EXPERIMENTAL INVESTIGATION ON BEHAVIOUR OF ECCENTRICALLY LOADED MODEL FOOTING ON WEAK SOIL

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY

IN

GEOTECHNICAL ENGINEERING

Submitted by:

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CANDIDATE'S DECLARATION

I, SHEETAL NIRANJAN,2K19/GTE/13 student of M tech (GEOTECHNICAL ENGINEERING),hereby declare that the Major Project I entitled "EXPERIMENTAL INVESTIGATION ON BEHAVIOUR OF ECCENTRICALLY LOADED MODEL FOOTING ON WEAK SOIL" which is submitted by me to department of Civil Engineering ,Delhi Technological University, Delhi In Partial fulfillment of the requirement of the award of the degree of Master Of Technology , is original and not copied from any source without proper citation .This work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title or recognition.

Place: Delhi SHEETAL NIRANJAN

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CERTIFICATE

This is to certify that the thesis entitled "**Experimental Investigation on behavior of eccentrically loaded model footing on weak soil"** submitted by **Ms. Sheetal Niranjan** in partial fulfillment of the requirements for the award of Master of Technology in **CIVIL ENGINEERING** with specialization in **GEOTECHNICAL ENGINEERING** at the **DELHI TECHNOLOGICAL UNIVERSITY, DELHI** is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

Place:

Date:

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Abstract

Foundation in simple term is a part of structure through which load is finally transmitted to the soil. Settlement of foundation is an important criteria to understand the failure of foundation. In eccentrically loaded footing the settlement observed is different from concentric loaded footing hence it becomes important to observe the change in settlement due to variation in load eccentricity.

Weak soil is a problematic condition on field as settlement observed is large hence geogrid material layer is reinforced at a suitable depth to observed the change in settlement.

In the present study eccentric load is applied on a rectangular model footing at different values of eccentricity and by placing geogrid layer at different depth below the footing in weak soil. A uniformly increasing load is applied at a particular load eccentricity & for a geogrid layer and the settlement is noted for different load values.

As a result the settlement observed in case of reinforced case is less than unreinforced case and with increase in eccentricity upto kern boundary the nature of settlement observed is different from the case when the load is applied outside the kern boundary.

ACKNOWLEDGEMENT

I thank my parents and God almighty for bestowing their blessings and grace in completion of major project.

I express my sincere thanks to the respective V.C. of DTU and respective HOD of Civil Engineering Department and to the University Management. As I write this acknowledgement, I must clarify that this is not just a formal acknowledgement but also a sincere note of thanks and regards from my side. I feel a deep sense of gratitude and affection for those who associated with the project and without whose co-operation and guidance this project could not have been conducted properly.

Words fail me to express my regards towards my project guide, Prof. A.K. GUPTA, Department of Civil Engineering, Delhi Technological University, Delhi for giving me an opportunity to work under his guidance, which really instilled in me the requisite confidence.

I also express my deep gratitude to all the Faculty members of Department of Civil Engineering

Last but not the least, I would like to thank my family and friends who stimulated me to bring this work to a successful close.

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CHAPTER 1

INTRODUCTION

The foundation is the part of the structure that serves as the interface between the loads and the underlying soil or rock.

Shallow foundations are laid at depths equal to or less than the width of the structure, B. Shear criteria and settling criteria are two parameters that must be met while designing a shallow foundation.

Weak or soft soil makes engineering projects dangerous to construct. Ground improvement procedures, in which the engineer forces the ground to adopt the project's requirements by altering the natural condition of the soil rather than having to adjust the design in response to ground natural constraints, are commonly utilised these days to enhance such soil.

The majority of the research is focused on the scenario of a vertical load applied to the centre foundation. Eccentrically loaded footings, on the other hand, have a different bearing capacity than centrally loaded footings.

Meyerhof (1953) created empirical methodology for calculating the ultimate bearing capacity of foundations subjected to eccentric forces.

The general stability of the foundation declines as a result of load eccentricity, as does settlement and tilting of the foundation, lowering the bearing capacity.

Estimating settlements of shallow foundations in cohesionless soils is still considered a major geotechnical difficulty from both a practical and theoretical standpoint.

The current research looks on how to improve the bearing capacity of poor soils by inserting reinforcement materials at various depths.

Reinforcement is a safe and effective way to improve the strength and stability of soils.

Soil reinforcing technology has evolved as a potential topic in civil engineering, particularly for foundation engineers looking to improve certain soil qualities. Rubber shreds, high density polyethylene (HDPE) strips, polypropylene fibres, and jute fibres have been used as fill along soil in embankments and retaining walls to improve specific soil properties.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

An eccentric footing is a spread or wall footing that must sustain a moment in addition to the axial column load. The eccentric footing is designed so that the C.G. (centre of gravity) of the superimposed load and the C.G. (centre of gravity) of the base area are the same, resulting in uniform bearing pressure. The use of geosynthetic materials to boost the bearing capacity and settlement performance of shallow foundations has garnered a lot of attention in the field of geotechnical engineering. Several studies have shown that incorporating reinforcements into the ground can increase the ultimate bearing capacity and settlement characteristics of the foundation. Here is a quick survey of the literature on eccentrically laden foundations. Below is a summary of the experimental investigation and numerical simulation.

2.2 Concentric loaded footing

Terzaghi (1948) provided a well-thought-out theory for calculating the ultimate bearing capacity of a shallow, rough, stiff, continuous (strip) foundation supported by a homogeneous soil layer extending to a great depth and subjected to vertical loading.

Terzaghi suggested the following relationships in soil

$$q_u = cN_c + qN_q + 0.5B\gamma N_\gamma \quad \text{for strip foundation}$$

$$q_u = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma \quad \text{for square foundation}$$

$$q_u = 1.3\,cN_c + qN_q + 0.3B\gamma N_\gamma \quad \text{for circular foundation}$$

The failure area in the soil under the foundation can be divided into three major zones:

1. Zone AFB. This is a triangular elastomeric zone positioned just beneath the foundation's bottom. The angle between the wedge's AF and BF sides and the horizontal is α (soil friction angle).

2. Zone BFG. The Prandtl's radial shear zone is located here.

3. Zone BGD. The Rankine passive zone is located here. As seen in figure 2.1, the slip lines in this zone make an angle of (45º - Φ /2) with the horizontal.

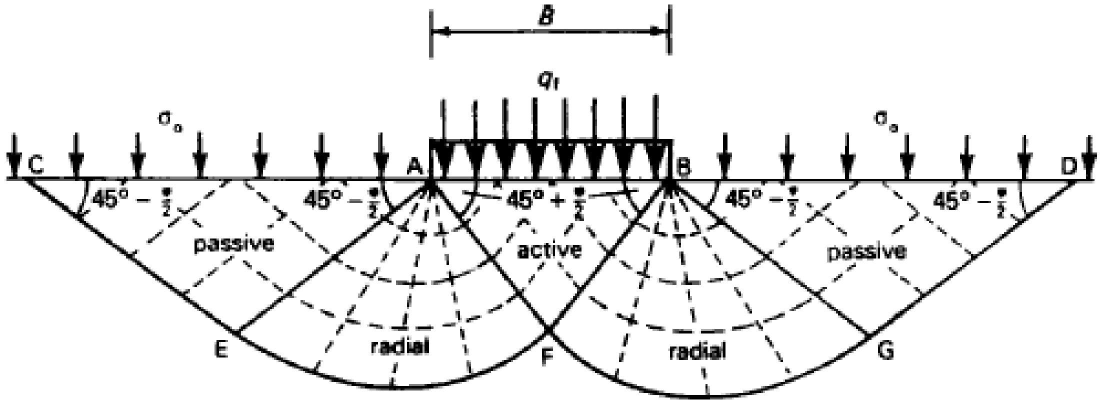

Figure 2.1: Failure surface in soil at ultimate load for a continuous rough rigid foundation (source: Terzaghi, 1948)

Meyerhof (1951) proposed a generalised equation for centrally vertically loaded foundations, which he referred to as

$$q_u = cN_c\, s_c\, d_c + qN_q\, s_q\, d_q + 0.5B\gamma N_\gamma$$

In the case of granular soil, the preceding equation (2.4) can be simplified to:

$$q_u = qN_q\, s_q\, d_q + 0.5B\gamma N_\gamma$$

Where q_u = ultimate bearing capacity;

$q = \gamma D_f$; D_f = depth of foundation;

γ= unit weight of soil;

B = width of foundation;

N_c, N_q ,N_γ = bearing capacity factors;

s_c, s_q, s_γ = shape factors;

d_c, d_q, d_γ = depth factors.

2.3 Eccentric vertical condition

Eccentric footings are non-concentrically weighted foundations. Furthermore, when footings are subjected to an axial load "P" plus a bending moment "M" or a lateral force "H," the stress distributions throughout the base will be uneven. Applying an axial load at an eccentricity "e" from the footing's centroid can also accomplish this.

Some of the most well-known studies in the literature dealing with numerical and experimental research on the behaviour of footings subjected to vertical and inclined loads on unreinforced and reinforced soil are briefly covered in the following paragraph. A few questions about the use of waste materials in different civil engineering projects are also briefly covered.

Meyerhof (1953) provided an empirical hypothesis that an eccentrically loaded footing may be viewed as a narrower centrally weighted footing. When a shallow foundation is subjected to an eccentric load, the contact pressure does not drop linearly from the toe to the heel, hence it is assumed that the contact pressure drops linearly from the toe to the heel. Meyerhof's recommended effective width B as

B’=B-2e

where e = load eccentricity

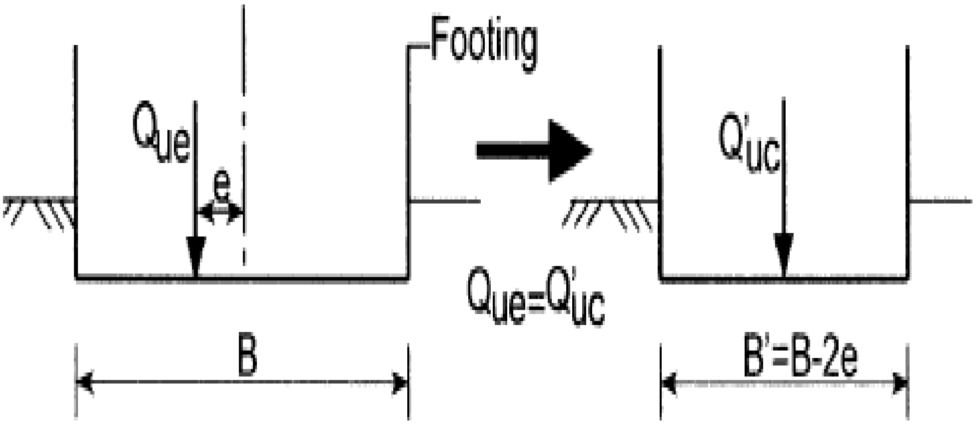

Figure 2.2: Eccentric loading on footing

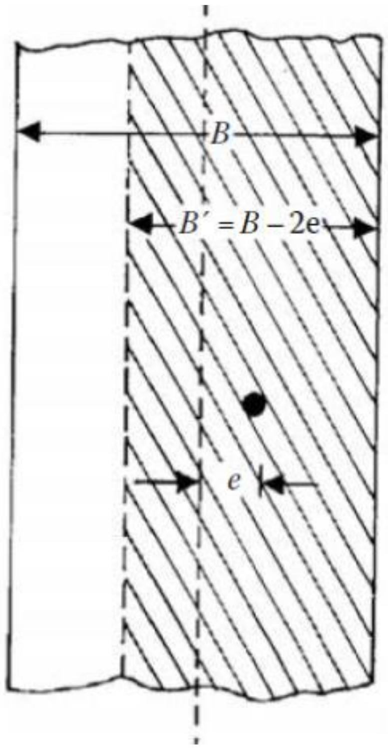

Figure 2.3: Plan view of eccentric loading on footing

This idea, which assumes that the load applies centrally along the effective contact width, can be used to determine the bearing capacity of a continuous foundation, as shown in Figure 2.2. As a result, given a continuous vertically laden foundation,

$C_1 = 1.0679 + B_1 + B_2$

where N_{cq}, $N_{\gamma q}$ = resultant bearing capacity factors for a central load and depend on ϕ and D/B' ; c = unit cohesion; γ = density of soil. For a continuous foundation, the shape factors are all one. The foundation's ultimate load per unit length Qu can be computed as

$Q_{ult} = q_u$ (A') (2.8)

Where A'= effective area =B'x1

He came to the conclusion that as the eccentricity of the footing increases, the average bearing capacity of the footing reduces in a parabolic fashion.

Prakash and Saran (1971): Prakash and Saran (1971) developed a detailed mathematical technique for estimating the ultimate bearing capacity of rough continuous surfaces foundations subjected to eccentric loads Figure depicts how this process works.On a continuous foundation, the predicted failure surface in a c– ϕ soil eccentric loading is a term used to describe a load that is applied in an unusual way. As illustrated in figure 2.3, the foundation's contact width with the soil is equal to Bx1.

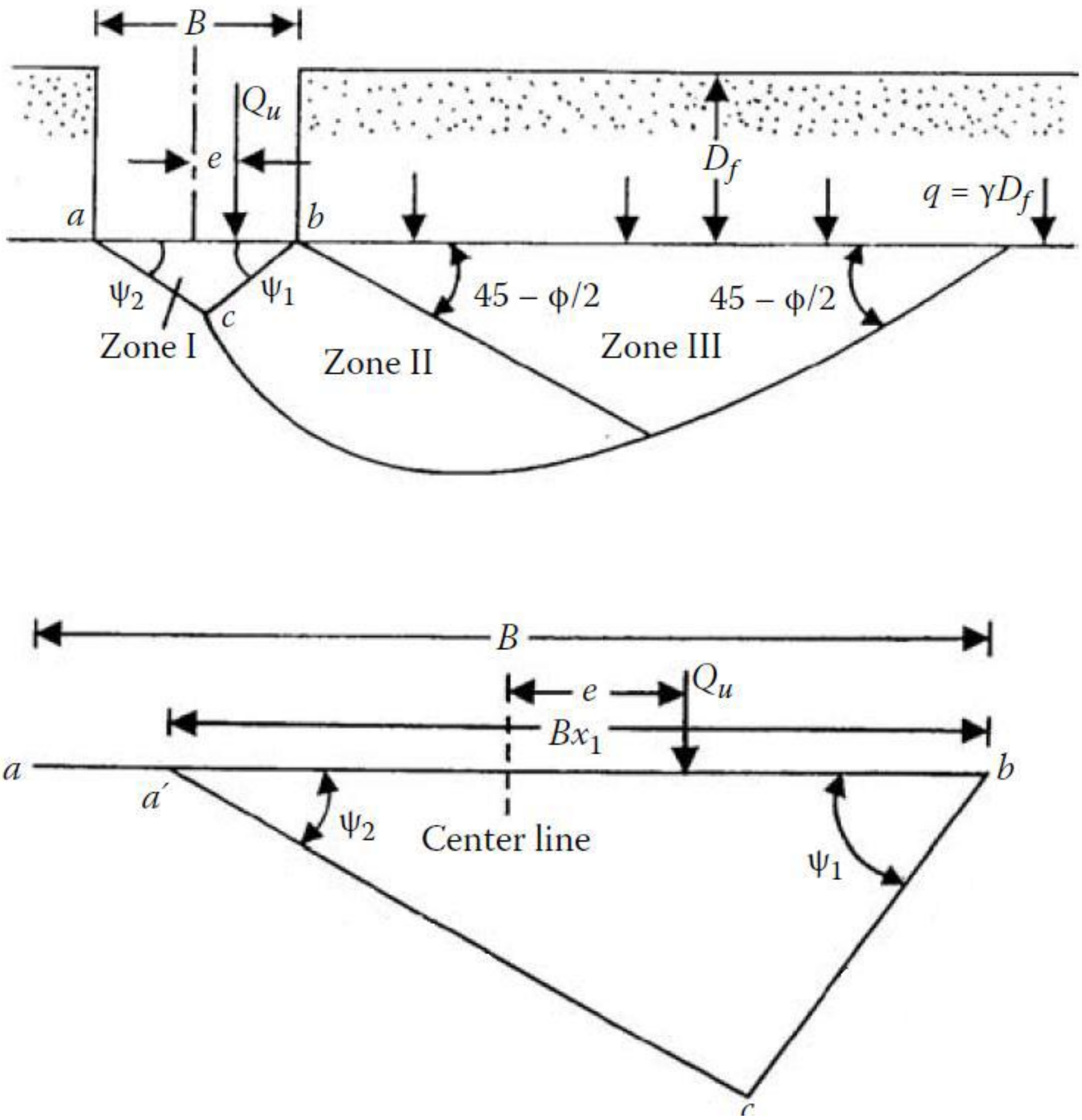

Figure 2.4: Eccentrically loaded rough continuous foundation (source: Das, B. M. 2009)

$q_u = Q_u /(B\times1) = 0.5\, \gamma\, B\, N_{\gamma(e)} + \gamma\, D_f\, N_{q(e)} + cN_{c(e)}$

Where $N_{\gamma (e)}$, $N_{q(e)}$, $N_{c(e)}$ = bearing capacity factors for an eccentrically loaded continuous foundation.

Ingra and Baecher (1983): The bearing capacity estimated by Ingra and Baecher (1983) using Terzaghi's superposition approach is partly theoretical and partly empirical. Many theoretical derivations, as well as experimental data from model testing and prototype footings, may be found in the literature. They assessed the effect of uncertainty in soil parameters on bearing capacity predictions deduced using statistical analyses of currently available experimental data.

The eccentricity factor, E_{γ}, is determined by the load offset in relation to the footing dimension (E_{γ} /B). According to a statistical research, friction angle and foundation size have little effect on E_{γ}. The carrying capacity of surface footings on sand has been investigated using an extension of Terzaghi's superposition approach. Data on footings with length-to-width ratios of 1 and 6 have been evaluated using statistical methods whenever possible. Theoretical 11 examination of foundations' ultimate bearing capability on cohesionless soil results in a wide range of options. Variations in bearing capacity factor N_{γ} show the most variation. When the friction angle is known, the relationship of N_{γ} to Φ seems to be the principal source of uncertainty in bearing capacity prediction.

Michalowski and You (1998): According to Michalowski and You (1998), The load supplied to the footing is assumed to be symmetric in traditional solutions to the bearing capacity problem. Eccentricity of the load is frequently accounted for in design by reducing the width of the footing, B, by twice the eccentricity, 2e, resulting in a B-2e effective width. Meyerhof proposed this concept, and it is now frequently used in geotechnical design. The effective width rule is the name given to this process. According to the literature, the method is conservative for cohesive soils and may overstate the carrying capacity for frictional soils. The purpose of this research is to find a limit analysis solution for eccentrically loaded strip footings, as well as to assess and comprehend the effective width rule in terms of plasticity analysis.

The bearing capacity problem of a footing subjected to eccentric loads will be solved using the kinematic approach of limit analysis. This backs with Salencon and Pecker's earlier findings. Only when the footing is 9 bound to the soil and the eccentricity is reasonably big (e/B > 0.25) can the effective width rule severely underestimate the bearing capacity for clays (Φ ≈0). With an increase in the density of cohesive-frictional soils, this underestimate reduces angle of internal friction. When the soil-footing interface is not bonded (tension cut-off interface), and when the eccentricity is minimal (e/B 0.1), the rule of effective width offers extremely reasonable estimations of the bearing capacity of eccentrically laden footings on cohesive or cohesive-frictional soils. In these situations, the effective width rule understates the optimal upper bound answer by no more than 8%. When the surcharge load is minor, however, it overestimates the bearing capability of merely frictional soils.

Mahiyar and Patel (2000): Mahiyar and Patel examined the finite-element analysis of an angle-shaped footing under eccentric pressure (2000). The soil is limited and lateral movement is inhibited by one side vertical projection of the footing, as shown in figure 2.4. The eccentricity width ratio (e_x/B) will determine the depth of footing projection. A mild steel square footing plate has been considered. It was given an angle shape by attaching it to a footing projection, which is a mild steel plate. Both plates are perpendicular to one other. The point load was applied at a variety of eccentricities (B = square footing width = 100 mm). The e_x /B values were adjusted for a specified depth of footing projection (D). Internal friction and cohesion c angles have been set to 27 ˚ and 1 kN/m^2 for internal friction and cohesion, respectively. Sand has a Young's modulus of elasticity E of 22,500 kN/m^2, while steel has a Young's modulus of elasticity E of 2.0×108 kN/m^2.

The eccentricity width ratio was changed from zero to 0.25 (up to 0.30 for 1.50B and 2.00B footing projection depths), and the depth of the footing projections was changed from 0.25B to 2.00B. The tilt can be minimized to almost zero for a given value of e_x /B by providing a vertical footing projection of sufficient depth at the edge closer to the load. For cohesion-less soil, this is independent of the footing material and the angle of internal friction. However, when Φ is greater, the ultimate bearing capacity will be greater. Under the same particular load intensity, the prototype footing tilts less than the model footing

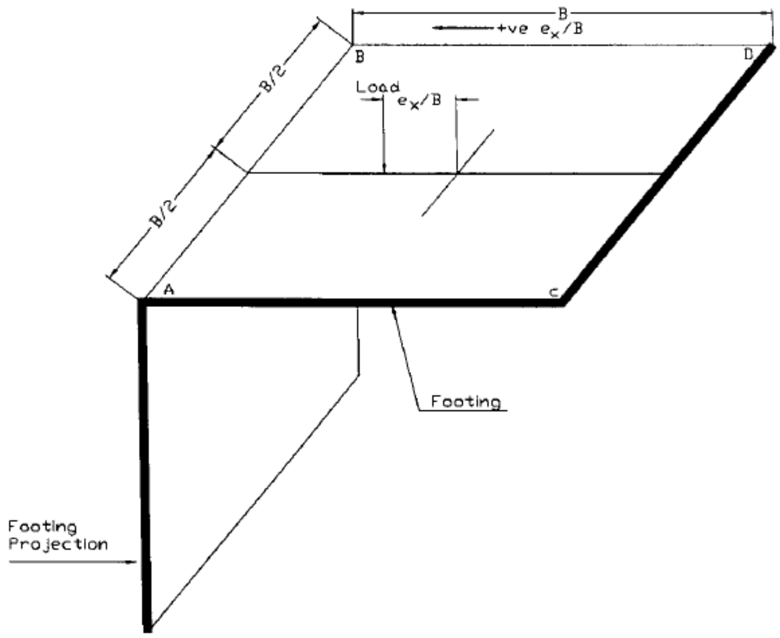

Figure 2.5: Sign Conventions for Load Position, e_x/B (source: Mahiyar and Patel, 2000)

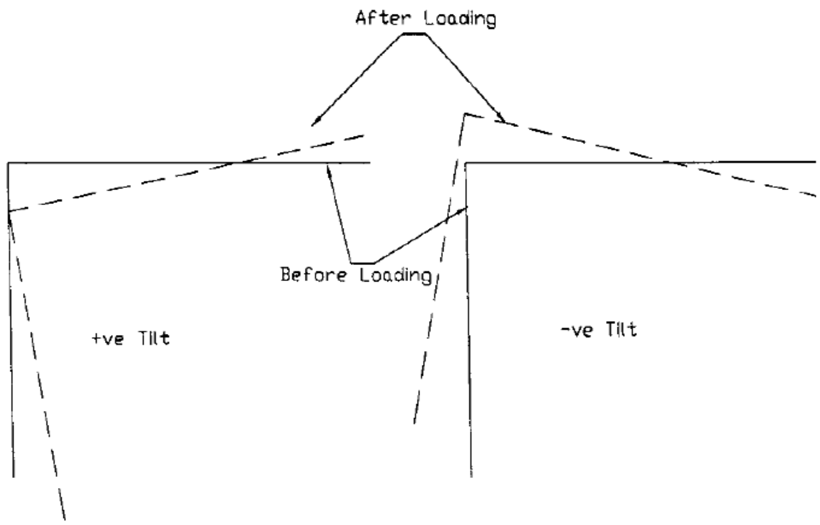

Figure 2.6: Sign Conventions for Tilt Footing(source: Mahiyar and Patel, 2000)

Emel Turker, Erol Sadoglu, Evrim Cure, and Bayram Ali Uzuner (2014) - A tank, model footing, sand, and a loading mechanism comprise the experimental set-up utilised to conduct the testing. At a depth of half the width of the footing, a single woven geotextile strip sheet was put horizontally below the footing's base. To investigate ultimate loads, failure surfaces, load–displacement curves, footing rotation, and so on, a series of bearing capacity tests were conducted with an eccentrically (e/B = 0, 1/12, 1/6, 1/3) loaded model surface (Df /B = 0) and shallow (Df /B = 0.25) strip footings (B = 80 mm) resting close to reinforced finite sand slopes. With increased eccentricity, the ultimate loads reduced. Due to a combination of eccentricity and slope, this decline has occurred. In comparison to unreinforced situations, the introduction of geotextile reinforcement enhanced ultimate loads.

2.4 Geogrid as reinforcement

Arghadeep Biswas, Md. Asfaque Ansari, Sujit Kumar Dash & A. Murali Krishna- Depending on layer thickness and subgrade strength, planar geogrid reinforcement installed at the sand-clay interface can significantly increase the performance of foundation beds. The study found a maximum of 5.6-fold improvement in bearing capacity for very soft clay subgrades of 7 kPa.

Manash Chakraborty, Jyant Kumar - The increase in bearing capacity associated with the use of reinforcement grows in lockstep with the increase in ϕ. When comparing two layers of reinforcement to a single layer of reinforcement, the improvement in bearing capacity is rather significant.

CHAPTER 3

PROJECT WORK

3.1 Objective

- To study the effect of eccentric load applied on shallow foundation.
- To study the settlement of weak soil in case of unreinforced soil and then by placing a reinforcement material at a suitable depth.
- To determine the suitable depth of placing the reinforcement.

3.2 Experimental Setup

- Weak soil
- Reinforcement material- geogrid layer
- Experimental model test tank
- Model footing
- Dial gauge

Weak soil

a. **Clays/Silts/Peats**: These soil deposits are commonly found around river mouths, along bay perimeters, and beneath marshes or lagoons. High-organic-content soil deposits are common in these low-lying areas, and they can be particularly bothersome. The low-lying land features where these difficult soils are often located are prone to flooding. As a result, before structures or roads can be built on such soil deposits, the grade level must be raised by compacting fill. Adding large amounts of compacted fill, on the other hand, places considerable loads on the soil, which can result in major settlements. For example, the New Jersey Meadowlands complex was created on marshlands along the Hackensack River in central New Jersey in the 1980s. The New Jersey Meadowlands complex, for example, was built in the 1980s on Hackensack River marshlands in central New Jersey, just a few miles west of downtown Manhattan (NYC). The following settlements were found in the soft soil as a result of fill placement:

- 0.25m during placement of the fill;
- 0.12m during the construction phase; and
- 0.10m over the ten following years.

b. **Loose Saturated Sands**: During severe ground motion, loose saturated sand deposits in seismically active areas are prone to liquefaction and settlement. The 1964 Niigata Earthquake in Japan was a perfect example. During the earthquake, many buildings built on loose saturated sand layers settled more than 1m, while others (particularly an apartment complex) tipped over on their sides. (Apartment buildings are hydrodynamically unstable constructions that will capsize if the soil liquefies.)

Geogrid: Geogrids are plastics with huge pores that have been moulded into an extremely open, gridlike shape. Geogrids are either stretched in one or two directions for increased physical qualities, or they are woven using innovative processes on weaving machinery. Typically used to strengthen weak soil and garbage heaps.

"They can be made by weaving or knitting yarns, heat-welding strips of material, or punching a consistent pattern of holes in sheets of material and stretching them into a grid." Geogrid is typically made from polymer materials such as polypropylene, polyethylene, or polyester. The development of tensile drawing methods for making reasonably rigid polymeric materials introduced the prospect of using such materials to reinforce soils for walls, steep slopes, roadway bases, and foundation soils" (Capaccio and Ward, 1974).

Figure 3.1: Geogrid material

Model Test Tank

All experiments are conducted at DTU's Geotechnical Engineering Laboratory. In a mild steel tank, the model testing are carried out. The top, bottom, and middle of the model tank are strengthened by steel angle sections to prevent lateral yielding during soil compaction in the tank and when applying force at the model footing during the experiment. The tank's two long sides are built of high-strength fibreglass. The tank is reinforced on all four sides to prevent bulging during testing. To lessen side friction, the inside walls of the tank were smooth.

Figure 3.2: Test tank

Model Footing

In the experiment, a model footing with the dimensions of length (L), width (B), and thickness (D) was used. The footing dimensions were chosen based on the size of the model tank. For a similar experimental set-up, Kumar and Bhoi (2008) found that the ratio of tank length to footing width has no effect on load– settling behaviour when the ratio is more than 12. As a result, the dimensions in this study are based on this finding, and the ratio is more than 12. The width of the test pit should not be less than 5 times the width of the test plate, according to IS 1888-1962, so that the failure zones can grow freely without interference from the sides. The bottom surface of the model footing was roughened by cementing a layer of sand with epoxy glue to promote friction between the footing foundation and the top soil layer. A robust steel plate was used at the top of the model footing to reduce bending while applying the weight. On the footing, load is imparted eccentrically.

Figure 3.3: Model footing

Laboratory Model test

Weak soil is poured into the test tank in layers. The model foundation was placed in the centre of the tank at a desirable D_f /B ratio. A loading assembly was used to manually apply load to the model foundation. Dial gauges installed on the model foundation are used to measure the settlement of the foundation.

3.3 Methodology

- The study involves preparing a model footing of isolated rectangular footing. An experimental model test tank is filled with weak soil and model footing is placed on it.
- Uniformly increasing load is applied to the model footing by using a weight block. The corresponding foundation settlement can be measured by using a dial gauge.
- Initially the test will be conducted on unreinforced soil and load vs settlement curves will be drawn. Reinforcement will be applied at predetermined depths below the base of the model footing in the second situation.
- Again the load is applied and settlement is observed and load vs settlement curve is drawn.

- The reinforcement used in this study is geogrid layer and the results are compared with unreinforced case.
- The depth of reinforcement is increased below the footing and settlement is observed while applying load.

3.4 Specifications

Soil Profile

Soil profile type can be established using shear wave velocity or the Standard Penetration Test (SPT) result and soil classification using grain size distribution data.
The limitations of a standard penetration test are numerous. Apart from the tests, the N value must be corrected by correlating it with the soil parameters.
One of the most significant drawbacks of using N values is determining an adequate N value for layered soil, particularly when coarse and fine-grained soils are interlayered. Furthermore, the areaal extent of soil profiles might vary greatly. Then deciding on the N value to employ in determining the soil profile type becomes incredibly challenging.
Because of the method's limitations, the shear wave velocity should be used as a supplement to the normal penetration test.
Despite the fact that the shear velocity approach is more reliable in defining the site, replacing the SPT method with shear wave velocity is challenging in India due to the cost of the equipment and trained staff required for its application. Despite its limitations and the numerous modifications that must be performed to the recorded N value, the standard penetration test is the most extensively utilised field test in India and around the world.
The criteria described below is as per Indian Seismic code, IS 1893 (Part 1): 2002
Soft Soils: All soft soils other than SP with $N<10$. The various possible soils are
1. Silts of Intermediate compressibility (MI);
2. Silts of High compressibility (MH);
3. Clays of Intermediate compressibility (CI);
4. Clays of High compressibility (CH);
5. Silts and Clays of Intermediate to High compressibility (MI-MH or CI-CH);
6. Silt with Clay of Intermediate compressibility (MI-CI);
7. Silt with Clay of High compressibility (MH-CH)

Geogrids: In the current experiment, a biaxial geogrid was used. The soil is strengthened by biaxial geogrid, which has tensile strength in two mutually perpendicular directions.

Model test tank: Dimensions of model test tank are as follows

Length- 100 cm

Width- 60 cm

Height- 50 cm

Model footing: Isolated rectangular footing dimensions are as follows

Length- 50 cm

Width- 7.5 cm

Height- 4 cm

Dial gauge: A single dial gauge are attached to the loading plate and least count of dial gauge is 0.01 mm.

Table1: Properties of geogrid material

PARAMETERS	VALUE
Type	BFX30
Polymer	Polypropylene Pp
Aperture size (W)	(43*43) mm^2
Aperture shape	Square
Rib width (w)	3 mm
Tensile strength (ASTM D 6637)	30 kN/m

Figure 3.4: Testing tank with loading plate

Figure 3.5: Eccentric loading on footing

Table 2: Properties of soil

C.B.R Soil Analysis		
Description		Brownish Clayey silt with high plasticity
IS classification		CH
Grain size analysis	Gravel (%)	0
	Sand (%)	7
	Silt (%)	61
	Clay (%)	32
Atterberg Limits	Liquid limit (%)	68.24
	Plastic limit (%)	28.85
	Plasticity index (%)	39.4
	Shrinkage limit (%)	12.42
Free swelling index (%)		36.4
Specific Gravity		2.74
Heavy compaction	OMC (%)	12.90
	MDD(gm/cc)	2.17
C.B.R	Soaked	4.70
	Un-soaked	8.02

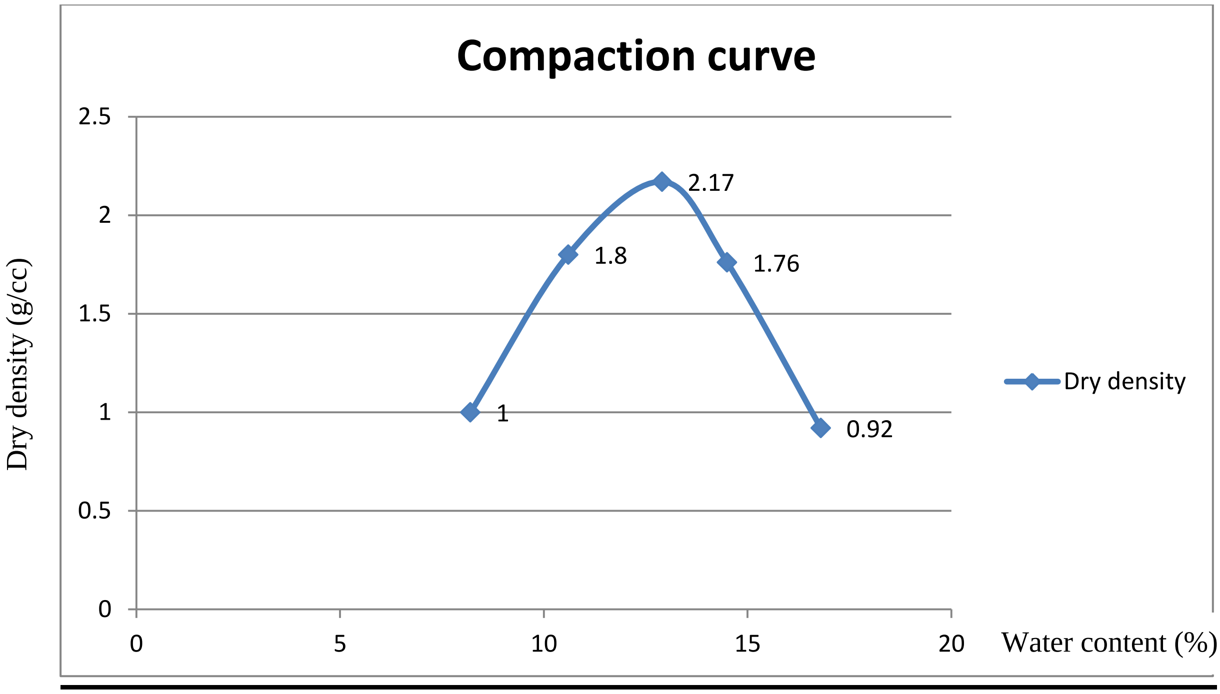

Figure 3.6: Compaction curve

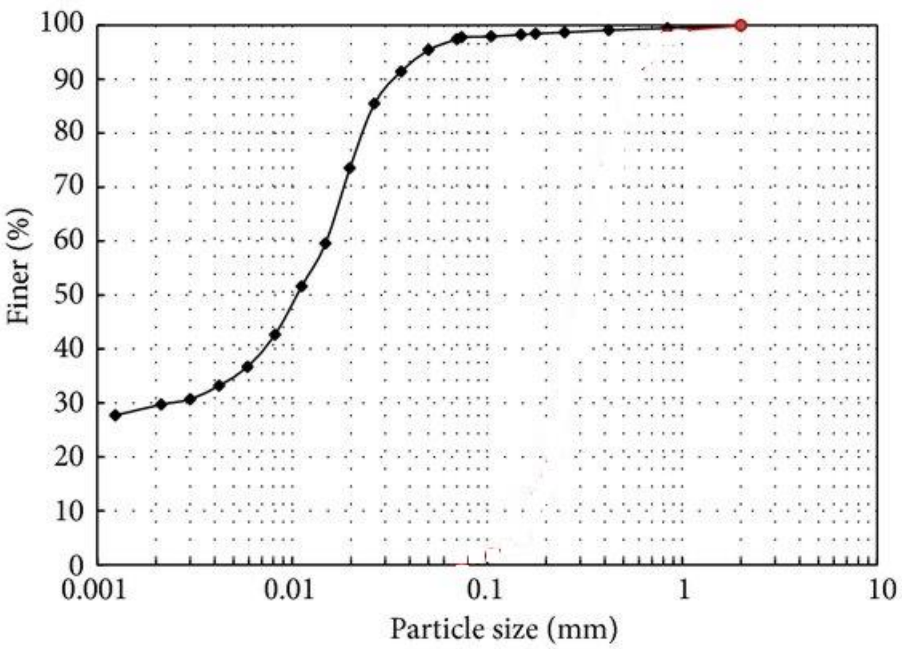

Figure 3.7: Particle size distribution curve

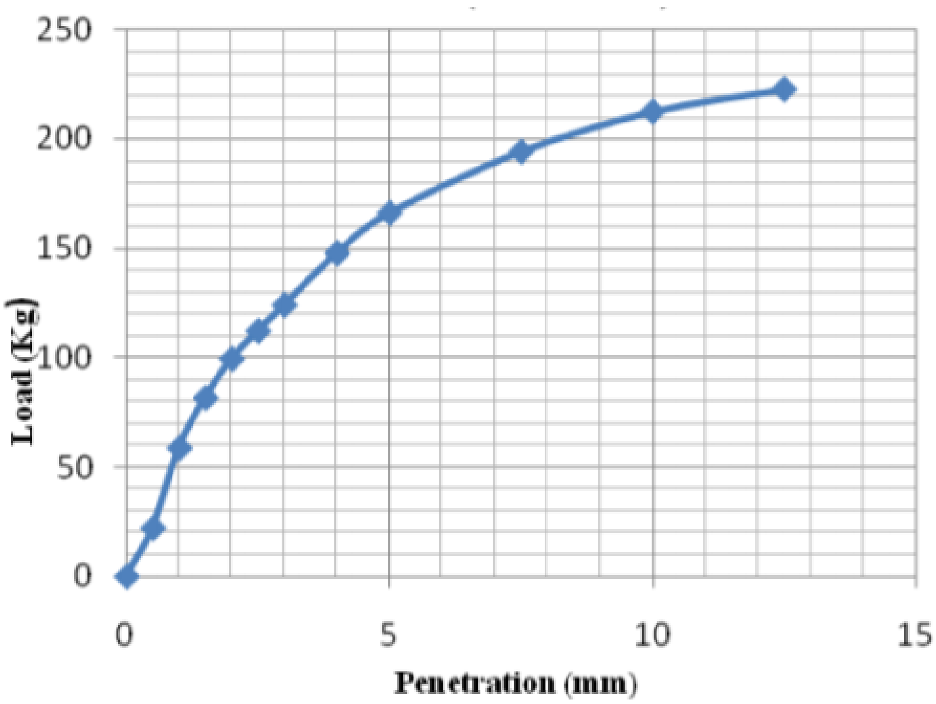

Figure 3.8: CBR Curve

3.5 Experimental Procedure

Preparation of model test tank: The tests are conducted in glass test tank of dimensions 100cm×60cm×50cm in length, width and height respectively. In all tests, the dry unit weight is kept same i.e., 1 Mg/m^3. Weak soil is filled in the tank in layers of 50 mm upto 40 cm of height. The sand quantity of 30Kg is deposited in the tank loosely and then lightly compacted with with a wooden hammer in the tank upto 50mm thickness. This process continued until the sand mass height reached 40 cm.

Placing of footing: Isolated rectangular footing is placed exactly on the middle of the tank and then soil is filled over it so that the depth of footing is 5cm and the overall depth becomes 45cm.

Hence according to Terzaghi for shallow foundation $D_f/B \leq 1$ is applicable here in this experiment.

Determination of eccentricity: In case of eccentric load, the footing is subjected to a point load P as well as a moment M, due to which the pressure develop is not uniform. The moment will be about the centroidal axis of footing area and can be expressed as M= P×e

Where e denotes the load's eccentricity with respect to the area's centroidal axis.

Large eccentricities create tension and cause a section of the footing to lift off the soil, hence load should be placed within the kern distance of a rectangular footing to maintain compression.

Hence, $e \leq (L/6)$ or $e \leq (B/6)$

The shape of kern in rectangular footing is rhombus.

In the present situation to keep the loading with in kern –

along length $e \leq 8.33$cm and along width $e \leq 1.25$cm.

For the experimental determination we take different values of eccentricity with in the kern and outside the kern to study the effect.

The load eccentricities values taken along length about the centroidal axis are 0cm, 5cm, 8.33cm, 10cm.

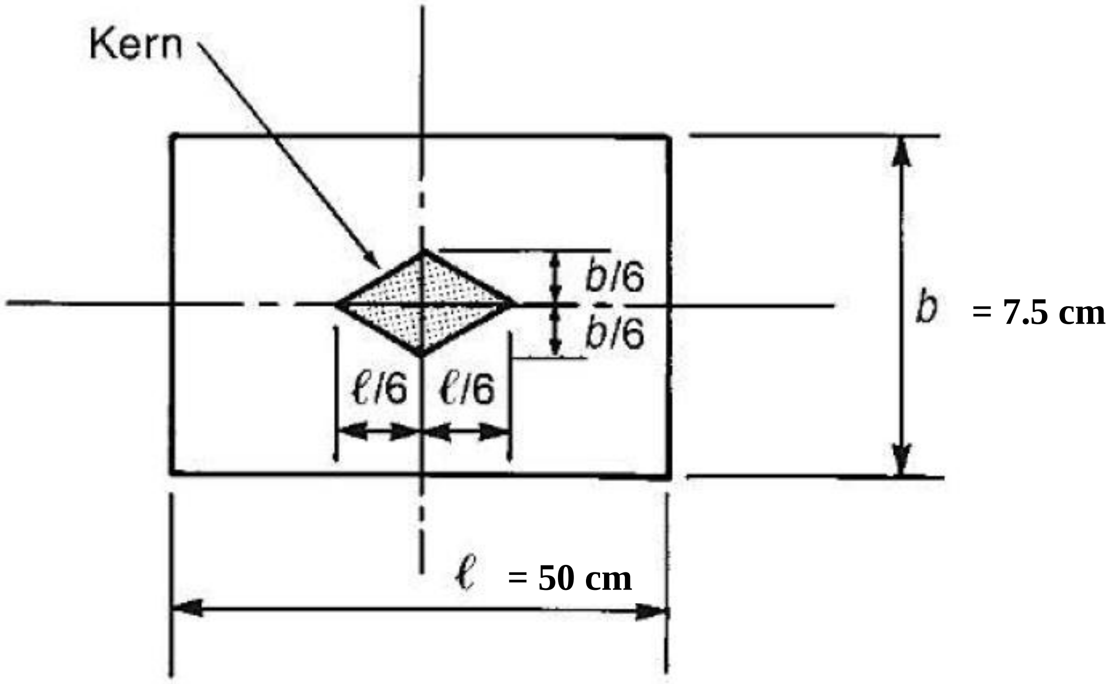

Figure 3.9: Kern area in rectangular footing

Distance of reinforcement below the footing: The first reinforcement is positioned ‘u' below the footing, whereas the second reinforcement is put ‘h' below the first reinforcement. To determine the optimum distance three test are carried out for u = 2cm, 4cm, 7cm and for h = 2cm, 4cm, 7cm. The dimensions of the reinforcement layer are same as the dimensions of the tank.

Application of load: A constant load of 49.05 N is applied and the corresponding settlement is recorded. The load is then increased and a sequence of load is applied as 98.1 N, 147.15 N, 196.2 N , 245.25 N and the corresponding settlement is recorded for every applied load. Each load is applied for unreinforced soil and then for soil reinforced with geogrid. Each load is applied for different values of eccentricities taken above and the settlement is observed.

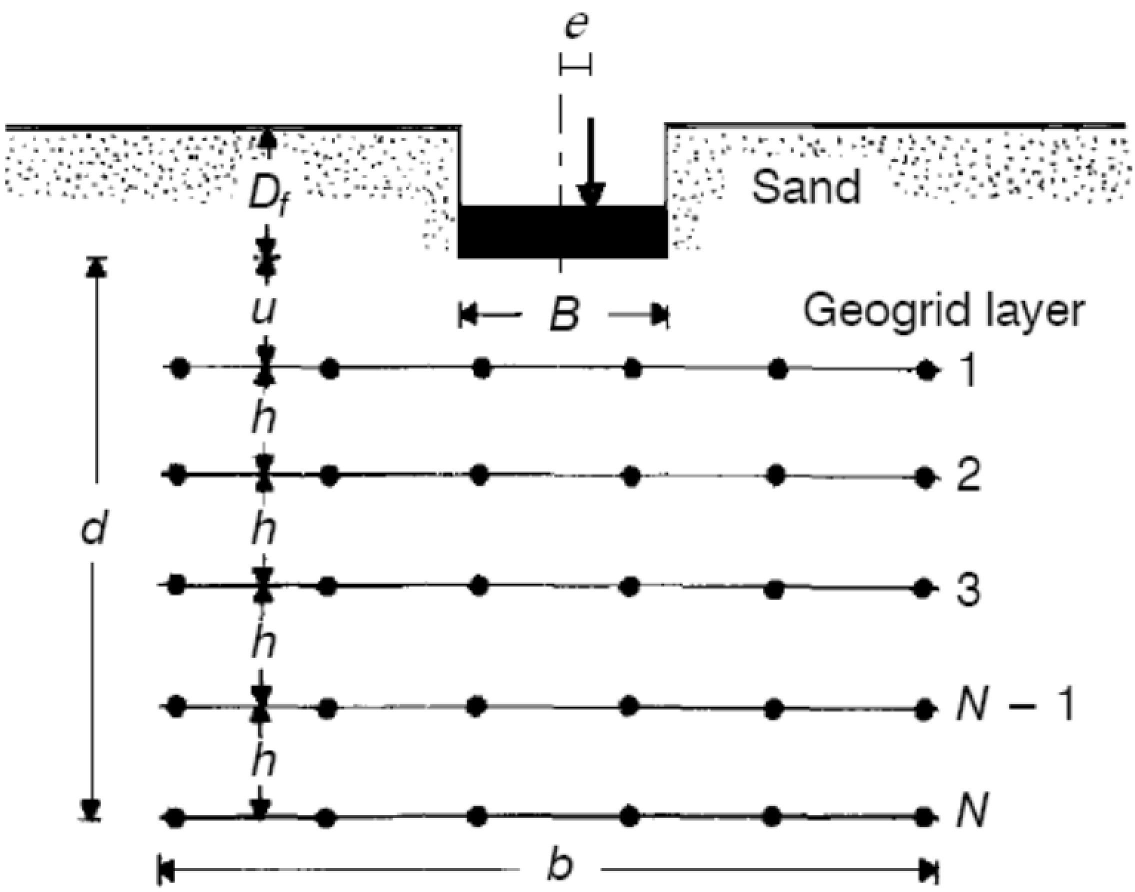

Figure 3.10: Schematic diagram of soil loaded with eccentric loading and with different layers of geogrid reinforcement

CHAPTER 4

OBSERVATIONS

4.1 Unreinforced soil case

In this case no reinforcement is placed below the footing and the loading is applied at different load eccentricities as 0cm, 5cm, 8.33cm, 10cm. The loading is increased in the same manner as discussed above and the corresponding settlement is recorded.

Table 3: Load – Settlement data for unreinforced, e=0cm

Load (N)	Eccentricity (cm)	Settlement (mm)
49.05	0	0.76
98.1		1.53
147.15		2.34
196.2		3.13
245.25		3.95

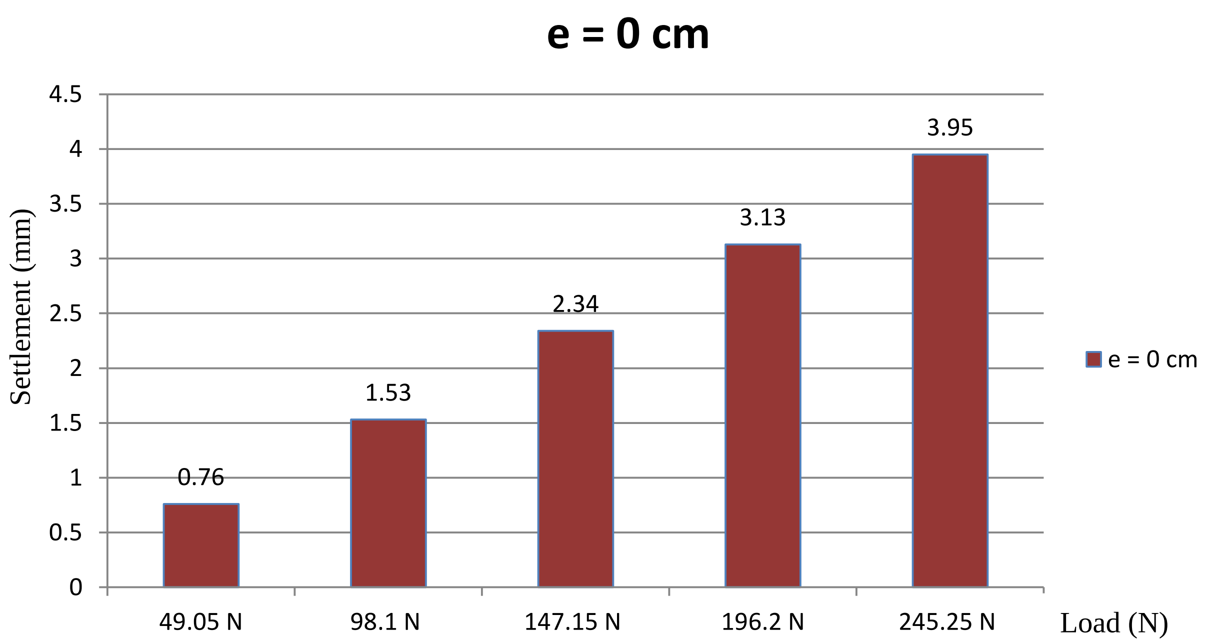

Figure 4.1: Load – Settlement variation for unreinforced, e=0cm

Table 4: Load – Settlement data for unreinforced, e=5cm

Load (N)	Eccentricity (cm)	Settlement (mm)
49.05	5	0.86
98.1		1.70
147.15		2.54
196.2		3.41
245.25		4.21

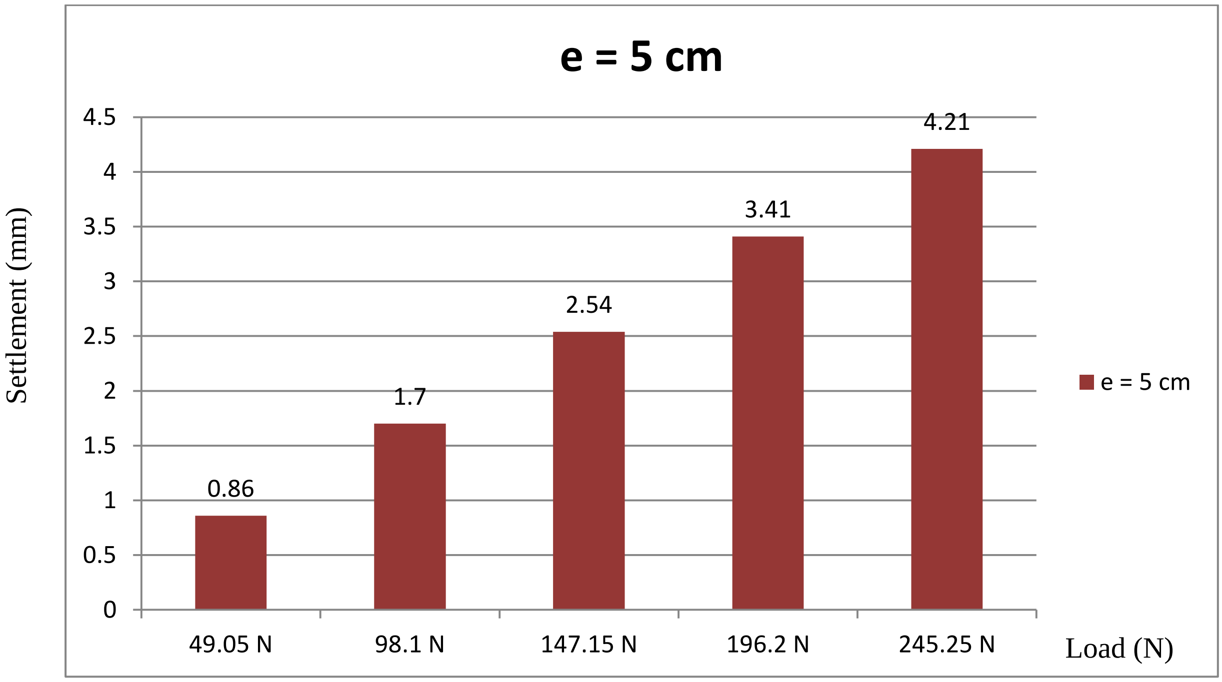

Figure 4.2: Load – Settlement variation for unreinforced, e=5cm

Table 5: Load – Settlement data for unreinforced, e=8.33cm

Load (N)	Eccentricity (cm)	Settlement (mm)
49.05	8.33	0.74
98.1		1.45
147.15		2.21
196.2		2.84
245.25		3.65

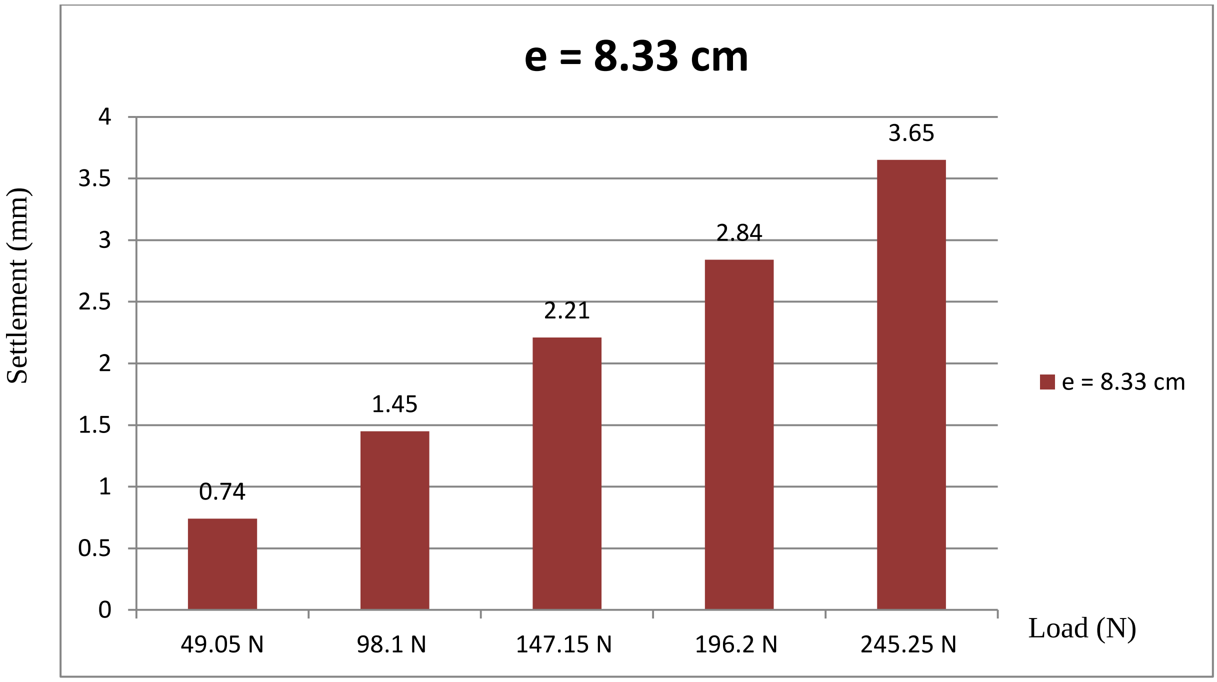

Figure 4.3: Load – Settlement variation for unreinforced, e=8.33cm

Table 6: Load – Settlement data for unreinforced, e=10cm

Load (N)	Eccentricity (cm)	Settlement (mm)
49.05	10	0.68
98.1		1.34
147.15		2.06
196.2		2.68
245.25		3.55

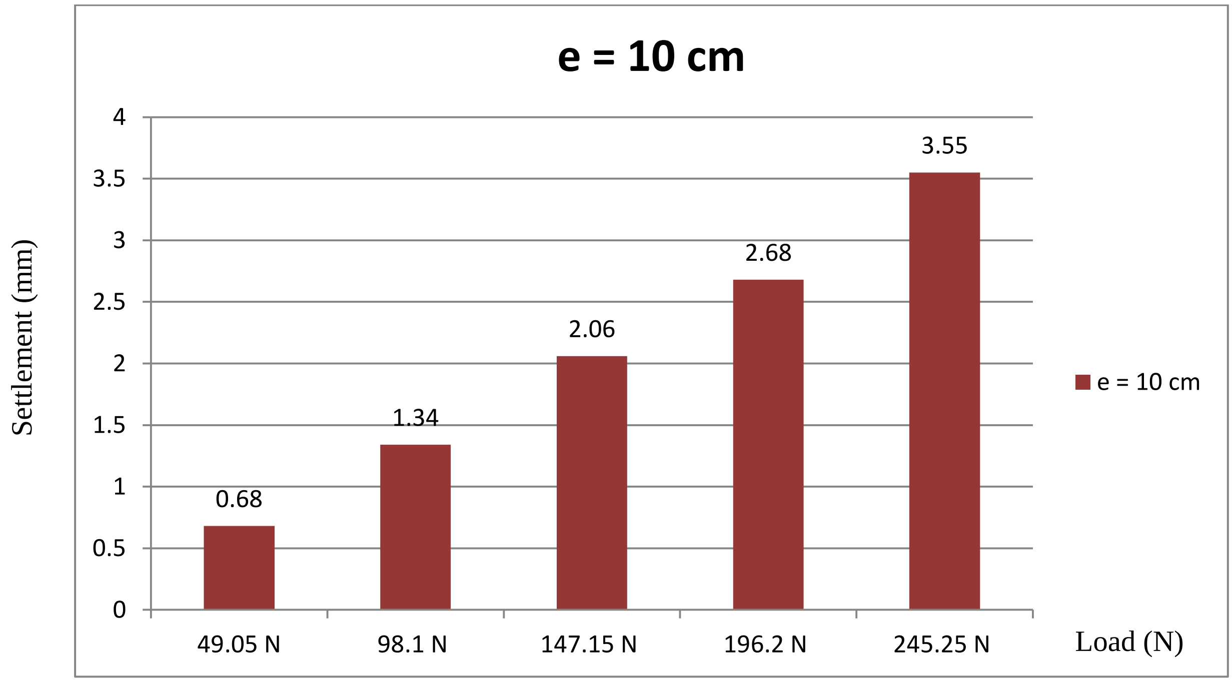

Figure 4.4: Load – Settlement variation for unreinforced, e=10cm

4.2 Reinforced soil case

4.2.1 First reinforcement layer at u = 2cm

At a depth of 2 cm, a single layer of reinforcement is applied, below the base of footing and load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

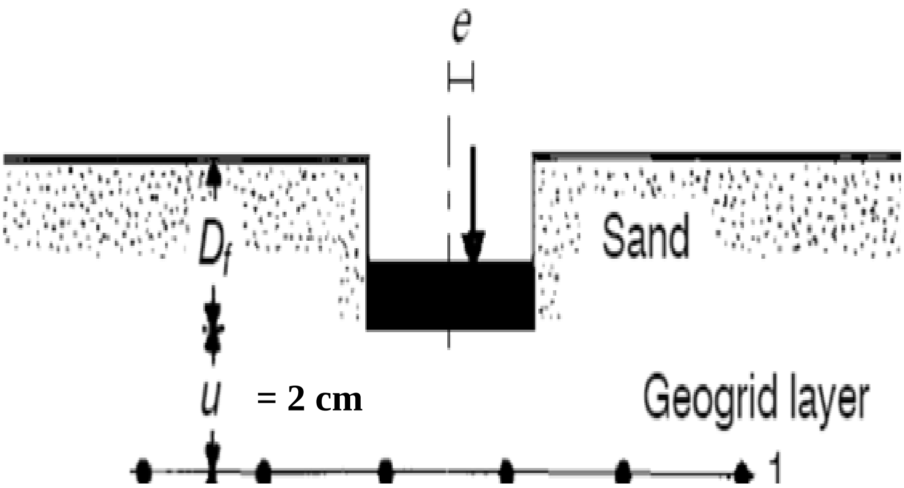

Figure 4.5: Schematic diagram of soil loaded with eccentric loading and with a single layers of geogrid reinforcement at u = 2 cm

Table 7: Load – Settlement data for single, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	0	2	0.52
98.1			1.01
147.15			1.56
196.2			2.06
245.25			2.56

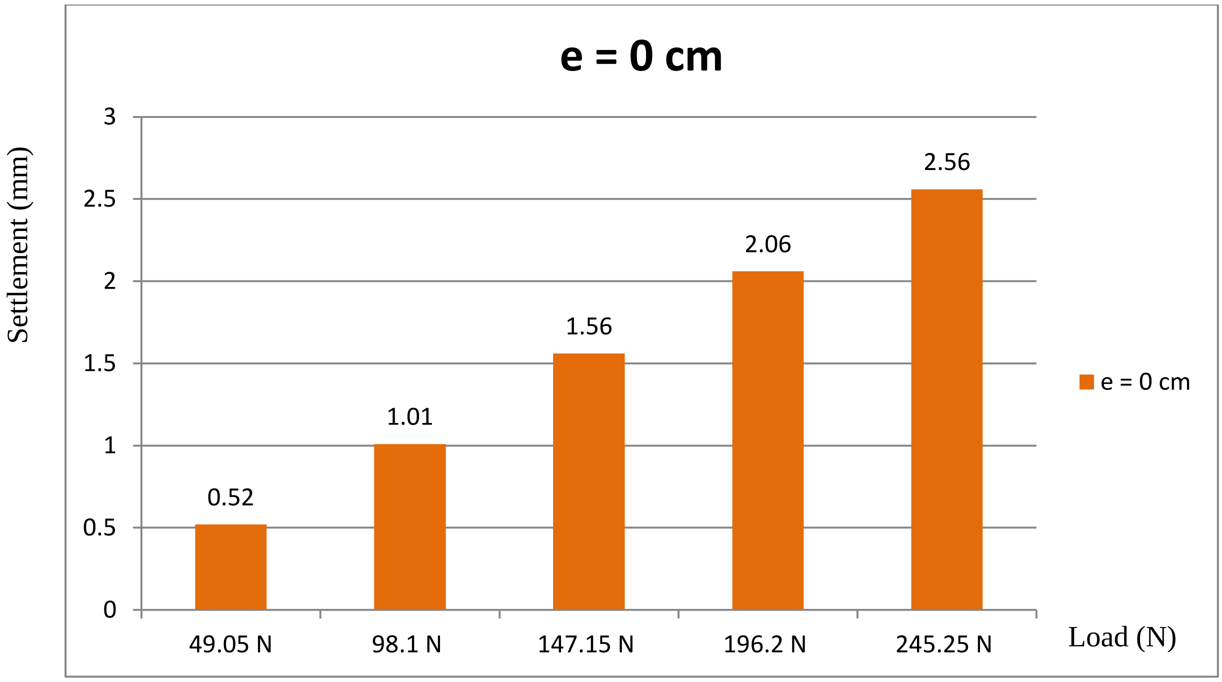

Figure 4.6: Load – Settlement variation for single, e=0cm

Table 8: Load – Settlement data for single, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	5	2	0.43
98.1			0.84
147.15			1.23
196.2			1.71
245.25			2.10

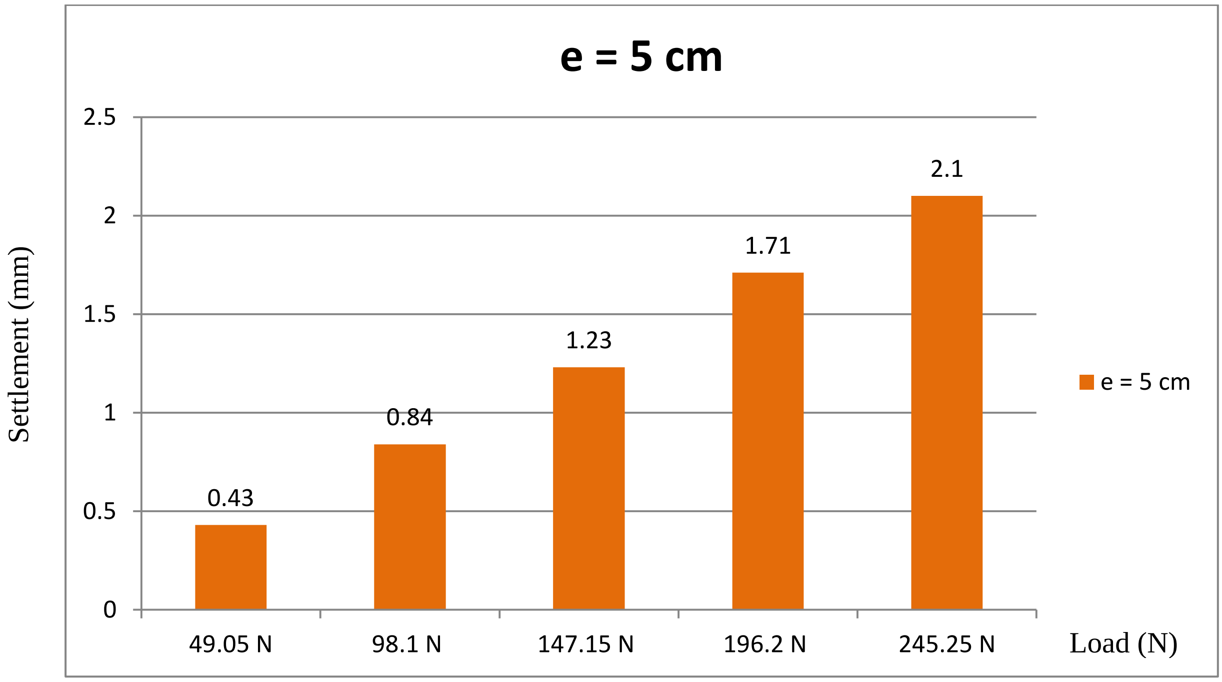

Figure 4.7: Load – Settlement variation for single, e=5cm

Table 9: Load – Settlement data for single, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	8.33	2	0.52
98.1			1.01
147.15			1.52
196.2			2.06
245.25			2.64

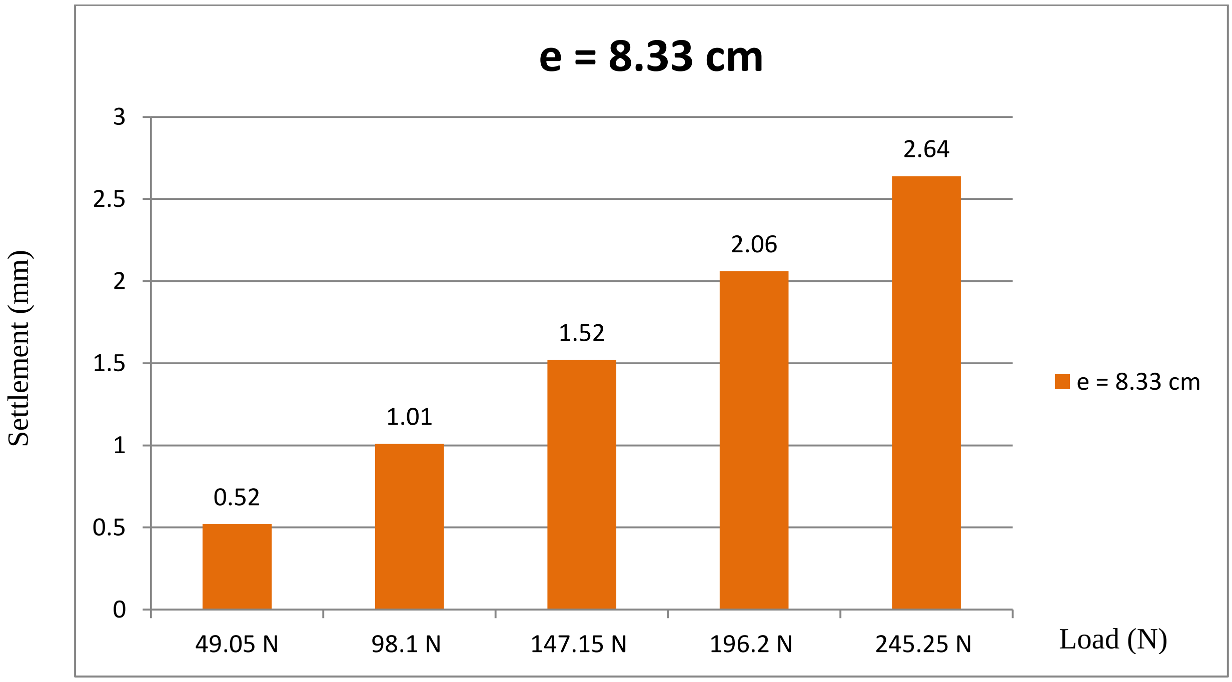

Figure 4.8: Load – Settlement variation for single, e=8.33cm

Table 10: Load – Settlement data for single, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	10	2	0.69
98.1			1.36
147.15			2.05
196.2			2.66
245.25			3.48

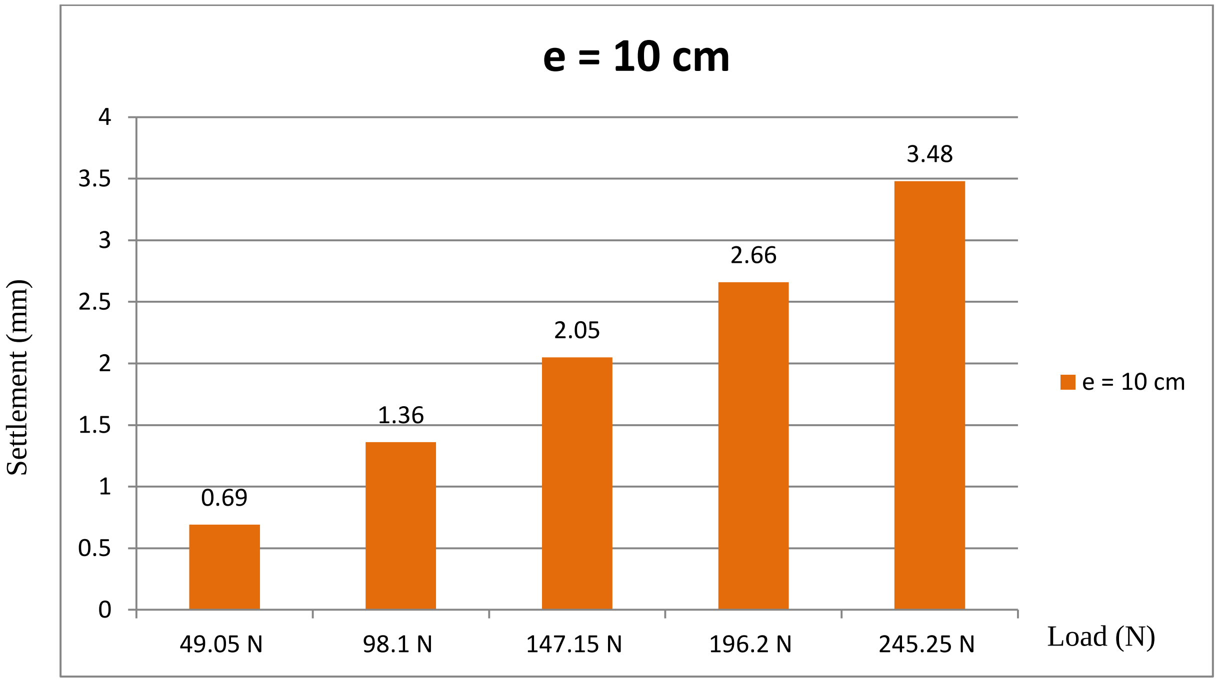

Figure 4.9: Load – Settlement variation for single, e=10cm

4.2.2 First reinforcement layer at u = 4 cm

At a depth of 4 cm, a single layer of reinforcement is applied, below the base of footing and load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

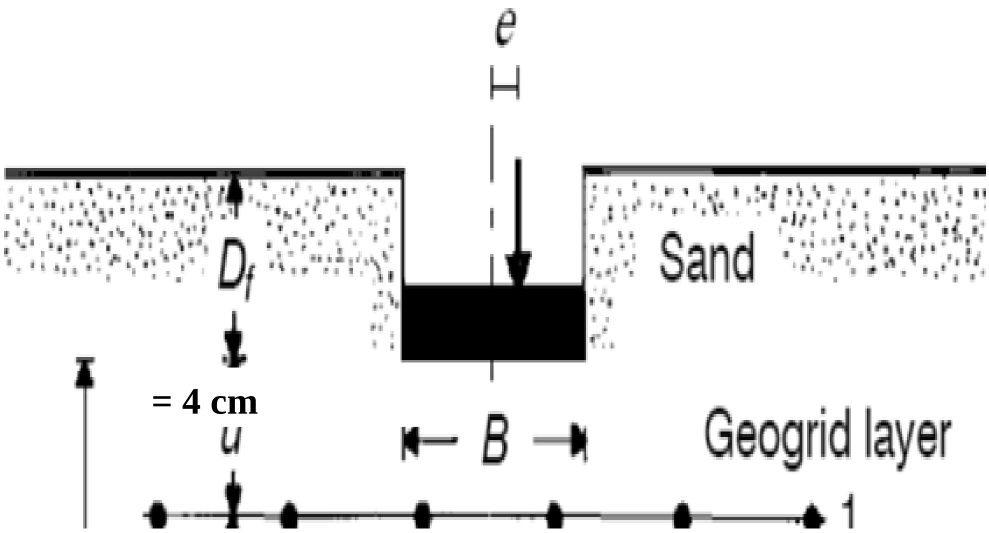

Figure 4.10: Schematic diagram of soil loaded with eccentric loading and with a single layers of geogrid reinforcement at u = 4 cm

Table 11: Load – Settlement data for single layer, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	0	4	0.51
98.1			1.04
147.15			1.50
196.2			2.08
245.25			2.56

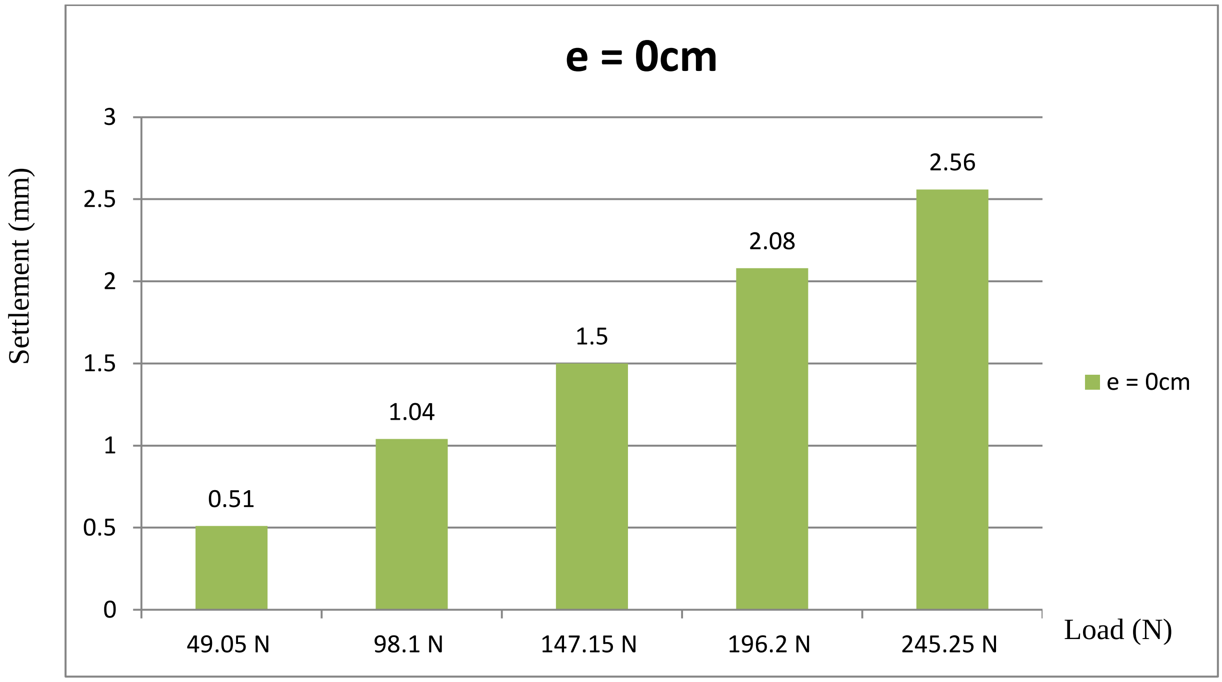

Figure 4.11: Load – Settlement variation for single layer, e=0cm

Table 12: Load – Settlement data for single layer, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	5	4	0.40
98.1			0.76
147.15			1.16
196.2			1.55
245.25			1.96

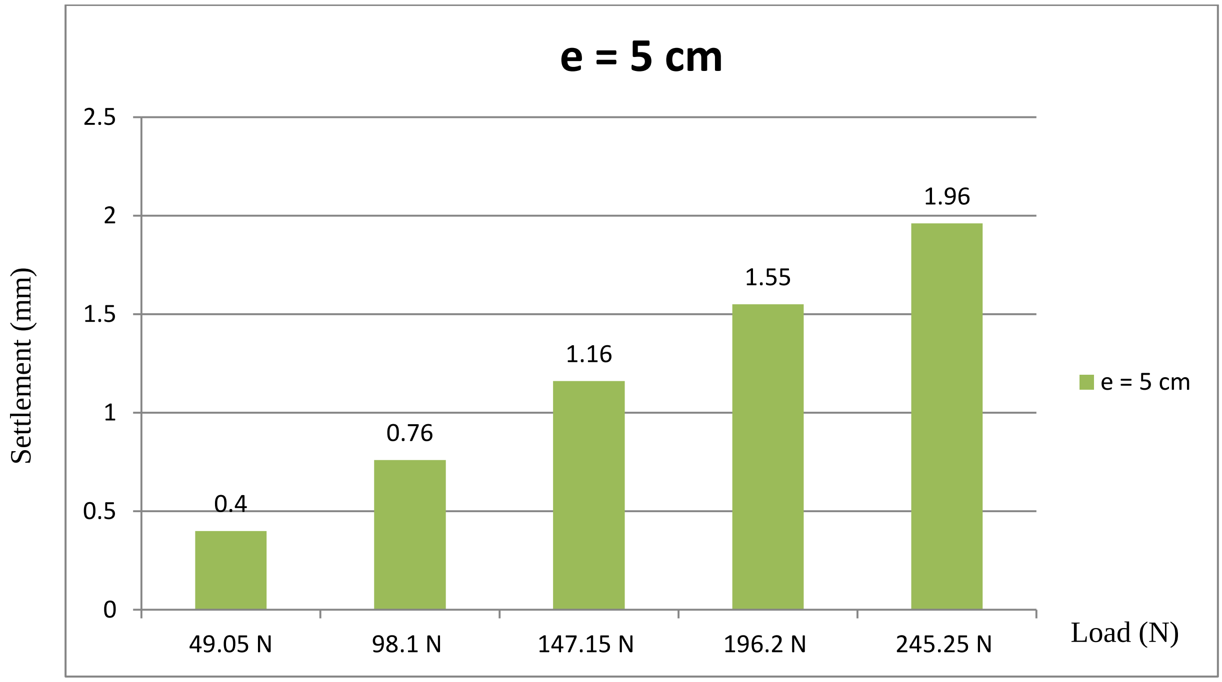

Figure 4.12: Load – Settlement variation for single layer, e=5cm

Table 13: Load – Settlement data for single layer, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	8.33	4	0.42
98.1			0.85
147.15			1.23
196.2			1.65
245.25			2.06

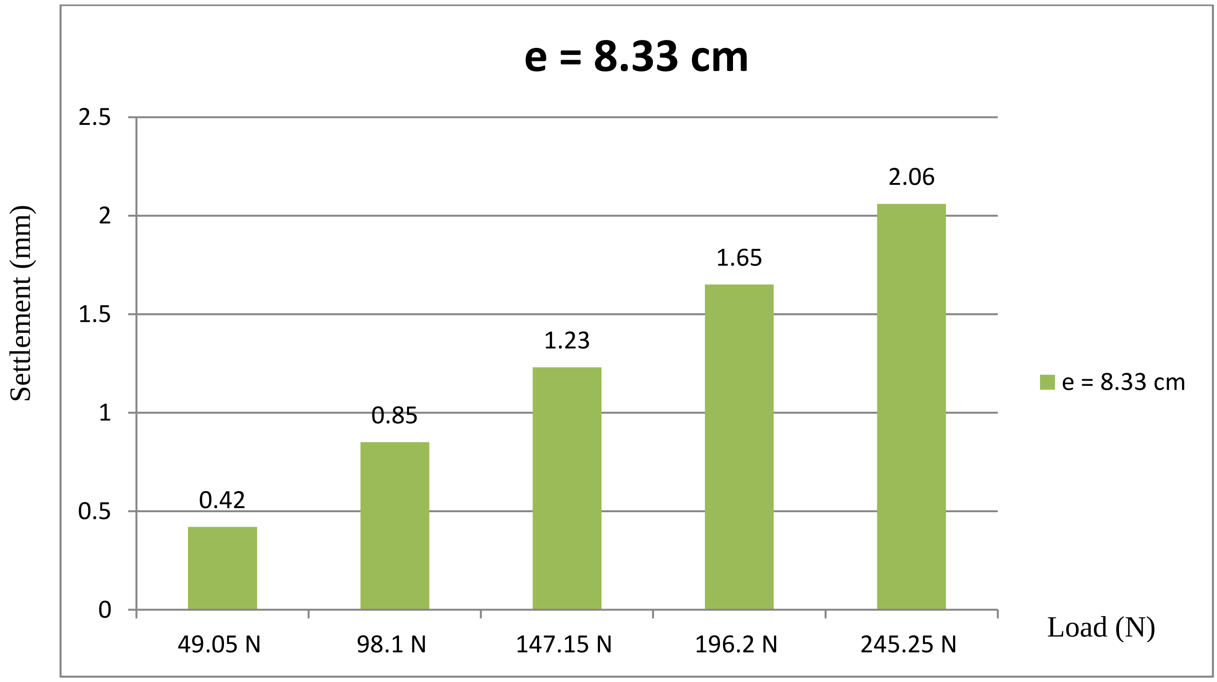

Figure 4.13: Load – Settlement variation for single layer, e=8.33cm

Table 14: Load – Settlement data for single layer, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	10	4	0.54
98.1			1.06
147.15			1.58
196.2			2.10
245.25			2.65

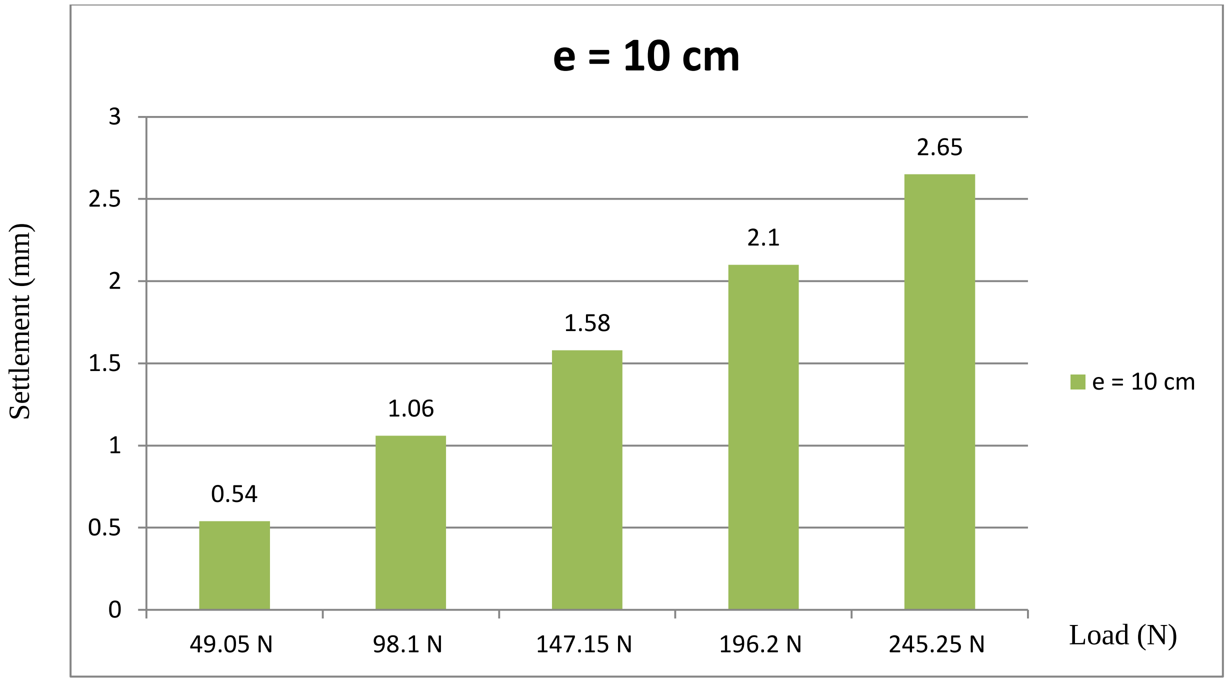

Figure 4.14: Load – Settlement variation for single layer, e=10cm

4.2.3 First reinforcement layer at u = 7 cm

At a depth of 7 cm, a single layer of reinforcement is applied, below the base of footing and load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

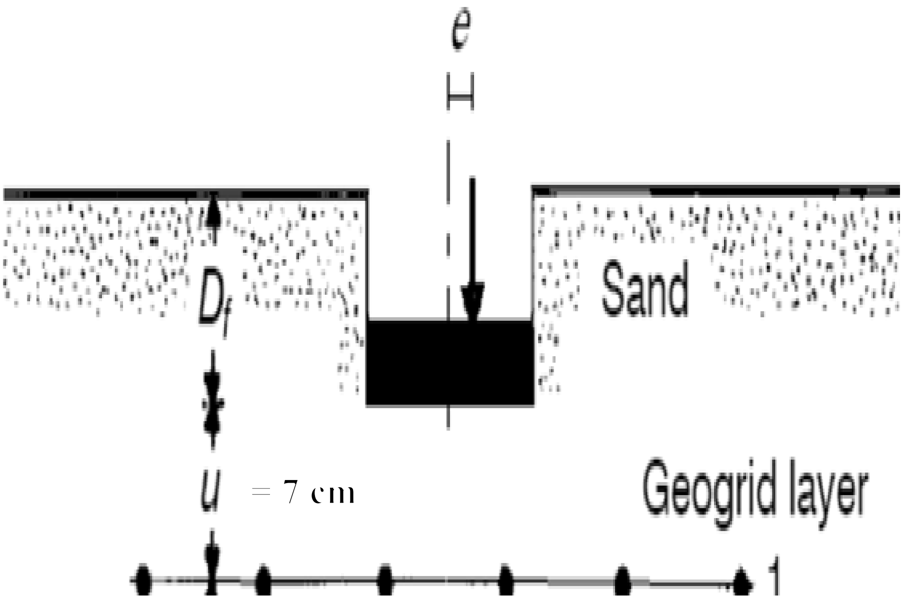

Figure 4.15: Schematic diagram of soil loaded with eccentric loading and with a single layers of geogrid reinforcement at u = 7 cm

Table 15: Load – Settlement data for single layer, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	0	7	0.43
98.1			0.86
147.15			1.29
196.2			1.72
245.25			2.12

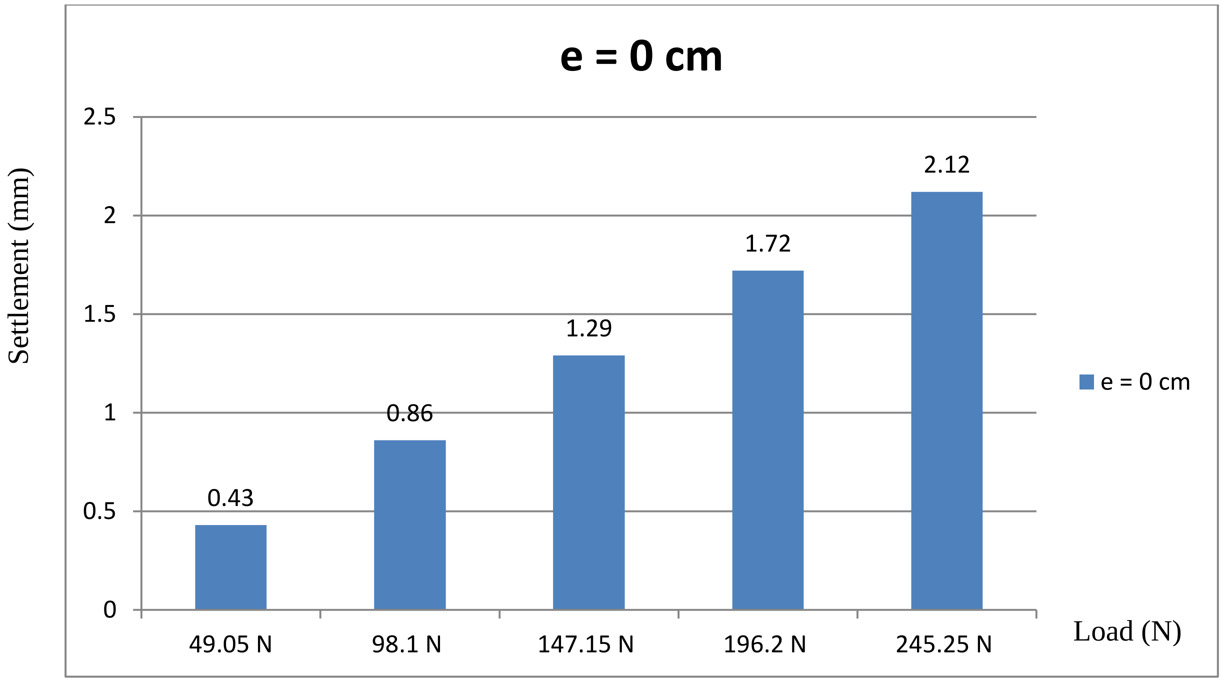

Figure 4.16: Load – Settlement variation for single layer, e=0cm

Table 16: Load – Settlement data for single layer, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	5	7	0.48
98.1			0.95
147.15			1.41
196.2			1.91
245.25			2.36

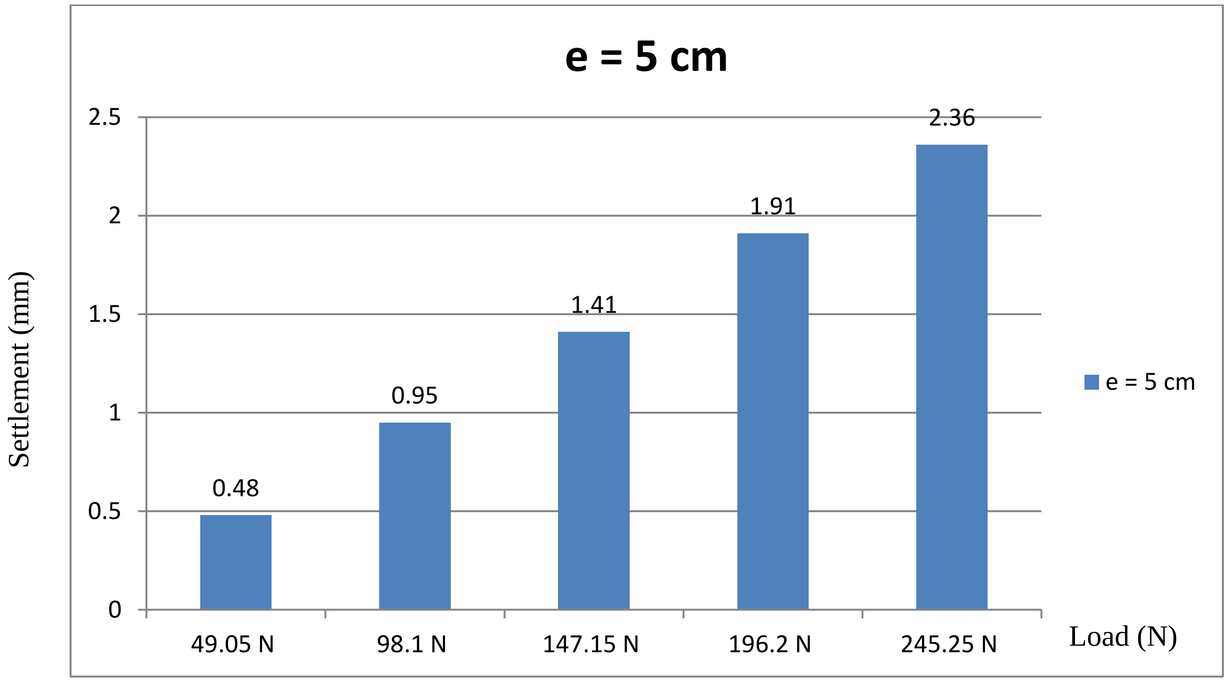

Figure 4.17: Load – Settlement variation for single layer, e=5cm

Table 17: Load – Settlement data for single layer, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	8.33	7	0.50
98.1			0.97
147.15			1.45
196.2			1.96
245.25			2.45

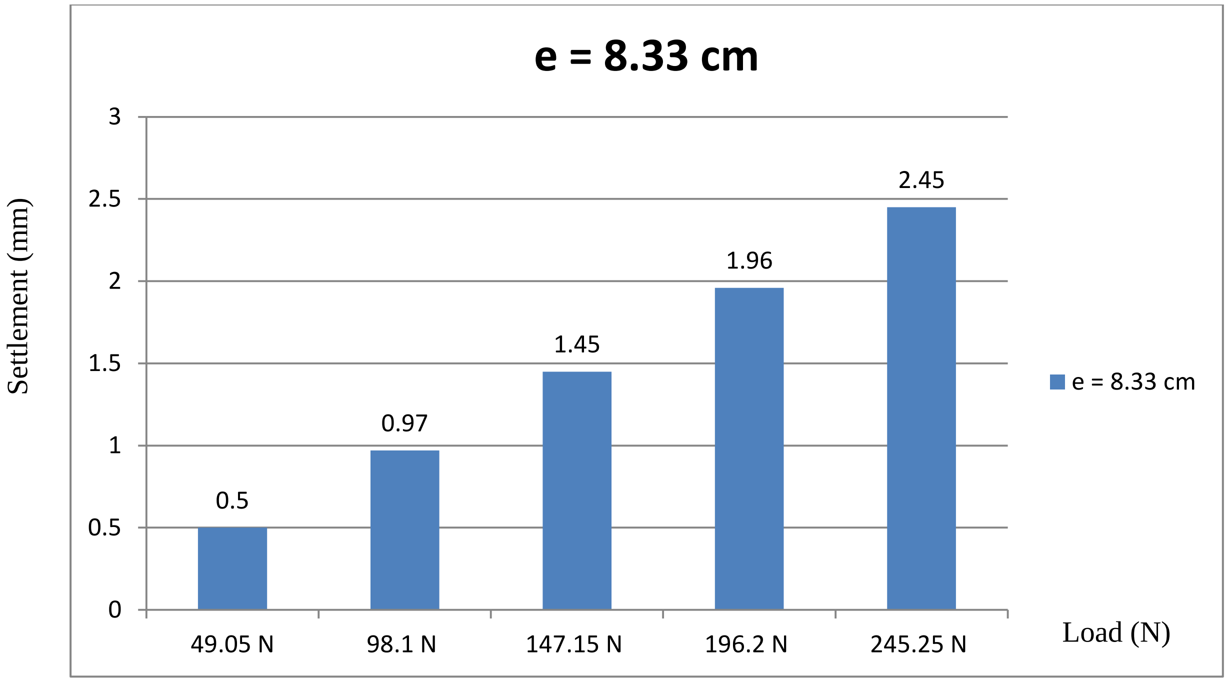

Figure 4.18: Load – Settlement variation for single layer, e=8.33cm

Table 18: Load – Settlement data for single layer, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	Settlement (mm)
49.05	10	7	0.55
98.1			1.08
147.15			1.63
196.2			2.16
245.25			2.71

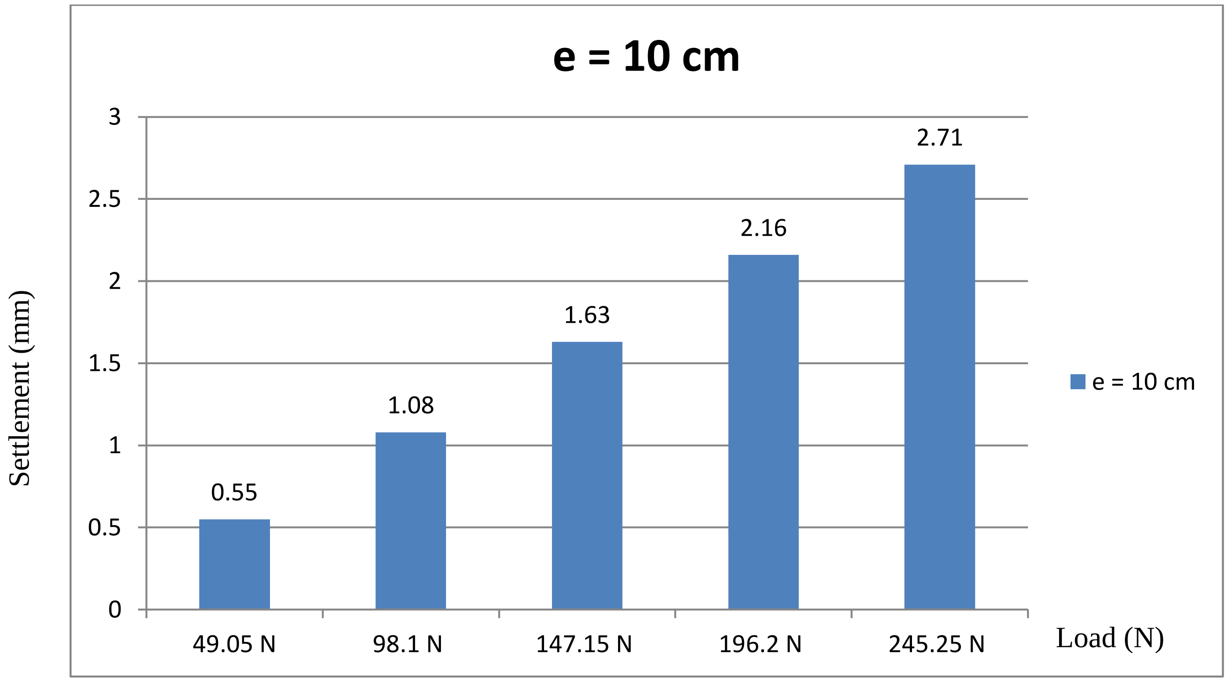

Figure 4.19: Load – Settlement variation for single layer, e=10cm

4.2.4 Two layers of reinforcement for u = 4cm and h = 2cm

In this case two layers of reinforcement are placed, first layer is placed at u = 4cm and second layer is placed at h = 2cm. From the previous case it has been observed that for the first reinforcement layer the optimum depth is 4 cm hence we will consider u = 4 cm for all the above cases. Load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

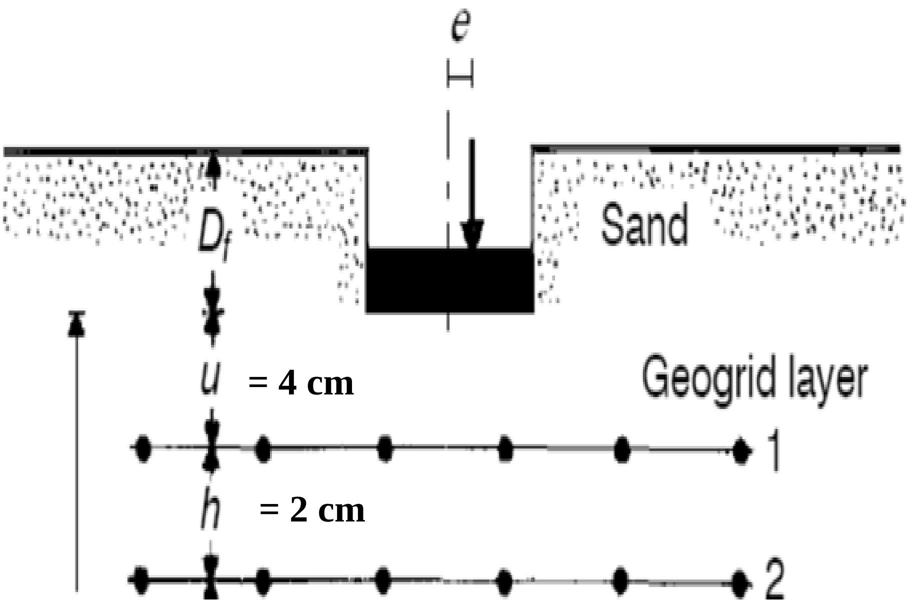

Figure 4.20: Schematic diagram of soil loaded with eccentric loading and with two layers of geogrid reinforcement at u = 4 cm and h = 2 cm

Table 19: Load – Settlement data for double layer, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	0	4	2	0.34
98.1				0.67
147.15				1.01
196.2				1.33
245.25				1.67

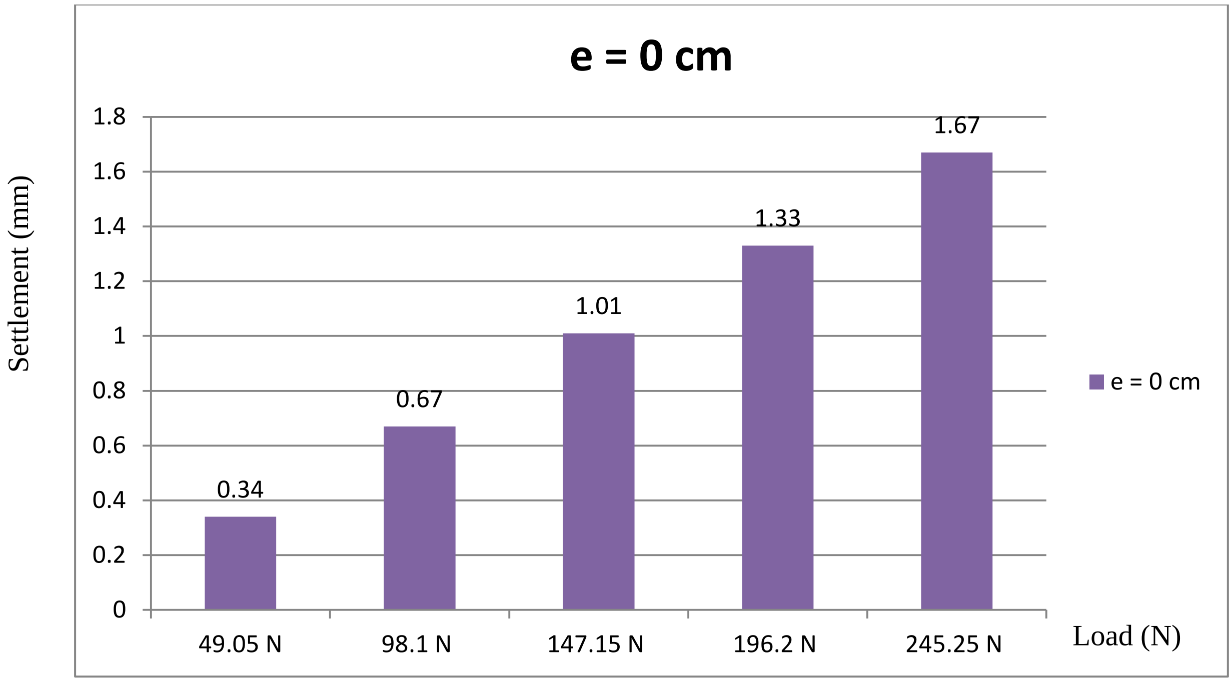

Figure 4.21: Load – Settlement variation for double layer, e=0cm

Table 20: Load – Settlement data for double layer, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	5	4	2	0.29
98.1				0.56
147.15				0.85
196.2				1.10
245.25				1.43

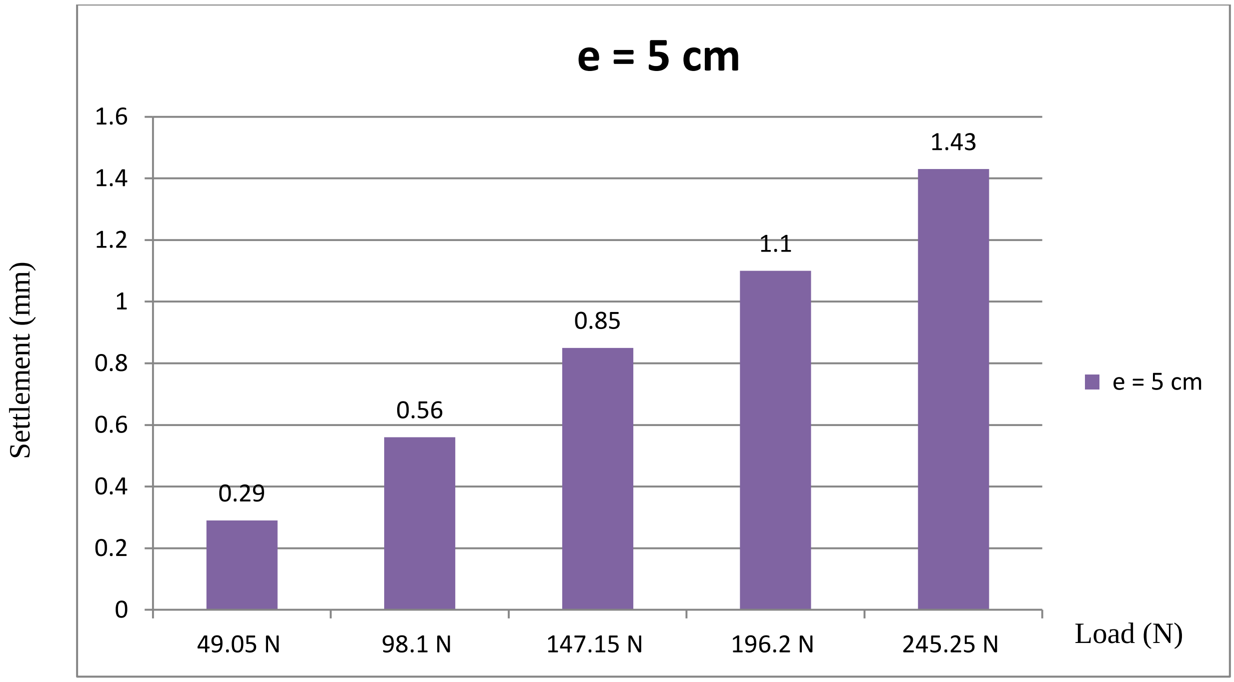

Figure 4.22: Load – Settlement variation for double layer, e=5cm

Table 21: Load – Settlement data for double layer, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	8.33	4	2	0.26
98.1				0.50
147.15				0.75
196.2				1.02
245.25				1.28

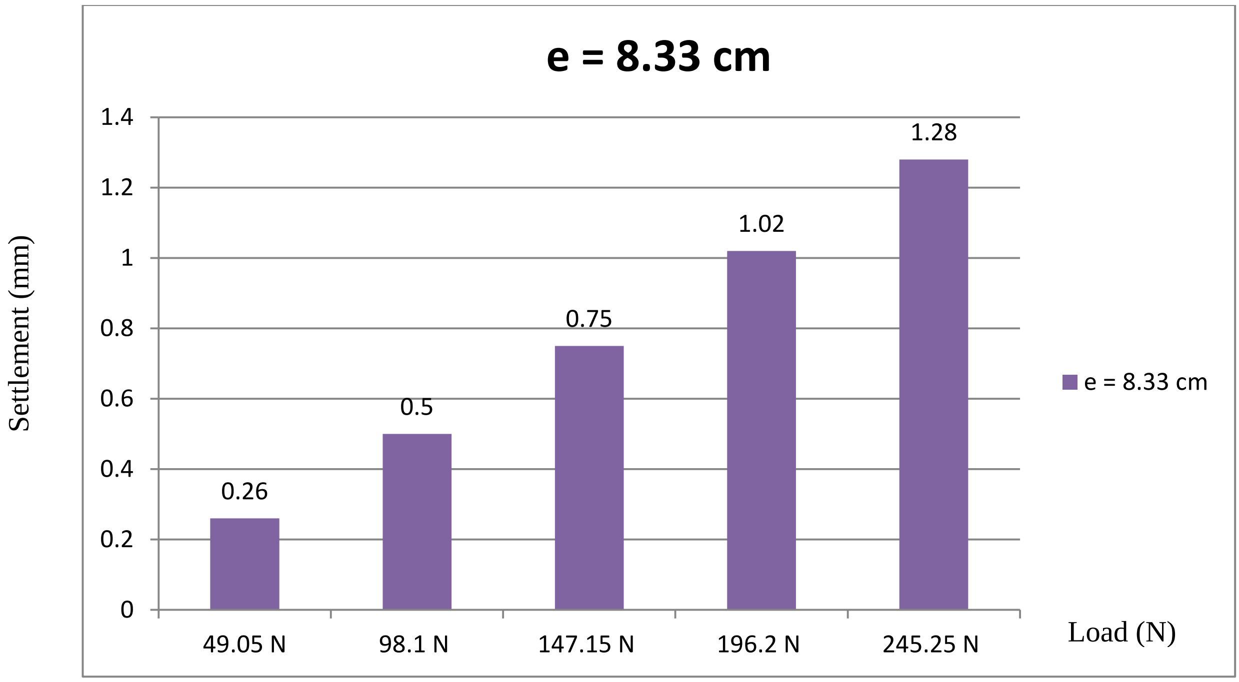

Figure 4.23: Load – Settlement variation for double layer, e=8.33cm

Table 22: Load – Settlement data for double layer, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	10	4	2	0.38
98.1				0.74
147.15				1.10
196.2				1.49
245.25				1.88

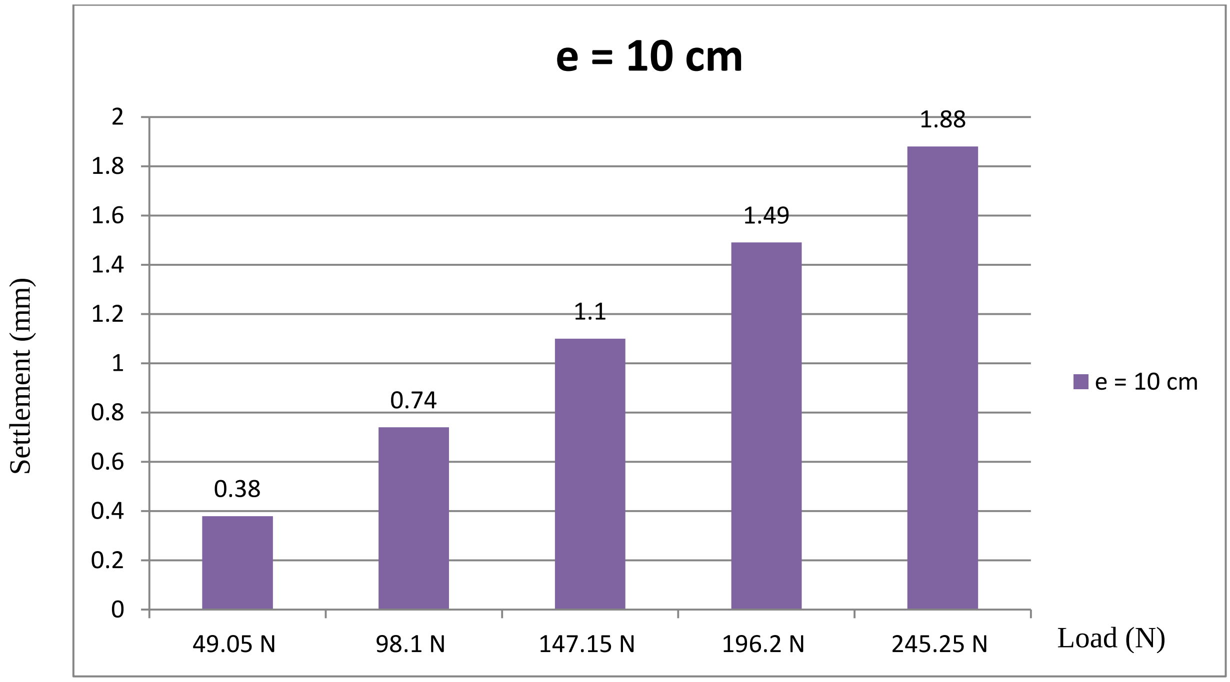

Figure 4.24: Load – Settlement variation for double layer, e=10cm

4.2.5 Two layers of reinforcement for u = 4cm and h = 4cm

In this case two layers of reinforcement are placed, first layer is placed at u = 4cm and second layer is placed at h = 4cm. From the previous case it has been observed that for the first reinforcement layer the optimum depth is 4 cm hence we will consider u = 4 cm for all the above cases. Load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

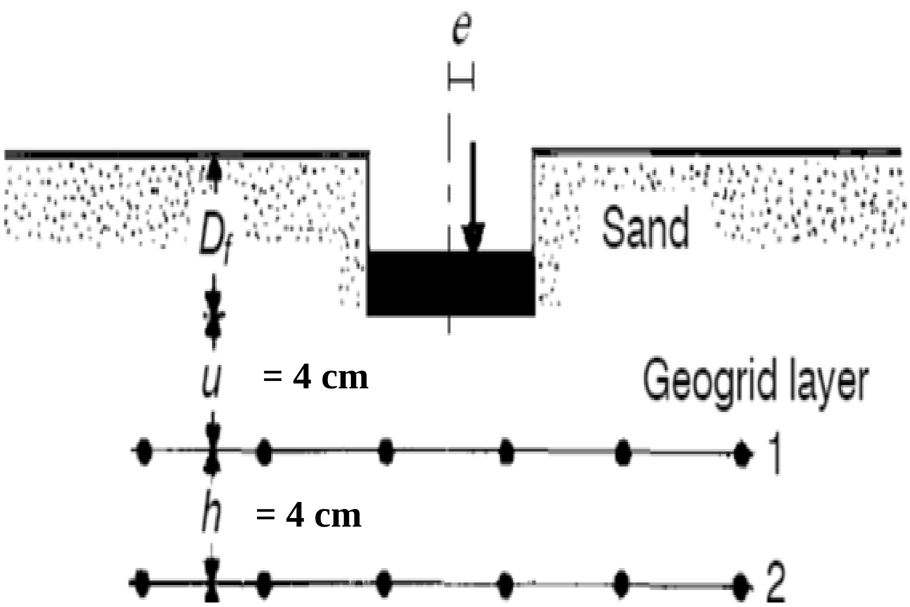

Figure 4.25: Schematic diagram of soil loaded with eccentric loading and with two layers of geogrid reinforcement at u = 4 cm and h = 4 cm

Table 23: Load – Settlement data for double layer, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	0	4	4	0.32
98.1				0.62
147.15				0.95
196.2				1.26
245.25				1.56

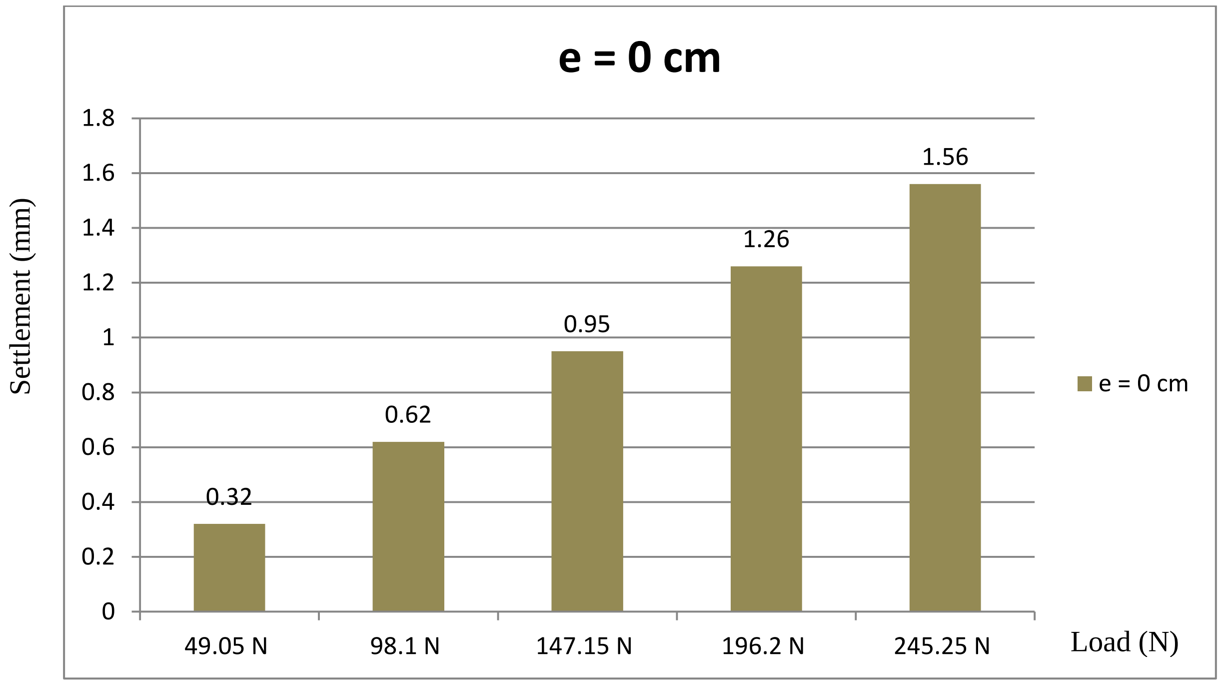

Figure 4.26: Load – Settlement variation for double layer, e=0cm

Table 24: Load – Settlement data for double layer, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	5	4	4	0.26
98.1				0.51
147.15				0.75
196.2				1.02
245.25				1.26

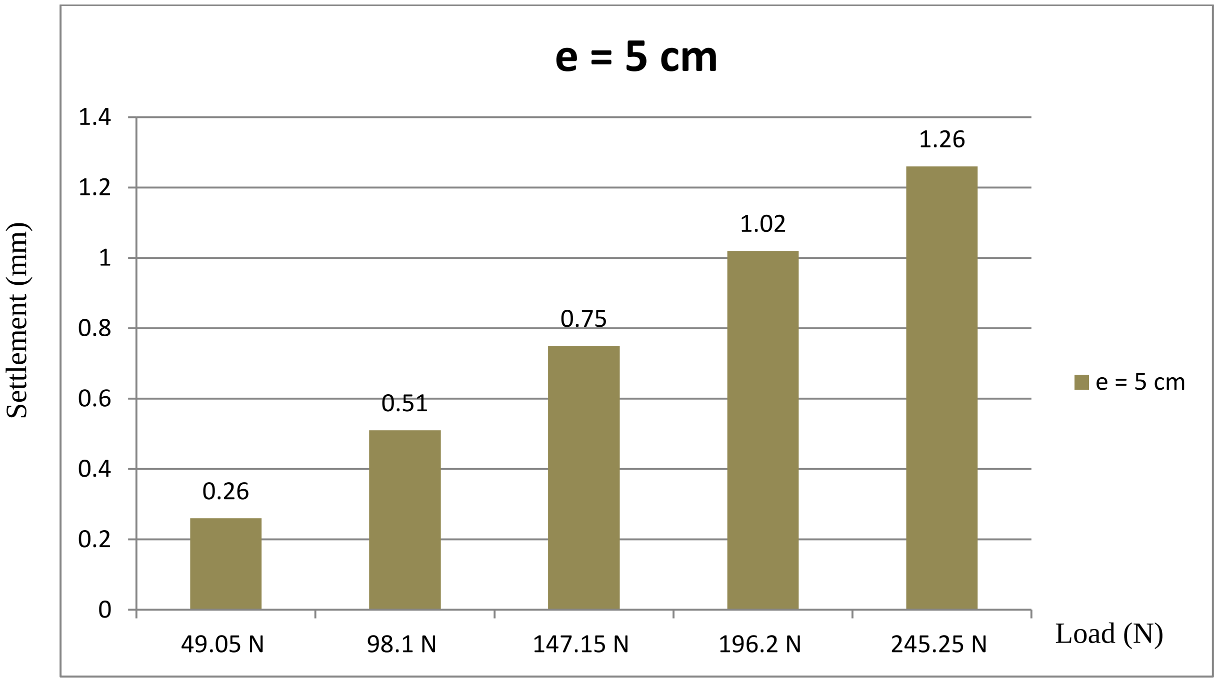

Figure 4.27: Load – Settlement variation for double layer, e=5cm

Table 25: Load – Settlement data for double layer, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	8.33	4	4	0.23
98.1				0.45
147.15				0.66
196.2				0.90
245.25				1.10

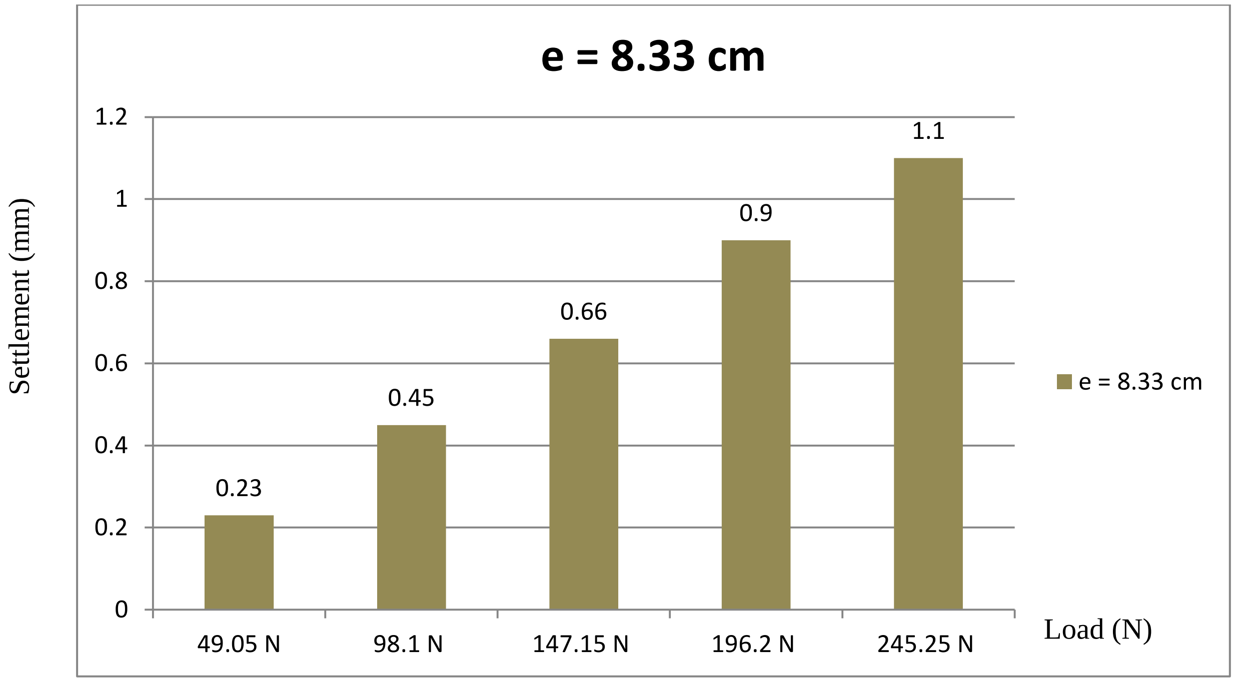

Figure 4.28: Load – Settlement variation for double layer, e=8.33cm

Table 26: Load – Settlement data for double layer, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	10	4	4	0.39
98.1				0.76
147.15				1.15
196.2				1.53
245.25				1.94

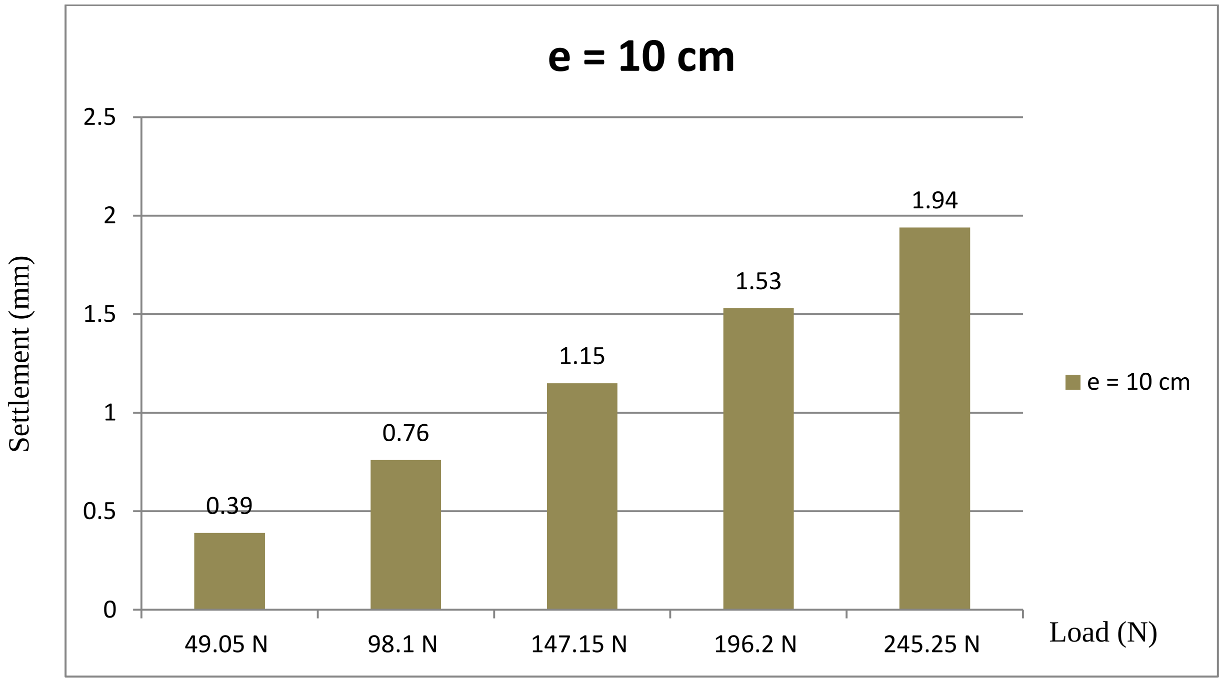

Figure 4.29: Load – Settlement variation for double layer, e=10cm

4.2.6 Two layers of reinforcement for u = 4cm and h = 7cm

In this case two layers of reinforcement are placed, first layer is placed at u = 4cm and second layer is placed at h = 7cm. From the previous case it has been observed that for the first reinforcement layer the optimum depth is 4 cm hence we will consider u = 4 cm for all the above cases. Load is applied similarly as in previous case of 49.05 N, 98.1 N, 147.15 N, 196.2 N, 245.25 N and each load is applied at eccentricity of 0cm, 5cm, 8.33cm, 10cm.

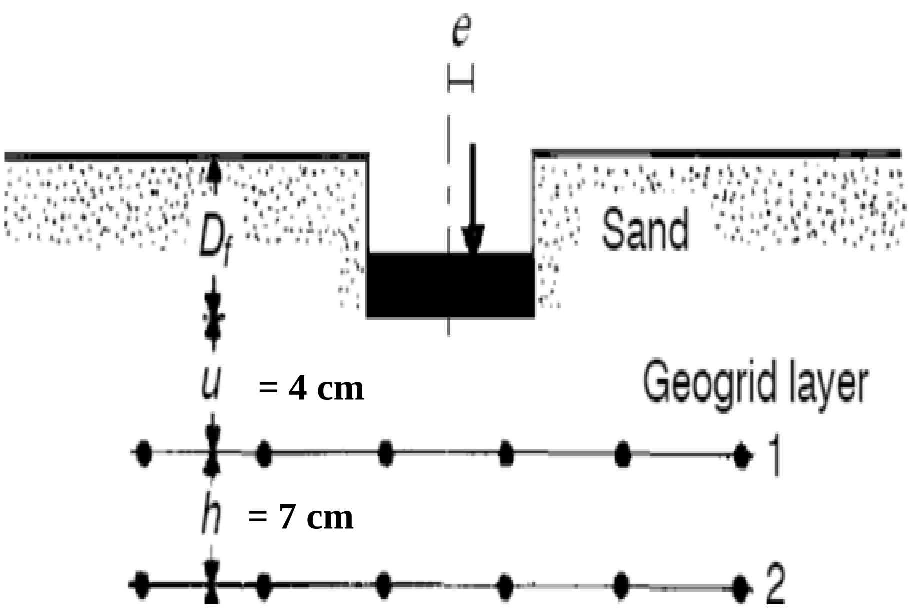

Figure 4.30: Schematic diagram of soil loaded with eccentric loading and with two layers of geogrid reinforcement at u = 4 cm and h = 7cm

Table 27: Load – Settlement data for double layer, e=0cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	0	4	7	0.33
98.1				0.65
147.15				0.96
196.2				1.29
245.25				1.63

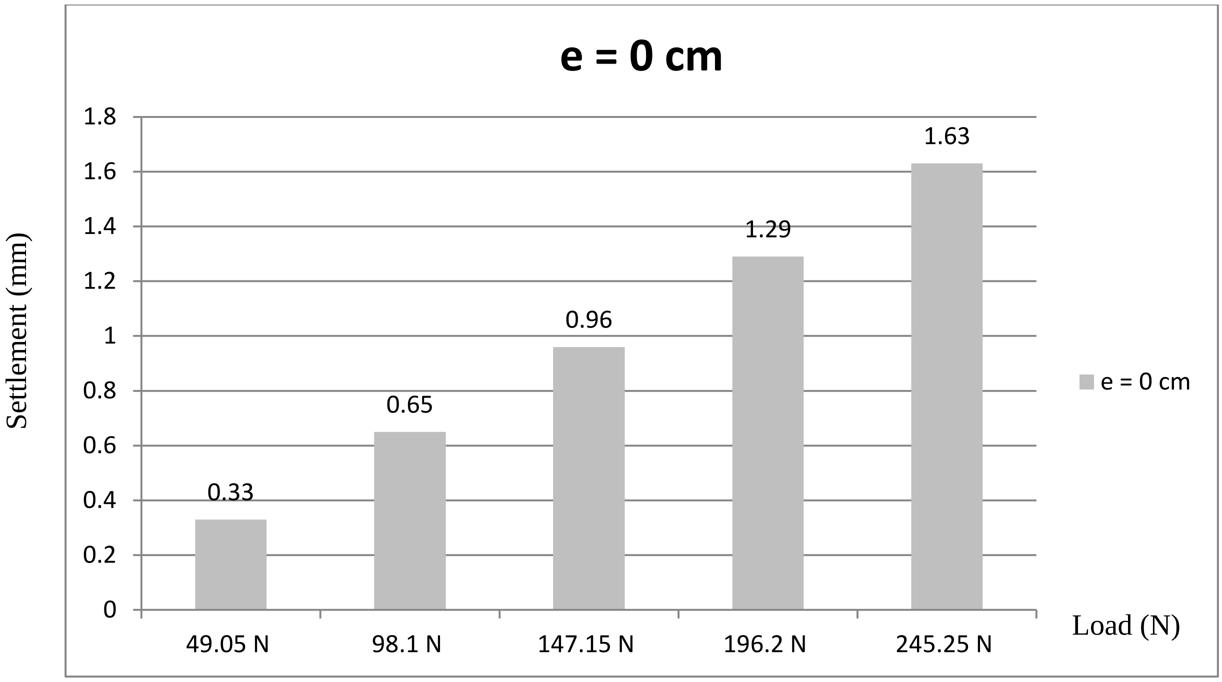

Figure 4.31: Load – Settlement variation for double layer, e=0cm

Table 28: Load – Settlement data for double layer, e=5cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	5	4	7	0.27
98.1				0.53
147.15				0.80
196.2				1.07
245.25				1.34

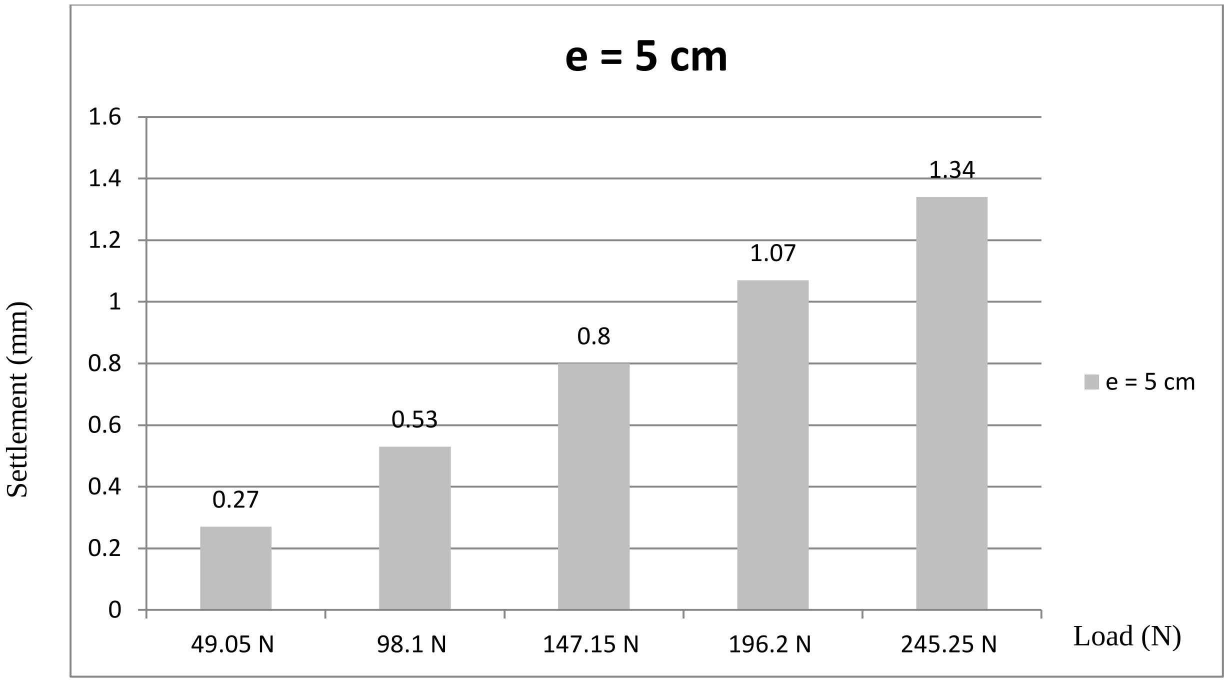

Figure 4.32: Load – Settlement variation for double layer, e=5cm

Table 29: Load – Settlement data for double layer, e=8.33cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	8.33	4	7	0.28
98.1				0.54
147.15				0.82
196.2				1.10
245.25				1.36

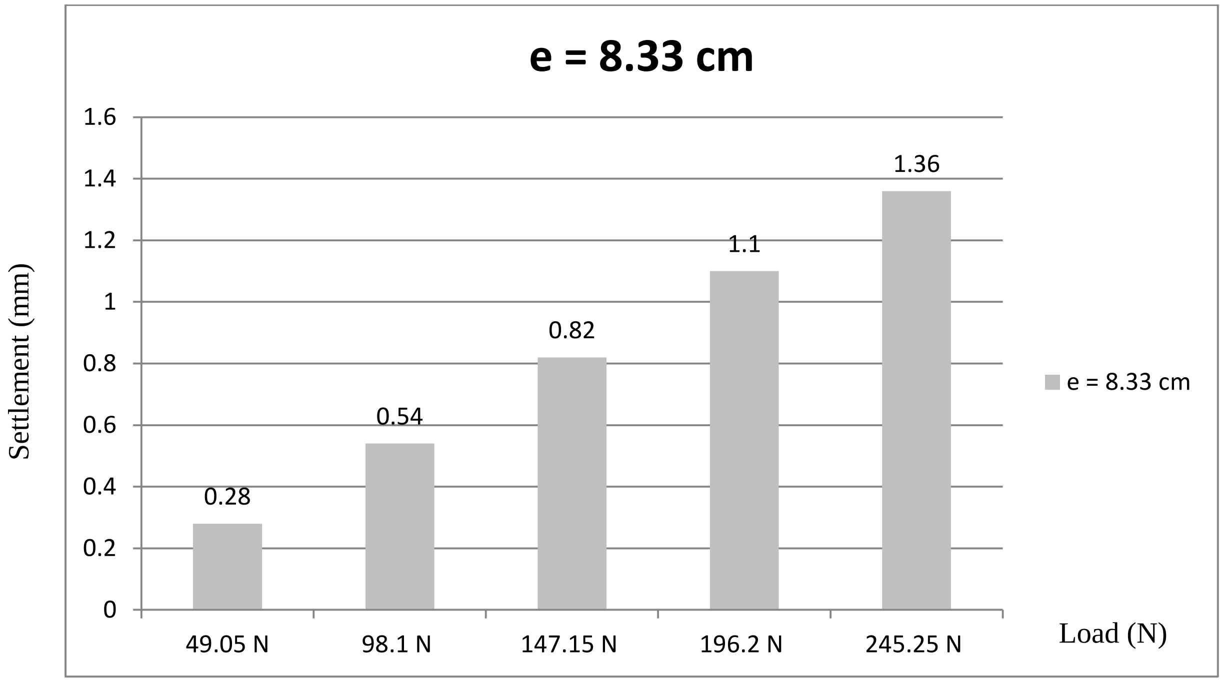

Figure 4.33: Load – Settlement variation for double layer, e=8.33cm

Table 30: Load – Settlement data for double layer, e=10cm

Load (N)	Eccentricity (cm)	u(cm)	h (cm)	Settlement (mm)
49.05	10	4	7	0.38
98.1				0.74
147.15				1.12
196.2				1.50
245.25				1.88

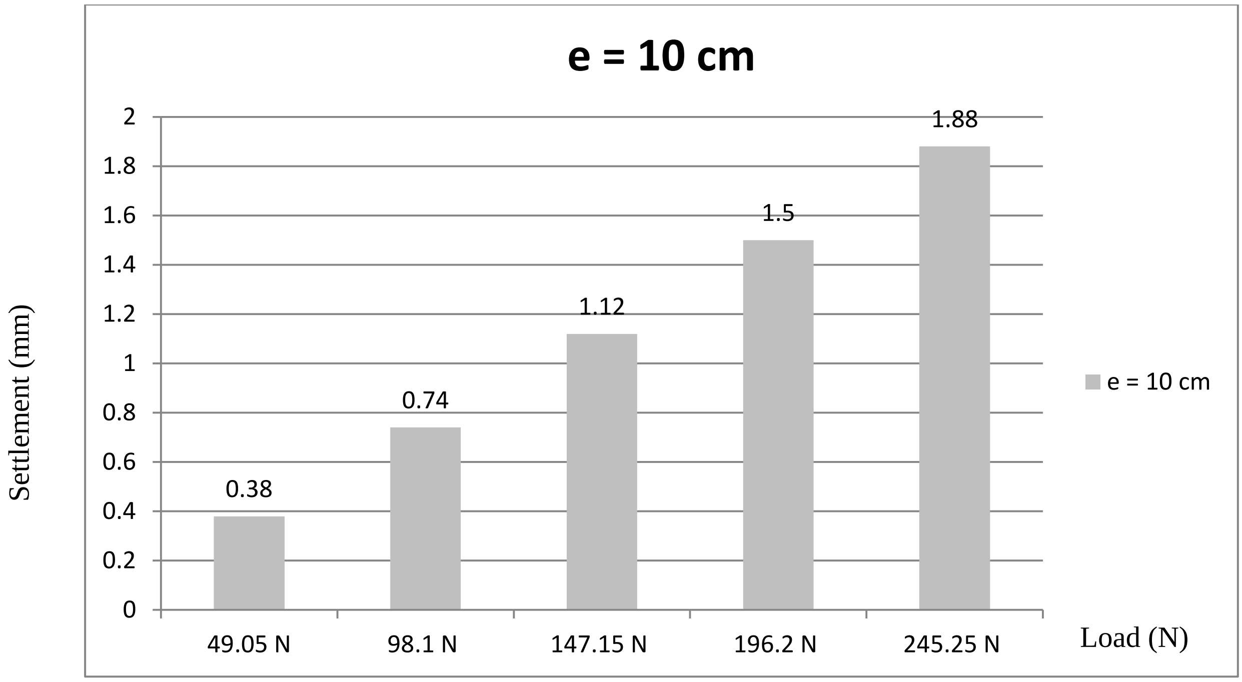

Figure 4.34: Load – Settlement variation for double layer, e=10cm

CHAPTER 5

RESULT AND DISCUSSION

5.1 Unreinforced soil case

Load vs settlement curves for the unreinforced soil case for different values of load at different values of eccentricity are observed and the effect of load eccentricity on settlement for different values of load is observed.

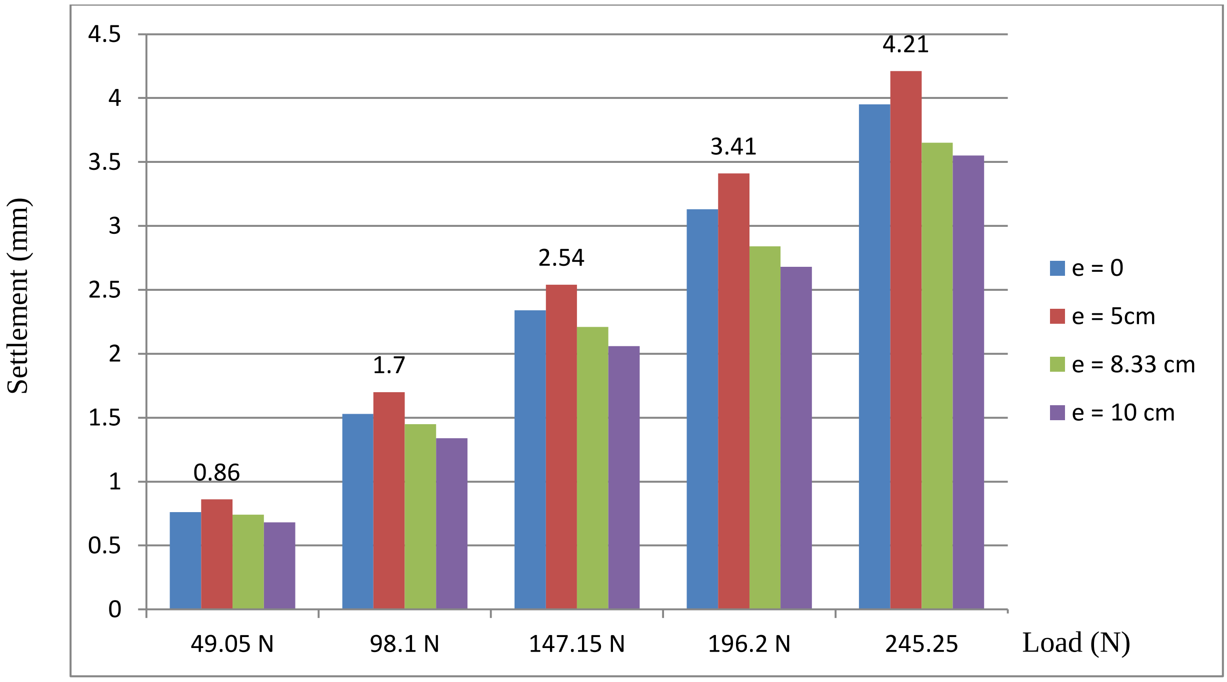

Figure 4.35: Load – Settlement variation for unreinforced soil

From the comparative figure it can be observed that the settlement increases with increase in load and as eccentricity increases from e = 0cm to e =5cm settlement increases and further increment in eccentricity upto kern boundary and beyond, settlement decreases and the maximum settlement is obtained when load eccentricity is between the kern area.

5.2 Reinforced soil case

5.2.1 First reinforcement layer at u = 2cm

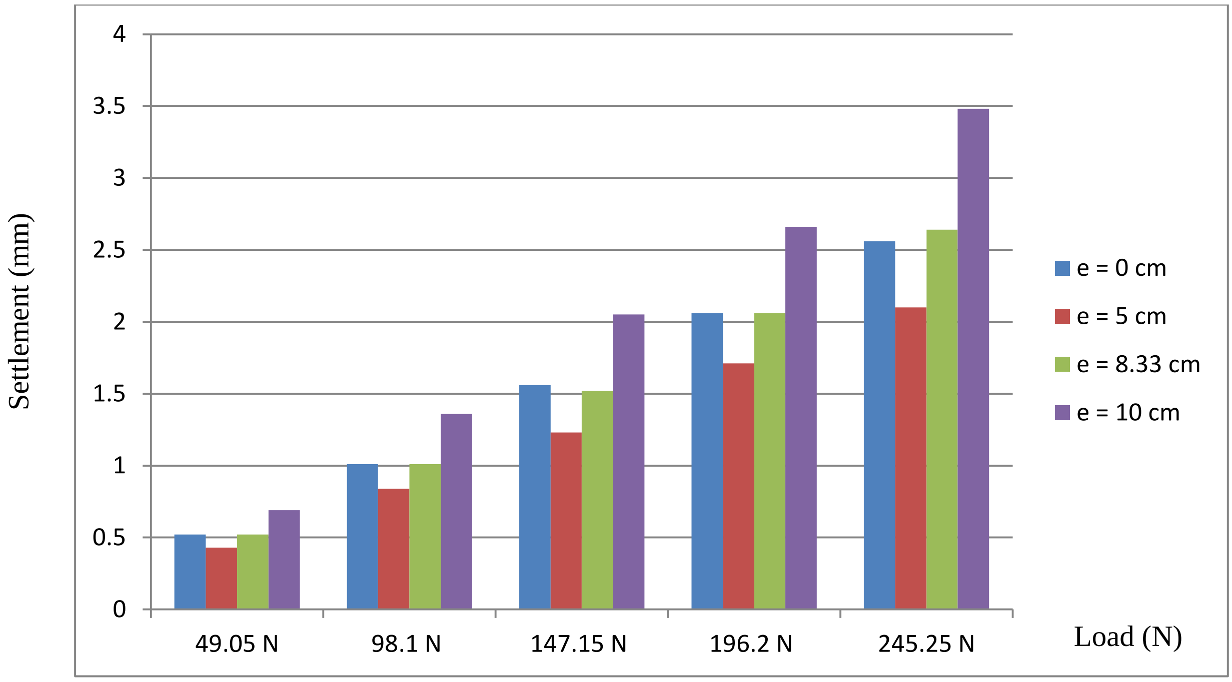

Figure 4.36: Load – Settlement variation for reinforced soil, u = 2cm

In this case when first reinforcement is placed at a depth of 2 cm below the base of footing then the soil between the footing and reinforcement is slightly compacted and from the above figure it has been observed that when load eccentricity increases from concentric to the kern boundary the settlement initially decreases and then again increases almost at the same value as concentric load and with load eccentricity outside the kern boundary, the settlement increases sharply.

5.2.2 First reinforcement layer at u = 4cm

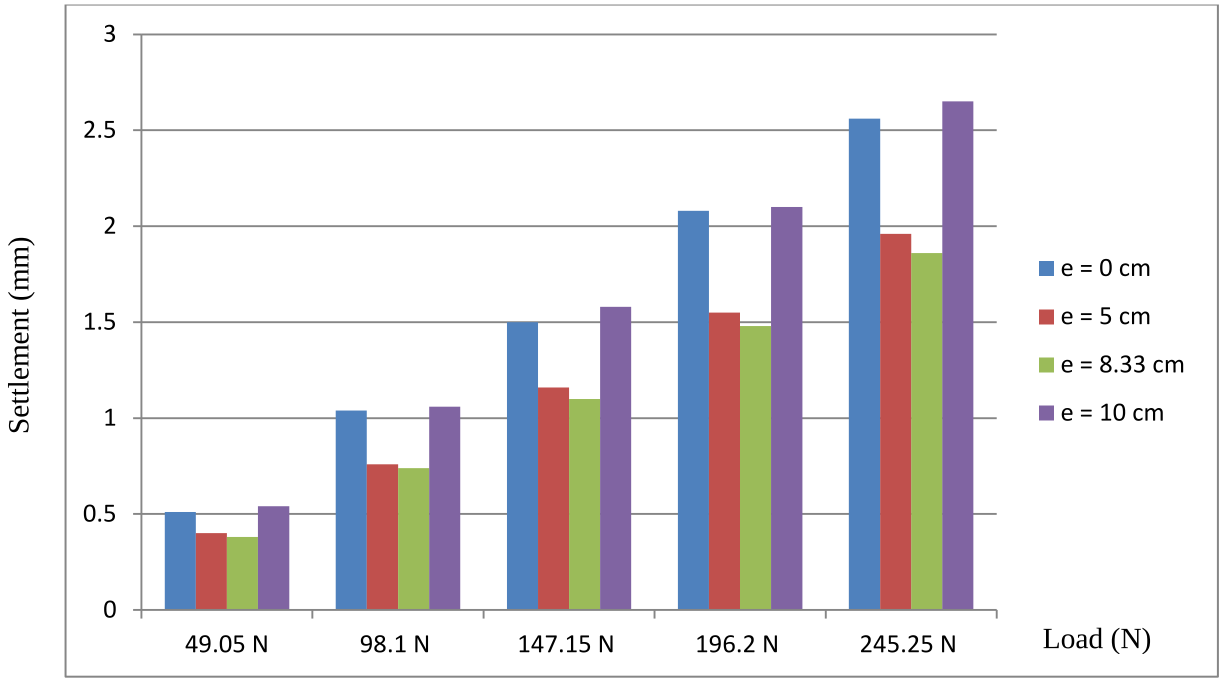

Figure 4.37: Load – Settlement variation for reinforced soil, u = 4cm

In this case geogrid reinforcement layer is placed at a depth of 4 cm below the base of footing.

In this case for a particular value of load when eccentricity increases from e = 0 cm to e = 5 cm, settlement decreases and with further increase in eccentricity from e = 5 cm to e = 8.33 cm settlement slightly decreases and beyond the kern boundary, settlement increases.

On comparing with other depth of reinforcement, the settlement obtained in this case is less than other hence this is the optimum distance for placing the first reinforcement layer.

5.2.3 First reinforcement layer at u = 7cm

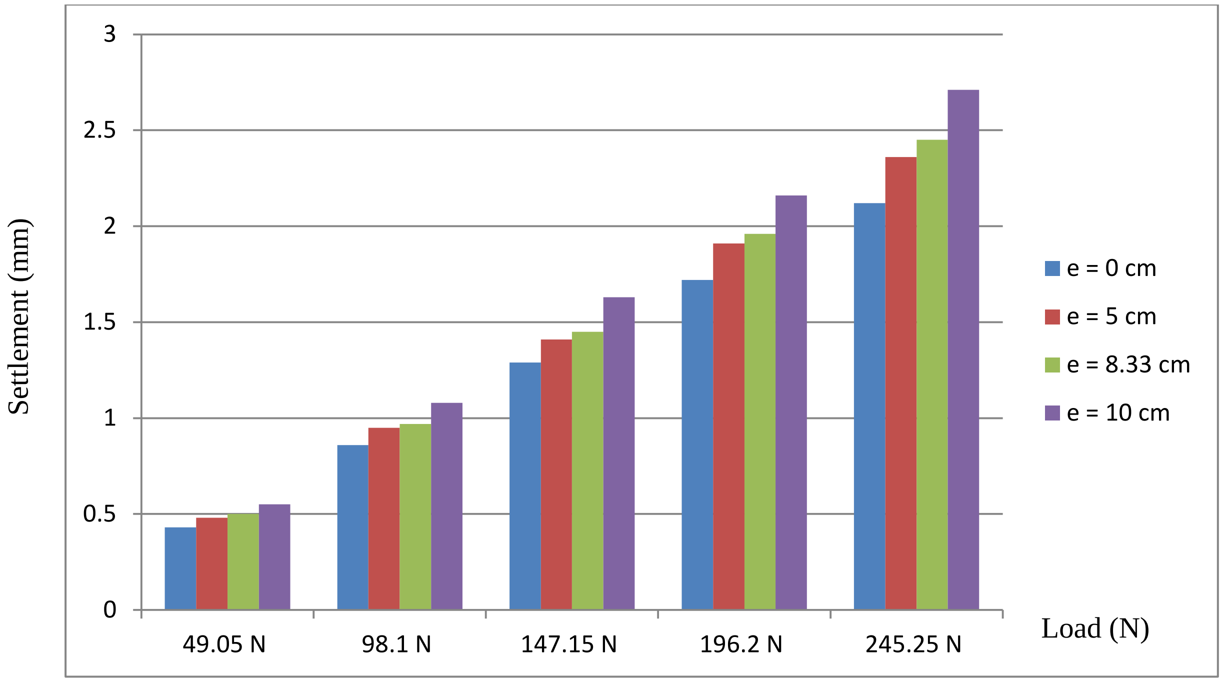

Figure 4.38: Load – Settlement variation for reinforced soil, u = 7cm

In this case geogrid reinforcement layer is placed at depth of 7 cm below the base of footing. With increase in eccentricity from e = 0 cm to e = 5 cm, settlement increases and with further increase in eccentricity from e = 5 cm to e = 8.33 cm settlement slightly increases and when load is beyond the kern boundary settlement continuous increasing trend.

5.2.4 Two layers of reinforcement for u = 4cm and h = 2cm

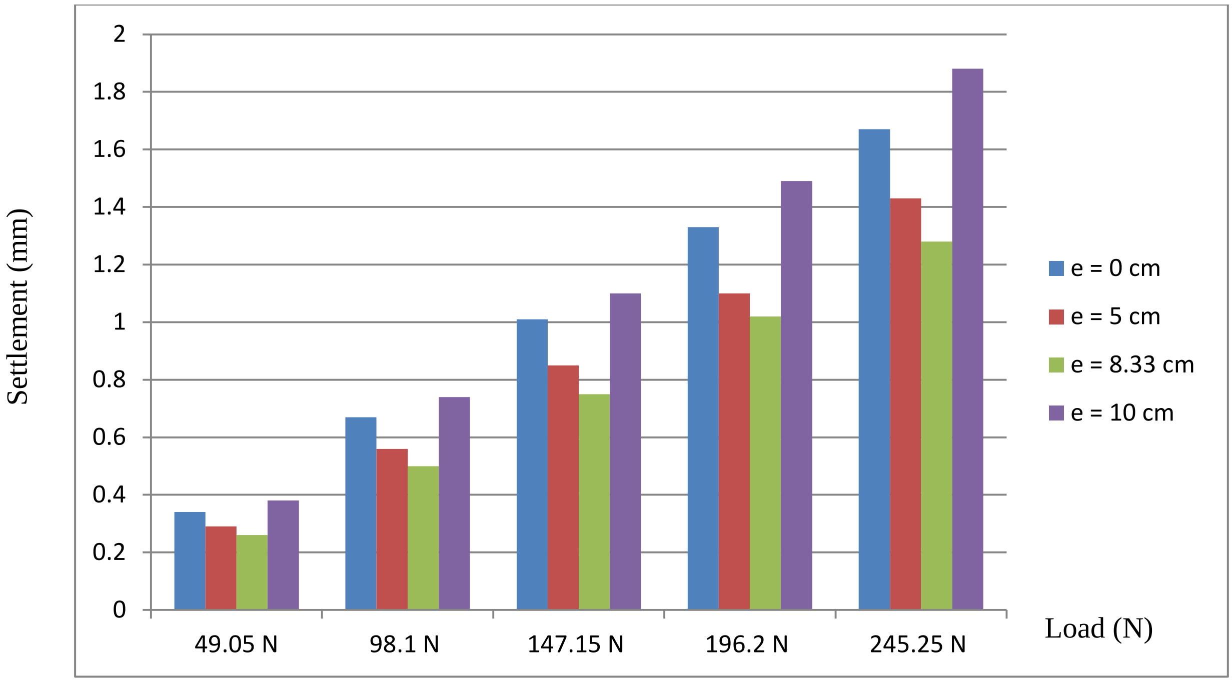

Figure 4.39: Load – Settlement variation for reinforced soil, u=4cm, h=2cm

In this case two layers of reinforcement are placed below the base of footing. For the first reinforcement layer the optimum depth is obtained as 4 cm hence we will fix the first reinforcement at 4 cm and vary the distance of second reinforcement. In this case when eccentricity increases from e = 0 cm to e = 5cm settlement decreases and with further increase in eccentricity from e = 5 cm to e = 8.33 cm settlement further decreases and for load outside the kern boundary settlement increases more than the concentric case.

5.2.5 Two layers of reinforcement for u = 4cm and h = 4cm

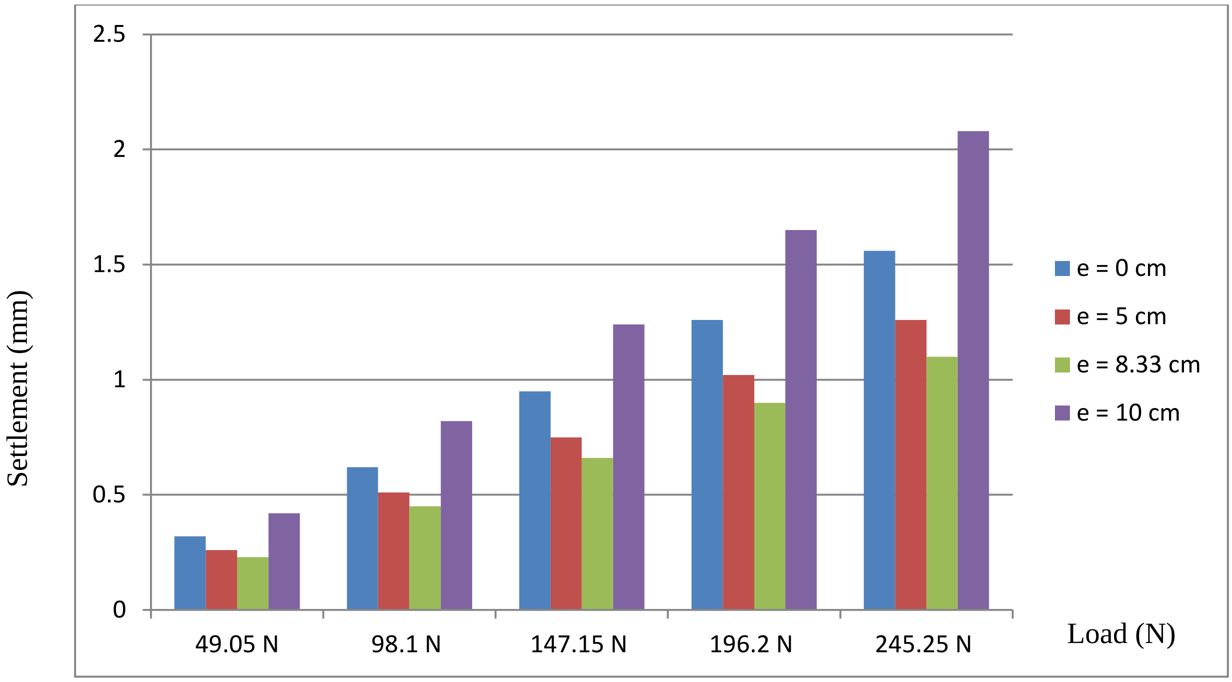

Figure 4.40: Load – Settlement variation for reinforced soil, u=4cm, h=4cm

In this case first reinforcement is placed at 4 cm and second reinforcement layer is also placed at depth of 4 cm. When eccentricity increases from e = 0 cm to e = 5 cm, settlement decreases and with further increase in eccentricity from e = 5 cm to e = 8.33 cm settlement further decreases and when load is beyond the kern boundary, settlement increases sharply.

On comparing h= 4 cm with other cases of depth of reinforcement, the settlement obtained here is less than other cases hence this is the optimum depth of placing the second reinforcement layer.

5.2.6 Two layers of reinforcement for u = 4cm and h = 7cm

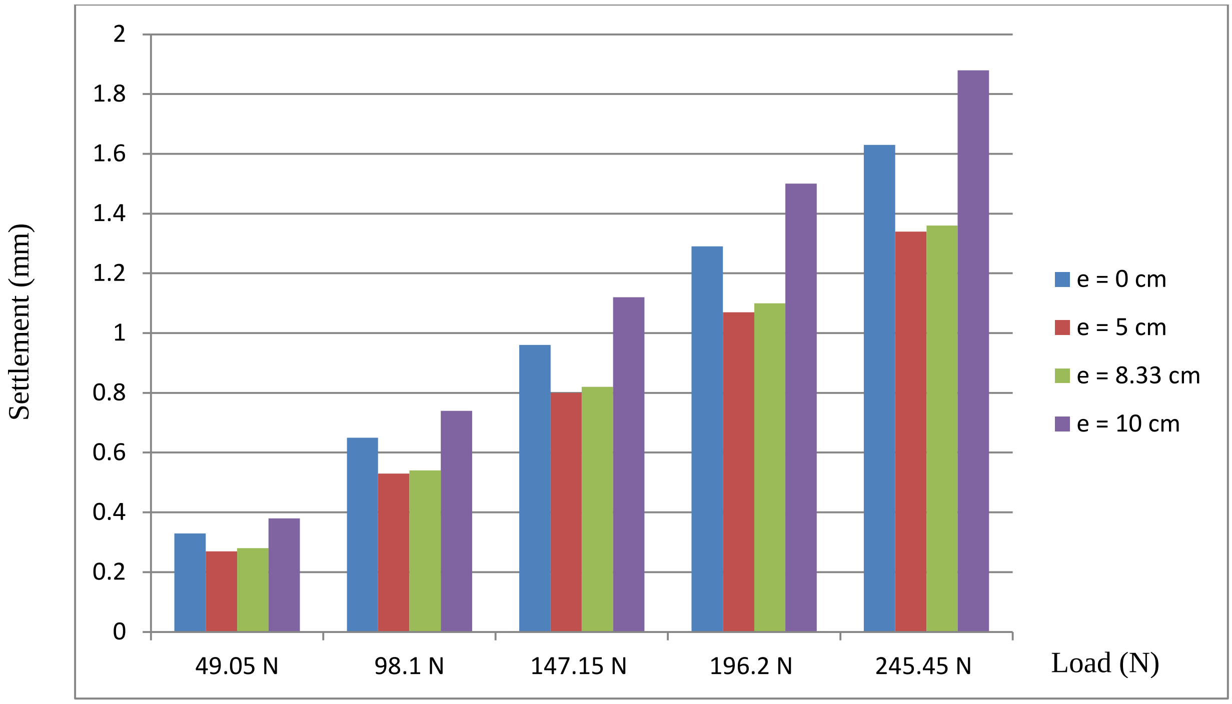

Figure 4.41: Load – Settlement variation for reinforced soil, u=4cm, h=7cm

In this first reinforcement layer is placed at a depth of 4 cm and second reinforcement layer is placed at depth of 7 cm. When eccentricity increases from e = 0 cm to e = 5 cm, settlement decreases and with further increase in eccentricity from e = 5 cm to e = 8.33 cm, settlement increases slightly. When load is beyond the kern boundary, settlement increases.

5.3 Comparitive study of first reinforcement for u = 2 cm, 4 cm and 7 cm

Load = 49.05 N

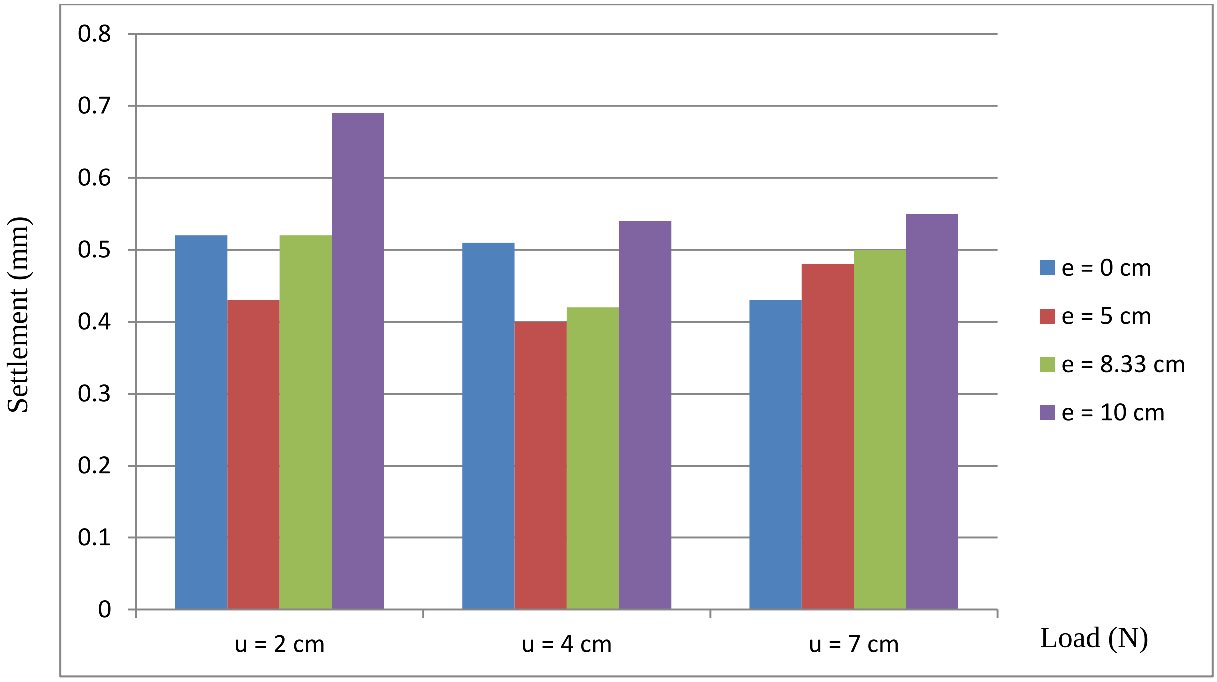

Figure 4.42: Load – Settlement variation for reinforced soil, u=2,4,7 cm

This is the comparison to obtain the optimum depth of reinforcement for the first reinforcement layer. For load 49.05 N, the settlement data obtained for u = 4 cm are more reliable than other cases.

For u = 4cm, the settlement obtained with in the kern boundary are less than the other cases and for outside the kern area the settlement is most suitable. Hence

u = 4 cm is the optimum depth of placing the first reinforcement layer.

5.4 Comparitive study of two reinforcement for u = 4cm and h = 2cm, 4cm & 7cm

For load = 49.05 N

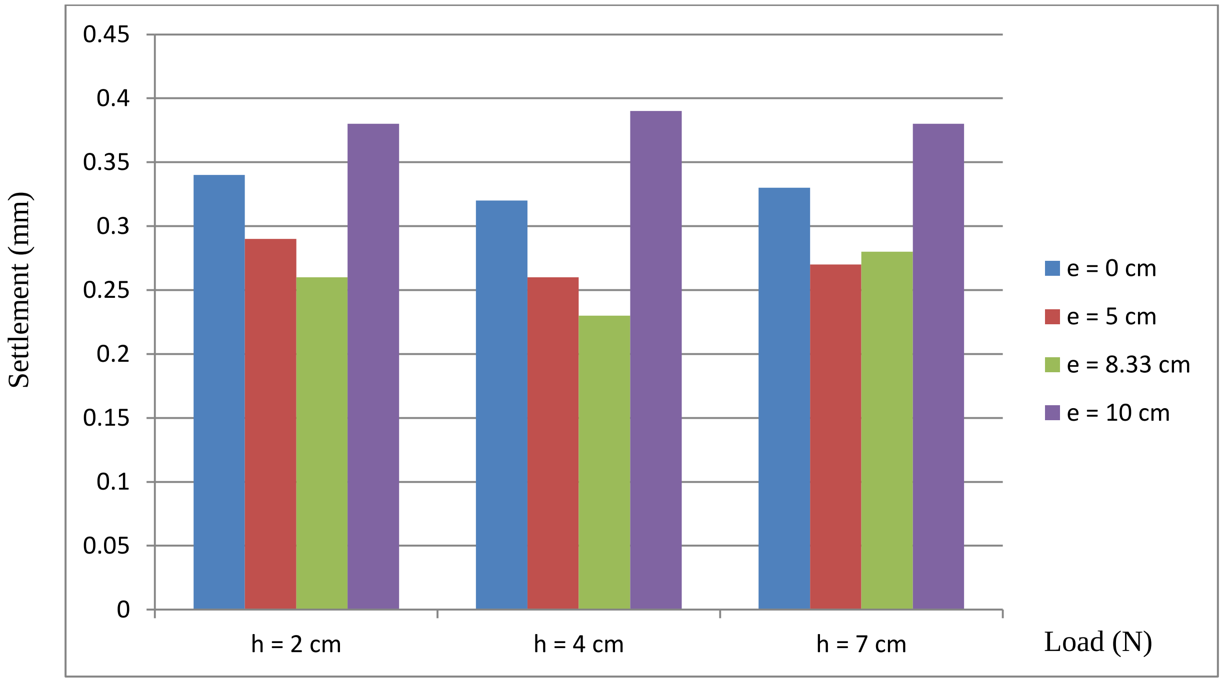

Figure 4.43: Load – Settlement variation for reinforced soil, u=4cm& h= 2,4,7 cm

This is the comparison to obtain the optimum depth of reinforcement for the second reinforcement layer. For load 49.05 N, the settlement data obtained for u = 4 cm & h = 4 cm are more reliable than other cases.

For h = 4 cm, the settlement obtained with in the kern boundary is less than other two cases and outside the kern boundary the settlement data is most reliable. Hence h = 4 cm is the optimum depth of placing the second reinforcement layer.

5.5 Comparative study for unreinforced and reinforced case

Load = 49.05 N

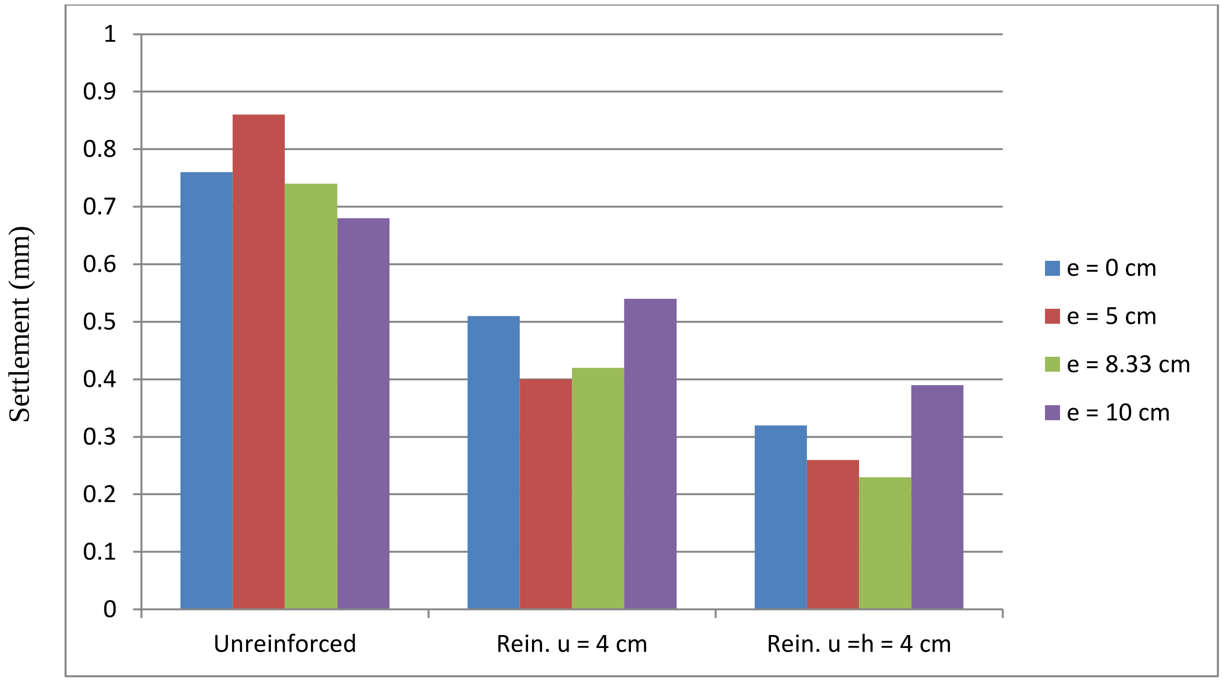

Figure 4.44: Load – Settlement variation for reinforced & unreinforced soil

This is the comparison of settlement for unreinforced, reinforced at u = 4 cm and reinforced at u = 4cm & h = 4cm case for different values of eccentricities at a constant load of 49.05 N.

In this case maximum settlement is observed in case of unreinforced case and by providing a single reinforcement layer, settlement decreases. Further by increasing the number of layer of reinforcement layer to two, the settlement decreases more.

CHAPTER 6

CONCLUSION & FUTURE SCOPE

6.1 Conclusion

From the present study, the following conclusions can be drawn:

1. In case of eccentric loading on unreinforced soil, settlement increases when load moves from e = 0 cm to e = 5 cm whereas due to further increment in load eccentricity from e = 5 cm to e = 8.33 cm and beyond, settlement decreases. In this case settlement increases when eccentricity is somewhat half of the kern distance with respect to origin and when eccentricity increases to kern boundary and outside the kern area, settlement increases.
2. In case of eccentric loading on soil reinforced with single layer of geogrid reinforcement,

a. the optimum depth of placing the reinforcement is obtained to be u = 4 cm. At u = 4 cm minimum settlement is observed for different values of eccentricities.

b. the settlement observed is less than the unreinforced case.

c. For u = 2 cm, settlement decreases when load moves from e = 0 cm to e = 5 cm and on further increase in eccentricity from e = 5 cm to e = 8.33 cm and beyond, settlement increases. In this case when eccentricity is near the half of kern boundary, settlement is found to be less than concentric case while when load is on kern boundary and beyond it, settlement increases more than concentric case.

d. For u =4 cm, settlement decreases when load moves from e = 0 cm to e = 5 cm and on further increase in eccentricity from e = 5 cm to e = 8.33 cm the settlement slightly decreases and beyond it, settlement increases. In this case when eccentricity is with-in the kern boundary, settlement is found to be less than concentric case while when load is outside the kern boundary, settlement increases more than concentric case.

e. For u = 7cm, settlement increases when load moves from e = 0 cm to e = 5 cm and on further increase in eccentricity from e = 5 cm to e = 8.33 cm the settlement slightly increases and beyond the kern boundary settlement decreases. In this case when eccentricity increases up-to kern boundary settlement increases and when load is outside the kern boundary, settlement decreases more than concentric case.

3. In case of eccentric loading on soil reinforced with two layer of geogrid reinforcement,

a. the first layer of reinforcement is placed at u = 4 cm as this is the optimum depth of placing on which the settlements are minimum and for the second layer of reinforcement the optimum depth is also 4 cm.

b. the settlement observed with two layers of reinforcement is less than single layer & unreinforced case.

c. For u = 4 cm & h = 2 cm, settlement decreases when load moves from e = 0 cm to e = 5 cm and on further increase in eccentricity from e = 5 cm to e = 8.33 cm the settlement further decreases and beyond the kern boundary settlement increases more than the concentric case. In this case when eccentricity increases upto kern boundary settlement decreases and when load is outside the kern boundary, settlement increases more than concentric case.

d. For u = 4 cm & h = 4 cm and u = 4 cm & h = 7 cm, the variation in settlement observed is same as in previous case for u = 4 cm & h = 2 cm.

4. By placing the reinforcement settlement decreases with respect to unreinforced case and by placing two layers of reinforcement, the settlement observed is less than single layer of reinforcement.

6.2 Future Scope

The present study pertains to the study of settlement when a uniformly increasing load is applied on it. In this project unreinforced case, single reinforced case and double reinforced case are studied. The future research work should address the below mentioned points:

- The present work can be extended with gradually increasing load
- The present work can be extended with more than two layers of reinforcement.
- The present work can be extended for different sizes of foundation at different depth of embedment.
- Large scale study should be carried out to validate the present developed settlement results.

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