Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Evaluation of the Interference Effect and Settlement of Closely Spaced Footings of Shallow Foundation for Two Adjacent Buildings Using FEM Analysis

A Research submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering.

(Geotechnical Engineering)

By:

Dekebi Chakeri

May 2021

Jimma, Ethiopia

Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Evaluation of the Interference Effect and Settlement of Closely Spaced Footings of Shallow Foundation for Two Adjacent buildings Using FEM Analysis

A research submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering.

(Geotechnical Engineering).

By: Dekebi Chakeri

Advisor: Damtew Tsige (Ph.D.)

Co-Advisor: Alemineh Sorsa (Ph.D. Candidate)

May 2021

Jimma, Ethiopia

DECLARATION

I, the undersigned, declare that the thesis entitled "**Evaluation of the Interference Effect and Settlement of Closely Spaced Footings of Shallow Foundation for Two Adjacent buildings Using FEM analysis**" is my own original work and that it has not been presented and will not be presented by me to any other University for similar or any other degree award.

Dekebi Chakeri ______________ ____________________.

Researcher Signature Date

This Research has been submitted for examination with my approval as university supervisor.

Dr Damtew Tsige ______________________________ ________________
Advisor Signature Date

Alemineh Sorsa ______________________ _______________

Co-Advisor Signature Date

APPROVAL SHEET

I, the undersigned certify that the thesis entitled: **"Evaluation of the Interference Effect and Settlement of Closely Spaced Footings of Shallow Foundation for Two Adjacent buildings Using FEM analysis"** is the work of Dekebi Chakeri and has been accepted and submitted for examination with my approval as university advisor in partial fulfillment of the requirements for degree of Master of Science in Geotechnical Engineering.

Name	**Signature**	**Date**
1. Main Advisor: Damtew Tsige (PhD)	__________________	_________________
2. Co Advisor : Alemineh Sorsa (PhD Candidate)	______________	_________________

As member of Board of Examiners of the MSc Thesis Open Defense Examination, We certify that we have read, evaluated the thesis prepared by Dekebi Chakeri and examined the candidate. We recommended that the thesis could be accepted as fulfilling the thesis requirement for the Degree of Master of Science in Geotechnical Engineering.

1. Dr. Indalu Tadele (PhD) __________________________ __________________

   [External Examiner] [Signature] [Date]

2. Tigist Mezmur (MSc.) ____________________ __________________

   [Internal Examiner] [Signature] [Date]

3. Mohammed Yassin (MSc.) ____________________ __________________

   [Chair Person] [Signature] [Date]

ACKNOWLEDGEMENT

Above all, I would like to thank the almighty GOD for his unending support and grace upon me during all my works and in all my life. My deepest gratitude goes to my advisor Dr. Damtew Tsige and my co-advisor Alemineh Sorsa for all their limitless efforts in guiding me through my work and for providing me useful reference materials.

Finally, my deepest appreciation goes to Jimma University, School of Graduate Studies, Jimma Institute of Technology, Civil Engineering Department and Geotechnical Engineering Stream.

ABSTRACT

Industrialization and urbanization has run to situations where Constructions of buildings are close to each other for the reason of restricted available space that causes footings of same or adjacent structures come closer. This condition causes interference, causing the stress zones under the base of foundation to overlap, resulting in bearing capacity loss and excessive foundation settling. The problem of adjacent footings contact is extremely critical in practice. The convectional solutions for bearing capacity and soil settling under shallow foundations do not take into account the interference effects of footings. Therefore, the study aims to assess the behaviour of two closely placed footings on the two layered clay soil. The sample collected from the site for laboratory analysis following ASTM testing procedures. The result of interference of two footings in terms of bearing pressure, settlement and tilt is observed using finite element analysis. The hardening soil model used to model the foundation soil medium using the finite element analysis program plaxis 3D. The spacing between the footings, load and the depth of the footings varied in a parametric analysis. The effect of spacing with each case on the load-settlement characteristic, settlement variance, and bearing pressure investigated and the effect of water variation investigated. The results are presented in terms of non-dimensional efficiency factors, which defined as the ratio of settlement or bearing pressure of interfering footings to that of the isolated footing, in order to determine the interference effect on the performance of the adjacent building's closest footing. It is found that the interference effect on the overlapping footings decreases with increase in the spacing between the footings and gets isolated footing at greater spacing ($S \geq 5B_L$) and increases when the spacing between footings decreases. The maximum interference has been found at $S/B_L = 0.5$. Similarly, when the groundwater level varies, the interference effect of overlapping footings changed. It is observed that, when water depth varies from 2.20m to 2.60m the settlement effect reduces from 33% to 20%. Thus, water level from footing depth has negative influence on the interaction of footings.

Keywords: Shallow foundation, Interference effect, Settlement, Bearing pressure, Tilting, Clayey soil, Finite element analysis

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACRONYMS AND ABBREVIATIONS

AASHTO American Association of State Highway and Transportation Officials

ASTM American Society for Testing and Material

B_L Base of Left Footing

B_R Base of Right Footing

C Cohesion Pressure

Cc Compressive Index

CH Highly plastic Clay Soil

CL Low plastic Clay Soil

cm Centimeter

Cs Swelling Index

Df Depth of Footing

dx Distance in x- direction

dy Distance in y- direction

dz Distance in z- direction

E Elastic modulus

E_{50}^{ref} Triaxial Loading Stiffness Reference

E_{oed}^{ref} Oedometer Loading Stiffness Reference

E_{ur}^{ref} Triaxial Unloading and Reloading Stiffness Reference

e Void ratio

e_{int} Initial Void Ratio

FEM Finite Element Method

Gs Specific Gravity

HB Hoek Brown Model

HS Hardening Soil Model

HSS Hardening Soil Model with Small Strain Stiffness

JIT Jimma Institute of Technology

JR Jointed Rock Model

K_o^{nc} Coefficient of Lateral Strain in Normal Consolidation

Kpa Kilo Pascal

LE Linear Elastic model

LL Liquid Limit

MC Mohr Coulomb Model

MCC Modified Cam Clay Model

MH Highly plastic Silt Soil

ML Low plastic silt Soil

OCR Over Consolidation Ratio

OH Highly Plastic organic Soil

OL Low Plastic organic Soil

P Pressure

PC Preconsolidation Pressure

PI Plastic Index

PL plastic Limit

pL Pressure on left Footing

pR pressure on Right Footing

Rf Friction Ratio

S Clear Spacing

SS Soft Soil Model

SSC Soft Soil Creep Model

UCS Unified classification System

νur Unloading and Reloading Poisons Ratio

γ Unit Weight

α Footing Size Non-Dimensional Factor

αv Coefficient of compressibility

β Footing Load Non -Dimensional Factor

Δδ Vertical Induced Stress

δm Over Consolidation Margins

σ_3 Chamber Pressure/Minor principal Stress

σ_1 Major principal stress

Δσ Deviatoric Stress

ζ Efficiency Factor

ζδ Settlement Efficiency Factor

ζ_γ Bearing capacity Efficiency Factor

ζθ Tilting efficiency Factor

ζδL Settlement Efficiency factor for Left Footing

ζδR Settlement Efficiency Factor for Right Footing

ζ_γL Bearing Capacity Efficiency Factor for Left Footing

ζ_γR Bearing Capacity Efficiency Factor for Right Footing

ζθL Tilting Efficiency Factor for Left Footing

ζθR Tilting Efficiency Factor for Right Footing

ηm Micrometer

ϕ Friction Angle

Ψ Dilatancy angle

CHAPTER ONE

1. INTRODUCTION

1.1 Background

The most important components of any structure is foundation that transmits a super structural loads to underlying soil layer, decreasing total and differential settlements through over distributing load stress, avoiding possible movement of structures, resist wind-driven uplifting or overturning forces, lateral forces caused by soil movement, and water penetration and dampness (Salahudeen and Sadeeq, 2016b).

Shallow foundations are an essential part of a structure because they guide the load to the soil underneath it at a shallow depth. When the depth of the foundations is less than three meters, or less than the width of the footing, it is considered shallow. During design of shallow foundations, two considerations should be taken into account: the soil's bearing capacity and complete settlement. However, the design of shallow foundations is commonly controlled by settlement rather than bearing capacity(Shahin, et al., 2002).

The reason of shortage of construction sites, rapid urbanization, and structural constraints, the structure or group of foundations may be forced to come up close to one another. Because of these conditions, the stress zones beneath the foundations can overlap, causing distraction in the failure mechanism, settlement, and bearing capacity responses of the footings in comparison. The spacing between two adjacent foundations has a significant impact on their load settlement behaviour. In recent years, numerical/theoretical as well as experimental analysis many researchers (Kumar and Bhoi, 2009; Ghosh and Sharma, 2010; Noorzad and Manavirad, 2012; Alimardani and Ghazavi, 2012; Shahein and Hefdhallah, 2013; Nainegali *et al.*, 2013; Basudhar e*t al*, 2013; Ghosh *et al.*, 2015; Alwalan, 2018; Nainegali and Ekbote, 2019) have been specified on different features of the problem. However, it has been noted in the literature that the majority of studies have been conducted for two or more interfering footings resting on the surface of sand soil, with only a few attempts made for footings resting on clay as foundation. Hence, this study is carried out to perceive the effects of interference on the two close footings resting on the layered clayey soil and evaluate the problem of interaction between adjacent footings.

1.2 Statements of the problem

In fast growing and rapid urbanization cities, there are many constructions of buildings are close to each other for the reason of restricted available space that causes destruction in failure mechanism(Ghosh and Sharma, 2010). In foundation design, the designer depends on bearing capacity and settlement of soil and has no knowledge about the interference of closely spaced footings, which is significant for altering characteristics and the condition of footings. In recent some researchers have been pointing that there is a probability of building failures for closely spaced footing resting on clay soil. Nainegali, (2019) noticed that there is significant tilt arises for the footings placed very close to each other on clay soil. Hence, studying on this the problem of interaction between adjacent footings on clay soil is very important. Moreover, in most of our country's towns specially Jimma town is one of the mostly dominated by clay soil, and there is a possibility of building failures mechanism to be occur due to closeness spaced footings of shallow foundation. Therefore, for a closely spaced building, it requires knowledge on how the interference effect and settlement of closely spaced footings of shallow foundation for adjacent buildings change the condition of footings.

1.3 Research Questions

The research questions that this study attempt to answer the problems during the study period are:

1. Can subsurface characteristics of soil/rock and groundwater variation affects the characteristics of interference effect of closely spaced footings of buildings?
2. Can the geometry and loading have influence on closely spaced footings?
3. What are the sensitive parameters more affects characteristics of interference effect of closely spaced footings?

1.4 Objectives

1.4.1 General objective

The general objective of this study is to evaluation of the interference effect and settlement of closely spaced footings of shallow foundation for two adjacent buildings using FEM analysis.

1.4.2 Specific Objectives

Based on the above general objective, the following points need to be identified as the specific objectives of the study.

1. To characterize Geotechnical conditions of soil under the footings
2. To develop numerical model of shallow foundation on clay soil
3. To identify sensitive parameters that can cause geotechnical failure of closely spaced footings of two adjacent buildings

1.5 Scope of the study

This research addresses the interference effect and settlement of closely spaced footings of shallow foundation for two adjacent buildings. Samples collected from study area where the depth of soil affected by foundation bearing pressures, settlement, and the effective depth H of the influence zone below the loaded area from H is 0.0 to about 5B in order to limit stress distribution not beyond the influence zone. For the intended purpose, Atterberg limits, grain size analysis, specific gravity, consolidation tests and triaxial test conducted. After laboratory test, the engineering properties of soil analyzed using micro soft excel. The analyses depend on plaxis 3D program and perform the finite element computation. For this study, the soil modeled by using hardening soil model, hence, soils classified under medium to stiff soil and deals with the settlement characteristics of two closely spaced perfectly rough isolate footings resting on two-layered clay soil for the vertical load applied to the footing.

1.6 Significance of the Study

Most literatures show that shallow foundation settlement for interference effect and settlement of closely spaced footings have been analyzed by using plaxis 2D by considering Mohr coulomb model. As soil is sheared, Mohr-Coulomb idealization suggests dilation at constant rate, which is impractical. Soils on shearing have variable volume change characteristics based on pre-consolidation strain, which MC cannot account for. In a variety of geotechnical applications, this model can be used to predict displacement and failure for different types of soils (Brinkgreve, 2005). According to this research soil modeled using Hardening soil model in contrast to the Mohr coulomb model, it accounts for stress-dependency of stiffness moduli and dilation variation. Initial soil condition, such as pre-consolidation, play a crucial role in soil deformation problems can also account in initial stress generation of hardening soil model.

Additionally, predicting foundation settlement by analytical approach has some limitations with complex soil properties and these limitations minimized by using finite element method, Plaxis 3D, which is becoming most applicable in the worldwide for modeling and analyzing complex geotechnical problems.

The significance of this research is to recommend and a knowledge interference effect and settlement of closely spaced footings of shallow foundation for two adjacent building, which it helps to avoid risk factors that are associated with foundation design that assists designer to be accurate and reliable for foundation design and interference effect between footings. In addition to this, the study can be used as references for the researchers who want to Perseus their study in related area.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Foundation and Load Distribution

The foundation is a part of a structure that transfers the super structural loads to the underlying foundation materials, specifically soils. Foundations are essential structures for transmitting and distributing the supported loads into the underlying soil structure, maintain soil pressures at all depths within tolerable levels, resisting soil shear failure and limiting soil settlements within tolerable levels. It has also two general groups of foundation based on depth, which are shallow and deep foundations. A foundation having the embedment depth lesser than its width called shallow foundations and if its load transferred to deep foundation material through piers, piles, and drilled shafts, they called deep foundations (Nimeri, et al., 2017).

A shallow foundation is a structure responsible for transferring super loads to the ground from the supper structure. Therefore, determining the soil's ability to bear loads without causing interesting displacement between the structure and the ground underneath it is a critical step in the design process. Several theories have been developed to deal with the actions of shallow foundations bearing capacity, settlement, failure surface, and other factors that are commonly used in practice. However, these theories are only applicable if the shallow foundations in are isolated and there is no interaction between footings. In reality, however, foundations are often close together and not isolated. As a result, individual footings' characteristic behavior during a bunch can differ from that of an isolated one. Engineers are forced to build footings that interfere with each other in some cases, such as area limits, since the geometry of the building or structures near each other requires engineers to build footings that interfere with each other to meet requirements. If not properly managed, this intrusion quantitatively leads to excessive settlement and severe structural damage, particularly when the distance between the footings is reduced. The interaction of closely spaced shallow foundations within the term of stress and failure zone can trigger an unbalanced distribution of stress within the soil, When compared to single footing action, which affects the determination of bearing ability and settlement of footings resting on soil (Shahein and Hefdhallah, 2013).

The vertical stresses within the soil mass are increased when a load is applied to the soil surface. The maximum stress impact is immediately underneath the loaded region, but it extends in all directions indefinitely (Shahein and Hefdhallah, 2013).

The distribution of the pressure depends on the mechanical properties of the soil in combination with the stiffness of the foundation plate. The flexural rigidity of a footing plate, resting on soil, is often very large compared to the deformability of the material below it. Depending on interaction and flexural rigidity between soil and foundation materials, foundation can be classified as flexible and rigid foundations. The stress distribution within the soil is influenced by the relative rigidity of the foundation to the soil mass. The elastic solutions provided are for flexible loads and do not take into account the relative rigidity of the soil foundation system; however, if the foundation is rigid, the stress increases are considerably lower (15 to 30% less for clays and 20 to 30% less for sands) than those computed from the elastic solutions (Shahein and Hefdhallah, 2013).

2.2 Foundations' Settlements

The bearing pressure of a footing is used to predict the settlement under a structure. Since alternative neighboring footings are also affecting the settlement of the footing, the estimated value of the settlement may not be adequate. As a result, the impact of adjacent footings must be considered when estimating the settlement of a foothold (Aytekin, 2016).

Foundation settlement begins as soon as construction begins, gradually increases as the load is increased, and, if allowed, continues until the load per unit area equals ultimate bearing pressure, at which point the soil supporting the foundation fails (Aytekin, 2016).

Rainfall infiltration influenced the settlement behavior of shallow foundations. Because of the high modulus of elasticity as matric suction increases, the influences of rainfall infiltration on soils have obvious strengthening effects for the bearing capacity of shallow foundations with decreasing settlement. In unsaturated soils, rainfall intensity also plays a significant role in evaluating the settlement of shallow foundations. Rainfall infiltration and a lack of matric suction are thought to be the causes of the extra settlement. Because of changes in matric suction, changes in settlements during rainfall have a major impact on the location of the bottom water table near the ground surface. Furthermore, soils with smaller permeability functions have a higher bearing capacity in response to rainfall infiltration than soils with larger permeability functions (Kim, et al., 2017).

Structures may settle uniformly or unevenly, thus total and differential settlements are used to describe them. The difference in settlement under various sections of a single structure is referred to as differential settlement. Most structures can withstand large total settlements, but only to a certain extent when it comes to differential settlements. The maximum amount of permissible differential settlement depends on the type of building, the type of equipment housed within, and the length of time that the settlement exists. If the settlement is not held within acceptable limits, the structure's intended use may be harmed, and its design life may be limited (Grant, et al., 1974).

Settlement is the vertical displacement of the ground under load that occurs because of a rise in effective stress(Birand et al., 2002). The stiffer soils result from a reduction in volume due to a rise in effective stress. Dewatering is one mechanism for increasing successful stress. The ground loses its buoyant impact and raises its self-weight during dewatering. The application of a surcharge load is another mechanism for increasing effective stress. Due to serviceability constraints or in determining allowable bearing pressure, the settlement is a critical criterion in the design of foundation systems.

Allowable settlement refers to the maximum amount of total settlement that can be tolerated by various foundation forms in various soils. (Birand et al., 2002) states that the maximum permissible settlement for isolated footings on sand is 40 mm, for rafts up to 65 mm, and for isolated footings and rafts on clay, the corresponding values are 65 mm and 100 mm, respectively. Furthermore, the maximum differential settlements between isolated foundations on sand and clay could be set at 25 mm and 40 mm, respectively, as the design limit(Birand et al., 2002).

Allowable settlement is the allowed maximum amount of total settlement recommended to different foundation and soil types. Total settlement and differential settlement must be considered by the engineer for each structure, because designing of some structures are very sensitive to differential settlement (Lymon et al, 2006).

The overall settlement of a foundation contains three components: immediate (elastic), Primary consolidation and secondary settlement (creep) and differential (distortion) settlement. Total settlement is the summation of these above components depending on type of soils, needed computation and in this thesis elastic and primary consolidation settlements will be discussed in detail by considering soil behavior(Al-Taie, et al., 2016).

2.2.1 Elastic Settlement

Deformation of dry soil, as well as moist and saturated soils, causes elastic settlement without a change in moisture content. In general, when compared to consolidation settlement, the immediate settlement of footings on clay is minimal, particularly for normally consolidated soils. It should be important, particularly in the case of highly plastic clays or organic soils. Immediate settlement is also known as elastic settlement because it is often measured using linearly elastic soil models and various studies are carried out to predict elastic settlement on fine-grained soils. If the clay is soaked, it is fair to say that as soon as a load is added to it, it will settle immediately (Aytekin, 2016).

Immediate and long-term settlement checks are an integral a part of foundation design. Therefore, reasonably accurate estimates of the immediate settlement of shallow foundations touching on clay are necessary, particularly for highly plastic clays or organic soils, that the immediate settlement is also significant. Since there is no room for alteration within the clay volume in the short term, immediate settlement is entirely due to the distortion of the clay under the shallow foundations. Since the soil stress-strain response is nonlinear even at small strains, design procedures based on linear elasticity are unable to predict soil deformations accurately. Hence, an on the spot settlement analysis that takes soil nonlinearity under consideration is required(Foye, et al., 2008).

2.2.2. Primary Consolidation Settlement

Primary consolidation settlement is a time-dependent process that occurs in clayey soils below the groundwater table as a result of the volume change in the soil caused by the expulsion of water from the void spaces (Aytekin, 2016).

In the laboratory, Oedometer cell (consolidation Test) is used to measure compression (i.e., deformation) and consolidation (i.e., rate of excess pore water dissipation) properties. To evaluate the relationship between load and rate of deformation of soils, the applied load and specimen deformation were carefully measured (Dante, et al., 2014).

2.2.3 Secondary Consolidation Settlement

Secondary consolidation settlement occurs in saturated clayey soils after the primary consolidation phase and is caused by the plastic modification of soil fabrics (Aytekin, 2016).

Secondary consolidation settlement, also defined as creep settlement, is caused by the time-dependent rearrangement of soil particles over a long period under constant effective stress. Consolidation stops when excess hydrostatic pressures within a fine-grained substrate are completely dissipated, according to basic consolidation theory (K. Mitchell and Kenichi, 2005). Secondary compression is critical for highly organic and fragile soils including soft clays and peats (Dante, et al., 2014). However, some long- term settlement may occur in sand soils especially in case of the soil subjected to vibration load and large amounts of settlement can occur if very loose sand soils subjected to vibration (Lymon et al., 2006). Loose sands below the water table cannot lose water immediately during earthquake and as a result, it subjected to liquefaction, because of the water flow out is time dependent.

2.3 Factors Causing Foundation Failure

There are varieties of causes for foundation failure. However, the most known causes are soil moisture movement, which includes the ground water table and unequal sub-soil settlement. Subsoil lateral movement, Atmospheric Action (Atmospheric Action) is a term. In this study the ground water effect, footing depth effect and effect spacing of close footings are reviewed.

2.3.1 Depth of Foundation

Due to the compressibility characteristics of the subsoil, footings built at a depth "Df" below the surface settle under the applied loads. The distribution of effective vertical pressure of the overburden (due to soil unit weight) and vertical induced stress due to net foundation pressure determine the depth to which the soil is compressed. In the case of deep compressible soils, the point is the lowest amount taken into account in the settlement analysis. The vertical induced stress is about 0.1 to 0.2qb, where qb is the net pressure at the foundation's base, and this depth refers to 1.5 to 2 times the footing's width. Because of the applied foundation pressure, the soil within this depth is compressed, causing more than 80% of the structure's settlement. The region of significant stresses is a depth DS. If the thickness of this zone is more than 3 m, the settlement should have analyzed by dividing the layer at depth not more than 3 m(Murthy, 2007).

The depth to which boreholes should be bored is assessed by the depth of soil affected by foundation bearing pressures and settlement, as well as the effective depth H of the influence zone below the loaded region (between H = 0 and H = 4B or 5B) (Joseph E. Bowles, 1997). It is possible that the soil can flow laterally from under the existing footing if the new footing is lower than the existing footing for adjacent footings. This may result in some further excavation, but more importantly, it may cause settlement cracks in the existing structure (Joseph E. Bowles, 1997).

2.3.2 Effects of Ground Water Level on Settlement

The amount of water in the soil has an effect on the modulus of elasticity. Since water binds the particles and thus increases the effective stress among particles through suction, low water content in the soil leads to high soil modulus. As the water content in the soil increases, the effective stress of the soil decreases, and the modulus of elasticity decreases as well (Araza, et, al.,2011).

2.3.3 Effects Spacing of footings

Spacing between footings lead to interference that causes the stress zones below the foundation causing distraction of the building. For the past, several years' researchers have studied the interference effect and settlement of closely spaced footings of shallow foundation to avoid risk factors that are associated with foundation design that assists designer to be accurate and reliable for foundation design and interference effect between footings.

(Terzaghi, 1943); stated that bearing capacity of footing is influenced by the presence of adjacent footing. In the modern world, due to space constraint in the urban areas, tall structures in close proximity to each other have become a common sight. In such cases, the behavior of footings is different from an isolated footing due to inference in the pressures bulbs from adjacent footings. However, the bearing capacity can be much higher when the footings are close and resting on a finite layer of soil. The interference between the pressures bulbs are found to improve the load carrying capacity of the footing, which can be economized. Extensive studies have been carried out in the past on the bearing capacity of such footings.

(Stuart, 1962); examined the effect of closely continuously spaced foundations in cohesion less soil on the base of the limit equilibrium method. He assumed that the medium is homogeneous soil that extends to great depths, and that the failure mechanism has the same geometry as Terzaghi's rupture. Three zones have developed underneath the shallow foundation: the Rankine passive zone, the radial zone, and the triangular wedges. Based on the center-to-center distance between the shallow foundations, it is concluded that as long as the rupture surfaces only overlap in the Rankine passive region, Terzaghi's formula does not need to be modified and can be applied directly. However, at the ultimate loads, the value settlement relative to individual footings will change. If there is overlapping in the radial zone, it is important to adjust the bearing capacity. Stuart uses the efficiency factor (ζ), which is a function of the spacing to width of the foundations and the soil friction angle, in this case. Since the efficiency factor is greater than one, as the center-to-center spacing between foundations reduces, the ultimate bearing capacity increases. If compared to isolated foundations, however, settlement would be more important. Stuart's observations do not take into account the impact of different parameters; detailed studies are needed to include certain parameters, such as the variation of elastic modulus with depth.

(Amer, 1962); Studied the settlement and tilt of two interfering footings on clay soil by taking both the footings of width 30 cm, average settlements and tilt were computed, using the above described analysis for different S/B ratio varies from 0.5 to 3. Isolated footing was analyzed for comparison purposes. Pressure-settlement and pressure tilt curves for rough strip footings resting on clay were obtained. It is obtained for particular pressure intensity, settlement and tilt generally increases with the decrease in S/B ratio. The tilt of the interfering footings takes place toward the center of the system (i.e. they tilt towards each other). The value of tilt depends on the magnitude of pressures and relative spacing. Non-dimensional relationships at factors of safety 2 and 3 have been obtained to predict maximum and minimum settlements of interfering footings by investigating footings of width (B = 10 cm, 20 cm, 30 cm, 40 cm and 60 cm). Finally, concluded ultimate bearing capacity of interfering footings is almost same as of isolated footings in case of clay. Therefore, from shear failure consideration the interfering footings may be designed as isolated footing and magnitude of settlement and tilt of the interfering footings is affected by S/B ratios. Therefore, proportioning of interfering footings should be carried out by actual estimation of settlement and tilt. Tilt of interfering footings takes place toward the center of the system (i.e. they tilt towards each other).

(Shahein and Hefdhallah, 2013), studied influence of neighboring footings and the adjacent footings decrease rate of stress dissipation with depth under the footing and thus thicken the depth or zone that is influenced by the surface stress. It is possible that the thickening would require a soft compressible layer. The interior or middle footing has the greatest impact, while corner footing has the low effect. Increased settlement caused by a rising number of adjacent footings because of loading or a change in the location of the footing in the layout, as compared to an individual footing. Considering the soil profile and spacing between footings, the increase in settlement may be up to 4 to 5 times that of a single footing. Increased settlement due to the influence of adjacent footings will result in a breach of the foundation system's maximum permissible settlement, requiring a change from isolated footings to raft foundation to meet the settlement requirements. With the reduction in spacing between footings, the increase in settlement due to the influence of adjacent footings increases. When the influence of neighboring footings is taken into account, the maximum differential settlement between adjacent footings may be up to 1.5 times the settlement measured for individual footings. When the settlement of a footing under a structure is measured solely based on the bearing pressure of that footing, the estimated value of the settlement might not be accurate enough because the settlement of adjacent footings may also affect the footing under consideration. As a result, the effect of adjacent footings must be considered when estimating the settlement of a footing.

(Nainegali and Ekbote, 2019); studied the interference of two nearby footings placing on clay medium. On a homogeneous clay medium, studies on two nearby surface and embedded strip footings are conducted. He concluded that interference has very little impact on footings on pure clay medium as compared to UBC, but it has a considerable effect on bearing pressure and settlement measured within the allowed range. Furthermore, major tilt is observed for footings that are put very close together. There is a decrease in bearing pressure and an increase in settlement in the region of significant interference (S/B = 0.5) as compared to isolated footing, calculated at permissible settlement and pressure, respectively.

2.4 Foundation Settlement Analysis Methods

The design of shallow foundations normally controlled by settlement rather than bearing capacity (Shahin, et al., 2002). Therefore, settlement estimation is a major concern and a crucial criterion in the foundation design process. Consistent and precise settlement prediction has yet be achieved using a range of methods ranging from strictly empirical to complex nonlinear finite elements. The most widely used methods were empirical/analytical methods, Experimental method, and Numerical modeling/Finite Element Method.

Numerical modeling is a powerful mathematical method that allows it imaginable to solve complex engineering problems. A model is a structure or system built to denote a physical concept or phenomenon (Salahudeen and Sadeeq, 2016). The finite element method (FEM) is a well-known numerical analysis technique that is widely used in civil engineering applications for both study and design of real-world Engineering problems (Ornek *et al.*, 2012). Numerical analyses used to model the constitutive behavior of soils. One of the mathematical methods for dividing continuous media into finite elements with different geometries is the finite element method (Salahudeen *et al.*, 2017). It has the advantage of more rationally idealizing the material behavior of soil, which is non-linear with plastic deformations and stress-path dependent (Ornek *et al.*, 2012).

The finite element method is also useful for determining deformation configurations and stress distribution during deformation and at the ultimate state. The finite element method's capabilities allow it to model the construction method and investigate the behavior of shallow footings and the surrounding soil during the construction process, not just at the limit equilibrium conditions (Laman and Yildiz, 2007). The finite element method (FEM) permits modeling complicated nonlinear soil behaviour using constitutive model, various geometrics with different boundary conditions and interfaces. It is capable of predicting the stresses, deformations, and pore pressures of a given soil profile (Subrahamaniam, 2011). Finite element analysis divides a continuous problem governed by differential equations into a finite number of parts (elements) that are definite by a finite number of parameters. A discretized finite element problem with uncertain nodal values was created from a persistent physical problem. Nodal values can be used to recover values within finite elements (Izumi and Sonohara, 2008).

CHAPTER THREE

3. MATERIALS AND METHODS

3.1 Study Area Description

The study area in Jimma Town, of the Oromia Region enclosed by Jimma Zone. It is situated 346 km south-west of Addis Ababa, with a latitude and longitude of 7°40′N 36°50′E and an average altitude of 1760 m above mean sea level. According to the Köppen climate classification, Jimma has a tropical rainforest climate. It has a long wet season that lasts from March to October every year. The average high and low temperatures are 27° and 17° Celsius, respectively, with an average rainfall of 1500 mm (Yonas, 2002).

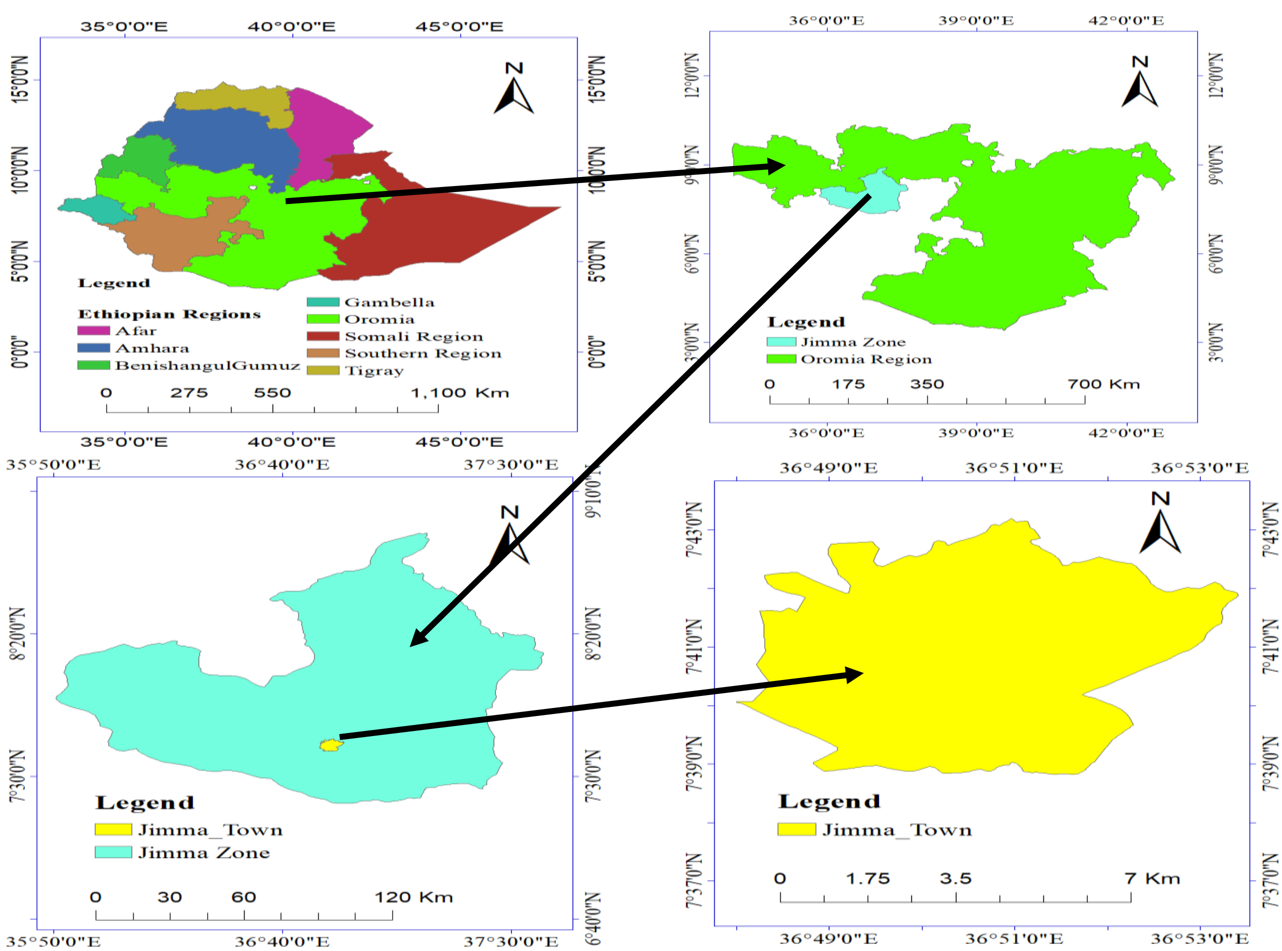


Figure 3.1: Study area map

In geological terms, the town is underlain by Tertiary volcanic rocks, most of which appear to be basalt. The area's rock unit is made up of medium to acid lava, forming the so-called traps formation (Yonas, 2002).

The study site characterized as fine-grained soils and two main types. The first type is red brown clay soil at the top and, massive dark grey clay soils formation have color ranging from gray to dark below the first layer appeared.

Figure 3.2: Study site

3.2 Study Design

This research started with a review of books, journal articles, and papers that established a basis of knowledge in this area. The data was gathered on the site, and several field visits (reconnaissance) were made to learn about the real situations that occurred in the workplace, which could be integrated into the analysis during the studies. The study accompanied by using both descriptive and analytical methods to describe the engineering properties of soil. This means that the methodology used in the work is the laboratory analysis of sample data and collected from the study area.

In this study, the soil tests conducted by using American Society for Testing and Materials (ASTM) a standard method of laboratory test procedures. Undisturbed samples used for determination of shear strength parameters; consolidation tests and triaxial test whereas disturbed samples used to conduct index properties tests such as specific gravity, liquid limit, plastic limit, plasticity index, and grain size analysis.

Microsoft excel and Microsoft word were used to conduct discussions on laboratory test results graphs and tables. The soil model was created with the plaxis 3D software, and the graphs were created with the Origin pro software. Finally, a comprehensive conclusion and recommendation about the results were made. The study's overall flow chart is depicted in Figure 3.3.

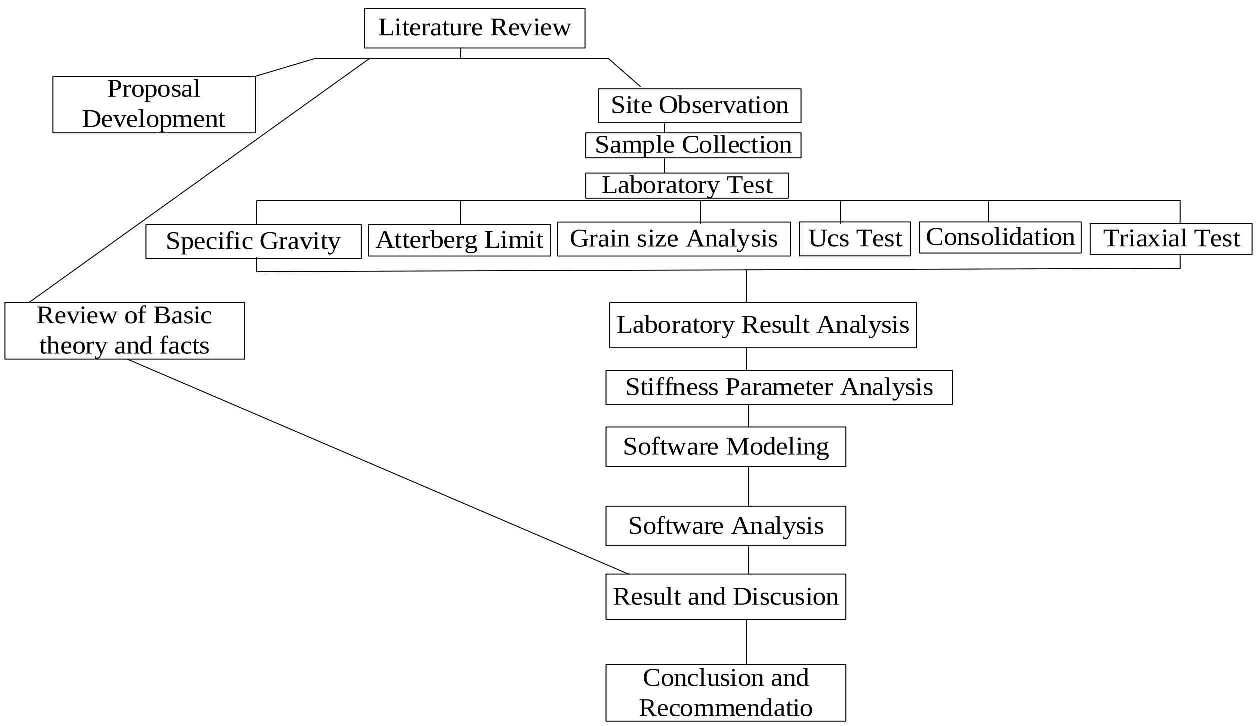


Figure 3.3: General Flow chart of the study

3.3 Data Collection Process

Soil samples collected for identification, classification and geotechnical engineering characterization. In this study, soil sample collected during excavation process from the study area and transported to Geotechnical Engineering Soil laboratory, Jimma Institute of Technology (JIT), and Ethiopian Construction Design and Supervision Works Corporation (Addis Ababa). Before selecting sampling areas, visual site investigation and information from administrator, residents and construction organization were collected to consider soil types and to take sample.

3.4 Laboratory Test Program

Samples collected from excavated test pits used to obtain index and engineering property of the soils. Collected samples used to classify soils based on USCS standards, for particle size distribution, to determine Atterberg limits (liquid limit, plasticity limit and plasticity index), specific gravity, 1D consolidation test and Triaxial test.

3.4.1 Standard Testing Procedures

The ASTM standards specified in Table 3-1 were used to establish the laboratory specifications for this study.

Table 3-1: Standard soil testing procedures

Test Type	ASTM	Sample Condition
Specific gravity	D 854-02	Disturbed
Grain size	D 422-98	Disturbed
Atterberg limit	D 4318-00	Disturbed
Consolidation test	D2435	Undisturbed
Unconfined compressive strength	D 2166-00	Undisturbed
Triaxial compression test	D 2850-99	Undisturbed

3.4.2 Specific gravity of soil

The specific gravity (Gs) of soil was calculated for use in the hydrometer method's particle size analysis and was evaluated using ASTM D854-02.

3.4.3 Grain Size Analysis

The mechanical sieving method and the hydrometer method are two methods to determine grain size measurement. Particles larger than 75 m are identified using sieve size, while particles smaller than 75 m are identified using the hydrometer test. For this research, both the wet sieve condition and the hydrometer method were used in accordance with ASTM D 422-98. The overall particle size distribution curve was generated by combining the results from the two methods.

3.4.4 Atterberg Limit

The plastic limit (PL), liquid limit (LL), and plasticity index (PI) of the soil were determined using this test. The ASTM D 4318-00 standard laboratory test procedure was used in this research. The plastic limit is the moisture content at which the soil paste begins to crumble when rolled to a diameter of 3 mm. The soil sample for the plastic limit test pass through a 425-m sieve, and the plastic limit is the moisture content at which the soil paste begins to crumble when rolled to a diameter of 3 mm. The liquid limit is assessed using the standard cup, which involves cutting a groove in the soil sample and raising and dropping the sample cup a given number of times before the groove barely closes. The liquid limit is the soil moisture content at which this occurs. The difference between LL and PL is the plasticity index (PI).

3.4.5 Consolidation test

This test was performed using (ASTM D2435) to calculate the rate and magnitude of soil consolidation and to predict the states of soil when the soil is restrained laterally and loaded axially. The test is One-dimensional compression test. The consolidation parameters was obtained by this test will use to determine the consolidation settlement and time of consolidation for a given loading state.

3.4.6 Unconfined Compressive Strength

On cohesive materials, the unconfined compression test can be performed. The aim of this lab is to determine the unconfined compressive strength of a cohesive soil sample, which used to measure the clay's unconsolidated undrained shear strength under unconfined conditions and to identify the stiffness properties of the soil. It was carried out using ASTM D 2166-00 procedures and the unconfined compression strength apparatus.

3.4.7 Triaxial Compression Test

The triaxial test is one of the most effective laboratory methods for assessing soil shear strength. The soil's unconsolidated undrained compressive strength was measured in an undisturbed state using a strain-controlled axial compression load with confining pressure. It was carried out using ASTM D 2850-99 procedures and a triaxial compression test apparatus.

3.5 Numerical modeling

Plaxis, a finite element program, was used to perform a 3-D non-linear finite element analysis of foundation settlement. Plaxis uses data from processed laboratory test results as input. Tables 3-2 and 3-3 show the soil properties and material properties of the footing used in numerical analysis and general computations, respectively. Soil layers were defined by means of boreholes which is a method specific with Plaxis 3D. Boreholes are used to enter data about the soil layers and water table. Boreholes are points in the drawing region where details about soil layers and the water table can be found. Structures were defined in horizontal work planes (Plaxis Material Models Manual, 2013). Plaxis 3D Manual contains more information on this topic.

3.5.1 Soil Elements

The soil layers was modeled by 6-node or 15-node triangular elements available for deformation and stress in soil in plaxis 2D. For displacements, the 15-node triangle has a fourth order interpolation and twelve stress points. Consequently, the 15-node triangle is more useful for complex problems, but it consumes more memory and performs calculations and operations slightly slower. The 15-node elements are more recommended (Brinkgreve, 2005). However, the quadratic tetrahedral 10-node elements are available in plaxis 3D (Plaxis 3D Manual, 2013).

3.5.2 Calculation types

Plastic calculation, Consolidation analysis, and Phi-c reduction are the three forms of calculations available in Plaxis. If the user wants to do an elastoplastic deformation study without taking into account the magnitude of excess pore pressures over time, they can choose the Plastic calculation. Time effects are not taken into account in a plastic calculation. With soft soils, a plastic calculation may be used, but the loading history and consolidation cannot be tracked; instead, an accurate estimate of the final situation is given. When it is important to track the evolution of excess pore pressure in soft soils over time, the consolidation analysis should be used. The phi-c reduction in Plaxis is a safety analysis that is useful when the situation in the problem necessitates the estimation of the safety factor. A safety analysis may be performed after each individual calculation step, but it is preferable to do so after all calculation phases have been established. It is particularly unwise to begin the calculation with a safety analogue (Brinkgreve, 2005). In this research, the pressure vs. settlement of footings was calculated using the plastic calculation phase.

3.5.3 Material Modeling

In modeling, it is essential to choose a soil constitutive model that is appropriate for the problem (Ti *et al.*, 1971). It is difficult to choose the most suitable soil model for use in numerical modeling. As a result, each model's concepts, advantages, limitations, and performance should be understood for each problem being modeled. It should also be familiar with the use of a constitutive model, which can be used to suit data from a variety of laboratory tests.

Based on unconfined compressive strength results, soil was categorized as medium to stiff clay and medium clay soil for layer 1 and layer 2 respectively in this report. The hardening soil model (HS) is the most effective soil model for medium and stiff soil in their numerical modeling. As a result, the HS model accounts for the soil's stress-dependent stiffness and overcomes the drawbacks of dilatancy and neutral loading. The stiffness of a soil can increase after loading due to the densification of soil particles, resulting in a lower volume of voids. As plastic deformation starts, this phenomenon is observed. The HS model takes into account the expansion and contraction of the shear yield surface, as well as the addition of a volumetric cap yield surface that causes soil stiffness to decrease as a result of high amplitude strain (Verghese, et al, 2013).

3.5.4 Parameters for the Soil Hardening Model

The triaxial loading stiffness E_{50}, the unloading-reloading stiffness E_{ur}, and the oedometer loading stiffness E_{oed} are used to describe the soil stiffness. These three input stiffness values are all related to a reference stress of 100 kN/m^2 (Brinkgreve and Vermeer 1998).

3.5.4.1 Elastic Soil Stiffness Parameter (E_{oed}^{ref})

Plaxis recommends using a simple correlation for determining E_{oed}^{ref} in the absence of a soil oedometer test, namely Eoedref E^{ref}_{50}. However, for this study, the E^{ref}_{oed} is obtained from the oedometer test. It is the slope of tangent Oedometric modulus in odometer test at the vertical stress σ_{ref} =100kN/m^2(Plaxis Material Models Manual, 2013).

3.5.4.2 Plastic Soil Stiffness Parameters (E_{50}^{ref} and E_{ur}^{ref})

Plastic soil stiffness parameters E_{50}^{ref} and E_{ur}^{ref} calculated based on confining pressure from the results of the triaxial compression tests. E^{ref}_{50} calculated as the ratio of 50% peak deviatoric stress to 50% strain(Wu, 2019) and E_{ur}^{ref} =3 E^{ref}_{50}(Plaxis Material Models Manual, 2013).

3.5.4.3 Angle of Dilation (ψ)

Dilatancy angle be observed in laboratory tests for cohesive soils, it can be assumed that dilatancy depends on the preconsolidation state. Hence, dilatancy angle can be taken as (Obrzud and Truty, 2018):

$\psi = 0$ ….. For normally and lightly over consolidated soil

$\psi = \frac{\phi}{6}$ …… For over consolidated soil

$\psi = \frac{\phi}{3}$ …. For heavily overconsolidated soil

3.5.4.4 Advanced parameters

v_{ur} is pure elastic parameter and its value used as 0.2 default setting and Ko^{NC} is an independent input parameters and plaxis uses as a default setting k_o^{NC} =1 –sinϕ (Plaxis Material Models Manual , 2013).

3.5.4.5 Dilatancy cut-off parameters

In this study, the drainage type selected as undrained (A) type. Hence, during undrained soil model void ratio remain constant and dilatancy cut-off does not help a dilatancy angle(Plaxis Material Models Manual , 2013).

3.5.4.6 Failure ratio parameter

Plaxis used an average value of the failure ratio as R_f = 0.9 default value(Plaxis Material Models Manual, 2013).

3.5.5 Soil Parameters for the models

The summary of the input parameters for software analysis from laboratory test results (oedometer and triaxial tests) used in this paper were numerated in following Table 3-2.

Table 3-2: Hardening soil model input parameters and method to obtain

Parameters	Soil type	
	layer 1/Reddish Brown	layer 2/Dark grey
C’	25.64	29.40
Φ’	8.58	6.54
Ψ	1.43	0
E_{50}^{ref} (mpa)	1.864	2.139
E_{ur}^{ref}(mpa)	5.592	6.418
E_{oed}^{ref}(mpa)	3.442	4.515
ν_{ur}	0.2	0.2
M(power)	1.00	1.00
K_o^{nc}	0.990	0.886
R_f	0.9	0.9
ɣb	17.23	15.99

3.5.6 Structural parameters

In Engineering, it acknowledged that the size and shape of a footing might have influences on the bearing pressure and settling characteristics of a footing. However, in in the construction of building opposed to rectangular footings the circular footings are used, hence the circular footings are more cost effective (Luévanos Rojas, 2016). Where as in direct vertical bearing loads, a square footing typically used because it uniformly distributes the load on the ground and provides the most stability to the column. When lateral loads are present, forces move from side to side and the shear wall is required to resist such forces. The shear wall, like the supporting footing, will have a length. For this case, it makes sense to have a rectangular footing. The square footing had higher bearing pressure than the circular footing, which may be contributed to the square footing's more confining impact, as the area of a square is greater than the area of a circle for the same lateral size (Patel and Bhoi, 2019).

During settlement, response of a smaller foundation width is less than that of a larger foundation width. (Kim, et al., 2017) expressed that larger foundation sizes exhibit higher settlement than smaller foundation sizes, however, as (Briaud, 2010) indicated, the size impact can be mitigated by creating load-normalized settlement. The different footing sizes of 1.0m * 1.0m, 1.5m * 1.5m, 2.5m * 2.5m and 3m * 3m were compared in the literature under different loading pressures. The result demonstrates that the 3.0m * 3.0m answer has a higher settlement value than the others. In the literature, (Murthy, 2002) also assessed the impact of this different footing sizes on settlement, demonstrating that a larger footing size results in a larger settlement for a given increase in soil pressure.

For this study, a square shape footing of size 2000mm (length) x 2000mm (width) located on the soil model selected, hence the direct vertical bearing loads is considered. The footing size and loading were varied based on sensitive parameters in numerical analysis for the case of assymmetrical condition. The concrete footings are assumed linear elastic with a Young's modulus of $24*10^6$ kN/m^2 and a Poisson ratio of 0.2.

Table 3-3: Structural parameters in numerical analysis

Parameters	**Unit**	**Values**
Unit weight	kN/m^3	24
Young's modulus	kN/m^2	$24*10^6$
Poisson's ratio	-	0.20
Material behavior	-	Linear (Isotropic)

3.5.7 Geometric Modeling

The typical square foundation placed on layered soil and subjected to vertical load is as presented in Figure 3.3. The thickness of top soil layer '5m' and the second soil layer '4m'. In the site, ground water was encountered at a shallower depth. As showed on figure 3.2, the ground water table displayed at the time of boring is located at a depth of 2.20m to 2.26m below the naturally existing ground surface. Hence, for the model simulation, the depth of ground water table considered and used for software analysis. The software analysis was accomplished by performing multiple trials to compute the settlement at the center of the footings by varying the domain size as well as the element size.

3.5.8 Loading condition

Loads are an important consideration in any building design because they describe the nature and magnitude of external forces that a structure must withstand in order to provide acceptable performance over its useful life. The intended use, configuration, and location of a building all affect the projected loads. Therefore, it is critical to apply design loads in a practical manner that the soil below the footings need to perceive the load safely. However, in this study in order to overcome this condition the load checked by increasing until the soil gets to be collapsed and has been optimized to 1000kN/m^2. In addition based on this load value, the footings condition have been studied for different cases of footing geometry. The geometry of the footing for symmetrical footing was ($B = B_L = B_R$ and loadings was $P = P_L = P_R$). In addition, asymmetric footings have been studied and asymmetric loading ($P_R = \beta \times P_L$, where β is the non-dimensional parameter) are the two loads that are applied to the width footings on the left and right sides B_L and B_R ($B_R = \alpha \times B_L$, where α is the non-dimensional parameter) respectively. The values of non-dimensional factors have been changed based on the cases study.

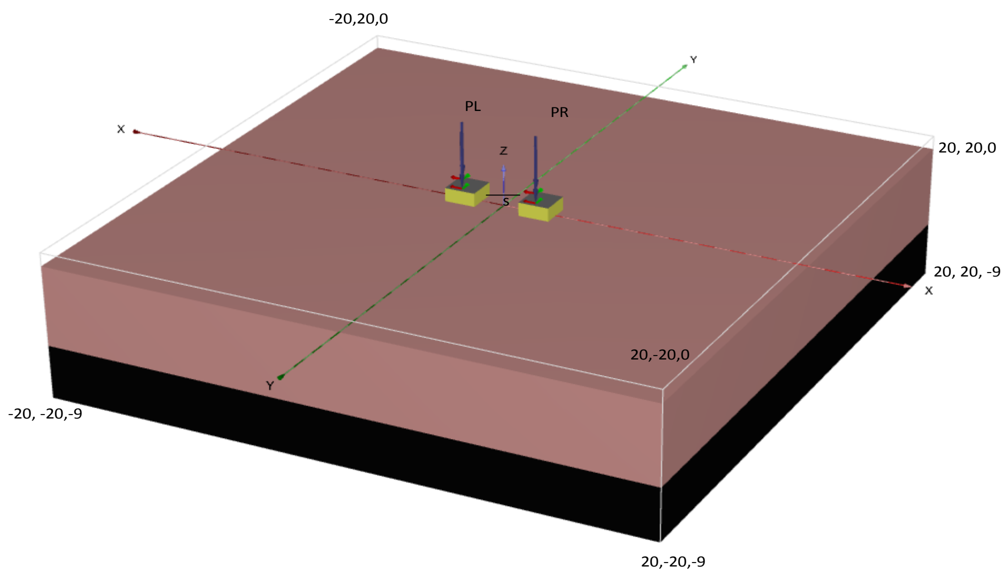


Figure 3.3: Geometrical Modeling of Footing

3.5.9 Finite element mesh, domain, and boundary conditions

Analyses are conducted using a defined soil domain (in the x, y, and z directions), and the length of the footing is sufficient in comparison to its width of the problem is defined in plane strain. The finite element meshes' lateral and bottom boundaries were set at 10B horizontally and 4B/5B vertically from the origin of coordinates.

Plaxis 3D, a commercially available finite element program, can now be used to perform two-dimensional finite element analysis. The domain and mesh size have a major impact on the FEM-calculated solutions. A detailed analysis must first be performed to correct the domain and subdivide the preferred domain into a finite number of elements in order to achieve a convergent solution.

As shown in Figure 3.4, boundaries for the soil domain were chosen at an acceptable long distance away from the edges of the footings on all sides. The vertical boundary is assumed unrestricted in the vertical direction but restricted in the horizontal. In both vertical and horizontal directions, the bottom horizontal boundary was restricted. Meanwhile there are no interface elements between the soil and the footing, any slippage between the two takes place within the soil. Since concrete footings poured against the ground shape a very rough interface, this is practical.

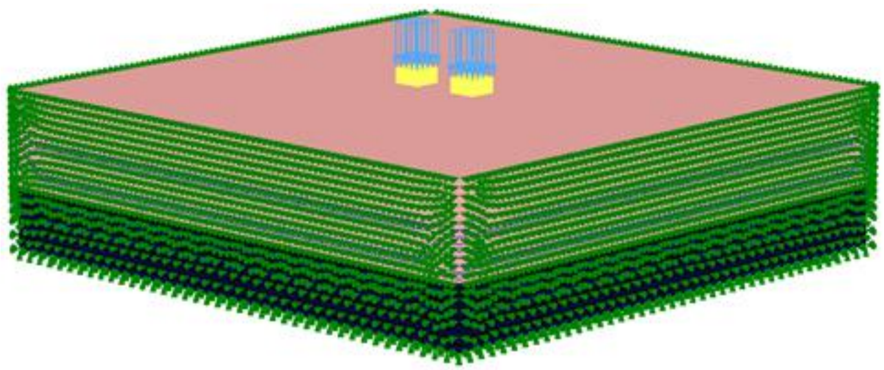


Figure 3.4: Geometrical Fixity of Model

3.6 Sensitivity Analysis of Parameters

The sensitivity analysis was accepted in order to determine the optimal domain and element sizes beyond which their effect on the computed results is insignificant and thus can be isolated. Depending on the footing geometry and loading, the following condition are considered:

1. Symmetrical footings and symmetrical loading (S/BL = 0.5, 1.5, 2.5, 3.5.5 and D/B =0.0, 1.0 (α =1; β/α = 1))
2. Symmetrical footings and asymmetrical loading (α=1;β/α =1.25, 1.50, 1.75, and 2.00);
3. Asymmetrical footings and symmetrical loading (α=1.5, 2.0; β/α=1);
4. Asymmetrical footings and Asymmetrical loading (α=1.5, 2.0; β/α= 1.50 and 2.00).
5. Effects of Ground Water Level on footing Settlement (2.20m and 2.60m).

CHAPTER FOUR

4. RESULT AND DISCUSSION

4.1 The Characterization of Geotechnical Conditions of Soil

4.1.1 Introduction

The investigation based on the results of soil tests in order to characterize the engineering properties of the soil where the foundation footing was laid. Specific gravity, Atterberg limits, unconfined compressive strength, consolidation tests, and triaxial test results analysis are the most relevant soil tests to recognize and classify the soil at which footings are placed.

4.1.2 Subsurface visualization

The subsurface characteristics of vertical profile under which the footing laid is shown in Figure 4.1. From investigation, it is identified that the layer comprises three different soil materials, namely, fill materials, red brown clay soil and dark gray clay soil. It is observed that, the ground water level varies from 2.2m to 2.6m.

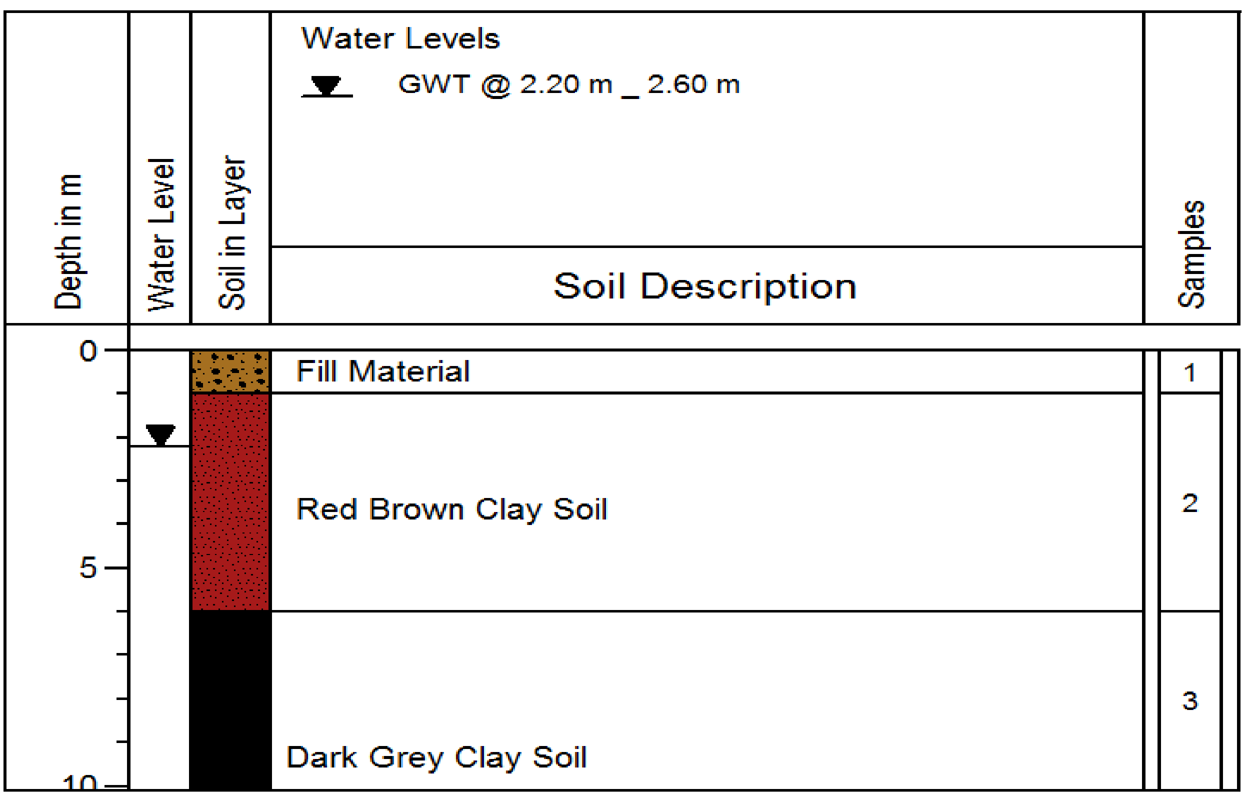


Figure 4.1: Soil profile in layers (using Geographical soil boring software)

4.1.2 Laboratory Test Results and discussion

4.1.2.1 Specific Gravity

The results for specific gravity test for soil layer one and layer two is shown in Table 4-1. The values obtained from the result can be used to identifying the soil type, classification, and for calculating consolidation test parameter. Test results detail presented in Appendix I.

Table 4-1: Specific gravity test results

Soil in layer	Specific Gravity (Gs)
Soil layer 1/Reddish brown	2.74
Soil layer 2/dark grey	2.65

The specific gravity of greatest soils under investigation lies within a narrow range of 2.5-2.85. However, organic soil containing specific gravity values such as 2.3 or less. On the other hand, soils containing iron oxide may have values above 3.0. So, from the specific gravity value of Table 4-1, the soils can be categorized as inorganic soils since their Gs values are greater than 2.5 - 2.85.

4.1.2.2 Grain size analysis

According to ASTM D422-98, grain size was determined using both wet-sieve and hydrometer methods. The combined results of the two methods, as well as the particle size distribution curves, are depicted in Figure 4.2, with further information available in Appendix II. The graph shows that the soil samples were graded as fine-grained soils, with over 90% passing the #200 sieve in both cases.

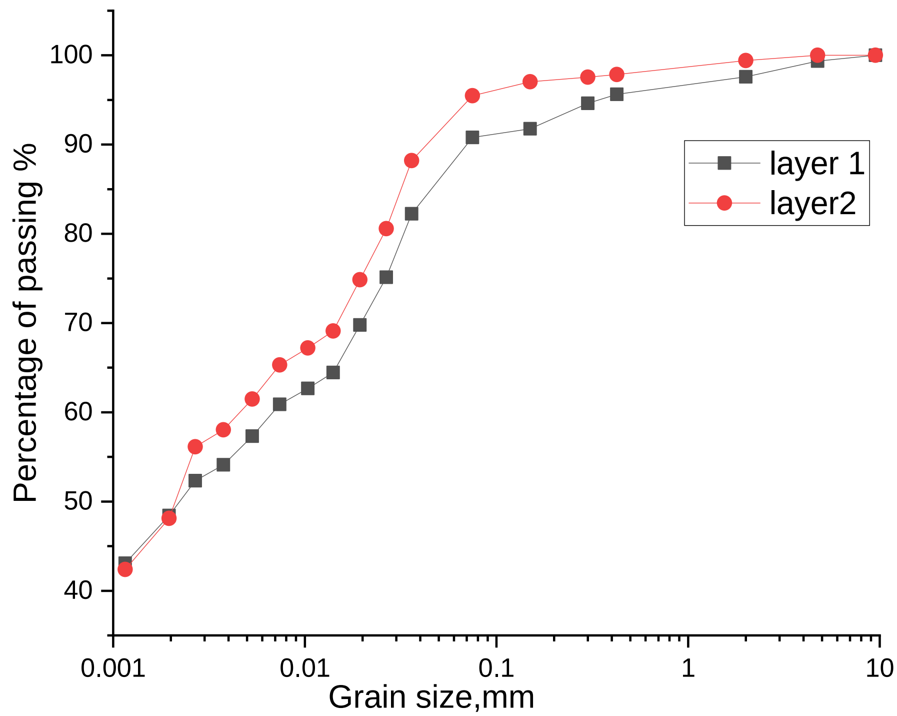


Figure 4.2: Summary of grain size distribution curves for both layers

In both cases, more than 90% of the total mass passes through a sieve size of 75m, as shown in Figure 4.2, as the results of grain size analysis. This indicates that almost all samples are fine-grained soil. From hydrometer analysis the size of more than 46% are less than 5ηm clay content as per ASTM in both soil layer 1 and 2. This gives shows the study area are highly covered with the clay soil.

4.1.2.3 Atterberg Limits

Table 4.2 shows the results of atterberg limit tests (liquid limit, plastic limit, and plastic index) performed in the laboratory for soil layers one and two. The portions of the samples that passed through the No. 40 (0.425mm) sieve used to prepare the sample for this analysis according to ASTM D4318 -98 (Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). The result is summarized in Table 4-2, while the detailed investigation is presented in appendix-III.

Table 4-2**:** Atterberg Limit test results

Soil in layer	Liquid limit (LL)	Plastic Limit(PL)	Plastic Index(PI)
Soil layer 1	63	29	34
Soil layer 2	75	32	43

This test results shown in table 4.2 indicates the soil have high liquid limit and plastic values. However, the values of soil layer 2 greater than soil layer 1 that indicates the second layer have more plasticity behavior than the first layer. In both cases, the values of liquid limit lies within range of 60 – 120 and plastic limit within range of 35 - 60, which indicates soil is Illite clay mineral type. From plasticity, and liquidity of Table 4-3, it can be said that the soil in its natural state is highly plastic fat clay in both layer.

Table 4-3: Atterberg limits and related parameters for various clay minerals (Glendinning, et al., 2015).

Clay Mineral Type	**Liquid Limit %**	**Plastic Limit %**
Montmorillonite	100 - 900	50 -100
Nontronite	37 - 72	19 – 27
Illite	60 - 120	35 – 60
Kaolinite	30 - 110	25 – 60
Hydrated Halloysite	50 – 70	47 – 60
Dehydrated Halloysite	33 - 55	30 – 45
Attapulgite	44 - 47	36 – 40
Chlorite	200 - 250	130 – 140

4.1.2.4 Soil classification

Figure 4.3 indicates the soil plasticity chart for fine-grained soil based on unified soil classification system (USCS), from the figure, it is shown that the soil type for layer one and layer two classified as highly plasticity clay (CH). This type of soil is more vulnerable for volume variation, creating settlement causes damage to the structure

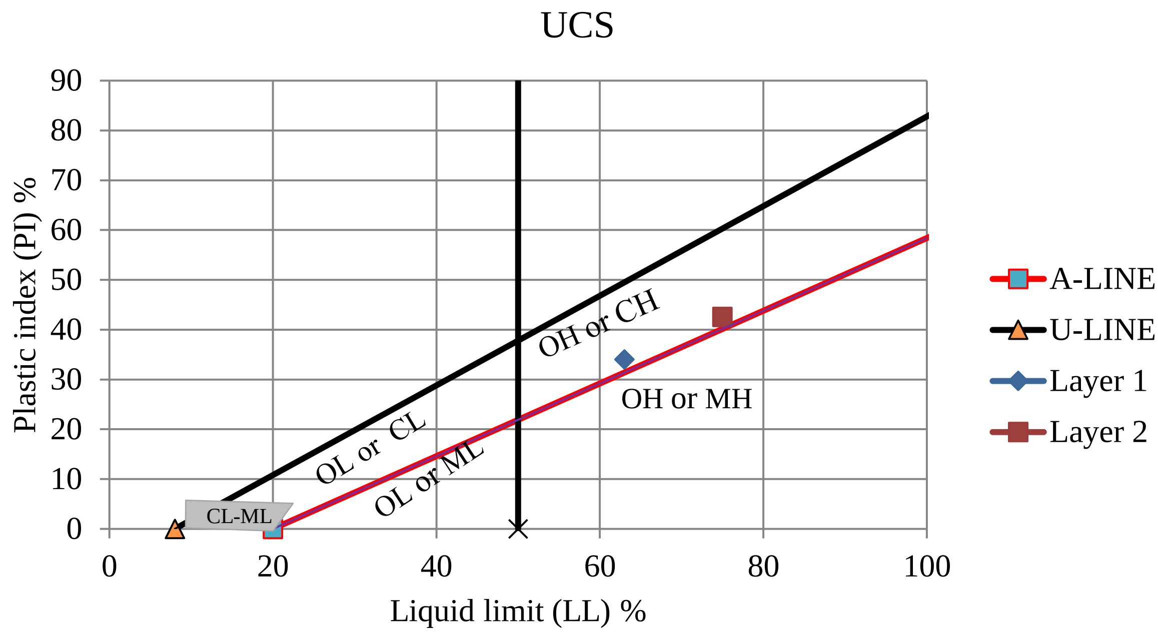


Figure 4.3: UCS soil classification

4.1.2.5 Unconfined Compression test

Shear strength is determined by two soil parameters: cohesion (inter-particle attraction) and angle of internal friction (inter-particle slip resistance). Unconfined compression test results for layer one and two are illustrated in Table 4-4.

Table 4-4: Unconfined Compression test results

Soil layer	qu (kpa)
Soil layer 1	109.84
Soil layer 2	86.80

The results from Table 4-4 show soil in first layer as medium to stiff clay soil and in second layer as medium clay soil based on Consistency and Unconfined Compression strength of Clays. Table 4-5 shows the general relationship between clay consistency and unconfined compression strength, which can be seen that clay shear strength is highly dependent on consistency. Clayey soils exhibits to hold water and difficult to drain. It make difficulty in dewatering in clay soil and high moisture clay makes to flow and tends catastrophic to the structures. The detail is presented in appendix IV.

Table 4-5: Clay's consistency and unconfined compression strength (Das, B., 1997)

Consistency	**qu (kN/m^2)**
Very Soft	0 - 24
Soft	24 – 48
Medium	48 - 96
Stiff	96 - 192
Very Stiff	192 - 383
Hard	>383

4.1.2.6 Consolidation test Results

This test is conducted to determine the rate and magnitude of soil consolidation and to predict the states of soil. The one dimensional consolidation test used to obtain compression parameters and consolidation parameter to predict the rate of settlement of structures. The pre-consolidation pressure p_c and the OCR also determined from this test. The detail is presented in Appendix V.

Pressure – void ratio curve

The pressure-void ratio curve can be achieved by calculating the void ratio of the sample at the end of each increment of load. The basic data used to determine this curve are natural moisture content, Specific gravity, density, cross sectional area and height of the sample, initial void ratio and applied loads. From these curve important parameters such as coefficient of compressibility (a_v), compression indexes (Cc), Swelling index (Cs) and pre-consolidation pressure (p_c) are determined. The summary of test results is presented in Table 4-6 and Figure 4.4 below.

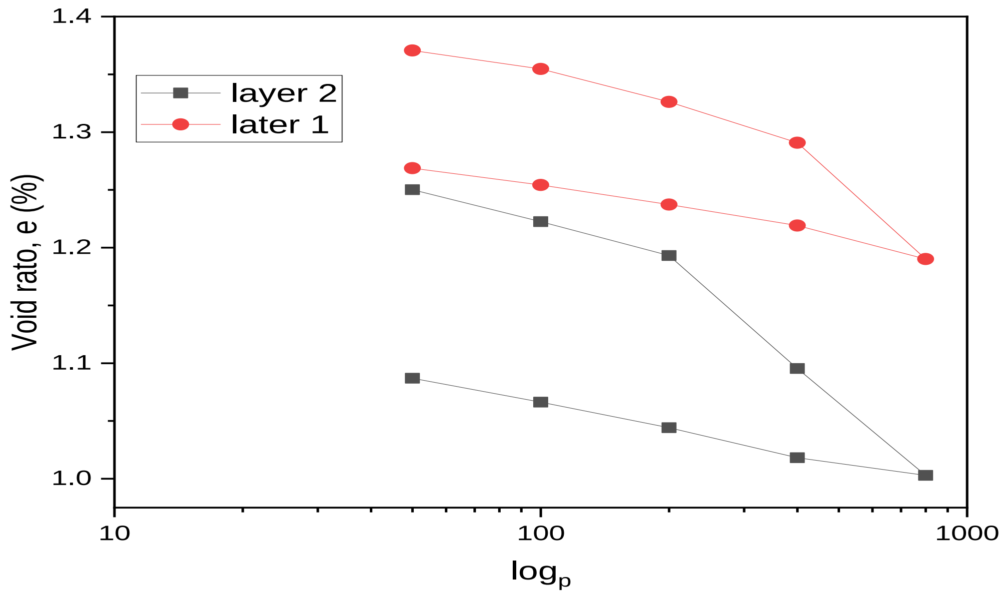


Figure 4.4: Void ratio and log pressure curve for soil layers 1 and soil layer 2

Table 4-6: Summary of applied pressure and void ratio of soils layers

Soil layer	Soil layer 1	Soil layer 2
Pressure, kpa	Void ratio, e (%)	Void ratio, e (%)
Loading stage		
50	1.371	1.250
100	1.355	1.222
200	1.326	1.193
400	1.291	1.095
800	1.190	1.003
Unloading stage		
400	1.219	1.018
200	1.237	1.044
100	1.254	1.066
50	1.269	1.087

Parameters and Pre-consolidation pressure

Compression parameters such as coefficient of compressibility (αv) and compression index (Cc), swelling index (Cs), and pre-consolidation pressure ($\boldsymbol{\sigma}$c) are obtained from a plot of void ratio versus log pressure.

Based on laboratory test results, there are a few graphical methods for evaluating the pre-consolidation pressure. The method proposed by Casagrande was the first and most commonly used (Bowles, J.E., 1996) . The method entails locating the point of maximum curvature on laboratory e-logp curve, drawing a tangent to the curve, and constructing a horizontal line.

After that, the angle formed by these two lines was bisected. The pre-consolidation pressure (Pc) corresponds to the abscissa of the point of intersection of this bisector with the upward extension of the inclined straight segment. The summary of the results is tabularized as in Table 4-7 and the detail is obtainable in the Appendix V.

Table 4-7: Summary of oedometer test results

Soil layer	Void ratio (e %)	Compression index,Cc	Swelling index,Cs	Preconsolidation pressure(kpa)	Overburden pressure(kpa)	Overconsolidation ratio, OCR	Overconsolidation margin(σm)
Layer 1	1.39	0.23	0.06	350	48.288	4.83	277.568
Layer 2	1.31	0.32	0.08	300	170.583	1.84	129.42

Two expressions OCR = σc/σo and σm = σc −σo used to classify soils with respect to their degree of over consolidation as indicated in Table 4.8, where OCR = the over consolidation ratio and σm = the over consolidation margin.

Table 4-8, shows the tested samples exhibit a moderately over-consolidated state as a result the soils deform gradually before the applied load reaches the pre-consolidation pressure stage. The over-consolidation ratio (OCR), which is pre-consolidation ratio divided by vertical overburden pressure, is greater than 3 and over consolidation margin is ranges between 100kpa and 400kpa for soil layer 1 and for soil layer 2 the over consolidation ratio ranges between 1 and 3 indicates soil was under Lightly over-consolidated. This shows that the soils have experienced higher effective vertical pressure in their past geological history and/or the soil might be pre-consolidated.

Table 4-8: Types of overconsolidated clay soils(Mohammed, 2015).

OCR	σm(kpa)	Classification
1-3	0-100	Lightly over-consolidated
3-6	100-400	Moderately over-consolidated
>6	>400	Heavily over-consolidated

4.1.2.7 Triaxial Test Result

The triaxial test performed using a triaxial compression test apparatus, according to the (ASTM D2850) procedures. From the triaxial compression tests, the fundamental parameters extracted are friction angle, soil cohesion, triaxial loading stiffness (E^{ref}_{50}), triaxial unloading stiffness (E^{ref}_{ur}) and Poisson's ratio (Vur). These parameters used in computer model to predict how the material behaves in the foundation interaction and settlement.

The Mohr-Coulomb strength parameters c and ϕ are the intercept and the angle of the Mohr-Coulomb failure envelope respectively. Alternatively, the c and ϕ values can be obtained from p-q diagram, as shown in Figure 4.5. The results have been illustrated, as shown in Table 4 -9.

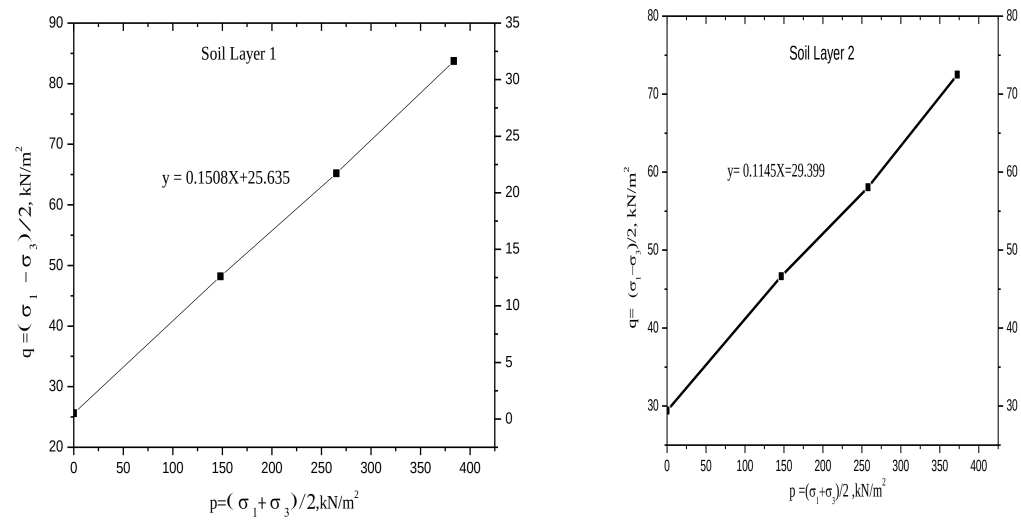


Figure 4.5: Graph for determination of **c** and ϕ (p vs q)

Table 4-9: Shear strength parameters from triaxial test result

Soil type	Soil layer 1	Soil layer 2
Φ(degree)	8.58	6.53
C(kpa)	25.64	29.40
Ɣ(kN/m^3)	17.23	16.00

4.1.3 Stiffness parameters results

4.1.3.1 Elastic Soil Stiffness Parameter

The values of Eoedref parameters required as input stiffness parameters of the HS Model for both subsoil layer are shown in Table 4-10. The values are in the order of 3442kN/m^2, and 4515 kN/m^2 for medium and medium to stiff clays, respectively.

Table 4-10: Oedometer Loading Stiffness Reference results

Soil type	Layer 1	Layer 2
E_{oed}^{ref} (mpa)	3.442	4.515

4.1.3.2 Plastic Soil Stiffness Parameters

From the values of Table 4-11, it can be seen that the stiffness parameters of layer one is less than layer two. It indicates that the soil stiffness increases with depth. The values required as input stiffness parameters of the HS Model as per their layers.

Table 4-11: Triaxial Loading Stiffness Reference and Triaxial unloading and reloading stiffness results

Soil type	Layer1/reddish brown	Layer 2/Dark grey
E_{50}^{ref}(mpa)	1.864	2.139
E_{ur}^{ref}(mpa)	5.592	6.418

4.1.3.3 Angle of Dilation (ψ

From consolidation test the tested samples exhibit over-consolidated state, as a result the soils deform gradually before the applied load reaches the preconsolidation pressure stage for soil layer 1 and lightly over consolidated soil for soil layer 2. Hence, the $\psi = \phi/6$ and $\psi = 0.0$ used to determine the dilatancy angle respectively.

Table 4-12: Shear strength parameter result

Soil type	Layer 1/reddish brown	Layer 2/dark grey
Ψ	1.43	0

4.2 Numerical modeling Analysis

4.2.1 Numerical modeling

The numerical results and discussions discussed in this section illustrate the effects of putting closely spaced footings on layered clay soil mass. In order to investigate the impact of closely spaced footings on bearing capacity, settling, and tilting of footings, more than 100 numerical models were run on interference footings over layered clay in this research. The non-dimensional factors Bearing Capacity factor, Settlement factor, and tilting factor are used to represent the bearing capacity ratio, settlement ratio, and tilting ratio of the footing on the layered clay soil, respectively. The analysis carried out using plaxis 3D for adjacent footing resting on layered soil for different spacing and different thickness of clay soil layer. The analysis carried out by using stiffness and shear strength parameters, applied load and consolidation parameters. The hardening model with 76004 elements used to mesh the soil material and with the average element size 0.5m. The generation of mesh based on the triangulation procedure. Fine mesh discretization defined for the model. The model is shown in Figure 4.6 for S/B_L =2.50.

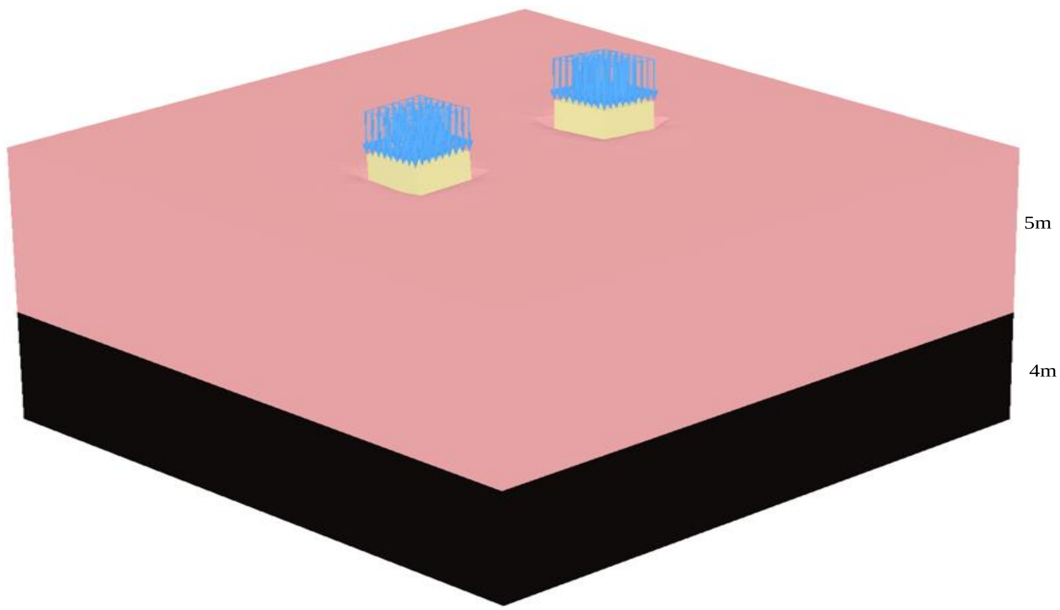


Figure 4.6: Footings loading model (S/B_L = 2.5)

4.2.2 Interference between Footings

In this research, the load is applied on the left and right footings at an angle of 90^0 with the horizontal, the interference of footings observed by increasing S/B_L ratio at S/B_L = 0.5, 1, 5, 2.5, 3.5, 5.0 and 5.5 are presented.

4.2.2.1 Isolate Footing

Figure 4.7 indicates the active zone appear under the footing base and radial shear zone failure starts to propagate after the footings settled. Hence, the footing is single footing it settled with some value and smaller values while it compares with settlement values of two parallel footing with closest footings.

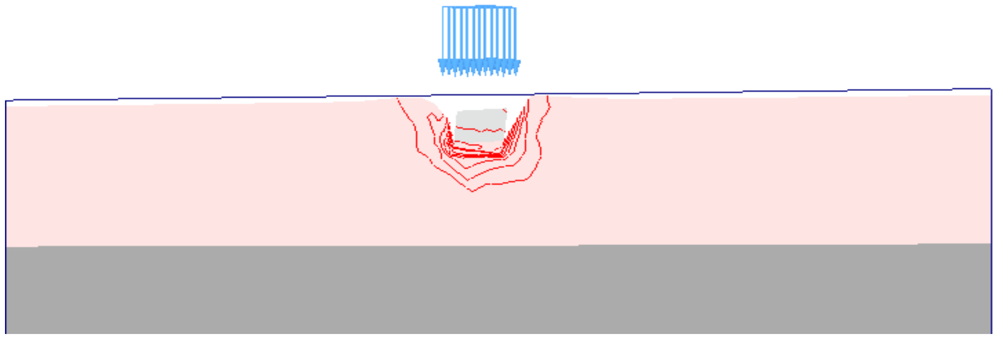


Figure 4.7: Settlement of Isolate footing

4.2.2.2 Two Parallel Footings (S/B_L = 5.5)

Placing two footings at wide spacing, as shown in Figure 4.8, the interference has no takes place. The intermediate zone appears between the footings, and there are no effect on stress values between these pairs of individual footings.

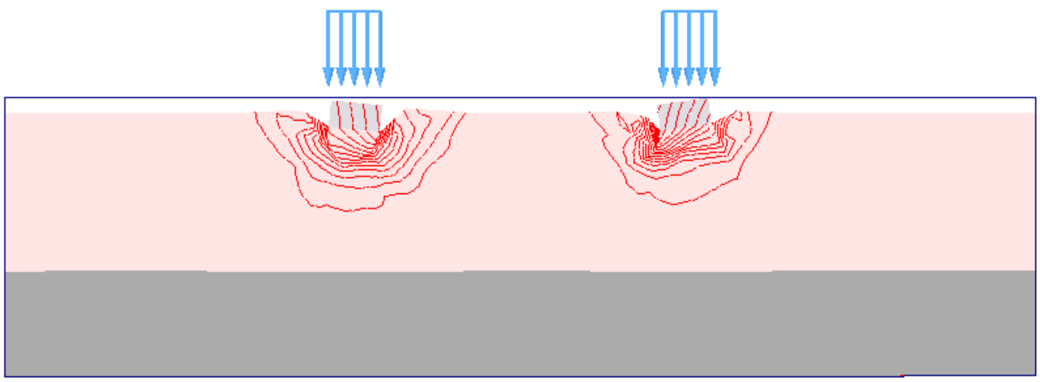


Figure 4.8: Settlement of Pair footings at S/B_L =5.5

4.2.2.3 Two Parallel Footings (S/B_L = 5.0)

Placing two footings at S/B_L = 5.0, as shown in Figure 4.9 the interference has no takes place. The intermediate zone appears between the footings, and there are no effect on stress values between these pairs of individual footings.

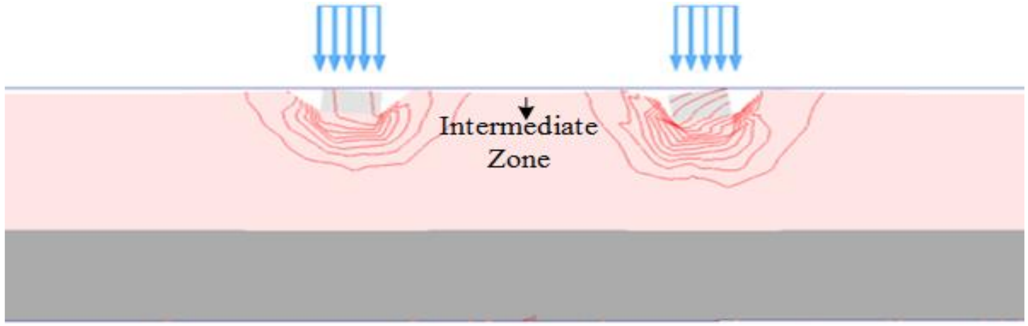


Figure 4.9: The Settlement of pair footings at $S/B_L = 5.0$

4.2.2.4 Two Parallel Footings (S/B_L =3.5)

As the clear spacing between two footings, starts to reduce the interference zone arrived to each other. At $S/B_L = 3.5$ the inter failure zones/intermediate zone between two footings starts to interfere.

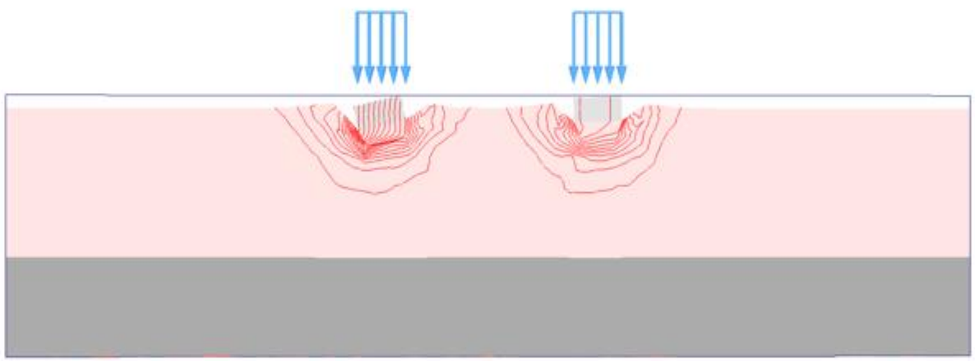


Figure 4.10: The Settlement of Pair footing at $S/B_L = 3.5$

4.2.2.5. Two Parallel Footings ($S/B_L = 2.5$)

As the spacing reduces, a condition shown in Figure 4.11 arises, where the passive zones/radial zone interpenetrate, the interference zones starts to interfere as the footings approach each other and the soil between the individual footings travels down with the footings.

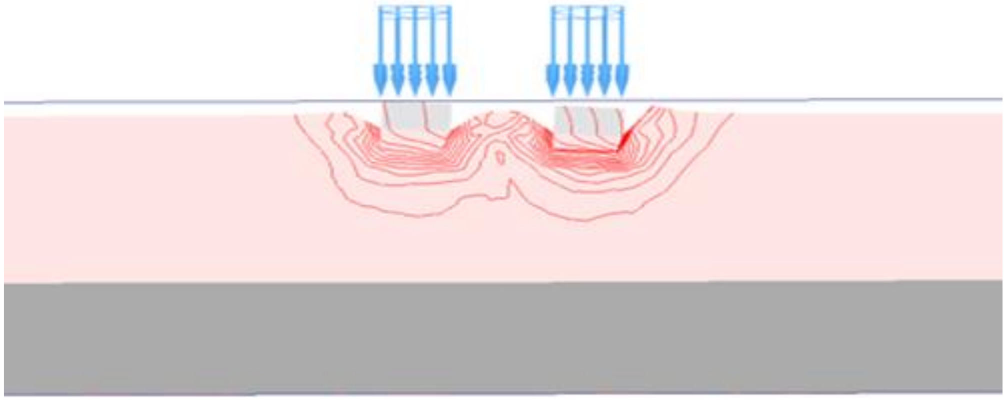


Figure 4.11: The Settlement of Pair footing at S/B_L = 2.5

4.2.2.6 Two Parallel Footings (S/B_L =1.5)

At closer spacing (S/B_L = 1.5), an intermediate condition arises as shown in Figure 4.12, in which the interference zone between the footings more disturbed, and travels down changes in the stress values.

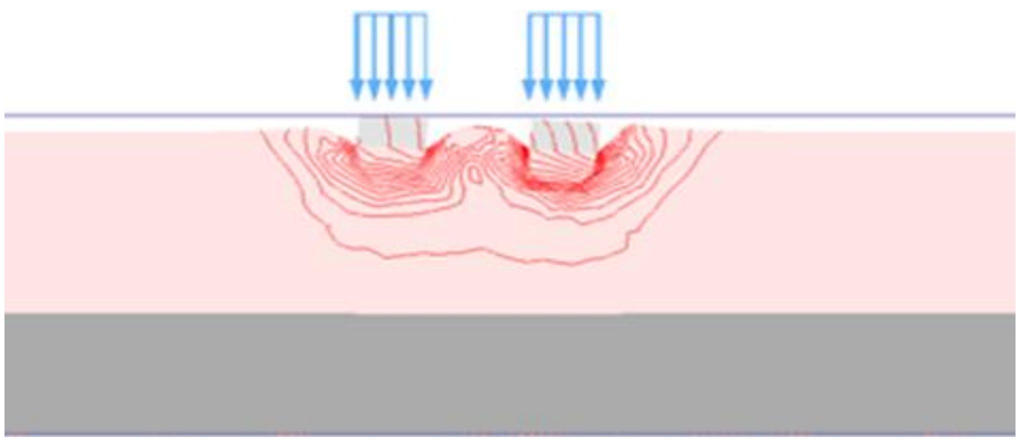


Figure 4.12: The Settlement of pair footings at S/B_L =1.5

4.2.2.7 Two Parallel Footings (S/B_L = 0.5)

When the spacing between the footings is very close (S/B_L = 0.5), an intermediate condition occurs, as shown in Figure 4.13, in which the size of the zone between the footings vanishes and the stress values change. The footings are collapsing on top of each other.

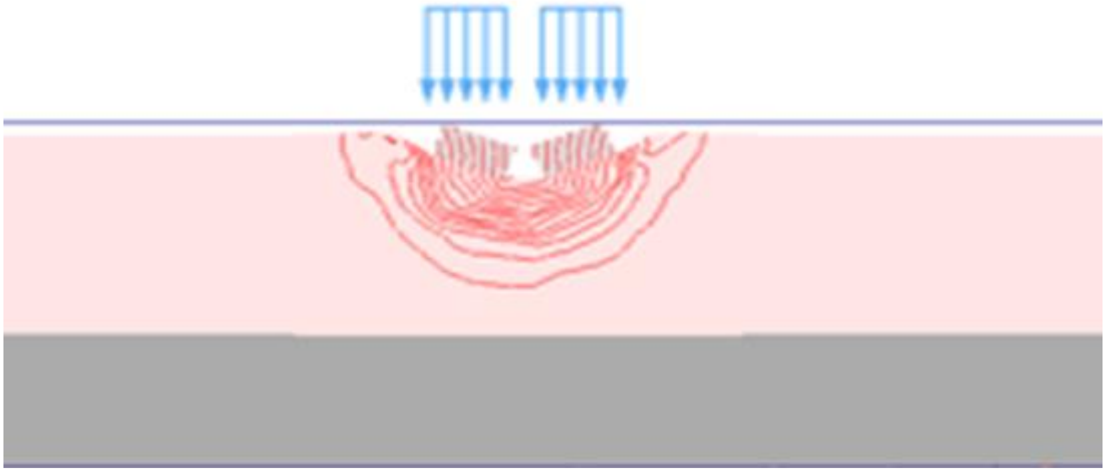


Figure 4.13: The Settlement of Pair footings at $S/B_L = 0.5$

4.2.2.8 Two Parallel Footings ($S/B_L = 0.0$)

As the footings come close together, the failure mechanism merges two isolated mechanisms into one; when the footings strike, it is geometrically similar to the isolated case, but twice the size. When the outer spirals collide, blocking happens, and the two footings become a single foundation. When the load is applied, the soil between the individual footings travels down with the footings. The interference zone vanishes, and the system reverts to the characteristics of a single foundation with a width twice the isolate footing's base. The load added to the footings acts as a simple twice of the individual footing load, causing further settlement to occur.

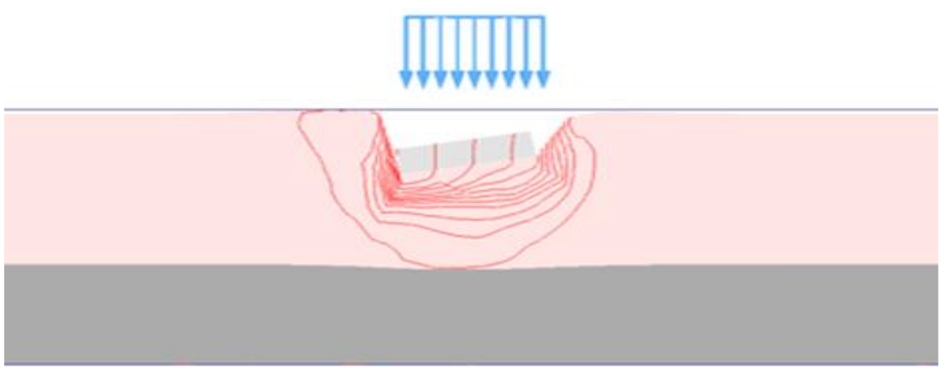


Figure 4.14: The Settlement of pair footings at $S/B_L = 0.0$

4.2.2.9 Two Parallel Footings with Assymmetrical Loading

As assymmetrical loading was applied to two parallel footings at very near spacing, the larger loading settled more and the stress values of the smaller loading changed. As a result, the smaller footings fail toward the larger footing.

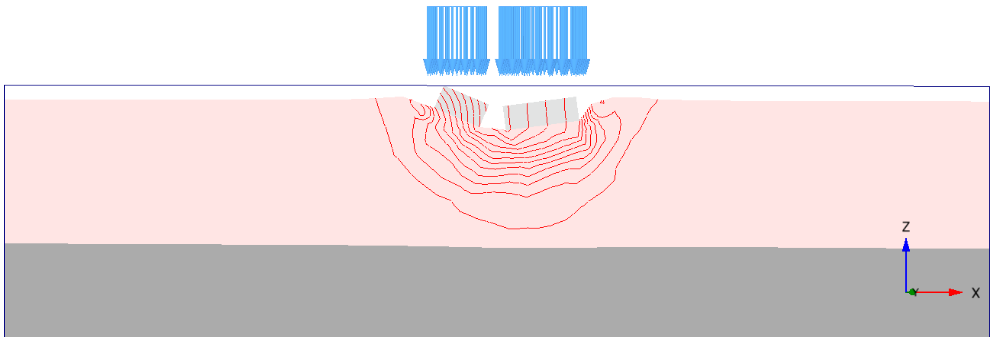


Figure 4.15: Settlement of assymmetrical loading two parallel footings

4.2.3 Pressure – Settlement

Pressure - Settlement obtained by increasing load with each step until failure of soil can be obtained. The ultimately bearing capacity obtained from drawing the tangent in load versus vertical settlement graph. For this study, the bearing capacity of soil obtained at allowable settlement for each case and foundation settlement values obtained from crossponding bearing pressure obtained from isolate footing at allowable settlement for comparison purpose.

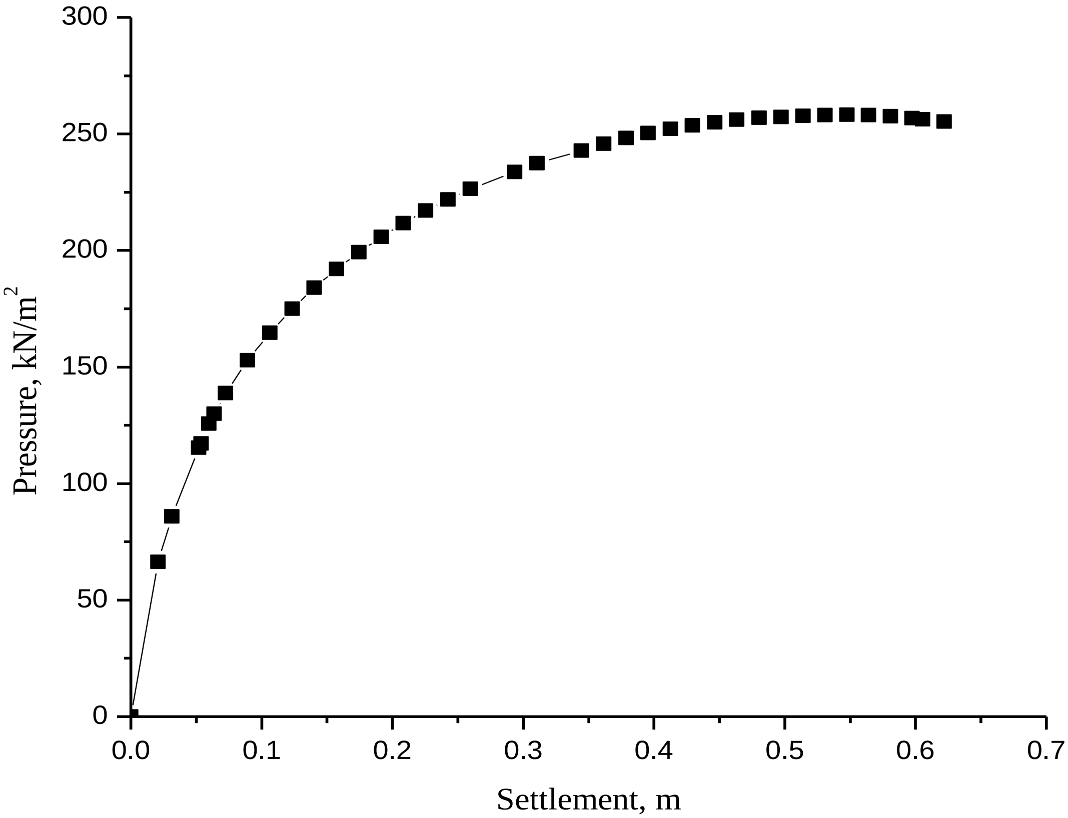


Figure 4.16: pressure – settlement curve

Pressure settlement curves, for isolated footings and two interfere parallel footings resting on soil surface with vertical load are tabulated and discussed here under various cases using the finite element model. It is to be noted that the pressure and settlement presented are obtained by averaging values below footing.

4.2.3.1 Symmetrical Footing and Symmetrical Loading

It observed that for the symmetrical cases, the pressure settlement curves obtained both from the left and right footings are almost identical. At $S/B_L = 0.5$, the bearing pressure is observed to be decreased by about 11.73% associated to isolated footing. This is due the overlapping of the stress zones of individual footing when placed close to each other. Similar kind of variation is seen for the settlements at working load. The settlement is increased by 33.33% when the footings are placed at $S/B_L = 0.5$ and the percentage increase in the settlement decreases with increase in S/B_L ratio. Detail values of bearing pressure and settlement is shown in following Table 4-13.

Table 4-13: Bearing pressure and settlement of footing for both symmetrical footing and loading.

S/B_L		0.5	1.5	2.5	3.5	5.0	5.5	Isolate
Bearing pressure(@75mm), kPa	Left	165.08	179.01	179.04	179.35	185.23	185.56	185.56
	Right	164.88	179.01	179.35	179.54	185.00	185.34	
δ, mm	Left	30.00	22.00	21.00	21.00	20.00	20.00	20.00
	Right	29.54	22.00	21.00	20.00	20.00	20.00	

4.2.3.2 Symmetrical Footing and Assymmetrical Loading

The bearing pressure calculated owing to the interference for left and right footings placed at different spacing are tabularized in Table 4-14. It is obtained that the bearing pressure of left footings decreases by 12% - 22.39% at S/B_L = 0.5 compared to that of isolated footing under different loading condition of right footing. It indicates that the left footing more influenced when the load on the right footing increases. However, at S/B_L = 5.0, the bearing pressure of two footings attain a value as that of isolated footing and remains constant with further increase in S/B_L ratio.

The settlements, δ obtained are presented in Table 4-15. It observed that the settlement of left increased by 33% - 35% at S/B_L = 0.5 compared to that of isolated footing. However, the stress contour at S/B_L = 5.0 of right footing do not overlap much with that of left footing and hence negligible interference phenomenon is observed.

Table 4-14: Bearing capacity and settlements of footing for symmetrical footing and assymmetrical loading.

S/BL	Bearing pressure @75mm, kpa							
	β/α =1.25		β/α =1.5		β/α =1.75		β/α =2.0	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	162.04	180.97	155.56	184.08	149.00	177.72	144.00	176.38
1.5	170.44	177.00	180	184.71	178.51	179.77	180.78	177.46
2.5	180.07	180.97	181.74	185.06	180.21	184.16	182.99	180.74
3.5	182.00	186.14	183.29	185.30	181.10	184.28	183.98	183.15
5	185.64	186.04	185.69	185.46	185.64	184.49	185.70	184.54
5.5	185.61	185.73	185.63	185.34	185.58	185.67	185.62	185.57

Table 4-15: Settlements of footing for symmetrical footing and assymmetrical loading.

	δ, mm							
S/BL	β/α =1.25		β/α =1.5		β/α =1.75		β/α =2.0	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	30.00	31.00	28.00	32.00	27.00	34.00	30.00	36.00
1.5	22.00	23.00	28.00	27.00	25.00	29.00	28.00	28.00
2.5	21.00	21.00	27.00	26.00	26.00	26.00	27.00	27.00
3.5	20.00	22.00	27.00	26.00	20.00	25.00	24.00	26.00
5	20.00	20.00	20.00	24.00	20.00	20.00	20.00	20.00
5.5	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00

4.2.3.3 Assymmetrical Footing and Symmetrical Loading

Table 4-16 shows the pressure and the settlement values obtained from pressure settlement curves for the case of assymmetrical footing with symmetrical loading footings. It is obtained that the bearing pressure of interfering footing of left footing decreases by 13%-14% at S/B_L = 0.5. However at all S/B_L= 5.0 the bearing capacity value remains same as that of isolated footing, revealing negligible or no effect of interference on bearing capacity. The settlement observed on left footing increased by 33% to 35% in settlement as associated to the isolated footing when the footing size changes.

Table 4-16: Bearing capacity and settlements of footing for assymmetrical footing and symmetrical loading.

S/BL	**Bearing Pressure @75mm, kPa**				**δ, mm**			
	D/B_L = 0.0 α=1.5, β =1		D/B_L = 0.0 α =2.0, β =1		D/B_L =0.0 α=1.5, β =1		D/B_L = 0.0 α=2.0, β =1	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	161.21	162.46	159.67	163.94	30.00	34.00	31.00	40.00
1.5	163.48	170.29	161.36	174.75	24.00	33.00	25.00	39.00
2.5	181.17	180.12	182.27	178.92	22.00	32.00	22.00	38.00
3.5	184.79	182.09	184.15	182.67	22.00	32.00	22.00	38.00
5.0	185.06	185.61	185.73	185.90	21.00	26.00	20.00	31.00
5.5	185.53	185.21	185.53	185.59	20.00	26.00	20.00	30.00

4.2.3.4 Assymmetrical Footing Size with Assymmetrical Loading

The pressure settlement curves obtained for case assymmetrical footing with assymmetrical loading footings. Table 4.17 shows that the bearing pressure of the interfering footing of the left footing decreases by 14.19 % -15.34% at S/B_L = 0.5, while the bearing capacity value remains the same as that of the isolated footing at all S/B_L= 5.0, indicating that interference has insignificant or no impact on bearing capacity. However, interference had a major impact on settlement on the left footing and increased 26% to 56.25% in settlement as related to the isolated footing when the footing size changes. From this cause, it noticed that the left footing more influenced when the load on the right footing and size the footing increases.

Table 4-17: Bearing capacity and settlements of footing for assymmetrical footing and assymmetrical loading

S/BL	Bearing Pressure @75mm.				δ, mm			
	D/B_L = 0.0		D/B_L =0.0		D/B_L =0.0		D/B_L =0.0	
	α=1.5,β/α=2		α=2, β/α =1.5		α=1.5,β/α =2		α = 2,β/α = 1.5	
	Left	Right	Left	Right	left	Right	left	Right
0.5	157.10	143.49	159.22	134.67	26.67	32.00	45.71	37.95
1.5	167.69	168.43	179.63	201.23	25.71	31.00	28.67	36.97
2.5	178.56	181.77	180.50	206.63	20.95	30.60	23.81	36.61
3.5	184.11	190.40	182.62	211.98	20.00	30.00	20.00	36.00
5.0	185.56	195.65	185.20	212.90	20.00	30.00	20.00	36.00
5.5	185.52	195.200	185.15	212.65	20.00	30.00	20.00	36.00

4.2.3.5 Assymmetrical Footing with Assymmetrical Loading at different depth

The pressure settlement curves obtained for case assymmetrical footing with assymmetrical loading footings at different depth. It observed that the settlement obtained from left footing more than the settlement from right footing. It indicated that the right footing settle with some value and the surface footing (left Footing) will disturbed and settled down more. When right footing is at surface and embedded to D/B_L =1.0, the bearing pressure of interfering footing increases 26.42 % to 32.11 % at S/B_L = 0.5, respectively, and the bearing capacity value remains the same at all S/B_L ratios, indicating that interference has negligible or no impact on bearing capacity. However, interference has a major impact on settlement on the left footing, raising settlement 45.46 % to 47.81% for the surface and embedded right footings, respectively, as comparison to the isolated footing. This means that the interference impact of footings increases as the depth of the right footing increases.

Table 4-18: Bearing capacity and settlements of footing for assymmetrical footing and assymmetrical loading (Left Footing @D/B_L = 0.0 and Right Footing @D/B_L =1.0)

S/BL	Bearing pressure @75mm, kPa				δ, mm			
	D/B_L = 0.0		D/B_L =1.0		D/B_L =0.0		D/B_L =1.0	
	α=1.5,β/α=2		α=2, β/α =1.5		α=1.5,β/α =2		α = 2,β/α = 1.5	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	136.54	165.09	125.98	173.13	36.67	47.00	34.49	45.82
1.5	160.27	176.22	180.41	206.40	25.71	37.00	22.37	42.55
2.5	178.56	189.57	182.00	209.97	22.95	33.6	20.41	39.27
3.5	181.18	193.48	182.42	211.58	20.00	30.00	18.00	36.00
5	185.56	195.00	185.56	212.00	20.00	30.00	18.00	34.00
5.5	185.56	195.00	185.56	212.00	20.00	30.00	18.00	34.00

4.2.3.6 Symmetrical Footing with Assymmetrical Loading at different depth

From Table 4.19 it is obtained that the bearing pressure of interfere footing increases by 15.33% to 20.20% for left footing at S/B_L = 0.5 when similar footing with only load differ at surface and embedded right footing. The interference on the settlement is observed on left footing and increase 44.44% to 46.42% in settlement as compared to the isolated footing. It can be considered that the interference effect increases as depth of right footings increases.

Table 4-19: Bearing capacity and settlements of footing for symmetrical footing and assymmetrical loading (Left Footing @D/B_L = 0.0 and Right Footing @D/B_L =1.0)

S/BL	Bearing pressure@75mm, kPa				δ, mm			
	D/B_L =0.0		D/B_L=1.0		D/B_L =0.0		D/B_L=1.0	
	α=1,β/α = 3.0		α= 1, β/α = 3.0		α=1, β/α = 3.0		α=1, β/α = 3.0	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	157.10	143.49	148.08	111.35	36.00	45.00	37.33	42.55
1.5	167.69	168.43	181.83	173.67	28.00	42.00	23.90	41.25
2.5	178.56	181.77	182.71	196.03	25.00	37.50	20.95	36.82
3.5	184.11	190.40	185.30	205.62	23.00	36.00	20.00	36.00
5	185.56	195.00	185.56	212.00	20.00	30.00	20.00	34.500
5.5	185.56	195.00	185.56	212.00	20.00	30.00	20.00	34.00

4.2.3.7 Effects of Ground water Variation at different depth

In this study, the effect ground water variable also studied in order to examine the influence of water level variation to the bearing capacity and the settlement is enumerated in Table 4.20. The pressure settlement curves observed at different depth of ground water level. It is observed that the settlement obtained when the ground water level near the ground is greater than ground water level is far from ground level. It is obtained that the bearing pressure of interfere footing decreases by 11.2% and 10.84% at $S/B_L = 0.5$ when the ground water level at 2.20m and 2.60m respectively. The settlement of interfering footing increased by 33% and 20% at $S/B_L = 0.5$ when the ground water level at of 2.20m and 2.60m respectively. The interference effect of footings decreases as the water level decreases, as it observed from these values.

Table 4-20: Bearing capacity and settlements of footing for effects of Ground water variation

S/BL	Bearing Pressure @75mm				δ, mm			
	D/BL = 0.0 GWT@2.20m		D/B_L =0.0 GWT@2.60m		D/BL = 0.0 GWT@2.20m		D/B_L = 0.0 GWT@2.60m	
	Left	Right	Left	Right	Left	Right	Left	Right
0.5	164.77	164.57	165.44	179.49	30.00	29.45	22.50	22.50
1.5	185.06	171.26	190.39	182.10	25.00	23.00	19.64	19.64
2.5	185.04	178.53	194.75	189.01	23.14	20.76	18.82	18.40
3.5	185.56	184.59	195.44	194.41	20.50	20.44	18.00	18.00
5	185.56	185.56	195.45	195.45	20.00	20.00	18.00	18.00
5.5	185.45	185.32	195.45	195.45	20.00	20.00	18.00	18.00

4.3. Parametric Studies

The parameters for this parametric studies evaluated based on varying size of footing and load with correspondence of spacing variation for each case until the failure of soil obtained to assess the ultimate bearing capacity of the soil. For isolated footing on clay soil the maximum allowable settlement is 75mm as per IS (1904). The interference effect is analyzed for bearing pressure corresponding to permissible settlement. The settlement effect studied corresponding to working load of 100kpa isolated footing, also studied that the non-uniform settlement at the base of footings which influences the tilting of the footings. The effect of depth difference of footings and the effect of ground water level variation also studied with respect to spacing ratio (S/B_L) between the footings.

The dimensionless parameter efficiency factor is used to study the effects. The efficiency factor (ζ_γ) is defined as the ratio of the bearing capacity of interfering footing to that of isolated footing when calculating the bearing capacity of an interfering foundation. The settlement efficiency factor (ξδ) is described as the ratio of interfering footing settlement to isolated footing settlement. The tilt ratio (ζθ) is calculated by dividing the difference in settlement between the inner and outer edges by the width of the footing. The bearing factor, settlement factor, tilting factor, and interaction factors studied in terms of interference effect for the left and right footings are properly labelled, and their values read from the corresponding axes as defined on the left and right.

4.3.1 Bearing pressure variation

4.3.1.1 Symmetric footing and Symmetric loading

The load varies with α=1.0, β/α=1.0 to describe the case of symmetrical footing and symmetrical loading conditions.

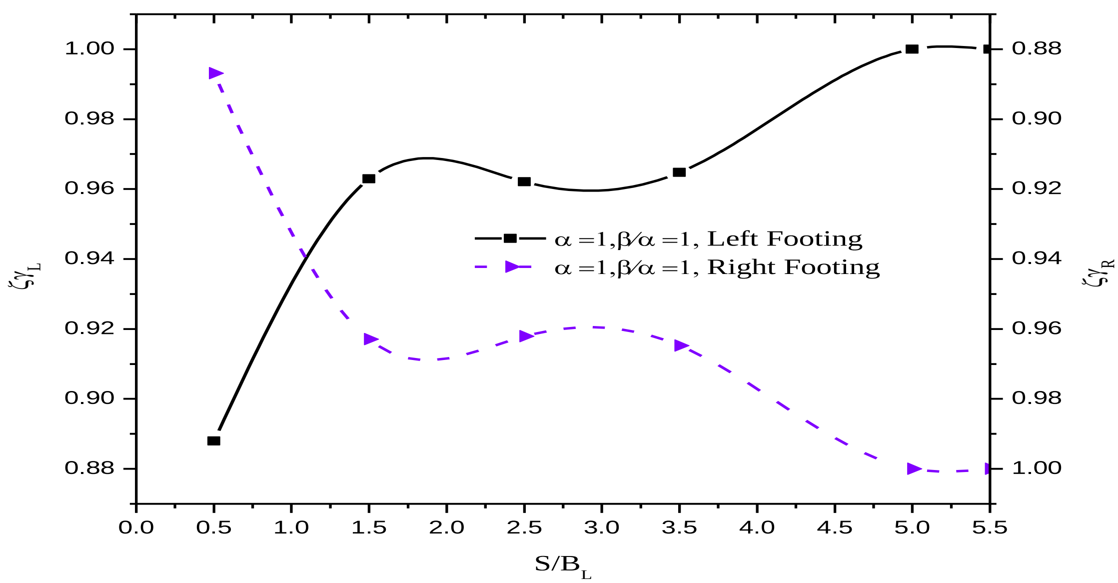


Figure 4.17: Variation of the Bearing Factors with S/B_L for; α=1; and β/α=1

It is observed that the bearing Pressure Variation corresponding the Spacing for symmetric footings and symmetric loading in Figure 4.17 the bearing ratio decreases at S/B_L = 0.5 and increases to $S/B_L \geq 5.0$ and become one. The maximum interference of footings observed at S/B_L= 0.5. It indicate that at S/B_L = 0.5 the footings are highly interfere to each other and $S/B_L \geq 5.0$ the footings act as isolate footing. It also observed that under symmetrical footings and symmetrical loading condition the left and right footing are equal interfere each other.

4.3.1.2 Symmetrical footings and Asymmetrical loading

The load differs with α=1.0, β/α=1.25,1.5,1.75 and 2.0 to characterize the case of symmetrical footing and asymmetrical loading conditions.

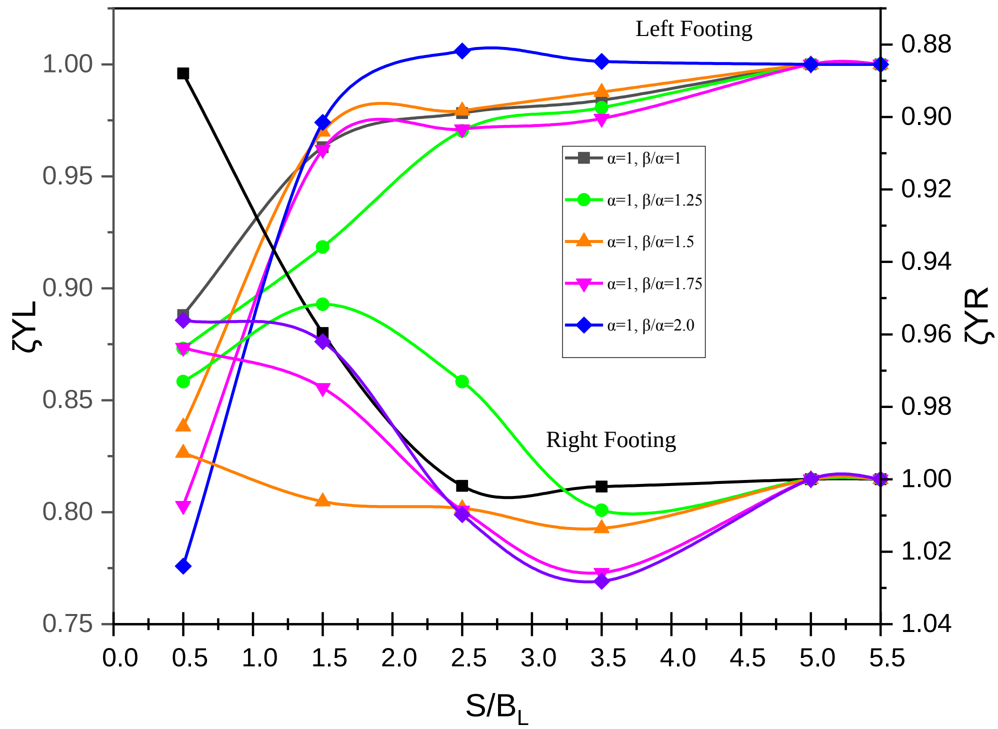


Figure 4.18: Variation of the Bearing Factors with S/B_L for; α=1; and β/α=1 - 2

Figure 4.18 shows the bearing factor plotted against the S/B_L ratio, which shows that the bearing ratio is less than one at $S/B_L = 0.5$. It shows that interfering footing has lower bearing pressure than isolated footing, and that bearing pressure increases as the S/B_L ratio increases, eventually reaching one at $S/B_L= 5.0$. This indicates the footing also act as isolated footing when the spacing ration $S/B_L \geq 5.0$. Under this condition, the larger load footing is the lesser the bearing capacity and gets more settle makes the smaller footing load to be more interfered.

4.3.1.3 Asymmetrical footings and Symmetrical loading

The Footing width varies with α =1.5 and 2.0, β =1.0 to indicates the case of assymmetrical footing size and symmetrical loading conditions.

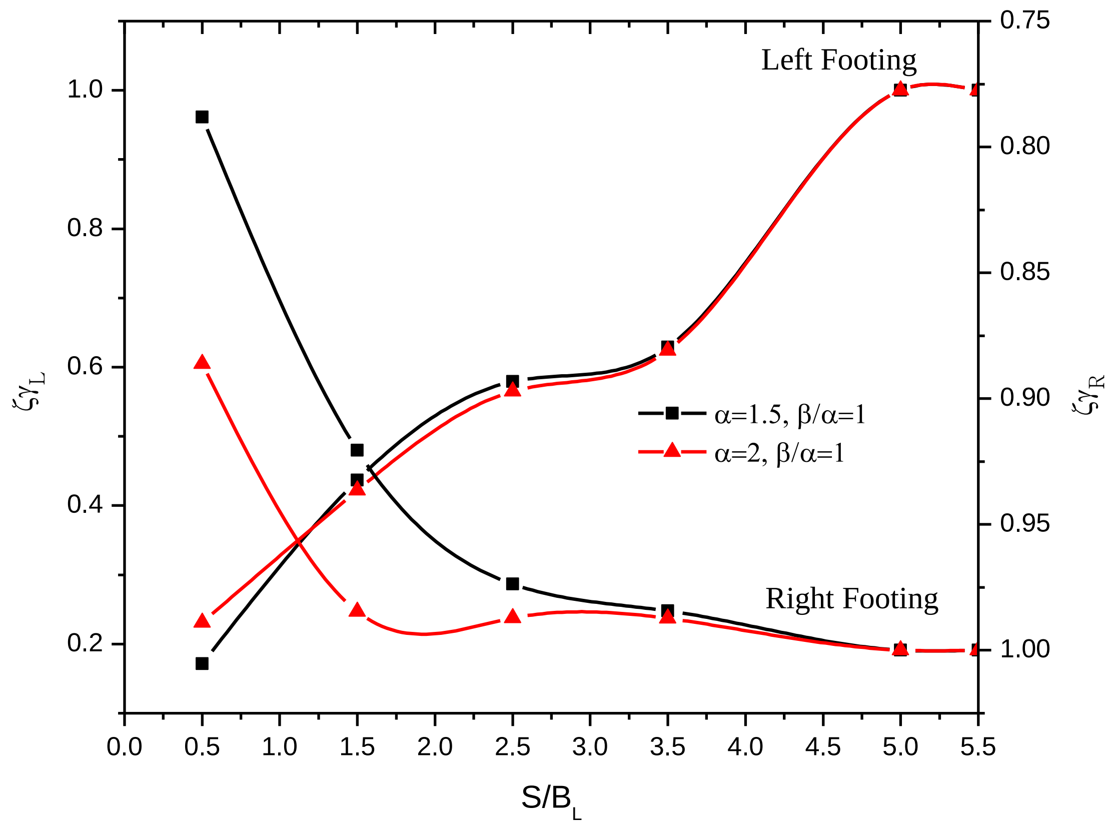


Figure 4.19: Variation of the Bearing Factors with S/B_L for; α =1.5, 2; and β/α =1

It was obtained that the bearing factor increases as the S/B_L ratio increases from 0.5, indicating that there is high interference at S/B_L= 0.5 and isolate footings at $S/B_L \geq 5.0$. The interaction factors for the left footing are greater than the values of the interaction factors for the right footing at a specified S/B_L ratio with such right footing increases in the width relative to the width of the left footing, indicating that the right footing interferes the left footings.

4.3.1.4 Asymmetrical footings and Asymmetrical loading

The Footing width varies with α =1.5 and 2.0 to represent the case of asymmetrical footing and the footing width varies with β/α = 1.50, and 2.00 to represent the case of asymmetrical loading conditions.

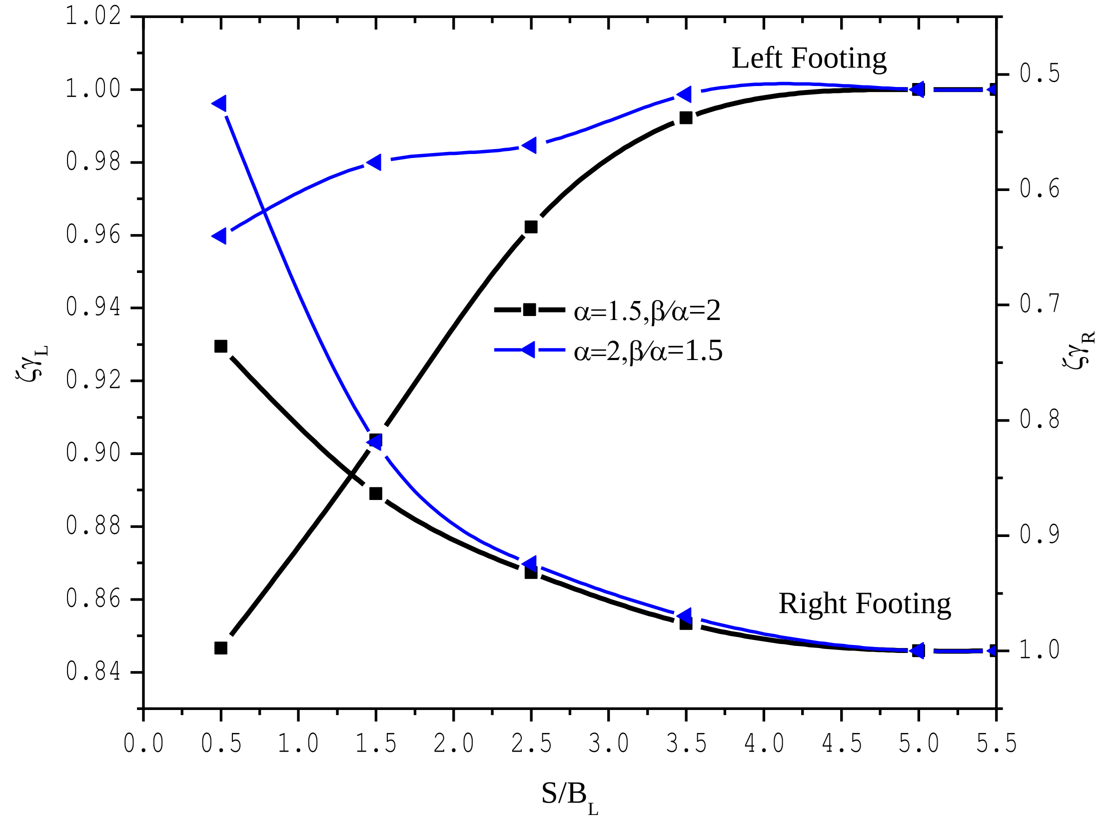


Figure 4.20: Variation of the bearing factors with S/BL for; α = 1.5, 2; and β/α =1.5, 2.

The bearing ratio is less than one for left footing and right footing at S/B_L = 0.5 and it increases as clear spacing increases. It implies that interfering footing has lower bearing pressure than isolated footing, as measured under acceptable settlement, and that the bearing pressure of interfering footing increases as the S/B_L ratio rises, eventually reaching one at S/B_L = 5 and more. The bearing factor for right footing less than the bearing factor of left footing as it considered from this study. It indicates that the large footing influences the smaller footing size and load. However, the footings have no interference effect when spacing ratio of footings $S/B_L \geq 5.0$; act as isolated footing.

4.3.1.5 Symmetrical footings and Asymmetrical loading at Different Depth Footings

The load varies with α =1.0, β/α = 3.0, the left footings at surface at D/B_L = 0.0, and the right footing embedded in soil at D/B_L=1.0 to characterize the case of symmetrical footing and asymmetrical loading conditions.

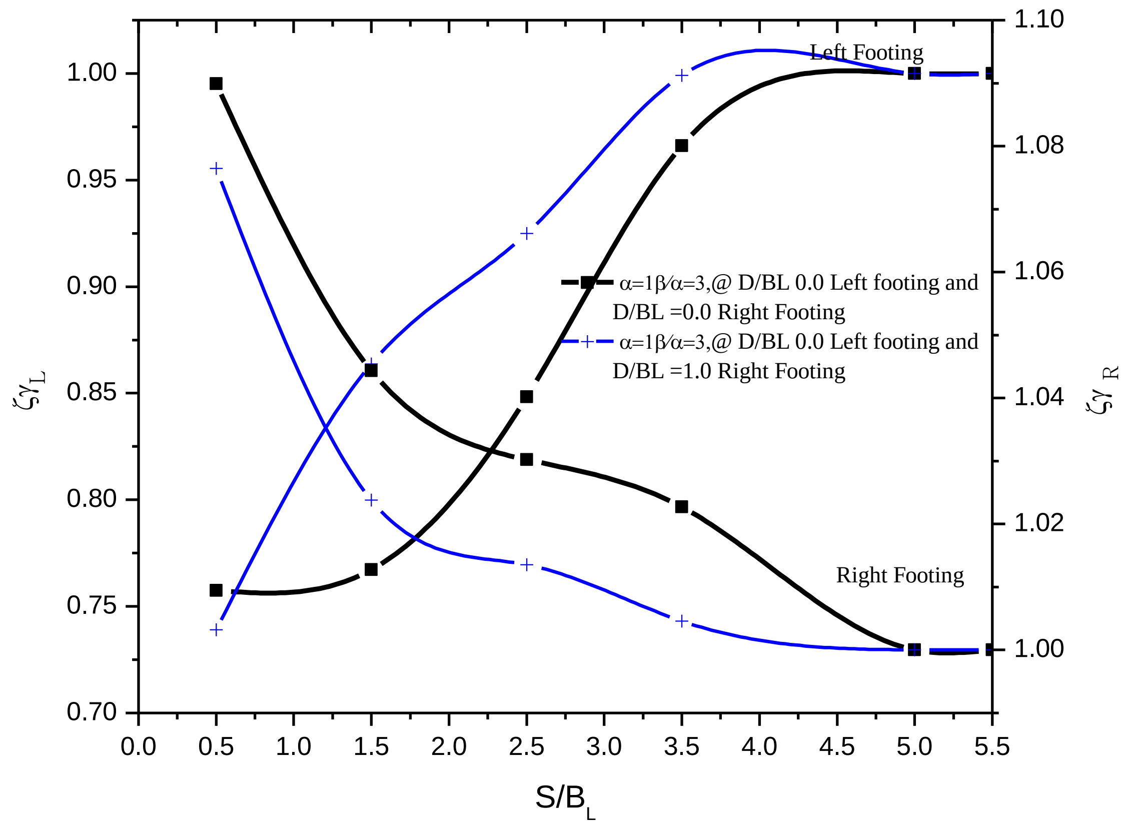


Figure 4.21: Bearing factor variation when right footing embedded in soil at D/B_L= 0.0, 1.0 with S/B_L for α = 1.0 and β/α = 2.0.

From symmetrical footings and asymmetrical loading at different depth footings it is observed the embedded footing was interfere the surface footing when the footings closer to each other. The bearing ratio is less than one at S/B_L = 0.5 for both left and right footing and it increases as the spacing between footings increases. Indicating that interfering footing has lower bearing pressure than isolated footing, as measured by allowed settlement, and that the bearing pressure of interfering footing increases as the S/B_L ratio increases, reaching one at $S/B_L \geq 5$.

4.3.1.6. Asymmetrical Footing and Asymmetrical Loading at Different Depth of Footings

The right footing is embedded at D/B_L =1.0, in Figure 4.22 shows the bearing ratio variations of the interaction factors with S/B_L for asymmetrical footing (α =1.5) and asymmetrical loading (β/α = 2.0). The plots of the asymmetrical footing and asymmetrical loading (α =1.5, β/α =2.0, and D/B_L = 0.0) are also shown for comparison.

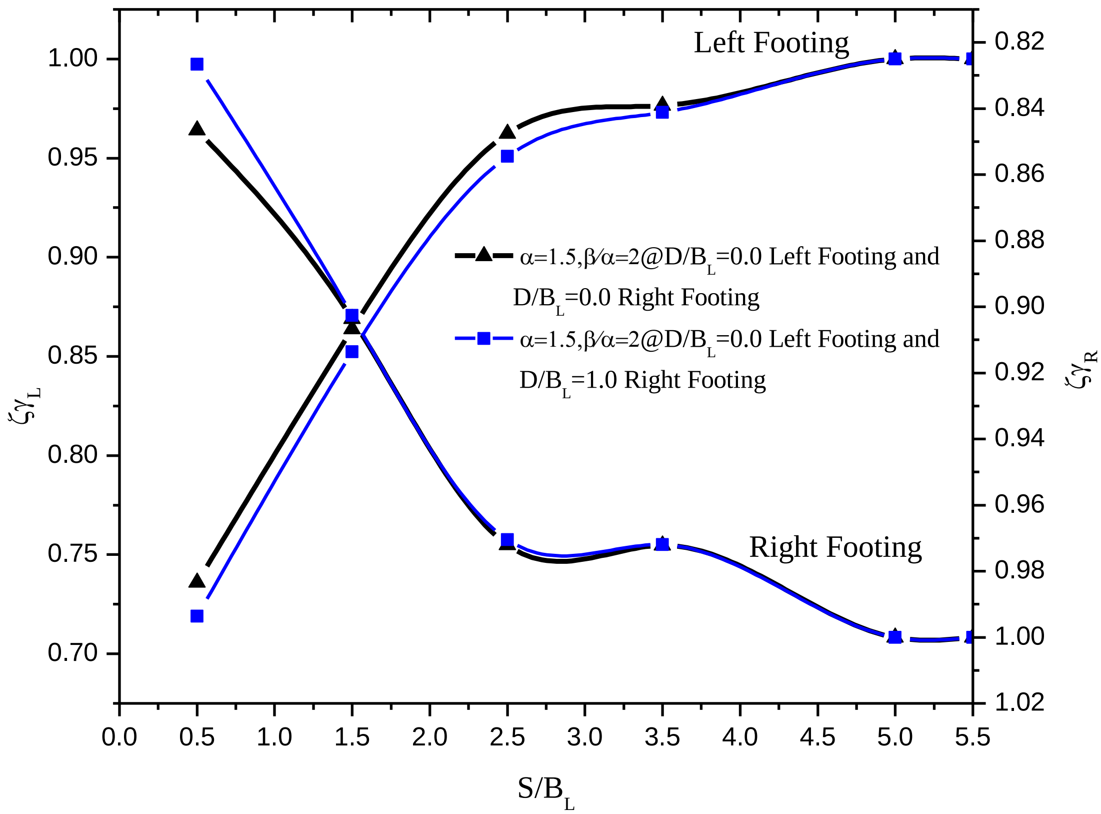


Figure 4.22: Bearing Factor Variation when Right Footing Embedded in Soil at D/BL = 0.0, 1.0 with S/B_L for α= 1.5 and β/α =2.0.

From this case, it observed that as S/B_L decreases the interference between two footings increases. The interaction factors for right footing while embedded in soil have more interference on surface footing. At the footing very close to each other (S/B_L= 0.5) the more interference of footings appear with the maximum bearing factor value. At spacing ratio between two footings S/B_L = 5.0 and more the footing act as isolated footing and there is no interference.

4.3.2 Settlement Variation

4.3.2.1 Symmetric footing and Symmetric loading

Symmetrical footing and symmetrical loading conditions the load varies with α=1.0, β/α=1.0 and shown as in Figure 4.23 of the settlement variation.

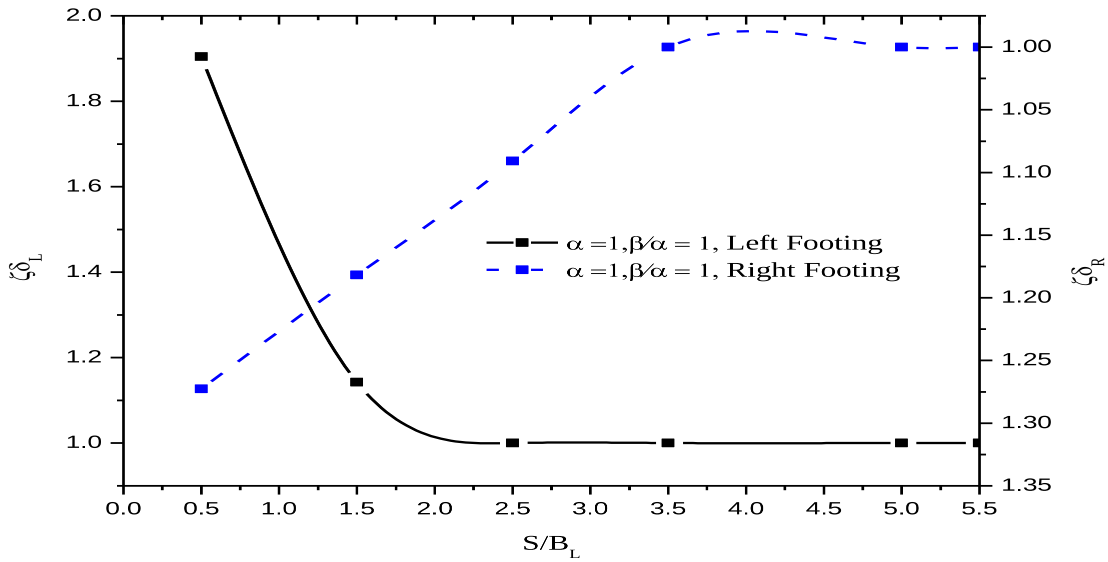


Figure 4.23: Settlement Efficiency Factor with Varying Spacing at α = 1 and β/α =1

It observed that for the symmetrical footing and symmetrical loading as the spacing ratio S/B_L increases the interference settlement decreases and become one when spacing between footings far as spacing ratio $S/B_L \geq 5.0$. This result indicates the footings act as individual footing when footings far apart as clear spacing ratio $S/B_L \geq 5.0$.

4.3.2.2 Symmetrical footings and Asymmetrical loading

Figure 4-24 shows the variance of settlement factor with respect to spacing ratio. The plots for β/α =1.0 reflect symmetrical footing and symmetrical loading, while the rest of the plots represent symmetrical footing and asymmetrical loading.

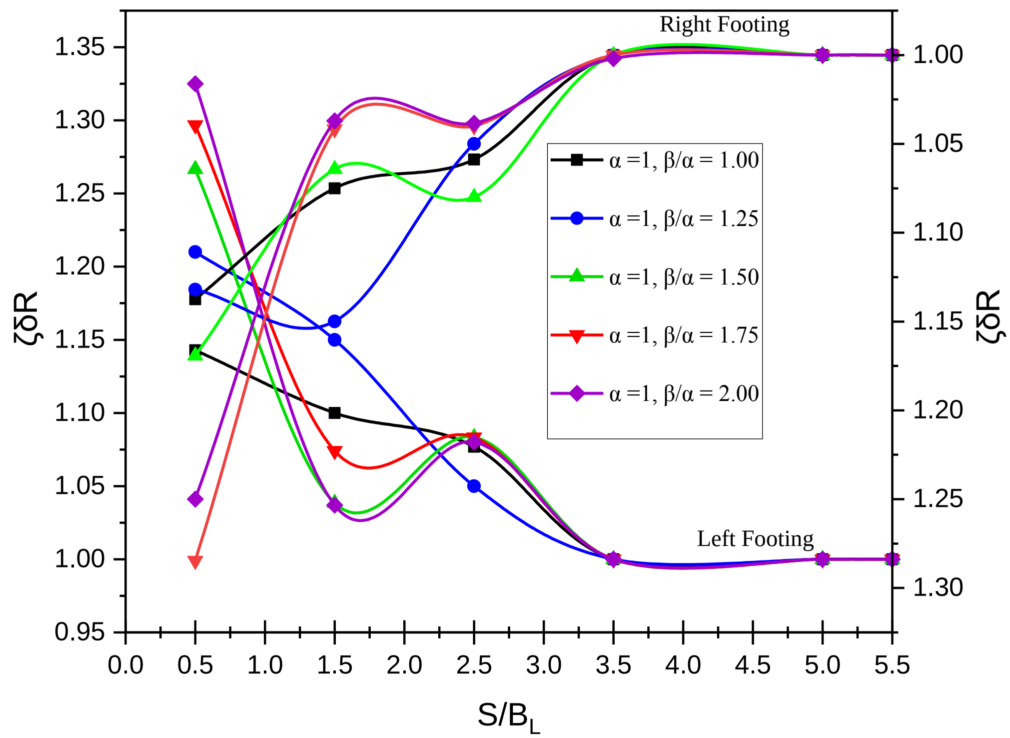


Figure 4.24: Variation of the Settlement Factors with S/B_L for; α=1 and β/a=1-2

It was observed that when asymmetrical loads are applied, the larger load settles down more and interferes with the smaller load. As the S/B_L ratio increases, the interaction factors decrease, eventually reaching a constant value of one at greater spacing $S/B_L \geq 5.0$ and while comparing with symmetrical footing and symmetrical loading the interaction factors of the left footing are equal to the interaction factors of the right footing for α=1.0 and β/α=1.0.

4.3.2.3 Asymmetrical footings and Symmetrical loading

In Figure 4.25, the results obtained for footing that are with asymmetrical footing size and symmetrical footing load.

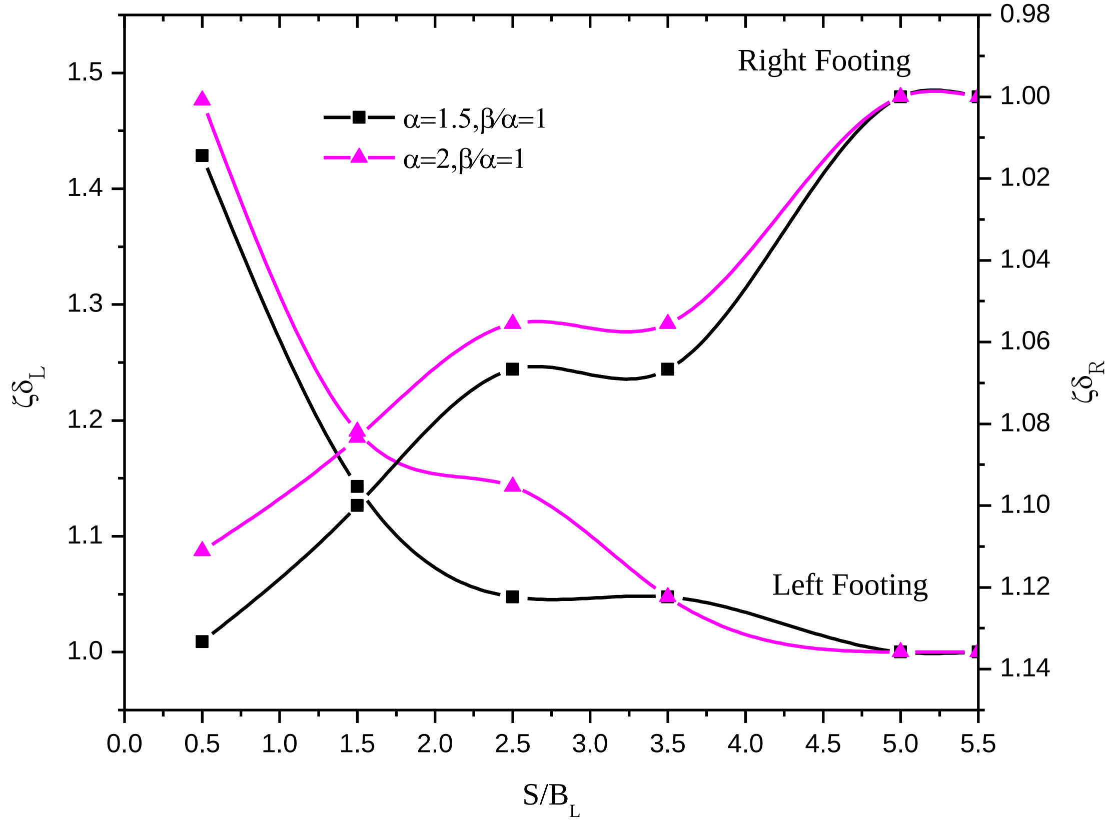


Figure 4.25: variation of the Settlement factors with S/B_L for; α =1.5, 2; and β/α =1

The interaction factors for the left footing are greater than the interaction factors for the right footing at a specified S/B_L ratio with an increase in the width of the right footing compared to the width of the left footing. The larger footing size interferes with the smaller footing size. With an increase in the S/B_L ratio, the interaction factors decrease and reach a value of unity, where the footings considered to be behaving as an isolated footing.

4.3.2.4 Asymmetrical footings and Asymmetrical loading

Figure 4.26 presents the variation of the interaction factors with S/B_L for asymmetrical footing (α=1.5) and asymmetrical loading (β/α=1.5 and 2.0).

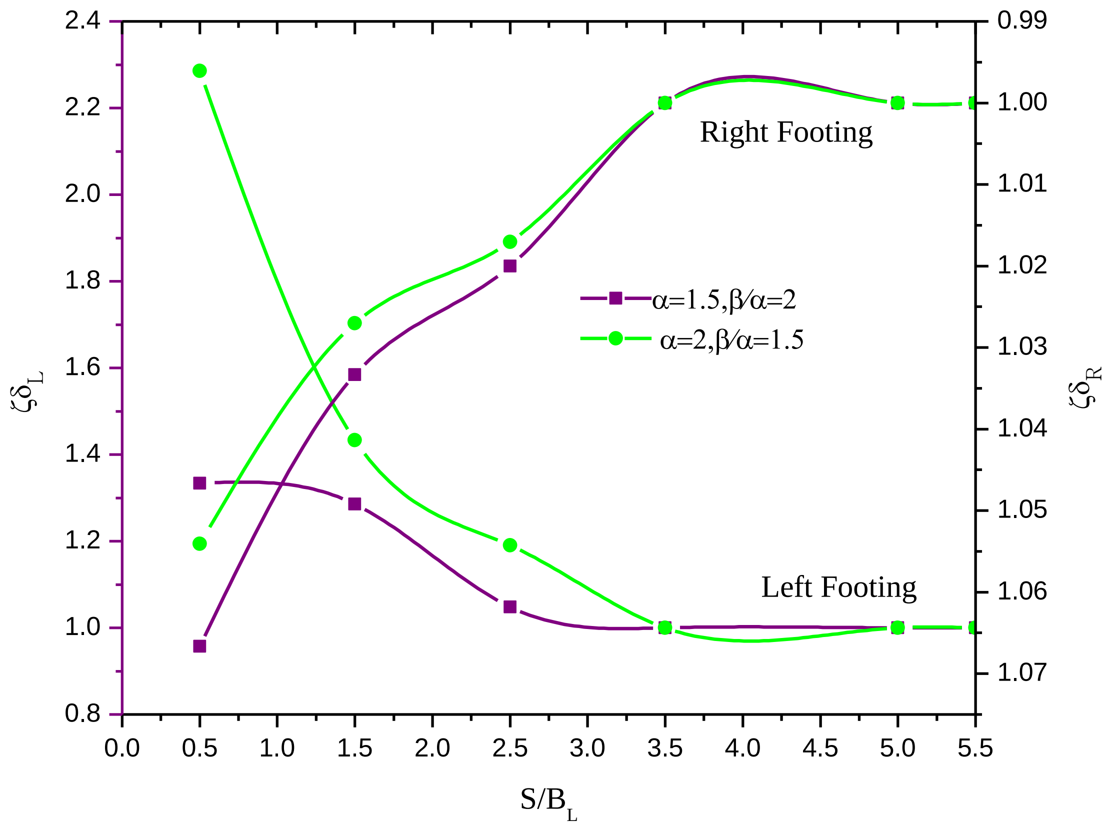


Figure 4.26: Variation of the Tilting Factors with S/B_L for; α = 1.5, 2; and β/α =1.5, 2.

It is observed that settlement factor decreases as spacing between footings increase. It is maximum at S/B_L = 0.5 and become unite at S/B_L = 5.0 and more. It indicates that footings act as isolate footing as space between footing far apart as $S/B_L \geq 5.0$. The interaction factors of the left footing increase with an increase in the width of the right footing, according to this study. This is attributable to the fact that a larger footing have a greater impact on a smaller footing.

Similar analysis was carried for α= 2.0 and the interaction factors for the left footing were found to be higher than the interaction factors for the right footing. This may be because the right footing's stress zone interferes significantly with the left footing's stress zone, and vice versa, with the left footings stress zone resulting in fewer interaction factors for the right footing. Furthermore, as the load intensity on the right footing increases, the zone of impact of the right footing increases, resulting in higher interaction factors for the left footing. It was also found that when the spacing ratio between the two footings is 5.0 or more, they act as an isolated footing.

4.3.2.5 Symmetrical footings and Asymmetrical loading at different depth footings

Figure 4.27 presents the variations of the interaction factors with S/B_L for asymmetrical footing (α=1) and asymmetrical loading (β/α=3.0) when right footing embedded at D/BL =1.0 and left footing placed at surface S/B_L = 0.0. For comparison purpose, the plots of the asymmetrical footing and symmetrical loading (α =1.0, β/α=3.0 and D/B_L = 0.0) are also presented.

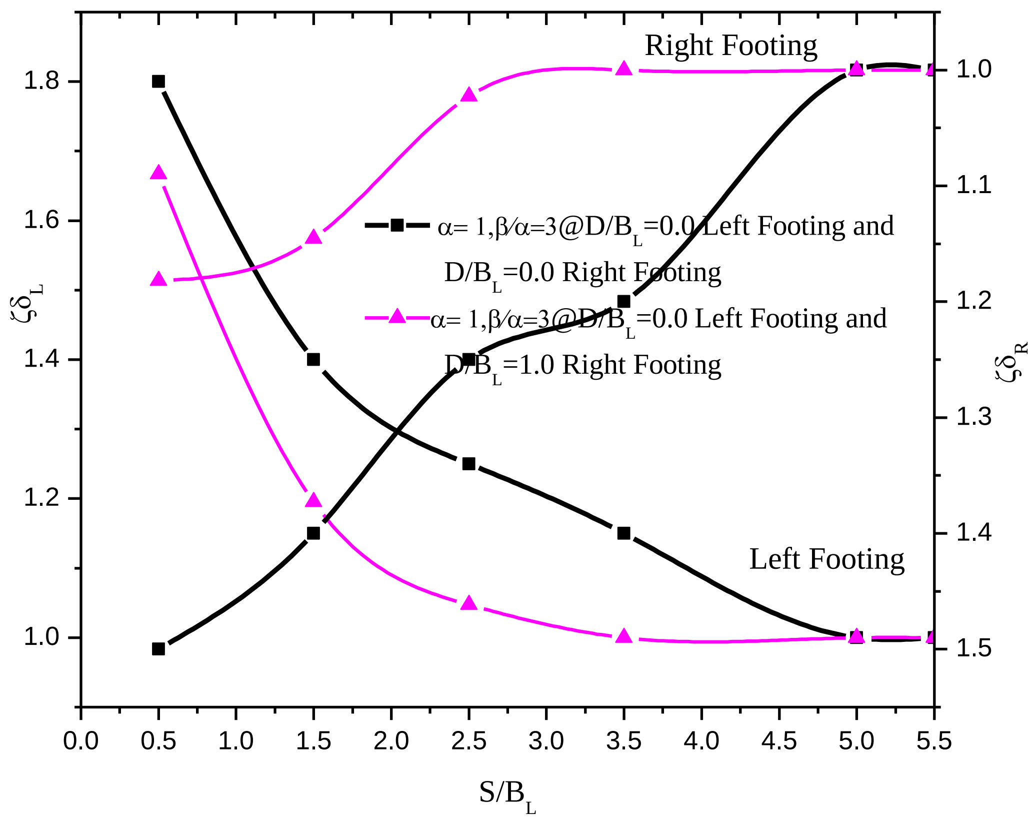


Figure 4.27: Settlement Factor Variation When Right Footing Embedded in Soil at D/B_L = 0.0, 1.0 with S/B_L for α = 1.0 and β/α =3.0.

In this case, it is obtained that, the interaction factors for left footing while right footing embedded in soil have more interference effect than when two footings placed at the surface. Like surface footing for the embedded footing, the interaction disappeared at $S/B_L \geq 5.0$. The larger settlement occur when the spacing between two footings close to each other.

4.3.2.6. Asymmetrical Footing and Asymmetrical Loading at Different Depth of Footings

Figure 4.28 presents the variations of the settlement interaction factors with S/B_L for asymmetrical footing (α=1) and asymmetrical loading (β/α=3.0) when right footing embedded at D/B_L =1.0. For evaluation purposes, the asymmetrical footing and asymmetrical loading (α =1.5, β/α=2.0 and D/B_L = 0.0) are also presented. The interaction factors for left footing while right footing embedded in soil have more interference effect than when two footings placed at surface. However, the left footing has less interfere effect on right footing while right footing embedded in soil to D/B_L = 1.0.

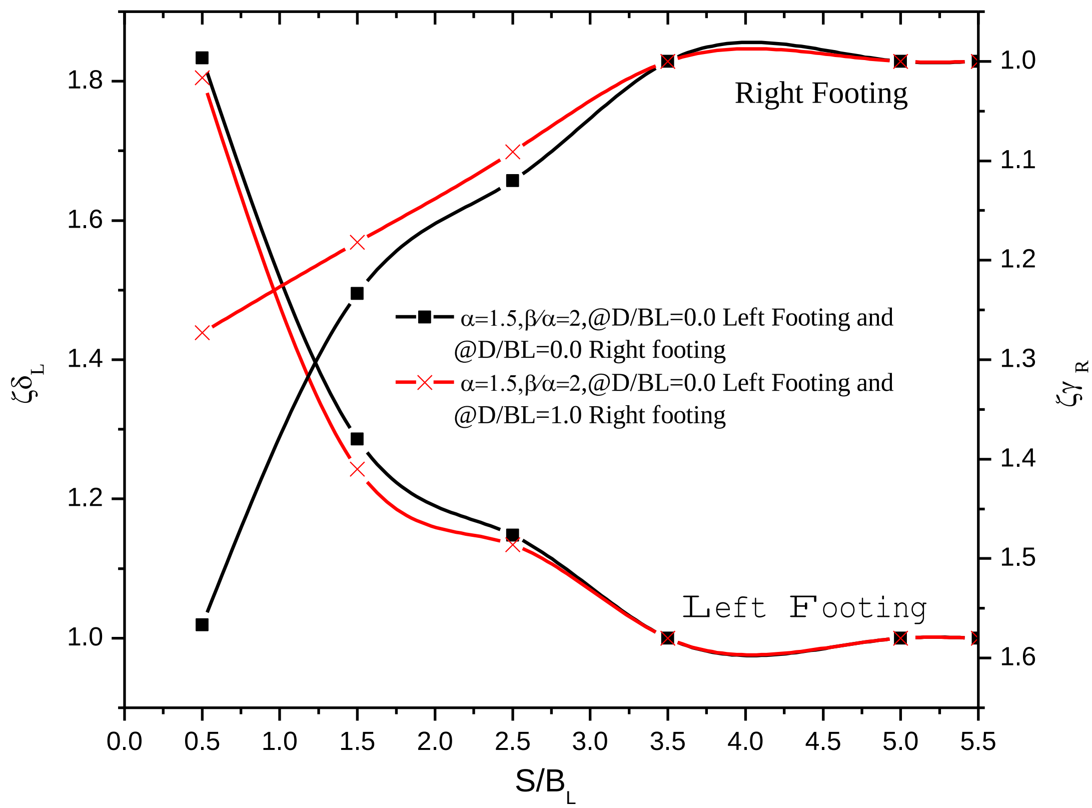


Figure 4.28: Settlement variation when right footing embedded in soil at D/BL = 0.0, 1.0 with S/B_L for α= 1.5 and β/α =2.0.

4.3.3 Tilting Variation

4.3.3.1 Symmetric footing and Symmetric loading

For symmetrical footing (α =1.0) and symmetrical loading (α/β =1.0), in Figure 4.29 shows the variations of the interaction tilting factors with S/B_L.

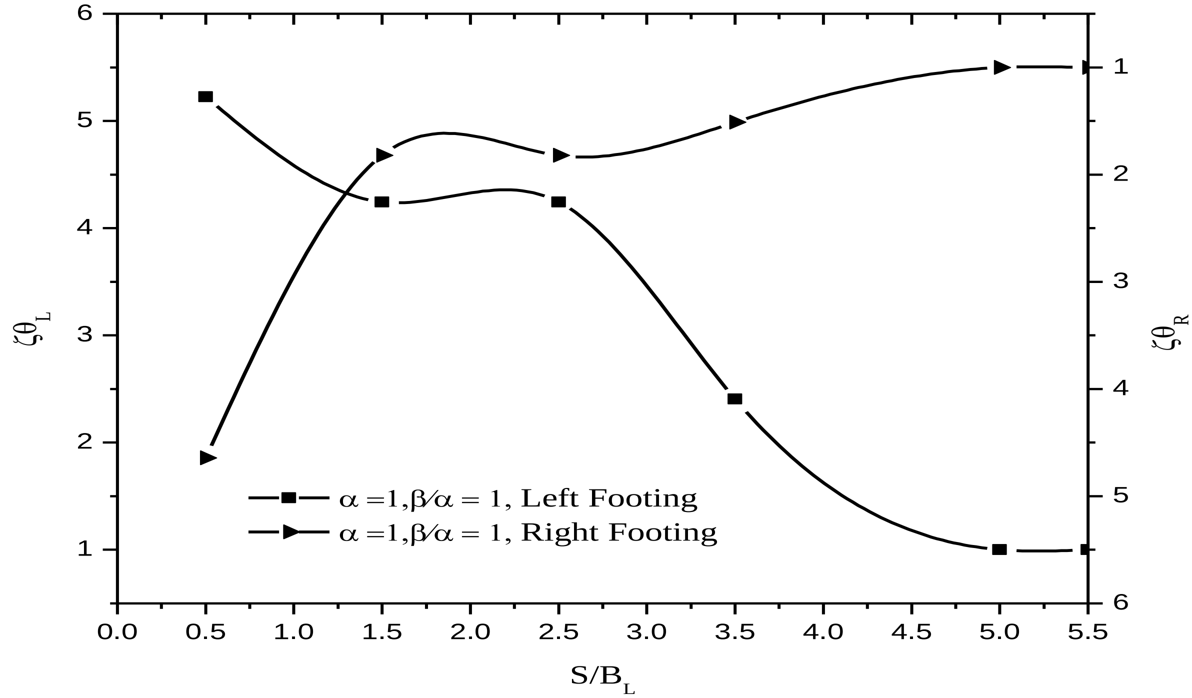


Figure 4.29: Variation of the tilting factors with S/B_L for; α=1; and β/α=1

From this study, it observed that the settlement at the base edge of the footing is not uniform for symmetrical footing and symmetrical loading since at mid between both footings the load come from two footings make unbalance load between footings which causes the tilting of footings. The effect of interference for footings is with some variation of the settlement indicates when the footings are set very close together, there is a noticeable tilt at $S/B_L = 0.5$ and decreases as S/B_L ratio increases. At footing clear spacing ratio $S/B_L = 5.0$ and more no interference take place between footings. The footings act as isolated footing.

4.3.3.2 Symmetrical footings and Asymmetrical loading

Figure 4.30 presents the variations of the interaction tilting factors with S/B_L for symmetrical footing (α=1.0) and assymmetrical loading (β/α=1.0, 1.25.1.50, 1.75, 2.0).

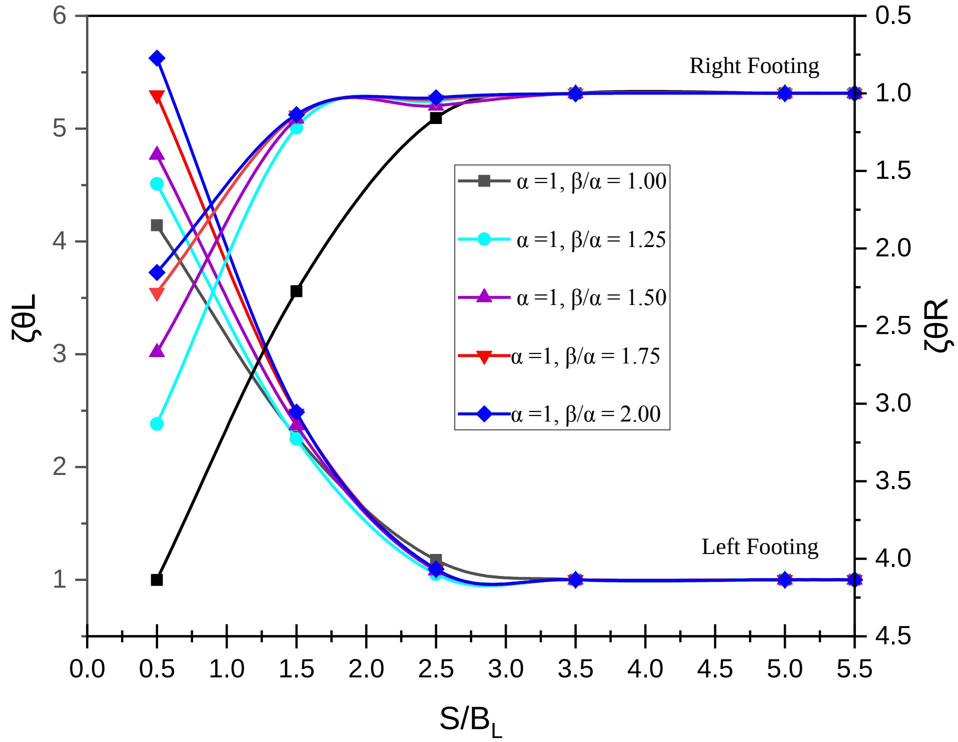


Figure 4.30: Variation of the tilting interaction factors with S/B_L for; α =1; and β/α =1 – 2

As can be seen in this case, the settlement at the footing's base is non-uniform due to asymmetric loading, which affects the footing's tilt. Maximum and minimum settlement were found at the inner and outer edges of the interfering footings, respectively. At S/B_L=0.5, a noticeable tilt in the footing is observed, which is important for practical considerations; however at large spacing S/B_L = 5.0 and more, no interference effect of the footings.

4.3.3.3 Asymmetrical footings and Symmetrical loading

Figure 4.31 shows that for asymmetric footing and symmetric loading, the settlement at the footing's base is non-uniform, influencing the tilt of the footing.

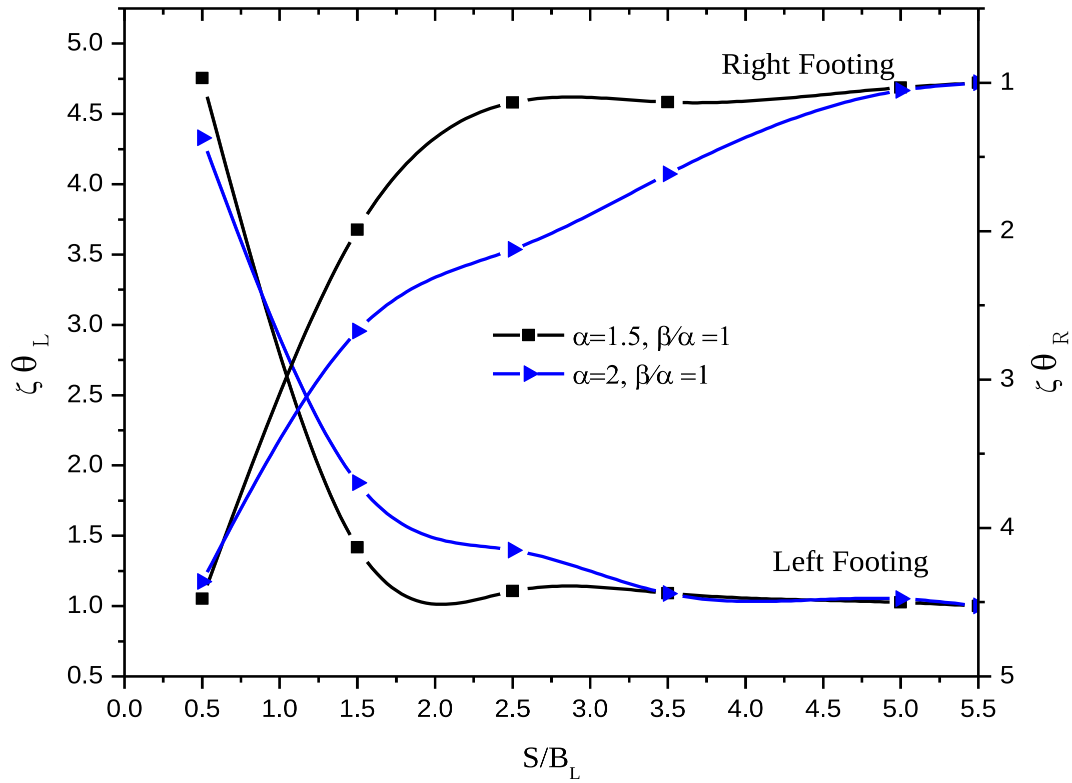


Figure 4.31: Variation of the Tilting Factors with S/B_L for; α = 1.5, 2; and β/α =1

In this case, it observed that as footing size increases the interaction footings also increases. The larger footing size influences the lesser footing size to be tilt. From this studies it also seen that the significant tilting considered at S/B_L = 0.5 and with no influences at S/B_L = 5.0 and more. It indicates that these footings act as single footing while far apart as $S/B_L \geq 5.0$.

4.3.3.4 Asymmetrical footings and Asymmetrical loading

The Footing width varies with α =1.5, 2.0 to represent asymmetrical footing and the footing width varies with β/α = 1.5 and 2.0 to indicate asymmetrical loading shown in Figure 4-32.

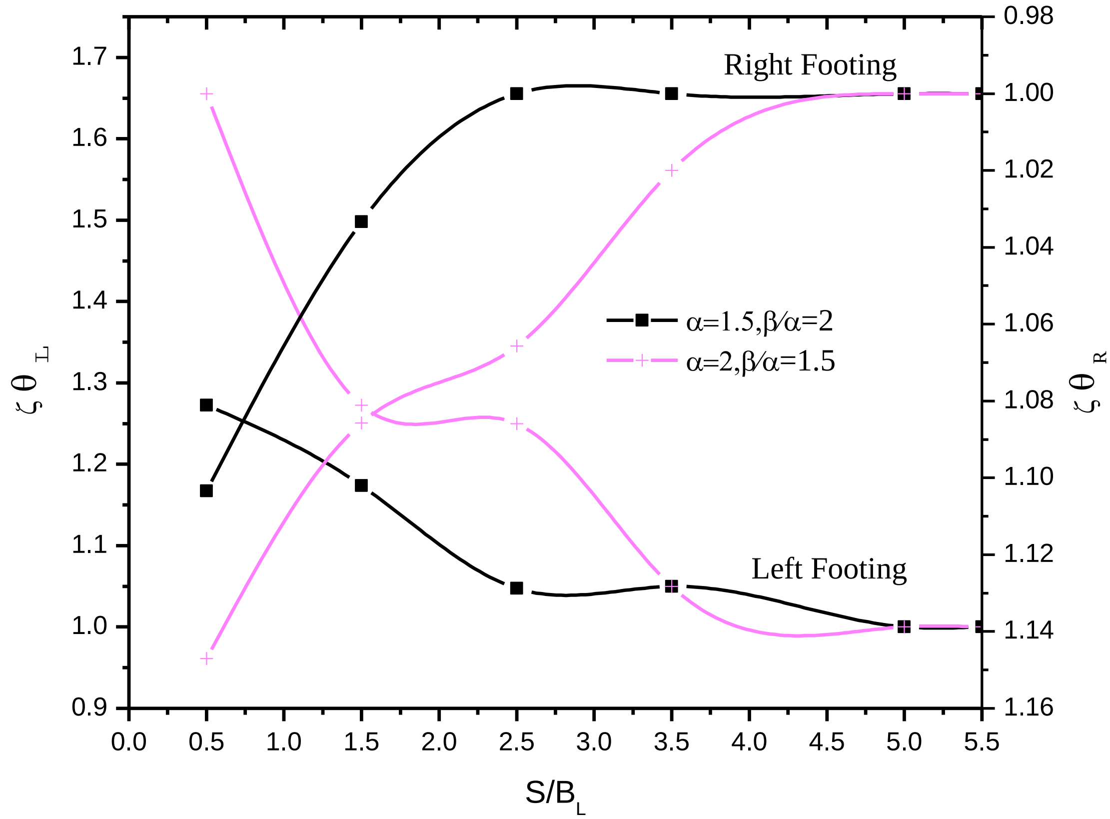


Figure 4.32: Variation of the tilting factors with S/B_L for; $\alpha = 2$; and $\beta/\alpha = 1.5$

It observed that the larger load and footing size settled, and influences the smaller footing size and load. The smaller footing has more tilting values due to the interference of the larger footing. At $S/B_L = 0.5$, the footing tilts significantly, which is well intentioned for practical reasons, and at $S/B_L \geq 5.0$, the footings behave as individual footing.

4.3.3.5 Symmetrical footings and Asymmetrical loading at different depth footings

The load varies with $\alpha = 1.0$, $\beta/\alpha = 3.0$, the left footings at surface at $D/B_L = 0.0$, and the right footing embedded in soil at $D/B_L = 1.0$ to characterize the case of symmetrical footing and asymmetrical loading conditions. The tilting factor has been observed as shown in Figure 4.33.

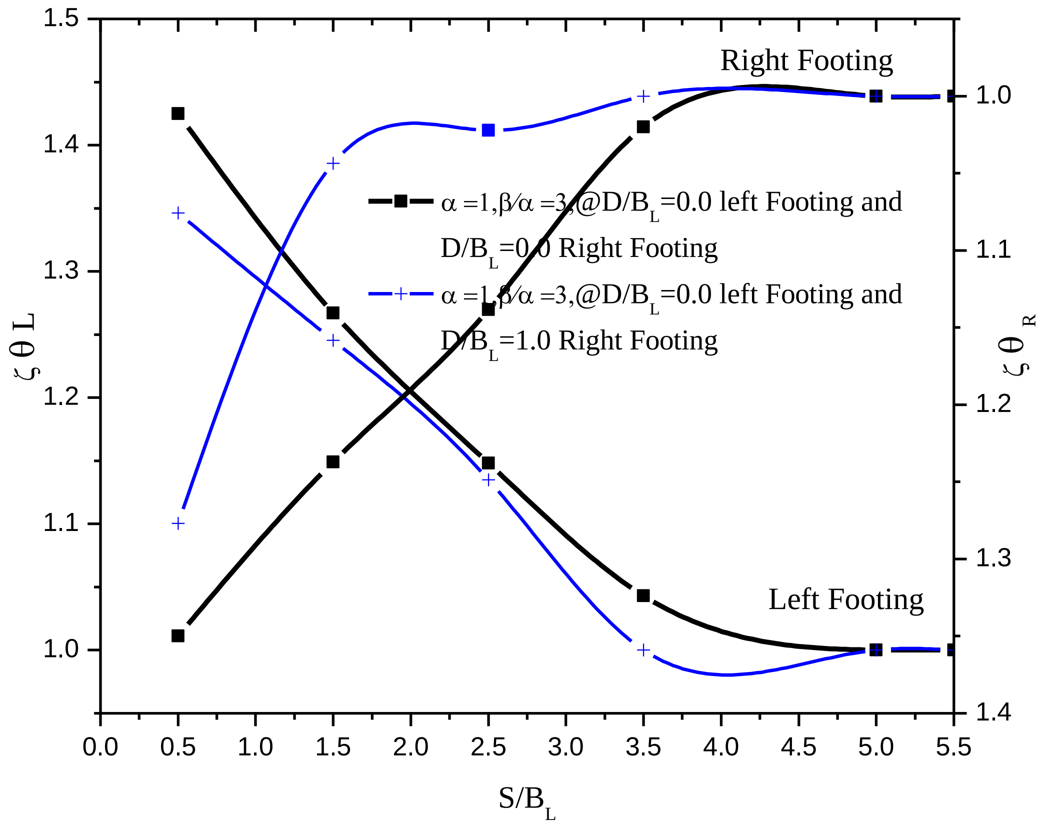


Figure 4.33: Tilting variation when right footing embedded in soil at D/B_L = 0.0, 1.0 with S/B_L for α = 1.0 and β/α = 2.0.

The interaction factors for tilting at surface D/B_L = 0.0 have more interference effect than when two footings placed at same level of depth with high value of load at right footing when compare to the same footing and load only vary with the depth of right footing embedded in soil to D/B_L =1.0. while the right footing embedded in soil the effect of left footing on right footing less almost near to constant that indicates there is no much tilting of embedded depth of right footing. However, the interference for left footing is higher than right footing, which indicates the effect of right footing on left footing at the surface of soil is more while right footing embedded into soil.

4.3.3.6. Asymmetrical Footing and Asymmetrical Loading at Different Depth of Footings

The load varies with α =1.5, β/α =2.0, the left footings at surface at D/B_L = 0.0, and the right footing embedded in soil at D/B_L =1.0 to characterize the case of assymmetrical footing and assymmetrical loading conditions. The tilting factor has been observed as shown in Figure 4.34.

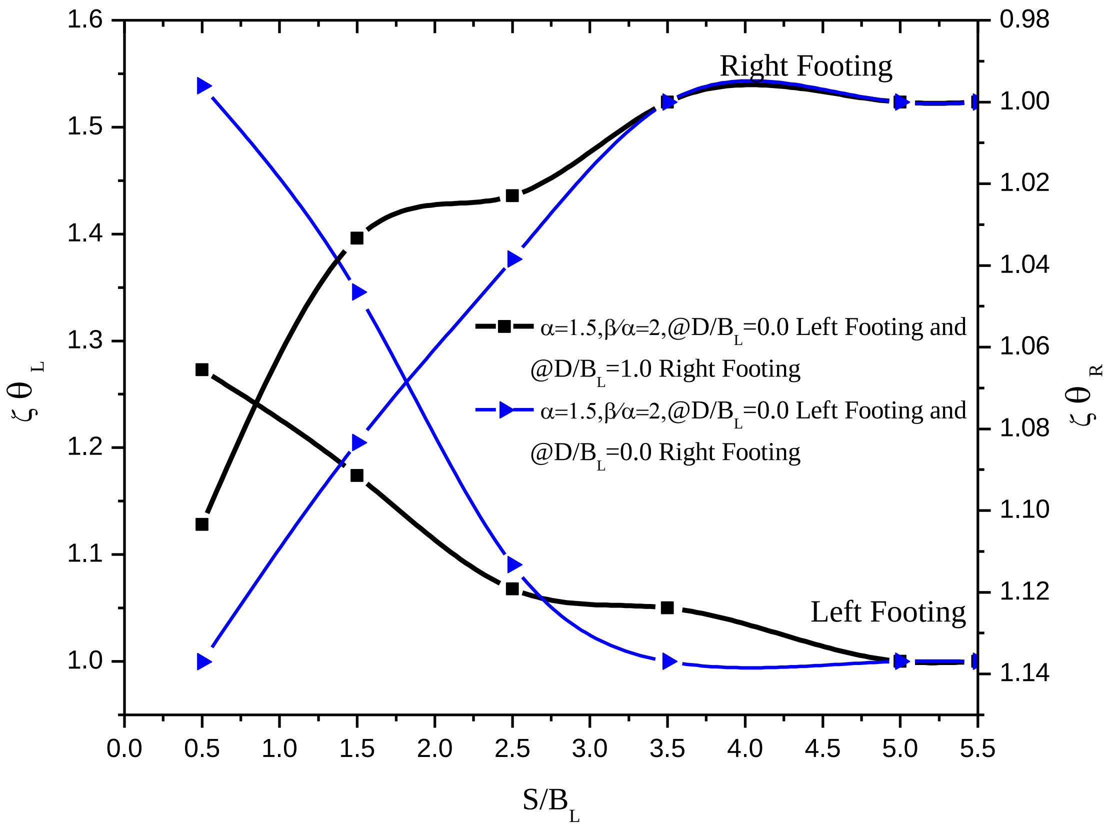


Figure 4.34: Tilting variation when right footing embedded in soil at D/B_L = 0.0, 1.0 with S/B_L for α = 1.0 and β/α =2.0.

The interaction factors for tilting at surface D/B_L = 0.0 have more interference effect than when two footings placed at same level of depth with high value of load at right footing when compare to the same footing and load, only by varying with the depth of right footing embedded in soil to D/B_L = 1.0. While the right footing embedded in soil the effect of left footing on right footing less in in interference that indicates there is no much tilting of embedded depth of right footing. However, the interference for left footing is higher than right footing, which indicates the effect of right footing on left footing at the surface of soil is more while right footing embedded into soil.

4.3.2 Effects of Ground Water Level

The parameters for this parametric studies evaluated based on varying water ground level from 2.20m to 2.60m with correspondence of spacing variation for each case until the failure of soil obtained to evaluate the bearing pressure, settlement, tilting for symmetrical footing and symmetrical loading.

4.3.2.1 Bearing factor variation with ground water tables variation

Figure 4.34 indicates the bearing factor ratio when ground water table about 2.20m and when the ground water table drop to about 2.60m from ground level. The study evaluated for symmetrical footing and symmetrical loading and compared both ground water level at 2.20m and 2.60m.

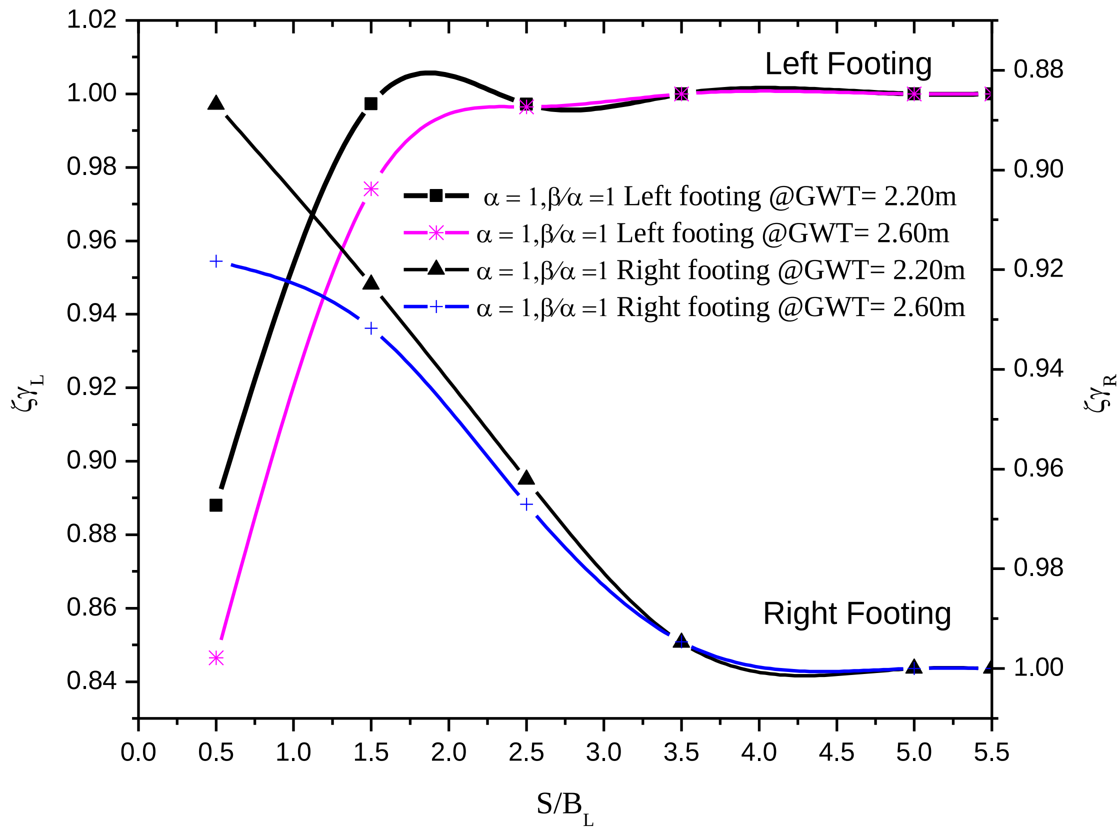


Figure 4.35: Bearing factor variation with ground water table variation

It obtained that the bearing factor ratio when ground water table about 2.20m is more than the bearing factor of when ground water table drop to about 2.60m from ground level. This indicates the bearing capacity of soil will have been decreases as water table near to the ground.

The interference effect of footing at water level increases to the ground level has more effect than while the ground water far from ground level. The maximum variation of bearing capacity have been studied when clear spacing about S = $0.5B_L$ and the interference of the footings starts to disappear when footings far apart S = $5.0B_L$ when ground water varies from 2.20m to 2.60m. This indicates the footing act as isolate footing when they are far apart as $S/B_L \geq 5$. It is observed that the settlement obtained when the ground water level near the ground is greater than ground water level is far from ground level and the bearing pressure of interfere footing decreases by 11.2% and 10.84% at $S/B_L = 0.5$ when the ground water level at 2.20m and 2.60m respectively.

4.3.2.2 Settlement variation with ground water tables variation

Figure 4.36 shows the interaction factors' variations with S/B_L for symmetrical footing, α=1.0 and symmetrical loading, β/α=1.0 while water tables varies from 2.20m to 2.60m for comparison purposes.

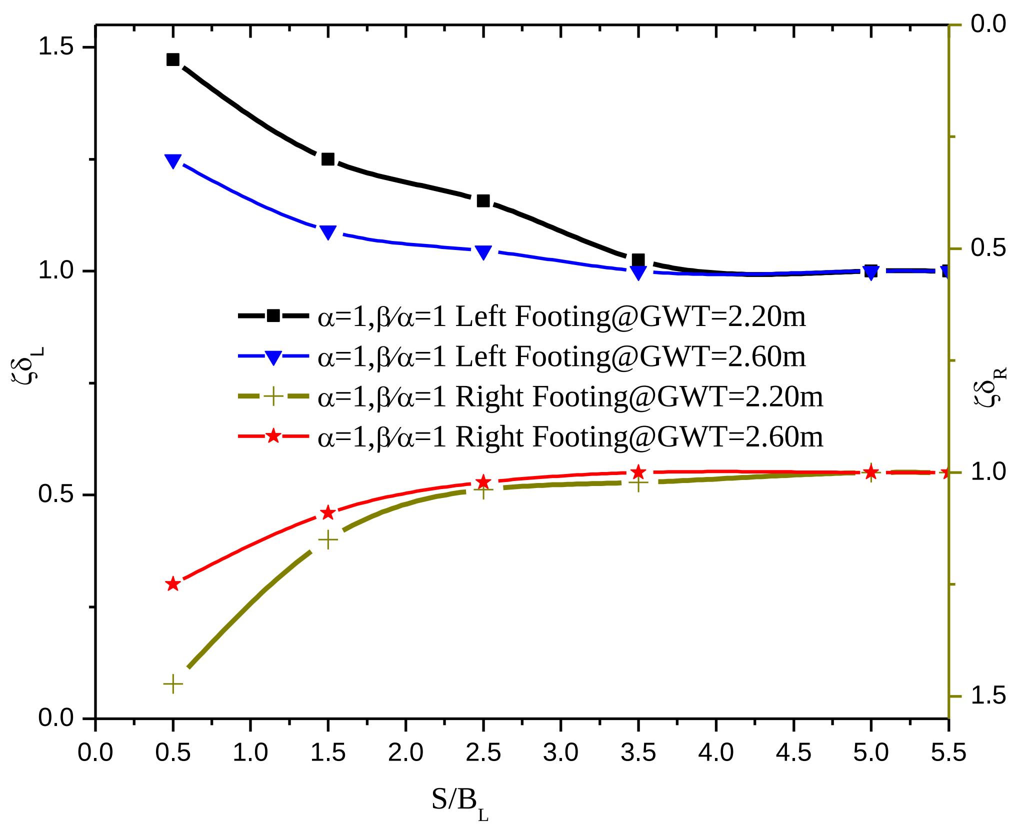


Figure 4.36: Settlement variation with ground water variation

It obtained that the interference effect of footing at water level increases to the ground level has more effect than while the ground water far from ground level. In case the effect of interference was, disappear when footing far from each other ($S/B_L \geq 5.0$) for ground water level reduced to 2.60m. However, the settlement of interfering footing increased by 33% and 20% at $S/B_L = 0.5$ when the ground water level at of 2.20m and 2.60m respectively.

4.3.2.3 Tilting variation with ground water tables variation

Figure 4.37 indicates the tilting factor ratio when ground water table about 2.20m and when the ground water table drop to about 2.60m from ground level. The study evaluated for symmetrical footing and symmetrical loading and the values of tilting factors illustrated as Figure 4.36.

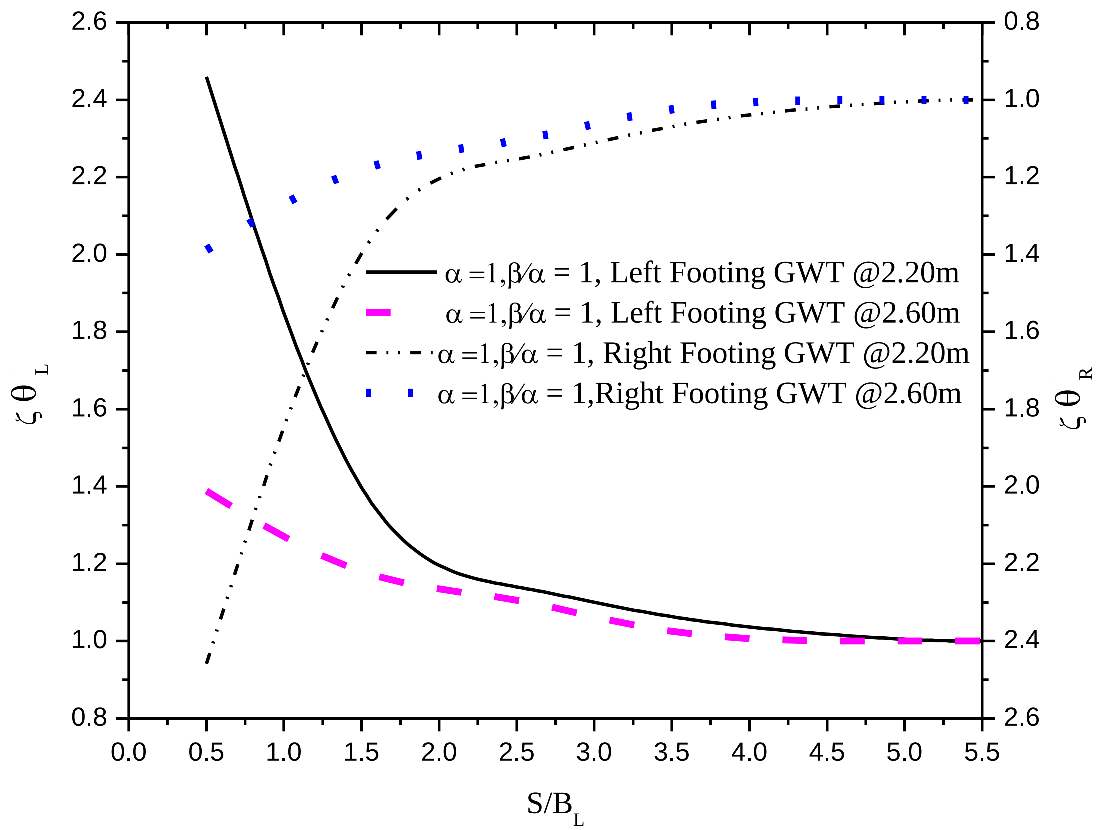


Figure 4.37: tilting variation with ground water variation

It is obtained that the tilting variations of the interaction factors when water level far from the ground level has less effect than while the ground water near to ground level. In this cases the effect of interference was become unit when footing far from each other ($S/B_L = 5.0$) shows both footings act as isolate footing for ground water level varies to 2.60m.

Summary of the Discussion

In a previous study, (Nainegali and Ekbote, 2019) compared the ultimate bearing capacity of interfering footings to that of isolated footings on clay soil, and found that at S = 0.5B, a difference of 2.8% and 9.12% in ultimate bearing capacity with that of isolated footing is observed for D/B = 0.0 and 1.0, respectively. It also noted that the signifying settlement of interfering footing is greater than isolated footing, decreases with increasing S/B ratio, and reaches a value of one at S/B = 5 and more, wherein footings can be stated to act individually. The increase in settlement at S = 0.5B is said to be about 36% for surface footings and 62% for embedded footings, respectively.

In present work the studies based on different condition for two closely spaced footing, however in all cases, the bearing pressure and settlement of interfering footings are at the maximum when the spacing between adjacent footings is about $0.5B_L$. When the spacing between adjacent footings is about $5B_L$, the bearing pressure and settling of the interfering footings, decreases and the isolated footing value reached. When comparing all of the situations, the assymmetrical footings size and assymmetrical loading condition resulted in the highest percent of bearing pressure and settlement impact. It shows that as the footing size and load increase, the interference effect increases. The interference effect of the footings, on the other hand, decreases as the depth of the footings increases, hence the stiffness soil increases with depth of soil and the effect reduces as ground water levels fall. It is also observed that, when water depth varies from 2.20m to 2.60m the settlement effect reduces from 33% to 20% which indicates water depth from footing depth have more influence on the interaction of footings. It can also be noticed that, significant settlement of interfering footing is greater than isolated footing is obtained at $S/B_L = 0.5$, wherein the bearing pressure of the interfere footings is not much significant in case of both surface and embedded footings, respectively. Table 4.21 shows the detailed outcome description for both events, as well as the impact of bearing pressure and footing settlement at $S = 0.5B_L$.

Table 4-21: Summary of the result discussion

Sect.	Footings condition	Effect on Left footing @ $S/B_L = 0.5$		Remarks
		Bearing Pressure	Settlement	
1	Symmetrical footing size and Symmetrical loading	11.73%	33.33%	
2	Symmetrical footing size and Assymmetrical loading	12% - 22.39%	33% - 35%	
3	Assymmetrical footing size and Symmetrical loading	13% - 14%	33% - 35%	Effect increases as footing size and load increases.
4	Assymmetrical footing and Assymmetrical loading	14.19 -15.34%	26% - 56.25%	
5	Assymmetrical footing size and Assymmetrical loading @ $D/B_L = 0.0$ and 1.0	26.42% - 32.11%	45.46% - 47.81%	Effect increases as depth of right footing increases.
6	Symmetrical footing size and Assymmetrical loading @ $D/B_L = 0.0$ and 1.0	15.33% - 20.20%	44.44% - 46.42%	
7	Ground water variation From 2.2m – 2.6m	11.2% - 10.84%	33% - 20%	Effect decrease as depth of Ground water reduced.

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

The following comprehensive conclusions obtained from the studies.

Interfering footings have a lower bearing capacity than isolated footings with equal footing and loading. With increasing distance between the footings, the bearing capacity progressively increases until it reaches the same value as the isolated footing.

Interfering footings settle more than isolated footings of equal width and loading. With decreasing in the clear spacing between the footings, the settlement continues to increase and as the spacing between footings is increased, the interaction factors decrease and gradually become unity, with the effect of interference being negligible.

In the case of asymmetrical loading and footing size, the footing with the greater width and load will have a significant impact on the footing with the smaller values. When two symmetrical footings with symmetrical loading and symmetrical footing size are placed at $S/B_L = 0.0$, they act as a single footing. However, they do not serve as a single footing for asymmetrical footing size and load since the larger size or load can settle and interfere the smaller footing.

When the footings are in the region of interference, the footings can tilt. When footings are close together, the tilt is greater, and as the spacing increases, the tilt decreases. For symmetrical footing and symmetrical loading, the direction of tilting of footings toward each other as spacing between footings near to each other. However, by the spacing between footings, the direction of the tilt of the footings can be reversed. When the larger footing settles, the smaller footings tilt toward the larger footing, causing the right footing to tilt toward the right side.

The interaction factors of the left footing increase as the depth of the right footing increases for different depths of footings. When two footings are mounted at the same level but have different loads, the interference effect is greater.

The bearing capacity, the settlement and the tilt of the interfering footings maximum when the spacing between adjacent footings is about $0.5B_L$ and attains a value as isolated footing at greater spacing ($S \geq 5B_L$). The interference effect of interfering footings when ground water level far from the footing depth decreases with increase in the spacing between the footings. However, when water depth varies from 2.20m to 2.60m the settlement effect reduces from 33% to 20%. Thus, water depth from footing depth have more influence on the interaction of footings.

5.2 RECOMMENDATION

The following recommendations made for application purpose and for the further studies conducted in the future.

- Study focused on two-layered clay soil for further it can be studied on other soil.
- The study focused on isolated footings, for further study another researcher can be study for others type of footings.
- This study focus on statical vertical loading it has to be consider for inclined loading and dynamic loading in the future.

REFERENCE

Al-Taie, E., Al-Ansari, N. and Knutsson, S. (2016) 'Evaluation of Foundation Settlement under Various Added Loads in Different Locations of Iraq Using Finite Element', *Engineering*, 08(05), pp. 257–268. doi: 10.4236/eng.2016.85022.

Alimardani, A. and Ghazavi, M. (2012) 'Behavior of closely spaced square and circular footings on reinforced sand', *Soils and Foundations*. Elsevier, 52(1), pp. 160–167. doi: 10.1016/j.sandf.2012.01.006.

Alwalan, M. F. (2018) 'Interaction of Closely Spaced Shallow Foundations on Sands and Clays: A review', *International Journal of Advanced Engineering Research and Science*, 5(9), pp. 101–110. doi: 10.22161/ijaers.5.9.11.

AMER, R. (1962) 'SETTLEMENT AND TILT of TWO INTERFERING FOOTINGS ON clay'.

Aytekin, M. (2016) 'Impact of adjacent footings on immediate settlement of shallow footings', *Challenge Journal of Structural Mechanics*, (March). doi: 10.20528/cjsmec.2016.01.001.

Briaud, JL. (2010) Introduction to Soil Moduli. Static2.docstoccdn.com/docs/42184625/1

Brinkgreve, R. B. J. (2005) 'Copyright ASCE 2005 69 Soil Constitutive Models Evaluation, Selection, and Calibration', *Geo-Frontiers Congress 2005*, pp. 69–98. doi: 10.1158/1538-7445.AM2013-1522.

Foye, K. C., Basu, P. and Prezzi, M. (2008) 'Immediate settlement of shallow foundations bearing on clay', *International Journal of Geomechanics*, 8(5), pp. 300–310. doi: 10.1061/(ASCE)1532-3641(2008)8:5(300).

Ghosh, P. *et al.* (2015) 'Experimental studies on interference of two angular footings resting on surface of two-layer cohesionless soil deposit', 9(4). doi: 10.1179/1939787914Y.0000000080.

Ghosh, P. and Sharma, A. (2010) 'Interference effect of two nearby strip footings on layered soil: Theory of elasticity approach', *Acta Geotechnica*, 5(3), pp. 189–198. doi: 10.1007/s11440-010-0123-2.

Glendinning, S., Jones, C. J. F. P. and Lamont-Black, J. (2015) *The Use of Electrokinetic Geosynthetics to Improve Soft Soils, Ground Improvement Case Histories: Chemical, Electrokinetic, Thermal and Bioengineering Methods*. Elsevier Ltd. doi: 10.1016/B978-0-08-

100191-2.00013-7.

Ismail, A. and Teshome, F. (2011) ‘Analysis of deformations in soft clay due to unloading’, p. 74.

IZUMI, M. and SONOHARA, M. (2008) ‘Introduction to the Feature’, *Igaku Toshokan*, 55(3), pp. 211–211. doi: 10.7142/igakutoshokan.55.211.

Jackson, N. and Dhir, R. K. (1996) *Compressibility of Soil*, *Civil Engineering Materials*. doi: 10.1007/978-1-349-13729-9_26.

K. Mitchell, J. and Kenichi, S. (2005) ‘Fundamentals of Soil Behavior, 3rd Edition’, p. 592.

Kim, Y., Park, H. and Jeong, S. (2017) ‘Settlement behavior of shallow foundations in unsaturated soils under rainfall’, *Sustainability (Switzerland)*, 9(8). doi: 10.3390/su9081417.

Kumar, J. and Bhoi, M. K. (2009) ‘Interference of Two Closely Spaced Strip Footings on Sand’, (April), pp. 595–604.

Laman, M. and Yildiz, A. (2007) ‘Numerical studies of ring foundations on geogrid-reinforced sand’, *Geosynthetics International*, 14(2), pp. 52–64. doi: 10.1680/gein.2007.14.2.52.

Luévanos Rojas, A. (2016) ‘A comparative study for the design of rectangular and circular isolated footings using new models’, *Dyna*, 83(196), pp. 149–158. doi: 10.15446/dyna.v83n196.51056.

‘lymon,2006.pdf.opdownload’ (no date).

Mohammed, J. (2015) ‘Soil & Soil Mechanics’, *Book*, (April), pp. 0–147. doi: 10.13140/RG.2.2.31964.39047.

Murthy, V.N.S. (2002) Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering. New York : Marcel Dekker Inc,.

Murthy, V. N. . (2007) ‘Advanced Foundation Engineering’, p. 795.

Nainegali, L. and Ekbote, A. G. (2019) *Interference of two nearby footings resting on clay medium*, *Lecture Notes in Civil Engineering*. Springer Singapore. doi: 10.1007/978-981-13-0368-5_7.

Nainegali, L. S. *et al*. (2013) ‘Interference of Two Asymmetric Closely Spaced Strip Footings Resting on Nonhomogeneous and Linearly Elastic Soil Bed’, (December), pp. 840–852. doi: 10.1061/(ASCE)GM.1943-5622.0000290.

Nainegali, L. S., Ghosh, P. and Basudhar, P. K. (2013) ‘Interaction of Nearby Strip Footings Under

Inclined Loading Interaction de semelles rapprochées soumises à des charges inclinées', pp. 3459–3462.

Nimeri, M., Elfass, S. and Norris, G. (2017) 'Load–settlement response of shallow foundations resting on granular soil', *Innovative Infrastructure Solutions*. Springer International Publishing, 2(1). doi: 10.1007/s41062-017-0080-9.

Noorzad, R. and Manavirad, E. (2012) 'Bearing capacity of two close strip footings on soft clay reinforced with geotextile'. doi: 10.1007/s12517-012-0771-7.

Obrzud, R. F. and Truty, A. (2018) 'The hardening soil model - a practical guidebook', 05, p. 205.

Ornek, M. *et al.* (2012) 'Numerical analysis of circular footings on natural clay stabilized with a granular fill', *Acta Geotechnica Slovenica*, 9(1), pp. 61–75.

P., S. (2011) 'Reliability Based Analysis of Slope , Foundation and Retaining Wall Using Finite Element Method', (June), p. 77.

Patel, M. and Bhoi, M. (2019) 'Effect of different shape of footing on its load-settlement behaviour (circular, square and rectangular)', *World Congress on Civil, Structural, and Environmental Engineering*, pp. 1–7. doi: 10.11159/icgre19.168.

'PLAXIS Material Models Manual 2013' (2013).

Salahudeen, A. B. *et al.* (2017) 'Investigation of Shallow Foundation Soil Bearing Capacity and Settlement Characteristics of Minna City Centre Development Site Using Plaxis 2D Software and Empirical Formulations', *Nigerian Journal of Technology*, 36(3), pp. 663-670–670.

Salahudeen, B. A. and Sadeeq, A. (2016a) 'Evaluation of Foundation Settlement Characteristics From Standard Penetration Test Results Using Empirical Models', (29), pp. 93–114.

Salahudeen, B. A. and Sadeeq, A. (2016b) 'EVALUATION OF FOUNDATION SETTLEMENT CHARACTERISTICS FROM STANDARD PENETRATION TEST RESULTS USING EMPIRICAL MODELS', (29), pp. 93–114.

Schanz, T. (2000) 'The hardening soil model : Formulation and verification', pp. 1–16.

Shahein, M. and Hefdhallah, A. (2013) 'Effect of Neighboring Footings on Single Footing Settlement', (April 2013), pp. 0–9.

Shahin, M. A., Maier, H. R. and Jaksa, M. B. (2002) ‘Predicting Settlement of Shallow Foundations using Neural Networks’, *Journal of Geotechnical and Geoenvironmental Engineering*, 128(9), pp. 785–793. doi: 10.1061/(asce)1090-0241(2002)128:9(785).

Shahin, M., Jaksa, M. B. and Maier, H. R. (2002) ‘Artificial Neural Network – Based Settlement Prediction Formula for Shallow Foundations on Granular Soils ARTIFICIAL NEURAL NETWORK − BASED SETTLEMENT PREDICTION’, (April 2015).

Stuart, J. G. (1958) ‘Interference Between Foundations , With Special’, 1. doi: 1962.12.1.15.

Terzaghi, K. (1943) ‘Theoretical Soil Mechanics’, *Theoretical Soil Mechanics*. doi: 10.1002/9780470172766.

Ti, K. S. *et al.* (1971) ‘A review of Basic Soil Constitutive Models for Geotechnical Application’, *Electronic Journal of Geotechnical Engineering*, 97(2), pp. 375–391. Available at: http://www.academia.edu/download/40591375/A_Review_of_Basic_Soil_Constitutive_models.pdf.

Verghese, S. J., Nguyen, C. T. and Bui, H. H. (2013) ‘EVALUATION OF PLASTICITY-BASED SOIL CONSTITUTIVE’, 5(2), pp. 672–677.

Wu, J. T. H. (2019) ‘Determination of Model Parameters for the Hardening Soil Model’.

Yonas, S. (2002) ‘a Historical Survey of Jimma Town’, (June), p. 173.

APPENDIX

Appendix I Specific gravity test results

Soil layer 1

Trial	1	2	3
mass of pychnometer, g	31.1	29.7	30
Mass of pychnometer with water at Ti, g	127.8	125.2	125.8
Initial temperature in degree Celsius, Ti, ◦c	23	23	23
Mass of pychnometer and soil , g	41.4	39.9	40
Mass of pychnometer, soil and water, g	134.3	131.6	132
Final temperature in degree Celsius, Tx, ◦c	25	25	25
Mass of soil,g	10.3	10.2	10
density of water at Ti	0.99732	0.99732	0.99732
density of water at Tx	0.99654	0.99654	0.99567
mass of pychnometer with water at final temperature,g	127.72	125.13	125.64
k	0.9983	0.9983	0.9974
specific gravity at Tx	2.76	2.73	2.74
Gs @20◦c	2.76	2.73	2.73
Gs @20 ◦c avg.	2.74		

Soil layer 2

Trial	1	2	3
mass of pychnometer, g	30.1	31.1	27.6
Mass of pychnometer with water at Ti, g	125.9	128	78.9
Initial temperature in degree Celsius, Ti, ◦c	19	19	19
Mass of pychnometer and soil , g	40.2	41	37.8
Mass of pychnometer, soil and water, g	132.14	134.1	85.2
Final temperature in degree Celsius, Tx, ◦c	22	22	22
mass of soil,g	10.1	9.9	10.2
density of water at Ti	0.99841	0.99841	0.99841

density of water at Tx	0.99777	0.99777	0.99777
mass of pychnometer with water at final temperature, g	125.84	127.94	78.87
k	0.99957	0.99957	0.99957
specific gravity at Tx	2.66	2.65	2.64
Gs @20◦c	2.66	2.65	2.64
Gs @20 ◦c avg.	**2.65**		

Appendix II Grain size analysis test results

Soil layer 1/Red brown soil

Sieve analysis

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.5	0	0	0	100
4.75	6.4	0.64	0.64	99.36
2	17.6	1.76	2.4	97.6
0.425	19.6	1.96	4.36	95.64
0.3	10.2	1.02	5.38	94.62
0.15	28.6	2.86	8.24	91.76
0.075	9.7	0.97	9.21	90.79
pan	907.9	90.79	100	0
sum	1000			

Hydrometer Analysis

Elapsed time, min	Temp. ^{0}c	Actual Hydr. reading,Rh	Hydr. reading corrected for meniscus(R’)	Effective Depth, L (mm)	Particle Diameter (mm)	Corr.Hydr .Rdg. R’’	% Finer P	% Adjusted Finer PA
1	21	51	52	7.8	0.037	45.2	88.63	80.47
2	21	49	50	8.1	0.026	43.2	84.71	76.91
4	21	47	48	8.4	0.019	41.2	80.79	73.35
8	21	45	46	8.8	0.014	39.2	76.86	69.79

15	21	42	43	9.2	0.010	36.2	70.98	64.44
30	21	40	41	9.6	0.007	34.2	67.06	60.88
60	21	38	39	9.9	0.005	32.2	63.14	57.32
120	22	36	37	10.2	0.004	30.4	59.61	54.12
240	22	34	35	10.6	0.003	28.4	55.69	50.56
480	21	32	33	10.9	0.002	26.2	51.37	46.64
1440	21	30	31	11.2	0.001	24.2	47.45	43.08

L_{eff} =16.3 - 0.1641R Gs=2.74 a = 0.6226(Gs/Gs -1) P= a*R" /Mo *100

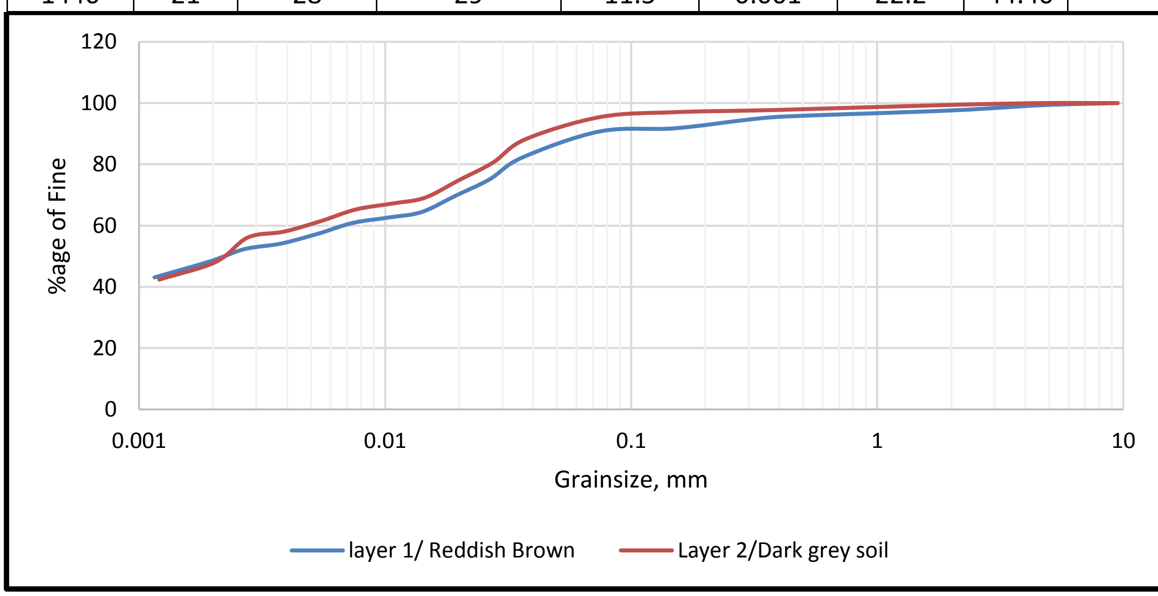
Soil layer 2/dark grey soil

Sieve Analysis

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.5	0	0	0	100
4.75	0	0	0	100
2	4.1	0.59	0.59	99.41
0.425	11	1.57	2.16	97.84
0.25	2	0.29	2.44	97.56
0.15	3.7	0.53	2.97	97.03
0.075	10.9	1.56	4.53	95.47
pan	668.3	95.47142857	100	0
sum	700			

Hydrometer Analysis

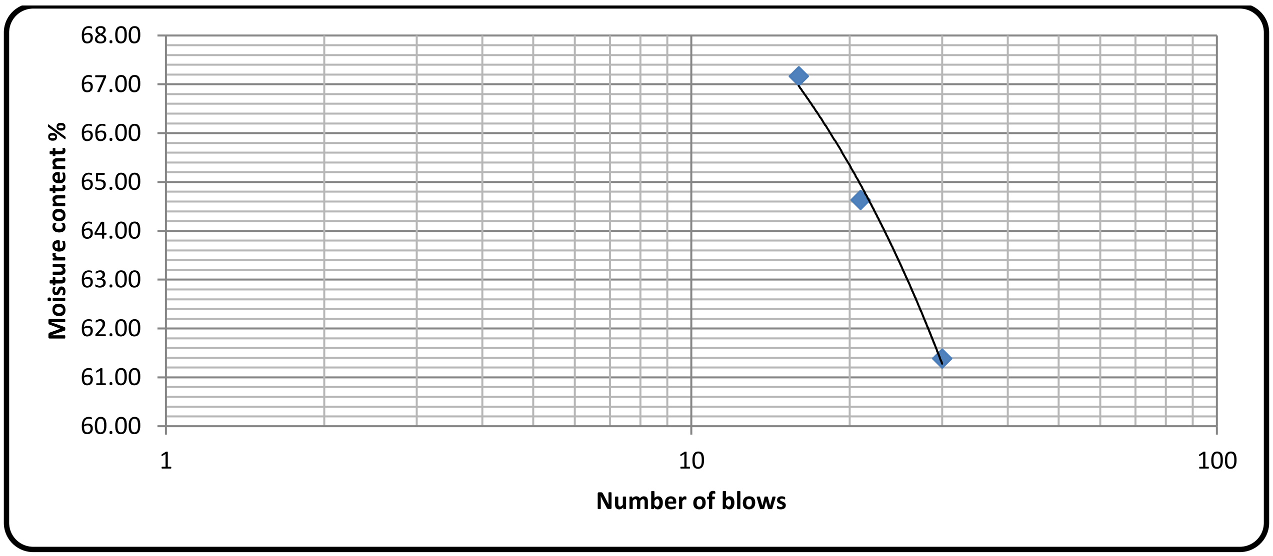
Elapsed time, min	Temp. ^{0}c	Actual Hydr. Reading, Rh	Hydr. reading corr. for meniscus(R')	Effective Depth, L (mm)	Particle Diameter (mm)	Corr. Hydr. Reading. (R'')	% Finer (P)	% Adjusted Finer (PA)
1	21	52	53	7.6	0.037	46.2	92.39	88.21
2	21	48	49	8.3	0.027	42.2	84.39	80.57
4	21	45	46	8.8	0.020	39.2	78.39	74.84
8	21	42	43	9.2	0.015	36.2	72.40	69.12
15	21	41	42	9.4	0.011	35.2	70.40	67.21
30	21	40	41	9.6	0.008	34.2	68.40	65.30
60	21	38	39	9.9	0.005	32.2	64.40	61.48
120	22	36	37	10.2	0.004	30.4	60.80	58.04
240	22	35	36	10.4	0.003	29.4	58.80	56.13
480	21	31	32	11.0	0.002	25.2	50.40	48.11
1440	21	28	29	11.5	0.001	22.2	44.40	42.39



Appendix III Atterberg limits test results

Soil layer 1

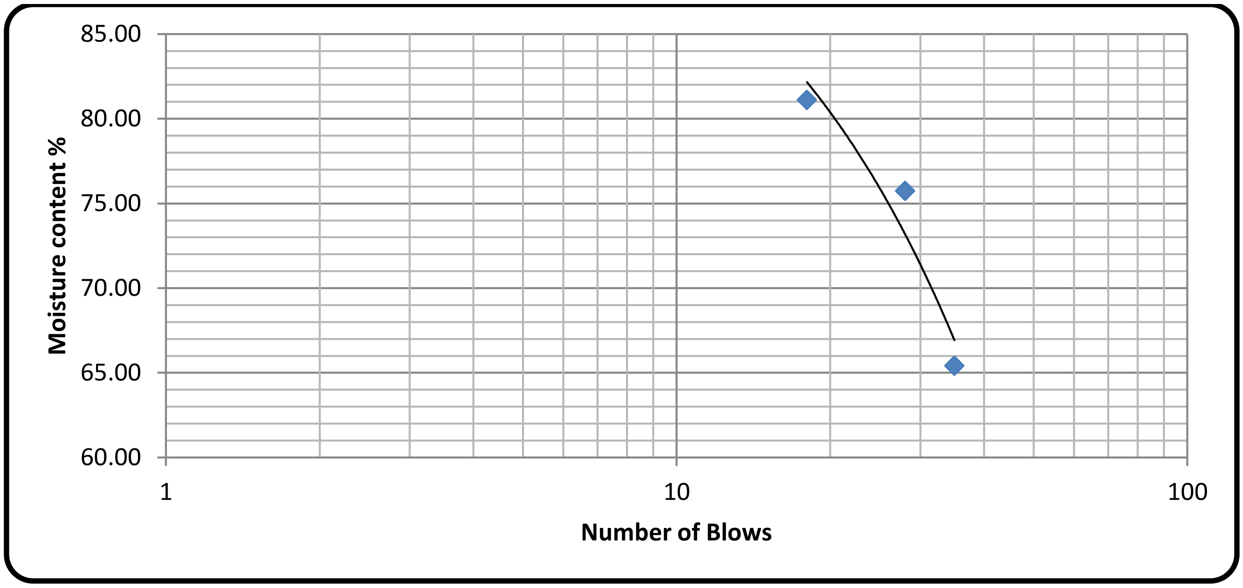
Determination	***Liquid Limit (D-4318)***			***Plastic Limit (D-4318)***	
Number of blows	30	21	16		
Test No	1	2	3	1	2
Wt. of Container, (g)	37.8	37.3	18	6.3	6.4
Wt. of container + wet soil, (g)	54.1	50.8	29.2	12.2	12.1
Wt. of container + dry soil, (g)	47.9	45.5	24.7	10.9	10.8
Wt. of water, (g)	6.2	5.3	4.5	1.3	1.3
Wt. of dry soil, (g)	10.1	8.2	6.7	4.6	4.4
Moisture container, (%)	61.39	64.63	67.16	28.26	29.55
Average				29	



Soil layer 2

Determination	**Liquid Limit (D-4318)**			**Plastic Limit (D-4318)**	
Number of blows	35	28	18		
Test No	1	2	3	1	2
Wt. of Container, (g)	17.8	28.7	18.1	6.5	6.4
Wt. of container + wet soil, (g)	31.1	50.3	32.1	11	12.1
Wt. of container + dry soil, (g)	25.84	40.99	25.83	9.9	10.7
Wt. of water, (g)	5.26	9.31	6.27	1.1	1.4

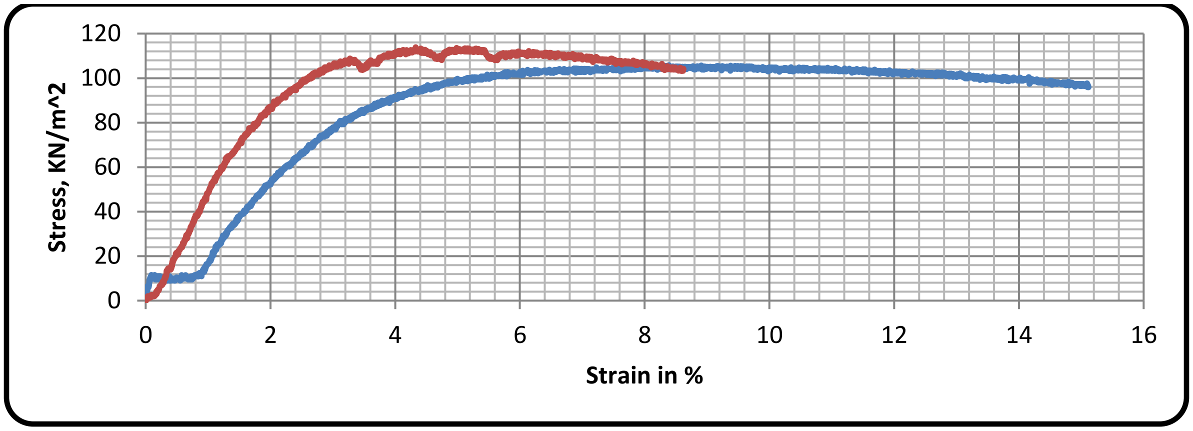
Wt. of dry soil, (g)	8.04	12.29	7.73	3.4	4.3
Moisture container, (%)	65.42	75.75	81.11	32.35	32.56
Average				32.46	



Appendix IV Unconfined compressive strength test results

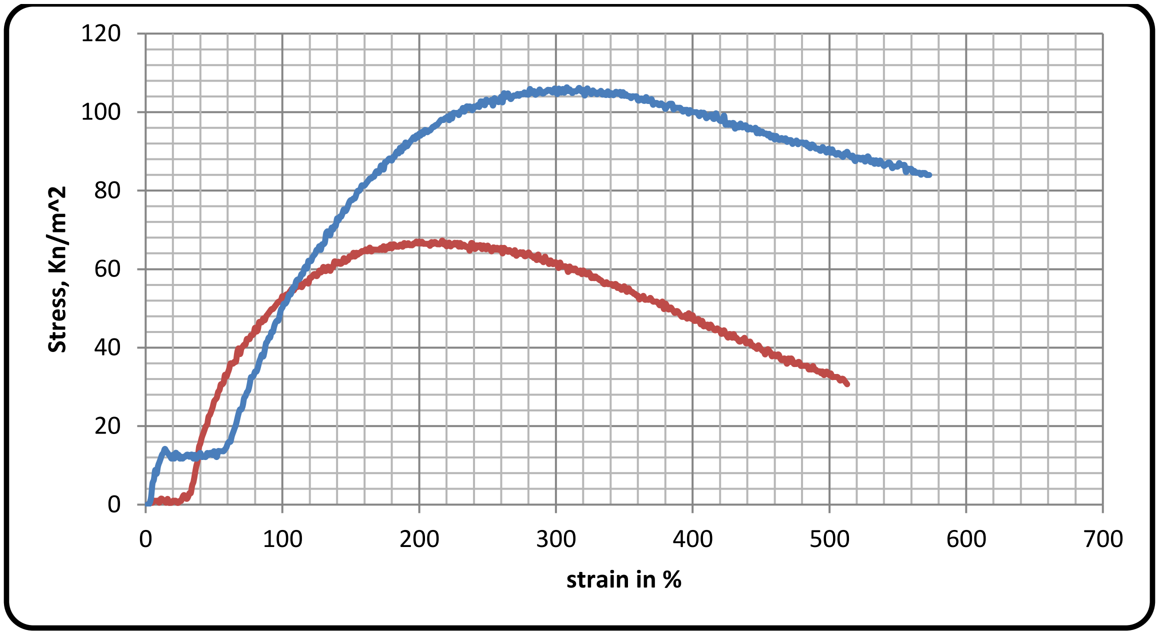
Soil layer 1

Trail		1	2	avg.
Density	(Kg/m^3)	1.94	1.83	1.88
Unit weight	(KN/m^3)	18.99	17.99	18.49
Moisture	(%)	45.74	46.69	46.21
Dry Unit weight	(KN/m^3)	12.73	12.27	12.50
Unconfined compressive strength (qu)	(KN/m^2)	113.87	105.81	109.84
Cohesion (c)	(KN/m^2)	56.93	52.90	54.92



Soil layer 2

Trail	1	2	avg.
Density (Kg/m^3)	1.67	1.67	1.67
Unit weight (KN/m^3)	16.42	16.43	16.42
Moisture (%)	56.06	55.91	55.99
Dry Unit weight (KN/m^3)	10.5196976	10.53731	10.53
Unconfined compressive strength (qu) (KN/m^2)	67.27	106.33	86.80
Cohesion (c) (KN/m^2)	33.63	53.16	43.40



Appendix V Consolidation test results

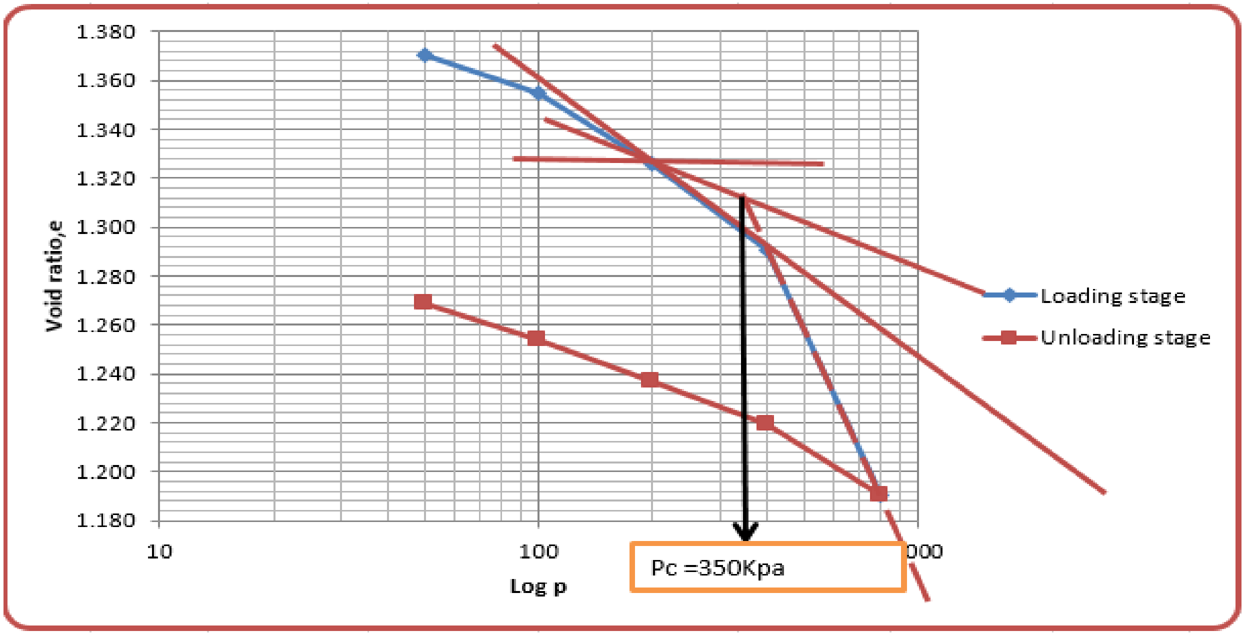
Soil layer 1

Time	Deformation @ 1kg	Deformation @ 2kg	Deformation @ 4kg	Deformation @ 8kg	Deformation @ 16kg	Unloading @ 8kg	Unloading @ 4kg	unloading @ 2kg	unloading @ 1kg
0.00	0.058	0.226	0.362	0.604	0.978	1.852	1.572	1.392	1.242
0.10	0.126	0.263	0.4	0.71	1.022	2.02	1.524	1.346	1.32
0.25	0.138	0.27	0.434	0.724	1.146	1.818	1.522	1.345	1.31
0.50	0.148	0.276	0.444	0.734	1.16	1.816	1.52	1.344	1.3
1.00	0.156	0.282	0.45	0.748	1.18	1.814	1.516	1.343	1.28
2.00	0.166	0.288	0.464	0.764	1.206	1.81	1.512	1.342	1.26
4.00	0.176	0.294	0.476	0.776	1.232	1.808	1.508	1.34	1.232
8.00	0.184	0.302	0.486	0.812	1.272	1.804	1.498	1.334	1.21
15.00	0.194	0.308	0.502	0.83	1.326	1.798	1.486	1.325	1.196

30.00	0.198	0.316	0.518	0.87	1.43	1.59	1.478	1.318	1.186
60.00	0.200	0.324	0.538	0.884	1.482	1.584	1.468	1.304	1.175
120.00	0.206	0.332	0.56	0.916	1.56	1.578	1.41	1.28	1.162
240.00	0.210	0.34	0.576	0.942	1.68	1.574	1.398	1.264	1.144
480.00	0.220	0.358	0.594	0.974	1.834	1.573	1.396	1.25	1.12
1440.00	0.226	0.362	0.604	0.978	1.852	1.572	1.392	1.242	1.112

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen,ΔH	Cumulative of change height of specimen	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]
50	0.120	0.170	0.22	2	19.830	9.9	0.058	0.162	0.162	0.019	1.371
100	0.264	0.312	0.36	18	19.688	9.8	0.226	0.134	0.296	0.035	1.355
200	0.424	0.512	0.6	38	19.488	9.7	0.362	0.238	0.534	0.064	1.326
400	0.720	0.810	0.9	6	19.190	9.6	0.604	0.296	0.83	0.099	1.291
800	0.898	1.359	1.82	20	18.641	9.3	0.978	0.842	1.672	0.200	1.190
400	2.016	1.813	1.61	2.8	18.187	9.1	1.852	0.242	1.43	0.171	1.219
200	1.528	1.474	1.42	50	18.526	9.3	1.572	0.152	1.278	0.153	1.237
100	1.346	1.298	1.25	70	18.702	9.4	1.392	0.142	1.136	0.136	1.254
50	1.328	1.224	1.12	6	18.776	9.4	1.242	0.122	1.014	0.121	1.269

Pressure (KPa)	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)	Coefficient of permiabiliility (kv) cm/min	Coefficient of volume of compressibility (mv) 10-4 (m2/kN)
50	*0.097*	0.23		1.80	0.722	0.753406
100	*0.011*			1.56	0.069	0.651966
200	*0.005*			1.37	0.028	0.572962
400	*0.030*			1.05	0.136	0.439626
800	*0.01*			1.05	0.040	0.440004
400	*0.0582*		0.06	1.81	0.466	0.757406
200	*0.0034*			3.28	0.049	1.371159
100	*0.0025*			5.98	0.064	2.501779
50	*0.0289*			11.27	1.410	4.714355



Soil layer 2

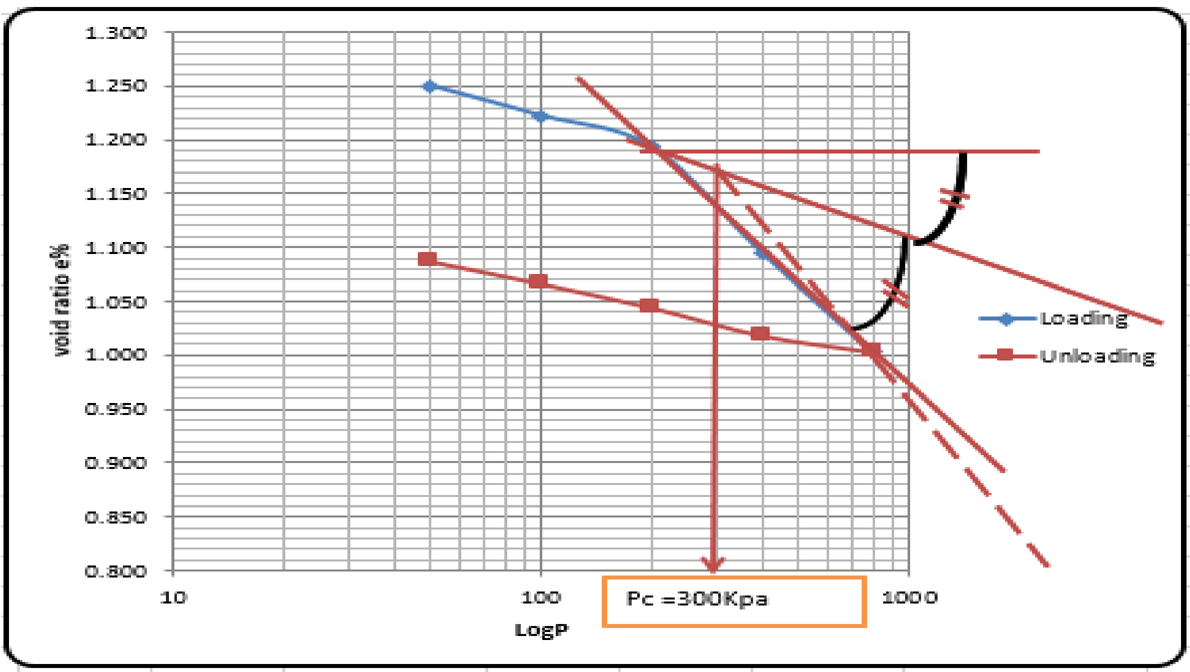
Time	Deformation @ 1kg	Deformation @ 2kg	Deformation @ 4kg	Deformation @ 8kg	Deformation @ 16kg
0.00	0.15	0.650	0.896	1.152	1.998
0.10	0.340	0.71	0.962	1.282	1.888
0.25	0.360	0.716	0.97	1.3	2.102
0.50	0.388	0.722	0.976	1.312	2.114
1.00	0.410	0.738	0.984	1.328	2.13
2.00	0.430	0.748	0.992	1.348	2.148
4.00	0.458	0.758	1.018	1.382	2.184
8.00	0.486	0.776	1.028	1.406	2.238
15.00	0.508	0.802	1.042	1.438	2.298
30.00	0.550	0.812	1.064	1.49	2.386
60.00	0.596	0.832	1.092	1.632	2.502

120.00	0.610	0.86	1.106	1.784	2.63
240.00	0.624	0.872	1.13	1.9	2.732
480.00	0.640	0.886	1.146	1.97	2.786
1440.00	0.650	0.896	1.152	1.998	2.818

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen,ΔH	Cumulative of change height of specimen
50	0.310	0.473	0.636	5.5	19.527	9.8	0.15	0.486	0.486
100	0.696	0.793	0.89	13	19.207	9.6	0.65	0.24	0.726
200	0.950	1.050	1.15	11	18.950	9.5	0.896	0.254	0.98
400	1.274	1.637	2	14	18.363	9.2	1.152	0.848	1.828
800	1.674	2.237	2.8	0.4	17.763	8.9	1.998	0.802	2.63
400	2.802	2.744	2.686	150	17.256	8.6	2.818	0.132	2.498
200	2.678	2.568	2.457	45	17.433	8.7	2.683	0.226	2.272
100	2.454	2.367	2.28	90	17.633	8.8	2.472	0.192	2.08
50	2.254	2.170	2.085	160	17.831	8.9	2.264	0.179	1.901

Pressure (KPa)	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)
50	0.056	1.250	*0.034*			17.29

100	0.084	1.222	*0.014*	0.32		2.46
200	0.113	1.193	*0.016*			1.32
400	0.211	1.095	*0.012*			2.23
800	0.303	1.003	*0.39*			1.10
400	0.288	1.018	*0.0010*		0.08	0.19
200	0.262	1.044	*0.0033*			0.65
100	0.240	1.066	*0.0017*			1.08
50	0.219	1.087	*0.0010*			2.00



Appendix VI Triaxial test results

Company Name

በኢ.ትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን

Ethiopian Construction Design & Supervision Works Corporation

Title: Geotechnical Laboratory Report	Document No: OF/ECDSWC/0996	Issue No. 1	Page No. 1 of 3

Lab no.:- 869/13-870/13 **Client Ref:-** FCEE/TiT/052/2013

Submitted by:- Dekebi Chakeri **Date Received:-** 14/10/2020

Project:- Evaluation of the interference effect and settlement of closely Spaced footings of shallow foundation for two adjacent buildings using FEM analysis

Station:- Jimma Town

Test Requested:- Triaxial-UU

Reported to:- Dekebi Chakeri **Reported on:-** 15/10/2020

No	Tests	Tests methods	Soil Test Results	
			Reddish Brown Lab No:869/13	Dark Gray Lab No:870/13
1	Triaxial-UU C(kPa) Φ (^{0})	ASTM D 2850	 25.64 8.58	 29.40 6.53

REMARK: The samples were collected and submitted to the laboratory by the client.

Central Laboratory E.C.D.S.W.Co Research Laboratory & Training Center

Reported by Bethlehem. B
Geotechnical Engineer

Checked by Biruk. A
Senior Geotechnical Engineer

Approved by Getu. D
Geotechnical Lab S/P Manager

Among the major services rendered by the Geotechnical and Material Laboratory Testing s/processes of Ethiopian Construction Design & Supervision Works Corporation are:

- ***In Geotechnical Laboratory**:-Testing the engineering properties of Soil Mechanics and Rock Mechanics.*
- ***In Material Testing Laboratory**:- Testing the engineering properties of various Construction materials, such as Aggregates, Asphalts/Bitumen/, Cements, Rocks, Water, Reinforcement steel bars, Hollow Blocks, Bricks, Ceramics, Tiles,Asphalt & Concrete Core Tests, Concrete Mix Designs, Asphalt Mix Designs, Sampling of the soil and construction materials, and so on.*

Please make sure that this document is the correct version before use

	Company Name በኢትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን Ethiopian Construction Design & Supervision Works Corporation			
Title	Triaxial Test (UU)	Document No OF/ECDSWC/0816	Issue No 1	Page No 2 of 3

Project :- Evaluation of the interference effect and settlement of closely Spaced footings of shallow foundation for two adjacent buildings using FEM analysis

Client :- Dekebi Chakeri

Location :- Jimma Town

Sample ID :- Reddish Brown

Depth (m) :- -

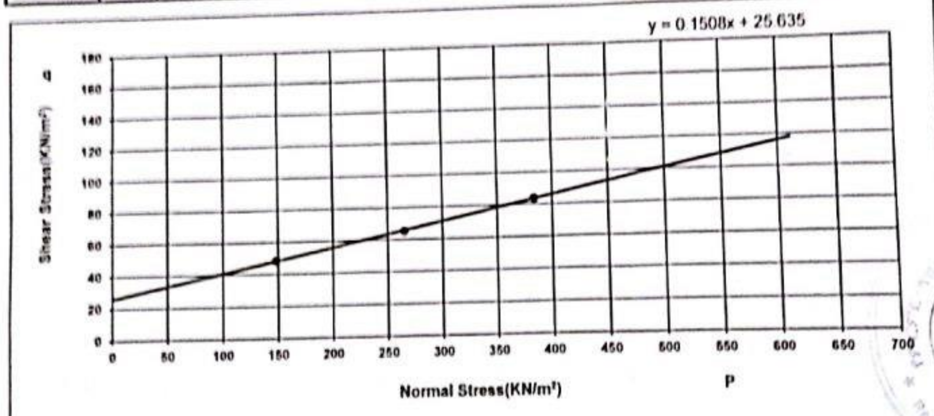
SPECIMEN DATA

Lab No.	869/13'
Initial height (cm)	7.40
Initial Area (cm²)	10.75
Initial Wet Weight (gm)	139.70
Final Dry Weight (gm)	98.60
Moisture content (%)	41.68

Test Type	TRIAXIAL TEST
Sample condition	Undisturbed
Initial diameter (cm)	3.7
Initial volume (cm³)	79.58
Bulk density (gm/cm³)	1.756
Dry density (gm/cm³)	1.239

p =	$1/2(\sigma_1+\sigma_3)$
q =	$1/2(\sigma1-\sigma3)$

					C (KN/m²)	Ø (Degrees)
Chamber press. (KN/m²)	(σ_3)	100	200	300	25.64	8.58
Deviator Stress, (KN/m²)	$(\sigma_1-\sigma_3)$	96	130	167		



Tested by : Mammo.H
Lab Expert

Processed by : Bethlehem.B
Geotechnical Engineer

Checked b Biruk. A
Senior Geotechnical Engineer

Approved Getu.D
Geotechnical Lab S/P Manager

Please make sure that this document is the correct version before use

Company Name			
	[illegible] Ethiopian Construction Design & Supervision Works Corporation		
Title: Triaxial Test (UU)	Document No OF/ECDSWC/0816	Issue No 1	Page No 1 of 1

Project :- **Evaluation of the interference effect and settlement of closely Spaced footings of shallow foundation for two adjacent buildings using FEM analysis**

Client :- **Dekebi Chakeri**

Location :- **Jimma Town**

Sample ID :- **Dark gray**

Depth (m) :- -

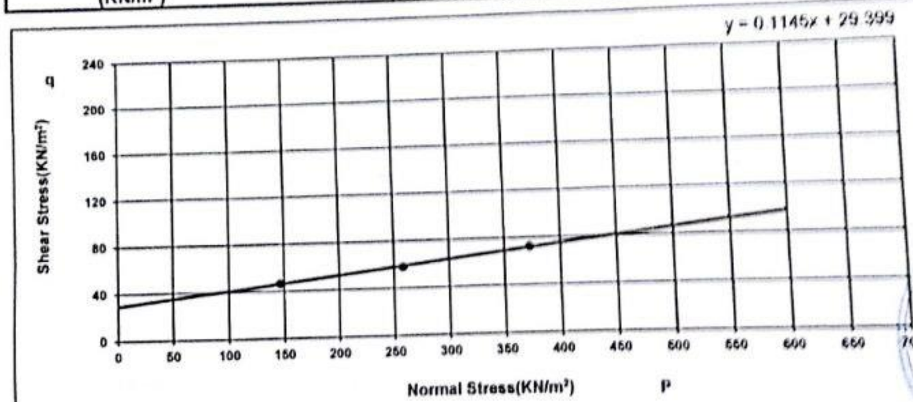
SPECIMEN DATA

Lab No.	870/13'
Initial height (cm)	7.40
Initial Area (cm^2)	10.75
Initial Wet Weight (gm)	129.70
Final Dry Weight (gm)	91.50
Moisture content (%)	41.75

Test Type	TRIAXIAL TEST
Sample condition	Undisturbed
Initial diameter (cm)	3.7
Initial volume (cm^3)	79.58
Bulk density (gm/cm^3)	1.630
Dry density (gm/cm^3)	1.150

P =	$1/2(\sigma_1+\sigma_3)$
q =	$1/2(\sigma_1-\sigma_3)$

					C (KN/m^2)	Ø (Degrees)
Chamber press. (KN/m^2)	(σ_3)	100	200	300	29.40	6.53
Deviator Stress, (KN/m^2)	($\sigma_1-\sigma_3$)	93	116	145		



Tested by : **Mammo.H**
Lab Expert

Processed by : **Bethlehem.B**
Geotechnical Engineer

Checked by **Biruk. A**
Senior Geotechnical Engineer

Approved b **Getu.D**
Geotechnical Lab S/P Manager

Please make sure that this document is the correct version before use

Appendix VII Some Activities in Laboratory