BE CIVIL ENGINEERING

EVALUATION OF SOIL REPLACEMENT THICKNESS FOR THE OPTIMIZE BEARING CAPACITY IMPROVEMENT IN SOFT CLAYS

FINAL YEAR PROJECT REPORT

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It is certified that the

Final Year Project titled

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has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

Dr. Kamran Akhtar, PhD **Dr. Mazhar Iqbal, PhD**

DEDICATION

Dedicated to our beloved parents whose prayers, best wishes and support was always with us during the course of this project. Along with our respectable, sincere and dedicated instructor who were always willing to put in their best to guide us throughout the tenure of the studies. Thank you so much.

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We are thankful to Almighty Allah for giving us strength and courage to complete this project. We would like to express our heartiest gratitude to the Commandant MCE and the Dean MCE CE Wing for providing resources and encouragement for completion of this project. We would also like to express our sincere gratitude to our advisors Dr. Mazhar Iqbal Arshad and Dr. Kamran Akhtar for supporting and encouraging us in our project. There suggestion and advices were pivotal in successful completion of this project.

ABSTRACT

Foundations are there to transfer the upcoming structural loads uniformly to the soil beneath. For this, the soil must have an adequate bearing capacity to sustain the upcoming structural load without failure. In case of Shallow foundations, failure is governed by two factors that are the shear failure of the soil and the excessive settlements of the foundation. To avoid these types of failures, either the site is changed or the bearing capacity of the soil is improved with the help of different techniques. One of such technique is the soil replacement technique.

In this project, the effect of the depth of soil replacement on the overall bearing capacity of the soil is studied with the help of experimentation as well as with different software. Firstly, the footing size is kept constant and against each footing size, the effect of soil replacement at different depths (i.e. 0.25d, 0.5d, 0.75d; where d= diameter of footing) against bearing capacity is studied. At last, all the scenarios are run on FEA software (PLAXIS) to analyze and counter check the efficiency of the results obtained. After analyzing all the results certain conclusions were drawn with respect to the bearing capacity improvement and its relation with depth of replacement.

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Chapter 1

INTRODUCTION

1.1 General

Foundations are provided beneath every structure to adequately transfer the upcoming structure load to the underlying soil surface. Every load passes through the structural elements and foundations and then dissipates in to the soil. This makes soil an important medium as ultimately it has to carry the whole structural load. It shows how crucial the soil investigation is and how important it is to properly study the soil strata and its properties for the effective foundation design.

Foundations must rest on the soil which has the capacity to bear the upcoming load. If the soil is poor and weak in load bearing, it can cause excessive settlements and can also result in the overall shear failures, resulting in the foundation failure. To avoid such incidents, foundations must be constructed over strong soil and if not possible, weak soil must be improved against load bearing.

Soil is improved with the help of various soil improvement techniques. One of Such techniques is "Soil Replacement Technique" which is being used extensively around the world. In soil replacement technique, depth of replacement is an important factor which governs the overall load bearing capacity of the soil. This research focuses towards the various effects of depth of replacement in governing the overall bearing capacity of the soil. This chapter addresses the basic problem, the scope and objectives of the project, Significance of the project and a conceptual framework and layout of the project.

1.2 Background

Pakistan is currently going through the evolution in the field of Road and transportation as numerous Highways and Motorways projects are being constructed all over the country. One of such project is Sialkot-Lahore Motorway (SLM). It is 89km long and with the cost of 44 billion it is expected to be completed in December 2018. It will help to reduce the travel time between Sialkot and Lahore to 50 minutes.

During the soil investigation, it was revealed that at some points the top strata i.e. up to 2m is composed of week clayey soil and having low bearing capacity which will not be able to withstand the foundation load of culverts. As, sand is easily available material thus soil replacement technique was selected to replace the weak clayey soil with compacted sand to increase the bearing capacity. A need of research was identified to examine the effect of depth of replacement on the overall bearing capacity of the soil.

1.3 Problem Statement

One of the major issues in the construction industry related to Geo-Tech is the handling of the weak soils having low bearing capacity. They include mostly loose silt or soft clays. Shallow foundations when built on these soils have low load carrying capacity and undergo large settlements. This requires the utilization of soil improvement techniques. In case of shallow foundations mostly soil replacement technique is used in which the weak soil is reinforced with the compacted granular fill up to a certain depth. Thus, there is a need to carry out the research to investigate the effect of this depth of replacement along with the width of footing on the bearing capacity of the soil.

1.4 Scope and Objectives:

To carry out this research a model study will be done with the approach to fulfill the following objectives:

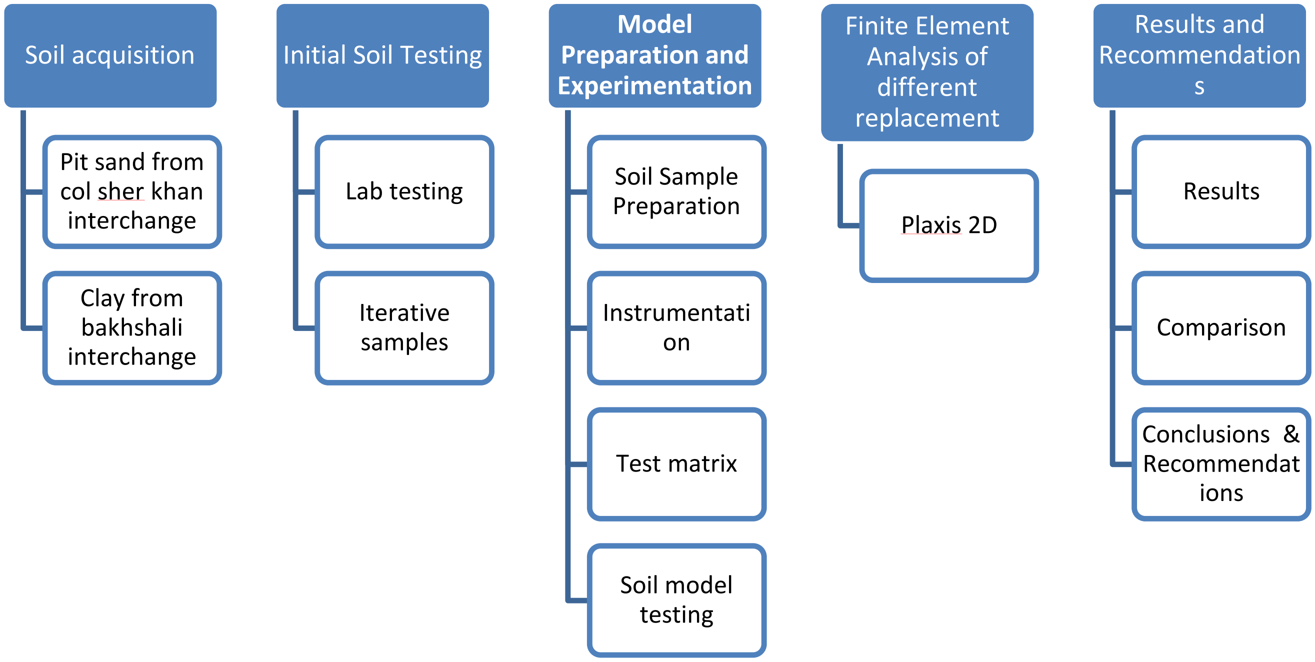
- Fabrication and Development of Lab Model for Testing Purpose
- To study the effect of depth of replacement on the bearing capacity of Soft clays
- To study the effect of size of footing with depth of replacement on the bearing capacity of soil
- To analyze the effect of depth of replacement on bearing capacity using Finite Element Analysis

1.5 Research Questions

This research work will cover the following questions:

- How the depth of replacement is related to bearing capacity in soil replacement technique?
- How the size of footing is related with the depth of replacement and bearing capacity?
- How close are the experimental results with those obtained from software using Finite Element Analysis?

1.6 Methodology



1.7 Research Significance:

The world is progressing and this progress demands the construction of various infrastructures. Construction industry has to come across the locations where the soil strata is weak and cannot bear the upcoming structure load. To counter this, often soil replacement technique is used in which the some portion of the weak soil is replaced with the compacted granular fill. This amount of bearing capacity improvement is related with the depth of replacement. Thus this research is of much importance as the conclusions drawn at the completion of this research are directly applicable on the field. This will tell us the amount of soil improvement with the certain depth of replacement. This research will also give guide us to select the possible footing size along with the certain depth of replacement to achieve a particular bearing capacity.

1.8 Organization of the Report:

This research report has been organized into the following chapters:

- **Chapter 2** explains the literature review and past studies in relation with the different bearing capacity methods, soil replacement technique of soil improvement, bearing capacity analysis of shallow footings, knowledge gaps in depth of replacement and footing size analysis and Finite element analysis of shallow footing over layered strata.
- **Chapter 3** describes the methodology of the research work, lab tests of the soil, soil classification and identification, method and sequence of model experiments, model assembly preparation and tests performed on the model samples.
- **Chapter 4** focuses on the results of the soil investigation, basic soil tests performed in the lab and the tests executed on model experiments on the samples. It discusses the effects of depth of replacement and size of footing on the bearing capacity. It focuses on the discussion of the results obtained on the modeled samples and how much the bearing capacity is improved for a specific depth of soil replacement or for a specific size of footing. It also compares the results obtained through the experiments and from the finite element analysis.
- **Chapter 5** gives the conclusions of the research work along with the recommendations for selecting a suitable depth of replacement and size of footing for a specific problem.

1.9 Conclusion:

This chapter accumulates the basic introduction of the research work, its scope and objectives along with the explanation of the problem. It specifies how the significance of the research along with the various research questions that it addresses. Research layout and the organization of the overall report of research work were also explained. Upcoming chapters will address the research work in details.

Chapter 2

Literature Review

2.1 Introduction

Bearing capacity of any soil has got prime importance as it determines the total load that the soil can withstand. Moreover, there are many cases in which the structure collapsed-in spite it was structurally stable- due to the reason of the low bearing capacity of the soil beneath. These lead researchers towards the study of bearing capacity of soil and exploration of various ways to improve it.

The most economical way of transferring the load is via shallow footing as it could be done with the help of simplest tools and requires minimal excavation. Moreover, a shallow footing gives the liberty in terms of its shape and size. While designing the shallow footing, one must consider the following two modes of soil failure that could occur:

1. Shear failure of the soil beneath
2. Failure due to excessive settlement

These two failures mainly depend upon the type and properties of the soil existing beneath the footing. If the soil is weak clay –having low bearing capacity- then it mostly leads towards the settlement failure, or in case of sands or strong clays it leads towards the sudden shear failure. Shear failure is considered the most critical as it causes sudden failure of the soil and ultimately the whole structure. But one should not neglect the adverse effects of settlement failure as it seizes the functionality of the structure especially in case of culverts beneath the highways and high rise building structure. Moreover, it could lead towards the differential settlement which could result in developing large cracks and ultimately become the cause of structural collapse.

This shows that bearing capacity analysis is the basic requirement before the designing and construction of the any structure. An engineer must perform the bearing capacity analysis of the soil beneath the structure in the very initial phases of the design process in order to avoid two types of failures discussed above. Knowing the importance of bearing capacity, an extensive amount of literature is available along with the improvement techniques that could help in bearing capacity improvement. This literature will be discussed in upcoming paragraphs in the light of our research project.

2.2 Methods for Determining Bearing Capacity

Lot of research has been done in the past for the determination of bearing capacity and various methods have been developed-for different scenarios- to calculate the bearing capacity. These methods are explained as follows:

2.2.1 Bearing Capacity via Insitu Test Methods

For any construction project, engineer must check the Insitu bearing capacity of the soil during the initial investigation phase and suggest any bearing capacity improvements if the capacity is low. To check the bearing capacity in the field following methods are usually adopted:

2.2.1.1 Plate Load Test

For the verification of the bearing capacity and design of shallow footing, insitu plate load test is conducted in the field using plate load testing apparatus. In the apparatus, a plate acts as a footing and load is applied through hydraulic pump. Settlement is measured against each load increment with the help of three to four dial gauges attached to the plate. At last the load versus settlement curve is obtained as shown below.

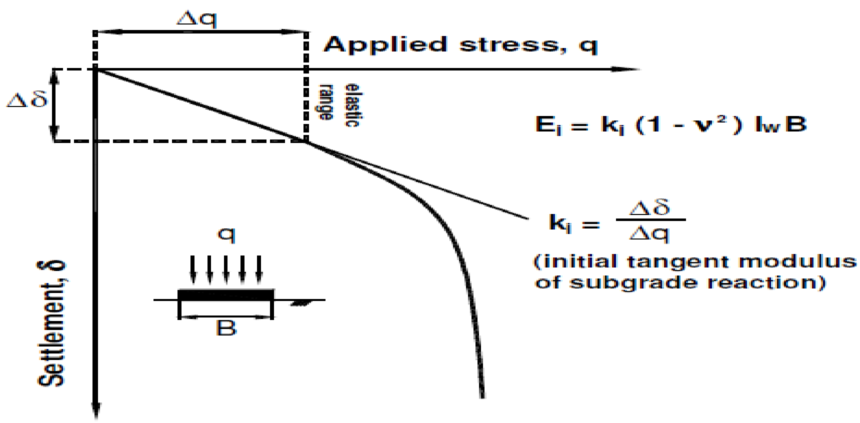


Figure 2.1 Applied Stress vs. Settlement Graph Using Plate Load Test

The Figure 2.1 shows the general behavior of the load vs. settlement curve in plate load test and generally the peak stress is observed at a point where the curve is changing from elastic state to non-elastic. This point represents the ultimate bearing capacity and is denoted by q_{ult}. There are cases in which there is soft clay or very weak soil, in such cases the well-defined failure is not observed but instead local or punching failure is observed. In these cases the load vs. settlement curve does not change but it is rather a straight line. In such cases, the q_{ult} is the stress in correspondence to the settlement of 10%-15% the diameter of the footing or plate.

Similarly, with the help of this test, elastic modulus could also be found which will be dependent upon the reaction provided by the soil i.e. ‘K’ to the exerted load. This reaction is defined as the slope of the load vs. settlement curve. The modulus of elasticity is calculated as follows:

$$E = \frac{(1-\nu^2)}{(\delta / q)} B I_w = k(1-\nu^2) I_w B$$

Here;

V= Poison Ratio; B= Footing diameter ; d/q= settlement/Stress; k= Modulus of soil reaction

Iw= Factor Dependent upon shape and flexibility of area under load

One must remember that the plate load test only represent the bearing capacity of the soil that is under 1.5 to 2.5 times the diameter of footing.

2.2.1.2 Standard Penetration Test:

The most reliable method of the insitu test is the standard penetration test. This test is the most commonly used method to get the idea about the strata of the soil underneath and to know about the profile and properties of the soil at various depths.

In this method an open ended pipe of 51mm diameter is driven having a sampler and weight of 63.5kg dropped from a free fall of 760mm. Neglecting the initial penetration of 150mm, the number of blows that are required to drive the sample to a depth of 300mm are noted and referred as N-SPT values. This N-SPT value is used extensively for the designing of foundations piles and other structures.

The Soil collected in the sample is then tested in the laboratory for the index properties.

2.2.1.3 CPT (Cone Penetrometer) Test:

There are many tests that are performed in the field for the characterization and classification of soils that include SPT, DCP, Vane shear Test and Plate load test. Among all of the tests, CPT test is the most commonly used insitu test that engineers prefer.

This test basically deals with measuring the resistance of the tip of the cone which is penetrated in the soil. With the help of the resistance values, classification of the soil is executed as each type of soil resists the cone differently. The resistance values have lot of other applications in geotechnical designs as they are commonly used in the design process of footings and in the evaluation of pile shaft and base capacities. Moreover, various additional gadgets and devices are linked with the CPT which enables CPT test for the wide range of geotechnical applications.

Tend et.al (1986) studied the use of CPT values in determining the bearing capacity of circular or square footing in intact clays and having no embedment. The expression used to evaluate the qult is as follows:

$$\text{Square / Circle:} \quad q_{ult} = 0.445(q_t - \sigma_{vo})$$

Mehrof (1956) Direct Method:

Mehrof (1956) -using CPT values- evaluated a direct method for the estimation of ultimate bearing capacity on sands in case of shallow footings. He presented an equation for ultimate capacity which is as follows:

$$q_{\text{ult}} = \bar{q}_c(B/C)(1 + D/B)$$

Here;

qc= Tip Resistance values obtained from CPT

B= Diameter of footing/ Footing Width

C= Empirical constant (Usually 12.2 in meters)

D= Depth of embankment below ground surface

The qc used in the relation is the average value of qc over depth which is equal to the width of the footing. Moreover, Mehroof suggested a value of 3 as factor of Safety to estimate the allowable bearing stress.

Tand (1995)

Tand et.al (1995) suggested the equation for estimation of ultimate bearing capacity for the lightly cemented medium dense sand using cone resistance:

$$q_{ult} = R_K q_C + \sigma_{vo}$$

Here;

Rk= Factor depending upon shape and depth (Varies from 0.14 to 0.2)

σ_{vo}= Vertical stress at the base of the footing

The graph below shows the correlation between the bearing capacity and cone resistance:

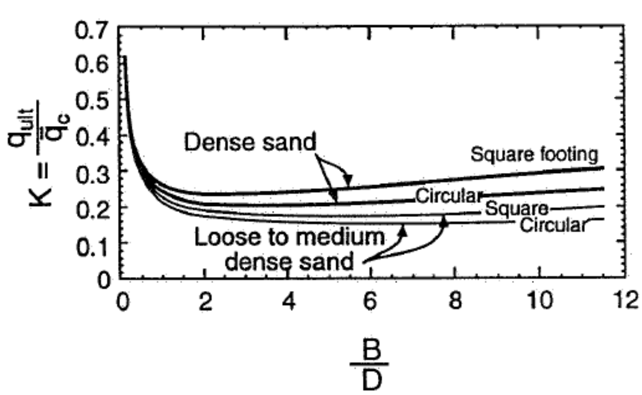


Figure 2.2 Correlation between Bering Capacity and Cone Resistance

It must be kept in mind that these direct methods are approximate and the results obtained via these will be conservative.

2.2.2 Bearing Capacity through Analytical Methods:

There are various analytical methods for the determination of bearing capacity of any soil. These analytical studies have been developed in the past by various researchers. These methods are explained as follows:

2.2.2.1 Terzaghi's Method

By slightly modifying the theory of bearing capacity by Prandtle in 1920, Terzaghi proposed its theory in 1943. The modification was related to the roughness of the footing and the location of footing at a certain depth below the ground surface represented by Df. In its analysis of strip footing having L/B greater than 5 he proposed a failure model as shown below:

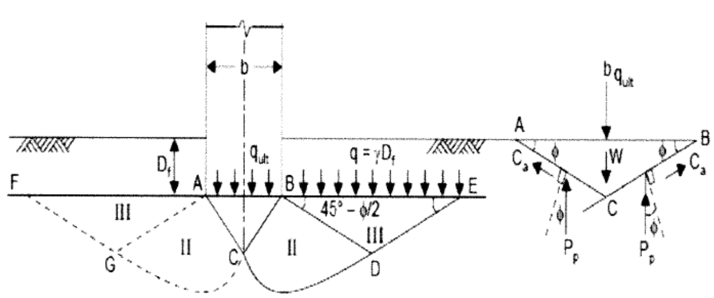


Figure 2.3 Failure model of Terzaghi

He then proposed a generalized expression for the calculation of bearing capacity. In his proposition, he considered the generalized shear failure conditions of the soil. The final expression which Terzagi extracted keeping in view the shape factors in relation with cohesion Sc and base terms i.e. Sy is as follows:

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5\gamma BN_\gamma s_\gamma$$

Here;

C= cohesion of soil

q= Overburden pressure at base of foundation

The bearing capacity factors Nc, Nq, and Ny are given by:

$$N_q = \frac{a^2}{acos^2(45 + \emptyset/2)}$$

$$a = e^{(o.75\pi - \emptyset/2)tan\,\emptyset}$$

$$N_c = (N_q - 1)\cot\emptyset$$

$$N_\gamma = \frac{\tan\emptyset}{2}\left(\frac{K_{p\gamma}}{cos^2\emptyset} - 1\right)$$

2.2.2.2 Meyerhof’s Method

The basic difference between the Meyerhof’s and Terzaghi’s method is the consideration of shear resistance of the overburden soil. Terzaghi’s method does not count for that. Thus in comparison to the Terzaghi’s method, Meyerhof’s (1963) method is more reliable as it caters for the shear resistance of the soil above the base of the foundation. Moreover, in his equation he proposed the solution for both the vertical loads and the inclined loads. The general shear failure proposed by Meyerhof is shown below:

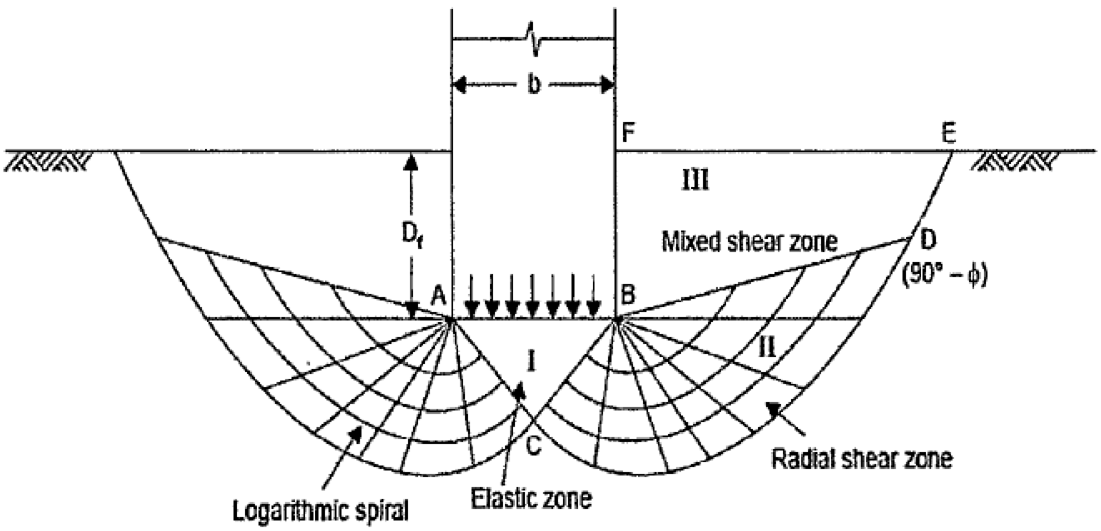


Figure 2.4 General Shear Failure Proposed by Meyerhof

Meyerhof's proposed equations are as follows:

$$\text{For vertical load } q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$$

Where;

$$\text{Inclined load } q_{ult} = cN_c d_c i_c + \bar{q} N_q d_q i_q + 0.5\gamma B' N_\gamma d_\gamma i_\gamma$$

$$N_q = e^{\pi tan\varphi}\, tan^2(45 + \varphi/2)$$

$$N_c = (N_q - 1)\cot\varphi$$

$$N_\gamma = (N_q - 1)\tan(1.4\varphi)$$

2.2.2.3 Hansen's Method

Hansen method is basically based upon the Meyerhof's method with two changes which is the addition of the following two factors:

1. Factor for Base Tilt
2. Factor for foundation slope

The proposed equation is as follows:

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

where

$$N_\gamma = 1.5(N_q - 1)\tan\varphi$$

2.2.2.4 Vesic's Method

Vesic's method uses the exact same equation as proposed by Hansen for the calculation of bearing capacity. In fact Vesic, Hansen and Meyerhof used the expression which was suggested by Prandte in 1921 which ultimately help in determining the values of Nq and Nc. In Vesic method the only change is in the equation of Ny. The expressions proposed by Vesic are as follows:

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

N_q and N_γ are same as Meyerhof's.

$$N_\gamma = 2(N_q + 1)\tan\varphi$$

2.2.2.5 Limit Equilibrium Method

This method has been used traditionally and the work of Terzaghi and Meyerhof is based on this method. This method provides an approximate and conservative solution of the bearing capacity determination problems as various assumptions about the stress distribution in soil are to be made to establish a bearing capacity equation which satisfies in equilibrium in terms of its resultant forces.

Saran, Sud and Handa with the help of this method and limit analysis approach provided a solution which helps in the evaluation of the bearing capacity of shallow foundations. According to their theory the equation of bearing capacity for a strip footing is as follows:

$$q_u = CN_c + qN_q + 1/2\gamma BN_\gamma$$

Where;

Nq, Nc, Ny are the bearing capacity factors &

q= Stress due to surcharge soil having depth D

This section of the chapter discussed all the methods which were based on both the Insitu testing and analytical studies for the determination of bearing capacity of soil. Engineers often come across sites that have soil of very low bearing capacity usually soft clay. In such cases some suitable soil improvement technique must be applied to improve its bearing capacity. Below are some of the modern techniques that are being used in the industry for bearing capacity improvement.

2.3 Modern Ground Improvement Techniques

Modern ground improvement techniques have less cost and are time efficient. Modern techniques can be used widely and have a large scope. The brief description of latest techniques that are used to increase the strength of soi is given below:

2.3.1 Vibro-Compaction Method for Ground strength Improvement

This method is used to densify the soil particles by vibration techniques. Vibro Compaction is ground strength improvement technique to make the soft soils to hard and dense end product. The main principle behind vibro compaction is simple vibration action so that loose particles of soil are converted to denser mesh. This technique is used to decrease subsoil settlement and to reduce risk of liquefaction.

2.3.2 Vacuum Consolidation of Soil

Vacuum consolidation is very effective for saturated soils. The construction site is covered by an air tight membrane in which vacuum is created by using pumps. Due to this technique 4.5m surcharge fill can be applied on the site after this treatment. Vacuum consolidation helps in reducing pore water pressure. This technique is used to reduce risk of failure and is used to construct mega structures in thick compressible fill during vacuum consolidation process. Question which arises here is that is the inner side displacement of the topmost layer likely to be greater for peat on explanation of its compressibility? According to Ong and Chai (2011), the lateral displacement not only depends on magnitude and loading but also depends on settlement and consolidations. This is reinforced by the experiential relation between y-axis and x-axis displacement by Mesri and Khan (2012).

2.3.3 Pre-compression or pre-loading

Pre-compression and pre-loading is simple and it has been used since very long ago. Pre-compression or pre-loading is the method in which addition vertical stress is applied on the soil to increase pore water pressure byRamli Mohamad (1992) and Johnson, S.J. (1970). If pore water pressure is reduced settlement will occur. Surcharging is very effective and economical method for ground strength improvement. This method is time consuming and hence it affects feasibility of the project. The soils treated are clays, soft clays, varied silts, organic silts. Considerations for design are slope stability, bearing capacity. This method has been in presence since the nineteen seventies, e.g. Johnson (1970).

2.3.4Thermal Stabilization

Due to heating soil particles are broken down and converted into crystalline form or glass product. Electric current is used to heat up the soil and hence it is converted into dense soil. Due to heating soil its properties are permanently changed. Depending upon various soils temperature range is from 300 to 1000 degree Celsius. This technique is used to restrict radioactive or polluted soil, densification of soil, soil stabilization.

2.3.5 Ground Freezing Technique for Ground Improvement

Freezing techniques is used to convert the in situ pore water into ice. The converted ice act as bonding agent like cement, mortar, glue etc and hence combine the material to one dense mesh and increase strength.

2.3.6 Vibro-Replacement of Stone Columns

This technique increases the range of soil that can be improved. Reinforcement of the soil with boulders and stones by top-feed method increases the strength of soil. Stone columns in compressive loads collapse in two main different ways: bulging (Hughes and Withers, 1974) and general shear failure.The stone columns and soil is improved by top feeding of stone columns. In cohesive underneath soil, extra pore water pressure is readily dissipated with the help of stone columns resulting in less settlements takes place more rapidly as compared to normal cohesive soils. Many researchers have tried to carry out solutions.

2.3.7 Mechanical Stabilization

A one segmental, precast stabilizing of earth wall will generate metallic or geo-synthetic reinforcement which is linked to precast pre-concrete or fabricated metal face panel which will create reinforced soil. We can achieve suitable strength by mixing the soil with some stabilizing material to an uninterrupted soil sums and attaining interaction by letting it infiltrate through soil voids.

2.3.8 Soil Nailing

The basic concept of soil nailing is to produce closely spaced concrete or metallic nails, to make soil in-situ and protect it from settlement. In presence of various factors such as the construction order, the fixing of nails, the link between the nails and the fronting are likely to guidance the conduct. The general design contains transferring of the resisting tensile forces which are generated in the additions into the ground through the friction mobilized at the boundaries. It is used for stabilization of railroads, highways and slope cuts and excavation holding structures in built-up areas for tall building and underground facility.

2.3.9 Sand Drains

Sand drains is a process of arranged and uniform consolidation within an embankment by intensifying the rate of drainage by moving a casing into embankment making vertical holes which are filled with a suitable grade of sand. This method increase consolidation rate of soil by providing drainage path to the water and decrease settlement .Designing drains according to the soil in situ properties and other parameters of soil in situ and in compliance with the consolidation theory .Application of surcharge on the ground surface results in an increased pore water pressure which results in drainage in horizontal and vertical directions. Driving the vertical drains reduces length of drainage path thus accelerating the consolidation process and helping the clay to gain strength

2.3.10 Grouting

Grouting is the instillation of pump able constituents into a soil or rock foundation to change the physical features of the formation. Grouting choice considerations are Site precise requirement, Soil type, Soil grout capability and Perviousness. Grouting can be prohibited by Failure of granular soils, Settlement under nearby foundations, Utilities destruction and Day lighting. Grouting can provide improved soil strength and severity, reduced ground movement and foreseeable degree of improvement. Although most samples distribute in sand and clay soils, a large amount of foundations distribute in sands and gravels and in Silty clay zone as well. Therefore, it can be said that there are some other factors such as jet-injection type, cement water ratio and working parameters that control the strength of jet-grout columns. Furthermore, depth may also definitely affect the strength of columns.

2.3.11 Lime Stabilization

It is a technique in which soil is scraped and pulverized up to certain depth in which hydrated lime is mixed with the problematic soil (clayey soil) in order to increase its bearing strength. Firstly when the hydrated lime react with the soil it releases the heat and its start absorbing the water content of soil chemically and thus evaporates additional moisture. This will continue slowly until a stage come where they reduce the soil moisture holding capacity and make soil dry. After that compaction is done by passing sheep foot roller over the soil. The moist curing is done in order to gain in strength of compacted subgrade layer. Soil is very old construction material and is simply available. The use of different chemicals increases its strength and stability hence improves its engineering properties. These properties can be improved by alteration of physical and chemical means.

2.3.12 Soil Replacement

This technique is mostly being used nowadays because of its economic benefits and due to availability of feasible material. In this method, the top surface of the weak layer -mostly soft clay- is removed up to a certain depth and is replaced with the improved soil i.e. the compacted sand. In this way the bearing capacity of the soil is improved. This technique is known as soil replacement. This technique is nowadays used as sand is easy available material and could be compacted easily. Moreover, it is economical then other soil reinforcement and improvement techniques.

2.4 Research Area

In our research project we will be applying the soil replacement technique and will be studying the effect of the depth of replacement on the bearing capacity of the soil. As, in the soil replacement method, the bearing capacity is dependent upon two variable i.e. the depth up to which the weak soil is being excavated and compacted improved soil is replaced and the size of the footing for transferring the load. Our research will focus on both of these aspects and will yield results about the effect of footing size and depth of replacement on the bearing capacity of the soil. Various researchers have done the work which is related to this research. Few of these are explained below.

2.5 Studies on Soil Replacement and Reinforcement

Following are the few of the studies which were done on the effects of soil replacement and soil reinforcement in relation with bearing capacity.

2.5.1 Improvement of Bearing Capacity by Partial Soil Replacement Technique

Fattah et al. (2015) conducted a bearing capacity analysis on footing over soft clay in which partial replacement technique was utilized. Soil was removed at different depths and was replaced with the granular fill of crushed stone. Total numbers of 8 tests were conducted in which 4 replacements were done in square pattern and 4 in trench pattern. The details of the soil replacement models were as follows:

Type	width of replacement (mm)	Depth of replacement (h) (mm)
square	100	100
square	100	150
square	200 (b = 50 mm)	100
square	200 (b = 50 mm)	150
trench	100	100
trench	100	150
trench	200 (b = 50 mm)	100
trench	200 (b = 50 mm)	150

Table 2.1 Soil Replacement Models

For all of these scenarios the improvement in the bearing capacity was estimated as follows:

Case	Type	q_u / c_u	$q_{treated}/q_{untreated}$
b=0 h=10 mm	Trench	16.5	4.9
b=0 h=15 mm	Trench	18.5	5.5
b=5 h=10 mm	Trench	24.5	7.3
b=5 h=15 mm	Trench	30.1	8.9
b=0 h=10 mm	Square	13.2	3.9
b=0 h=15 mm	Square	15.3	4.5
b=5 h=10 mm	Square	18.2	5.4
b=5 h=15 mm	Square	28	8.12

Table 2.2 Bearing Capacity Improvement

The deduced results show that the bearing capacity in terms of trench soil replacement was improved by 4.9 to 8.9 times and in case of square replacement it was 3.9 to 8.1 times. Hence, the conclusion was drawn that the improvement ratio of soil replacement via trench is more as compared to square soil replacement as shown below.

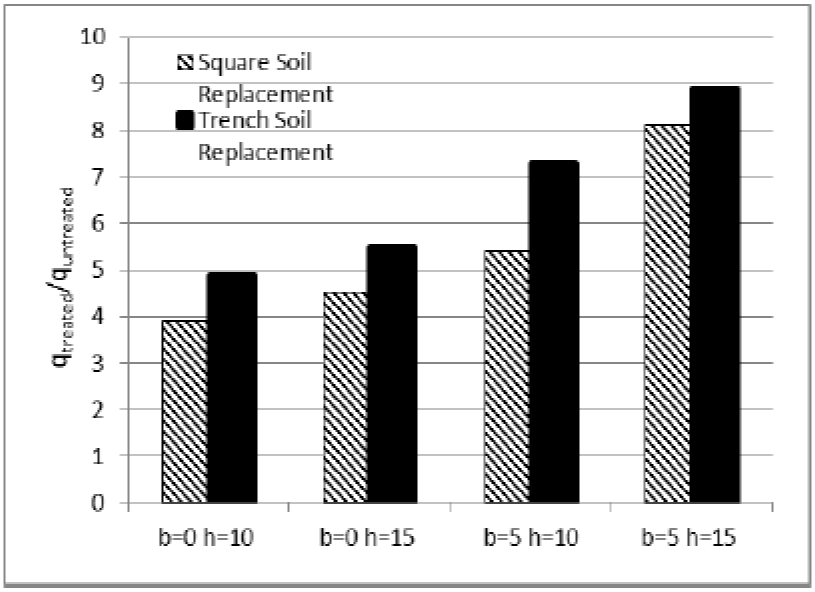
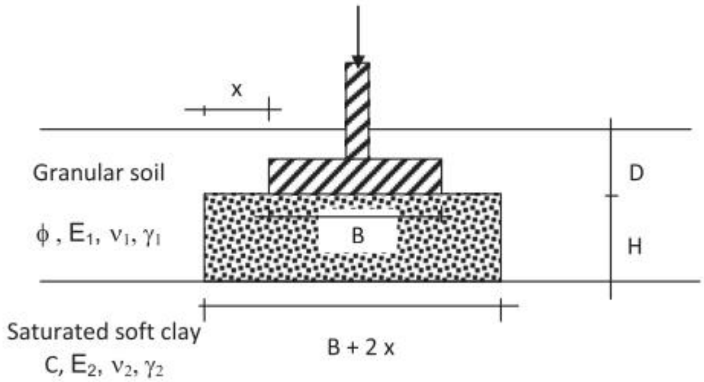
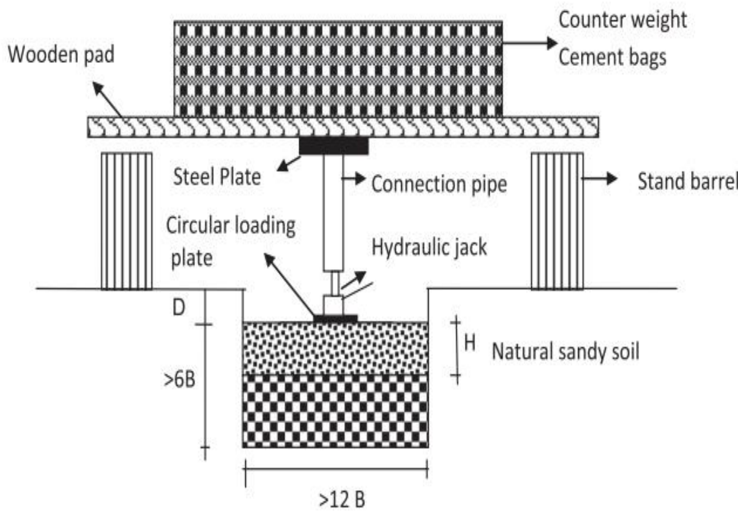


Figure 2.5 Comparison of Soil replacement

2.5.2 Bearing Capacity of Circular footing resting on Granular Soil overlaying soft soil

Ibrahim (2016) conducted the research on the effect of circular footing resting on granular soil overlaying soft soil on the bearing capacity along with the effects of changing the angle of internal friction angle and the density of granular soil on the load carrying capacity of soil. The model and properties of the layers used are shown below:





Element	Soil type	Model	C (kPa)	ϕ	ψ	γ (kN/m^3)	E_s (kN/m^2)	v
Upper layer (drained)	Medium to loose sand (series 1)	Mohr Column	1	35	5	19.0	20000	0.30
	Very dense sand (series 2)	Mohr Column		45	12	22	50000	0.29
Lower layer (undrained)	Soft clay	Mohr Column	21	0	0	20	4000	0.50
Footing	Steel	Elastic	–	–	–	–	2E8	0.30

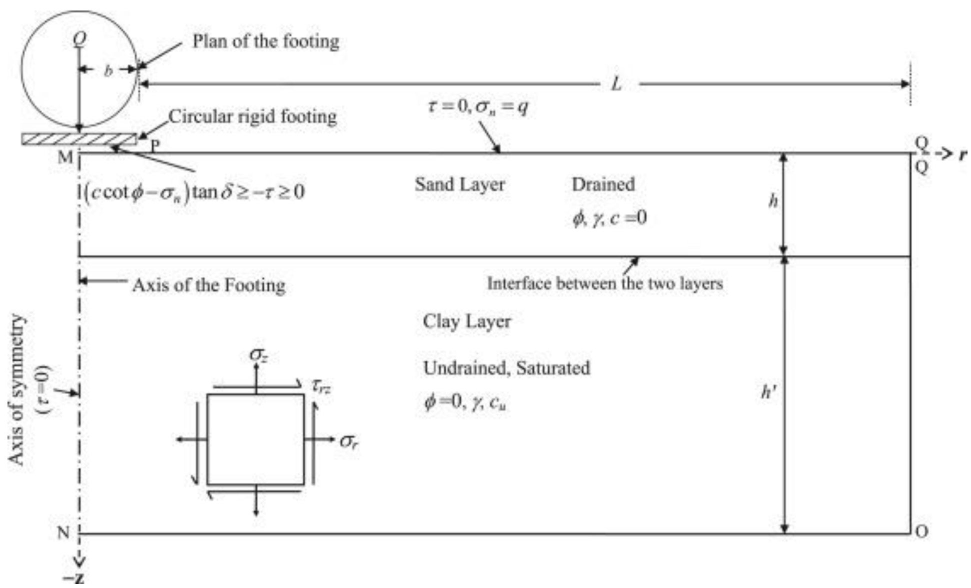
Figure 2.6 Testing Assembly and Properties

The conclusions were drawn that when the density of the granular material i.e. sand changes from medium dense to dense, the ultimate bearing capacity increases up to 67%. Moreover, the bearing capacity increases with the increase in the depth of the layer of granular material but this increase is limited up to the depth that is two times that of diameter of footing. Similarly, increase in bearing capacity is observed with the increase in the internal friction angle of the granular soil.

2.5.3 Studies Using Finite Element Analysis

2.5.3.1 Jyant Kumar (2015)

Kumar (2015) also investigated the effects of improvement of the soil with soil replacement technique. He performed a finite element analysis using FLAC on the circular footing and having



a soil strata of two layers i.e. clay overlaying sand. In his research he calculated the change in the improvement of bearing capacity with the change in the density of the overlaying sand. He presented his work in the form of various graphs which shows the incremental percentage of the bearing capacity with the help of a ratio that represent the bearing capacity with the replaced layer to the bearing capacity of footing lying straight on the clayey strata. Optimum thickness of the overlaying sand layer was also calculated and it was found that the increase in the density of the sand and surcharge load q increases the optimum thickness. His model and few of the results are shown below:

Figure 2.7 Chosen domain and stress boundary conditions for circular footing on two-layer sand–clay media

b (m)	h/b	ϕ (deg)	ψ (deg)	c_u (kPa)	$c_u/(\gamma b)$	$p_u/(\gamma b)$			
						Present result	Lee et al. (2013a)[a]	Lee et al. (2013b)[b]	Craig and Chua (1990)[a]
3	2.06	36.7	7.1	17.70	0.56	28.88	22.39	19.88	–
4	1.02	38.7	9.6	16.30	0.41	8.67	7.43	7.67	–
	1.56	37.5	8.1	17.70	0.42	15.17	12.29	12.30	–
5	0.82	38.9	9.9	16.30	0.33	7.05	5.40	5.37	–
	1.24	37.8	8.5	17.70	0.33	10.29	8.43	8.35	–
6	0.68	39.1	10.1	16.30	0.28	4.65	4.08	4.04	–
	1.04	38.2	9.0	17.70	0.28	7.28	6.05	6.20	–
7	0.88	38.5	9.3	17.70	0.24	5.53	4.61	4.80	–
	1.00	38.1	7.6	42.00	0.58	9.32	–	–	7.53
	1.36	37.7	7.1	41.00	0.57	10.64	–	–	8.85
8	0.78	38.6	9.4	17.70	0.21	4.34	3.92	3.92	–

[a]By using centrifuge test.
[b]By using a new conceptual model based on failure mechanism developed by Lee et al. (2013b).

Figure 2.8 Comparison with various researchers

2.5.3.2 Bearing Capacity Evaluation of Footing Using ABAQUS

Mosadegh (2015) performed Finite Element Analysis on the footing over layered soil strata. Bearing Capacity of the strip footing was evaluated on both one layer and two layer soil. In his research he uses ABAQUS for the finite element analysis and applied it to get the effects of various parameters on the bearing capacity of the footing. He at the end concluded that the bearing capacity of the footing reduces as the height of the cohesive soil increases and ultimately it cause the increase in the settlement. His model and results are as follows:

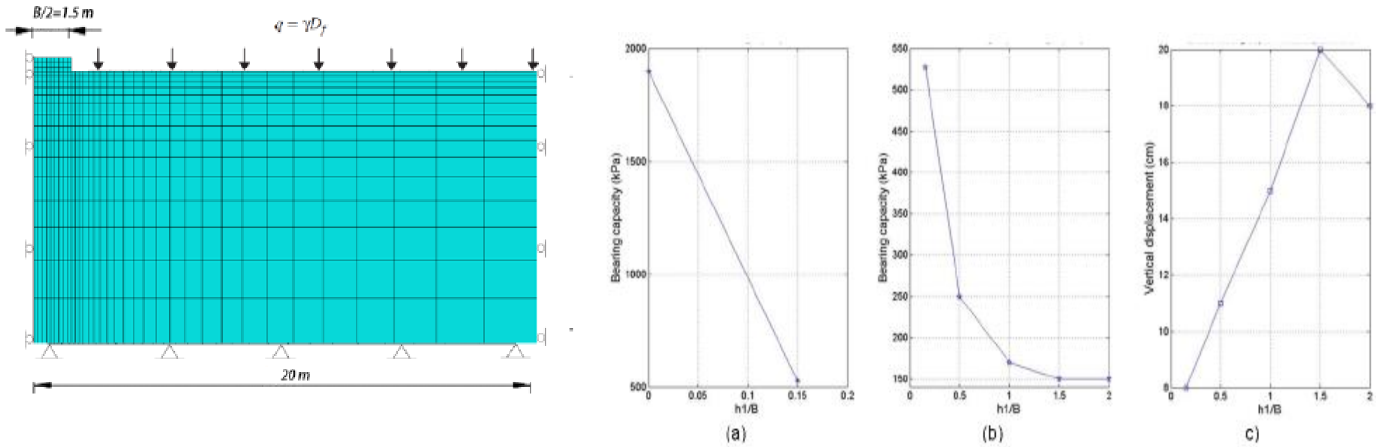


Figure 2.9 Model and Results

2.6 Conclusion

This chapter focused on the previous studies that were conducted to evaluate the bearing capacity and how it is affected by soil replacement technique. It also explained the main research area of our project and what objectives we could deduce with the help of this literature review, research studies and experimentations. The work of various researchers was explained along with the output of the results that they extracted. Moreover, the relevance of all the literature to our study was also explained. Thus, to conclude this chapter focused on the previous studies on the bearing capacity analysis and soil replacement.

Chapter 3

Methodology

3.1 Introduction

This chapter covers the methodology of the research work in sequence. It will cover all the basic tests that were performed in the lab as well as on the model assembly. Along with that it will also explain the procedure for the preparation of the testing assembly, test matrix, preparation of soil model and the details of all the tests that were performed on the assembly. This methodology is pictorially represented by the flow chart shown below:

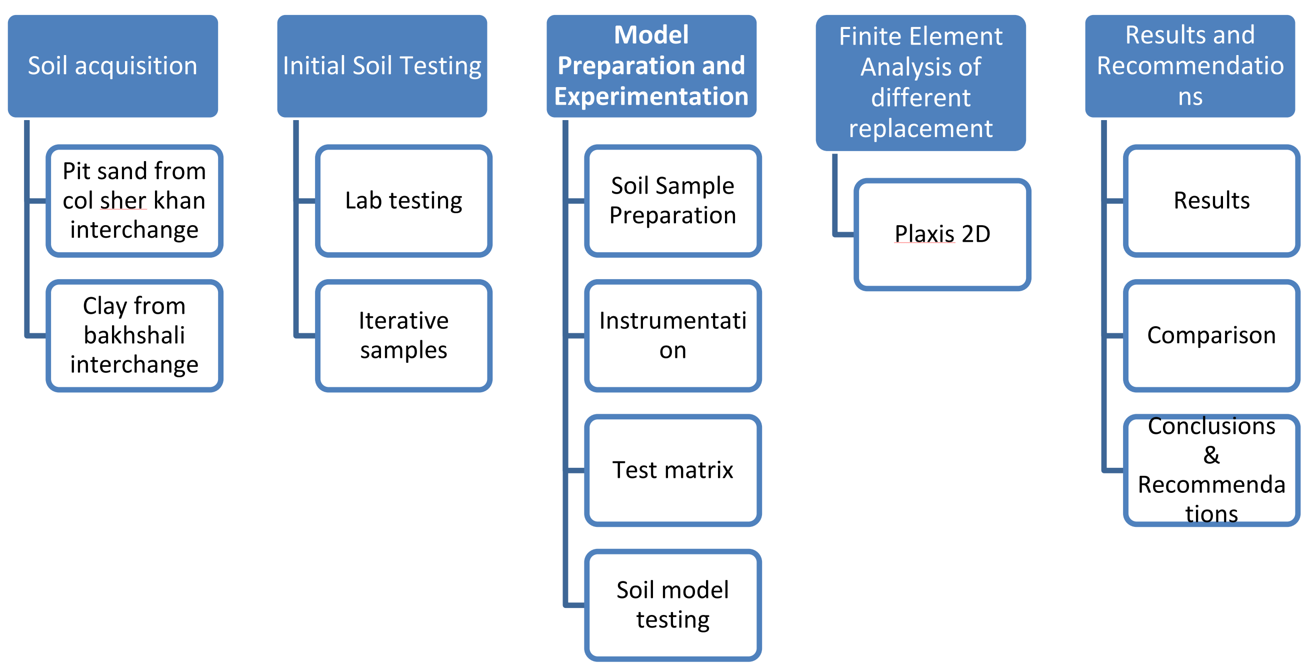


Figure 3.1 Methodology

3.2 Soil Acquisition

For this research, two type of soil was used. Pit-Sand was acquired from the field near Col. SherKhan Interchange. It was then passed through No. 10 Sieve to clean it from large boulders. Similarly, Clay was brought from the site near to Bakshali Interchange. Following lab tests were performed to know the properties of both the soil.

Figure 3.2 Sand and Clay Acquisition

3.3 Lab Testing

Several lab tests were performed to check the properties of the soil brought. These tests are as follows:

3.3.1 Sieve Analysis Test

The sieve analysis test was performed on both the soils brought from the field. This test is used to classify the soil based on the (%) grain size distribution. To perform this test, Clay was pulverized so that the lumps are broken and then samples of both clay and sand were oven-dried. After all the moisture was lost, both the samples of Clay and sand were separately passed through a set of sieves that were arranged in descending order of their sizes. The quantity of soil retained on each sieve was calculated.

Figure 3.3 Sieve Analysis Test

3.3.2 Atterberg Limits

Atterberg limit test is performed according to procedure given by ASTM D 4318. This test is performed to check the liquid limit, plastic limit and plastic index of the clayey (Fine Particles) soil. As the water reacts with the clayey or silty soil, their consistency changes. Their reaction with water causes change in their behavior which includes the shear strength change which is much critical to study. These changes in the properties of the clayey soil could be examined via Atterberg limit tests. These tests include:

1. **Liquid Limit Test:**

   This test is performed to check the amount of water that will change the behavior of the soil from plastic to liquid. For this test Casagrande apparatus is used. A certain amount of water was mixed and placed in the apparatus and number of blows against each soil sample of specific moisture content is noted. Liquid limit is that moisture content (%) of soil at which the bottom of the grove closes 13mm when the number of blows reaches 25.

Figure 3.4 Liquid Limit using Casagrande Apparatus

2. **Plastic Limit Test:**

   The moisture content at which 3.2mm diameter threads of soil crumbles is known as Plastic limit. The threads are made at different moisture content and then rolled on smooth surface to check.

Using both the above tests and values of Liquid Limit (LL) and Plastic Limit (PL), the Plastic Index (PI) was calculated which shows the water content at which the soil is behaves as plastic. PI is calculated as follows:

Plastic Index (PI) = Plastic Limit (PL) – Liquid Limit (LL)

Figure 3.5 Oven Dried Samples of PL and LL

3.3.3 Specific Gravity Test

This test is performed to check for the degree of saturation of the soil along with the void ratio. It is the ratio of the weight of the soil to the weight of the water having same volume. This test is performed according to ASTM D854-14.

Figure 3.6 Specific Gravity Test

3.3.4 Direct Shear Test

This test is used to check for the shear properties of the soil. This test is performed on the granular soils. Undisturbed sample is placed in the shear box and confining stress is applied. The sample is then sheared and the displacements are recorded at regular intervals. Similarly, the test is repeated at various confining stresses and the shear parameters of the soil are obtained. These are the Cohesion (c) and friction angle (Φ). This test is performed on both the medium dense sand and the dense sand. It is performed as specified in ASTM D 3080.

Figure 3.7 Direct Shear Test Apparatus

3.4 Model Preparation

For conducting this research, a model assembly was prepared. A round drum made of steel sheet having the following dimensions was prepared:

- Diameter of the drum = 5ft

- Height of the drum = 5ft

The diameter of the drum was deliberately kept large to minimize the boundary effects during the experimentation. The height of the drum was large enough to handle the soil strata of two types.

3.5 Conditions to achieve

For this research, fully saturated medium dense sand is placed at the bottom 3 ft. of the drum and drum is filled with water till that point. On the top of the sand, clay having specific moisture content is placed and consolidated up to certain strength. To know about the specific moisture content at which the clay will be consolidated and the time intervals of the loading iterations were done on the very small scale.

3.5.1 Iteration Samples

Two samples of clay were prepared having different moisture contents (15% and 20%) and placed over the layer of saturated sand. The thicknesses of the layers were maintained as 1/10th of the thickness of the sample which is to be placed originally in the drum. The loading was incrementally applied after regular interval to normally consolidate the sample. At the end the undrained shear strength was measured using vane shear tests as well as the unconfined compression test. The strength against a loading time intervals and moisture was noted and applied on the larger scale in drum.

Figure 3.8 15% and 20% moisturized Clay iterative samples

3.6 Soil Sample Preparation

Soil sample was prepared using the following process:

3.6.1 Sand

Sand obtained from the site of Col. Sher-Khan Interchange was to be placed in the bottom 3 ft. of the drum in medium dense conditions. Sandy soil was passed through a large #10 sieve to separate any soil lumps and to obtain clean sand. To achieve the medium density of the sand, sand was rained through the #10 sieve from a specific height i.e. 400mm. The density of the sand was measured by placing a proctor mold at three different locations of the drum. Average density was calculated with the help of the three individual proctor molds.

3.6.2 Pulverization of Clay

The clay obtained from site was in the form of lumps which were hard and difficult to break. To make a proper sample out of it and to properly mix the required water content, these lumps were broken and the soil was pulverized. The pulverized soil sample was then passed through the #10 sieve to separate any large boulders.

Figure 3.9 Pulverization of Clay

3.6.3 Clay Sample Preparation

After pulverization, specific water was added in the clay and mixed thoroughly to obtain the required moisture content (15%). The moisture was added to the small batches of the clay (10 kg clay samples). These samples were then placed over sand layer until a clay layer of 1 ft. was obtained. Then layer of the soil was then straighten and covered with the plastic sheets to avoid any moisture loss due to atmospheric temperature during consolidation. This process is repeated after each test to make the moisture content equal to the required (15%).

Figure 3.10 Mixing Clay for Uniform Moisture Mixing

3.6.4 Loading Mechanism

Loading was applied incrementally after a specific interval to consolidate the clayey soil to achieve a specific strength. A large plate was used for the purpose of uniform distribution of the load. Load was increased after regular intervals as found in the iterated samples. After the completion of the loading time, strength of the soil was checked to verify it against the desired strength.

Figure 3.11 Loading Mechanism

3.7 Soil Strength Measurement

After the completion of loading process, un-drained shear strength of the soil is calculated. The basic aim towards this step is to confirm that all the samples prepared separately for each test have equal strength. This will ultimately help in comparison of the results. Following tests were used to calculate the shear strength parameter.

3.7.1 Vane Shear Test

Vane shear test is the in-situ test which is performed to calculate the un-drained shear strength of the in-situ soil. This test was performed after the completion of the loading process of each test sample. It was performed around the four points of the samples and the average value was then compared to the required strength.


Figure 3.12 Vane Shear Tests

3.7.2 Penetrometer Test

This is another in-situ strength measurement test which directly gives the value is tons/ft^2. This test was also performed on each sample and on all the four corners of the sample and then averaged to calculate the uniform strength of the sample. This was also then compared to the strength required and with the results of Vane shear test.

Figure 3.13 Penetrometer Tests

3.7.3 Unconfined Compression Test:

This is the accurate laboratory test which is performed on the undisturbed sample taken from the field. This test was also performed after the regular interval to authenticate the results of in-situ vane shear and Penetrometer tests. The undisturbed sample was taken from the clay and placed in the UCC machine. Loading was applied and the settlement against the incremental load was noted. The parameters achieved by this were the peak shear strength and the unconfined shear strength i.e. Su.

Figure 3.14 Unconfined Compression Test

3.8 Testing Matrix

There was total number of 12 tests that were performed on the soil having same strength properties. These tests were related with the increment in depth of replacement with the increase in the footing size. The footing sizes used for this research were of 4inch, 6inch and 8inch in diameters. The details of the testing are as follows:

Type of Footing	Number of Tests	Tests on Each Footing
4inch in Diameter	1	Without any Replacement
	2	With 0.25D Replacement
	3	With 0.5D Replacement
	4	With 0.75D Replacement
6inch in Diameter	5	Without any Replacement
	6	With 0.25D Replacement
	7	With 0.5D Replacement
	8	With 0.75D Replacement
8inch in Diameter	9	Without any Replacement
	10	With 0.25D Replacement
	11	With 0.5D Replacement
	12	With 0.75D Replacement

Figure 3.15 Testing Matrix for experimentation

3.9 Procedure for Testing

For performing these tests, following apparatus was used:

3.9.1 Instrumentation

To carry out the bearing capacity analysis, the apparatus used was the field CBR machine. The machine was fitted with an I-beam which was casted in wall. This beam acts as a reaction beam against the loading. The three footings placed under the CBR for these tests were the steel plates of 4, 6 and 8 in diameter. The deflection was measured with the help of dial gauge which was fitted with the CBR machine and placed on the rod which was supported by field CBR stands. The loading was recorded with the help of the proving ring fitted in the CBR machine whose 1 reading represents 10 pounds.

Figure 3.16 CBR machine with Dial gauge

Figure 3.17 CBR load test

3.9.2 Procedure:

Following procedure was adopted to accomplish these 12 tests:

1. After the consolidation of the clay, the CBR machine was fitted along with the dial gauge.
2. Load was increased incrementally and the respective settlement was noted against the applied load. Load was noted with the help of proving ring inside CBR machine and the settlement was noted using dial gauge attached to the bottom of CBR machine.
3. For the first experiment load was applied directly on the clay using the three different footing plates of different sizes. Loading with the respective settlement was noted.

Figure 3.18 Footings

4. The soil was again consolidated after each test and the strength was measured after each consolidation using the three methods of strength measurements that are Vane Shear Test, Penetrometer test and Unconfined Compression test.
5. Now the soil was replaced till a specific depth dependent upon the diameter of the footing plate e.g. till 0.25D, 0.5D and 0.75D.

Figure 3.19 Removing of soft clay up to a depth

6. Sand was used as a replacement material and was compacted by raining effect that is passing the sand through sieve and dropping it from a minimum height of 800mm. The optimum moisture content which was calculated as discussed above was first added to make the sand sample first.

Figure 3.20 Sand replacement with raining compaction

7. After replacement, CBR machine was again fitted and the test was performed to get the load and settlement data which was ultimately used to see the improvement in the load carrying capacity.
8. After the test, the moisture content of the sample was tested and increased to reach at 15%.
9. The loading is then applied incrementally to consolidate the soil sample and then the same procedure explained above is repeated for each of the 12 tests.

3.10 Finite Element Analysis

Finite element analysis was done using the software Plaxis 2D. All the tests were repeated on the Plaxis by developing a same model having same properties as of the experimental model. The harden soil model (HSM) was used to analyze the footing research as it provides results using Elasto-plastic formation in comparison to the Mohr Coulomb which only provides elastic formation results.

Load vs. settlement curves were obtained as a result of the analysis using which the improvement with variable footing size and changing depth of replacement was analyzed. Moreover, the results obtained were compared with the experimental results for verification.

3.11 Conclusion

This chapter has discussed the methodology of the research work and explained in details all the lab tests and the procedures adopted to carry out the experimentation. It discusses the methodology in relation to the model preparation, soil sample preparation, iteration samples for knowing the loading combination and moisture content and then the testing that is carried on the model soil samples. At the end, it discusses about the output results and the procedure adopted for the finite element analysis.

Chapter 4

Experimental and Finite Element Results and Analysis

Part A: Experimental Results and Analysis

4.1 General

This chapter discusses the results and analysis of all the experimental investigation done in the lab or on the model experimentation. By analyzing the model tests and their results, further analysis will be done to observe the change in bearing capacity with the change in depth of replacement and to see how it is affected by varying the size of the footing. Below is the explanation and results of the lab tests:

4.2 Sieve Analysis Test

There were two types of soil borrowed from the site, i.e. sand and clay. Sieve Analysis was performed on both the samples and their Gradation curves are drawn.

4.2.1 Sand

The sieve test for sand yields the results as **Fine Clean sand (SP)** in USCS and **A-1-b** in AASHTO. Gradation curve is shown below:

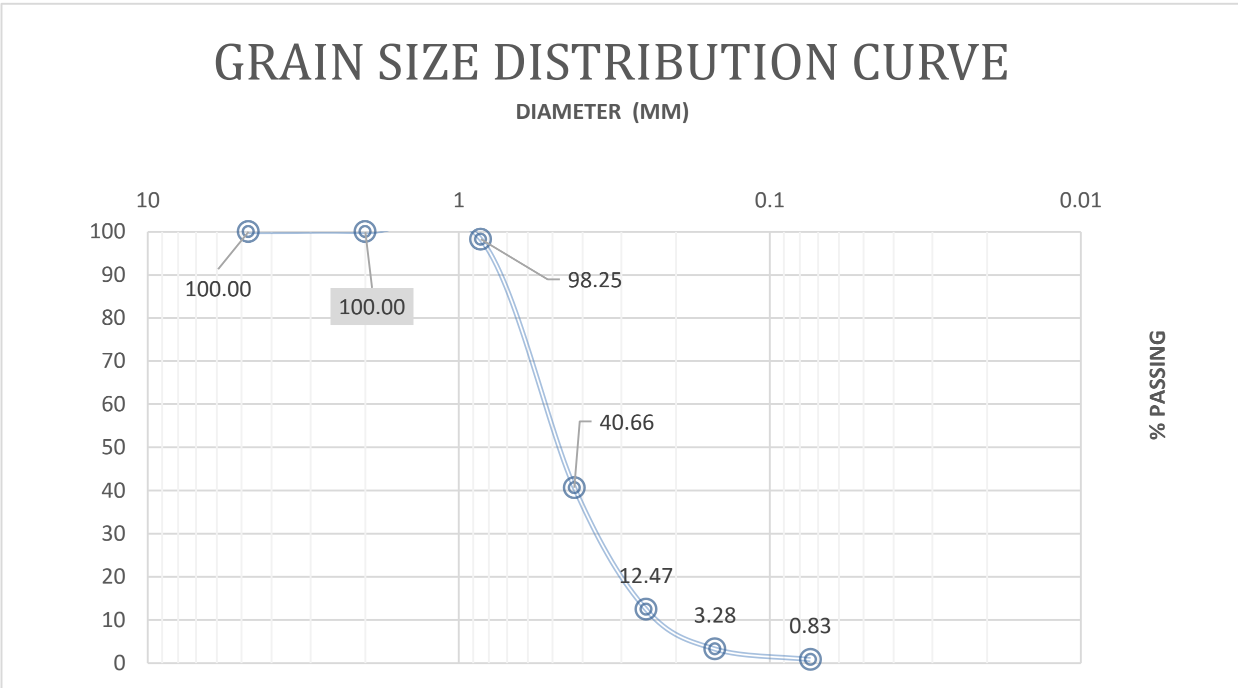


Figure 4.1 Grain size distribution curve

4.2.2 Clay

Similarly, sieve test on clay sample was performed. Before the test, soil was pulverized to break the lumps of the clayey soil. After the pulverization, soil was passed through the set of sieves and the gradation curve obtained is as follows:

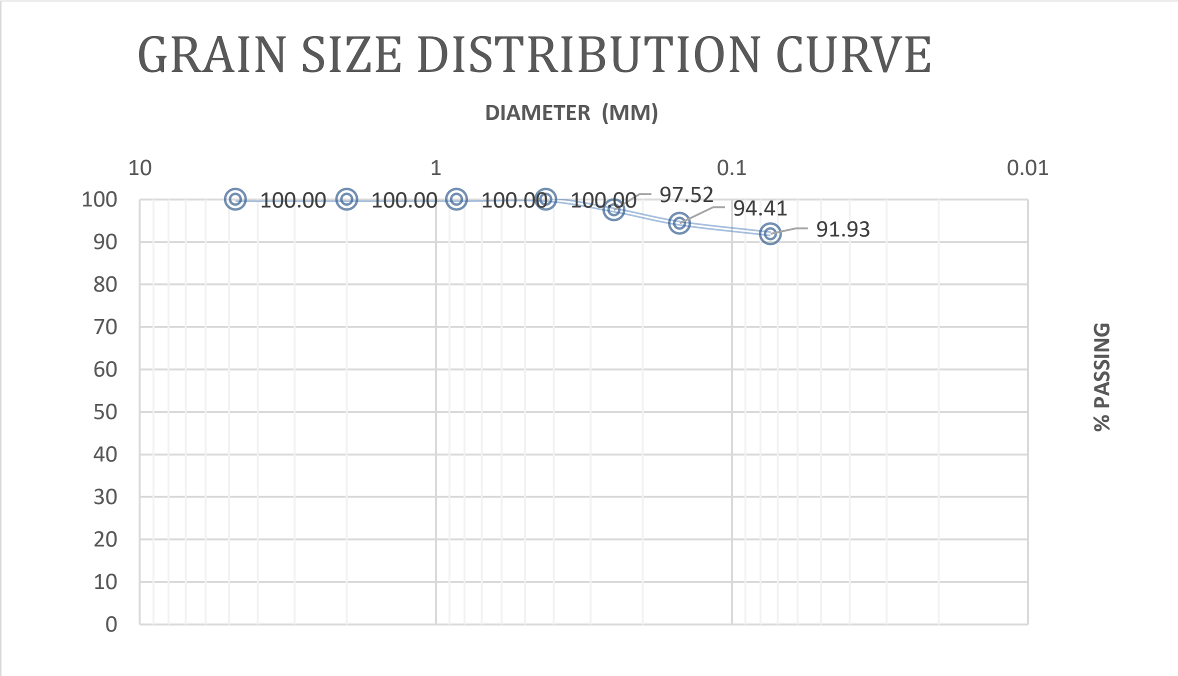


Figure 4.2 Grain size distribution curve for clay

Classification of clayey soil is **CL** in USCS and **A-4(8)** in AASHTO

4.3 Attenberg Limit Tests

Attenberg test was performed on the clayey soil sample and the results are as follows:

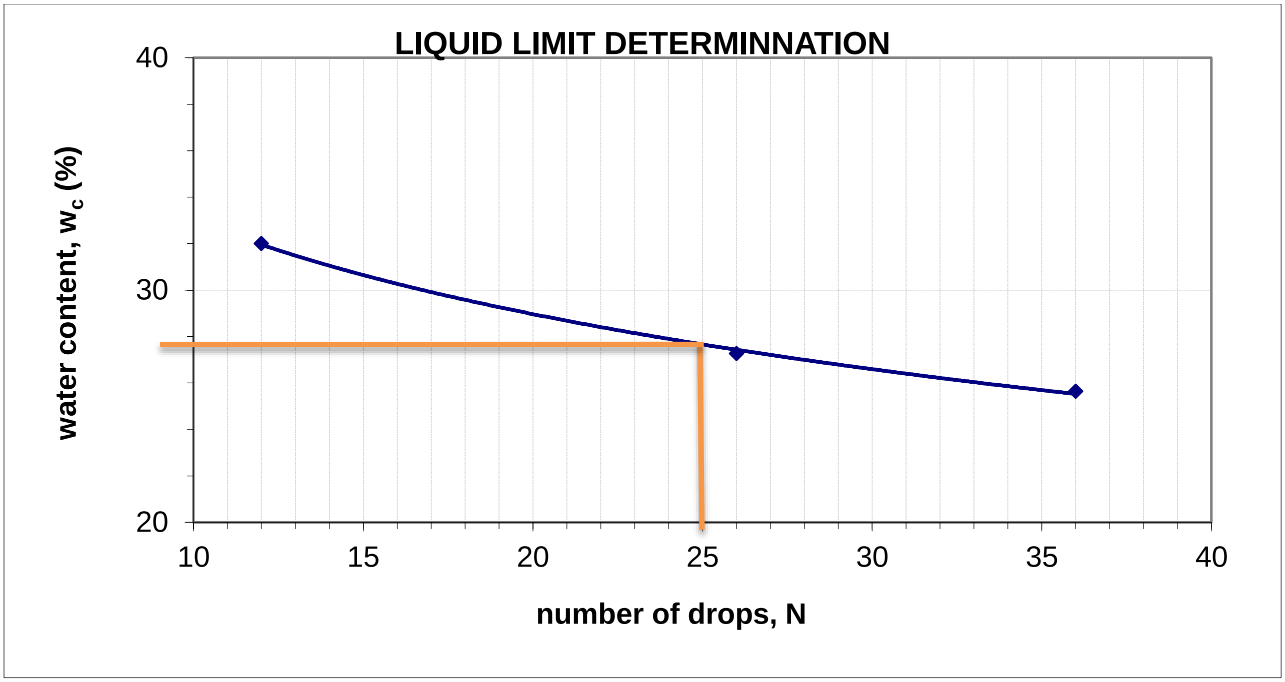


Figure 4.3 No. of blows vs. Water content

LIQUID LIMIT (LL) =	27.5	%
PLASTIC LIMIT (LL) =	17.5	%
PLASTIC INDEX (PI) =	10.0	

4.4 Specific Gravity

Specific Gravity of the sand was also determined as it is also a very important factor and its value obtained after lab tests and applying correction is as follows:

Specific Gravity with temperature correction: **2.66**

4.5 Direct Shear Test:

Direct Shear test was done on sand. This tests results in the shear strength parameters and give us the values of cohesion, peak friction angle and critical state friction angle. The results of the tests are as follows:

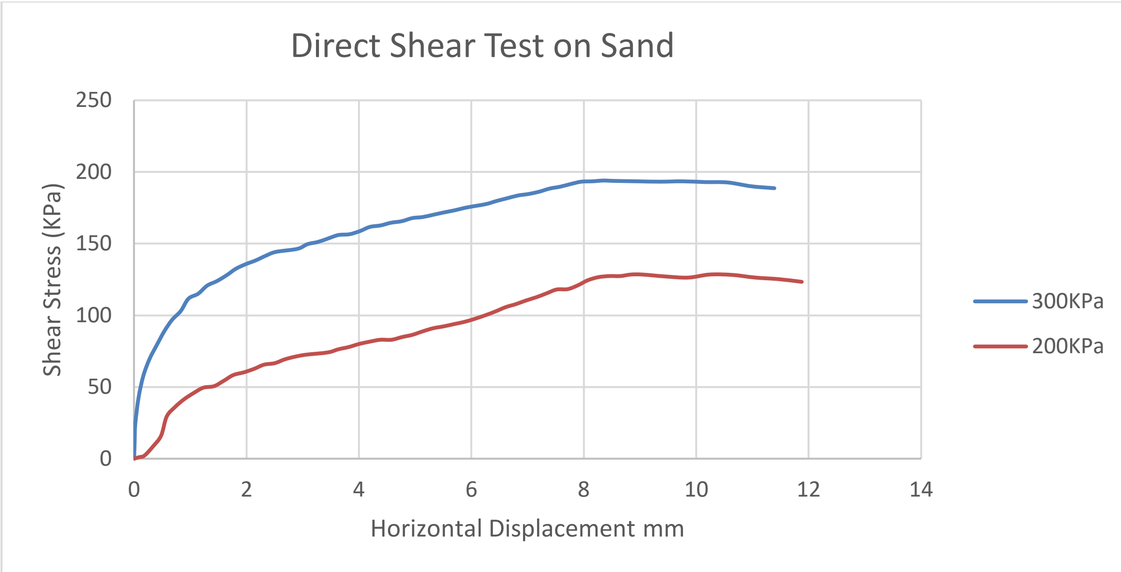


Figure 4.4 Direct Shear Test

4.5.3 Shear strength Parameters

Shear strength parameters obtained with the help of direct shear test are as follows:

Medium Dense Sand	Parameters	Dense Sand	Parameters
c (cohesion)	2	c (Cohesion)	1
Peak Friction Angle (Phi p)	38	Peak Friction Angle (Phi p)	32
Phi c (degree)	29	Phi c (degree)	29

Figure 4.5 Sand shear strength parameters

4.6 Soil Strength Measurement

Strength of clay is defined by its shear strength which was measured with the help of three methods:

1. Vane Shear Test
2. Penetrometer Test
3. Unconfined Compression Test

These tests were firstly applied on the iterative samples and then on the larger samples of drum. The results of these tests are as follows:

4.6.1 Vane Shear Test

This rest was performed after the preparation of clay samples before each test. This test was repeated 12 times for the one type of clay layer having 15% moisture content. The test was performed at all the four corners of the samples and then averaged to get the mean value. Results are attached in Appendix B. The value from all the shear tests ranges as follows:

Vane Shear Test	Value
For the 12 tests on Clay having 15% moisture content	Range: 40-42 KPa

Table 4.6 Vane Shear test strength measurement

4.6.2 Penetrometer Test

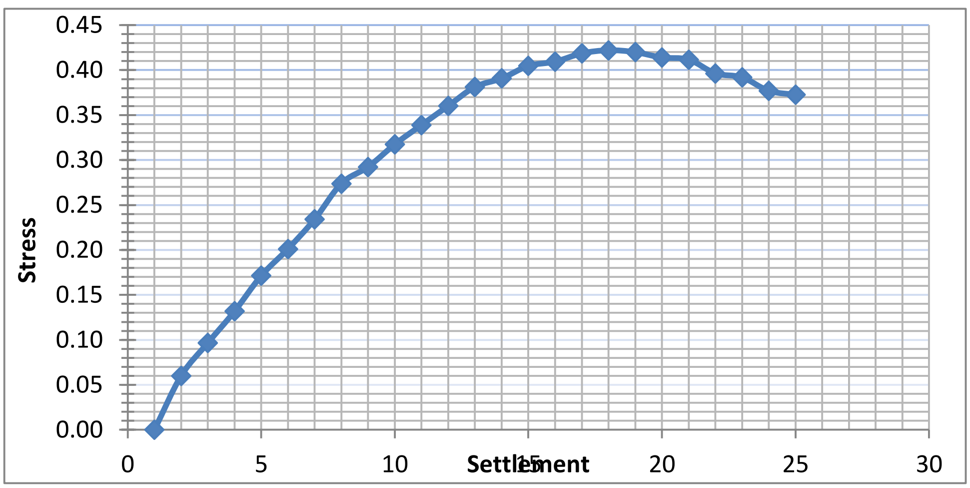
This test was also performed to verify the in-situ test results of vane shear test. This test gives directly the values in tons/ft^2. This test is also performed on all the four sides of the sample and the range of the value obtained for all the tests are as follows:

Penetrometer Test	Value
For the 12 tests on Clay having 15% moisture content	Range: 0.37-0.41 tons/ft^2 Range: 39-43.2KPa

Table 4.7 Penetrometer Results

4.6.3 Unconfined Compression Test

Unconfined compression tests were performed in the lab on the undisturbed samples taken from the samples just after each test. These samples were taken to verify the results of both insitu vane shear tests and penetrometer tests. The results obtained with the help of this sample finally defines the strength of the soil. The values of UCCT were as follows:



Unconfined Compression Test	Value
For the 12 tests on Clay having 15% moisture content	Range: 41-42KPa Average Value: 42KPa

Table 4.8 UCC test results

This shows the accuracy of the samples and all the samples were of same strength. Thus, a comparison analysis could be done on the results obtained on these similar samples. One of the results of UCC test are shown as follows:

4.7 Model Experimentation Results

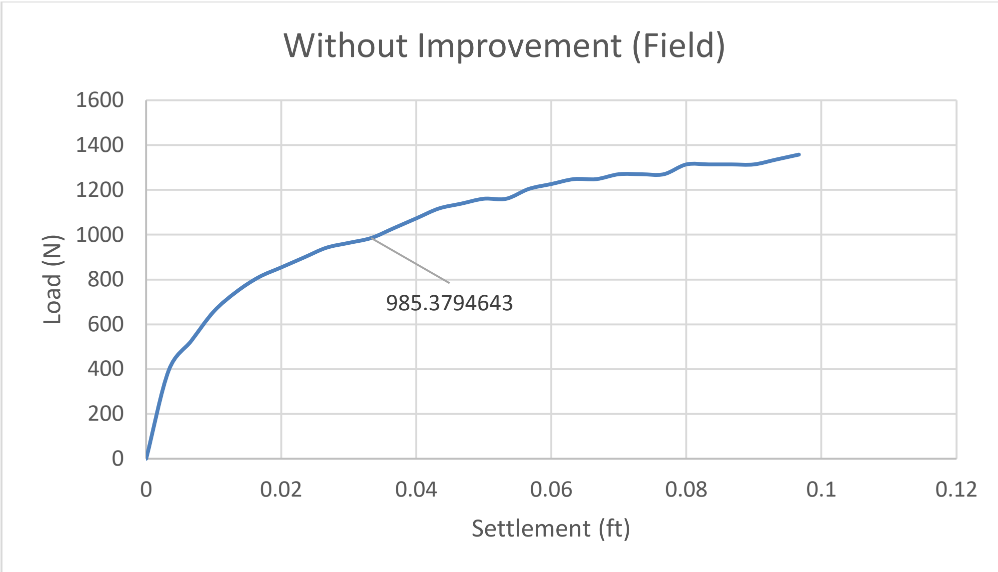
Model Experimentation was conducted with the aim to check for the improvement in bearing capacity with the increase in the depth of replacement of compacted sand in the weaker clayey strata. Moreover, footing size was also varied to check for the effect of size of footing on bearing capacity and to finally analyze the best option in case of improvement by soil replacement technique. Three footing were selected for the analysis having diameters of 4inch, 6inch and 8inch. The depth of replacement was varied at 0.25 times diameter of footing, 0.5D and 0.75 times diameter. CBR machine with the proving ring showing load and dial gauge showing displacement was used to obtain the load vs. settlement curves for each footing and each replacement. The results of the testing are shown below:

4.7.1 For Footing of 4inch Diameter

For the 4 inch diameter footing plate, following tests were conducted:

4.7.1.1 Without Replacement

For the first test, 4 inch diameter footing was placed directly over the weak clayey layer. The load was gradually applied and corresponding settlement was noted. The load vs. settlement curve obtained is shown below:

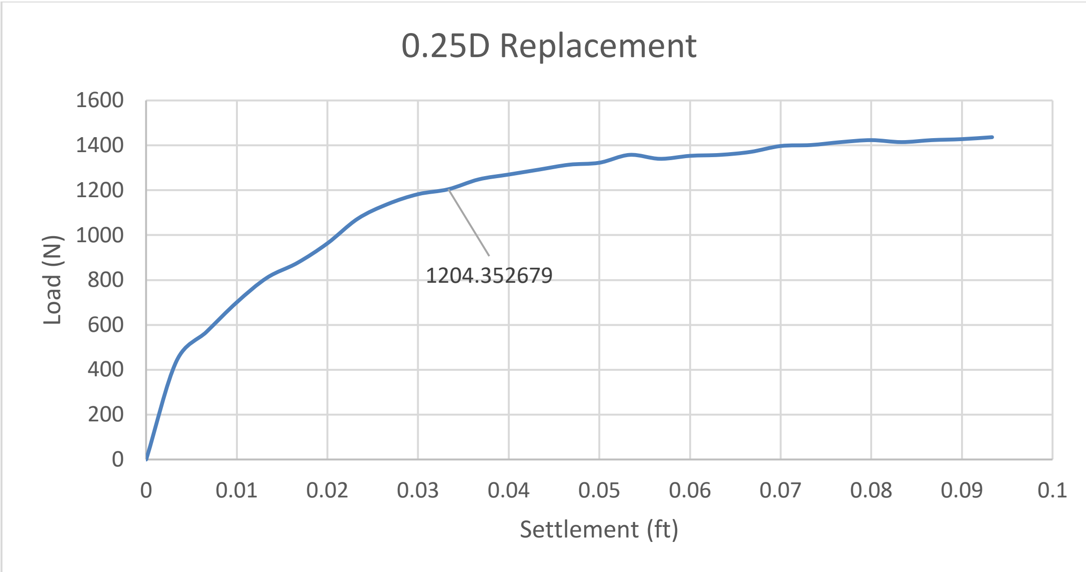


By analyzing the curve, we can see that there is not a definite failure and no definite peak is observed thus, by 10% displacement rule the load against settlement of 10% the diameter was considered. Thus,

F load at 10% settlement= 985.38 N= 9.8538KN

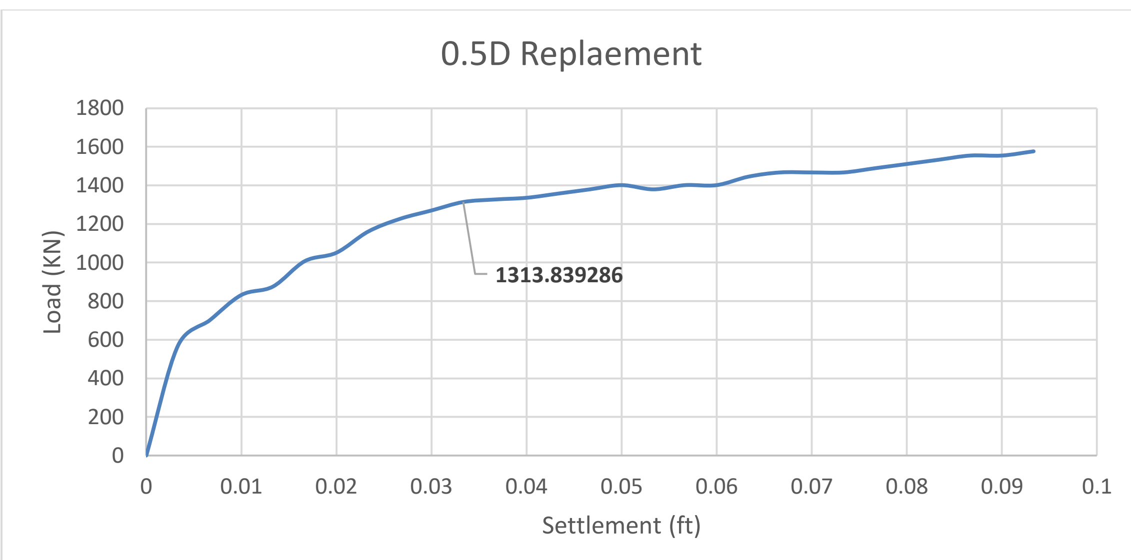
4.7.1.2 With 0.25D Replacement

Now, the clayey soil was excavated to a depth of 0.25 times diameter and was filled with the compacted sand having optimum moisture content. Soil was not compacted with proctor mold hammer or vibratory compactor because it could result in the compaction of weak clayey soil beneath. Thus, soil was compacted with the help of raining effect and was filled till the upper level of clay. Then, the similar procedure was adopted to check the load vs. settlement curve. The results obtained are as follows:



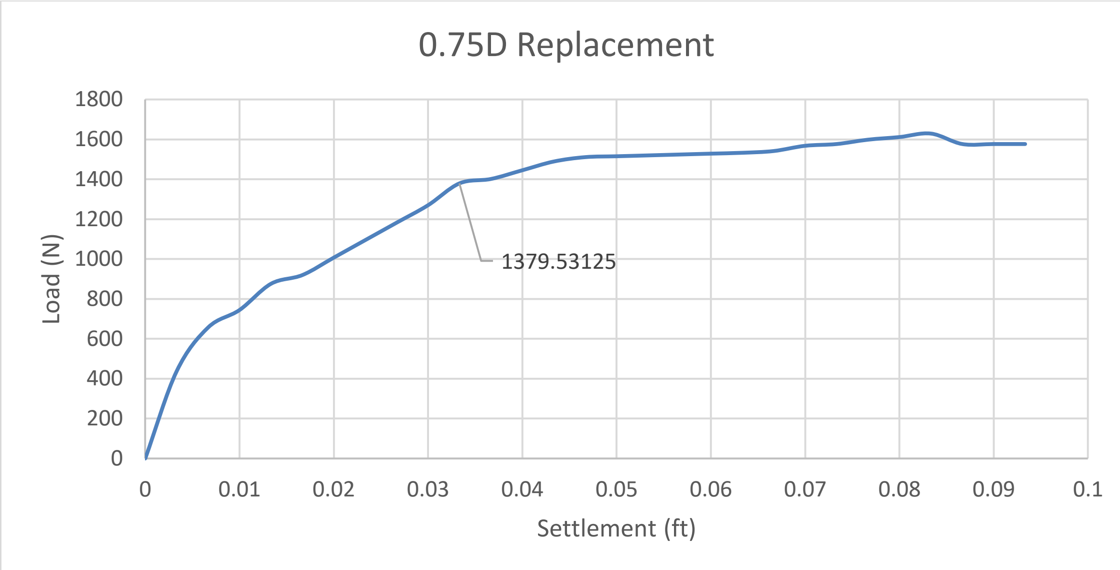
4.7.1.3 With 0.5D Replacement

Similar procedure was adopted with the depth increment of the replaced layer up to 0.5times the diameter of footing. The results of Load vs. Settlement curve are as follows:



4.7.1.4 With 0.75D Replacement

Same procedure was repeated but with the depth of compacted soil increased up to 0.75 times the diameter of footing. The load vs. settlement curve obtained is as follows:



4.7.1.5 Comparison

The combined load vs. settlement curve is shown below:

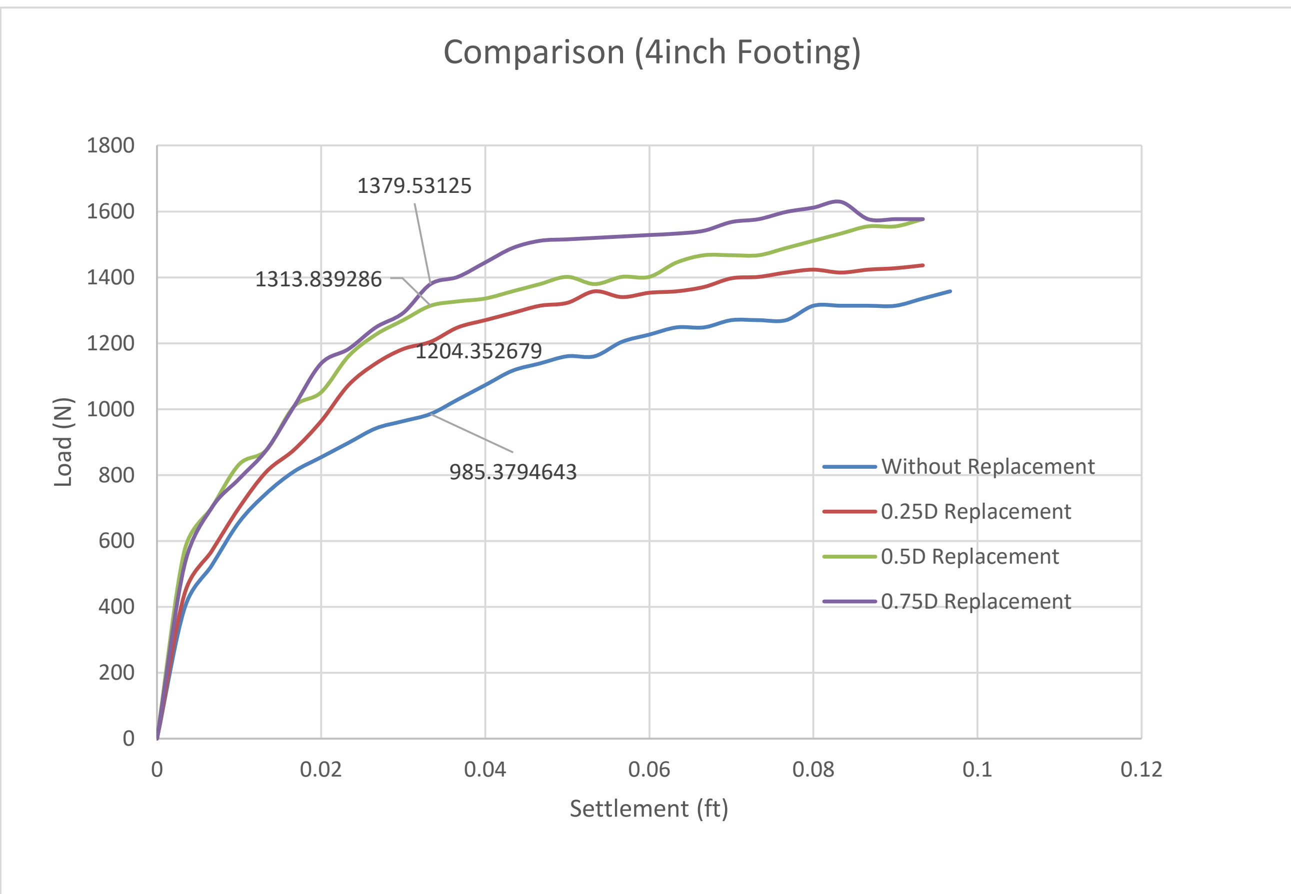


Figure 4.9 Comparison 4inch Footing

It can be seen from the curve that with the increase in the depth of replacement, the load carrying capacity of the soil increases. For example, for this footing the load against the 10% settlement was increased almost 22% at 0.25D replacement as compared to the load at 10% settlement of the test without replacement.

It is also seen that after 0.25D replacement, the increase in the load carrying capacity decreases and the behavior becomes lot stiffer. This stiffness with the increase in depth is shown below:

Replacement(4inch Diameter footing)	**Load(N)**	**% Increase from without Replacement**
Without Replacement	985.379	0%
With 0.25D Replacement	1204.35	22%
With 0.5D Replacement	1313.84	9.5%
With 0.75D Replacement	1379.5	5%

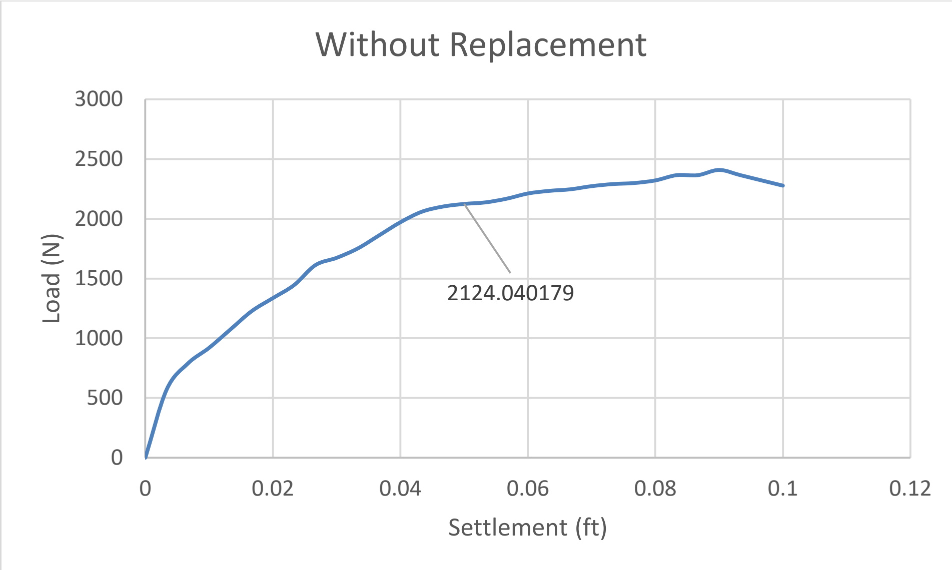
Table 4.10 Results comparison of 4 inch footing

4.7.2 For Footing of 6inch Diameter

The results of the 6inch diameter footing without and with different replacement are shown below:

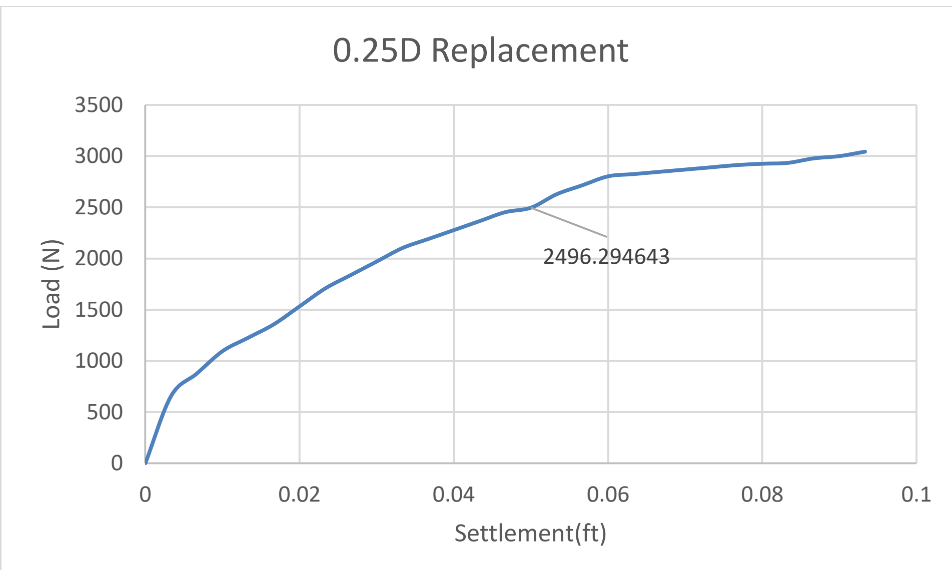
4.7.2.1 Without Replacement

Load vs. settlement curve for 6inch footing directly residing on clay layer without any replacement is as follows:



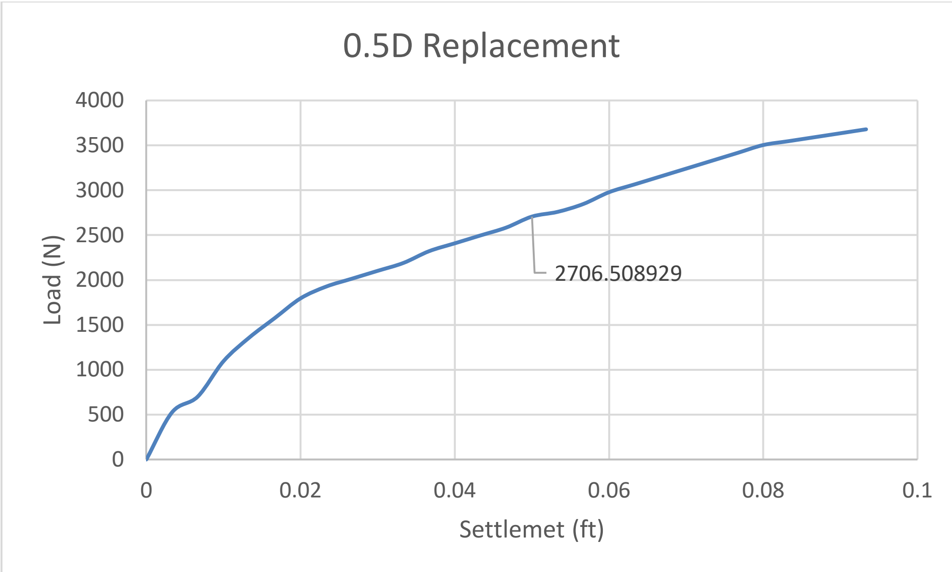
4.7.2.2 With 0.25D Replacement

Load vs. settlement curve for 0.25D replacement is as follows:



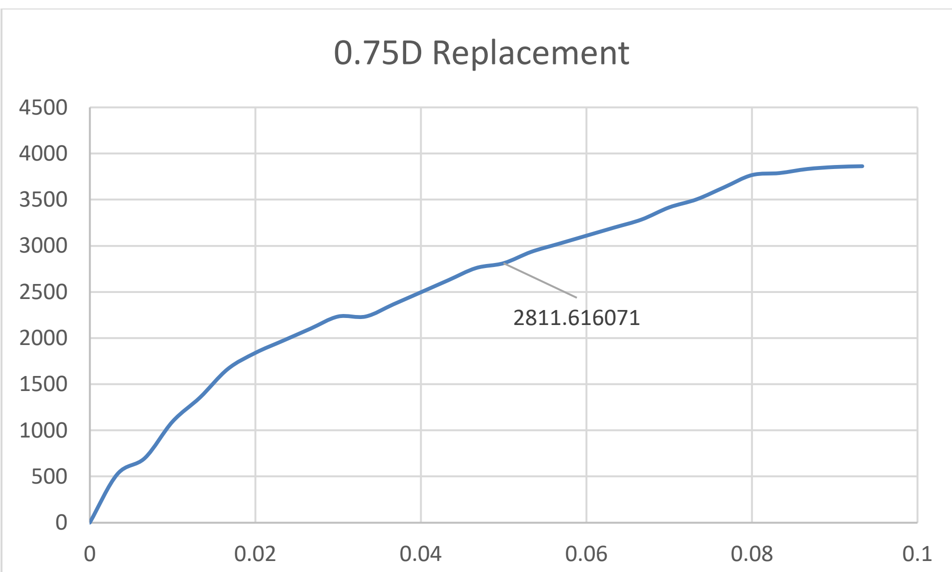
4.7.2.3 With 0.5D Replacement

Load vs. settlement curve for 0.5D replacement is as follows:



4.7.2.4 With 0.75D Replacement

Load vs. settlement curve for 0.75D replacement is as follows:



4.7.2.5 Comparison

By comparing the second footing, we can observe that the load carrying capacity of the footing is much increased as compared to the 4 inch footing size but if we compare the improvement percentage of 6inch footing with the 4 inch, we can see a decrement in improvement percentage. This shows that as the soil replacement depth is increased, the stiffer behavior is exhibited by the soil. This is due to the increment in the size of the footing on the similar conditions of soil and partial soil replacement. This is shown by the cumulative graph as follows:

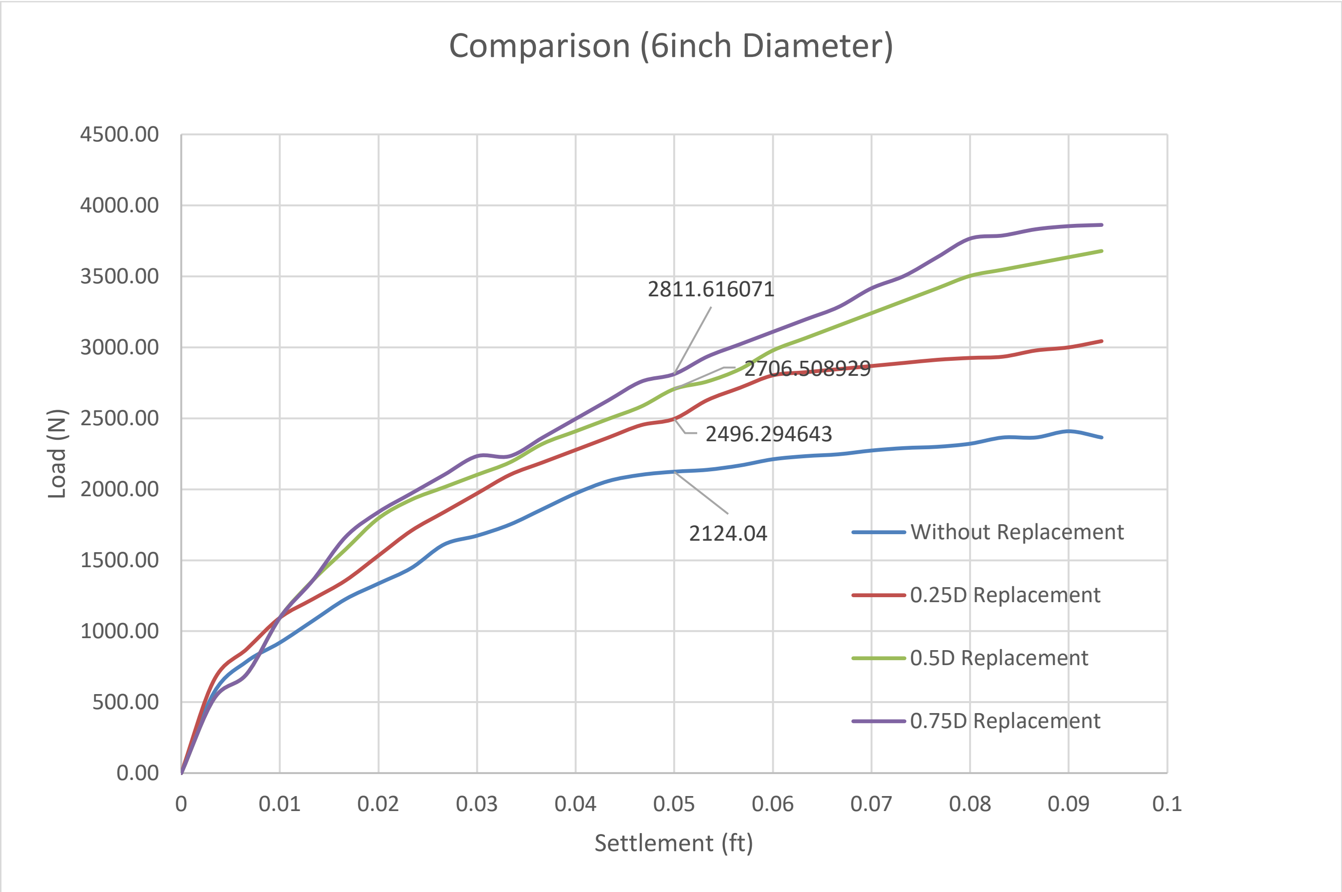


Figure 4.11 Comparison 6inch Footing

The percentage (%) increase in the load with replacement is as follows:

Replacement (Diameter = 6inch)	Load (N)	% Increase from without Replacement
Without Replacement	2124.04	0%
With 0.25D Replacement	2496.2946	17%
With 0.5D Replacement	2706.533	8%

With 0.75D Replacement	2811.625	4%

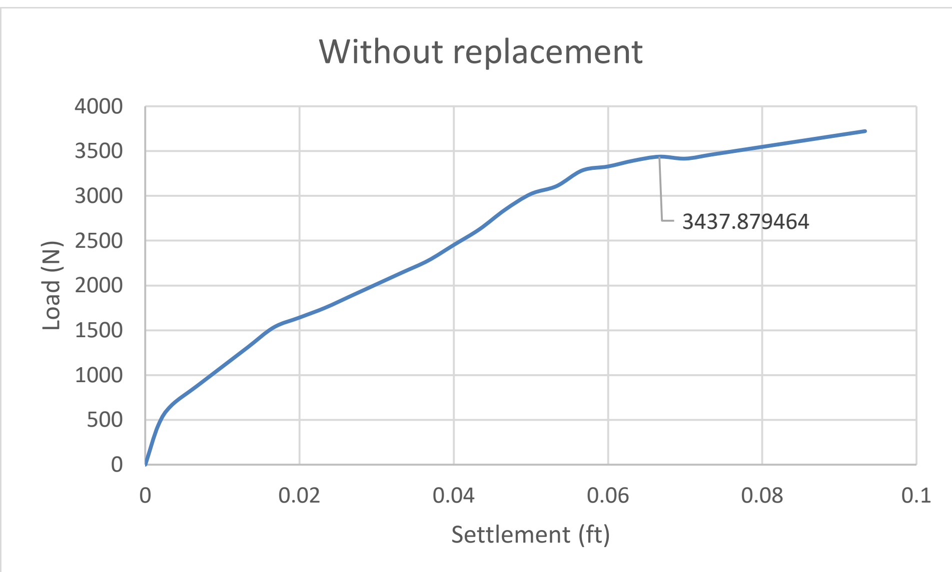
Table 4.12 Results comparison of 6 inch footing

4.7.3 For Footing of 8inch

Similarly, the footing size was increased up to 8 inches so that an efficient comparison could be made for the effect of footing size along with replacement. The results of 8inch diameter footing are as follows:

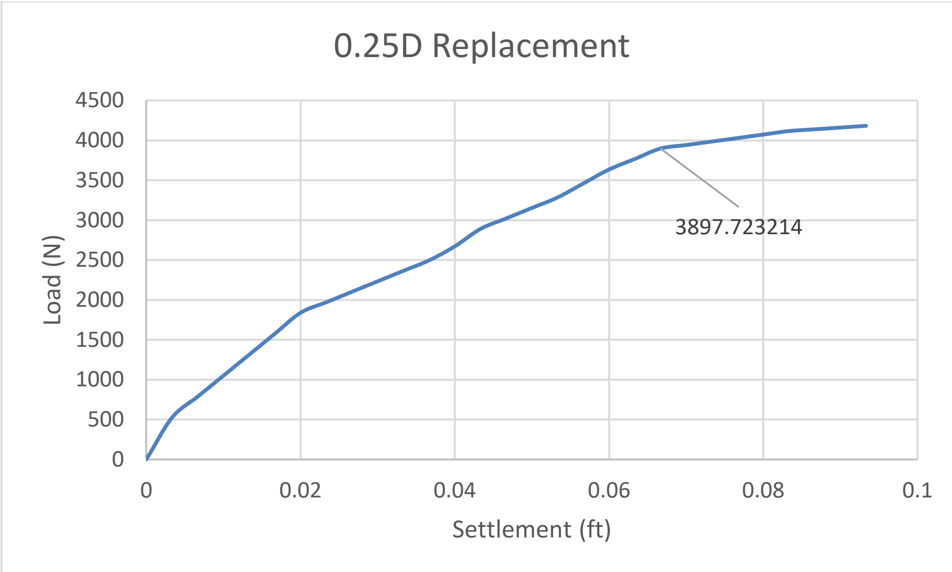
4.7.3.1 Without Replacement

Load vs. settlement curve for 8inch footing directly residing on clay layer without any replacement is as follows:



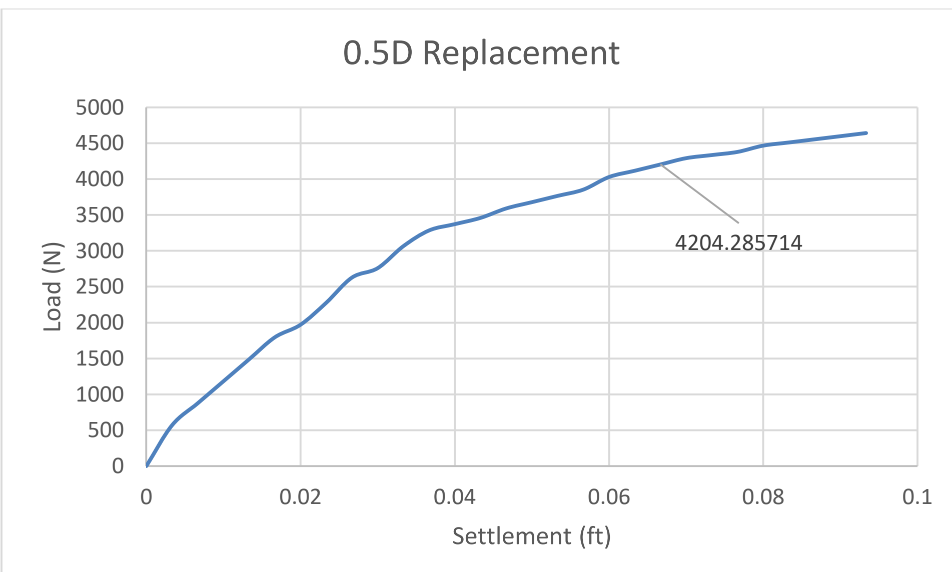
4.7.3.2 With 0.25D Replacement

Load vs. settlement curve for 0.25D replacement is as follows:



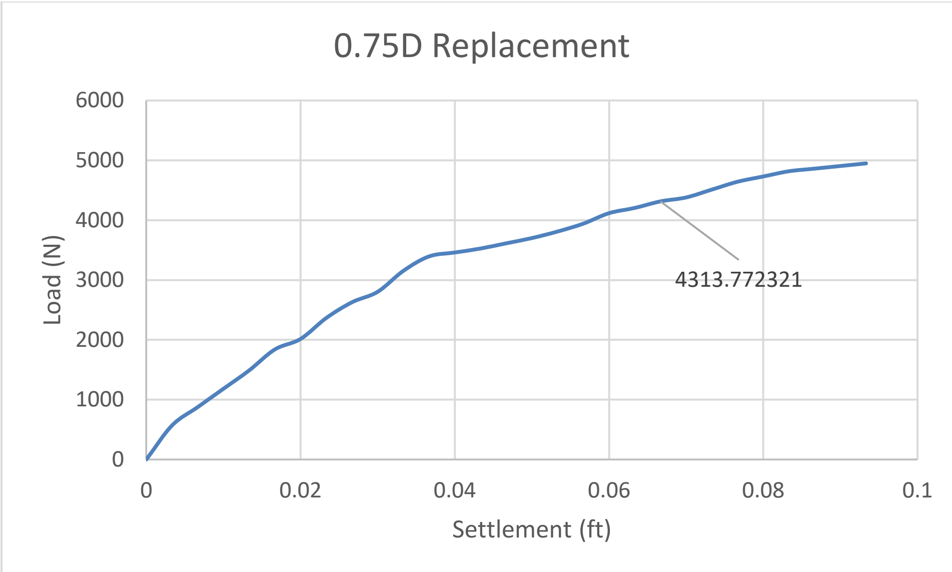
4.7.3.3 With 0.5D Replacement

Load vs. settlement curve for 0.5D replacement is as follows:



4.7.3.4 With 0.75D Replacement

Load vs. settlement curve for 0.75D replacement is as follows:



4.7.3.5 Comparison

Similar trend was seen in case of 8inch diameter footing whereas the percentage in increment is further decreased as the footing size increases. But on the other hand, the capacity of load carrying is much improved as compared to smaller footing sizes. This is shown by the graph below:

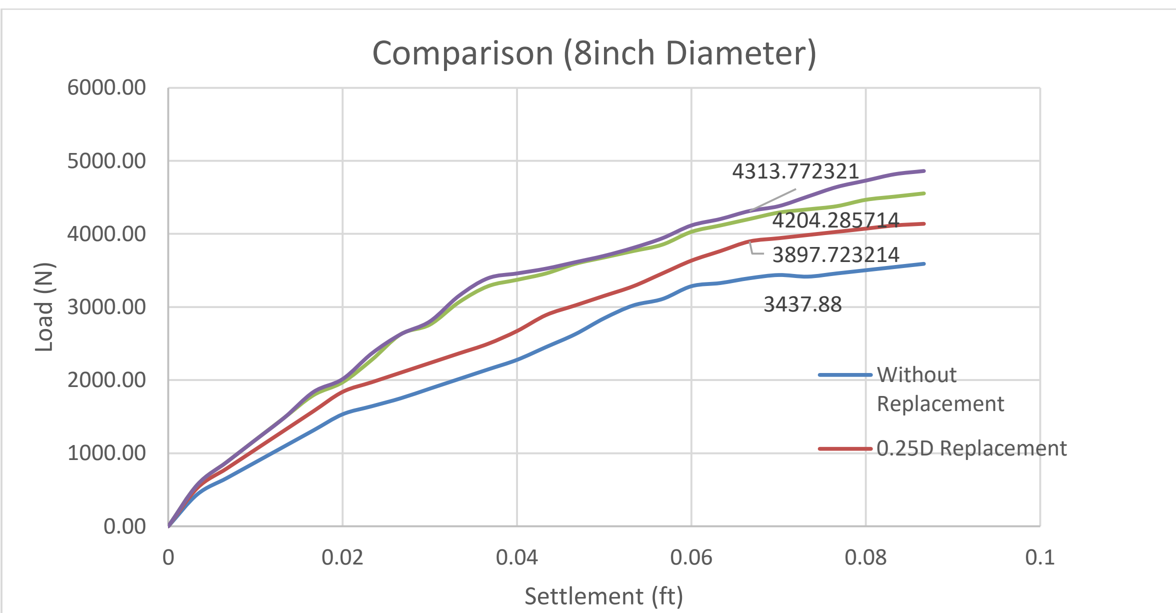


Figure 4.13 Comparison 8inch Footing

Replacement (Diameter = 6inch)	Load (N)	% Increase from without Replacement
Without Replacement	3437.87	0%
With 0.25D Replacement	3897.723	13%
With 0.5D Replacement	4204.28	7.3%
With 0.75D Replacement	4313.772321	2.6%

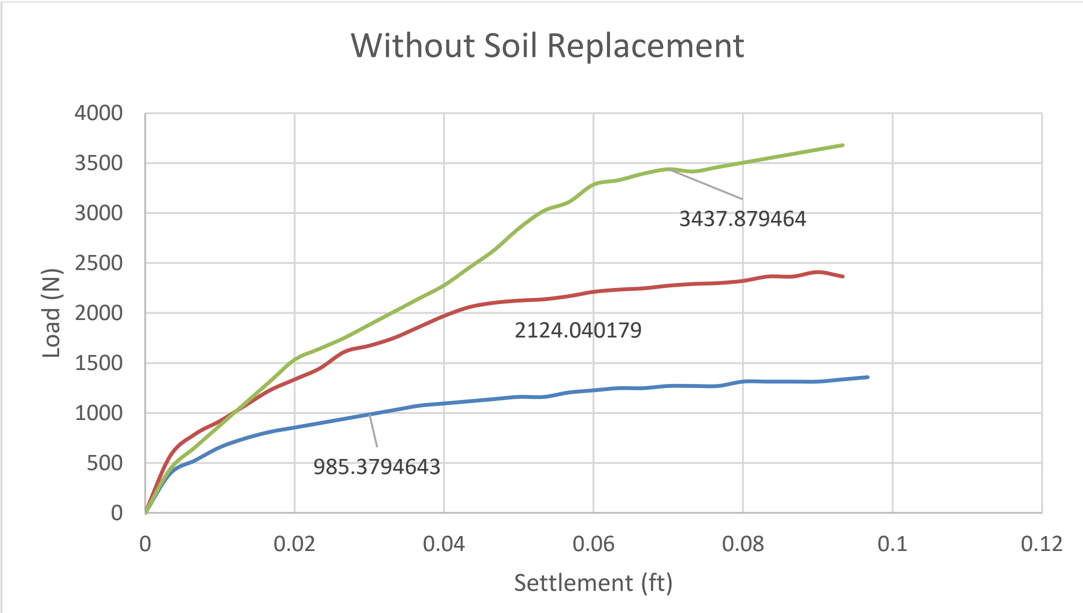
Table 4.14 Results comparison of 8 inch footing

4.8 Comparison between Footings

By comparing the results we can observe that the load carrying capacity of the footing increases with the increase in the footing size as well as with the increase in the depth of replacement. The results obtained shows that the increase in the capacity is much governed by the size of the footing. For a single footing case, the load carrying capacity increases with the increase the depth of replacement but this increase is much less as compared to the improvement obtained by increasing the size of the footing. Moreover, the increase in load carrying capacity due to increase in depth of replacement decreases as we move towards larger depths of displacements. This means that the maximum increment in load carrying capacity is observed during the initial replacement and then it decreases for larger depths of replacements. The effect of size of footing is explained below with comparison at each replacement.

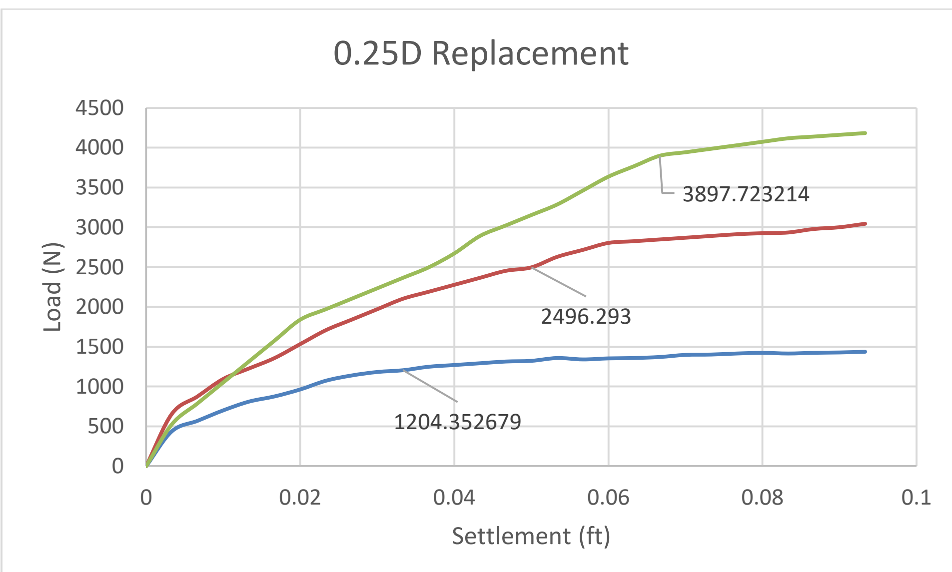
4.8.1 Without Replacement

Without replacement and directly placing the foundation over the soft clayey strata, footing size plays an important role. As, the footing size increases, the load carrying capacity increase with it. It is shown as follows:



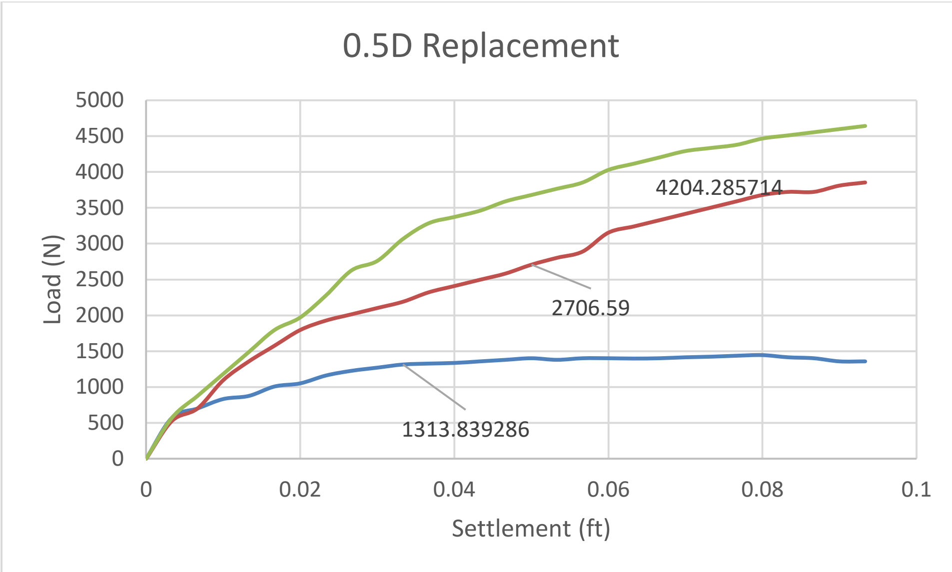
4.8.2 With 0.25D Replacement

Similarly, the effect of footing size could be analyzed with 0.25D replacement.



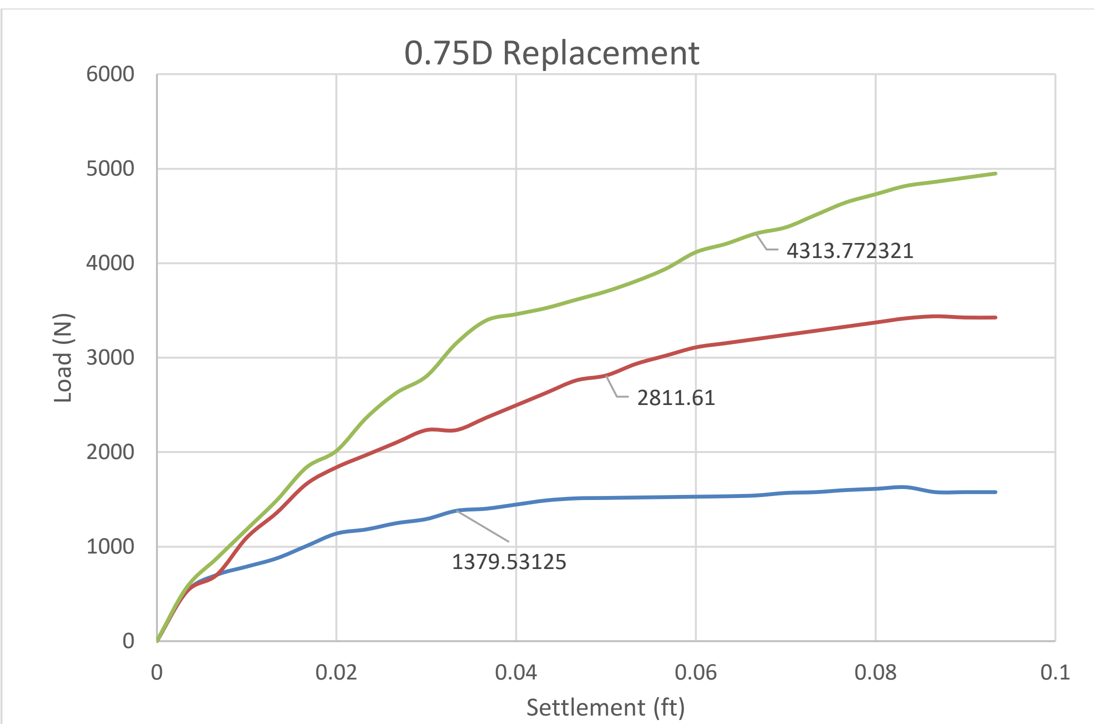
4.8.3 With 0.5D Replacement

Similarly, the effect of footing size could be analyzed with 0.5D replacement as follows:



4.8.4 With 0.75D Replacement

Similarly, the effect of footing size could be analyzed with 0.75D replacement with the help of following graph:



4.8.5 Effect of footing size

With the help of the above graphs, we can deduce that the effect of increment of footing is much on the bearing capacity of the soil. The bearing capacity of the soil increases as the footing size increases. Moreover, it also shows that the effect of soil replacement is enhanced multiple times if the footing size is increased with it. This also shows how the footing size could improve the load carrying capacity better than the replacement technique.

4.9 Correlation based on soil replacement and footing size

With the help of the results obtained using this research, a correlation is developed which will help to predict the bearing capacity on the basis of depth of soil replacement and size of footing. Here, depth of soil replacement is represented by 'H', diameter of footing as 'D', reinforced soil bearing capacity as 'qr' and unreinforced soil as 'qo'. A graph is plotted representing the BCR (Bearing Capacity Ratio = qr/qo) on Y axis and the H/D ratio on x- axis. The results are as follows:

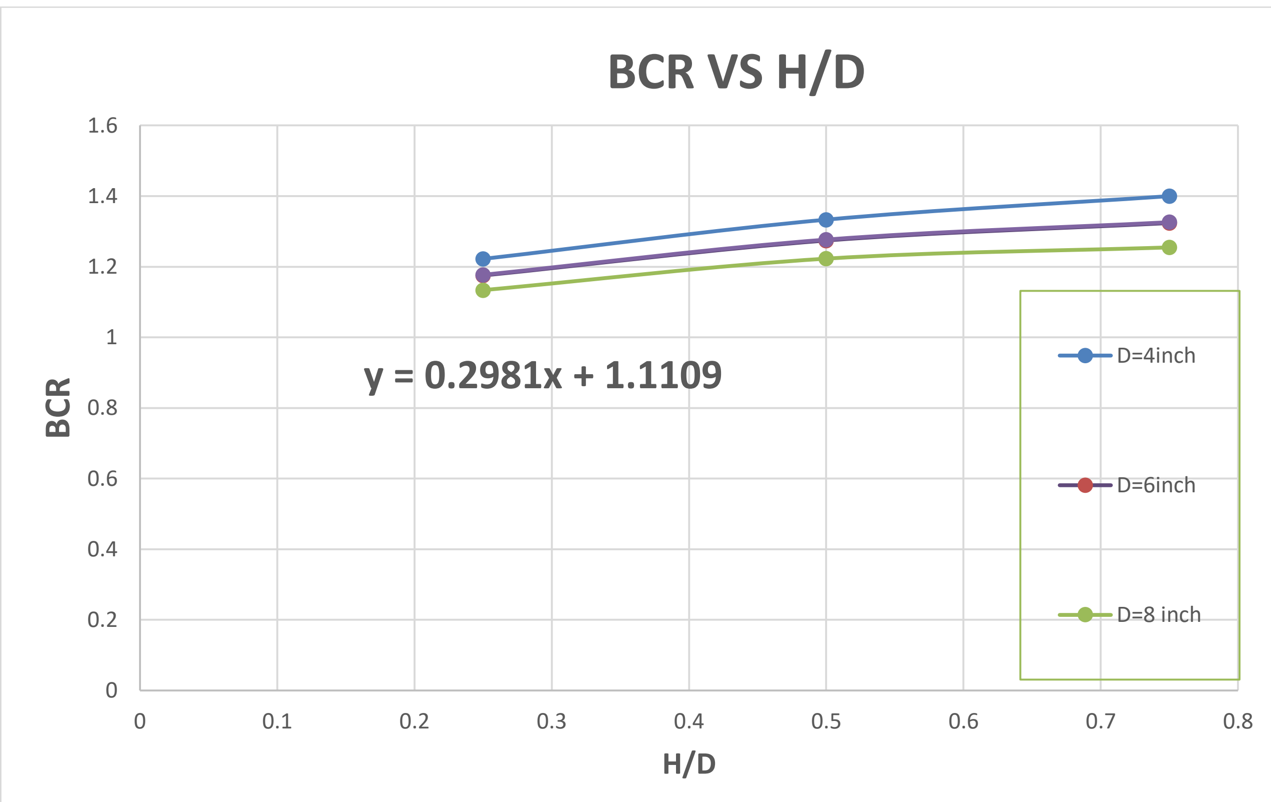


Figure 4.15 Correlation based on soil replacement and footing size

Thus, the average equation developed from the curves is as follows:

$$\mathbf{BCR = 0.2981(H/D) + 1.1109}$$

Thus with the help of this equation, we can determine the bearing capacity improvement ratio with respect to the footing diameter and thickness of the replaced layer.

4.10 Conclusion

This chapter has addressed all the experimental tests performed in the lab for the basic testing as well as the tests performed on the model assembly. The results of the model tests were explained in details and were correlated to develop certain conclusions from them. Effects of replacement for each footing diameter were compared and incremental percentages were explained. Along with that, the effect of footing size with respect to each replacement depth was explained. At the end an equation was developed for estimating the Bearing capacity improvement ratio with the inputs of diameter and thickness of the replaced layer.

Part B: Finite Element Results and Analysis

4.11 General

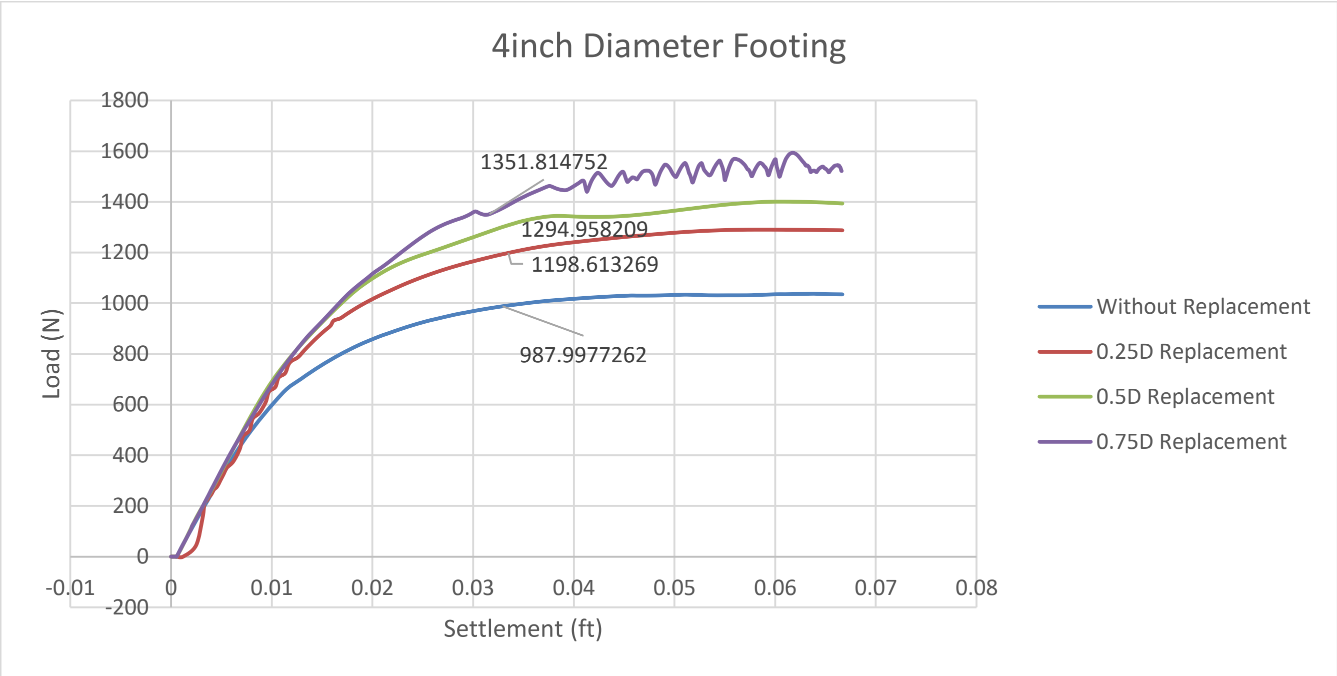
This part of chapter discusses the finite element analysis done on the basis of same experiment models and the analysis of the results obtained. Along with the comparison of the field results, the failure mechanism was also explained explaining how the foundation will fail under reinforced soil. Below are the results of all the results obtained.

4.12 Finite Element Results

Following are the combined results of load vs. settlement curves obtained using Plaxis 2D:

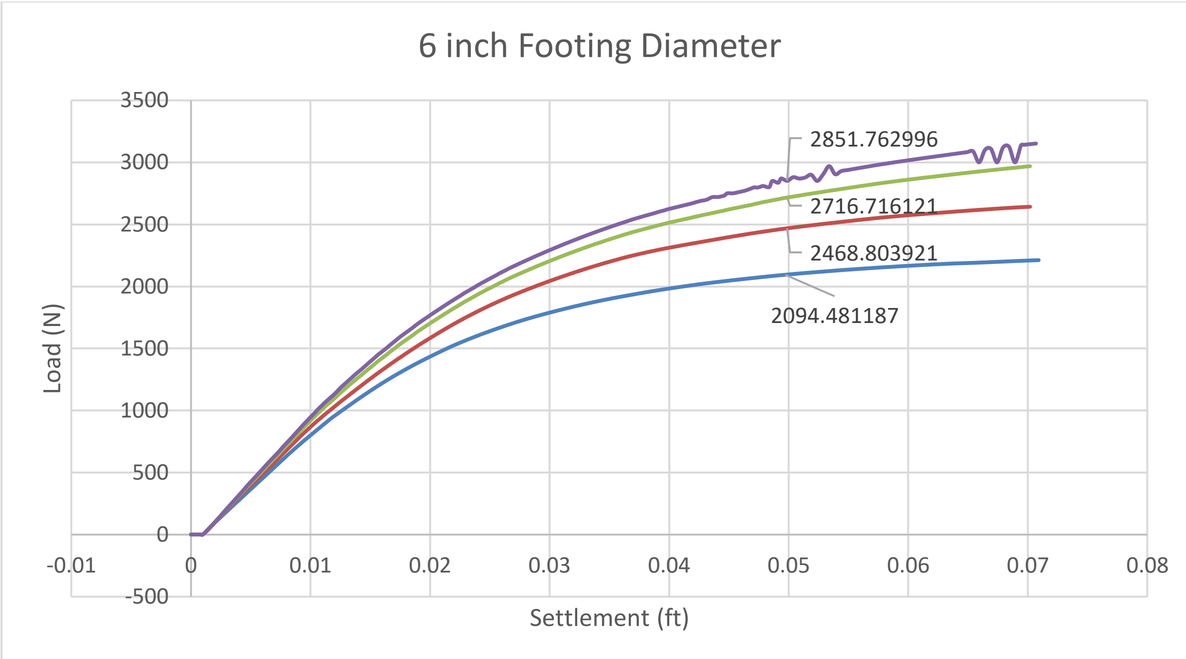
4.12.1 For 4 inch Diameter Footing

The results obtained with the help of Plaxis 2D for 4 inch footings is as follows:

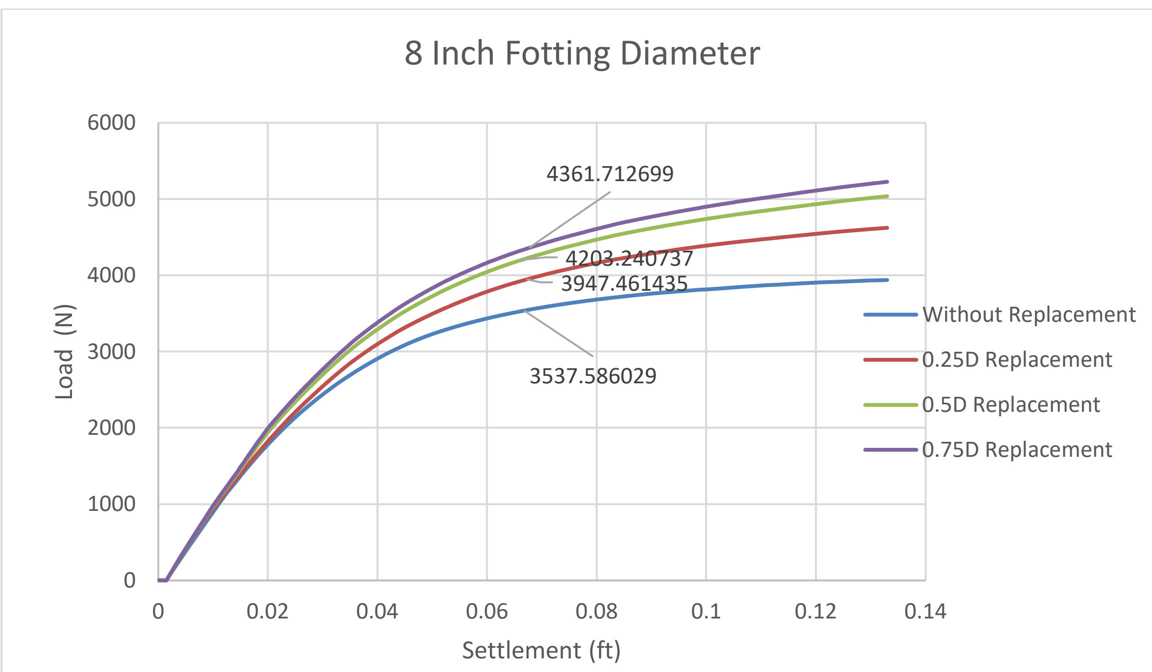


4.12.2 For 6inch Diameter Footing

The combined results of 6inc footing are as follows:



4.12.3 For 8inch Diameter Footing



4.12.4 Comparison to field

The graphs above exhibit almost similar behavior to that of the data obtained from the field. The difference in the values obtained by using finite element analysis and field data is shown with the help of the table below:

Footing		Difference			% Difference
Footing Size	Replacement	Plaxis Value(N)	Field Value (N)	Difference(N)	
4 inch	Without	987.99	985.379	2.611	0.265
	0.25D	1198.61	1204.35	5.74	0.479
	0.5D	1294.958	1313.839	18.8812	1.458
	0.75D	1351.8147	1379.531	27.71655	2.050
6Inch	Without	2094.4811	2124.04	29.5589	1.411
	0.25D	2468.8039	2496.295	27.4907	1.114
	0.5D	2716.7162	2706.533	10.1832	0.376
	0.75D	2851.76299	2811.625	40.13799	1.428
8Inch	Without	3537.58	3437.87	99.71	2.900
	0.25D	3947.6	3897.723	49.877	1.280
	0.5D	4237.24	4204.28	32.96	0.784
	0.75D	4362.72	4313.772	48.94768	1.135

We can see from the results that the difference is less than 2% and is varying. This variation is due to the experimental variation and the small variation in the soil samples for each test.

4.13 Failure Mechanism

After performing the analysis on the Plaxis, the output also generate the visual representation of failure mechanism. This is explained as follows:

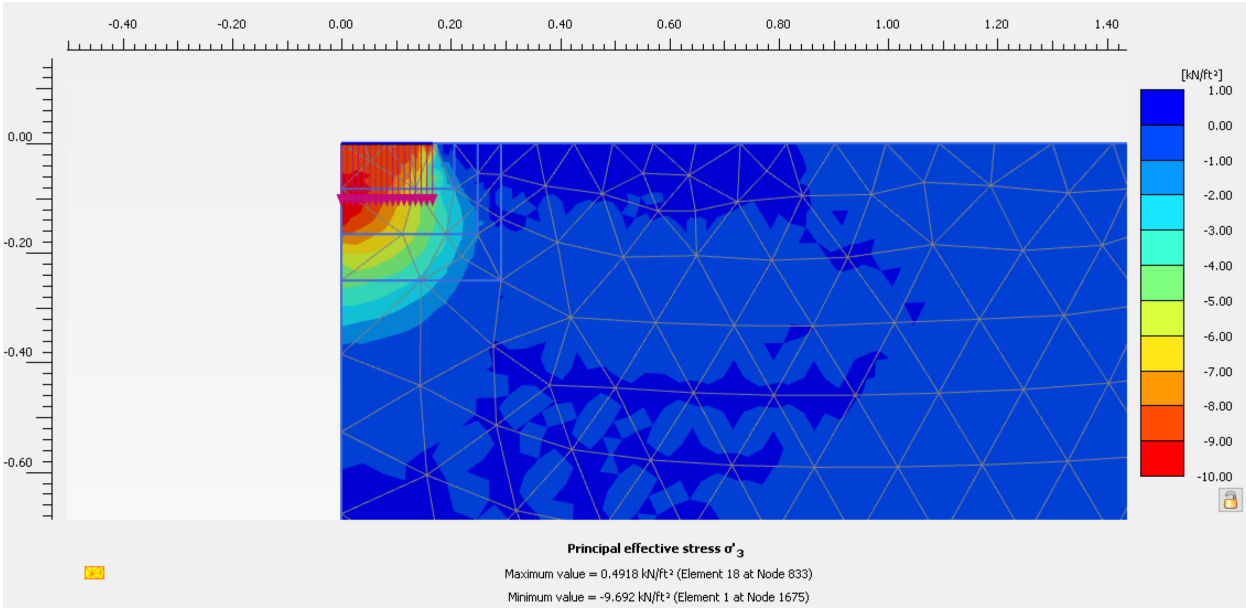


Figure 4.16 Failure pattern (without replacement)

This is the failure pattern of the footing without reinforced soil and is directly placed over clay layer. We can observe that all the stresses are developing right beneath the footing and a proper failure is shown by failure bulb.

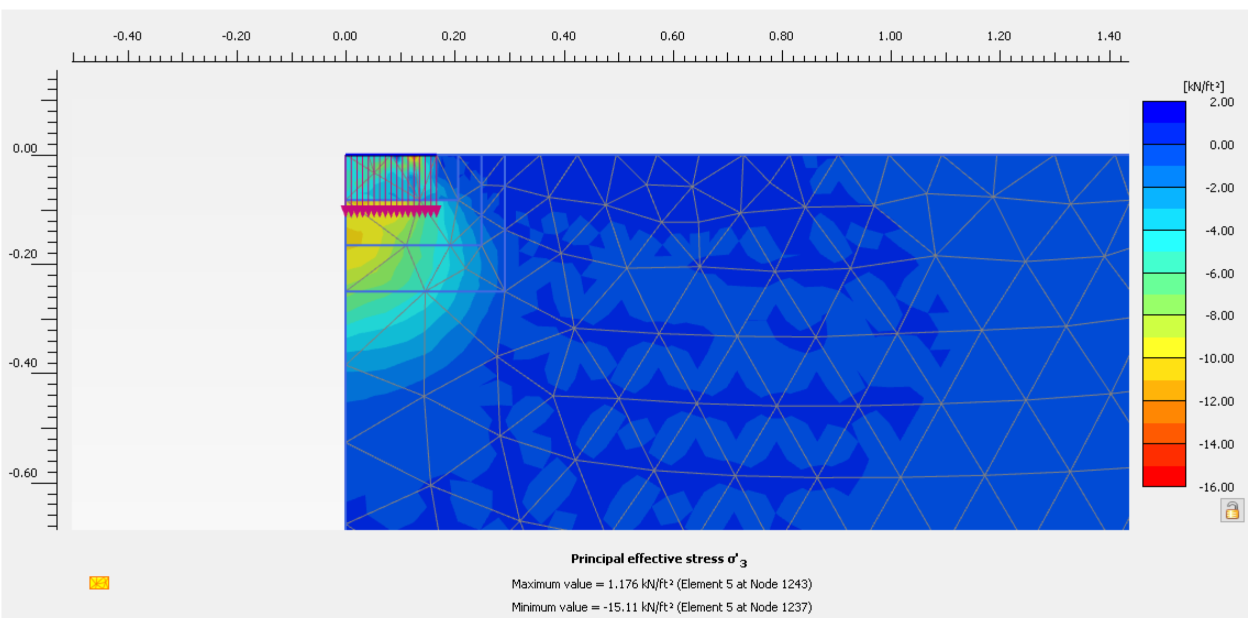


Figure 4.17 Failure Pattern (0.25D Replacement)

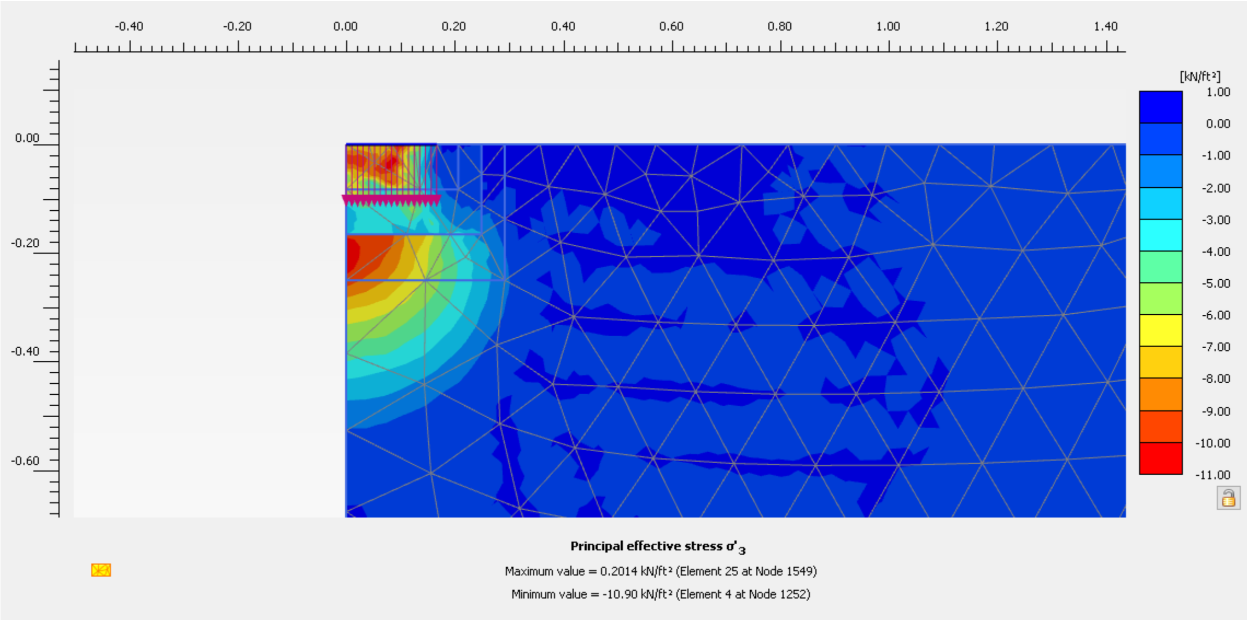


Figure 4.18 Failure Pattern (0.5D Replacement)

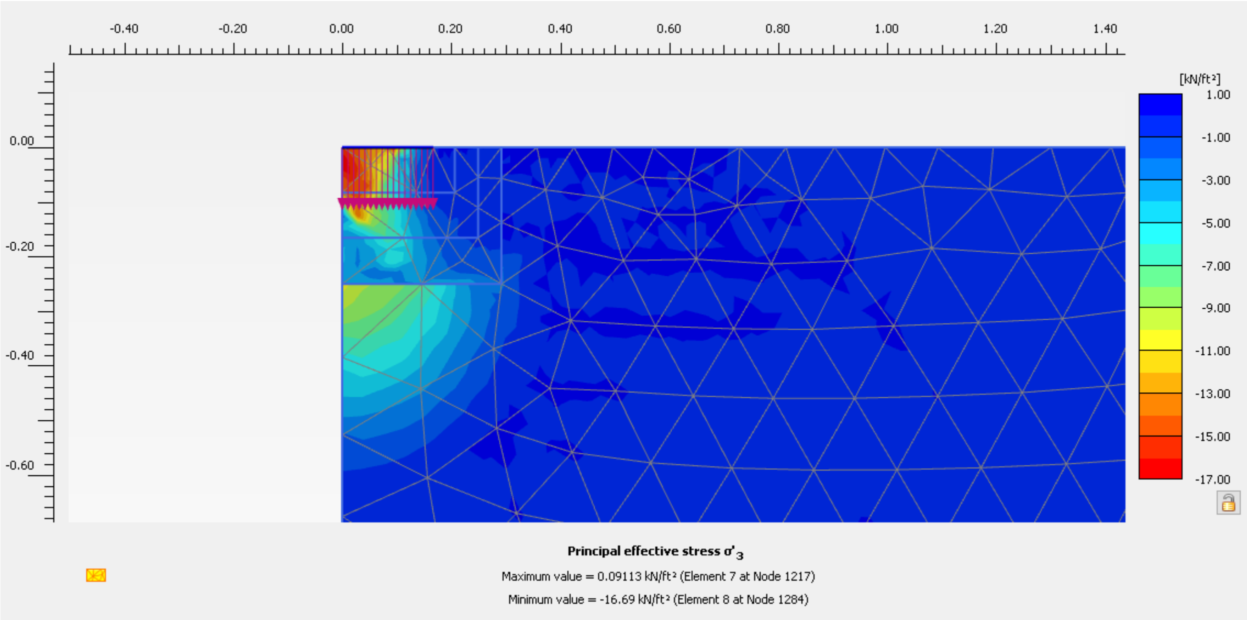


Figure 4.18 Failure Pattern (0.75D Replacement)

We can observe that as we are increasing the depth of the replacement, the stresses in the zone of replacement are not taking a proper failure bulb but rather are straight due to the stiffness of the sand layer. This helps in taking more load and improving the soil bearing capacity.

Chapter 5

Conclusions and Recommendations

5.1 General

This chapter focusses towards the finding of the research and the conclusion that are obtained with both the field experimentation and the Finite Element Analysis. Moreover, this chapter also suggests the recommendations for further research in this area and explains what could be researched in future related to this topic.

5.2 Conclusions

Following are the conclusion drawn after the completion of the research:

- Bearing Capacity increases with the depth of replacement but this increase is maximum during the first phase of replacement e.g. in our research it was 0.25 times the diameter of footing
- Load carrying capacity also increases with the size of the footing and this increment is much larger as compared to that achieved by soil replacement technique
- The percentage increase in the bearing capacity from the unreinforced soil decreases with the increase in the footing size.
- For a single replacement depth, the diameter change can increase the bearing capacity nearly twice.
- Bearing Capacity of the reinforced soil could be found using the following equation developed from this research:

$$\mathbf{BCR\ (q_r/q_o) = 0.2981\ (H/D) + 1.1109}$$

5.3 Recommendations

Following are the recommendations for the further research in this area:

- The reinforced material used in this research was sand, one could see the effect of soil replacement using GM material to develop different relations for the estimation of ultimate bearing capacity
- Same analysis could also be done on the larger footing sizes to see the and correlate the results
- Sand of variant behavior having different strength parameters could be used to develop bearing capacity relations for each type of sand
- Clay layer used here was of 1 ft. It could be increased and effect of replacement on the depths larger than the footing diameter could be analyzed

ANNEXTURE A

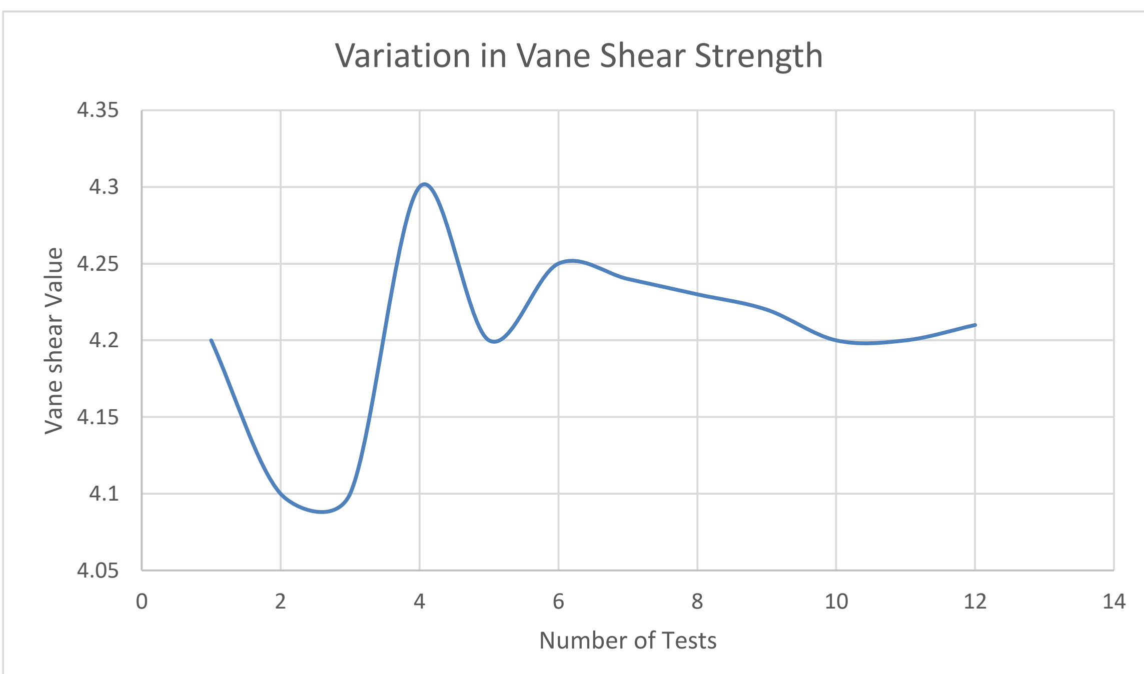
DIRECT SHEAR TEST ON SAND

300KPa		200KPa	
Shear Stress	H. Settlement	Shear Stress	H. Settlement
0	0	0	0
0.01	12.04352679	0.092	1.070535714
0.02	24.08705357	0.184	2.141071429
0.08	42.82142857	0.336	8.564285714
0.166	58.0765625	0.48	16.05803571
0.278	69.85245536	0.58	29.43973214
0.406	79.48727679	0.73	36.13058036
0.536	88.85446429	0.89	41.48325893
0.676	96.88348214	1.058	45.76540179
0.83	103.0390625	1.23	49.51227679
0.966	111.6033482	1.422	50.5828125
1.14	115.0825893	1.594	54.3296875
1.298	120.7029018	1.764	58.34419643
1.474	123.9145089	1.95	60.21763393
1.644	127.9290179	2.132	62.62633929
1.81	132.4787946	2.31	65.5703125
1.986	135.6904018	2.502	66.64084821
2.166	138.3667411	2.682	69.3171875
2.342	141.5783482	2.868	71.190625
2.522	144.2546875	3.058	72.52879464
2.906	146.3957589	3.446	74.13459821
3.082	149.6073661	3.63	76.27566964
3.27	151.2131696	3.818	77.88147321
3.452	153.621875	4.002	80.02254464
3.634	156.0305804	4.19	81.62834821
3.83	156.5658482	4.38	82.96651786
4.014	158.7069196	4.58	82.96651786
4.192	161.6508929	4.766	84.83995536
4.384	162.7214286	4.954	86.44575893
4.57	164.5948661	5.136	88.85446429
4.762	165.6654018	5.32	90.99553571
4.946	167.8064732	5.51	92.33370536
5.14	168.609375	5.698	93.93950893
5.328	170.2151786	5.886	95.5453125
5.516	171.8209821	6.07	97.68638393
5.705	173.2929688	6.252	100.0950893
5.892	175.0325893	6.432	102.7714286
6.082	176.3707589	6.61	105.7154018
6.272	177.7089286	6.794	107.8564732
6.456	179.85	6.975	110.3989955
6.642	181.7234375	7.158	112.6738839
6.828	183.596875	7.338	115.3502232
7.02	184.6674107	7.518	118.0265625
7.208	186.2732143	7.716	118.2941964
7.392	188.4142857	7.896	120.9705357
7.582	189.7524554	8.07	124.4497768
7.768	191.6258929	8.254	126.5908482

ANNEXTURE B

Vane Shear Test Variation

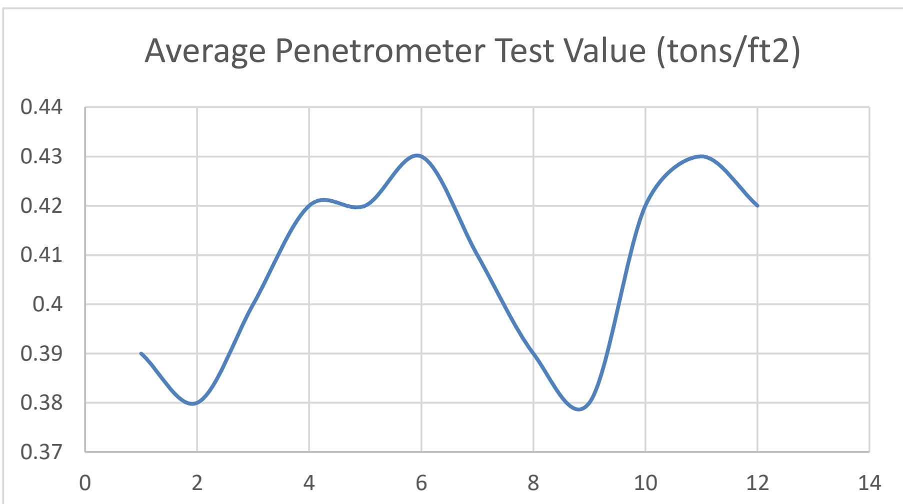
Footing Size	Tests	Average Vane Shear Test Value
4 Inch	1	4.2
	2	4.1
	3	4.1
	4	4.3
6 Inch	5	4.2
	6	4.25
	7	4.24
	8	4.23
8 Inch	9	4.22
	10	4.2
	11	4.2
	12	4.21



ANNEXTURE C

Penetrometer test variation

Footing Size	Tests	Average Penetrometer Test Value (tons/ft2)
4 Inch	1	0.39
	2	0.38
	3	0.4
	4	0.42
6 Inch	5	0.42
	6	0.43
	7	0.41
	8	0.39
8 Inch	9	0.38
	10	0.42
	11	0.43
	12	0.42



Annex D

Unconfined Compression Test

Deformatiom		Axial Strain	Strain %	Area of Sample cm^2	X-Sec Area cm^2	Proving Ring Dial Reading	Load Kgf	Stress kg/cm^2
DR	(mm)							
0	0.0	0.0000	*0.00*	*23.77*	23.77	0	0	***0.00***
50	0.50	0.0045	*0.45*	*23.77*	23.88	5	1.4305	***0.06***
100	1.00	0.0091	*0.91*	*23.77*	23.99	8.1	2.31741	***0.10***
150	1.50	0.0136	*1.36*	*23.77*	24.10	11.1	3.17571	***0.13***
200	2.00	0.0182	*1.82*	*23.77*	24.21	14.5	4.14845	***0.17***
250	2.50	0.0227	*2.27*	*23.77*	24.32	17.1	4.89231	***0.20***
300	3.00	0.0273	*2.73*	*23.77*	24.43	20	5.722	***0.23***
350	3.50	0.0318	*3.18*	*23.77*	24.55	23.5	6.72335	***0.27***
400	4.00	0.0364	*3.64*	*23.77*	24.66	25.2	7.20972	***0.29***
450	4.50	0.0409	*4.09*	*23.77*	24.78	27.5	7.86775	***0.32***
500	5.00	0.0455	*4.55*	*23.77*	24.90	29.5	8.43995	***0.34***
550	5.50	0.0500	*5.00*	*23.77*	25.02	31.5	9.01215	***0.36***
600	6.00	0.0545	*5.45*	*23.77*	25.14	33.5	9.58435	***0.38***
650	6.50	0.0591	*5.91*	*23.77*	25.26	34.5	9.87045	***0.39***
700	7.00	0.0636	*6.36*	*23.77*	25.38	35.9	10.27099	***0.40***
750	7.50	0.0682	*6.82*	*23.77*	25.51	36.5	10.44265	***0.41***
800	8.00	0.0727	*7.27*	*23.77*	25.63	37.5	10.72875	***0.42***
850	8.50	0.0773	*7.73*	*23.77*	25.76	38	10.8718	***0.42***
900	9.00	0.0818	*8.18*	*23.77*	25.89	38	10.8718	***0.42***
950	9.5	0.0950	*9.50*	*23.77*	26.26	38	10.8718	***0.41***
1000	10.0	0.1000	*10.00*	*23.77*	26.41	38	10.8718	***0.41***
1100	11.0	0.1100	*11.00*	*23.77*	26.71	37	10.5857	***0.40***
1200	12.0	0.1200	*12.00*	*23.77*	27.01	37	10.5857	***0.39***
1300	13	0.1300	13.00	*23.77*	27.32	36	10.2996	***0.38***
1400	14	0.1400	14.00	*23.77*	27.64	36	10.2996	***0.37***

References:

Meyerhof, G. G. (1974). Ultimate bearing capacity of footings on sand layer overlying clay. *Canadian Geotechnical Journal*, *11*(2), 223-229.

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