EVALUATING BEARING CAPACIY OF SHALLOW FOUNDATION ON LAYERED SOIL USING FINITE ELEMENT BASED SOFTWARE

By

Samson Sileshi

A Thesis Submitted to

The department of Civil Engineering,

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

Adama, Ethiopia

December, 2020

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BY: SAMSON SILESHI AYELE

ADVISOR: SIRAJ MULUGETA, PhD

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Approval of Board of Examiners

We, the undersigned, members of the Board of Examiners of the final open defense by Samson Sileshi Ayele have read and evaluated his thesis entitled “Evaluating Bearing Capacity of Shallow foundation on layered soil using Finite Element Based Software.” and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirement of the Degree of Masters in Civil Engineering (specialization in Geotechnical Engineering).

Name	Signature	Date
Samson Sileshi		
Name of student		
Siraj Mulugeta, PhD		
Advisor /Supervisor/		
Yadeta Chamdessa, PhD		
Chairperson		
Argaw Asha, PhD		
Internal Examiner		
Tezera Firew, PhD		
External Examiner		
Head of Department		
School Dean		
Post Graduate Dean		

Declaration

I hereby declare that this MSc Thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

Name: Samson Sileshi Ayele

Signature: ___________

This MSc Thesis has been submitted for examination with my approval as thesis advisor.

Name: Siraj Mulugeta (PhD)

Signature:

Date of submission: ………..

ACKNOWLEDGEMENT

First, I would like to thank the almighty GOD for all the blessings I have received and the strength he gave me through my life time and during my research work. I would like to express my sincere appreciation and deep gratitude to my advisor, Dr. SIRAJ MULUGETA, who devoted time and effort to this research project. I am extremely grateful for all his support, encouragement and advice throughout my studies.

I would like to acknowledge ASSOSA UNIVERSITY for the scholarship program and sponsoring me. Also I would like to extend my deepest appreciation for ASTU Civil and Environmental engineering staff member for their hand work and all their support throughout my studies specially Dr. ARGAW ASHA.

Finally, my last and most important acknowledgement goes to my FAMILIY for their highly practical advice and valuable contributions to every aspect of my life. I am forever indebted to you for the never ending support provided to me and all the sacrifices made. At last but not least l would like to give my thanks to my friends in life and classmates for their encouragements and advices throughout my studies Maye GOD bless you all.

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LIST OF ACRONYMS AND ABBREVATONS

B	Width of square footing
C	Cohesion of Soil
D	Foundation Depth
Dw	Depth of Water Table below Ground Surface
E	Young's Modulus of Elasticity of Soil
FEA	Finite Element Analysis
FEM	Finite Element Method
H	Thickness of Top Layer in Two-Layer Subsoil System
H_1	Thickness of first Layer in three -Layer Subsoil System
H_2	Thickness of second Layer in three -Layer Subsoil System
Ka	Active Earth Pressure Coefficient
Kp	Passive Earth Pressure Coefficient
Ko	Coefficient of lateral earth pressure at rest
L	Length of Footing
σv	Total Vertical Pressure
σh	Total Horizontal Pressure
β	Inclination Angle of the Backfill to the Horizontal
φ	Angle of Internal Friction of Soil
ψ	Dilatancy Angle of Soil
γ	Bulk Density of Soil
ν	Poisson's Ratio of Soil
q	Ultimate Bearing Capacity of Soil
Nc, Nq, Nγ	Bearing Capacity Coefficients

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. It's found that as the thickness of upper layer to width ratio increase the bearing capacity increase for stronger layer overlying softer layer whereas for softer layer overlying stronger layer the bearing capacity decreases. And also as the length to width ratio of footing increases the bearing capacity of the soil decreases. Ground water also significantly reduces the bearing capacity of foundation. Spacing of 1.5 times the width of the footing is also recommended as minimum value.

Key words: Bearing capacity, layered soil, Finite Element Method

CHAPTER ONE

INTRODUCTION

1.1 Background

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 seismic 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.

In designing our foundation there are two parts: the ultimate bearing capacity of the soil under the foundation, and the tolerable settlement that the footing can undergo without damaging the superstructure. The ultimate bearing capacity aims in determining the load that the soil under the foundation can handle before shear failure. The magnitude of the ultimate bearing capacity depends on the mechanical characteristics of the soil and the physical characteristics of the footing.

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.

Research on the ultimate bearing capacity problems can be carried out using either analytical solutions or experimental investigations. The former could be studied through theory of plasticity or finite element analysis, while the latter is achieved through conducting prototype, model and full-scale tests. A satisfactory solution is found only when theoretical results agree with those obtained experimentally.

In the literature, over the last four decades, several reports can be found dealing with the problem of foundations resting on layered soils. At first, researchers based their studies on the results of prototype laboratory model testing in order to develop empirical formulae to predict the ultimate bearing capacity of these footings. Recently, theories based on finite element and numerical analyses were presented that gave more rational solutions as compared to the previous ones.

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.

This study has developed a better understating on estimation of bearing capacity of soil in layered soil profile. A soil data has been accessed from secondary source of data and the soil profile and material parameters has been used as an input. The investigation tool utilized was numerical modelling with PLAXIS 3D finite element based software. A series of 3D numerical analysis has been conducted to analysis bearing capacity of layered soil profile. Investigation concerning the key parameters that mainly influence the bearing capacity of soil has been addressed and discussed in depth by a researcher through an intensive literature review based on the analyses result.

1.2 Statement of problem

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.

1.3 Research objectives

1.1.1 General objective

The general objective of this research is to evaluate the bearing capacity of layered soils using finite element analysis Based software.

1.1.2 Specific objectives

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

1.4 Significance of the study

This study helps to evaluate bearing capacity shallow foundation 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 gathered from literature.

1.5 Scope of the study

The study mainly focused on the estimation of bearing capacity of layered soil under vertical and static loading. The analysis is extended to study the influence of key design parameters such as soil properties, layering of soil, upper layer thickness to width ratio, spacing between two footings and effect of ground water. The result from PLAXIS 3D code was then compared with empirical method.

1.6 Organization of the thesis

The content of this thesis is organized in to five chapters: Chapter 1 provides general introduction with the research objectives, scope and significance of the study. Chapter 2 outlines a full detailed literature review of bearing capacity determination, including historical development, and previous research works. Chapter 3 presents material parameters, soil profile adopted for modeling, preliminary checks for sensitivity analyses. In Chapter 4 Result and Discussion regarding the influence of key design parameters is also furnished. The last chapter, Chapter Five, discusses the conclusions derived from this study and limitations in the analyses and recommendations for future work.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

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 (Murthy, 2007).

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) (Murthy, 2007).

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 (Murthy, 2007).

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2.2 Bearing capacity equations

2.2.1 Ultimate bearing capacity (Qu)

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 Modes of failure

There are 3 possible mode of failure depending on type of soil and foundation size and depth. The first mode of failure is general shear failure which is usually encountered in stiff clay and dense sand underlying shallow foundation. The general shear failure mode is accompanied by the occurrence of a failure surface and the inability to maintain the applied pressure. As shown in (figure 2.1 a) there is a distinctive peak in the pressure versus settlement curve shown in the figure, which corresponds to the ultimate bearing capacity, qu.

The second mode of failure is local shear failure which is encountered in medium-stiff clay and medium-dense sand. It is characterized by the lack of a distinct peak in the pressure versus settlement curve as shown in (figure 2.2 b). In the case of local shear failure, determination of the ultimate bearing capacity is usually governed by excessive foundation settlements, as indicated in the figure. The local shear failure mode is accompanied by a progressive failure surface that may extend to the ground surface after qu is reached (Figure 2.1 b).

The third mode of failure is punching shear failure usually occurs in loose sands and soft clays. This type of failure is accompanied by a triangular failure surface directly under the foundation. As in local shear failure, punching failure is also characterized by the lack of a distinctive ultimate bearing capacity as shown in (figure 2.1 c). Thus, the ultimate bearing capacity in this case is taken as the pressure corresponding to excessive foundation settlements.

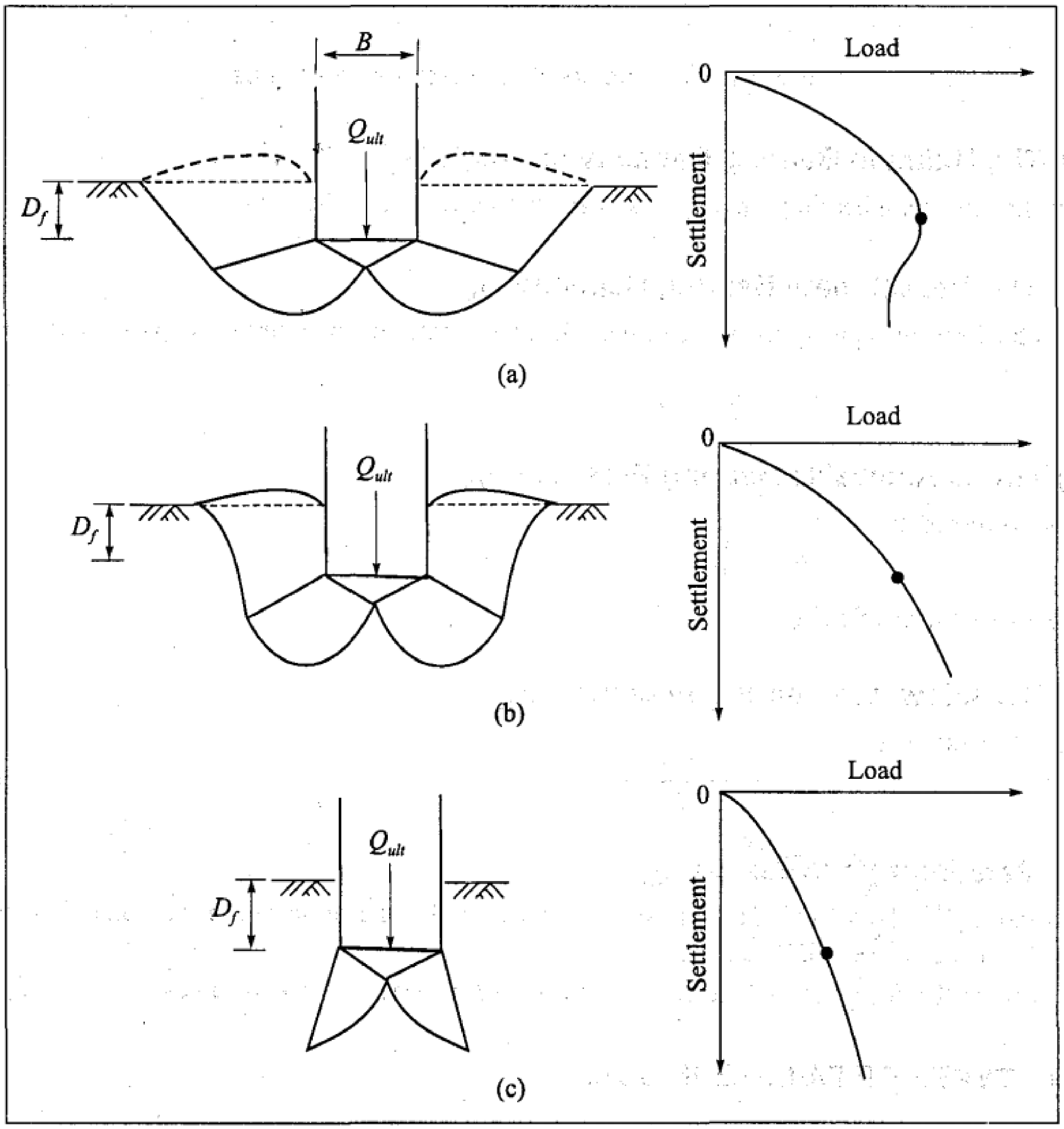


Figure 2. 2 Mode of bearing capacity failure (Vesic 1963): (a) general shear failure, (b) local shear failure, (c) punching shear failure

2.2.3 Bearing capacity of layered soil

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 nonhomogeneous 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 (Murthy, 2007).

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.

2.2.4 Punching shear model

As proposed by Hanna and Meyerhof the punching shear model was based on assumption that the failure plane as inverted uplift problem. Thus, at ultimate load the upper stronger soil mass having approximately truncated pyramidal shape pushed in to the weaker soil mass. So that the ultimate bearing capacity can be estimated as (Das, 2017);

$$Q_u = q_b + 2\left(\sum_{i=1}^{n} CiHi/B\right) + \sum_{i=1}^{n-1} Hi(\gamma iHi + 2\sum_{j=1}^{i-1} Hi\gamma i + 2D)\frac{Ksitan\Phi}{B} - \sum_{i=1}^{n} \gamma iHi \text{ ....equ2.1}$$

For rectangular footing equation 2.1 can be modified by introducing shape function Where K_{si}, ,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.

The soil beneath any foundation system are naturally stratified though within each layer we can consider as homogenous as in one soil property, although the properties of adjacent soil are quite different. If a footing is placed on stratified soil and the thickness of the upper layer beneath the footing is quite large compared with the width of the footing, usually 2 times the width, then realistic estimate of ultimate bearing capacity van be determined using conventional bearing capacity theory based on the property 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. 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.

$$C_{av} = \frac{C_1H_1 + C_1H_1 + C_1H_1 + \cdots + C_1H_1}{\sum H_i} \quad \text{……..equa2.2}$$

$$\Phi_{av} = tan^{-1}\frac{tan\varphi_1H_1 + tan\varphi_2H_2 + tan\varphi_3H_{13} + \cdots + tan\varphi_nH_n}{\sum H_i} \quad \text{……..equa2.3}$$

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

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

There are many methods to solve bearing capacity problems the most common of them are limit equilibrium method, limit analysis method, semi-empirical approach and finite element method. In recent decades the finite element method has been getting a lot of attention in solving geotechnical problems by many researchers. In practice engineers are giving priorities to less complicated solutions to simplify computation as experimental analysis are time consuming and commonly used limit equilibrium methods are becoming less 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.

2.3 Numerical limit analysis models

Finite element formulation of the upper bound theorem and lower bound theorem are briefly described here. 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 (J.S. Shiau, Bearing capacity of a sand layer on clay by finite element limit analysis, 2003).

The proposed lower bound formulation handles both two- and three-dimensional stress fields and, in the former case, employs the linear elements. Note that 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.

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 modelled using appropriate variables, and the optimum (minimum) internal power dissipation is obtained as the solution to a mathematical programming problem (J.S. Shiau, Bearing capacity of a sand layer on clay by finite element limit analysis, 2003).

2.4 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 (O.C. Zienkiewicz, 2005)

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.

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. Solutions have been obtained for few standard cases by classical methods, whereas solutions can be obtained for all problems by finite element analysis. In case where complex problems are faced such shape, boundary conditions, and loading the classic approach makes a drastic assumptions whereas the finite element method considers the problem as it is. 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. If structure consists of more than one material, it is difficult to use classical method, but finite element can be used without any difficulty.

2.5 Modeling using PLAXIS 3D FOUNDATION

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.

Work planes 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 work plane 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.

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. 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. On the contrary, stresses and strains are calculated at individual stress points (Gaussian integration points) rather than at the nodes.

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.

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 (PLAXIS 3D FOUNDATION Reference Manual).

2.6 Review on some related literature

Button (1953) analyzed bearing capacity of strip footing underlying two layers of clay soil. 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.

Reddy and Srinivasan (1967) extended the work of button to include the effect of non-homogeneity and anisotropy of the soil with respect of shear strength. The basic assumption in determining bearing capacity are: the failure surface is cylindrical, the coefficient of anisotropy is the same at all point in the soil medium, the soil in each layer is either homogenous with respect to shear strength or varies linearly with depth with respect to shear strength

Brown and Meyerhof (1969) investigated foundations resting on stiff clay layer overlying soft clay layer deposit and soft clay layer overlying stiff clay layer. They assumed that the soil fails by punching through the top layer in the first case and with full development of bearing capacity in the second case. From the obtained experimental result using empirical relations equations and charts were developed giving appropriate bearing capacity factors. But these results are essentially experimental and therefore strongly affected by characteristics of clay tested.

The purpose of their research is to present a result of series of model footing test carried out on two-layered clay soils, and the models have many limitations. First, they are limited to one type of clay though their strength was varied, and the deformation properties remains constant. Secondly, studies was limited to surface loading only using rigid strip and circular footing with rough base. Thirdly, all studies were made in terms of undrained condition using $\phi = 0$ analysis.

They also conducted a series of footing tests on homogeneous clay. And found that failure patterns beneath the footing is function of the physical modes 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 the conventional Mohr-coulomb concept of cohesion and friction.

Meyerhof (1974) investigated the case of 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 (MEYERHOF G. G., 1974).

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 is not always the case

Meyerhof and Hanna (1978) considered the case of strong layer overlying weak layer and weak layer overlaying strong layer. The analysis of different failure mode were compared with the results of model tests of strip and circular footing on layered sand and clay. They developed theories to predict the bearing capacity of layered soil under vertical loading and inclined loading conditions (A. M. HANNA, 1978).

In case of strong layer overlaying weak layer, considering the failure as inverted uplift problem, an approximate theory of 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. The authors concluded that the ultimate bearing capacity of footings on a dense layer overlying a weak layer can be expressed by inclination factors in conjunction with punching shear coefficients, which depend on the shear strength parameters and bearing capacity ratio of the layers under vertical loads.

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. They also observed that when the foundations are subjected to inclined loads the influence of non-uniformity, including anisotropy of the soil became more important than under vertical loads

Hanna and Meyerhof (1979) extended their previous theory of bearing capacity of two layered soil in to three layered soil. Their analysis was well compared with the test results of strip and circular footing on three layered soil. Only one case was considered in this paper, that for footings subjected to vertical loads and resting on subsoil consisting of two strong layers overlying a weak deposit (Meyerhof, 1979).

The same theoretical failure mechanism was assumed by considering a soil mass of the upper two layers is pushed into the lower layer, and the same forces acting on the failure surface was assumed as well. Formulas and charts were developed and can be used in designing foundations having the same conditions. Model tests on rough strip and circular footings under central vertical loads were made on the surface of three-layer sand consisting of two dense upper layers and a loose lower one. By comparing the results of the model tests with the results of the punching theory, good agreement was found. Briefly, this paper is an extension of the previous theory in order to include the case of the three-layer soil. But, it is restricted to only one case of three-layer soil, and it needs more development to include all possible cases of three-layer soils

Hanna and Meyerhof (1980) extended their previous work to cover the case of were dense sand deposit overlying soft clay deposit and presents their result in the form of design chart. 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 (MEYERHOF A. M., 1980).

Thus, in the previous theory of punching, the bearing capacity formula depends to a large extent on the value of the average mobilized angle of shearing resistance δ on the assumed failure planes, which was previously estimated as $2\phi/3$. The exact value of δ is difficult to calculate, and approximate value will affect the results of the calculation of the bearing capacity. However, these difficulties may be overcome by expressing the angle δ in the dimensionless form (δ/ϕ_1), where ϕ_1 is the friction angle of the upper sand layer. The design charts presented in this paper, together with the punching theory previously developed by the authors, can be utilized to predict the ultimate bearing capacity of footings on a dense sand layer overlying a soft clay deposit. These charts give more accurate results than the previous formulas and charts, but still estimation of the real bearing capacity since they are based on experimental results, the properties of sand and clay and the method used to model the tests can affect the charts and therefore the results.

Hanna and Meyerhof (1981) investigated experimentally the ultimate bearing capacity of footing subjected to axially inclined loads by conducting model tests on circular and strip footing 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 Manual. 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 (G. G. MEYERHOF, 1981)

Hanna (1987) investigated the ultimate bearing capacity of strip footing in homogenous and layered sand soil experimentally on model strip footing and using finite element model using nonlinear stress-strain relationship to simulate testing conditions. The result obtained from the finite element model are in good agreement with the corresponding experimental values. The investigation was only limited to sand and their theoretical model was used to generate data for the purpose of comparison with existing theories (HANNA, 1987).

J. S. Shiau, A. V. Lyamin, and S. W. Sloan (2003) investigated ultimate bearing capacity of strip footing resting on sand layer over clay using rigorous plasticity solution. They use advanced upper and lower bound theorem to determine the ultimate bearing capacity. In their study the assumed the soil layer obeys an associated flow rule and range of parameters including depth of sand layer with width ratio (D/B), friction angle of sand, the undreained shear strength of clay, and the effect of surcharge are considered in their study. Their study compares these solutions obtained by lower and upper bound theories with other published results (J.S. Shiau, Bearing capacity of a sand layer on clay by finite element limit analysis, 2003).

ZENON SZYPCIO, KATARZYNA DOŁŻYK (2006) investigated ultimate bearing capacity of strip and square footing resting strong layer overlying weak layer and weak layer overlying strong layer. They uses finite element based software PLAXIS version 8 to analyze the ultimate bearing capacity and compared with various methods (ZENON SZYPCIO, 2006).

Vipin Chandra Joshi, Rakesh Kumar Dutta, and Rajnish Shrivastava (2015) investigated ultimate bearing capacity of circular footing on layered soil. They use punching shear failure mechanism followed bay projected area approach and presents theoretical equation for ultimate bearing capacity. The proposed bearing capacity equation has been derived as a function of upper and lower layer properties. Wide range of parametric study including Non-Dimensional parameter ($c/\gamma 1\ D'$), the friction angle of the sand layer (ϕ), Mobilised angle of shearing resistance (δ), Load spread angle (α) in sand layer, Non-Dimensional parameter (H/D') were carried out. The results of the parametric study were compared with the available equations in literature for the circular footing. Further, the results were validated with the experimental results reported in literature by other investigator (Vipin Chandra Joshi 1, 2015).

Table 2. 1Summary of review on some related literature

year	Authors	Authors findings
1953	Button	Analyzed bearing capacity of strip footing underlying two layers of clay soil. The bearing capacity factor used depends on the upper soil layer and on the ratio of the cohesions of the lower/upper clay layers.
1967	Reddy and Srinivasan	Extended the work of button to include the effect of non-homogeneity and anisotropy of the soil with respect of shear strength.
1969	Brown and Meyerhof	Investigated foundations resting on stiff clay layer overlying soft clay layer deposit and soft clay layer overlying stiff clay layer. From the obtained experimental result using empirical relations equations and charts were developed giving appropriate bearing capacity factors.
1974	Meyerhof	Investigated the case of sand layer overlying clay: dense sand on soft clay and loose sand on stiff clay. 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 and also loose sand overlying stiff clay
1978	Meyerhof and Hanna	Considered the case of strong layer overlying weak layer and weak layer overlaying strong layer. They developed theories to predict the bearing capacity of layered soil under vertical loading and inclined loading conditions.
1979	Hanna and Meyerhof	Extended their previous theory of bearing capacity of two layered soil in to three layered soil. Formulas and charts were developed and can be used in designing foundations having the same conditions.
1980	Hanna and Meyerhof	Extended their previous work to cover the case of were dense sand deposit overlying soft clay deposit and presents their result in the form of design chart.
1981	Hanna and Meyerhof	investigated experimentally the ultimate bearing capacity of footing subjected to axially inclined loads by conducting model tests on circular and strip footing 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.
1987	Hanna	Investigated the ultimate bearing capacity of strip footing in homogenous and layered sand soil experimentally on model strip footing and using finite element model using nonlinear stress-strain relationship to simulate testing conditions. The result obtained from the finite element model are in good agreement with the corresponding experimental values.
2003	J. S. Shiau, A. V. Lyamin, and S. W. Sloan	Investigated ultimate bearing capacity of strip footing resting on sand layer over clay using rigorous plasticity solution.

2006	ZENON SZYPCIO, KATARZYNA DOŁŻYK	Investigated ultimate bearing capacity of strip and square footing resting strong layer overlying weak layer and weak layer overlying strong layer. They uses finite element based software PLAXIS version 8 to analyze the ultimate bearing capacity and compared with various methods
2015	Vipin Chandra Joshi, Rakesh Kumar Dutta, and Rajnish Shrivastava	Investigated ultimate bearing capacity of circular footing on layered soil. They use punching shear failure mechanism followed bay projected area approach and presents theoretical equation for ultimate bearing capacity.

CHAPTER THREE

NUMERICAL MODELING

3.1 Introduction

In this chapter numerical modeling of different foundation type are modeled in layered soil profile. The soil parameter used in the analysis of bearing capacity are incorporated from reference material.

In the analysis vertical loading is only considered for the rectangular footing used for analysis and for the given load three type of soil layers were considered. In order to have good modeling it is necessary to carry out preliminary check that is sensitivity analysis such as mesh geometry, distance to the boundary to ensure the accuracy of numerical modeling.

3.2 Soil property

The soil property used in the analysis are obtained from reference material and an average values were taken and modeled using Mohr-Columb (MC) constitutive relation.

Table 3. 1 Soil parameter used for modeling taken from Joseph E. Bowles "foundation analysis and design fifth edition"

Parameter	Symbol	Unit	Soil type			
Soil type			Dense sand	Loose sand	Stiff clay	Soft clay
Model utilized	Model		Mohr columb	Mohr columb	Mohr columb	Mohr columb
Soil unit weight	γunsat	KN/m3	20	12	18	16
	γsat	KN/m3	22	14	20	19
Young modulus	Eref.	KN/m2	81000	10000	75000	5000
Poisson's ratio	ν	-	0.5	0.25	0.3	0.4
Cohesion	Cref.	KN/m2	1	1	200	15

Friction angle	ϕ	-	40	30	6	6
Diltancy angle	ψ	-	0	0	0	0

Note: the soil parameter are taken from Bowles.

Table 3. 2 Structural parameter from Joseph E. Bowles "foundation analysis and design fifth edition"

Footing type	Footing dimension	L/B ratio	Thickness d(m)	Density of material γ (KN/m2)	Modulus of elasticity E (N/m2)	Poisson's Ratio. v	Shear Modulus, G (N/m2)
Square	2m x2m	1	0.4	24	30000000	0.2	12500000
Rectangular	2m x3m	1.5	0.4	24	30000000	0.2	12500000
Rectangular	2m x4m	2	0.4	24	30000000	0.2	12500000

3.3 Geometric model

The soil stratum for the structural system are listed below

Case 1

Layers	Soil type
Layer 1	Dense sand
Layer 2	Soft clay

Case 2

Layers	Soil type
Layer 1	Loose sand
Layer 2	Dense sand

Case 3

Layers	Soil type
Layer 1	Dense sand
Layer 2	Stiff clay
Layer 3	Soft clay

Table 3. 3 Models used in PLAXIS 3D foundation for bearing capacity analysis

Model No	Soil type used	H/B	L/B	spacing between footing	Consideration of Effect of ground water
1	Dense sand/ soft clay	0.5	1	B	No
1b	Dense sand/ soft clay	0.5	1	B	Yes

2	Dense sand/ soft clay	1	1	B	No
2b	Dense sand/ soft clay	1	1	B	Yes
3	Dense sand/ soft clay	2	1	B	No
3b	Dense sand/ soft clay	2	1	B	Yes
4	Dense sand/ soft clay	0.5	1.5	B	No
4b	Dense sand/ soft clay	0.5	1.5	B	Yes
5	Dense sand/ soft clay	1	1.5	B	No
5b	Dense sand/ soft clay	1	1.5	B	Yes
6	Dense sand/ soft clay	2	1.5	B	No
6b	Dense sand/ soft clay	2	1.5	B	Yes
7	Dense sand/ soft clay	0.5	2	B	No
7b	Dense sand/ soft clay	0.5	2	B	Yes
8	Dense sand/ soft clay	1	2	B	No
8b	Dense sand/ soft clay	1	2	B	Yes
9	Dense sand/ soft clay	2	2	B	No
9b	Dense sand/ soft clay	2	2	B	Yes
10	Dense sand/ soft clay	0.5	1	1.5B	No
10b	Dense sand/ soft clay	0.5	1	1.5B	Yes
11	Dense sand/ soft clay	1	1	1.5B	No
11b	Dense sand/ soft clay	1	1	1.5B	Yes
12	Dense sand/ soft clay	2	1	1.5B	No
12b	Dense sand/ soft clay	2	1	1.5B	Yes
13	Dense sand/ soft clay	0.5	1.5	1.5B	No
13b	Dense sand/ soft clay	0.5	1.5	1.5B	Yes
14	Dense sand/ soft clay	1	1.5	1.5B	No
14b	Dense sand/ soft clay	1	1.5	1.5B	Yes
15	Dense sand/ soft clay	2	1.5	1.5B	No
15b	Dense sand/ soft clay	2	1.5	1.5B	Yes
16	Dense sand/ soft clay	0.5	2	1.5B	No
16b	Dense sand/ soft clay	0.5	2	1.5B	Yes

17	Dense sand/ soft clay	1	2	1.5B	No
17b	Dense sand/ soft clay	1	2	1.5B	Yes
18	Dense sand/ soft clay	2	2	1.5B	No
18b	Dense sand/ soft clay	2	2	1.5B	Yes
19	Dense sand/ soft clay	0.5	1	2B	No
19b	Dense sand/ soft clay	0.5	1	2B	Yes
20	Dense sand/ soft clay	1	1	2B	No
20b	Dense sand/ soft clay	1	1	2B	Yes
21	Dense sand/ soft clay	2	1	2B	No
21b	Dense sand/ soft clay	2	1	2B	Yes
22	Dense sand/ soft clay	0.5	1.5	2B	No
22b	Dense sand/ soft clay	0.5	1.5	2B	Yes
23	Dense sand/ soft clay	1	1.5	2B	No
23b	Dense sand/ soft clay	1	1.5	2B	Yes
24	Dense sand/ soft clay	2	1.5	2B	No
24b	Dense sand/ soft clay	2	1.5	2B	Yes
25	Dense sand/ soft clay	0.5	2	2B	No
25b	Dense sand/ soft clay	0.5	2	2B	Yes
26	Dense sand/ soft clay	1	2	2B	No
26b	Dense sand/ soft clay	1	2	2B	Yes
27	Dense sand/ soft clay	2	2	2B	No
27b	Dense sand/ soft clay	2	2	2B	Yes
28	Loose sand/dense sand	0.5	1	B	No
28b	Loose sand/dense sand	0.5	1	B	Yes
29	Loose sand/dense sand	1	1	B	No
29b	Loose sand/dense sand	1	1	B	Yes
30	Loose sand/dense sand	2	1	B	No
30b	Loose sand/dense sand	2	1	B	Yes
31	Loose sand/dense sand	0.5	1.5	B	No
31b	Loose sand/dense sand	0.5	1.5	B	Yes

32	Loose sand/dense sand	1	1.5	B	No
32b	Loose sand/dense sand	1	1.5	B	Yes
33	Loose sand/dense sand	2	1.5	B	No
33b	Loose sand/dense sand	2	1.5	B	Yes
34	Loose sand/dense sand	0.5	2	B	No
34b	Loose sand/dense sand	0.5	2	B	Yes
35	Loose sand/dense sand	1	2	B	No
35b	Loose sand/dense sand	1	2	B	Yes
36	Loose sand/dense sand	2	2	B	No
36b	Loose sand/dense sand	2	2	B	Yes
37	Loose sand/dense sand	0.5	1	1.5B	No
37b	Loose sand/dense sand	0.5	1	1.5B	Yes
38	Loose sand/dense sand	1	1	1.5B	No
38b	Loose sand/dense sand	1	1	1.5B	Yes
39	Loose sand/dense sand	2	1	1.5B	No
39b	Loose sand/dense sand	2	1	1.5B	Yes
40	Loose sand/dense sand	0.5	1.5	1.5B	No
40b	Loose sand/dense sand	0.5	1.5	1.5B	Yes
41	Loose sand/dense sand	1	1.5	1.5B	No
41b	Loose sand/dense sand	1	1.5	1.5B	Yes
42	Loose sand/dense sand	2	1.5	1.5B	No
42b	Loose sand/dense sand	2	1.5	1.5B	Yes
43	Loose sand/dense sand	0.5	2	1.5B	No
43b	Loose sand/dense sand	0.5	2	1.5B	Yes
44	Loose sand/dense sand	1	2	1.5B	No
44b	Loose sand/dense sand	1	2	1.5B	Yes
45	Loose sand/dense sand	2	2	1.5B	No
45b	Loose sand/dense sand	2	2	1.5B	Yes
46	Loose sand/dense sand	0.5	1	2B	No
46b	Loose sand/dense sand	0.5	1	2B	Yes

47	Loose sand/dense sand	1	1	2B	No
47b	Loose sand/dense sand	1	1	2B	Yes
48	Loose sand/dense sand	2	1	2B	No
48b	Loose sand/dense sand	2	1	2B	Yes
49	Loose sand/dense sand	0.5	1.5	2B	No
49b	Loose sand/dense sand	0.5	1.5	2B	Yes
50	Loose sand/dense sand	1	1.5	2B	No
50b	Loose sand/dense sand	1	1.5	2B	Yes
51	Loose sand/dense sand	2	1.5	2B	No
51b	Loose sand/dense sand	2	1.5	2B	Yes
52	Loose sand/dense sand	0.5	2	2B	No
52b	Loose sand/dense sand	0.5	2	2B	Yes
53	Loose sand/dense sand	1	2	2B	No
53b	Loose sand/dense sand	1	2	2B	Yes
54	Loose sand/dense sand	2	2	2B	No
54b	Loose sand/dense sand	2	2	2B	Yes

Model No	soil type used	H1/B, H2/B	L/B	spacing between footing	Consideration of Effect of ground water
55	dense sand/stiff clay/soft clay	0.5, 0.5	1	B	No
55b	dense sand/stiff clay/soft clay	0.5, 0.5	1	B	Yes
56	dense sand/stiff clay/soft clay	1, 0.5	1	B	No
56b	dense sand/stiff clay/soft clay	1, 0.5	1	B	Yes
57	dense sand/stiff clay/soft clay	1, 1	1	B	No
57b	dense sand/stiff clay/soft clay	1, 1	1	B	Yes
58	dense sand/stiff clay/soft clay	0.5, 0.5	1.5	B	No
58b	dense sand/stiff clay/soft clay	0.5 ,0.5	1.5	B	Yes

59	dense sand/stiff clay/soft clay	1, 0.5	1.5	B	No
59b	dense sand/stiff clay/soft clay	1, 0.5	1.5	B	Yes
60	dense sand/stiff clay/soft clay	1, 1	1.5	B	No
60b	dense sand/stiff clay/soft clay	1, 1	1.5	B	Yes
61	dense sand/stiff clay/soft clay	0.5, 0.5	2	B	No
61b	dense sand/stiff clay/soft clay	0.5, 0.5	2	B	Yes
62	dense sand/stiff clay/soft clay	1, 0.5	2	B	No
62b	dense sand/stiff clay/soft clay	1, 0.5	2	B	Yes
63	dense sand/stiff clay/soft clay	1, 1	2	B	No
63b	dense sand/stiff clay/soft clay	1, 1	2	B	Yes
64	dense sand/stiff clay/soft clay	0.5, 0.5	1	1.5B	No
64b	dense sand/stiff clay/soft clay	0.5, 0.5	1	1.5B	Yes
65	dense sand/stiff clay/soft clay	1, 0.5	1	1.5B	No
65b	dense sand/stiff clay/soft clay	1, 0.5	1	1.5B	Yes
66	dense sand/stiff clay/soft clay	1, 1	1	1.5B	No
66b	dense sand/stiff clay/soft clay	1, 1	1	1.5B	Yes
67	dense sand/stiff clay/soft clay	0.5, 0.5	1.5	1.5B	No
67b	dense sand/stiff clay/soft clay	0.5, 0.5	1.5	1.5B	Yes
68	dense sand/stiff clay/soft clay	1, 0.5	1.5	1.5B	No
68b	dense sand/stiff clay/soft clay	1, 0.5	1.5	1.5B	Yes
69	dense sand/stiff clay/soft clay	1, 1	1.5	1.5B	No
69b	dense sand/stiff clay/soft clay	1, 1	1.5	1.5B	Yes
70	dense sand/stiff clay/soft clay	0.5, 0.5	2	1.5B	No

70b	dense sand/stiff clay/soft clay	0.5, 0.5	2	1.5B	Yes
71	dense sand/stiff clay/soft clay	1, 0.5	2	1.5B	No
71b	dense sand/stiff clay/soft clay	1, 0.5	2	1.5B	Yes
72	dense sand/stiff clay/soft clay	1, 1	2	1.5B	No
72b	dense sand/stiff clay/soft clay	1, 1	2	1.5B	Yes
73	dense sand/stiff clay/soft clay	0.5, 0.5	1	2B	No
73b	dense sand/stiff clay/soft clay	0.5, 0.5	1	2B	Yes
74	dense sand/stiff clay/soft clay	1, 0.5	1	2B	No
74b	dense sand/stiff clay/soft clay	1, 0.5	1	2B	Yes
75	dense sand/stiff clay/soft clay	1, 1	1	2B	No
75b	dense sand/stiff clay/soft clay	1, 1	1	2B	Yes
76	dense sand/stiff clay/soft clay	0.5, 0.5	1.5	2B	No
76b	dense sand/stiff clay/soft clay	0.5, 0.5	1.5	2B	Yes
77	dense sand/stiff clay/soft clay	1, 0.5	1.5	2B	No
77b	dense sand/stiff clay/soft clay	1, 0.5	1.5	2B	Yes
78	dense sand/stiff clay/soft clay	1, 1	1.5	2B	No
78b	dense sand/stiff clay/soft clay	1, 1	1.5	2B	Yes
79	dense sand/stiff clay/soft clay	0.5, 0.5	2	2B	No
79b	dense sand/stiff clay/soft clay	0.5, 0.5	2	2B	Yes
80	dense sand/stiff clay/soft clay	1, 0.5	2	2B	No
80b	dense sand/stiff clay/soft clay	1, 0.5	2	2B	Yes
81	dense sand/stiff clay/soft clay	1, 1	2	2B	No
81b	dense sand/stiff clay/soft clay	1, 1	2	2B	Yes

3.4. Preliminary check to ensure the accuracy of models used in numerical analysis

The accuracy of the finite element model depends not only on the level of sophistication of the material model, but also on the boundary condition of the domain, on the number and type of the elements in to which the domain is discretized and other modeling issues

3.4.1. Influence of distance to the boundary

External boundaries are an artificial representative of real forces, extensions and conditions that define the solution of a finite element model. It is not possible to include real natural extension of a mass of soil or some events applied on it, so finite element code (PLAXIS 3D Foundation) enables the user to substitute the reaction of these extensions and events as a restraint, displacements and forces at the boundary. Consequently, the user can quantify these effects and assign them to the studied model with minimum effect on the accuracy of this model. Positions of boundary restraints can significantly affect finite element simulation results. Since the generated reaction forces and displacement in these boundaries can influence the impact of the applied forces on the zones of interest. So to avoid any restriction that reflected on the accuracy of the finite element model, a user should select the location of this boundaries to be sufficient distant from any zone of interest, but at the same time the user should consider them not to be exceedingly far, costing more time in the analyzing process

Practically the sensitivity analysis was carried out on the model for only the side boundary. Bottom boundary is already considered as a natural boundary as it was stiff enough at 8m deep. The location of distance to the boundary from the footing center ranged from 2B-12B.

The sensitivity analysis was conducted on a selected two 2m square footing with 4m spacing between footings. The vertical displacement (*uy*) was measured at the points A, B, C, D, E, F, G, and H. Point A and B are at the point of load application of footing 1 and 2 respectively, C, D are at the base of the footing 1 and 2 respectively, E and F are 1m below the base of the footing 1 and 2, and G and H are points 2m below the base of footing 1 and 2 respectively. The soil profile adopted for the subsequent FEA.

The accuracy of relevant boundary distance is determined by comparing the normalized error for vertical displacement (uy) at a selected point where the side boundary is located at a distance xB against that of 12B.

Normalized error for vertical displacement, Uy=$\frac{UxB-U12B}{U12B}$

Where

Uy (xB) = is the vertical displacement at selected point for the case of side boundary located at xB from the center of footing pad.

Uy (12B) = is the vertical displacement at selected point for the case of side boundary located at 12B from the center of the footing pad.

The footings were loaded to 400kpa and their vertical displacement (Uy) and normalized error for different boundary distances are listed below.

Table 3. 4 Influence of the distance to boundary upon the settlement of two 2 m square footing

Distance to boundary	Vertical displacement (Uy)								Normalized error (%)							
	A	B	C	D	E	F	G	H	A	B	C	D	E	F	G	H
2B	25.1	24.9	24.8	24.5	23.8	23.5	20.4	20.2	19.2	22	19	22	19.2	21.8	22	24
4.5B	28.4	29.2	28.1	28.9	27	27.8	23	23.6	8.7	7.8	8.8	7.8	8.5	7.6	12	11
6.5B	29	33.4	28.7	33.1	27.4	31.8	24.3	28.1	6.6	5.5	6.7	5.5	7.04	5.8	6.9	5.7
8.5B	32	31.9	30.7	31.6	29.6	30.6	26.4	26.7	1.8	0.6	0.4	0.6	0.32	1.5	1	0.7
12B	31.1	31.7	30.8	31.4	29.5	30.1	26.1	26.6	0	0	0	0	0	0	0	0

It appears that positioning the boundary closer than 8.5B to the center of the footing induces more settlement in the pad footing. The distance to the boundary from the center of the footing is conservatively chosen at 8.5B for all the subsequent FEA.

3.4.2. Mesh sensitivity analysis

The influence of the number of elements upon the accuracy of the finite element model was investigated by conducting a mesh sensitivity analysis. Mesh sensitivity analysis was conducted for a reasonably selected two 2m x 2m footings with 4m spacing between them three. Where the accuracy of coarse and medium meshes are compared against the fine meshes.

The vertical displacement (*uy*) was examined for the mesh sensitivity analysis. *uy* was measured at the points A, B, C, D, E, F, G, and H. Point A and B are at the point of load application of footing 1 and 2 respectively, C, D are at the base of the footing 1 and 2 respectively, E and F are 1m below the base of the footing 1 and 2, and G and H are points 2m below the base of footing 1 and 2 respectively. The accuracy of coarse and medium meshes is determined by comparing the normalized error for vertical displacement (uy) against fine meshes.

Normalized error for vertical displacement, $uy = \left(\frac{Uy,f - Uy}{Uy,f} \right)$

Where

Uy= vertical displacement at the selected point for the coarse and medium meshed elements.

Uy,f = is vertical displacement at the selected point for fine meshed elements.

Table 3. 5 Vertical displacement at points A, B, C, D, E, F, G, and H

Mesh	No of mesh element	Vertical displacement (Uy)								Normalized error (%)							
		A	B	C	D	E	F	G	H	A	B	C	D	E	F	G	H
Coarse	1910	26.8	26.1	26.4	25.8	24.9	24	23	21.9	7.6	9.6	8	9.7	7.7	11	6.1	9.1
Medium	1520	28.5	28.5	28.1	28.2	26.8	27	24	24	1.4	1.2	1	1.2	0.5	1	0.3	0.3
Fine	3408	29	28.9	28.7	28.6	26.9	27	24	24	0	0	0	0	0	0	0	0

The number of elements used for each mesh and the normalized error for the vertical displacement is shown in above. It is clear that the values for medium meshes are converging towards fine mesh and also the normalized error between medium and fine meshes for the vertical displacement is quite low (maximum error is not greater than 5%).

3.5. Model generation using PLAXIS 3D foundation

In this study an attempt has been made to analysis bearing capacity of shallow foundation resting on layered soil profile using numerical modeling and analyze using finite element software package PLAXIS 3D FOUNDATION. The following figures show input procedures, mesh generation and output curves of a PLAXIS model involved in the present study.

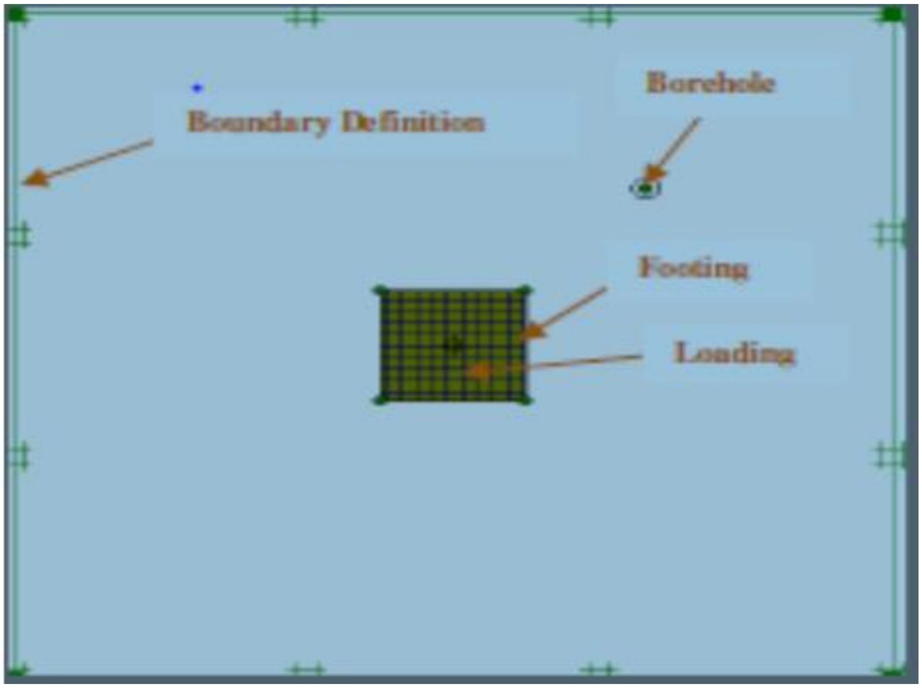
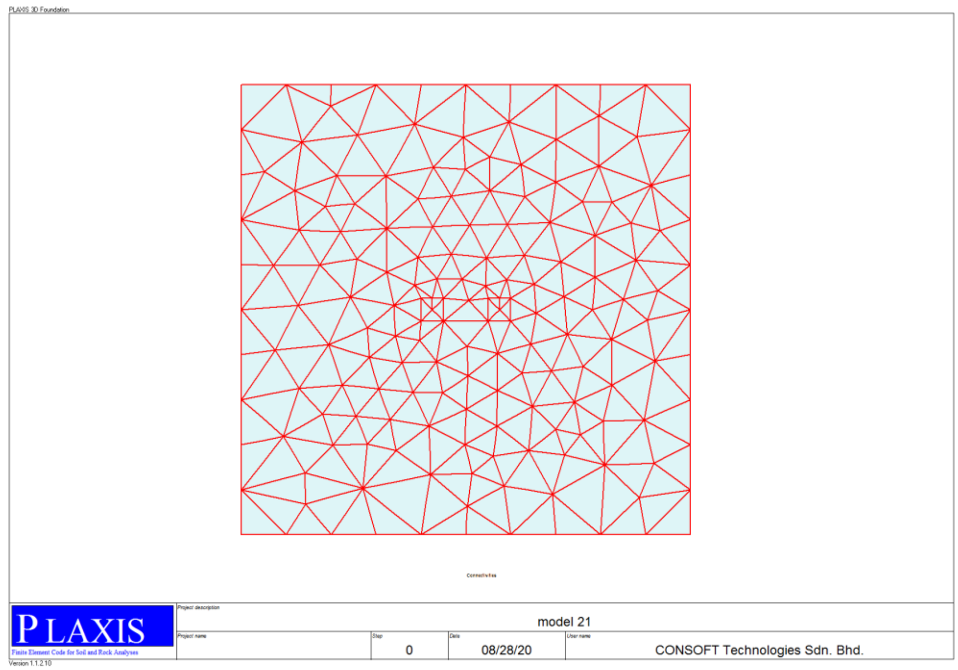


Figure 3. 15 PLAXIS 3D Input window



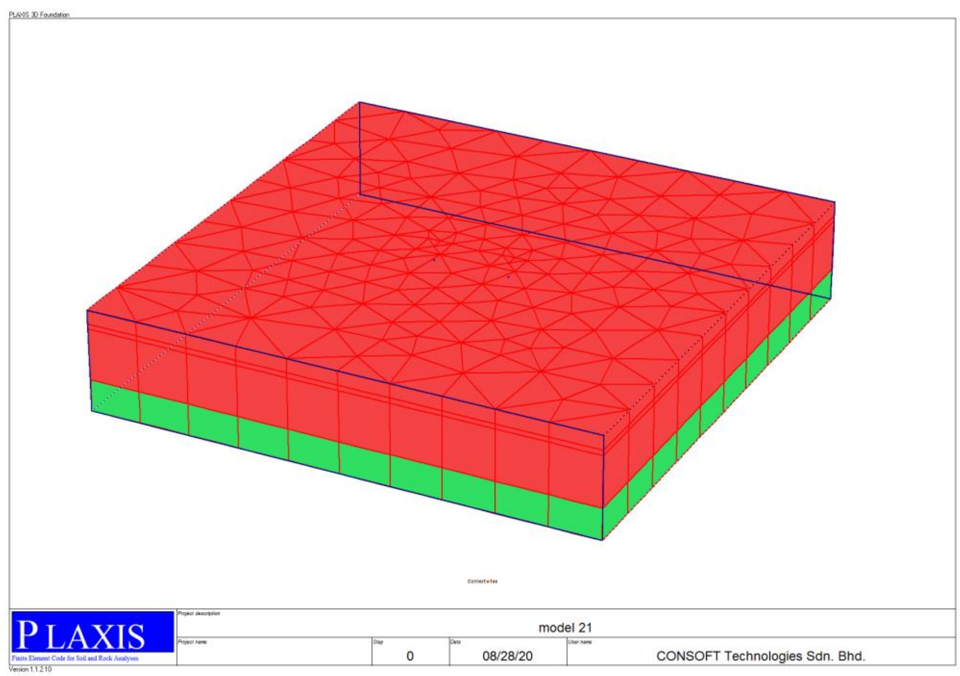
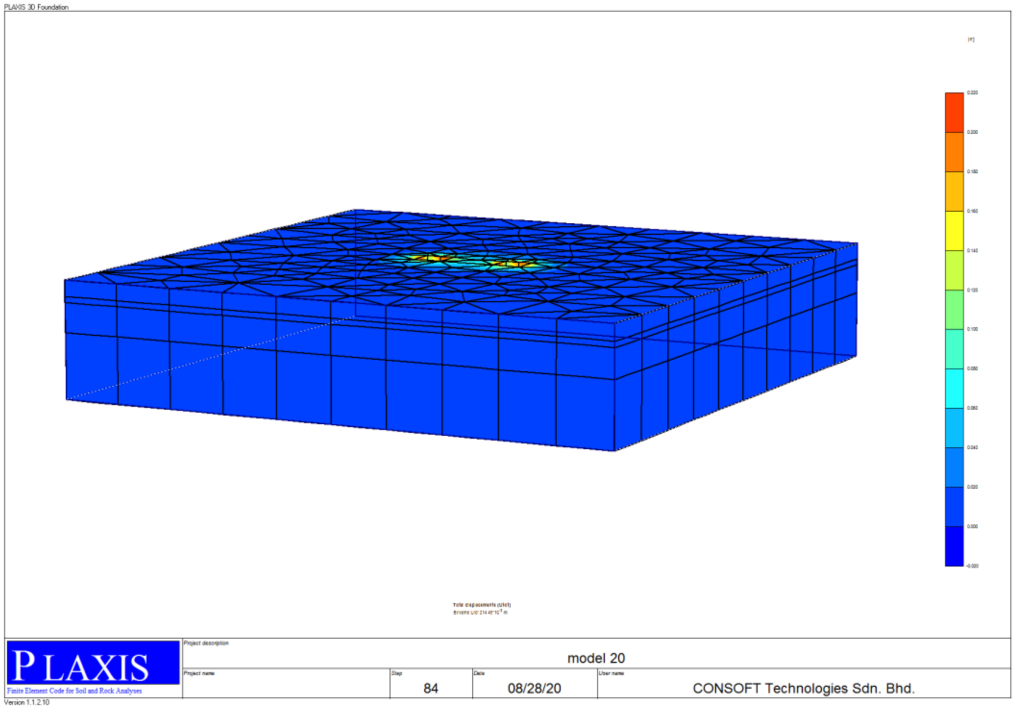


Figure 3. 16 2D and 3D mesh generation in PLAXIS 3D



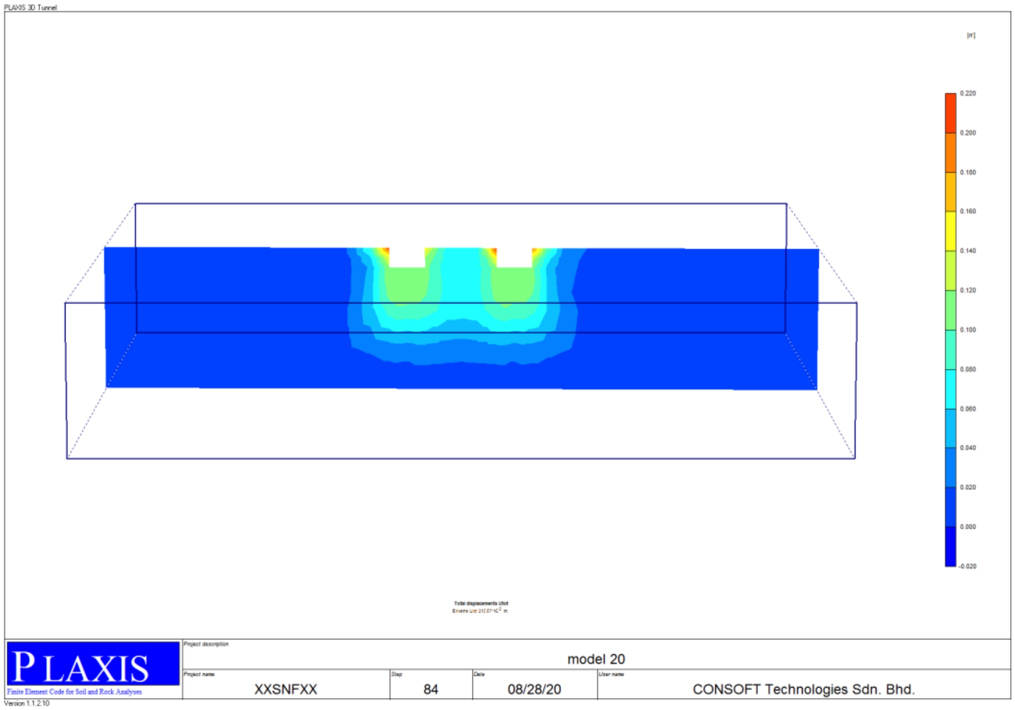


Figure 3. 17 Settlement in PLAXIS 3d

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. The ultimate bearing capacity is determined at maximum (50mm) settlement as stated in EBCS7 (EUROPEAN STANDARD ETHIOPIAN ANNEX, 2013). Such pressure-settlement curve and its tabular presentation are given below.

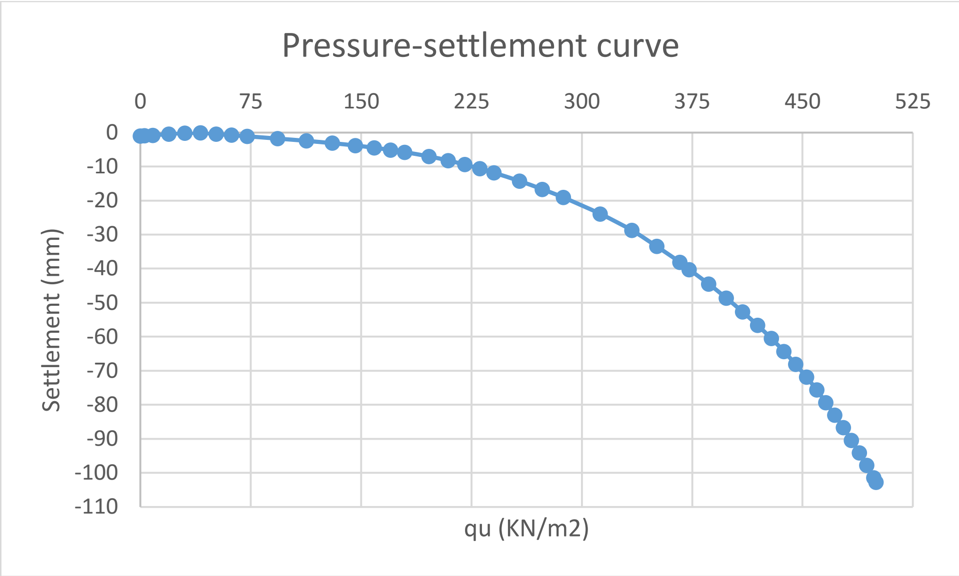


Figure 3. 18 Pressure settlement plot

3.6. 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, ground water effect, spacing between footing, ration of upper layer thickness to width of the footing etc. on ultimate bearing capacity of shallow footings total 162 finite element modeling has been carried out in PLAXIS 3D Foundation some of those model results are given below:

3.6.1 Extreme settlement

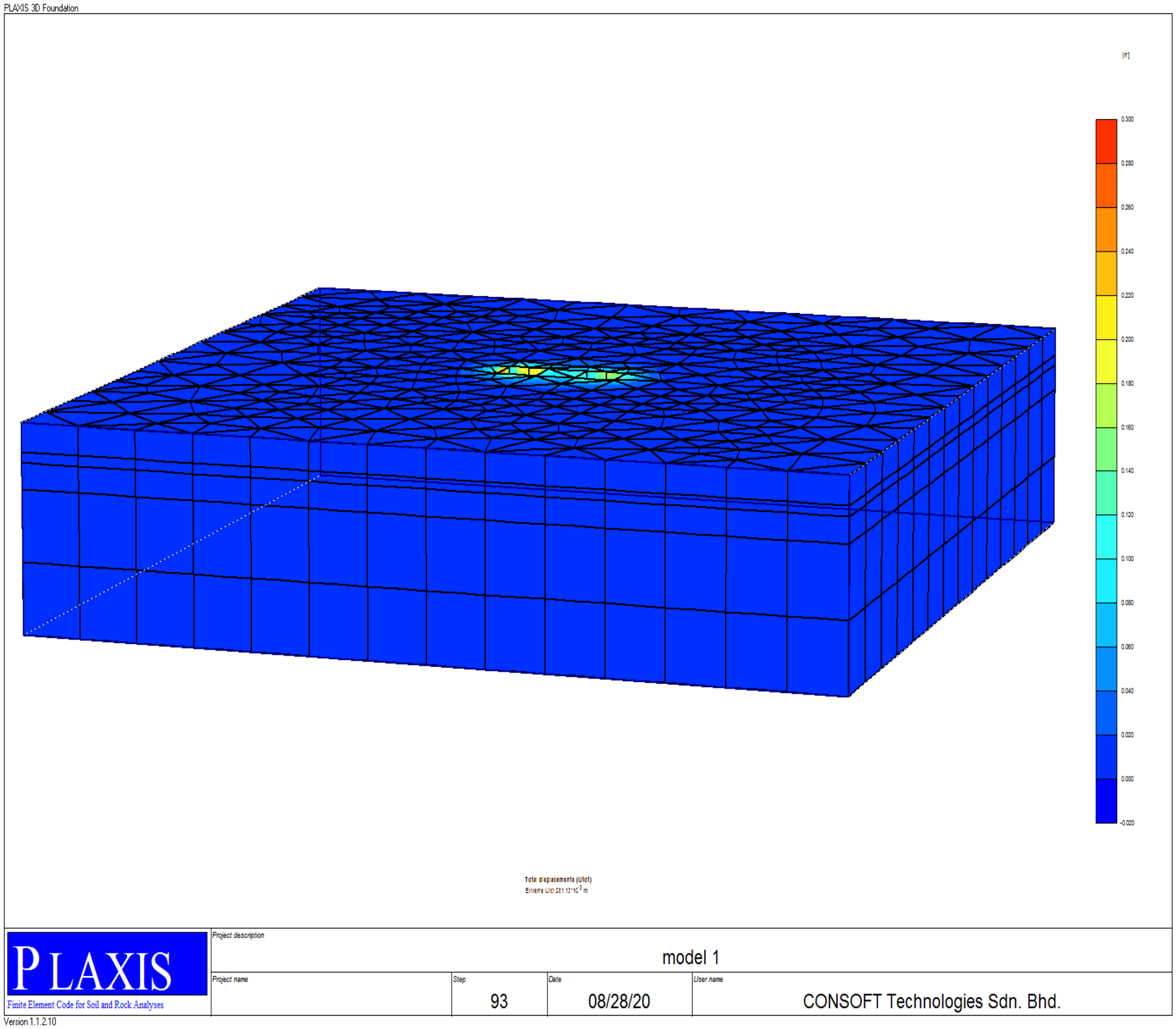


Figure 3. 19 extreme settlement for Model 1 dense sand overlying soft clay with thickness of upper layer to width ratio of 0.5, length to width ratio of 1 and spacing between two footings equal to the width of the footing.

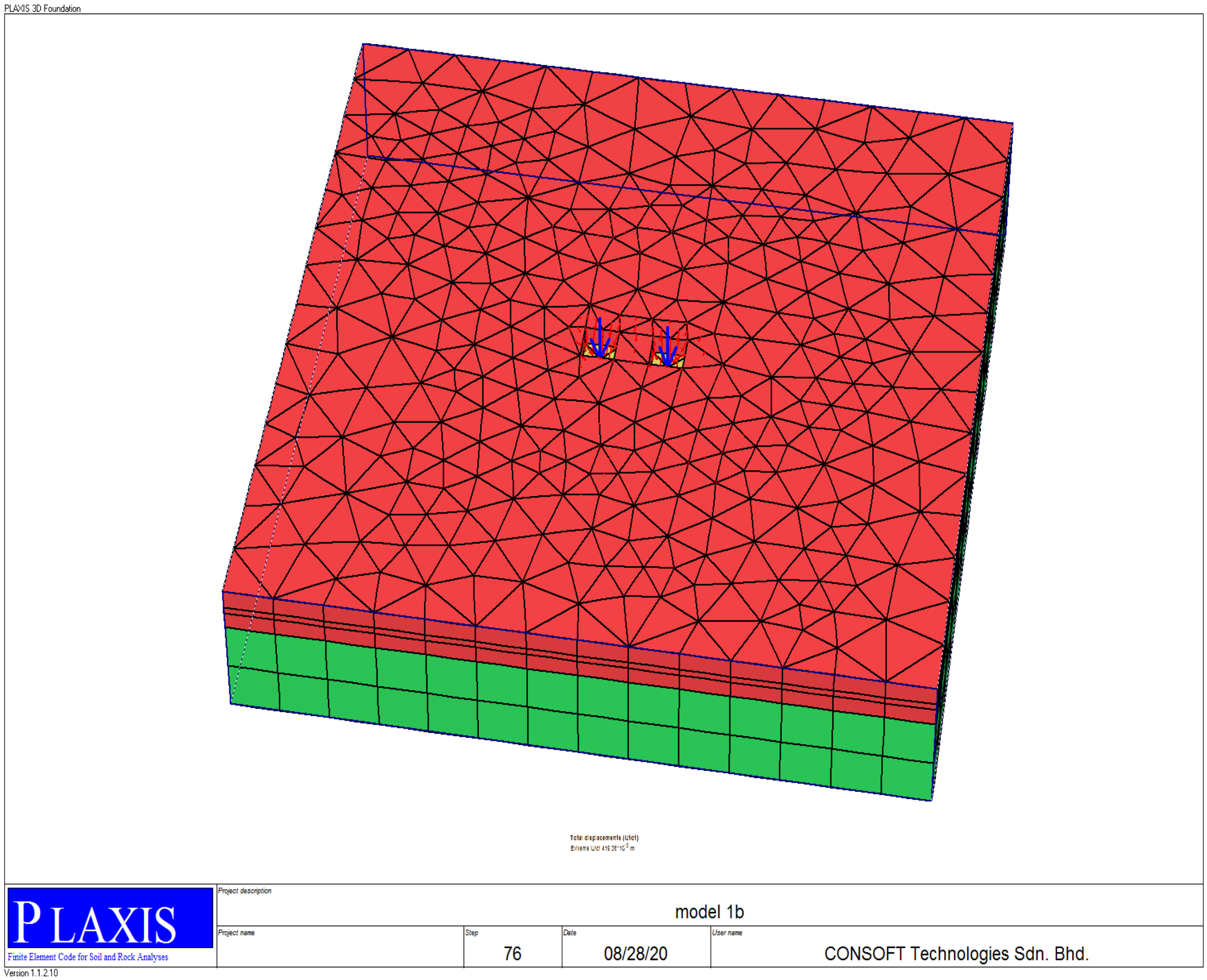


Figure 3. 20 extreme settlement for Model 1b dense sand overlying soft clay with thickness of upper layer to width ratio of 0.5, length to width ratio of 1 and spacing between two footings equal to the width of the footing and also considering the effect of ground.

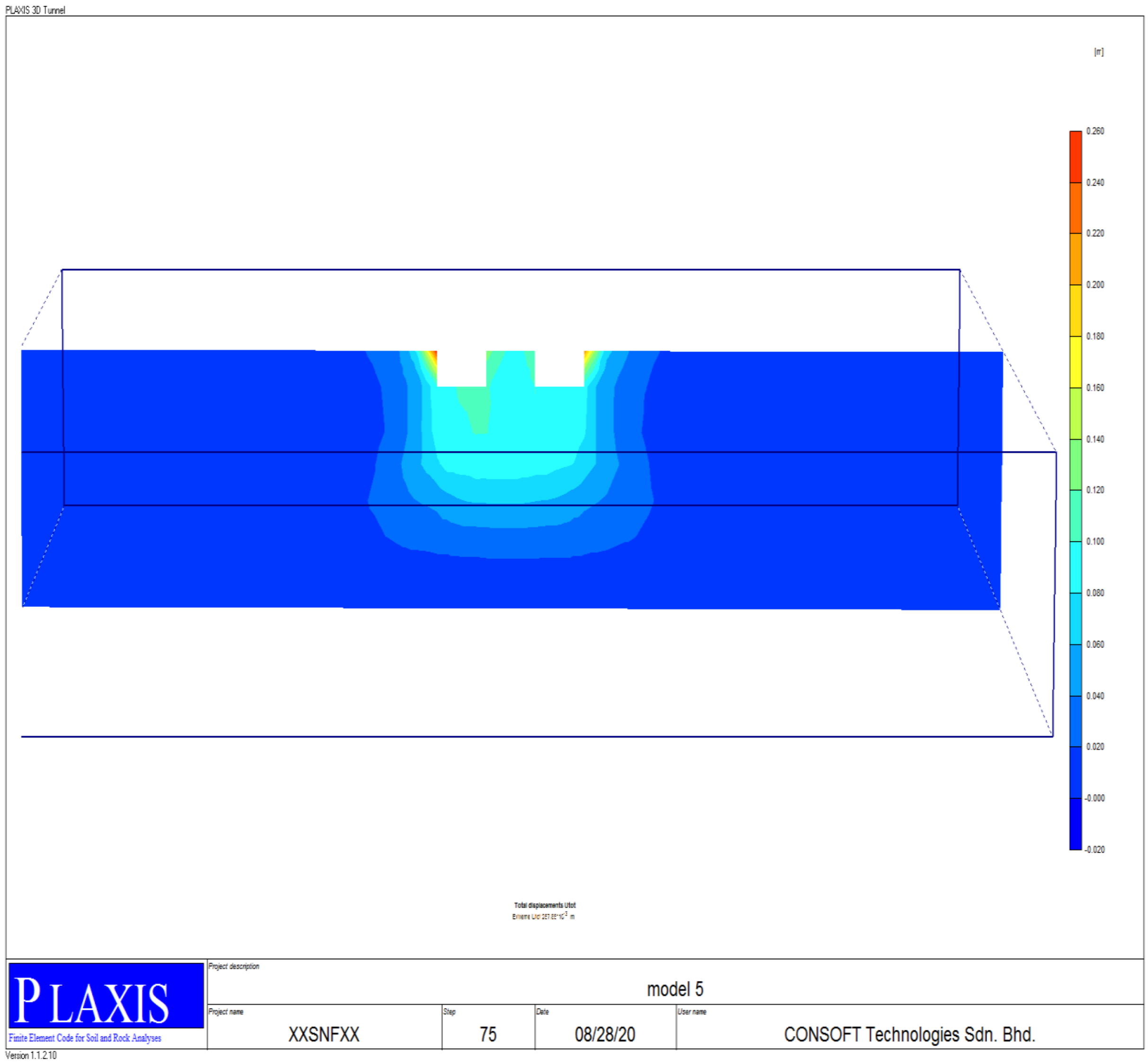


Figure 3. 21 extreme settlement for Model 5 dense sand overlying soft clay with thickness of upper layer to width ratio of 1, length to width ratio of 1.5 and spacing between two footings equal to the width of the footing.

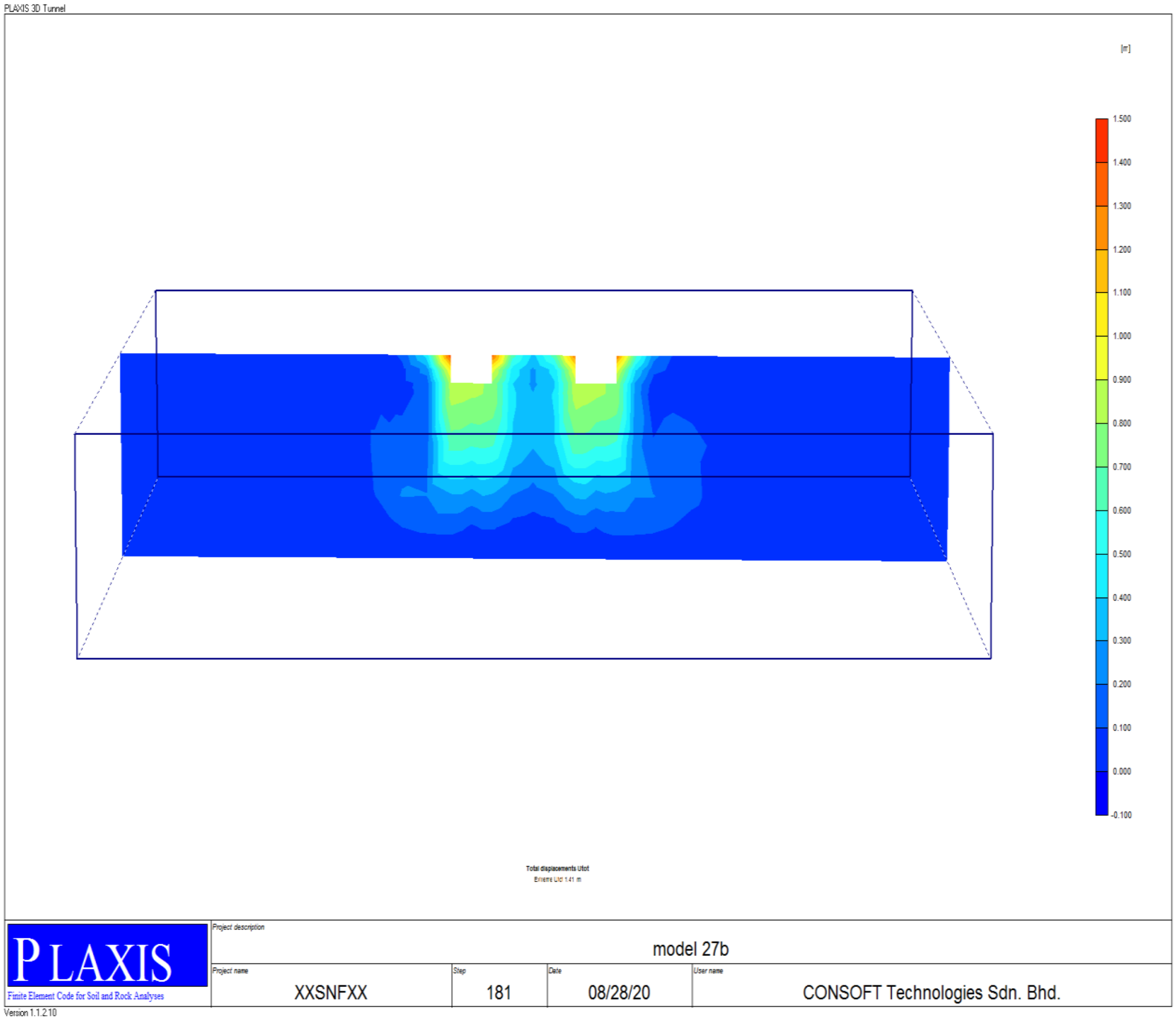


Figure 3. 22 settlement for Model 27b dense sand overlying soft clay with thickness of upper layer to width ratio of 2, length to width ratio of 2 and spacing between two footings equal to two times the width of the footing and also considering the effect of GWT

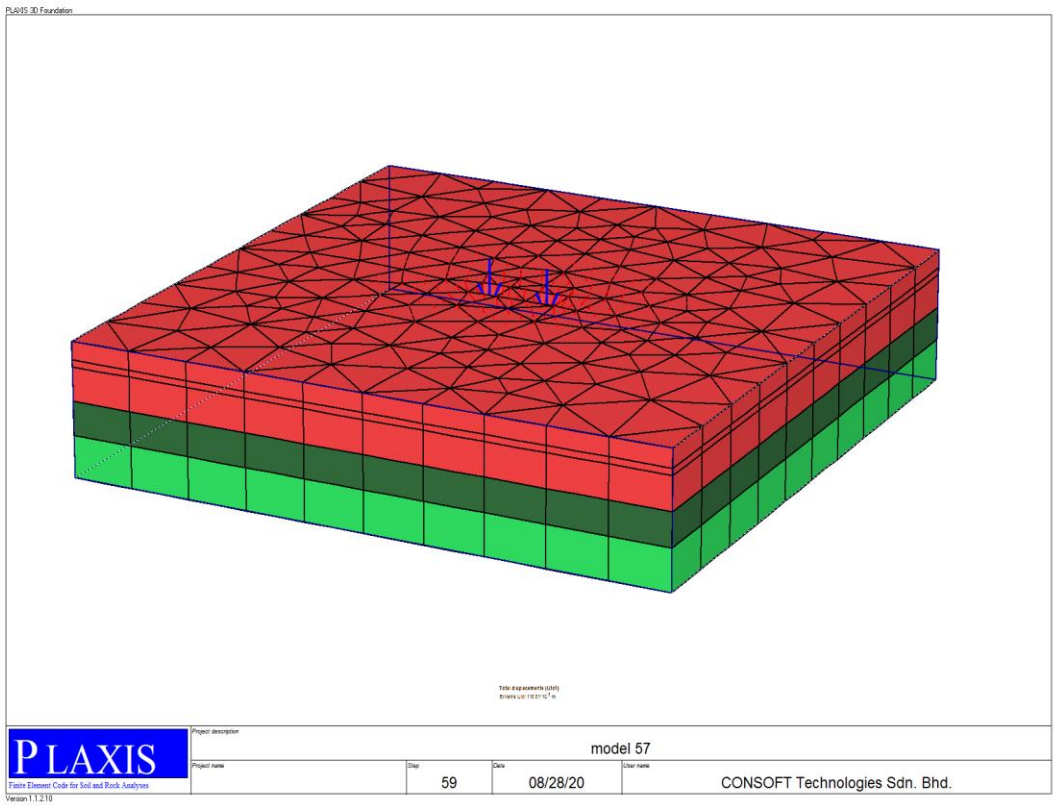


Figure 3. 23 Model 57 dense sand/ stiff clay/ loose sand with thickness of dense sand layer to width ratio of 1, stiff clay layer to width ratio of 1, length to width ratio of 1 and spacing between two footings equal to width of the footing.

3.6.2. Pressure-settlement plot

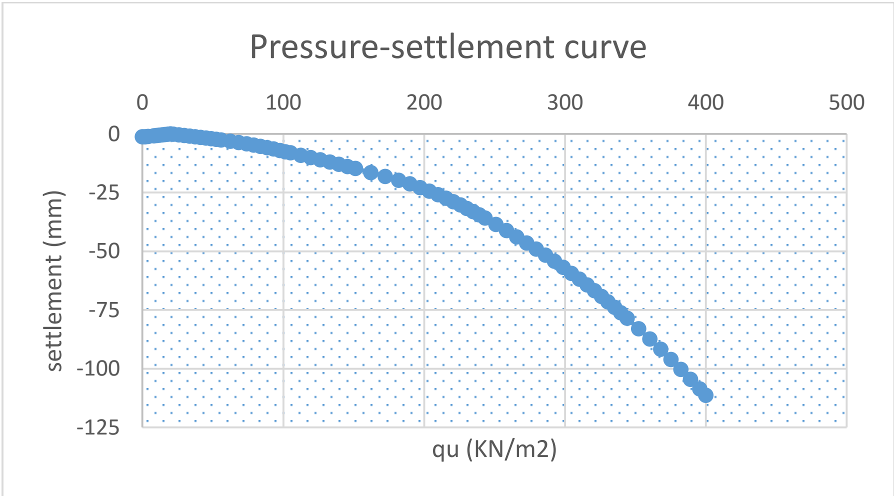


Figure 3. 24 load -settlement curve for model 1

Model 1, L/B=1, h/B=0.5, and spacing between footing = B, the bearing capacity is =289kpa for maximum settlement of 50mm as per EBCS 7.

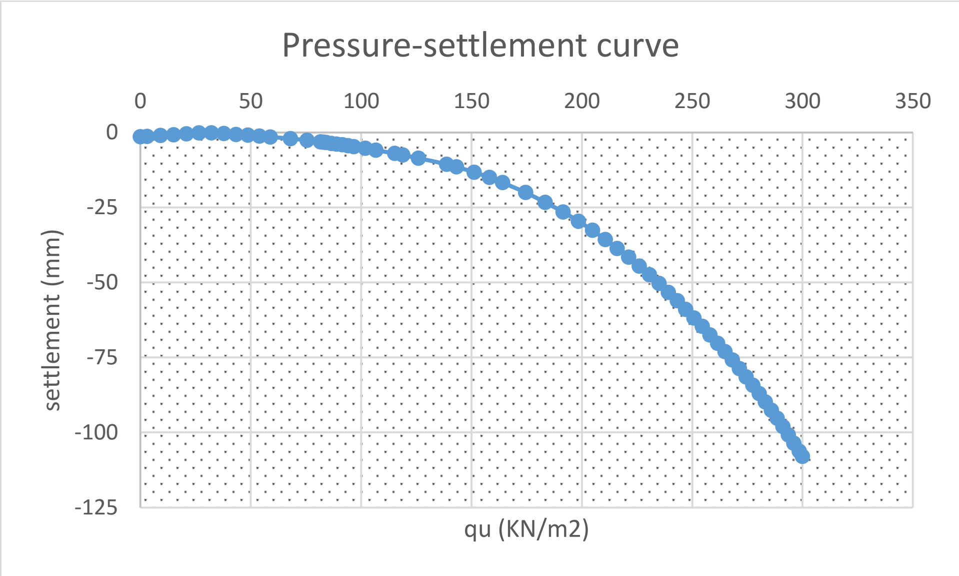


Figure 3. 25 load -settlement curve for model 1b

Model 1b, L/B=1, h/B=0.5, spacing between footing = B, and also considering the effect ground water the bearing capacity is =227kpa for maximum settlement of 50mm as per EBCS 7.

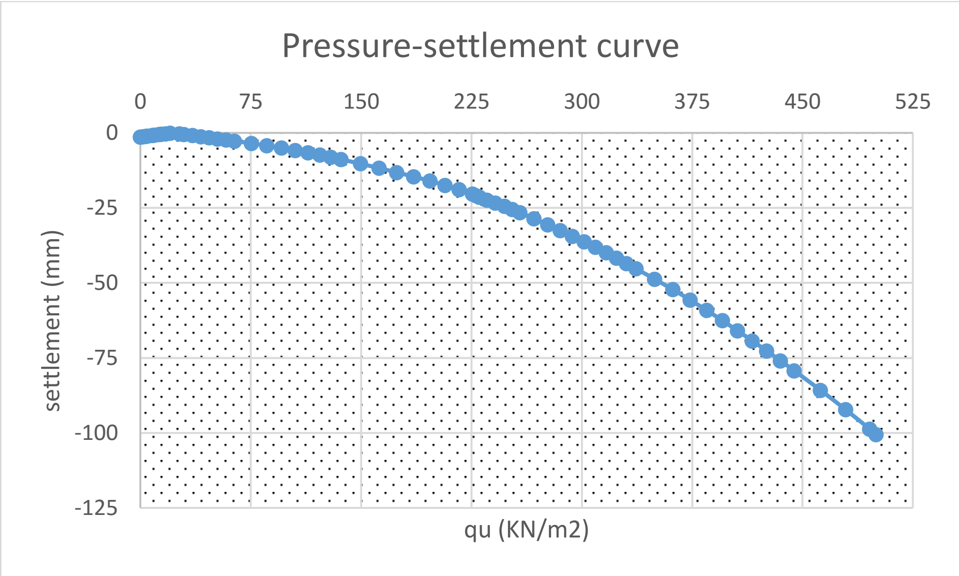


Figure 3. 26 load -settlement curve for model 5

Model 5, L/B=1.5, h/B=1, and spacing between footing = B, the bearing capacity is =356kpa for maximum settlement of 50mm as per EBCS 7.

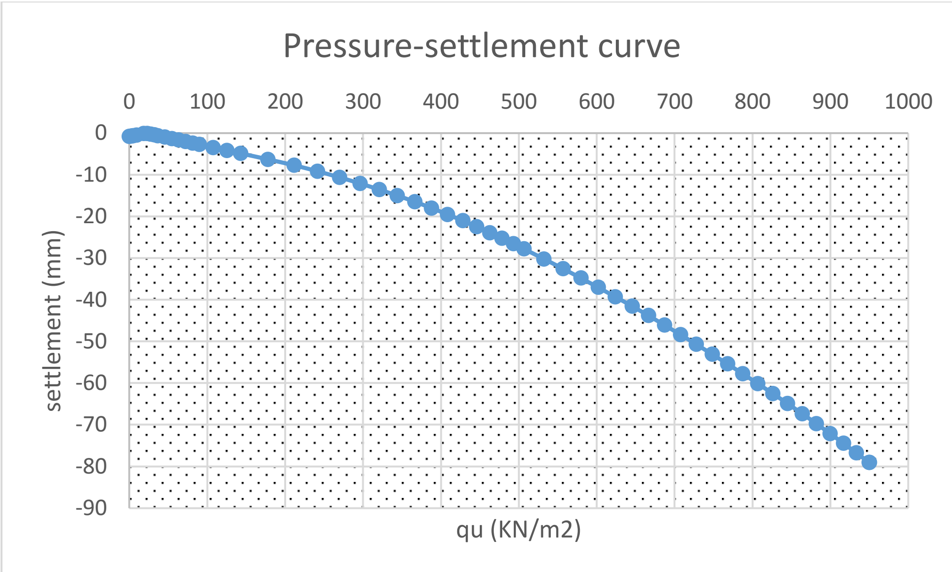


Figure 3. 27 load -settlement curve for model 27

Model 27, L/B=2, h/B=2, and spacing between footing = 2B, the bearing capacity is =720kpa for maximum settlement of 50mm as per EBCS 7.

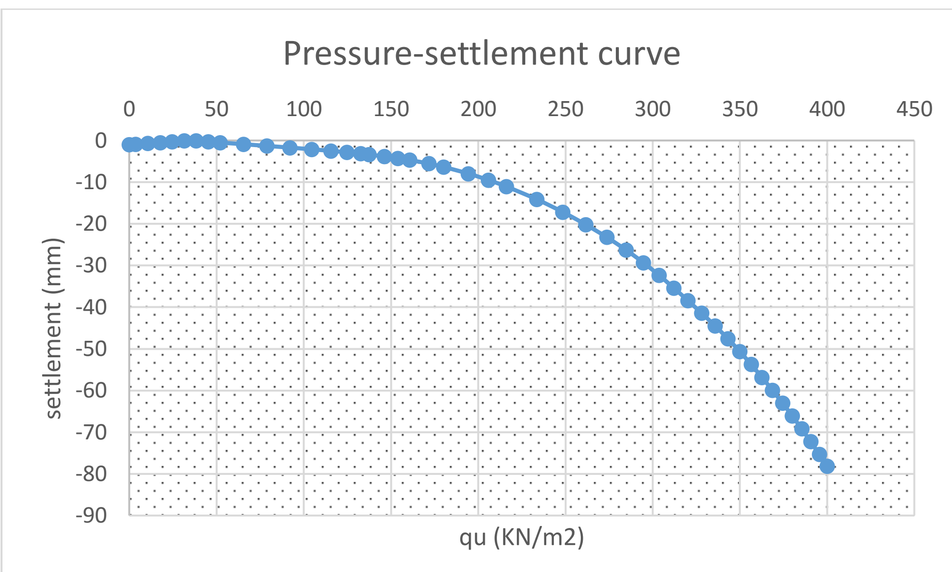


Figure 3. 28 load -settlement curve for model 59b

Model 59b, L/B=1.5, h/B=1, spacing between footing = B, and also considering the effect ground water the bearing capacity is =349kpa for maximum settlement of 50mm as per EBCS 7.

3.7. Empirical method for calculating bearing capacity

A total of 51 calculation has been done for comparison purpose. The calculation has been done in modified Hanna and Meyerhof method. [21] Sample calculations for two layered, three layered and are presented below.

i. For two layered soil

- For dense sand overlying soft clay

$q_u = q_b$ + (1+ B/L) $(2\frac{C_a H}{B})\lambda_a$ + (1+B/L) $\gamma_1 H^2(1 + 2\frac{2D_f}{H})$ $(\frac{K_s \tan\Phi_1}{B})$ λ_s - $\gamma_1 H \leq q_t$ …………equa3.1

$q_b = C_2 N_{c(2)}\lambda_{cs(2)} + \gamma_1(D_f + H)N_{q(2)}\lambda_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)}\lambda_{\gamma s(2)}$ ……………………..…equa3.2

$q_t = C_1 N_{c(1)}\lambda_{cs(1)} + \gamma_1(D_f + H)N_{q(1)}\lambda_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)}\lambda_{\gamma s(1)}$ ………………………..equa3.3

Where λ_a, λ_s = Shape factors

$\lambda_{cs(1)}, \lambda_{qs(1)}, \lambda_{\gamma s(1)}$ = shape factors for the top soil layer

$\lambda_{cs(2)}, \lambda_{qs(2)}, \lambda_{\gamma s(2)}$ = shape factors for the bottom soil layer

$N_{c(1)}, N_{q(1)}, N_{\gamma(1)}$ = bearing capacity factors for the top soil layer

$N_{c(2)}, N_{q(2)}, N_{\gamma(2)}$ = bearing capacity factors for the bottom soil layer

$\gamma_1, and\ \gamma_2$ are unit weight of the top and bottom layers respectively

B and L are width and length of the footing.

$D_f\ and\ H$ are the depth of the footing and thickness of the top layer respectively

Model 1 B=2m, B/L= 1, H/B=0.5, H=0.5B=1m,

For ϕ1= 40, Nc_1=75.31, Nγ_1 =93.69, Nq_1= 64.20 ϕ2= 6, Nc_2=6.81, Nγ_2 =0.11, Nq_2= 1.72

$\lambda_{cs(1)}$ = 1.92, $\lambda_{qs(1)} = \lambda_{\gamma s(1)}$ = 1.46, $\lambda_{cs(2)}$ = 1.2, $\lambda_{qs(2)} = \lambda_{\gamma s(2)}$ = 1

$q_1 = C_1\ N_{c(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)}$

q_1 = 1x75.31 + 0.5x20*2*93.61= 1949.11

$q_2 = C_2\ N_{c(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)}$

q_2 = 15x6.81 + 0.5x16*2*0.11= 103.75

${}^{q_2}/_{q_1}$ = 0.05, from figure K_s=2.3 c_a=0.67

q_b= 15x6.81x1.2 + 20(1.5+1)1.72x1 + 0.5x2x16x0.11x1

q_b= 210.18

q_u= 210.18+ (1+1/1)(2x0.67x1x0.5) + (1+1/1)x20x1^{2}x(1+$\frac{2x1.5}{1}$)$\frac{2.3tan40}{2}$ –20x1

q_u = 365.9 KN/m^2

q$_t$= 1x75.31x1.92 + 20x1.5x64.20x1.42 + 0.5x20x2x93.69x1.46

q$_t$= 5615.26 KN/m^2

Therefore q_u = 365.9 KN/m^2

Considering the ultimate bearing capacity and the uncertainties involved in evaluating the shear strength parameters of the soil, the allowable bearing capacity q_{all}cane obtained as

$$q_{all} = \frac{q_u}{F.S}$$

For normal structure and isolated footing a factor of safety of 2 – 3 can be used to calculate the allowable bearing capacity. Factor of safety of 2.5 is used to calculate the allowable bearing capacity is this research.

Therefore $q_{all} = \frac{365.9}{2.5}$ = 146.4 Kpa

For loose sand overlying dense sand

$$q_u = \frac{1}{2}\gamma_1 B \lambda^*_{\gamma s} N_{\gamma(m)} + \gamma_1 D_f N_{q(m)} \lambda^*_{qs} \leq \frac{1}{2}\gamma_2 N_{\gamma(2)} \lambda_{\gamma s(2)} + \gamma_2 D_f N_{q(2)} \lambda_{qs(2)} \quad \text{equa3.4}$$

Where $N_{\gamma(m)} = N_{\gamma(2)} - \left[\frac{H}{D(\gamma)}\right]\left[N_{\gamma(2)} - N_{\gamma(1)}\right]$……………………………equa3.5

$N_{q(m)} = N_{q(2)} - \left[\frac{H}{D(q)}\right]\left[N_{q(2)} - N_{q(1)}\right]$…………………………equa3.6

$N_{\gamma(2)}, N_{q(2)}$ = Meyerhof's bearing capacity factors with reference to soil friction angle φ_2

$N_{\gamma(m)}, N_{q(m)}$ = modified bearing capacity factors

$\lambda^*_{\gamma s}, \lambda^*_{qs}$ = modified shape factors

$\lambda_{\gamma s(2)}, \lambda_{qs(2)}$ = Meyerhof's shape factors with reference to soil friction angle φ_2

Model 28, B=2m, B/L= 1, H/B=0.5, H=0.5B=1m,

For ϕ2= 40, Nγ_2 =93.69, Nq_2= 64.20 $\lambda_{qs(2)} = \lambda_{\gamma s(2)}$ = 1.46,

For ϕ1= 30 D_q /B = 1.7, D_q =3.4, D_γ/B=0.85, D_γ=1.7

λ^*_{ys}= 0.95 λ^*_{qs}= 1

$N_{y(m)}$= 93.69 – (1/1.7)(93.69 – 15.67) = 47.81 $N_{q(m)}$= 64.2 – (1/3.4)(64.2 – 18.4) = 50.73

q_u = 0.5*12*2*0.95*47.81 + 12*1.5*1*50.73

<= 0.5*20*1.46*93.69 +20*1.5*1.46*64.20

q_u= 1458.2 <=4180

q_u= 1458.2 KN/m^2

Considering the ultimate bearing capacity and the uncertainties involved in evaluating the shear strength parameters of the soil, the allowable bearing capacity q_{all}cane obtained as

$$q_{all} = \frac{q_u}{F.S}$$

For normal structure and isolated footing a factor of safety of 2 – 3 can be used to calculate the allowable bearing capacity. Factor of safety of 2.5 is used to calculate the allowable bearing capacity is this research.

Therefore $q_{all} = \frac{1458.2}{2.5}$ = 583.3 Kpa

ii. for three layered soil

Dense sand / medium sand / soft clay

$$q_u = q_b + (1+ \text{B/L}) \left[\frac{K_{s1}\, H_1^2 tan\Phi_1}{B} + \frac{K_{s2}\gamma^- H_2^2 tan\Phi_1}{B}\ (1 + \frac{2H_1}{H_2})\right] - \gamma^-(H_1 + H_2) \leq q_t$$

Where

$\gamma^- = \left(\frac{\gamma_1+\gamma_2}{2}\right)$, γ_1 and γ_2 are the unit weights of the upper and middle layer respectively.

K_{s1} , K_{s2} = are the punching shear coefficients for the upper and middle layers, respectively.

q_b, q_t = are the ultimate bearing capacities on a very thick bed of lower layer and upper layer, respectively.

Model 55, B=2m, H1=1m, H2=2m, L/B=1

For ϕ1= 40, Nc_1=75.31, Nγ_1 =93.69, Nq_1= 64.20 ϕ2= 6, Nc_2=6.81, Nγ_2 =0.11, Nq_2= 1.72

ϕ3= 6, Nc_3=6.81, Nγ_3 =0.11, Nq_3= 1.72

q_1 = 1937.3, q_2 = 1364 q_3 = 103.92 q_2/q_1 = 0.71, K_{s1}=11 q_3/q_2 = 0.08, K_{s2}=1

q_b = 238.72

q_u = 238.72 + (1+1/1)[11*0.5*10*1^2*tan40*0.5 + 1*19*tan6*(1+2*1/1)] -19(1+1)

q_u= 435.28KN/m^2

3.8. Comparison of results

A comparison based on FEM and empirical method has been done. It's widely presented in parametric study.

Table 3. 6 comparison of result obtained from Empirical method and PLAXIS 3d

Model No	Bearing capacity (KN/m2)					% difference		
	conventional method		FEM(PLAXIS 3d)					
	q_{ult}	q_{all}	S=B	S=1.5B	S=2B	S=B	S=1.5B	S=2B
1	365.9	146.4	289	341	348	-97.4044	-132.923	-137.705
1b	267.5	107	234	306	298	-118.692	-185.981	-178.505
2	593.25	237.3	420	494	516	-76.9912	-108.175	-117.446
2b	427.14	170.9	287	365	358	-67.9345	-113.575	-109.479
3	1319.5	527.8	878	1002	1035	-66.3509	-89.8446	-96.097
3b	852.22	340.9	463	542	540	-35.817	-58.9909	-58.4042
4	312.52	125	248	282	301	-98.4	-125.6	-140.8
4b	243.85	98	203	261	282	-107.143	-166.327	-187.755
5	521.94	209	356	390	423.5	-70.3349	-86.6029	-102.632
5b	366.5	146.6	250	312	337	-70.5321	-112.824	-129.877
6	1128.4	451.36	680	751	813	-50.6558	-66.386	-80.1223
6b	737.27	295	404	465	498	-36.9492	-57.6271	-68.8136
7	297	119	217	264	272	-82.3529	-121.849	-128.571
7b	232.78	94	207	250	262	-120.213	-165.957	-178.723
8	485.5	194	308	345	375	-58.7629	-77.8351	-93.299
8b	346.42	138.6	264	297	320	-90.4762	-114.286	-130.88
9	1038	415.2	605	675	720	-45.7129	-62.5723	-73.4104
9b	678.2	271.28	398	446	473	-46.7119	-64.4058	-74.3586
28	1458.2	583.3	547	551	605	6.223213	5.537459	-3.72021
28b	1095	438	398	481	447	9.13242	-9.81735	-2.05479
29	692.34	276.94	393	503	410	-41.908	-81.6278	-48.0465
29b	678	271.2	330	394	368	-21.6814	-45.2802	-35.6932
30	509	203.6	282	310	298	-38.5069	-52.2593	-46.3654

30b	390.5	156.2	255	296	278	-63.2522	-89.5006	-77.977
31	1467	586.8	515	510	514	12.23586	13.08793	12.40627
31b	1098	439.2	346	429	462	21.2204	2.322404	-5.19126
32	693	277.2	364	450	467	-31.3131	-62.3377	-68.4704
32b	678	271.2	284	323	360	-4.71976	-19.1003	-32.7434
33	512	204.8	259	276	279	-26.4648	-34.7656	-36.2305
33b	392	156.8	223	245	262	-42.2194	-56.25	-67.0918
34	1473	589.2	490	494	518	16.83639	16.1575	12.08418
34b	1103	441.2	339	404	432	23.1641	8.43155	2.085222
35	693	277.2	348	353	349	-25.5411	-27.3449	-25.9019
35b	678	270	280	329	354	-3.7037	-21.8519	-31.1111
36	514	205.6	249	269	274	-21.1089	-30.8366	-33.2685
36b	392	156.8	215	248	262	-37.1173	-58.1633	-67.0918
55	435.3	218	502	702	598	-130.275	-222.018	-174.312
55b	283.5	142	353	403	394	-148.592	-183.803	-177.465
56	789.07	394.5	706	780	770	-78.9607	-97.7186	-95.1838
56b	479.8	239.9	407	463	456	-42.507	-62.1148	-59.6639
57	815	407.5	850	950	900	-108.589	-133.129	-120.859
57b	571.3	285.6	497	562	548	-107.17	-134.264	-128.429
58	357.72	178.86	460	620	560	-168.366	-224.276	-213.094
58b	259.7	129.85	320	380	366	-146.438	-192.645	-181.864
59	672.5	336.25	606	680	670	-75.7541	-97.2158	-94.3155
59b	425.67	212.8	460	540	521	-116.165	-153.759	-144.831
60	689.17	344.8	755	863	843	-124.535	-156.654	-150.706
60b	394.5	197.3	480	550	514	-143.284	-178.763	-160.517
61	300.1	150.5	456	543	523	-202.99	-260.797	-247.508
61b	230.65	116.7	301	370	350	-157.926	-217.052	-199.914
62	622	311	577	668	641	-80.7078	-109.208	-100.752
62b	399.02	199.5	445	518	498	-123.058	-159.649	-149.624
63	638.6	319.3	723	820	805	-132.476	-163.666	-158.842

63b	367.6	183.8	465	525	505	-152.992	-185.637	-174.755

The percentage difference shows that bearing capacity estimation using empirical formulas are lower than that of calculation done using FEM (PLAXIS 3D).

CHAPTER FOUR

DISCUSION OF RESULTS

Different cases has been considered in parametric study. The parametric studies considered in this thesis are: upper layer thickness to width ratio, spacing between two footings, foundation type i.e. length to width ration of the footing, and the effect ground water.

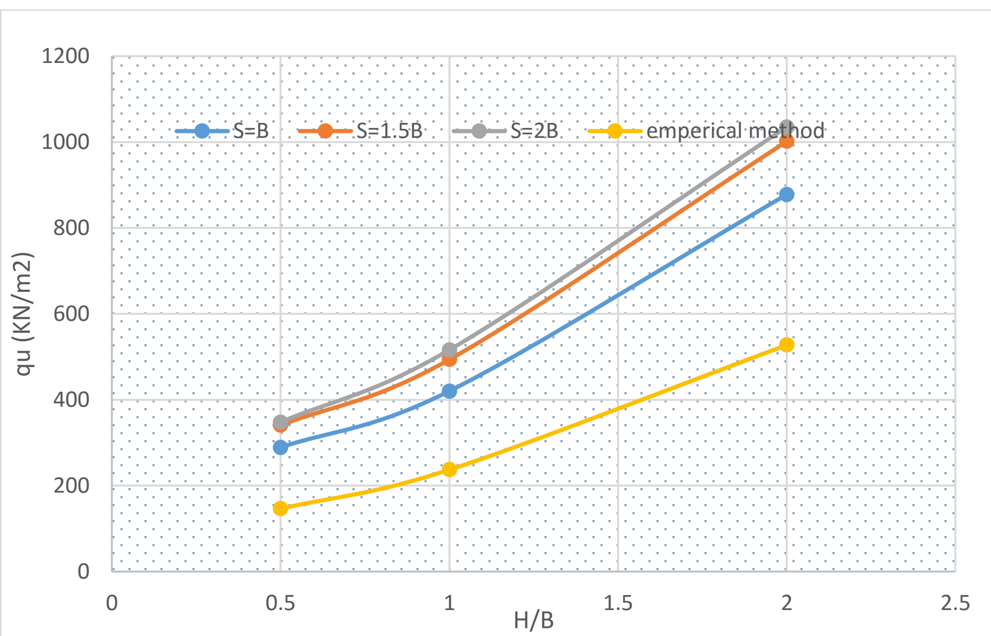
In many cases settlement criteria will control the allowable bearing capacity so as per EBCS 7 for normal structure with isolated footing a total settlement of 50mm is often acceptable. So in this thesis this provision is considered in estimating the bearing capacity.

4.1. For two layered soil profile

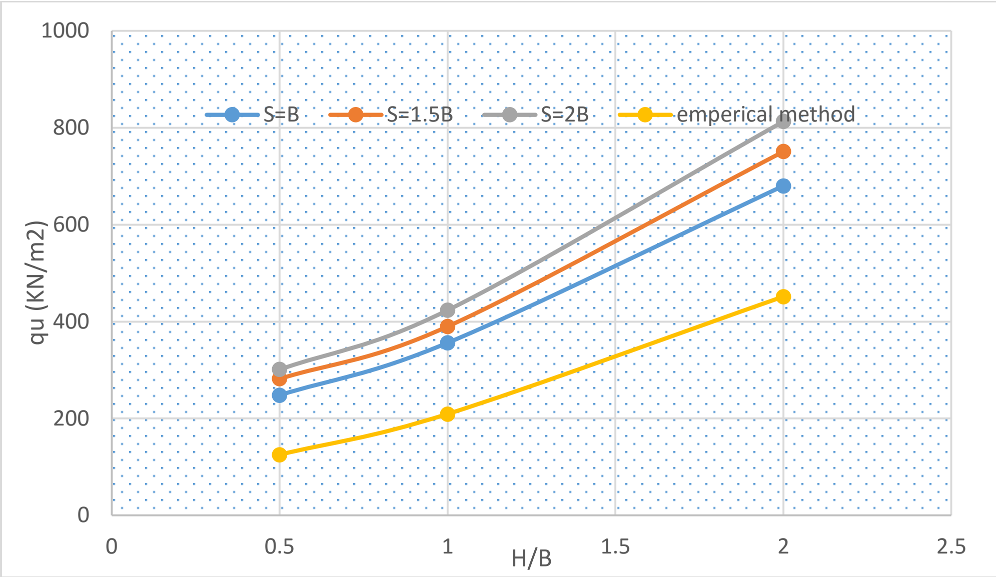
4.1.1 Stronger layer overlying weaker layer

For the case stronger layer overlying weaker layer stratum a sample of dense sand overlying soft clay has been considered in this case

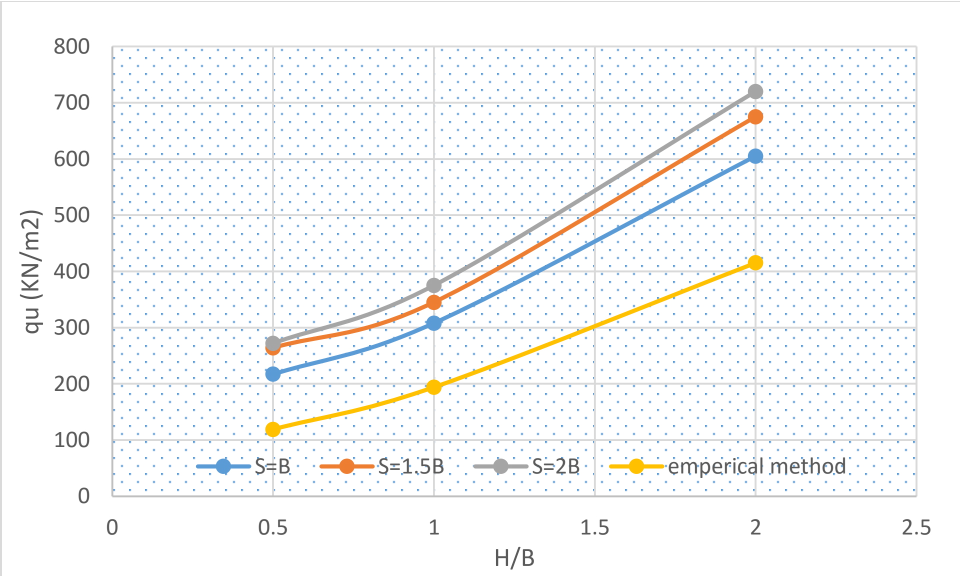
i. Effect of upper layer thickness to width ratio



(a)



(b)



(c)

Figure 4.1 Variation of bearing capacity with H/B (a) L/B=1, (b) L/B=1.5, (c) L/B=2

From the above graphs it can be seen that as thickness of upper layer increases the bearing capacity also increase all case considered in this study. And also bearing capacity estimation using empirical method gives lower value as compared with results obtained from PLAXIS 3D FOUNDATION software.

For the two layered soil profile considering for the case of stronger layer overlying weaker layer as the thickness of the uppermost layer (i.e. the stronger layer) increases the bearing capacity also increases.

ii. Effect of spacing between two footings

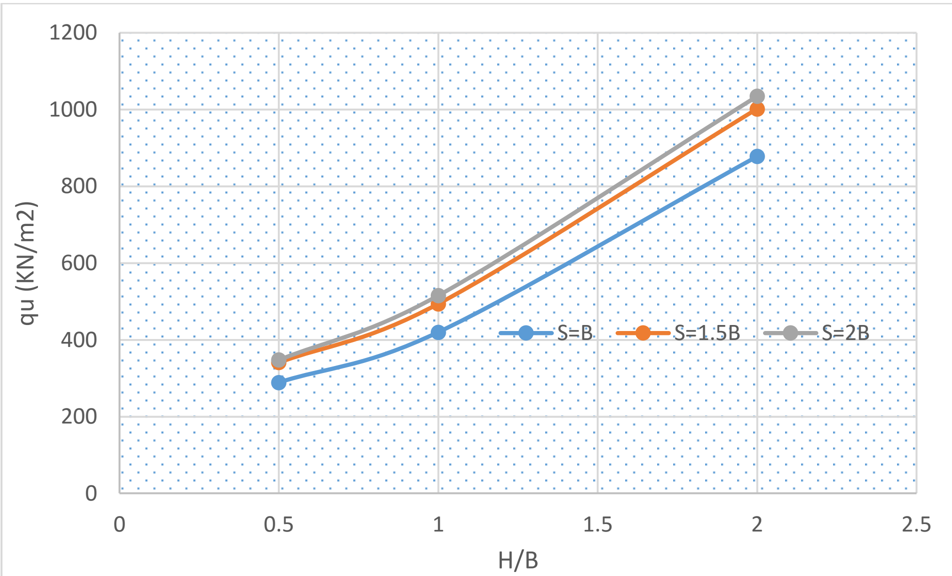
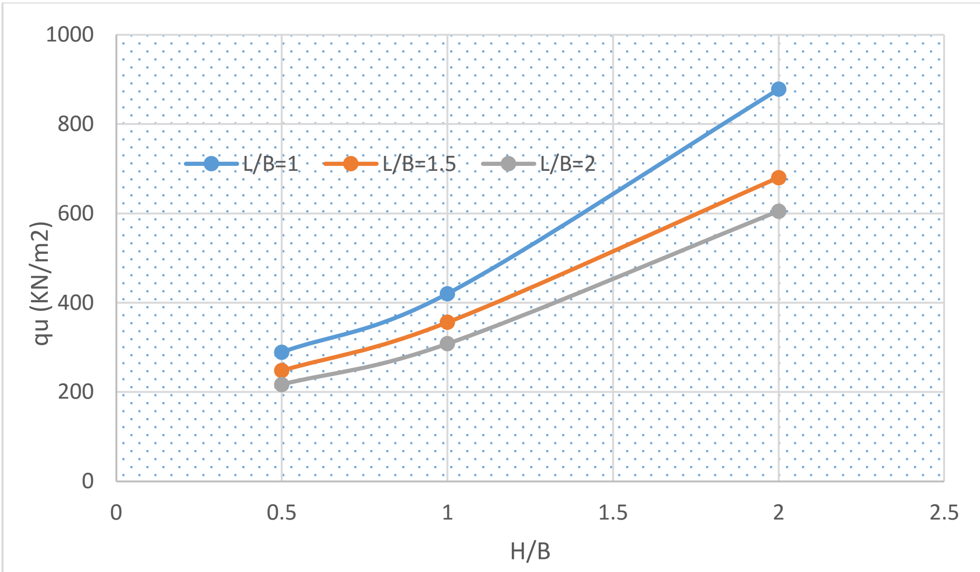


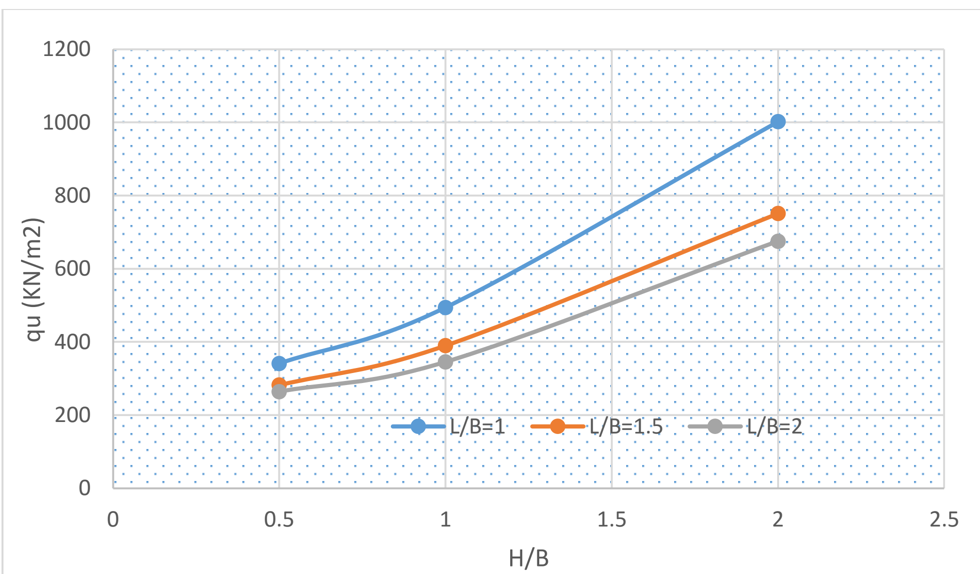
Figure 4.2 Variation of bearing capacity with H/B for length to width ratio of 1

From the above graphs it can be observed that as the spacing between the two footings decreases the bearing capacity also decreases.

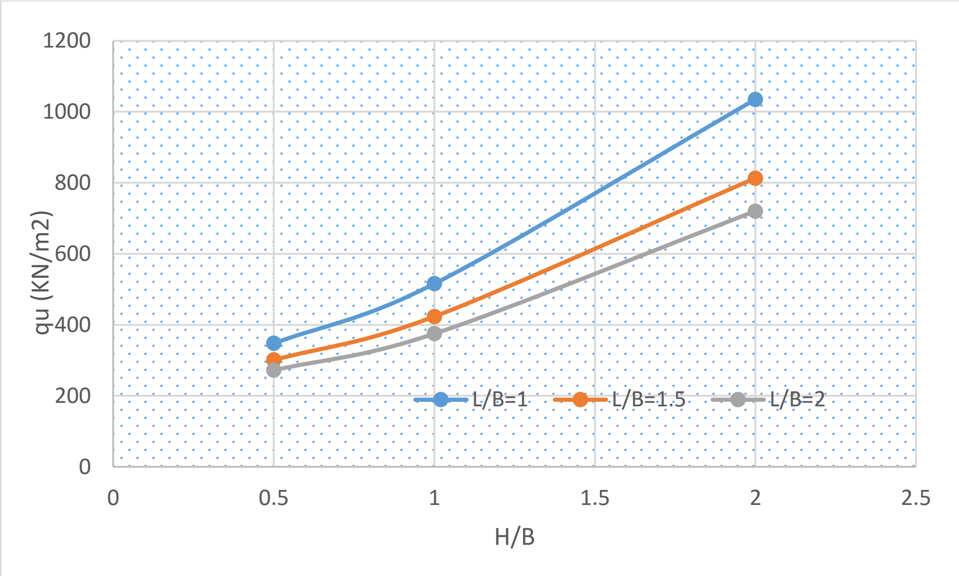
iii. Effect of length to width ratio of footings


1000
800
600
400
200
0
qu (KN/m2)
L/B=1
L/B=1.5
L/B=2
0
0.5
1
1.5
2
2.5
H/B

(a)


1200
1000
800
600
400
200
0
qu (KN/m2)
L/B=1
L/B=1.5
L/B=2
0
0.5
1
1.5
2
2.5
H/B

(b)



(c)

Figure 4.3 variation of bearing capacity with H/B for (a) S=B, (b) S=1.5B, (c) S=2B

From the above graph it can be seen that as the length to width ratio increases bearing capacity of the soil decrease and also has greater effect as H/B increase.

iv. Effect of ground water

The presence of groundwater significantly affect the bearing capacity of shallow foundation by reducing the shear strength of the soil base of the footing by reducing the apparent cohesion due to suction or weak cementation bond. Moreover the unit weight of the submerged soil is about half of the dry soil.

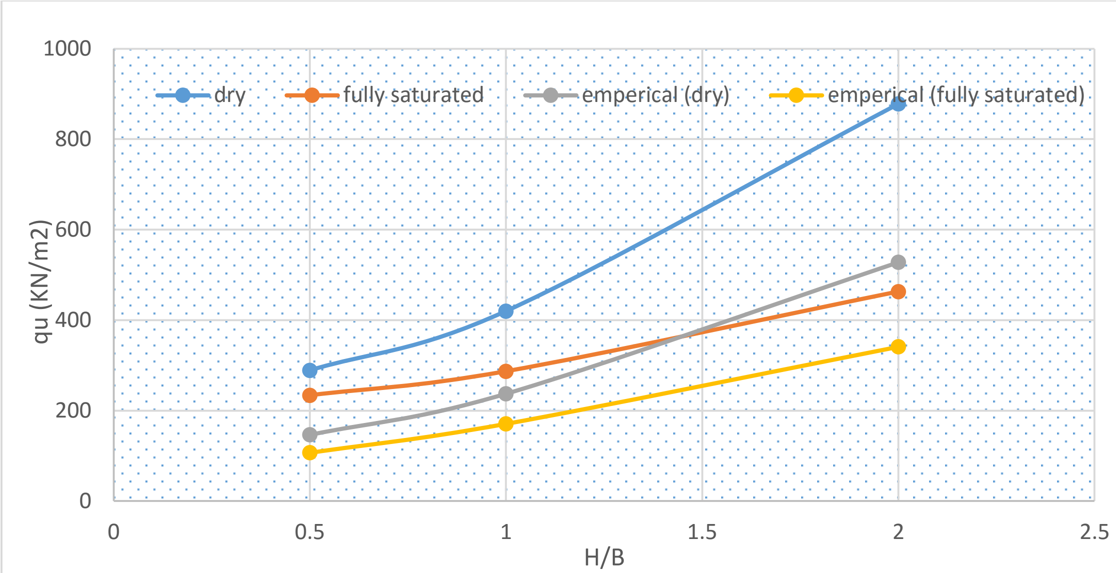


Figure 4.3 variation of bearing capacity with H/B for L/B=1 and S=B

From the above graph it can be seen that ground water has a great effect bearing capacity of soil. Also it can be seen that empirical method of estimating bearing capacity gives lower values compared with that of results obtained using PLAXIS 3d software. And also as the thickness of the dense sand layer increases the effect of ground water on bearing capacity also increases.

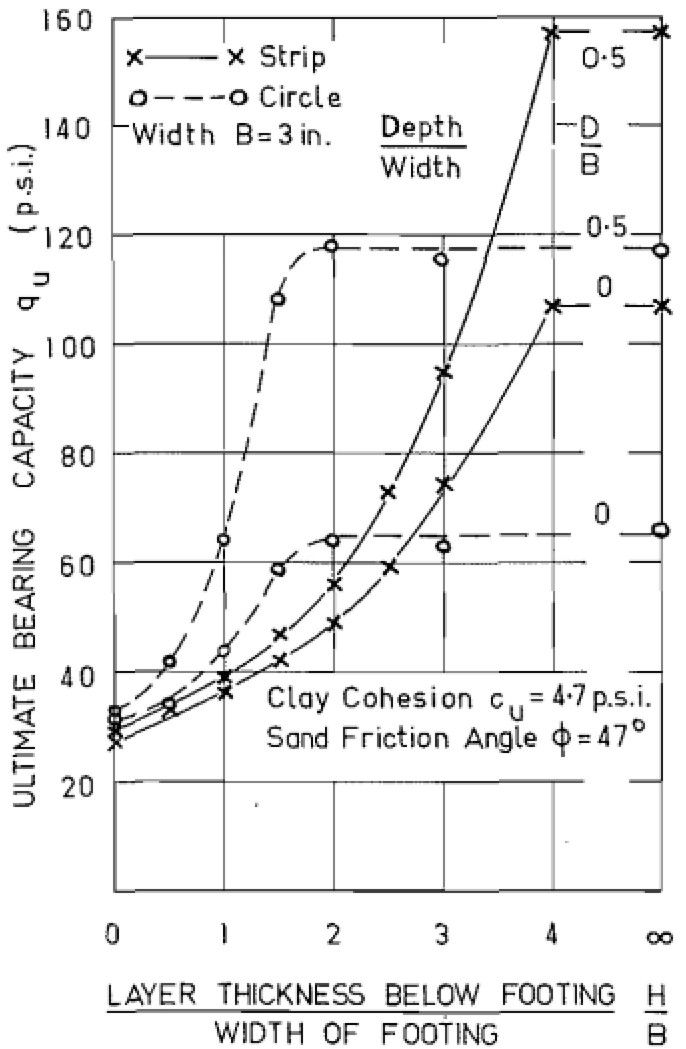


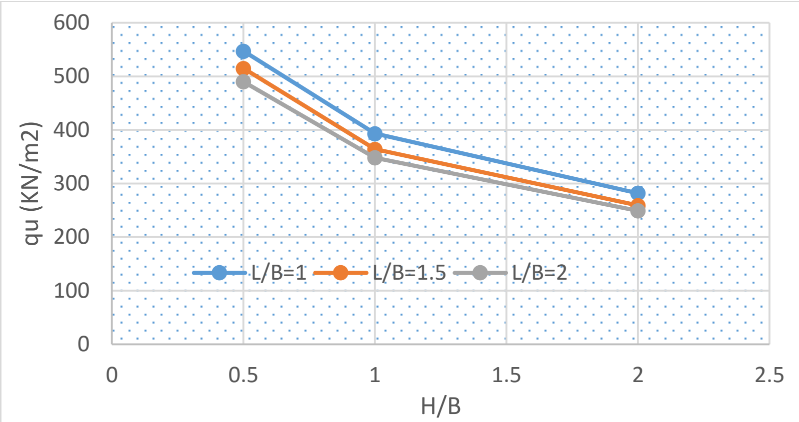
Figure 4.5 Typical results of model footing tests on dense sand overlying soft clay (G.G Meyerhof 1974)

Comparing the results obtained from the PLAXIS 3D foundation (finite element based software) with that of result obtained from model test by G.G Meyerhof its can be observed that in both case as the ratio of the upper layer to width increases the bearing capacity also increases.

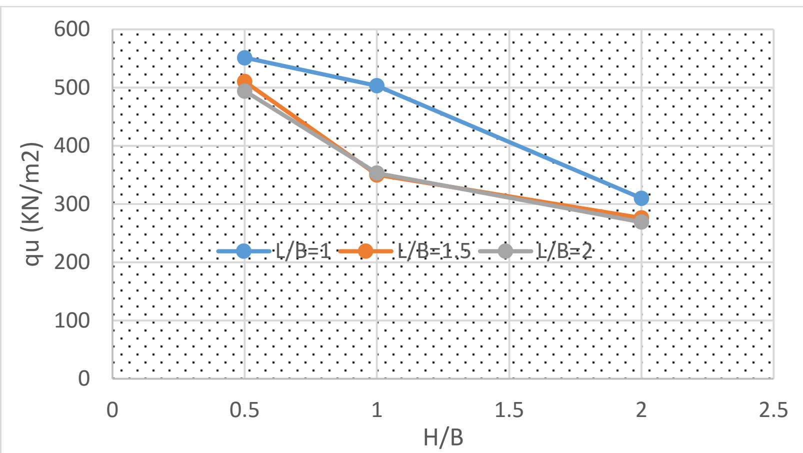
4.1.2 for weaker layer overlying stronger layer

For the case weaker layer overlying stronger layer stratum a sample of loose sand overlying dense sand has been considered in this case.

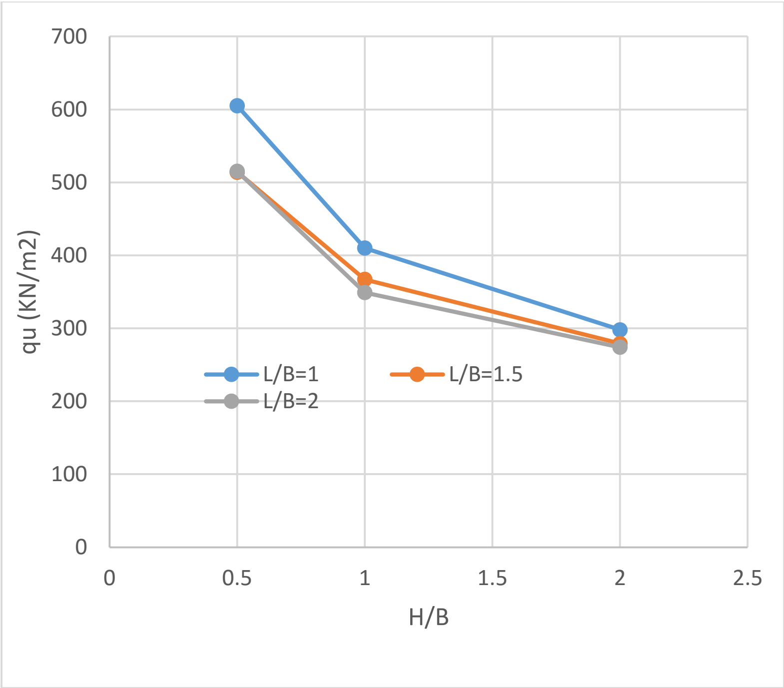
i. Effect of upper layer thickness to width ratio



(a)



(b)

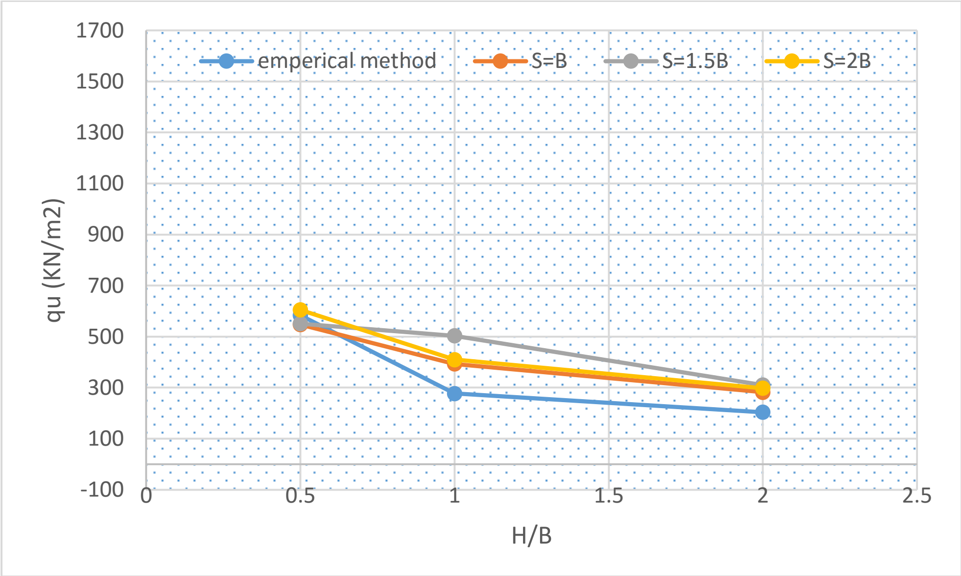


(c)

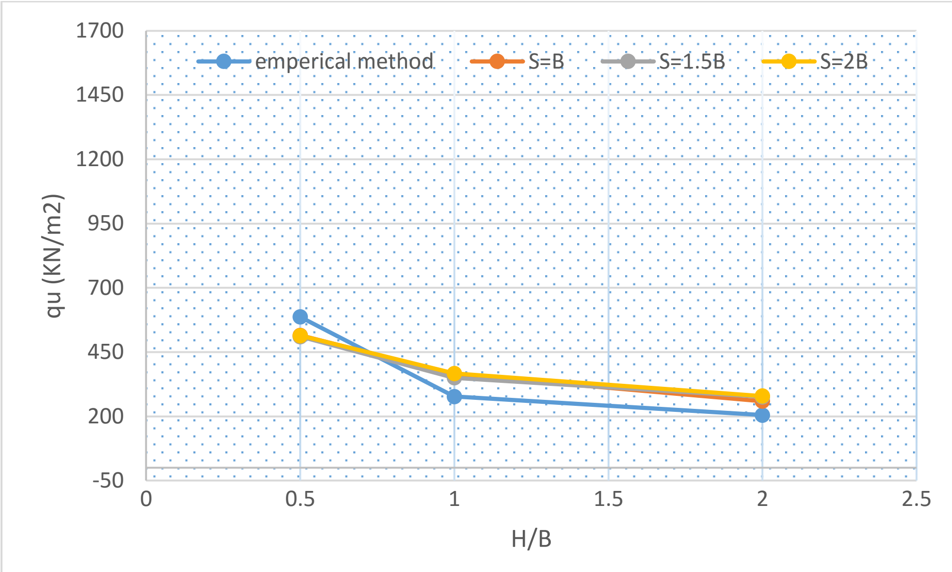
Figure 4.6 variation of bearing capacity with H/B for (a) S=B, (b) S=1.5B, (c) S=2B

From the above graphs it can be seen that as thickness of upper layer increases the bearing capacity decreases for the cases considered in this study.

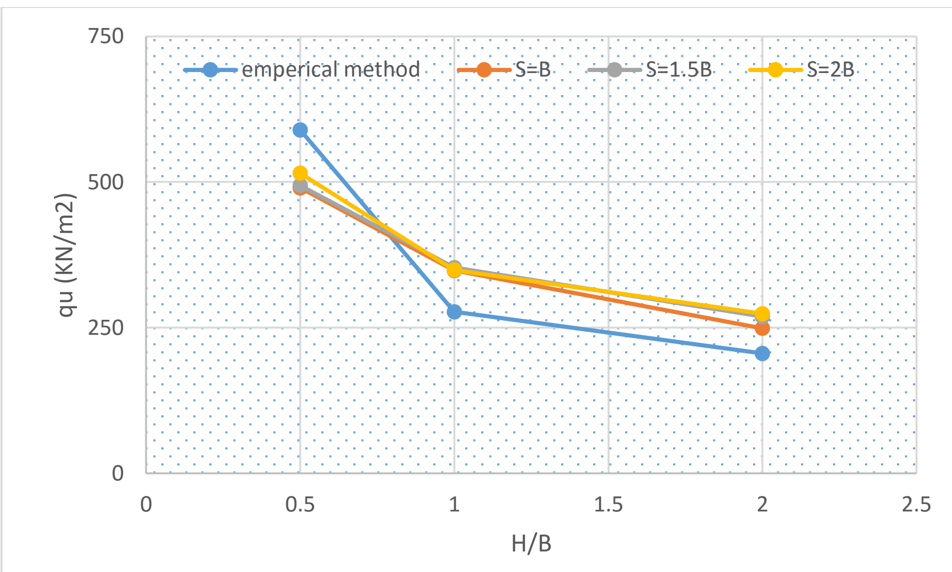
ii. Effect of spacing between two footings



(a)



(b)



(c)

Figure 4.7 Variation of bearing capacity with H/B for (a) L/B=1, (b) L/B=1.5, (c) L/B=2

From the above graph (a) it can be seen that for L/B=1 and spacing of 1.5B yields max value for H/B>= 1 whereas for H/B= 0.5 spacing of 2B gives max value, but for L/B=1.5 and 2 as it seen from graph (b) and (c) effect of spacing between footings doesn't much of difference on the bearing capacity of footing. And also it can be seen that empirical method of determining bearing capacity yields lower value than that of bearing capacity determination using FEA (PLAXIS 3D) except when the ratio of the upper layer to width ratio is 0.5 that this case empirical method gives higher value as compared with that of FEA .

iii. Effect of length to width ration of footings

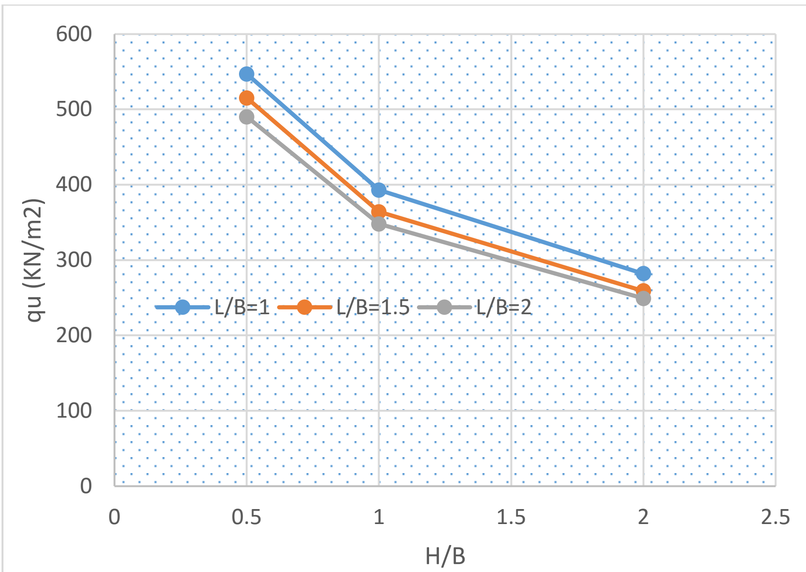


Figure 4.8 Variation of bearing capacity with H/B for spacing between two footings equal to B.

From the above graph it can be seen that as L/B ratio increases the bearing capacity of soil decreases.

iv. Effect of ground water

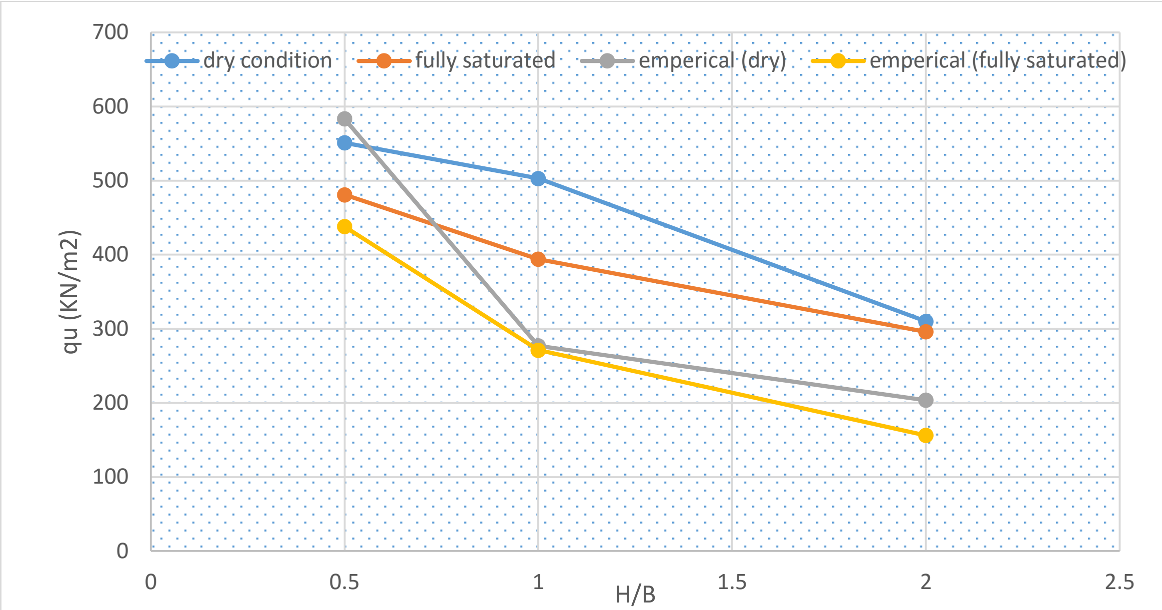


Figure 4.9 Variation of bearing capacity with H/B for L/B=1 and S=1.5B

From the above graph it can be seen that ground water has a great effect bearing capacity of soil. Also it can be seen that empirical method of estimating bearing capacity yields higher values compared with that of results obtained using PLAXIS 3d software packages for H/B =0.5 but for H/B ≥ 1 it give lower value compared with FEA.

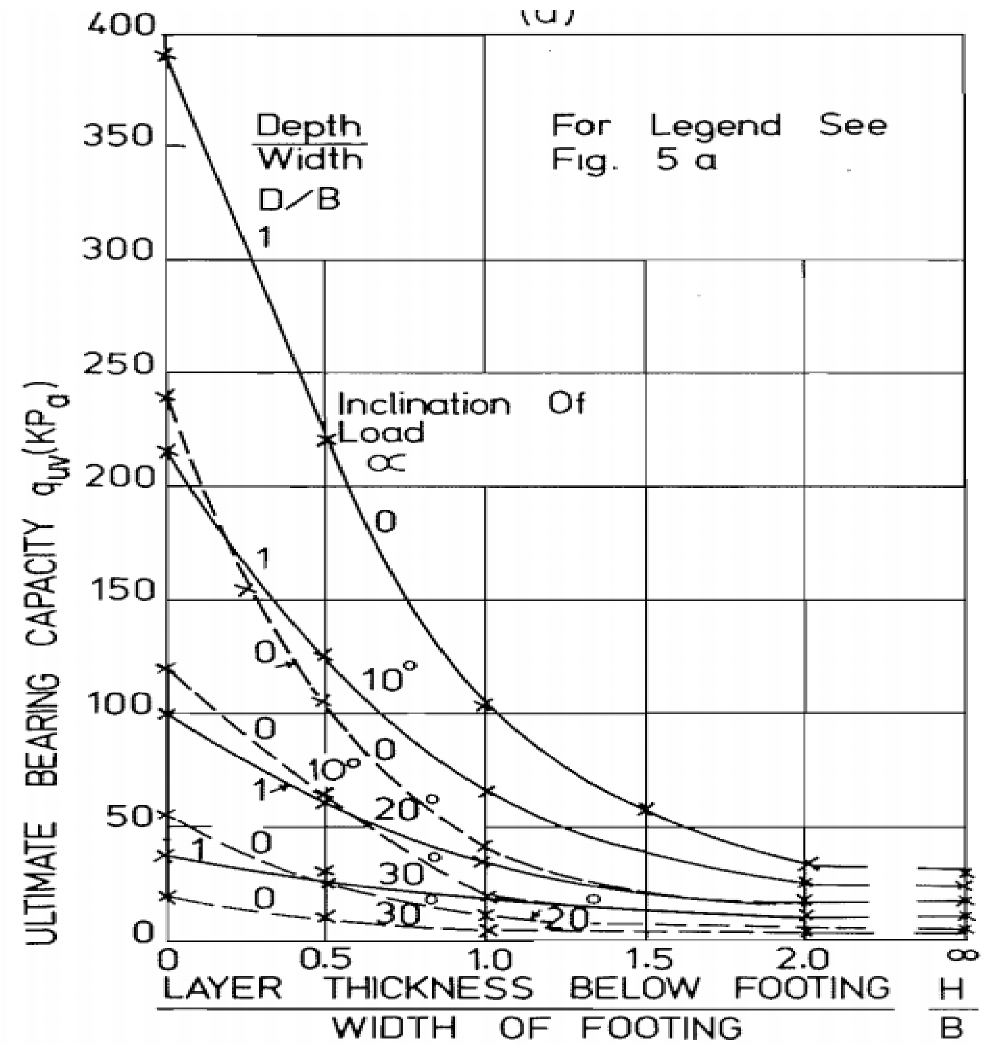


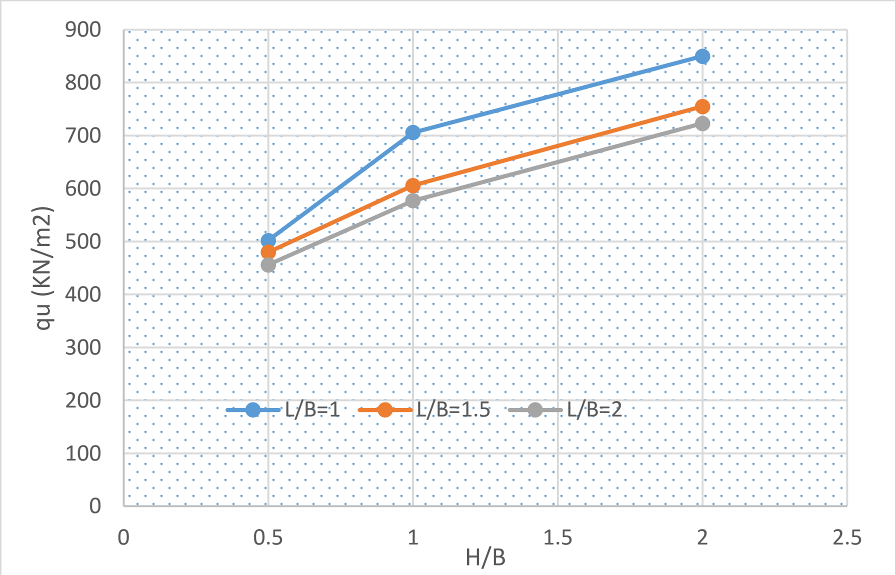
Figure 4.10 Typical results of model footing tests on loose sand overlying dense sand for strip footings

Comparing the results obtained from the PLAXIS 3D foundation (finite element based software) with that of result obtained from model test by G.G Meyerhof its can be observed that in both case as the ratio of the upper layer to width increases the bearing capacity decreases.

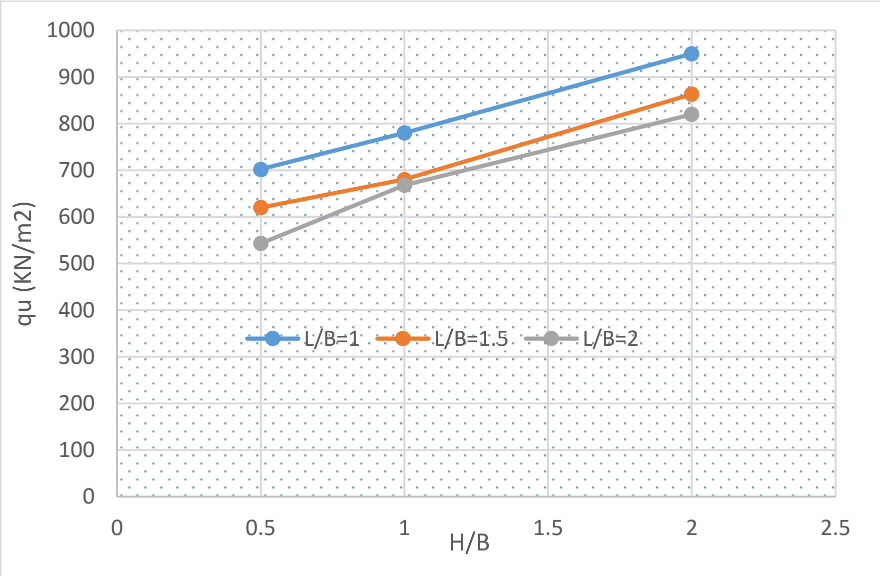
4.2. For three layered soil

For the three layered soil stratum a sample two strong layer overlying weaker layer was studied. That is for the case dense sand / stiff clay / soft clay

i. Effect of upper layer thickness to width ratio



(a)

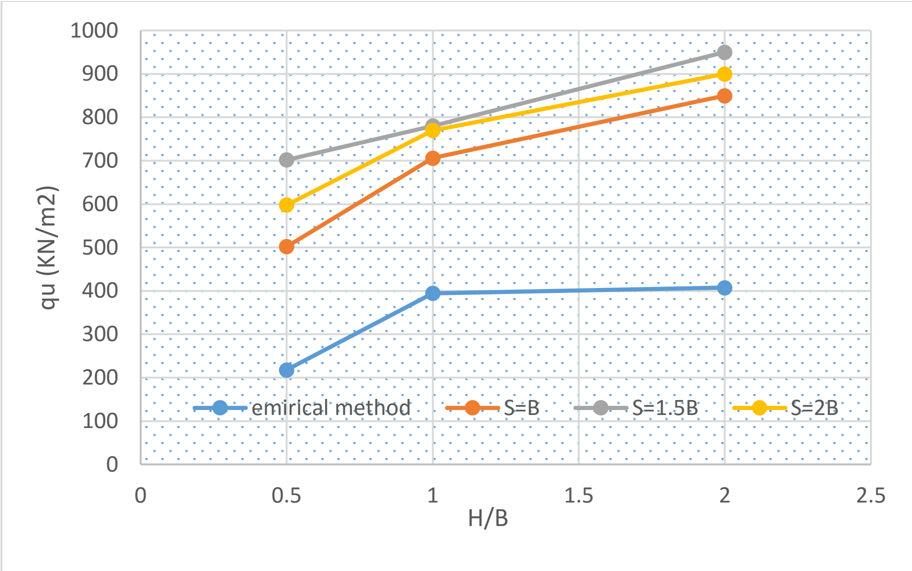


(b)

Figure 4.11 Variation of bearing capacity with H/B for (a) S=B, (b) S=1.5B,

From the above graphs it can be seen that as thickness of upper layer increases the bearing capacity also increase all case considered in this study.

ii. Effect of spacing between two footings



(a)

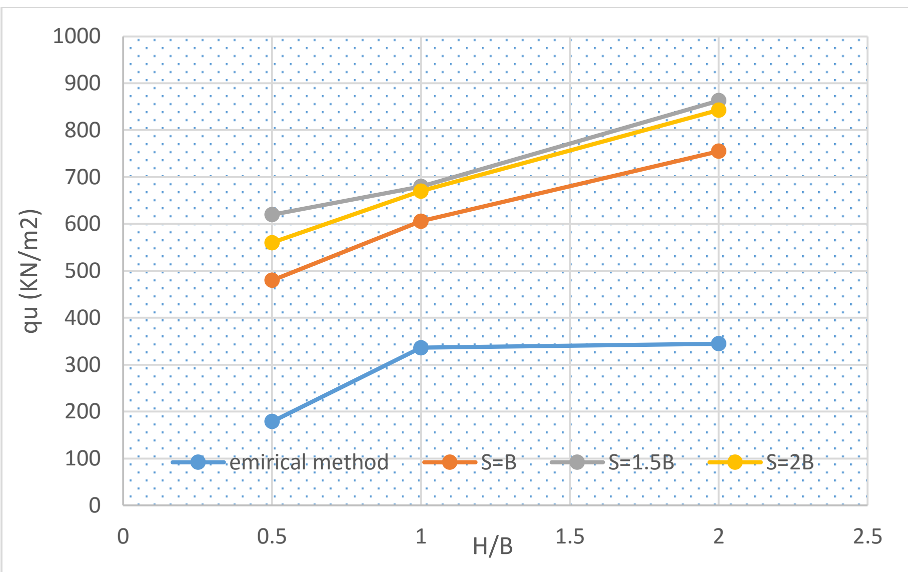


Figure 4.12 Variation of bearing capacity with H/B for (a) L/B=1, (b) L/B=1.5,

From the above graphs it can be observed that as the spacing between the two footings increases the bearing capacity also increases. And also we can see that the values bearing capacity obtained using empirical method are lower that of values obtained from analysis of bearing capacity using FEM (PLAXIS 3D).

iii. Effect of length to width ration of footings

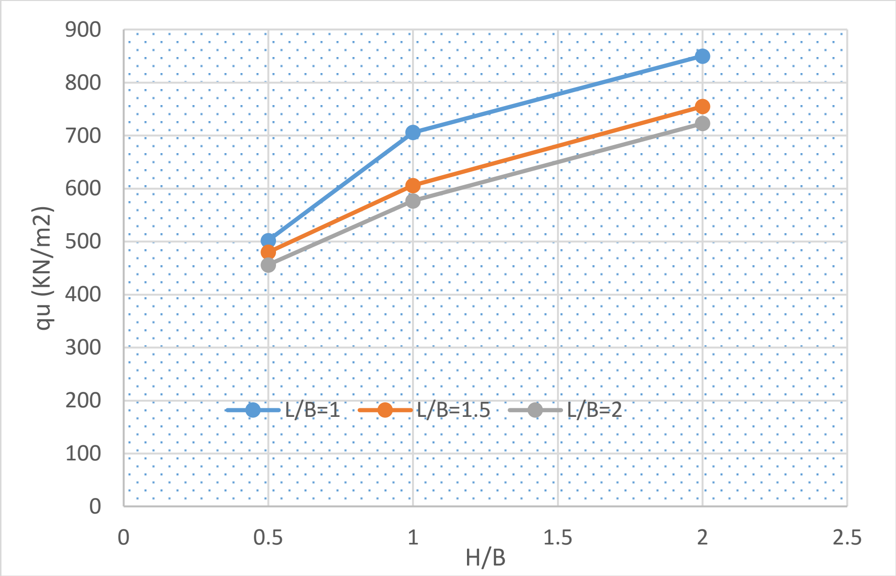


Figure 4.13 Variation of bearing capacity with H/B for spacing b/n footing equal to B

From the above graph we can see that as length to width ratio of footing increases bearing capacity of soil decreases.

iv. Effect of ground water

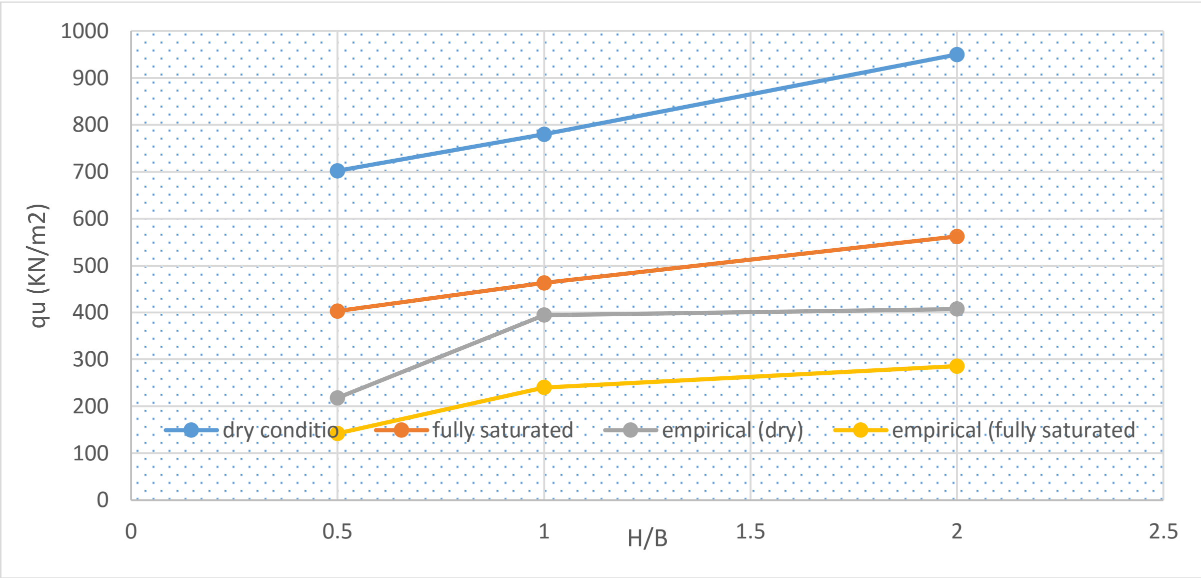


Figure 4.14 Variation of bearing capacity with H/B for L/B=1 and S=1.5B

From the above figure we can observe that ground water significantly reduces bearing capacity of the soil and values obtained from empirical method are lower compared with results obtained from PLAXIS 3D which finite element based software.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study a serious 162 Finite Element Analysis were conducted and settlement of each model were used to determine the bearing capacity of layered soil profile and compared with empirical method of estimation of bearing capacity. The key parameters investigated in this study which affects bearing capacity of soil are: soil properties, layering of soil, ratio of upper layer to width of the footing, spacing between two footings, length to width ratio of footing, and effect of ground water table on bearing capacity of layered soil. The study mainly focus of estimation of bearing capacity in layered soil under the application of vertical and static loading only. Based on the results from FEA and empirical method of estimation of bearing capacity the following conclusion can be drown.

- For cases of stronger layer overlying weaker soil layer (dense sand overlying soft clay) values obtained from empirical method are much lower than that of values obtained from FEA. And also as ratio of upper layer thickness to width of footing (H/B) increase from 0.5 to 2 the percentage difference between empirical method and increases up to -35 to -118 %, -58 to -166 % and -58 to 168 % for spacing between footings of B, 1.5B, and 2B respectively.
- From those results it can be seen that as the spacing between the two footing increases beyond 1.5B the bearing capacity decreases so a spacing of 1.5B between two adjacent footings could be used as minimum spacing.
- For cases of weaker layer underlying stronger soil layer (loose sand overlying dense sand) values obtained from empirical method are much lower than that of values obtained from FEA except for case when H/B is equal to 0.5 in this this FEA slightly gives a higher value . And also the percentage difference between empirical method ranges from -3 to -63 %-, 9-to -89 % and -2 to -77 % for spacing between footings of B, 1.5B, and 2B respectively.
- From those results it can be seen that as the spacing between the two footing increases beyond 1.5B the bearing capacity decreases so a spacing of 1.5B between two adjacent footings could be used as minimum spacing.

- For the case of three layered system values obtained from empirical method value gives much lower than that of results obtained from FEA. The percentage difference between empirical method and FEA ranges from -75% to -152%, -62% to 260%, and -59% to 259% for spacing between footings of B, 1.5B, and 2B respectively, so spacing of 1.5B as a minimum value can be used.
- It has been observed that as the thickness of the upper two stronger layer increases the difference between values obtained from empirical and FEA tends to increases.
- For all cases considered in this study it’s been observed that as the ratio of length to width ration (L/B) of the footing increases the bearing capacity of soil tend to reduce in both of estimation methods.
- For all cases considered in this study it’s been observed that ground water greater reduces the bearing capacity of soil in both of estimation methods and also as thickness of sand layer increases the effect of ground water on bearing capacity tends to increase.

5.2. Recommendations

A number of issues have been addressed in this study and various useful conclusions are drawn. But there are still some limitations in the analyses and unsolved problems worth further investigation. These limitations are presented here, together with some recommendations for future work.

- In this study only vertical loading conditions were analyzed so the effect of load inclination on bearing capacity of layered soil profile should be studied in the future.
- The study only focused on static loading so the effect of dynamic loading in bearing capacity of layered soil could be future research area.
- The study uses Mohr-Coulomb constitutive model in the Finite Element analysis software package (PLAXIS 3D) so for the future work different constitutive model could be used in FEA with different software packages such as ABAQUS, ANYSIS etc.

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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:

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

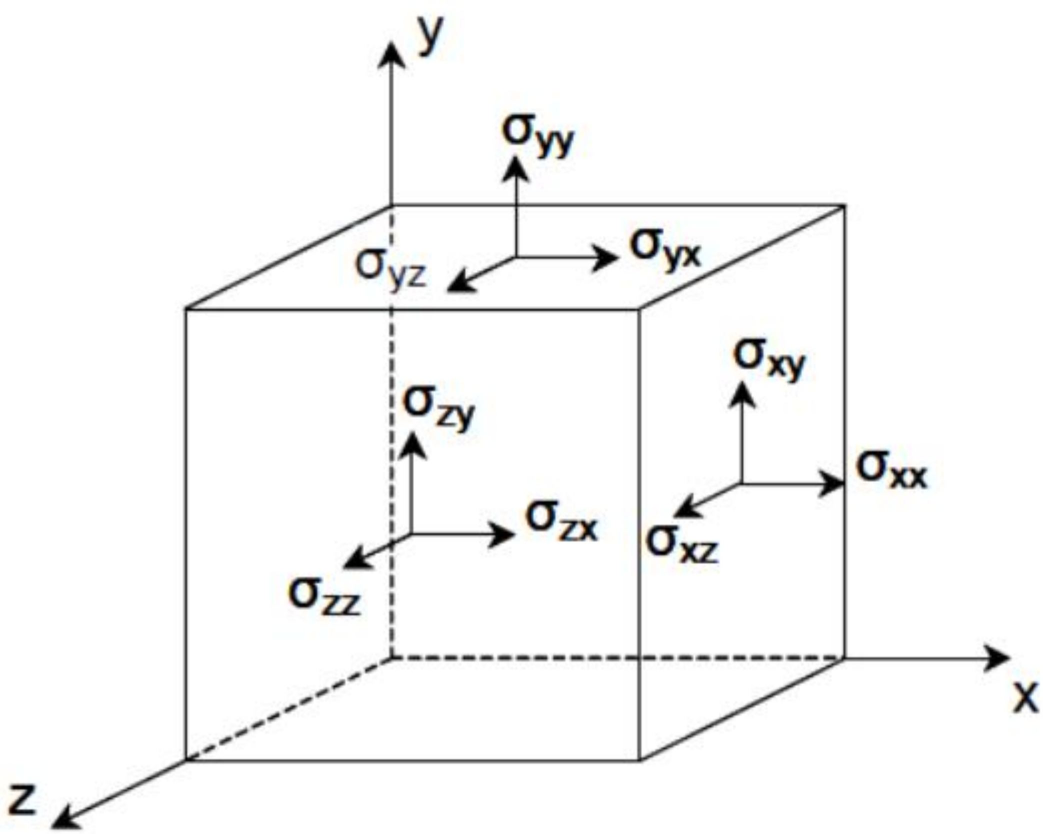

Figure A. 1: General three-dimensional coordinate system and sign convention for stresses

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 general states of stress. In fact, this condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition consists of six yield functions when formulated in terms of principal stresses:

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

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

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

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

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

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

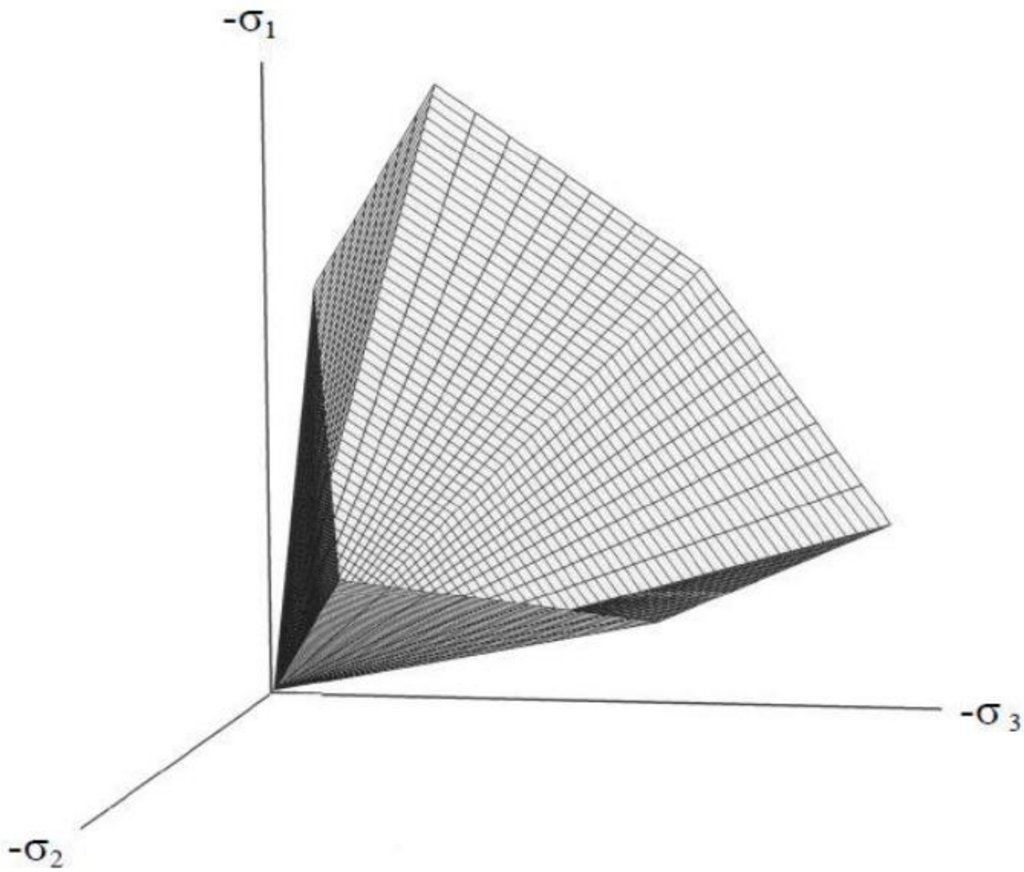

Figure A. 2 The Mohr-Coulomb yield surface in principal stress space (c = 0)

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 [°]

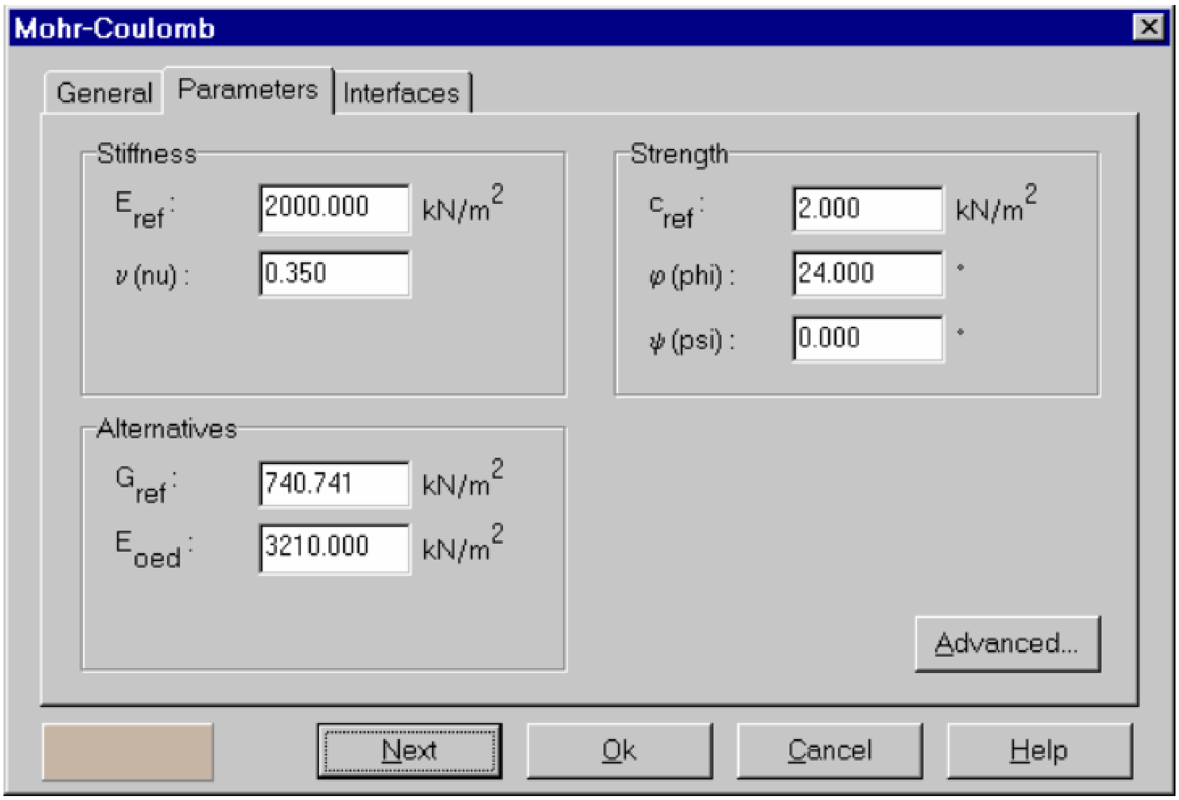

Figure A 2 1: Parameter tab sheet for Mohr-Coulomb model

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 geo materials show a nonlinear behavior 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 tunneling and excavations, on needs *Eur* instead of *E*50. For soils, both the unloading modulus, *Eur*, 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*M weight* 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-dimensional compression it is easy to select a Poisson's ratio that gives a realistic value of *K*0. Hence, ν is evaluated by matching *K*0. This subject is treated more extensively in Appendix A of 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, non-experienced users 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 sometimes obtained 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 a particular project.

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

Dilatancy angle (ψ)

The dilatancy angle, ψ (psi), is specified in degrees. Apart from heavily over consolidated layers, clay soils tend to show little dilatancy (ψ ≈ 0). The dilatancy of sand depends on both the density and on the friction angle. For quartz sands the order of magnitude is ψ ≈ ϕ - 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 PLAXIS Outputs

Effective mean stress

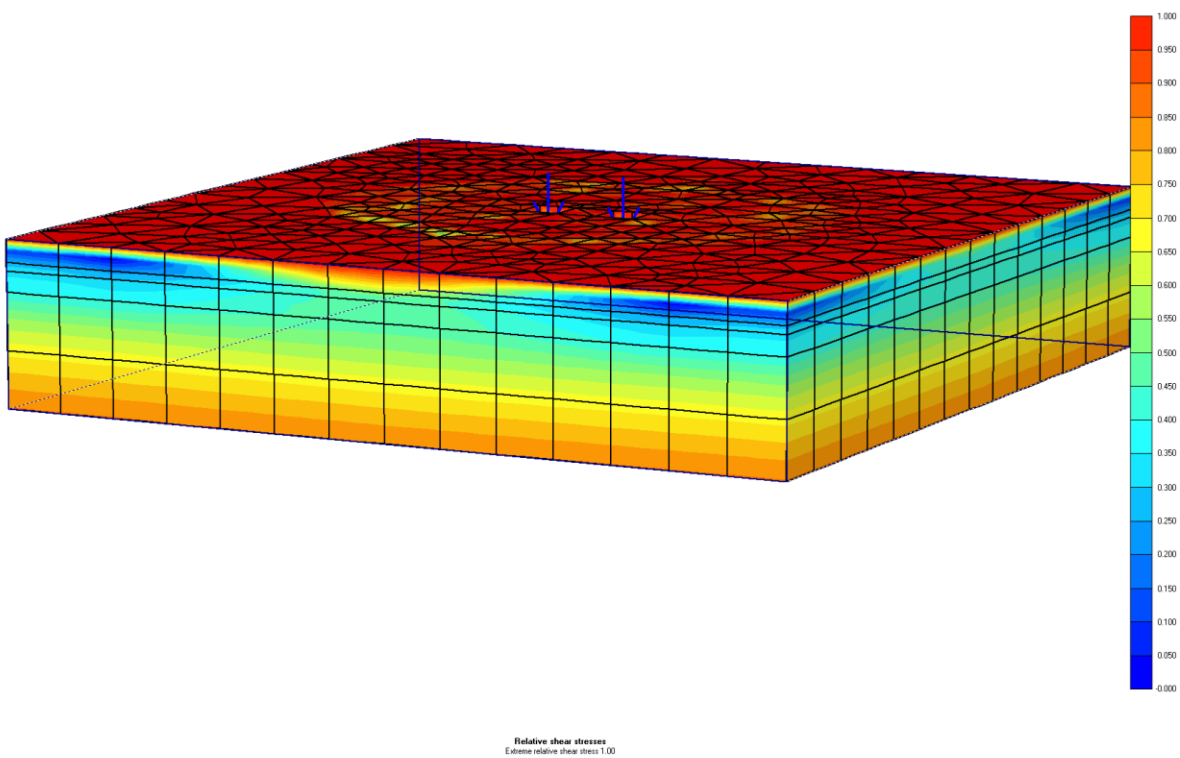

Figure A.3 1 relative shear stress for dense sand overlying soft clay

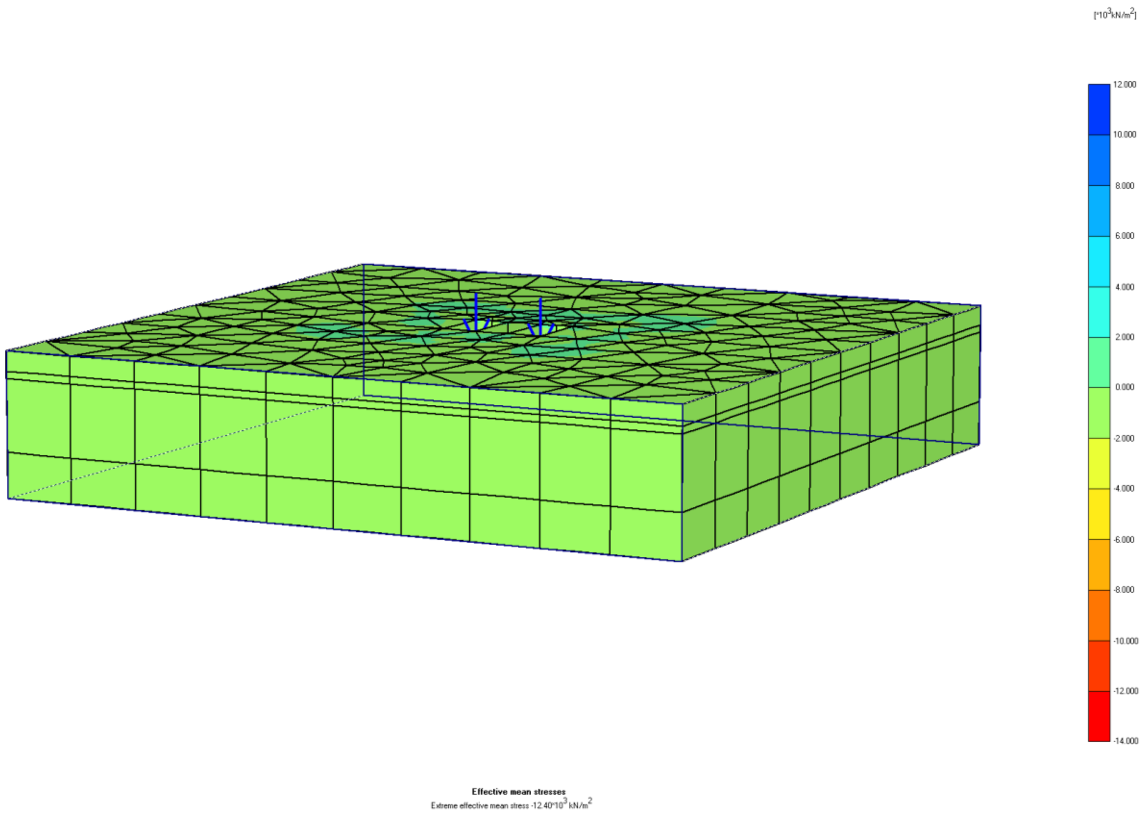

Figure A.3 2 effective mean stress for loose sand overlying dense sand

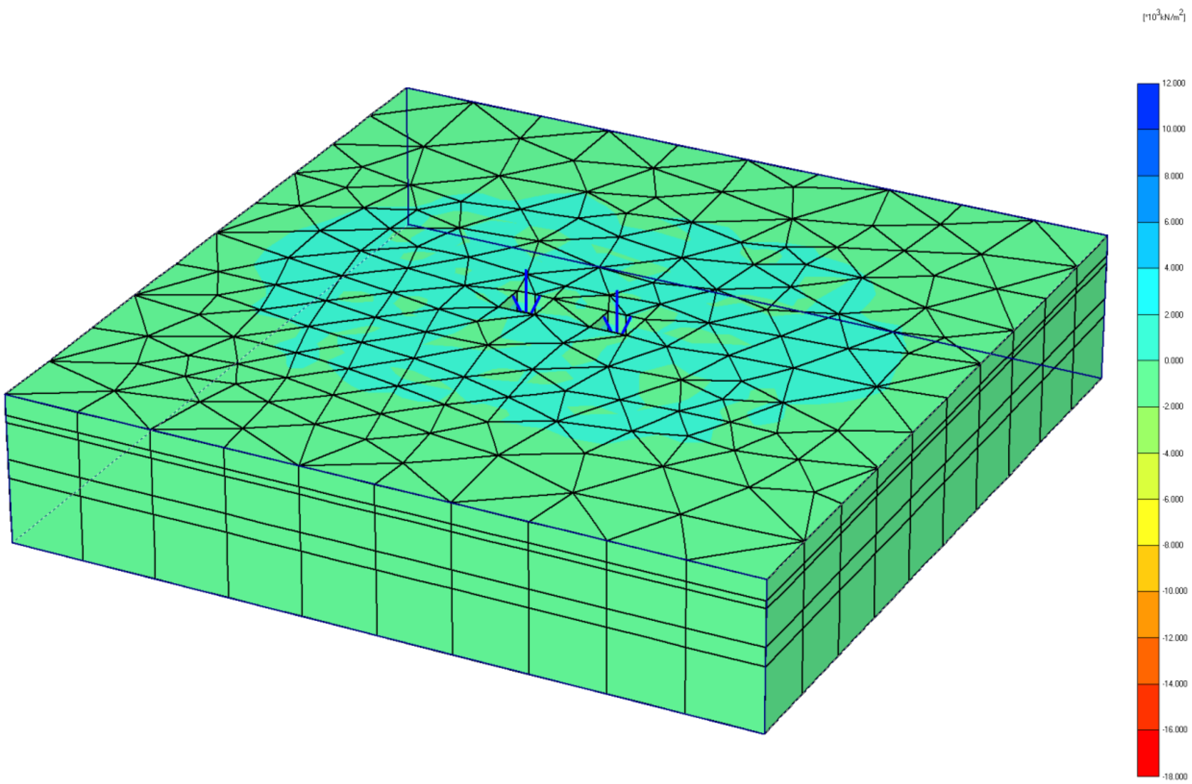

Figure A.3 3 effective mean stress for dense sand / stiff clay / soft clay