-scale stone column construction [74, 112], in comparison to soil displacement and force penetration techniques. Stone columns were cast in a PVC casing with an internal diameter of 40 mm and a thickness of 2 mm, as seen in Figure 3.12, and sunk into the investigated soil using a hydraulic jack. The main reason for using top-down methods was to avoid soil caving during borehole construction.

Figure 3.12 Hollow pipe of diameter 40mm

3. **Unreinforced stone columns casting:** In this analysis, floating columns were modeled. After filling the model tank to a depth of 20 cm with soil, a hollow cylindrical pipe was mounted inside. The soil inside the PVC casing was gathered with a diameter of 38 mm and washed out. To prevent wall scratching and quick casing recovery, the inner walls of the casing were greased before adding the stone aggregates, and the aggregates were compacted using IS light compaction with a 2.6 kg hammer. The height of fall and volume of blows were measured by trial and error until a desired relative density of 65 percent (23 kN/m^3) was obtained, which has the attributes for successful load transfer (strength) over its drainage capacity since the accompanying soil is permeable soil. According to published literature, relative densities of 50 to 80 percent have been used in the case of clayey soil domains [74,112, 197]. Around the same time, the aggregates were pumped into the container, tamped, and the casing was recovered. The difference in relative density during aggregate placement was 65 ± 2%.

3.5.2 GROUP STONE COLUMNS (THREE AND FOUR COLUMNS)

1. **Filling the soil**: The first step is similar to the unreinforced Single column. For group of 3 & 4 stone columns compaction was done by using a rammer to deliver a series of 15 blows per 10 cm. The soil was then filled to a height of 50 cm above the tank's bottom, and the stone columns were cast simultaneously.

2. **Hollow pipe casing:** Before casting of hollow pipe it is very essential to decide the spacing of stone columns. So as per as per Castro et al (2017) the tributary area is transformed into a circle (cylinder) of the same cross – sectional area having an equivalent diameter of 1.05 times spacing between stone columns (s) for triangular arrangement and 1.13 times spacing between stone columns (s) for square arrangement as shown in Figure 3.13 The soil replacement method is used to finish the casting of group of 3 & 4 stone columns. Three hollow pipes are used for casting of stone columns for 3 unreinforced stone columns and four hollow pipes are used for casting of 4 unreinforced stone columns as shown in Figure 3.14.

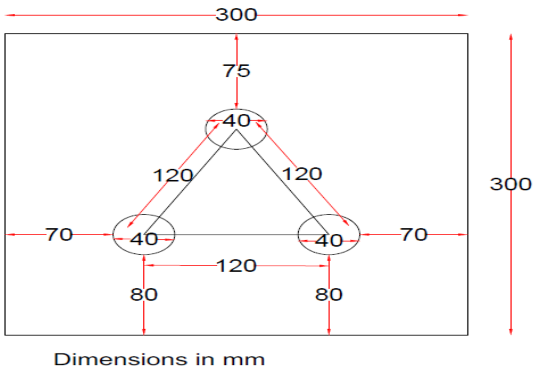


Figure 3.13 (a) Triangular Arrangement of Stone Columns

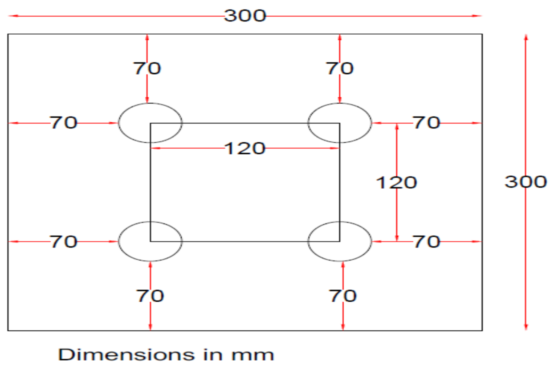


Figure 3.13 (b) Square Arrangement of Stone Columns

Figure 3.14 (a) Un-reinforced group of 3 columns

Figure 3.14 (b) Un-reinforced group of 4 columns

3. Unreinforced stone columns casting: In this analysis, floating columns were modeled. After filling the model tank to a depth of 20 cm with dirt, a hollow cylindrical pipe was mounted inside .The soil inside the PVC casing was gathered with a screw augur with a diameter of 38 mm and washed out. To prevent wall scratching and quick casing recovery, the inner walls of the casing were greased before adding the stone aggregates, and the aggregates were compacted using IS light compaction with a 2.6 kg hammer.

3.6 STONE COLUMNS CONSTRUCTION FOR VERTICALLY ENCASED STONE COLUMNS

3.6.1 SINGLE STONE COLUMN

Stone columns were encased along with geotextile reinforcement and the procedure followed for that has been given below:

1. **Soil filling:** The method of filling the soil is identical to that of an unreinforced stone column. A hollow pipe was placed after compaction and tracer positioning on two layers of 10 cm each.
2. **Column casting:** In the tank, the geotextile encased stone columns were placed. The insides of the pipe are greased to minimize internal friction. When the pipe was being removed side by side, a rod was used to tamp the crushed aggregate stone. The casting process was similar to the unreinforced stone columns method, which allows the

geotextile to serve as a confinement to hold the aggregates. Vertical encasement provided is shown in Figure 3.15.

Figure 3.15 Vertical encasement

3.6.2 GROUP STONE COLUMNS (THREE AND FOUR COLUMNS)

Stone columns were encased along with geotextile reinforcement and the procedure followed for that has been given below:

1. **Soil filling:** The method of filling the soil is identical to that of an unreinforced stone column. A hollow pipe was placed after compaction and tracer positioning on two layers of 10 cm each.
2. **Column casting:** In the tank, the geotextile encased was contained. The insides of the pipe are greased to minimize internal friction. When the pipe was being removed side by side, a rod was used to tamp the crushed aggregate stone. Vertical encasement provided for group of stone columns is shown in Figure 3.16.The casting process was similar to the unreinforced stone columns method, which allows the geotextile to serve as a confinement to hold the aggregates.

Figure 3.16 Vertically encased group of stone Column (3 & 4)

3.7 STONE COLUMNS CONSTRUCTION FOR HORIZONTALLY REINFORCED STONE COLUMNS

3.7.1 SINGLE STONE COLUMN

The stone column casting with horizontal circular discs was carried out in following steps:

1. **Pipe Formation:** To plan for the positioning of spherical horizontal discs, pipes are scaled every 3 cm and labeled with ink, as seen in Figure 3.17.

Figure 3.17 (a) Pipe markings at every 3cm

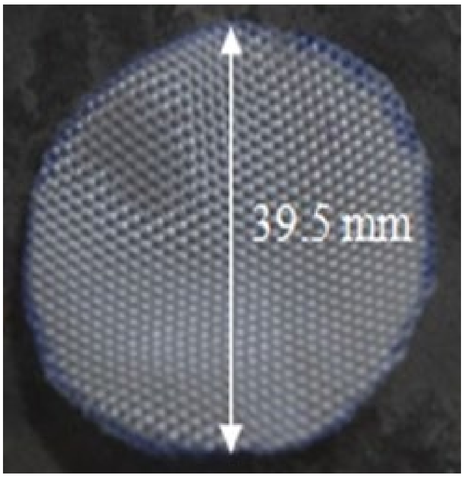


Figure 3.17 (b) Horizontal circular disc of diameter 39.5mm

2. **Column casting:** The geotextile material was cut into circular discs with a diameter of 39.50 mm and placed 3 cm apart. According to literature [10], lateral reinforcement spacing of d/2 (where d is the stone column's diameter) or s/d = 0.5 (where's' reflects reinforcement spacing) results in the largest improvement in failure stress. Additionally, an increase in the x/l ratio from 0.5 to 1.0 was attributed to failure, where "x" refers to horizontal support spacing from the top of the stone column and "l" refers to column thickness. Failure burden rises from 18 percent for x/l = 0.5 to 25 percent for x/l = 1 according to research.

   According to the x/l ratio and s/d ratio variance parameters, the discs were spaced 3 cm apart in the study, resulting in an x/l ratio of 0.1 to 0.9 and a s/d ratio of 0.75. The tubing was submerged 20 cm under the surface. Once the stone aggregates had been filled and tamped, discs were mounted at each marking using a pipe with a smaller diameter than the casting pipe. The disparity in relative density inside the casted stone column was assumed to be 65 ± 2% during aggregate positioning. Since the relative density differential between casted columns and horizontal reinforcement was widely assumed to be slightly higher, precise compaction of a stone sheet thickness of just 30 mm was difficult to achieve.

3.7.2 GROUP STONE COLUMNS (THREE AND FOUR COLUMNS)

1. **Pipe formation:** Similar to single horizontally reinforced stone columns the first step is repeated for 3 and 4 Stone columns.
2. **Column casting:** The model tank was depicted as a cylinder with an impact zone diameter hemmed in the surrounding soil and a group of stone columns using the unit cell technique. A row of stone columns is typically arranged in a triangular (three) or square (four) pattern. For ease of theoretical analysis, the tributary area was converted into circles (which were cylinders in 3D) with comparable cross-sectional area in group projects, where each column is a kind of tributary area for the surrounding soil in hexagon form for triangular grid and square form for square grid. As a result, for a square distribution [254], the diameter of corresponding unit cell was determined to be 1.05 times column spacing ‘s', and for a triangular distribution, it was calculated to be 1.13s.

3.8 FEASIBILTY TEST

A research was conducted prior to the key trials to fine-tune the method for mixing the materials and compacting the soil layers, as well as to determine the feasibility of auguring the soil and building the stone column hole without it collapsing, in order to identify any potential issues before the actual tests. This permitted the homogeneity of residual soils within the tank to be determined, as well as the initial water content of the soil and the water content after 48 hours of drying. It also ensured that the viability of compressing the stone used as a fill was assessed, as well as the amount of blows and volume of compaction needed to attain the required relative density of stone.

3.9 REPEATABILITY OF TEST

While it is well recognised that more test repetitions lead to more precise and consistent outcomes, only a few experiments in this study were replicated once. Because of the large size of the test cells, the time it took to shape the soil bed and install the stone columns, and the limited period of time allotted to each task, repeating all of the experiments several times was extremely difficult. At least three trials of each collapse test have been performed to ensure the measurement of reliable and accurate values. The measurement at any certain load of three repetitions is averaged and computed by summing up the three values and then dividing them by three. The average values of final load are shown in Table 3.4.

Table 3.4 Average values of experimental results (Final Load)

Type of Reinforcement	Experimental Results			Average Value (kN)	Final Settlement (mm)
	End Load (kN) (First Trial)	End Load (kN) (Second Trial)	End Load (kN) (Third Trial)		
Single un-reinforced column	7	7.20	7.10	7.10	30.00
Single vertically encased column	14	14.70	14.20	14.30	30 .00
Single horizontally reinforced column	14.5	15.5	15	15.00	30.00
Group of 3 unreinforced columns	17.00	16.90	17.70	17.20	30.00
Group of 3 vertically encased columns	21.80	22.10	22.70	22.20	30.00
Group of 3 horizontally reinforced columns	22.40	22.50	22.60	22.50	30.00
Group of 4 unreinforced columns	20.50	20	20.70	20.40	30.00
Group of 4 vertically encased columns	24.70	24.40	24.10	24.40	30.00
Group of 4 horizontally reinforced columns	24.90	24.80	25	24.90	30.00

3.10 TESTING PROCEDURE

Test procedure involved in the current research work consist of test tanks, loading frames , pumping unit, hydraulic jack and devices for measurement of settlements and load. The size of the tanks was decided carefully in view of the diameters of the column to be tested, the zones of influence in both vertical and horizontal directions. The tank dimensions were chosen so that the distance between the column's edge and the tank's side has no bearing on the failure segment. For stone columns, the area replacement ratio (Ar) varies between 10% and 35% [112].

The thickness of loading plate was evaluated using a trial and error technique to ensure there was no plate deformation under load. According to Ali et al [10], the plate measurements were kept such that Ar = 26 percent for a group of four stone columns and Ar = 18 percent for a group of three stone columns. [10], in which Ar = 25% was used as a constant. A modified plunger with an 80 mm diameter was used to apply the load, and the tank dimensions were 3 to 5 times the diameter of the filled area for a unit cell. By adding a plunger to the UTM, a compressive load of 1kN/min was applied. Once a settlement of 30 mm had been reached, the load was halted. By dividing the total load by the footing area, which is used to measure the applied vertical stress, the footing pressure was measured. In the case of a group of stone columns, however, the load on a single stone column was not measured. The columns collapsed at various loads due to different reinforcement structures and their effect on increasing the load bearing strength of the soil. We encountered early penetration into the soil when we modelled floating columns, prior to the creation of major hoop stresses as expected by the model (Ali et al, 2013) [10].

3.11 NUMERICAL MODELING

3.11.1 GENERAL

In order to have a precise solution to a geotechnical problem, the conditions of compatibility, material performance, equilibrium and boundary criteria of displacement and forces need to be satisfied and in recent years, it has been found that these requirements are entirely fulfilled by numerical methods of analysis. Ever since the development, tremendous advancements in numerical methods have been made because of the wide utility of this new age technology and the software that can perform complex calculations in a relatively short span of time. Finite element method (FE), finite difference method (FD), boundary element method (BE), and discrete element method (DE) are some of the most commonly used numerical methods.

The method used for this analysis is finite element model (FEM), which brings the infinite number to a finite quantity in the form of ordinary or partial differential equations after analysing their behaviour. The elements can be arranged in any manner and thus can be used to model any shape. The method makes it possible to find solutions for the problems with complex geometry and problematical non-linear equations without the prerequisite of different forms of analytical solutions. The FEM technique proves to be efficient in handling the issues of non-finite periphery conditions, composite equations and behaviour as a continuum. The strength of the method lies in the fact that the changes in material stiffness evaluated even at elemental level can be easily accommodated. It also permits the application of various boundary criteria to achieve a globally acceptable estimated solution to a physical problem.

3.12 PLAXIS 2-D

The entire finite element modelling using Plaxis 2D and 3D has now been revised thoroughly. The finite element modelling in Plaxis 2D is carried out in an axisymmetric environment for single stone column. The 15 – noded triangle element is used to model the soil and related volume clusters as it depicts a highly level of accuracy in predicting the stresses (Fig. 3.18).

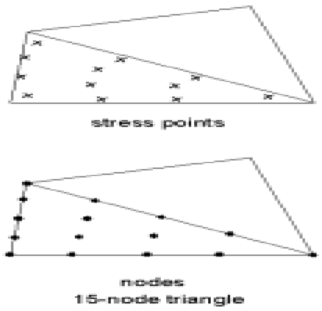


Figure. 3.18 15 – node triangle element used (Ref: Plaxis)

The axisymmetric model used signifies that the radial strains of the model are equal in all direction, εx = εz. As the name implies the structures in the model is symmetrical along the vertical Y axis and the model is rotated about the Y axis which results in a circular excavation. In Plaxis 2D, the rotating axis is always at the left boundary. Since in case of a single stone column, the strains in both x and z axis is equal, axisymmetric modelling leads to lesser calculation time and ease of convergence. The modelled axisymmetric model is given in Fig. 3.19.

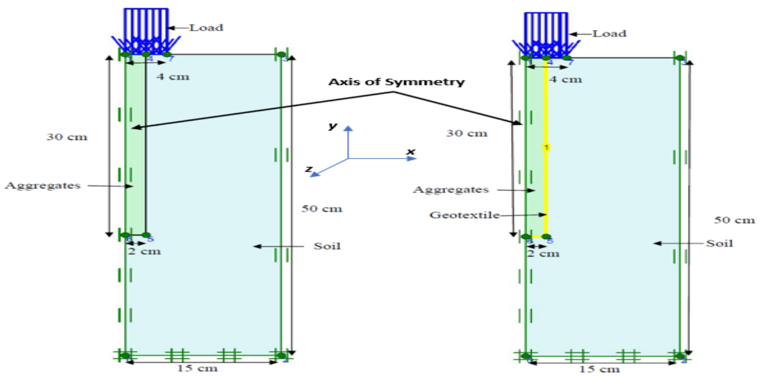


Figure 3.19 Axisymmetric model of unreinforced and vertically reinforced stone columns

Stone columns under the loading undergo a vertical settlement and lateral displacement. Hence to replicate the actual failure of stone column under the load, the modelled stone columns are allowed displacement in the vertical y – direction. The base of the model is restricted in both x and y directions. The lateral boundary is set free in x direction and is allowed displacement in y – direction.

The modelling of aggregates and surrounding soil medium has been carried out using the soil models available in Plaxis code. As per the literature, the most used soil model is the Mohr – Coloumb.

3.12.1 MATERIAL PROPERTIES

The Mohr-Coulomb modeled the non-linear behavior of the soil into two bilinear lines, as presented in Fig. 3.20.

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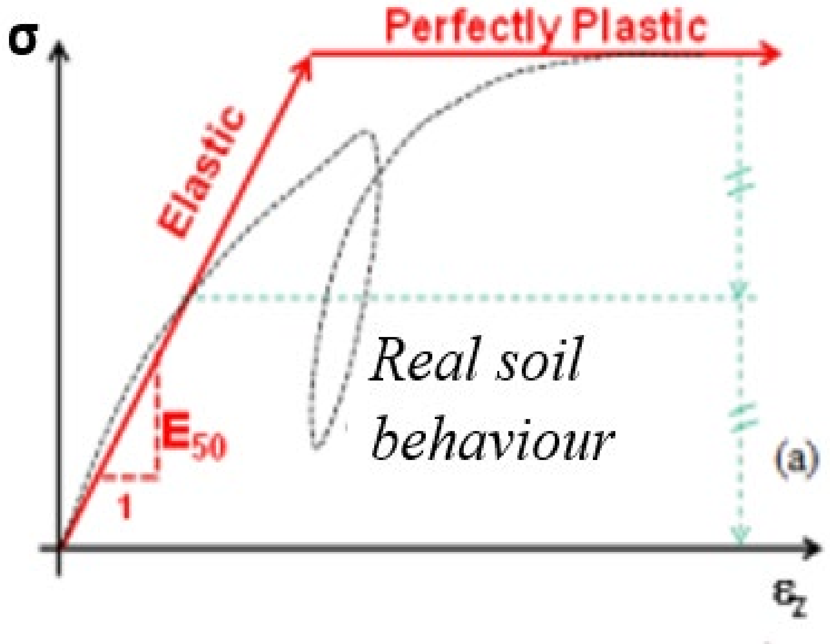


Figure. 3.20 Simplification of real soil behavior by Mohr – Coulomb model

The Mohr-Coulomb soil model requires 5 input parameters, in particular, Young's modulus E, Poisson's proportion ν, cohesion c, friction angle of soil φ, and a dilatancy angle ψ for modelling of soil. Since the surrounding soil used in the study is sand, drained behavior leading to development of no excess pore water pressures under loading has been employed. The Mohr – Coulomb soil model is also used to simulate the gravelly soil i.e. infill of stone columns: aggregates. The only short coming of modelling aggregates in Plaxis is the non –

availability of determining the size of aggregates. The unit weight and shear strength parameters of the modelled soil and aggregates are adopted as determined in the lab testing as given in Table 3.5. For poisson's ratio and dilatancy angle, standard values for sand have been used as per Plaxis.

Table 3.5. Properties of soil and Aggregates

Parameters	Soil	Aggregates
Soil	Sand	-
Cohesion (c)	1.96 kN/m^2	0.10 kN/m^2
Friction angle (ϕ)	20°	42°
Bulk Unit Weight ($\gamma_{unsat.}$)	19.66 kN/m^3	22.78kN/m^3
Saturated Unit Weight ($\gamma_{sat.}$)	21.75 kN/m^3	23.25 kN/m^3
Modulus of Elasticity	20,000 kN/m^2	55,000kN/m^2
Poisson ratio	0.30	0.30

3.12.2 MODEL CONFIGURATION

Geotextile reinforcement was modeled using the "geogrid" element. The geogrid element is primarily a line element unable to sustain compression loads and with two translational degrees of freedom i.e. u_x and u_y at each node.The geogrid element in conjunction with the 15 – noded triangle element coincides at 5 – node points for evaluation of stress as shown in Fig. 3.21.

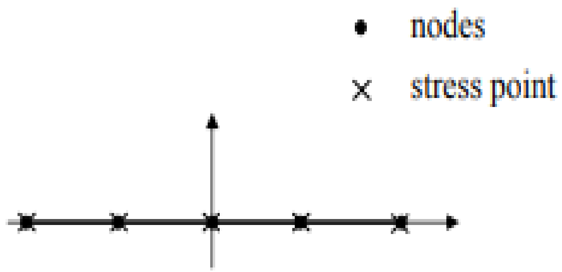


Figure. 3.21 Position of nodes and stress points

The defining parameter for modelling the stiffness of the geotextile used for reinforcing stone columns is the elastic axial stiffness EA where E = Young modulus of the geotextile material, A = cross sectional area of geotextile per m. The axial stiffness of a geogrid element used for modelling the geotextile is defined as the ratio of the axial force per unit width and the axial strain (Eqn. 3.1). Therefore, EA has units of force per unit width.

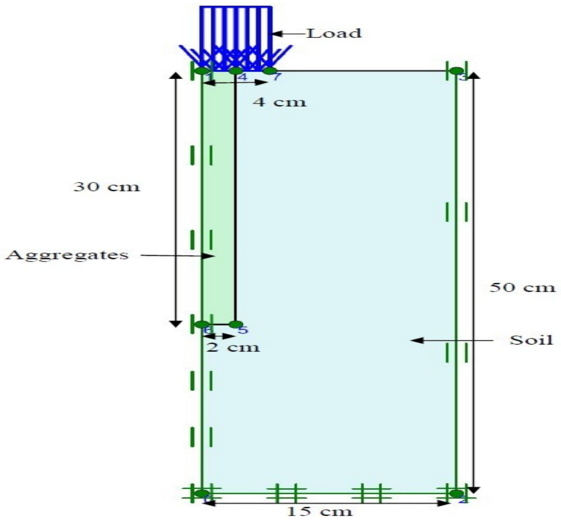
$$EA = \frac{Applied\ force\ in\ longitudinal\ direction}{\Delta l / l} \tag{3.1}$$

The yield strength of the geogrid is defined by the value of plasticity denoted as N_P. In the present case, the N_P is adopted as per provided by the manufacturer of geotextile given Table 3.6.

Table 3.6 Woven Polypropylene Geotextile properties

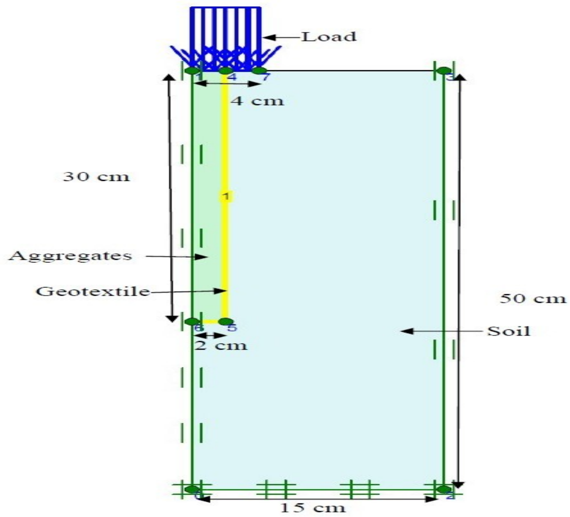
Parameter	Value
Yield Tensile Strength (N_P)	45kN/m
Modulus of Elasticity (E)	150,000.00 kN/m^2
Mass/area	200g/m^2
Axial Stiffness, EA	75000 kN/m

Geotextile reinforcement was modeled using the "geogrid" element. The geogrid element possesses only one (axial) degree of freedom at each node, and is subjected to tensile forces only. The configured model used for analysis is as depicted in Figure 3.22. The model is indicative of the geometry and boundary conditions as simulated in the analysis.



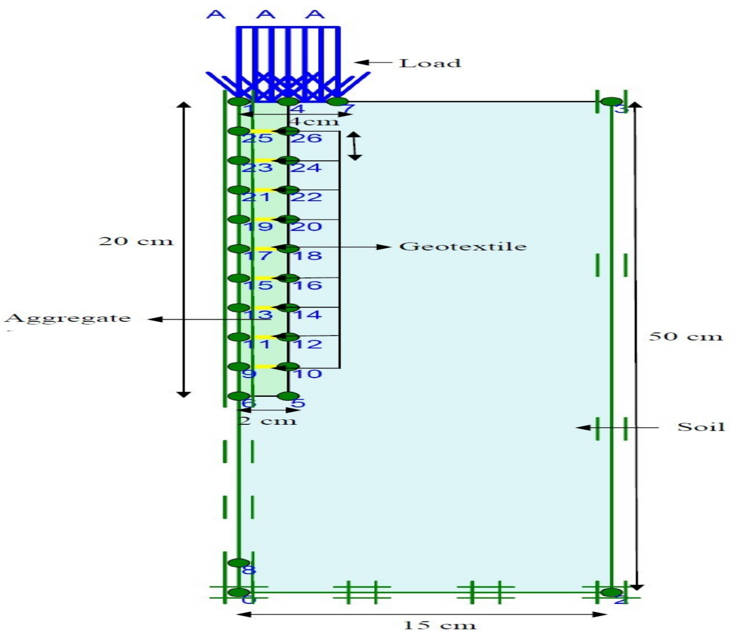
Unreinforced column

(a)



Vertically reinforced column

(b)



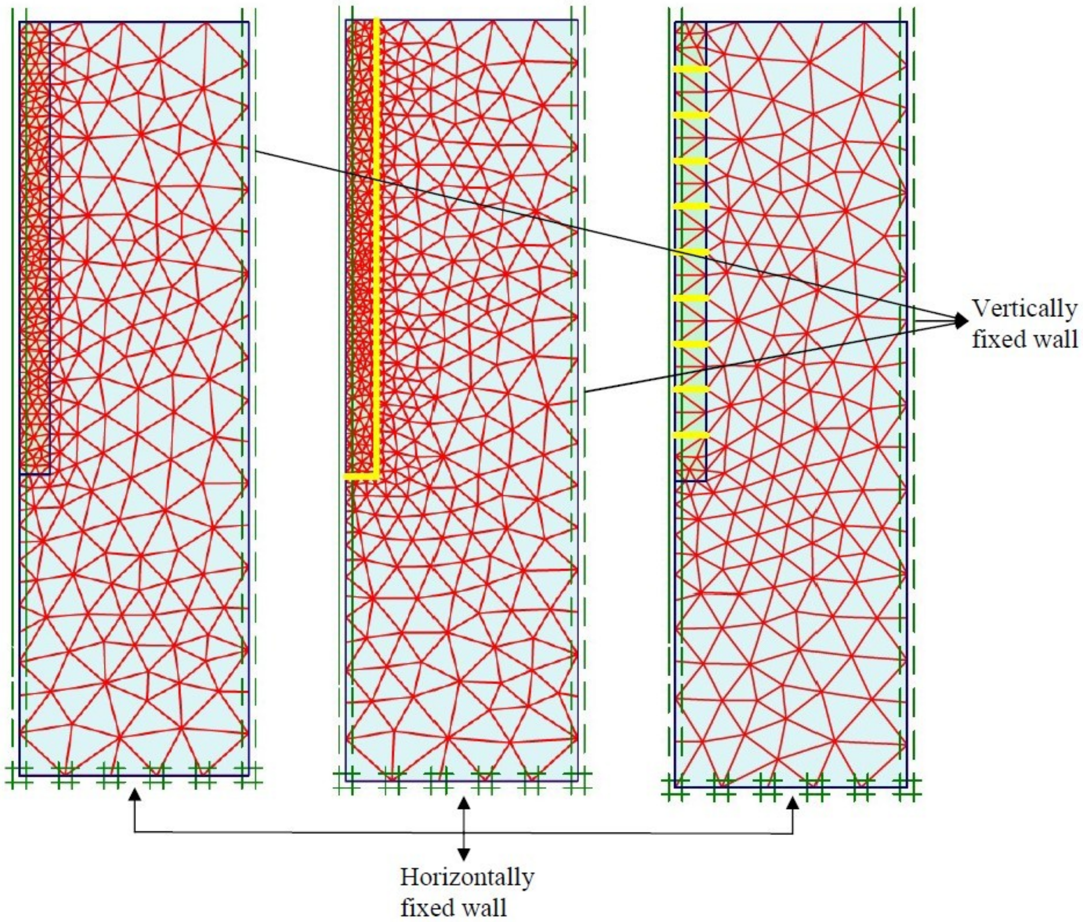
Horizontally reinforced

(c)

Figure 3.22 Geometry Model with all structural elements

3.12.3 MESH GENERATION

Once the geometry of the model is characterized and material properties are assigned, there arises a need to divide the geometry into finite elements in order to permit finite element calculations. This organization of finite elements is called a mesh. Plaxis 8.0 permits fully automatic mesh generation in majorly these forms – very coarse, coarse, medium, fine and very fine. For evaluation of deformations and stresses precisely, the mesh generation is made fines near the soil stein column interface. This also enables capturing of bulging failure occurring during loading of stone columns. The meshing has been generated with finer mesh near the interface and progressively growing to coarse towards the lateral boundaries. Figure 3.23 depicts the generated 2D mesh for all the three cases of unreinforced SC, vertical encased SC and horizontally reinforced SC.



Unreinforced (a) | vertically reinforced (b) | Horizontally reinforced (c)

Figure 3.23 Initial Mesh Generated

Once the composite ground has been discretised, the initial ground water condition and equilibrium stresses are specified. This is generally achieved by positioning the phreatic line at specified location. This parameter becomes critical are situations involving undrained soil conditions, since Plaxis mainly works on the effective stress condition. However, in the present study, drained condition was modelled by locating the phreatic line at the base of the model. This signifies that all pore pressure and external water pressure developed is taken as zero during calculation. The initial vertical stresses are developed using the Eqn. 3.2 given as:

$$\sigma'_{v,0} = \sum M_{weight} \left(\sum_i \gamma_i \cdot h_i - p_w \right) \tag{3.2}$$

Likewise, the initial horizontal stresses are evaluated using the coefficient of earth pressure at rest (K_0). The K_0 value is calculated as per the Jacky's formula given by Eqn. 3.3 and horizontal stresses using Eqn. 3.4:

$$1 - \sin \varphi \tag{3.3}$$

$$\sigma'_{h,0} = K_0 \cdot \sigma'_{v,0} \tag{3.4}$$

With the modelled sand domain with embedded stone column, the calculation phase is initiated. The loading on the stone column is applied using the uniformly distributed line load as available in the Plaxis code. The loading and the corresponding boundaries are activated prior to the calculation is started. In the present study, plastic calculation is used for evaluating the deformation and failure of both unreinforced and reinforced stone columns. The main reason for using plastic analysis is based on the fact the original undeformed state of stone column reinforced ground condition is taken as the starting point for development of the stiffness matrix. Also, the plastic calculation is considered as most suitable for elastic – plastic deformation without considering the decay for excess pore water pressure with time. The same procedure of modelling and analysis is also adopted for group of stone columns. However, considering the limitation of axisymmetric condition for group of three and four columns, the behaviour of group of stone columns is also studied using Plaxis 3D.

3.13 PLAXIS 3D

The approximate Finite Element Method (FEM) has proven itself as a valuable tool for the study of complex engineering problems. The method's theory requires the use of simulated work to estimate the spread of stress and pressure over a continuum. For geotechnical applications PLAXIS 3D is a three-dimensional FE program precisely designed for such applications. For mimicking the complex behaviour of small group of stone columns this program is quite ideal and was adopted for the consequent finite elemental analysis. According to this program, the both soil and stone behaviour is simulated with advanced constitutive models, which are described in the chapter. In addition, it is required to carry out a number of introductory checks, such as mesh sensitivity and distance to the boundary to obtain precise numerical analyses.

The numerical analysis was carried out by using PLAXIS 3D software (finite element method) to compare the load-settlement of experimental investigation and model test. The boundary condition was carried out as well via employing Mohr-coulomb for the stone column and Hardening soil for sand, A drained behavior was assumed for the column and clay. Fifteen nodded triangular was used for the process of meshing. Medium deformation was restricted to all these boundary conditions that were used to represent the behavior of the stone column surrounding by the soil and typical deformation and mesh for the stone column.

In PLAXIS 3D program, several phases of analysis must be defined, in each phase, the program makes the required calculations. The second phase of the present work included calculation of initial stresses and limit stress in the soil. The calculation consists of three phases except the initial phase for generating the initial stresses with active groundwater table. The process of setting the stone columns was chosen in phase one. Phase two was to simulate the elements of the stone columns. The load was selected in phase three to consider settlement and stresses in the stone columns and surrounding soil. Calculations in Phase 1 include estimating the initial stresses, the effective stresses of the soil are calculated by using K_o procedure where $K_o=1-\sin\phi$ is the lateral earth pressure coefficient at rest which defines the relationship between horizontal and vertical stresses in the soil.

The work planes are horizontal planes layers with different y – coordinates for defining the discontinuities in the form of objects, loads and constructions stages. In the present study, the 3 work planes are defined at the depth of 0 mm, 300 mm and 500 mm. The stone columns in case of Plaxis 3D are modelled using the massive circular pile technique. The stone columns are modelled with zero thickness and diameter equal to 40 mm. The piles are then provided with the properties of aggregates as given Table 3.5. Like Plaxis 2D, the surrounding soil is modelled using the Mohr-coulomb model with properties as given in Table 3.5. The vertical geotextile reinforcement is modelled using the wall element with thickness d = 1 mm and unit weight, stiffness as per Table 3.6. For horizontal reinforcement, the geotextiles are modelled using plate element. The plate elements are based on the Mindlin's plate theory and are allowed deform due to shearing and bending. The plate element also undergo elongation in length under the application of an axial force. The stiffness of the plate element is also selected as per Table 3.6.

3.13.1 MATERIAL PROPERTIES

In Mohr-Coulomb model which idealises soil as an elastic-perfectly plastic material. The behaviour of soil before failure is approximated by Hooke's law of elasticity. The failure of soil is based upon the Mohr-Coulomb failure criterion which is defined by two parameters, angle of internal friction (ϕ) and cohesion (c). This failure criterion is an extension of Coulomb's friction theory and its yield surfaces in principal stress space. The parameters are as shown in Table 3.5 & 3.6.

3.13.2 MODEL GENERATION

3.13.2.1 MODELING OF SOIL BEHAVIOUR

In PLAXIS 3D modeling, the important data that need to be taken into account is the type and hydrostatic state of the soil under investigation. Indeed, all the integrated parameters must represent the effective response of the soil, particularly emphasizing on the relations between the stresses and the deformations of the skeleton of the soil. The presence of pore water in the soil significantly affects the response of the soil and therefore must also be taken into consideration. As the cohesiveless soils do not develop excess pore pressure during loading and there are two methods i.e. Undrained and Drained with the help of which such soils can be modelled with PLAXIS 3D software.

3.13.2.2 SOIL-COLUMN INTERFACE MODELLING

PLAXIS 3D software provides interface components for modelling interaction between smooth and rugged materials, such as pile walls/basement walls and soil. These components will mimic slip displacements and gaps that are natural and parallel to the interface. The 16 node elements consist of 8 pairs of nodes (2 nodes at the same point; 1 for the soil and 1 for the wall).

In addition to Coulomb criterion which is adopted to differentiate between elastic and plastic behaviour of element is demonstrated as elastic-plastic,. With a strength reduction factor (R_{inter}) loss of strength at interface is modelled, which is relationship between interface strength to that of soil strength through cohesion (c) and friction angle (φ):

$$c_i = R_{inter} c_{soil} \tag{3.5}$$

$$\tan\varphi_i = R_{inter} . \tan\varphi_{soil} \le \tan\varphi_{soil} \tag{3.6}$$

The elements have zero thickness in actual, however, a virtual thickness is assigned to determine element stiffness. Gap and slip displacements are calculated from the odometric ($E_{oed,i}$) and shear (G_i) moduli, respectively. The moduli are given by the expression:

$$E_{oed,i} = 2G_i \frac{1-\nu_i}{1-2\nu_i} \tag{3.7}$$

Where ν_i=0.45

A rigid interface elements (i.e. R_{inter}= 1) was adopted by Guetif et al. [86]on the basis that there is tight interlocking of stone columns with the surrounding soil and thus a perfect bond exists along the column-soil interface. However Gäb et al. [260], Elshazly et al. [261], Domingues et al. [262] and many other authors prefer model a perfect bond along the column-soil interface by neglecting interface elements.

The column arrangements with interface elements yield lower settlement improvement factors which mean that they over-predict the settlement of stone columns. This probably attributes to elastic-plastic material model and the increased Poisson's ratio (ν_i=0.2→0.45) assigned to interface elements in PLAXIS 3D program. An increased Poisson's ratio yields lower shear moduli with an increase in slip displacement.

The interface elements' impact also becomes more marked with an increasing number of columns. The punching of closely-spaced columns into the underlying soil occurs, whereas columns at higher A/A_C tend to bend and swell. Therefore, with decrease in ratio A/A_C, most of the applied load is shifted along the side of columns and interface elements inducing more displacement.

The strong interlock of columns into the surrounding soil makes it reasonable to model the column-soil interface in the subsequent FEA by omitting interface components, according to the work of several scholars. To have vertical protection, the stone columns were encased in geotextile. The downward displacement of the encased stone columns–geotextile interface is completely determined by friction between the soils. Due to hoop stresses caused by column bulging collapse and the high angular presence of aggregates, it is assumed that the aggregate – geotextile interface is crucially higher than the soil – geotextile interface. Thus, settlement loss in vertically encased floating stone columns between soil and geotextile is primarily determined by mobilised interface friction

The interface test has been repeated as per the reviewer's suggestion. As seen from Table 3.7, soil – to – soil interface depicts higher interface angle of approximately 20° with a small cohesion value of 1.15 kN/m^2. The interface friction for soil – to – geotextile interface was found to be around 17.4° with a cohesion value of 2.5 kN/m^2 (Fig. 3.24). The lower value of soil – geotextile interface depicts that during loading as the hoop stresses are developed within the stone column, the geotextile confinement restricts the transfer of stresses to the surrounding soil. Due to this the hoop stresses are transferred to the stone column base with induces the settlement of stone column. Now as the stone column settles, a shearing resistance is mobilized between the aggregate – geotextile – soil interface. Since the soil – geotextile interface has a smaller interface friction value; it signifies that it will be mobilized prior to the mobilization of soil – soil interface. This process continues till complete mobilization of shearing resistance at the soil – geotextile interface is mobilized. In this way the geotextile adds to the additional shearing resistance as well as confinement.

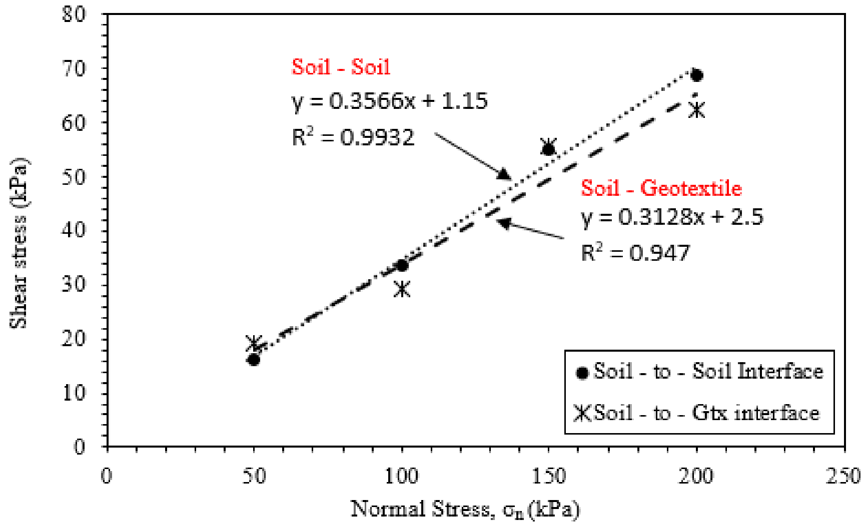


Figure 3.24 Interface friction angles

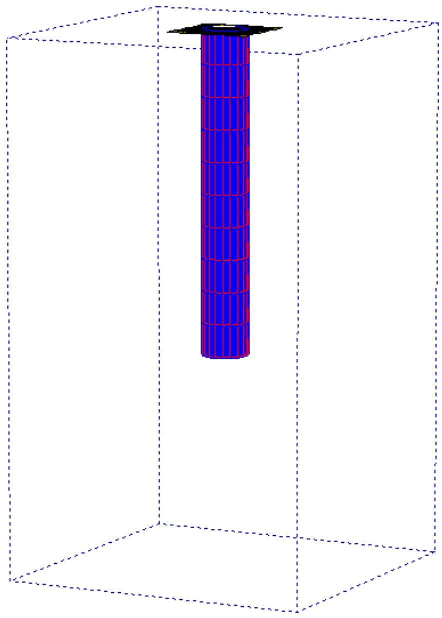
Table 3.7 Summary of test results for different interfaces by the conventional direct shear box.

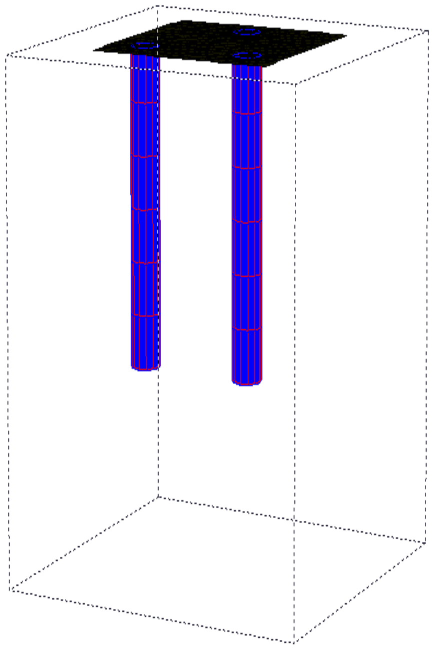
Type of interface	**Normal stress, σn (kPa)**	**Shear stress peak (kPa)**	**Friction angle (ϕ)**	**Cohesion, c (kPa)**
soil - soil	50	16.3	**19.62° ≈ 20°**	**1.15**
	100	33.7		
	150	55.1		
	200	68.6		
soil-geotextile	50	19.1	**17.37° ≈ 17.4°**	**2.5**
	100	29.2		
	150	55.7		
	200	62.4		

3.13.2.3 MESH GENERATION

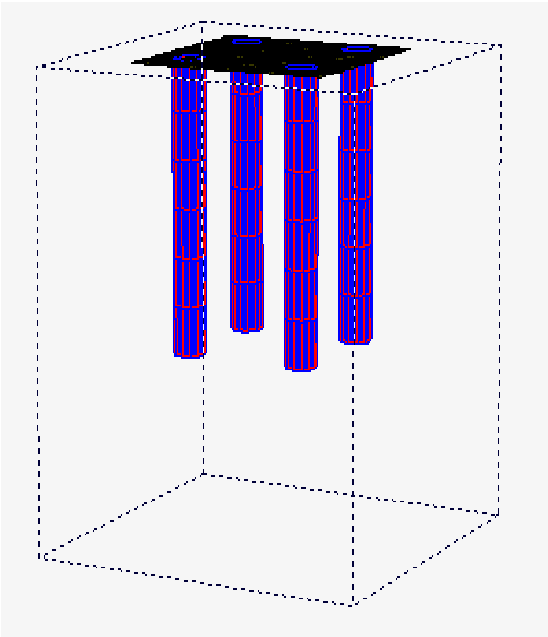
A mesh sensitivity analysis describes the effect of the number of elements upon the accuracy of the FEM. In the subsequent parametric studies, both the number of columns and column spacing are varied beneath pad footings which results in various footing sizes. Mesh sensitivity analyses are carried out for six different sizes of footing, and the accuracy of fine, medium, and very fine meshes is compared.

Vertical displacement (u_y) and mean effective stress (p') were measured. While conducting mesh sensitivity analyses, the examination of stresses in the zone of interest must be done carefully, as the distribution of stress within elements is derived from lower order equations than the displacement. Therefore, stress converges slower than displacement with increasing mesh density and thus the distribution of stress within an element will not be as accurate as the displacement. Vertical displacements were also examined for the mesh sensitivity analysis as settlement performance of stone columns is the main focus of this thesis. The generated 3-D mesh is shown in Figure 3.25.





(a) Generated Mesh for Single Stone Column (b) Generated Mesh for 3 Stone Columns



(c) Generated Mesh for 4 Stone Columns.

Figure 3.25 Generated 3-D mesh

By comparing the normalised error for vertical displacement (u_y) and mean effective stress (p') against very fine meshes, the rightness of medium and fine meshes is determined:

Normalised error for vertical displacement is calculated as given by Equation (3.8):

$$u_y = \left(\frac{u_{y,vf} - u_y}{u_{y,vf}} \right) \times 100 \tag{3.8}$$

Normalised error for mean effective stress,

$$p' = \left(\frac{p'_{vf} - p'}{p'_{vf}} \right) \times 100 \tag{3.9}$$

3.13.2.4 INFLUENCE OF DISTANCE TO BOUNDARY AND BOUNDARY CONDITIONS

A zone of soil surrounds the footings modeled in the subsequent parametric studies that do not undergo lateral displacement along its outer boundary. Boundary conditions should not influence the results that are why it’s requisite to locate the boundary at a sufficient distance from the footing. The first step in the modeling software is to specify the three dimensional geometry of the soil sample to be modeled. Each model is further subdivided into three groups; group of soils (sands), group of stone columns, and group of geosynthetic materials while configuring the geometry of the models. The steps followed are given below:

1. The coordinates of the points which form the geometry of the soil model (x, y, z) are given in the Cartesian coordinate system;
2. A uniformly distributed load is applied to a specific area by specifying the coordinates;
3. Choosing the material properties; if the types of material already described in input parameters of PLAXIS, then only the selection of the suitable type is required, otherwise the characteristics of the soil have to be tested by carrying out laboratory tests and then the input values for soft soil are introduced as done in our study;

CHAPTER-4

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 GENERAL

The main focus in this chapter is a comparison of behaviour of the encased stone column with that of the conventional stone column (i.e. ordinary stone column) both installed in weak cohessionless soil beds of identical properties. In order to bring out their relative performances, single stone column and group of stone columns (3 & 4) with and without encasement were formed at cohessionless soil beds independently and load tested using universal testing machine. Unreinforced, vertically encased geosynthetics and by providing horizontally reinforced geosynthetics in stone columns have also been studied for load settlement. The experimental data was analyzed in order to assess the relative change in soil bearing capability.

4.2 FINDINGS FROM MODEL TESTING

4.2.1 LOAD – SETTLEMENT BEHAVIOR OF SAND BED

Load test on a pure sand bed without stone column was performed initially to find determine the settlement of the modelled sand layer. Since the investigation was mainly on studying the behaviour of unreinforced and reinforced stone columns, the details were not provided in the thesis. However, as per the reviewer's suggestion, the testing results as now shown in Figure 4.1. It was seen from load – settlement response of pure sand bed without stone column that the top surface depicts a settlement of 50 mm under a maximum load of 4 kN and thereby the load becomes constant.

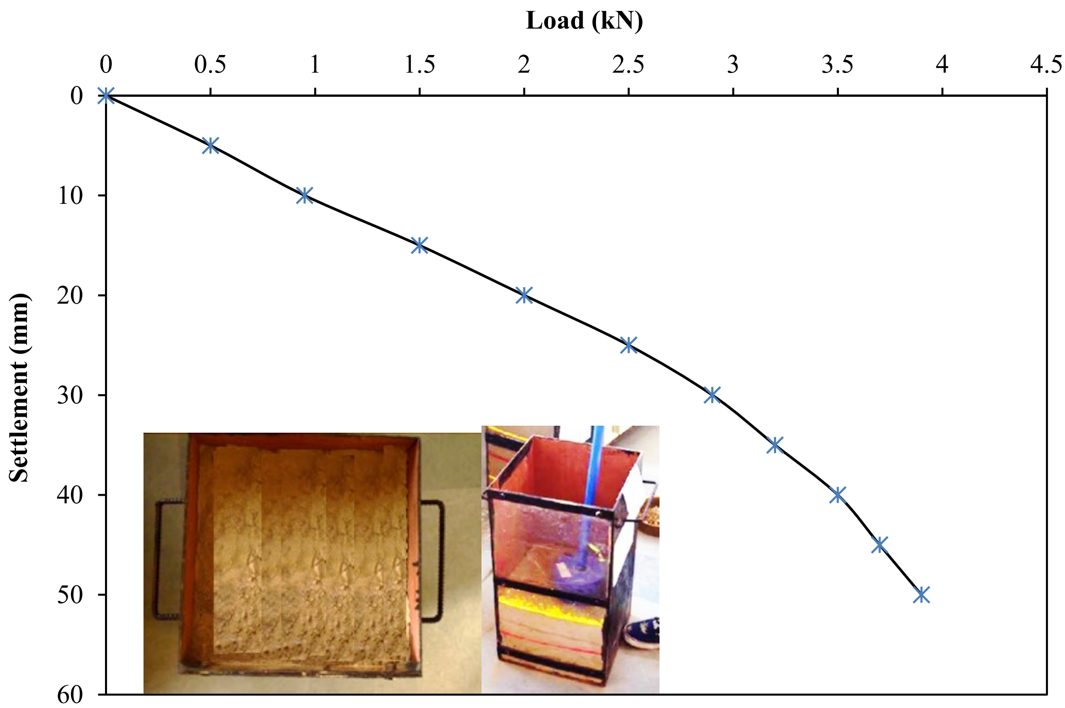


Figure 4.1 Variation in load – settlement for Sand Bed

4.2.2 LOAD – SETTLEMENT BEHAVIOR OF SINGLE STONE COLUMN

Figure 4.2 demonstrates the load difference for settlement of both unreinforced and reinforced (vertical and horizontal) single stone columns. By comparing reinforced and unreinforced stone columns, it was discovered that reinforced stone columns could carry more Load. Both vertical and horizontal reinforced groups respond in identical manner. In case of vertical reinforcement, the increase in confinement effect generates hoop stresses in the interior the stone columns, which are not dissipated under loading and thus gets constrained. This is equivalent to a lateral pressure rise from the surrounding surface, which counterbalances column sand expansion and allows for effective load transmission to the column's foundation. Similarly, in case of horizontal circular discs, the column deformation mobilises the interface friction. In this type of reinforcing, the total stone column length is partitioned into 10 mm sections, with circular geosynthetic discs at regular intervals of 3.0 cm. This reduces the aspect ratio, with a consequence of reduction in the tendency of column bulging and increase in significant load transfer.

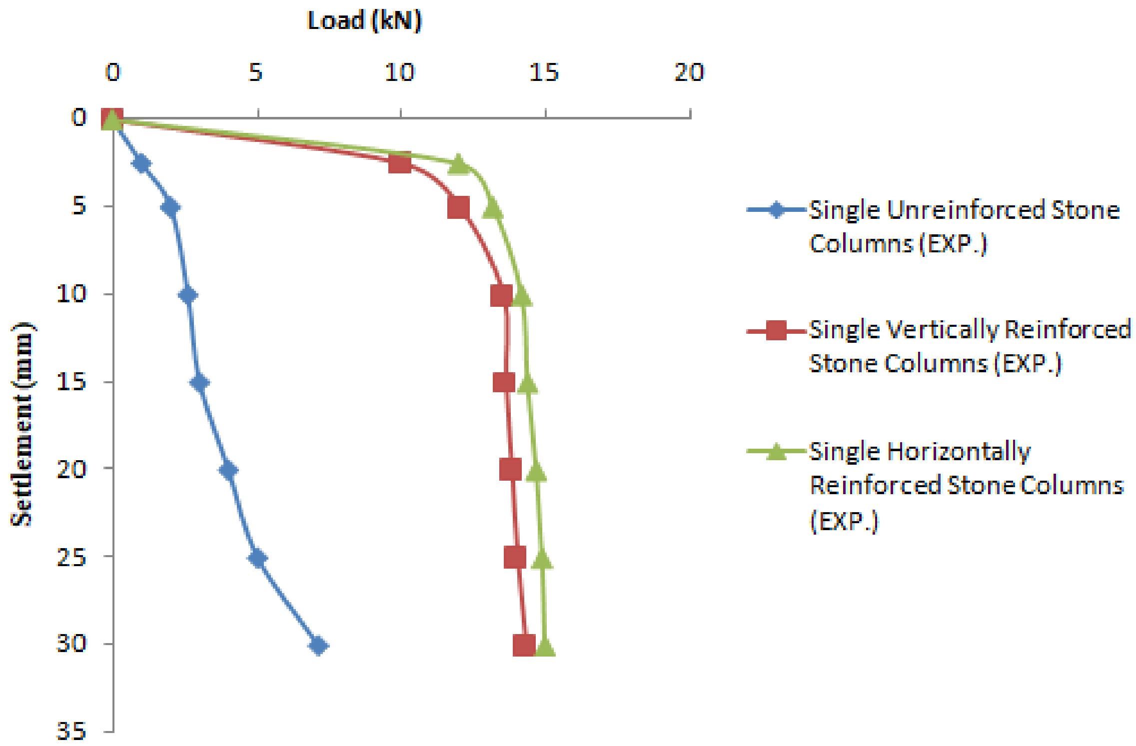


Figure 4.2 Load – settlement variation for single stone column

4.2.3 LOAD–SETTLEMENT BEHAVIOR OF GROUP STONE COLUMNS (3 & 4)

4.2.3.1 LOAD–SETTLEMENT BEHAVIOR OF THREE STONE COLUMNS

Figure 4.3 illustrates the experimental results on load vs. settlement for a three stone columns, both for unreinforced columns and geosynthetically reinforced columns (vertically and horizontally).In this case also, reinforcement enhances the load bearing tendency of unreinforced stone columns increases. Moreover, both vertical and horizontal reinforcement show similar response. Thus stone columns go through settlement in non-appearance of any end-bearing. The reason for this is same as discussed above for single stone columns.

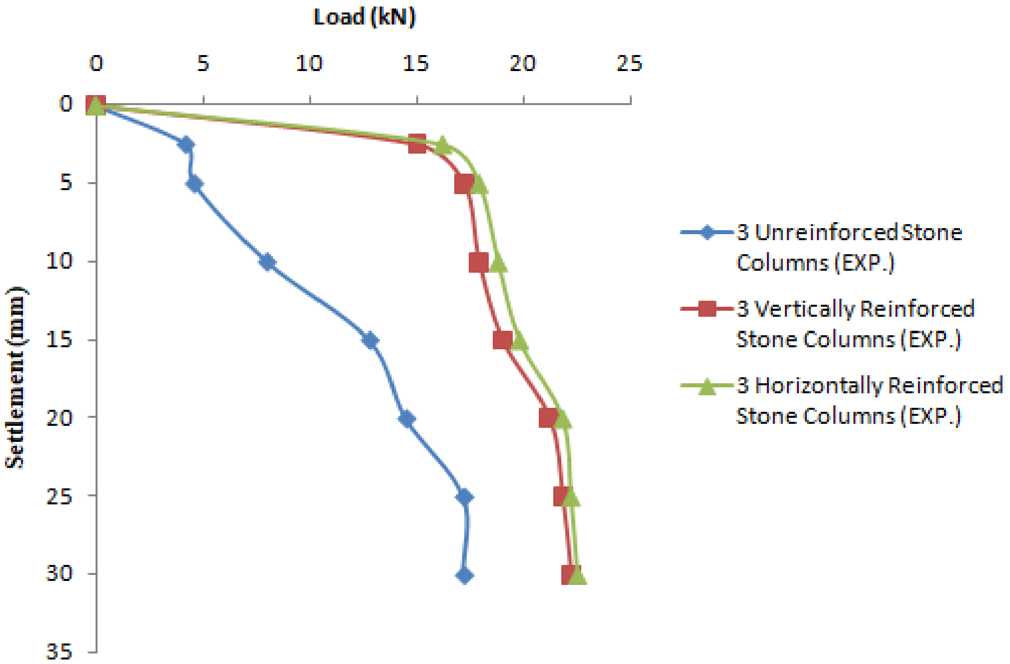


Figure 4.3 Variation in load settlement for three stone columns in group.

4.2.3.2 LOAD–SETTLEMENT BEHAVIOR OF FOUR STONE COLUMNS

Figure 4.4 indicates the load–settlement profile for four stone columns in group. Both vertical and horizontal reinforced stone columns exhibit the same reaction behaviour. Furthermore, the efficiency of reinforced stone columns has been found to be superior to that of unreinforced stone columns. For vertical and horizontal reinforced columns, the action is related to resistance by hoop tension and mobilisation of interface friction, respectively. Furthermore, there is tension between the soil-geotextile interfaces in the case of vertically encasement columns, which influences the downward rotation of floating type stone columns. Because of the more angular existence of aggregates and the production of hoop stresses as the column extends, the aggregate–geotextile interface is believed to be considerably higher than the soil–geotextile interface. In this way, the mobilised tension between the soil and the geotextile interface plays a part in the breakdown of settlement in vertically encased floating stone columns. The current research used a direct shear test to assess the importance of interface friction among geotextile and soil (DST). Geotextile was mounted in the lower mould's shearing surface and soil was placed in the upper mould of the DST equipment to perform the examination.

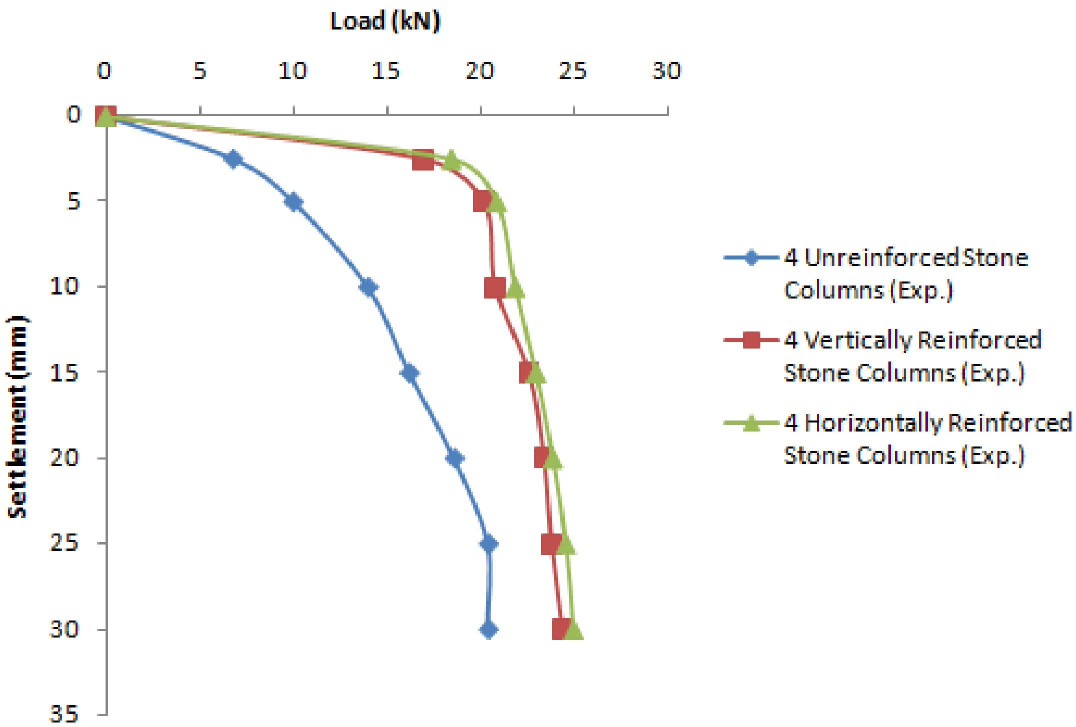


Figure 4.4 Variation in load – settlement for a group of four stone columns

The following observations have also been apparent from **Figures 4.2, 4.3** and **4.4**.

a) In the case of unreinforced columns, early settlement has been identified at loadings of less than 2.0 kN, 5.0 kN and 7.0 kN, however for Single, three and four stone columns.

b) With an arrangement of Single, three and four stone columns, traditional unreinforced columns can tolerate loads of about 5 kN, 14.5 kN and 18.6 kN till 20 mm settlement. Following that, the columns begin to settle without bearing any load. As a consequence, the diagrams are in constant step.

c) The vertically reinforced columns restrict settlement to 15 kN and 17 kN, respectively, for the arrangement of 3 or 4 stone columns.

d) The vertically reinforced columns, unlike the reinforced stone columns, have not collapsed up to a load of 22.20 kN for 3 columns and 24.40 kN for 4 columns. Leading to the limited expansion of vertically encased columns and reduced column penetration, it has also been concluded that vertically reinforced columns have a greater bearing capacity than unreinforced ones.

e) For assemblies of three or four stone columns, horizontally reinforced columns displayed no settlement up to loadings of 16.2 kN and 18.4 kN, respectively.
f) Horizontally reinforced columns have been found to have a similar response to vertically encased columns. The horizontally reinforced columns were found to resist failure until a 30 mm settlement was achieved at load applications of 22.20 kN and 24.40 kN, respectively, for groups of three and four stone columns. This column failure may be attributed to both the effects of bulging and buckling because to shear failure
g) Horizontal reinforcement has been shown to be preferable to vertical reinforcement in the case of sandy soils. This discovery contradicts what has been seen in previous research on coherent soils.

4.4 LOAD RATIO IMPROVEMENT

4.4.1 LOAD RATIO FOR SINGLE STONE COLUMN

The load ratio (L.R.) parameter is used to calculate the performance of stone columns in terms of ultimate bearing efficiency. The Load ratio is calculated as the ratio of ultimate load carried by reinforced soil to the ultimate load carried by with no stone column. Figure 4.5 depicts the L.R difference of settlement for stone columns with diameters of 40 mm with and without stone column reinforcement. As can be shown in Fig 4.5, load ratio for stone columns with diameter of 40 mm ranges from 0.51 for unreinforced stone columns to 0.55 for vertically reinforced and 0.58 for horizontally reinforced stone columns. The variation in load ratio can be accounted for the improved load transfer attained by reinforced stone columns than unreinforced stone columns. Moreover, it can also be seen that horizontally reinforced stone columns render 5% higher load transfer than vertically reinforced and about 13% higher load transfer than unreinforced stone columns.

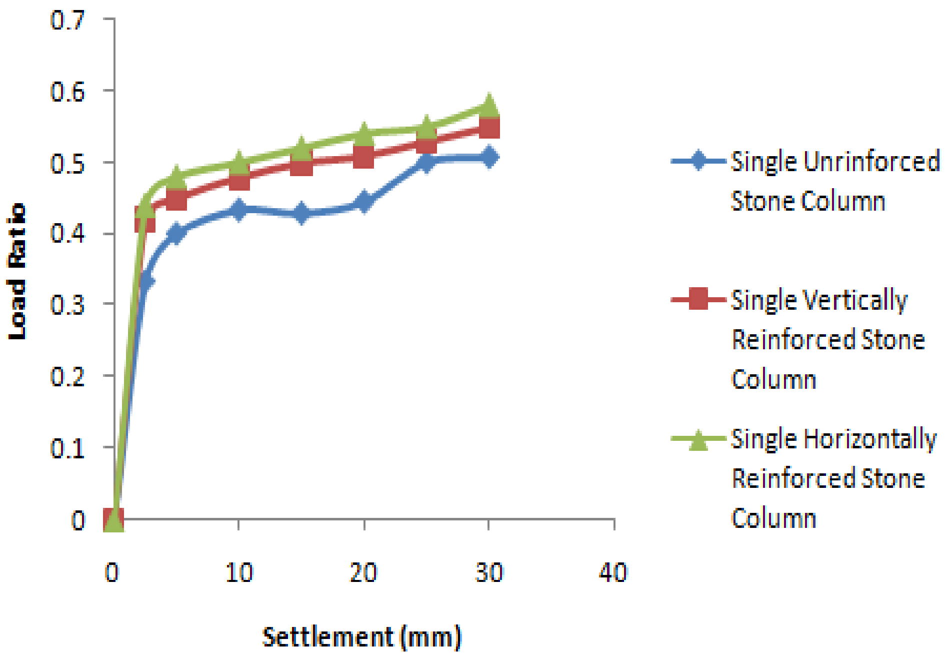


Figure 4.5 Variation of load ratio versus settlement for single stone column

4.4.2 LOAD RATIO FOR GROUP STONE COLUMN (3 & 4)

This is because support material keeps columns from bulging by providing lateral containment for three and 4 stone columns. Figures 4.6 & 4.7 also shows that raising the loading up to a settlement of around 30 mm raises the value of L.R, while increasing settlement beyond 30 mm reduces the value of L.R in unreinforced stone column due to bulging and exceeding the column's final strength for three & four stone columns. The difference of LR increases with loading plate displacement.

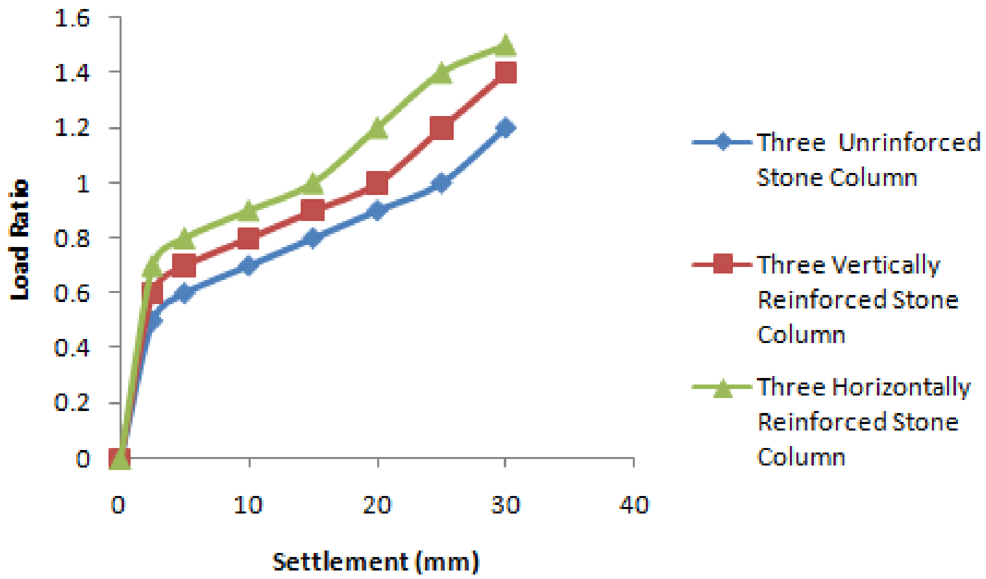


Figure 4.6 Variation of load ratio versus settlement for 3 stone columns

As can be seen from Fig. 4.6 and Fig. 4.7, for three group of stone columns, L.R values ranging from 1.2 - 1.5 whereas for four group of stone columns, LR varies between 1.4 - 1.5. Thus, it can be stated that load transfer obtained using horizontally reinforced stone columns is similar for both three and four stone columns. However, for vertically reinforced and unreinforced, load transfer from along the stone column length increases as the number of columns are increased. It is also evident that for equal number of stone columns (i.e. three) the load transferred by the stone column from top to its bottom is 25% more when the columns are horizontally reinforced as compared to unreinforced. In comparison to vertically reinforced stone column group of three columns, about 7% higher load is transferred with stone columns reinforced by horizontal discs.

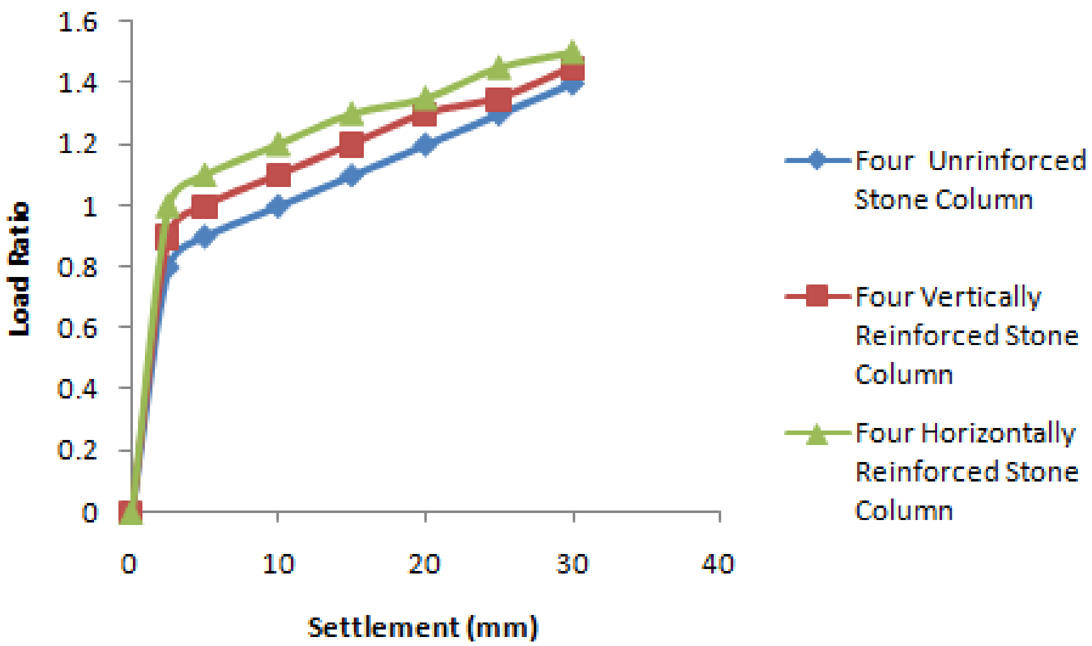


Figure 4.7 Variation of load ratio versus settlement for 4 stone columns

Thus, based on the above mentioned rationales, it can be deduced that mobilization of shearing stress due to lateral restrain against bulging at the aggregate – geotextile – aggregate interface render higher load transfer efficiency as compared to shearing mobilized due to stone column bulging and simultaneous settlement at vertical interface of aggregate – geotextile – surrounding soil.

4.5 STRESS CONCENTRATION RATIO

4.5.1 STRESS CONCENTRATION RATIO FOR SINGLE STONE COLUMN

The exterior load is split between stone columns and soft soil based on the ratio of column hardness to soft soil hardness. Since the column hardness is greater than that of the underlying soft soil, the stresses on the columns are higher than those on the soft soil. The stress concentration ratio (SCR) values does not remain constant for all stone columns, as can be seen from Fig. 4.8, and varies as column settlement increases. Figure 4.8 also reveals that the ultimate value of SCR for columns with a diameter of 40 mm is varies from 3 for unreinforced single stone column to 4 for vertically reinforced single stone column and reaches to 4.2 as the single stone column is horizontally reinforced. For single columns, it can be observed that under gradually increasing compressive load at settlement of 8mm, unreinforced stone column takes 28.5% less stresses as compared to horizontally reinforced stone columns. However, in comparison to vertically reinforced stone columns, unreinforced stone columns render 25% less stress development within the stone columns. When comparing both the reinforced single stone columns, it is found that horizontally reinforced stone columns have 5% greater stress development within the stone column. This higher stress development further supports the fact that higher load ratio are attained with horizontal reinforcement of stone columns.

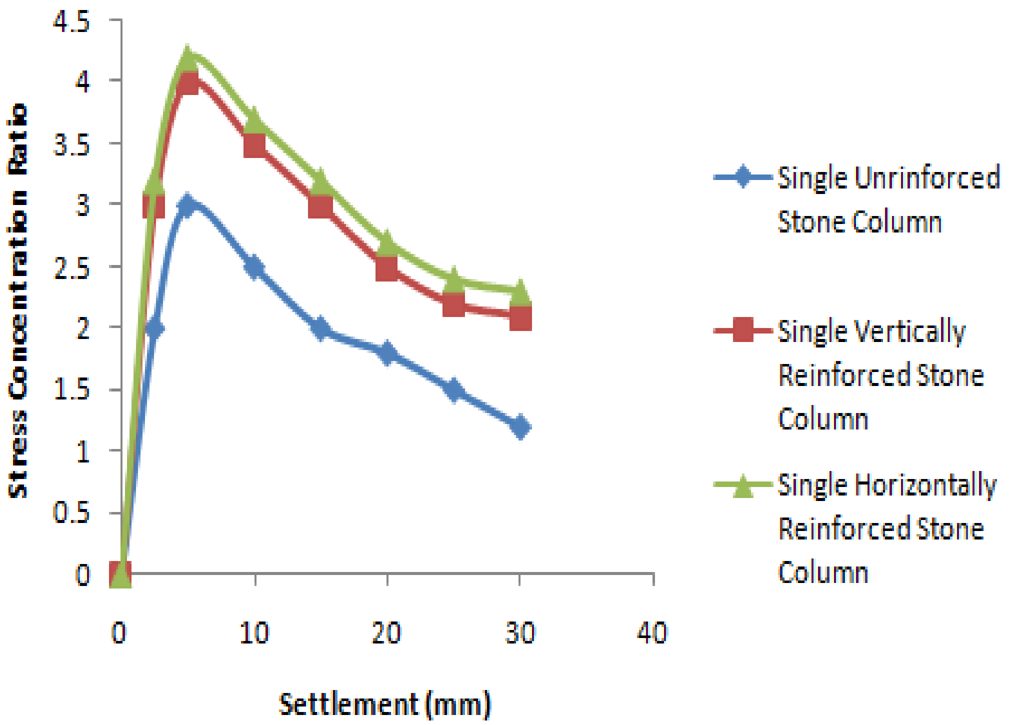


Figure 4.8 Variation of Stress concentration ratio versus settlement for Single stone columns

It can also be observed from Fig. 4.8 that as the settlement is increase from 8mm to 30 mm, the stress concentration gradually fades off and attains a constant value of 2.48 for both vertically and horizontally reinforced stone columns. For unreinforced stone column, at 30 mm a much lower SCR value of 1.25 is obtained. The decrease in the SCR value with increase in settlement can be accounted for the release of stress from stone column to the surrounding soil which is marked by the inevitable settlement of both unreinforced and reinforced stone columns.

4.5.2 STRESS CONCENTRATION RATIO FOR GROUP STONE COLUMN (3 & 4)

It can be seen from Fig. 4.9 and Fig. 4.10, that for both three and four stone columns groups, SCR value increases up to settlement of 10mm and thereby declines as the settlement reaches 30 mm. Beyond 30 mm,. a constant value of SCR is reached. The stone column travels backward and rearranges the stone column grains in unreinforced during the first step of filling up to 10 mm displacement. The material density of the column is significantly increased as a result of this. As a result, the grains mechanically interlock. Granular content continues to migrate laterally into the underlying soft soil as load and settling increase. The load is gradually transferred to the soft soil as a result of this. The SCR reduces as a result. The reinforcement material in encasement adds to the lateral isolation.

As compared to unreinforced, the SCR value reduces marginally. As seen in Fig. 4.9, SCR = 6 is obtained for three unreinforced stone column group, SCR = 7 for three stone column group reinforced vertically and SCR = 7.5 for horizontally reinforced three stone column groups. These SCR values are found to increase as the number of stone columns are increased from there to four for which SCR = 6.4 for four unreinforced stone column group, SCR = 7.9 for four stone column group reinforced vertically and SCR = 8.8 for horizontally reinforced four stone column group. The increase in SCR value from three to four stone columns can be accounted for higher stiffness of reinforced composite soil attained with greater number of stone columns. Similar to single stone column, it is obvious that low lateral restrainment around unreinforced stone columns lead to smaller generation of stresses within the stone column. The maximum generated stress is obtained for horizontally reinforced stone columns for both three (about 7%) and four (about 11.4%) stone column groups as compared to vertically reinforced stone column groups.

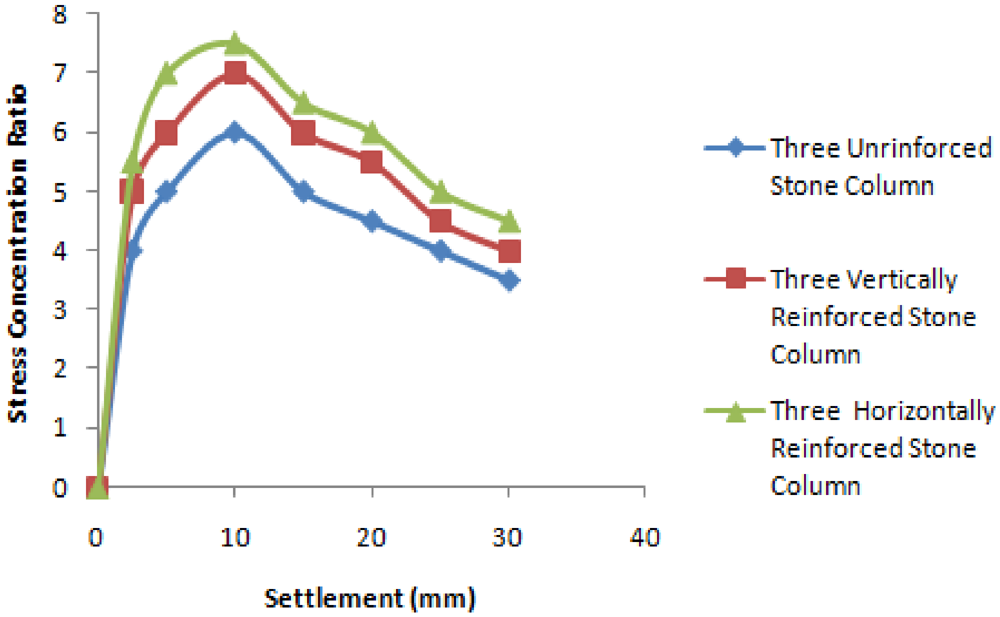


Figure 4.9 Variation of Stress concentration ratio versus settlement for 3 stone columns

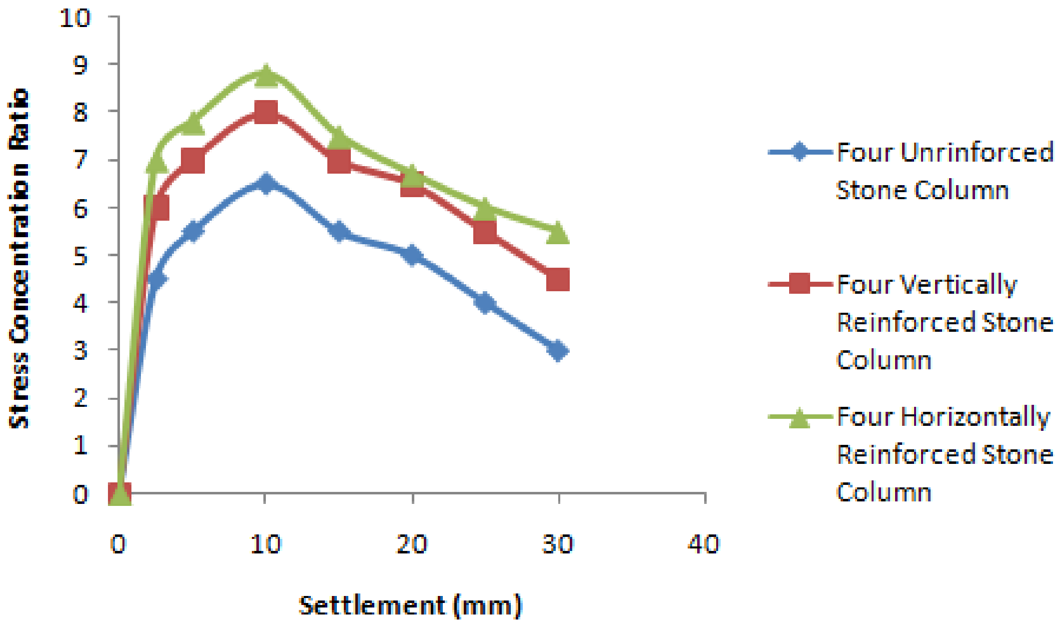


Figure 4.10 Variation of Stress concentration ratio versus settlement for 4stone columns

This implies that during group action, with higher lateral stiffness of composite ground, higher load at equal settlement is taken up by the stone columns which are reinforced horizontally as compared to vertically reinforced stone columns. It further reveals that higher restrainment to bulging at regular interfaces developed due to introduction of geotextile discs is more that the circumferential restrainment rendered by the vertical confinement.

4.6 DEFORMATIONAL BEHAVIOUR OF SINGLE & GROUP STONE COLUMNS

4.6.1 EXHUMATION OF THE COLUMNS

The columns were exhumed in order to examine the failure history of the columns. It was expected that the columns would bulge at the waist, based on previous analysis. [10].The following is a summary of the exhumation process and the failure pattern that resulted:

a) Exhumation of the Unreinforced Stone Column

The partial exhumation has been carried out as the whole exhumation would lead to probable fragmentation of the unreinforced column. To evaluate the failure sequence, the perpendicular distance between the neck and the bottom of the column to the wall is measured as shown in Figure 4.11.

Figure 4.11 (a) Excavated unreinforced stone columns

Figure 4.11 (b) Intervals of perpendicularity between the column's collar and the model tank's Wall.

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Figure 4.11 (c) The distance seen between the end of the column and the model tank's wall.

As seen in Figure 4.11, the perpendicular distance between the column's neck and the tank wall is 14cm, while the distance between the column's bottom and the tank wall is 15cm.The prior gap is less than the latter, suggesting that the column has collapsed due to neck bulging.

(b) Exhumation the Vertically Encased Stone Column

In this situation, the vertical encased was supposed to serve as a sac, stopping the column from disintegrating; column exhumation as shown in Figure 4.12 began at the top and continued down into the model tank. The detected failure trend was then photographed. Here also, the failure at the neck due to bulging was reported.

Figure 4.12 (a) Excavated Single stone column with bulging at the neck

Figure 4.12 (b) Excavated 3 stone columns with bulging at neck

Figure 4.12 (c) 4 stone columns with bulging necks were excavated.

(c) Exhumation of the Horizontally Reinforced Stone Columns

Around the same way as the vertically encased column was excavated, the horizontally reinforced stone column was also excavated. The column finally broke, and the breakdown pattern was deduced from the impression left on the surrounding soil. The same is depicted in Figure 4.13 (a) –(b).

Figure 4.13 (a) Top view of the column during exhumation

Figure 4.13 (b) Side view of the column after exhumation depicting bulging at neck

Thus, both singular and group of stone columns have been noticed to experience bulging at the neck. Wood et al. observed the same failure pattern, hypothesizing that as the area replacement ratio is increased, the bulging of columns increased in the upper zone of the soil layers, causing load to be transferred to a deeper depth. Until a bulging collapse, the floating form stone columns will fail in end bearing in the weak underlying layer. In general, however, for subsurface conditions, expansion is the most common regulating failure mechanism [199].

CHAPTER-5
FINITE ELEMENT ANALYSIS RESULTS AND DISCUSSIONS

5.1 GENERAL

This chapter presents the results of finite element modelling and the comparison of results of finite element analysis with experimental results. The main focus of this chapter is to bring out the mechanism with which the stone column behaviour installed in weak soil responds to the load and also to find the reason for the improvement in the behaviour of stone column because of encasement (i.e. reinforcing the column

5.2 NUMERICAL MODELING RESULTS

The results of numerical modelling using Finite Elemental Modelling were compared to the vertical load-settlement profile obtained from experimental data (FEM). This was achieved using the PLAXIS 3D programme, which was tested by a model test using a tank with dimensions of 30 cm * 30 cm * 55 cm. A 40 cm diameter, 30 cm long column was inserted into the tank's middle and is loaded with a 20 cm * 20-cm square plate.

Since the stress conditions in a single and groups of 3 or 4 stone columns are uniform, a finite element analysis was performed. To replicate the collapse of sand and stone columns, the Mohr – Coulomb soil relaxation model was used, with the reinforcement medium being linear elastic. The soil and stone columns were discretized using 15-noded triangular parts, and the tank's bottom was operated in both x and z directions. The FE model is analysed for displacement increments in the same manner as experimental experiments are. The results from numerical modelling have been found to be in close agreement with those from experimental procedures. The laboratory assembly process of stone columns isn't really taken into consideration in the current FE simulation, and instead is modelled as embedded parts.

5.2.1 LOAD–SETTLEMENT BEHAVIOR OF SINGLE STONE COLUMN

Figure 5.1 illustrates the load settlement actions of unreinforced and reinforced (vertical and horizontal) single stone columns obtained from Plaxis 3D. The figure indicates that reinforced columns have a greater load carrying capability than unreinforced ones. Furthermore, the response for vertical and horizontal reinforced groups is distinct. Since vertical reinforcing enhances the containment effect, this can be explained. As a consequence, hoop stresses produced inside the stone column when it is loaded are not dissipated and are reduced. This leads to a rise in lateral pressure from the underlying soil, which stops the stone column from bulging and guarantees optimal load distribution to the rim. The interface tension is mobilised when the column deforms in the case of lateral strengthening in the shape of circular discs. Furthermore, circular discs spaced at 3 cm intervals are used to segment the overall length of stone columns into 10 mm sections. The risk of column bulging is minimised as a result of the lower aspect ratio, and significant load transfer is achieved. The community of stone columns however, settles in both cases due to a lack of functional end-bearing.

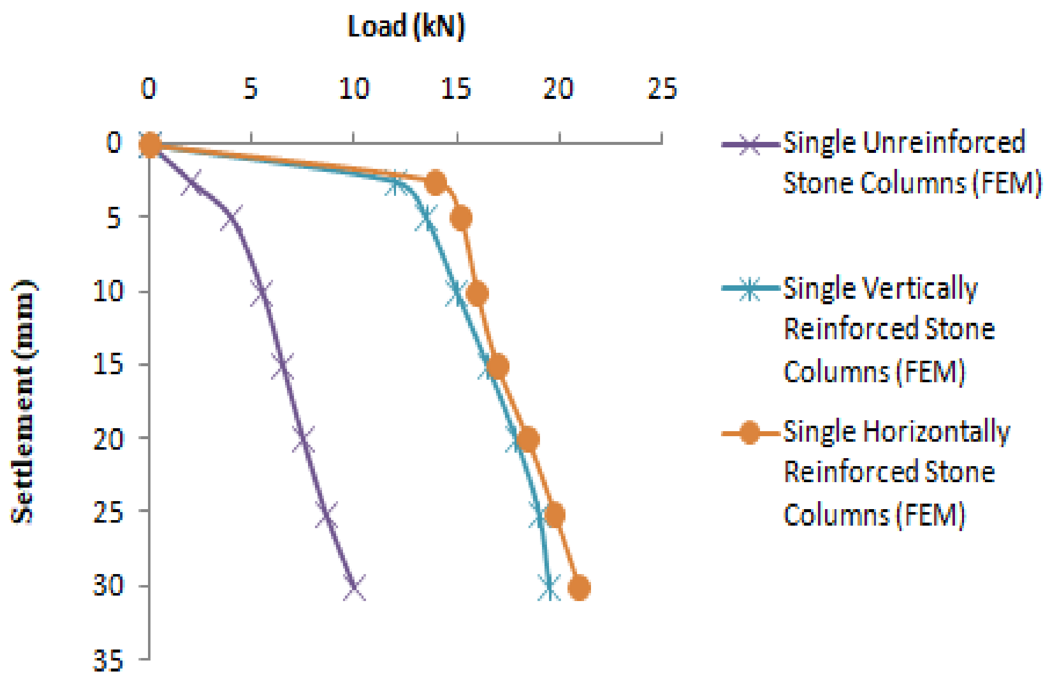


Figure 5.1 Load – settlement variation for Single stone column

5.2.2 LOAD SETTLEMENT BEHAVIOR OF GROUP STONE COLUMNS

5.2.2.1 LOAD SETTLEMENT BEHAVIOR OF THREE STONE COLUMNS

Figure 5.2 illustrates the load settlement intervention of three stone columns, for unreinforced and strengthened (vertically and horizontally). The figure reveals that reinforced stone columns can handle more load than unreinforced columns. Furthermore, the responses of both the vertical and horizontal strengthened classes are similar. Since vertical reinforcing enhances the containment effect, this can be explained. As a consequence, hoop stresses produced inside the stone column when it is loaded are not dissipated and are reduced. A group of three stone columns settles in the absence of any possible end-bearing in both unreinforced and strengthened conditions in the same manner as a single stone column settles in the vicinity of any available end-bearing.

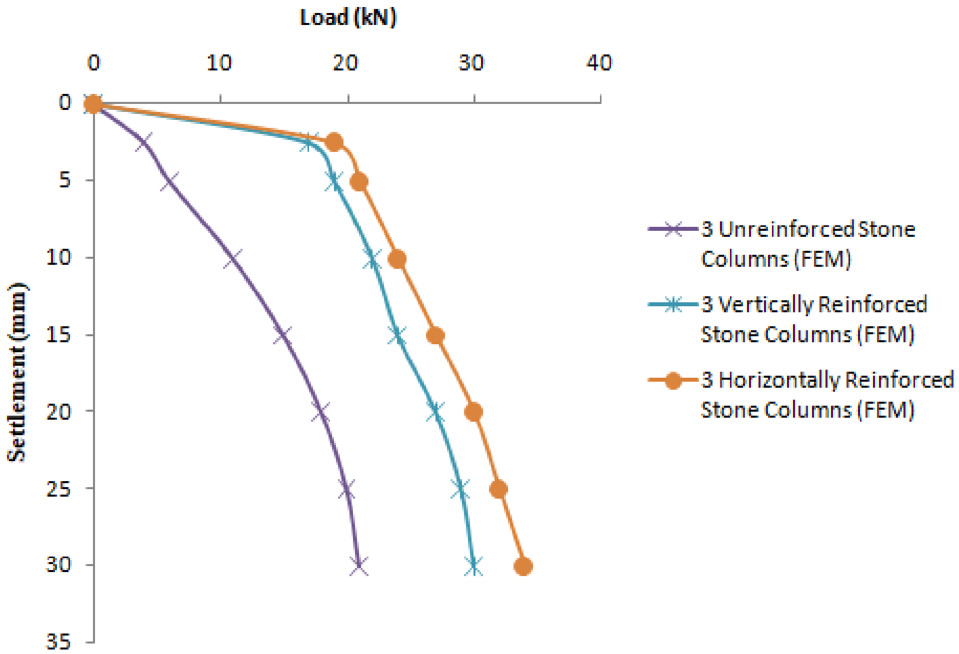


Figure 5.2 Variation in load settlement for three stone columns

5.2.2.2 LOAD–SETTLEMENT BEHAVIOR OF FOUR STONE COLUMNS

Figure 5.3 illustrates the load settlement actions for group of four columns. The same nature of vertical and horizontal reinforced stone columns is found once more. Furthermore, reinforced stone columns outperform unreinforced stone columns. The behaviour of vertically and horizontally reinforced columns can be connected to hoop stress resistance and interface friction mobilisation, respectively. The aggregate–geotextile interface is supposed to be considerably higher than the soil–geotextile interface due to the high angular nature of aggregates and hoop stresses caused by bulging of column. As a result, the mobilised interface tension between soil and geotextile plays a part in the settlement collapse of vertically encased floating stone columns.

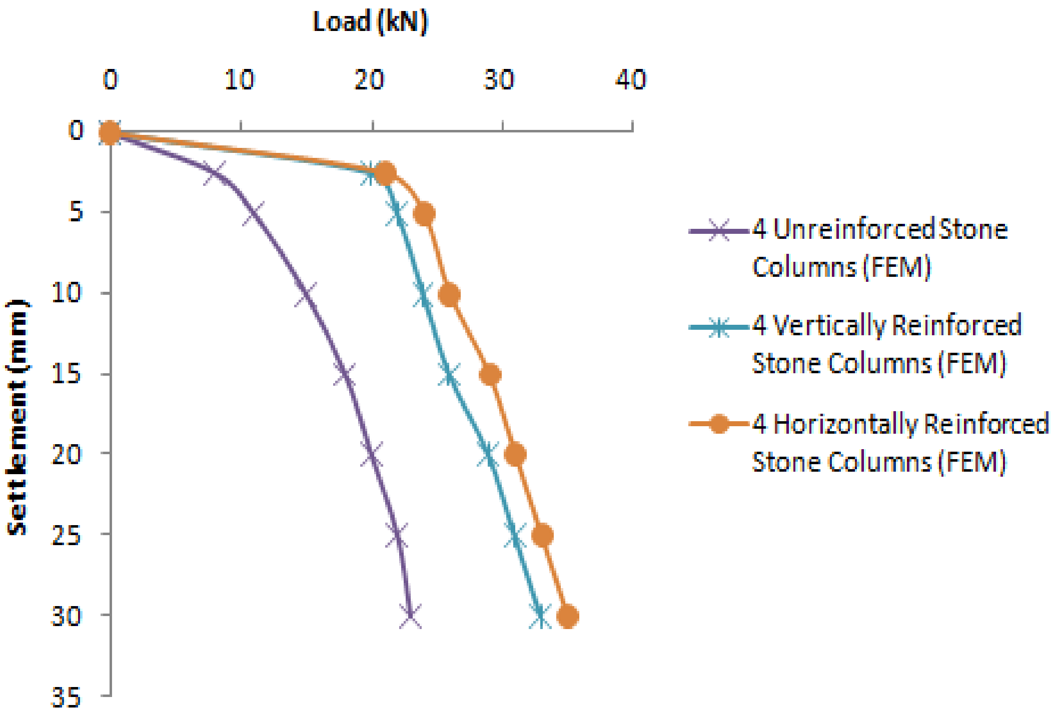
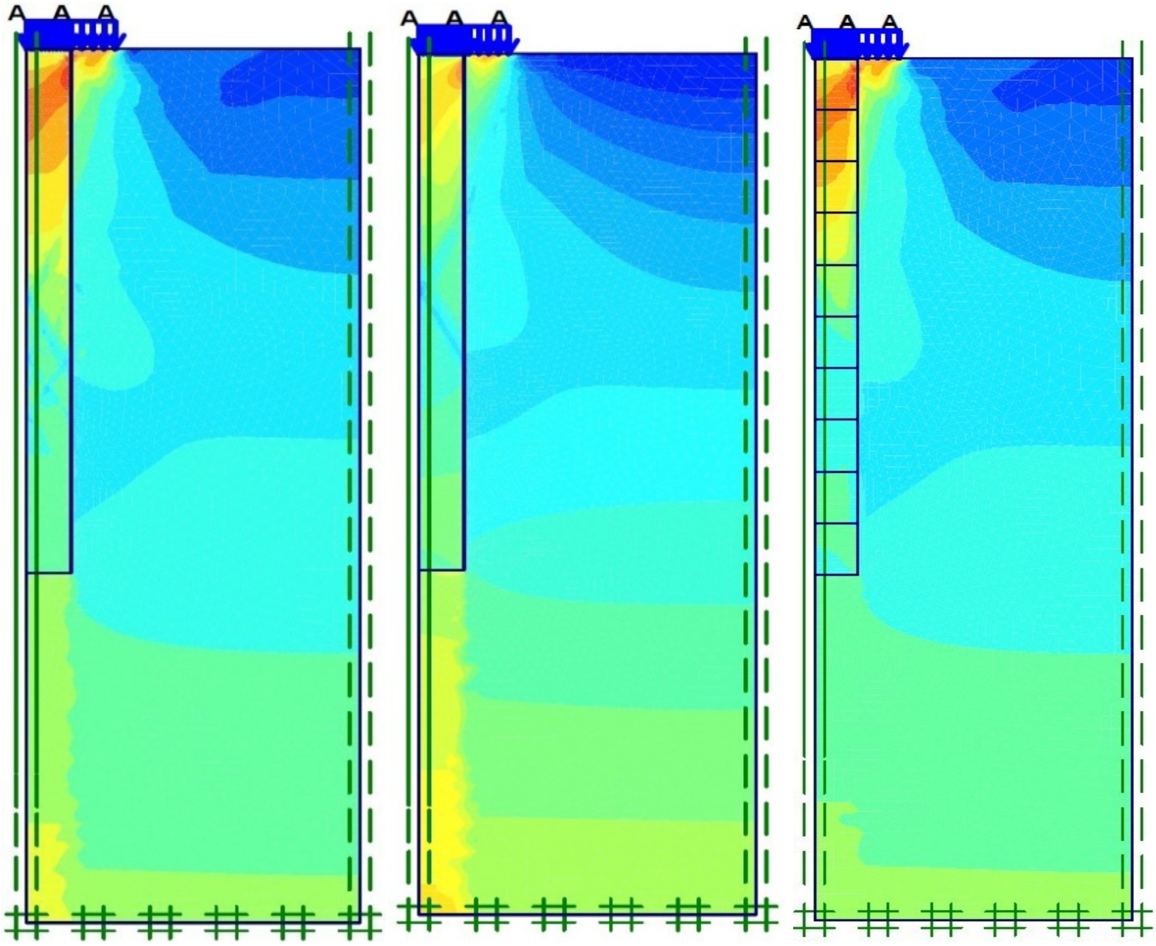


Figure 5.3 Variation in load settlement for group of four columns

5.3 DEFORMATIONAL BEHAVIOUR OF STONE COLUMNS

5.3.1 PLAXIS 2-D

The simulated model was subjected to a uniformly distributed load. Figure 5.4 illustrates the loading. The final loads that are achieved during testing of stone columns at 60mm settlement, namely 9.80 kN for unreinforced, 11.90 kN for vertically encased, and 15.80 kN for horizontally reinforced columns. When the stresses produced in all three cases are contrasted, the red portion appears to be the most conspicuous in the unreinforced column and the least conspicuous in the vertically reinforced column. This indicates that the pressures in the unreinforced column are higher than those in the reinforced columns.



Unreinforced Vertically reinforced Horizontally reinforced

(a) (b) (c)

Figure 5.4 Deformed mesh for Single stone columns

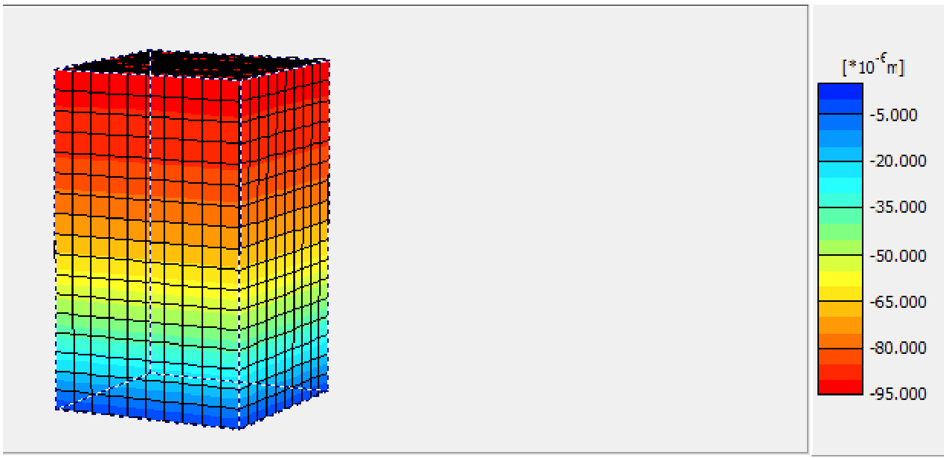
5.3.2 PLAXIS 3-D

The squared shaped tank was modelled for three and four stone columns in the current work to reduce the geometrical limitations associated with PLAXIS. The vertical load applied in terms of specified displacement, taking into account the material's rigid behaviour. In the x, y and z ways, the model's bottom boundaries were constrained. Vertical displacements of stone columns were only allowed in the z-direction to allow vertical displacements of stone columns. The soil's modulus of elasticity, which was used in numerical modeling, was determined using a consolidation laboratory test with pressures ranging from 100 to 200 kPa

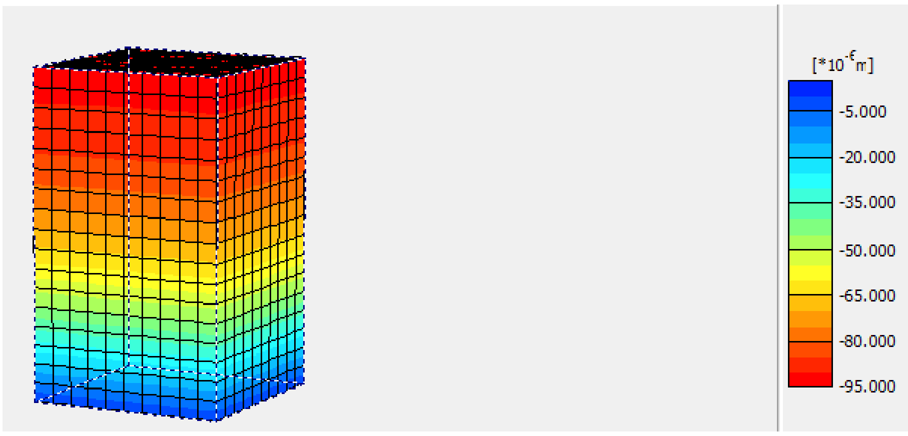
[41]. For geotextile modelling, the Plaxis code's Geogrid element was used, and the parameter of geotextile axial stiffness (EA) was taken into account to accurately model the behaviour. Using the Mohr – Coulomb soil model, the geotextile is modelled as elastic materials and the soil is modelled as elasto – plastic. The properties of the soil used were shown to be identical to those found in the lab. It is concluded that the column deforms mainly due to bulging with no shear, and that the soil-column interface is dependent on the method of implementation, whose shear properties can vary considerably. As a result, no interface region was used for unreinforced columns in this situation.

However, an interface reduction ratio (R_{inter}) value has been used to model the geotextile–soil interface for vertically reinforced columns, while an interface friction value derived from DST was used to model the aggregate–geotextile interface for horizontal reinforcement. Fine meshing around the stone columns and coarser meshing in the radial direction is used to mesh the whole model in three dimensions. In-situ stresses were measured using Jacky's formula before load was applied (1- sin). To load stone columns, a vertically pre-defined displacement was used, which was then evaluated using a plastic scale. The failure load is measured at different displacements before the predetermined displacement is achieved.

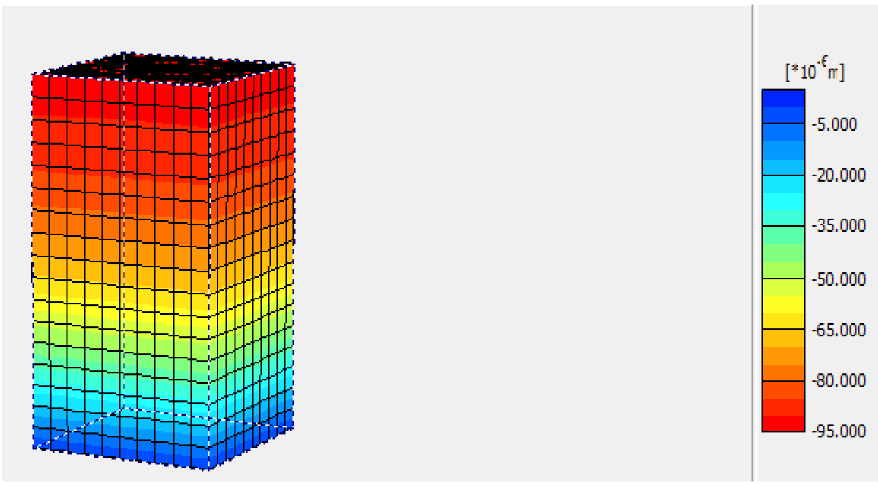
The deformed shapes of single, group of three or four stone columns after loading are shown in **Figures 5.5, 5.6, and 5.7.** According to a systematic observation, deformation occurs due to bulging and lateral deformation in single and group of 3 and 4 columns. Because vertical reinforcement has no bending stiffness, it can help to reduce the lateral deformation of the columns.



(a). Un-reinforced Single Column

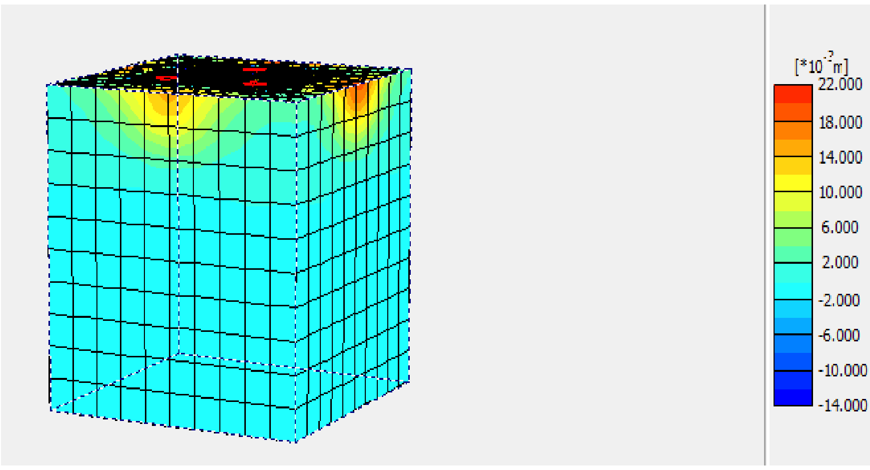


(b). Vertically encasement Single Column

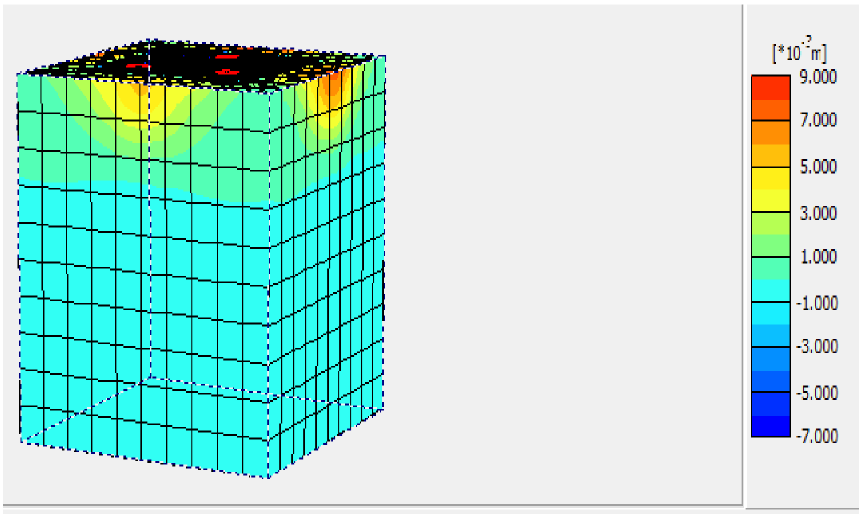


(c). Horizontally reinforced Single Column

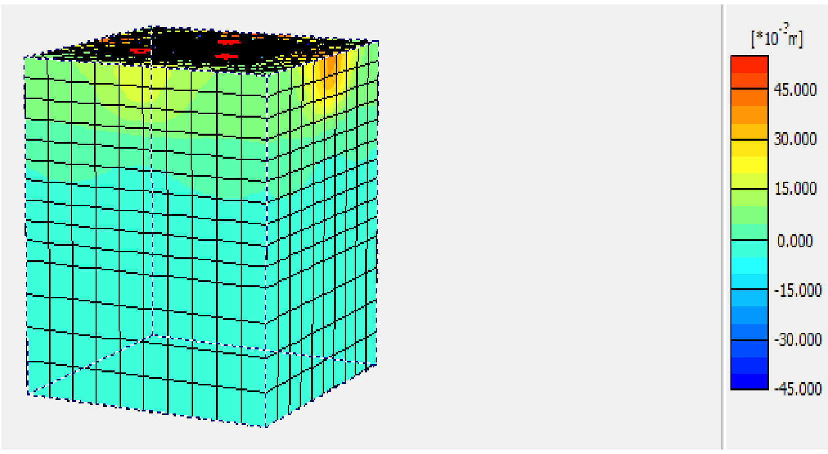
Figure 5.5 Single stone columns with deformed mesh



(a). Unreinforced 3 Columns

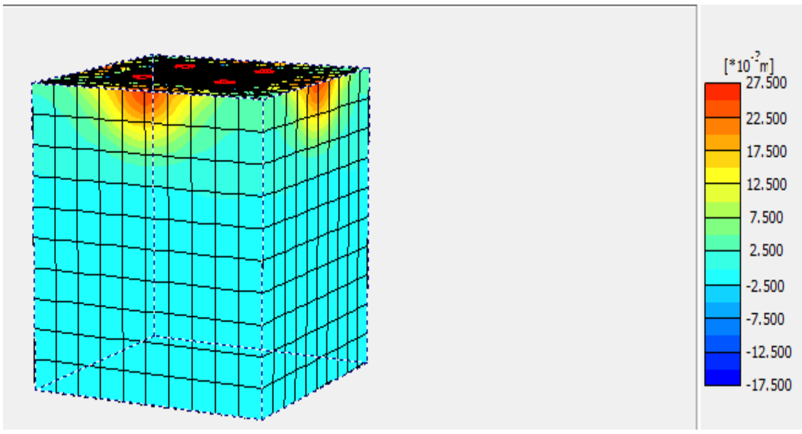


(b). Vertically encased 3 Columns

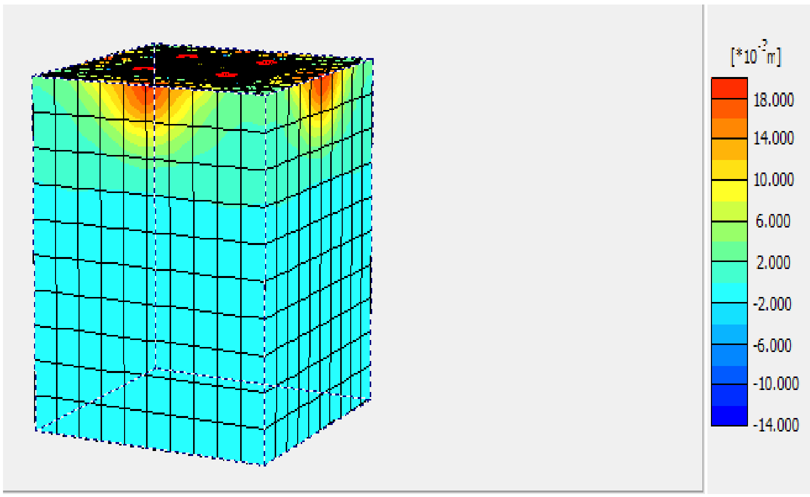


(c). Horizontally reinforced 3 Columns

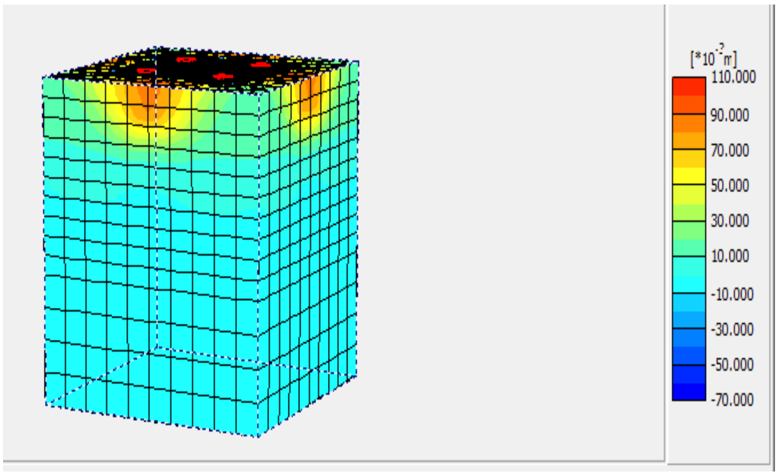
Figure 5.6 Mesh that has been deformed for a group of three columns



(a). Un-reinforced 4 Columns



(b). Vertically encased 4 Columns



(c). Horizontally reinforced 4 Columns

Figure 5.7 Mesh that has been deformed for a group of four stone columns

5.4 VALIDATION OF EXPERIMENTAL FINDINGS & COMPUTATIONAL SIMULATION IN THREE DIMENSIONS

Figures 5.8, 5.9, and 5.10 displays a comparison of load bearing capability with settlement for both experimental and finite element analysis under both vertical and horizontal reinforcement arrangements. The load expected by model testing for a group of three unreinforced stones is virtually equivalent in both cases. However, the finite element approach has shown that horizontally reinforced stone columns have a greater load carrying capacity than vertically reinforced stone columns. These observations are consistent with those obtained by experiments. In contrast to unreinforced columns, load carrying capacity of columns increases with vertical containment or horizontal reinforcement, according to estimates. This is due to the longitudinal reinforcing that avoids the lateral enfolding of aggregates into the neighboring soft soil, which restricts column bulging. In the case of horizontal reinforcement due to lateral displacement of aggregates, the shearing resistance between the geotextile and aggregate interface is mobilized as load is applied. This interface mobilisation inhibits the extension of the columns.

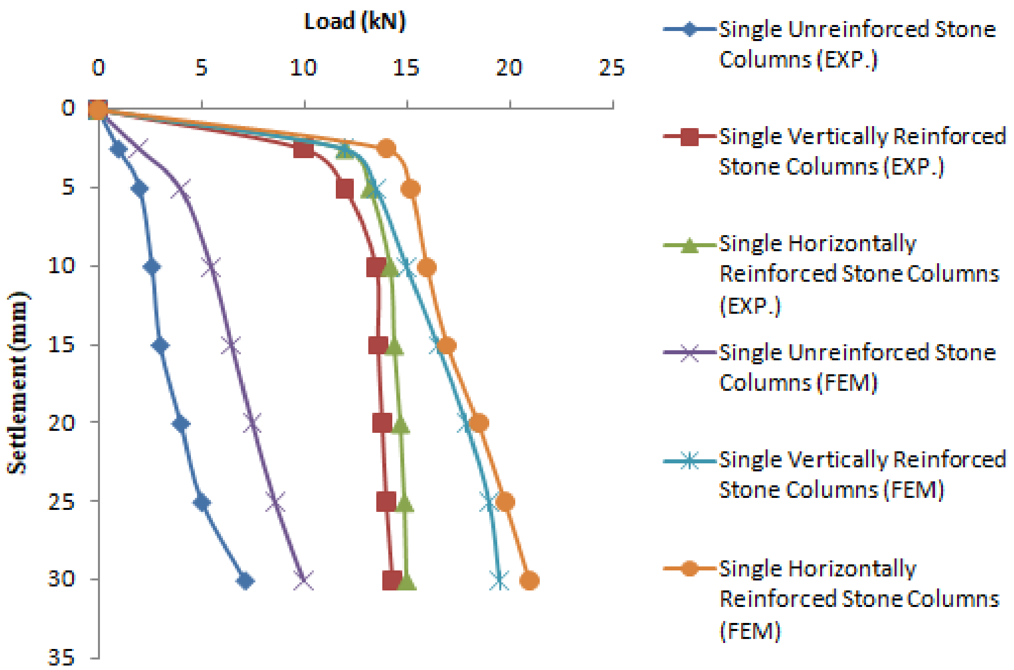


Figure 5.8 Variation in load settlement for single stone columns

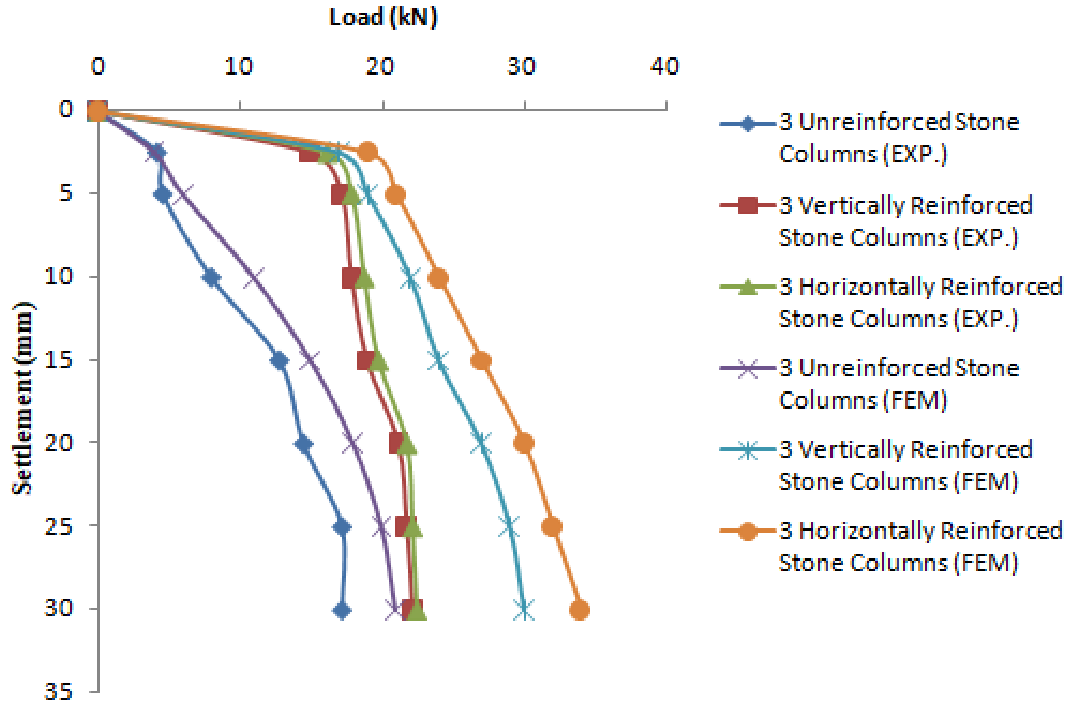


Figure 5.9 Variation in load settlement for three stone columns

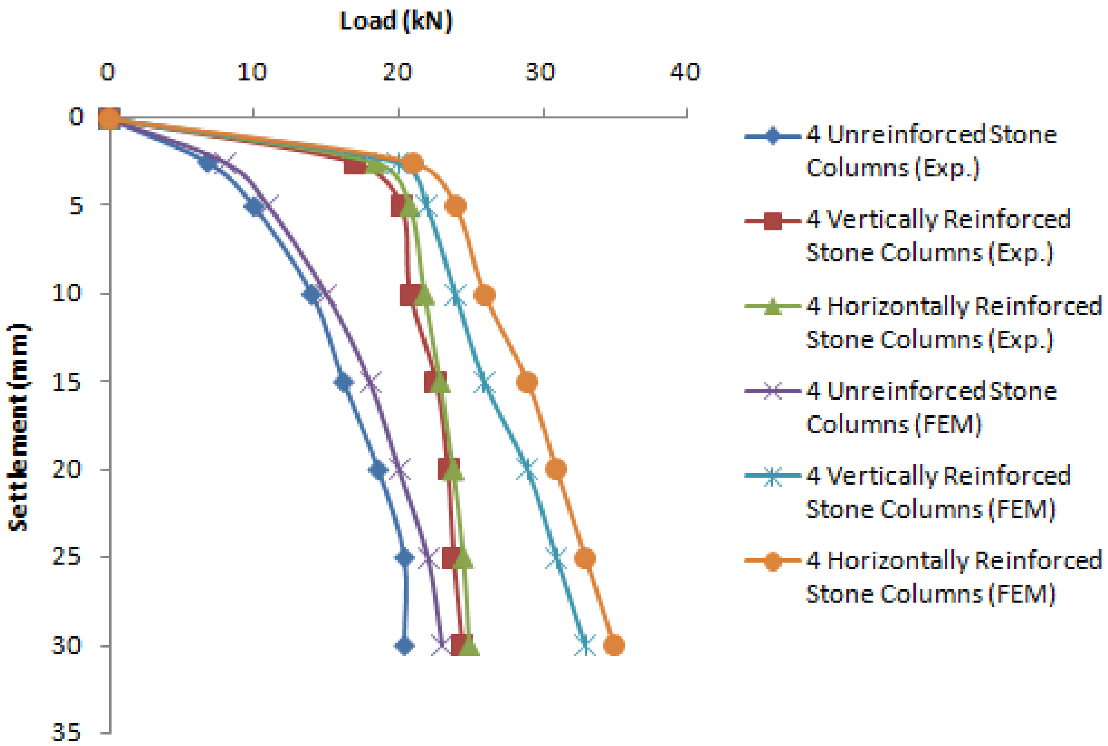


Figure 5.10 Variation in load settlement for four stone columns

Another notable difference between the experimental and numerical simulation results is that the numerical simulation results for single, three and four stone columns indicate higher levels of load capacity. The overestimation is due to the finite model not accounting for soil disruptions caused by stone column moulding and construction, and the stone columns were also treated as if they were buried in the soil prior to loading.

As a result, changes in soil properties that are used in experimentation are not allowed in computational simulation. The finite element represents the load bearing potential of soil after the preliminary shear strength parameters have been determined. With a standard deviation of less than 8.13 and a median variance of just 28.78 percent, Table 5.1 indicates a good correlation between experimental and FEM values.

Table 5.1 Unreinforced and reinforced stone columns' experimental and FEM values

Type of Reinforcement	Experimental Results	FEM Results	**Final Settlement (mm)**	**Standard Deviation**	Coefficient of Variance (COV) (%)
	End Load (kN)	**End Load (kN)**			
Single un-reinforced column	7.10	10.00	30.00	2.05	23.98
Single vertically encased column	14.30	19.50	30 .00	3.67	21.75
Single horizontally reinforced column	15.00	21.00	30.00	4.24	23.57
Group of 3 unreinforced columns	17.20	21.00	30.00	2.68	14.06
Group of 3 vertically encased columns	22.20	30.00	30.00	5.51	21.13
Group of 3 horizontally reinforced columns	22.50	34.00	30.00	8.13	28.78
Group of 4 unreinforced columns	20.40	23.00	30.00	1.83	8.47
Group of 4 vertically encased columns	24.40	33.00	30.00	6.08	21.18
Group of 4 horizontally reinforced columns	24.90	35.00	30.00	7.14	23.84

5.5 PREDICTIONS BASED ON THEORIES

The design methods for predicting bearing strength of stone column stabilised bed are few and among the methods, the method based on combined resistances of stone column due to bulging and the bearing strength of clay around the column is widely used in design offices. The Indian Standards code of Practice (IS 15284 Part 1, 2003) [37] also recommends this method. From the experimental observations and numerical analyses of current research it is shown that the stone column bulges and the soil around the column provides lateral resistance and also shares the part of the load as a bearing support. The stress condition in the column is in passive state and the passive pressure coefficient is more or less equal to K_{pcol}. Thus the method recommended in IS 15284 Part 1 (2003) [37] is used to determine the bearing strength of stone column stabilised bed and compared with the experimental results of the present study

The maximum load that traditional stone columns changed sand surface can withstand was calculated using method mentioned in IS:15284 [Indian standard IS (2003)] [37]. The stone column that can withstand a load proportional to the amount of (1) the stone column's load carrying capacity due to the surrounding soil's resistance to lateral deformation due to axial load and the total load that the stone column can withstand. (2) Soil between columns provides protection. Since there was no surcharge, there was no effect on the loading power of the columns. The limiting axial stress v on the columns (i.e. the maximal pressure on the columns) is given by

$$\sigma_v = (\sigma_{ro} + 4c_u)Kp_{col} \tag{5.1}$$

Where σ_{ro} represents the column's original effective radial measured at a depth of two times its diameter. $Kp_{col} = \tan^2(45 + \phi/2)$ Where ϕ is the friction angle of aggregates. The calculation was used to measure the amount of support given by the sand in contact with the loading plate, $q_{safe} = c_u + N_c / FS$, here the N_c was taken equal to 5.14 since the frictional angle was zero and FS i.e. the safety factor is assumed to be 2.5.

The maximum load corresponding to a settlement of 30 mm, as calculated from the trails for different diameters of conventional stone columns, was measured and compared to the limiting load that can be applied to the stone columns. The results' precision (as shown by repeated experiments) indicates that the soil bed and stone columns in the model studies are uniform. The data comparison shows less than 5% variation in the results from different tests.

The vertical pressure on the encased stone columns was predicted by analytical method based on hoop tension theory. The extension of the columns was discovered to occur mainly in the upper part of the column across a height of around 4 times the column diameter. This radial bulging induces vertical strains in the columns that can be considered as the effect of compression in the top zone of the column. Assume that there was no volume change during the deformation of the stone columns. Thus the circumferential strain i.e. hoop strain ϵ_c in the reinforcement material can be related to the vertical or axial strain, ϵ_a using the relation:

$$\varepsilon_c = \frac{1-\sqrt{1-\varepsilon_a}}{\sqrt{1-\varepsilon_a}} \tag{5.2}$$

The vertical strain in the stone column was determined by dividing the estimated surface displacements by the column's height, which was nearly four times its diameter. The load strain profile from the stitched geosynthetic width tension experiments was used to quantify the hoop compression pc, which was then used to evaluate the hoop compression pc using the equation.

$$p_c = \frac{2T}{d} \tag{5.3}$$

The column diameter is denoted by d. The hoop compression increases as the diameter of the stone column gets reduced. The vertical tension on vertically encased stone columns can be measured using the following formula:

$$\sigma_v = (\sigma_{ro} + 4c_u + p_c)Kp_{col} \tag{5.4}$$

In present study, c_u value for encased columns was zero and the vertical stress on the columns corresponding to 30 mm settlement was calculated in case of woven geotextile.

5.5.1 RECOMMENDATIONS FOR ENCASED STONE COLUMNS

The following guiding principles for geosynthetic encased stone columns were planned based on the findings of the current research study:

1. Depending on the pressure loading Po from the structure, stone columns with appropriate parameters such as diameter (d) and spacing (s) are considered. The strain on the unit cell is believed to be entirely borne by the stone column alone in the cell in a standard unit cell made up of stone columns and the surrounding soil.

   Load on SC= applied pressure p_o * Area of the unit cell, A.

   Where $A = \pi \times (0.525s)^2$ for triangular grid and $A = \pi \times (0.564s)^2$ for square grid

   As a consequence, the weight on the stone column equals the load on the unit cell divided by the area of the stone column Ac.
2. Calculation is used to measure the limiting stress on a typical stone column (1).
3. The equation yields the additional containment, pc that is necessary.

$$p_c = \frac{(p_o - \sigma_v)}{Kp_{col}} \tag{5.5}$$

4. The resulting hoop stress can be determined using the formula

$$T = \frac{p_c d}{2} \tag{5.6}$$

5. In the encasement, the hoop strain can be obtained from equation (2) using the value of ϵ_a which is calculated by

$$\varepsilon_a = \frac{\delta}{4d} \tag{5.7}$$

6. The discussion allows the selection of suitable geosynthetic that can result in tensile strength, T at the strain level ϵ_c.

Another significant difference between the experimental and theoretical simulation results is that the numerical simulation results for two, three, and four stone columns suggest higher load capacities as shown in Table 5.2. The results show the good agreement between theoretical and experimental value having maximum value of coefficient of variance 25.80%.

Table 5.2 Experimental and Theoretical values

Number of column Arrangement	Reinforcement Orientation	Model Testing	Theoretical	Final Settlement (mm)	Coefficient of Variance (COV) (%)
		Load (kN)	Load capacity Equation given by IS 15284 Part1 (kN)		
Single Stone Column	Unreinforced	7.1	10.2	30	24
	Vertically	14.3	20	30	22
	Horizontally	15	----	30	----
Three Stone Columns	Unreinforced	17.2	22.2	30	16.67
	Vertically	22.2	17.31	30	25.80
	Horizontally	22.5	----	30	----
Four Stone Columns	Unreinforced	20.4	23.4	30	16.67
	Vertically	24.4	19.73	30	25.80
	Horizontally	24.9	----	30	----

CHAPTER-6

CONCLUSIONS

6.1 GENERAL

In this chapter a summary of the contents of the investigation and salient conclusions derived from testing, finite element and validations using codal provisions are presented. The chapter also covers up suggestions for future research work.

6.2 CONCLUSIONS

In this research, a total of 33 model tests comprising of unreinforced single and group of three/four stone columns, vertically reinforced single and group of three/four stone columns, horizontally reinforced single and group of three/four stone columns are investigated under compressive loading in cohesionless medium. The model stone columns have a diameter of 40 mm and length of 300 mm. The results of model testing has been validated using results from finite element axisymmetric Plaxis 2D for single unreinforced and reinforced stone columns. Similarly, validation of testing results for three and four stone column group for both unreinforced and reinforced stone columns has also been carried out using three – dimensional finite element code Plaxis 3D. Further, results from testing are also validated using theoretical relationships used by field engineers as given through codal provisions. Based on the results and subsequent discussions, following conclusions have been reached:

1. Unreinforced stone column provided a 54.92% increment in bearing capacity in comparison with only sand bed, Vertically encased stone columns provides 47.33% increase in the bearing capacity whereas horizontal reinforcement provides an increment of 49.65 % in the bearing capacity, as can be observed from the model testing results in comparison with Unreinforced stone columns.
2. Vertically encased 3 stone columns provided a 76.44% increment in bearing capacity whereas horizontal reinforcement 3 stone columns provides an increment of 77.47% in

the bearing capacity, as can be observed from the model testing results in comparison with Unreinforced 3 stone columns.

3. Vertically encased 4 stone columns provided a 81.92% increment in bearing capacity whereas horizontal reinforcement 4 stone columns provides an increment of 83.60% in the bearing capacity, as can be observed from the model testing results in comparison with Unreinforced 3 stone columns.
4. Geosynthetic encasement adds lateral confinement to stone columns against bulging by mobilizing hoop pressures, while friction horizontal reinforcing avoids bulging. Horizontal soil reinforcement outperformed encased columns in terms of load bearing capacity.
5. Compared to unreinforced stone columns, geotextile strengthening necessitates the mobilisation of higher hoop stresses and the accomplishment of greater load bearing strength all at the same time. Furthermore, horizontal reinforcement increases shearing tolerance among aggregate–geotextile–aggregate during bulging, resulting in higher load potential than vertically encased floating columns.
6. Bulging loss occurs between d and 2d from the top of the column, where d is the stone column's diameter. The lateral bulging of unreinforced stone columns is significantly higher than that of reinforced stone columns. As a result, reinforced stone columns will support greater loads than unreinforced stone columns.
7. The stiffness of the encasement material influences the load-settlement behavior of reinforced columns. Thus it can be concluded that greater the stiffness of the encasement/confining material, lesser is the bulging.
8. From the variation of load ratios and stress concentration ratios, it can be concluded that mobilization of shearing stress due to restrained bulging as obtained by inclusion of Geotextile within the stone column is significantly more beneficial as compared to vertically encasing the floating stone column. Also reinforcing the stone columns leads to increased load transfer from top of the column to its bottom and also reduces transfer of load to the surrounding soil.
9. For horizontally reinforced stone columns, stress concentration is independent of the number of stone columns when increased from three to four. Thus, it can be concluded that mobilization of shearing stress within the stone column body is significant in restrainent of bulging in comparison to shearing stresses mobilized due to stone column settlement.

10. A 3D FE study shows that encased (vertically and horizontally) columns collapse due to restricted bulging. According to both model tests and FE review, there is an appropriate average deviation of around 28% in load bearing capability to corresponding settlement, which is a reasonable agreement. Moreover it can be concluded that without modelling the installation effect in FE analysis, failure modes obtained are comparable to testing failure modes but a higher load carrying capacity than experimental values is obtained. Therefore, for detailed simulation of stone column behavior in finite element, a method for installing stone columns is suggested.
11. From the validation of model testing results with available codal provisions, it can be concluded that empirical relationship as reported overestimate the load carrying capacity for a particular settlement for both unreinforced and vertically encased stone columns due to absence of consideration for installation effect in the reported relationships. Moreover, it can also be suggested to have thorough model and large scale testing results for development of empirical relationship for horizontally reinforced stone columns which are virtually non – existent currently.

6.3 SCOPE OF FURTHER WORK

The current state-of-the-art leaves certain identifiable gaps that are of significant importance for a better understanding of behavior of stone column reinforced ground and its prediction towards response under load. There is scope for further research on the following topics:

1. The behaviour of reinforced stone columns in layered soil domain can be studied by the future researchers. Keeping in view the actual site conditions, sandwiching of a sand layer between clay layers or vice versa is easily encountered for which stone columns are found to perform satisfactorily. However, use of reinforced stone columns in such situation has been rarely investigated or quantified. In such scenario, both the drainage of the surrounding soil as well as the interface friction between soil – reinforcement interface must be accounted. Also, the different response of installation for both cohesive and cohesionless soil will be affecting the overall failure mode and load – settlement characteristics of stone columns. Moreover, the length of encasement, location of horizontal discs, its effects for soil of varying stiffness will be a significant contribution to the available literature which is focused on primarily soft cohesive soil and purely cohesionless soils only.

2. Another area of study for stone columns can be assessing the feasibility of using modified aggregates as filler material for stone columns. Material such as crumbed rubber infused stone columns, aggregates from C&D wastes, or other waste material used for backfilling can be evaluated as potential stone column material.
3. Since a combination of vertical and horizontal reinforcement was not evaluated in this study, the bearing ability of vertically encased and horizontally reinforced stone columns is comparable. Furthermore, there are almost no field experiments on mixed vertical and horizontal reinforcement. As a result, prospective experiments may use small-scale or field-scale testing to study stone columns that are supported both vertically and horizontally.
4. For assessing load capacity and settlement of individual reinforced and unreinforced columns, more sophisticated instrumentation is required. For assessing bulging and further evaluating reinforcing operation, strains formed along stone columns should be recorded and evaluated in vertical and horizontal stone column reinforcement.
5. Calculating differences in ground pressure through stone columns reveals valuable details regarding the failure modes of respective reinforced stone columns.
6. Detailed finite element numerical study with consideration of installation effects for both single and group of stone columns. Also, more comprehensive testing at both laboratory and full scale to see for feasibility of installation of horizontally reinforced stone columns so as to develop relationship which can be directly used by the field engineers without cumbersome preliminary site investigation. These will significantly contribute to the Indian standards used top reinforced stone column designing which have inadequate information or guidelines regarding horizontally reinforced stone columns.

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