Experimental Studies on Reinforced Granular Bed overlying Sand Layer

Badooru Durga Prasad

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Department of Civil Engineering

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Durga Prasad

(Signature)

Badooru Durga Prasad

(Student Name)

CE13M1015

(Roll No)

Approval Sheet

This thesis entitled "Experimental Studies on Reinforced Granular Bed overlying Sand Layer" by Badooru Durga Prasad is approved for the degree of Master of Technology from IIT Hyderabad.

Dr. B. Venkatesham
Assistant Professor,
Department of Mechanical Engineering,
Indian Institute of Technology Hyderabad

Dr. S. Sireesh
Associate Professor
Department of Civil Engineering
Indian Institute of Technology Hyderabad

Dr. B. Munwar Basha
Assistant Professor
Department of Civil Engineering
Indian Institute of Technology Hyderabad

Dr. B. Umashnakar
Assistant Professor
Department of Civil Engineering
Indian Institute of Technology Hyderabad

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Dedicated to

My Guide

Abstract

Use of reinforcement is one of the oldest and widely used methods to improve the response of soil due to loads acting on it. Apart from the use of reinforcement, placing a granular bed over the poor soil is also commonly practiced ground improvement techniques. In the present study, the effect of combination of use of reinforcement and replacement of existing poor soil with a competent soil layer on the load - settlement response is studied. Two types of reinforcement- geogrid and road mesh- were used and a 100mm thick aggregate layer was laid over sand to study the improvement in the load carrying capacity. A large-scale test chamber of size equal to 1 m x 1m x 1 m was used to perform the experiments. The loading plate consisted of 200 mm width square footing and an actuator of 10T capacity was used to apply the load in a displacement-rate controlled mode. Parameters varied in this study were (a) depth of the reinforcement, (b) width of the reinforcement, (c) relative density of underlying soil layer, (d) type of reinforcement, and (f) the number of layers of geogrid reinforcement. The improvement in the load carrying capacity is quantified in terms of load improvement factor.

Nomenclature

SP – Poorly-graded sand

SEM – Scanning electron microscope

D_{10} – Effective particle size

D_{30} – Particle size corresponding to 30% finer

D_{60} – Particle size corresponding to 60% finer

s – Settlement of model footing

B – Width of model footing

h – Thickness of the overlying aggregate layer

u – Depth of first layer of reinforcement from the bottom of the footing

J – Stiffness of reinforcement

s/B – Settlement ratio

u/B – Optimum depth ratio

D_R – Relative density of underlying soil layer

L_r – Width or length of the reinforcement

LIF – Load Improvement Factor

C – Apparent cohesion of soil

ϕ – Friction angle of soil

Contents

Chapter 1

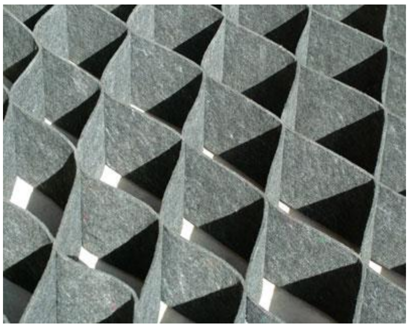
Introduction

1.1 Overview

Rise in land cost and decrease in the availability of proper construction sites resulted in choosing sites with poor soil conditions that possess low shear strength and stiffness characteristics. For sites with poor soil condition, deep foundations are preferred in order to transfer the structural load onto the deep firm stratum. However, these methods are costly. In search of cost-effective methods, one can prefer shallow foundations. But shallow foundations are laid on top of existing soil resulting in transferring the structural load mainly to the top soil stratum. If the top existing soil is weak then it will result in failure of the structure. In order to avoid this problem, modifications in the foundation soil can be done. Conventional modification techniques used are increasing the dimensions of the footing or replacing the existing soil with stronger material or a combination of both. In addition to these methods, the most popular and cost effective technique is use of geosynthetics to reinforce the foundation soil. This can be done by either reinforcing the existing poor soil or replacing the poor soil with stronger granular fill with the inclusion of reinforcement in it. Thus, the resulting composite soil mass will improve the load carrying capacity and also helps in providing better pressure distribution on the top existing soil. Depending upon the existing field conditions and structural load, one can select most appropriate and cost effective technique to improve the soil condition.

Over last few decades the use of geosynthetics as soil reinforcement became very popular. The various functions of geosynthetics include separation, reinforcement, filtration or drainage. The most common types of geosynthetics are Geotextiles, Geogrids, Geonets, Geocells, Geofoams and Geocomposites. Figure 1.1 shows various types of geosynthetics.

(a)



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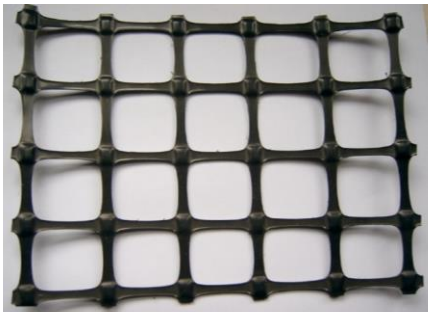
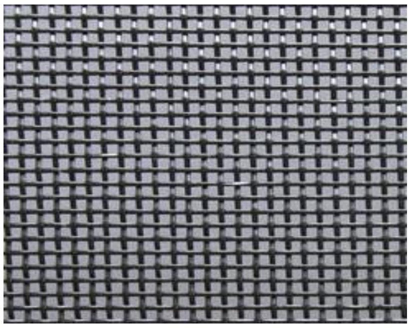
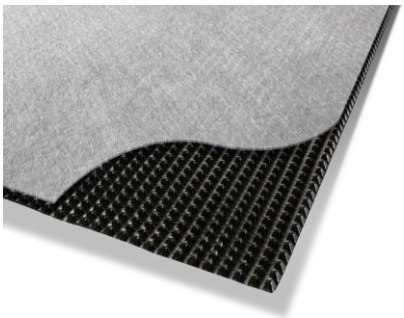


(c)



(d)

(e)



(f)

Fig 1.1: Various types of geosynthetics:

(a) Geonet (Source: http://www.hznai.com/products/Geonet)

(b) Geocell (Source: http://www.kometa.by/Geogrid.htm**)**

(c) Geogrid (Source**:** http://www.terrafixgeo.com/products/geogrids)

(d) Geotextile (Source: http://www.bonartf.com/en/x/61/hf---high-flow-wovengeotextiles**)**

(e) Geofoam (Source: http://versatechinc.net/products/geofoam/tucker-tunnel-fill/)

(f) Geocomposite (Source: http://www.abgltd.com/products/pozidrain.html)

The present study involves investigating the improvement in the load carrying capacity of the foundation soil by the inclusion of reinforcement. Accordingly, experiments were conducted by reinforcing the poor soil and also by placing a reinforced granular bed over the poor soil. The parameters varied in this study includes depth of the reinforcement, width of the reinforcement, relative density, type of reinforcement and the number of layers of reinforcement. The improvement in the load carrying capacity of the foundation soil is quantified by using a non-dimensional parameter- load improvement factor (I_f)- defined as ratio of load carrying capacity of the reinforced soil foundation at a given settlement to the load carrying capacity of the unreinforced soil foundation at the same settlement.

1.2 Objectives of the study

The objective of this research includes

1. To determine the optimum depth of reinforcement when a single layer of reinforcement placed in soil only. The optimum depth is defined as the depth of the reinforcement below the bottom of the footing at which the composite soil mass (soil with reinforcement) results in the maximum load carrying capacity).
2. To determine the optimum depth of the reinforcement when single layer of reinforcement was placed in layered system with a strong granular fill overlies a poor soil.
3. To determine the effect of width of the reinforcement by varying the reinforcement width as a function of footing width in the poor soil and stronger granular fill.
4. To determine the effect of relative density on the load carrying capacity of various test configurations considered in the study. The relative density, D_R, equal to 50% and 70% were chosen for the underlying soil layer.
5. To determine the effect of type of reinforcement (viz., geogrid and road mesh).
6. To determine the effect of number of reinforcement layers in overlying aggregate layer.

1.3 Organization of the study

This dissertation is divided into six chapters. The following is a brief summary of the contents in each chapter.

Chapter 2 presents an extensive literature review related to experimental study, numerical analysis and analytical study of reinforced soil foundation.

Chapter 3 gives the material properties used in the experiments and also various tests performed on the materials.

Chapter 4 presents the experimental test program including test setup and organization of the test series for conducting the experiments.

Chapter 5 provides the results and discussions of experimental study.

Chapter 6 outlines the conclusions drawn from this research work.

Chapter 2

Literature Review

2.1 Introduction

Over the last few decades, extensive research works have been carried out to investigate the behavior of reinforced soil foundations. All these studies indicate that the use of reinforcement can increase the bearing capacity and reduce the settlement. Various researchers evaluated the benefits of using reinforcement through bearing capacity ratio (BCR), which is defined as the ratio of the bearing capacity of the reinforced soil foundation at a given settlement to that of the unreinforced soil foundation at the same settlement. Several research works aimed at varying different parameters that would contribute to the BCR value. The results of the experimental studies available in the literature showed that better improvements were obtained when the reinforcement is placed within a certain depth beyond which no significant improvement will occur. The parameters studied by researchers include:

1. Number of layers of reinforcement (N)
2. Spacing of the first layer reinforcement (u)
3. Total depth of the reinforcement (d)
4. Vertical spacing between reinforcement (h)
5. Width or length of the reinforcement (L)
6. Type of reinforcement and stiffness of reinforcement (J)
7. Soil type
8. Embedment depth of the footing (D_f)
9. Shape of the footing

Fig 2.1 shows a typical geosynthetic reinforced soil foundation and the descriptions of various geometric parameters.

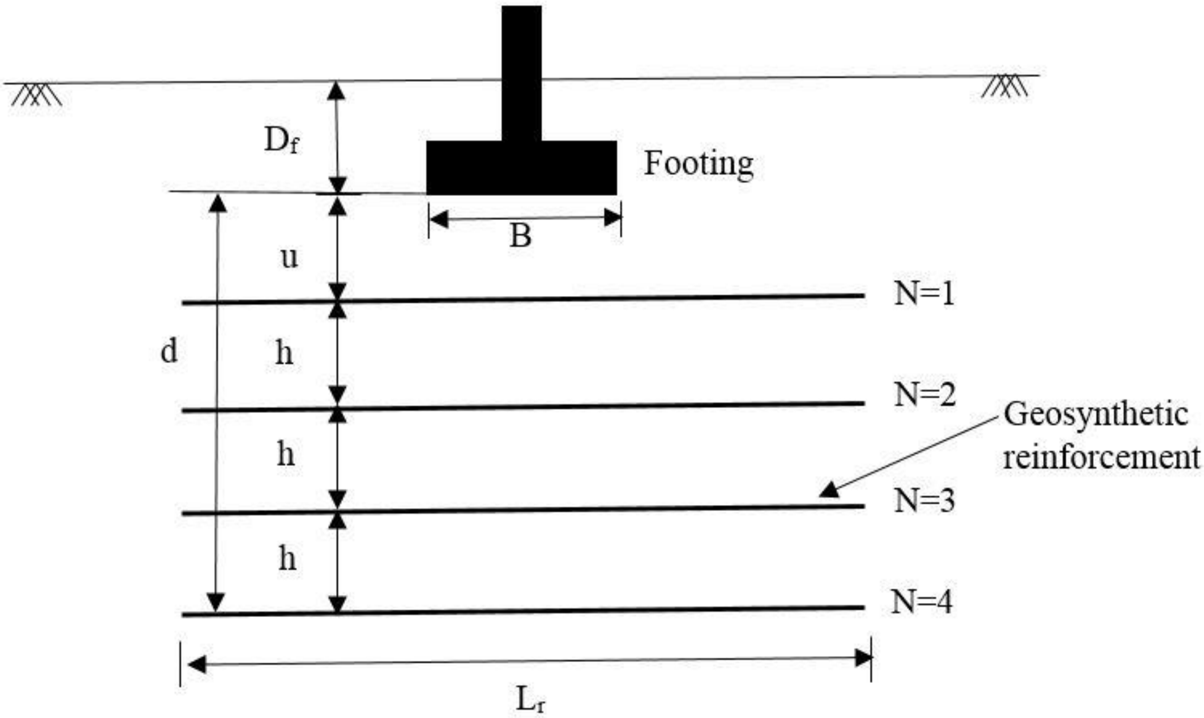


Fig 2.1 Geosynthetic reinforced soil foundation

The geosynthetic-reinforced soil foundations have been used in the civil engineering constructions such as houses, bridges, dams, retaining walls, etc. When the geosynthetics are placed in the soil, they impart additional shear strength to the soil mass. This is mainly because as the soil cannot take tensile load, the geosynthetic materials placed in the soil mass will act as tensile members which in turn helps in improving the strength and controlling the settlements. To be effective, the reinforcement placed in the soil must intersect the potential failure surfaces in the soil mass. Strains in the soil mass generate strains in the reinforcements, which in turn, generate tensile loads in the reinforcement. These tensile loads act to restrict soil movement and thus impart additional shear strength. This results in composite soil/reinforcement system having significantly greater shear strength than the soil mass alone.

2.2 Background work

In the last few decades, several researchers have extensively worked to improve the bearing capacity and to reduce the settlements with the inclusion of geosynthetics in the foundation soil.

Fragaszy et al. (1984) conducted a series of laboratory model tests to investigate the influence of soil density and reinforcement length on the load-settlement behavior. The model tests were conducted in a rectangular fiberboard box with inside dimensions of 0.56 m (width), 1.22 m (length), and 0.36 m (height) and the model footing used was 7.6 cm x 15.2 cm rectangular steel plate. Tests were performed on both unreinforced and reinforced subgrades prepared with relative densities of 51%, 61%, 71%, 80% and 90%. All the model test were conducted with three layers of reinforcement keeping the top reinforcement layer spacing as 2.54 cm and vertical spacing between the reinforcement is kept as 2.54 cm.

The test results indicated that amount of improvement in the bearing capacity was dependent on two design criteria: the bearing capacity at a settlement ratio (s/B) of 0.04 and 0.10. Settlement ratio (s/B) is defined as the ratio of the settlement (s) to the width of the footing (B). At a settlement ratio of 0.04, the bearing capacity ratio (BCR) increased from 1.2 to 1.5 with increase in the relative density from 51% to 90%, while at a settlement ratio of 0.10, BCR is almost constant (1.6 to 1.7) no matter how much the soil density was. The magnitude of BCR increased from 1.3 to 1.7 with increasing the length of the reinforcement from 3.0 to 7.0 B, after which the improvement became negligible.

Yetimoglu et al. (1994) performed both laboratory tests and numerical analysis to investigate the bearing capacity of rectangular footing on geogrid-reinforced sand. The model tests were conducted in a 70 cm (width), 70 cm (length) and 100 cm (depth) steel box and the model footing used was 127 mm (length) and 101.5 mm (width) rectangular steel plate. The parameters investigated in this study were depth of the first layer of reinforcement, vertical spacing of reinforcement layers, number of reinforcement layers, and the size of reinforcement.

Based on both the laboratory model tests and numerical analysis, the following findings were reported: (1) the optimum spacing ratio (u/B) of top reinforcement layer was found to be around 0.3 and 0.25 in reinforced sand when single layer and multi-layer reinforcement were used, (2) the optimum vertical spacing ratio (h/B) between the reinforcement layers was determined as 0.2 to 0.4 depending up on the number of layers of reinforcement, (3) the influence depth was approximately 1.5B and the effective width ratio (L_r/B) of reinforcement was 4.5, and (4) an increase in the reinforcement stiffness beyond a certain limit would result in insignificant increase in the BCR value.

Das et al. (1994) conducted experiments to study the effect of width of the strip footing on the bearing capacity of the reinforced soil foundation. Six different model strip footings were used for this study having widths of 50.8 mm, 76.2 mm, 101.6 mm, 127 mm, 152.4 mm and 177.8 mm and length was kept as 304.8 mm. All the tests were performed in a box having dimensions as 1.96 m long, 0.305 m wide and 0.914 m deep. Tests were performed on reinforced and unreinforced foundations with soil prepared at relative densities equal to 55%, 65% and 75%.

Test results indicated that as the width of the foundation increases even though there is increase in the bearing capacity of the unreinforced and reinforced soil foundation, there is decrease in the bearing capacity ratio (BCR). When the footing width is equal to or greater than 130 to 140 mm, the magnitude of BCR was practically constant irrespective of the relative density.

Adams et al. (1997) performed large-scale model tests on geosynthetic reinforced soil foundations. The tests were conducted in a 6.9 m long, 5.4 m wide, and 6 m deep concrete box. The various dimensions of the square footings used are 0.3 m x 0.3 m, 0.46 m x 0.46 m, 0.61 m x 0.61 m and 0.91 m x 0.91 m. The parameters investigated in this study include number of reinforcement layers, spacing between reinforcement layers, and the top reinforcement layer spacing.

Test results indicated that there is significant increase in the bearing capacity and the ultimate bearing capacity ratio increased to 2.6 when the number of geogrid layers was kept as three. However, the amount of settlement required for this improvement is about 20 mm and may be unacceptable on some foundation applications. Test results also showed that when the top layer reinforcement spacing is maintained less than 0.25B, there were beneficial effects of reinforcement even at low settlement ratio (s/B).

DeMerchant et al. (2002) conducted an experimental study of plate load tests on geogrid reinforced light weight aggregate. Tests were performed in a 2.2 m wide, 3.2 m long and 1.6 m deep laboratory test pit. A 305-mm-diameter circular footing was used in the test. Parameters varied in this study were soil density (compact and very loose), width of the reinforcement, top reinforcement layer spacing, number of reinforcement layers, and tensile strength of the reinforcement.

Test results indicated that the compaction of the aggregate results in a considerable increase in the subgrade modulus values. The subgrade modulus values for lightweight aggregates are 8 MN/m^3 (loose) and 38 MN/m^3 (compact). The effective width of the reinforcement was 4B and the influence depth of the reinforcement is 1B. At smaller displacement (displacement less than 6 mm), stiffer geogrid is less beneficial than flexible geogrid.

Shin et al. (2002) conducted small-scale laboratory model tests to determine the bearing capacity of strip foundation on the geogrid-reinforced sand. The model tests were conducted in a 174 mm wide, 1000 mm long and 600 mm deep steel box. The dimensions of the model strip footing used were 172 mm long, 67 mm wide and 77 mm thick. Tests were conducted by preparing the samples at a relative density of 74%. Parameters varied in this study were embedment depth of the footing keeping the top layer spacing ratio (u/B), vertical spacing ratio between reinforcement (h/B) and width ratio of reinforcement (L_r/B) as constant. The model test results indicated that the influence depth for placing reinforcement was 2B and also concluded that for a given reinforcement-depth ratio, u/B, h/B and L_r/B, the bearing capacity ratio with respect to the ultimate load increases with the increase in the embedment ratio (D_f/B).

Boushehrian and Hataf (2003) performed both experimental studies and numerical analysis to investigate the bearing capacity of model circular and ring footings on reinforced sand. Tests were conducted in a cylindrical tank with 1 m diameter, 1 m height and 4 mm thick. Parameters varied in this study includes depth of the first layer of reinforcement, stiffness of the reinforcement, vertical spacing between the reinforcement layers and number of reinforcement layers.

Based on both the laboratory model tests and numerical analysis, the following findings were reported: as the depth of the topmost layer increases there is decrease in the improvement of bearing capacity and if the depth of the topmost layer is placed beyond 0.4B then there is continuous decrease in the BCR value. As the stiffness of the reinforcement increases beyond 300 kN/m, there is no effect of stiffness on the BCR both in ring and circular footings indicating that choosing a stiffer reinforcement will not always results in higher BCR value.

Laman et al. (2003) performed model studies of ring foundations on geogrid-reinforced sand. Tests were conducted in a tank whose dimensions are 700 mm (length), 700 mm (width) and 700 mm (depth). Tests were performed with five different model ring footings, one was circular and the remaining four footings with inner radius (r) to outer radius (R) varied as 0.3, 0.53, 0.65 and 0.75. Parameters varied in the tests were optimum ring width ratio (r/R), effect of depth of first layer reinforcement, number of reinforcement layers and effect of reinforcement length on the bearing capacity of ring foundation.

Based on the laboratory model tests the following findings were reported: (1) the optimum ring width ratio (r/R) was 0.3B, (2) the optimum location of the first layer of reinforcement to obtain maximum benefit was found to be 0.3B, (3) the optimum number of geogrid layers is 4, (4) the optimum length of the reinforcement is 3B, beyond which there was no significant improvement in the bearing capacity ratio value and (5) the effective depth of reinforcement zone was 1.2 times the width of the footing.

Yoon et al. (2004) conducted experiments using waste tires as reinforcement to investigate the improvement in the bearing capacity. Treads and sidewalls of tires were combined to make the tire mat. Plate load tests were conducted to study the reinforcing effect by varying the parameters such as relative density, embedded depth, number of layers of reinforcement and size of the mat. Relative densities of 40%, 55% and 70% of sand was prepared to investigate the effect of soil improvement by tire mat. For the experimental study, plate load tests were carried out in a test chamber having dimensions of 2 m wide, 2 m long and 1.5 m deep. The width of the loading plate used for the experiment was 350 mm. Test results indicated that the sand reinforced with waste tires had more than twice the bearing capacity of loose sand. The improvement in bearing capacity due to the tires decreased with increase in the relative density. The settlement reduction due to tire reinforcement with combination of treads and sidewalls was as much as about 70% for loose sand and 34% for dense sand. The size of the tire mat should be at least five times the load plate widths for full improvement of sands.

Kumar et al. (2007) conducted experiments to investigate the bearing capacity of the strip footing resting on reinforced layered soil. Parameters varied in this study were number of layers of reinforcement, thickness of top layer of soil and thickness of bottom layer of soil. Dimensions of the strip footing used for the test were 0.15 m x 1.19 m. The inside dimensions of the tank were fixed as 1.8 m long, 1.19 m wide and 1.2 m deep. Depth of the topmost layer of reinforcement and vertical spacing between adjacent layers of reinforcement were fixed as 0.3B and 0.2B respectively. Test results indicated that reinforcing the subsoil after replacing the top layer of soil with a well-graded soil is beneficial as the mobilization of soil-reinforcement frictional resistance will increase. There was increase in the ultimate bearing capacity (3 to 4 times) for a well-graded reinforced sand (2-4 layers of reinforcement) with thickness equal to width of the footing was laid over the poor soil.

Basudhar et al. (2007) performed experiments on circular footing resting on geotextile-reinforced sand bed. Analytical and numerical analyses were also conducted and compared with the experimental observations. The parameters varied in the study were size of the footing (30, 45 and 60 mm diameters); number of reinforcement layers, reinforcement placement pattern, bond length and the relative density (45%, 73% and 84%). All the experiments were performed in a tank having dimensions 0.44 m x 0.44 m x 0.21 m. Dry Ganga sand and woven geotextile were used for the experiments.

Test results indicated that the stress-settlement plots corresponding to the unreinforced case shows local shear failure whereas for reinforced case it shows linearly elastic-plastic failure. For relative densities of 73% and 84% there was sharp peak stresses where the failure occurs and beyond this point strain softening was observed. But for a relative density of 45% there was no strain softening. Two reinforcement patterns were adopted; one was rectangle and the other was circular and concluded that rectangular shape of reinforcement was more preferable compared to circular shape reinforcement for overall strength and settlement improvements. To compare the experimental results with analytical methods, method proposed by Janbu et al. (1956) was adopted. For numerical analysis, FLAC software was used. It was observed that with increase in number of reinforcement layers there was decrease in the settlement rate but there was a substantial increase in the bearing capacity ratio value comparatively.

Latha et al. (2009) performed laboratory tests and numerical simulations to investigate the bearing capacity of the square footing resting on the geosynthetic reinforced soil foundation. Parameters varied in this study were type and tensile strength of reinforcement, number of reinforcement layers, layout and configuration of the reinforcement. Experiments were performed in a tank of dimensions 900 x 900 x 600 mm and the width of the square footing was 150 mm. The sample was prepared with a relative density of 70%. Numerical analysis was performed by using Flac 3D software.

Based on both the laboratory model tests and numerical analysis the following findings were reported: (1) the effective depth of the reinforcement zone is twice the width of the footing, optimum spacing of reinforcing layers was 0.4B, (2) optimum width of the reinforcement was 4B and (3) layout and configuration plays a major role in increasing the bearing capacity than the tensile strength of the reinforcement.

Zidan (2012) performed a series of finite element analysis to investigate the behavior of square footing resting over reinforced sand. The parameters that are varied in the study were number of geogrid layers, location of top layer of reinforcement, spacing between the reinforcement. PLAXIS 2D software was used for the numerical analysis. The model geometry was simulated by means of an axisymmetric model in which the circular footing and the load were positioned along the axis of symmetry. Both the soil and the footing were modelled with 15-noded elements. The depth and width of the model were taken as 15 times the diameter of the footing. Hardening soil model was used in the study to simulate the non-linear behavior of the sand.

Numerical analysis results indicated that the depth of topmost layer plays an important role in the improvement of the reinforced soil behavior and reported that the optimum depth was 0.19 times the width of the footing. The effect of geogrid was negligible when the ratio of depth of first layer to the footing diameter is equal to 0.5. The improvement also increases with decrease in the spacing between the layers when the topmost layer is placed at 0.2. But when the topmost layer reinforced is placed beyond 0.3 then there was negligible effect of spacing between the layers.

Abu-Farsakh et al. (2013) performed six large scale field tests to investigate the behavior of foundations on geosynthetic-reinforced silty clay marginal embankment soil. Parameters varied in this study includes top reinforcement layer spacing, number of reinforcement layers, vertical spacing between layers, type of reinforcement, embedment depth (D_f) and shape of the footing. The model tests were conducted in a 1.5 m long, 0.91 m wide and 0.91 m deep steel test box. The model footings used in the tests were 25.4 mm thick steel plates with dimensions of 152 mm width (B) for square footing and 152 mm wide, 254 mm long for rectangular footing. Three types of geogrid and one type of geotextile were used as reinforcement in the tests.

Test results indicated that optimum top reinforcement layer spacing was 0.33B for the embedded square model footing (D_f/B =1.0) on geogrid reinforced sand. The influence depth of reinforcement was 1.25B regardless of the type of reinforcement and embedment depth. He also reported that geogrid reinforced soil foundations were performing better than geotextile reinforced soil foundations.

Yadu et al. (2013) performed experiments to investigate the effect of length of the geogrid layers on BCR of geogrid reinforced granulated blast furnace slag (GBS) overlaid on soft subgrade soil system. Experiments were conducted in a test tank whose dimensions were 1.8 m (length), 0.305 m (width) and 0.914 m (height). The model footing used for the test was made of a rigid steel plate with dimensions of 305 mm length, 76.2 mm width and 25.4 mm thickness. Soil sample was prepared with a relative density of 85% and the thickness of the GBS was 2B. Optimum depth of the first layer reinforcement and the vertical spacing between the reinforcement layers were maintained as 0.33B and the number of reinforcement layers used in the study was equal to 5. Different lengths of the reinforcement (L_r) used were 2, 4, 6, 8, 10 and 12 times the width of the footing (B).

Test results indicated that there was significant increase in the magnitude of the BCR value when the L_r/B ratio was kept as 4. There was no significant improvement in BCR value when L_r/B is kept beyond 4. Granulated blast furnace slag with geogrid (L_r/B of 2.0) increases the settlement reduction ratio (SRR, defined as the percentage reduction in the settlement due to stabilized case relative to the unstabilized case at a constant load) as 84% at ultimate bearing capacity of soil bed. There was no significant improvement in the SRR when L_r/B is kept beyond 2.

Elsaied et al. (2015) performed laboratory tests to investigate the influence of soil confinement on circular footing resting on a granular soil. Nine hollow cylinders with varying heights and diameters were installed around the footing model for soil confinement purpose. Parameters varied in this study were diameter, height and depth of the cylinder. Number, width and position of the geogrid layers were also investigated in the study. Three dimensional stiffened framed tank of inner dimensions 1 m (length), 1 m (width) and 0.6 m (depth) was used to perform the experiments and the diameter of the circular footing used in the study was 200 mm.

Test results indicated that if the diameter of the cylinder is kept same as that of footing diameter then the improvement is more since it behaves like a deep foundation (one unit). When the height of the confinement is increased then the soil behavior is enhanced since the footing load was transferred to the deeper layers. When the width of the geogrid is equal to 0.25D it was observed that there is improvement in bearing capacity around 7.5 times compared with the non-confining case. The optimum depth of the top layer reinforcement when single layer is used is 0.25 times the footing diameter and it was observed that increase in number of layers of the geogrid has little effect on the improvement.

Chapter 3
Material Properties

3.1 Introduction

This chapter presents the characteristics of the materials like sand, aggregate and reinforcement used in the research work. Various tests performed on the sand, aggregate and reinforcement are also presented.

3.2 Sand and Aggregate

Locally available river sand and aggregates were used for all the experiments. Grain-size distribution, the maximum unit weight, and the minimum unit weight of sand were obtained according to ASTM D422, ASTM D4253 and ASTM D4254 respectively. Figure 3.1 shows the grain-size distribution curve of sand, while Figure 3.2 shows the morphology of the sand particles captured at a magnification factor equal to 60X using Scanning Electron Microscope (SEM). Table 3.1 provides the physical properties of the sand used in the study. Sand was classified as poorly-graded sand (SP) as per the Unified Soil Classification System (USCS). The maximum unit weight of sand reported was obtained from the vibratory method. Locally available aggregate was used as strong granular fill whose size lies between 4.75mm and 9.5mm.

Table 3.1: Physical properties of river sand used in the study

Parameter	**Value**
D_{10} (mm)	0.29
D_{30} (mm)	0.48
D_{60} (mm)	0.7
Coefficient of curvature, C_c	1.13
Coefficient of uniformity, C_u	2.4
Specific gravity, G_s	2.65
Maximum unit weight (kN/m^3)	17.8
Minimum unit weight (kN/m^3)	15.1

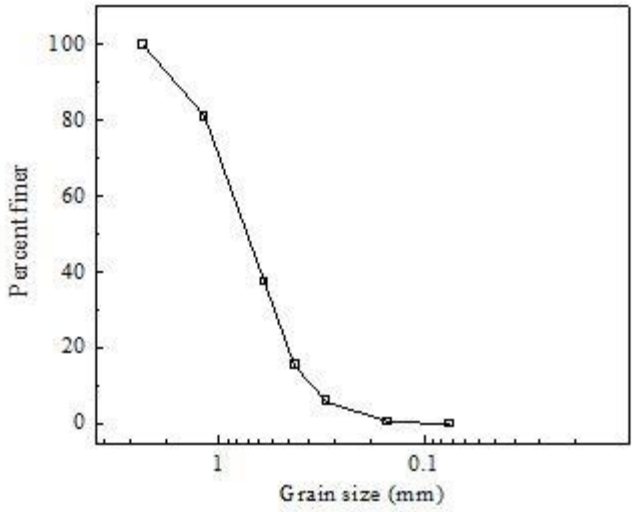


Fig 3.1: Grain-size distribution of the river sand

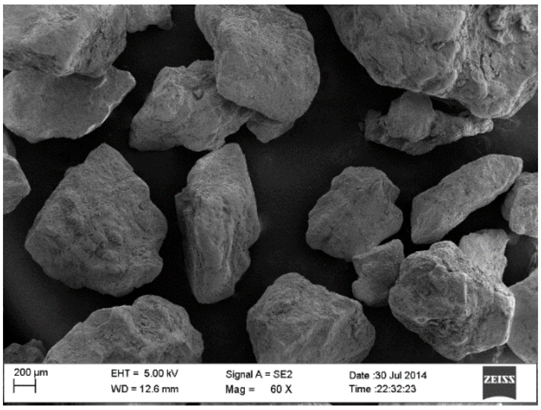


Fig 3.2: Morphological image of river sand using SEM

3.2.1 Direct shear test

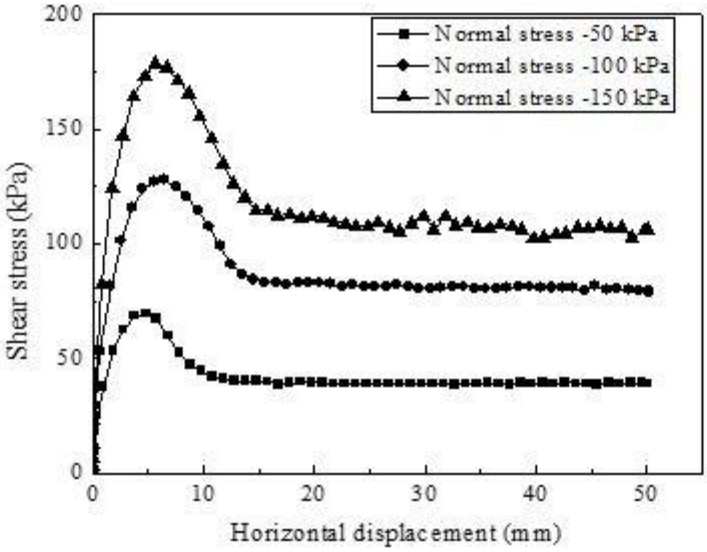
To determine the shear strength parameters of sand and aggregate, large-scale direct shear test apparatus was used. The inner dimensions of the direct shear test box were equal to 300 mm x 300 mm x 200 mm (Fig 3.3). To achieve the relative density equal to 70%, raining technique was adopted. Firstly, calibration studies were done and fall height equal to 20 cm was found to provide a desired relative density equal to 70%. In the case of aggregates, the shear strength parameters were determined after the sample was prepared using compaction technique. Compaction was done in four layers. Number of blows required for the compaction of the aggregate was obtained based on energy imparted to the aggregate sample to the compaction energy of the standard compaction test, and was found to be equal to 192. Once the sample was prepared, the top surface was leveled using a spatula and leveling was checked using a level tube and then loading plate was placed over the compacted surface. The large-scale direct shear apparatus consisted of two load cells, horizontal load cell and vertical load cell, both having a maximum capacity of 44 kN. The maximum allowable horizontal displacement of the lower box was equal to 50 mm. Two linear variable displacement transducers (LVDT's) were used to measure the horizontal displacement of the lower box and the vertical displacement of the sample.

Fig 3.3: Large-scale direct shear apparatus

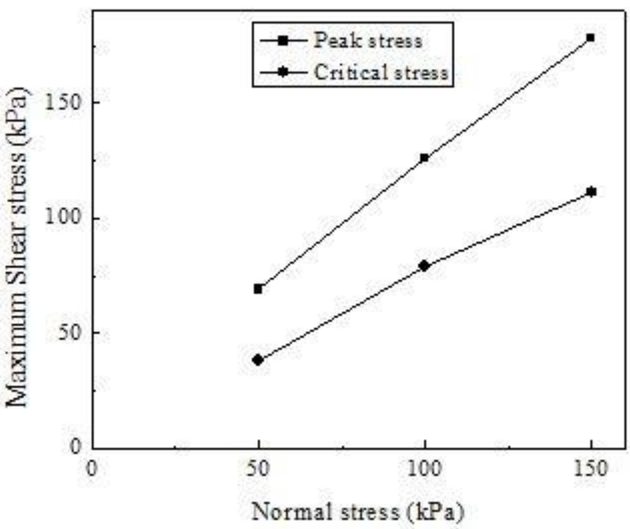
Once the sample was prepared and the loading plate was placed over the levelled top surface, the whole box assembly was pushed in position to the direct shear testing machine. Adjustments were made through to make sure that the center of the box matches exactly under the vertical load cell. The vertical load cell was lowered till it touches the ball placed at the center of the loading plate. After the load cell touches the ball the vertical movement was stopped and the bolt connections were done to ensure that the upper box did not move during shearing stage.

Two stages were involved in the direct shear test - consolidation stage and shearing stage. Once all the bolt connections were done, the sample was consolidated under a given normal stress for five minutes to facilitate the uniform application of required normal stress. The loading was done using the software. The shearing stage was started by removing the shear bolts. During the shearing stage, the upper box was held in position and the lower box was moved. The rate of displacement was 1 mm/min was maintained and shearing was continued till a horizontal displacement of the lower box reached a value equal to 50 mm.

Tests were repeated for different normal stresses to determine the strength parameters. Figs. 3.4a and 3.4b and Figs 3.5a and 3.5b show the variation of shear stress with the horizontal displacement and shear strength envelope of sand and aggregates respectively.



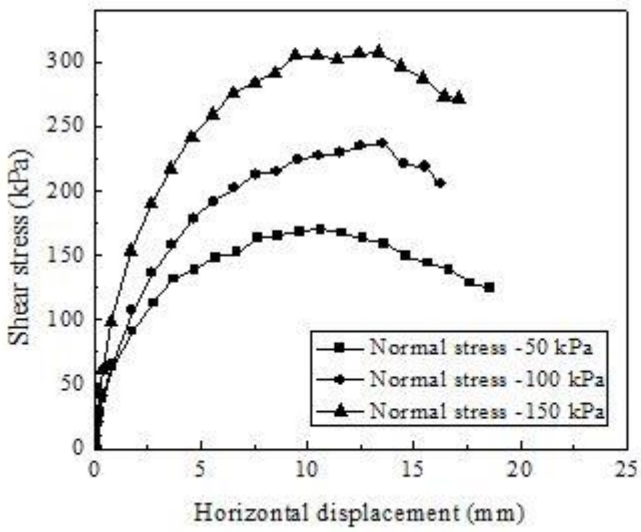
(a)



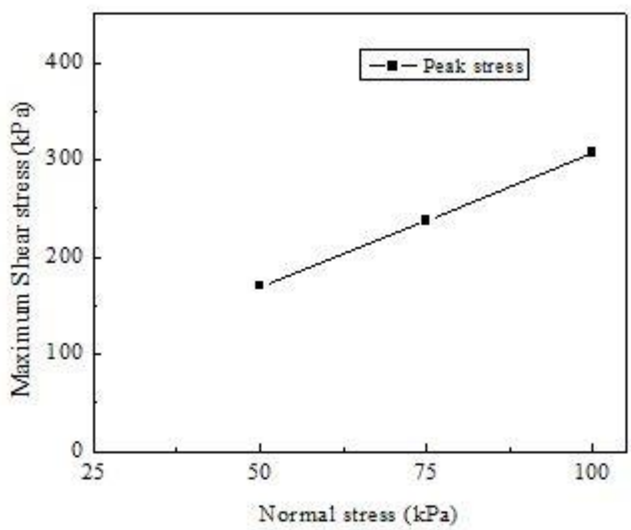
(b)

Fig 3.4: (a) Shear stress vs. horizontal displacement for unreinforced sand, and (b) shear strength envelope for unreinforced sand

Shear strength parameters of river sand, apparent cohesion and peak friction angle, were found to be 15 kPa and 47°. While the apparent cohesion and peak friction angle of aggregates were found to be 31 kPa and 70°.



(a)



(b)

Fig 3.5: (a) Shear stress vs. horizontal displacement for aggregates, and (b) shear strength envelope for aggregates

3.2.2 Interface direct shear test

Interface direct shear test was conducted using large-scale direct shear apparatus. The procedure and preparation of the sample was same but the only difference was placing the geogrid at the interface of two boxes (Fig 3.6).

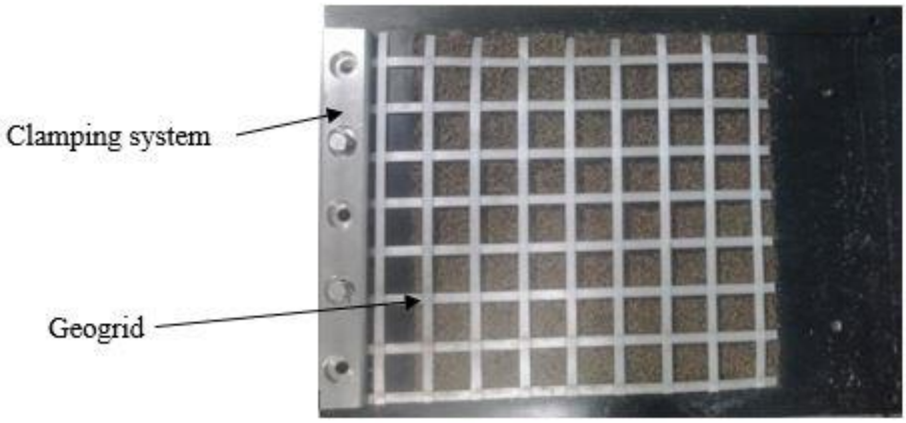
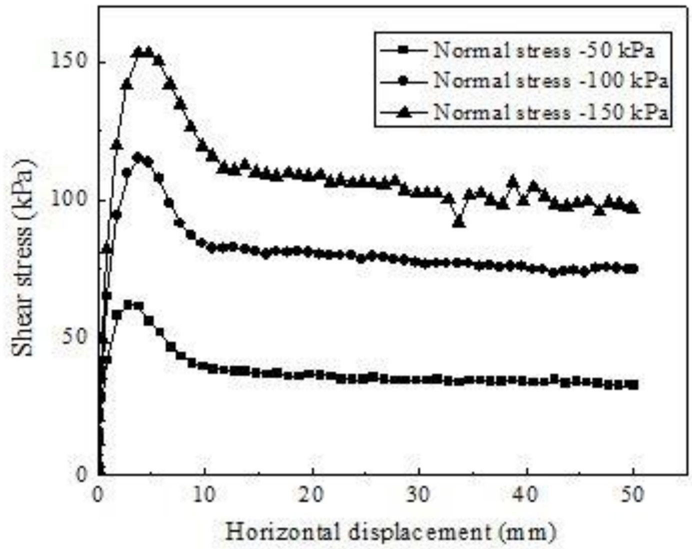
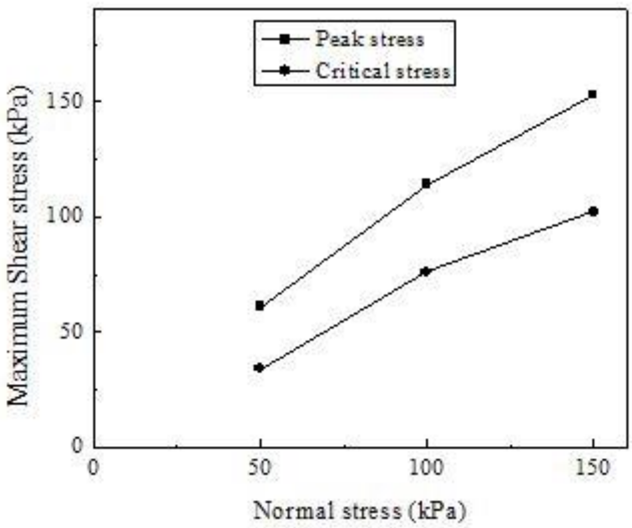


Fig 3.6: Clamping of geogrid on the top of lower shear box

Tests were repeated for different normal stresses to determine the interface shear strength parameters of the river sand. Figs 3.7a and 3.7b show the variation of interface shear stress with horizontal displacement and shear strength envelope of the reinforced sand. Interface shear strength parameters between geogrid and sand were found to be 17 kPa and 42°. The interface friction angle at peak state was found to be 10.6% lower than the friction angle of the unreinforced sand.



(a)



(b)

Fig 3.7: (a) Shear stress vs. horizontal displacement for reinforced sand, and (b) Interface shear strength envelope for reinforcement and sand

3.3 Reinforcement

Two different types of reinforcements were used for experiments. First type of reinforcement was geogrid and the second type was Road mesh

3.3.1 Geogrid

Geogrid is a geosynthetic material used for soil reinforcement. Geogrid reinforcement is made of stretched monolithic with polypropylene flat bars with welded junctions. Geogrid used in the experiments was manufactured by Secugrid. Fig 3.8 shows the biaxial geogrid used in the experiments and Table 3.2 provides the properties of the geogrid.

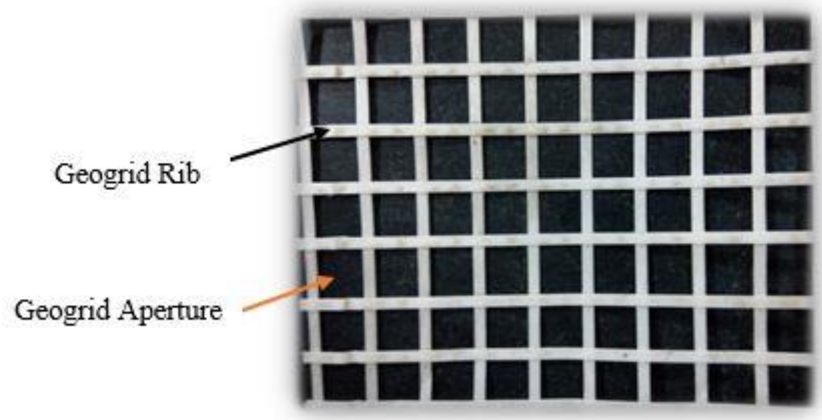


Fig 3.8: Biaxial Geogrid used in the experiments

Table 3.2: Properties of geogrid used in the study

Property	Value
Raw material	Polypropylene (PP)
Maximum Tensile strength (kN/m)	40
Tensile strength at 2% elongation (kN/m)	16
Tensile strength at 5% elongation (kN/m)	32
Aperture size (mm)	31 x 31
Rib thickness (mm)	0.85

3.3.2 Road mesh

Fig 3.9 shows the road mesh used in the experiments. Road mesh is a bi-directional high strength steel reinforcement. It is manufactured from double twisted hexagonal steel wire mesh with a transverse rod that is woven into the mesh. Transverse rod is woven into the mesh at approximately 175 mm interval. The distance between the axis of two consecutive twists is denoted as "D" (also called opening of the mesh). Road mesh used in the experiments has a opening distance (D) equal to 105 mm. The diameter of the steel mesh was equal to 2.5 mm and the diameter of the transverse rod was 5.3 mm. The wire used for the manufacture of the road mesh has a tensile strength in the range of 380-550 MPa. The dimensions of the road mesh used in the experiments were 800 mm x 800 mm.

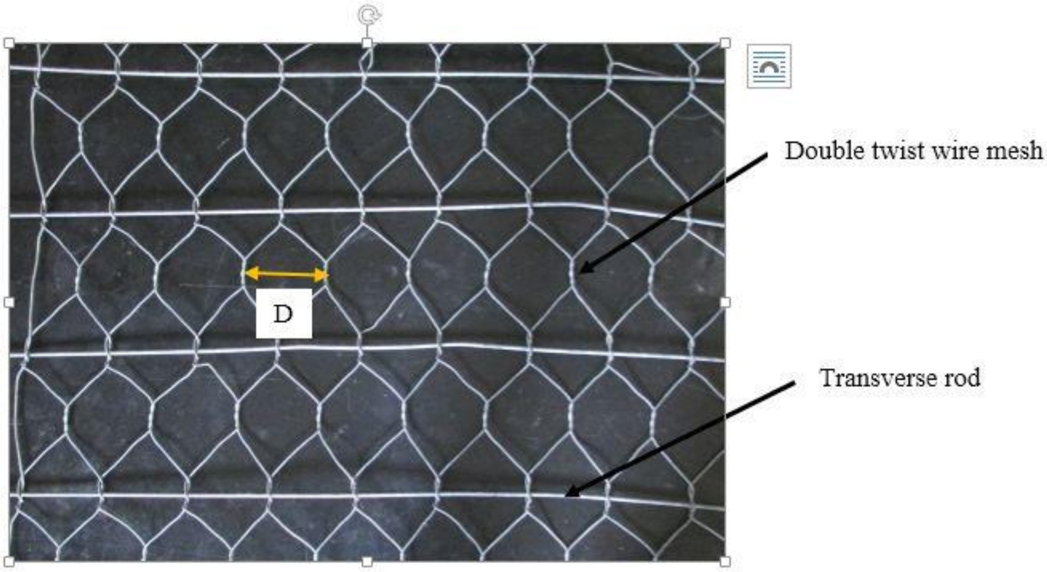


Fig 3.9: Road mesh used in the experiments

Chapter 4

Experimental work

4.1 Introduction

This chapter presents the experimental work conducted in the laboratory which includes instrumentation, sample preparation, test procedure, test series, and step by step procedure followed in the station manager software used for loading and the checklist used during the experiment. The photographs showing the test frame, compaction equipment, laying of geogrid during the sample preparation are also provided.

4.2 Instrumentation

In this study, a large-size tank equal to 1 m x 1 m x 1 m was used to study the load-settlement behavior of square-shaped model footing of size equal to 200 mm x 200 mm and 30mm in length, width, and thickness, respectively. Static load was applied on a loading plate using a computer-controlled, servo-hydraulic actuator. Loading to the plate was done using MTS Multi-Purpose Test Ware (MPT) with the help of hydraulic power unit (HPU) and hydraulic servo manifold (HSM). A 10T actuator was attached to a reaction frame of height equal to 3.5 m. Reaction frame facilitates the side ward movement of actuator. An in-built load cell and LVDT present inside the actuator unit was used to measure the load and the displacement of the loading plate. Figure 4.1 shows the photograph of the test frame used to perform the experiments.

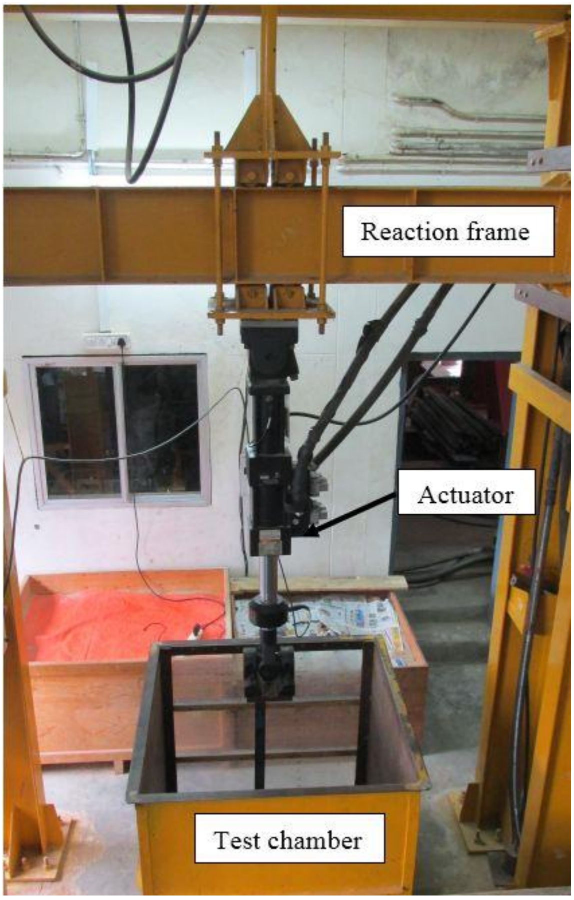


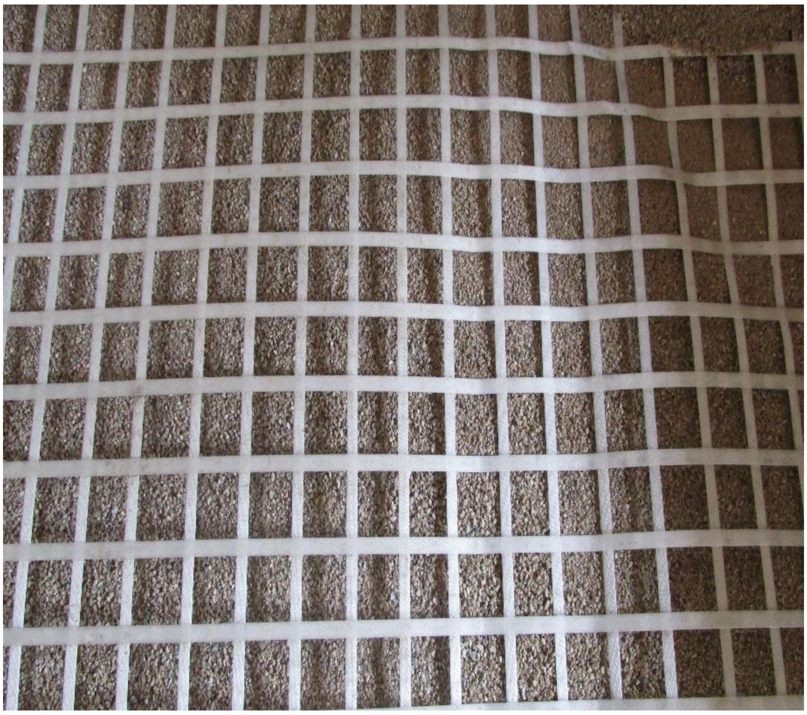
Fig 4.1: Photograph of test frame along with test chamber used for testing

4.3 Sample Preparation

Sand bed was prepared using vibratory method. Sand bed was prepared up to a depth of 800 mm. Total depth was divided into five layers. By knowing the unit weight and volume of each sand layer, weight of the sample to be taken was calculated. Calculated weight of the sand was taken and poured into the test tank. Sand was poured from negligible fall height to maintain uniform preparation of each layer before the compaction. Once the total weight of the sand required for each layer was poured into the test tank, the surface of the tank was levelled. After levelling, measurements (height) of the sand layer were taken. Measurements were taken at different locations in the test tank. Total nine measurements were taken before compacting the sand layer. Once the readings were taken, compaction of the sand layer was done using plate vibrator. To compact each layer, a pneumatically operated, impact-type piston vibrator manufactured by NAVCO (Model: BH-2 IGO) was used. The vibrator was connected to a pressure source through a pressure line, and a steel plate of dimensions equal to 300 mm x 300 mm x 10mm (in length, width and thickness) was bolted to the bottom of the vibrator. The weight of vibrator with the steel plate was about 18 kg (Fig 4.2).

Fig 4.2: Pneumatic vibrator used for compaction

Compaction of the sample was done by placing the plate vibrator over the sample and moving the plate vibrator throughout the test tank uniformly. One bar pressure (100 kPa) and 0.25 bar (25 kPa) pressure were used to compact the specimen to achieve relative densities equal to 70% and 50%, respectively. Once the compaction of sand layer was finished, top surface of the compacted sand surface was levelled and measurements were taken to check that each layer was compacted to the required relative density. For the case of aggregate layer overlying a sand layer, a 100 mm (h/B =0.5) thick layer of aggregates was laid over the prepared uniform sand bed and compacted. Square footing was placed over the prepared bed. Reinforcement was placed in the desired location calculated from the bottom of the footing. Figs 4.3a and 4.3b show photographs of geogrid placed in sand and aggregate layers, respectively.



(a)

(b)

Fig 4.3: Placing geogrid in (a) sand layer, and (b) aggregate layer

4.4 Test Procedure

Uniform bed was prepared in the test tank by using the plate vibratory method as discussed in section 4.2. Figures 4.4 and 4.5 shows the schematic views of the test bed in case of sand alone and layered system of sand with aggregate.

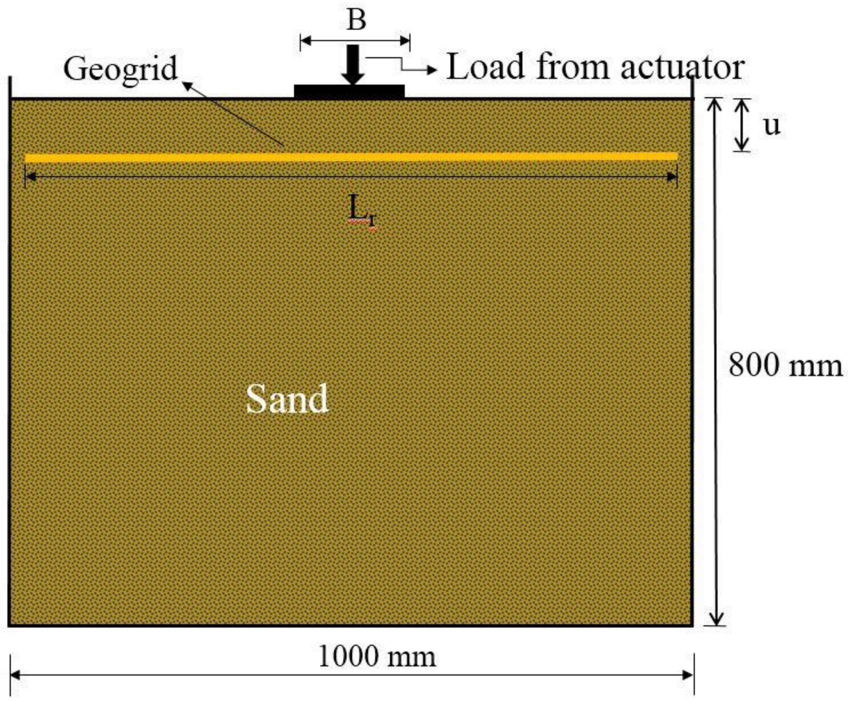


Fig 4.4: Schematic view of test bed in case of sand alone

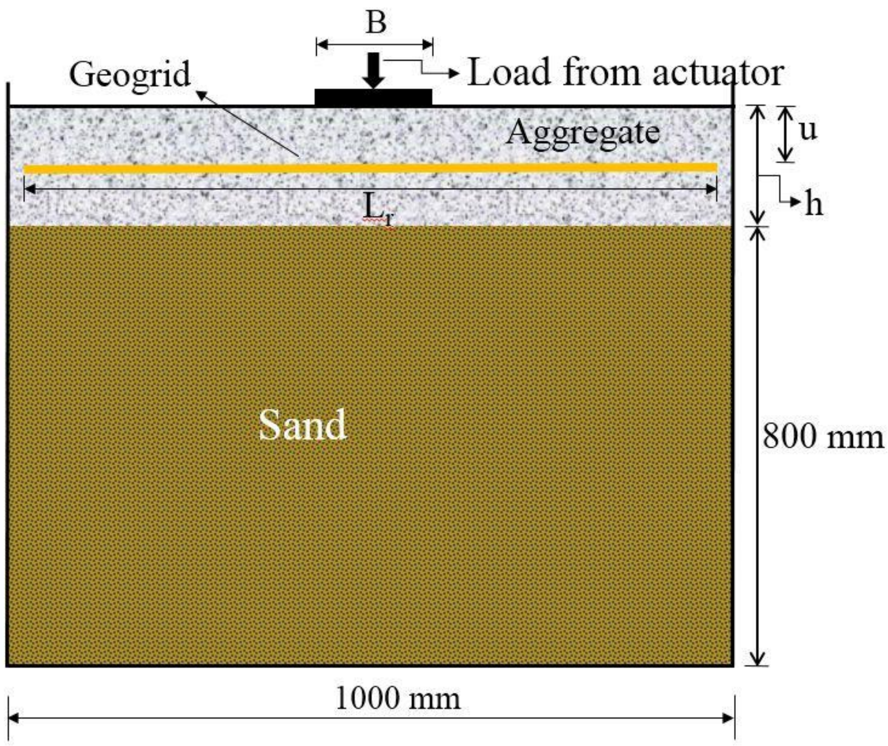


Fig 4.5: Schematic view of test bed in case of sand with aggregate

A square-shaped rigid plate of dimensions equal to 200 mm x 200 mm x 30 mm (length, width, and thickness) was placed on the prepared sand bed. Square footing was placed exactly at the center of the prepared bed. A provision was made on the square plate for seating of the plunger. A plunger was used to connect the actuator and square plate. After preparation of sample, plunger was fitted to the swivel of actuator and the actuator was moved down by enabling the manual command in the software (which will be discussed in Section 4.6). The actuator was moved till the plunger touches the ball placed at the center of the square footing. For few tests to monitor the surface settlement profiles, four LVDTs were placed over the surface of the prepared bed at a distance of 150 mm and 225 mm on either side of the footing. Figure 4.6 shows position of plunger over the square footing and the four LVDTs placed on either side of the square footing over the prepared bed.

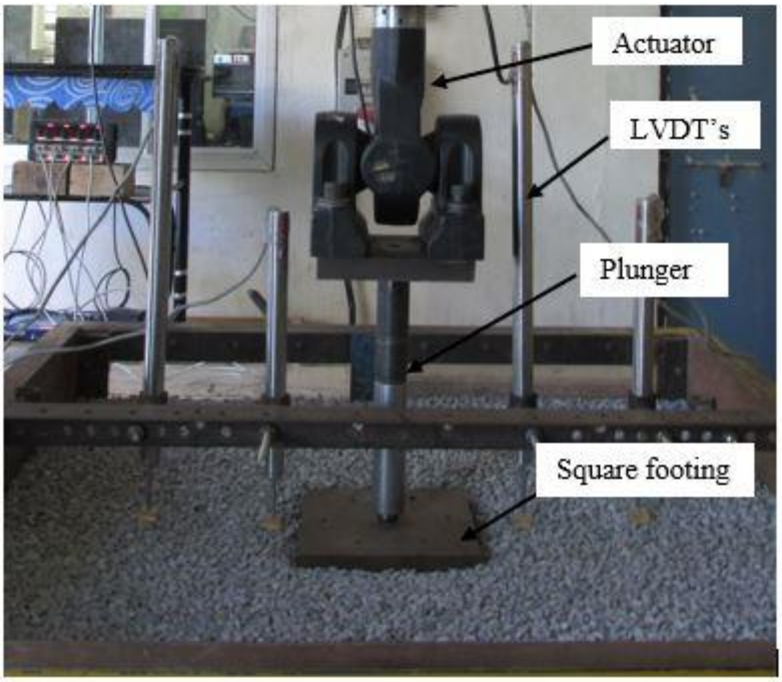


Fig 4.6: Placing of square footing and LVDTs over the prepared bed

Loading for all experiments was done in displacement-controlled mode. The rate of displacement was maintained as 1 mm/minute and test was performed until a vertical displacement equal to 50 mm was reached. MPT software records the values of load and settlement for every 10 seconds interval. Figs 4.7a and 4.7b show the test setup after allowing 50 mm of prescribed displacement in case of sand and sand with aggregate, respectively. Once the test was finished, load and settlement values were directly obtained from the software.

(a)

(b)

Figure 4.7: Photograph showing the test setup at the end of 50 mm prescribed displacement on (a) sand alone, and (b) aggregate overlying sand layer

4.5 Test Series

All the experiments were conducted in a sequential order by designing the test series (A – K). Each test series was designed to study the effect of different parameters. The most important parameters studied in this study were

a) Optimum depth of the first reinforcement layer in sand and in aggregate layer
b) Width of the reinforcement in sand and in aggregate layer
c) Relative density of sand layer
d) Type of reinforcement
e) Number of layers in aggregate layer

Test series A includes conducting experiments on square footing resting on unreinforced sand prepared with relative densities equal to 50% and 70%. Test series B aims at changing the thickness of the aggregate layer which is placed over the uniformly prepared sand bed.

Test series C and D were conducted by varying the depth of the reinforcement in sand and also aggregate layer laid over sand bed. The objective of this series of tests (i.e., C and D) is to find out the optimum depth of the reinforcement in sand and in aggregate layer laid over sand respectively.

Test series E and F were conducted by varying the width of the reinforcement in sand and also aggregate layer laid over sand bed. The objective of this series of these test series (i.e., E and F) is to find out the effect of width of the reinforcement in sand and in aggregate layer laid over sand respectively.

Test series G and H were conducted by preparing the sand bed and also aggregate layer overlaid on sand with relative density of 50%. The objective of this series of tests (i.e., G and H) is to find out the load settlement response in case of sand and also in aggregate layer laid over sand by placing the geogrid at optimum locations which were found in test series C and D.

Test series I and J were conducted by varying the reinforcement type in sand and also aggregate layer laid over sand bed. The objective of this series of tests (i.e., I and J) is to find out the effect of type of reinforcement in sand and in aggregate layer laid over sand respectively.

Test series K includes determining the effect of number of layers of reinforcement. All the Test series (A to J) were performed with single layer of reinforcement. To see the effect of number of layers of reinforcement in case of layered system (sand with aggregate), geogrid was placed at optimum depth ratio obtained from Test series D and second layer of reinforcement was placed at 0.5B, i.e., at the interface of the sand and aggregate layer. Two tests were performed in this test series by preparing the samples with two relative densities (50% and 70%) using geogrid reinforcement and width of the reinforcement used in this Test series was 4B. Table 4.1 shows the summary of the test series considered in this study.

Table 4.1: Details of the test series used in the study

Test Series	**Parameters varied in the tests**				
	Type of reinforcement	**u/B**	**b/B**	**h/B**	**D_R**
A	Geogrid	-	-	-	50 % , 70%
B	Geogrid	-	-	0.1,0.25,0.5	70%
C	Geogrid	0.15,0.3,0.45,0.6	5	-	
D	Geogrid	0.15,0.3,0.45	5	0.5	
E	Geogrid	0.45	3,4,5	-	
F	Geogrid	0.3	3,4,5	0.5	
G	Geogrid	0.45	4	-	50%
H	Geogrid	0.3	4	0.5	
I	Road mesh	0.45	4	0	70% and 50%
J	Road mesh	0.3	4	0.5	70% and 50%
K	Geogrid	0.3 and 0.5	4	0.5	70% and 50%

4.6 Station manager software

Once the uniform sand bed was prepared, square footing was placed over the prepared bed. Then chilling unit and the hydraulic power unit (HPU) were switched 'ON' and the actuator was manually operated to facilitate its movement to the center of the tank. Static load was applied on a loading plate using a computer-controlled, servo-hydraulic actuator. Station manager software was used for operating the actuator. The detailed step by step procedure of operation of the station manage software was given below.

- Station manager software was opened and ‘244_22A.cfg’ configuration file was selected.
- It contains of the following windows.
  - Station manager
  - Manual command
  - MPT procedure editor
  - Meters
  - Detectors
- Multi-Purpose Test Ware was selected in the station manager window and the hydraulic power unit (HPU) and hydraulic servo manifold (HSM) in the station manager window were switched ON.
- After that, in the manual command window select the displacement mode and operate the movement of the actuator by enabling the manual command option. The total allowable movement of the actuator was 150 mm.
- MPT procedure editor was used to input the rate of displacement and also total displacement.
- Detectors were used to set the limiting values for displacement and force to make sure that load application will be stopped automatically if it exceeds this particular limiting value.

4.7 Check list used during the experiment

Table 4.2 shows the check list that was used in the experiments to make sure that everything was done in an orderly manner.

Table 4.2: Check list used in the experiments

S.No	Description	Yes	No
1	Whether Chilling Unit and Hydraulic Power Unit were ON		
2	Whether HPU was ready to use (module selection)		
3	Whether secondary pump button in the Chilling unit was switched on after achieving 13.5° temperature		
4	Whether actuator was bolted to the reaction frame and positioned vertical		
5	Whether Loading surface/plate was horizontal		
6	Whether Station Manager: Selection of Configuration file and parameter		

	set (244_22A.cfg, unreinforcedsand_30_12) were selected correctly		
7	Reset the Interlock and switch ON the HSM and HPU in the station manager window		
8	Whether Actuator was in full contact with loading plate (use manual command)		
9	Whether sufficient actuator displacement was available		
10	Whether appropriate rate of displacement and relative end level entered in MPT procedure editor		
11	Whether toggle execute mode is ON		
12	Whether displacement/force Detectors set appropriately (interlock/disabled)		
13	Where the procedure was saved in the new specimen file		
14	Once the test was finished, switch OFF the HPU and HSM and close the software		

Chapter 5

Results and Discussion

5.1. Introduction

In this Chapter, the load-settlement behavior of the loading plate placed on soil and on a strong granular fill (aggregate) over the poor soil with and without the inclusion of the reinforcement is provided. Parameters like depth of the reinforcement, width of the reinforcement, type of reinforcement, and relative density of sand layer are varied and the results are discussed in this chapter.

5.2. Load improvement factor

The improvement in the performance due to the provision of aggregate layer over a sand layer (for both the cases - with and without reinforcement) was quantified by using a non-dimensional parameter, load improvement factor (I_f), defined as

$$I_f = q_r/q_o$$

where, q_r is the bearing pressure of the reinforced foundation soil at a given settlement, and q_o is the bearing pressure of the unreinforced foundation soil at the same settlement.

5.3. Unreinforced case

5.3.1. Sand alone

First, the experiments were performed on a sand bed to know the load-settlement response of square footing resting on it. Tests were performed by preparing uniform sand bed with two different relative densities (50% and 70%) to know the effect of relative density. Figure 5.1 presents the variation of bearing pressure with settlement ratio for Test series A. Results showed a significant improvement in the bearing pressure as the density of sand increases. The bearing pressure vs. settlement curve of unreinforced sand prepared with a relative density of 50% (medium dense sand) shows a progressive failure whereas sand prepared with 70% relative density shows a general shear failure with a peak behavior at a settlement ratio of 15% with the corresponding bearing pressure equal to 343 kPa. Fragaszy et al. (1984) conducted experiments with different relative densities and observed similar behavior. For a settlement ratio of 10%, there was an improvement of 39.2% in the bearing pressure of the unreinforced sand prepared with 70% relative density when compared with sand bed with 50% relative density.

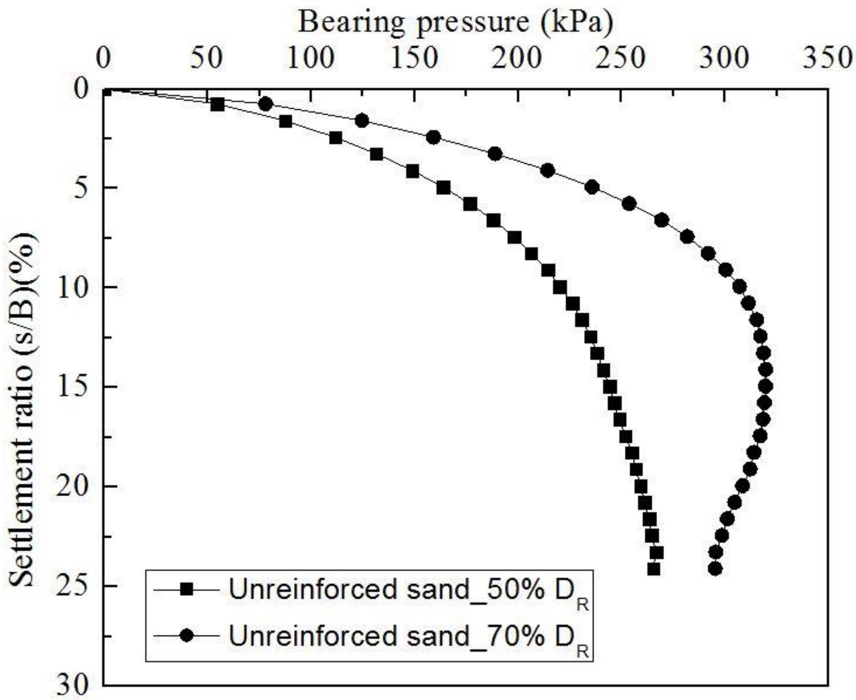


Fig. 5.1: Variation of bearing pressure with settlement ratio for unreinforced sand - Test series A

5.3.2. Layered system- Aggregate layer overlying sand layer

Placement of a stronger fill over the poor soil is one of the ground improvement techniques adopted to improve the loading carrying capacity of the footing. Accordingly, an aggregate layer with varying thickness 0.1B, 0.25B and 0.5B was adopted (Test series B). All the experiments in this test series were performed with underlying sand at a relative density equal to 70%. Figure 5.2 shows the variation of bearing pressure with settlement ratio by varying the thickness of the aggregates. Results showed that as the thickness of the aggregate layer increases there is an increase in the bearing pressure of the footing. For the thickness of the aggregate layer equal to 100 mm, the bearing pressure was found to be maximum. Hence for further tests, the thickness of aggregates was maintained as 100mm (i.e., h =0.5B). For a settlement ratio (s/B) equal to 10%, there was an increase of 80.8% in the bearing pressure of footing resting on 100 mm-thick aggregate layer placed over the sand when compared with the unreinforced sand.

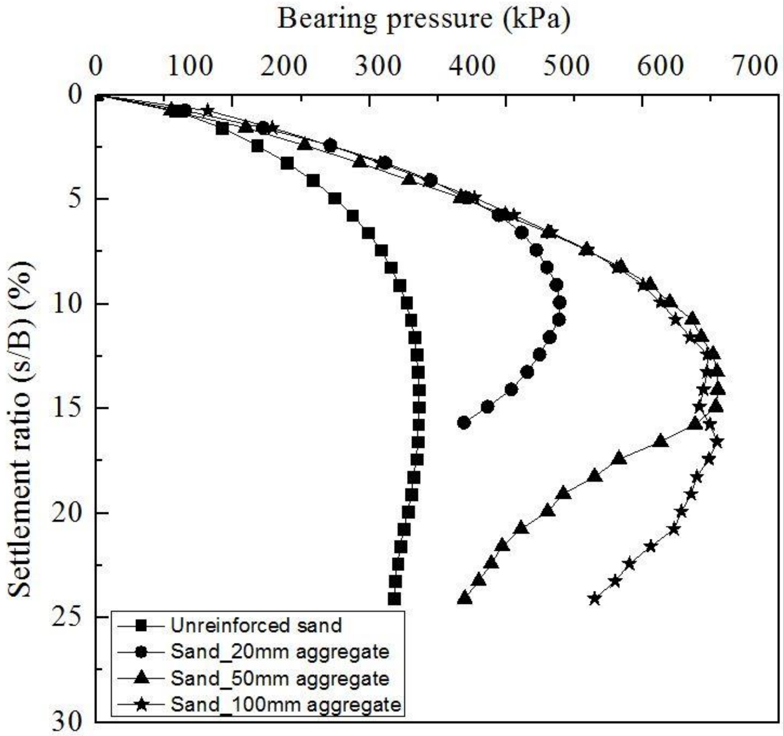


Fig.5.2: Variation of bearing pressure with settlement ratio for unreinforced layered system - Test series B

5.4. Effect of depth of the reinforcement

5.4.1. Sand alone

To increase the load carrying capacity of the sand, geogrid reinforcement was used. Single layer of geogrid was used to reinforce the sand. Location of the geogrid in the foundation soil plays a significant role in the load-settlement response of the footing. Therefore, to determine the optimum depth of the reinforcement which is defined as the depth of the reinforcement at which the composite soil mass (soil with geogrid) results in maximum bearing pressure, the depth of the reinforcement (u) was varied as 0.15, 0.3, 0.45 and 0.6 times the width of the footing (B) (Test series C). All the experiments were performed by preparing the sample at a relative density of 70%. Figure 5.3 presents the variation of bearing pressure with settlement ratio for different depths of geogrid in sand. From Figure 5.3, it is clear that as the depth ratio (u/B) increases from 0.15 to 0.45, bearing pressure also increases and increasing the depth ratio (u/B) beyond 0.45 resulted in decrease in the bearing pressure. For a settlement ratio of 10%, as the depth of the geogrid was varied as 0.15B, 0.3B, 0.45B and 0.6B, the percentage increase in the bearing pressure was 23.2%, 53.8%, 65.9% and 31.5% compared with the unreinforced sand respectively. Therefore, the optimum depth of the reinforcement when single layer of geogrid was placed in the sand layer was 0.45B with 65.9% increase in the bearing pressure compared with the unreinforced sand. Figure 5.4 shows the variation of load improvement factor with settlement ratio for Test series C. For a settlement ratio equal to 10%, there was an increase of 24.8% in the load improvement factor when the geogrid was placed at u = 0.3B compared with the geogrid placed at u = 0.15B. When the geogrid was placed at u = 0.45B, there was increase of 7.9% in the load improvement factor compared with the geogrid placed at u = 0.3B for a settlement ratio equal to 10%. Similarly, when the geogrid was placed at u = 0.6B, there was a decrease of 20.8% in the load improvement factor compared with the geogrid placed at a depth of 0.45B indicating that when the geogrid was placed beyond 0.45B, there was a decrease in the load improvement factor and hence the optimum depth of the reinforcement when single layer of geogrid reinforcement was placed in the sand bed was 0.45 times the width of the footing.

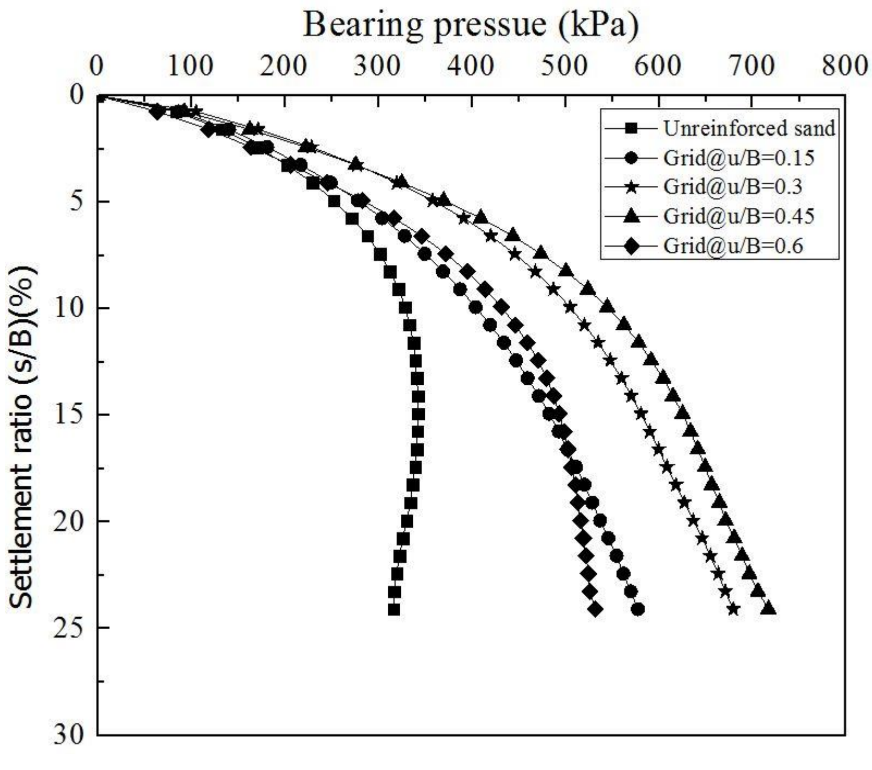


Fig.5.3: Variation of bearing pressure with settlement ratio for different depths of geogrid in sand- Test series C

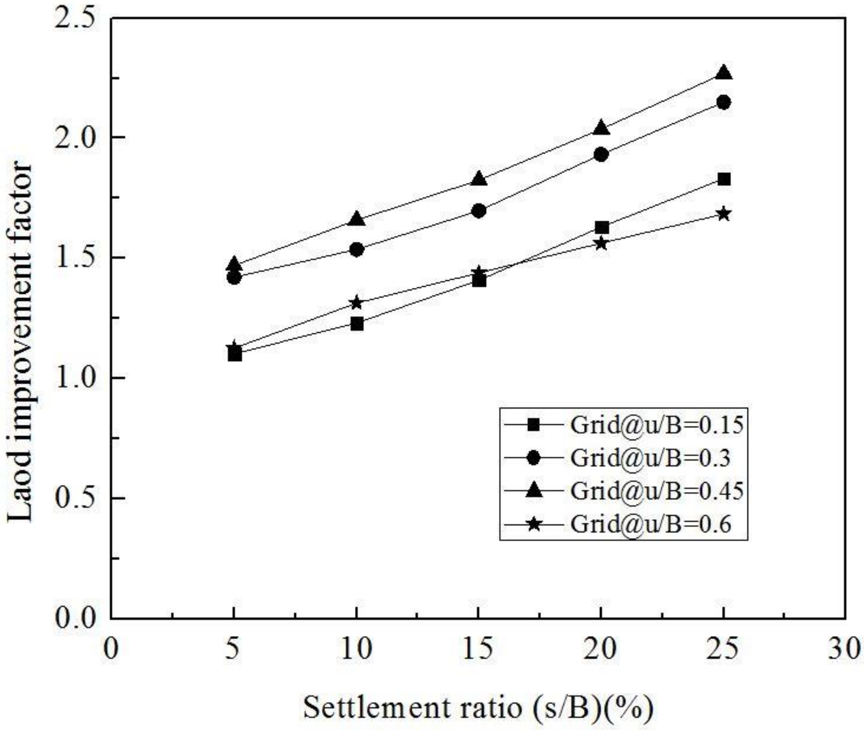


Fig.5.4: Variation of load improvement factor with settlement ratio for different depths of geogrid in sand - Test series C

5.4.2. Layered system- Aggregate layer overlying sand layer

To increase the load carrying capacity of the layered system (sand layer underlying 100 mm thick aggregate layer) further, geogrid reinforcement was placed in the aggregate layer. Single layer of geogrid was used to reinforce the aggregate layer. Therefore, to determine the optimum depth of the reinforcement the depth of the reinforcement (u) was varied as 0.15, 0.3 and 0.45 times the width of the footing (B) (Test series D). All the experiments were performed by preparing the bed with underlying sand layer to a relative density equal to 70%. Figure 5.5 shows the variation of bearing pressure with settlement ratio for different depths of geogrid in aggregate layer. From Figure 5.5, it is clear that as the depth ratio (u/B) increases from 0.15 to 0.3, bearing pressure of footing also increases and further increase in the depth ratio (u/B) resulted in decrease in the bearing pressure of footing. For a settlement ratio equal to 10%, as the depth of the geogrid was varied as 0.15B, 0.3B and 0.45B, the corresponding increase in the bearing pressure of footing was equal to 15.5%, 26.5% and 6.4% compared with the unreinforced layered system, respectively. Therefore, the optimum depth of the reinforcement when the single layer of geogrid was placed in the aggregate layer was 0.3B with 26.5% increase in the bearing pressure when compared with the unreinforced layered system. Figure 5.6 presents the variation of load improvement factor with settlement ratio for Test series D. For a settlement ratio of 10%, there was an increase of 71% in the load improvement factor when the geogrid was placed at u = 0.3B compared with the geogrid placed at u = 0.15B. When the geogrid was placed at u =0.45B, there was a decrease of 75.7% in the load improvement factor compared with the geogrid placed at u = 0.3B for a settlement ratio equal to 10%, indicating that when the geogrid was placed beyond 0.3B, there was decrease in the load improvement factor and hence the optimum depth of the reinforcement when single layer of geogrid reinforcement was placed in the aggregate was 0.3 times the width of the footing.

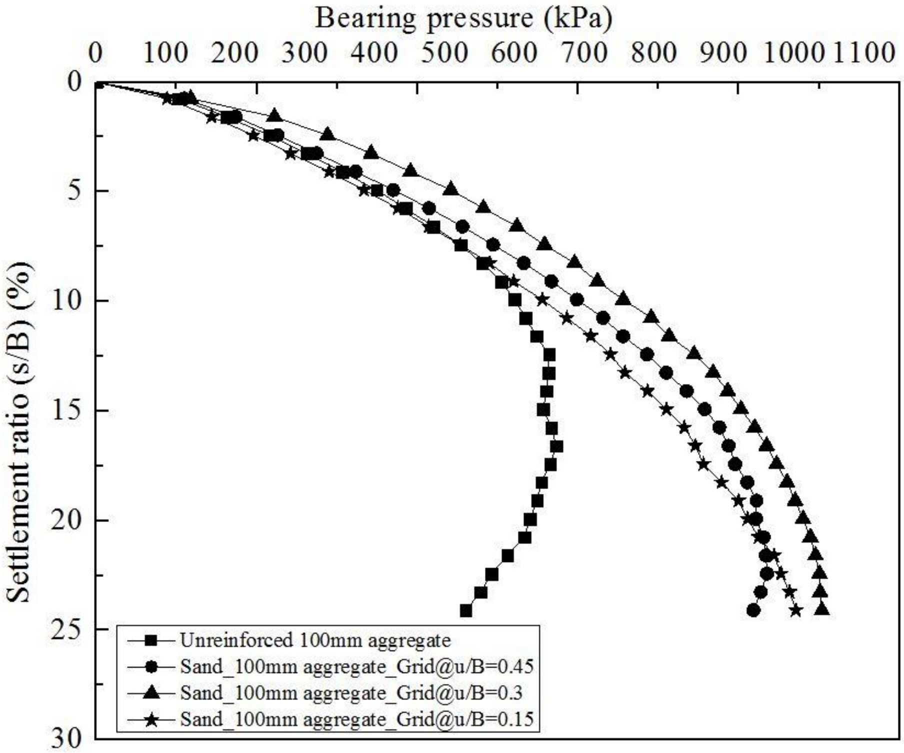


Fig.5.5: Variation of bearing pressure with settlement ratio for different depths of geogrid in aggregate layer overlying a sand layer-Test series D

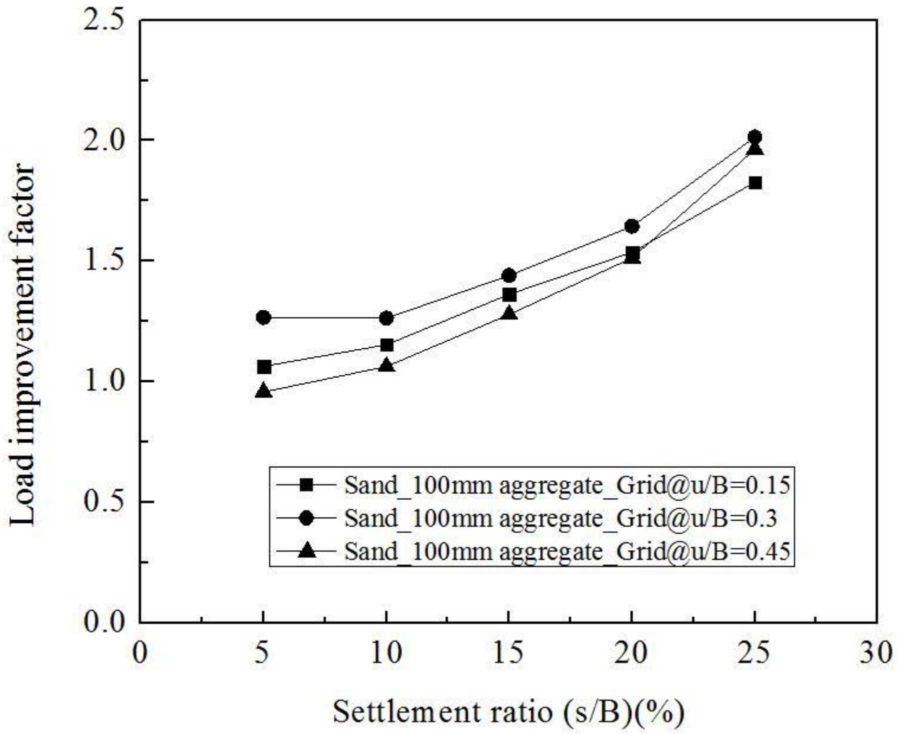


Fig.5.6: Variation of load improvement with settlement ratio for different depths of geogrid in aggregate layer overlying a sand layer - Test series D

5.5. Effect of width of the reinforcement

5.5.1. Sand alone

The experimental results in Test series C indicated that the optimum depth of the reinforcement when single layer of reinforcement was placed in sand bed was 0.45B (refer to Fig.5.4). The width of the reinforcement for all the experiments in the Test series C was maintained the same as width of the tank (i.e., five times the width of the footing). To determine the effect of width of the reinforcement on the load-settlement behavior of the footing, Test series E was designed. Two tests were performed in this Test series by varying the width of the reinforcement as three and four times the width of the footing. The reinforcement was placed at the optimum depth which was determined from Test series C and the width of the reinforcement was varied. Figure 5.7 presents the variation of bearing pressure of footing with settlement ratio for different widths of geogrid in sand. As the width of the reinforcement increases from 3B to 5B, there was increase in the bearing pressure. Even though the bearing pressure was maximum when the width of the reinforcement was five times the width of the footing, there can be boundary effects on the geogrid reinforcement. Therefore, the width of the reinforcement was considered as 4 times the width of the footing for all the experiments in the remaining Test series prepared with sand bed only.

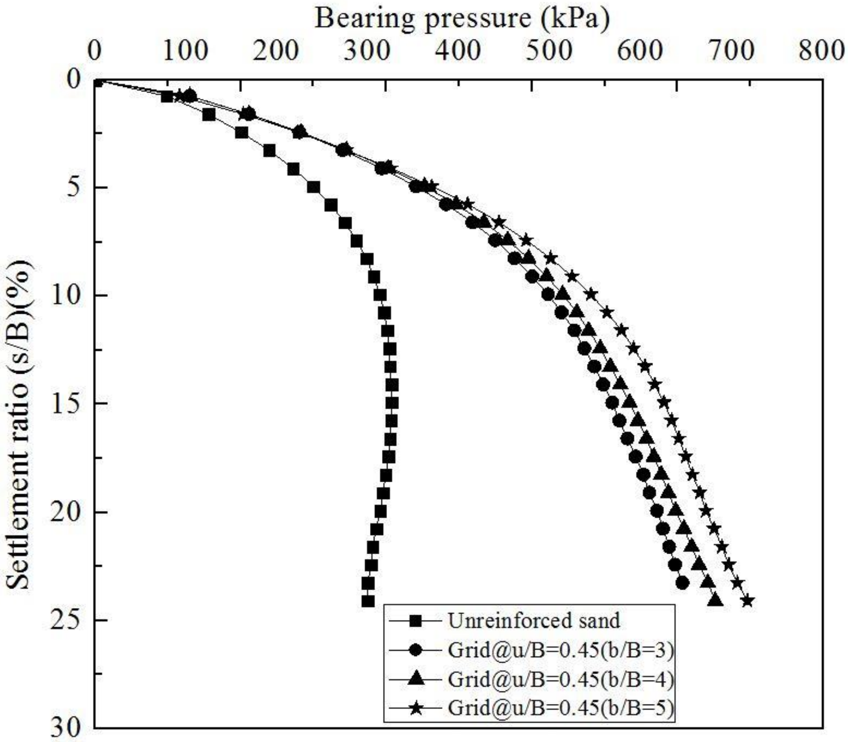


Fig.5.7: Variation of bearing pressure with settlement ratio for different widths of geogrid in sand-test series E

5.5.2. Layered system- Aggregate layer overlying sand layer

The experimental results in Test series D indicated that the optimum depth of the reinforcement in aggregate was 0.3B. The width of the reinforcement for all the experiments in the Test series D was maintained as same as the tank size (i.e., five times the width of the footing). To determine the effect of the width of the reinforcement on the load - settlement behavior of footing, Test series F was designed. Two tests were performed in this test series by varying the width of the reinforcement as three and four times the width of the footing. The reinforcement was placed at the optimum depth which was determined from the Test series D and the width of the reinforcement was varied. Figure 5.8 presents the variation of bearing pressure with settlement ratio for different widths of geogrid placed in aggregate layer. As the width of the reinforcement increases from 3B to 5B, there was no improvement in the bearing pressure and hence the width of the reinforcement was maintained as four times the width of the footing for the experiments with aggregate in the remaining test series.

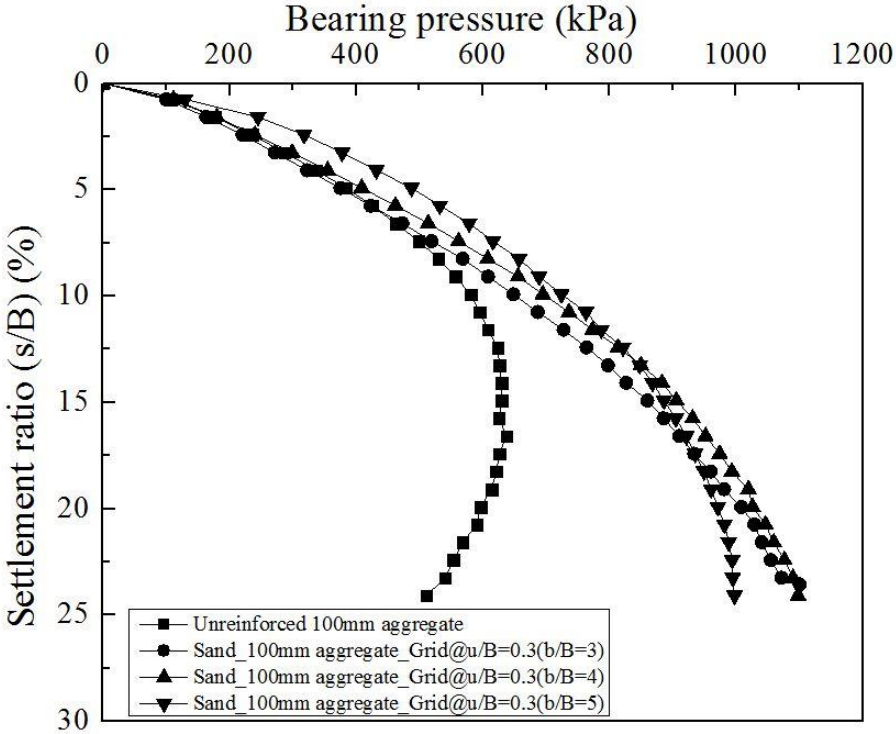


Fig.5.8. Variation of bearing pressure with settlement ratio for different widths of geogrid placed in aggregate layer -Test series F

5.6. Effect of relative density

5.6.1. Sand alone

The relative density is one of the important factors influencing the load-settlement behavior of footing placed on reinforced beds. Hence in this study, tests were performed by preparing the sand beds with two different relative densities, i.e., 50% and 70%. Geogrid was placed at the optimum depth of 0.45B and the width of the reinforcement was kept as 4B. Figure 5.9 shows the variation of bearing pressure with the settlement ratio for different relative densities. For a settlement ratio (s/B) of 10%, there was an increase of 37.2% in the bearing pressure of the reinforced sand compared with the unreinforced sand prepared for 50% relative density. Similarly, sand bed prepared with a relative density of 70%, there was an increase of 57% in the bearing capacity of the reinforced sand compared with unreinforced sand.

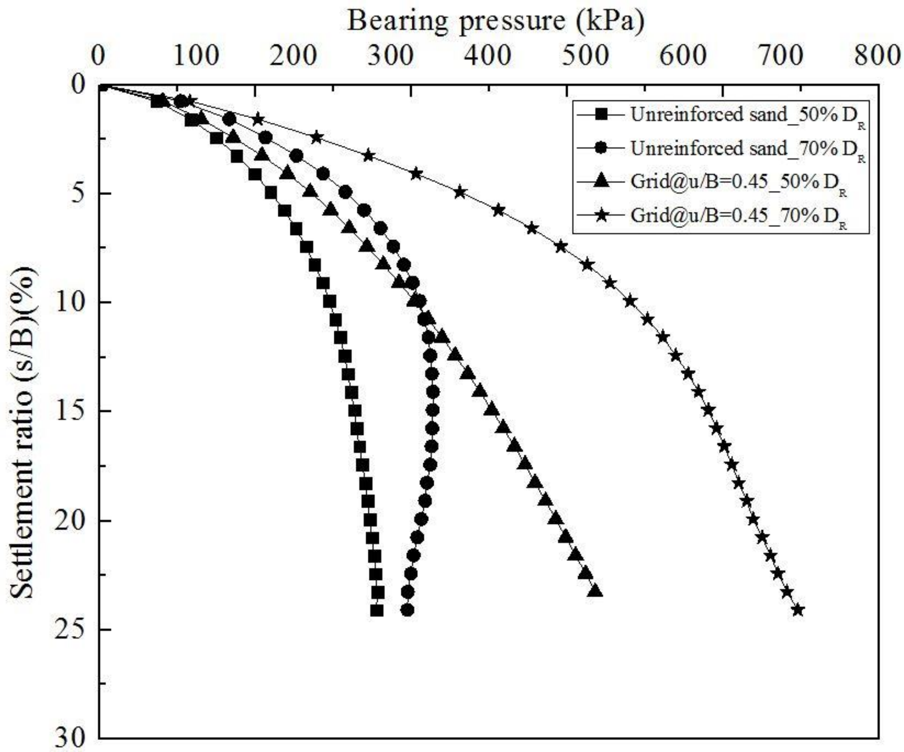


Fig.5.9: Variation of bearing pressure with settlement ratio for two relative densities in sand- Test series G

5.6.2. Layered system- Aggregate layer overlying sand layer

Once the optimum depth and width of the reinforcement were found out in case of aggregate layer overlying the sand layer, experiments were performed with 50% relative density to determine the effect of relative density. Geogrid was placed at optimum depth which was found out in Test series D and the width of the reinforcement was maintained as 4B. Figure 5.10 presents the variation of bearing pressure with settlement ratio for two relative densities in case of layered system. For a settlement ratio (s/B) of 10%, there was an increase of 12.8% in the bearing capacity of the reinforced sand prepared with 50% relative density when compared with the unreinforced sand prepared with the same relative density. Similarly, sand beds prepared with a relative density of 70%, there was an increase of 21.1% in the bearing capacity of the reinforced sand compared with unreinforced sand.

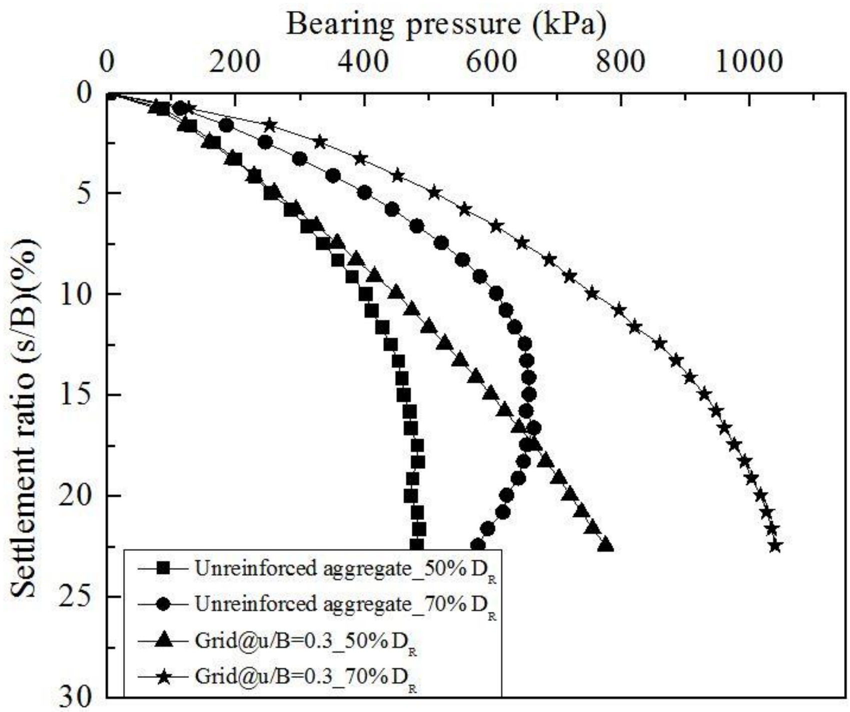
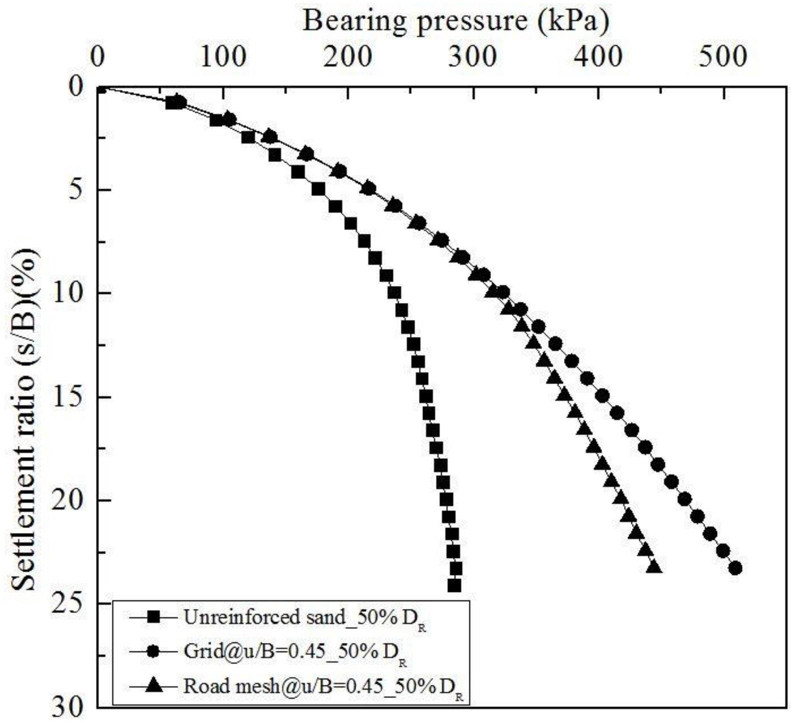


Fig.5.10 Variation of bearing pressure with settlement ratio for two relative densities for the case of layered system - Test series H

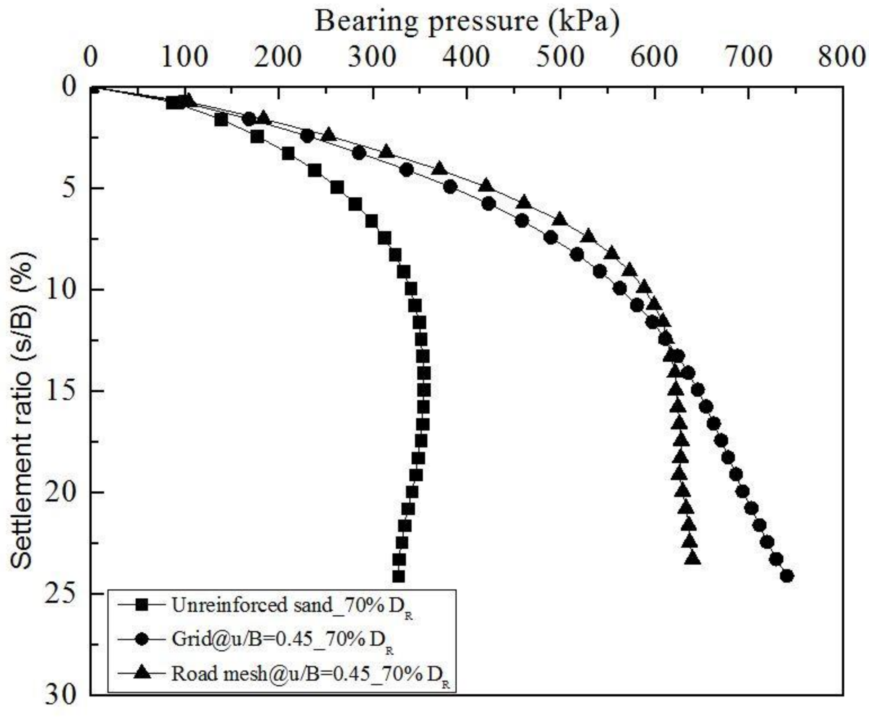
5.7. Effect of type of reinforcement

5.7.1. Sand alone

To determine the effect of type of reinforcement, road mesh was used as reinforcement. Road mesh was placed at optimum depth of 0.45B in the case of sand alone and the width of the road mesh reinforcement was maintained as 4B. Two tests were performed with road mesh by preparing the sand bed with two different relative densities – 50 % and 70% – and the results were compared with the geogrid reinforcement. Figs. 5.11a and 5.11b show the comparison of load-settlement behavior of footing on sand beds reinforced with road mesh and with geogrid reinforcement, respectively. For a settlement ratio (s/B) equal to 10%, in case of sand bed prepared with a relative density of 50%, there was a decrease of 2.5% in the bearing pressure of footing when road mesh was used instead of geogrid. In the case of sand bed prepared with a relative density of 70%, there was an increase of 4.4% in the bearing pressure of footing when road mesh was used instead of geogrid which is not a very significant improvement. Hence, the load-settlement behavior of footing placed on sand beds reinforced with road mesh is similar to that of beds reinforced with geogrid reinforcement.



(a)

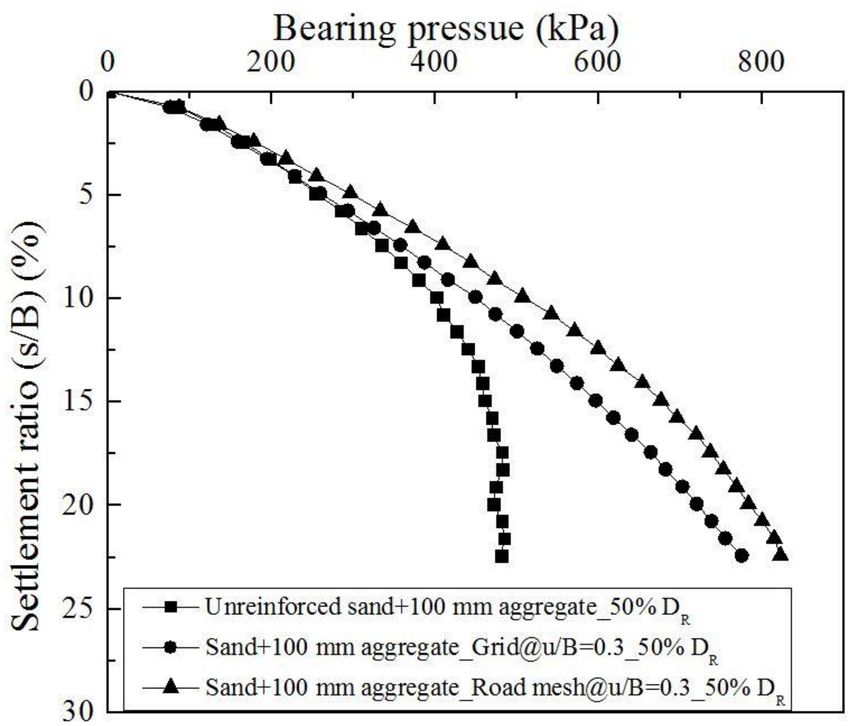


(b)

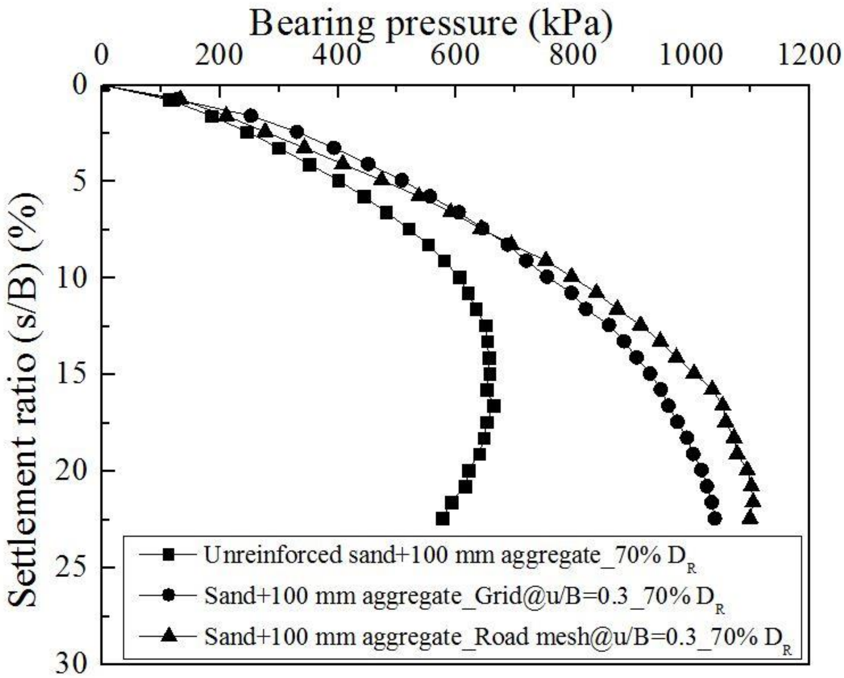
Fig.5.11. Effect of type of reinforcement placed in sand alone for (a) a relative density of sand bed equal to 50%, and (b) a relative density of sand bed equal to 70% - Test series I

5.7.2. Layered system- Aggregate layer overlying sand layer

To determine the effect of type of reinforcement in case of aggregate layer, road mesh was used. Road mesh was placed at optimum depth of 0.3B in the aggregate layer and the width of the road mesh reinforcement was maintained as 4B. Two tests were performed with road mesh by preparing the sample bed with two different relative densities – 50 % and 70% and the results were compared with the geogrid reinforcement. Figs. 5.12a and 5.12b show the comparison plots of layered system reinforced with road mesh and geogrid reinforcements respectively. For a settlement ratio (s/B) of 10%, there was an increase of 13.5% in the bearing pressure when road mesh was used instead of geogrid when underlying sand beds were prepared at a relative density equal to 50%. For a relative density of sand beds equal to 70%, there was an increase of 4.7% in the bearing pressure of footing when road mesh was used instead of geogrid. Hence, the load-settlement behavior of footing resting on road mesh reinforced layered system shows a significant improvement compared to geogrid reinforced layered system for the case of sand beds prepared at relative density equal to 50%.



(a)

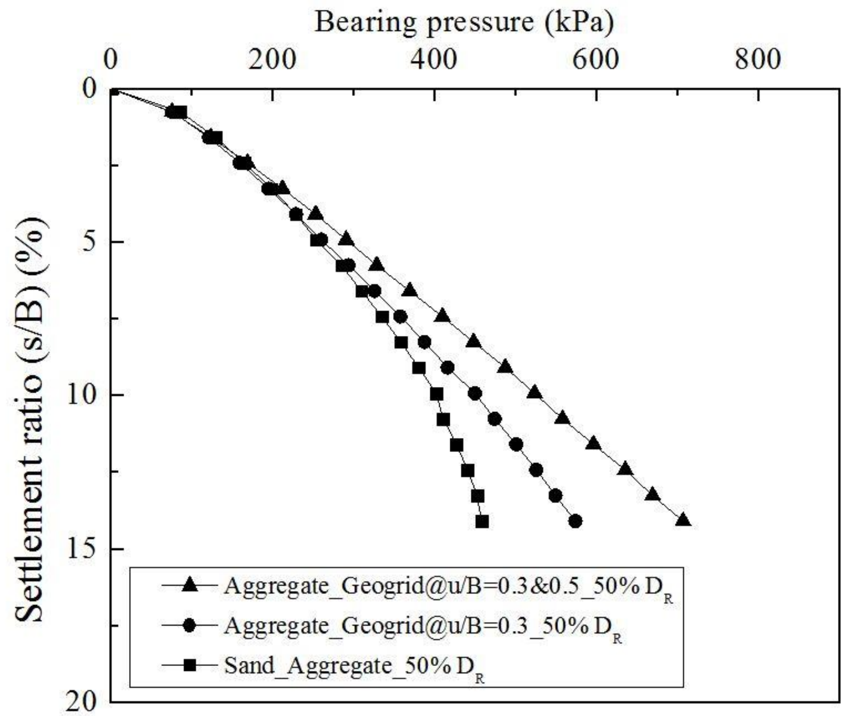


(b)

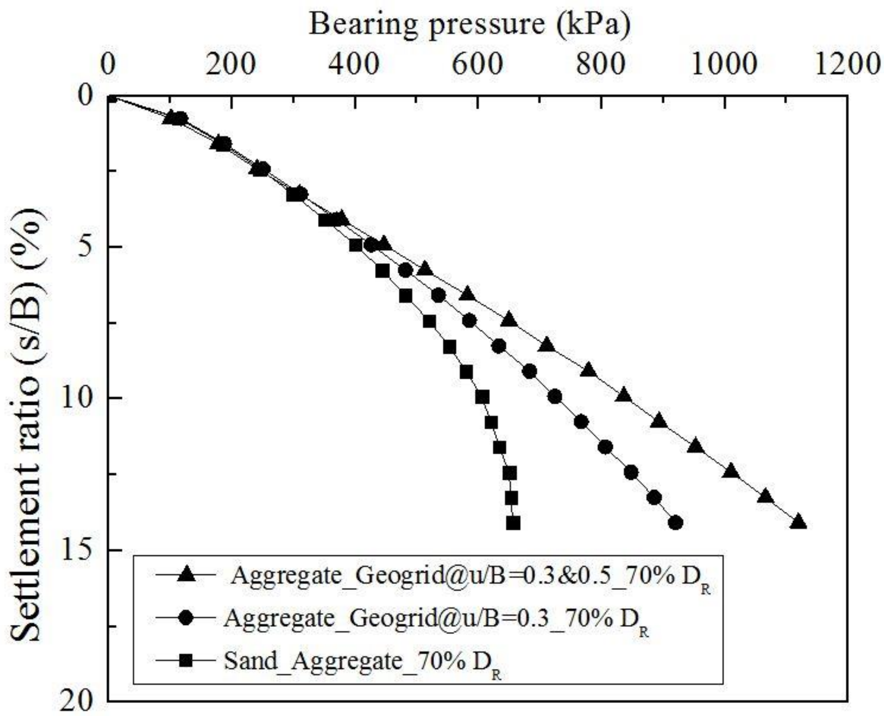
Fig.5.12: Effect of type of reinforcement in aggregate layer overlying sand beds prepared at (a) a relative density of 50%, and (b) a relative density of 70% - Test series J

5.8. Effect of number of reinforcement layers

To determine the effect of number of layers of reinforcement, geogrid was placed in aggregate layer at optimum depth (equal to 0.3B) and also at the interface of the sand and aggregate (equal to 0.5B). Two tests were performed by varying the relative densities as 50% and 70%. Figs. 5.13a and 5.13b show the variation of bearing pressure of footing with settlement ratio when two reinforcement layers were used. For a settlement ratio (s/B) equal to 10%, there was an increase of 16.7% in the bearing pressure of the footing resting on two layered reinforced case when compared with the bearing pressure of the footing resting on single layer reinforced case for a relative density of 50%. For a relative density of 70%, there was an increase of 15.5% in the bearing pressure of the footing resting on two layered reinforced case when compared with the bearing pressure of footing resting on single layer of reinforced case.



(a)



(b)

Fig.5.13: Effect of number of reinforcement layers for layered system with the relative density of underlying sand beds equal to (a) 50%, and (b) 70% - Test series K

Chapter 6

Conclusions

Based on the results of the present study, the following conclusions can be drawn:

1. The inclusion of reinforcement in sand and also in aggregate layers resulted in increase in the bearing pressure of footing.
2. Optimum depth of the reinforcement when geogrid is placed in sand layer alone is found to be 0.45 times the width of the footing. With the inclusion of geogrid in sand layer at a depth equal to 0.45B, there was an increase of 66% in the bearing pressure of footing when compared with the unreinforced case.
3. Increase in the width of the geogrid reinforcement showed an increase in the load carrying capacity of footing for settlement ratio (s/B) >10% when the geogrid reinforcement was placed in sand layer.
4. For a settlement ratio (s/B) equal to 10%, there was an improvement of 39.2% in the bearing pressure of footing resting on unreinforced sand prepared with 70% relative density compared to 50% relative density.
5. For a settlement ratio (s/B) of 10%, there was an increase of 37.2% in the bearing pressure of footing resting on geogrid reinforced sand compared with the unreinforced sand for 50% relative density. While, the increase in the bearing pressure of footing was equal to 57% for geogrid reinforced sand compared with unreinforced sand for 70% relative density.
6. No significant improvement in the bearing pressure of footing with the inclusion of road mesh compared with the geogrid reinforcement.
7. The increase in the thickness of the aggregate layer as 0.1B, 0.25B and 0.5B, there was increase in the bearing pressure of footing. When 100 mm thick aggregate layer was laid over the sand there was an increase of 20.2% in the bearing pressure of footing compared with the unreinforced sand for 50% relative density. While, the corresponding increase was equal to 80.8% for dense sand beds prepared at 70% relative density.
8. Optimum depth of the reinforcement when geogrid is placed in aggregate layer is 0.3 times the width of the footing. With the inclusion of geogrid in aggregate layer

at a depth of 0.3B, there was an increase of 26.5% in the bearing pressure of footing when compared with the unreinforced case.

9. Increase in the width of the reinforcement did not show significant effect on the load carrying capacity of footing when geogrid reinforcement was placed in aggregate.
10. For a settlement ratio (s/B) equal to 10%, there was an increase of 12.8% in the bearing pressure of the geogrid reinforced layered system compared with the unreinforced layered system for 50% relative density of sand beds. While the corresponding increase was equal to 26.3% for 70% relative density of sand beds.
11. Inclusion of road mesh in place of geogrid reinforcement in aggregate layer showed significant improvement in the bearing pressure of footing for 50% relative density compared with the layered system prepared with 70% relative density.
12. When two layers of geogrid reinforcement were included in the aggregate layer, for a settlement ratio (s/B) equal to 10%, there was an increase of 16.7% in the bearing pressure of the footing resting on two layered reinforced case when compared with the bearing pressure of the footing resting on single layer reinforced case for a relative density of 50%. For a relative density of 70%, for a settlement ratio of 10%, there was an increase of 15.5% in the bearing pressure of the footing resting on two layered reinforced case when compared with the bearing pressure of the footing resting on single layer of reinforced case.

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