INFLUENCE OF CONFINENESS ON BEARING CAPACITY OF SAND

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE AWARD OF DEGREE
OF

MASTER OF TECHNOLOGY

IN

GEOTECHNICAL ENGINEERING

SUBMITTED BY

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UNDER THE SUPERVISION OF

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CANDIDATE'S DECLARATION

I, **ANKIT KUMAR**, (**2K16/GTE/04)** student of M.tech (GEOTECHNICAL ENGINEERING), hereby declare that the Project Dissertation entitled "**INFLUENCE OF CONFINESS ON BEARING CAPACITY OF SAND"** which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for 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 Associateship, Fellowship or other similar title or recognition.

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CERTIFICATE

I hereby certify that the Project Dissertation titled "**INFLUENCE OF CONFINESS ON BEARING CAPACITY OF SAND"** which is submitted by ANKIT KUMAR, (2K16/GTE/04) , Civil Engineering Department, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by him under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Place: Delhi **(ANIL KUMAR SAHU)**

Date PROFESSOR & SUPERVISOR

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ABSTRACT

This project presents the influence of confineness on the bearing capacity and settlement of the locally available sand under vertical loading. In this, confining cells of different heights and widths have been used to confine the sand. The project presents the comparison of the behavior of the sand in unconfined condition and confined condition which has been created using different cells. The results showed that confinement increased the bearing capacity of sand drastically. When small width cell and footing has been used, the soil-cell footing behaves as one unit. Keywords: cohesionless soils; model test; square footing; ultimate bearing capacity; soil confinement.

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LIST OF ABBREVIATIONS

SYMBOL	TITLE
BCIF	Bearing Capacity Improvement Factor
C_u	Uniformity coefficient
C_c	Coefficient of curvature
DTU	Delhi Technological University
G	Specific gravity
GTE	Geotechnical Engineering
IS	Indian Standard

CHAPTER-I INTRODUCTION

1.1 GENERAL

Soil should be capable of transferring load without shear failure and settlement. This has been the primary focus of the researcher. Soil can be improved by different methods viz grouting, compaction, reinforcement etc. due to paucity of good soil and land we have to use the land that have marginal soil properties. Due to this problem in situ treatment of soil to improve its properties has risen. Confinement of soil is also one of the techniques to improve the ground strength. Many researchers studied this method and gave their results. Confining help in reduction of the lateral movement of soil due to which bearing capacity of soil is increased.

Providing confinement by using cells made of metal plays a significant role in the field of geotechnical engineering. This technique of providing cell below foundation level has not got much attention in the field of foundation engineering. For foundation design settlement and bearing capacity are the criteria which to be checked for the safety of the shallow foundation. Settlement is the more prominent criterion as compare to bearing capacity of sand. This project also follows the same approach as settlement is the most important criterion, so by using confinement cell arrest of the settlement take place which make the footing and structure safe in settlement.

1.2 Objectives

On the basis of literature survey, the following objectives are framed-

1. To study the effect of soil confinement on bearing capacity of sand.
2. To study the effect of soil confinement in the reduction of settlement of footing.
3. To study the variation of bearing capacity with respect to height and width of cell.
4. To study the variation of bearing capacity and settlement when cell is placed at different depth.

1.3 Organisation of the project work

- The cells of different width and height have been designed and used to make graphs, which shows variation of bearing capacity and settlement.
- This project work divided into 5 chapter.
- Chapter 1 deals with the introduction and objective of the research.
- Chapter 2 contains literature review.
- Chapter 3 deals with materials and methods.
- Chapter 4 deals with the results & discussion
- Chapter 5 deals with conclusions and recommendations for the future work.

CHAPTER 2 LITERATURE REVIEW

In order to observe the effect of confineness on the bearing capacity of sand the literature survey has been made and which is reflected in the following section.

The structural measures for foundations are widely used in weak soil conditions to support column loads. Sometimes the excavation needs to be braced during foundation construction. One of the available solutions is to use side support to the excavation during construction. Due to the problems associated with the removal these supports, they are provided as part of the permanent structure. Accordingly, it consists of two parts; it is to deals with the structural analysis of the footing if the side supports are used as end supports for the foundation (Sawwaf and Nazer, 2005). Secondly, the effect of these supports on the lateral movement of the soil underneath the foundation is to be investigated as the effect of the lateral confinement on the bearing capacity of the sands. While there are several solutions for the first problem, such as isolating the foundation from the side supports. But the effect of lateral confinement by these side supports on the foundation behavior is not well understood. Swwaf and Nazer (2005) studied the effect of confinement on the bearing capacity of sand and have found an improvement in bearing capacity as high as 17 times as that without confinement.
Rajagopal et al. (1999) studied the strength of confined sand by carrying out a large number of triaxial compression test to study the influence of geocell confinement on the strength and stiffness behaviour of granular soils. Rea and Mitchell (1978) conducted a series of model plate load test on circular footings supported over sand filled square shaped paper grid cells was carried to identify different modes of failure and arrive at optimum dimensions of the cell. Dash et al.(2001a) conducted a load test for a strip footing on homogeneous dense sand (relative density of 70%) beds, however, indicate that an 8-fold increase in bearing capacity could be achieved with the provision of geocell in the foundation sand.

V. A. Guido, D. K. Chang, and M. A. Sweeney (1986) compared the results of laboratory model tests used to study the bearing capacity of geogrid and geotextile reinforced earth slabs. The parameters studied were the coefficient of friction between the geotextile and the soil, pull-out resistance between the geogrid and the soil, depth below the footing of the first layer of reinforcement, vertical spacing of the layers, number of layers, width size of a square sheet of reinforcement, and tensile strength of the reinforcement. For both geogrids and geotextiles, after an optimum number of layers or width of reinforcement, the bearing capacity did not increase. In addition, the bearing capacity was largest for those geogrid and geotextile reinforced earth slabs where the first layer was closest to the footing and the spacing between the layers was the smallest. Bearing capacity increased directly with increasing reinforcement tensile strength for the geotextile; however, for the geogrid, aperture size and reinforcement tensile strength must be looked at simultaneously.

K.H. Khing, V.K. Puri et al. (1993) performed laboratory-model test results for the bearing capacity of a strip foundation supported by a sand layer reinforced with layers of geogrid are presented. Based on the present model test results, the bearing-capacity ratio with respect to the ultimate bearing capacity, and at levels of limited settlement of the foundation, has been determined. For practical design purposes, it appears that the bearing-capacity ratio at limited levels of settlement is about 67–70% of the bearing-capacity ratio calculated on the basis of the ultimate bearing capacity

Madhavi Latha Gali and Rajagopal Karpurapu (2000) proposed a simple method for the preliminary design of geocell supported embankments. This method uses a general-purpose computer program for the stability analysis of geocell supported embankments. In this program, the geocell layer at the base of the embankment was considered as an equivalent soil layer, whose shear strength properties were obtained from relevant equations. For the given configuration of the embankment, the dimensions and properties of the geocell layer required to be provided at the base to sustain the given surcharge pressure can be obtained from simple trial analyses using the computer program. The validity of the procedure was verified by conducting stability analysis of models of geocell supported embankments, constructed and tested in laboratory. The minimum factor of safety obtained from the slope stability program for the experimental configurations of embankments at observed failure surcharge pressure was obtained as nearly one for most of the cases, suggesting that this procedure can be successfully used for the preliminary design of geocell supported embankments. A design problem describing the steps involved in the analysis is presented.

A. Kumar, M. L. Ohri, and R. K. Bansal (2007) performed Bearing capacity tests of strip footings on reinforced layered soil. The ultimate bearing capacity of strip footings resting on subsoil consisting of a strong sand layer (reinforced/unreinforced) overlying a low bearing capacity sand deposit has been investigated. Three principal problems were analysed based on results obtained from the model tests as follows: (1) the effect of stratified subsoil on the foundations bearing capacity; (2) the effect of reinforcing the top layer with horizontal layers of geogrid reinforcement on the bearing capacity; (3) effect of reinforcing stratified subsoil (reinforced and unreinforced) on the settlement of the foundation. It has been observed 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.

V. K. Singh, A. Prasad and R.K. Aggarwal (2007) presented the results of laboratory model tests on the effect of soil confinement on the behavior of a model footing resting on Ganga sand under eccentric – inclined load.

Bestun J. Nareeman and Mohammed Y. Fattah (2012) performed experiment to show the effect of Soil Reinforcement on Shear Strength and Settlement of Cohesive-Frictional Soil. This study investigates the effect of soil reinforcement using geonet on the shear strength, consolidation and swelling of silty soil. The tests that carried out are classified into two categories: First; tests on soil without reinforcement and second tests on soil with reinforcement. The loading test was conducted on small scale model using different layers of reinforcement. The results showed that the shear strength parameters could be improved by using geonet reinforcement. Moreover, the settlement and swelling of silty soils are decreased by using geonet.

Musa Alhassan and I L Boiko (2013) performed test to check the patterns of load-settlement characteristic of statically loaded shallow foundation models with different vertical cross-sectional shapes on both unreinforced and reinforced soft clay soils are presented. Models of shallow foundations with rectangular, wedge and T-shape vertical cross-sections were studied. The study generally shows that reinforcement of soil under shallow foundations with deferent vertical cross-sectional shapes increases bearing capacity and reduces settlement of the subsoil base. Evaluation of Bearing Capacity Ratio (BCR) shows that foundations with rectangular vertical cross-sectional shapes have higher BCR values than those foundations with T and wedge vertical cross-sectional shapes.

D Chakraborty and Jyant Kumar (2014) proposed a method to determine the ultimate bearing capacity of a strip footing placed over granular and cohesive-frictional soils that are reinforced with horizontal layers of reinforcements. The reinforcement sheet is assumed to resist axial tension but not bending moment. The analysis was performed by using the lower bound theorem of the limit analysis in combination with finite elements. A single layer and a group of two layers of reinforcements were considered. The results were obtained for different values of the soil internal friction angle. The critical positions of the reinforcements, which would result in a maximum increase in the bearing capacity, were established. The required tensile strength of the reinforcement to avoid its breakage during the loading of the foundation was also computed

D Chakraborty and Jyant Kumar (2015) A method is presented for determining the ultimate bearing capacity of a circular footing reinforced with a horizontal circular sheet of reinforcement placed over granular and cohesive-frictional soils. It was assumed that the reinforcement sheet could bear axial tension but not the bending moment. The present research is an extension of recent work with strip foundations reinforced with different layers of reinforcement. Results were obtained for different values of the soil internal friction angle (ϕ). The optimal positions of the reinforcements, which would lead to a maximum improvement in the bearing capacity, were also determined. The variations of the axial tensile force in the reinforcement sheet at different radial distances from the center were also studied

Prof. Harish and Shravan Balu (2016) studied the improvement of bearing capacity of square footing on the soft soil layer by using the sand piles with or without confinement. In this study, square footing of dimension 50mm x 50mm x 20mm and the skirts of dimension 50mm x 50mm are used and varying lengths decided based on the depth of soft soil layer. A sand bed is provided beneath soft clay layer of fixed depth. In order to study the improvement of bearing capacity of the square footing, the load-settlement behavior of the model foundation is tested in Universal Testing Machine. A series of load tests were carried out to investigate the effect of partially replaced sand pile with or without confinements by skirts. The results show that the improvement of load bearing capacity is very nominal using both partially replaced sand piles with or without skirts. The chosen method can considerably alter the stress displacement curve of the foundation resting on the soft soil layer, notably decreases the settlement and the replaced soil acts as a deep foundation. It was observed in the study that the skirt length has a significant role in improving the bearing capacity and reduction in the settlement of footing.

Renaningsih et al (2017) performed twelve laboratory experiments on steel circular footing with various diameters and skirt lengths. In addition, the type of soil used in this study was sand soil in which the formation of water content and compaction method was maintained. The laboratory tests indicated that skirts are very effective to improve the ultimate bearing capacity, as they can increase the length hence the ultimate bearing capacity can be enhanced by 4.70 times in certain study condition. Skirts are also capable to reduce the settlement. In general, the analysis on the same value of load of 2.00 kN indicated the decrease of the settlement is in accordance with the increase of the skirt length attached on the circular footing

A B Listyawan et al.(2018) focused on the investigation of the effect of skirt on circular footing to improve the footing performance in resisting vertical load on clay soil. There are nine samples of circular footing tested in vertical load system in laboratory in the diameter of 75 mm, 100 mm and 150 mm with length of skirt 100 mm and 150 mm on clay by keeping the similar water content and compaction method. The results show that the skirt effectively reduces the foundation settlement on clay which is observed on similar load 1 kN. The observations on L/D ratio on similar diameter also show that the higher L/D ratio the smaller settlement. On the observations of settlement 3 mm with similar diameter show that the longer the skirt the higher load retained by footing.

F.W. Jawad et al.(2019) investigated the response of skirted footing resting on sandy soil of different states to lateral applications of loads on a small-scale physical model manufactured for this purpose. The parameters studied are the distance between the footing and the skirt and its depth. The results show that the presence of the skirt behind the footing loads to an increase in bearing load and a reduction in the lateral movement whereas the skirt near or adjacent to the footing edge causes maximum increases in bearing load and reduction in lateral movement, for skirted footing. The ratio between the wall distance and the width of the footing has no effect when it is greater than one. On the other hand, the state of the soil influences the bearing load and lateral movement with different ratio of wall distance and wall depth to the width of the footing, especially when the wall distance to the footing width is less than one and the state of the soil has no effect on the bearing load and lateral movement when the ratio is more than one.

CHAPTER 3 MATERIALS & METHODS

3.1 MODEL TEST APPARATUS

In order to see the effect of confiness on settlement characteristics of square footing and bearing capacity of sand the following test apparatus were made in laboratory.

a) Footing

A square model footing of size 50 x 50 mm and 6 mm thick made of mild steel with grooves at top was used.

Fig-1- Footing (50mmx50mm)

b) Model test tank

Model tests were conducted in a test tank, having inside dimensions of 1000 x 1000 mm in plan and 1200 mm deep.

Fig-2-Tank Top View (1000mmx1000mm)

Fig-3-Tank Side View (depth 1200mm)

c) Confining cells

The confining cells are made of MS plate with different dimensions. The internal sizes of square cells used are 100mm, 125mm, 200 mm and 250 mm. The heights of square cells used are 100mm,125mm, 200 mm and 250 mm. The thickness of the all the confining cells is 6 mm.

Fig-4-Confining Cells

d) Loading arrangement

Fig-5-Hydraulic Jack

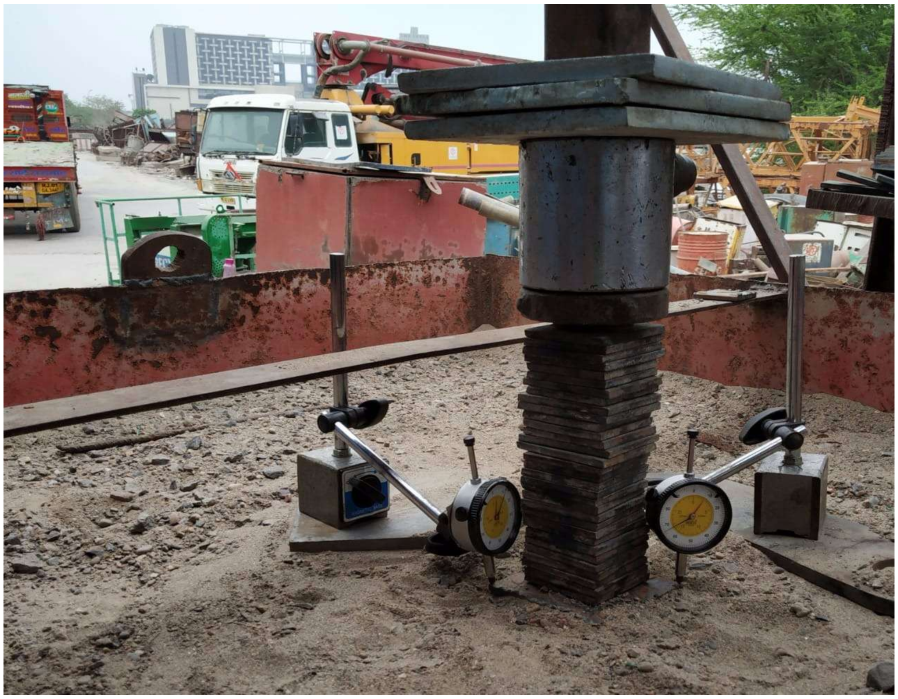

Fig-6-Hydraulic Jack on Footing

3.2 Test materials

Sand was obtained from the Yamuna river bank near Wazirabad bridge. Location as shown in the figure 7.

Fig-7-Location of Sample Collection

The properties of sand are evaluated in the DTU laboratory. The properties evaluated are particle size analysis, specific gravity and direct shear test.

3.3 LABORATORY TEST

3.3.1 Test conducted on sand

3.3.1.1 PARTICLE SIZE DISTRIBUTION OF SAND

As per IS-2720 (PART 4) 1985, for particle distribution, take 1 kg sand, pass it through the sieves arranged in order. Measure the weights of soil collected on each sieve. Using this cumulative percentage can be found out. After putting all the value in table given in IS code particle size distribution curve is obtained from the table.

3.3.1.2 Specific Gravity(G) of sand

As per IS: 2386 (Part III) -1963

Fig-8- Empty Pycnometer

Fig-9-Pycnometer with Sand

Fig-10-Pycnometer with Water

1) Take 1000gm sample and sieved this sample though 10 mm IS sieve and washed properly so that fine particles should be removed .
2) Take 500 gm of sand (C) and put that sample into pycnometer
3) Fill the pycnometer partially with water and remove the air by using glass rod. Then fill the pycnometer with water and fill the cone of pycnometer with the help of wash bottle.
4) Record the weight of pycnometer (A)
5) Record the weight of pycnometer with water only (B)
6) Weight of oven dry sample (D)

3.3.1.3 Direct shear test on sand

Fig- 11-Direct Shear Test Apparatus

According to IS-2720-PART-13-1986 procedure for direct shear test is as follows

a) To conduct this test , the need of direct shear apparatus consist of shear box that is horizontal divided in two parts having connecting pins, spacing screw, base plate with cross grooves on the top face, loading pad with steel ball on the top, two pairs of grooves plates one plain and other perforated, sets of weight for normal load, electric motor, hand wheel, rollers, locking pins, clutch and gear, for different weights of loading for different soil, sampler, rammer.
b) Take a specimen of soil to be tested from undisturbed sample. Measure the size of the box that is 6cm *6cm.
c) Assemble the two-half using the connecting pins. Place the base plate inside the box. Place one groove plate such that groves are perpendicular to the shear.
d) Place sample and place another groove perpendicular to the direction of shear. Place loading plate on metal plate.
e) Place pin on the box and place the shear box inside the container
f) Place the dial gauge for horizontal and vertical displacement and set all the dial gauge to zero.
g) Set the clutch and gear for shear displacement 1.25mm/min to sample. Switch on the machine.
h) Continue apply the shear force till quick back on the pointer on the dial gauge.
i) Conduct at least 3 tests on separate specimen with same density water content and apply different normal load.
j) Record all the observations.

k) Area of specimen at failure $A_O(1- \delta h/3)$ where δh is the horizontal displacement and A_o is the original area
l) Shear stress = Shear Force/ Area of Specimen at failure

3.3.1.4 Load settlement test

According to IS: 1888-1971 procedure for Load settlement test is as follows

Procedure

a) Take the tank of suitable dimensions. Fill it with sand, whose bearing capacity is to be analysed. Place the cell inside the sand such that the top surface of the sand coincides with the top of the cell. Place the plate of thickness 6 mm over the surface of the sand.

Fig-12- Placing of Confining Cell

Fig-13-Confining Cell on Ground Level

b) Now load the plate using hydraulic jack at a uniform rate and plot the graph load versus settlement

c) Repeat step 1 and 2 for different dimensions of cell and plot the graph

d) To study the variation of bearing capacity vs settlement with respect of depth, repeat step 1 and 2 using 100 mm cubical cell and 50 mm plate. Initially the cell is placed such that its top touches the surface of the sand. Subsequently it is placed at the depth of 200mm, 400mm, 600mm, and 800mm respectively

e) Finally plot the graph for each case and analyses the result.

CHAPTER 4 RESULTS AND DISCUSSION

This section gives all the result obtains from different test performed in the laboratory

4.1. Result of test performed on sand

4.1.1 Particle Size Distribution of Sand

Table-1-Particle Size Distribution of Sand

Particle Diameter(mm)	Mass Retained(gm)	Cumulative Mass Retained(gm)	Percentage Retained (%)	Total Passing (%)
4.75	84	84	8.4	91.6
2.36	154	238	23.8	76.2
1.18	126	364	36.4	63.6
0.6	76	440	44	56
0.3	354	794	79.4	20.6
0.15	166	960	96	4
0.075	0	0	0	0

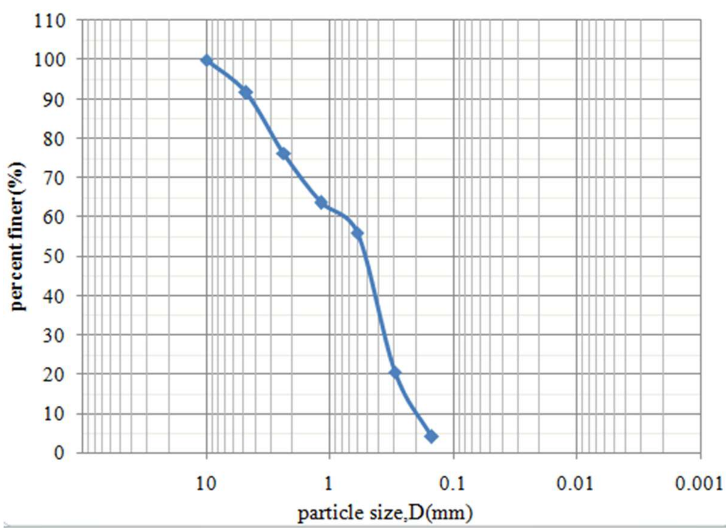

Fig-14- Particle Size Distribution Curve of Sand

The particle size distribution curve shows 91.6% soil is passing through 4.75mm sieve and the percentage passing through 75 micron is nil. This shows the soil pass under the category of sand. From the curve D_{10} , D_{30} and D_{60} sizes are 0.204mm, 0.379mm and 0.905mm respectively. The value of coefficient of uniformity and coefficient of curvature are 4.43(<6) and 0.77 (not falling between (1 to 3) respectively. Therefore as per codal recommendation it is poorly graded sand.

4.1.2 <u>Specific Gravity(G) of sand</u>

1) Record the weight of pycnometer (A) = 1936gm
2) Record the weight of pycnometer with water only (B)= 1624 gm
3) Weight of oven dry sample (D) = 497gm
4) Specific gravity = [D/(C-(A-B))] *100
5) Specific gravity of sand 2.64

Specific gravity on the basis of result obtain is found to be 2.64 which may be in the range of quartz mineral.

4.1.3 Direct shear test on sand

Table-2-Direct Shear Test Readings

Normal stress (kg/cm²)	Shear stress (kg/cm²)
0.5	0.36
1	0.72
1.5	1.089

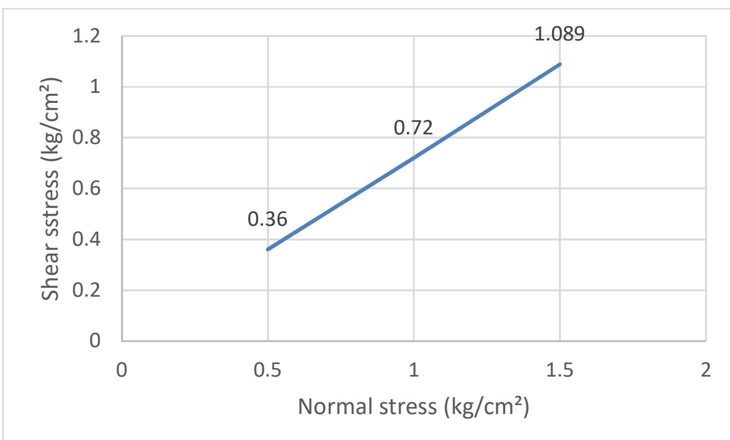

Fig-15-Shear Stress Versus Normal Stress Plot

The curve between normal stress and normal stress is drawn as in figure 15. These stresses are obtained on the basis of area at failure. From the curve the value of angle of shear resistance id found to be 36^0. Since it is dry sand therefore cohesion is taken as nil.

Friction angle of sand = 36°, Cohesion of sand = 0 kg/cm^2

On the basis of experimental result all the properties of Yamuna sand is summarized in Table 3.

Table-3- Properties of Yamuna Sand

S. No	Characteristics of sand	Values
1.	D_{10}(mm)	0.204217
2.	D_{30}(mm)	0.379661
3.	D_{60}(mm)	0.905263
4.	C_u (Uniformity Coefficient)	4.43
5.	C_c (coefficient of curvature)	0.77
6.	I. S. Classification	SP
7.	G (Specific Gravity)	2.64
8.	Φ (Friction angle)	36^0

4.2 Load settlement behaviour of various combination of modelled placed and its confinements

a) Pressure Settlement behaviour with change of cell width

First of all, the load settlement curve of dry sand is obtained for the plate of 50 mm square. It is observed that settlement increasing with the increasing of pressure as shown in Table 4. There is continuous increase of settlement at the pressure of 700kPa. Since the settlement of the plate is due to deformation of the soil inside the pressure bulb beneath the plate, due to increase in the pressure the size of pressure bulb also increases therefore settlement increases.

Table-4-Pressure – Settlement Behaviour of 50mm Plate without confinement

Applied Pressure (kPa)	Settlement (mm)
0	0
196	12
294	16
490	36
600	48
690	93

Table-5-Pressure – Settlement Behaviour of 50mm Plate with Cell of 100mm

Applied Pressure (kPa)	Settlement (mm)
0	0
490	2
1470	4
3430	6
5390	8
5495	14

Table-6-Pressure – Settlement Behaviour of 50mm Plate with Cell of 125mm

Applied Pressure (kPa)	Settlement (mm)
0	0
392	4
1764	6
2450	15
3136	30
3239	40

Table-7-Pressure – Settlement Behaviour of 50mm Plate with Cell of 200mm

Applied Pressure (kPa)	Settlement (mm)
0	0
343	9
1176	12
1960	33
2444	39
2552	43

Table-8-Pressure – Settlement Behaviour of 50mm Plate with Cell of 250mm

Applied Pressure (kPa)	Settlement (mm)
0	0
294	10
980	12
1568	37
2156	43
2250	47

b) Pressure Settlement behaviour change with different cell depth from ground level

Table-9-Pressure – Settlement Behaviour of 50mm Plate with Cell of 100mm at Ground level

Applied Pressure (kPa)	Settlement (mm)
0	0
490	2
1470	4
3430	6
5390	8
5495	14

Table-10-Pressure –Settlement Behaviour of 50mm Plate with Cell of 100mm at 200mm Depth

Applied Pressure (kPa)	Settlement (mm)
0	0
433	2
1324	4
3233	6
5300	8
5500	10

Table-11-Pressure – Settlement Behaviour of 50mm Plate with Cell of 100mm at 400mm Depth

Applied Pressure (kPa)	Settlement (mm)
0	0
420	2
1200	4
2300	6
4300	8
4600	10

Table-12-Pressure –Settlement Behaviour of 50mm Plate with Cell of 100mm at 600mm Depth

Applied Pressure (kPa)	Settlement (mm)
0	0
300	2
980	4
1500	6
2100	8
2225	10

Table-13-Pressure – Settlement Behaviour of 50mm Plate with Cell of 100mm at 800mm Depth

Applied Pressure (kPa)	Settlement (mm)
0	0
260	2
390	4
580	6
690	8
700	10

4.3 Estimation of bearing capacity

a) Bearing capacity of 50mm plate without confinement= 294 kPa

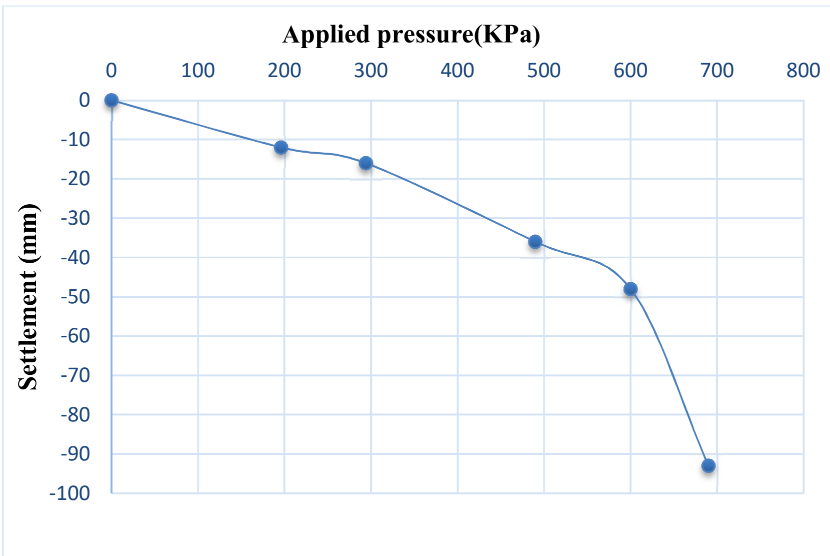

Fig-16- Applied Pressure with Settlement of 50mm Plate without Confinement

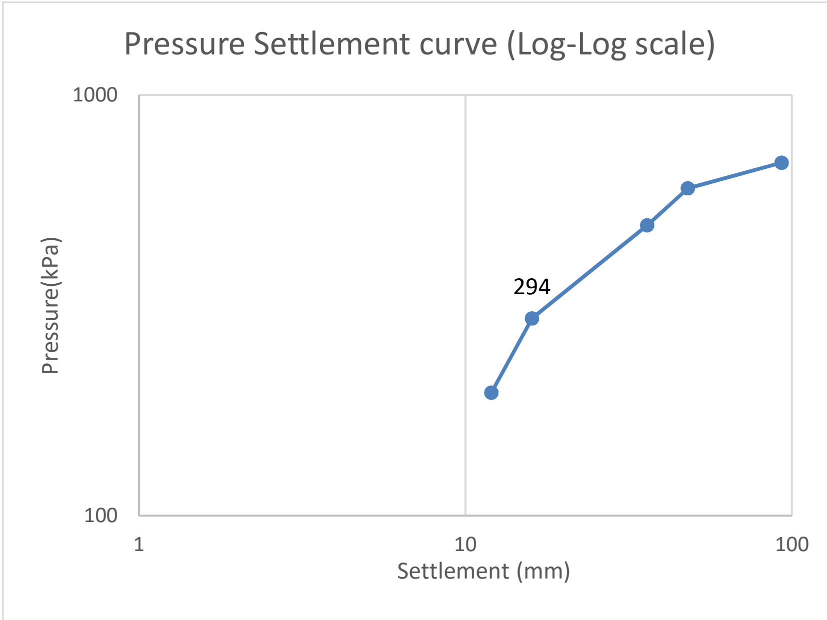

Fig-17- Pressure Settlement Curve (Log-Log scale) of 50mm Plate without Confinement.

b) Bearing capacity of 50mm plate with cell of 100mm = 5390 kPa

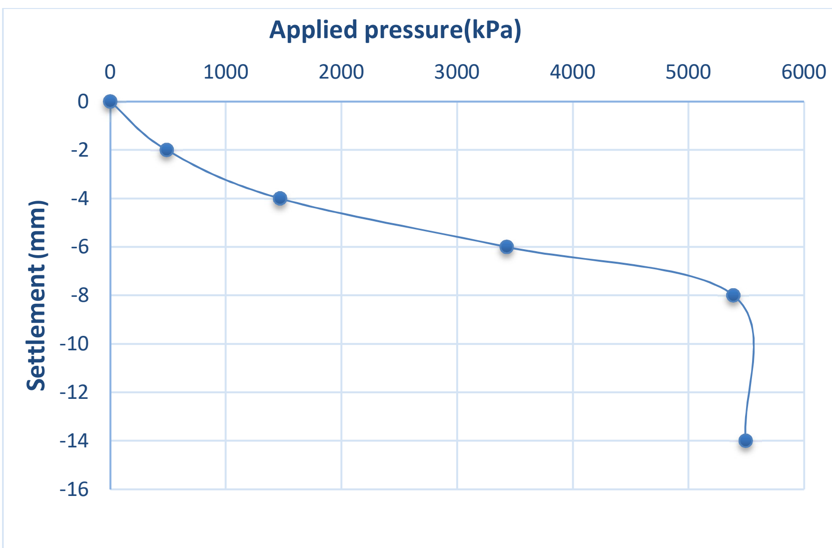

Fig-18- Applied Pressure with Settlement of 50mm Plate with Cell of 100mm

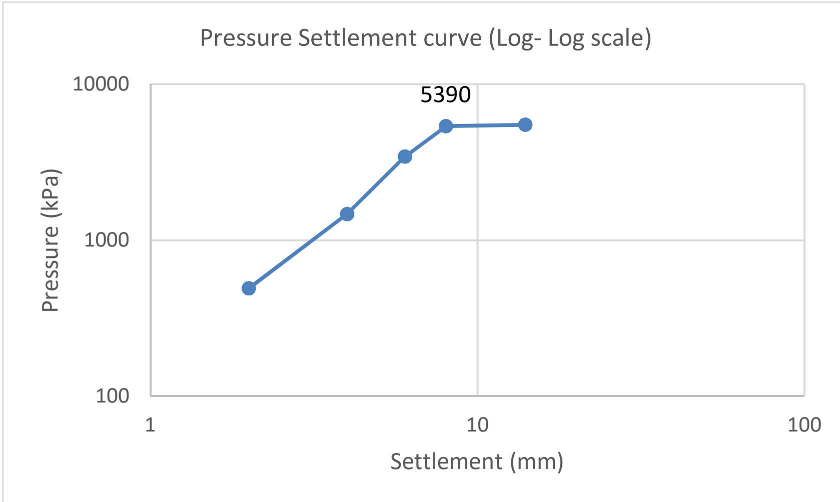

Fig-19-Pressure Settlement Curve (Log- Log scale) of 50mm Plate with Cell of 100mm

c) Bearing capacity of 50mm plate with cell of 125mm=1764 kPa

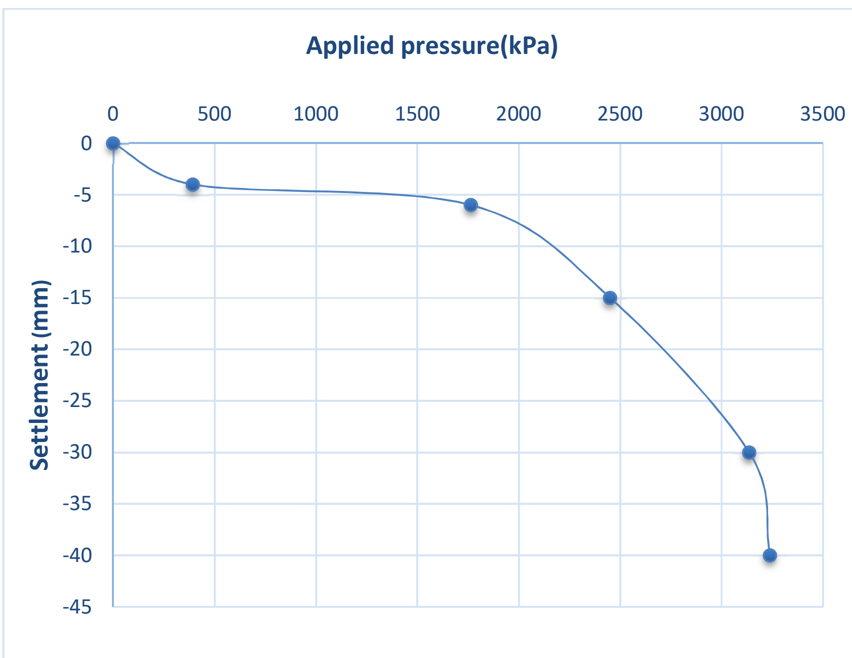

Fig-20- Applied Pressure with Settlement of 50mm Plate with Cell of 125mm

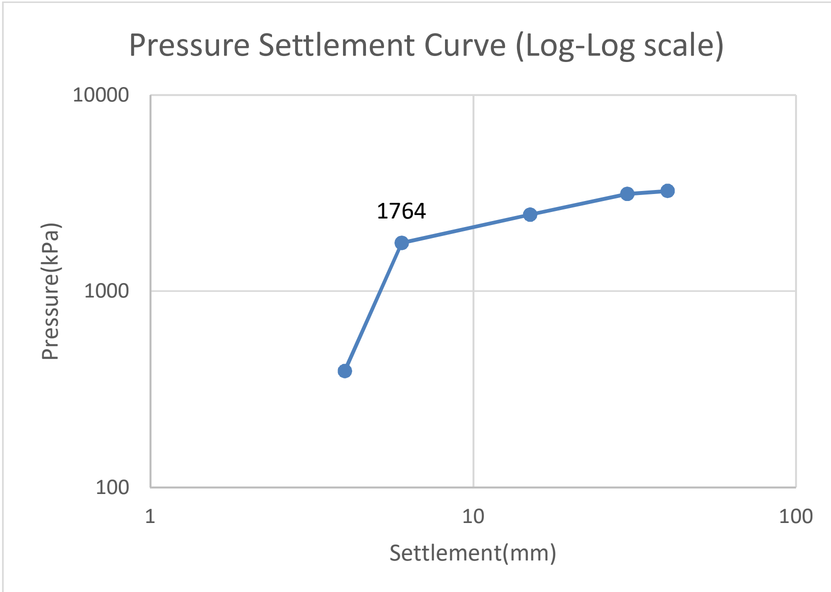

Fig-21- Pressure Settlement Curve (Log-Log scale) of 50mm Plate with Cell of 125mm

d) Bearing capacity of 50mm plate with cell of 200 mm = 1176 kPa

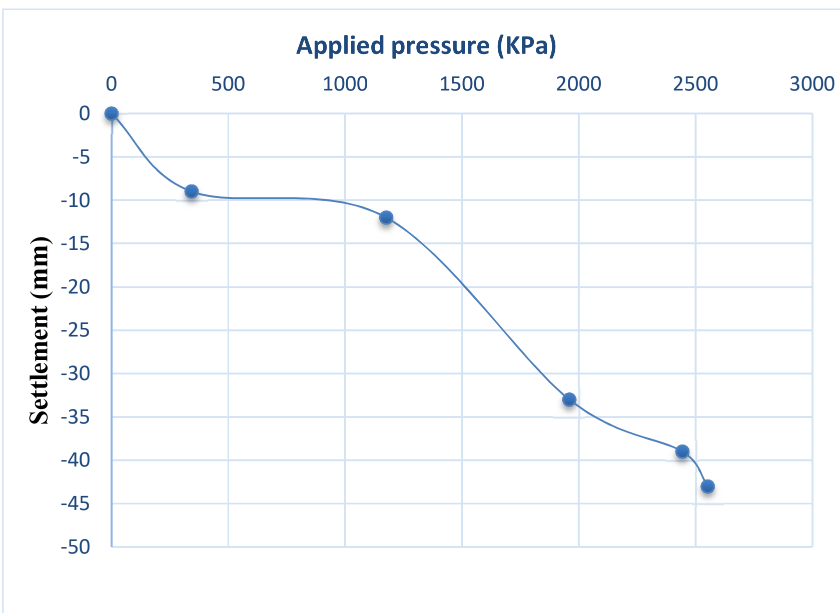

Fig-22- Applied Pressure with Settlement of 50mm Plate with Cell of 200mm

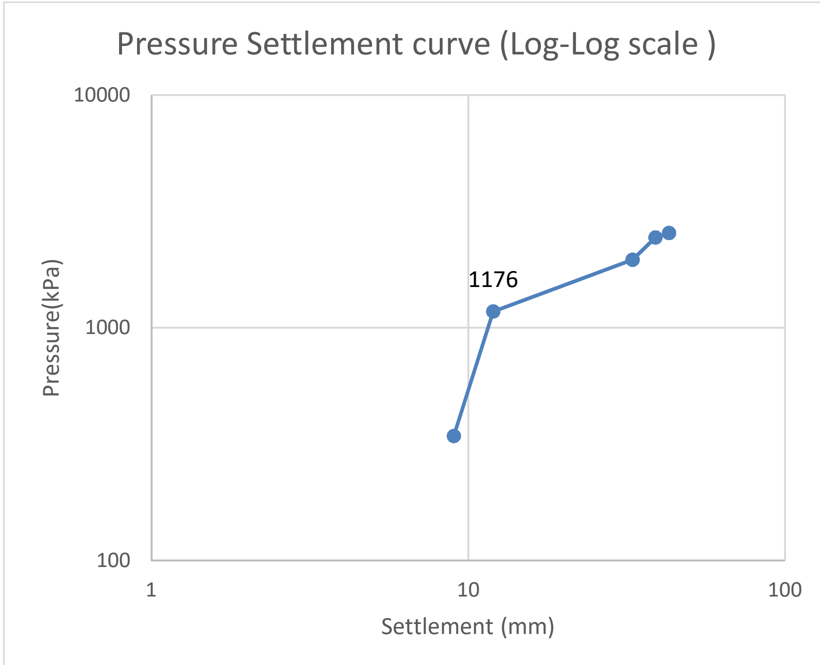

Fig-23- Pressure Settlement Curve (Log-Log scale) of 50mm Plate with Cell of 200mm

e) Bearing capacity of 50mm plate with cell of 250 mm=980 kPa

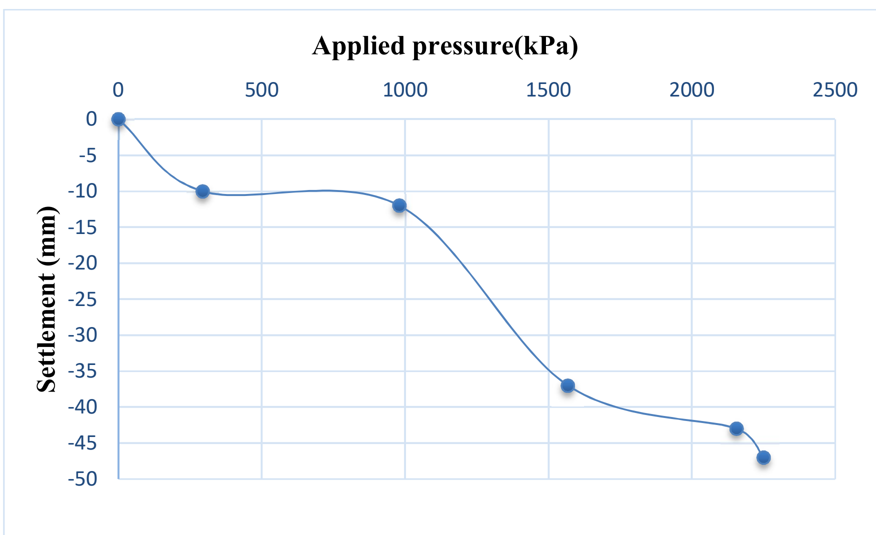

Fig-24- Applied Pressure with Settlement of 50mm Plate with Cell of 250mm

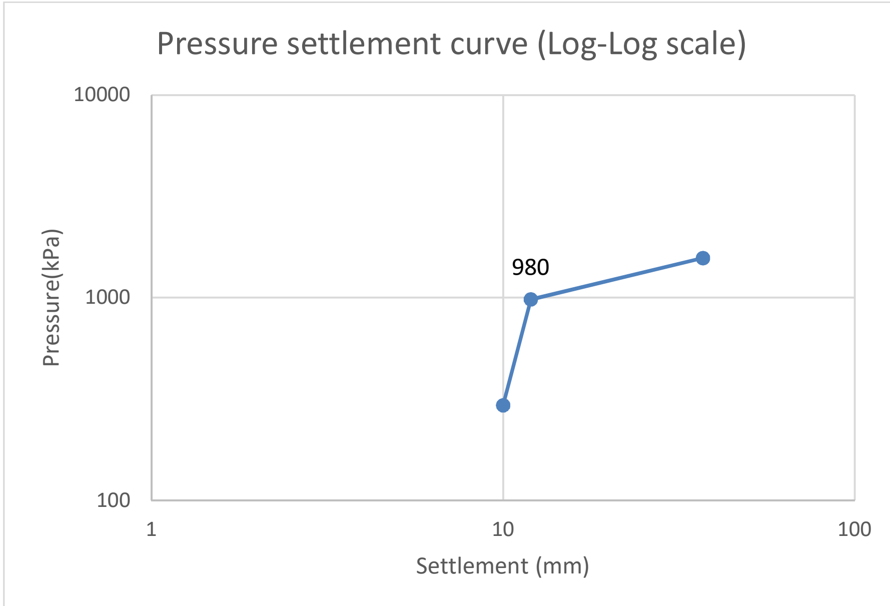

Fig-25- Pressure Settlement Curve (Log-Log scale) of 50mm Plate with Cell of 250 mm

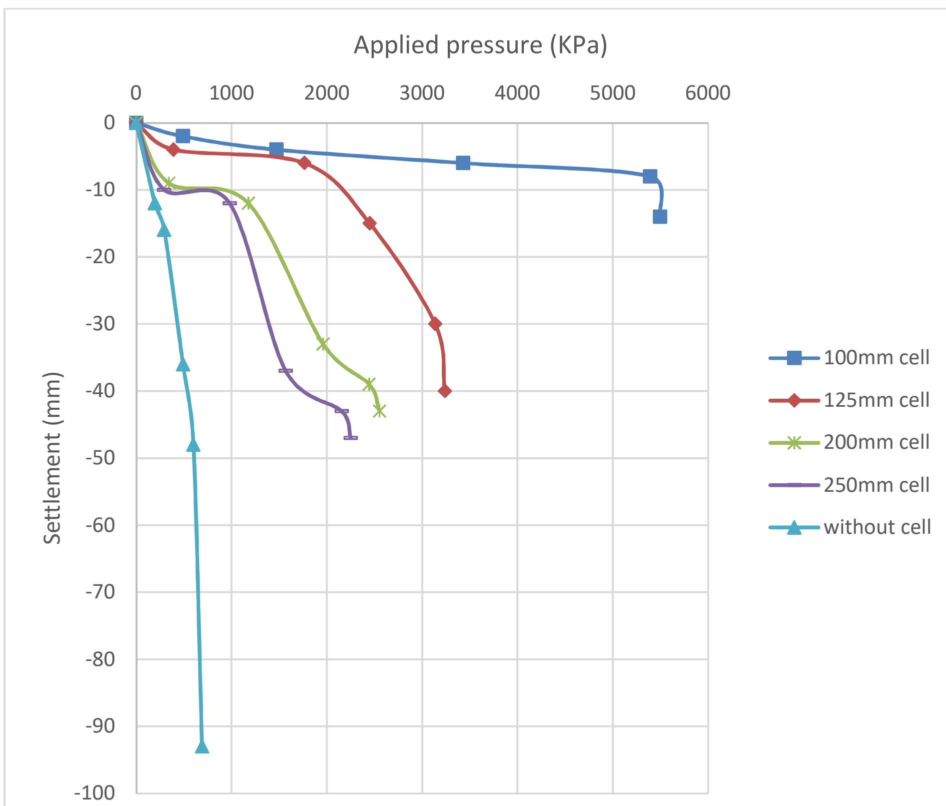

Fig-26- Pressure – Settlement behaviour of 50mm plate with cell of different width

Table-14- Ultimate Bearing Capacity of Plate with Different Cell Size

Plate size (mm)	Cell Size(mm)	Ultimate Bearing Capacity of Plate (kPa)
50	100	5390
50	125	1764
50	200	1176
50	250	980

f) Bearing capacity of 50mm plate with cell of 100mm at ground level = 5390 KPa

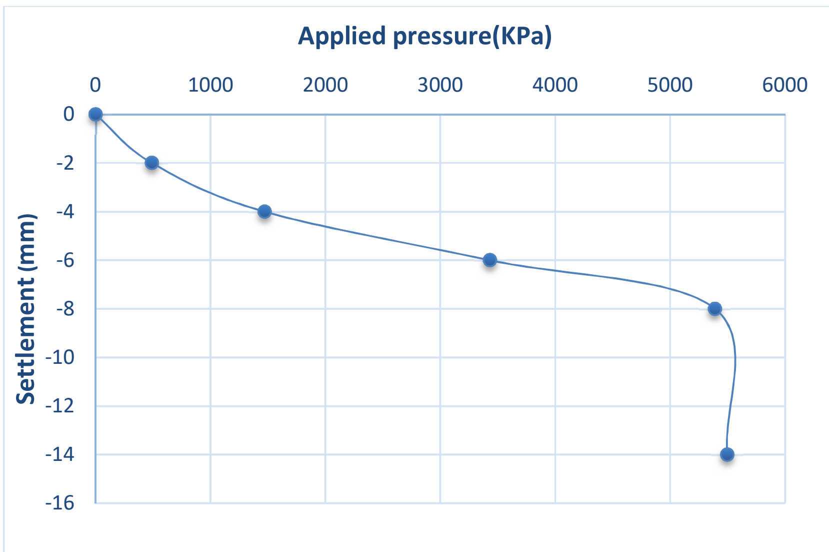

Fig-27- Applied Pressure with Settlement of 50mm Plate with Cell of 100mm at Ground Level

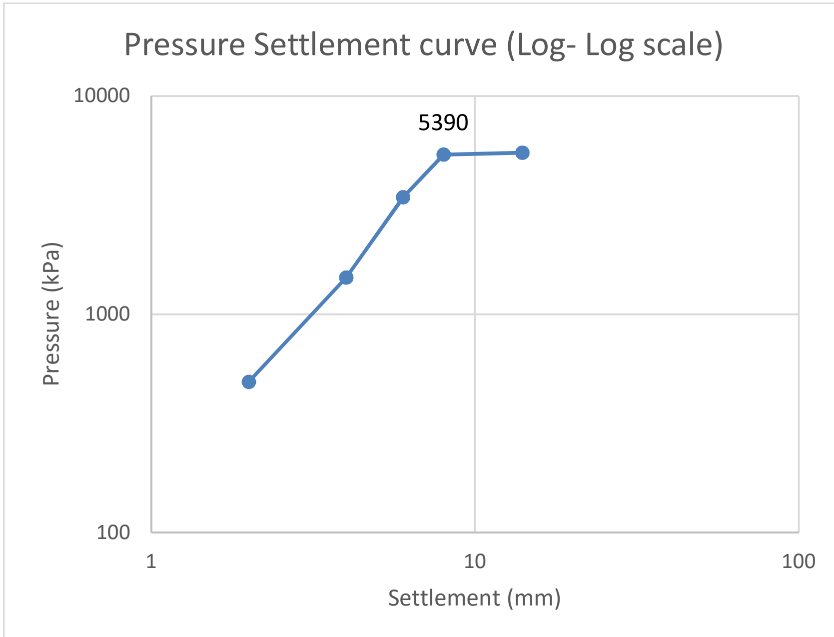

Fig-28-Pressure Settlement Curve (Log- Log scale) of 50mm Plate with Cell of 100mm at Ground Level

g) Bearing capacity of 50mm plate with cell of 100mm at 200mm depth= 5300 kPa

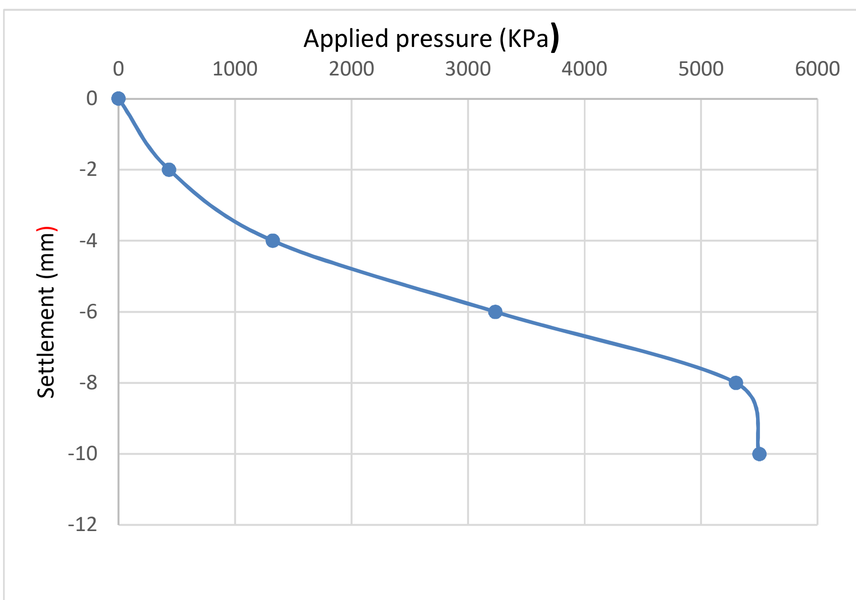

Fig-29- Applied Pressure with Settlement of 50mm Plate with Cell of 100mm at 200mm Depth

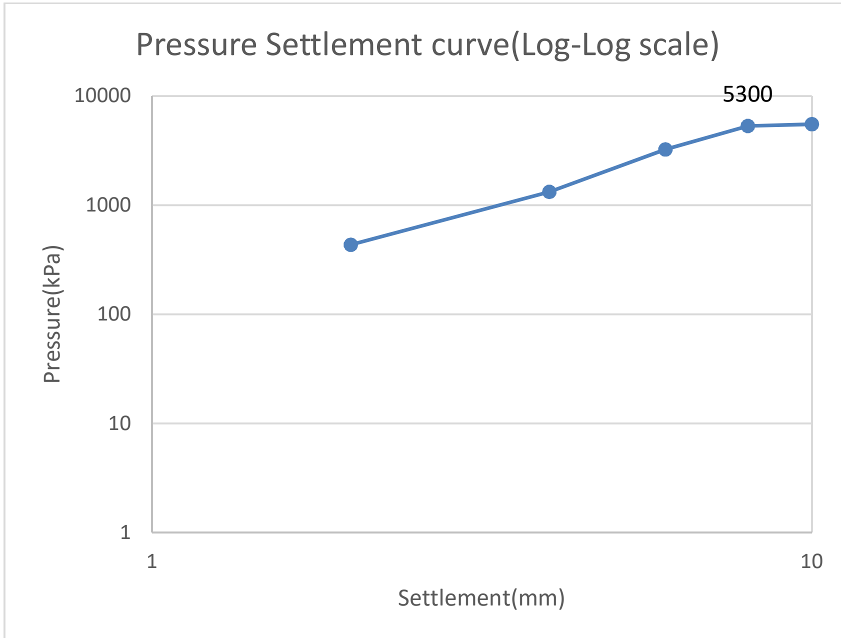

Fig-30-Pressure Settlement Curve (Log- Log scale) of 50mm Plate with Cell of 100mm at 200mm Depth

h) Bearing capacity of 50mm plate with cell of 100mm at 400 mm depth = 4300 kPa

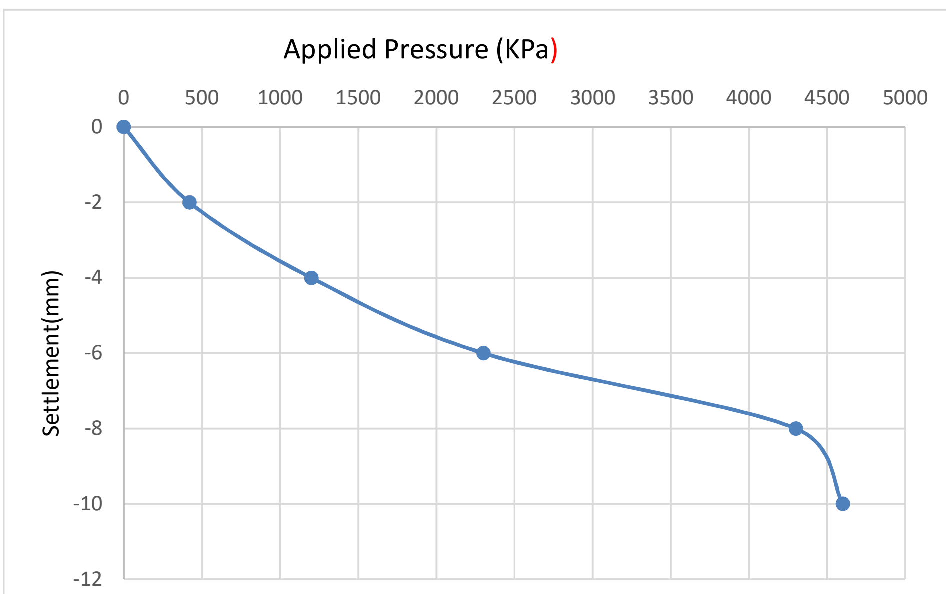

Fig-31- Applied pressure with Settlement of 50mm Plate with Cell of 100mm at 400mm Depth

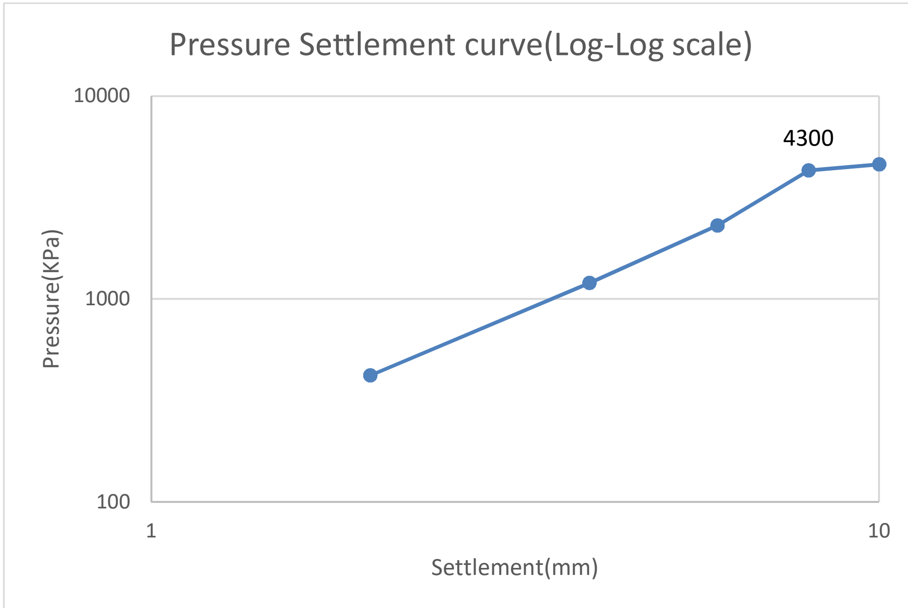

Fig-32-Pressure Settlement Curve (Log- Log scale) of 50mm Plate with Cell of 100mm at 400mm Depth

i) Bearing capacity of 50mm plate with cell of 100mm at 600 mm depth 2100 kPa

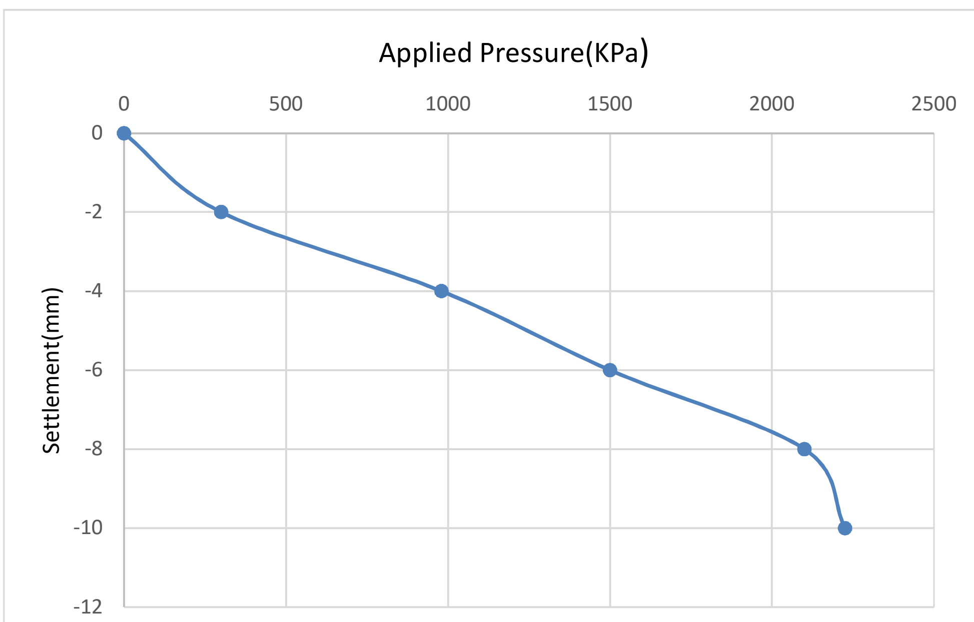

Fig-33- Applied Pressure with Settlement of 50mm Plate with Cell of 100mm at 600 mm Depth

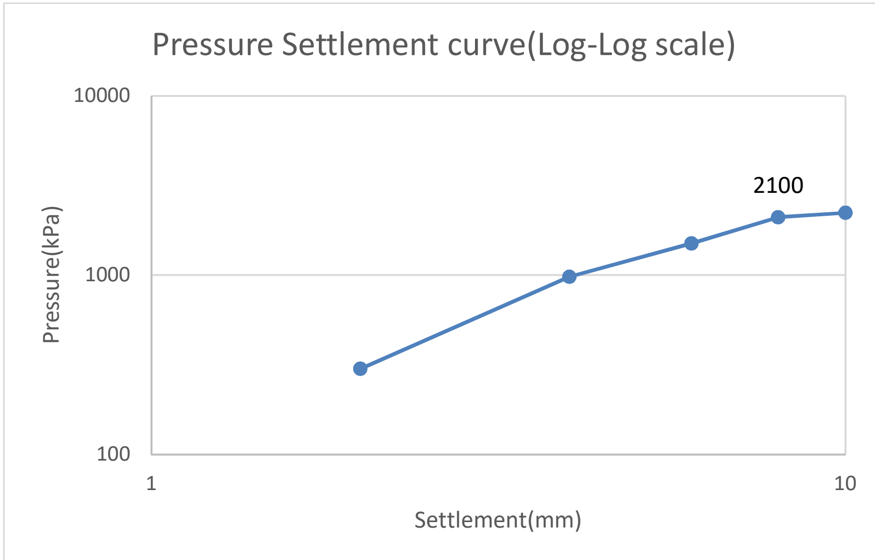

Fig-34-Pressure Settlement Curve (Log- Log scale) of 50mm Plate with Cell of 100mm at 600 mm Depth

j) Bearing capacity of 50mm plate with cell of 100mm at 800 mm depth =690 kPa

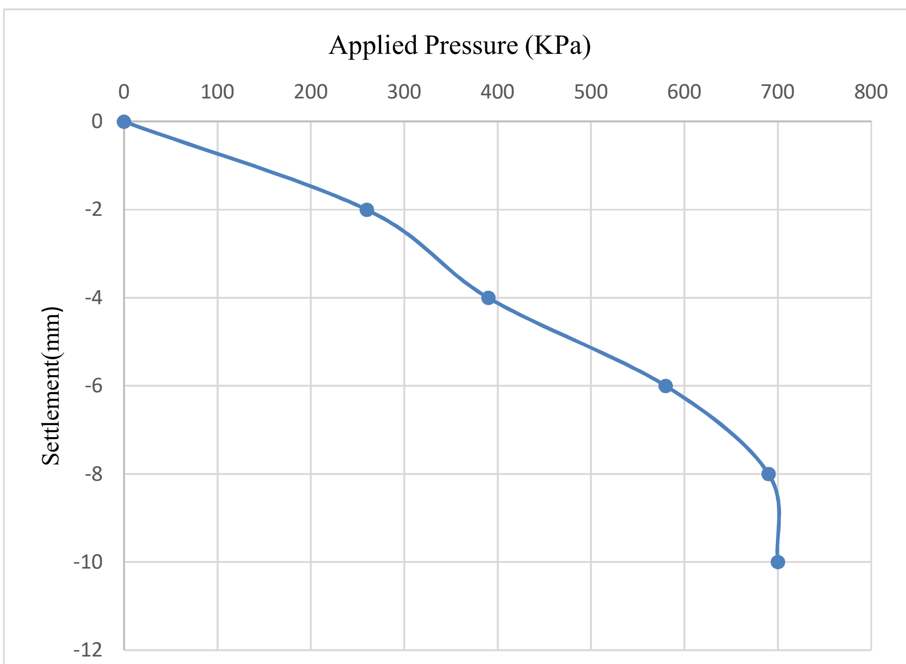

Fig-35- Applied Pressure with Settlement of 50mm Plate with Cell of 100mm at 800mm Depth

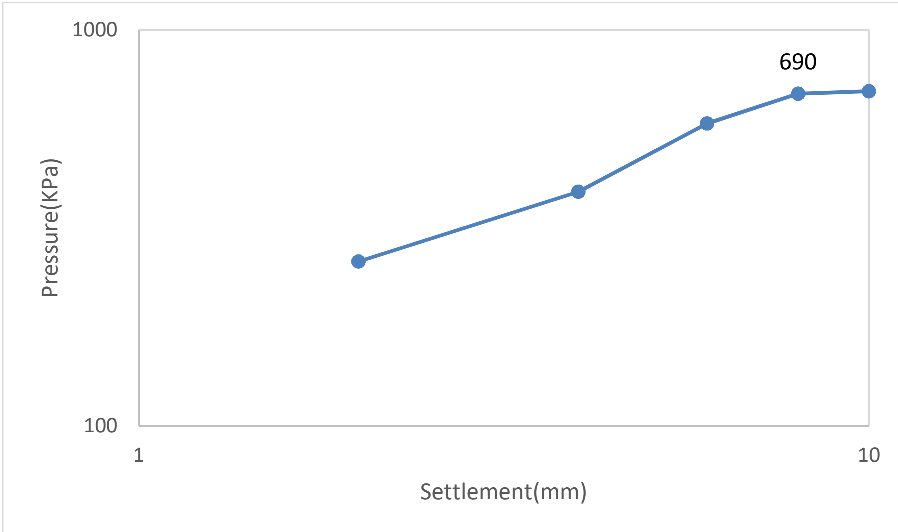

Fig-36-Pressure Settlement curve (Log- Log scale) of 50mm Plate with Cell of 100mm at 800 mm Depth

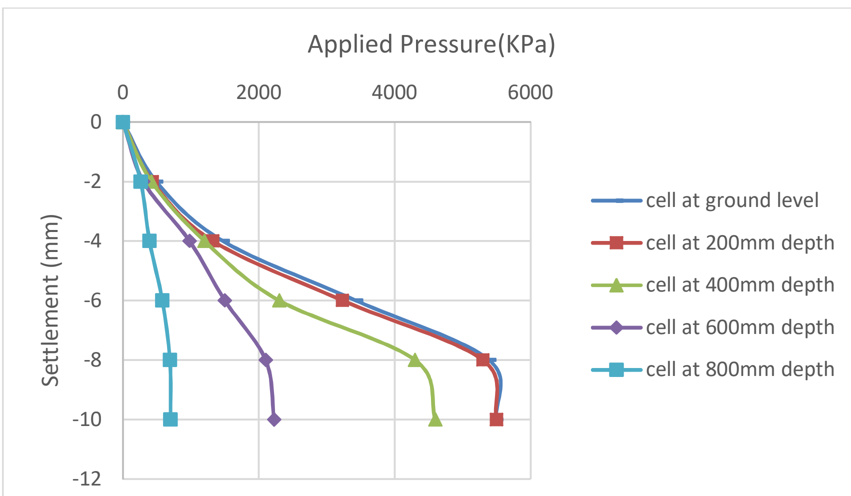

Fig-37- Pressure – Settlement Behaviour of 50mm Plate with Cell of 100mm at Different Depth from the Ground Level

Table-15- Ultimate Bearing Capacity of Plate with Same Cell Size at Different Depth

Plate size (mm)	Cell Size(mm)	Depth of cell Below Ground Level(mm)	Ultimate Bearing Capacity of Plate (kPa)
50	100	0	5390
50	100	200	5300
50	100	400	4300
50	100	600	2100
50	100	800	690

4.2 Discussion

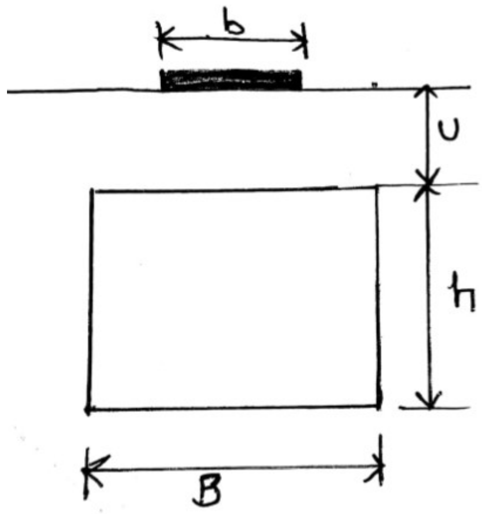

Fig-38- Geometric Parameters of Model

In given figure - **b** is width of plate, **h** is height of cell, **B** is width of cell, **u** is depth below which cell is placed from the ground level.

Non-dimensional factor is used that is Bearing Capacity Improvement Factor (BCIF), for representing the improvement of bearing capacity due to soil confinement. BCIF is defined as ratio of increase in ultimate bearing capacity with the confinement to ultimate bearing capacity without confinement.
Bearing Capacity Improvement Factor (BCIF)= (Y-X)/X
Y= Ultimate bearing capacity of sand with confinement.
X= Ultimate bearing capacity of sand without confinement.

Table-16-Bearing Capacity Improvement Factor (BCIF) Behaviour with change of {B (Width of Cell)/b (Width of plate)}

B (Width of Cell)/b (Width of plate)	Bearing Capacity Improvement Factor (BCIF)
0.5	17.33
2.5	5
4	3
5	2.3

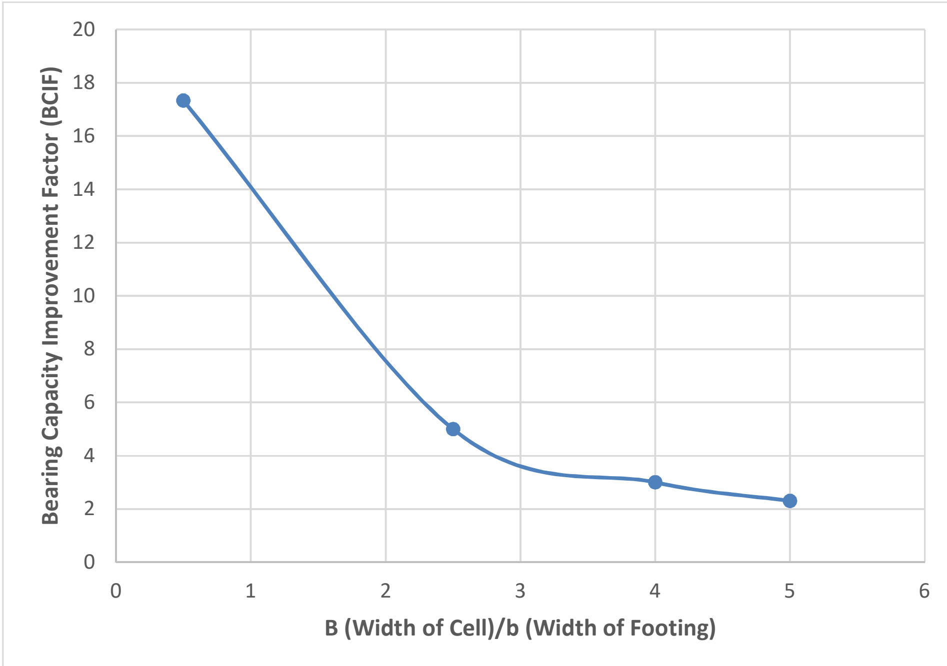

Fig-39- Variation of Bearing Capacity Improvement Factor (BCIF) with B (Width of Cell)/b (Width of plate)

In order to scrutinize the effect of cell width on the plate operation, four cells with width of 100,125,200, and 250 mm were used. Above Fig 39 shows variation of BCIF versus normalized cell width with a same plate width of 50 mm. A significant decrease in the bearing capacity of the plate supported on sand with the increase of cell width B/b was noticed. While performing the model tests, it was noticed that as failure proceeded in tests carried out with small cell width cell and sand within the cell act as one unit. When tests carried out with larger cell width, this behaviour was seen in starting, but as the load was increased it was not seen. An upgrade in bearing capacity as high as 17.33 times more than that in the absence of soil confinement. It is easy to understand that the best outcome of soil confinement could be get with a (B/b) ratio between 0.5 to 1.0 with the maximum increase in the bearing capacity at a ratio of about 0.5. As the lateral displacement is decreased due to the confinement of soil due to this vertical displacement also decrease and increase in bearing capacity is observed. This behaviour is seen up to the point till the cell, plate and sand act as one unit. The behaviour is similar to that seen in deep foundations (caissons and pile).

Table-17-Bearing Capacity Improvement Factor (BCIF) Behaviour with Change of {u (Depth of Cell Below Ground Level)/ b (Width of plate)}

u (Depth of Cell Below Ground Level)/ b (Width of plate)	Bearing Capacity Improvement Factor (BCIF)
0	6.8
4	6.6
8	5.2
12	2

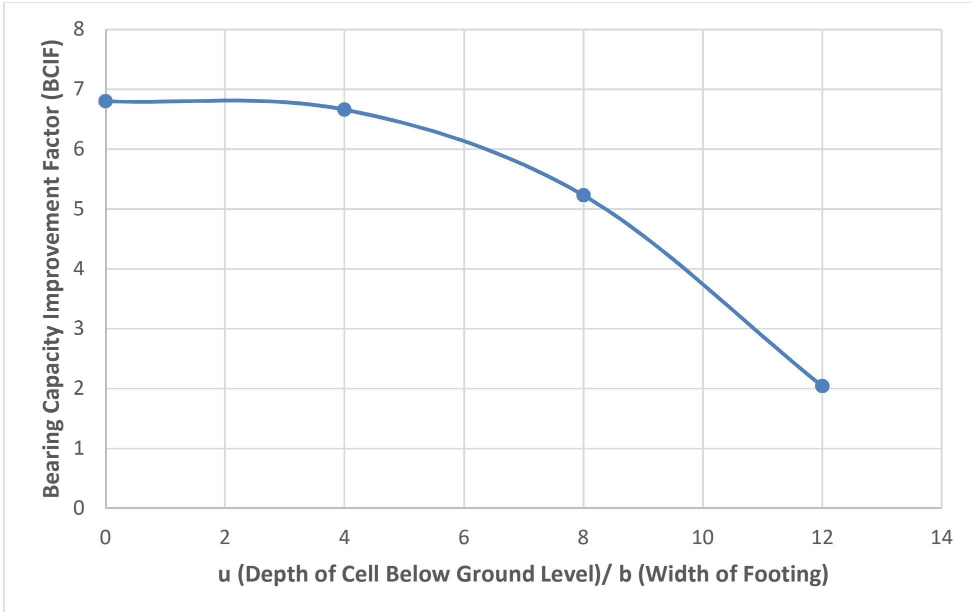

Fig-40- Variation of Bearing Capacity Improvement Factor (BCIF) with u (Depth of cell Below Ground Level)/ b (Width of plate) with different cell depth

In the above fig. variation of u/b and BCIF is shown taking 800mm depth cell as a reference. As u/b was increase BCIF started decrease. Maximum BCIF is obtained when u/b is zero that is when plate was on ground. With an advancement in the depth of plate below the ground level, the sand between the cell top and bottom of plate deforms laterally and, therefore, the BCIF decreases and the vertical settlements increase.

CHAPTER 5- CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

5.1 Conclusion

1) Bearing capacity of the confined soil is increased very significantly as compared to unconfined soil.
2) Based on the experiment, bearing capacity of isolated footing can be increased by using different width cells.
3) At the same level of settlement, load carrying capacity increasing with decrease in size of cell, in order words at any level of pressure on the plate, the settlement increasing with increase in the size of cell.
4) As depth of the placement of cell is increased, bearing capacity of the footing is decreased. This is due to the lateral displacement of the soil above the cell.

5.2 Recommendations for future work

This work can further procced with the help of circular, rectangular shape confining cells. Loading position can be change to eccentric loading and inclined loading. These works can be repeated for soils other than sand. Also the effect of water table and the effect of variation in the particle size of sand can be observed.

REFERENCES

1. A B Listyawan et al(2018)"Bearing capacity of circular skirted footing on clay soil" IOP Conference Series Materials Science and Engineering 403(1).
2. A. Kumar, M. L. Ohri, and R. K. Bansal (2007) "Bearing capacity tests of strip footings on reinforced layered soil," Geotechnical and Geological Engineering, vol. 25, no. 2, pp. 139–150.
3. Bestun J. Nareeman and Mohammed Y. Fattah (2012) "Effect of Soil Reinforcement on Shear Strength and Settlement of Cohesive- Frictional Soil" International Journal of GEOMATE, Vol. 3, No. 1 (Sl. No. 5), pp. 308-313
4. Dash, S., N. Krishnaswamy and K. Rajagopal (2001 a) "Bearing capacity of strip footing supported on geocell-reinforced sand." Geotextile and Geomembrane, 19, 535-256.
5. Dash, S., K. Rajagopal and N. Krishnaswamy (2001b) "Strip footing on geocell reinforced sand beds with additional planar reinforcement." Geotextile and Geomembrane, 19, 529-538.
6. D Chakraborty and Jyant Kumar (2014) "Bearing Capacity of Strip Foundations in Reinforced Soils" International Journal of Geomechanics 14(1):45-58.
7. D Chakraborty and Jyant Kumar (2015) "Bearing Capacity of Circular Footings on Reinforced Soils" International Journal of Geomechanics 15(1):04014034 ·
8. F.W. Jawad et al.(2019) **"**Experimental Investigation of Skirt Footing Subjected to Lateral Loading" The Open Civil Engineering Journal 13(1):20-25.
9. IS: 2720 **(**1985**)** "Methods of test for soils". Grain size analysis, Part 4
10. IS: 2386 (1963) "Methods of test for soil''. Specific gravity, Part 3
11. IS: 2720 (1986) "Methods of test for soil". Direct shear test, Part 13
12. IS: 1888 (1971) "Methods of test for soil". Direct shear test, Part 13
13. K.H. Khing, V.K. Puri et al. (1993) "Bearing capacity of strip foundation on geogrid-reinforced sand" Article in Geotextiles and Geomembranes 12(4):351–361
14. Lutenegger, A. J., and Adams, M. (2007) "Scale effects of shallow foundation bearing capacity on granular material." ASCE, 133 (10), 1192-1202.
15. Madhavi Latha Gali and Rajagopal Karpurapu (2000) "Design of Geocell Supported Embankments" Conference Paper, 2nd Asian Geosynthetics Conference, At Kuala Lumpur, Malaysia.
16. Meyerhof, G.G. (1965) "Shallow foundations", J. of SMFD, ASCE, Vol. 91, SM2, 21-31.
17. Musa Alhassan and I L Boiko (2013) "Effect of vertical cross-sectional shape of foundation and soil reinforcement on settlement and bearing capacity of soils" International Journal of Civil Engineering and Technology 4(2):80-88·
18. Prof. Harish and Shravan Balu (2016) "A Model Study to Analyze the Improvement of Bearing Capacity of A Square Footing on Soft Soil with Sand Pile With or Without Confinements" International Research Journal of Engineering and Technology (IRJET) , Volume: 03.

19. Rajagopal, K., N. Krishnaswamy and G. Latha (1999) "Behavior of sand confined with single and multiple geocells", Geotextile and Geomembrane, 17, 171-184.
20. Rea, C. and J.K. Mitchell (1978) "Sand reinforcement using paper grid cells", Proc. Symposium on Earth Reinforcement, Pittsburg, ASCE, 644-663.
21. Renaningsih et al (2017)"Method to increase ultimate bearing capacity of skirted circular footing"AIP Conference Proceedings.
22. Sawwaf, M.E., and A. Nazer (2005) "Behavior of circular footing resting on confined granular soil." Journal of Geotechnical and Geoenvironmental Engineering, 131(3), 359-366.
23. V. K. Singh, A. Prasad and R.K. Aggarwal (2007) "Effect of Soil Confinement on Ultimate Bearing Capacity of Square Footing under Eccentric–Inclined Load." Electronic Journal of geotechnical Engineering, Vol.12.
24. V. A. Guido, D. K. Chang, and M. A. Sweeney, "Comparison of geogrid and geotextile reinforced earth slabs," Canadian Geotechnical Journal, vol. 23, no. 4, pp. 435–440, 1986.
25. Verma, B.P., and A.N.R. Char (1986) "Bearing capacity tests on reinforced sand subgrades," Journal of Geotechnical Engineering. 112(7), 701-706.
26. Vesic, A.S. (1973) "Analysis of ultimate loads of Shallow foundations." Journal of Soil Mechanics and Foundation Division, Vol. 99(1), 45-73.