ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
SCHOOL OF CIVILAND ENVIRONMENTAL ENGINEERING

Evaluating Bearing Capacity of Layered Soils Using Finite element analysis software

By:
Hlina Belachew

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES, ADDIS ABABA UNIVERSITY, IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN CIVIL ENGINEERING [GEOTECHNICAL ENGINEERING]

Advisor: Dr-Eng. Henok Fikre

Dec, 2017

ADDIS ABABA INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

Evaluating Bearing Capacity of Layered Soils Using Finite Element Analysis Software

By Hlina Belachew

June, 2017

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES, ADDIS ABABA UNIVERSITY, IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN CIVIL ENGINEERING

Advisor: - Dr-Ing Henok Fikre

Addis Ababa University

School of Graduate Studies

Institute of Technology

Evaluating Bearing Capacity of Layered Soils Using Finite Element Analysis Software

By:

Hlina Belachew

Approved by Board of Examiners

Dr-Ing Henok Fikre	______________	______________
Advisor	signature	Date
______________	______________	______________
Internal examiner	signature	Date
______________	______________	______________
External Examiner	signature	Date
______________	______________	______________
Chairman	signature	Date

Declaration

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor **Dr-Ing Henok Fekre** and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have been duly acknowledged.

Name: HLINA BELACHEW

Signature: ______________

Place Institute of Technology

Addis Ababa University

Addis Ababa.

Date: ________________

Acknowledgement

I would like to express my sincere and deepest gratitude to my advisor Dr-Ing Henok Fikre for his critical comments, suggestions in guiding and supervising throughout all my research work.

I am very grateful to Ethiopian Roads Authority (E.R.A) for sponsoring my M.Sc. Program. Next I am indebted to Jeroccia Geotechnical Services and Engineering P.L.C Management and Staffs for their positive critical comments and for giving me relevant reference documents. Also I am very grateful to Jara Mengistu, Yohannes Afewerk, Melat Birhanu, Anteneh Geremew and Mekonen Araya for giving me valuable comments, documents, advices and encouragements.

Above all I am very thankful to the Almighty God, who is always with me in each and every step of my life. I thank God for everything He did and made education compatible with the rest of my life. Finally I would like to express my deepest gratitude to my parents, my friends and all who contributed to this research work in one way or another.

Abstract

The bearing capacity of the foundation is a primary concern in the field of foundation engineering. Predicting ultimate bearing capacity of a footing on layered soil is very important as it is a requirement for any design. The failure mechanism of a soil under footing and the bearing capacity value mainly depend on soil properties of each layer and the layer thickness.

It is well established that safe bearing capacity of soil is affected by various factors such as the depth of ground water table (GWT), soil properties, layering of soils, size and shape of the foundation, depth of foundation etc. among many other factors. Hence most of the studies focused on homogenous soils but in practice foundations often consist of layered soils, it is very important to consider layered soils.

In this thesis a numerical model is developed using PLAXIS. Finite element analysis is carried out using Mohr-coulomb failure criteria to represent three dimensional soil models. For Comparison purpose the layered soil bearing capacity has been analyzed by Meyerhof and Hana method which is classical approach method. After that by comparing results from FEM and classical approach the analysis has been presented.

Key Words: Bearing Capacity, layered soil, Plaxis, Finite Element Analysis

Table of Contents

List of Tables page no

List of Figures page no

LIST OF SYMBOLS:

B = Least Lateral Dimension or Width of Footing (m)

L = Length of Footing (m)

C = Cohesion of Soil (kN/m^2)

φ = Angle of Internal Friction of Soil (Degree)

ψ = Dilatancy Angle of Soil (Degree)

γ = Bulk Density of Soil (kN/m^3)

E = Young's Modulus of Elasticity of Soil (kN/m^2)

ν= Poisson's Ratio of Soil

D = Foundation Depth (m)

H = Influence Depth of Foundation in Subsoil (m)

h = Thickness of Top Layer in Two-Layer Subsoil System (m)

K_0 = Earth Pressure Coefficient At Rest

K_a = Active Earth Pressure Coefficient

K_p = Paasive Earth Pressure Coefficient

σ_v = Total Vertical Pressure (kN/m^2)

σ_h = Total Horizontal Pressure (kN/m^2)

β = Inclination Angle of the Backfill to the Horizontal

q = Ultimate Bearing Capacity of Soil (kN/m^2)

N_c, N_q, N_γ = Bearing Capacity Coefficients

D_w = Depth of Water Table below Ground Surface (m)

Chapter one

1. Introduction

1.1. General

The bearing capacity of the foundation is a primary concern in the field of foundation engineering. The self-weight of the structure and the applied loading such as: dead load, live load, wind load etc. should be transferred to the soil safely and economically. The load at which the shear failure of the soil beneath the foundation occurs is called the ultimate bearing capacity of the foundation. The magnitude of the ultimate bearing capacity depends on the mechanical characteristics of the soil and the physical characteristics of the footing.

Foundation design consists of two distinct parts: the ultimate bearing capacity of the soil under the foundation and the tolerable settlement that the footing can undergo without affecting the superstructure. The ultimate bearing capacity aims at determining the load that the soil under the foundation can handle before shear failure; while, the calculation of the settlement caused by the superstructure should not exceed the limits of the allowed deformation for stability, function and aspects of construction.

From previous reviews on the subject, the majority of the bearing capacity theories involve homogenious soils under the foundation soil properties were assumed to remain constant for the bearing capacity analysis, and therefore analytical solutions,like Terzaghi's bearing capacity theory,matched with the experimental results. However, in cases where the soil properties vary with depth, most of these theories cannot be implemented , and the analytical solutions that take into consideration the non-homogeniety of the soils are approximations, and hence the results are inaacurate.

Layered soil profiles are often encountered whether naturally deposited or artificilally made. With in each layer , the soil may be considered as homogeneous.The ultimate load failure surface in the soil depends on the shear strength parameters of the soil layers such as; the thickness of the upper layer to the width of the footing. Therefore, it is important to determine the soil profile and to calculate the bearing capacity accordingly.

Geotechnical engineers often should solve problems in layered soil while the majority of existing studies have mostly focused on homogeneous continuum. Predicting ultimate bearing capacity of footings on layered soil is very important as it is a requirement for any design and the failure mechanism of soil under footing and the bearing capacity value mainly depend on soil properties of each layer and the layer thickness.

In recent years, Finite Element Method (FEM) has been widely used in geotechnical studies to investigate soil behavior. In practice, for bearing capacity analysis engineers are seeking less complicated solutions to simplify computations as experimental analysis is time consuming and commonly used solutions such as limit equilibrium are no longer applicable. Therefore, computer programs developed based on the finite element method have been receiving much attention over recent decades as the powerful tool for solving complex cases.

1.2. Importance of Study

Foundations are the most important components of any structures and any damage to either the foundation or the foundation results in catastrophic failure of super structure. Hence, focus should be on providing a strong base to any structure. Bearing capacity of soil is a parameter widely used in the design of foundation and the objective of foundation design engineer is to provide and proportion a foundation that keeps the stresses in the foundation soil well within the limits of safe bearing capacity.

The finite element method is used to solve physical problems in engineering analysis and design. It is a numerical method for obtaining approximate solutions to many complex problems encountered in engineering analysis. In order to model such a problem with a reasonable amount of data a limited number of points may be defined and the shape between the points approximated by functions. Finite element methods are important for layered soils since it is less complicated and fast.

1.3. Problem Statement

Safe bearing capacity of foundation is one of the important stability problems in geotechnical engineering, which depends upon the bearing capacity and the allowable settlement of foundation.

Finite Element Method allows modeling complicated nonlinear soil behavior through constitutive model, various geometrics with different boundary conditions & interfaces. It can predict the stresses, deformations and pore pressures of a specified soil profile, therefore predicting ultimate bearing capacity of footings on layered soil is very important as it is a requirement for any design and the failure mechanism of soil under footing and the bearing capacity value mainly depend on soil properties of each layer and the layer thickness.

1.4. Objective and Methodology of the Research

1.4.1. Objective of the Study

The general objective of this research is to evaluate the bearing capacity of layered soils using finite element analysis software and to compare the result with the conventional method.

1.4.1.1. Specific Objective of the Present Study

- To evaluate bearing capacity of layered soil using finite element analysis software (Plaxis-3d)
- To compare the results of evaluation of bearing capacity of layered soils using PLAXIS (finite element analysis software) and those from other theories based on foundation type, geometry, soil type and different loading.
- To conduct Parametric studies

1.5. Scope and Limitations of the Present study

The present study focuses on the bearing capacity of layered soils, which the stability of the foundations are analyzed. Properties of soil considered in this geometric model simulation.

Finite Element Method (FEM, PLAXIS 3D) is used in the analysis where three dimensional foundations will be considered.

The present study focuses on the bearing capacity of a soil medium on ground by loadings of square footing which the stability of the foundations are analyzed. Properties of soil considered in this geometric model simulation. Finite Element Method (FEM, PLAXIS 3D) is used in the analysis where two dimensional footing is considered.

1. Study is limited to footing subjected to static loading only.

2. Soil is idealized to Mohr-coulomb yield criterion.

1.6. Significance of the Study

This thesis helps to evaluate bearing capacity of layered soils using finite element analysis with Mohr- coulomb failure criteria and to compare the results of evaluation of bearing capacity from PLAXIS and those from other theories.

1.7. Methodology

The methods that has been used for this thesis includes document review, hand calculation and Using a finite element software Plaxis. Previous research studies that are found to be relevant to the objectives of this research has been reviewed and the results of evaluation of bearing capacity from PLAXIS and those from conventional theories has been compared. 3D Finite Element Analysis Model in Plaxis has been done and compared with other methods of calculating bearing capacity analysis.

1.8. Organization of the thesis

The first chapter consists of the introductory part. The second Chapter following this one tried to point out the major works done related to this matter. It pointed out the experiments performed and the formulas developed by different scholars specialized in this Geotechnical topic. It also reviewed case studies and thesis reports done on PLAXIS and their results will be discussed.

The third Chapter is on modeling. This chapter will focus on the foundation model, geometry, following that plaxis model has been developed and using Empirical method calculations were performed. It will also give a brief description procedures and the geometrical input for the model in the software mentioned. In the fourth Chapter Parametric studies are conducted and results are compared and data was analyzed at last conclusion and recommendations are given.

Chapter Two

2. Literature Review

2.1. Introduction

The determination of bearing capacity of soil based on the classical earth pressure theory of Rankine (1857) began with Pauker, a Russian military engineer (1889), and was modified by Bell (1915). Pauker's theory was applicable only for sandy soils but the theory of Bell took into account cohesion also. Neither theory took into account the width of the foundation. Subsequent developments led to the modification of Bell's theory to include width of footing also.

The methods of calculating the ultimate bearing capacity of shallow strip footings by plastic theory developed considerably over the years since Terzaghi (1943) first proposed a method by taking into account the weight of soil by the principle of superposition. Terzaghi extended the theory of Prandtl (1921).

Prandtl developed an equation based on his study of the penetration of a long hard metal punch into softer materials for computing the ultimate bearing capacity. He assumed the material was weightless possessing only cohesion and friction.

Taylor (1948) extended the equation of Prandtl by taking into account the surcharge effect of the overburden soil at the foundation level. No exact analytical solution for computing bearing capacity of footings is available at present because the basic system of equations describing the yield problems is nonlinear. On account of these reasons, Terzaghi (1943) first proposed a semi-empirical equation for computing the ultimate bearing capacity of strip footings by taking into account cohesion, friction and weight of soil, and replacing the overburden pressure with an equivalent surcharge load at the base level of the foundation. This method was for the general shear failure condition and the principle of superposition was adopted. His work was an extension of the work of Prandtl (1921). The final form of the equation proposed by Terzaghi is the same as the one given by Prandtl.[19]

2.2. Bearing capacity equations

2.2.1. Ultimate Bearing Capacity (Q**u**)

A foundation, often constructed from concrete, steel or wood, is a structure designed to transfer loads from a superstructure to the soil underneath the superstructure. In general, foundations are categorized into two groups, namely, shallow and deep foundations. Shallow foundations are comprised of footings, while deep foundations include piles that are used when the soil near the ground surface has no enough strength to stand the applied loading. The ultimate bearing capacity, $q_{u,}$(kPa) is the load that causes the shear failure of the soil underneath and adjacent to the footing.

2.2.2. Bearing Failure Modes

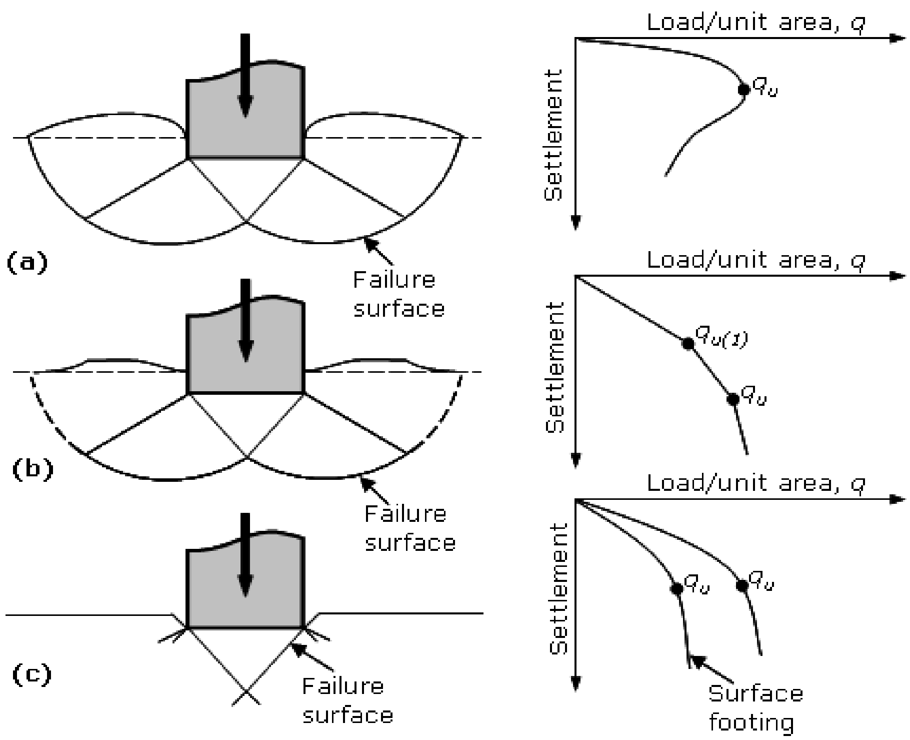


Figure 2.1: Modes of bearing failures (a) General shear (b) Local shear and (c) Punching shear.

Relative density of the soil and size of the foundation are among the major factors that affect the mode of bearing failure likely to occur. The modes of bearing failure are generally separated into three categories: The *general shear failure* (Fig. 2.1 a) is usually associated with soils of low compressibility such as dense sand and stiff cohesive soils. In this case, if load is gradually applied to the foundation, settlement will increase. At a certain point – when the applied load per unit area equals to the ultimate load q_u – a *sudden* failure in the soil supporting the foundation will take place. The failure surface in the soil will extend to the ground surface and full shear resistance of the soil is developed along the failure surface. Bulging of the soil near the footing is usually apparent.

For the *local shear failure* (Fig. 2.1 b), which is common in sands and clays of medium compaction, the failure surface will gradually extend outward from the foundation but will not reach the ground surface as shown by the solid segment in Fig. 2.1 b. The shear resistance is fully developed over only part of the failure surface (solid segment of the line). There is a certain degree of bulging of the soil.

In the case of *punching shear failure*, a condition common in loose and very compressible soils, considerable vertical settlement may take place with the failure surfaces restricted to vertical planes immediately adjacent to the sides of the foundation; the ground surface may be dragged down. After the first yield has occurred the load-settlement curve will be steep slightly, but remain fairly flat.[18]

2.2.3. Bearing Capacity of Layered Soils

Most of the theoretical analysis dealt with so far is based on the assumption that the subsoil is isotropic and homogeneous to a considerable depth. In nature, soil is generally non-homogeneous with mixtures of sand, silt and clay in different proportions. In the analysis, an average profile of such soils is normally considered. However, if soils are found in distinct layers of different compositions and strength characteristics, the assumption of homogeneity to such soils is not strictly valid if the failure surface cuts across boundaries of such layers.[19]

Geotechnical engineers often should solve problems in layered soil while the majority of existing studies have mostly focused on homogeneous continuum. Predicting ultimate bearing capacity of footings on layered soil is very important as it is a requirement for any design and the failure mechanism of soil under footing and the bearing capacity value mainly depend on soil properties of each layer and the layer thickness. Terzaghi and Peck for the first time in 1948 analyzed strip footing behavior on sand overlaying clay which followed by many researchers.

Layered soil profiles are often encountered whether naturally deposited or artificially made. Within each layer, the soil may be considered as homogeneous. The ultimate load failure surface in the soil depends on the shear strength parameters of the soil layers such as; the thickness of the upper layer; the shape, size and embedment of footing; and the ratio of the thickness of the upper layer to the width of the footing. Therefore, it is important to determine the soil profile and to calculate the bearing capacity accordingly.

In many circumstances it is possible that a foundation is located on a layered and the interface of the soil layering is at a shallow depth. Depending on the ratio H/B it is possible that the ultimate bearing capacity of the foundation.

Button, 1953 analyzed the bearing capacity of a strip footing resting on two layers of clay. He assumed that the cohesive soils in both layers are consolidated approximately to the same degree. In order to determine the ultimate bearing capacity of the foundation, he assumed that the failure surface at the ultimate load is cylindrical, where the curve lies at the edge of the footing. The bearing capacity factor used depends on the upper soil layer and on the ratio of the cohesions of the lower/upper clay layers. [1]

Reddy and Srinivasan, 1967 extended the work of Button to include the effect of the non-homogeneity and anisotropy of soil with respect to the shear strength. The basic assumptions involved in determining the ultimate bearing capacity are: the failure surface is cylindrical, the coefficient of anisotropy is the same at all points in the foundation medium, the soil in each layer is either homogenous with respect to the shear strength in each layer varies linearly with depth and for the soil at failure, the $\phi = 0$ consolidation is valid. [2]

Brown and Meyerhof, 1969 investigated foundations resting on a stiff clay layer overlaying a soft clay layer deposit, and the case of a soft layer overlying a stiff layer. They assumed that the footing fails by punching through the top layer for the first case, and with full development of the bearing capacity of the lower layer in the second case. Equations and charts giving the appropriate modified bearing capacity factors were given derived from the empirical relationships obtained based on the experimental results. The results of the investigation are summarized in charts, which may be used in evaluating the bearing capacity of layered clay foundations, but these results are essentially experimental, and therefore are strongly affected by the characteristics of the clay tested. [3]

The purpose of this paper is to present the results of a series of model footing tests carried out on two-layered clay soils, and the models have many limitations. First, they are limited to one type of clay, although the strength of the clay was varied, the deformation properties remained constant. Second, studies were limited to surface loading only, using rigid strip and circular footings with rough bases. Third, all studies were made in terms of the undrained shear strength of the clay, using the "$\phi = 0$" analyses.

They also conducted a series of tests on footings in homogenous clay. They observed that the pattern of failure beneath a footing is a function of the physical mode of rupture of the clay, which is strongly dependent on the structure of the clay. The failure mechanism of the structure of the clay is not adequately defined by conventional Mohr-coulomb concepts of cohesion and friction.

Meyerhof, 1974 (18) investigated the case sand layer overlying clay: dense sand on soft clay and loose sand on stiff clay. The analyses of different modes of failure were compared with the results of model test results on circular and strip footings and field data. [14]

In the case of dense sand overlying a soft clay deposit, the failure mechanism was assumed as an approximately truncated pyramidal shape, pushed into the clay so that, in the case of general shear failure, the friction angle ϕ of the sand and the undrained cohesion c of the clay are mobilized in the combined failure zones. Based on this theory, semi-empirical formulae were developed to calculate the bearing capacity of strip, and circular footings resting on dense sand overlying soft clay. He conducted model tests on strip and circular footings on the surface and at shallow depths in the dense sand layer overlying clay. The results of these tests, and the field observations were found to agree with the theory developed.

In the case of loose sand on stiff clay, the sand mass beneath the footing failed laterally by squeezing at an ultimate load. Formulae for the ultimate bearing capacity of strip and circular footings were developed. Model tests were carried out on strip and circular footings, and the results also agreed with the theory developed.

Theory and test results showed that the influence of the sand layer thickness beneath the footing depends mainly on the bearing capacity ratio of the clay to the sand, the friction angle ϕ of the sand, the shape and depth of the foundation.

This paper is limited to vertically loaded footings, and does not include eccentric or inclined loads, it is also limited to sand over clay, and has no solution for clay over sand. In the case of dense sand on soft clay, the theory considers simultaneous failure of the sand layer by punching, and general shear failure in the clay layer, which isn't always the case.

Meyerhof and Hanna, 1978 considered the case of footings resting in a strong layer overlying weak deposit and a weak layer overlying strong deposit. The analyses of different soil failure were compared with the results of model tests on circular and strip footings on layered sand and clay. They developed theories to predict the bearing capacity of layered soils under vertical load and inclined loading conditions. [15]

In the case of a strong layer overlying a weak deposit, considering the failure as an inverted uplift problem, an approximate theory of the ultimate bearing capacity was developed, At failure, a soil mass, roughly shaped like a truncated pyramid, of the upper layer is pushed into the underlying deposit in the approximate direction of the applied load. The forces developed on the actual punching failure surface in the upper layer are the total adhesion force and a total passive earth pressure inclined at an average angle δ acting upwards on an assumed plane inclined at an angle α to the vertical. The analysis for strip footings was extended to circular and rectangular footings, and approximate formulae for the bearing capacity of strip, rectangular, and circular footings were developed, taking into consideration the case of eccentric and inclined loading as well. Model tests on rough strip and circular footings under central inclined loads at varying angles α were made on the surface and at shallow depth in different cases of two layered soils of sand and clay, where good agreement was found between the theoretical and experimental results.

In the case of a weak layer overlying a strong deposit, considering that the weak soil mass beneath the footing may fail laterally by squeezing, which is the same theory as from the previous paper developed the theory of the ultimate bearing capacity. The bearing capacity can be estimated by the approximate semi-empirical formula. Model tests were also carried out on strip and circular footings under vertical and inclined loads, and the results of the tests were compared to the theoretical ones.

This paper is a development of the previous theory (Meyerhof 1974), taking into consideration all possible cases of two different layers of subsoil, and also including the effect of inclined and eccentric loading on the ultimate bearing capacity of strip, rectangular and circular footings. This theory and the failure mechanism considered are approximations of the real failure mechanism, which depends on many factors.

Hanna and Meyerhof, 1980 extended the previous theory to cover the case of footings resting on subsoil consisting of a dense layer of sand overlying a soft clay deposit, and they presented the results of this analysis in the form of design charts. It is a kind of revision of the assumptions previously used in the punching theory of the previous papers in order to reduce their effects on the analysis. They showed that when the clay layer is relatively weak, passive failure of the sand layer may be accompanied by a failure surface that extends downwards into the clay. In this case, values of *K*p obtained by assuming that failure is confined to the sand layer will be too large. Hanna and Meyerhof used a limit-equilibrium approach to study this case and obtained values for a coefficient of punching shear, *K*s, which is related to *K*p by the equation *K*p tan d = *K*s tan f¢. They presented the results of the analysis in the form of charts in which the relationship between *K*s and f¢ is plotted as a function of the clay shear strength, *s*u, and the ratio d/f¢. These charts are not presented in a non-dimensional form, however, and so are appropriate only for the values of sand unit weight and layer thickness that were adopted in their preparation (Burd and Frydman1996). Furthermore, the precise details of the limit-equilibrium calculation used by Hanna and Meyerhof are not clear, and so it is not straightforward to repeat their analysis.[10]

Hana and Meyerhof, 1981 investigated experimentally the ultimate bearing capacity of footings subjected to axially inclined loads by conducting tests on model strip and circular footings on homogeneous sand and clay. The results were analyzed to determine the inclination factors, depth factors and the shape factors incorporated in the general bearing capacity equation for shallow foundations. These values were compared with the recommended values given in the Canadian Foundation Engineering Manuel. The values of these factors given in the manual agree reasonably well with the experimental ones, except for the depth and shape factors, for which the theoretical values are on the conservative side when applied to inclined loads. [16]

Abdel-Baki et al., 1993 investigated the effect of a single strong reinforcement layer, placed within granular soil, on the bearing capacity of footings subjected to eccentric, inclined and concentric loads. The effect of the reinforcement on the bearing capacity was investigated experimentally. [1]

The results of the experimental tests proved that the reinforcement had a considerable effect on the bearing capacity of the footings. The bearing capacity of a reinforced layer was about three times the unreinforced one. The effect of the length of the reinforcement was also investigated, and it was found that there is no significant effect on the bearing capacity if the reinforcement length is extended over 1.25 times the width of the footing. The effect of reinforcement continuity below the footing was also investigated in this paper, and it was found that the bearing capacity from the reinforcement decreased as the gap in the reinforcement increased and reached zero when the gap width was equal to the footing width.

Merifield*et al* examined the ultimate bearing capacity of a footing on a two-layer undrained clay profile employing numerical limit analysis. The upper and lower bound limit results were presented in the form of bearing capacity factors in terms on the depth ratio and strength ratio, which can be used in designing of foundation on soft-over-strong or strong-over soft clay deposit.

2.3. Developed M-H method

The punching shear model proposed by Meyerhof and Hanna is based on a well-established theory and provides a useful insight into the behavior of the subsoil consisting of two-layered soils. However, in case that the subsoil consists of layered soils more than two layers, it will have difficulties. On the basis of failure mechanism assuming that, at the ultimate load, a soil mass of upper layers is vertically punched into the lower layer, the ultimate bearing capacity can be presented as:

$$Qu = qb + 2\sum_{i=1}^{n}\frac{CiHi}{B} + \sum_{i=1}^{n-1} Hi(\gamma iHi + 2\sum_{j=1}^{i-1} Hj\gamma j + 2D)\frac{Ksi\tan\varphi}{B} - \sum_{i=1}^{n-1}\gamma iHi \ldots\ldots 2.1$$

For Square footing, introducing shape factor in to eqn 2.1the equation yields to

$$Qu = qb + 2\sum_{i=1}^{n}\frac{CaiHi}{B} + \sum_{i=1}^{n-1} Hi(\gamma iHi + 2\sum_{j=1}^{i-1} Hj\gamma j + 2D)\frac{Ksi\tan\varphi}{B} - \sum_{i=1}^{n-1}\gamma iHi \ldots\ldots 2.2$$

Where K_{si}, γi, φ,ci and Hi are coefficients of punching shear, the value of the unit weight , the angle of internal friction, the value of cohesion and thickness of every layer, respectively. This equation is a general governing equation. Therefore, it is widely used in the thesis. [21]

Naturally occurring soils are often deposited in layers. Within each layer the soil may, typically, be assumed to be homogeneous, although the strength properties of adjacent layers are generally quite different. Bearing capacity for layered soils is not limited to two layers but also for three and four layers can be calculated also. If a foundation is placed on the surface of a layered soil for which the thickness of the top layer is large compared with the width of the foundation, then realistic estimates of the bearing capacity may be obtained using conventional bearing capacity theory based on the properties of the upper layer. Note that there will not be many cases of a two- (or three-) layer cohesive soil with clearly delineated strata. Usually the clay gradually transitions from a hard, over consolidated surface layer to a softer one; however, exceptions may be found, primarily in glacial deposits. In these cases, it is a common practice to treat the situation as a single layer with a worst-case su value. A layer of sand overlying clay or a layer of clay overlying sand is somewhat more common, and the stratification is usually better defined than for the two-layer clay. An increase in shear strength with depth could be approximated by addition of "soils" with the same ϕ and Ɣ properties but increased cohesion strength. Footings in Anisotropic soil usually occurs in *cohesive* soils where the undrained vertical shear strength $S_U$is different (usually larger) from the horizontal shear strength. This is a frequent occurrence in cohesive field deposits but also is found in cohesion less deposits.

A possible alternative for ϕ-c soils with a number of thin layers is to use average values of c and ϕ in the bearing capacity equations

$$c_{av} = \frac{c_1H_1 + c_2H_2 + c_3H_3 + \cdots + c_nH_n}{\sum H_i}$$

$$\phi_{av} = \tan^{-1}\frac{H_1\tan\phi_1 + H_2\tan\phi_2 + \cdots + H_n\tan\phi_n}{\sum H_i}$$

Where C_i = cohesion in stratum of thickness *Hi*; *c* may be 0

ϕ = angle of internal friction in stratum of thickness Hi; ϕ may be 0

In multi-layered subsoil each soil layer is assumed to be homogenous with thickness H_i. The footing is regarded as a rigid body, since it is stiffer than the soils. Several important examples exist of foundation engineering problems in which it may be necessary to include the effect of soil layers in an assessment of bearing capacity. Shallow offshore foundations, for example, generally have large physical dimensions; potential failure surfaces may therefore extend a significant distance below the soil surface. Any soil layers within the depth of these failure surfaces would be expected to influence the failure load.

Methods of solving bearing capacity of footing are classified in four major approaches: limit equilibrium, limit analysis approach, semi empirical approach and finite element method. In recent years, Finite Element Method (FEM) have been widely using in geotechnical studies to investigate soil behavior. In practice, for bearing capacity analysis engineers are seeking less complicated solutions to simplify computations as experimental analysis is time consuming and commonly used solutions such as limit equilibrium are no longer applicable. Therefore, computer programs developed based on the finite element method have been receiving much attention over recent decades as the powerful tool for solving complex cases. Hence, the application of FEM to evaluate bearing capacity evaluation of a footing on a layered soil is the objective of this study.

Another commonly used semi-empirical approach is the load spread model, in which the sand layer is assumed to merely spread the load to the underlying clay foundation. This technique ignores the shear strength of the sand and transforms the problem to the classical one of estimating the bearing capacity of a clay layer that is subjected to a surcharge. [19]

A more rigorous approach, based on the upper bound theorem of limit analysis, has been used by to calculate the bearing capacity of multilayered soils. These studies consider various rigid block mechanisms that, at the point of collapse, assume power is dissipated solely at the interfaces between adjacent blocks. After optimizing the geometry to furnish the minimum dissipated power, the mechanism that gives the lowest value is used to compute the best upper bound on the limit load. To give confidence in the accuracy of the solutions obtained from upper bound calculations, it is desirable to perform lower bound calculations in parallel so that the true result can be bracketed from above and below [12]. Unfortunately, due to the difficulty in constructing statically admissible stress fields by traditional methods, this is rarely done in practice. [8]

Although conventional finite element analysis can be used to predict the bearing capacity of multilayered soils, the estimate so obtained is neither an upper bound nor a lower bound on the true value. Moreover, great care needs to be exercised with displacement finite element predictions in the fully plastic range, as the results can be very inaccurate due to the occurrence of locking . This phenomenon, which is characterized by a constantly rising load–deformation response, occurs when the displacement field becomes over constrained by the requirements of an incompressible plastic flow rule. To estimate undrained limit loads accurately using displacement finite elements in plane strain, it is prudent to avoid using low order Formulations unless they are used with some form of selective integration. [16]

2.4. Numerical limit analysis models

This section gives a very brief summary of the finite element formulations of the upper and lower bound theorems. The lower bound limit theorem states that if any equilibrium state of stress can be found which balances the applied loads and satisfies the yield criterion as well as the stress boundary conditions, then the body will not collapse. Stress fields that satisfy these requirements, and thus give lower bounds, are said to be statically admissible. The key idea behind the lower bound analysis applied here is to model the stress field using finite elements and use the static admissibility constraints to express the unknown collapse load as a solution to a mathematical programming problem. For linear elements, the equilibrium and stress boundary conditions give rise to linear equality constraints on the nodal stresses, and the yield condition, which requires all stress points to lie inside or on the yield surface, gives rise to a nonlinear inequality constraint on each set of nodal stresses. The objective function, which is to be maximized, corresponds to the collapse load and is a function of the unknown stresses. For linear elements, this function is also linear. After all the element coefficients are assembled, the final optimization problem is thus one with a linear objective function, linear equality constraints, and nonlinear inequality constraints. The lower bound formulation proposed handles both two- and three-dimensional stress fields and, in the former case, employs the linear elements and it incorporates statically admissible stress discontinuities at all inter-element boundaries and special extension elements for completing the stress field in an unbounded domain. Unlike displacement finite element meshes, each node is unique to a single element and several nodes may share the same coordinates. Although the stress discontinuities increase the total number of variables for a fixed mesh, they also introduce extra "degrees of freedom" in the stress field, thus improving the accuracy of the solution. Along a given discontinuity the normal and shear stresses must be continuous, but the tangential normal stress may be discontinuous.[13]

As the formulation of FEM uses linear elements in both two and three dimensions, the objective function and equality constraints are linear in the unknowns. As discussed previously, the equality constraints arise because the stress field needs to satisfy equilibrium in the continuum and the discontinuities, as well as the stress boundary conditions. Some additional constraints may also be required to incorporate special types of loading along the domain boundaries. After assembling the various objective function coefficients and equality constraints for the mesh, and imposing the nonlinear yield inequalities on each node, the lower bound formulation of Lyamin and Sloan leads to a nonlinear programming problem .[13]

The upper bound theorem can be formulated in terms of finite elements by adopting a similar approach to that just described for the static case. This theorem states that the power dissipated by any kinematically admissible velocity field can be equated to the power dissipated by the external loads to give a rigorous upper bound on the true limit load. A kinematically admissible velocity field is one that satisfies compatibility, the flow rule, and the velocity boundary conditions. In a finite element formulation of the upper bound theorem, the velocity field is modeled using appropriate variables, and the optimum (minimum) internal power dissipation is obtained as the solution to a mathematical programming problem.

2.5. Basics of Finite Element Analysis

Finite element analysis is a method of solving continuous problems governed by differential equations by dividing the continuum into a finite number of parts (elements), which are specified by a finite number of parameters. [20]

A problem is solved by dividing the larger geometry into small elements, which are interconnected with nodes. Each element is assigned an element property. In solid mechanics, the properties include stiffness characteristics for each element. This force displacement relationship is expressed as the element stiffness matrix, the nodal displacement vector of the element and in the nodal force vector.

1. In classical methods exact equations are formed and exact solutions are obtained where as in finite element analysis exact equations are formed but approximate solutions are obtained.
2. Solutions have been obtained for few standard cases by classical methods, whereas solutions can be obtained for all problems by finite element analysis.
3. Whenever the following complexities are faced, classical method makes the drastic assumptions ‘and looks for the solutions:
   a. Shape
   b. Boundary conditions
   c. Loading

   To get the solution in the above cases, rectangular shapes, same boundary condition along a side and regular equivalent loads are to be assumed. In FEM no such assumptions are made. The problem is treated as it is.

4. When material property is not isotropic, solutions for the problems become very difficult in classical method. Only few simple cases have been tried successfully by researchers. FEM can handle structures with anisotropic properties also without any difficulty.
5. If structure consists of more than one material, it is difficult to use classical method, but finite element can be used without any difficulty.
6. Problems with material and geometric non-linearity cannot be handled by classical methods.

There is no difficulty in FEM. Hence FEM is superior to the classical methods only for the problems involving a number of complexities which cannot be handled by classical methods without making drastic assumptions. For all regular problems, the solutions by classical methods are the best solutions. In fact, to check the validity of the FEM programs developed, the FEM solutions are compared with the solutions by classical methods for standard problems.

2.5.1. Modeling by Plaxis

In the present study finite element analysis has been carried out using PLAXIS 3D FOUNADATION software. PLAXIS 3D FOUNADATION is a three-dimensional finite element program especially developed for the analysis of foundation structures. It combines simple graphical input procedures, which allows the user to automatically generate complex finite element models, with advanced output facilities and robust calculation procedures.

PLAXIS 3D FOUNDATION program consists of four basic components, namely Input, Calculation, Output and Curves. In the Input program the boundary conditions, problem geometry with appropriate material properties are defined. The problem geometry is the representation of a real three-dimensional problem and it is defined by work-planes and boreholes. The model includes an idealized soil profile, structural objects, construction stages and loading. The model should be large enough so that the boundaries do not influence the results. Boreholes are points in the geometry model that define the idealized soil layers and the groundwater table at that point. Multiple boreholes are used to define the variable soil profile of the project. During 3D mesh generation soil layers are interpolated between the boreholes so that the boundaries between the soil layers coincide with the boundaries of the elements.

Workplanes are horizontal planes with different y-coordinates that show the top view of the model geometry. They are used to draw, activate and deactivate the structural elements and loads. Each workplane holds the same geometry lines but vertical distance between them may vary. Within work-planes, points, lines and clusters are used to describe a 2D geometry mode.

After the creation of 2D geometry model in a work-plane, a 2D finite element mesh is automatically generated based on the composition of the clusters and lines in 2D geometry model. 2D finite element mesh is composed of 6-nodes triangles. However, the 3D finite element mesh is the extension of 2D mesh into the third dimension and it is generated after generating 2D mesh. The 2D mesh generation in the program is fully automatic while 3D mesh generation is semi-automatic. Mesh dimensions should be appropriately defined, to prevent the effects of boundary conditions. The 2D mesh should be constructed before proceeding to the 3D mesh extension.

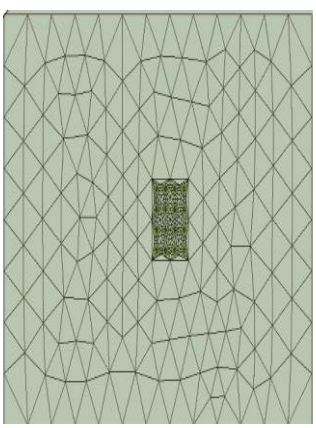


Figure 2.2a : 2D Mesh Generation of FE Model in PLAXIS

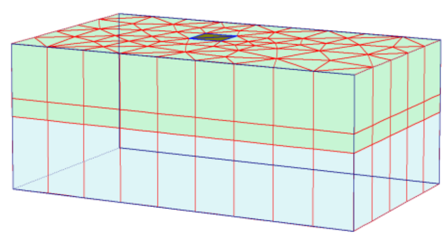


Figure 2.2 b 3D Mesh Generation of FE Model in PLAXIS

Typical 2-D and 3-D meshes used in this study are presented in Figure: 2.2(a) & 2.2(b) shown above. The mesh element size can be adjusted by using a general mesh size varying from very coarse to very fine and also by using line, cluster and point refinements. Very fines meshes should be avoided in order to reduce the number of elements, thus to reduce the memory consumption and calculation time. The program does not allow entering a new structural element or a new soil cluster after the mesh is generated. If a new element or cluster is added to the geometry model, the mesh generation should be repeated with the new input.

3D finite element mesh is composed of elements, nodes and stress points. While generating the mesh, the geometry is divided into 15-node wedge elements. These elements are composed of the 6-node triangular faces in x-z direction and 8-node quadrilateral faces in y-direction. In addition to the volume elements, which are generally used to model the soil, compatible 3-node line elements, 6-node plate elements and 16- node interface elements may be generated to model structural behavior and soil-structure interaction respectively. The wedge elements as used in the 3D Foundation program consist of 15 nodes. The distribution of nodes over the elements is shown in figure –.2. Joining elements are connected through their common nodes. During a finite element analysis, displacement values are calculated at the nodes and a specific node can be selected before calculation steps in order to generate the load displacement curves. In this thesis a medium mesh generation is used and the medium mesh generation is for 250 elements with n_c =100.

On the contrary, stresses and strains are calculated at individual stress points (Gaussian integration points) rather than at the nodes. A 15-node wedge element contains 6 stress points that shown in Figure 2.3.

However, stress and strain values at stress points are extrapolated to the nodes for the output purposes. At the bottom of the 3D finite element mesh, total fixities were used that restrain the movements in both horizontal and vertical directions. For upper part, 3D finite element mesh had no fixities.

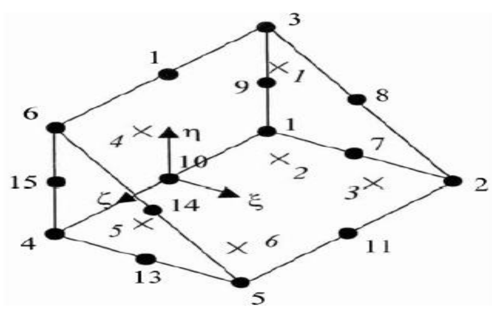


Figure: 2.3 Distribution of nodes (●) and stress points (x) in a 15-node wedge element

After defining the model geometry and generation of 3D mesh, initial stresses are applied by using either K0-procedure or gravity loading. The initial stresses in the soil are affected by the weight of the soil and history of the soil formation. Stress state is characterized by vertical and horizontal stresses. Initial vertical stress depends on the weight of the soil and pore pressures; whereas initial horizontal stresses are related to the vertical stresses multiplied by the coefficient of lateral earth pressure at rest. The K0-procedure may only be used for horizontally layered geometrics with a horizontal ground surface and, if applicable, a horizontal phreatic level. In the present study the K0-procedure has been used.

In this analysis the effect of ground water table is not taken into account. If it would have been taken into account then the initial stress would have been calculated by effective stress consideration. As the effect of ground water table is not considered it has been done by the total stress consideration only.

$$\sigma h=K_o\sigma v$$

where Ko is based on jacky's formula (1944) which is written as:

$$K_o=1- \sin\varphi$$

As in all PLAXIS programs the 3D Foundation program has convenient procedures for automatic load stepping and for the activation and deactivation of loads and parts of the geometry (Staged Construction).

Staged construction is a very useful type of loading input. In this special PLAXIS feature it is possible to change the geometry and load configuration by deactivating or reactivating loads, volume elements or structural objects as created in the geometry input. Staged construction provides an accurate and realistic simulation of the various loading, construction and excavation processes. This option can also be used to reassign material data sets to clusters or structural elements.

As mentioned above the construction stages are defined by activating or deactivating the structural elements or soil clusters in the work-planes and a simulation of the construction process can be achieved.

In every construction step, the material Properties, geometry of the model, loading condition and the ground water level can be redefined. During the calculations in each construction step, a multiplier that controls the staged construction process (ΣMstage) generally starts at zero and is expected to reach the ultimate level of 1.0 at the end of the calculation phase.[17]

Chapter Three

3. Modeling

3.1. Introduction

In this section of the research, soils are modeled with adjacent foundation loads with different foundation types in which the details will be discussed in the subsequent sections.

The first part explains the soil parameter which is used in the analysis of the bearing capacity. The representative soil samples are incorporated from reference material. In the second part description and results of parametric study on the same soils and selected parameters are presented.

It has been taken one type of load for isolated type of foundation load and for the given load one to four types of soil layers were considered. The depth is 8m for all the cases. There are 2 cases in which the four layers are positioned. The first one is soft saturated clay, silty clay, stiff clay and dense gravel respectively. Whereas dense gravel, stiff clay, silty clay and soft saturated clay are in the second case respectively. In the analysis it is tried to analyze the difference and also the similarity between conventional methods which has been known for many years by Geotechnical Engineers and also FEM which has gain much acceptance in recent years. From the result it has shown that the bearing capacity calculation for FEM is higher than conventional methods in many cases. Since FEM has taken into account approximate solutions it analyzed the soil condition and gives higher values than that of conventional method which takes the soil as a homogenous and isotropic even though the real soil condition is not like that, most of the time.

3.2. Soil Properties

The soil parameters considered for this study are obtained from reference materials. An average value has been taken for the accuracy and modeled using Mohr columb (MC) constitutive relation. The detail properties of the soil model are presented in Appendix A.2.

The Detailed Soil Properties which are taken as input for plaxis 3D software are presented in the table below.

3.2.1 Soil Parameters for the models

Table 3.1: Parameters of soil type 1

Parameter	**Symbol**	**Unit**	**Soil type**			
Soil Type			**Dense Gravel**	**Stiff Clay**	**Silty clay**	**Saturated soft clay**
model utilized	model		Mohr columb	Mohr columb	Mohr columb	Mohr columb
Soil unit weight	γ_{unsat}	kN/m^3	20	18	17	10
	γ_{sat}	kN/m^3	22	20	19.5	12.5
Young's modulus	$E_{ref.}$	kN/m^2	200000	75000	7000	2000
poisson's ratio	ν	-	0.35	0.3	0.3	0.5
Cohesion	$C_{ref.}$	kN/m^2	5	40	30	12.5
Friction angle	ϕ	-	40	6	6	6
Diltancy angle	ψ	-	0	0	0	0

Note:- Soil parameters are taken from Bowles.

Table 3.2: Parameters of soil type 2

Parameter	Name	Unit	Soil type			
Soil Type			Loose sand	Stiff saturated clay	Soft clay	Dense sand
model utilized	model		Mohr columb	Mohr columb	Mohr columb	Mohr columb
Soil unit weight	γ_{unsat}	kN/m^3	12	16.5	16	20
	γ_{sat}	kN/m^3	14	19.5	19	22
Young's modulus	$E_{ref.}$	kN/m^2	10000	15000	5000	81000
poisson's ratio	ν	-	0.25	0.4	0.4	0.35
Cohesion	$C_{ref.}$	kN/m^2	1	25	15	1
Friction angle	ϕ	-	30	6	6	40
Diltancy angle	ψ	-	0	0	0	0

3.3 Structural parameters

The structural parameters include the isolated footing is given below.

Table 3.3: Parameters of isolated footing foundation

Footing Legend	Footing type	Footing Dimension	L/B Ratio	Thickness d(m)	Density of material γ (KN/m^2)	Modulus of Elasticity E, (KN/m^2)	Poisson's Ratio. v	Shear Modulus, G (KN/m^2)
F-I	square	2.5m x2.5m	1	0.4	24	$3x10^7$	0.2	$1.25x10^7$

3.4 Geometric Model

3.4.1. Type of Soil and Geometry

- The Soil stratum for structural systems listed below

i. For One Layered Soil

Case 1: For One Layered Soil

Layers	Soil Type
Layer 1	Dense Gravel

Case 2: For One Layered Soil

Layers	Soil Type
Layer 1	Dense sand

Case 3: For One Layered Soil

Layers	Soil Type
Layer 1	Stiff Saturated clay

ii. For Two Layered soils

Case 1: For Two Layered Soil

Layers	Soil Type
Layer 1	Dense Gravel
Layer 2	Soft Clay

Case 2: For Two Layered Soil

Layers	Soil Type
Layer 1	Dense Sand
Layer 2	Saturated Soft clay

Case 3: For Two Layered Soil

Layers	Soil Type
Layer 1	Dense sand
Layer 2	Loose sand

Case 4: For Two Layered Soil

Layers	Soil Type
Layer 1	Stiff Saturated clay
Layer 2	Soft clay

iii. For Three layered Soils

Case 1: For Three Layered Soil

Layers	Soil Type
Layer 1	Soft saturated clay
Layer 2	Silty clay
Layer 3	Stiff clay

Case 2: For Three Layered Soil

Layers	Soil Type
Layer 1	Dense Gravel
Layer 2	Stiff clay
Layer 3	Silty Clay

iii. For Four Layered Soils

Case 1: For Four Layered Soil

Layers	Soil Type
Layer 1	Soft saturated clay
Layer 2	Silty clay
Layer 3	Stiff clay
Layer 4	Dense Gravel

Case 2: For Four Layered Soil

Layers	Soil Type
Layer 1	Dense Gravel
Layer 2	Stiff clay
Layer 3	silty clay
Layer 4	Soft Saturated Clay

Table 3.4: Program of PLAXIS 3D Model for Bearing Capacity Analysis

Models for soil combinations with h/B					
Model No	Footing	h/B	soil No	No of soil layers	Soil Type
1		0.6	S1	Two layers	S1-dense gravel/ soft clay S2-dense sand/sat soft clay S3-dense sand/loose sand S4-stiff sat clay/ soft clay S5-dense gravel/stiff sat clay S6-soft sat clay/stiff sat clay S7-soft sat clay/silty clay S8-dense gravel/stiff clay S9-dense sand/soft sat clay S10-soft clay/dense gravel S11-sat soft clay/ dense sand S12- loose sand/dense sand
2			S2		
3			S3		
4			S4		
5		1.2	S1		
6			S2		
7			S3		
8			S4		
9			S5		
10			S6		
11			S7		
12			S8		
13			S9		
14			S10		
15			S11		
16			S12		
17		1.6	S1		
18			S2		
19			S3		
20			S4		
21		0.6	Sa	Three Layers	Sa-Dense gravel/ Stiff Sat clay/ Soft sat clay Sb- Soft Sat clay/ Stiff sat clay/ dense gravel Sc-soft sat clay/silty clay/Stiff clay Sd-Dense gravel/stiff clay/silty clay
22			Sb		
23		1.2	Sa		
24			Sb		
25			Sc		
26			Sd		
27		2	Sa		
28			Sb		
29		0.4	Si	four layers	Si-Soft Clay/silty clay/stiff clay/dense gravel Sii-dense gravel/Stiff clay/Silty clay/soft sat clay
30			Sii		
31		1.6	Si		
32			Sii		

3.5 Model generation Using Plaxis 3D Software

In the present investigation attempt has been made to study the bearing capacity of shallow footings (e.g. square footings) resting on layered cohesive soils using numerical modeling and analysis in finite element software package PLAXIS 3D. The following figures show input procedures, mesh generation and output curves of a PLAXIS model involved in the present study.

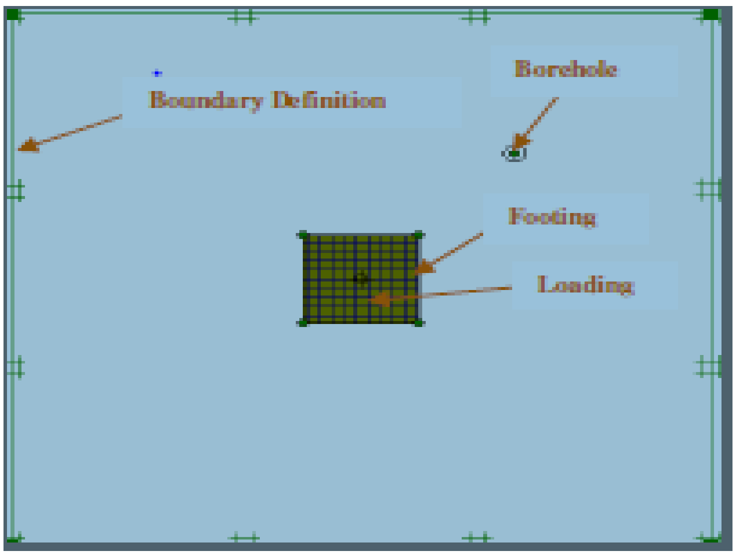


Figure 3.7: PLAXIS 3D Input Window

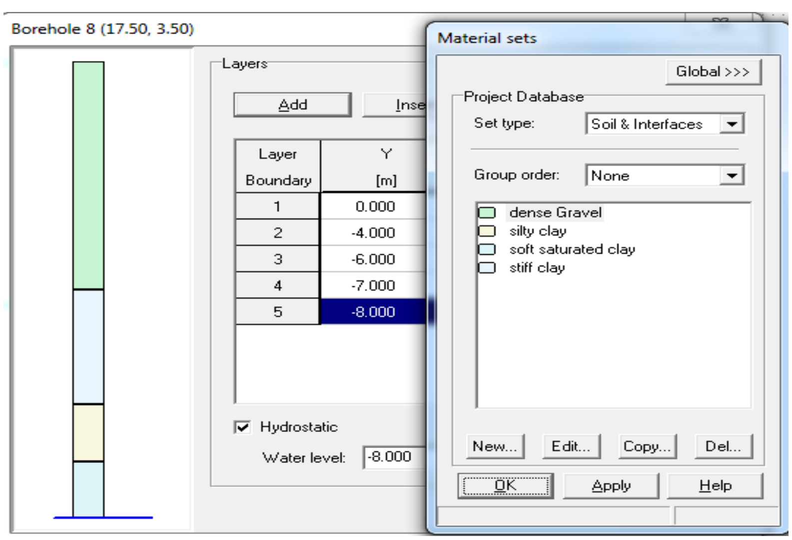


Figure 3.8: Borehole and Soil Parameters Definition

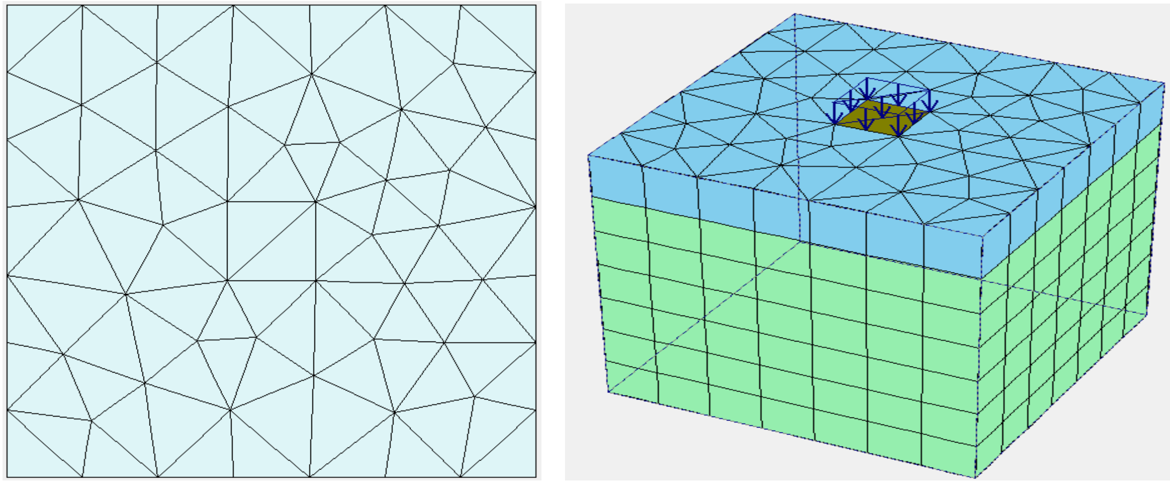


Figure 3.9: 2D and 3D Mesh Generation of FE Model in PLAXIS

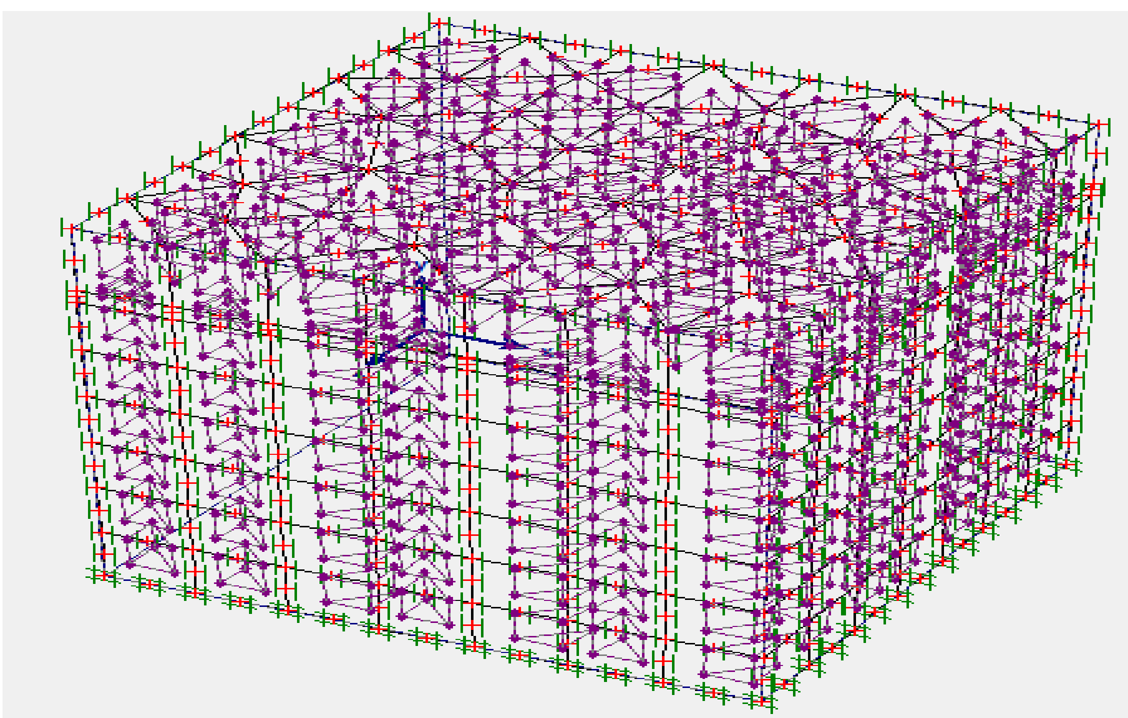


Figure - 3.10: Distribution of nodes (x) and stress points (●) in PLAXIS Model

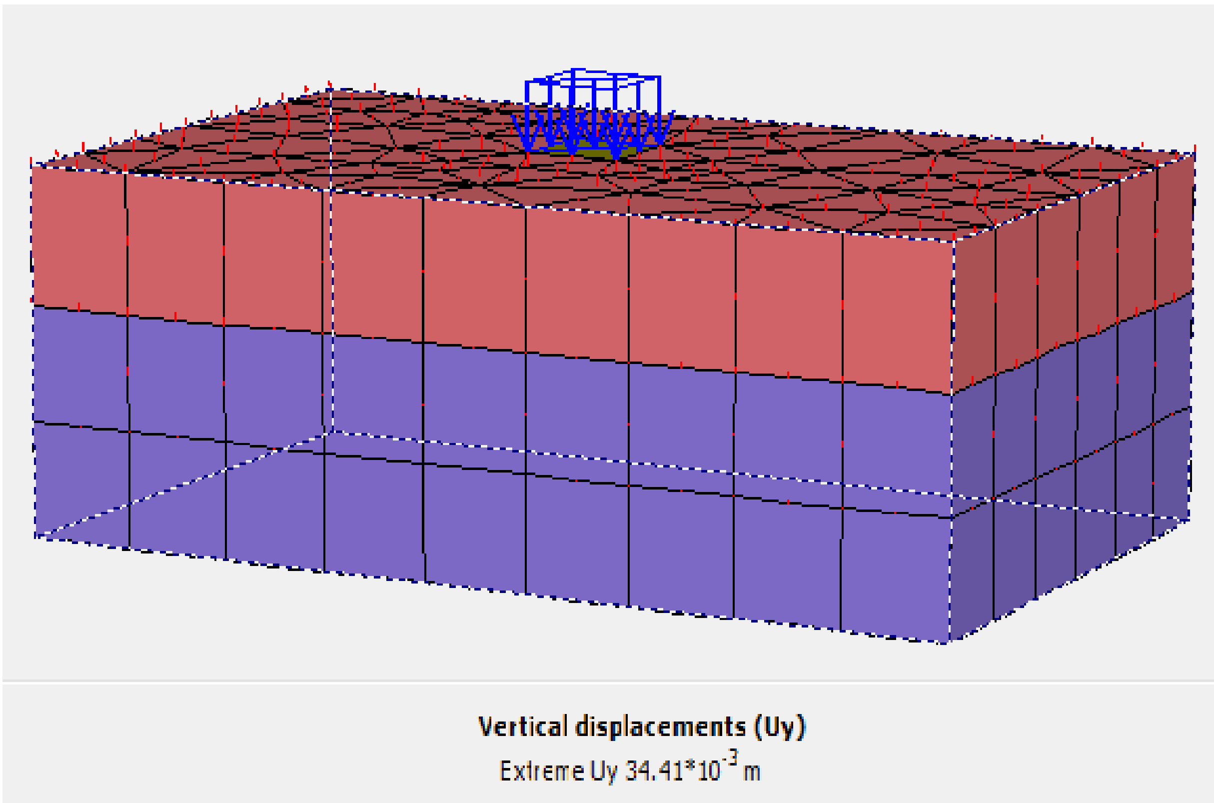


Figure - 3.11: Settlement of a model in PLAXIS Model

Performing staged construction by increasing load with each step increment until failure and extracting settlement contour at the founding level as presented above a pressure-settlement curve can be attained to ultimately evaluate the bearing capacity for that set of soil parameters and foundation system. Such pressure-settlement curve and its tabular presentation are given below.

Table 3.5: Pressure settlement values

load (KN/m^2	Settlement
10	1.41E-03
50	2.60E-03
100	5.12E-03
150	1.06E-02
200	2.16E-02
250	4.39E-02
300	9.48E-02

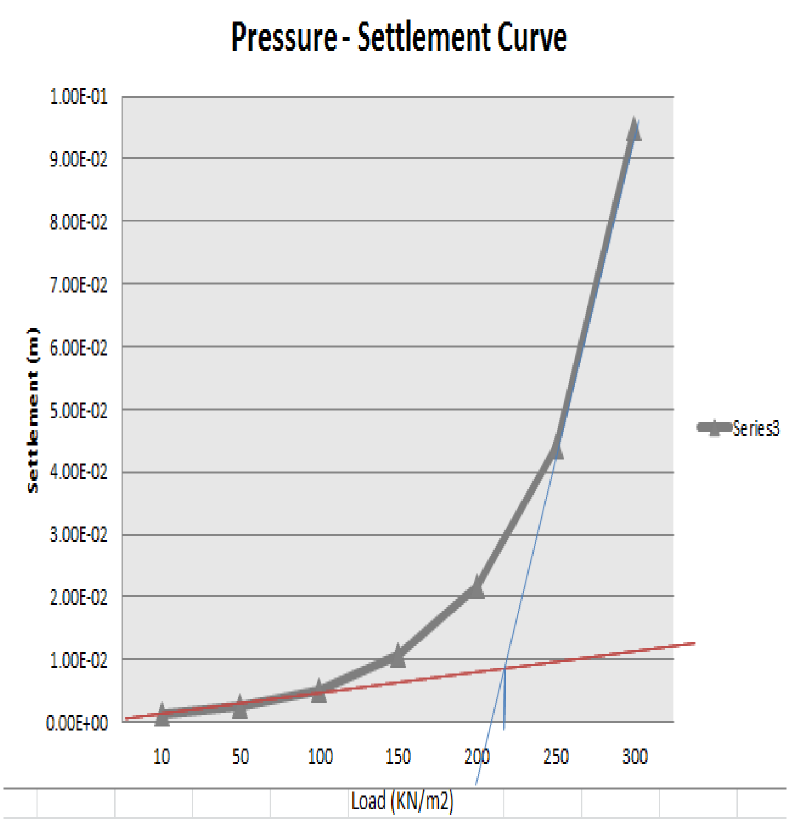


Figure 3.12: Pressure – settlement plot

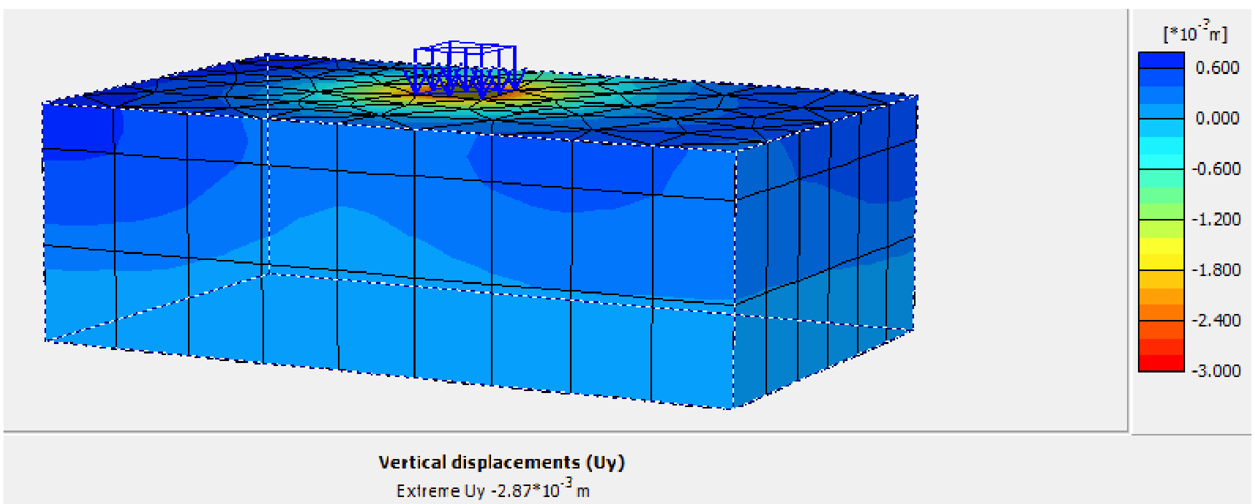
3.5.1 Presentation of results

To establish correlation between analytical results with various classical bearing capacity theories and to carry out a parametric study to understand the influence of different parameters like footing width, thickness of different layers etc. on ultimate bearing capacity of shallow footings total 37 finite element modeling has been carried out in PLAXIS 3D Foundation:

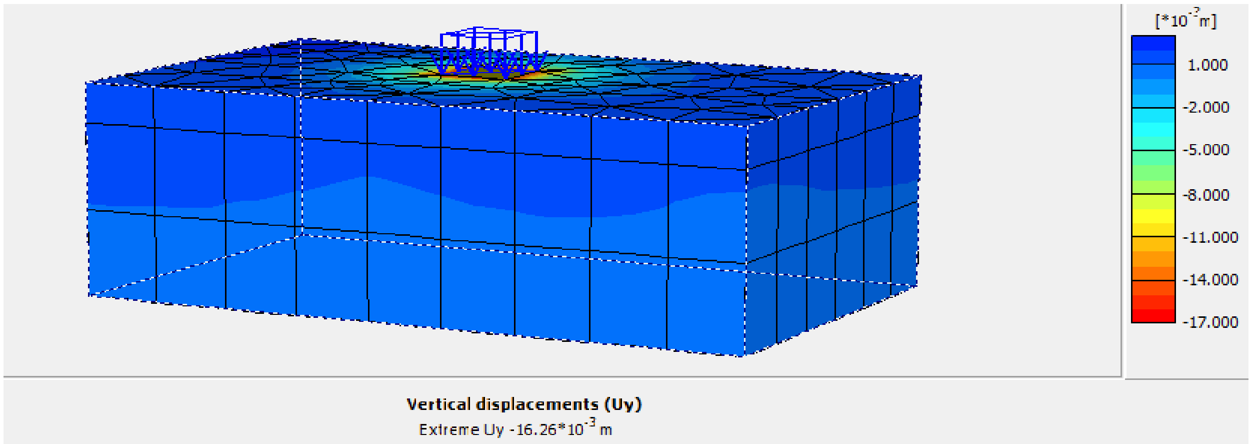
The foundation base model is isolated footing with different layers with dimensions of width 2.5m and depth of 1.5m. Different types of soil layer exists below the natural ground is stratified in two, three and four layered soils.

Model No-1:

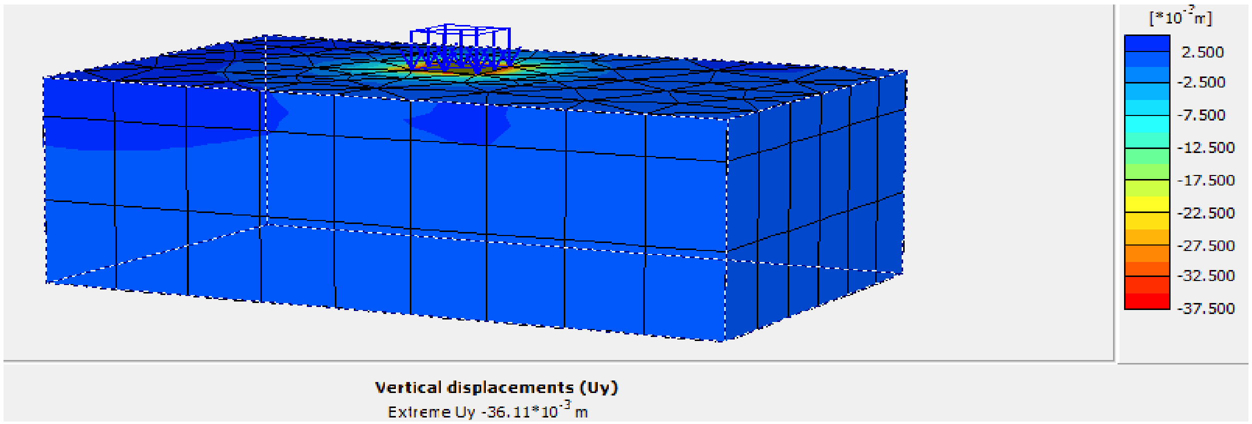
The following pictures are obtained from Plaxis output represent settlement values for each loading.



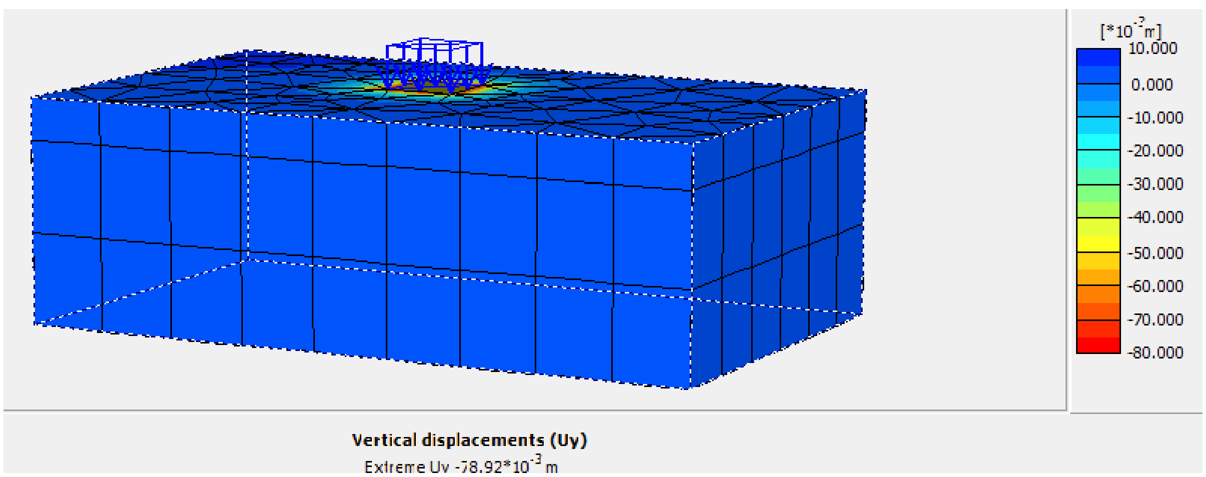
Model 1



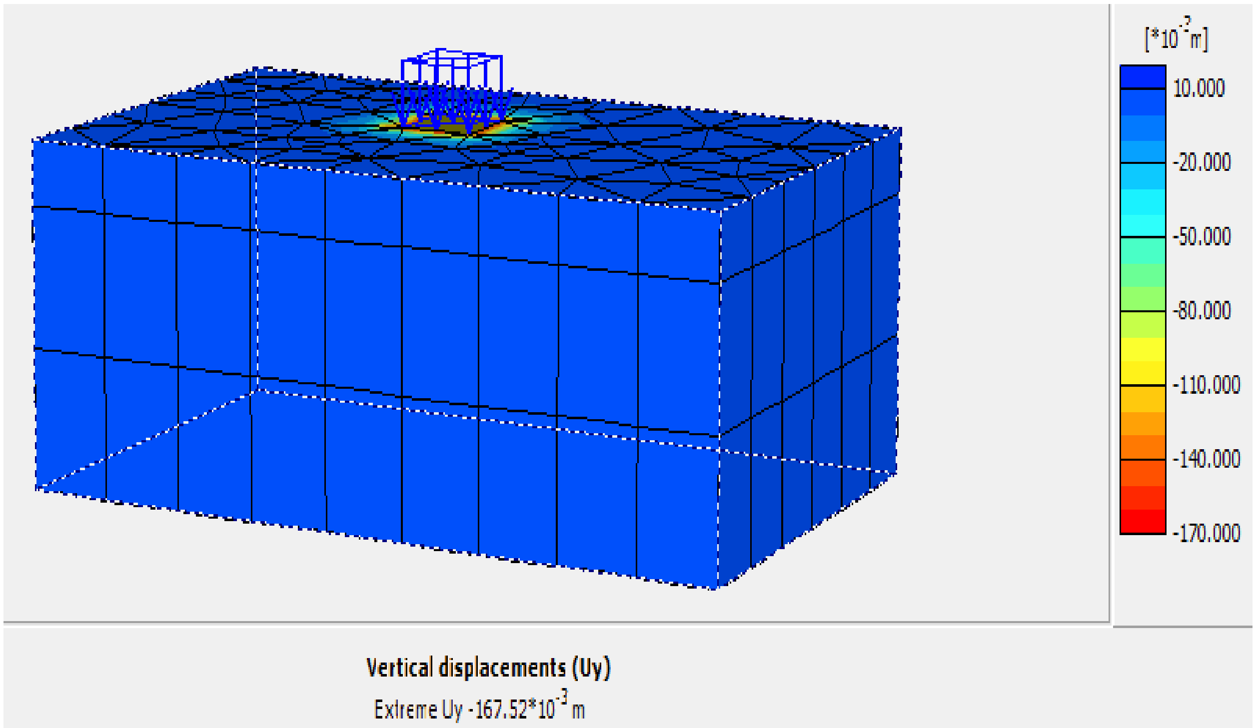
Model 2



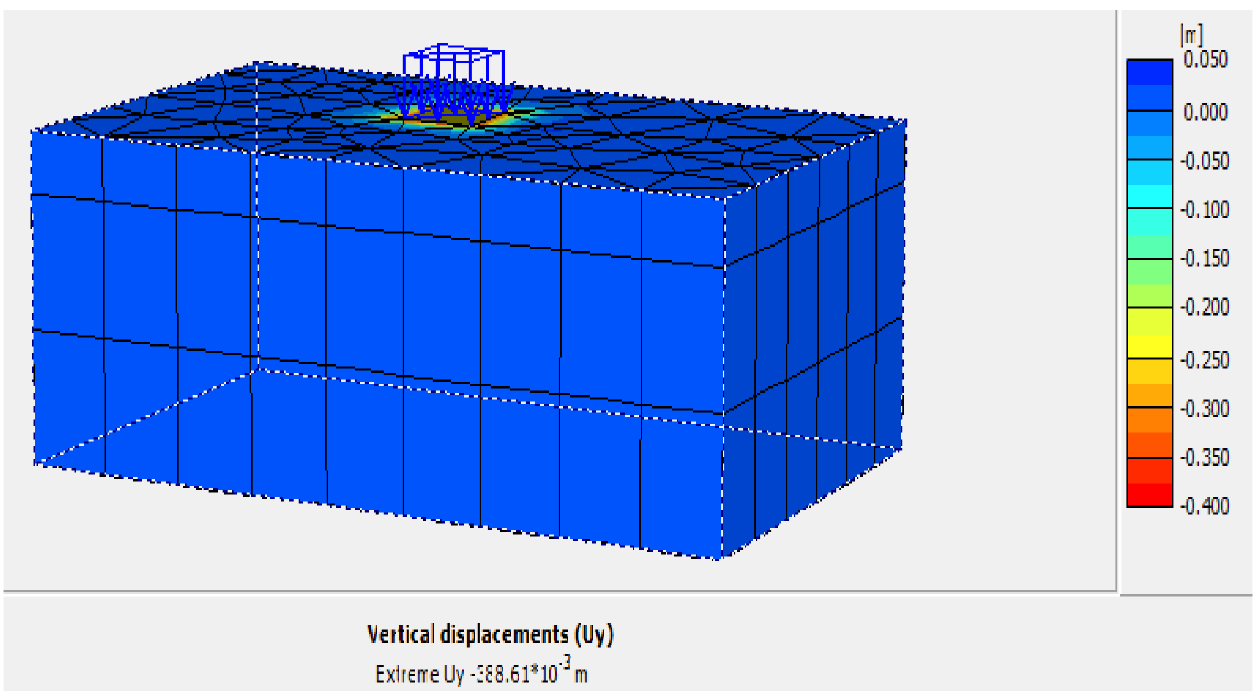
Model 3



Model 4

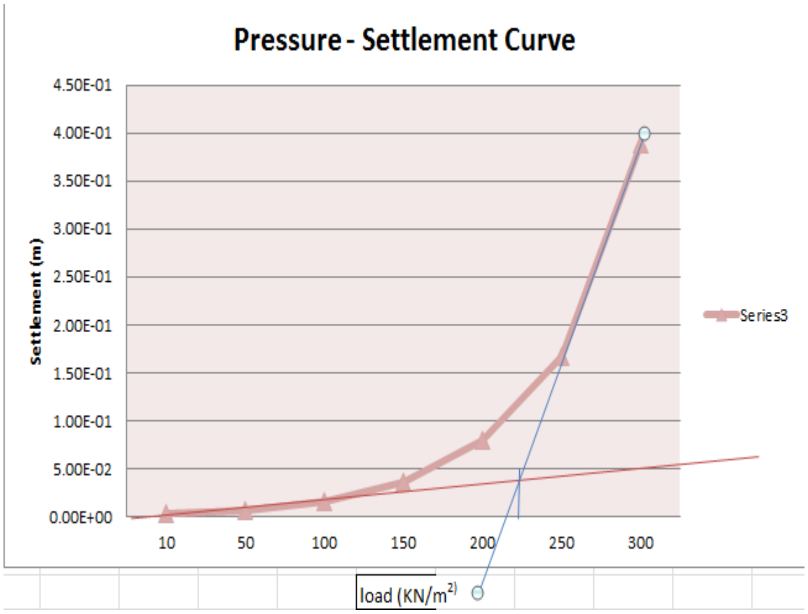


Model 5



Model 6

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



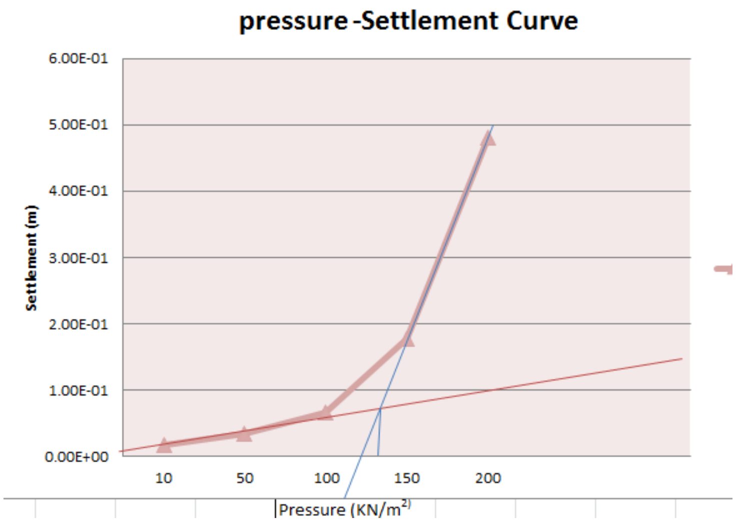
pressure (KN/m$^{2)}$	Settlement(m)
10	2.87E-03
50	6.75E-03
100	1.63E-02
150	3.61E-02
200	7.89E-02
250	1.68E-01
300	3.89E-01

L/B=1, h/B=0.6, Bearing Capacity = 223 KN/m^2

○ The graph shows the relationship between settlement and load for dense gravel over soft clay and as it is clearly shown it is increasing in an exponential scale. The tangent line shows both steep and mild state.

Model –2:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

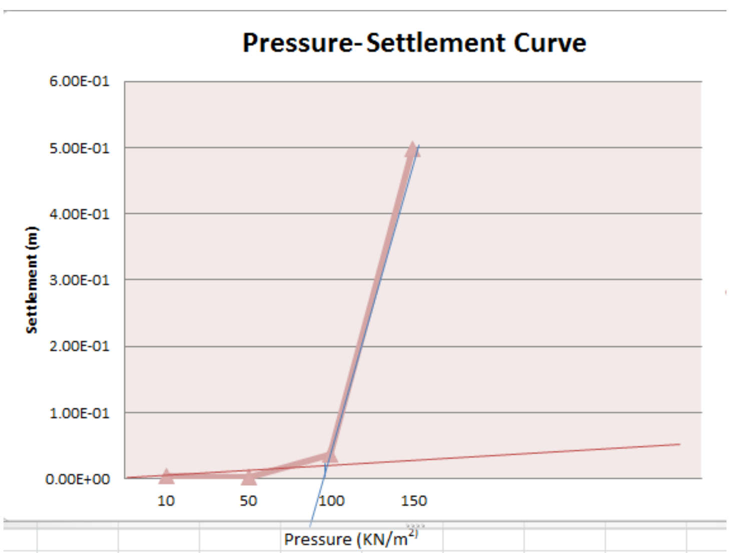


Pressure ($KN/m^{2)}$)	Settlement(m)
10	1.63E-02
50	3.50E-02
100	6.53E-02
150	1.78E-01
200	4.80E-01

L/B=1, h/B=0.6, Bearing Capacity = 135 KN/m^2

Model – 3:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

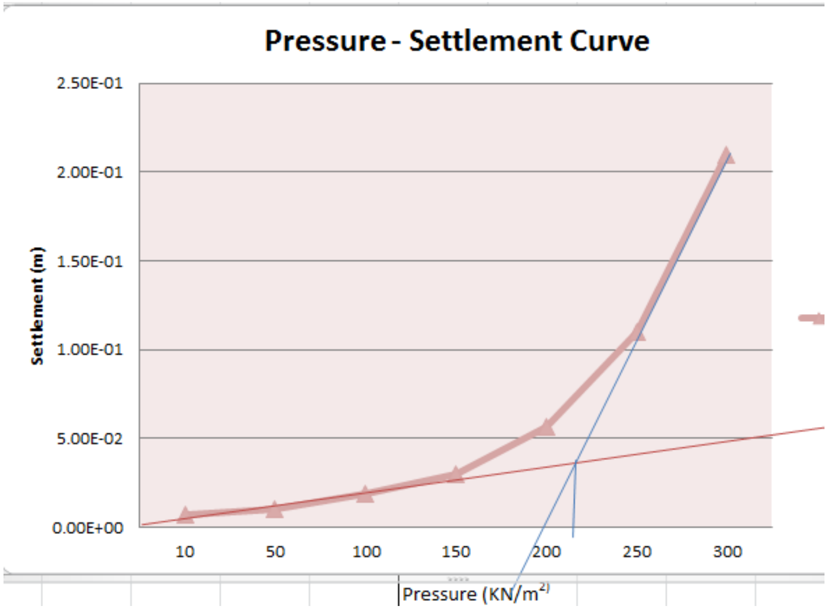


Pressure ($KN/m^{2)}$)	Settlement(m)
10	4.10E-03
50	2.50E-03
100	3.69E-02
150	4.98E-01

L/B=1, h/B=0.6, Bearing Capacity = 100 KN/m^2

Model –4:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

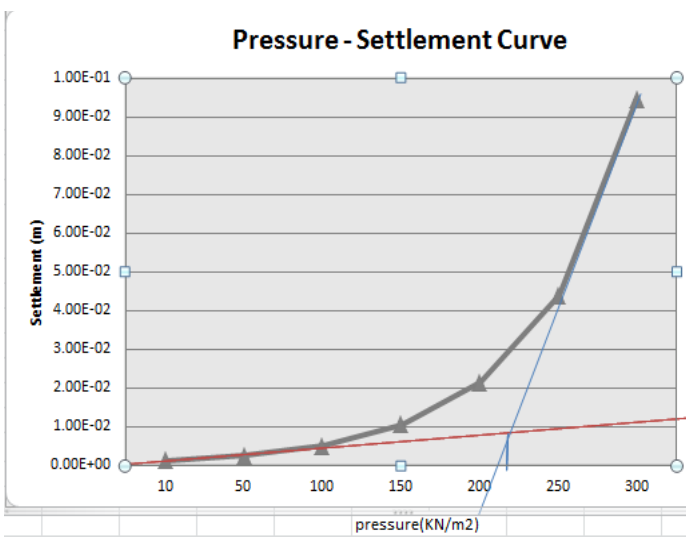


Pressure (KN/m$^{2)}$	Settlement(m)
10	7.01E-03
50	1.00E-02
100	1.86E-02
150	2.96E-02
200	5.68E-02
250	1.10E-01
300	2.10E-01
350	4.03E-01

L/B=1, h/B=0.6, Bearing Capacity = 210 KN/m^2

Model –5:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

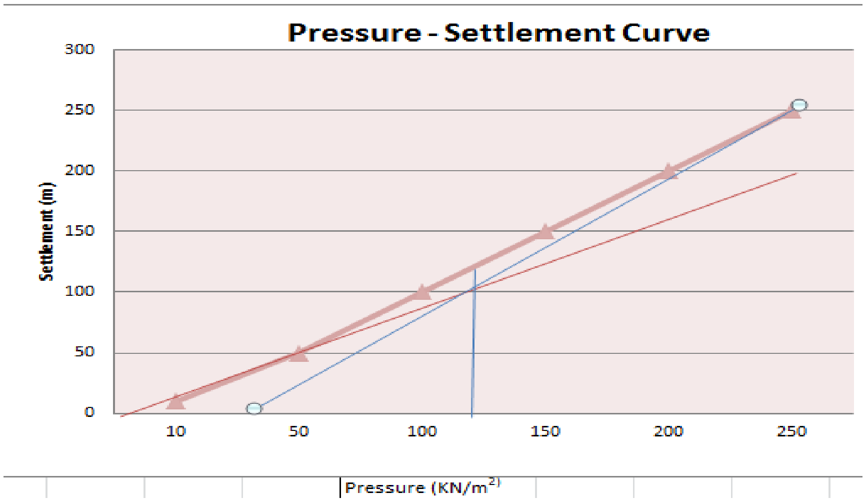


Pressure (KN/m$^{2)}$	Settlement(m)
10	1.41E-03
50	2.60E-03
100	5.12E-03
150	1.06E-02
200	2.16E-02
250	4.39E-02
300	9.48E-02

L/B=1, h/B=1.2, Bearing Capacity = 227 KN/m^2

Model –6:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



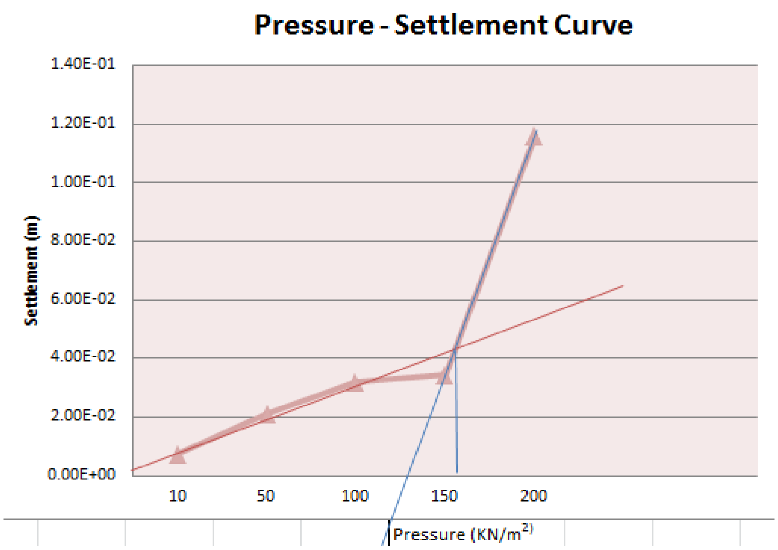
Pressure (KN/m$^{2)}$	Settlement(m)
10	2.01E-02
50	3.00E-02
100	4.08E-02
150	6.01E-02
200	1.27E-01
250	2.89E-01

L/B=1, h/B=1.2, Bearing Capacity = 152 KN/m^2

○ The graph shows the relationship between settlement and load for dense sand over lain soft saturated clay with h/b=1.2 and as it is clearly shown it is increasing in linear scale.

Model –7:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



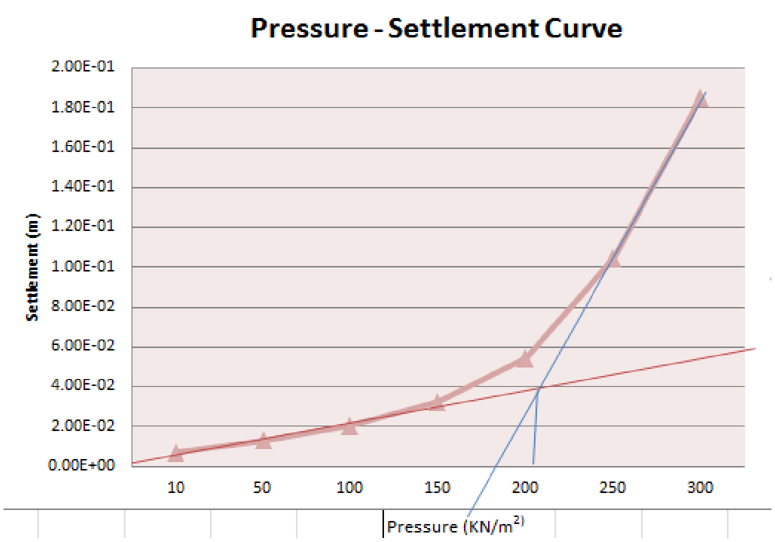
Pressure (KN/m$^{2)}$	Settlement(m)
10	7.28E-03
50	2.10E-02
100	3.20E-02
150	3.44E-02
200	1.16E-01

L/B=1, h/B=1.2, Bearing Capacity = 157 KN/m^2

○ The graph shows the relationship between settlement and load for dense sand over loose sand with h/b=1.2 and as it is clearly shown it is increasing in linear scale.

Model –8:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

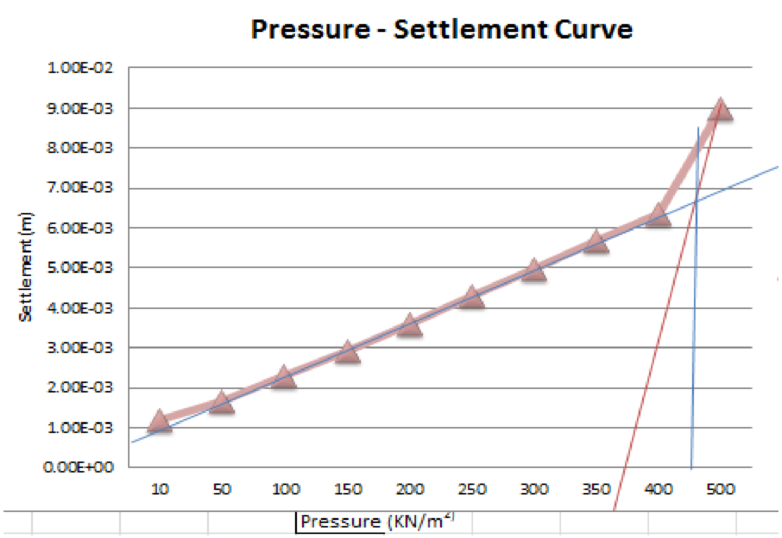


Pressure (KN/m$^{2)}$	Settlement(m)
10	7.02E-03
50	1.29E-02
100	2.05E-02
150	3.26E-02
200	5.45E-02
250	1.05E-01
300	1.85E-01

L/B=1, h/B=1.2, Bearing Capacity = 220 KN/m^2

Model –9:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



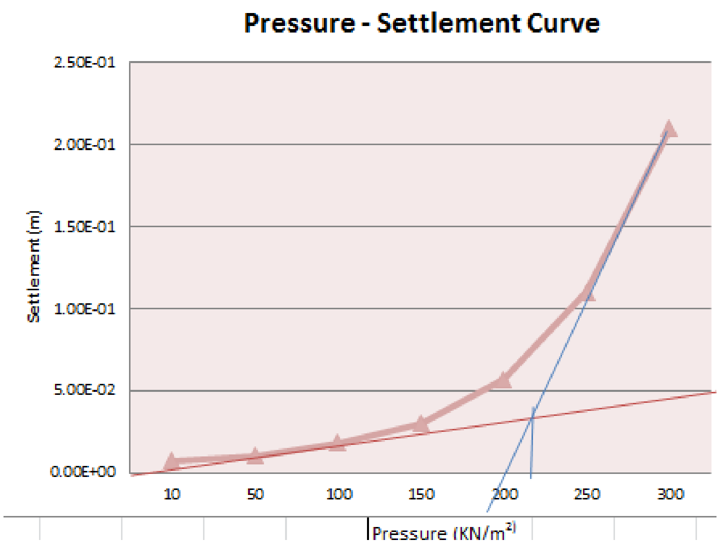
Pressure (KN/m$^{2)}$	Settlement(m)
10	1.18E-03
50	1.67E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03
350	5.70E-03
400	6.35E-03
500	9.00E-03

L/B=1, h/B=1.2, Bearing Capacity = 480 KN/m^2

○ The graph shows the relationship between settlement and load for dense gravel over stiff saturated clay with h/b=1.2 and as it is clearly shown it is increasing in linear scale.

Model –10:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



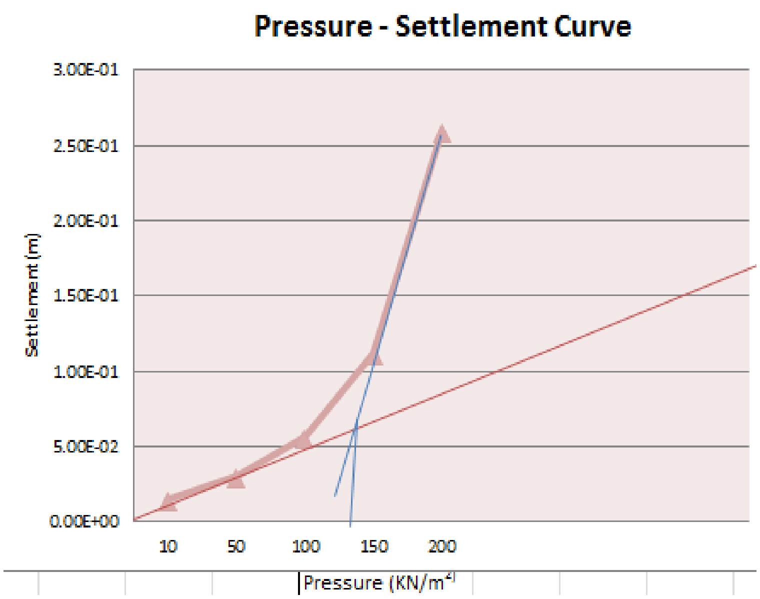
Pressure (KN/m$^{2)}$	Settlement(m)
10	7.01E-03
50	1.00E-02
100	1.80E-02
150	3.00E-02
200	5.68E-02
250	1.10E-01
300	2.10E-01
350	4.54E-01

L/B=1, h/B=1.2, Bearing Capacity = 220 KN/m^2

Model –11:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

Pressure (KN/m$^{2)}$	Settlement(m)
10	1.50E-02
50	3.00E-02
100	5.60E-02
150	1.12E-01
200	2.59E-01

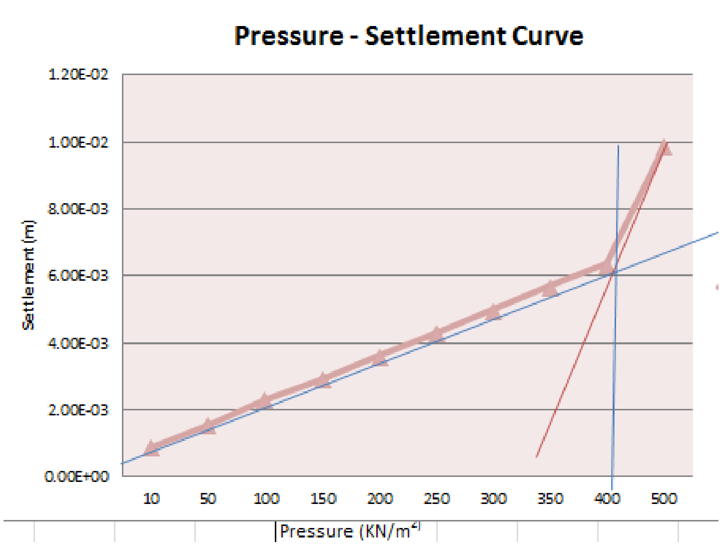


L/B=1, h/B=1.2, Bearing Capacity = 140 KN/m^2

○ The graph shows the relationship between settlement and load for soft saturated clay over silty clay with h/b=1.2 and as it is clearly shown it is increasing in linear scale. The tangent line shows a steep line.

Model –12:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



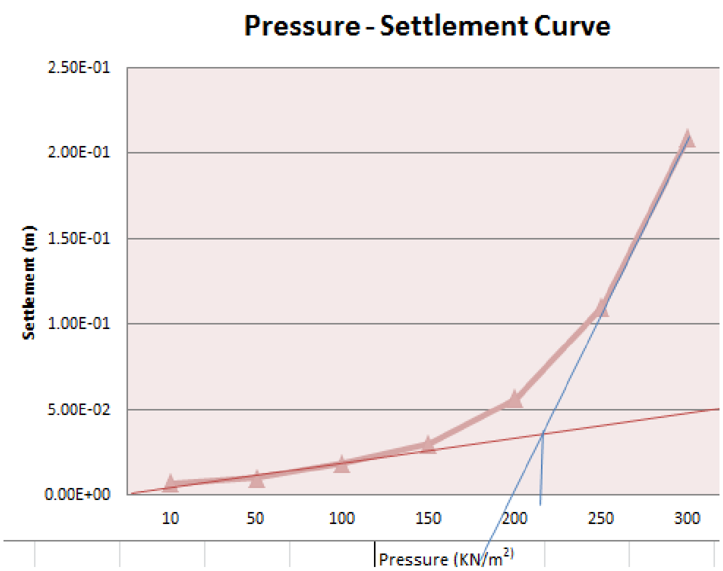
Pressure (KN/m^2)	Settlement(m)
10	9.00E-04
50	1.57E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03
350	5.70E-03
400	6.35E-03
500	9.90E-03

L/B=1, h/B=1.2, Bearing Capacity = 405 KN/m^2

○ The graph shows the relationship between settlement and load for dense gravel over stiff clay with h/b=1.2 and as it is clearly shown it is increasing in linear scale.

Model –13:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



Pressure (KN/m$^{2)}$	Settlement(m)
10	6.90E-03
50	1.00E-02
100	1.90E-02
150	3.00E-02
200	5.68E-02
250	1.10E-01
300	2.09E-01
350	3.99E-01

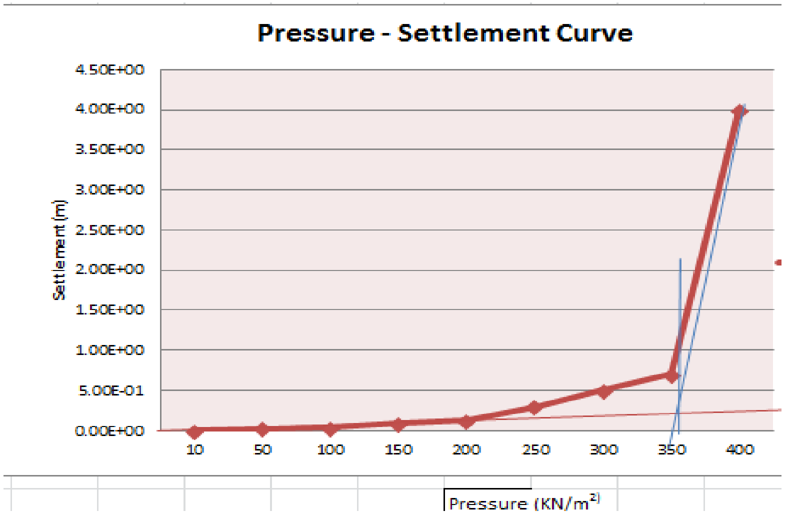
L/B=1, h/B=1.2, Bearing Capacity = 355 KN/m^2

Model –14:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

Pressure (KN/m$^{2)}$	Settlement(m)
10	5.02E-03
50	2.92E-02
100	4.00E-02
150	9.00E-02
200	1.30E-01
250	3.00E-01
300	5.00E-01
350	7.00E-01

400	4.00E+00

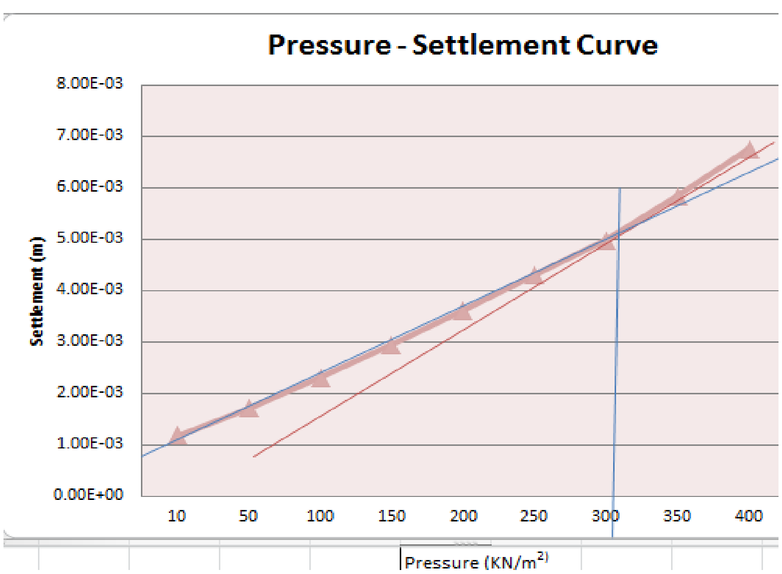


L/B=1, h/B=1.2, Bearing Capacity = 355 KN/m^2

○ The graph shows the relationship between settlement and load for soft clay over dense gravel with h/b=1.2 and the graph shows mild slope up to 350KN/m2 and after that it turns to steep slope.

<u>Model –15:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



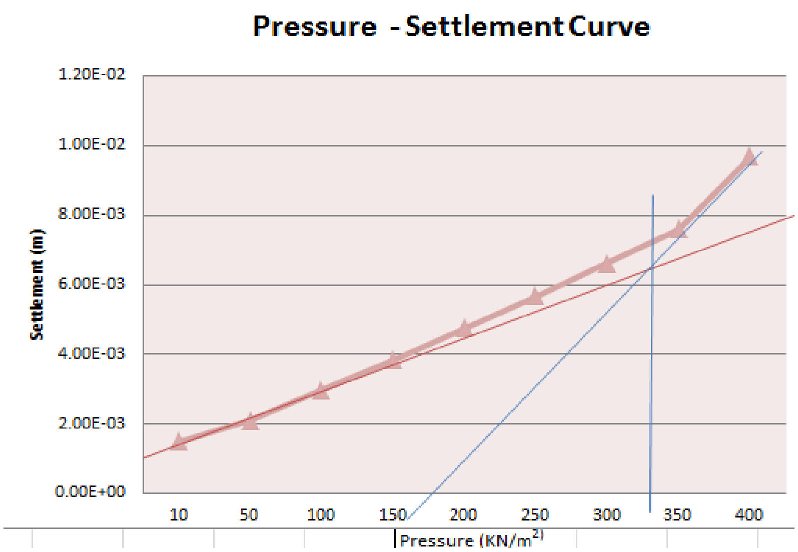
Pressure (KN/m^{2})	Settlement(m)
10	1.18E-03
50	1.71E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03
350	5.83E-03
400	6.75E-03

L/B=1, h/B=1.2, Bearing Capacity = 305 KN/m^2

○ The graph shows the relationship between settlement and load for saturated soft clay over dense sand with h/b=1.2 and as it is clearly shown it is increasing in linear scale

Model –16:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



Pressure (KN/m$^{2)}$	Settlement(m)
10	1.50E-03
50	2.10E-03
100	2.99E-03
150	3.85E-03
200	4.76E-03
250	5.68E-03
300	6.62E-03
350	7.61E-03
400	9.69E-03

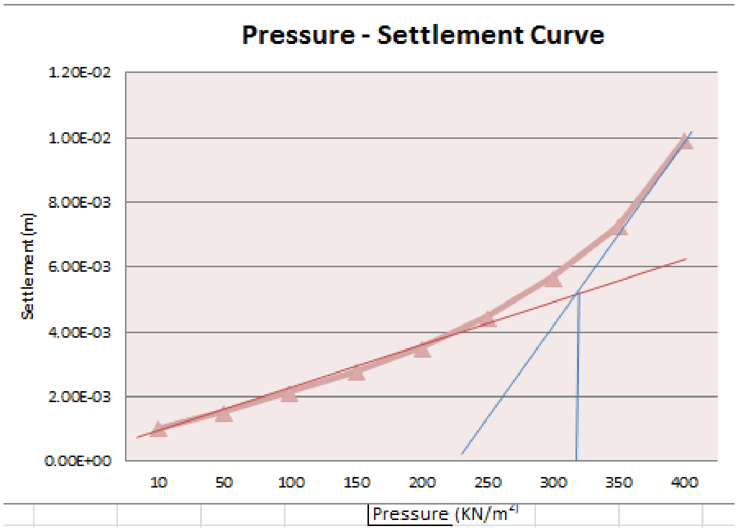
L/B=1, h/B=1.2, Bearing Capacity = 342 KN/m^2

○ The graph shows the relationship between settlement and load for loose sand over dense sand with h/b=1.2 and as it is clearly shown it is increasing in linear scale

Model –17:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

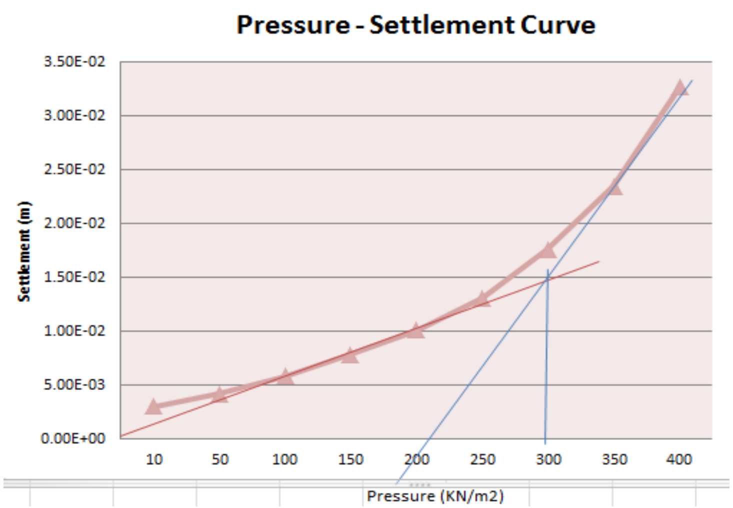
Pressure (KN/m$^{2)}$	Settlement(m)
10	1.04E-03
50	1.53E-03
100	2.15E-03
150	2.80E-03
200	3.53E-03
250	4.47E-03
300	5.69E-03
350	7.29E-03
400	9.96E-03



L/B=1, h/B=1.6, Bearing Capacity = 320 KN/m^2

<u>Model –18:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



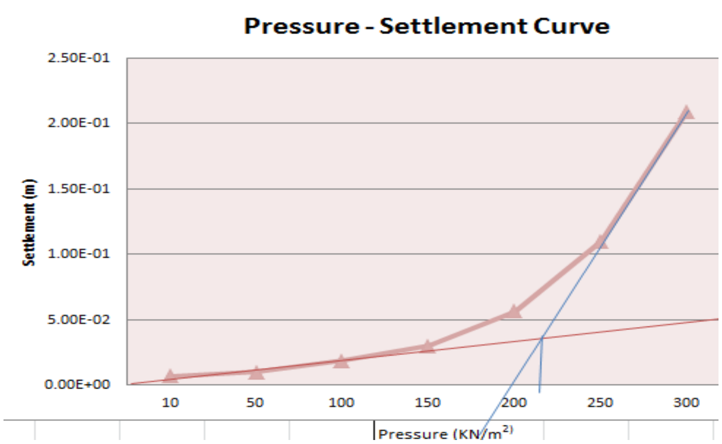
Pressure (KN/m^2)	Settlement(m)
10	3.04E-03
50	4.20E-03
100	5.88E-03
150	7.84E-03
200	1.01E-02
250	1.31E-02
300	1.76E-02
350	2.35E-02
400	3.27E-02

L/B=1, h/B=1.6, Bearing Capacity = 300 KN/m^2

<u>Model –19:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

Pressure (KN/m^2)	Settlement(m)

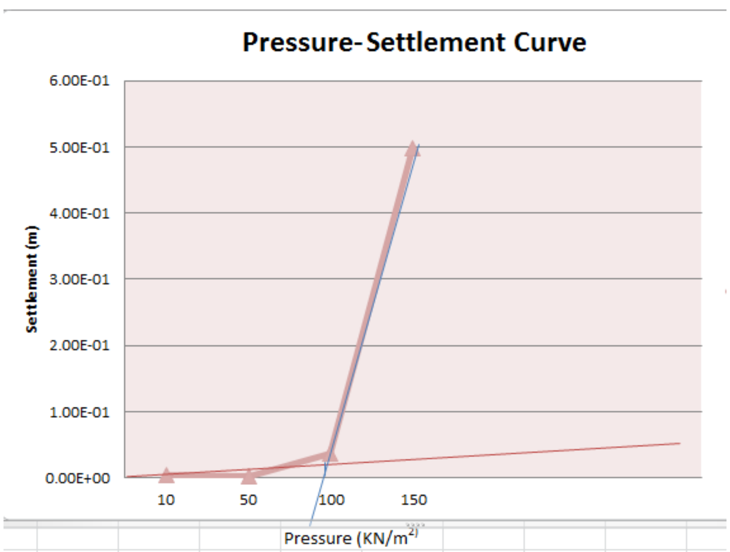


10	6.90E-03
50	1.00E-02
100	1.90E-02
150	3.00E-02
200	5.68E-02
250	1.10E-01
300	2.09E-01
350	3.99E-01

L/B=1, h/B=1.6, Bearing Capacity = 225KN/m^2

<u>Model –20:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



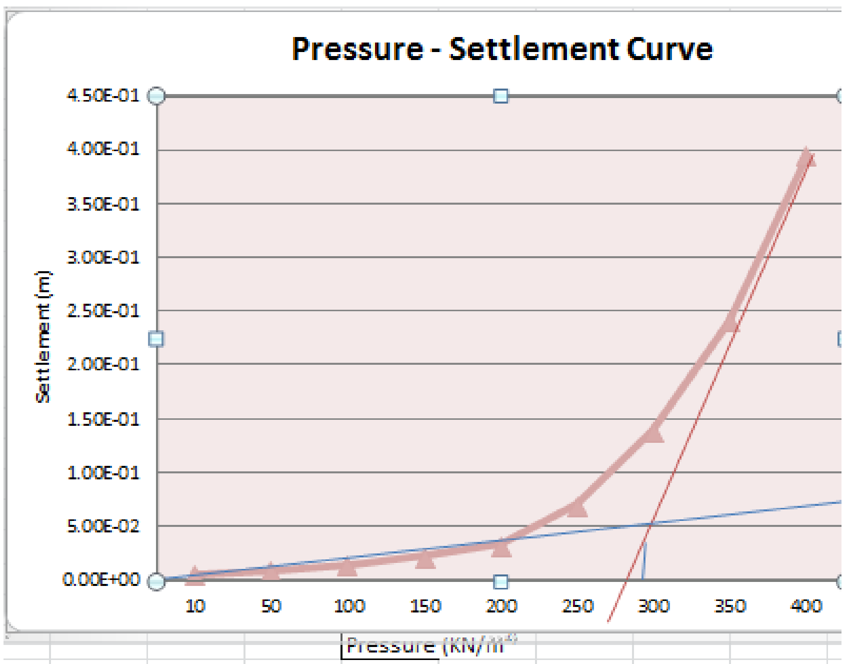
Pressure (KN/m^2)	Settlement(m)
10	8.29E-03
50	1.27E-02
100	1.82E-02
150	2.46E-02
200	3.79E-02
250	6.23E-02
300	1.03E-01
350	1.81E-01
400	3.23E-01

L/B=1, h/B=1.6, Bearing Capacity = 92KN/m^2

○ The graph shows the relationship between settlement and load for stiff saturated clay over soft clay with h/b=1.6 and as it is clearly shown it is increasing in linear scale and the slope is mild up to 100KN/m2 and after that it turns to mild state.

Model –21:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



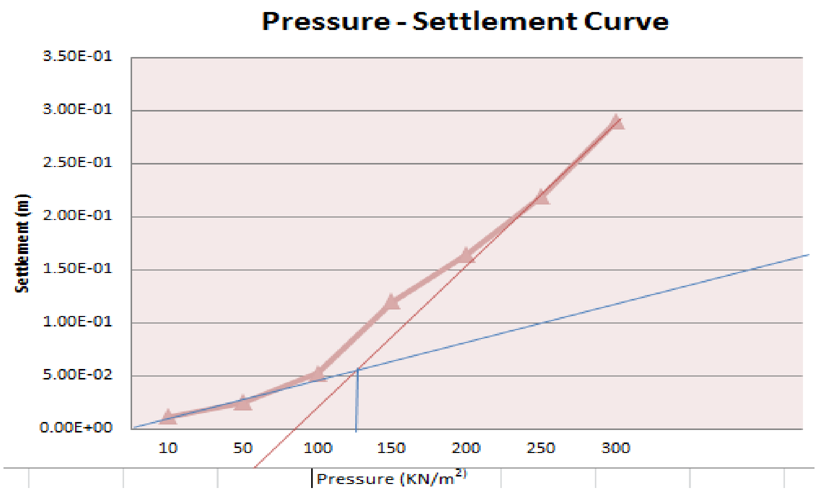
Pressure (KN/m$^{2)}$	Settlement(m)
10	5.16E-03
50	8.96E-03
100	1.47E-02
150	2.23E-02
200	3.34E-02
250	7.05E-02
300	1.39E-01
350	2.41E-01
400	3.95E-01

L/B=1, h/B=0.6, Bearing Capacity = 290KN/m^2

Model –22:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

Pressure (KN/m$^{2)}$	Settlement(m)
10	1.23E-02
50	2.56E-02
100	5.32E-02
150	1.21E-01
200	1.65E-01
250	2.20E-01
300	2.90E-01

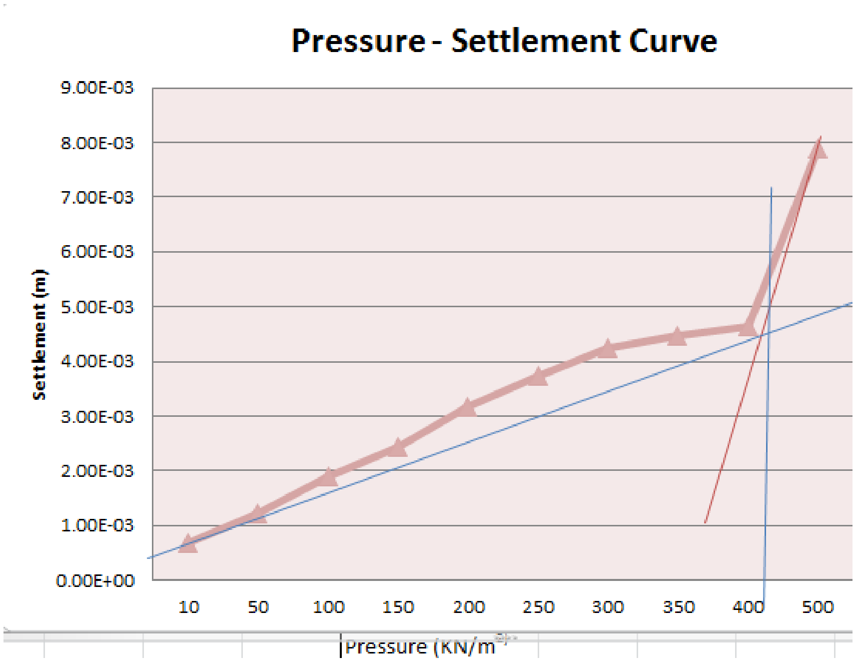


L/B=1, h/B=0.6, Bearing Capacity = 140KN/m^2

○ The graph shows the relationship between settlement and load for soft saturated clay, stiff clay and dense sand with h/b=0.6 and as it is clearly shown it is increasing in linear scale

Model –23:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

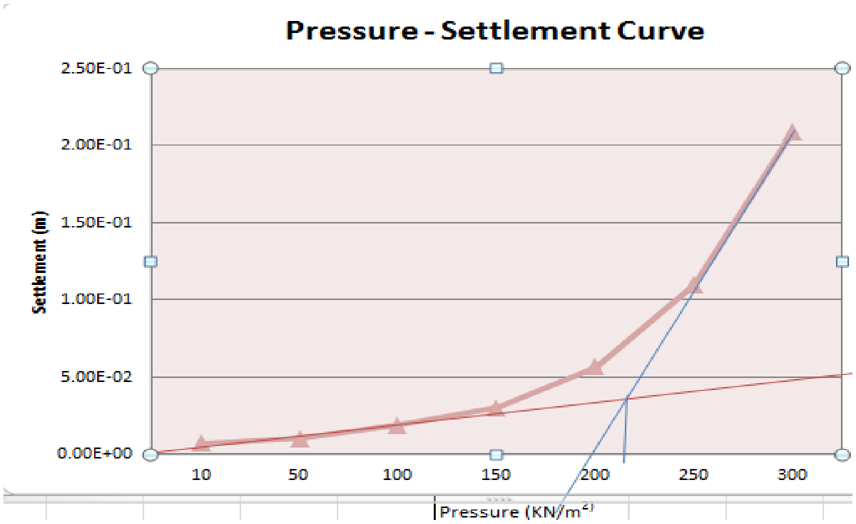


Pressure (KN/m$^{2)}$	Settlement(m)
10	6.90E-04
50	1.21E-03
100	1.91E-03
150	2.44E-03
200	3.16E-03
250	3.73E-03
300	4.24E-03
350	4.47E-03
400	4.63E-03
500	7.89E-03

L/B=1, h/B=1.2, Bearing Capacity = 408KN/m^2

Model –24:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



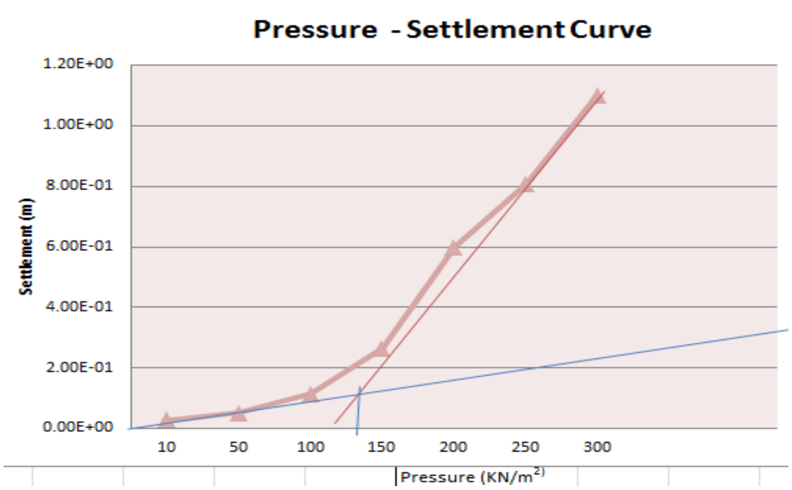
Pressure (KN/m$^{2)}$	Settlement(m)
10	6.90E-03
50	1.00E-02
100	1.90E-02
150	3.00E-02
200	5.68E-02
250	1.10E-01
300	2.09E-01
350	3.99E-01

L/B=1, h/B=1.2, Bearing Capacity =113 KN/m^2

Model –25:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

Pressure (KN/m$^{2)}$	Settlement(m)
10	2.89E-02
50	5.23E-02
100	1.16E-01
150	2.64E-01
200	6.00E-01
250	8.09E-01
300	1.10E+00



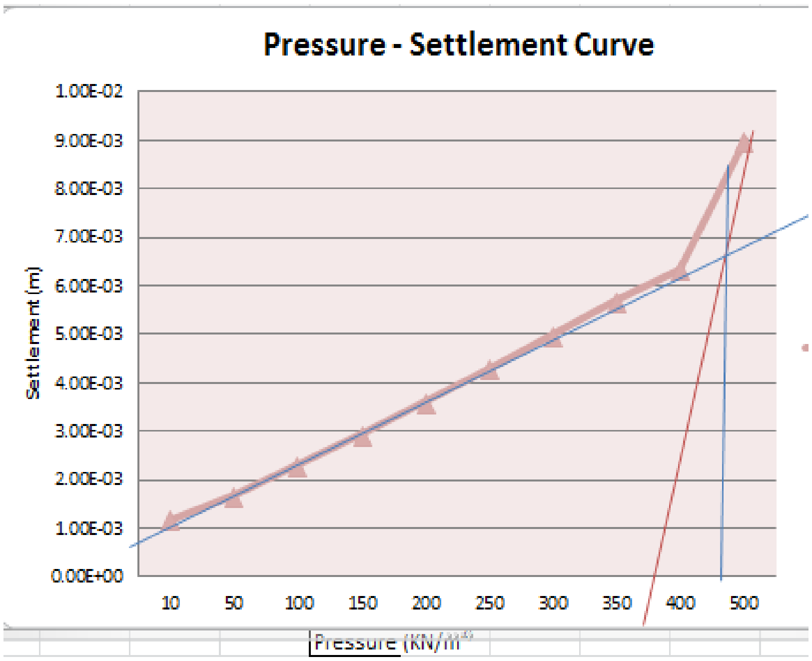
Pressure (KN/m$^{2)}$	Settlement(m)

L/B=1, h/B=0.6, Bearing Capacity = 135KN/m^2

Model –26:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

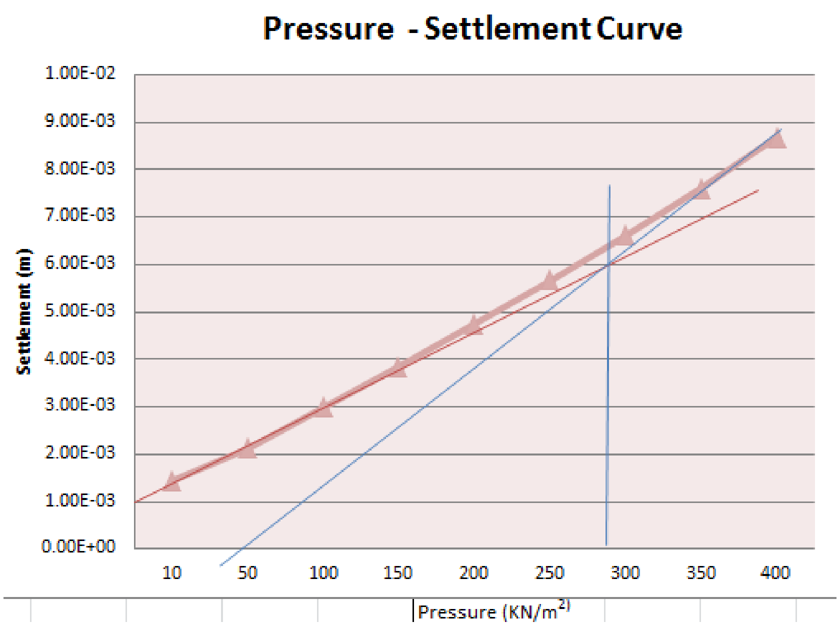
10	1.18E-03
50	1.67E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03
350	5.70E-03
400	6.35E-03
500	9.00E-03



L/B=1, h/B=0.6, Bearing Capacity =471 KN/m^2

<u>Model –27:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling

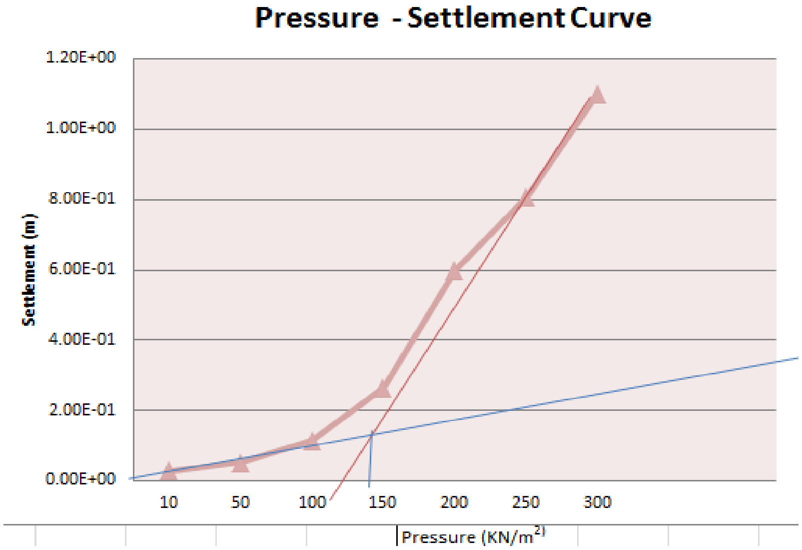


Pressure (KN/m$^{2)}$	Settlement(m)
10	1.44E-03
50	2.13E-03
100	2.99E-03
150	3.85E-03
200	4.76E-03
250	5.68E-03
300	6.62E-03
350	7.61E-03
400	8.69E-03

L/B=1, h/B=0.6, Bearing Capacity = 292KN/m^2

Model –28:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



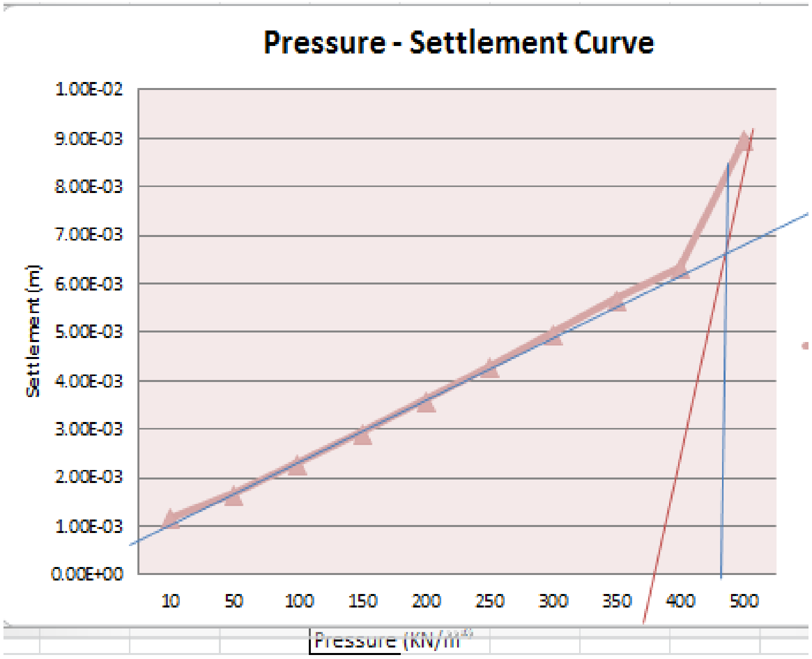
Pressure (KN/m$^{2)}$	Settlement(m)
10	2.89E-02
50	5.23E-02
100	1.16E-01
150	2.64E-01
200	6.00E-01
250	8.09E-01
300	1.10E+00

Pressure (KN/m$^{2)}$	Settlement(m)
10	1.18E-03
50	1.67E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03

L/B=1, h/B=2, Bearing Capacity = 110KN/m^2

Model –29:

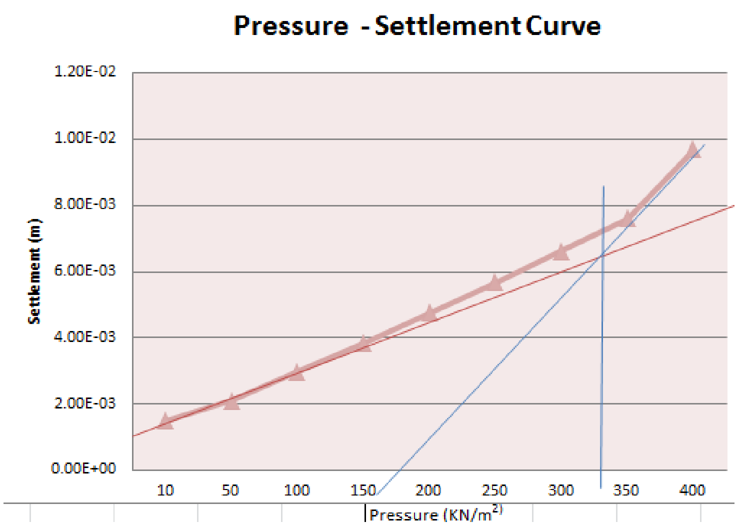
Pressure– Settlement Plot from PLAXIS Finite Element Modeling:



350	5.70E-03
400	6.35E-03
500	9.00E-03

L/B=1, h/B=0.4, Bearing Capacity = 470KN/m^2

<u>Model –30:</u> Pressure– Settlement Plot from PLAXIS Finite Element Modeling:

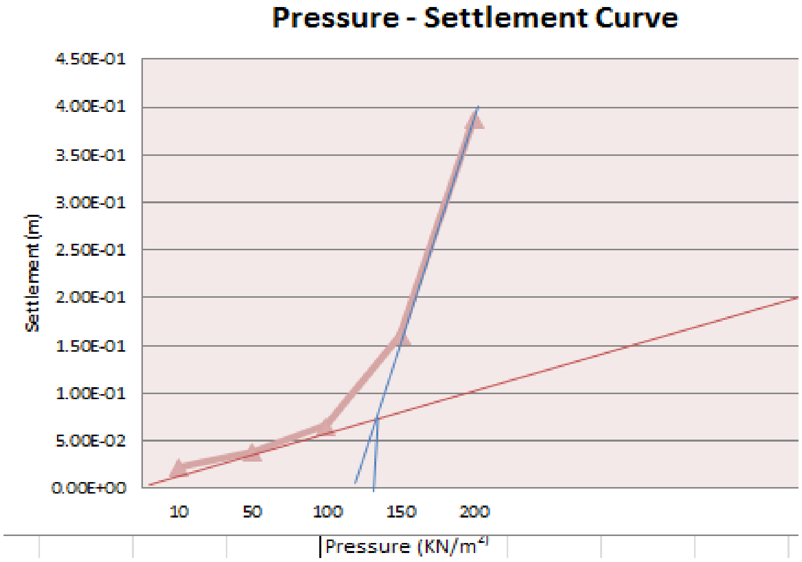


Pressure (KN/m^2)	Settlement(m)
10	5.02E-03
50	2.92E-02
100	4.00E-02
150	9.00E-02
200	1.30E-01
250	3.00E-01
300	5.00E-01
350	7.00E-01
400	4.00E+00

L/B=1, h/B=0.4, Bearing Capacity = 342KN/m^2

<u>Model –31:</u>

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:

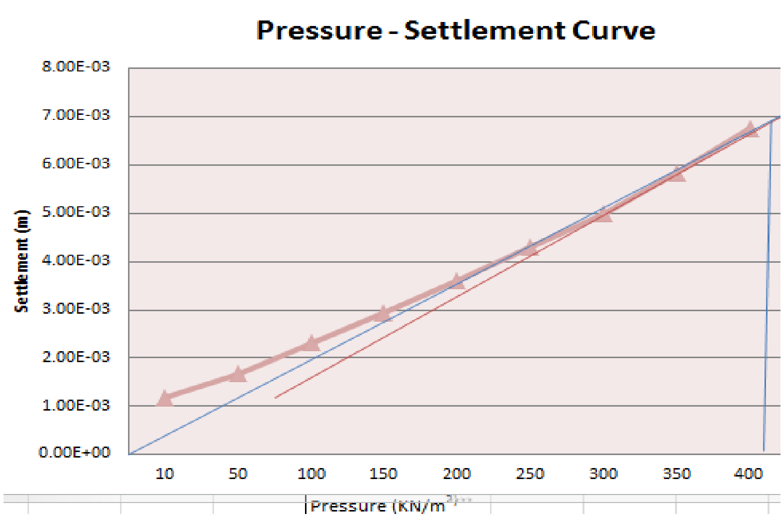


Pressure (KN/m²)	Settlement(m)
10	2.33E-02
50	3.93E-02
100	6.70E-02
150	1.62E-01
200	3.88E-01

L/B=1, h/B=1.6, Bearing Capacity = 135KN/m^2

Model –32:

Pressure – Settlement Plot from PLAXIS Finite Element Modeling:



Pressure (KN/m²)	Settlement(m)
10	1.18E-03
50	1.67E-03
100	2.31E-03
150	2.94E-03
200	3.61E-03
250	4.30E-03
300	4.98E-03
350	5.83E-03
400	6.75E-03

L/B=1, h/B=1.6, Bearing Capacity = 410KN/m^2

3.5. Empirical method of calculating bearing capacity

A total of 33 hand calculation has been done for comparison purpose. Hand calculation has been done in modified Hanna and Meyerhof method. [21] Sample calculations for two layered, three layered and four layered are presented below.

i. For two layered soil

- For dense gravel underlain soft clay

$$Q_u = q_b + \frac{2C_1H}{B} + \gamma_1 H^2 \quad 1 + \frac{2D}{H} \quad \frac{K_s \tan\varphi}{B} - \gamma_1 H$$

q_b = 100 for soft clay

For dense gravel:-

$$\varphi = 40, Nc = 75 \; and \; N\gamma = 53$$

For Soft clay:-

$$\varphi = 6, Nc = 6.9 \; and \; N\gamma = 0.1$$

$$q1 = C1Nc1 + \frac{1}{2}\gamma 1BN\gamma 1$$

q1= 1700

$$q2 = C2Nc2 + \frac{1}{2}\gamma 2BN\gamma 2$$

q2= 105.875

q2/q1 = 0.1

From the table for $\varphi 1 = 40, \frac{q2}{q1} = 0.1, \quad Ks = 3$

therefore,

Qu= 164KN/m2

ii. for three layered soil

soft saturated clay, stiff saturated clay, underlain dense gravel (1.5m, 2m,4.5m)

$$Q_u = q_b + 2\sum_{i=1}^{n} \frac{C_iH_i}{B} + \sum_{i=1}^{n-1} H_i(\gamma_i H_i + 2\sum_{j=1}^{i-1} H_j\gamma_j + 2D)\frac{K_{si}\tan\varphi}{B} - \sum_{i=1}^{n-1}\gamma_i H_i$$

$$2\sum_{i=1}^{n}\frac{C_i H_i}{B} = 2\times \left(12.5\times\frac{1.5}{2.5} + 25\times\frac{2}{2.5} + 5\times\frac{4.5}{2.5}\right)$$

=73

$$2\sum_{j=1}^{i-1} H_j\gamma_j = 2\times\left(1.5\times 12.5 + 19.5\times 2\right) + 2\left(18.75 + 39\right)$$

=115.5

$$\sum_{i=1}^{n-1} H_i\left(\gamma_i H_i + 2\sum_{j=1}^{i-1} H_j\gamma_j + 2D\right)\frac{K_{si}\tan\varphi}{B} = 44.8$$

$= 44.8$

$\sum_{i=1}^{n-1}\gamma_i H_i = (17.69 * 1.5 + 1.5 * 19 + 22 * 10.5)$

=286.1

$$q1 = C1Nc1 + \frac{1}{2}\gamma 1BN\,\gamma 1$$

q1= 88.625

$$q2 = C2Nc2 + \frac{1}{2}\gamma 2BN\,\gamma 2$$

q2= 106

for ø=6, q1/q2=1.2 ks=0

$$q3 = C3Nc3 + \frac{1}{2}\gamma 3BN\,\gamma 3$$

q3= 1700

q3/q2=1.2 ks=0

qu=455.3 KN/m^2

iii. For four layered soil

dense gravel, stiff clay, silty clay, soft saturated clay

$$Q_u = q_b + 2\sum_{i=1}^{n}\frac{C_i H_i}{B} + \sum_{i=1}^{n-1} H_i(\gamma_i H_i + 2\sum_{j=1}^{i-1} H_j\gamma_j + 2D)\frac{K_{si}\tan\varphi}{B} - \sum_{i=1}^{n-1}\gamma_i H_i$$

$$2\sum_{i=1}^{n}\frac{C_i H_i}{B} = 2\times \quad 5\times\frac{1}{2.5} \ + \ 40\times\frac{1}{2.5} \ + \ 30\times\frac{2}{2.5} \ + (12.5\times\frac{4}{2.5})$$

=124

$$2\sum_{j=1}^{i-1} Hj\gamma j = 2\times \quad 1\times 20 \ + \ 17\times 1. \ + \ 18\times 2 \ + \ 12.5\times 4$$

=246

Ks1=4, Ks2=1,Ks3=1,Ks4=1

Q_u=q_b+124+400-246.

Qb=328.6kN/m^2

3.6 Comparision of Results

A comparision basing on FEM and Emperical method has been done. It is widely presented in the parametric studies .

Table 3.9: Parameters of Imposed load for the Models

Model No	h/B	bearing capacity		%age difference of plaxis with conventional method
		conventional method	Plaxis (FEM)	
1	0.6	164	223	26.4573991
2		113	135	16.2962963
3		112	100	-12
4		160	210	23.80952381
5	1.2	230	227	-1.321585903
6		197	152	-29.60526316
7		182	157	-15.92356688
8		152	200	24
9		474.8	480	1.083333333
10		200	220	9.090909091
11		142.5	140	-1.785714286
12		474.25	405	-17.09876543
13		280	355	21.12676056
14		223	210	-6.19047619
15		292	305	4.262295082
16		382	342	-11.69590643
17	1.6	360	320	-12.5
18		326	300	-8.666666667
19		197	225	12.44444444
20		58.86	92	36.02173913
21	0.6	310	290	-6.896551724
22		212	140	-51.42857143
23	1.2	392	408	3.921568627
24		190	113	-68.14159292
25		254.25	135	-88.33333333
26		519.36	471	-10.26751592
27	2	420	292	-43.83561644
28		165	110	-50
29	0.4	486.6	470	-3.531914894
30		328.6	342	3.918128655
31	1.6	152	135	-12.59259259
32		449	410	-9.512195122

The %age difference value shows that the percentage increase in Plaxis with that of conventional theories and the negative values shows that increase in conventional method than that of the Plaxis. The comparison result shows a reasonable values.

Chapter Four

4. Comparative Studies

Different cases have been considered in the parametric studies. The parametric study consists of four things. The study has been done for different types of soils and number of layers based on a given loading condition.

4.1. The foundation model

- Isolated footing of dimension (2.5 mX 2.5m) for different soil types and layers is considered.

4.2 Comparative studies based on top soil thickness

The parameters for this parametric study are combinations of different soil types from stronger layer to weak deposit and vice versa. Based on the soil type, the thickness of the top soil plays a significant role on the bearing capacity. According to the keys given above for the types of soils the parametric study has been conducted as follows.

Table 4.a: Cases on S.S.2

Soil Type	h/B	Cases
S1-dense gravel/ soft clay S2-dense sand/sat soft clay S3-dense sand/loose sand S4-stiff sat clay/ soft clay	- h/B = 0.6 - h/B= 1.2 - h/B=1.6	1
Sa-Dense gravel/ Stiff Sat clay/ Soft sat clay Sb- Soft Sat clay/ Stiff sat clay/ dense	- h/B = 0.6 - h/B= 1.2 - h/B= 2	2
Si-Soft Clay/silty clay/stiff clay/dense gravel Sii-dense gravel/Stiff clay/Silty clay/soft sat clay	- h/B= 0.4 - h/B= 1.6	3

4.5 Results of Parametric Study based on top soil thickness

This section defines the various cases studied and presents the results of the analysis. The combination of the main parameters employed for each particular case is also specified.

The responses studied include the bearing capacity evaluation with different, settlement of foundation due to lateral equivalent static loads.

In The analysis finite element software plaxis is used to get the output of the above cases in different soil layers and load categories.

Table 4.1: bearing capacity for S1 for h/B

h/B	hand calculation	plaxis
0.6	164	223
1.2	230	227
1.6	360	320

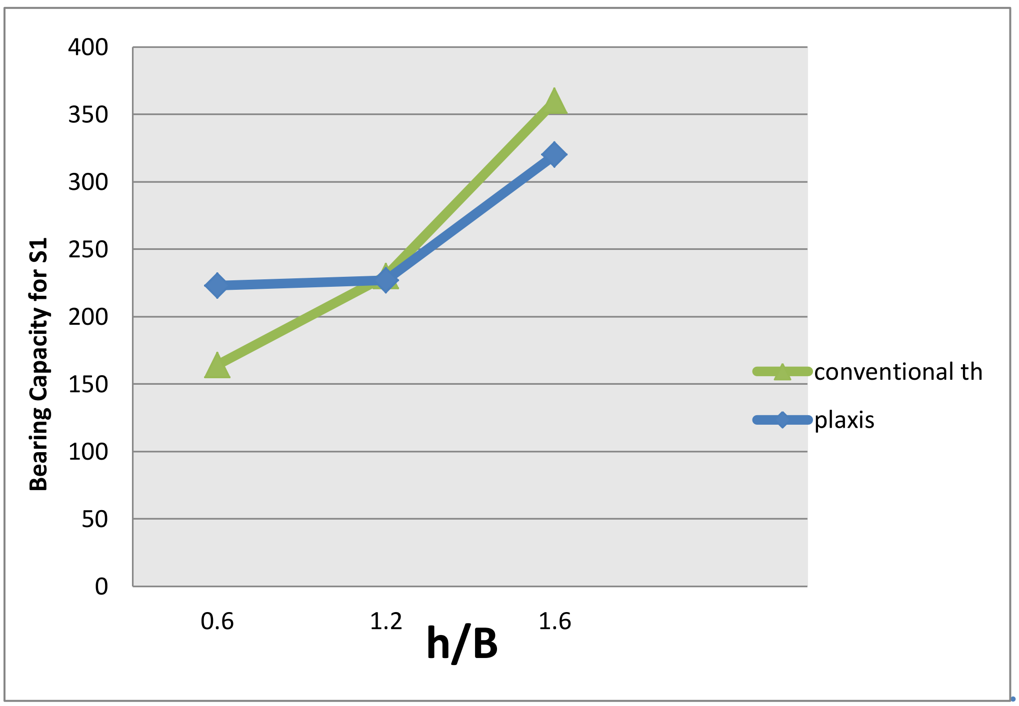


Figure 4.1: Bearing capacity comparison for S1

Table 4.2: bearing capacity of S2 for h/B

h/B	hand calculation	plaxis
0.6	113	135
1.2	197	152
1.6	326	300

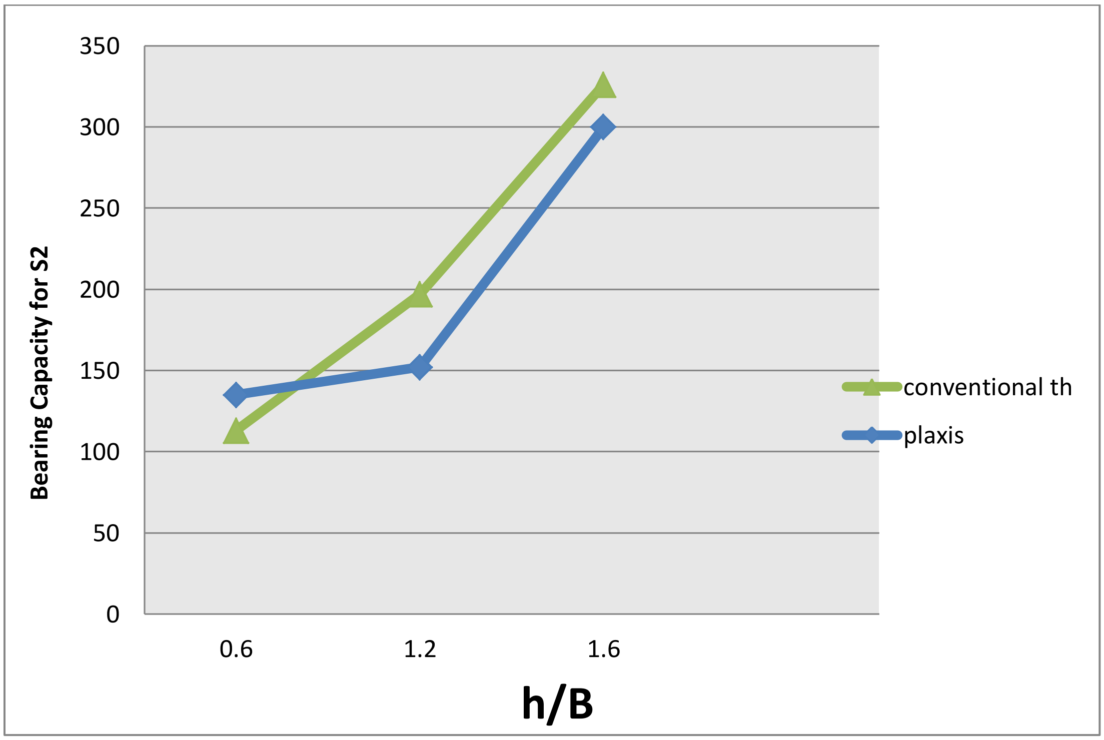


Figure 4.2: Bearing capacity comparison for S2

Table 4.3: bearing capacity of S3 for h/B

h/B	hand calculation	plaxis
0.6	112	100
1.2	182	157
1.6	197	225

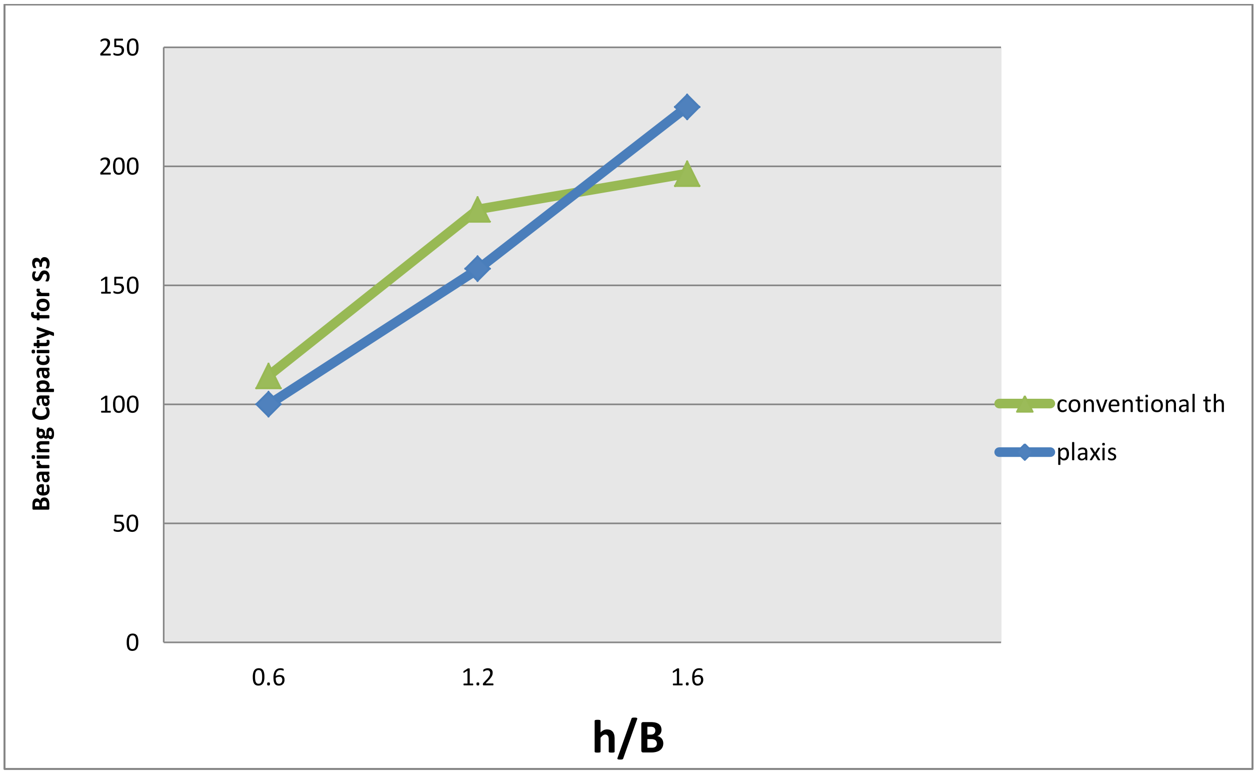


Figure 4.3: Bearing capacity comparison for S3

Table 4.4: bearing capacity of S4 for h/B

h/B	hand calculation	plaxis
0.6	160	210
1.2	152	200
1.6	58	92

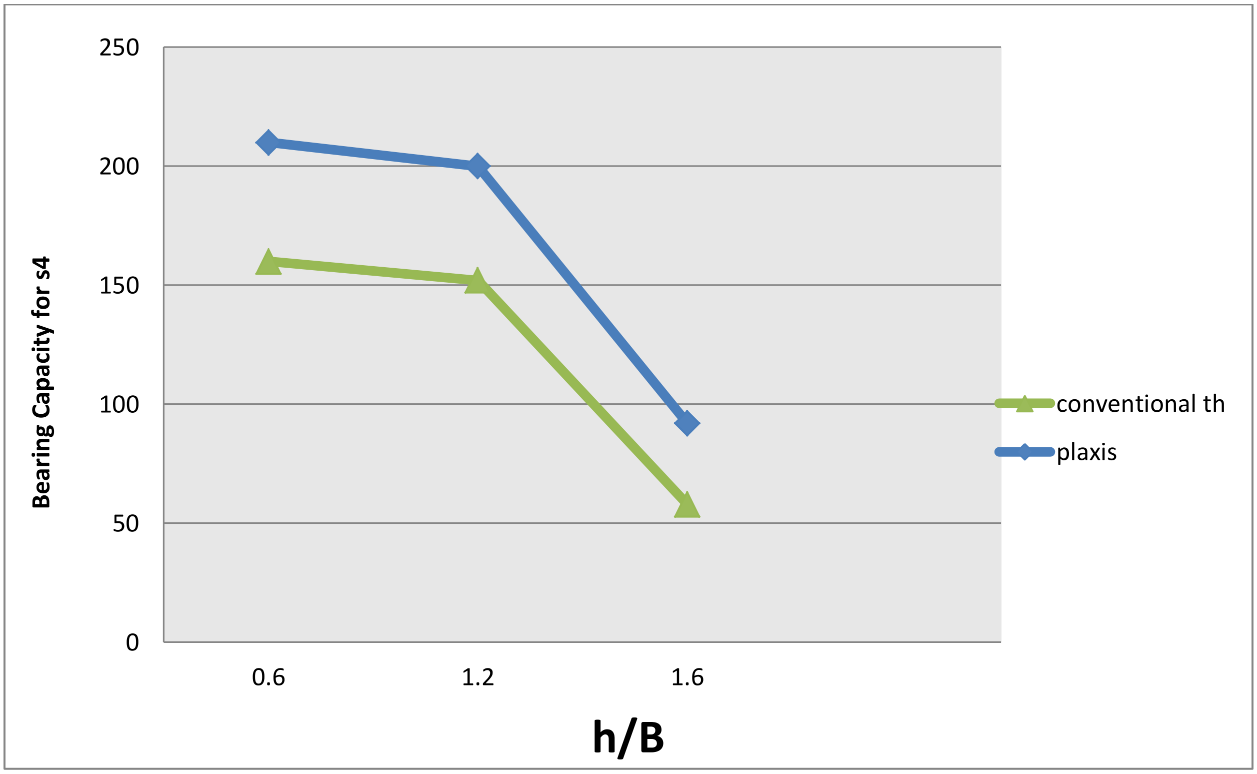


Figure 4.4: Bearing capacity comparison for S4

Table 4.5: bearing capacity of Sa for h/B

h/B	hand calculation	plaxis
0.6	310	290
1.2	392	408
2	420	480

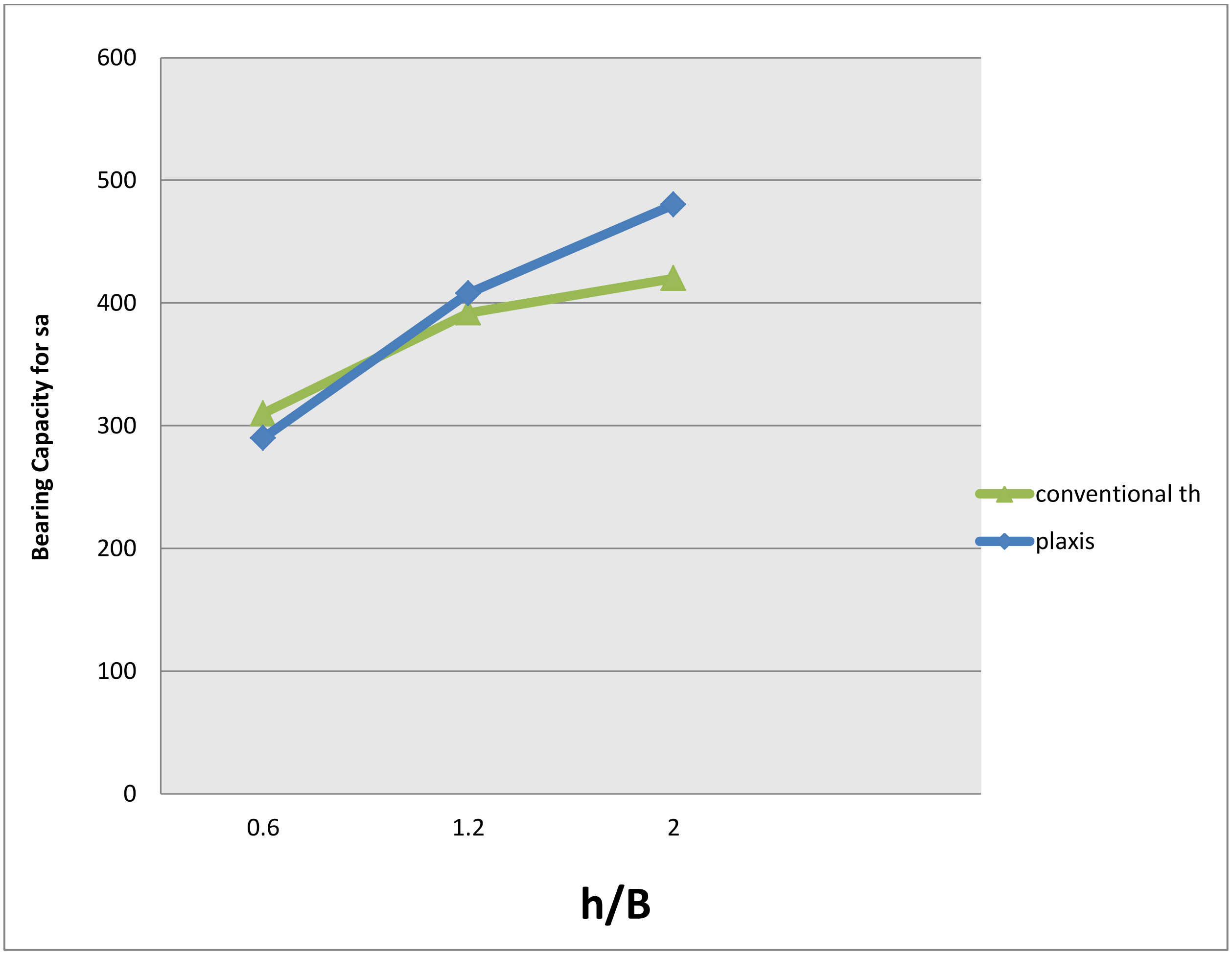


Figure 4.5: Bearing capacity comparison for Sa

Table 4.6: bearing capacity of Sb for h/B

h/B	hand calculation	plaxis
0.6	212	140
1.2	190	113
2	165	110

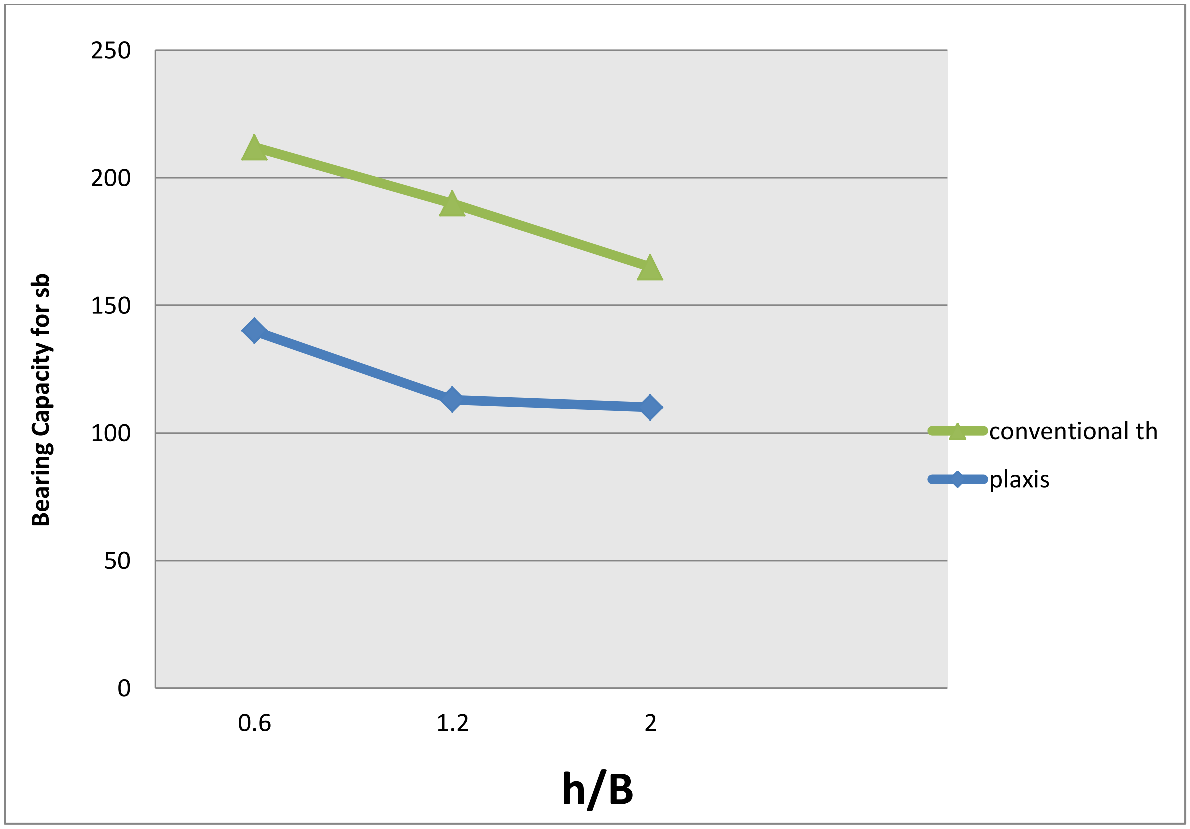


Figure 4.6: Bearing capacity comparison for Sb

Table 4.7: bearing capacity of Si for h/B

h/B	hand calculation	plaxis
0.4	486	470
1.6	152	135

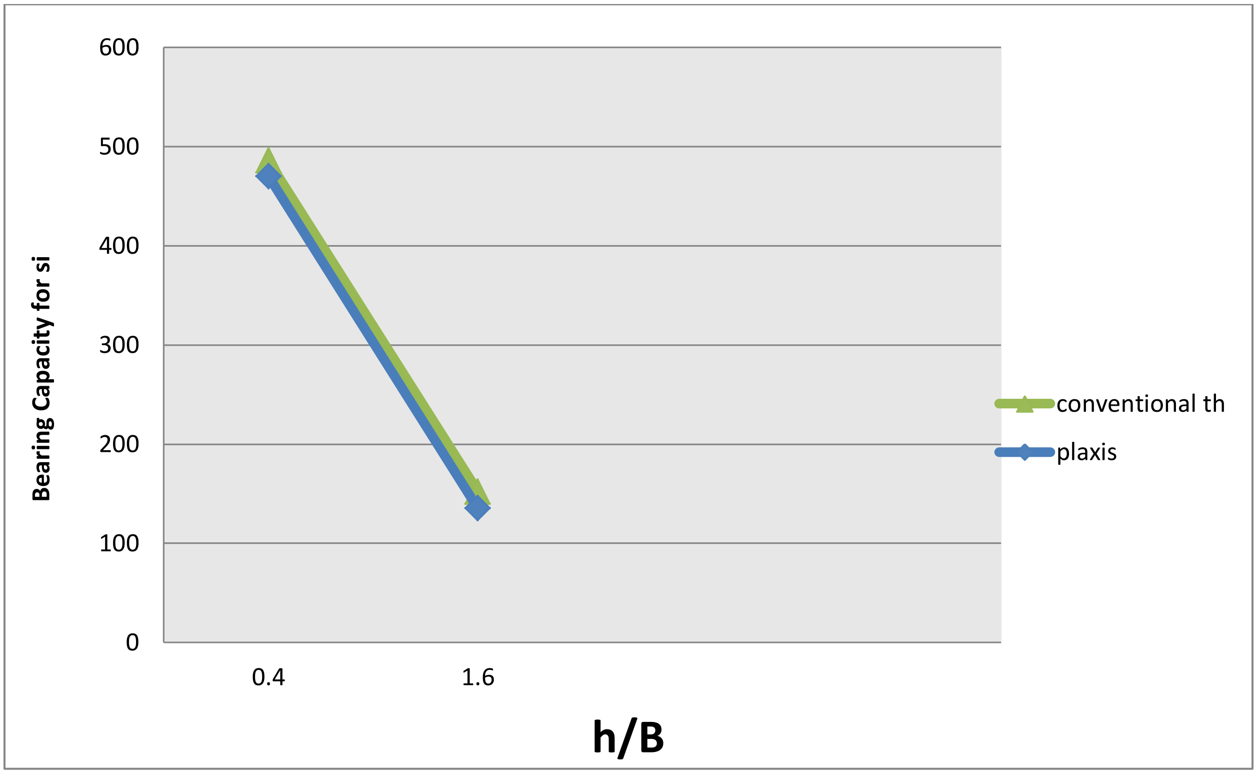


Figure 4.7: Bearing capacity comparison for Si

Table 4.8: bearing capacity of Sii for h/B

h/B	hand calculation	plaxis
0.6	212	140
1.2	190	113
2	165	110

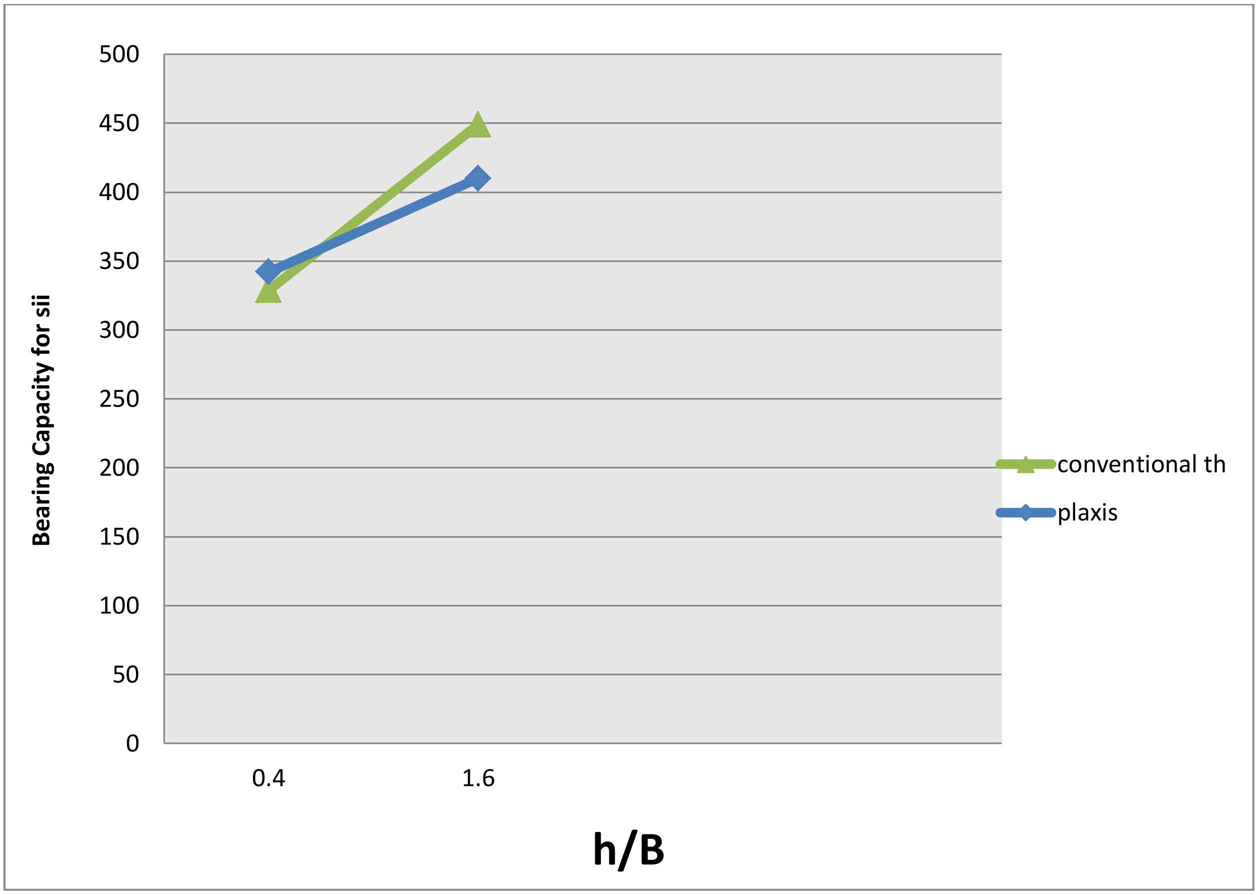


Figure 4.8: Bearing capacity comparison for Sii

In this chapter parametric studies are carried out to evaluate the bearing capacity of layered soils and effect of soil layers on the bearing capacity of a foundation, in addition to that to compare the results of finite element analysis software Plaxis.

Chapter Five

5. Discussions on results

From The results of Plaxis and empirical calculations, It has been observed that, the results obtained in the plaxis are substantialy different from empirical calculations. There is a difference of percentage variation up to 43%. This is due to the consideration that is taken by plaxis which is in detail analysis of the soil layers than that of a mere assumption of conventional method.

A good correlation between analytical results and theoretical results has also been found for layered soil system, provided, when the influence depth is varied with varying the thickness of the top layer in a fashion demonstrated above. While using the conventional theoretical approaches for layered soil stratum, the consideration of influence depth in those theoretical calculations help in commenting on the failure mechanism and assessing the failure surface in the sub soil.

In the case of two layered, three layered and four layered soil types By using FEM analysis software (Plaxis) their respective bearing capacity has been analyzed and plaxis software is more economical than conventional method since the pressures that we get from it is most of the time lower than conventional method.

For S1 when the h/B increases from 0.6 to 1.6 the bearing capacity also increases from 223 to 320 KN/m^2. For soil type S2, when the h/B increases from 0.6 to 1.6 the bearing capacity also increases from 135to 300 KN/m^2, For soil type S3 the bearing capacity also increases from100 to 225 KN/m^2 For soil type Sa the bearing capacity also increases from290 to 480KN/m^2 , For soil type Sii the bearing capacity also increases from140 to 110 KN/m^2 it shows that as the top soil is stronger than the underlain soil. This shows the increase in the bearing capacity of the soil layer varies considerably for different soil types with increase in depth. For the soil types with a slight change in bearing capacity, the settlement of the building increases and it shows that it is not proper to construction.

Whereas, For soil type Si the bearing capacity decreases from470 to135 KN/m^2 also For soil type Sb the bearing capacity decreases from 470 to135 KN/m^2 and For soil type S4 the bearing capacity decreases from 210 to 92 KN/m^2, it shows that as the top soil is stronger than the underlain soil.

Chapter Six

6. Conclusion and Recommendation

As we can see from both the Plaxis and empirical computations, the bearing capacity is influenced by the number of layers in the underlain soils and with the thickness of the top soil.

Similarly from consideration of influence depth in bearing capacity, both the conventional theoretical approaches and finite element software can infer the effect. For the first case of soil model S1, S2, S3, S9, and Sii the bearing capacity increases as h/B increases from 0.6 to 1.2. It shows that the top soil is stronger than the underlain soil. .

The settlement decreases and also FEM gives lower values than that of classical method in many cases, it implies that FEM is economical. Since FEM is able to analyze in different loading conditions with their precede, it is also better for showing failure line. In FEM the stresses are analyzed in all direction with their respective settlement. For the case of weak soil overlaid by a strong soil deposit where the top layer is relatively thick ($h \geq 0.5B$), the failure mechanism may be limited in the top layer only and the strength of the remaining lower layer has minimal influence. but due to uniformly graded nature of the coarse soil permeability is the danger, Since there is no ground water analysis in this study.

In the second cases Si, Sb, S4 the bearing capacity decreases as h/B decreases from 0.4 to 2. It shows that the top soil is weaker than the underlain soil. But as ($h < 0.5B$) i.e. when the top layer is relatively thin the influence of lower strong clay layer is higher.

Output of finite element modeling in PLAXIS 3D gives the proximity up to 50%. In the case of weak soil overlaid by strong soil deposit where the top soil is relatively thick the failure mechanism relatively limited in the top layer.

Scope for Future Works

.The further studies may be extended towards the following directions.

- Other shapes of foundations can be considered. Axisymmetric problems can be analyzed using PLAXIS.
- Other than Mohr-Coulomb model, advanced soil models such as Hardening Soil model, Soft Soil Creep model and user defined models can be used.
- Study of bearing capacity of footings under dynamic load in layered soil can be done.
- Other FEM softwares like Abaqus can be used for analysis.

Reference

[1] Abdel-Baki, S., Raymond, G. P. & Johnson, P. *"Improvement of the Bearing Capacity of Footings by a single Layer of Reinforcement"*,1993

[2] Ahmed S.Al-Agha,"*Basics of Foundation Engineering with solved problems*", 2015

[3] Alemayehu, T., "*Principles of Foundation Engineering*", Addis Ababa: Faculty of Technology, Addis Ababa University, 1992

[4] *Adrinam Paul "ultimate bearing capacity of shallow footings on layered cohesive soil", master's thesis, 2014*

[5] Bowles, J.E."*Foundation analysis and design*."*McGraw-Hill*, New York. (2002).

[6]Brinkgreve, R.B.J ,. "*PLAXIS 2D*-Version *Manual"*.Balkema (2009).

[7] Chen and Davidson, Florkiewicz and Michalowski *," Bearing Capacity determination by limit Analysis*"(1995)

[8]Davis Chen and Sloan, *"bearing capacity of a sand layer on clay by FEM*". 1989

[9]EBCS 7, "*Ethiopian Buildings Codes of Standards*", Ministry of works and urban

Development, Addis Ababa-Ethiopia, 1995

[10]Hanna, A. M. & Meyerhof G. G.,"*Design Charts for Ultimate Bearing Capacity of Foundations on Sand Overlying Soft Clay."* Canadian Geotechnical Journal, vol. 17, n. 2, pp. 300-303, 1980

[12] Hanna, A. M. & Meyerhof G. G."*Experimental Evaluation of Bearing Capacity of Footings Subjected to Inclined Loads*", Canadian Geotechnical Journal, vol. 18, n.4, pp. 599-603", 1981

[13]Lyamin and Sloan*,"bearing capacity of sand layer on clay by FEM*" Canadian Geotechnical Journal ,2011

[14] Meyerhof, G.G."*Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay*." Canadian Geotechnical Journal, vol. 11, pp. 2g23-229, 1974

[15] Meyerhof, G. G. & Hanna, A. M. "*Ultimate Bearing of Foundations on Layered Soils Under Inclined Load"* Canadian Geotechnical Journal, vol. 15, n. 4, pp. 565-572, 1978

[16] Nagtegaal et al. 1974; Sloan and Randolph, "Rigorous plasticity solutions for bearing capacity of layered soils " 1982

[17]Plaxis3D user Manuel, Version 2

[18]Prof.T.G.Sitharam Indian Institute of Science ,Bangalore
http://nptel.ac.in/courses/105108069/2 (Lecture 1)

[19]V.N.S Murthy, "*Geotechnical Engineering Principles and practices of soil mechanics and foundation engineering*"

[20Yuan Fan-fan, "*methods for estimating ultimate bearing capacity of layered foundation*", 2003

[21] Zienkiewicz and Taylor," the finite element method", 2005

Appendices

A.1 PLAXIS 3D FORMULATIION

A.1.1 GENERAL DEFINITIONS OF STRESS

Stress is a tensor which can be represented by a matrix in Cartesian coordinates:

$$\underline{\underline{\sigma}} = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix}$$

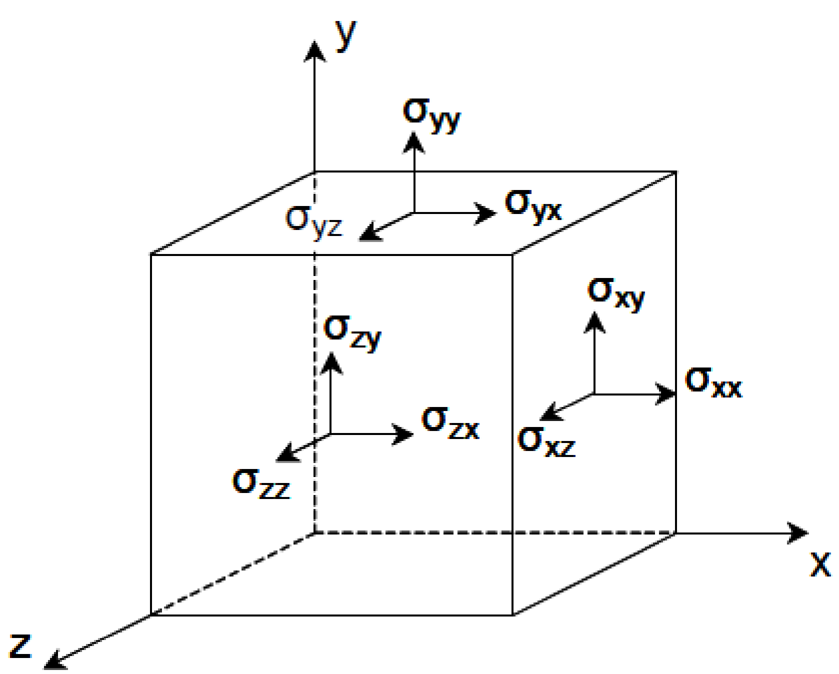


Figure A.1: General three-dimensional coordinate system and sign convention for stresses

In the standard deformation theory, the stress tensor is symmetric such that $\sigma xy = \sigma yx$, $\sigma yz = \sigma zy$ and $\sigma zx = \sigma xz$. In this situation, stresses are often written in vector notation, which involve only six different components.

A 1.2- Mohr-Columb Soil Model

FORMULATION OF THE MOHR-COULOMB MODEL

The Mohr-Coulomb yield condition is an extension of Coulomb's friction law to generalstates of stress. In fact, this condition ensures that Coulomb's friction law is obeyed inany plane within a material element.

The full Mohr-Coulomb yield condition consists of six yield functions when formulated in terms of principal stresses:

$$f_{1a} = \frac{1}{2}(\sigma'_2 - \sigma'_3) + \frac{1}{2}(\sigma'_2 + \sigma'_3)\sin\varphi - c\cos\varphi \leq 0$$

$$f_{1b} = \frac{1}{2}(\sigma'_3 - \sigma'_2) + \frac{1}{2}(\sigma'_3 + \sigma'_2)\sin\varphi - c\cos\varphi \leq 0$$

$$f_{2a} = \frac{1}{2}(\sigma'_3 - \sigma'_1) + \frac{1}{2}(\sigma'_3 + \sigma'_1)\sin\varphi - c\cos\varphi \leq 0$$

$$f_{2b} = \frac{1}{2}(\sigma'_1 - \sigma'_3) + \frac{1}{2}(\sigma'_1 + \sigma'_3)\sin\varphi - c\cos\varphi \leq 0$$

$$f_{3a} = \frac{1}{2}(\sigma'_1 - \sigma'_2) + \frac{1}{2}(\sigma'_1 + \sigma'_2)\sin\varphi - c\cos\varphi \leq 0$$

$$f_{3b} = \frac{1}{2}(\sigma'_2 - \sigma'_1) + \frac{1}{2}(\sigma'_2 + \sigma'_1)\sin\varphi - c\cos\varphi \leq 0$$

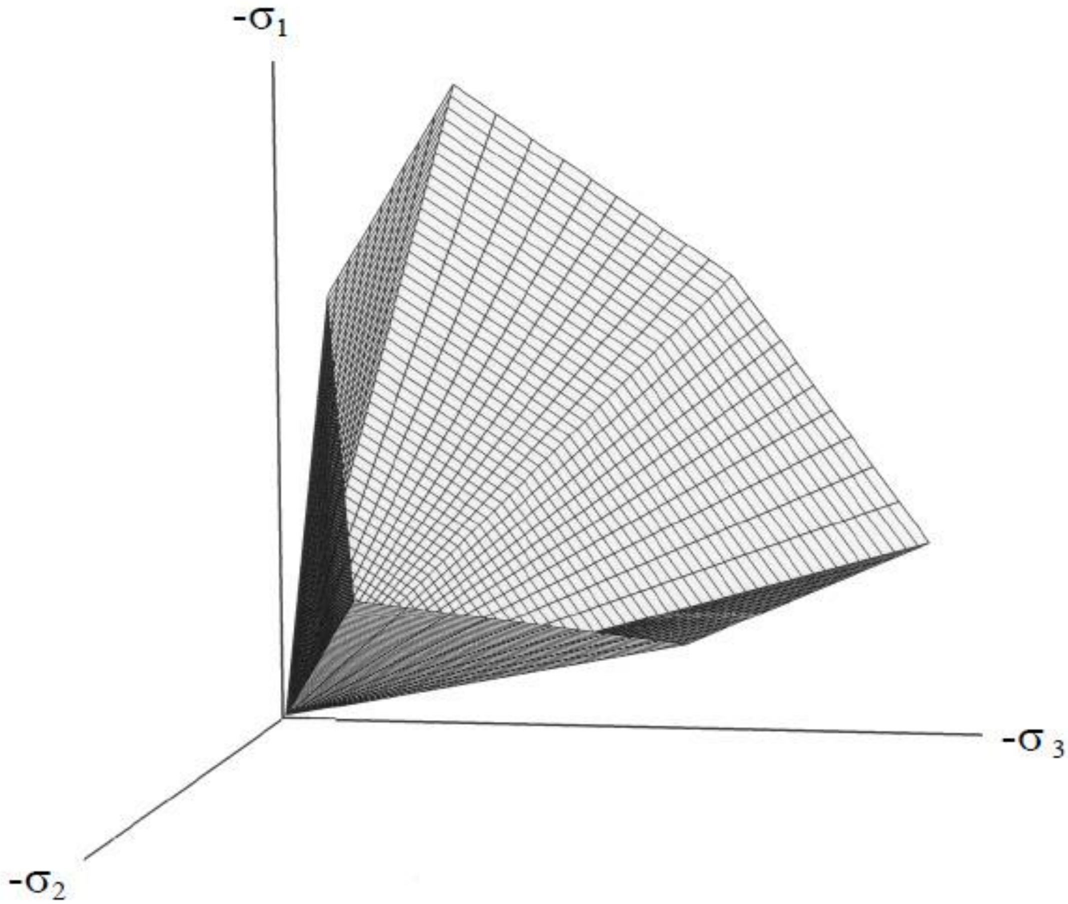


Figure A.2: The Mohr-Coulomb yield surface in principal stress space (*c* = 0)

A1.2 Basic parameters of the mohr-coulomb model

The Mohr-Coulomb model requires a total of five parameters, which are generally familiar to most geotechnical engineers and which can be obtained from basic tests on soil samples.

These parameters with their standard units are listed below:

E : Young's modulus [kN/m2]

ν : Poisson's ratio [-]

ϕ : Friction angle [°]

c : Cohesion [kN/m2]

ψ :Dilatancy angle [°]

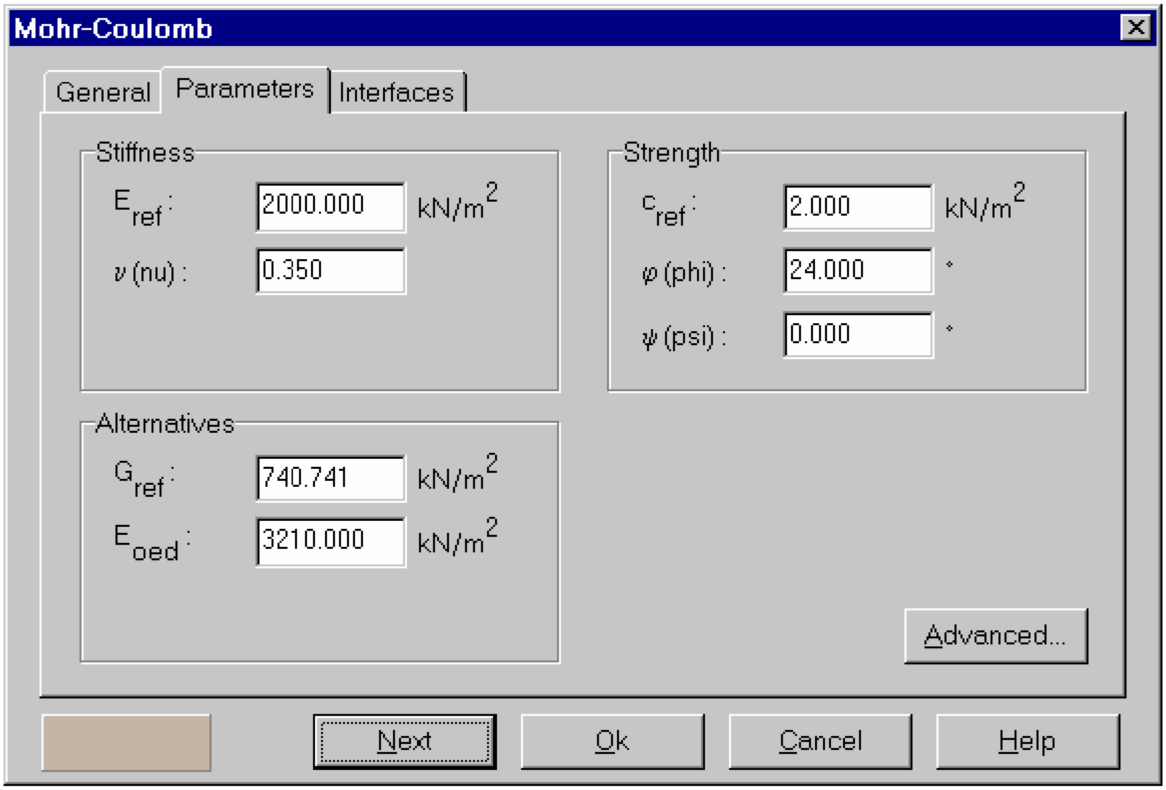


Figure A.2: Parameter tab sheet for Mohr-Coulomb model

Young's modulus (E)

PLAXIS uses the Young's modulus as the basic stiffness modulus in the elastic model and the Mohr-Coulomb model, but some alternative stiffness moduli are displayed as well. A stiffness modulus has the dimension of stress. The values of the stiffness parameter adopted in a calculation require special attention as many geomaterials show a nonlinear behaviour from the very beginning of loading. In soil mechanics the initial slope (tangent modulus) is usually indicated as E_0 and the secant modulus at 50% strength is denoted as E_{50} . For materials with a large linear elastic range it is realistic to use E_0, but for loading of soils one generally uses E_{50}. Considering unloading problems, as in the case of tunnelling and excavations, one needs E_{ur}instead of E_{50}. For soils, both the unloading modulus, E_{ur}, and the first loading modulus, E_{50}, tend to increase with the confining pressure. Hence, deep soil layers tend to have greater stiffness than shallow layers. Moreover, the observed stiffness depends on the stress path that is followed.

Poisson's ratio (ν)

Standard drained triaxial tests may yield a significant rate of volume decrease at the very beginning of axial loading and, consequently, a low initial value of Poisson's ratio (ν).

For some cases, such as particular unloading problems, it may be realistic to use such a low initial value, but in general when using the Mohr-Coulomb model the use of a higher value is recommended.

The selection of a Poisson's ratio is particularly simple when the elastic model or Mohr-Coulomb model is used for gravity loading (increasing Σ*Mweight*from 0 to 1 in a plastic calculation). For this type of loading PLAXIS should give realistic ratios of $K0 = \sigma_h / \sigma_V$ .As both models will give the well-known ratio of $\sigma h / \sigma v = \nu / (1-\nu)$ for one-dimensionalcompression it is easy to select a Poisson's ratio that gives a realistic value of *K*0. Hence,νis evaluated by matching *K*0. This subject is treated more extensively in Appendix Aof the Reference Manual, which deals with initial stress distributions. In many cases one will obtain ν values in the range between 0.3 and 0.4. In general, such values can also be used for loading conditions other than one-dimensional compression. For unloading conditions, however, it is more appropriate to use values in the range between 0.15 and 0.25.

Cohesion (c)

The cohesive strength has the dimension of stress. PLAXIS can handle cohesionless sands(c =0), but some options will not perform well. To avoid complications, nonexperiencedusers are advised to enter at least a small value (use c > 0.2 kPa). PLAXIS offers a special option for the input of layers in which the cohesion increases with depth.

Friction angle (ϕ)

The friction angle, ϕ (phi), is entered in degrees. High friction angles, as sometimesobtained for dense sands, will substantially increase plastic computational effort. The computing time increases more or less exponentially with the friction angle. Hence, high friction angles should be avoided when performing preliminary computations for aparticular project.

The friction angle largely determines the shear strength by meansof Mohr's stress circles.The Mohr-Coulomb failure criterion proves to be better for describing soil behaviour than the Drucker-Prager approximation, as for the latter the failure surface tends to be highly inaccurate for axisymmetric configurations.

Dilatancy angle (ϕ)

The dilatancy angle, ψ (psi), is specified in degrees. Apart from heavily overconsolidated layers, clay soils tend to show little dilatancy ($\psi \approx 0$). The dilatancy of sand depends on both the density and on the friction angle. For quartz sands the order of magnitude is $\psi \approx \phi$ - 30°. For ϕ-values of less than 30°, however, the angle of dilatancy is mostly zero. A small negative value for ψ is only realistic for extremely loose sands.

A.3 PlaxisOutPuts

- Effective stress values

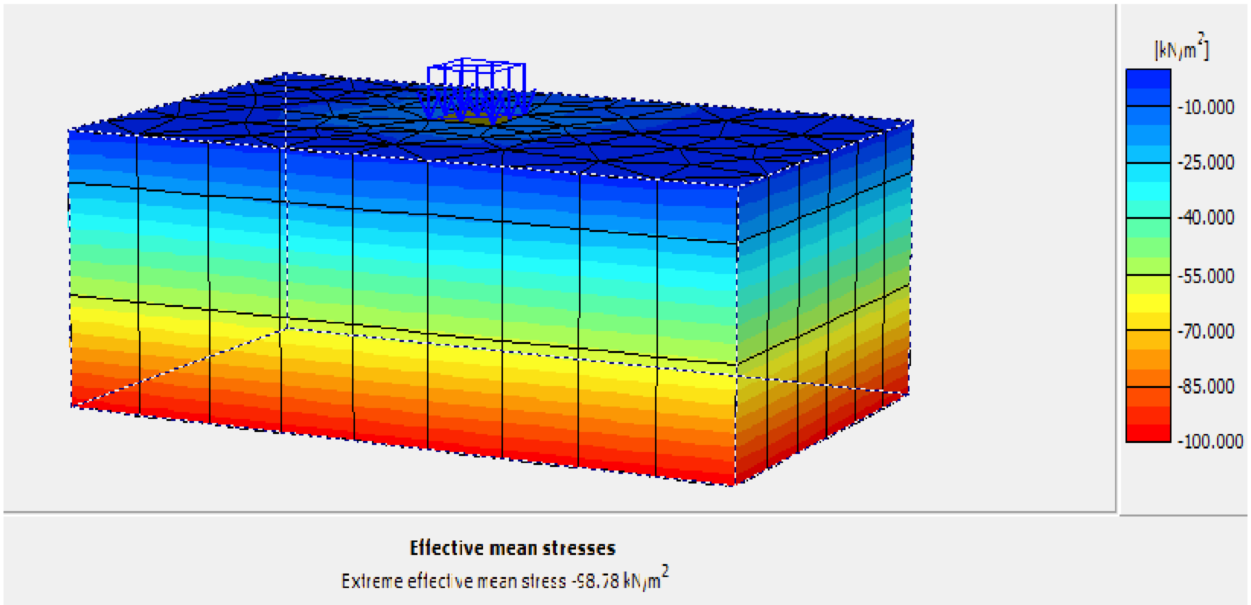


Fig A.3 effective stress of dense gravel on silty clay

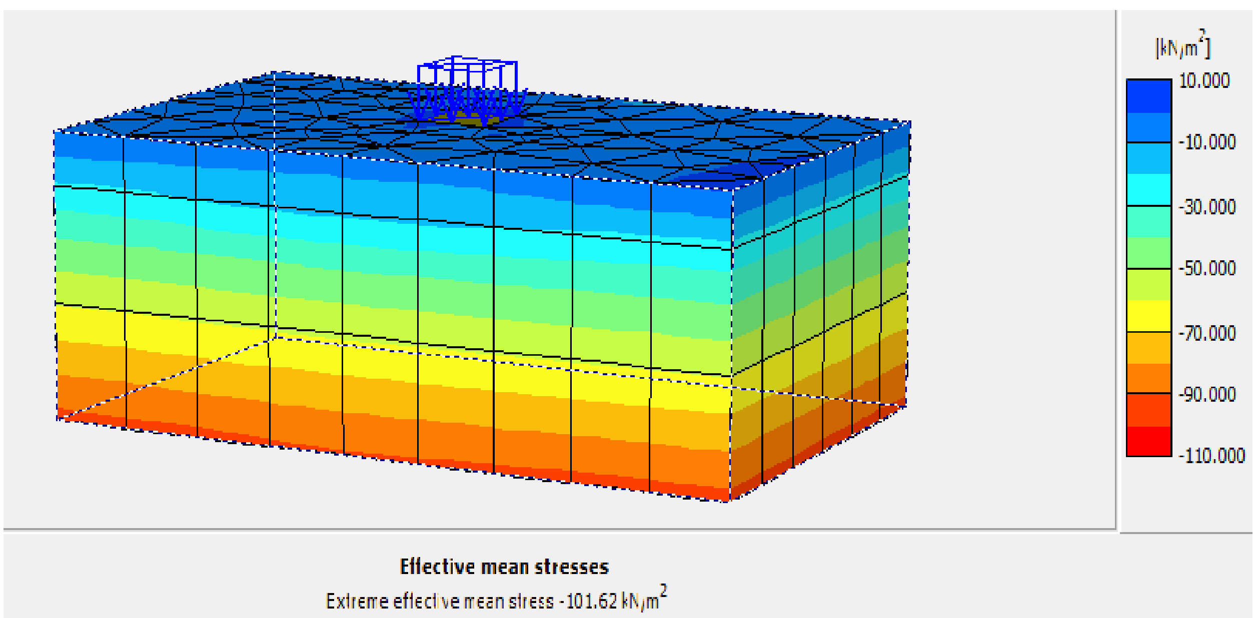


Fig A.4 effective stress of dense sand over loose sand

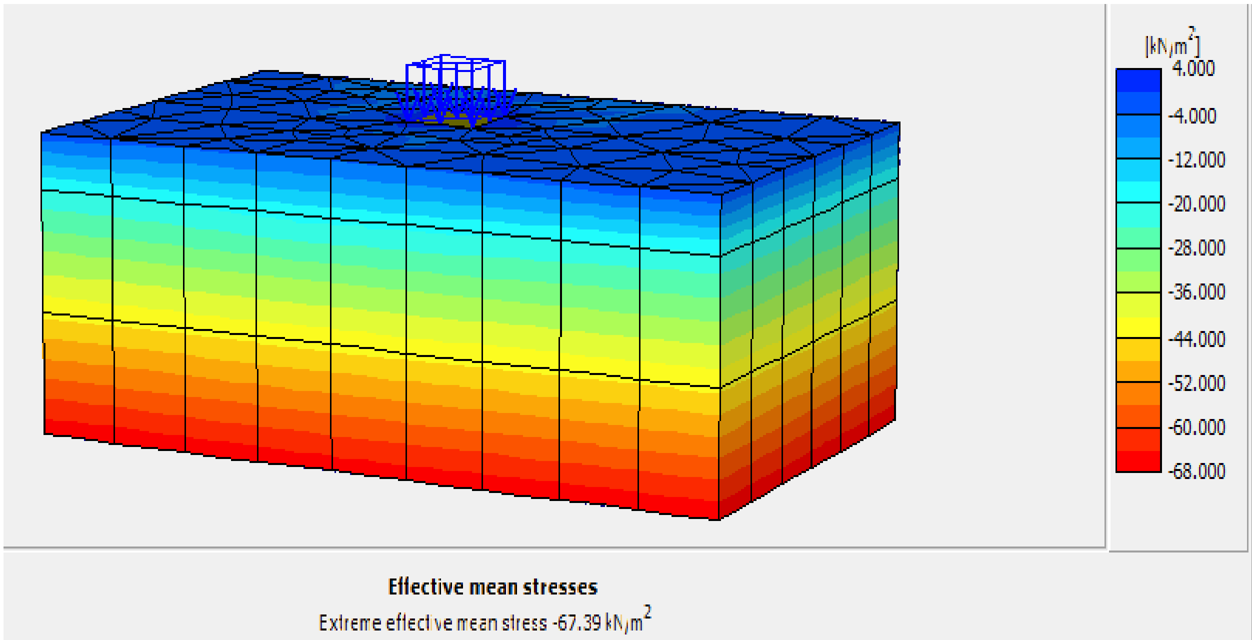


Fig A.5 effective stress of dense sand over saturated soft clay

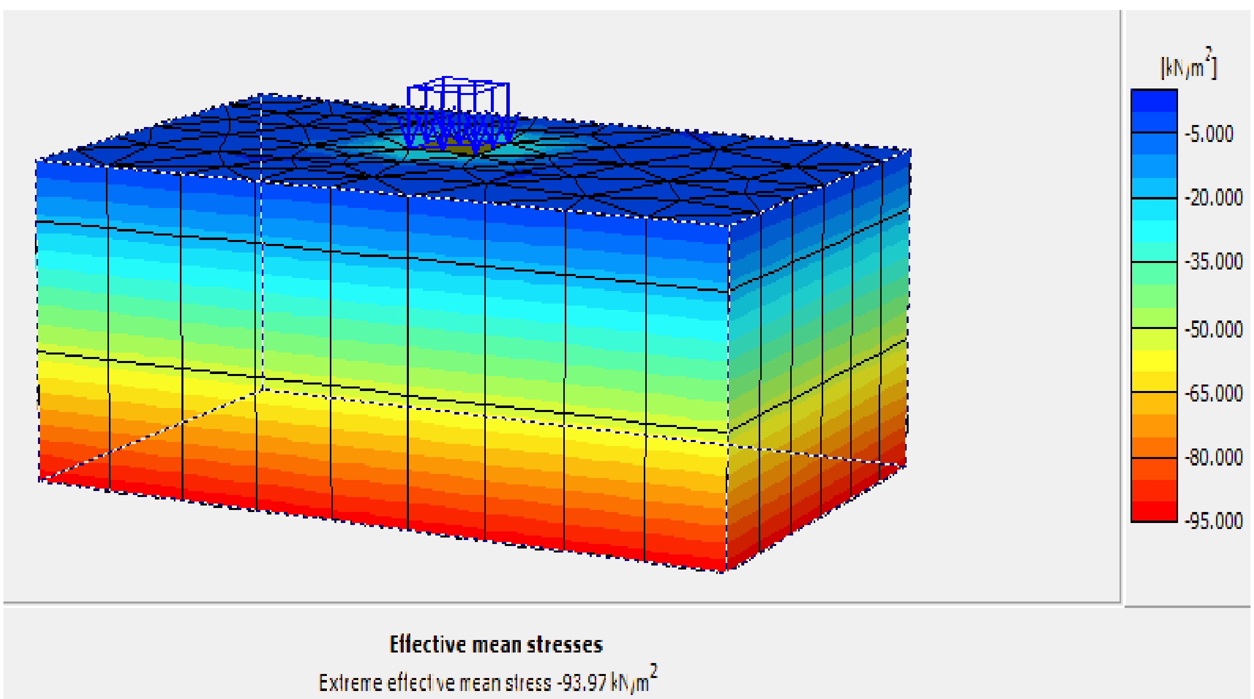


Fig A.6 effective stress of soft clay on saturated stiff clay