EXPERIMENTAL STUDIES ON SINGLE STONE COLUMNS WITH TIRE-DERIVED AGGREGATE (TDA)

by

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To my beloved my wife who have always been there to support and encourage me.
To God Almighty for all his mercies.
This thesis is dedicated to you.

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Abstract

Given the increasing land prices, the need for construction development on unsuitable lands (from a geotechnical perspective), environmental restrictions on the use of quality lands, etc., there is an ever-increasing need for the development of new methods for improving substandard lands. Choosing the right method for improvement in a specific site is of utmost importance. Depending on the project conditions, including the location of implementation, adjacent structures, available technologies, and the project's economy, one can benefit from one of the soil improvement methods. One of the applications of deep foundations is when the soil at shallow depths does not have sufficient bearing capacity, and the loads must be transferred to the harder layers below. Stone columns can be referred to as an ideal option for improving loose layered soils, clays, and loose silty sands. Considering the implementation conditions, the stone column method is more known as an empirical method and requires more extensive research to predict the ultimate strength of the soil reinforced by this method. Factors such as column length, column diameter, gradation of column materials, protective coating on the column, and the arrangement of columns when employing a group of columns should be considered, and different results can be obtained by changing any of these factors. Therefore, the need for further research in this field is felt.

This document focuses on the study and modeling of stone columns covered with a protective layer, featuring gravel mattress on the columns, as well as the utilization of tire-derived aggregate (TDA). The study has explored the impact of reinforcement on the load-bearing capacity of footings by examining encased stone columns. It is hoped that through acquiring the necessary expertise in this field and conducting extensive research, new solutions or at least ones that complement previous studies in this area can be provided.

LIST OF ABBREVIATIONS AND SYMBOLS USED

ABBREVIATIONS

ASTM	American Society for Testing and Materials
DAQ	Data Acquisition System
FDA	Finite Difference Analysis
FEA	Finite Element Analysis
HDD	Horizontal Directional Drilling
LP	Linear Potentiometer
SF	Stiffness Factor
TDA	Tire-Derived Aggregate
UTM	Universal Testing Machine

SYMBOLS USED

As	Cross-sectional area of the stone column
B	Width of the loaded area
C	Undrained shear strength of clay soil
Cc	Compression coefficient
W	Equivalent width of a row of stone columns
Df	Depth of foundation embedment
Fe and Fq	Dimensionless load-bearing capacity parameters
H	Thickness of the soil layer
K	Lateral earth pressure coefficient due to surcharge
N	Stress concentration ratio
Ko	At-rest lateral earth pressure coefficient
Q	Applied stress
Z	Distance from the stone column tip to the ground surface
E	Modulus of elasticity
aS	Ratio of the improved area
dS	Diameter of the stone column
Kas	Active lateral earth pressure coefficient of the stone column
e0	Initial porosity ratio
Kpe	Passive lateral earth pressure coefficient of soil
qult	Bearing capacity
Θ	Angle of the vertical plane of slip in each stone column
M	Poisson's ratio
cμ	Stress reduction coefficient in clay
Σ	Vertical stress
σro	Initial radial stress in the stone column
σz	Amount of stress added due to surcharge at depth Z
Sφ	Internal friction angle of stone column materials
Ψ	Angle between the failure plane and foundation
NC, Nq, and Nγ	Dimensionless load-bearing capacity parameters dependent on soil and stone column material characteristics and the ratio of the improved area
qS	Bearing capacity of soft soil
γs and γc	Specific weight of stone column materials and clay soil
T	shear stress of soil improved with stone columns

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Chapter One: Introduction

2.1 MOTIVATION

With the increasing land prices, the need for construction development on unsuitable lands (from a geotechnical perspective), environmental restrictions on the use of prime lands, etc., there is an ever-growing need for developing new methods for improving substandard lands. Choosing an appropriate method for soil improvement at a specific site is of paramount importance. Depending on the project conditions, including the location, nearby structures, available technologies, and project economics, one of the soil improvement methods can be utilized. Deep foundation applications are used when the soil at shallow depths lacks sufficient bearing capacity, and loads need to be transferred to the harder layers below. Deep foundations can be divided into three categories:

- Conventional slender systems or piles

- Bulky systems including deep holes and caissons

- Deep foundations stabilized with columnar elements like stone columns and deep mixing method

Stone columns can be considered an ideal option for improving soft layered soils, clay, and loose silty sands. By implementing stone columns, we can achieve an adequate level of shear strength, slope stability, allowable settlement amount, and safety factor against liquefaction. Applications of stone columns include reinforcing the foundation of embankments or large structures, bases of highways, etc. Given the execution conditions, the stone column method is more recognized as an empirical approach, and extensive research is required to predict the ultimate strength of soil reinforced with this method. In using stone columns, factors such as column length, column diameter, column material gradation, protective coating on the column, column arrangement during the use of column groups, etc., should be considered, and by altering any of these factors, different results can be obtained. Hence, the need for further research in this area is felt.

Implementing stone columns in very soft soils, due to the lack of all-around pressure, exacerbates the occurrence of lateral buckling (bulging phenomenon) in the stone column, leading to a decrease in the column's load-bearing capacity. Due to the identified performance weakness of unreinforced stone columns in the event of lateral expansion in soft soil and the occurrence of bulging buckling, geosynthetic coatings are considered one of the reinforcement methods for stone columns. This coating leads to an increase in the load-bearing capacity of the stone columns.

Used tires are special wastes and differ from other wastes in terms of chemical composition, size, and shape. Tires are made of polymeric materials that do not easily degrade in nature. Disposing of them poses numerous challenges, and burning them causes environmental pollution. Since waste producers in most of countries are not charged any fees, there are no facilities in waste disposal sites for volume reduction or shredding of tires. Consequently, they are dumped in their original form, occupying a significant volume of waste disposal sites. Therefore, efforts should be made to use these tires in civil engineering projects such as the construction of retaining walls, slope stabilization, and stone columns.

In this thesis, an attempt has been made to analyze, using a laboratory model, the amount of displacement and stress that can be borne by the columns in both confined and unconfined stone columns, with or without gravel mattress and the surrounding soil.

2.2 RESEARCH OBJECTIVES

The focus of this study has been on the following topics:

1. Investigating the use of single stone columns for soil improvement and assessing the increase in soil bearing capacity compared to the initial state.

2. Examining the use of Tire-Derived Aggregate (TDA) as alternative materials to sand in stone columns to enhance resistance and address environmental issues.

3. Investigating the optimal amount of Tire-Derived Aggregate (TDA)for use in stone columns.

2.3 RESEARCH SCOPE

Laboratory experiments were performed on a loading plate apparatus and a large-scale box with dimensions of 90*90 and a height of 120 centimeters were used. To fill the box, sandy soil with characteristics provided in subsequent chapters was used. The soil, with a specified and constant moisture content, was poured into 5-centimeter layers inside the box and compacted to a specified level using a special compaction hammer. Then, a casing was installed at the center of the box to create a single stone column. The soil inside the casing was emptied using an auger and filled to the desired level with either stone materials(aggregate) or a combination of stone materials and shredded worn-out tires (TDA). It was then compacted to the desired level using a special compaction hammer. Subsequently, a loading plate was placed on top of the stone column, and the load was applied by a hydraulic jack at a constant speed of one millimeter per minute until reaching a settlement of 5 centimeters. The results of the applied load and settlement were collected and presented in the final chapter.

2.4 THESIS LAYOUT

The structure of this thesis comprises five chapters:

Chapter 1 delves into the study's motivation, research objectives, and scope.

Chapter 2 explores soil improvement techniques, various methods, and similar studies on stone columns. It also examines the factors involved in selecting this method for soil improvement, as well as the use of geosynthetics and tire-derived aggregates in stone columns.

Chapter 3 describes the experimental investigation; starting with the Material and Substance Studies, continued by preparation of sand bed and creating stone columns, and finalized by an explanation of the test method.

Chapter 4 delves into examining the results obtained from the experiments and comparing the results obtained under different conditions for stone columns. It also discusses the reasons behind the results obtained regarding load-bearing capacity and settlement.

Chapter 5 offers a concise conclusion and recommendations for future research advancements focused on utilizing TDA in stone columns.

CHAPTER 2: BACKGROUND AND Technical Literature

3.1 Introduction

Generally, various soil improvement methods are employed in different regions with the objectives of improving soil quality, remediation, stabilizing excavations, controlling groundwater, and pollution control. Before the construction of the desired structures, soil characteristics such as compressibility and shear strength must be evaluated, and based on these and considering the loads and soil type, the appropriate improvement method should be selected. After improvement, suitable shallow foundations can be used. Although the use of unreinforced stone columns in soft soil deposits has advantages for foundations placed on them, they are generally suitable for soils with undrained shear strengths of about 5 to 10 kPa and may not provide sufficient lateral support to prevent excessive radial expansion for strengths less than this. Therefore, recent research has focused on the use of harder and more durable geosynthetics as a cover for stone columns.

This chapter discusses some soil improvement methods, particularly stone columns, some implementation and design requirements for them, and the use of geosynthetics in the construction of these columns.

3.2 Soil Improvement

Extensive areas, especially along coastlines, are formed by thick layers of soft marine clay with low shear strength and high compressibility. With the ongoing developments in construction in recent years, a number of airports, industries, and other infrastructure facilities are being built in these coastal areas. This highlights the necessity of using land with such weak layers, challenging designers with thick deposits of soft clay. Although the use of foundations in combination with piles can meet the requirements of a proper design, the negative or breaking forces and the long length of the piles often make their use expensive. Therefore, soil improvement methods and techniques, which often consider economic considerations, are preferred. By definition, the improvement of in-situ geotechnical properties of soil for use in a new and more robust structure is called improvement.

The goal of soil improvement is to increase the bearing capacity and reduce the compressibility of natural soils and artificial embankments against applied loads or to reduce their permeability. However, some economic, temporal, or environmental limitations sometimes make certain improvement methods, such as compaction or stabilization, unsuitable. In these cases, the use of stone columns as one of the reinforcement methods has proven its capabilities as a suitable alternative for most of these methods. When large relative settlements are permitted, this soil improvement method is successfully used to increase bearing capacity and reduce settlements in structures such as liquid storage tanks, natural and artificial embankments, low-rise buildings, bridge foundations, and stabilizing soil slopes on soft grounds.

3.3 Types of Soil Improvement Methods

Soil improvement methods are generally divided into three main categories: stabilization, reinforcement, and compaction. These methods include compaction methods, adhesion creation methods, reinforcement methods, drilling-replacement methods, physical and chemical alteration methods, and finally, biological conversion methods. Their application range, advantages, limitations, etc., are shown in Table 1-1.[1]

3.4 Soil Improvement by Stone Columns

The implementation of stone columns is one of the soil modification methods to increase the bearing capacity or reduce the settlement of the soil under the foundation. This improvement method is one of the most widely used soil improvement methods because they are relatively low-cost, easy to install and execute, and effective in performance. The improvement of foundations for low-rise buildings, liquid storage tanks, stabilization of embankments and slopes, bridge foundations, etc., are among the applications of stone columns. The reason for using stone columns is the high shear strength of the materials and the lateral confinement provided by the surrounding soil. This method is based on replacing 15 to 35 percent of the volume of poor soil by drilling holes with a specific diameter, depth, and spacing, and filling the holes with sand, gravel, or crushed stone and compacting them into vertical columns. The main idea of this method is to reduce the force on the soft soil without fundamentally changing the soil structure. Stone columns or granular piles are often used to reinforce soft clays, silts, and loose silty sands.

Advantages of stone columns include:

1. Reducing settlement and accelerating consolidation settlement due to reducing the flow path length and relative settlements.

2. Simple installation of stone columns.

3. Uniformity with depth.

4. Reducing the potential for liquefaction.

5. Increasing the bearing capacity of the soil under the foundation, especially in clayey soils or loose sandy layers with silt, and reducing the dimensional foundation. [2]

Table 3-1 Characteristics of Different Soil Improvement Methods [1]

Method	Suitable Soil Type	Effective Depth (meters)	Typical Placement and Final Spacing	Available Improvement Extent	Advantages	Limitations
Dynamic Compaction	Coarse-grained soils, saturated sands and silty sands, partially saturated sands	Between 10 to 20	Square arrangement at intervals of 2 to 6 meters	Dr = 80% $N1_{(60)}$= 25 q_c = 10-15Mpa	Inexpensive, simplicity	Limited depth of influence, noise and vibration pollution
Vibratory Compaction	Sands, silty sands, and gravelly sands (with less than 20% fines)	30	Square or triangular arrangement at intervals of 1.5 to 3 meters	Dr=80% $N1_{(60)}$=25 q_c = 10-15Mpa	Uniformity with depth	Requires special equipment, unsuitable in cobbly soils
Explosive Compaction	Saturated silty sands	Unlimited	Square or triangular arrangement with spacings of 3-8 meters	Dr=75% $N1_{(60)}$=20-25 q_c = 10-12Mpa	Inexpensive, simple technology	Noise and vibration pollution
Stone Columns by Replacement	Soft soils, clayey sands or silty sands, silty clays	30	Square or triangular arrangement at intervals of 1.5 to 3 meters	$N1_{(60)}$=20 q_c = 10-12Mpa	Drainage, uniformity with depth	Requires special equipment, unsuitable in cobbly soils
Micro Piles	All soils except very soft clays	-	Depends on the application.	Transfers loads through weak soils.	Supports structures	Expensive, potential for leakage around structures

Table 3-2 Continuation of Table 2-1 Characteristics of Different Soil Improvement Methods

Method	Suitable Soil Type	Effective Depth (meters)	Typical Placement and Final Spacing	Available Improvement Extent	Advantages	Limitations
Soil Anchoring	All soils except very soft clays	Unlimited	Covers an injected anchor area of 1 to 5 square meters	Stabilization of cut slopes and excavations	Flexible system, can accommodate large movements, high resistance to dynamic movements	Excavation must remain open until anchors are installed
Sand Drains	Sands, silty sands	20	Spacings chosen to minimize excess pore pressure ratio	Reduces pore water pressure	Inexpensive, no need for overall area modification	May require close spacing
Mixing	Cement for improving sands, lime for improving clays and clayey sands	In Shallow depths	-	From high-density fill materials to cemented materials	Can be designed up to the desired improvement level	Results depend on the number of additives and site compaction.
Deep Soil Mixing	All soils	20	Depends on the application	Dependent on the size, strength, and type of equipment and soil	High strength, ground reinforcement	Requires specialized equipment, fragile elements created.

Table 3-3 Continuation of Table 2-1 Characteristics of Different Soil Improvement Methods

Method	Suitable Soil Type	Effective Depth (meters)	Typical Placement and Final Spacing	Available Improvement Extent	Advantages	Limitations
Excavation must remain open until anchors are installed	Soils with medium to high compressibility, silty sands, silts, clays	Up to 65 meters, and over 20 meters requires a crane	Square or triangular arrangement with spacings of 1.5 to 6 meters	Depends on the final consolidation pressure	Inexpensive, simplicity	Unsuitable if there is a barrier above the permeable layer
May require close spacing	In all soils (challenging in highly plastic clays)	Unlimited	Depends on the application	Depends on the size, strength, and type of equipment and soil	Controlled improvement, feasible in confined urban areas	High cost
Results depend on the number of additives and site compaction.	All soils	Shallow depth	Depends on the application	Slope stabilization	Harmonizes with nature	Slopes remain stable at 1.5H:IV as long as the plants are alive
Requires specialized equipment, fragile elements created.	All soils	In Shallow depths	-	From high-density fill materials to cemented materials	Can be designed up to the desired improvement level	Expensive, requires retaining structures during excavation

3.4.1 Factors Affecting the Selection of Stone Columns

Stone columns are used for stabilizing soft soils present at construction sites. Below is a summary of factors influencing the feasibility of using stone columns:

1. Stabilization and Overall Stability: One of the best applications of stone columns is their use in stabilizing soil against large works such as embankments, tank loads, etc., and providing overall stability to control settlement.

2. Design Loads: The design loads on stone columns should be relatively uniform and limited between 20 to 50 tons per column.

3. Optimal Improvement: The best improvement can be achieved for compactable silty soils and clays located near the ground surface, where the shear strength range is from 15 to 50 kilonewtons per square meter. Economically, it is advantageous if the bearing layers are identified to be about 6 to 10 meters deep.

4. Special Precautions: Special precautions must be taken when stone columns are used in sensitive soils and soils containing organic and vegetative soil because of the high compressibility of organic and vegetative soils, lesser lateral support might be provided, and larger deformations might occur. The vibro-replacement method should not be used when the thickness of the organic soil layer is more than 1 to 2 times the diameter of the stone column [3].

3.4.2 Construction of Stone Columns

The improvement of soft soils with stone columns (sand or gravel) can be achieved through drilling, replacing, and various compaction methods. The main construction methods used by many companies are presented below.

3.4.2.1 Wet Top-Feed Vibro-Replacement Method

In this method, a hole or pit is formed in the ground using a water jet, and the drill is lowered to the desired and specified depth. The pit is emptied without a casing by washing out with water pressure, and then stone is added. The added stone layers, about 0.3 to 0.6 meters thick, are compacted using electrical devices or water or air pressure. The wet method is generally used at sites where the stability of the hole or pit is questionable. Therefore, it is used at sites where the underlying layers are formed of very soft to stiff soils and have a high groundwater level. Figure 1-1 shows the drilling device with a water jet, and Figure 1-2 illustrates the steps of the wet top-feed vibro-replacement method.

Figure 3-1 Water Jet Drilling [4]

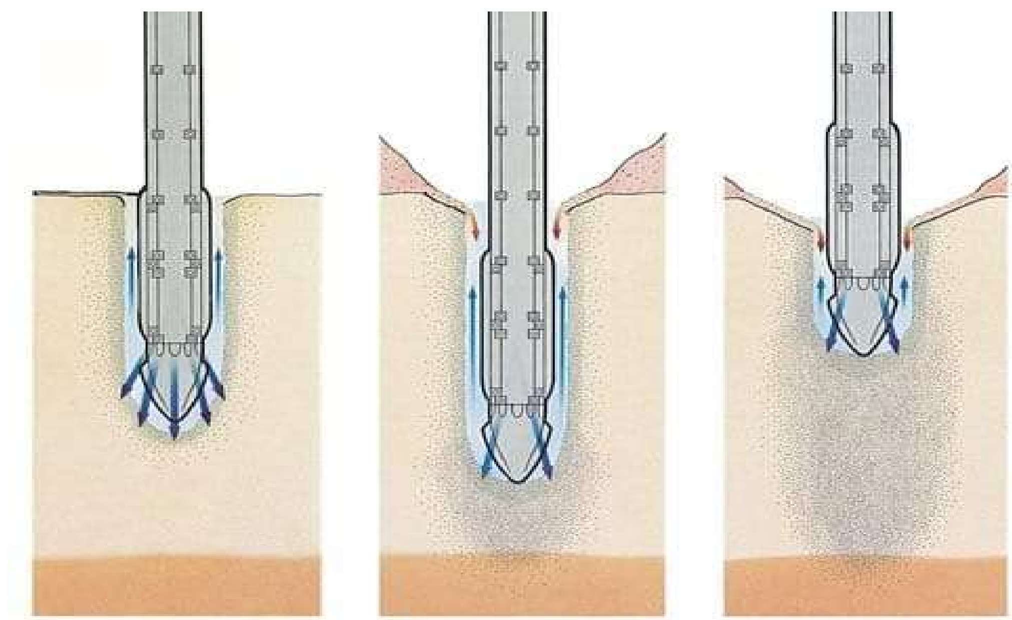

Figure 3-2 Stages of the Wet Vibration Replacement Method [4]

The Vibro-Displacement method is a dry process similar to the Vibro-Replacement method. The primary difference between Vibro-Replacement and Vibro-Displacement is that water is not used during the initial formation of the hole in the dry method. Many constructors use both methods. In the Vibro-Displacement method, the hole must be able to remain open during the entry and exit of the compacting tamper while the vibrator is active. Therefore, for Vibro-Displacement, the soil should ideally show an undrained shear strength of about 40-60 kilonewtons per square meter, and the groundwater level at the site should be relatively low.

Figures 1-3 and 1-4 illustrate the steps of the dry Vibro-Displacement method using top-feed and bottom-feed approaches.

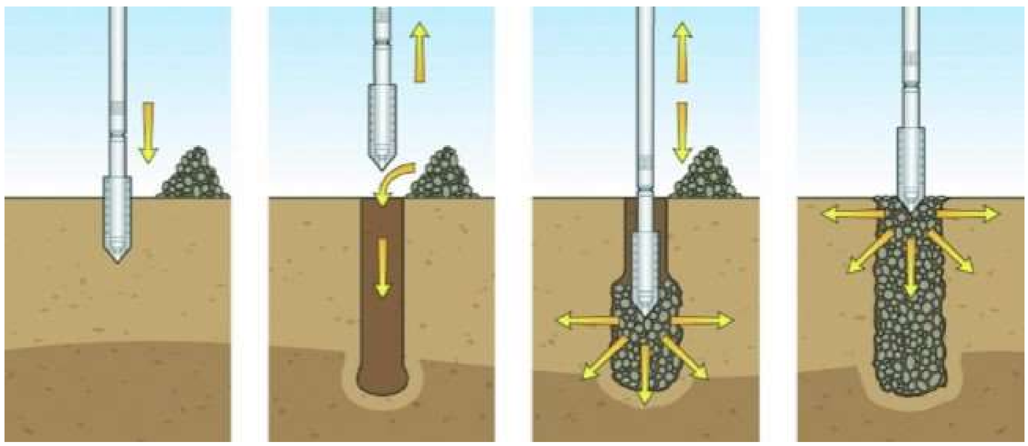

Figure 3-3 Stages of the Dry Vibration-Displacement Method - Top Injection [5]

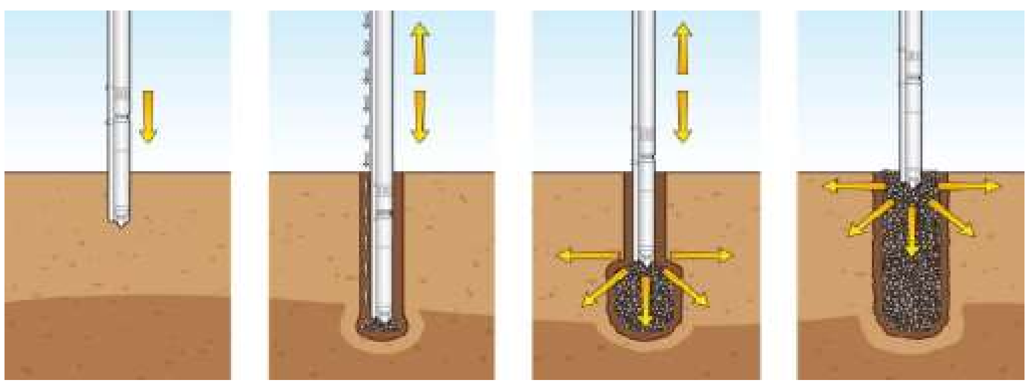

Figure 3-4 Stages of the Dry Vibration-Displacement Method - Bottom Injection [5]

3.4.2.2 Crushed Stone Columns

Crushed stone columns are created by driving an open or closed-ended pipe into the ground or drilling to form a hole, which is then replaced with a mixture of sand and stone. When casing is used in the initial stages within the soil, the potential for slippage and collapse of the hole's walls is eliminated. As a result, this technique is used in many soft soils. Stone columns are typically constructed using an electric or hydraulic (hydrodynamic) vibrator with a vibrating tamper shaped like a cylinder. The vibrator includes a hydraulic or electronic motor mounted in a cylindrical casing approximately 360-460 mm in diameter and 2 to 4.5 meters in length. The motor's power is generated by eccentric weights that create lateral vibration and compaction force. Depending on the selected device, the lateral force varies and can range from approximately 12 to 28 tons. Heavy casing sheets are added to the sides of the vibrator to protect it from wear and tear due to moving up and down in the soil.[6]

3.4.2.3 Rigid Stone Columns

In Europe, for various works, cement is added to compacted stone columns, resulting in the formation of rigid stone columns. The cost of rigid columns in the USA is similar to that of conventional stone columns. The added cost of the cement used in rigid columns is roughly equivalent to the acceleration of work compared to conventional columns.[7]

3.4.2.4 Special Considerations

Stone columns constructed using vibration techniques have been widely used in Europe since around 1950. As previously discussed, when an organic soil layer is observed, all organic materials must be completely removed from the hole. During removal, a large-diameter pit may form within the layer. A tamper with a diameter of 100 mm can be used to form columns within organic soil layers. In general, specific studies are recommended for constructing stone columns in silty soils and sensitive clays. During the construction period, which is subject to vibration, they face a significant reduction in strength. In soft soils, the application of a granular cover that causes bulging to transfer deeper (Figure 1-

5) results in an increase in the ultimate capacity of the column. Additionally, creating a cover helps distribute the load to the stone columns. The cover should have a thickness of approximately 0.3 to 1 meter and be made of sand, gravel, or crushed stone. A granular cover can also reduce hydrostatic pressure. Sometimes, the soil between constructed columns is removed and replaced with granular materials. Figure 1-5 shows the effect of a compact granular cover on the performance of a stone column.[7]

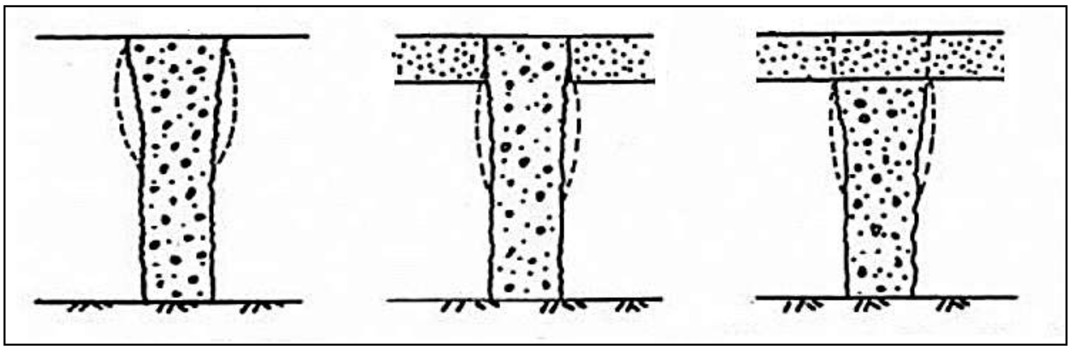

Figure 3-5 The Impact of Dense Granular Coating on the Performance of Stone Columns [7]

To fully assess the feasibility of implementing stone columns at specific sites, sufficient details about the underlying layers must be available. Constructors need accurate borehole logs, soil grain size classification of the underlying layers, shear strength, the number of blows required in the SPT test, or the results of dynamic penetrometer tests. The geological history of sensitive soil sediments is usually required. Parameters such as undrained shear strength, consolidation parameters, specific gravity, and groundwater level are typically needed for evaluation.

3.4.3 Characteristics of Stone Column Materials

The gravel and crushed stone used for filling the columns must be clean, hard, durable, non-weathered, and free from organic materials, debris, and other erodible substances. One of the required tests to determine the soundness and health of the fill materials for the column is the Magnesium Sulfate test. In the Magnesium Sulfate soundness test, the percentage of material weight loss should not exceed 15%. Table 2-4 shows the gradation used in the vibro-replacement method.[7]

Table 3-4 Grading Used in the Construction of Stone Columns [7]

Sieve Size	NO.1 Passing Percentage	NO.2 Passing Percentage	NO.3 Passing Percentage	NO.4 Passing Percentage
4	-	-	100	-
3.5	-	-	90-100	-
3	90-100	-	-	-
2.5	-	-	25-100	100
2	40-90	100	-	65-100
1.5	-	-	0-60	-
1	-	2	-	25-100
0.75	0-10	-	0-10	10-55
0.5	0-5	-	0-5	0-5

Generally, the use of grades NO.1 or NO.2 is recommended for stone column materials. In areas with soft organic soils and where rapid construction is required, grade NO.2 should be considered. If the specified gradation is not available, grade NO.3 can be used. Additionally, gradation NO.2 or NO.4 can be utilized in situations where materials with larger grain sizes are not available.

The following laboratory tests should be conducted and reviewed to ensure the quality and suitability of the materials used in stone columns:

- Grain size distribution according to AASHTO T-27 to determine the distribution of different size particles within the material.

- Specific gravity according to AASHTO C127 to determine the density of the materials relative to water, which is essential for calculating the weight and volume relationships.

- Loose unit weight according to ASTM C29 to determine the weight of the material in its natural, loose state, which helps in estimating the volume needed for the stone columns.

- Compacted unit weight according to ASTM C29 to determine the weight of the material once it has been compacted, providing an indication of how much the material will settle and compact under pressure.

The sand used in stone columns should be hard, either natural or manufactured, and free from organic materials, any erodible substances, and waste. The sand should be well-graded with less than 15% passing through a No. 200 sieve and should have a minimum particle size of 0.2 millimeters. These specifications ensure that the sand provides the necessary strength and stability to the stone columns, contributing to the overall effectiveness of the soil improvement.

3.4.4 Fundamentals of Analysis of Unreinforced Stone Columns

The arrangement of stone columns is commonly in an equilateral triangular pattern, although a square layout can also be used. The triangular arrangement tends to accommodate a greater number of stone columns within a fixed area compared to the square layout, which is why the triangular method is often preferred. Figure 1-6 illustrates the triangular arrangement of stone columns in a group configuration.[8]

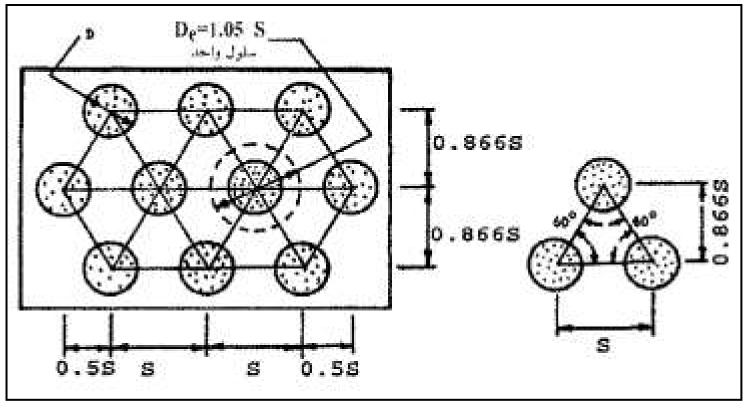

Figure 3-6 Triangular Arrangement in the Group Layout of Stone Columns [8]

For the purpose of calculating settlement and load-bearing capacity, each stone column and the surrounding soil are considered as a single unit cell.

As depicted in Figure 1-7. This conceptual model simplifies the analysis by treating the complex interaction between the stone column and the soil as a more manageable system.

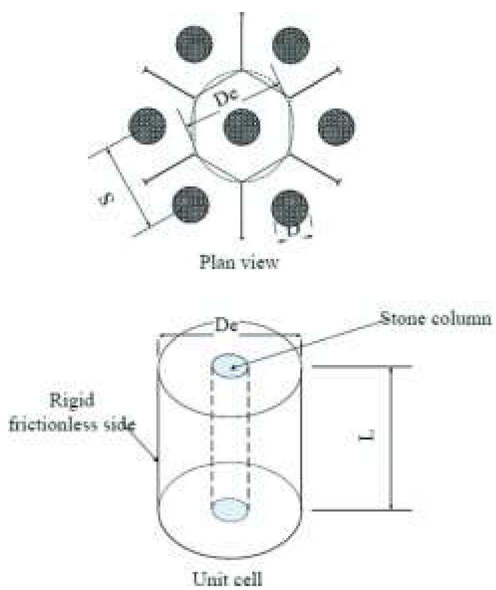

Figure 3-7 Assuming the Stone Column and Surrounding Soil as a Single Cell [8]

This approach allows engineers to estimate the performance of stone columns in terms of their ability to reduce settlement and increase the load-bearing capacity of the ground.

When using a triangular layout, the soil around each stone column, which forms a regular hexagon, can be accurately approximated by a circle of equivalent area. For an equilateral triangular arrangement, the effective diameter of the circle is given by:

$De=1.05S$

For a square arrangement of stone columns:

$De=1.13S$

Where:

- S is the center-to-center distance between the stone columns,
- De is the equivalent diameter of the circle corresponding to a regular hexagon, encompassing one stone column and the surrounding soil affected by it.

The volume of soil replaced by the stone column materials significantly impacts the increase in load-bearing capacity and reduction in soil settlement. For this purpose, a ratio known as the area replacement ratio, a_S, is defined as:

$a_S=A_S(A_C+A_S)$

Where:

- A_S is the horizontal area of the stone columns,
- A_C is the horizontal area of the soil surrounding the stone columns,
- A is the total horizontal area encompassing both the soil and the stone columns.

The area replacement ratio can be written as:

$a_s = C_1(D/S)2$

Where:

- D is the diameter of the stone column,
- C_1 is a coefficient dependent on the arrangement of the stone columns; for a square arrangement $C_1 = \pi/4$, and for an equilateral triangular arrangement $C_1 = \pi/2\sqrt{3}$

Studies and experience have shown that when a load is applied to the ground surface, stress concentration in the stone column is greater compared to the surrounding soil, as illustrated in Figure 1-8. This stress concentration is crucial for understanding the load transfer mechanism from the stone columns to the surrounding soil and the overall improvement in ground performance.

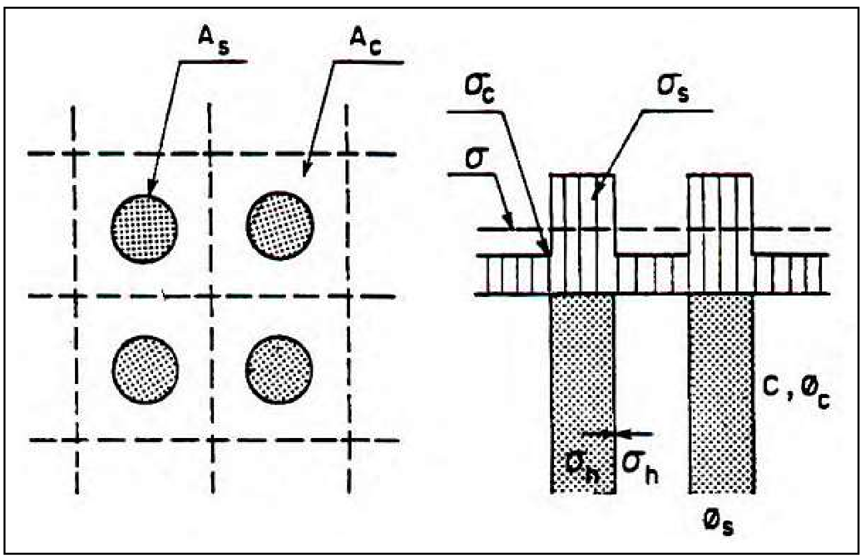

Figure 3-8 Stress Concentration in Soil Improved with a Stone Column [8]

In this soil improvement method, since the settlement of the stone column and the surrounding soil is almost the same, the load is distributed based on the stiffness ratio. Since the stone column has greater stiffness compared to the surrounding soil, it carries a larger share of the load, resulting in stress concentration within it. The distribution of vertical stresses in a single cell can be represented by a dimensionless coefficient known as the stress concentration factor.

$n=\sigma_s/\sigma_c$

Where:

- n is the stress concentration factor,
- σ_s is the stress within the stone column,
- σ_c is the stress within the surrounding soil.

The value of the stress concentration factor n depends on the stiffness of the stone column and the area replacement ratio, and it increases with the increase in the stiffness of the stone column and the area replacement ratio. Barksdale and Bachus (1983) calculated the stress concentration factor to range between 2 to 5. However, Aboshi et al. (1979) and Bergado et al. (1987) calculated the stress concentration factor to reach up to 9. The high stress concentration factor provided by Bergado et al. might be due to the very high rigidity of the loading plate, which causes a significant stress concentration in the stone column. In Thailand, for improving the soft clay bed (Bangkok clay) of an embankment, a stone column with an area replacement ratio of 0.06 was used, and a stress concentration factor of 2 was obtained. It was also observed that with an increase in the load, the stress concentration factor decreased to 1.45. The average stress (σ) applied to the unit cell area is defined as follows:[9]

$\sigma=a_s\sigma_s+\sigma_c(1-a_s)$

Using the stress concentration factor, the area replacement ratio, and the average stress, the relationship between the stress in the stone column and the soil stress can be written as:

$\sigma_s=n\sigma/ [1+(n-1)\ a_s] =\mu_s\sigma$

$\sigma_c=n\sigma/ [1+(n-1)\ a_s] =\mu_c\sigma$

Where:

- σ is the average stress applied to the unit cell area,
- μ_s is the ratio of stress in the stone column to the average stress,
- μ_c is the ratio of stress in the soil to the average stress.

3.4.5 Types of Failure in Stone Columns

Stone columns in soft soils typically extend to a firm layer. However, if the soft soil layer is thick, the stone column acts like a floating pile and transfers the applied load to the soil through the frictional interface between the stone column and the surrounding soil. Stone columns may fail individually or as a group. Figure 1-9 shows the possible failure modes of an individual stone column in homogeneous soft soil.

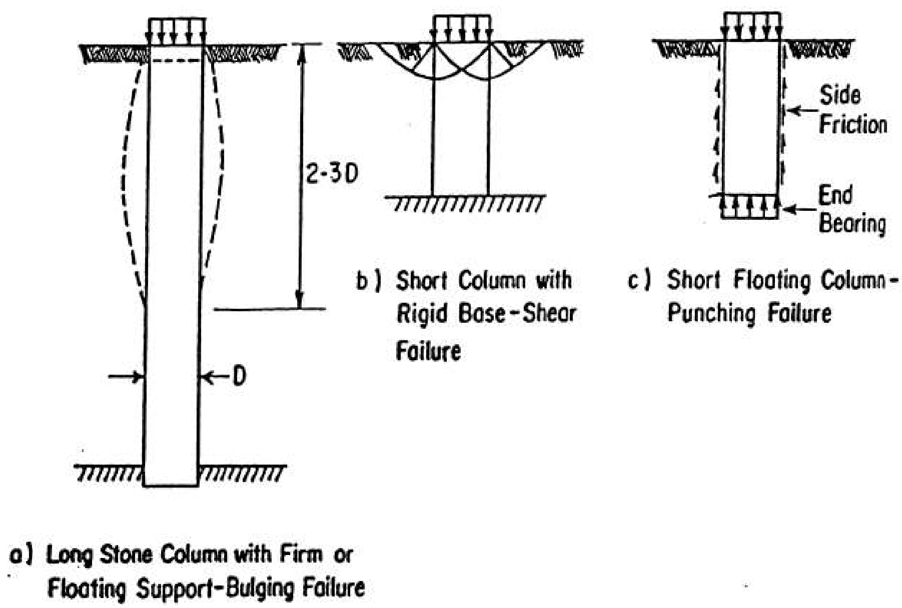

Figure 3-9 Types of Failure of a Single Stone Column in Homogeneous Cohesive Soil: (a) Failure due to Punching, (b) Failure due to General Shear, (c) Failure due to Lateral Buckling [10]

Figure 1-9 shows three types of failure that can occur in a stone column:

- Punching: Occurs in short columns constructed in soft soil (Figure 1-9a).
- General Shear: Occurs in short columns resting on a firm base (Figure 1-9b).
- Bulging: This type of failure is due to insufficient lateral support from the surrounding soil. Typically, this failure occurs at a depth of 2D to 3D. For stone columns longer than 4 to 6 times their diameter, failure usually occurs through lateral expansion (Figure 1-9c).

In the case of layered cohesive and non-homogeneous soils, the types of failures are shown in Figure 1-10. If there is a weak soil layer (1-3 meters thick) on the ground surface, the failure will be of general shear or lateral expansion type, and this surface layer will increase the settlement susceptibility of the stone column and reduce its load-bearing capacity. However, experience has shown that if a weak soil layer thicker than 1D surrounds the stone column, the column is at risk of lateral expansion, and if the thickness of the weak soil layer is less than 1D, the column is not at risk.

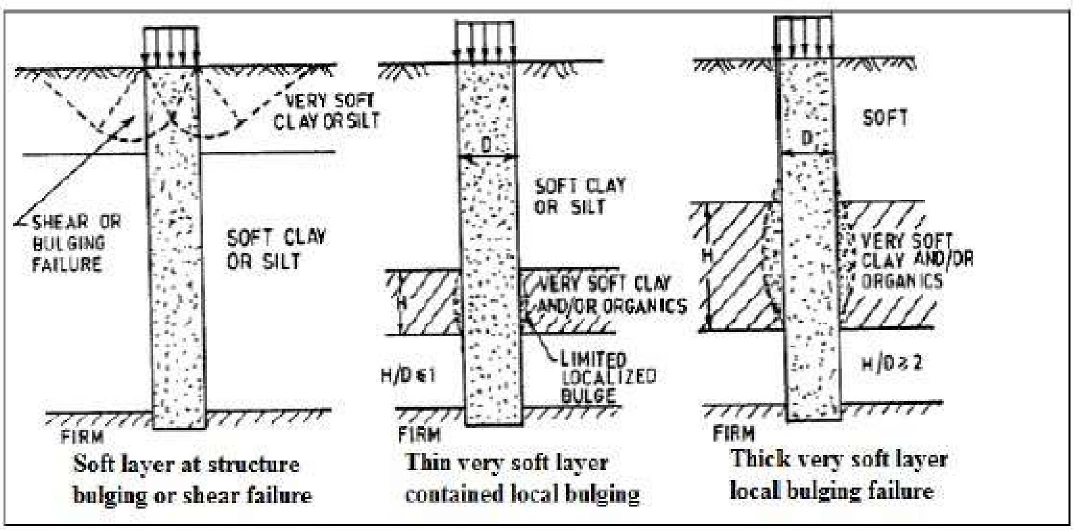

Figure 3-10 shows the failure modes of an individual stone column in non-homogeneous cohesive soil.[10]

3.4.6 Load-Bearing Capacity of a Single Stone Column

For a single stone column longer than 4 to 6 times its diameter, failure typically occurs through lateral expansion. In homogeneous soils, this type of failure often happens near the ground surface where the soil's lateral resistance is low. Lateral expansion usually occurs at a depth of 2D to 3D. Mori observed that for stone columns longer than 2D, only a small portion of the vertical load applied to the stone column reaches its end. To explain this phenomenon, Hughes and Withers conducted laboratory experiments with a model of soil and stone column in soft soil. They found that under load, the stone column undergoes lateral expansion, causing the stone aggregates to penetrate into the surrounding soil and transfer the force through shear.[6]

Finite element analyses have shown that when the applied load reaches the ultimate bearing capacity, failure occurs near the ground surface as vertical displacement (sliding) between the soil and the stone column, causing the stone column to sink into the soil. Typically, the failure of stone columns starts from the ground surface and progresses downwards.

3.4.7 Lateral Expansion Theory in Stone Columns

To calculate the bearing capacity of a stone column, the surrounding soil is considered as a confining stress factor (3σ). When the length of the stone column exceeds 4D, it undergoes lateral expansion under load, creating a resistance state in the surrounding soil. For stone columns made of granular materials with an internal friction angle (ϕ_s) and no cohesion (Cs=0), and confining pressure (3σ) from the surrounding soil, the allowable vertical stress (σ_1) due to the applied load at the top of the stone column is given by:

$$\sigma_1 = \sigma_3 tan^2\left(45 + \frac{\emptyset_s}{2}\right) = \sigma_3 \frac{1 + \sin\emptyset_s}{1 - \sin\emptyset_s} = \sigma_3 k_p$$

Using the elastic-plastic theory proposed by Gibson and Anderson (1970) for cohesive soils, considering the lateral (radial) expansion of the stone column and the resistance state created in the surrounding soil, Hughes (1974) and Withers presented an equation for calculating the ultimate undrained lateral stress (σ_3) that the surrounding soil can bear, using the CPT test:

$$\sigma_3 = \sigma_{r_0} + c\left[1 + \ln\frac{E_c}{2c(1+\vartheta)}\right] \quad (1\text{-}10)$$

Where:

- σ_3 is the ultimate undrained lateral stress exerted on the stone column by the soil.
- σ_{r0} is the initial (in situ) total lateral stress.
- E_c is the elastic modulus of the surrounding soil.
- c is the undrained shear strength of the soil.
- ν is the Poisson's ratio of the soil surrounding the stone column.

By substituting equation (1-10) into equation (1-9), we get:

$$\sigma_1 = \left[\sigma_{r_0} + c\left[1 + \ln\frac{E_c}{2c(1+\vartheta)}\right]\right]\left(\frac{1+\sin\emptyset_s}{1-\sin\emptyset_s}\right) \quad (1\text{-}11)$$

Typically, the internal friction angle for crushed stone is in the range of 45-42 degrees, and for regular gravel, it is in the range of 42-38 degrees.[6]

3.4.8 Vesic's Lateral Expansion Theory

In Vesic's lateral expansion theory, considering the assumptions of the previous method, for the soil surrounding the stone column, in addition to cohesion (c), the internal friction angle (ϕ_s) is also taken into account. Vesic ultimately presented the ultimate lateral stress (σ_3) that the surrounding soil can bear as follows:

$\sigma_3 = cF_c' + qF_q'$

Where:

- c is the cohesion of the soil surrounding the stone column.

- q is the average of the principal stresses (σ1+σ2+σ3) / 3 applied at the depth of failure.
- Fq′ and Fc′ are the expansion coefficients of the stone column well, and their values depend on the internal friction angle of the soil surrounding the stone column and the stiffness index (Ir). The stiffness index is defined as follows:

$$I_r = \frac{E_c}{2(1+v)(c+q\tan\varphi c)}$$

Vesic ultimately presented the bearing capacity of the stone column as follows:

$$q_{ult}=[\, cF_c' + qF_q'].\left(\frac{1+\sin\emptyset_s}{1-\sin\emptyset_s}\right)$$

Figure 1-11 provides diagrams for determining the values of Fq′ and Fc′.[11]

It's important to note that:

1. The value q in equations 1−131−13 and 1−141−14 includes the initial stresses present due to the soil's weight and the additional stress applied due to the applied surcharge. The value of q should be calculated at the depth where failure (lateral expansion) is likely to occur. Given that the applied surcharge at the ground surface is divided between the stone column and the surrounding soil according to the stress concentration factor, the effect of the surcharge in calculating q should only consider the portion pertaining to the surrounding soil.
2. The short-term and long-term bearing capacities of stone columns can be accurately predicted using the presented method.
3. For calculating the bearing capacity of stone columns using Vesic's method for soft to stiff soils (without organic matter), an elastic modulus of 11C is suggested, and for very soft soils or soils containing organic matter with a plasticity index greater than 30, an elastic modulus of 5C is recommended.

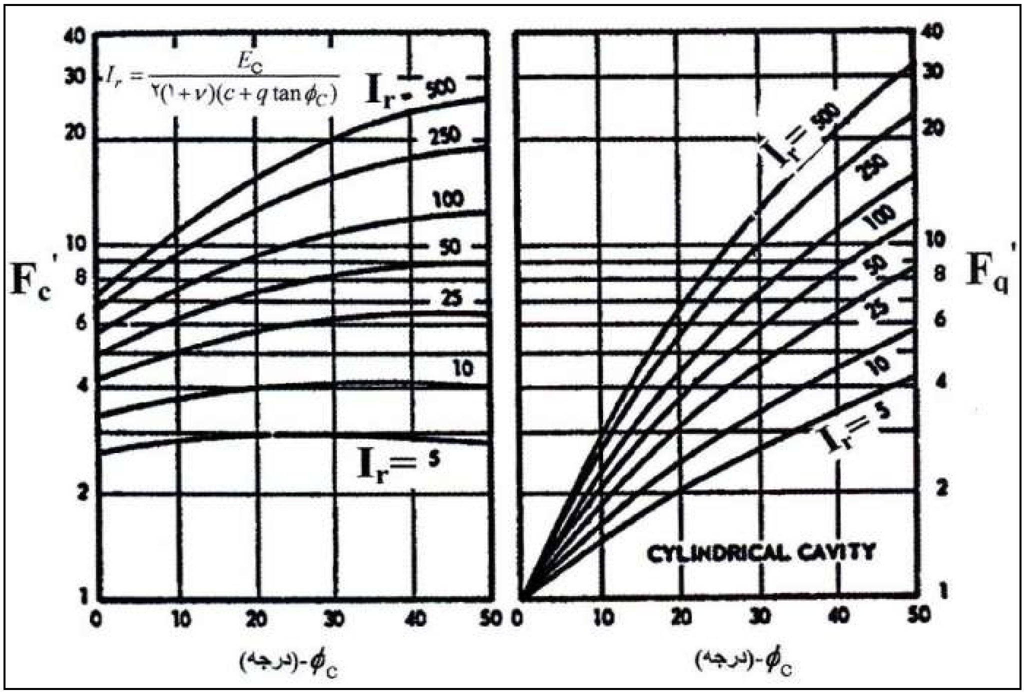

Figure 3-11 Diagram for Calculating Fq' and Fc' Using the Vesic Method [11]

3.4.9 Load-Bearing Capacity of Short Stone Columns

For the bearing capacity of short stone columns, as previously discussed, failure modes include general shear or local shear. When failure occurs due to punching, the bearing capacity is the sum of end-bearing resistance and skin friction of the stone column. In short stone columns near the ground surface, where lateral stress (σ) is low, if the stone column rests on a firm base, failure occurs by general shear. Madhav and Vitkar (1980) provided a formula for calculating the bearing capacity of short stone columns assuming the trench is filled with non-cohesive granular material and the surrounding soil is cohesive and lacks internal friction angle, in a plane strain state as follows:

$$\mathrm{qult} = \mathrm{cNC} + \mathrm{Df}\Upsilon_c\mathrm{Nq} + \frac{\Upsilon\mathrm{cBN}\Upsilon}{2}$$

Where:

- Nc, Nq, and Nγ are the bearing capacity factors.

- B and Df are the width of the footing and the depth of embedment, respectively.
- γ_c is the saturated or moist unit weight of the cohesive soil.[12]

Figure 1-12 provides diagrams for calculating the necessary factors for determining the bearing capacity using Madhav and Vitkar's method.

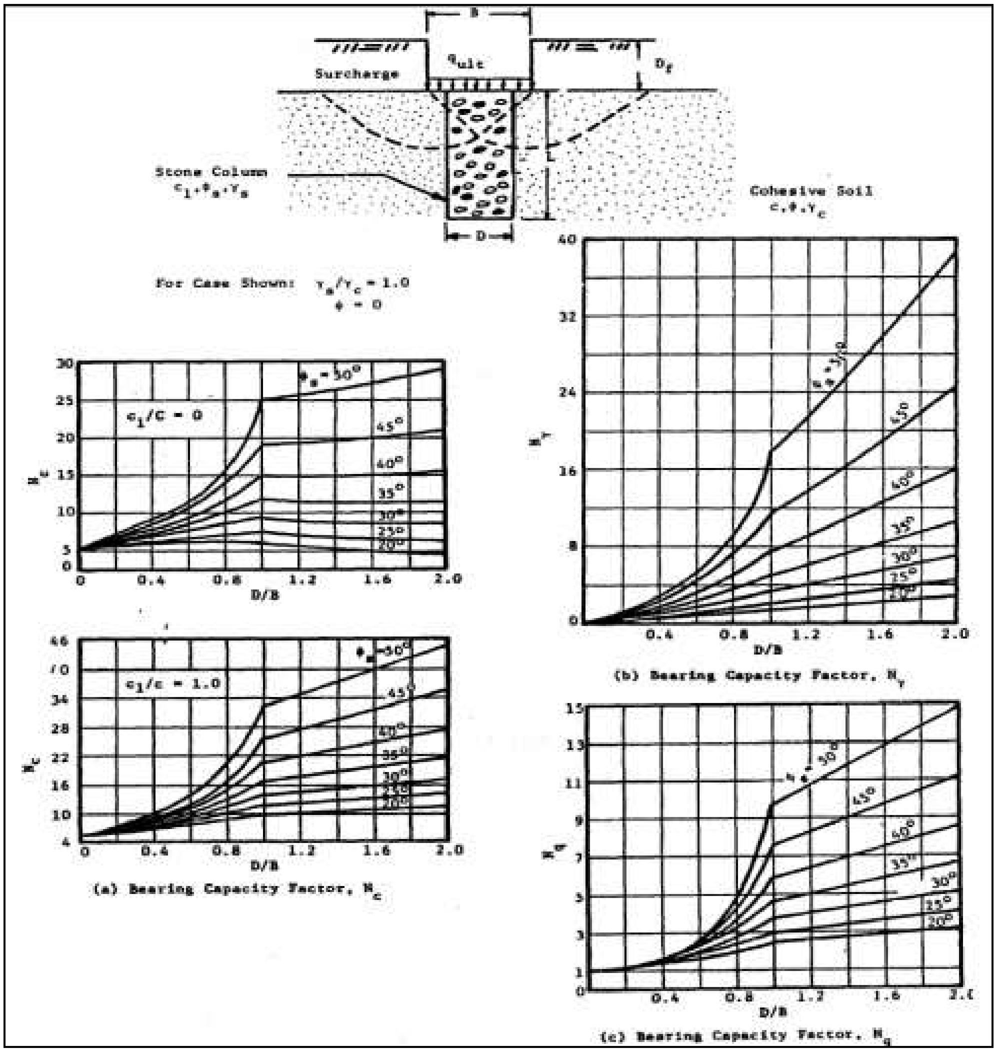

Figure 3-12 Diagrams for Calculating Load Capacity Coefficients of Short Stone Columns Using the Madhav and Vitker Method [12]

Table 1-3 presents various formulas proposed by different researchers for calculating the bearing capacity of stone columns, compiled by Aboshi and colleagues, for three different failure modes. [13]

Table 3-5 Formulas proposed for calculating the bearing capacity of stone columns for three different types of failure [13]

Type of Failure	Formula	Source
Lateral Expansion	$q_{ult} = (Y_c Z K_{pc} + 2C \sqrt{K_{PC}}) (1+Sin\varphi s / 1- Sin\varphi s)$	Greenwood(1970)
	$q_{ult} = [cF_c' + qF_q'] (1+Sin\varphi s / 1- Sin\varphi s)$	Vesic (1972) Datye and Nagaraju (1975)
	$q_{ult} = (1+Sin\varphi s / 1- Sin\varphi s) (\sigma_{r0} + 4c)$	Hughes and Withers (1974)
	$1+Sin\varphi s / 1- Sin\varphi s(\varphi_{r0}+4+K_0 qs) (W/B)^2 + (1- (W/B)^2$ $q_{ult} =+) qs$	Madhav et al. (1979)
General Shear	$q_{ult} = Cnc+ (1/2) \gamma_c B N_\gamma + Y_c D_f N_q$	Madhav and Vitkar (1978)
	$q_{ult} = 2As (K_{cp}^* q +2c\sqrt{K_{pc}}) + (1/K_{as}) (3ds K_{PC} Y_C (1-(3ds /2L)))$	Wong (1975)
Sliding	$q_{ult} = (1/2) Y_C B\tan^3\psi+2c \tan^2\psi+2(1-a_s) c \tan\psi$ $\psi = 45^o + \tan^{-1}(\mu_s a_s \tan\varphi s) / 2$	Barksdale & Bachus (1983)
	$\tau = (1-a_s) c +(Y_S Z+\mu_S \sigma z) a_s \tan\varphi s \cos^2 0$ $\mu_S = n / 1+(n-1) a_s$	Aboshi et al (1979)

Explanation of Table (2-5) Parameters:

As= Cross-sectional area of the stone column

B = Width of the loaded area

C = Undrained shear strength of clay soil

Cc = Compression coefficient

W = Equivalent width of a row of stone columns

Df = Depth of foundation embedment

Fe and Fq = Dimensionless load-bearing capacity parameters

H = Thickness of the soil layer

K = Lateral earth pressure coefficient due to surcharge

N = Stress concentration ratio

Ko = At-rest lateral earth pressure coefficient

Q = Applied stress

Z = Distance from the stone column tip to the ground surface

E = Modulus of elasticity

a_S = Ratio of the improved area

d_S = Diameter of the stone column

Kas = Active lateral earth pressure coefficient of the stone column

e_0 = Initial porosity ratio

Kpe = Passive lateral earth pressure coefficient of soil

q_{ult} = Bearing capacity

θ = Angle of the vertical plane of slip in each stone column

μ = Poisson's ratio

c_μ = Stress reduction coefficient in clay

σ = Vertical stress

σ_{ro} = Initial radial stress in the stone column

σ_z = Amount of stress added due to surcharge at depth Z

S_ϕ = Internal friction angle of stone column materials

ψ = Angle between the failure plane and foundation

N_C, N_q, and N_γ = Dimensionless load-bearing capacity parameters dependent on soil and stone column material characteristics and the ratio of the improved area

q_S = Bearing capacity of soft soil, approximately equal to $\frac{2}{3} C_0 N_0$

γ_s and γ_c = Specific weight of stone column materials and clay soil

τ = hear stress of soil improved with stone columns

3.4.10 Application of Geogrid in Reinforcing Stone Columns

While the use of geotextiles is suitable for specific applications, their use can be limited due to the significant relative settlements that result from minimal compaction during installation and throughout the loading phase. Therefore, recent research has focused on the application of stiffer and more durable geosynthetics such as geogrids as a wrapping for stone columns. Given that very soft soils lack sufficient lateral support, the construction and application of conventional stone columns, considering the low undrained shear strength of such soils, is nearly impossible. This problem can be solved by wrapping the stone column materials with geosynthetics. Additionally, when stone columns are used in sensitive clays, they encounter specific limitations. The lack of lateral support increases the settlement of the base. Moreover, the clumped clay particles around the stone column reduce the radial drainage capability of the column. To overcome these limitations and improve the effectiveness of stone columns in terms of resistance and compressibility, stone columns should be wrapped with geogrids or geocomposites. Murugesan & Rajagapol (2006) note that since stone columns in soft soils cannot provide the required bearing capacity due to the lack of lateral support from the soil, they recommend the use of geosynthetic wraps for quicker and more economical installation, as well as to increase the resistance and stiffness of the stone column and prevent lateral dispersion. An example of geogrids used for reinforcing stone columns is shown in Figure 1-13. [14]

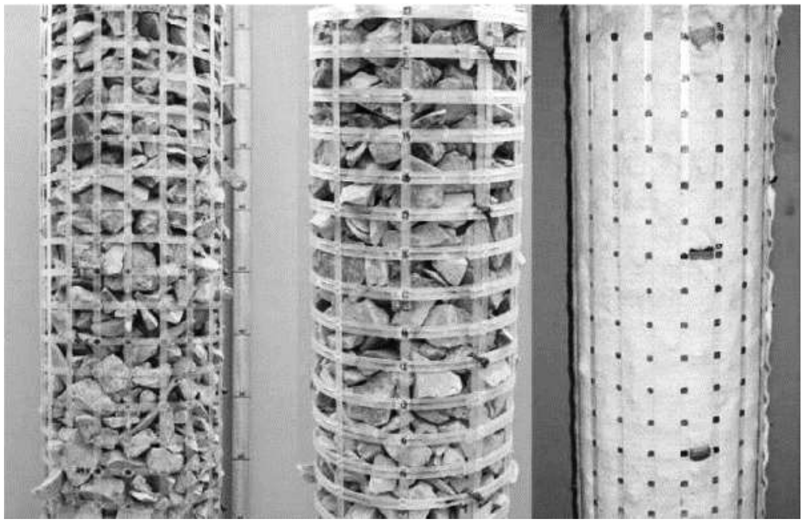

Figure 3-13 Examples of Geogrids Used in Stone Columns [14]

They explain that wrapped stone columns have a much higher bearing capacity and are under less pressure with less lateral expansion compared to conventional stone columns. The results show that the confining lateral stresses increase significantly with the use of wraps around the column. Furthermore, as the stiffness of the wrap increases, the lateral stress transferred to the surrounding soil decreases, making the bearing capacity of the wrapped column less dependent on the soil resistance compared to unwrapped columns.

3.4.10.1 Analytical Foundations of Reinforced Stone Columns

The analysis of stone columns is conducted using the axial symmetry idealization of a cylindrical unit cell comprising a stone column and the surrounding soil. Considering the assumption that the settlement of soft soil and the stone column (SC = SS) is equal, the radial changes of the geosynthetic (lateral buckling in the stone column) are taken into account (Equation (1-16) Raithel and Kempfert 2000). This equation can be solved through an iterative process. Due to the relatively time-consuming nature of the calculation process, numerical methods must be used to solve this equation and obtain the radial changes of the stone column (lateral buckling):

$$\Delta r_c = \frac{K_{a,c}\cdot\left(\frac{1}{a_E}\cdot\Delta\sigma_0 - \frac{1-a_E}{a_E}\cdot\Delta\sigma_{v,s} + \sigma_{v,0,c}\right) - K_{0,s}\cdot\Delta\sigma_{v,s} - K_{0,s}{*}\cdot\sigma_{v,0,s} + \frac{(r_{geo}-r_c)\cdot J}{r_{geo}^{2}}}{\frac{E^*}{(1/a_E-1)\cdot r_c} + \frac{J}{r_{geo}^{2}}} \quad (1\text{-}16)$$

In this equation, Ka,c represents the active state effective earth pressure coefficient for the stone column, $K_{0,S}$ is the at-rest effective earth pressure coefficient for soft clay, and $K_{0,S}*$ is the at-rest effective earth pressure coefficient for clay when the stone column is installed using the vibro-replacement method. a_E is the modified area ratio, J is the stiffness of the geogrid, r_{geo} is the geogrid radius, and r_c is the stone column radius. E* is the soil's modulus of elasticity, derived as a factor of the soil's modulus of elasticity from the consolidation test (Eoed,s), $\Delta\sigma_0$ is the difference in existing stress, $\Delta\sigma_{v,s}$ is the difference in vertical stress induced in the soil, and $\sigma_{v,0,c}$ is the existing vertical stress for the stone column. The following assumptions are considered in all presented equations:

1. The surface settlement in soft soil and the stone column is the same.

2. The single stone column is based on end-bearing capacity.

3. The active earth pressure coefficient is used for the stone column.

4. When the vibro-replacement method is used to install the stone column, the at-rest earth pressure coefficient is calculated using the equation KS = $K_{0,\,S}$ = 1-sinΘ, and if the vibro-displacement method is used for column installation, a higher earth pressure coefficient KS = $K_{0,\,S}*$ is used. Additionally, the behavior of geotextile is considered as a material with linear elastic properties. [15]

3.4.10.2 Advantages of Reinforced Stone Columns

1. Geosynthetic wrapping prevents lateral dispersion of the stone column when installed in very soft soil, minimizing damage to the columns and allowing for faster installation.

2. The application of geosynthetic wrapping creates additional confining pressure due to significant twisting, increasing the bearing capacity of the stone column.

3. Small-scale testing of column groups showed that the confinement of adjacent columns significantly reduces lateral buckling. It is expected that the lower lateral earth pressure and soil stiffness near the ground surface will lead to greater radial expansion in the upper part of the column group in full-scale testing. Thus, partial wrapping of the upper section of the column model is included in the testing program to observe the significant effect on vertical and radial deformations.

4. Laboratory observations on the bearing capacity and settlement of columns wrapped in geogrid and geotextile concluded that the bearing capacity of stone columns increases with the stiffness of the geofabric materials used in wrapping the sand columns.

5. Wrapping the upper part of the stone column up to twice the column diameter is sufficient to improve the bearing capacity. However, the lateral expansion behavior of stone column groups is not well understood.

6. The settlement of the stone column significantly decreases with increased geogrid stiffness. Additionally, for single columns, radial expansion is typically limited to the upper part of the column, and it is expected that wrapping only in the lateral expansion area sufficiently strengthens the column.

3.5 Geosynthetics:

3.5.1 Introduction to Geosynthetics

Geosynthetics are textiles and covers made from fibers produced from petroleum derivatives, whose primary property is their imperishability against internal soil agents. Geosynthetics are used as separators, filters, drains, reinforcements, and protectors, and are divided into very diverse forms.

3.5.2 History of Geosynthetic Materials

The historical background of the construction and application of geosynthetic materials dates back to 1950. At that time, single-strand sheets were used as filters for woven geotextiles to control erosion in the state of Florida, USA. Bob Koerner was the first to propose the initial designs for the application of geosynthetic materials in water and soil projects, thus earning the title of the father of geotextiles. [16]

In the mid-1960s, the US Army Corps of Engineers (COE) explored the possibility of using woven geotextiles as an alternative to granular filters in erosion control and slope protection systems. This organization introduced the first technical design criteria for geotextiles as filters and presented a set of technical standards in this field in 1975. At the same time, the use of geotextiles for riverbank protection was introduced, such that by 1977, about two million square meters of geotextile were used solely for protecting the banks of a river in the Netherlands, and this technique is currently considered a standard and accepted method in that country. [17]

In America, the use of single-strand woven geotextiles increased from the late 1970s, and the US Army Corps of Engineers popularized the use of these materials for filtering and erosion control as a standard. Following the activities of the US Army Corps of Engineers and the exchange of information with European engineers, the first geotextile conference was held in 1977, and the use of geotextiles as filters and drains was accepted by many experts and engineers in various fields. [18]

The application of geosynthetic materials in Iran, unfortunately, has not seen significant development due to a lack of precise and scientific awareness among employers and engineers, as well as the scarcity and high cost of these materials. So far, it has only been used in a few projects and in a limited manner.

3.5.3 Reasons for Producing and Using Geosynthetics

- Economic efficiency, speed, and ease of implementation

- Flexibility in design and implementation

- Longer lifespan and less maintenance

- Prevention of quality degradation during implementation

- Harmony with the natural environment

- Extensive application in various structures

3.5.4 Types of Geosynthetics and Specific Symbols

- Geogrid (GG)

- Geotextile (GT)

- Geomembrane (GM)

- Geonet (GN)

- Geocell (GL)

- Geomat (GA)

- Biotextile and Biomatt (BT)

- Geosynthetic clay liner (GCL)

- Geosynthetic clay liner (GCL)

- Synthetic composite geomembrane (GMS)

- Bituminous geomembrane (GMB)

3.5.5 Classification of Geosynthetics

Geosynthetics are classified into four main groups:

1) Geogrids

2) Geotextiles

3) Geomembranes

4) Geocomposites

3.5.5.1 Geogrids

Geogrids, as a type of geosynthetics, are polymeric products usually made in the form of regular mesh networks in one or two directions. These networks, especially the gaps between them, allow soil particles or stone materials to engage well with them, giving the geogrid and its surrounding materials good lock and bond properties. Thus, geogrids used in fine soil layers act as tensile-resistant elements and can effectively contain forces and tensile deformations in areas where tensile stresses and deformations occur in the soil. Figure 1-14 shows examples of uniaxial and biaxial geogrids. [19]

Figure 3-14 Uniaxial and Biaxial Geogrids [19]

3.5.5.2 Geotextiles

Geotextiles are often made from polypropylene polymers. Polypropylenes have a specific gravity less than one (γ=0.9), and are strong and durable. Polypropylene fibers and filaments are used in the production of both woven and non-woven geotextiles. High-strength polyester fibers are also used in geotextile production. Polyesters have a higher specific gravity, excellent strength, and are compatible with most soils found in the environment. Examples of geotextiles are shown in Figure 1-15. [19]

Figure 3-15 Examples of Geotextiles [19]

Geotextiles are generally divided into two main types:

1) Woven:

These geotextiles are made from single-strand monofilaments, multi-filament yarns, or slit film yarns woven together. Woven geotextiles are used for sediment control, encapsulating fines and sludge, and stabilizing roads and pathways.

2) Nonwoven:

This type of geotextile is made from short fibers (usually between 2.5 – 10 cm) or long fibers randomly distributed in layers, resembling a felt-like network. These felt-like networks are then passed through machines that bond the layers together. Nonwoven geotextiles are used in drainage systems, soil erosion control, and for stabilizing roads and pathways on moisture-sensitive soils.

3.5.5.3 Geomembranes

Geomembranes serve as a highly resistant and cost-effective barrier with a long lifespan, finding applications across various industries. For instance, in water and wastewater sectors, geomembranes are used to construct lagoons, water conveyance channels, ponds, pools, artificial lakes, and landfills for municipal, industrial, and hazardous wastes. Since geomembranes come into contact with soil, they can be combined with geotextiles or geogrids as needed. Figure 1-16 shows examples of geomembranes. [19]

Figure 3-16 Examples of Geomembranes [19]

3.5.5.4 Geocomposites (Combination of Geogrid with Geotextile)

Geogrids, due to their polymer structure and properties, have limited ultimate strength. In a composite system, a geogrid combined with a geotextile can be used for easy development of fine soil surfaces, followed by the application of fine soil. Geocomposites, such as Colbondrain (deep drainage) and Encadrain (horizontal and vertical drainage), are used in consolidating soft soils by facilitating the removal of water from the soil, collecting and channeling leakage water, and in contact with structures using vertical and horizontal drains. Figure 1-17 shows examples of geocomposites. [19]

Figure 3-17 Examples of Geocomposites [19]

3.5.6 Soil-Reinforcement Interaction Mechanism

The functioning and behavior of reinforced soil are based on the interaction between two different materials: soil particles, which bear compressive stresses, and reinforcement elements, which bear tensile stresses. The combined action of these two increases the strength, stiffness, and ductility of the soil. The interaction between the reinforcement and soil includes:

1- Friction between the reinforcement and soil

2- Adhesion between the reinforcement and soil, which is a constant value depending on the adhesion and surface roughness of the reinforcement.

3- Bearing stress between the reinforcement and soil, which is essentially derived from compressive stress.

The joint friction angle between soil and geosynthetics is a crucial factor in designing these structures. The use of reinforcements, due to the tensile forces in them, provides additional shear stress in the soil mass, enhancing the reinforced soil mass's strength. Consequently, horizontal deformations are reduced, thereby increasing the overall stability of the structure. The significant increase in geosynthetics' use in reinforced earth structures has led to the development of various testing methods to evaluate the interaction properties between them (soil-geosynthetic). Research conducted so far has evaluated the interaction properties of reinforcements in granular soils, mainly due to the widespread use of granular backfills in reinforced soil walls and embankments.

3.6 Tires

Given that rubber and plastic remain in the environment for many years without biodegrading, they pose significant environmental harm. With urbanization on the rise, the use of automobiles has increased, and so has the average distance traveled by each vehicle. One environmental impact of this trend is the production of used tires, which are not environmentally friendly and are among the most troublesome and voluminous wastes. Therefore, everyone is looking for ways to manage this issue. Depending on the type of vehicle, tires are produced with various features and sizes. The details of these features, which are inscribed on each tire, are as follows:

Figure 3-18 Specifications marked on a car tire [20]

3.7 Technical Literature Overview

Hughes and Withers (1974) concluded that stone columns in soft soils act like columns in a triaxial chamber with confined cell pressure. They proposed the following equation to determine the maximum vertical stress that a stone column (gravel or sand) can withstand before reaching ultimate stress:

$$q_{uil} = \frac{(1 + sin\phi')}{(1 - sin\phi')}(\sigma_{ro} - u + 4c)$$

where c and u represent undrained shear strength and pore water pressure, respectively. ϕ' is the internal friction angle of the column materials, q_{uil} is the vertical capacity of the column, and σ_{ro} is the initial total radial stress in the soil before the column is constructed. They also demonstrated that any increase in the column length beyond a depth-to-diameter ratio of 6.3 cm does not increase the load-bearing capacity of the column. Finally, the authors emphasized that in practical applications, loads are generally applied to both the column and the surrounding clay soil. Although the applied load will lead to soil consolidation and increase radial stiffness, this increased resistance will not significantly enhance the column's capacity. [8]

Siva Kumar (2004) conducted a consolidated undrained triaxial test on a sand column with a diameter of 32 cm and a column-to-sample clay height ratio of 0.4, 0.6, 0.8, and 1, with a length of 20 cm. Furthermore, the study examined the effect of increased lateral confinement by a geogrid cover. The sand columns were prepared by moisture compaction and freezing methods. The failure mechanism indicated that in short columns, the buckling phenomenon occurred at the clay's bottom surface, whereas in fully penetrated columns, it was observed along the length of the column. Stress-strain analysis showed a greater percentage reduction in pore water pressure for fully penetrated columns, suggesting a potential delay in settlement in compacted sand columns under undrained shear conditions. [22]

Mckelvey (2004) conducted a study on the deformation-load behavior of a group of stone (sand) columns. The materials used in the experiment included kaolin clay and substances with similar properties to kaolin clay. The kaolin slurry was consolidated under a vertical pressure of 140 kPa for 8 days. The internal diameter of the load application chamber was 413 mm, and its length was 1200 mm. After one-dimensional consolidation, the chamber's length was about 500 mm, and its drained shear strength was estimated to be around 32 kPa. At the end of the consolidation, pressure was removed from the sample, and a stone column with a diameter of 25 mm was placed in the kaolin sample and filled with sand through a wired mesh. After installing the columns, a plate for load application was placed on top. For the kaolin samples, four sand columns with a square cross-section and length-to-diameter ratios of 6 (column length 150 mm) and 10 (column length 250 mm), and a replacement area of 24%, were used. The modeled column base was controlled by a strain control system at approximately 0.0064 mm. Loading was stopped when the vertical displacement of the column base reached 40 mm. Placing sand columns with a length of 150 mm and a replacement area of 24% increased the load-bearing capacity by 30%. Extending the column length to 250 mm only resulted in a 5% increase in resistance. The authors concluded that increasing the column length beyond 6 times the column diameter does not significantly increase the load-bearing capacity. However, the stiffness for columns of lengths 150 mm and 250 mm was 4 and 7.5 times greater than that of the unreinforced clay, respectively. The authors determined that longer columns are more suitable for controlling displacements. Observations of the post-failure samples showed that the failures were due to bulging, bending, or shearing. In longer columns, deformations were concentrated in the upper region, while shorter columns tended to bulge or bend outward towards other columns and sink about 10 mm into the soft clay bed. The stress concentration factor was less than 2 for shorter columns and more than 4 for longer columns, calculated immediately after load application. In later stages of load application, the stress concentration factor reached 3, regardless of the column length. [23]

Kim and Lee (2005) conducted a centrifuge test that included a loading area of 25*10*25 cm of a group of sand and gravel piles with replacement ratios of 30, 40, and 50 percent (diameters of 2.2 cm and length of 15 cm). The clay (CH) was prepared lime-treated, then consolidated in the centrifuge chamber under a pressure of 50 kPa. The pile installation process involved initially freezing the columns and arranging them orderly. After placing the columns, an effective consolidation pressure of 150 kPa was applied to the system. The undrained shear strength was then calculated using a laboratory vane. While the clay and columns were being loaded, an increase in the applied load of 1 kg-f was controlled in the test, while the settlement of the plate was also calculated. The load-settlement graphs exhibited a bilinear behavior, based on which the authors defined failure at the intersection of the two curves. The results indicated that the failure stress increased with the replacement ratio, and generally, this ratio for stone columns was higher than for sand columns. The failure stresses for the space with the highest replacement of 50% were about 30 to 40 times higher than the control clay soil. The failure stresses in the stone (gravel) columns were about 1.25 to 2.6 times higher than those in the sand columns. [24]

The research by Black et al. (2006) involved testing consolidated kaolin clay with a diameter of 30 cm and a height of 40 cm. These samples were prepared by liquefying the clay and consolidating it under a pressure of 75 kPa. Columns made from crushed basalt with a diameter of 2.5 cm were prepared using a wet compaction method. The samples were then placed in a triaxial chamber, subjected to isotropic confining pressure of 75 kPa, and loading continued while the horizontal (σ'_1) and vertical (σ'_3) stresses were increased to 125 kPa and 100 kPa, respectively. A back pressure of 200 kPa was maintained on the sample throughout the experiment. Loads were applied independently by a plate with a diameter of 6 cm at a rate of 0.8 kPa/h to ensure full drainage of the sample. This process took 2 to 3 weeks to achieve a settlement of 15 to 20 mm. Using stone columns with a length-to-diameter ratio of 6 to 10 and a replacement area in the ground of 17%, they managed to achieve an increase in load-bearing capacity between 12% and 28%. For samples with longer columns, the stress concentration factor was 1.83. This small stress factor was related to the small replacement ratio and drained loading conditions as long as the n values for the drained conditions were higher than those for undrained conditions. Observations of failure modes showed that shorter columns did not have a significant change in diameter, whereas longer columns showed significant changes in their upper regions. The authors concluded that the optimal length-to-diameter ratio for the columns ranges between 2 to 10. [25]

White (2007) conducted four load tests on a group of stone columns and single pounded columns in an area with a 13-meter-thick layer of alluvial clay soil, which was very hard due to frost and had a 1-meter dry layer on top. A concrete foundation with a thickness of 46 cm and a width of 229 cm was used for loading the group of columns. The stone columns occupied 35% of the total area of the soil and column assembly. The test results on both the group of columns and the single column showed that the upper part of the shorter columns moved more than that of the longer columns. A comparison between the test results of the single long and short columns indicated that the shorter columns exhibited greater stiffness. [26]

Black et al. (2007) prepared kaolin clay samples with a length of 20 cm and a diameter of 10 cm using a one-dimensional consolidation setup. These samples were reinforced with a column of sand frozen with a diameter of 2/3 cm (unit weight of 18.9 kN/m^3) or a sand column with a diameter of 2 cm. The lengths of the columns varied between 12 and 20 centimeters. The kaolin samples were also consolidated under a pressure of 100 kPa. Depending on whether the samples were drained or undrained, the strain rate was 0.167% per hour, and the stress rate was 1.25 kPa per hour. The deviatoric stress for fully penetrated columns showed a 33% increase compared to the control sample for a 10% area replacement. For partially penetrated columns, the increase in deviatoric stress was marginal. For the group of columns, the deviatoric stress increased by 55% for a 12% area replacement. Thus, a 20% increase in the replacement area led to a 20% increase in capacity. The authors concluded that for undrained loading, the increase depended on the resistance of the group of columns related to the percentage of the replaced area. In drained tests, the deviatoric stress at 2% strain increased from 92 kPa for undrained to 104 kPa for the group of columns (3 columns with a diameter of 2 cm) and to 112 kPa for samples reinforced with a single column with a diameter of 2/3 cm. [27]

Ambily and Gandhi (2007) conducted a series of experiments with stone columns of 10 cm diameter arranged in a triangular pattern within a 45 cm thick clay sample. The clay was prepared in a tank with heights ranging from 21 cm to 83.5 cm and a height of 50 cm. This clay, with an undrained shear strength of 14.7 to 30 kPa, was utilized for the experiments. For single column tests, the clay tank diameter varied from 21 cm to 42 cm, while for the group of seven columns, the tank diameter was 83.5 cm. The columns also had a height of 45 cm. The clay preparation involved compaction processes. The materials used in the columns included crushed stones with diameters ranging from 2 mm to 10 mm, prepared to a density of 16.62 kN/m^3, resulting in a friction angle of 43 degrees. Both surface and column loading were employed. The applied load was at a displacement rate of 0.0625 mm/min and was controlled until a settlement of 10 mm was reached at specific times. When the entire surface was loaded, no bulging was observed in the columns. After direct loading on the columns, bulging was observed at a distance of 0.5D from the column head. Based on the results obtained, the ratio of the controlled or limited axial stress to the related shear strength of the surrounding clay was independent of the shear strength of the soil and constant for the assumed (S/d) and the internal friction angle of the column materials. In the test where the entire surface was loaded, the failure in the column was not due to the confining effect of the unit cell environment. However, the stiffness of the reinforced composite significantly increased. They also proposed a design method for stone columns in soft surrounding clay and used the results of this experimental work along with numerical analysis (FEM) to develop their proposed method, presenting the following diagram (Figure 2-1) based on numerical analysis.[28]

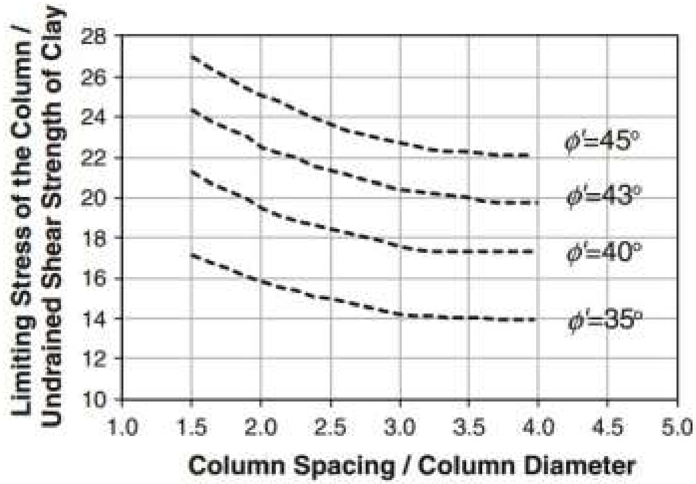

Figure 3-19 Design Chart for Stone Columns by Ambily and Gandhi (2007) [28]

They argued that a portion of the stress applied to the column is transferred to the surrounding soil. They also defined the factor (β), which is the ratio of the stiffness of the soil reinforced with stone columns to the stiffness of the unreinforced soil, and found it to be independent of the soil's surrounding strength.

Andreou (2008) conducted triaxial tests on kaolin clay reinforced with stone columns made of Houston sand and gravel. The experiments were designed to investigate the effects of drainage conditions and loading speed, and were carried out under both drained and undrained conditions. The columns used for reinforcing the soil had a diameter of 2 cm and a height of 20 cm, representing a cross-sectional area ratio of 4%. The results indicated that the increase in load-bearing capacity of the stone columns is related to the drainage conditions and the loading ratio. Specifically, the maximum deviatoric stress that the stone column could withstand under drained conditions was twice as much as that under undrained conditions, highlighting the significant impact of drainage on the performance of stone columns in reinforcing clay soils.[29]

Najjar (2010) conducted 31 undrained triaxial tests on normal consolidated kaolin samples with a height of 142 cm and a diameter of 1.7 cm, which were created and caged by ordinary quartz sand columns and tested. Confining pressures of 100, 150, and 200 kilopascals were applied to the sample. The results showed that the sand columns improved the undrained shear resistance, with the increase for the full penetration of the sand column ranging between 17.4% to 72.8%, and for the cross-sectional ratios of 7.9% and 17.8% respectively. The increase in undrained resistance was accompanied by a decrease in pore water pressure and an increase in bearing capacity [30]. Figure 2-2 shows an example of the triaxial test on stone columns.

Figure 3-20 An Example of a Triaxial Test on Stone Columns [30]

Black (2011) conducted a triaxial test on clay samples with a diameter of 30 cm and a depth of 40 cm that had been consolidated. Kaolin was consolidated under a pressure of 150 kpa, resulting in a shear strength of 35 kpa. Gravel columns with diameters of 2.5 cm, 2.8 cm, and 3.2 cm (cross-sectional ratios of approximately 17% and 40%) were placed in the soil using the replacement method. These columns were compacted by dropping 1 kg weights from a height of 5 cm to achieve a density of 15.5 kN/m^3. The comparison of settlements for columns penetrating the bedrock layer showed that settlement decreased with increasing depth, and foundation loading revealed that improvement factors for settlement increased with the L/D ratio for a given cross-sectional ratio, although this increase appears to have occurred between L/D ratios of 8 to 10. [31]

Stuedlin and Holtz (2012) conducted 20 vertical load tests on single and group stone columns with a diameter of 76 cm placed at depths of 305 and 457 cm in clay soil. The in-situ analysis showed a significant difference between the site test and the average undrained shear strength in the upper 5 to 6 meters, which ranged between 44 to 70 kilopascals. This test was designed to investigate the effect of the column compaction method and the impact of pre-drilling and mixed cement coating on column performance. [32]

Mahmoud Ghazavia & Javad Nazari Afshar (2013) conducted 18 tests on single columns and 3 tests on column groups to study the effect of reinforcement casing and its type and length with different materials on single columns and groups of columns. In these tests, the columns had diameters of 60, 80, and 100 millimeters and were made of crushed stone materials, spaced 150mm apart.

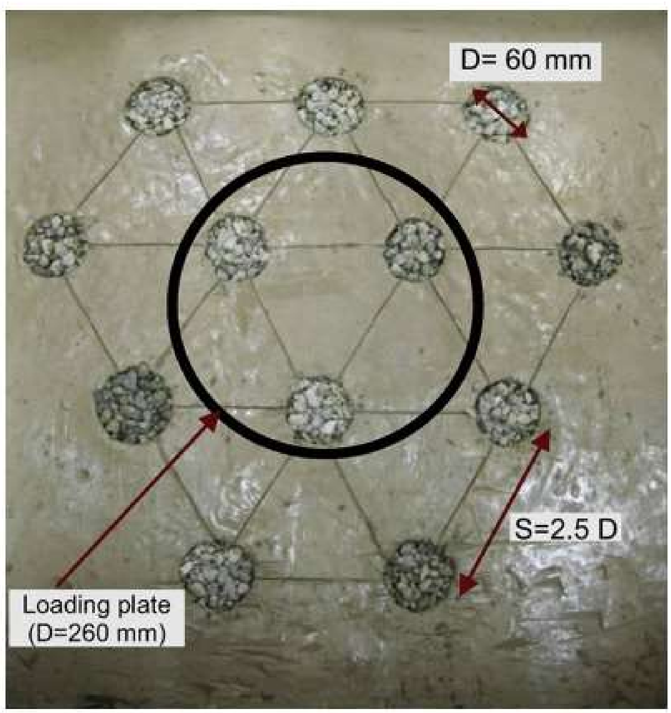

Figure 3-21 shows the group arrangement of stone columns.[33]

They concluded that the increase in load-bearing capacity and the increase in radial strain are directly proportional to the column diameter. Additionally, column failure is in the form of bulging and with an increase in diameter for a column that is half-covered with geotextile, the impact of the column on the load-bearing capacity of soft soil increases. [33]

Figure 3-22 shows an example of a vertical loading test on stone columns [33].

3.8 Conclusion:

This chapter discussed some soil improvement methods, especially stone columns, some execution and design requirements for them, and the use of geosynthetics and tires in constructing these columns and reviewed some of the research conducted by scholars in recent years. The next chapter will describe the experiments conducted using the foundation simulator device.

Chapter Three: laboratory model

4.1 introduction

In this chapter, detailed explanations of the experiments conducted on soil improvement with stone columns are provided, along with related diagrams and tables. A pile simulator device was designed for the planned experiments, as shown in Figure 3-1. The various parts of the device are described below:

- Jack: A hydraulic jack system with a maximum vertical force of 4 tons is used to apply vertical force to the soil and column samples.

- Metal Box: Soil and column samples are prepared in a large metal box measuring 120x120x90 cm.

- Device Frame: The main frame of the device consists of a solid metal frame on which a hydraulic jack load application system is installed. This frame has high stiffness against the load applied by the jack to ensure there is no displacement under the stress applied by the jack, and in other words, to ensure that all the force applied by the jack is directed downwards into the soil and no strain is induced in the frame.

- Displacement Control System: This part of the device is used to control the displacement of the load application jack system on the test sample with an accuracy of one hundredth of a millimeter. To measure the displacement, two LP numbers were used, which were placed on both sides of the loading plate to measure both the amount of displacement and to ensure the uniformity of the settlement.

- Force Control System: To accurately measure the force applied by the jack to the test sample, a sensor is placed between the jack and the test sample.

- DAQ: stands for "Data Acquisition," which is the process of collecting, measuring, and analyzing real-world physical conditions and converting the resulting samples into digital numeric values that can be manipulated by a computer. A DAQ system typically includes sensors, DAQ measurement hardware, and a computer with programmable software.

The sensors respond to physical phenomena such as temperature, pressure, or force, and convert these into electrical signals. The DAQ hardware then converts these analog signals into digital data that can be analyzed by software. This process allows for the monitoring, control, and analysis of physical properties in a wide variety of applications, from simple temperature monitoring to complex industrial automation and control systems.

Figure 4-1 Test Box with hydraulic jack and DAQ

4.2 Material and Substance Studies

4.2.1 sandy Soil

In this section, studies on materials and substances, specifically sandy soil, are discussed. Based on the problem statement presented in this thesis, experiments were conducted on a soil sample available in the laboratory, which has a high similarity to the soil found in the area. Initially, disturbed soil samples were collected for preliminary testing. Various tests were conducted on the soil, including particle size distribution, determination of initial soil moisture, determination of the specific gravity of undisturbed soil, Atterberg limits, and determination of minimum and maximum particle densities, among others. The results of these tests are briefly presented in Table 3-1. Additionally, detailed descriptions of these tests are provided below.

Table 4-1 Characteristics of Sandy Soil

Property of sand:

Parameters	Value
Plastic limit (%)	20
Optimum moisture content (%)	18
soil unit weight (γ) (kN/m^3)	16.8
Poisson's ratio (v)	0.2
Cohesion (C) (kpa)	1
Internal friction angle (ϕ)	36
USCS Classification Symbol	SP

4.2.1.1 Determination of Initial Soil Moisture

To determine the initial moisture content of the soil, 20 soil samples were collected from various parts of the area in a scattered manner. The test for determining the initial moisture percentage was conducted as follows: a certain amount of soil was placed in sampling containers, and their weight was recorded. The samples were then placed in an oven for 24 hours at a temperature of 105 degrees Celsius. After this period, the samples were weighed again, and the weight of the water in the samples was determined by subtracting these two weights. By dividing this weight by the dry weight, the moisture percentage was determined, resulting in an average moisture content of 6%.

4.2.1.2 Grain Size Distribution

For this test, approximately 1.5kg of soil, representing the overall soil volume, was washed through a No. 200 sieve, and the residue remaining on this sieve after drying in the oven was prepared for standard sieve analysis. The procedure involves drying the soil and placing it on a series of sieves, which is then shaken for 5 to 7 minutes on a shaker. The soil remaining on each sieve is weighed and recorded, and the percentage passing through each sieve is calculated. These percentages are plotted on a semi-logarithmic graph against the particle sizes. The soil passing through sieve No. 200 was also used for the hydrometer test, where the soil was mixed with 4% sodium hexametaphosphate solution and diluted to 200ml with distilled water. The mixture was placed in a graduated cylinder, and a hydrometer was placed inside. Readings were taken every 24 hours, and the values were converted to particle diameters using relevant formulas and included in the grain size distribution chart. The grain size distribution chart is presented in Figure 3-2.

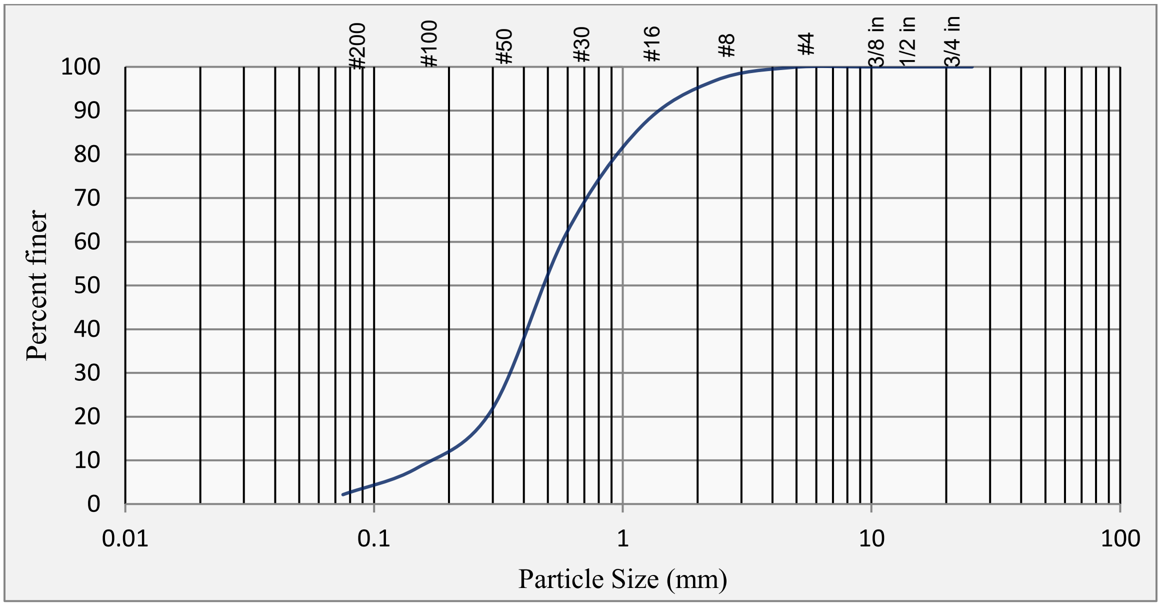

Figure 4-2 Soil Grading Diagram for sand

4.2.1.3 Atterberg Limits

To determine the mechanical properties of the soil, tests for determining the plastic and liquid limits were conducted. The plastic limit test, known as the Casagrande test, involves initially passing a certain amount of soil through a No. 40 sieve, then adding water to reach saturation, and leaving the soil for 24 hours to allow water to encompass all soil particles. After this, some of the prepared soil is placed in the Casagrande device's cup, a groove is made, and the device's handle is turned to apply blows to the cup. The number of blows counted should reach 25, but since obtaining this exact number can be difficult, numbers between 20 and 30 blows are also considered acceptable. If the test is repeated and the number of blows is less, it indicates low moisture, requiring additional water. If the numbers are above the mentioned range, it suggests excessive moisture, which should be reduced by kneading. If a number other than 25 is obtained, it is multiplied by a coefficient to convert it to this value. In the plastic limit test, after saturating the soil, a thread of 3.2mm diameter is made. If the soil starts to crack at this stage, it indicates the desired moisture content. If no cracks are formed, it means the moisture is excessive and should be reduced by kneading. The results of these tests are presented in Table 3-1.

4.2.1.4 Determination of sand Particle Density

For conducting this experiment, we first select a required amount of soil and determine its moisture content. Then, we place the soil in three layers inside the test cylinder and compact each layer with 25 hits using a special hammer that has a constant weight and a fixed drop height to ensure uniform compaction energy in all tests. Before doing this, we measure the internal dimensions of the cylinder and calculate its exact volume, and also determine the weight of the empty container. After compacting the soil in the mold, we weigh it, and by subtracting the weight of the empty container, we obtain the amount of soil in the mold. By dividing this weight by the volume of the mold, we obtain the specific weight of the soil. Then, we remove the soil from the mold to take a sample for determining the moisture content. In the next step, we add water to the soil sample, amounting to two to three percent of the soil's weight, and repeat the above experiment process to determine its specific weight and moisture content. With the addition of water and compaction, it is observed that the weight of the soil sample in the mold increases, and after passing a specific point, the weight of the sample decreases. After observing this decrease in weight, the experiment is concluded, and with the data of specific weight and moisture content from repeated tests, a graph of specific weight versus moisture content of the sample is drawn. This graph, showing the peak observed, represents the maximum specific weight of the soil against the optimum moisture for achieving this level of compaction. This graph is shown in Figure 3-3.

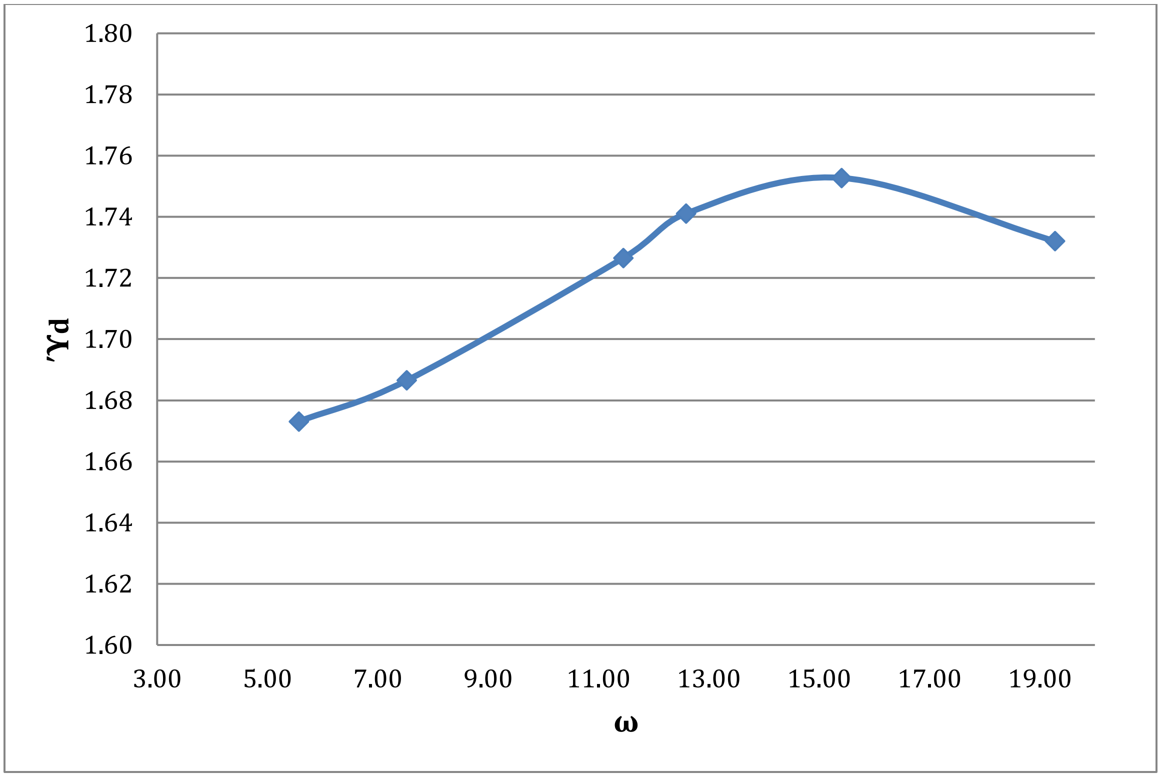

Figure 4-3 Soil Density

4.2.2 Gravel Aggregates

For the gravel materials used to fill the columns in the test box, aggregates with a maximum dimension of 1/6 of the column diameter were used. Various tests were conducted on the gravel sample to determine the characteristics of the gravel particles, and the results are presented in Table 3-2. The description of these tests is detailed below.

Table 4-2 Characteristics of Gravel

Property of Gravel:	
Parameters	Value
Specific gravity	2.7
Maximum dry unit weight (kN/m^3)	16.6
Minimum dry unit weight (kN/m^3)	14.9
Bulk unit weight for test at 68% relative density (kN/m^3)	16
Internal friction angle (4) at 68% relative density	46
Uniformity coefficient (Cu)	2.16
Curvature coefficient (Cc)	1.15
Unified system classification	GP

4.2.2.1 Grain Size Analysis

About 1.5 kg of soil, representing the overall soil volume, was washed through a No. 200 sieve, and the remaining material on this sieve after drying in an oven was prepared for grain size analysis using standard sieves. The grain size distribution chart is presented in Figure 3-4.

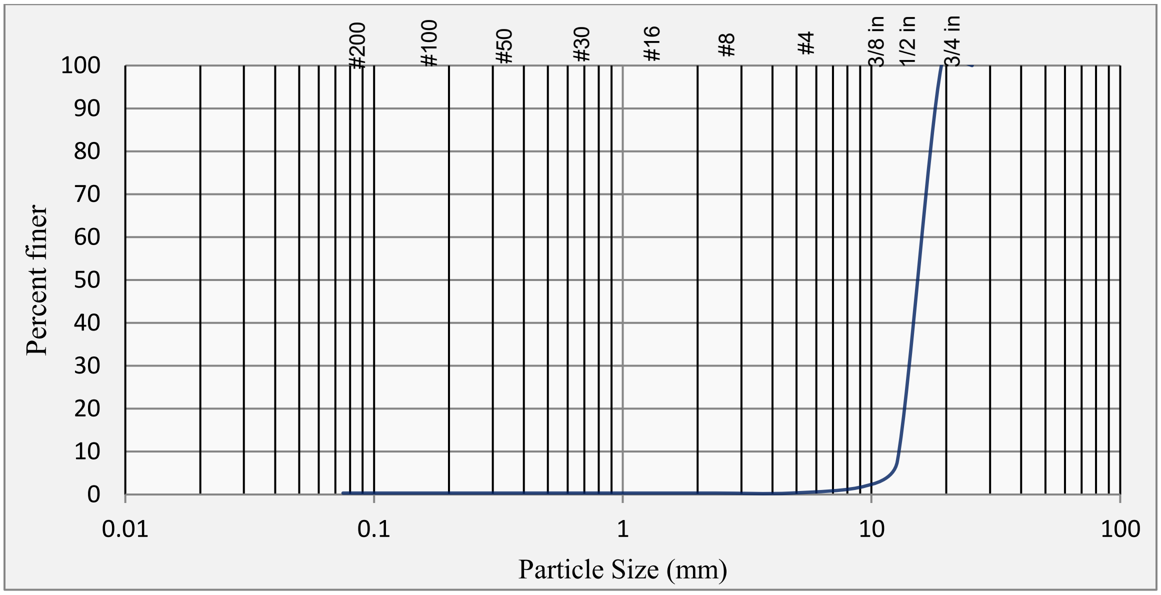

Figure 4-4 Aggregate Grading Diagram

4.2.2.2 Direct Shear Test on Aggregates

To determine the parameters C and ϕ of the aggregates, a direct shear test was conducted. The procedure involved placing the aggregates into the device's box and applying a vertical load that represents the natural overburden pressure on the soil. The device's chamber was then filled with water and allowed 16-18 hours for the water to saturate all soil particles. Following saturation, a shear force was applied to the soil using the device, and the maximum force the soil could withstand before failing was recorded. This test was repeated at least three times with different overburden pressures, and from the obtained data, along with Mohr-Coulomb circles, the parameters C and ϕ were derived. The results are presented in Table 3-2.

4.2.3 Geotextiles

In several experiments, geotextile coverings made of polypropylene were used around the columns. The specifications of the used geotextiles are provided in Table 3-3.

Table 4-3 Characteristics of Geotextile

Specifications	Unit	Value
Ultimate Tensile Strength	kN/m	**9**
Strain at Ultimate Strength	%	55
Secant Stiffness at Ultimate Strain	kN/m	16.5
Thickness	mm	3
Mass	g/m^2	140

4.2.4 Tire-Derived Aggregate (TDA)

Considering the need to use recycled tire chips to replace the gravel materials in stone columns, the available resources in the laboratory were utilized, which consisted of Tire-Derived Aggregate (TDA) of various sizes. To begin, a portion of the TDA was selected and visually inspected, and parts containing wires and impurities were removed from the collection. In the next step, from a dimensional perspective, samples with sizes similar to the gravel materials used in the columns and smaller than 1/6 of the column diameters (less than 10 millimeters) were chosen. These were then sieved out of the collection and further examined. In Figure 3-5, a sample of TDA is displayed.

Figure 4-5 TDA Size

The replacement of tire chips with gravel materials was done by weight. In this regard, in two experiments, 10 and 20 percent of gravel materials were replaced by weight with TDA, and the results were compared with the case of a stone column made with pure gravel materials.

Figure 4-6 TDA Replacement

Additionally, the results of the experiments on crumb rubber, aimed at finding various parameters of this material, have been presented in Table 3-4.

Table 4-4 TDA Parameters

Property of TDA:	
Parameters	Value
Particle unit weight (γ)(kN/m^3)	11.3
Maximum dry unit weight (kN/m^3)	4.3
Minimum dry unit weight (kN/m^3)	3.4
Maximum void ratio (e_{max})	2.32
Minimum void ratio (e_{min})	1.6

4.3 Preparation of sand Bed

To achieve the desired moisture content, the required moisture amount for a 2-centimeter-thick layer in the main box with dimensions of 1.2×0.9×0.9 meters was first calculated. Then, as 2-centimeter soil layers were created in this box, moisture was uniformly added to the mentioned box and was protected for 7 days by a polypropylene cover that prevents moisture exchange with the environment, and its surroundings were completely sealed with adhesive tape. After ensuring the correct moisture level through a moisture content test, the soil was removed from the box and stored in an insulated plastic bag for testing. Considering the specific weight of the soil ($\gamma = 17.7\ kN/m^3$), the soil was added to the main box in several stages and each layer was compacted until the determined specific weight was achieved, and this process continued until the box was completely filled. After filling the box, 2 days were allowed for the soil moisture to equilibrate in the sample, and then tests were conducted on the columns.

4.4 Creating Individual Columns in sand

To fill the box with sandy soil, the inside of the box was first sealed to prevent moisture loss. Then, based on the specific weight set for the soil, the prepared soil with the desired moisture content was weighed for a 5-centimeter layer and poured into the main box. After laying the soil to the aforementioned thickness, the height of the soil surface was measured from a base level. If the soil layer thickness exceeded 5cm, it was compacted using a specially made compactor for the test until the desired thickness was achieved. The compactor consists of a 30*30cm metal plate accompanied by a weight that is raised to a fixed height of 30 centimeters and dropped freely onto the plate, delivering an impact with a total weight of approximately 30 kilograms. To ensure uniform compaction during the soil compaction process, the weight was dropped from a specified height a fixed number of times onto the soil.

Figure 4-7 tamper for compaction bed sandy soil

The box was filled in this manner and leveled off, then the center of the box was measured and marked for the creation of a stone column. Before creating the space for the stone column, the compacted soil in the box was given 2 days, during which the surface of the box was covered with two layers of plastic to prevent moisture exchange with the surrounding environment. A 50mm diameter tube was used to create stone columns in the center of the box. For the placement of the metal tube, both the inside and outside of the tube were coated by thin layer of oil and driven into the soil vertically using a uniform hydraulic jack force. During the tube insertion, its verticality was continuously checked to promptly correct any deviations from vertical alignment. After placing each tube, the soil inside them was removed using an auger, for this task, a thick nylon layer with a hole in the center matching the diameter of the tube was placed on the soil bed. All the soil extracted from within the tube was carefully collected into a sealed plastic bag. After the soil removal was completed, the plastic bag was quickly sealed and weighed. This method allows for verifying the compaction consistency of different soil layers, the overall specific weight, and by determining the moisture content of the extracted soil, the accuracy and uniformity of the soil bed moisture can be assured, and at this point, the height of the stone column was precisely measured to ensure the correct height. To remove the tube from the soil, hydraulic jack force was used in such a way that three holes were drilled at the top of the tube, 120 degrees apart, to evenly distribute the pulling force across the tube's surface, ensuring the tube remained vertical and the inner wall of the created hole was not damaged. A wire passed through these holes was connected to the jack, and the jack and tube were gently moved upwards, pulling the tube out of the soil. The hole was then filled with aggregates with a specific weight of 1.6 kN/m^3. To achieve this specific weight, a special 2kg compactor was used, dropped from a height of 10 centimeters. Initially, the column height was divided into 5 parts, and the required amount of aggregate was weighed, poured into the hole, and compacted 10 times with the special compactor after filling each section. The height was accurately measured, and if there was any deviation from the required length, compaction was repeated until the desired height was reached. This process continued until the soil surface was reached, and the column was completely filled with aggregates. Figure 3-9 shows the placement of the tube in sandy soil and Figure 3-10 presents the prepared single column with a diameter of 50 millimeters in the box.

Figure 4-8 Placement of Tube in Soil

Figure 4-9 Creating a Single Column with a Diameter of 50 mm in sand

4.5 Single Column Test Method

This experiment was conducted using a controlled displacement method, with settlements up to 50 mm for each column. To conduct the experiments, four sets of variations were considered, and for each, three sets of experiments with changes in the amount of TDA materials were taken into account. The schematic representation of the experiments is shown in the figure below.

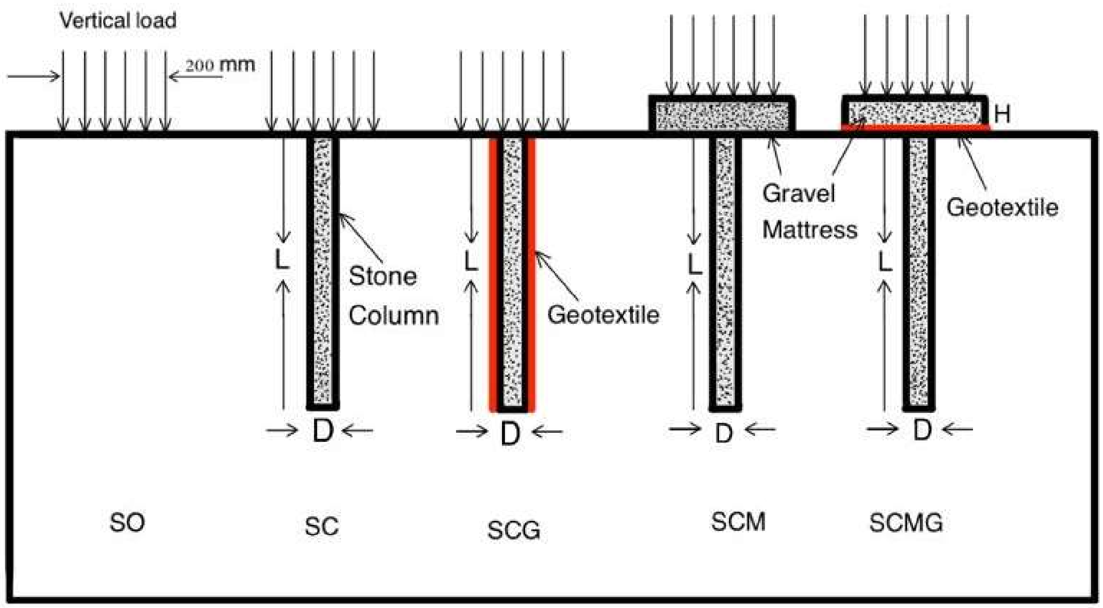

Figure 4-10The schematic representation of the experiments

Additionally, the title and abbreviation of each experiment are as follows:

- SO: sand bed
- SC: sand bed with stone columns
- SC10: sand bed with stone columns and replacement 10% TDA inside the columns
- SC20: sand bed with stone columns and replacement 20% TDA inside the columns
- SCG: sand bed with stone columns and geotextile cover, around stone column
- SCG10: sand bed with stone columns and geotextile cover, around stone column and replacement 10% TDA inside the columns

- SCG20: sand bed with stone columns and geotextile cover, around stone column and replacement 20% TDA inside the columns
- SCM: sand bed with stone columns and gravel mattress top of the stone column
- SCM10: sand bed with stone columns and gravel mattress top of the stone column and replacement 10% TDA inside the columns
- SCM20: sand bed with stone columns and gravel mattress top of the stone column and replacement 20% TDA inside the columns
- SCMG: sand bed with stone columns and geotextile + gravel mattress, top of the stone column
- SCMG10: sand bed with stone columns and geotextile + gravel mattress, top of the stone column and replacement 10% TDA inside the columns
- SCMG20: sand bed with stone columns and geotextile + gravel mattress, top of the stone column and replacement 20% TDA inside the columns

Initially, with the stone column's center positioned under the center of a 20 cm diameter and a thickness of 3 cm plate (to ensure the rigidity of the loading plate) attached to a hydraulic jack, loading began at a constant speed of 1 mm/min. The force applied was recorded by a load cell against the displacement of the loading plate. The displacement was measured by two LPs with an accuracy of 0.01 millimeters, which were installed after setting up the loading plate on them, and the vertical alignment of each was checked in both directions. For columns with geotextile, cylindrical geotextiles of appropriate diameters were prepared and placed inside the columns excavated in sandy soil. To simulate stone columns with TDA, the required amount of TDA was weighed, replaced with the column's aggregates, poured into the columns, and compacted until the desired specific weight was achieved. Figure 3-11 shows a stone column after loading, where bulging at the top of the column is observed. After the experiment and obtaining the results, the box was emptied and all the above steps were repeated for the next experiment.

Figure 4-11 Bulging of the Stone Column After Loading

4.6 Conclusion

In this chapter, the focus is on examining the experiment, the type and method of preparing the soil bed, and how to ensure the uniformity of experiments. It also discusses how loads are applied and the response of the soil bed. In the next chapter, the discussion will revolve around the results of the experiments and the graphs depicting various experiment types.

CHAPTER FOUR: RESULTS AND DISCUSSION

5.1 Introduction

In this chapter, an analysis of the results obtained from the plate loading tests on individual stone columns under various conditions is conducted. This includes the use of geotextiles for confinement, different percentages of Tire-Derived Aggregate (TDA) as substitutes for stone materials, and the use of gravel mattress, among others. Furthermore, discussions are held regarding the reasons for the increase in load-bearing capacity of stone columns and determining the optimal percentage for substituting TDA.

5.2 Effect of Geotextile Cover and gravel mattress on the Load-Bearing Capacity of a Single Column

The load-settlement diagram for a displacement of up to 50 mm, for a column with a diameter of 50 millimeters, is shown in Figure 3-12. By comparing the diagrams, it can be concluded that the use of a gravel mattress and a geotextile cover beneath the gravel mattress on top of the stone column increases the load-bearing capacity of the soil-column assembly by about 50%. This seems to be due to the increased force application area and the increased interaction surface between the soil and the stone column in resisting the load, which enhances the column's load-bearing capacity. Furthermore, the use of geotextile beneath the gravel mattress results in approximately a 73% increase in load-bearing capacity. This increase is attributed to the broader distribution of force over the top of the stone column, which enhances the overall load-bearing capacity of the assembly. Additionally, referring to the graph below, it is evident that using stone columns in the soil bed reduces the settlement of the soil under loading. Under a load of 9.2 kN, which is the maximum tolerable load by unreinforced soil, a settlement of approximately 10.5 mm occurs. However, under the same load, the settlement in soil improved with stone columns is around 7 mm. With the combination of stone columns and geotextile under gravel mattress, the settlement is reduced to 4 mm, which is equivalent to the settlement in soil reinforced with stone columns and a gravel mattress on top. However, when force exceeds 11 kN, the settlement notably decreases when using a geotextile underneath the gravel mattress compared to the case without it. Under a load of 12.5 kN, the settlement without geotextile is around 12 mm, whereas with the use of geotextile under the gravel mattress, the settlement is limited to 9.5 mm.

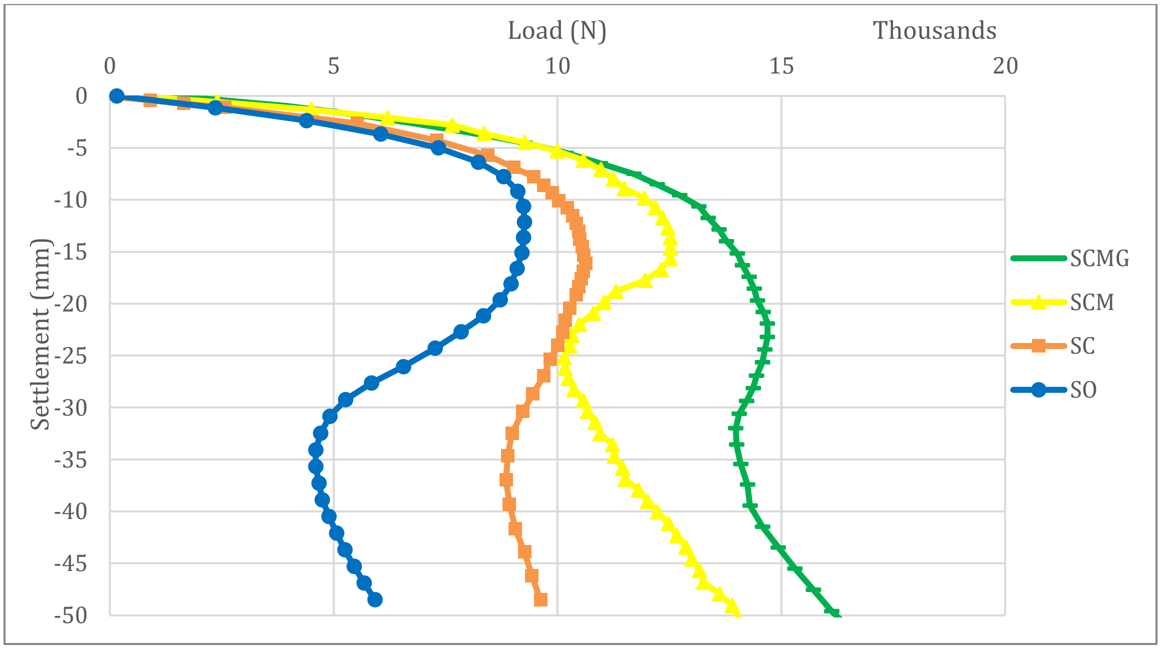

Figure 5-1 Load-Settlement Diagram for a 50 mm Column

Table 5-1 Comparison of Load-Bearing Capacity Changes with Gravel Mattress and Geotextile Above the Column

Test Type	Load-Bearing Capacity of the Column (N)	Percentage Increase in Load-Bearing Capacity
Soil without Stone Column (SO)	5922	-
Soil with Stone Column (SC)	9615	1
Soil with Stone Column and gravel Mattress (SCM)	14457	50
Soil with Stone Column, gravel Mattress, and Geotextile (SCMG)	16609	73

5.3 Effect of Using Tire-Derived Aggregate (TDA) on the Load-Bearing Capacity of Columns

Comparing the diagrams, it can be concluded that replacing 10% of the stone column material with used TDA increases the load-bearing capacity of the column by 38%, while a 20% replacement results in a 19% increase in load-bearing capacity. It appears that due to the compressibility of the TDA, these materials absorb the pressure applied to the column, thereby increasing the overall load-bearing capacity. However, when the amount of these materials is increased to 20%, given that the density of these materials is lower than that of the stone materials, it leads to a decrease in the column's load-bearing capacity compared to the previous case. Also as illustrated in the graph below, the use of Tire-Derived Aggregate (TDA) as alternative materials also leads to a significant reduction in soil settlement when combined with stone columns. Under a load of 9.2 kN, which was explained in the previous section for comparison purposes, the settlement in the case of stone columns with 10% replacement of materials with TDA is 1.2 mm, while this value increases to 3.9 mm when the replacement percentage is increased to 20%. Moreover, under a load of 11 kN, the settlement in the case of 10% material replacement with TDA is limited to 3.2 mm, whereas with 20% replacement, the settlement reaches approximately 10.8 mm.

These results indicate that the use of Tire-Derived Aggregate (TDA) as alternative materials instead of gravel in stone columns can lead to a noticeable increase in the soil's load-bearing capacity. Moreover, considering the reduction in settlement of the soil-column system, the utilization of these alternative materials in structures sensitive to settlement can significantly contribute to increasing the factor of safety against settlement. Also, it can be inferred that there is an upper limit to the use of TDA, where replacing these materials with gravel materials increases the column's load-bearing capacity and reduce the settlement until this limit is reached, but beyond this point, the column's load-bearing capacity will decrease and settlement will increase.

Figure 4-2 shows the load-settlement diagram for a stone column with 10% and 20% replacement of TDA with gravel materials. Additionally, the percentage increase in load-bearing capacity in this case is shown in Table 4-2.

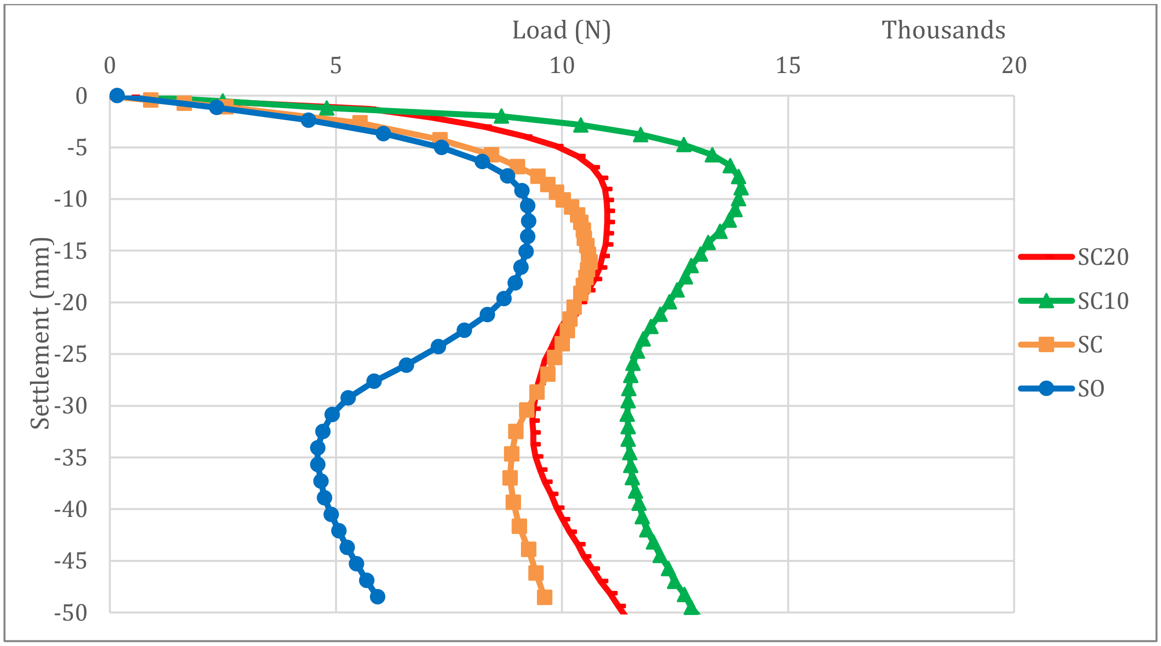

Figure 5-2 Load-Settlement Diagram for Stone Column with TDA Replacement

Table 5-2 Comparison of Load-Bearing Capacity Changes in Columns with TDA Replacement

Test Type	Load-Bearing Capacity of the Column (N)	Percentage Increase in Load-Bearing Capacity
Soil without Stone Column (SO)	5922	-
Soil with Stone Column (SC)	9615	1
Soil with Stone Column and 10% TDA (SC10)	13289	38
Soil with Stone Column and 20% TDA (SC20)	11433	19

5.4 Examination of a Single Stone Column Covered with Geotextile and Material Replacement

In this section, the load-bearing capacity of single stone columns covered with geotextile and also replacing 10% and 20% of the gravel column materials with TDA, is compared to an uncovered stone column in full. If 10% TDA are used along with a geotextile cover around the column, the stone column's load-bearing capacity increases by approximately 58%, whereas without the use of TDA, the increase in load-bearing capacity is only 43%. Replacing 20% of the materials with TDA results in a 34% increase in load-bearing capacity, which is less than the case of a simple column with geotextile cover, indicating that the optimal replacement threshold has been exceeded.

Regarding the use of geotextile cover, due to the confinement of the column, the area around the stone column acts like lateral support, thereby reducing column buckling and increasing the load-bearing capacity of the column.

By referring to the relevant graph, it can be observed that using geotextile as a confinement around the stone column not only increases the load-bearing capacity of the stone column but also reduces the settlement significantly. For instance, under a reference load of 9.2 kN, the settlement in soil improved with stone columns is approximately 7 mm, whereas in the soil improved with stone columns and geotextile confinement, the settlement is around 3.8 mm. Moreover, by replacing some of the gravel materials in the column with TDA and having geotextile as confinement, the settlement decreases noticeably. In the case of 20% material replacement with TDA, the settlement is about 1 mm, and in the case of 10% material replacement with TDA, the settlement is approximately 1 millimeter also (Under small loads, the settlement in both cases of 10% and 20% material replacement is approximately equal). Furthermore, in another comparison, under a load of 12.2 kN, the settlement for stone columns with 20% material replacement with TDA is about 3.5 mm, and for 10% material replacement, the settlement is limited to 2 mm. This indicates less settlement against higher load tolerance and suggests a significant potential for using TDA as an alternative material.

Figure 4-3 shows the load-settlement diagram for the stone column in the mentioned conditions.

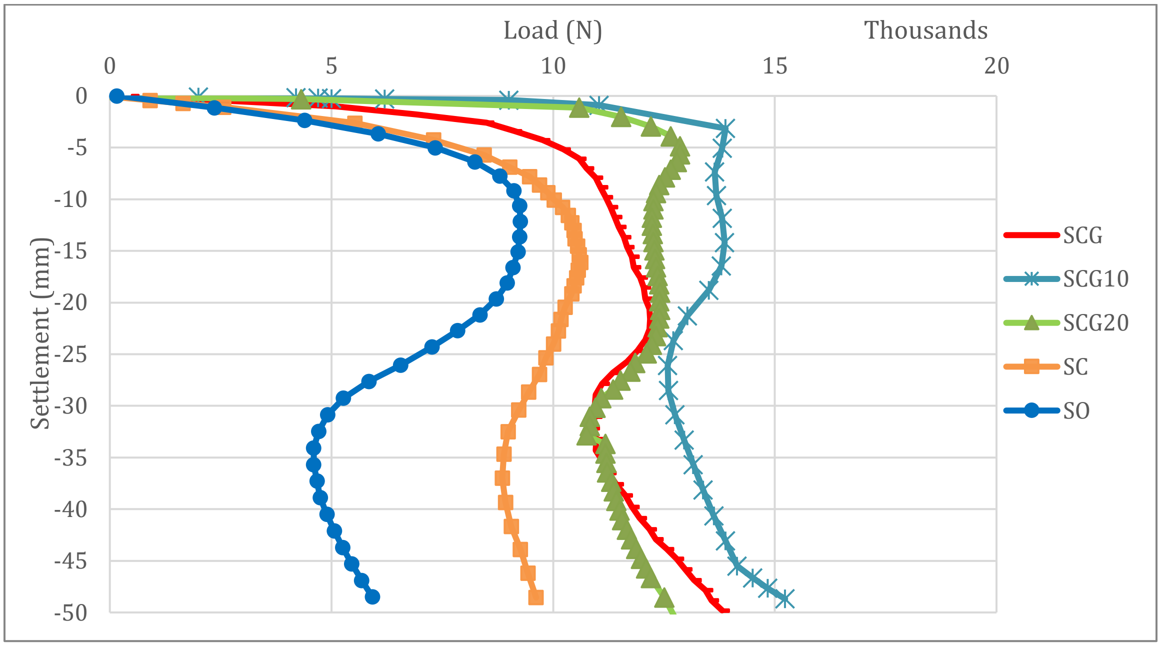

Figure 5-3 Load-Settlement Diagram for Stone Column Using Geotextile and Material Replacement

Table 5-3 Comparison of Load-Bearing Capacity Changes in Columns with TDA Replacement and Geotextile Encasement

Test Type	Load-Bearing Capacity of the Column (N)	Percentage Increase in Load-Bearing Capacity
Soil without Stone Column (SO)	5922	-
Soil with Stone Column (SC)	9615	1
Soil with Stone Column and Geotextile Cover (SCG)	13810	43
Soil with Stone Column with 10% TDA and Geotextile Cover (SCG10)	15228	58
Soil with Stone Column with 20% TDA and Geotextile Cover (SCG20)	12933	34

5.5 Examination of a Single Stone Column with gravel mattress and Material Replacement

In this section, the load-bearing capacity of single stone columns accompanied by a gravel mattress, as well as replacing 10% and 20% of the gravel column materials with TDA, is compared to a simple stone column. It is observed that replacing 10% of the gravel materials with TDA materials shows an increase in the load-bearing capacity of the stone column by approximately 55%. Replacing 20% of the materials shows an increase in load-bearing capacity of about 12% compared to the simple stone column, which suggests that this level of replacement does not significantly enhance the load-bearing capacity.

This section, which pertains to the examination of the gravel mattress on top of the stone column, again demonstrates a reduction in settlement when using a gravel mattress compared to cases with simple stone columns. For instance, against a reference load of 9.2 kN, the settlement in a stone column is 7 mm, while in the case of soil improvement with a stone column along with a gravel mattress on top, the settlement is approximately 4 mm, indicating a reduction in settlement under a constant load. Additionally, in the case of replacing 20% of the stone column materials with TDA, the settlement is 4 mm, and for 10% material replacement with TDA, the settlement is limited to about 2 mm. Furthermore, when comparing with higher loads, this reduction in settlement becomes more pronounced. For example, against a load of 12.3 kN, the settlement in a stone column with a gravel mattress on top is approximately 11.5 mm, while with 10% material replacement with TDA, the settlement is limited to 4.2 mm, indicating a significant reduction in settlement and the effectiveness of using TDA instead of gravel materials. However, with an increase in the amount of TDA to 20% of the stone column materials, the settlement reaches around 7.5 mm, which does not show a significant difference. Therefore, it seems that this level of TDA material replacement does not provide acceptable results in terms of load-bearing capacity and settlement reduction.

Figure 4-4 displays the load-settlement diagram for the stone column in the mentioned conditions.

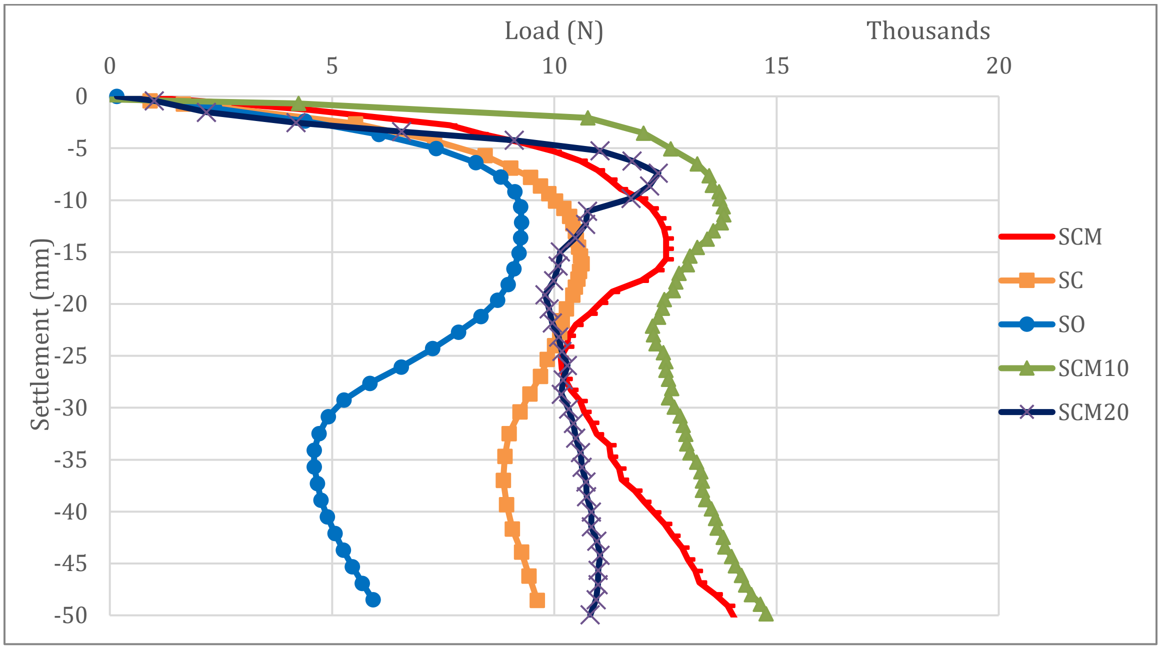

Figure 5-4 Load-Settlement Diagram for Stone Column Using Gravel Mattress and Material Replacement

Table 5-4 Comparison of Load-Bearing Capacity Changes in Columns with TDA Replacement and Gravel Mattress

Test Type	Load-Bearing Capacity of the Column (N)	Percentage Increase in Load-Bearing Capacity
Soil without Stone Column (SO)	5922	-
Soil with Stone Column (SC)	9615	1
Soil with Stone Column and gravel Mattress (SCM)	14457	50
Soil with Stone Column and 10% TDA with gravel Mattress (SCM10)	14896	55
Soil with Stone Column and 20% TDA with gravel Mattress (SCM20)	10806	12

5.6 Examination of Single Stone Columns with gravel mattress, Geotextile, and Material Replacement

In this section, the load-bearing capacity of single stone columns accompanied by a gravel mattress and geotextile beneath the gravel mattress, as well as replacing 10% and 20% of the gravel column materials with TDA, is compared to a simple stone column. It is observed that the simultaneous use of a gravel mattress and geotextile beneath it increases the column's load-bearing capacity by about 73% compared to the simple case. Replacing 10% of the gravel materials with TDA results in an approximate 95% increase in load-bearing capacity, while increasing the use of TDA to 20% results in a 57% increase in load-bearing capacity compared to the simple case, which is about 16% less than the case with the gravel mattress and geotextile. Although this replacement might not be as successful in terms of increasing load-bearing capacity, from an environmental and economic perspective, this replacement can be completely justified compared to the situation before material replacement. It not only has the potential to remove 20% of the TDA by weight from the environment but also can reduce the use of stone materials in such columns by 20%, which can be economically beneficial in large projects.

The final section related to soil improvement with stone columns and the use of a gravel mattress with a layer of geotextile underneath it. As indicated in the graph, the settlement significantly decreases when using the combination of a gravel mattress with geotextile, which is highly acceptable. In this scenario, under a load of 9.2 kN, the settlement in the stone column with a gravel mattress alone is 7 mm, while in the combination of stone column with a gravel mattress and geotextile, this value decreases to 4.5 mm. In the combination of a gravel mattress with geotextile and replacing 10% of the stone column materials with TDA, the settlement is 4.5 mm, and in the case of replacing 20% of the stone column materials, the settlement is 5.5 mm, indicating a reduction in settlement under the applied load. Moreover, when comparing with higher loads, under a load of 14.2 kN, the settlement in the stone column with a gravel mattress and geotextile is 17 mm, and in the case of replacing 10% of the materials with TDA, the settlement is reduced to 9 mm. However, in the case of replacing 20% of the materials with TDA, the settlement reaches 20 mm. In this scenario, replacing materials by 10% seems to provide more acceptable results compared to other cases. Figure 4-5 shows the load-settlement diagram for the stone column in the mentioned conditions.

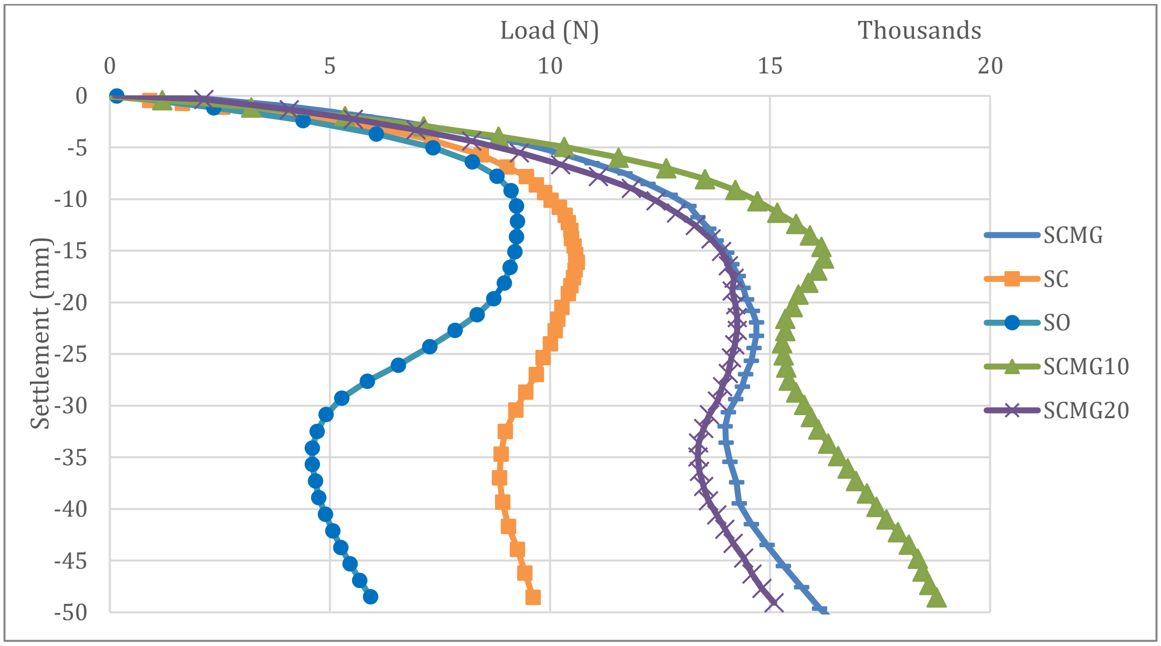

Figure 5-5 Load-Settlement Diagram for Stone Column Using Geotextile, Gravel Mattress, and Material Replacement

Table 5-5 Comparison of Load-Bearing Capacity Changes in Columns with TDA Replacement along with Gravel Mattress and Geotextile

Test Type	Load-Bearing Capacity of the Column (N)	Percentage Increase in Load-Bearing Capacity
Soil without Stone Column (SO)	5922	-
Soil with Stone Column (SC)	9615	1
Soil with Stone Column, gravel Mattress, and Geotextile (SCMG)	16609	73
Soil with Stone Column, 10% TDA, gravel Mattress, and Geotextile (SCMG10)	18783	95
Soil with Stone Column, 20% TDA, gravel Mattress, and Geotextile (SCMG20)	15094	57

5.7 Conclusion

This chapter provided detailed explanations about the conducted experiments and presented the related diagrams and tables. Among the most significant findings, an increase in the load-bearing capacity of the soil-column assembly was observed with the replacement of 10% of the gravel materials with TDA.

Chapter FIVE: Conclusion

6.1 Introduction

Previous studies have investigated the effects of stone columns on enhancing the ultimate strength of weak soils. This research also considers the utilization of TDA and their impact on the load-bearing capacity of these columns, alongside controlling and validating previous studies.

This chapter will examine the results obtained from the series of experiments and conclude with suggestions for future research.

6.2 General Conclusion

In this study, loading tests were conducted on soil reinforced with 50 mm diameter stone columns within sandy soil with 6% moisture content. The columns, built to a height ten times their diameter, were tested and their results compared with those of unreinforced soil. The following results were obtained:

1. Observations indicate bulging of the column occurred within a depth range of D (column diameter) to 2D from the top of the columns.

2. The load-bearing capacity of soil reinforced with stone columns without covering increased compared to unreinforced soil, with a greater increase for columns covered with geotextile.

3. The extent of bulging was less for columns covered with geotextile compared to uncovered columns.

4. Stone columns act as drains once placed in the soil.

5. Using 10% by weight of TDA in place of gravel materials shows a significant increase in the load-bearing capacity of the stone columns.

6. Increasing the use of TDA to 20% by weight of gravel materials shows a decrease in load-bearing capacity compared to before the replacement, but this increase in usage is justifiable from an environmental and economic perspective.

7. Simultaneous use of a gravel mattress, geotextile cover beneath the gravel mattress, and replacement of materials with TDA nearly doubles the load-bearing capacity of the columns, which is very cost-effective.

6.3 Suggestions

1. The effects of using geotextiles with different strengths could be tested.

2. The impact of a gravel layer made from the same materials as the columns with various thicknesses on the columns could be investigated.

3. The optimal amount for replacing gravel materials with TDA should be examined and tested.

6.4 Conclusion

This chapter analyzed the results from laboratory tests and presented the overall findings. In the end, suggestions for possible future research related to this study have been offered.

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