Correlation of Soil Parameters and Bearing Capacity Analysis of Piled Raft Foundation for Dhaka - Chittagong Elevated Expressway

Tansir Zaman Asik

Sakif Ibna Matin

Mashuk Rahman

ISLAMIC UNIVERSITY OF TECHNOLOGY

2016

Correlation of Soil Parameters and Bearing Capacity Analysis of Piled Raft Foundation for Dhaka - Chittagong Elevated Expressway

Tansir Zaman Asik
Sakif Ibna Matin
Mashuk Rahman

(125409)
(125408)
(125403)

A THESIS SUBMITTED

FOR THE DEGREE OF BACHELOR OF SCIENCE IN CIVIL ENGINEERING

DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

ISLAMIC UNIVERSITY OF TECHNOLOGY

2016

PROJECT REPORT APPROVAL

The thesis titled "Correlation of Soil Parameters and Bearing Capacity Analysis of Piled Raft Foundation for Dhaka - Chittagong Elevated Expressway" submitted by Tansir Zaman Asik, Sakif Ibna Matin and Mashuk Rahman, St. No. 125409, 125408 and 125403 has been found as satisfactory and accepted as partial fulfillment of the requirement for the Degree Bachelor of Science in Civil Engineering.

SUPERVISOR

Dr. Hossain MD. Shahin

Professor

Department of Civil and Environment Engineering (CEE)

Islamic University of Technology (IUT)

Board Bazar, Gazipur, Bangladesh

DECLARATION OF CANDIDATE

We hereby declare that the undergraduate research work reported in this thesis has been performed by us under the supervision of Professor Dr. Hossain MD. Shahin and this work has not been submitted elsewhere for any purpose (except for publication).

Dr. Hossain MD. Shahin
Professor,
Department of Civil and
Environment Engineering (CEE)
Islamic University of Technology (IUT)
Board Bazar, Gazipur, Bangladesh
Date:

Tansir Zaman Asik
Student No: 125409
Academic Year: 2015-16
Date: /11/2016

Sakif Ibna Matin
Student No: 125408
Academic Year: 2015-16
Date: /11/2016

Mashuk Rahman
Student No: 125403
Academic Year: 2015-16
Date: /11/2016

DEDICATION

We dedicate our thesis work to our family. A special feeling of gratitude to our loving parents.

We also dedicate this thesis to our many friends who have supported us throughout the process. We will always appreciate all they have done.

ACKNOWLEDGEMENTS

"*In the name of Allah, Most Gracious, Most Merciful*"

All the praises to Allah (SWT) for giving us the opportunity to complete this book. We wish to express our sincere gratitude to Professor Dr. Hossain MD. Shahin for providing us with all the necessary facilities, giving undivided attention and fostering us all the way through the research. His useful comments, remarks and engagement helped us with the learning process throughout the thesis. We are also grateful to Sultan Al Shafian, research assistant, in the Department of Civil & Environmental Engineering. We want to wish our heartiest gratitude and thanks to 'Noor-Zaman Engineering Foundation' and 'SMEC International Pty Ltd' for their helping hand to us for performing field level soil investigation and collection soil sample from different depths in soil layers from selected places. We would like express gratitude to all of the departmental faculty members for their help and support. We are also grateful to our parents for their encouragement, support and attention and for being ravished patrons.

We also place on record, our sense of gratitude to one and all, who directly or indirectly, have contributed to this venture.

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

CHAPTER 5 Results and Discussions

CHAPTER 6 CONCLUSION

ABSTRACT

Keywords: Soil parameters, Correlation, Compression Index, Constitutive Model, Bearing Capacity, Settlement, Finite Element Method etc.

Determination of soil parameters is one of the important tasks in geotechnical engineering. Sometimes, in many projects only basic soil parameters such as index properties of soils are determined through laboratory tests. In such cases, correlations of soil parameters available in literature are used in getting the other necessary soil parameters. In many soils, parameters determined by using available correlations significantly differ from the test results. In this research the validity of the correlations of soil parameters available in literature has been checked for the ground where Dhaka-Chittagong expressway will be constructed. Here, the correlation of compression index (C_c) which is an important soil parameter for clayey soils is taken into consideration. There are many empirical formulae that relate compression index to other soil parameters such as void ratio (e_0), liquid limit (L_L), natural water content (w_n) and many others. Several different tests have been carried out to determine compression index (C_c), void ratio (e_0), liquid limit (LL), and natural water content (w_n). It is found the available correlations of the compression index cannot properly express the test results for the ground investigated in this research. Therefore, three correlations for soil parameters relating compression index and liquid limit, compression index and void ratio, compression index and natural water content have been proposed in this study. Elastoplastic constitutive model parameter identification is an important task for proper modeling of any soil. In this research, subsoil characteristics of study locations are presented based on field and laboratory test results. Elasto-plastic constitutive model parameters of study locations soil has been determined for extended sub-loading tij model. Using these parameters, bearing capacity of piled raft has been estimated for 0.05% settlement of soil section. Considering the effect of settlement in 2D Finite Element analysis have been conducted. It is found that bearing capacity determined by the conventional methods match well with the results of the numerical simulations.

LIST OF SYMBOLS

C_c	Compression index
C_s	Swelling index
e_o	Initial void ratio
LL	Liquid limit
PL	Plastic limit
PI	Plasticity index
W_n	Natural water content
G_s	Specific gravity
G_T	Specific gravity at T^0 C
Δe	Variation of void ratio
$\Delta \log\sigma'$	Variation of effective stress
W_s	Weight of dry soil
V	Volume of soil sample
P_c	Preconsolidation pressure
q_u	Ultimate compression stress
σ	Stress
€	Strain
λ	Compression index for FEM *tij* simulation
Ḱ	Swelling index for FEM *tij* simulation

LIST OF SYMBOLS

RCs	Critical stress ratio
N	Void ratio at 98 KPa
β	Shape of yield surface
aAF	Influence of density
aIC	Influence of confining pressure
a_v	Co-efficient of compressibility
m_v	Co-efficient of volume compressibility
C_v	Co-efficient of consolidation
C_u	Undrained shear strength

LIST OF FIGURES

LIST OF FIGURES

LIST OF TABLES

Chapter 1 Introduction

1.1 General

The purpose of laboratory test in geotechnical engineering is to find out the soil parameters which will be used to performing analyses of full-scale lateral load tests. The laboratory tests included soil classification, unit weight, strength and consolidation. Sometimes it is not possible to determine all the parameters of soil due to different problems. But there are empirical equations established. From these equations if we know the value of one parameter, we can determine others. We have established some correlation between soil parameters for our country soil and we have compared them with the existing equations. The basic parameters of soil are water content, void ratio, liquid limit and compression index. We have established correlation between compression index (C_c) and liquid limit (LL), void ration (e_0) and water content (W_n). The word water content means the ratio of the weight of water and the weight of solid particles. It indicates the weight percent of moisture compared to the mass of the solid phase of soil. Void ratio indicates the compactness of soil. It may be defined as the ratio of volume of void space and the volume of solids. When water is added to dry soil, it changes its state of consistency from hard to soft. Liquid limit can be defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flow. In another word the amount of water which is responsible for the change of consistency of soil is called liquid limit.

Chapter 1 Introduction

Compression index indicates the variation of the void ratio (e_0) as a function of the change of effective stress plotted in the logarithmic scale. To calculate the compression index oedometer test is performed. The standard oedometer test is one of the most commonly used tests in geotechnical laboratory testing program. Oedometer test is very important in the field of geotechnical engineering but it is very time consuming and if we miss one data then we cannot calculate the actual result. To avoid Oedometer test we can use some empirical equation for finding out the compression index, (C_c).

The problem of designing deep foundations is related to many civil engineering structures as it is becoming more common and frequent to construct structures on soft soils. Pile foundation is a popular deep foundation type used to transfer superstructure load into subsoil and bearing layers. However, accurate prediction of piles' settlement is particularly difficult concerning complicated consolidation process and pile-soil interaction. [Kazimierz, 2015] Piles are commonly used to transfer superstructure load into subsoil and a stiff bearing layer. As it was emphasized by [Lambe and Whitman, 1969], a pile foundation, even in the case of single pile, is statically indeterminate to a very high degree. The proper solution to a given pile foundation problem requires empirical knowledge and the results of pile tests at the actual site.

1.2 Objectives of the study

1. To determine values of different soil parameters from laboratory tests.
2. To check the validity of existing correlations between the parameters for our country soil.
3. To make correlation between the parameters.
4. To calculate load bearing capacity of piled raft foundation.

1.3 Scope of the study

1. The identified correlations can help us to determine values of certain parameters in future.
2. There will be no need to find all the values of the all parameters by performing difficult laboratory tests.
3. This will minimize both duration and cost of a project.
4. Load bearing capacity of piled raft foundation of soil can make us identified the ultimate bearing capacity of that soil.

Chapter 2 Literature Review

2.1 General

Literature review has been done to identify the so far studies related to this field. Literature review for our research has divided into two parts: one is correlation of soil parameters and another is load bearing capacity of piled raft foundation.

2.1.1 Correlation of soil parameters

Over the decades, various empirical models have been developed to correlate Cc with various index properties of soils such as the liquid limit, natural water content, plasticity index, specific gravity and void ratio [Skempton,1944; Nishida,1956; Cozzolino,1961; Terzaghi and Peck,1967; Sowers,1970; Azzouzetal,1976; WrothandWood,1978; Mayne,1980; Park and Lee,2011; Mohammadzadeh, 2014] Table 2.1 presents some of the well-known empirical prediction equations in this field. Nearly all these relationships were derived by performing multiple linear regression analysis.

Table 2.1: Some of the well-known empirical prediction equations for C_c

Reference	Equation	Applicability
Skempton (1944)	C_c=0.007(LL-10)	Remolded clays
Nishida (1956)	C_c=1.15(e_0-0.35)	All clays
Cozzolino (1961)	C_c=0.43(e_0-0.11)	Brazilian clays
Terzaghi and Peck (1967)	C_c=0.009(LL-10)	Normally consolidated clays
Sowers (1970)	C_c=0.75(e_0-0.50)	Soils of very low plasticity
Azzouz etal. (1976)	C_c=0.40(e_0-0.25) C_c=0.01(ω-5) C_c=0.006(LL–9)	All natural soils
Wroth and Wood(1978)	C_c=0.50×PI×G_s	All remolded normally consolidated clays
Mayne (1980)	(LL-13)/109	All clays
Koppula (1981)	0.01ω	Chicago and Alberta clays
Herrero (1983)	0.01ω-0.075	Normally consolidated clays
Nagaraj and Murty (1985)	C_c=0.2343(LL/100)G_s	All inorganic clays
Park and Lee (2011)	C_c=0.49(e_0-0.11) C_c=0.014(LL-0.168)	Korean natural soils

2.1.2 Load bearing capacity of piled raft foundation

Bearing capacity is estimated by limit analysis using upper bound and lower bound theory. Therefore, in estimation of bearing capacity such parameters should be considered. Now-a-days Finite Element Method is widely used in different fields of Geotechnical Engineering. So, such condition can also be applied for bearing capacity estimation. However, the accuracy of the FE analysis depends on the constitutive models of soils. Available constitutive models such as Cam-clay model (Roscoe and Burland, 1968), Drucker-Prager Model, Mohr-Coloumb Model cannot properly consider or explain soil behavior of different densities. However, in this study extended sub-loading *tij* model [Nakai and Hinokio, 2004; Nakai , 2011] is used which can consider influence of intermediate principal stress on the deformation and strength of soils, dependence of the direction of plastic flow on the stress paths, influence of density and/or confining pressure and bonding effect on the deformation and strength of soils [Shahin, 2004; Nakai, 2010; Nakai, 2011].

2.2 Summary

From different research paper review we have come to know that different correlation between soil parameters are identified for different countries, but it is not available for Bangladeshi soil. And in bearing capacity estimation sub-loading *tij* model for FEM analysis is very much convenient.

Chapter 3 Methodology

3.1 General

As the study has a wide insight on a variety of aspects, different methods were adopted in order to achieve the objective of this study properly. And by implementing these methods, a direct approach has been set out to fulfill the scope of the study. In this chapter, the methods adopted and implemented are discussed thoroughly.

3.2 Study Area

For the soil investigation, we have selected several places (figure 3.1) in Narayanganj (23.60°N to 90.50°E with an area of 683.14 km^2) and Comilla (23.27°N to 91.12°E with an area of 3,146.30 km^2) districts. In Narayanganj, we have collected soil from Sonargaon (23°38′51″N to 90°35′52″E with an area of 171.02 km^2) and Bandar (23°37′N to 90°31.5′E with an area of 55.84 km^2) upazilla. In Comilla, we have collected soil from Comilla Sadar Dakshin (23°22′N to 91°12′E with an area of 241.66 km^2) and Chouddagram (23°13′N to 91°19′E with an area of 268.48 km^2) upazilla. All these locations are shown in figure 3.1. In this study, the physical and geotechnical properties are carried out with the help of field observations and different laboratory tests.

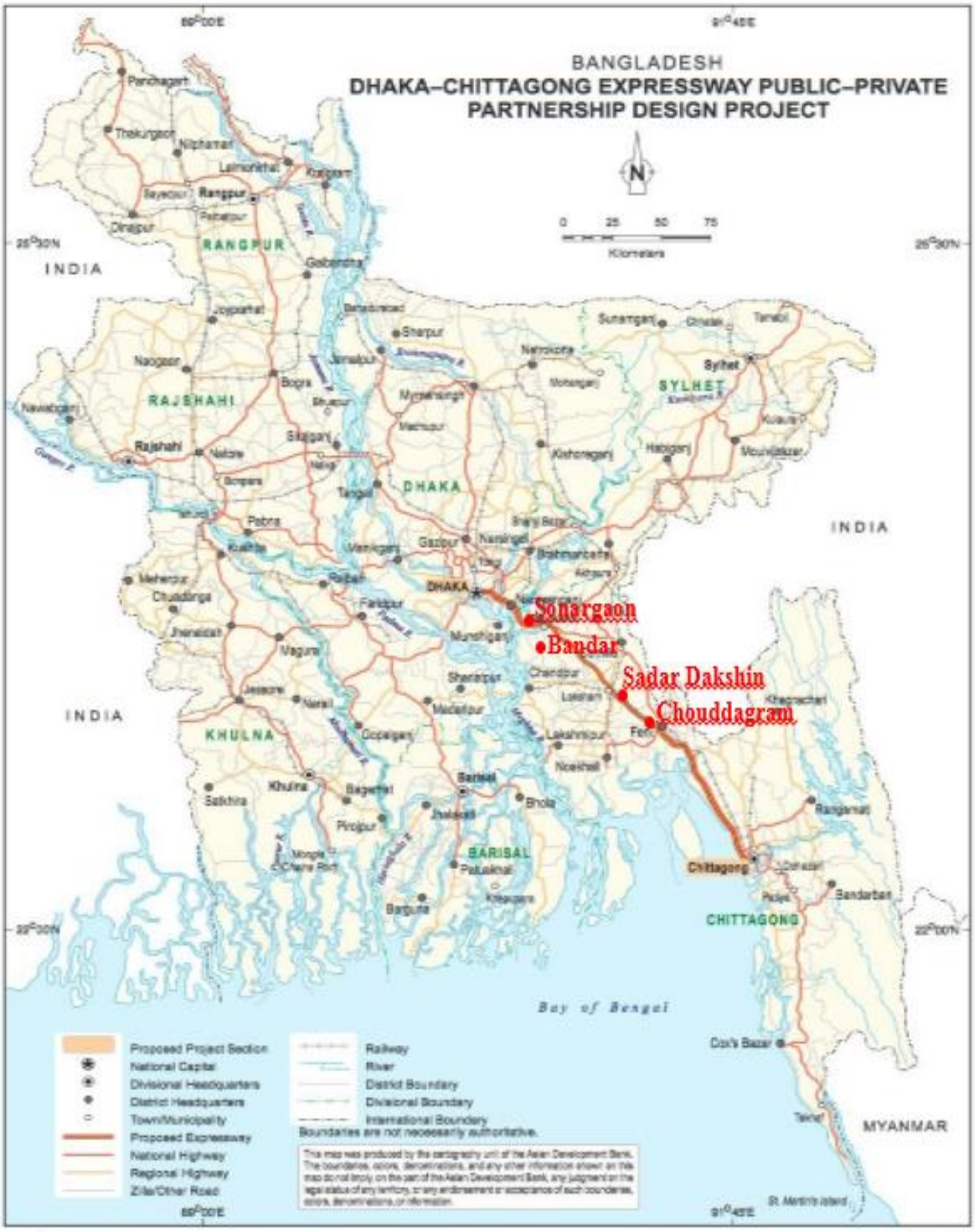

Figure 3.1. Proposed Dhaka-Chittagong elevated expressway

3.3 Material Collection

Soil samples are collected as boring sample using Shelby tubes. It is thin-walled open-tube samplers are designed for taking samples in soft and firm cohesive soils. These samplers have a much lower area ratio (approximately 10%) than U100 samplers and therefore give less disturbed samples. However, some disturbance is caused due to friction of the sample on the inside of the sample tube. Each tube has one end that is chamfered to form a cutting edge and the upper end includes holes for securing the tube to a drive head. Shelby tubes are useful for collecting soils that are particularly sensitive to sampling disturbance, including fine cohesive soils and clays. The tubes can also be used to transport samples back to the lab as well.

Figure 3.2. Shelby Tubes

So, the samples were undisturbed. The length of the each tube was 450 mm. We have collected samples from different depths of earth i.e. 5m, 10m, 15m, 20m and 30m below from the earth surface. These samples are then tested in laboratory by different experimental procedures.

Figure 3.3. Sample extraction from Shelby tube

3.4 Laboratory Experiments

We have performed several laboratory tests in the laboratory to determine various soil parameters. The tests we have performed are described briefly here.

3.4.1 Moisture Content of Soil

Water content or moisture content is the quantity of water contained in a material, such as soil (called soil moisture).

We have determined moisture content of soil. We have followed procedure described below:

i) Clean the container, dry it and weigh it with the lid (Weight ‘M_1’).

ii) Take the required quantity of the wet soil specimen in the container and weigh it with the lid (Weight ‘M_2’).

iii) Place the container, with its lid removed, in the oven till its weight becomes constant (Normally for 24hrs.).

iv) When the soil has dried, remove the container from the oven, using tongs.

v) Find the weight ‘M_3’ of the container with the lid and the dry soil sample.

Figure 3.4. Weight measurement of can

Water content or Moisture content of soil is measured to find out the quantity of water the soil sample has. We use the following formula to measure moisture content:

$$W_N = \frac{M2 - M3}{M3 - M1} * 100$$

An average of three determinations had been taken. The data we got is shown in Table 3.1.

Where,

W_n = Moisture content of soil (%)

M1 = Mass of empty can

M2 = Mass of wet soil + Can

M3 = Mass of dry soil + Can

Table 3.1: Moisture content measuring of soil sample

Moisture Content			
Can no.	43	66	54
can mass, g (M1)	28.90	26.60	34.30
Mass of wet soil + Can (M2)	53.90	56.60	69.30
Mass of dry soil + Can (M3)	48.10	49.50	61.10
Mass of dry soil, g (M3-M1)	19.20	22.90	26.80
Water content, %	30.21	31.00	30.60
Average	30.60		

A sample calculation:

Weight of water, M2-M3= 53.90-48.1=5.8

Weight of solid, M3-M1= 48.10-28.90=19.20

Water Content, W_n= (5.8/19.20)*100%=30.21%

3.4.2 Specific Gravity of Soil

Specific gravity (G_s) is defined as the ratio of the weight of an equal volume of distilled water at that temperature both weights taken in air.

We have determined specific gravity of soil. We have followed procedure described below:

i) First we had cleaned and dried pycnometer. Then we had taken water into the pycnometer up to the mark and taken weight W1

ii) Then we had put the water out and taken 50 gm of oven dried soil in the pycnometer and took some water into it.

iii) Then we took the pycnometer and submerged it into boiling water and stirred it for 10 minutes. After 10 minutes we pulled the pycnometer out of water and kept it in rest to get cool down.

iv) After that we filled the pycnometer up to mark with water and taken weight W2. We have determined the water temperature and from chart we got specific gravity of water at that temperature.

v) Then from these value we calculated specific gravity three times and taken the average value.


Figure 3.5. Laboratory test of determination of Specific gravity of soil.

We have measured specific gravity (G_s) of soil samples (Table 3.2), to calculate the soil properties like Void Ratio (e_0), Degree of Saturation etc. Data we collected during the test:

Table 3.2: Specific gravity measuring of soil sample

Determination No	1	2	3
Pycnometer No	1	2	3
Evaporating Dish No	15	10	28
Weight of dry Soil, w_s (gm)	50	50	50
Weight of Pycnometer + water (filled to the mark)= W_1 (gm)	352.92	353.82	355.92
Temperature of the Water, T^0 C	33	33	33
Weight of Pycnometer + Water (filled to the mark) + Soil= W_2	384.08	385.12	387.47
Weight of equal volume of water as the soil solids= W_w(gm)=(W_1+Ws)-W_2	18.84	18.7	18.45
Specific Gravity of Water= G_T at T^0 C	0.9957	0.9957	0.9957
Gs at T^0 C=(W_s/W_w) x G_T	2.64	2.66	2.70
Average specific gravity, G_s		2.67	

Sample Calculation:

Weight of dry Soil, w_s = 50gm

Weight of Pycnometer + water (filled to the mark)= W1 = 352.92gm

Weight of Pycnometer + Water (filled to the mark) + Soil= W2 = 384.08gm

Weight of equal volume of water as the soil solids= W_w =(W_1+Ws)-W2 =18.84gm

Specific Gravity of Water= GT = 0.9957

Specific gravity, Gs = (Ws/Ww)*GT = 2.64

3.4.3 Atterberg Limit of Soil

Liquid Limit is the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flow. The water content at which a soil will just begin to crumble when rolled into a thread approximately 1/8" (3 mm) in diameter. Plasticity index is the difference in moisture content of soils between the liquid and plastic limits expressed in percentage.

We have done Atterberg limit test to calculate Liquid Limit (LL) and Plastic Limit (PL) and Plasticity Index (PI) (Table 3.3) of the soil samples. (Figure 3.6) (Figure 3.7)

Table 3.3: Atterberg limit measuring of soil sample

			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				17	24	28
Can Number	---	---	12	53	47	66	65	51
Mass of Empty Can	M_C	(g)	34.36	32.83	24.84	26.62	29.21	29.17
Mass Can & Soil (Wet)	M_{CMS}	(g)	39.97	39.18	30.44	36.47	40.00	39.53
Mass Can & Soil (Dry)	M_{CDS}	(g)	38.60	37.63	29.05	33.71	37.09	36.83
Mass of Soil	M_S	(g)	4.24	4.80	4.21	7.09	7.88	7.66
Mass of Water	M_W	(g)	1.37	1.55	1.39	2.76	2.91	2.70
Water Content	w	(%)	32.3	32.3	33.0	38.9	36.9	35.2

3.4.3.1 Determination of Liquid limit (LL)

The procedure of determining liquid limit of soil:

i) Place a portion of the paste in the cup of the liquid limit device.

ii) Level the mix so as to have a maximum depth of 1cm.

iii) Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

iv) For normal fine grained soil: The Casagrande's tool is used to cut a groove 2mm wide at the bottom, 11mm wide at the top and 8mm deep.

v) For sandy soil: The ASTM tool is used to cut a groove 2mm wide at the bottom, 13.6mm wide at the top and 10mm deep.

vi) After the soil pat has been cut by a proper grooving tool, the handle is rotated at the rate of about 2 revolutions per second and the no. of blows counted, till the two parts of the soil sample come into contact for about 10mm length.

vii) Take about 10g of soil near the closed groove and determine its water content

viii) The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

ix) By altering the water content of the soil and repeating the foregoing operations, obtain at least 5 readings in the range of 15 to 35 blows. Don't mix dry soil to change its consistency.

x) Liquid limit is determined by plotting a ‘flow curve’ on a semi-log graph, with no. of blows as abscissa (log scale) and the water content as ordinate. Then after plotting we have determined the water content at 25 blow. That is the liquid limit (LL).

Figure 3.6. Casagrande cup in action

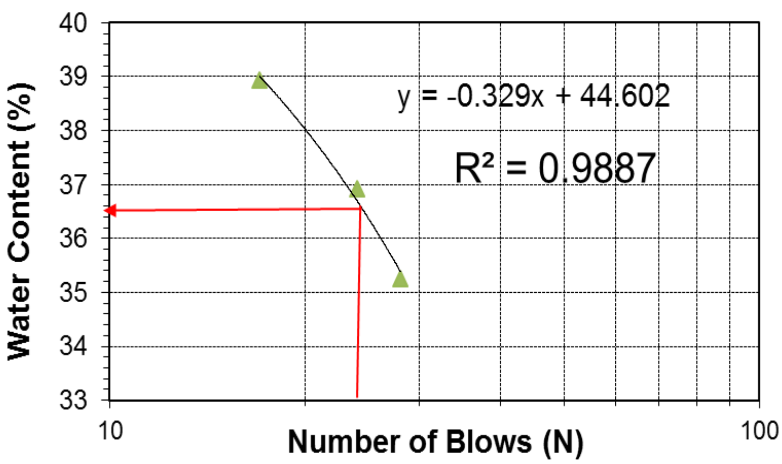

Figure 3.7. Determination of liquid limit (LL)

Here from Table 3.3 and Figure 3.7 we get LL = 36

3.4.3.2 Determination of Plastic limit (PL)

The procedure of determination of plastic limit of soil:

i) Take about 8g of the soil and roll it with fingers on a glass plate. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia.

ii) If the dia. of the threads can be reduced to less than 3mm, without any cracks appearing, it means that the water content is more than its plastic limit. Knead the soil to reduce the water content and roll it into a thread again.

iii) Repeat the process of alternate rolling and kneading until the thread crumbles.

iv) Collect and keep the pieces of crumbled soil thread in the container used to determine the moisture content.

v) Repeat the process at least twice more with fresh samples of plastic soil each time.

From Table 3.3 we get PL = 33

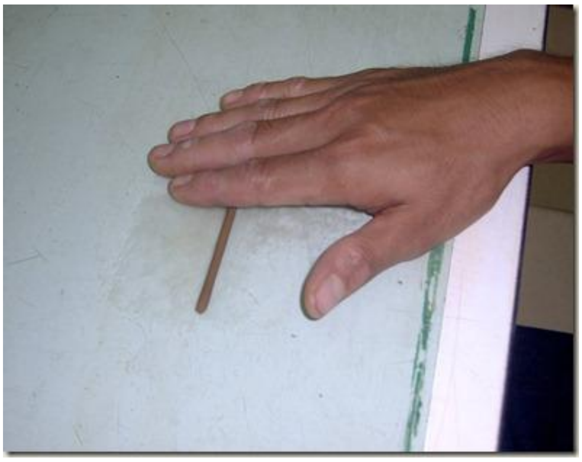

Figure 3.8. Determination of plastic limit

3.4.3.3 Determination of Plasticity Index (PI)

Plasticity index is the difference in moisture content of soils between the liquid and plastic limits expressed in percentage.

Plasticity index PI = LL-PL

= 36-33

= 3

3.4.4 Consolidation Test of Soil

For performing consolidation test of soil we have done Oedometer test (Figure 3.11) of soil sample which measures soil's consolidation properties i.e. Compression Index (C_c) and Void Ratio (e_0). Oedometer tests are performed by applying different loads to a soil sample and measuring the deformation response. The results from these tests are used to predict how a soil in the field will deform in response to a change in effective stress.

Oedometer Test Procedure:

i) Clean and dry the metal ring. Measure its diameter and height. Take the mass of the empty ring.

ii) Press the ring into the soil sample contained in a large container at the desired density and water content. The ring is to be pressed with hands.

iii) Remove the soil around the ring. The soil specimen should project about 10mm on either side of the ring. Any voids in the specimen due to the removal of large size particles should be filled back by pressing the soil lightly.

iv) Trim the specimen flush with the top and bottom of the ring.

v) Remove any soil particles sticking to the outside of the ring. Weigh the ring with the specimen.

vi) Take a small quantity of the soil removed during trimming for the water content determination.

vii) Saturate the porous stones by boiling them in distilled water for about 15min.

viii) Assemble the Consolidometer. Place the bottom porous stone, bottom filter paper, specimen, top filter paper and the top porous stone, one by one.

ix) Position the loading block centrally on the top porous stone. Mount the assembly on the loading frame. Centre it such that the load applied is axial. In the case of the lever loading system, counterbalance the system.

x) Set the dial gauge in position. Allow sufficient margin for the swelling of the soil.

xi) Connect the mould assembly to the water reservoir having the water level at about the same as the soil specimen. Allow the water to flow into the specimen till it is fully saturated.

xii) Take the initial reading of the dial gauge.

xiii) Apply an initial setting load to give a pressure of 2kg to the assembly so that there is no swelling. Allow the setting load to stand till there is no change in the dial gauge reading or for 24 hours.

xiv) Take the final gauge reading under the initial setting load.

xv) Apply the first load increment to apply a pressure of 4kg, and start the stop watch. Record the dial gauge readings at 0.05, 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60minutes.

xvi) Increase the load to apply a pressure of 8kg and repeat the step (15). Likewise increase the load to apply a pressure of 16, 32 or upto the desired pressure.

xvii) After the last load increment had been applied and the readings taken, decrease the load to 1/4 of the last load and allow it to stand for 24 hours. Take the dial gauge reading after 24 hours. Further reduce the load to 1/4 of the previous load and repeat the above procedure. Likewise, further reduce the load to 1/4 previous and repeat the procedure. Finally reduce the load to the initial setting load and keep it for 24 hours and take the final dial gauge reading.

xviii) Dismantle the assembly. Take out the ring with the specimen. Wipe out the excess surface water using a blotting paper.

xix) Take the mass of the ring with the specimen.

xx) Dry the specimen in the oven for 24 hours and determine the dry mass of specimen.

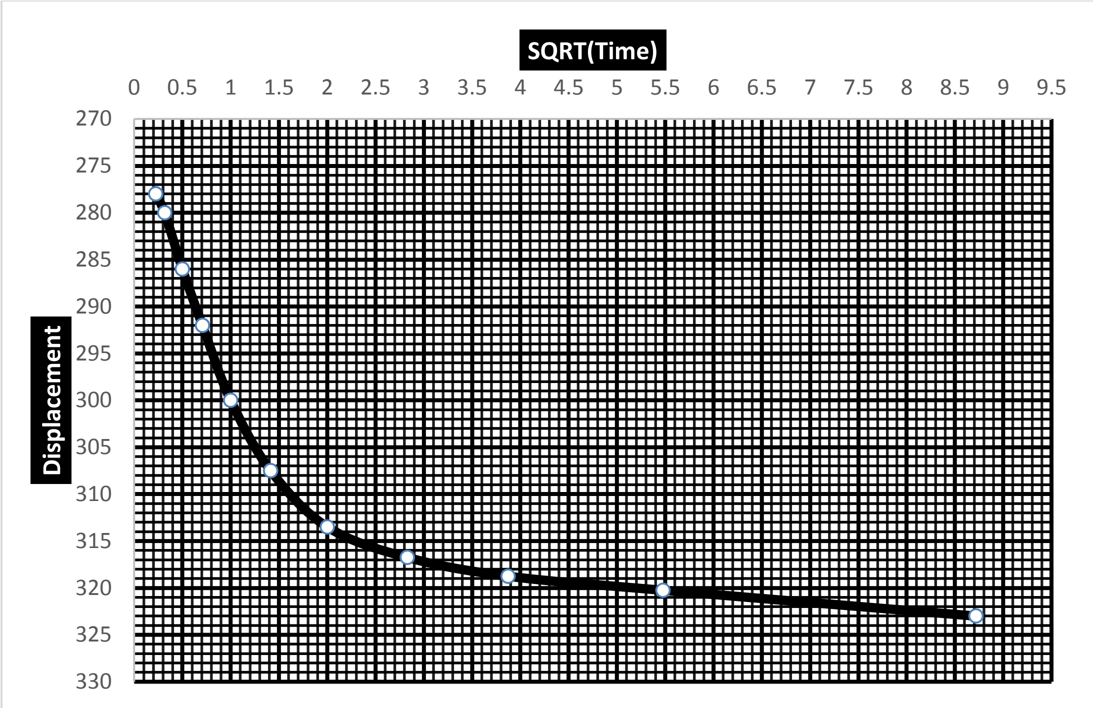

Figure 3.9. Displacement vs SQRT graph

From figure 3.9 we have determined value of t_{90}, d_0 and d_{90}

We use the following formulae for calculating C_c and e_0. (Figure 3.10)

$$\mathbf{C_c = \Delta e / \Delta \log\sigma'}$$

Where,

C_c = Compression Index

Δe = Variation of void ratio

$\Delta\log\sigma'$ = Variation of effective stress

$$\mathbf{e_0 = (G_s \times V) / W_s - 1}$$

Where,

e_0 = Void ratio

G_s = Specific gravity of soil

V = Volume of the soil sample

W_s = Weight of dry soil

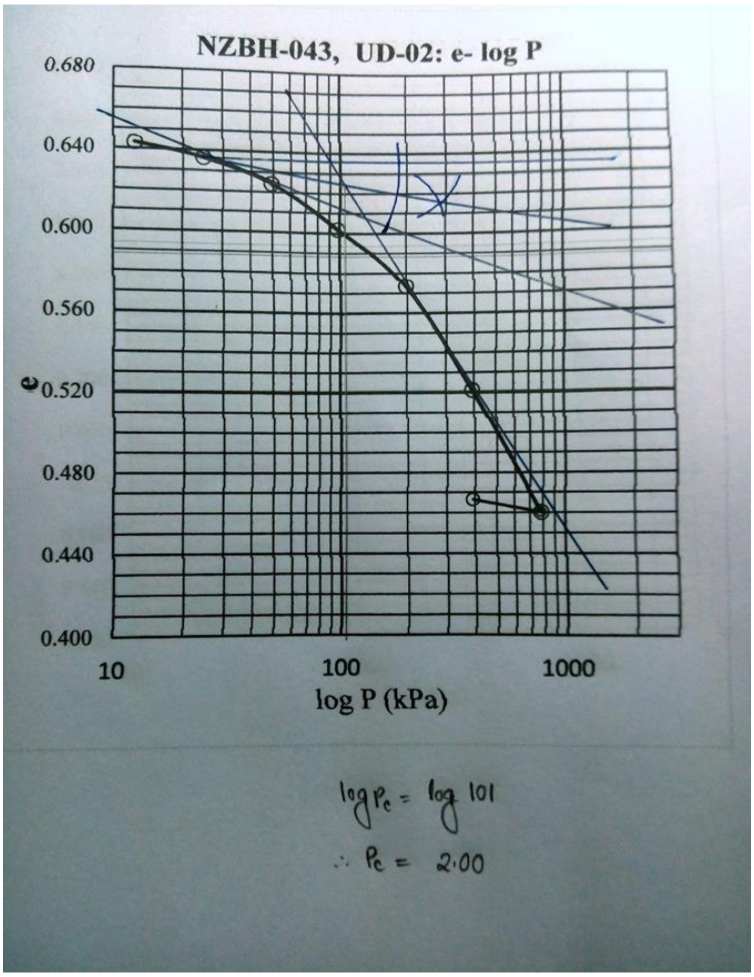

Figure 3.10. Void ratio vs effective stress curve

Figure 3.11. Oedometer test with sample

3.4.5 Unconfined Compression Test

To determine unconfined compressive strength of soil (q_u) we have done unconfined compression test of soil (Figure 3.12) (Figure 3.13).

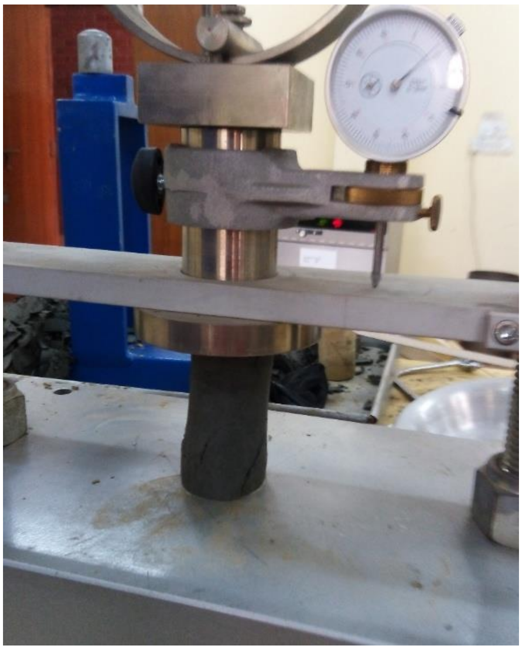

Figure 3.12 Unconfined compression test Equipment with cracked sample

Procedure of Test:

1. Take two frictionless bearing plates of 38 mm diameter.

2. Place the specimen on the base plate of the load frame (sandwiched between the end plates).

3. Place a hardened steel ball on the bearing plate.

4. Adjust the center line of the specimen such that the proving ring and the steel ball are in the same line.

5. Fix a dial gauge to measure the vertical compression of the specimen.

6. Adjust the gear position on the load frame to give suitable vertical displacement.

7. Start applying the load and record the readings of the proving ring dial and compression dial for every 5 mm compression.

8. In UC test, the commonly used loading rate is 1.25 mm/min. For harder specimens 1.5 mm/min or 2.25 mm/min can also be used.

9. Continue loading till failure is complete, and then draw the sketch of the failure pattern in the specimen.

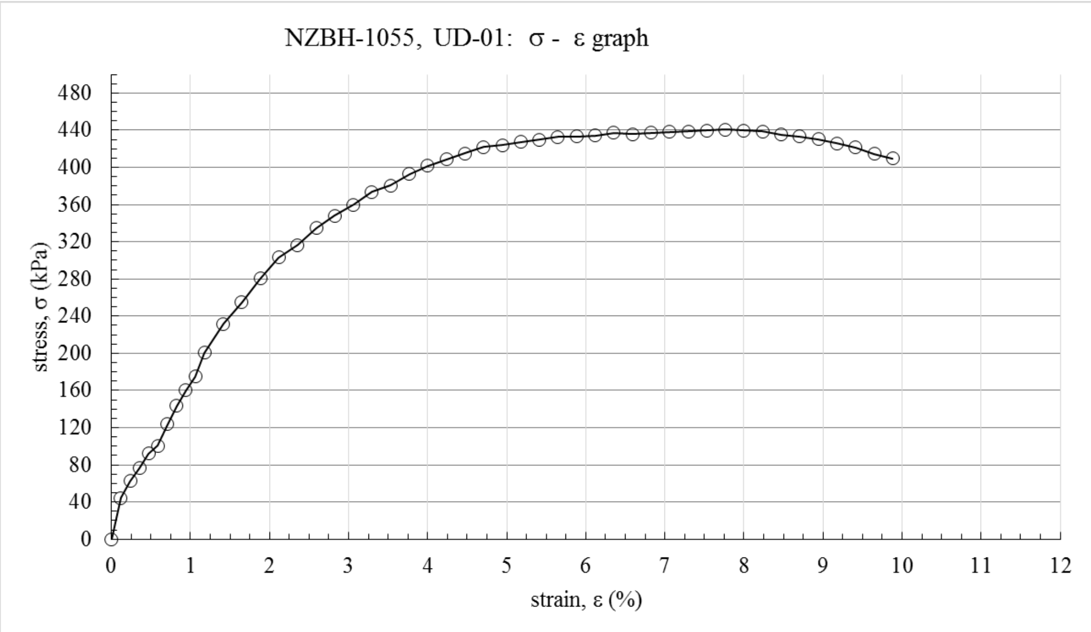

Figure 3.13 Stress vs. strain diagram

3.5 Parameter Identification for Constitutive Modeling

An elastoplastic constitutive model for soils, called the extended subloading t_{ij}-model [Nakai, 2011], is used in the finite element analyses. This model, despite the use of a small number of material parameters, can describe properly the following typical features of soil behaviors [Nakai and Hinokio, 2004 & Nakai, 2011]:

(i) Influence of intermediate principal stress on the deformation and strength of geomaterials.

(ii) Dependence of the direction of plastic flow on the stress paths.

(iii) Influence of density and/or confining pressure on the deformation and strength of geomaterials.

(iv) The behavior of structured soils such as naturally deposited soils.

A brief description of the above mentioned features of this model can be made as follows:

Influence of intermediate principal stress is considered by defining yield function f with modified stress t_{ij} (i.e., defining the yield function with the stress invariants (t_N and t_S) instead of (p and q). The yield function is written as a function of the mean stress t_N and stress ratio $X \equiv t_S/t_N$ based on t_{ij} by Eq.(3.5.1).

$$f = \ln\frac{t_N}{t_{N0}} + \varsigma(X) - \left(\ln\frac{t_{N1e}}{t_{N0}} - \ln\frac{t_{N1e}}{t_{N1}} \right) = 0 \tag{3.5.1}$$

Here, t_{N1} determines the size of the yield surface (the value of t_N at X=0), t_{N0} is the value of t_N at reference state and t_{N1e} is the mean stress t_N equivalent to the present plastic volumetric strain which is related to the plastic volumetric strain ε_v^p as

$$\varepsilon_v^p = \frac{\lambda - \kappa}{1+e_0} \ln\left(\frac{t_{N1e}}{t_{N1}}\right) \tag{3.5.2}$$

The symbols λ and κ denote compression index and swelling index, respectively, and e_0 is the void ratio at reference state. In this research, the expression for $\varsigma(X)$ is assumed as,

$$\varsigma(X) = \frac{1}{\beta}\left(\frac{X}{\mathrm{M}^*}\right)^\beta \text{ (material parameter)} \tag{3.5.3}$$

The value of M* in Eq.**Error! Reference source not found.** is expressed as follows using principal stress ratio $X_{CS} \equiv (t_S/t_N)_{CS}$ and plastic strain increment ratio $Y_{CS} \equiv (d\varepsilon_{SMP}{}^{*p}/d\gamma_{SMP}{}^{*p})_{CS}$ at critical state:

$$\mathrm{M}^* = \left(X_{CS} + X_{CS}{}^{\beta-1} Y_{CS}\right)^{1/\beta} \tag{3.5.4}$$

and these ratios X_{CS} and Y_{CS} are represented by the principal stress ratio at critical state in triaxial compression R_{CS}.

In elastoplastic theory, total strain increment consists of elastic and plastic strain increments as

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + d\varepsilon_{ij}^p \tag{3.5.5}$$

Here, plastic strain increment is divided into component $d\varepsilon_{ij}{}^{p(AF)}$, which satisfies associate flow rule in the space of modified stress t_{ij}, and isotropic compression component $d\varepsilon_{ij}{}^{p(IC)}$as given in Eq.(3.5.6).

$$d\varepsilon_{ij}^p = d\varepsilon_{ij}^{p(AF)} + d\varepsilon_{ij}^{p(IC)} \tag{3.5.6}$$

The components of strain increment are expressed as,

$$d\varepsilon_{ij}^{p(AF)} = \Lambda \frac{\partial f}{\partial t_{ij}} \quad and \quad d\varepsilon_{ij}^{p(IC)} = K \left\langle dt_N \right\rangle \frac{\delta_{ij}}{3} \tag{3.5.7}$$

Here, Λ is the proportionality constant, δ_{ij} is Kronecker's delta and <> denotes Macauley bracket. Dividing plastic strain increment into two components as in Eqs.(3.5.6) and (3.5.7), for the same yield function, this model can take into consideration, i.e., the dependence of the direction of plastic flow on the stress paths. Adding the term $G(\rho)$ in the denominator of the proportionality constant Λ of normal consolidated condition, influence of density is considered. The proportionality constant Λ is expressed as

$$\Lambda = \frac{\dfrac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij}}{\dfrac{1+e_0}{\lambda-\kappa}\left(\dfrac{\partial f}{\partial t_{kk}} + \dfrac{G(\rho)}{t_N} + \dfrac{Q(\omega)}{t_N}\right)} = \frac{df_\sigma}{h^p} \tag{3.5.8}$$

and $K = \dfrac{1}{\dfrac{1+e_0}{\lambda-\kappa}\left(1+\dfrac{G(\rho)}{a_{kk}}\right)} \cdot \dfrac{1}{t_{N1}}$ (3.5.9)

the stress-strain behavior of structured soil can be described by considering not only the effect of density described above but also the effect of bonding. Two state variables ρ related to density and ω representing the bonding effect are used to consider feature (iv). The following relationships for $G(\rho)$ and $Q(\omega)$ are adopted in the model:

$$G(\rho) = sign(\rho)a\rho^2 \quad and \quad Q(\omega) = b\omega \tag{3.5.10}$$

Where a and b are material parameters.

The parameters of subloading t_{ij} model are fundamentally the same as those of the Cam clay model [Roscoe and Burland, 1968], except for the parameter a, which is responsible for the influence of the density and the confining pressure. Parameter β controls the shape of the yield surface. The performance of the constitutive model has already been checked in numerical simulations [Shahin, 2004, Shahin, 2011; Nakai, 2010].

For getting parameters of the constitutive model, consolidation tests for study locations soils (Figure 3.1) have been carried out in laboratory. Figure 3.13 shows the relations between void ratios and mean effective stress in logarithmic scale. From these curves, compression index λ, swelling index $\acute{K}$ and void ratio at 98kPa, N are obtained for both soils by using Eq. 3.5.11; 3.5.12; 3.5.13. Using these values and fitting the computed curve parameter a (density parameter) of sub-loading t_{ij} model is obtained.

$$\lambda = 0.434 \times Cc \tag{3.5.11}$$

$$\acute{K} = 0.434 \times Cs \tag{3.5.12}$$

$$N = \text{Void ratio at 98 KPa} \tag{3.5.13}$$

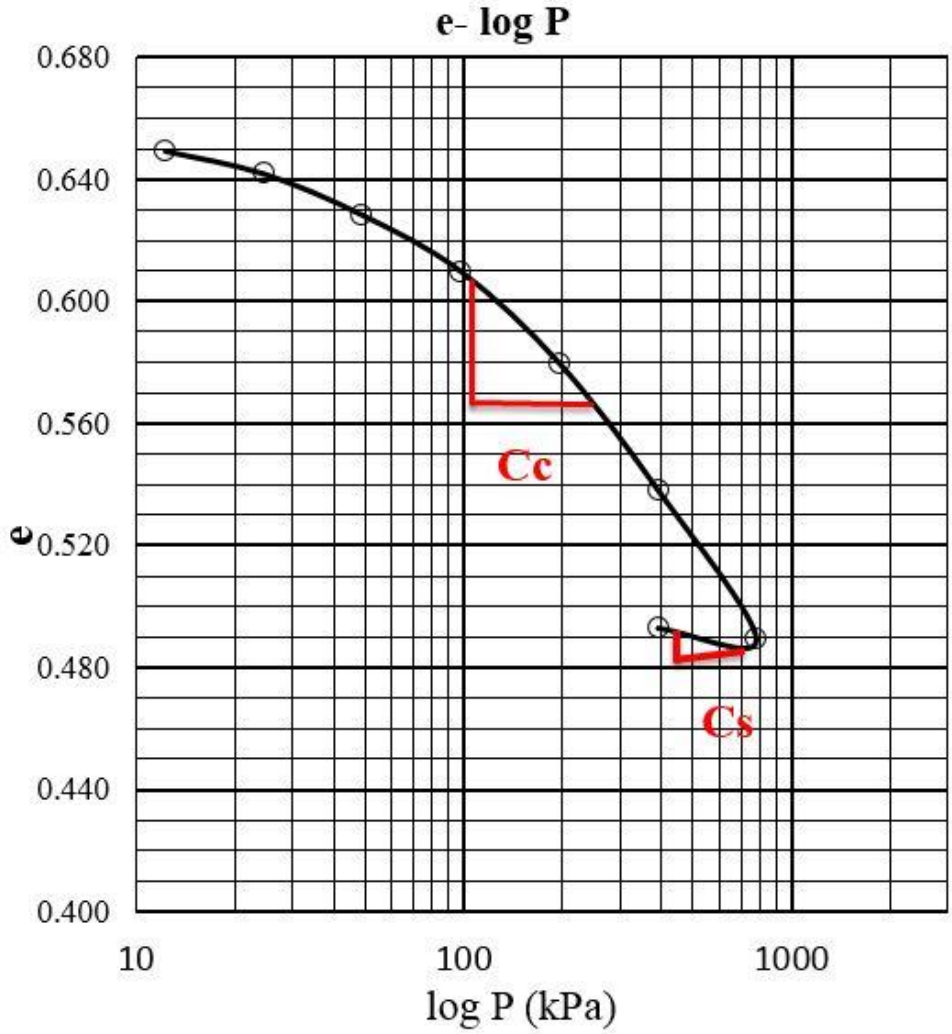

Figure 3.14. Calculation of C_c and C_s from e vs. logP curve

3.6 Layers of Soil Section with Piled Raft Foundation

From Figure 3.15 we can see a section of piled raft foundation with different layers of soil.

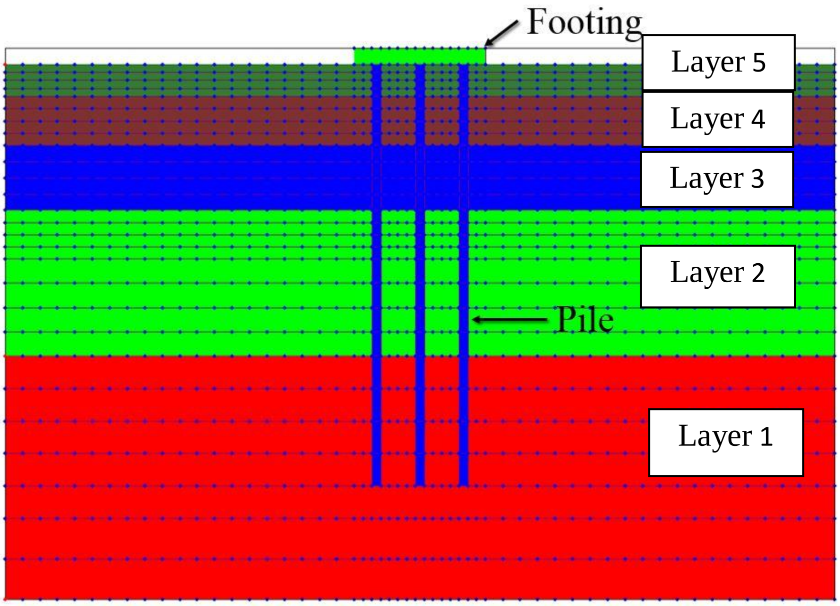

Figure 3.15. Layers of piled raft foundation soil section

3.7 Mesh of Soil Section

Mesh generation is the practice of generating a polygonal or polyhedral mesh that approximates a geometric domain. The term "grid generation" is often used interchangeably. Typical uses are for rendering to a computer screen or for physical simulation such as finite element analysis or computational fluid dynamics.

Figure 3.16 is the mesh with dimension of the same section which has been done for simulation work.

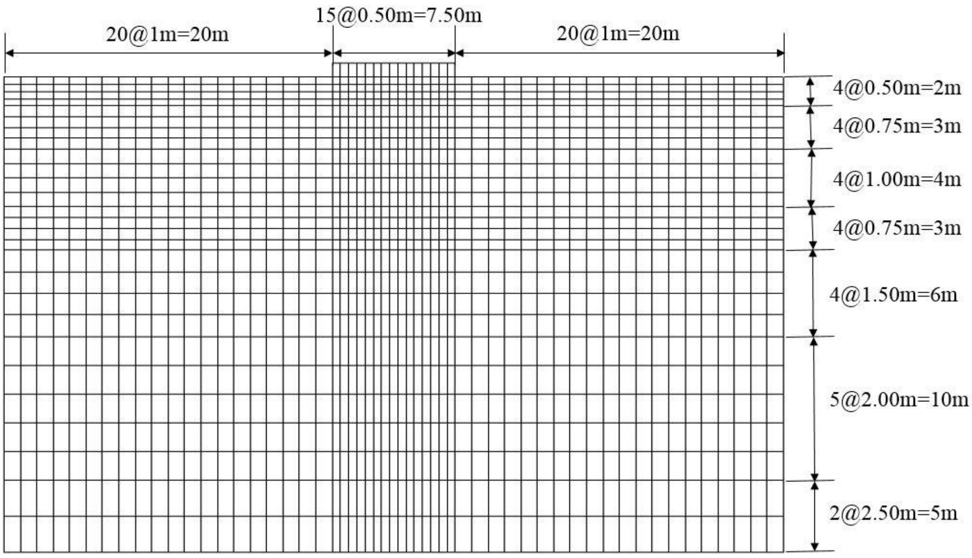

Figure 3.16. Finite Element Mesh for piled raft foundation

3.8 Conclusion

In this chapter, different methods adopted to achieve the objectives of the study are thoroughly discussed. Different parameters of soil are explained in order to relate it to the study result. Experimental method is important in order to set out the scope of the study. So, the methodology is followed by the result and discussion in the next chapter.

Chapter 4 Soil Characteristics at the Study Locations

Physically Narayanganj district is characterized by alluvial formations caused by several rivers such as Shitalakshya, Meghna, Old Brahmaputra, Buriganga, Balu and Dhaleshwari. Comilla district is mainly formed of olive grey silty loam and dark grey silty loam soil. By observing and testing we have found similarity among the soils of study locations in different depths which are shown in Figure 4.1.

Table 4.1: Simulation parameters

Soil Layers	Depth (m)	λ	Ḱ	β	RCs	N	e0	aAF	aIC
1	0.00-2.00	0.0700	0.00450	2.00	3.20	1.100	0.800	030	030
2	2.01-5.00	0.1038	0.00829	1.60	3.98	0.865	0.879	800	800
3	5.01-9.00	0.1018	0.00803	1.60	4.00	0.868	0.880	850	850
4	9.01-18.00	0.0819	0.00983	1.60	4.00	0.778	0.789	800	800
5	18.01-33.00	0.0879	0.00894	1.60	4.00	0.602	0.620	800	800

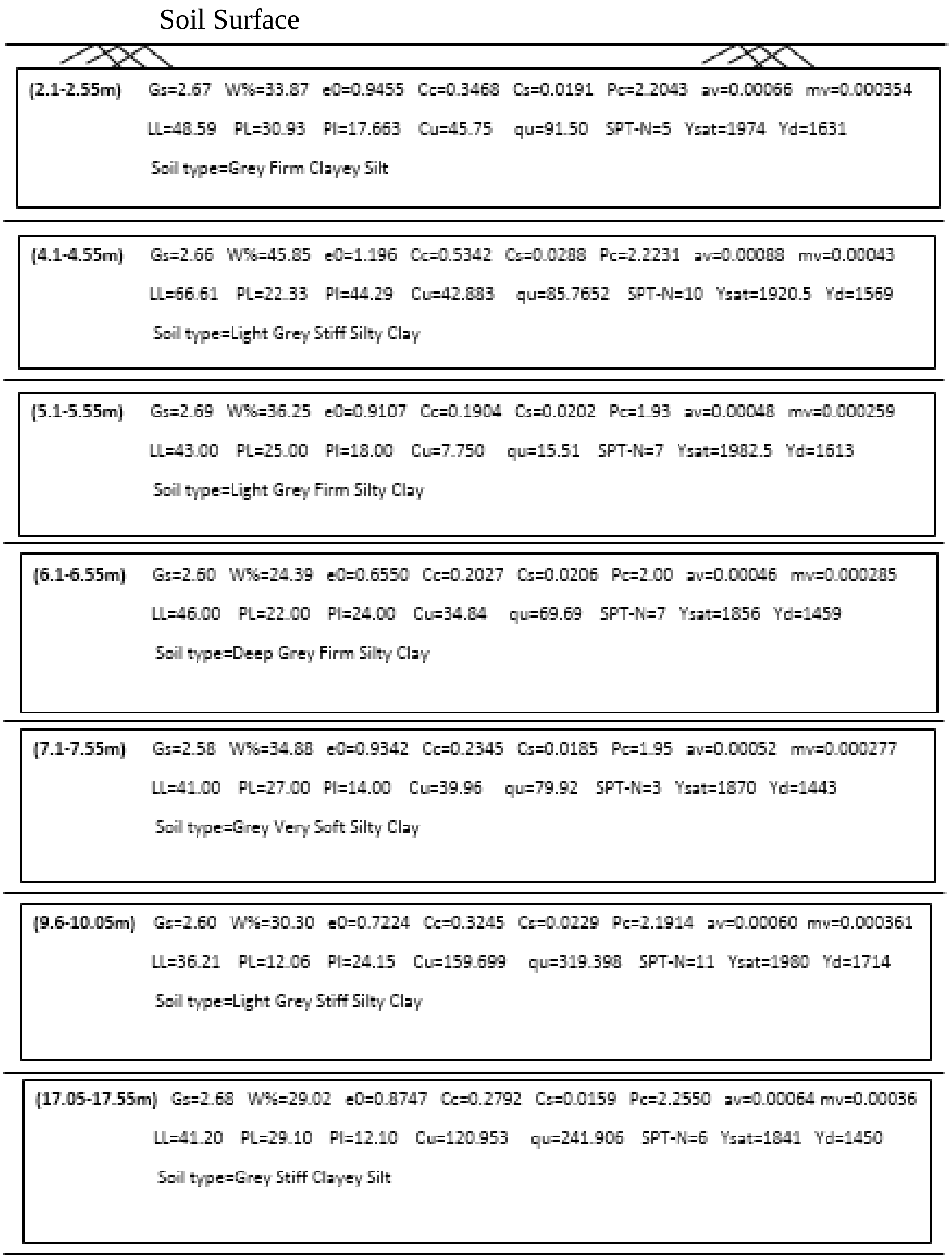

Figure 4.1. Soil parameters from laboratory tests

Chapter 5 Results and Discussions

5.1 General

This chapter deals with the presentation of results obtained from various tests and simulation conducted on soil. The main objective of the research program was to determine soil parameters and establish correlation between some parameters. Another objective was to determine the bearing capacity of piled raft foundation.

5.2 Correlation of Soil Parameters

Many developed countries have different correlation of soil parameters for their soil type. But for our country soil type there is no such established equation. Our target was to establish such equations. These correlations will save our time and labor because using the equations we can determine several parameters if we know one parameter value. So there will be no need to perform many laboratory tests.

5.2.1 Cc vs. LL

From Skempton (1944), **C_c=0.009(LL-10)**; from experiment we have proposed, **C_c=0.006(LL-10)** (Figure 5.1). Co-efficient of Skempton's equation was 0.009, but we have found 0.006 which is very much relevant according to our data.

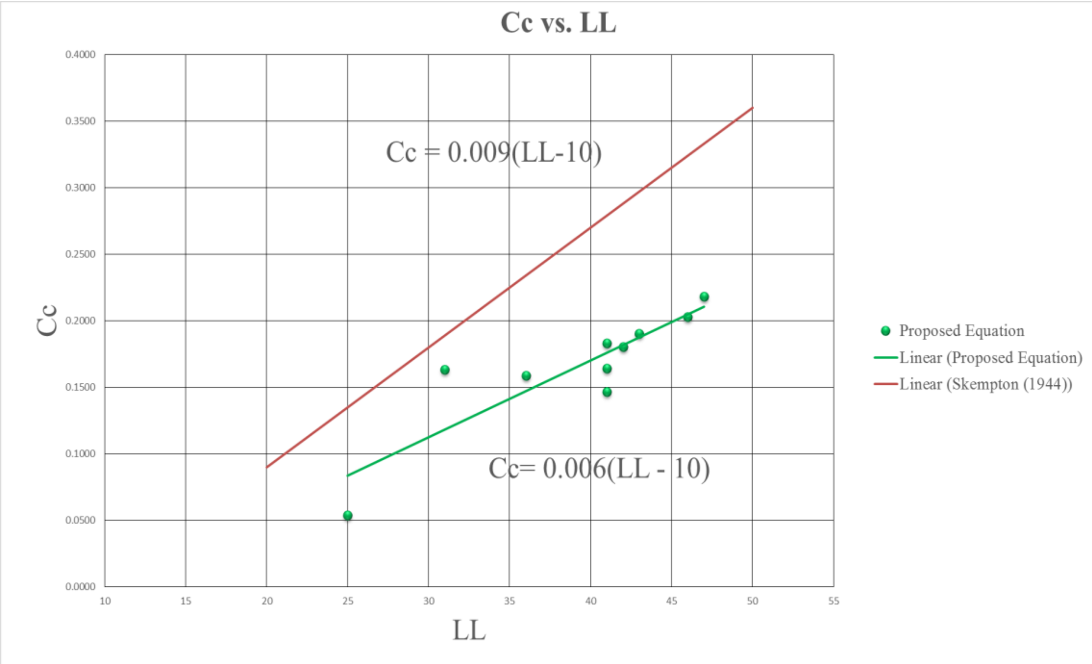

Figure 5.1. Compression index (C_c) vs. Liquid limit (LL)

5.2.2 C_c vs. W_n

From Rendon-Herrero (1980), **C_c=0.0115w_n**; from experiment we have proposed, **Cc=0.0065w_n** (Figure 5.2). Co-efficient of Rendon-Herrero's equation was 0.0115, but we have found 0.0065 which is very much relevant according to our data.

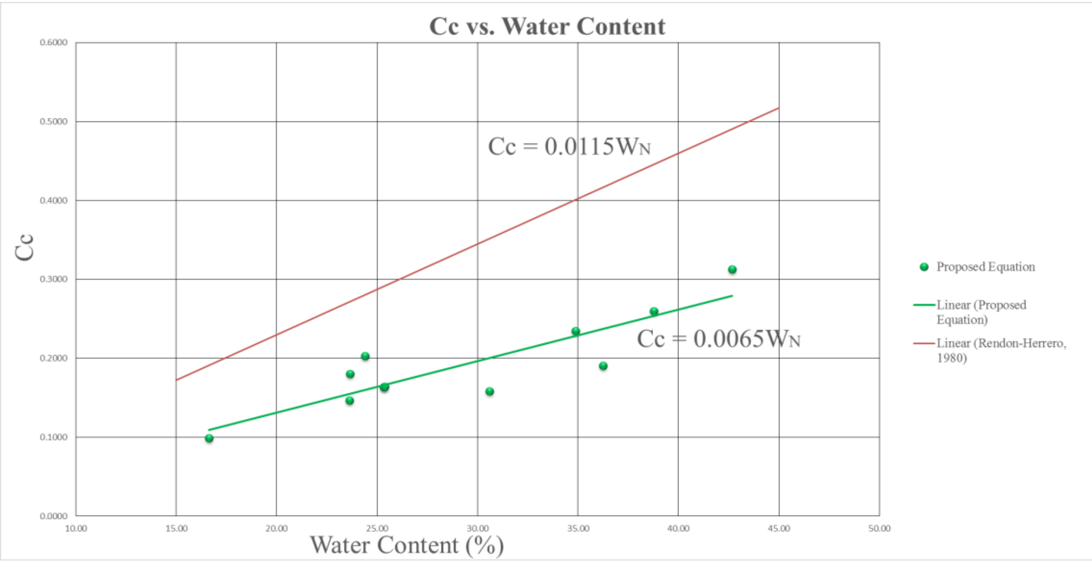

Figure 5.2. Compression index (C_c) vs. Water Content (w_n)

5.2.3 C_c vs. e_0

From Nishida (1956), **C_c=1.15(e_0-0.27)**; from experiment we have proposed, **C_c=0.4024(e_0-0.24)** (Figure 5.3). Co-efficient of Nishida's equation was 1.15, but we have found 0.4024 which is very much relevant according to our data.

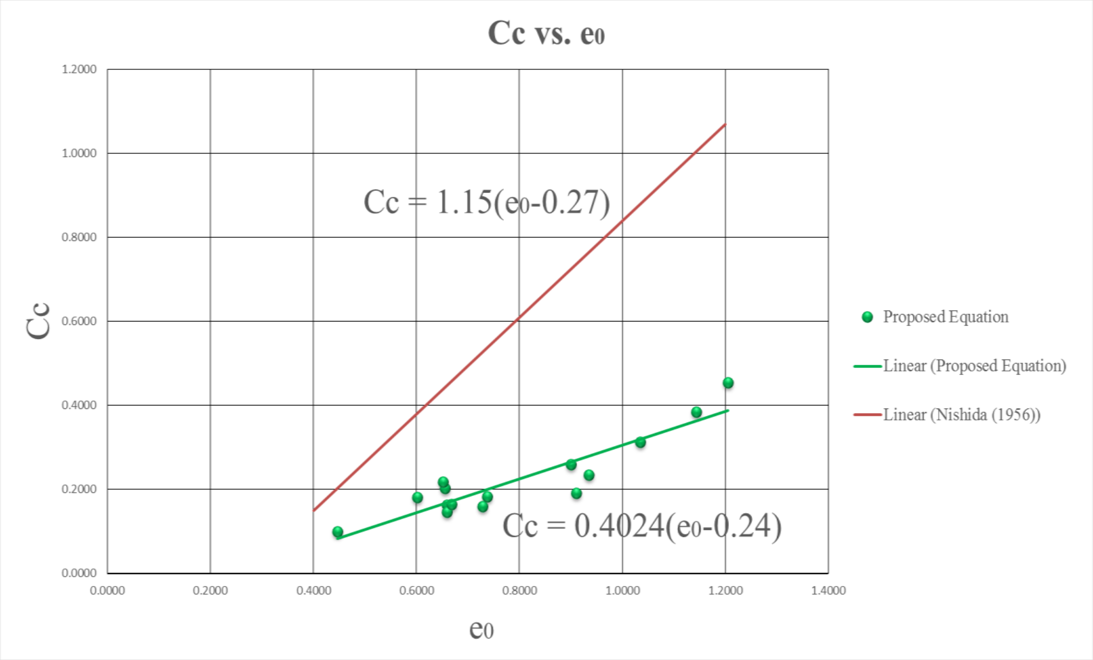

Figure 5.3. Compression index (C_c) vs. Void ratio (e_0)

5.3 Load Bearing Capacity

The bearing capacity of soils is perhaps the most important of all the topics in soil engineering. Soils behave in a complex manner when loaded so, it is important to know the bearing capacity of soils. Soil when stressed due to loading, tend to deform. The resistance to deformation of the soil depends upon factors like water content, bulk density, angle of internal friction and the manner in which load is applied on the soil. The maximum load per unit area which the soil or rock can carry without yielding or displacement is termed as the bearing capacity of soils.

5.3.1 Initial Stress Distribution of the Ground

Figure 5.4 shows the initial distribution of stress without piled raft foundation. Here we can see the stress in the deepest layer is highest.

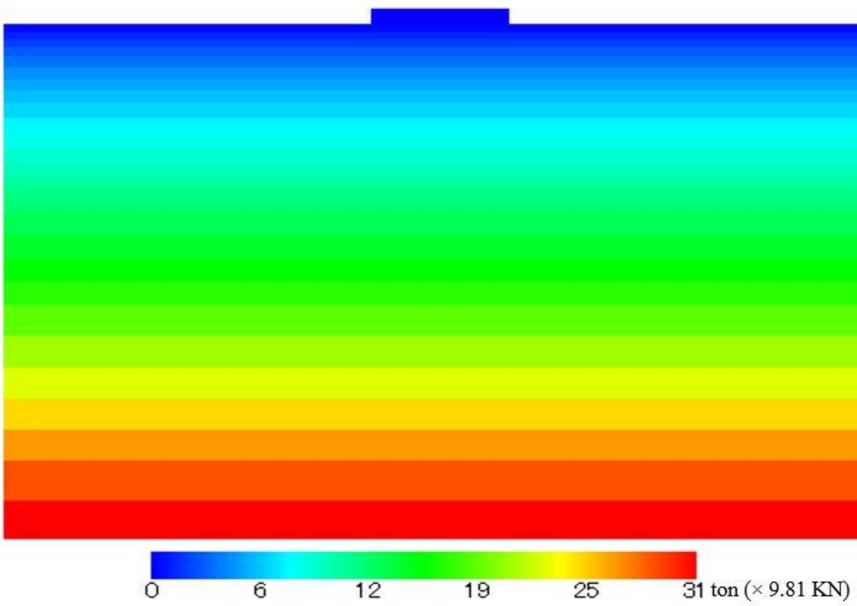

Figure 5.4. Stress distribution without piled raft foundation

5.3.2 Stress Distribution of the Groud with Structure Load

Figure 5.5 shows the initial distribution of stress with piled raft foundation. Here we can see hoe the piles are distributing the loads in the soil layer.

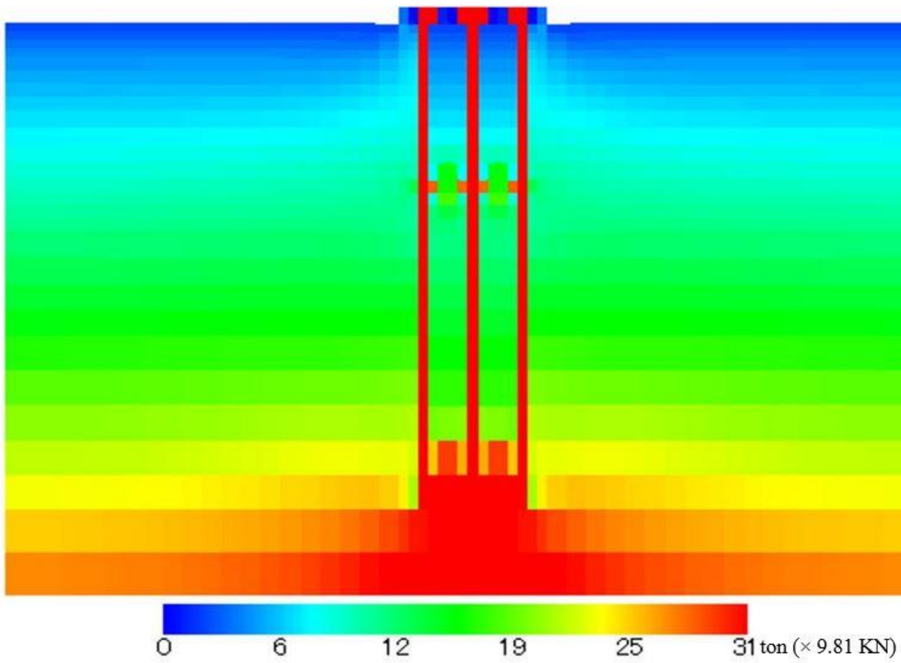

Figure 5.5. Stress distribution with piled raft foundation

5.3.3 Load-Displacement Relation

This the final result of our study through simulation. This figure 5.6 shows the load bearing capacity of soil. For 0.05% settlement the soil can take 880 ton load.

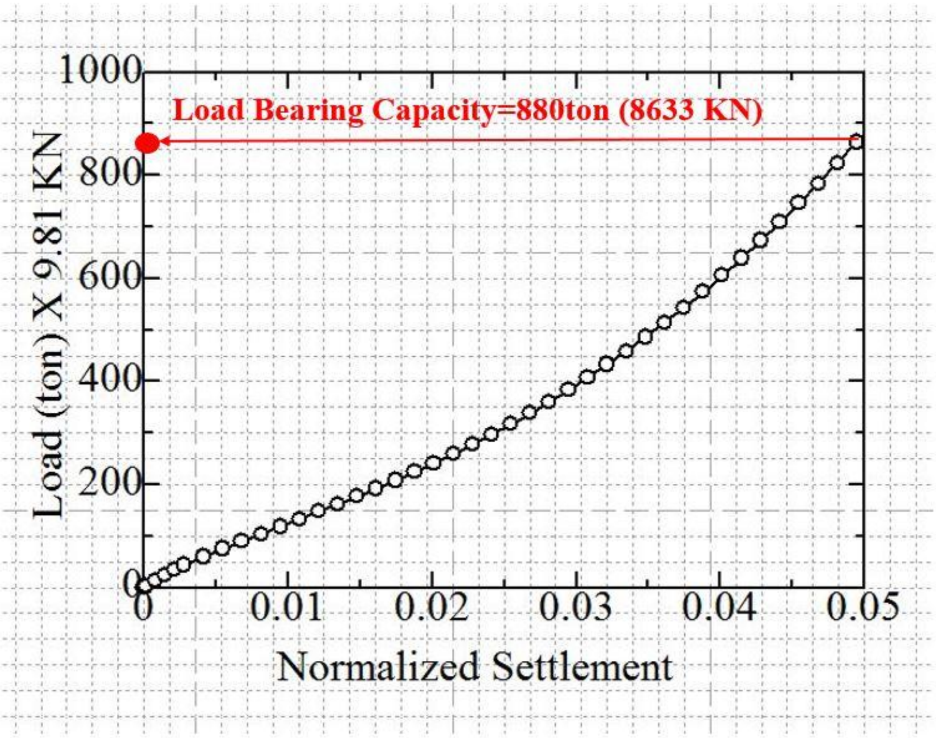

Figure 5.6. Load vs. Settlement curve

Chapter 6 Conclusion

6.1 Reviews on Completed Research Work

6.1.1 Correlation of Soil Parameters

The following points can be concluded from this research.

I. The existing correlations of soil parameters are not matched with the correlations, which found from sample soil investigation and analysis of the selected sites.

II. The proposed correlations are very much relevant for the selected sites as they have shown strong relation among the parameters.

Our proposed correlations of soil parameters are-

- ❑ **Cc=0.006(LL-10)**
- ❑ **Cc=0.0065WN**
- ❑ **Cc=0.4024(e0-0.24)**

6.1.2 Load Bearing Capacity

Load bearing capacity for 0.05% vertical settlement of soil = 880 ton or 8633 KN.

6.2 Future Research

Calculation of ultimate bearing capacity of piled raft foundation and pile foundation varying-

- Number of piles
- Length of piles
- Diameter of piles
- Changing the height of the water table

References

Azzouz, A.S., Krizek, R.J., Corotis,R.B., (1976). Regression analysis of soil compressibility. *Soils Foundation* 16(2), 19–29

Cozzolino, V.M. (1961). Statistical forecasting of compression index. *Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering*, Paris, vol.1, pp.51–53

Danial Mohammadzadeh S., Jafar Bolouri Bazaz, AmirH.Alavi, (2014). An evolutionary computational approach for formulation of compression index of fine-grained soils. *Engineering Applications of Artificial Intelligence*, 33(2014), 58–68

Kazimierz Józefiaka, Artur Zbiciaka, Maciej Maslakowski, Tomasz Piotrowskib, (2015). Numerical modelling and bearing capacity analysis of pile foundation. *XXIV R-S-P seminar, Theoretical Foundation of Civil Engineering (24RSP) (TFoCE 2015),* Procedia Engineering 111 (2015), 356 – 363

Mayne, P.W., (1980). Cam-clay predictions of undrained strength. *J.Geotech. Eng. Div. ASCE 106(11)*, 1219–1242

Mohammad S. Islam, H.M. Shahin, S. Banik, F. Azam (2013). Elasto-plastic constitutive model parameters and their application to bearing capacity estimation for Dhaka sub-soil. *Journal of Civil Engineering (IEB)*, 42 (2), 171-188

Nakai, T. and Hinokio, T. (2004). A simple elastoplastic model for normally and over consolidated soils with unified material parameters, *Soils and Foundations*, 44(2), 53-70

Nakai, T., Shahin, H.M., Zhang, F. Hinokio,M., Kikumoto, M., Onaha, S. and Nishio, A. (2010). Bearing capacity of reinforced foundation subjected to pull-out loading in 2D and 3D conditions. *Geotextiles and Geomembranes*, 28(3), 268-280

References

Nakai, T. Shahin, H.M., Kikumoto, M., Kyokawa, H., Zhang, F.and Farias, M.M. (2011). A simple and unified three-dimensional model to describe various characteristics of soils. *Soils and Foundations,* 51(6), 1149-1168

Nishida. Y., (1956). A brief note on compression index of soils. J. *Soil Mech. Found. Div., ASCE 82 (SM3)* (1027-1-1027-14)

Park,H., Lee,S.R., (2011). Evaluation of the compression index of soils using an artificial neural network. *Comput.Geotech*.38, 472–481

Roscoe, K. H. and Burland, J. B. (1968). On the generalized stress-strain behaviour of wet clay. *Engineering Plasticity*, Cambridge: 535-609

Skempton, A.W. (1944). Notes on the compressibility of clays. *Quart. J.Geol. Soc. Lond*. 100, 119–135

Sower, G.B., (1970). Introductory Soil Mechanics and Foundation, 3rd Ed. *The Macmillan Company of Collier-Macmillan Ltd*, London

Terzaghi, K., Peck, R.B., (1967). Soil Mechanics in Engineering Practice. *John Wiley & Sons Inc*., New York.

T.W. Lambe, R.V. Whitman, Soil Mechanics, *Wiley*, New York, 1969

Wroth,C.P., Wood,D.M., (1978). The correlation of index properties with some basic engineering properties of soils. *Can.Geotech.J*. 15, 137–145