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LOAD SETTLEMENT BEHAVIORS OF SINGLE PILES WITH DIFFERENT DIAPHRAGM HEIGHTS

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I hereby declare that I wrote this Master's Thesis titled " Load Settlement Behaviors of Single Piles with Different Diaphragm Heights" in consistent with the thesis writing guide of the Graduate School of Natural and Applied Sciences, Firat University. I also declare that all information in it is correct, that I acted according to scientific ethics in producing and presenting the findings, cited all the references I used, express all institutions or organizations or persons who supported the thesis financially. I have never used the data and information I provide here in order to get a degree in any way.

16 August 2021

Manal Maher ISMAEL

PREFACE

I thank Allah almighty and praise him, for being the best and graceful above all. I thank him for achieving what I aspire to in completing my master's degree. I extend my sincere thanks and appreciation to my supervisor Assoc.Prof.Dr. Hüseyin Suha AKSOY, for his patronage of this study, as he provided me with the inquiries I needed that had the greatest impact on the completion of this study. I also express my great gratitude to my dear parents and all members of my beloved family for their patience and intense support for me to complete this study. I also thank everyone who helped, provide me with assistance and provided me with the necessary information to complete this study.

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ELAZIG, 2021

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In this study, the behavior of single steel piles under axial loading is analyzed, in terms of settlement, friction and load transfer mechanisms are concerned. Four different sand relative densities (Dr) of 10%, 40%, 65% and 90% were used for two single steel piles. These piles have 60 cm length, 5 cm diameter and two different diaphragm height (2.5 and 5 cm). 8 model experiments were carried out under axial load in the laboratory. Nonlinear computer modeling was then performed using the finite element-based program Plaxis 3D. The results of the both methods, the load-settlement ratio and the ratio of the load transferred to the pile base are given in the graphs and both Plaxis and experimental results are compared with each other.

The results showed that the internal friction angle of the sand and the bearing capacity of the steel piles increased with increasing relative density. In addition, when the height of the pile diaphragm increases, the bearing capacity of the pile increases. The overall results show that the finite element analysis results are in good agreement with the experimental results.

Keywords: model piles, sand, friction angle, load-settlement analysis, finite element method

ÖZET

Load Settlement Behaviors of Single Piles with Different Diaphragm Heights

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Bu çalışmada, tekil çelik kazıkların eksenel yükleme altındaki davranışı, oturma, sürtünme ve yük aktarım mekanizmaları açısından incelenmiştir. İki adet tekli çelik kazık için %10, %40, %65 ve %90 olmak üzere dört farklı rölatif sıkılıkta (Dr) hazırlanmış kum zemin kullanılmıştır. Kullanılan kazıklar 60 cm uzunlukta, 5 cm çapta olup iki farklı diyafram yüksekliğinde (2,5 ve 5 cm) imal edilmiştir. Laboratuvarda eksenel yük altında 8 adet model deney yapılmıştır. Daha sonra Plaxis 3D sonlu elemanlar programı kullanılarak nonlineer bilgisayar modellemesi gerçekleştirilmiştir. Elde edilen sonuçlar, yük-oturma ilişkisi ve kazık tabanına aktarılan yük oranı grafiklerle verilerek sonlu elemanlar analizi sonuçları ve deneysel sonuçlar birbirleriyle karşılaştırılmıştır.

Sonuçlar, artan rölatif sıkılık ile kumun içsel sürtünme açısının ve çelik kazıkların taşıma kapasitesinin arttığını göstermiştir. Ayrıca kazık diyaframının yüksekliği arttıkça kazık taşıma kapasitesi de artmaktadır. Genel olarak, sonlu elemanlar analizi sonuçlarının deneysel sonuçlarla iyi bir uyum içinde olduğu görülmektedir

.

Anahtar Kelimeler: model kazıklar, kum, sürtünme açısı, yük-oturma analizi, sonlu elemanlar yöntemi

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SYMBOLS AND ABBREVIATIONS

Symbols

SP : Poorly Graded Sand
D_{10} : Effective Grain Size
D_{30} : Particle size at 30 % finer
D_{50} : Median Grain Size
D_{60} : Particle size at 60% finer
N_q : Bearing Capacity Factor
P-Δ effect: Lateral Deformation of the Pile
γ_d : Dry density
$\gamma_{d\,min}$: Minimum Dry Unit Weight
$\gamma_{d\,max}$: Maximum Dry Unit Weight
e_{max} : Maximum Void Ratio
e_{min} : Minimum Void Ratio
D_{max} : Maximum Grain Size
D_{min} : Minimum Grain Size
Gs : Specific Gravity
Cc : Coefficient of Curvature
Cu : Coefficient of Uniformity
K : Pile Stiffness Factor
L : Length of the Pile
D : Demeter of the Pile
Dr : Relative Density (%)
e : Void Ratio
δ' : Effective Skin Friction Angle between Sand and Pile materials
ϕ' : Effective Internal Friction Angle of the Sand
Ra, Ry, Rz: Surface Roughness Parameters
B : Short Dimension of the Foundation
D_{tank} : Diameter or Short Dimension of Model Tank
D_{pile} : Diameter of Pile

Abbreviations

ASTM : American Society for Testing and Materials
SSI : Soil Structure Interaction
USCS : Unified Soil Classification System
LVDT : Linear Variable Differential Transformer
ABAQUS: Is a Software Suite for Finite Element Analysis.
NP : Neutral Plane
FRP : Fiber-Reinforced Polymer
SWCC : Soil-Water Characteristic Curve
FEA : Finite Element Analysis

1. INTRODUCTION

1.1. Pile History

Since humans first attempted to create safe housing and across rivers and streams as fluctuating water levels introduced some element of uncertainty in the travel arrangements, pushing hard pegs or piles into the ground has provided a way to reduce life-threatening conditions. Evidence of narrow settlements is found on the shores of the lake, for example, in Switzerland, Italy, Scotland and Ireland. Some of these settlements are thought to have been in use around 4000 years ago and were sometimes very important.

The earliest record of attempts to pretend to be Herodotus is sometimes referred to as "the father of history", a Greek writer and traveler who lived in the fourth century BC. He noticed that the Paeonions tribe of Thracian lived in houses built on piles suspended at the bottom of the lake. The piles are driven under some form of communal organization. This system has provided a unique and efficient way to create a growing expanding settlement. In Britain, for example, a Roman Tyne Bridge was spanned off in Corbridge using piles to sustain construction. It was found when the old bridge was taken and found that it was from oak black and they were about 3 meters long.

Wood piles prone to wetting and drying cycles are well known to be much less durable than permanently submerged piles. In the nineteenth century, great changes began to occur in terms of motive power and materials. Metal piles, mostly in the form of cast iron tubes, were used in important buildings due to their durability. Machines are now available to carry a wide range of pile diameters from 50 mm to over 2 m, while other machines are designed to enable concrete or plaster to be driven into a pile through a hole in the ground that can be inserted into the ground. At the same time, driven piles developed and there are two main forms in the ground action, namely the pre cast pile and the cast-in place pile. Single piles and group piles design has progressed steadily in recent times with most of this work being performed by skilled engineers especially foundation engineers. Operational pressure systems are developed for their own design, while computer-based technologies for analyzing design processes for large and complex pile groups gain additional performance.

In modern research on site investigation has become a prerequisite for making reasonable decisions about both the design of the piles and the selection of appropriate construction methods, most importantly, they are available in the wide range of methods and equipment that are currently available .It is clear that as the ever-increasing pile stresses, resulting in strong incentives for the economical use of materials, piles must be sound and durable. The integrity of different types of piles has been studied and presented in a wide range of ground conditions and has been presented in various reports of the Construction Industry Research and Information Association [1, 2].

While the assessment of methods and developments in the engineering knowledge of the piling industry has changed in important ways throughout its history, and while the pace of change has been very rapid, especially in recent years. The equipment or method is used, a great deal of confidence should be given to the people whose skills, experience and sincere efforts make the engineering concepts a credible reality. It has always been this way [3].

1.2. General

A superstructure in a particular location may not be sufficient to support buildings, dams and bridges, and it may not be secure due to load capacity or to support major loads. Pile foundations are one of the most common types of foundations [4].

The piles are mainly deep foundation. Piles are required when the bearing capacity of the shallow foundations is insufficient to support the upper structure. The upper structure creates vertical forces due to the additional weight and loads, which are transmitted axially to the pile and through its shaft and base to the ground, and may reach a more solid layer. Therefore, the analysis of the charge-transport mechanism on individual substrates under axial load is an essential basis for capturing deep foundations. It is extremely important to carefully study the natural interaction of the pile with the soil. Settlement analysis is also fundamental, since the maximum permissible settlement of a foundation is remains the most important factor in its design. Therefore it must be estimated carefully [5].

To predict the exact behavior of the piles, you must take into account the interaction of the superstructure. The interaction between substrate, superstructure and soil is called (Soil Structure Interaction) SSI. Numerous analyzes have been reported in the literature, considering the interactive effects of frames on shallow foundations [6-8].

Below are situations where the use of a pile foundation system can be [9]:

- When the level of groundwater is high.
- Heavy and uneven loads are imposed from the superstructure.
- Other types of foundations are expensive or impossible.
- When the soil is compacted at a shallow depth.
- When the soil condition is not good and it is impossible to dig to the required depth.
- If it becomes impossible keeping foundation dry by pumping or other procedures due to the heavy flow of seepage.
- When it is possible to scoured, due to the location close to river bed or the shoreline, etc.
- Where a deep channel or drainage system exists near the structure.

1.3. Type of Pile Foundation

Pile foundations can be classified based on function, materials and installation process.

1.3.1. Classification Based on Function or Use

According to the function, the piles are classified into sheet piles, load bearing piles, end bearing piles, friction piles and soil compaction piles [10].

1. **Sheet Piles:**

This type of pile is mainly used to provide lateral support. It usually withstands side stresses like loose soil, water flow, etc. It is commonly used for coffer embankment, trench panels, beach protection, etc. It is not used to provide vertical support to the structure. It is commonly used for the following purposes:

- Retaining walls construction.
- River bank erosion Protection.
- Save some loose soil around basic trenches.
- Separation of the foundation from the neighboring land.
- Determine the location of the land to increase its carrying capacity.

2. **Load Bearing Piles:**

The main purpose of this type of foundation is to transfer vertical loads from the structure to the soil. These foundations transmit loads across the soil, with poor support properties of the bearing layer. Loading piles may be classified as drainage piles according to the mechanism through which the loads are transferred from the substrate to the ground [10].

3. **End Bearing Piles:**

These type of piles, the load passes through the lower end of the pile. The lower end of the pile is placed in a rock or strong layer of soil. Piles are usually in a transitional layer of strong and weak slayer. As a result, the load is transported securely to a stronger layer by means of the piles as a transport shaft [11].

4. **Friction Pile:**

Friction piles transfer the load by frictional force between the pile surface and the surrounding piles soil from the structure to the soil such as sandy soil, solid clay, etc. Friction can develop along the pile or the specified pile length, depending on the soil layer. Generally in a friction pile, the surface of the piles acts to transfer the load to the soil from the structure.

5. **Soil Compactor Piles:**

Sometimes the piles pushed at closed intervals by compaction to increase the bearing capacity of the soil [11].

1.3.2. Classification of Piles Based on Materials and Construction Method

Basis on the materials of pile construction and process of installation load-bearing piles can be classified as shown in Figure 1.1 [12]:

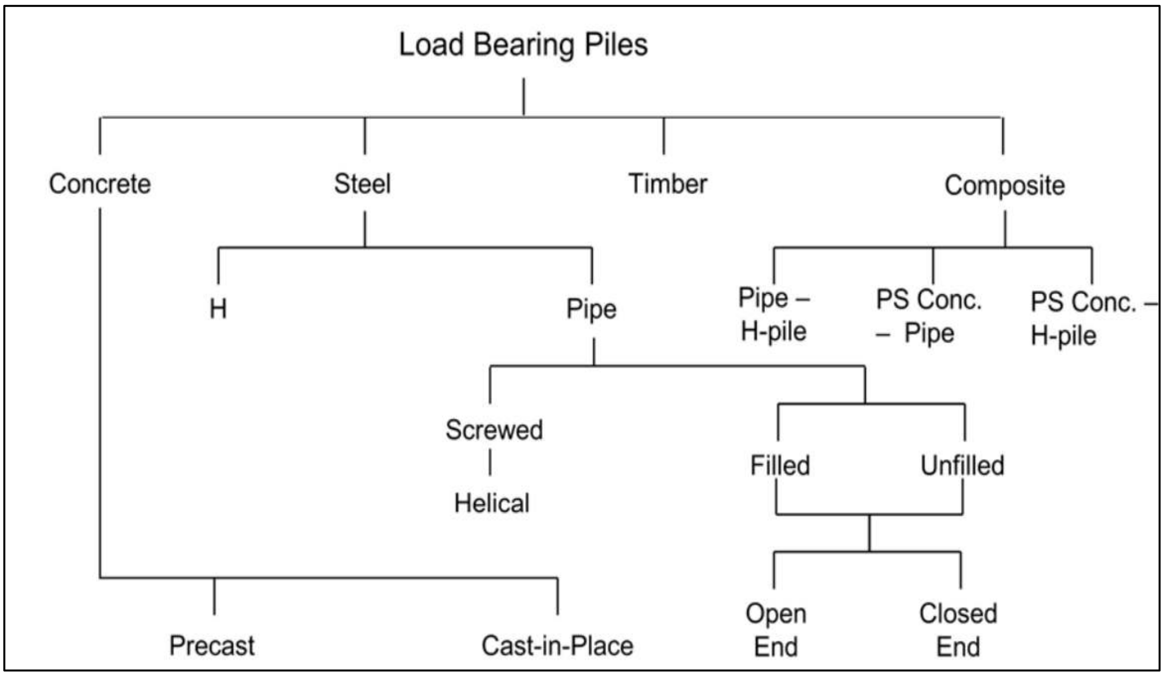


Figure 1.1. Classification of pile foundation

1. **Timber Piles**

Timber piles are the oldest of all piles that have been used in the modern era. Usually these piles are pre-cast displacement piles installed in driving method, or a less common method such as vibration. Timber piles usually have a taper cross section and a round made from the trunks of Southern pine or Douglas fir is pushed with a small end used as a toe pile. These piles should not be used to push through rocks, dense gravel, or as a piles to hold toes on rocks because they damage the head of the pile and the toe during difficult driving. Excessive exposure to timber piles can cause the fibers to break or crush at the top of the pile. If the timber piles are below the permanent water level then sustainability is generally not considered.

However, if the timber piles is subjected to repeated cycles of drying and moistening, or if it locate above the water table, this causes the piles to be damaged and decayed by insects, and also bacteria or fungi may attack the pile and this causes the pile to rot or damage the heap. The operational life of timber piles reduce if they do not treated with wood preservatives. Relatively high loading capacity is obtained in the natural convergence of pile shafts. If applied well, it is an economical, safe and effective solution.

2. Concrete piles

Concrete piles are a common foundation structural element used to support coastal structures such as oil wells, bridges, and floating airports. The use of offshore buildings is still relatively new from a technical point of view and much research is needed in this area. The offshore construction load consists of two components: side wave loads and vertical structural loads. The relationship between the two components of the load has a great influence on the way the reaction interacts and the pressure distribution in the piles. Also, the piles will exhibit a different reaction when it subject a small load of structural from the other large load structural.

3. Steel Piles

Steel piles are installed to resistance or design depth by vibration hammers. Driven piles increase geotechnical capacity effectively during compacting the soils at the toe and the soil displacement around the shaft during installation. Steel piles are open end or closed end. The power moving the piles is provided by a percussion hammer or high-frequency vibrating hammer. Figure 1.2 shows typical illustration of the steel piles, and Table 1.1 shows technical summary of the steel piles.

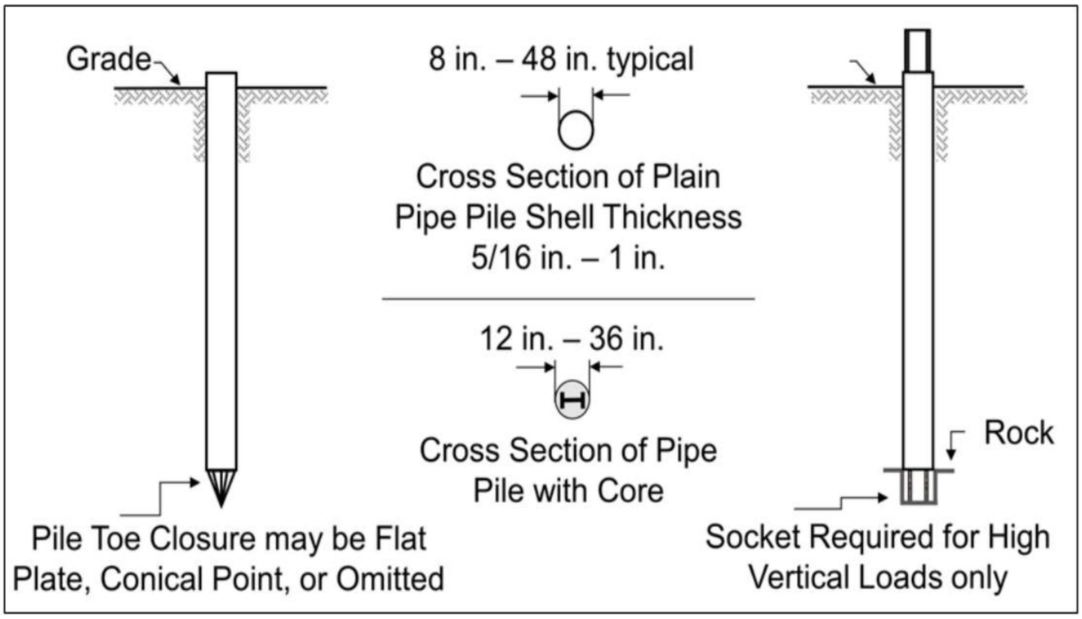


Figure 1.2. Steel piles typical illustration

Table 1.1. Steel Piles Technical Summary

Pile type	Steel-pipe pile
Typical lengths	15 to 200 feet.
Material specifications	ASTM A252 Grade 2 or 3, API 5L, or API 2B - for pipe. ACI 318 - for concrete (if filled). ASTM A572 - for core (if used).
Typical factored resistance	100 to 1,250 kips (closed end, $D \leq 30$ in.) with concrete fill. 660 to 6,500 kips (open end, 16 in. $\leq D \leq 72$ in.) no concrete.
Maximum driving stress	$\sigma_{dr} = 0.9\ \phi da\ Fy$. $\phi_{da} = 1.00$ for non-composite during driving. F_y = Yield strength of steel.
Advantages	•Closed end pipe can be internally inspected after driving. • Low soil displacement for open end installation. • High factored resistances depending on section. • Open end pipe with shoe can be used for obstructions. • Open end pipe can be cleaned out and driven further. • Easy to splice.
Disadvantage	•Vulnerable to corrosion where exposed and in corrosive soil conditions. • Potential soil displacement from larger closed end pipe.
Remarks	•Provides high bending resistance where unsupported length is loaded laterally. • Open end not recommended as a friction pile in granular material due to tendency for length and cost overruns.

- Closed End Steel Pipe

Closed end steel piles are made of either seamless welded or spiral steel tubes. Pipe piles of larger volume are available, but they are often exist with an open end. Piles shall be classified according to ASTM A252. Closed end steel piles can be used as toe bearing piles, friction piles, a combination of both of them, or rock piles. It is relatively easy to connect, so it is most commonly used when variable piles length is required. Piles can either remain open or can be filled with concrete. Steel reinforcement is only required if the concrete in the pile can be loaded due to conditions such as height, large transverse loads, or unsupported pile length. In most cases, the piles are driven from the head of the pile.

- Open End Steel Pipe

Open end pipe are often installed when severe driving conditions, large penetration depths, or debris are expected. Similar to closed pile, open piles can be seamless, laminated and welded or spiral steel piles. Open piles are specified by classification related to ASTM A252.

Large-diameter open piles are usually driven on heavy sand or light rock or can be used as friction piles. An open ended pile usually passes a large diameter through the soil while driving due to the great effect of weak soil. However, under constant loading conditions these piles usually overlap by providing significant resistance at the toe. Before placing concrete, reinforcement steel and dowel resisting can be added as needed. These piles can also be attached to the inner or outer steel cutting shoes to reduce the possibility of damaging toe.

4. Composite pile

This type of pile is consist of two different materials or materials, where one is pushed on top of the other, so that two piles can work together to perform the function of one pile. In this combination, piles takes advantage of the good qualities of the two materials. This is considered economical because it is allowed to use the anti-corrosion properties of one material strongly or cheapness of the other[12-14].

1.3.3. Types of Pile Based on Installation

The piles are classified according to their installation method:

- Driven Piles: steel, wooden or precast concrete piles are driven to a specific position using special equipment to press the piles.
- On-site cast piles: only concrete piles are cast on site. Piles are excavated and poured with concrete. Reinforcements are added as needed.
- Piles driven and cast on site: These are piles that combine driven pile with cast on site pile. Casing or casing can be used [14, 15].

1.4. Installation of Pile Foundations

At the first, piles are putting vertically on the ground level then they were pushed or driven on to the soil by a machine to drive piles .A pile driver is a machine that picks up vertical piles and pushes them to the ground. The blow is repeated by raising a large weight and puling it to the top of the pile. Piles must be driven into the soil until they reach the point of refusal, which is a point that piles can no longer be driven into the ground**.** The installing method is the main consideration for the structural integrity of pile foundation. The driven piling method is perfect choice as it provides maximum loading capacity per pile with minimal disturbance of the bearing soil around the pile. Since all the piles have a surface touching to the surrounding soil, the piles must be sufficiently separated from each other to distribute the load evenly [16].

Appropriate type of pile can be selected depending on categories of the following main factors:

- The type and location of the structure
- The conditions of the soil
- Durability

1.5. Pile Analysis

One of the major aspects of designing piling foundation is the load settlement procedure for single piles. In addition, the piles behavior is influenced by several factors such as behavior of non-linear mechanical of the soil, the properties of the piles itself, and installation method [17].

Generally, the settlement design of pile axial load is very small. The accuracy of the estimating settlement methods in predicting the small vertical settlement value of a single pile is variable due to errors arising from the design record measurement estimates, such as soil characteristics, and those derived from the geotechnical calculation methods themselves, and there is no agreement as to what is the most appropriate method. [17-20].

In most existing building structures, the calculation of standard material and technical properties for building materials and work is a fairly reliable confirmation of building elements, since the rationale for the calculation and standardized material properties is a numerical description of correct behavior. In geotechnical engineering design, numerical accuracy of usual models is low (and so the reliability confirmation is by calculating) due to the complex behavior of soil and rocks, the range of heterogeneity and parameters , and the relationship between man-made and natural materials, Estimation of piles and settlement capacity are a mixture of theoretical and experimental to date [21-23].

With regard to the analysis of the piles settlement, the load distribution of the single piles under the axial load and the field loading behavior of the piles were analyzed in several methods. According to (Poulos and Davis, 1980) it can be divided in three categories [5]:

1. Method of load-transfer, this implies a comparison between the resistance of the pile and the movement of the pile at various points in its length.
2. A flexible method based on theory using the equation described in (Mindlin, 1936), for surface loads in near-infinite mass (such as the Poulos and Davis method), or another analytical formula that gives the correspondence between the displacement of the pile and the surround soil for each element (such as the method of Randolph and Wroth).
3. Numerical methods such as finite element method.

Poulos and Davis pointed that out immediately (elastic) achieves most final settlements, and as indicated by Murthy, there are also enhanced transformations of sandy and even clay soils. In the case of piles in sand, rapid settlement includes almost all final settlement. The direct position of the pile can also be calculated using the semi-experimental method that suggested by Vesic [24].

The methods of measuring settlement include load transfer method, flexible analysis method, shear shift method, standardized relationships and finite element method, as well as other simplified analytical methods [25, 26]. It has been proposed in recent decades to calculate the axial settlement of a single pile under axial load. The method of load transfer, which describes the behavior of deformation on a pile-soil, was first developed by Seed and Reese (1957) [27] and later expanded by other researchers, such as Coyle and Reese (1966) , Chernauskas (1992) and Paikowsky et al. (1994) [28-30].

However, these transferring load functions ignore the reactive effects of soil motion around the pile and are not used to adjust the settlement analysis of the group of piles. The load transfer method is probably the technique most used to investigate problems of single axial piles, particularly it is useful if the soil behavior is not clearly linear or the soil surrounding the pile is layered. In this method, the pile formation model is a pile product with a non-linear spring on top, supported by a series of elements supported by separate spring-loaded that represent the soil's resistance to the impermeability of the surface.

It is very important to specify requirements for pile analysis that can be calculated from faults in the pile foundation practical. This analysis should be able to deal with the following aspects:

- Soil profile with non-horizontal layers.
- Different soil profiles along different piles within a group.
- Piles of different lengths or diameters in the same group take into account the interchange between different piles.
- Defection a long structural defects or changes in diameter and size on the piles.
- Added or active piles, during the load process, sections are created to simulate the installation of the update piles.
- The load is imposed by the general construction load in addition to the earth movement.

In addition to these possibilities, the analysis should be able to take into account the aforementioned facts about pile behavior, in particular the nonlinear interaction of piles, topography, and movement of the piles material [31].

1.6. Objectives and Methodology

In this study, the theoretical relationship between single piles settlement and load is developed to measure the detailed settlement amount of piles and soil, analysis designed based on the study of load transport on the piles. Processed soil has different internal friction angles, and the skin friction angle is determined between the soil and the steel. Additionally, this document provides for a complete settlement analysis. This thesis has three main objectives:

1. Description the theory of single piles analysis under the axial load.
2. The effects used in relation to the final program results is Plaxis 3D version 20.

3. Flexible formation of single piles under axial load, confirm their results by comparing experimental figures and Plaxis 3D curves and finally case study analysis.

1.7. Layout of the study

This thesis consists of five chapters, the first being the introduction.

Chapter 2 presents the results modelling of a single pile under vertical loading that the ones obtained by other authors under similar conditions.

Chapter 3 presents the tools and devices and how they work (their purpose) in detail that have been used in the practical experiments, as well as explain in detail the tests that were carried out for the single pile under vertical axial load in the sandy soil.

Chapter 4 presents the results of the experimental modelling of single piles under axial loading and the results obtained from the Plaxis 3D. These are compared these two results with each other.

Chapter 5 list the concluding remarks derived from this thesis.

2. LITERATURE REVIEW

2.1. Introduction

As Professor R. B. Peck said, "driving piles for a foundation is a brutal and dangerous process." The relationships are complex between the piles and the surrounding soil. The insertion of the piles changes the soil character, and the intense strains are placed locally close to the piles. With the influence of pile groups and pile shapes, the non-homogeneity of soils makes understanding pile-soil interaction difficult. For driven piles into soft rock or to hard rock, the structure and strength of the mass of rocks include the degree of weathering, the joints, and the bedding planes, make it difficult to understand the interaction of the rock piles. The widespread generalization of pile behavior is impractical. Success in designing foundations for piles requires an understanding of the importance of many of the factors involved. The natural complexity of moving the piles makes it necessary to use a practical method of semi-experimental design and pay attention to the important elements with subtle details.

Reaching to the optimum solution for a driven pile foundation, the foundation engineer must consider the terrain/rock behavior and parameters, applied boundary conditions, load combinations and calculated loads, performance of the requested project, and costs of the construction. The foundation, current practices of design and construction of the foundations upon which the foundation is located [12].

2.2. Body of review

Mattes and Poulos (1969), a single compressible floating pile's settlement behavior has been analyzed and the factors affecting the settlement for a wide range of values of L/d length-to-diameter ratio and the stiffness factor K of the pile is analyzed, which are relative measurements of the pile's compressibility. The K value has a significant effect on the shear stress distribution and settlement along the pile. The immediate ratio to final settlement is slightly affected only by K, while the settlement decline due to pile-based expansion is almost independent of K. The pile acting as incompressible pile at K value as L/d increases pile incompressibility increase. The comparison between the reported pile behavior from field measurements in clay and theoretical pile behavior shows a good agreement [32].

Poulos (1979), provide an analysis developed previously to pile settlement in homogeneous soil and extend to the case of nonhomogeneous soils. A comparison with finite element solutions shows that the analysis provides adequate accuracy solution for practical purposes. A series parametric solutions were performed which the modulus increase linearly with depth for a pile in a Gibson soil. It has been found that the analysis in layered soil to the piles is applicable, except where piles are found in more compressible soils than overlying layers. Then briefly examination of determining the soil modulus values, finally some comparisons have been presented between the measured pile behavior and the theoretical pile behavior, referring to the possible consequences practical under the assumption that the soil profile is nonhomogeneous more than homogeneous [33].

Kuwabara (1989), performed an elastic theory based on boundary element analysis on the behavior of the piled raft foundations under vertical loading. Characteristics of load transfer and settlement for piled raft foundations which the raft is placed on a homogeneous isotropic elastic semi-space soil are opposed with free-standing single piles and group piles. Although the raft provides 20-40% of the load applied directly to the soil, the reduction in the number of settlement due to the presence of the raft is very small. A large difference in the load of the piles is recognized at the top of the shafts between the two types of group piles with raft and without raft, only a small difference has been found in the rest part of the shafts. The soil pressure in contact with the raft is relatively uniform in the inner area surrounded by the piles [34].

Aristonous M. et al. (1991), Developed a simplified procedure for the effects of soil plasticity incorporating, separation and pile slippage into the soil analysis of single and two piles. The model proposed is based on a nonlinear representation of Winkler-type soil resistance for a single pile. Allow separation and sliding on both sides of the pile. A close agreement of the results between the two analyses was observed. It has been observed that the piles subjected to cyclic axial loads corresponding to 60% of their ultimate capacity can be exposed several load cycles without apparent damage [35].

Chung lee and kuo chiang (2007), made a series of tests of centrifugal models were performed to assess soil deformation and its effective detrimental on adjacent single piles in saturated sandy soil. Although deep ratio has been found to have a significant effect on the bending moments distribution a long the piles, the both of working load and depth ratio on the pile determining of the axial load profile. Shallow tunneling near long piles causes both negative and positive bending moments, while deep tunneling causes large negative bending moments. more loads have been transferred to the end of the pile wich resulted repairing. pile settlement rapidly increased with the result due to the large amount of ground loss of a at the end bearing resistance [36].

Jaeyeon et al. (2012), by numerical analysis the settlement behavior of square raft pile in clay soil have been investigated. Analysis of nonlinear three-dimensional finite elements have been performed using a pile-soil slip interface model for different pile numbers, pile positions, various types of loads and pile lengths under rafts. Aspects settlement for design efficient of raft piled under vertical loads have been discussed for hard clay and soft clay soils. It can be notice that the change in the rate of reduction of soft clay is relatively larger compared to the hard clay, while the rate of reduction of soft clay is relatively smaller than the hard clay [37].

Qian and Zhong Zhang (2012), present a nonlinear analysis for a simple analytical approach of the load displacement presented response of a single pile. Two models have been adopted for the proposed approach. Nonlinear softening relationship have been used in one model to simulate the deterioration behavior between the pile–soil and skin friction unit displacement developed at the pile-soil interface, and bilinear load–displacement relationship adopted by the other model to capture the response of end of the pile. Depending on the two models that have been proposed, a very efficient computer program for nonlinear load analysis and settlement behavior for a single pile was developed. A comparison has been made between the analytical methods, field well-documented pile loading experimental cases, and the present method. Comparative results show that the proposed method generally agrees well with the behavior observed in the field and with the calculated results from other approaches [26].

Chen et al. (2013), were performed investigation to the reaction of the piles under axial loading. Many heavy instrument piles were used for the experiment. The piles have been tested at different loading levels and up to 50,000 different cycles were used in each test. The stability of the settlement was determined to be largely related to the characteristics of the cyclic loads that have been applied. It has been found that if the cyclic loading amplitude is lower than a certain level, the pile will not produce increasing of settlement. Due to the very large number of loading cycles, a simple method is proposed to predict a single pile settlement. The idea of a cyclic deformation scheme was also developed to analyze the effect of cyclic load characteristics developed on the deformation behavior [38].

Weiping et al. (2014), in there paper a new hyperbolic model developed to describe the behavior of load transfer of the pile-soil interface that reflects the increasing properties of the initial shear stiffness consolidation. Includes loading, unloading, step-by-step and reverse load of the pile and soil interface. Depending on the model, pile-soil interface under head load of the pile and surcharge, shearing history have been studied the. In addition, the effect of pile’s head load magnitude and the driving time of pile on (neutral plane) NP and investigating drag load. Negative skin friction and NP elevation have been found depending on consolidation. An interesting result that has been experimented with in pile-soil during consolidation and the interface of the pile is the cyclic shear. The size of the pile head magnitude significantly affects the negative skin friction of the pile, for this reason, the greater the load on the pile head, the lower the NP elevation and the lower the resistance on the shaft of the pile. Installing piles after reaching a certain degree of consolidation in the soil can reduce the pile settlement and drag load of the pile. The capacity of the pile is decreased by consolidating the surrounding soil [39].

Reddy et al. (2015), were carried out Laboratory experiments to examine the single piles behavior in the sand under lateral load and uplift load combination. The model pile's dimensions were confirmed using the law of physical scale, based on the material properties adopted in the model and the prototype of the pile foundations. The results show that the load stress deflection behavior is nonlinear for lateral load and uplift load tests, and also the combined loading case. It is observed that the piles behavior under independent loads different considerably from comparing the combined loads. It has been found that the ultimate lateral /uplift load capacity increase considerably under the combined loading [40].

Abdrabbo and Naema (2015), analyzed and quantified drag and drop, which is located in a single pile. The effect of the load on the head of the pile was investigated. Analyze unshielded and shielded piles. Three dimensional non-linear analyses were performed with ABAQUS 6.12. The study examines the size of the extra loaded area, the location of the tip of the pile and the load on the pile head in the drag load and down drag imposed on the pile. Based on the numerical results, this study found that sacrificing piles that are unloaded piles "hang" the soil between the piles within the group, thus reducing the vertical effective pressure around the shield piles. Numerical results showed that the mean value of the shear stress mobilization factor moved along the central shielded pile without charge ranged between 0 and 0.27 in the neutral plane (NP). The long-term drag charge of a shielded floating pile is about 67% of the long-term drag charge of unshielded floating single pile [41].

Aksoy et al. (2016), performed Surface roughness tests on materials (FRP, steel and wood) that have been widely used in the driven pile construction. It has been observed that steel is the smoothest material compared to wood, which is the roughest material. Shear tests Interface were performed at the interface between the soil and these materials. The angle of the skin friction of between different soils that have been measured and material. Analyzing the internal friction angle (ϕ) of mixture soils with sandy clay, it was found that ϕ decreases with increasing clay ratio. However, with the clay content is about of 30%, rate of ϕ increased slightly. A slight increase in the internal friction angle of the sand-clay mixture was observed with an increase in the clay content [42].

Al-Khazaali et al. (2016), were presented The experimental results were paved with models pile applied to both sand and a glacial till spheres until different soil absorption values were applied with a flexible Mohr-Coulomb model. A numerical model study was performed with the commercial Plaxis 2D software to simulate the charge establishment behavior of the prototype pile. Mechanical properties of the soil, and the interface at different levels in the suction alone is estimated that this face is composed by using a simple mechanical means required in order to measure the properties of water in abundance and owner curve (SWCC). It is a compromise between good behavior and measure the load settlement and model piles [43].

Yenginar et al (2016), summarized the load factor (N_q) was determined on saturated and dry sand of 9 piles with different lengths and diameters, and consisted of cast-in-situ reinforced piles in sandy soil. The piles were then loaded with static piles loading tests in both saturated and dry sand. The actual piles capacity was measured directly using the load cell placed under the pile. Finally, the test results were compared with the following theoretical values. Such as that obtained through. Janbu (1976), Vesic (1977) and Meyerhof (1976) methods [44-46]. The main results can be summarized as follows:

1. When the soil changed from dry to saturated state, the loss of the pile base obtained by the test results was greater than the theoretical loss.
2. Experimental results on dry and saturated sand showed that the basic capacity of the piles was less than the capacity of the Janbu, Vesic and Meyerhof methods.
3. In comparing response to dry sand clumps, the saturated sands reach the limit capacity point at low settlement.
4. In theory, pile resistance is only related to effective soil stress and internal friction angle of the soil. However, in addition to these parameters, the experimental results show that the placement of the pile also affects the strength of the resistance point of the pile.
5. The value of N_q is related to the friction angle of the soil, and is also depend on the diameter of the pile, friction angle, density of the soil, effective stress and the degree of soil moisture.
6. As the L / D ratio (pile length / pile diameter) increases, the value of N_q decreases [47].

Elsamny et al. (2017), carried out an experimental study program was conducted to investigate the distribution of friction over the pile and the transferred load of piles in cohesionless soils was reduced as well as effect of two piles. However, the test program consisted of an experiment single and two groups of piles under an axial compressive load. The distance between the piles is 3 pile diameter. The program consisted of placing a test piles on dense sand that was placed in a soil chamber and exposed to an axial load of compacted soil. This research reached the following conclusions:

1. For single pile, 86% of the applied load resists lateral friction and 14% resistance of the tip.
2. For groups of two piles, 96% of the applied load resists by lateral friction and 4% resistance at the top.
3. The efficiency have been found for groups of two piles with a spacing of 3 times the diameter of the pile [48].

Han et al. (2017), present a study to axial load applied of non-displacement piles at the head of the pile to perform a realistic analysis for finite element. Analytics succeed in understanding configurations and developing shear bands below the base of the pile and along the pile shaft during the pile loading. Analysis revealed that the load response to the pile settlement of determining the pile load to the axial load is closely related and that the local behavior of the sand near the pile governed. The lateral soil pressure coefficient K was found to increase with increasing relative density of the sand but decrease when the initial confining stress increase [49].

Limjibhai and Mehta (2018), in the laboratory, a series of experiments were performed on model piled rafts, groups of piles and non-piled raft in cohesionless soil. The purpose of the experimental program is to investigate the foundation system behavior under vertical load. Model tests show that with increasing the pile length increases the size of the compressive load, thus increasing the ultimate load capacity and settlement, and therefore the slenderness ratio of the pile significantly affects the vertical load capacity of the pile's capacity increases the ultimate load capacity [50].

Wenjun Lu and Ga Zhang (2018), In their study present a series of experiments of centrifugal models were performed under different vertical-horizontal loading conditions to investigate the rules and mechanisms of the effect of loading on the reaction of a piles in sand. Vertical loading causes a decrease in the stiffness in the pile as a lateral deformation effect of the "P- Δ effect". The effect of soil compaction plays a dominant role in the initial horizontal load, and the P- Δ effect becomes more difficult with increasing horizontal load. The combined effect of these two effects seems to be the opposite is that the waiting time of the residues decreases first and then increases with increasing horizontal load. Additional stabilization of the pile is accomplished by horizontal loading and is positively related to pre-vertical loading [51].

Kenawi (2018), studied the response of piles under axial and transverse loads, for this analysis version 8.2 of the finite element Plaxis 2D software was used. For simulating the piles as a beam element with linear elastic properties. The soil behavior was simulated using a hardened soil model under drainage conditions. In this paper, we observe the bending moment and the lateral displacement along the length of the pile. The influence of the lateral force position, lateral force and axle load values were read from the pile response. As a result, it was found that the length of the pile and the movement of the corners according to the bending moment distribution were greatly influenced by the position and value of the load. However, the effect of the normal force value on the piles response is not significant [52].

Al-Kinani and Mahmood (2020), described a boring prototype model of a single and group of piles under vertical axial load. The drilled pile model used in the test was built on soft clay soil 2000 mm long and 150 mm in diameter. In addition, other theoretical methods such as Poulos 'and Vesić's methods were used to assess the establishment of a boring piles settlement, and then compared with piles load test data based on the rapid pile load test presented in (ASTM-D1143, 2007). In general, the theoretical method for predicting the settlement of a bored pile by Paul Wessik has a higher settlement value for a group of single and groups of piles compared to the settlement results obtained from the field test data. Therefore, it is not recommended to use soft clay soils. In contrast, the results of 90% of Hansen, Butler and Hoy can be considered as a reliable interpretation method for calculating the settlement of groups and single boring piles [53].

3. MATERIAL AND METHOD

3.1. Introduction

In this chapter properties of materials and equipment and how to use them will be explained in details, also it will explain the methods and test procedure that have been used for this study. A laboratory investigation was performed on model systems. Laboratory tests were conducted on single pile models to investigate the settlement behavior of the axial load pile raft system, and also to determining skin friction angle (δ) between sand and steel piles and internal friction angle of the sand (ϕ) direct shear test has been provided.

3.2. Materials

Some materials were used in this study such as steel piles and sand. In this section, these materials and their properties will be explained and described in detail.

3.2.1. Sand Properties

The sand used in the model pile load test was uniform gradation river sand brought from Murat River in Elazığ at Turkey. This sand has been washed and dried by the oven at 105 C^{O} and sieved by passing the sieve with an aperture of 1 mm and retained on the sieve #200 (0.075 mm). Figure 3.1 shows the sample of sand.

Figure 3.1. sand

According to (ASTM D4254-16) relative density determined, and according to (ASTM D422-63 and ASTM D854-14) specific gravity test and sieve analysis have been done. According to (USCS) Unified Soil Classification System (ASTM D2487) the sand was classified as poorly graded sand (SP). Table 3.1 explain index properties of the sand. Figure 3.2 shows grain size distribution of sand.

Table 3.1. Index properties of the sand

Index Property	Value
(D_{10}, D_{30}, D_{50}, D_{60})	(0.19 ,0.33,0.48,0.55) mm
Classification (USCS)	SP
Maximum dry unit weight ($\gamma_{d\,max}$ kN/m^3)	17.66
Minimum dry unit weight ($\gamma_{d\,min}$ kN/m^3)	14.32
(e_{max})	0.898
(e_{min})	0.539
Maximum – Minimum grain size, D_{max} - D_{min}	1 – 0.074
Specific gravity (G_s)	2.77
(C_u)	2.895
(C_c)	0.98

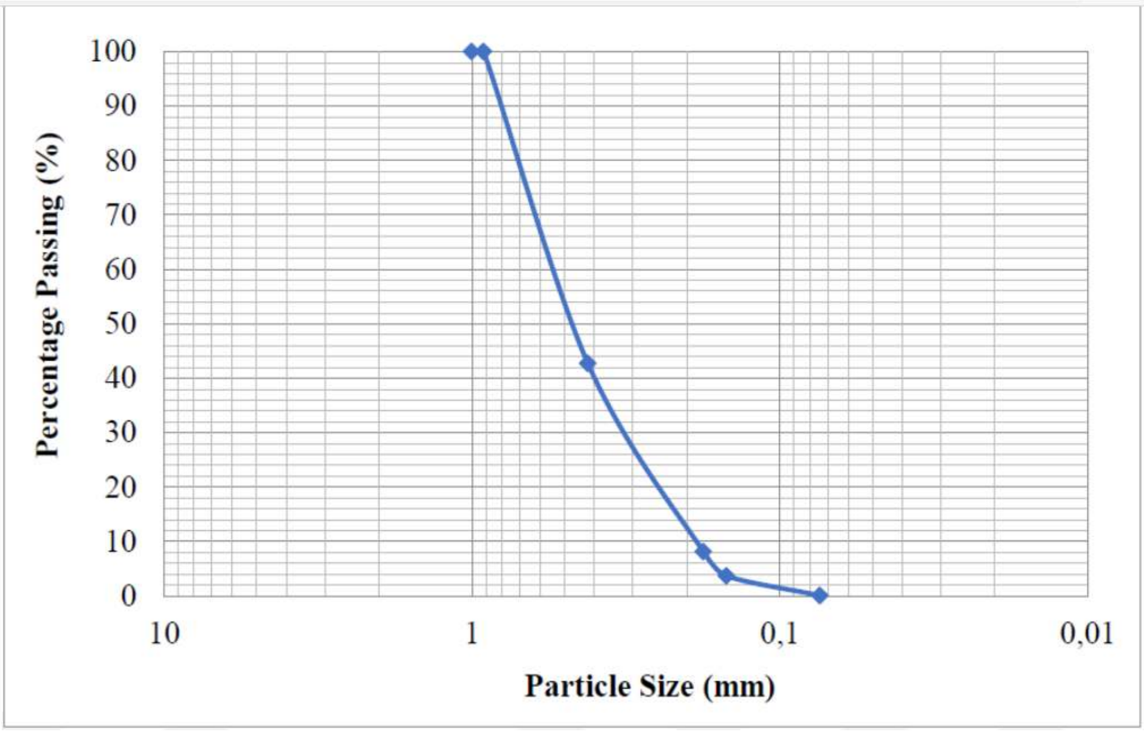


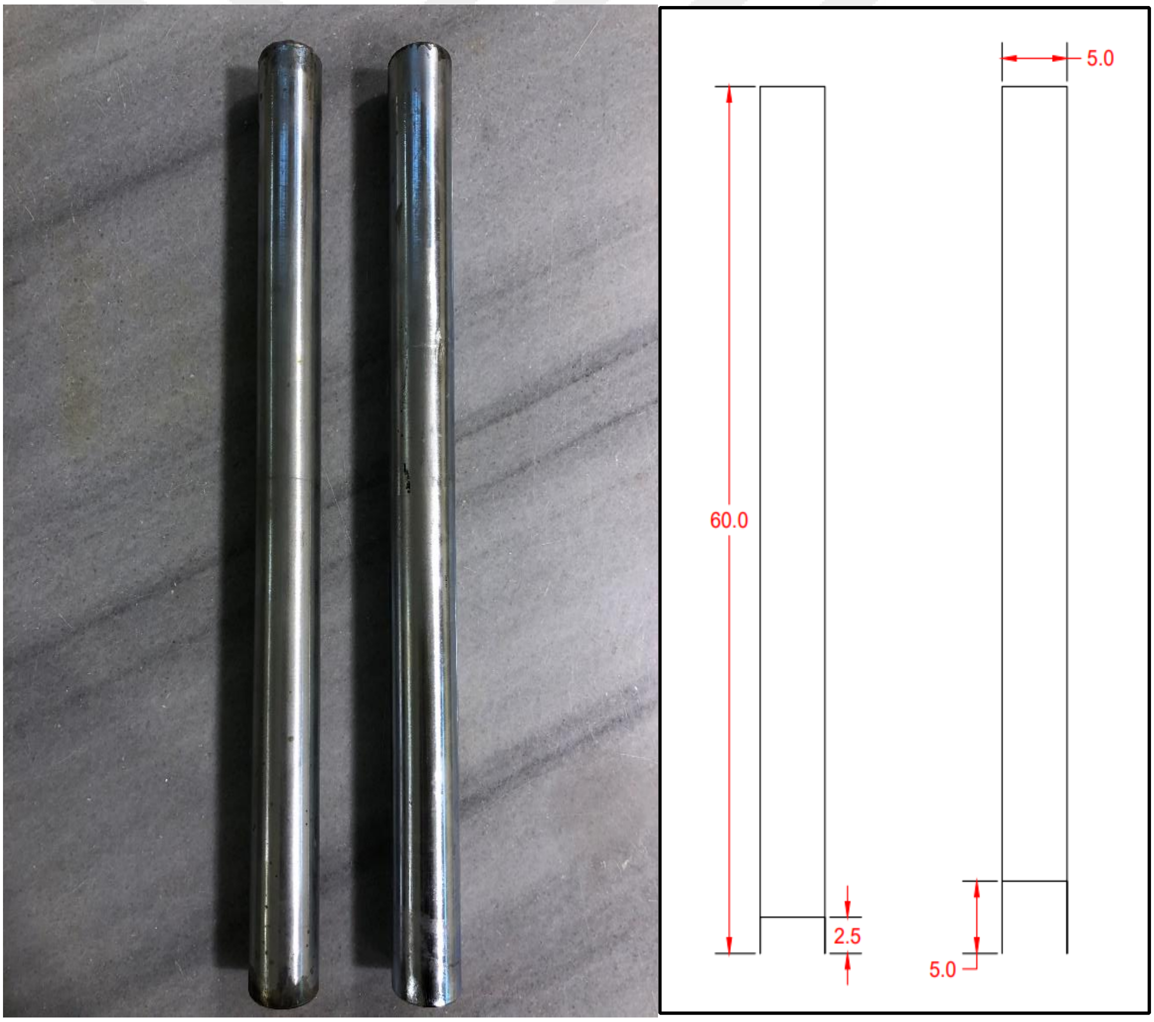
Figure 3.2. Grain size distribution curve of sand

3.2.2. Piles Properties

Two model piles with different diaphragm heights were fabricated for this research. Both piles were made from steel st-37 with dimensional 60cm length (L) and 5cm diameter (D), with 2.5 cm and 5 cm diaphragm heights. Figure 3.3 shows the model of steel piles and there dimensions. Table 3.2 shows the property of the steel (St-37).

Table 3.2. Properties of steel

property	value
Compression strength (Mpa)	240
Tensile strength (Mpa)	360
Tensile Elasticity Modulus (Gpa)	210
Density (gr/cm3)	7.85



(a) (b)

Figure 3.3. (a) Steel piles model (b) piles dimensions

3.3. Relative Density Calibration of the Sand

The relative density depends on the dry density of the soil without cohesion to the density (maximum and minimum). The degree of compacting non-cohesive soil can be determined depending on the relative density. This method cover determining the water content and maximum dry density of non-cohesion materials when compacted by vibration machine. It is used to determine the relative density of non-cohesive soils according to ASTM, which a well-defined moisture-density relationship curve will not produce by impact of compaction, and the maximum impact density method is generally lower than the vibration method.

Before placing sand into the tank, suitable relative density of the sand have to be determined. For this reason the sand have to be calibrated by laboratory experimental test. For compacting sand a hand compactor used to determine four relative density of the sand for this study. A Makita brand electro-pneumatic hammer used for this reason, a steel tamper plate of dimensions (15 X 15 cm) was produced for sand compaction. A plastic plate of the same dimensions of the steel base of machine is added to the bottom of steel base by screw in order to not crush the sand particles as shown in Figure 3.4.

Figure 3.4. Makita electro-pneumatic hammer

A wood box have dimensional (42 X 27.8 X 11.2 cm) was used for determining relative density of the sand. And four values of relative density found Dr=10%, Dr=40%, Dr=65% and Dr=90% by trail testing. For relative density Dr=10% the sand poured into the wooden box from a height of 15 cm through a plastic tube, for relative density Dr=40% the sand poured into the wooden box from a height of 10 cm every 10 cm then compacting every layer of sand by hammer for a moment (less than one second), for relative density Dr=65% the sand poured into the wooden box from a height of10 cm every 10 cm then compacting every layer of sand by hammer for 2 seconds, for relative density Dr=90% the sand poured into the wooden box from a height of 10 cm every 5 cm then compacting every layer of sand by hammer for 8 seconds. For each trail test a sample of sand have been taken to find water content of sand in order to find dry unit weight. Table 3.3 shows the details of compaction process, void ratio, dry unit weight and relative density of the sand.

Table 3.3. Details of compaction process, dry unit weight, void ratio and relative density of the sand

Compaction Test	Dry Unit Weight (γd) kN/m^3	Void Ratio (e)	Relative Density (Dr %)
Sand pours from a height of 15 cm Until the tank is filled to the required depth	14.6	0.86	10
Sand pours from a height of 5 cm every 10 cm of sand and compacting every layer for a moment	15.5	0.75	40
Sand pours from a height of 5 cm every 10 cm of sand and compacting every layer for 2 seconds	16.3	0.66	65
Sand pours from a height of 10 cm every 10 cm of sand and compacting every layer for 8 second	17.3	0.57	90

3.4. Direct Shear Test

Direct shear testing is a field or laboratory test performed by geotechnical engineers to identify internal friction angle of the sand (ϕ) and skin friction angle (δ) between sand and steel piles. The direct shear test involves the testing of a box with dimensional (6 X 6 X 2 cm) of the soil, which is held laterally and sheared along a mechanically involved horizontal plane while under pressure applied along a normal plane to the shear plane. Shear strength was measured at regular intervals through data loggers. Four samples were subjected to different normal pressures until the shear strength attained the maximum sustainable value. For all the test strain controlled direct shear test machine were performed with constant shear rate (0.5 mm/min) horizontal displacement. The relationship between the measured failure of the shear stress and the normal applied stress obtained would determine the shear strength and internal friction parameters.

a) Internal Friction Angle of Sand (ϕ)

For determining the friction angle of sand (ϕ) for this study by shear box device which consists of two metallic plates, two screws, two porous stones, a loading cap and a gripper disk where the normal stress is applied. Different relative density have been used ((Dr= 10%, 40%, 65% and 90%).for each relative density tree different direct shear tests have been done with tree different normal stress (54.5, 109, and 163.5 kPa) according to ASTM D3080-72(1989).Then Internal friction angle of the sand (ϕ) calculated from the results of the experiment. Table 3.4 shows the internal friction angle of the sand (ϕ). Figure 3.5 shows relative density curve of the sand. As shown in the graph the value of friction angle of sand increase by increasing relative density of the soil.

Table 3.4. Internal friction angle of sand

Relative density (Dr %)	Internal friction angle (ϕ)
10	41.0°
40	44.3°
65	48.7°
90	52.4°

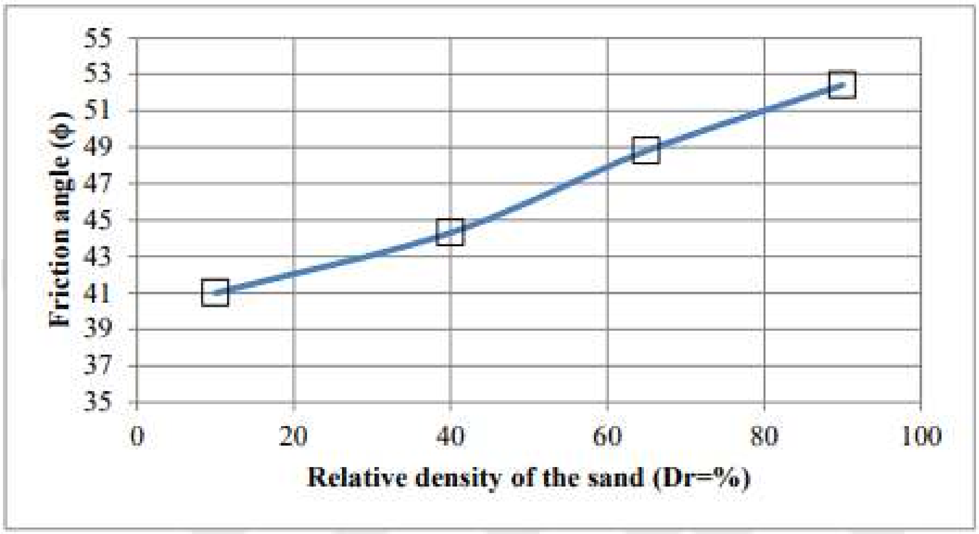


Figure 3.5. Relative density of the sand

b) Interface Shear Test

Direct shear box used to determine the friction angle between sand and steel piles (δ) for all relative density that have been used. Twelve interface shear test have been done for steel pile. In each trial, a sample pile model was placed in bottom half of the shear box and sand was poured in higher half with required relative density and sheared at constant velocity 0, 5 mm/min Figure 3.6 shows the configuration of the shear box. Table 3.5 shows the skin friction angle between sand and steel pile depending on relative density of the soil. From Figure 3.7 it is clear that skin friction angle increase by increasing relative density till (Dr=60%) but it decreased by relative density (Dr=90%).

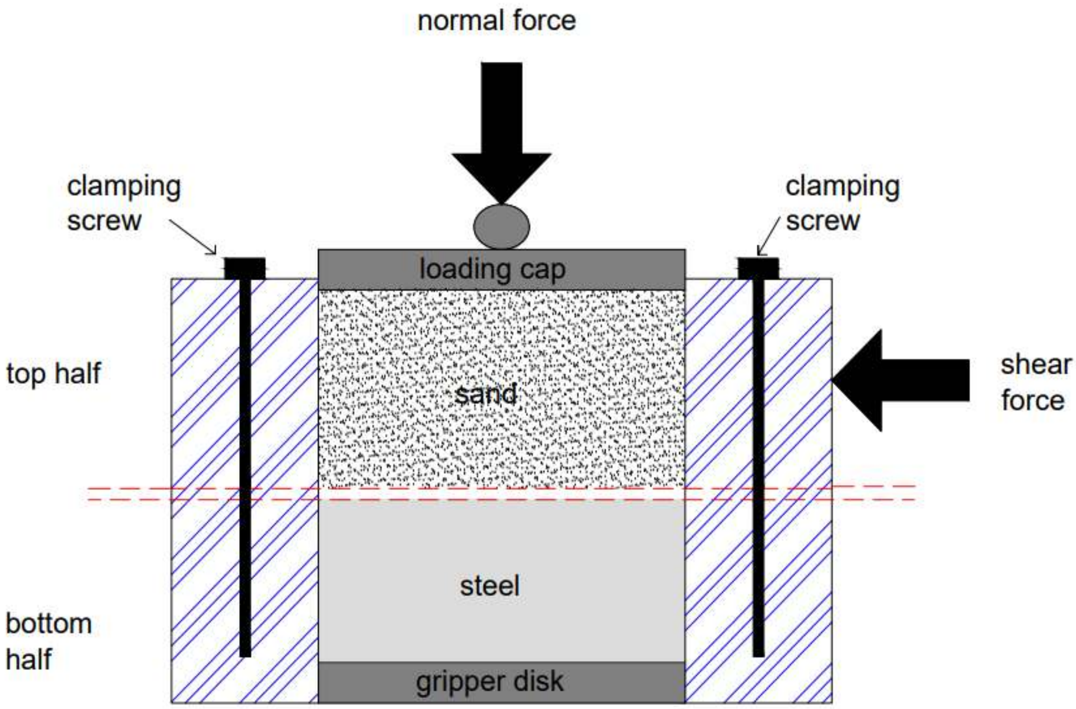


Figure 3.6. Configuration of the shear box

Table 3.5. Skin friction angle between sand and steel pile

Relative density (Dr %)	Friction angle between sand and steel (δ)
10	18.04°
40	19.28°
65	20.7°
90	18.8°

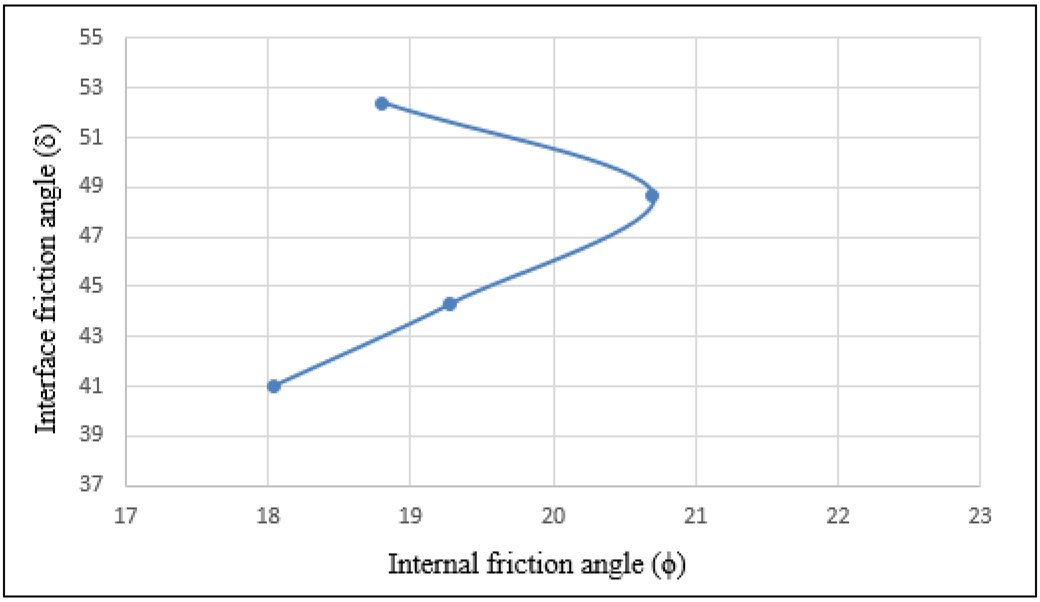


Figure 3.7. Skin friction angle sand-steel pile

3.5. Surface Roughness Determination of the Steel Piles

Surface roughness is a measure of the surface texture of a specific material. This roughness is measured by actual vertical surface deflections of its ideal shape. The surface of the material is rough if these deflections are large and is considered smooth if these deflections are small. In general, roughness is a high frequency and short wave component of the measured surface. In practice, knowing the amplitude and frequency is often necessary to ensure that the surface is suitable.

The roughness for different materials can be determined by experiments The relative roughness - the ratio between the absolute roughness of the steel pile and sand .The roughness of a surface was usually measured by means instrument which a stylus moves through the surface. The result is usually expressed as Ra, or mean roughness, and this is the mean value of the deflection effect above and below the center line. The value of Ra is typically measured in micrometers. Objectives such as surface roughness (Ra) and the measurement of power consumption used for the assessment of cutting force are recorded during machining. A further factor for surface roughness description is Ry which measures the root-mean-square deflection in a profile. R_Z is the maximum surface roughness. The surface roughness parameters were calculated by using (Mutitoyo SJ-201), eqs. (3.1) to (3.3) [42].

$$Ra = \frac{1}{N}\sum_{i=1}^{N}|Yi|$$

(3.1)

$$Ry = \ Yp + \mathrm{Yv}$$
(3.2)

$$Rz = \frac{1}{5}\sum_{i=1}^{5} Ypi + \frac{1}{5}\sum_{i=1}^{5} Yvi$$
(3.3)

It emerged from the results of the experiment of steel pile that the value of Ra=0.55 and Ry=3.43.

3.6. Axial Load Test

3.6.1. Preparation

Large-scale experimental setup was designed and fabricated specifically for these experiences, which is consist of a tank with dimensions of 75 cm length, 75 cm width and 100 cm depth .It also includes of a computer program system, two steel piles with dimensions of 60cm length, 5 cm diameter with diaphragms (2.5 and 5 cm) and some other tools that make up the system, the laboratory model test set up has been presented in Figure 3.8 which explain the devices , their names and their places in detail.

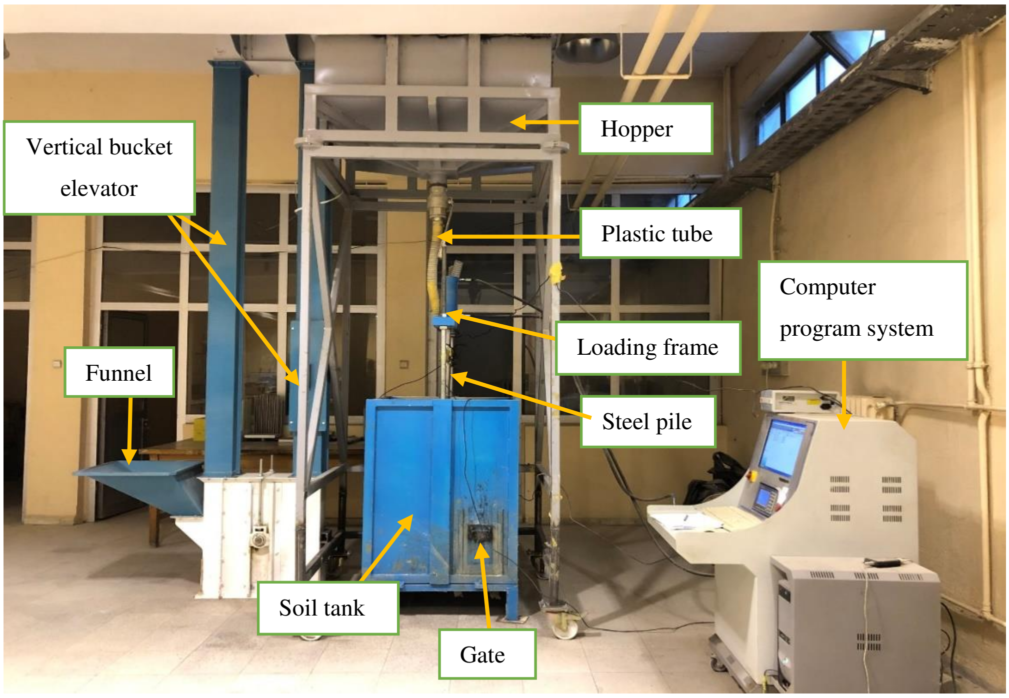


Figure 3.8. Model tank and loading frame

To reduce the measure of internal influence, some authors have suggested some values such as Vipulanandan et al put value 50 [54], Peterson (1988) put value 80 [55] and Loukidis and Salgado (2009) put value 100 [56], they said that in order to avoid that kind of problems [57].

The ratio of the average tank diameter to the model piles diameter (D_{tank}/D_{pile}) which equal to 15. This ratio must be greater than the minimum suggested value Bolton et al. (1999) which is 8, also diameter of the model pile (5 cm) must be greater than ($20*D_{50}$) main diameter of the soil particles which equal to 1 cm. To minimize internal scale effects as the minimum value required to eliminate the effect of scale on the base [58].

On the top of the loading frame there is a 4 tons hydraulic loading unit that can be moved vertically. This mechanical jack is installed in the inner side of the top section at the same distance from two ends of the bearing frame. It can be regulated by a hydraulic charge speed regulator. During the tests, using the data acquisition device, loading and settlement values are recorded through an interface program by transferring them to a computer program system. The sand was placed above the hopper by vertical bucket elevator, from there, the sand was distributed into the tank by means of a plastic tube. The optimal combination of dropping the sand from a specified height was established after a few tests. The objective was to produce samples of thick sand at relative density of 10%, 40%, 65% and 90%, respectively. It is important to note that in such cases, the tests simulate the section of a pile near the base of the tank, rather than the entire pile. The top head of each pile has been attached the detachable loading shaft vertically by special types of epoxy adhesive material to ensure that the piles well attached to the loading shaft as shown in Figure 3.9.

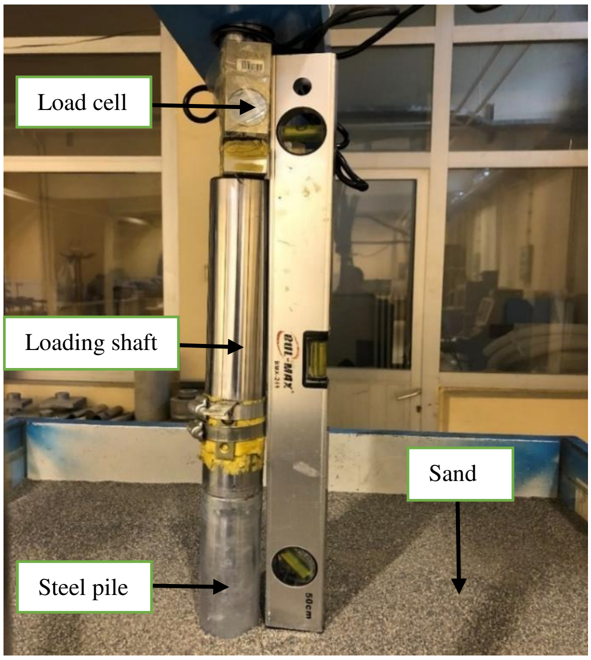


Figure 3.9. Pile attached to the loading shaft

To get the bending behavior of piles, two strain gauges (LVDT) linear variable differential transformer brand Opkon are attached to the pile. Settlement of the pile were measured with LVDTs. The most common inductive transducer converts linear motion into an electrical signal. The maximum measurement deflection for LVDTs was 15 cm. These LVDTs were connected to the computer by electrical wires, and the data were analyzed and recorded by a designed software were used to acquire the displacements, strains and axial load. Figure 3.10 shows LVDTs.

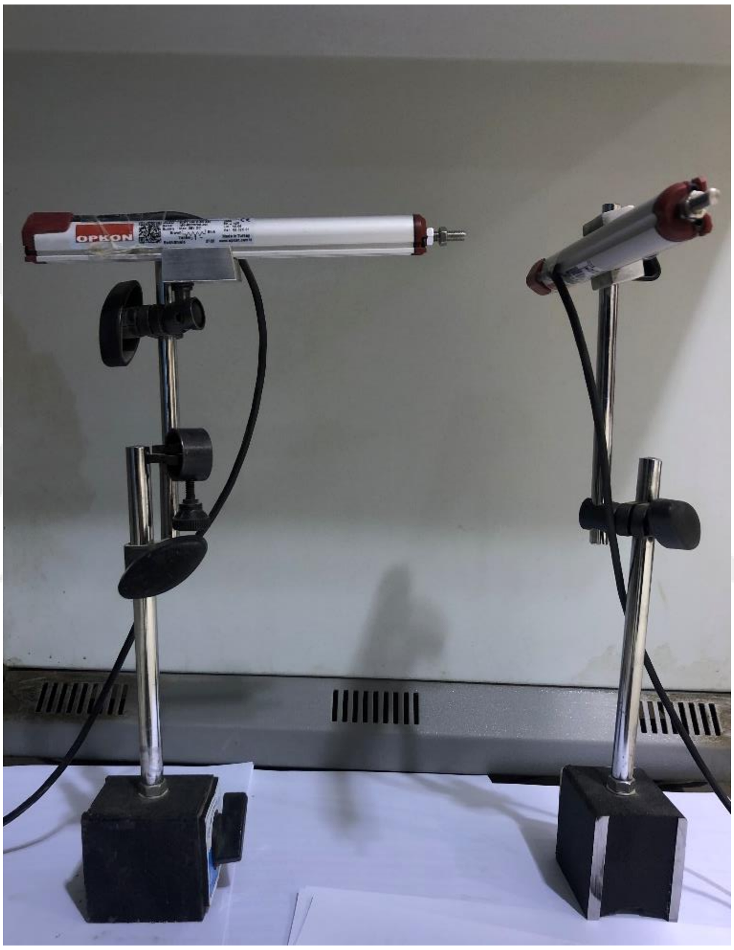


Figure 3.10. LVDTs

3.6.2. Test procedure

The method chosen to prepare sand samples with different densities in the chamber was by putting sand into the funnel, from there the sand was pulled through electrical vertical bucket elevator. Sand stored on the top of the tank in a hopper. From the hopper, sand poured through a specific plastic tube into the tank at specific heights. So that the height of the sand in the tank reaches 90 cm. This process was carried out in multiple stages for every relative density (Dr).

For relative density (Dr=10%) sand is poured through the plastic tube from a height of 15 cm until it reaches a height of 40 cm in the tank. This height was found by avoid the affecting the material properties on the pile bearing capacities, because stress from the load applied to the foundation can be effects on soil layers up to a depth of (1.5B to 2B). This height have been chosen according to (40 cm> 2B), B is short dimension of the foundation. And after reaching the required depth the sand is leveled with a wooden ruler, to ensure that the surface is leveled as shown in Figure 3.11.

Figure 3.11. Leveling and compacting sand in the tank

For relative density (Dr=40%) sand pours from a height of 10 cm every 10 cm depth of sand then compacting the sand by compacter for one moment (less than one second),then leveling the surface of the sand by wooden ruler. This process is repeated for 4 times to reach 40 cm depth of the tank depth.

For relative density (Dr=65%) sand pours from a height of 10 cm every 10 cm of sands depth then compacting it by compacter for 2 seconds then leveling the surface of the sand by wooden ruler. This process is repeated for 4 times to reach 40 cm depth of the tank.

For relative density (Dr=90%) sand pours from a height of 10 cm every 5 cm depth of the sand then compacting the sand by compacter for 8e seconds, then leveling the surface of sand by a wooden ruler. This process is repeated for 8 times to reach main depth of the tank (40 cm). The Figures (3.12, 3.13, and 3.14) show an experimental method of putting sand into the tank.

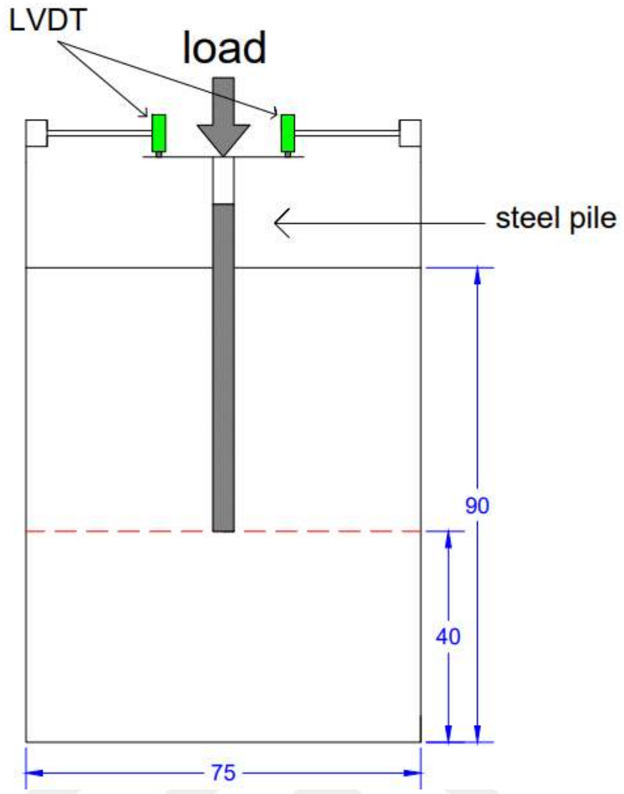


Figure 3.12. Experimental method of putting sand into the tank for (Dr=10%)

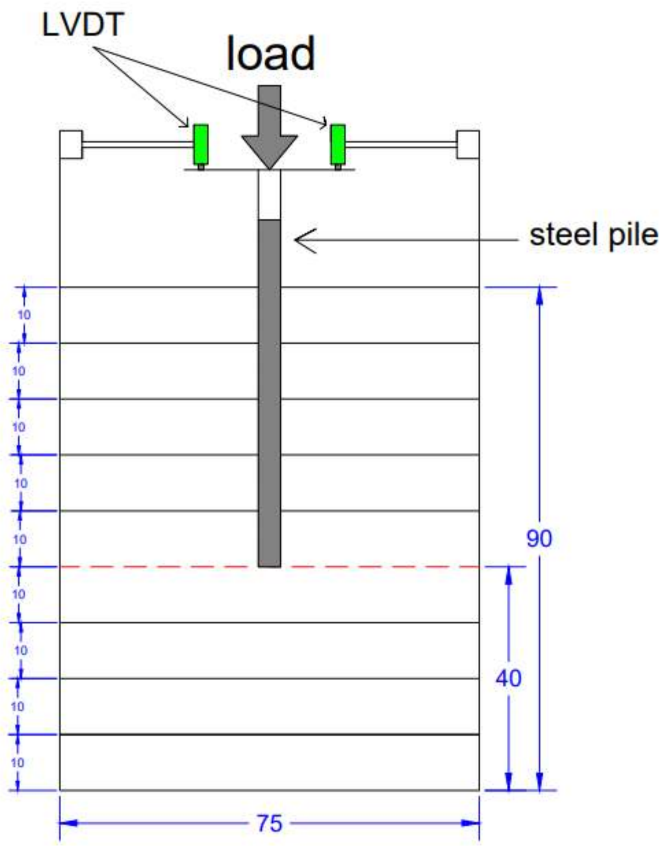


Figure 3.13. Experimental method of putting sand into the tank for (Dr=40%, 65%)

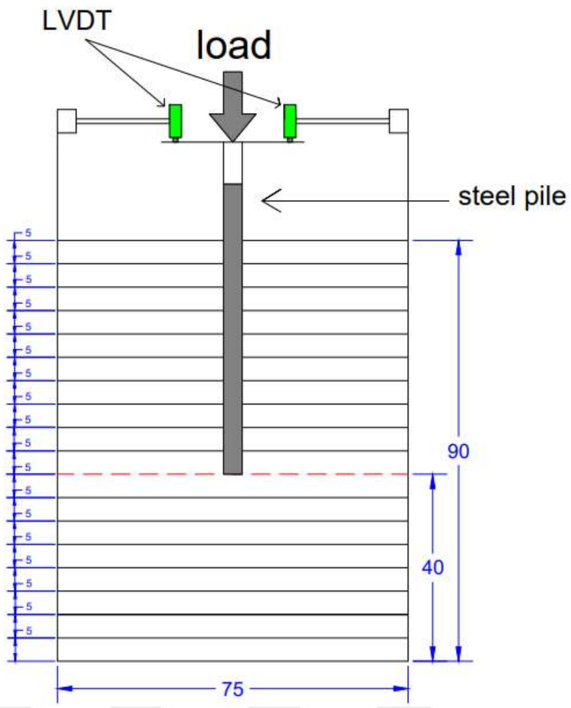


Figure 3.14. Experimental method of putting sand into the tank for (Dr=90%)

After that, the pile installed on the detachable loading shaft on load cell. The column is lowered by hydraulic jack until the pile base touched in the sand. After that, the process of pouring sand into the tank is repeated, as previously explained, until the depth of the sand in the tank reach 90 cm. To measure settlement of the pile two LVDTs installed on the pile as shown in Figure 3.15, the axial load displacement is downloaded to the pile, data and the information is collected in a computer.

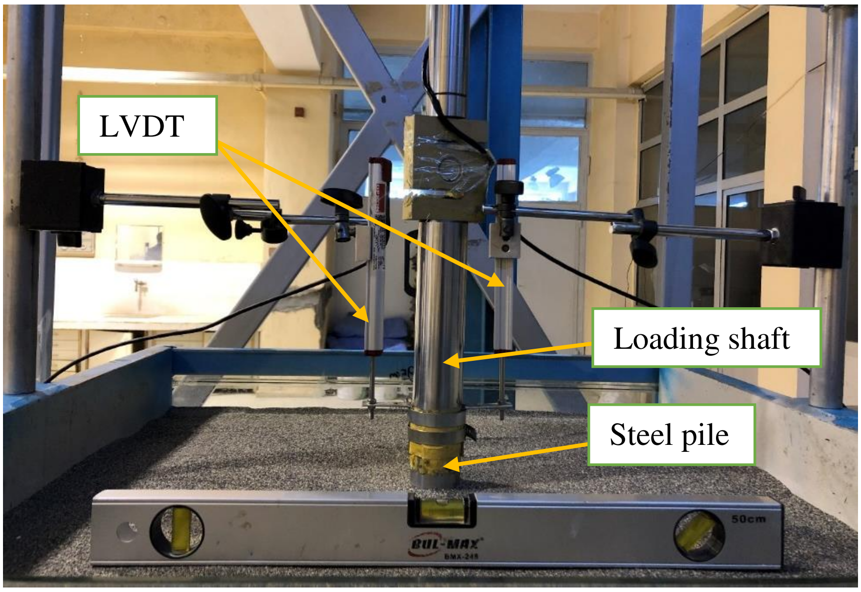


Figure 3.15. LVDTs installation on the pile

4. RESULTS AND DISCUSSION

4.1. Introduction

In this chapter, the results obtained from the experimental results of model piles will be presented, and the results of both piles with 2.5 and 5 cm diaphragm heights will be compared, as well as the experimental results will be compared with the results obtained from the effects used regarding the final results of the Plaxis 3D program.

4.2. Experimental Result

Two single piles with different diaphragm heights (2.5 and 5 cm) is considered for understanding the vertical axial load on settlement of piles in sandy soil with relative densities equal to (10%, 40%, 65% and 90%). The experimental results for the single pile are shown in Figures (4.1, 4.2, 4.3 and 4.4).

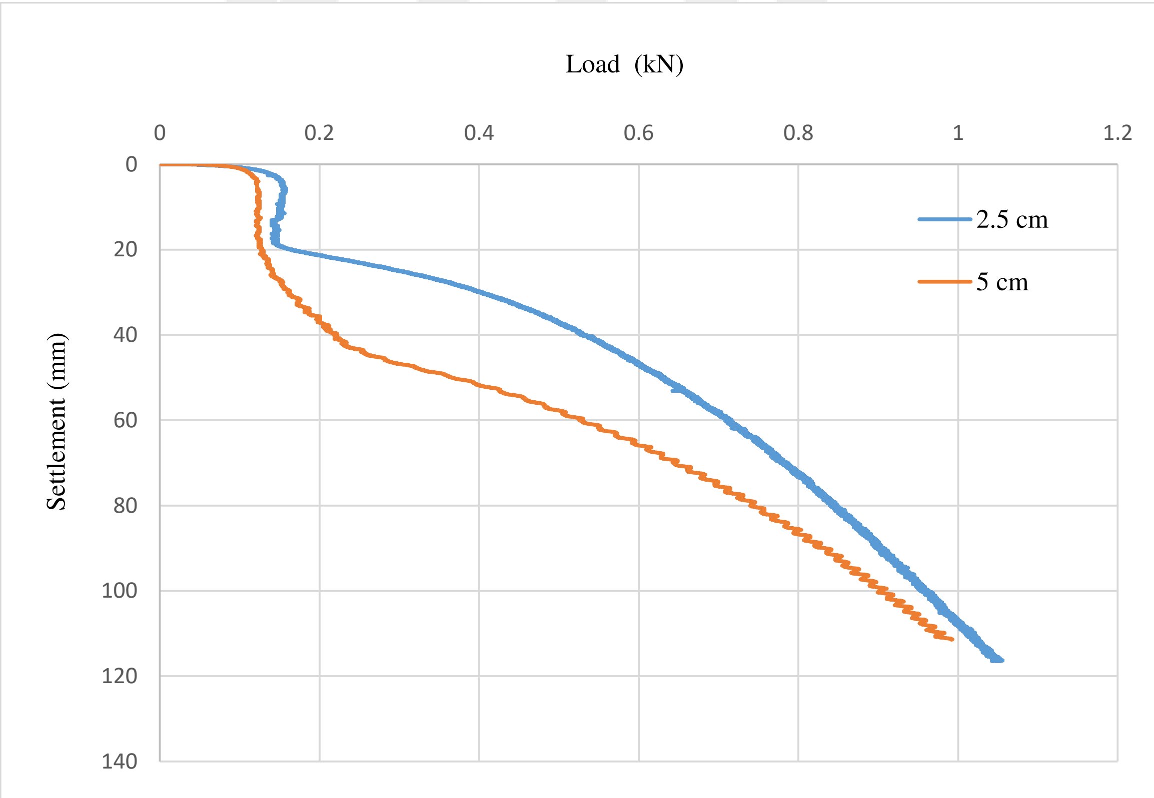


Figure 4.1. Load-Displacement curves of model piles for total bearing capacity test at Dr = 10%

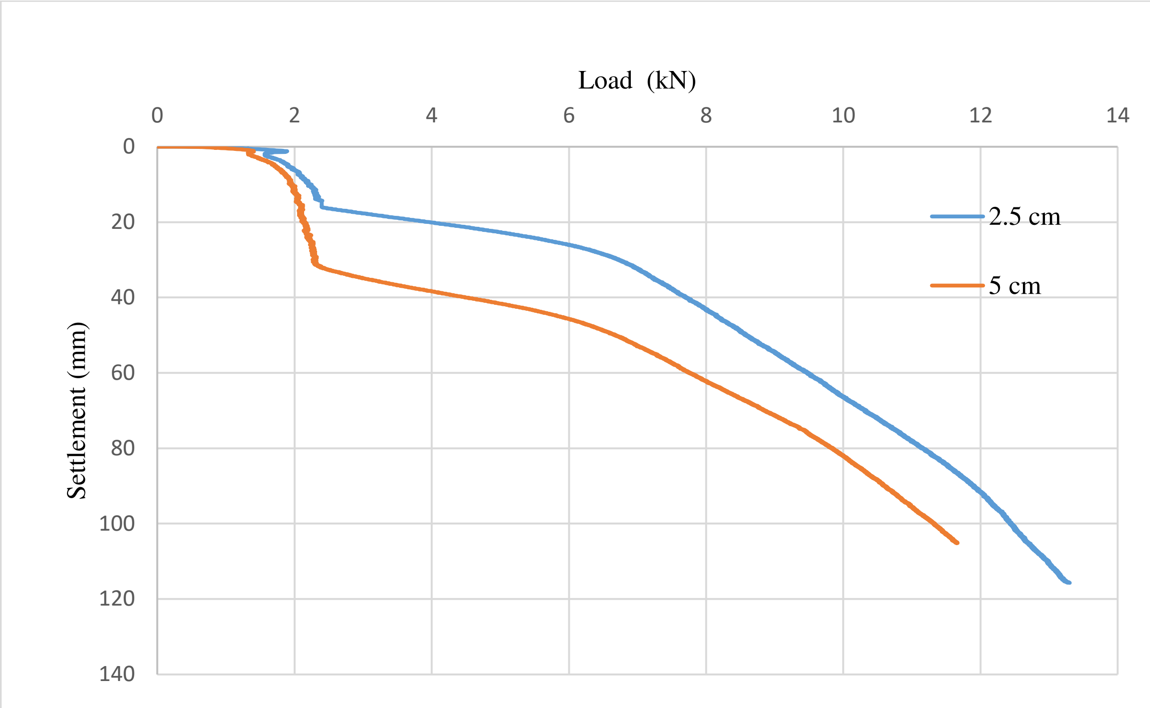


Figure 4.2. Load-Displacement curves of model piles for total bearing capacity test at Dr = 40%

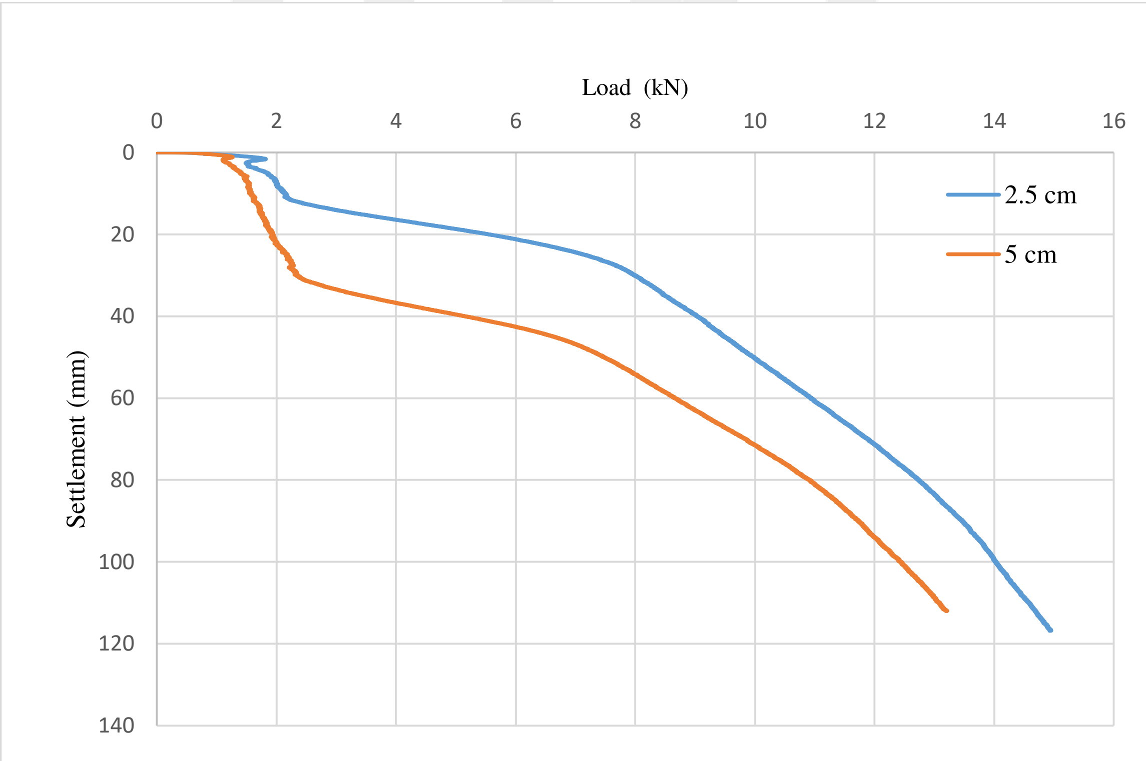


Figure 4.3. Load-Displacement curves of model piles for total bearing capacity test at Dr = 65%

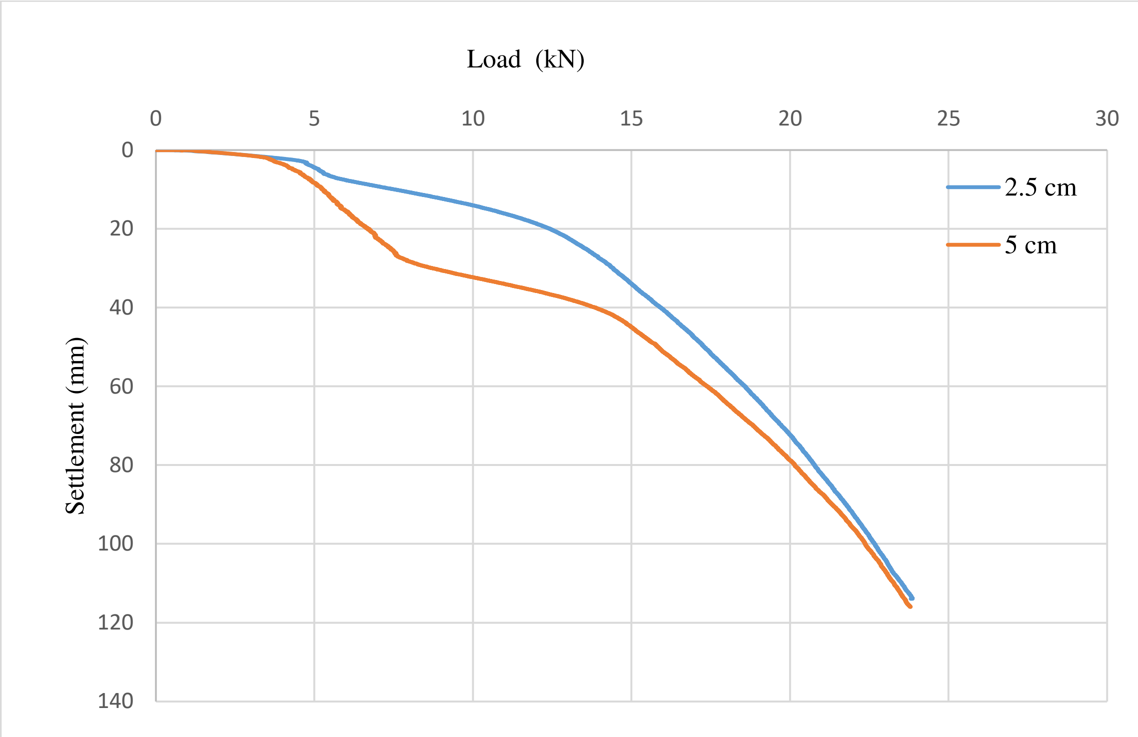


Figure 4.4. Load-Displacement curves of model piles for total bearing capacity test at Dr = 90

The experimental results show that the vertical axial on the pile and settlement increased with increasing relative density (Dr) of sand in a non-linear relationship. The ultimate bearing capacity Table 4.1 have been selected from the load-settlement experimental curves of the model piles, for Dr=10% the ultimate bearing capacity selected on the experimental curve by 15% of the pile diameter but the ultimate bearing capacity for the other Dr selected from the experimental graphs.

The total bearing capacity and the settlement results of single piles for the (2.5 and 5 cm) diaphragms height increases with increasing relative density Dr% of the sand in a non-linear relationship. The Figures (4.5 and 4.6) show the total ultimate bearing capacity and settlements of steel piles versus the relative density of the sand.

Table 4.1. Experimental results of the total bearing capacity of the model piles

Dr%	Load (kN)		Settlement (mm)	
	2.5 cm	5 cm	2.5 cm	5 cm
10	0.35	0.37	27.075	50.5
40	2	2.216	20.35	49.15
65	5	5.43	17.57	38.6
90	10.02	12.5	14.2	36.73

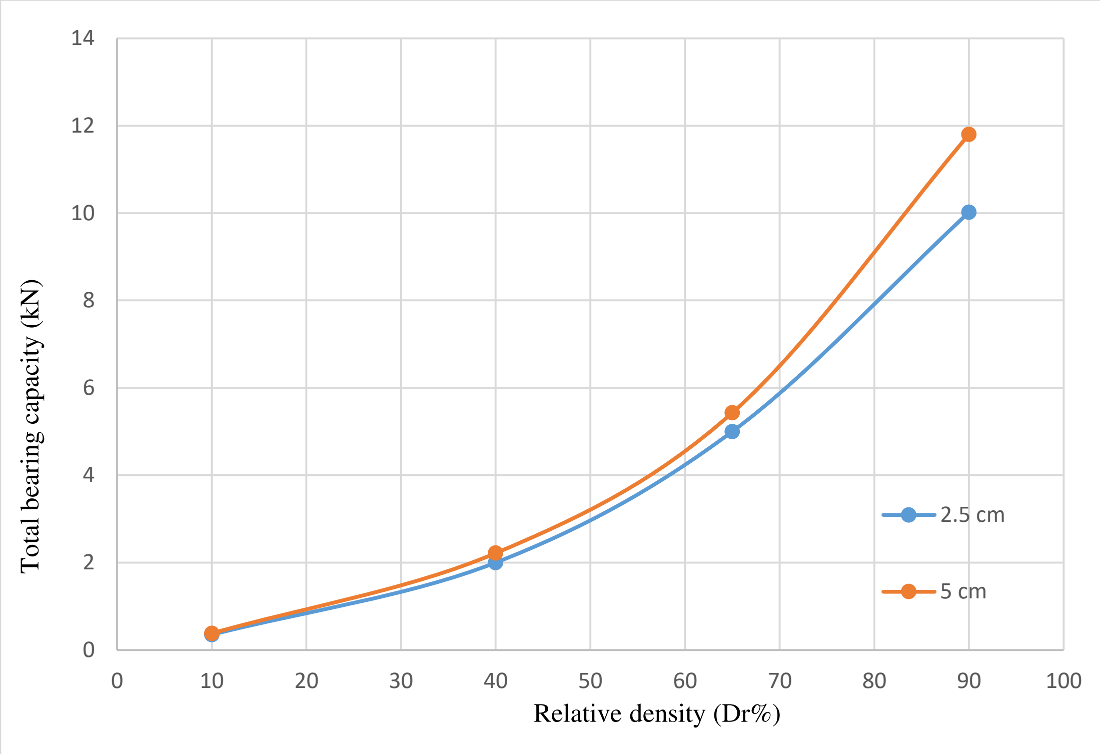


Figure 4.5. Total bearing capacity of model piles versus relative density

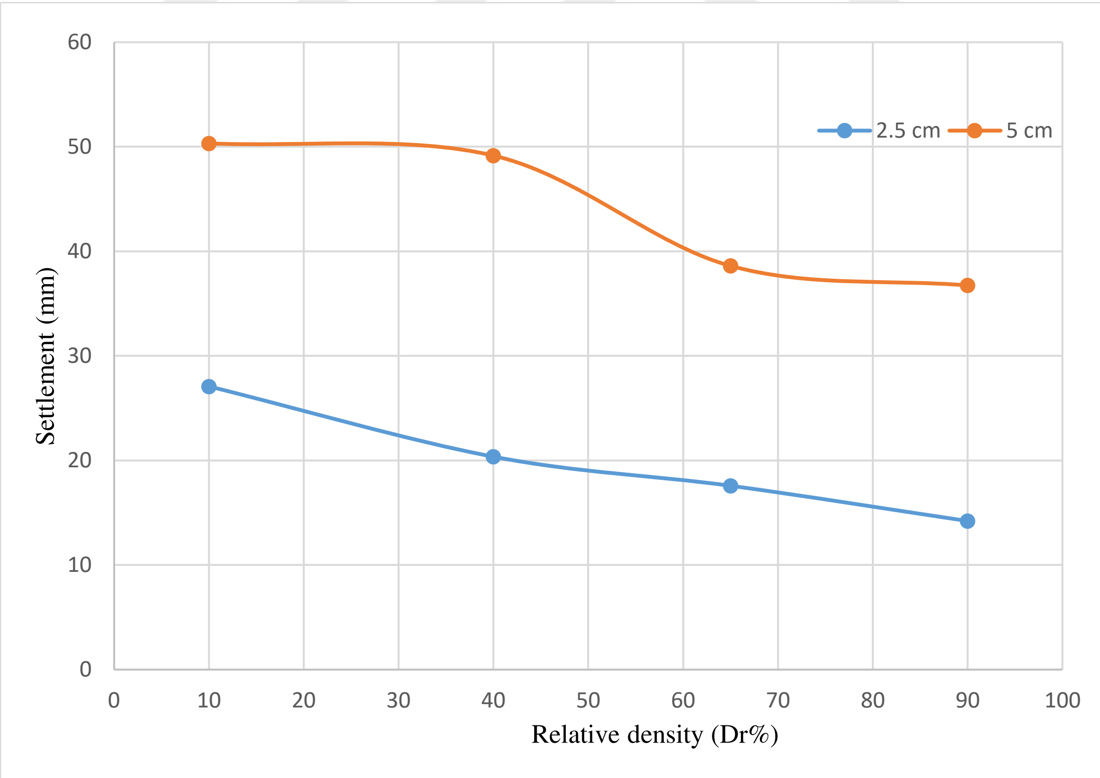


Figure 4.6. Displacement versus relative density for the total bearing capacity of model piles

4.3. Finite Element Analyses

In this section, the finite element method program Plaxis 3D version 20 have been used to model single pile's behavior under axial loading. Its results are compared with the experimental results.

4.3.1. Introduction

The Finite Element Analysis (FEA) methods are important to computational techniques for approximate results of a wide variety of "real-world" engineering problems with complex different fields under common boundary conditions. The FEA has been an important step in modeling or understanding physical phenomena in various Specialties of engineering. Usually the physical phenomena are based on the continuum of a materials (solid, liquid or gas) under including many different fields. From point to point the vary field variables, for that reason there are an infinite number of solutions in the domain. The basis FEA depend on the domain decomposition estimation of finite solution onto the limited subdomains (elements) numbers where approximate systematic solution have been performed or a weighted residual method. In practice, the FEA reduces the problem by dividing the domain into number of elements and specifying unknown field variables in terms of the estimated function that is calculated in each element, and reduces the infinite number of unknowns. These functions (also called interpolation functions) are defined by the values of the variables specifically field at some point called nodes. Usually nodes are located along the boundaries of the elements and connect elements adjacently. The ability to discretize irregular domains by finite elements makes this method a practical and valuable analytical tool for solving the initial, boundary, and intrinsic value problems arising in various engineering disciplines [59].

4.3.2. Geometry

The finite element method-based program Plaxis 3D have been validated for both piles. The analyzed pile is subject to axial loading, applied through prescribed displacements. Both the soil and the interface are modelled using the Mohr-Coulomb failure criterion. The interface strength factor, R_{inter}, is calculated based on the friction angle of the soil. These conditions are as similar as possible with the ones of the other simulations, the most significant difference being the interface, shear modulus, friction angle and cohesion could all be manually entered in the software. The Plaxis 3D simulation is axisymmetric and 15-node triangular elements are used as shown in Figure 4.**7**.

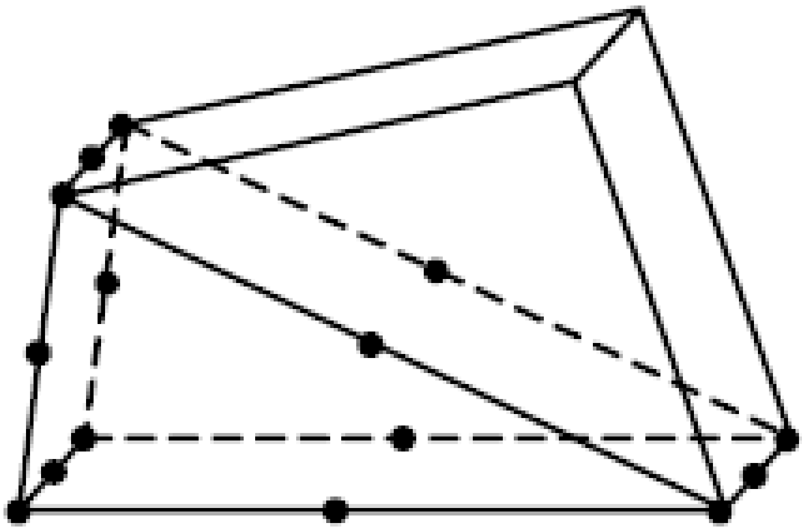


Figure 4.7. 15-Node wedge element in 3D Plaxis program

The model consists of the steel piles 60 cm long and 5 cm diameter with diaphragms of (2.5 and 5 cm) height and four different Dr%. The water table is 0 because the soil is dry. The total dimensions of the model tank is (75 x 75 x 100 cm), as used in the previous simulations. Table 4.2 and 4.3 shows the materials (sand and steel) parameters used in the Plaxis 3D simulation.

Table 4.2. Sand parameters used in Plaxis 3D program

Dr%	Average Dry Unit Weight (γd), (kN/m^3)	Modulus of Elasticity of the Sand (E) (kN/m^2)	Poisson's ratio	Internal friction angle of the sand (ϕ)°	Dialatancy angle of the sand (ψ)°	R_{int}	Cohesion kN/m^2
10	14.6	3100	0.3	41	11	0.7	0.001
40	15.5	28000	0.3	44.3	14.3	0.7	0.1
65	16.3	60000	0.3	48.7	18.7	0.7	0.3
90	17.3	120000	0.3	52.4	22.4	0.7	0.5

Table 4.3. Steel pile parameters used in Plaxis 3D program

Parameters	Steel pile
Average Dry Unit Weight (γd), (kN/m^3)	30.79
Modulus of Elasticity of the Piles (E) (kN/m^2)	$2 * 10^8$
Poisson's ratio	0.3

The generated mesh in Plaxis 3D Output program should be always inspected to ensure that it fits the requirements of the calculation, Figure 4.8 and 4.9 shows before applying load and after applying load settlement of the pile in the 3D dimensional mesh.

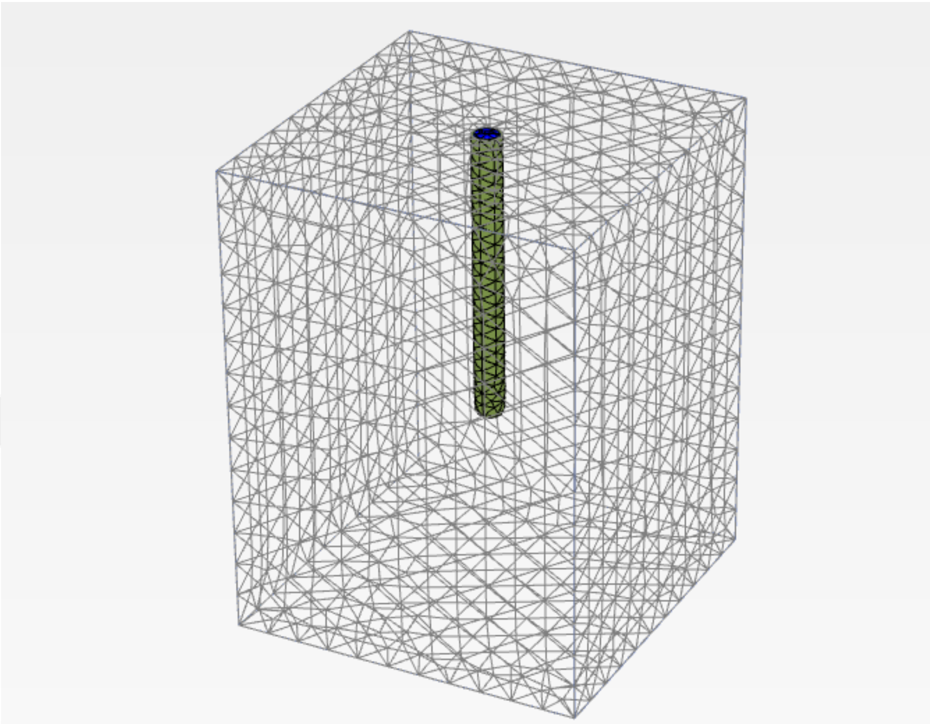


Figure 4.8. 3D dimensional mesh generated before applying load in Plaxis 3D program

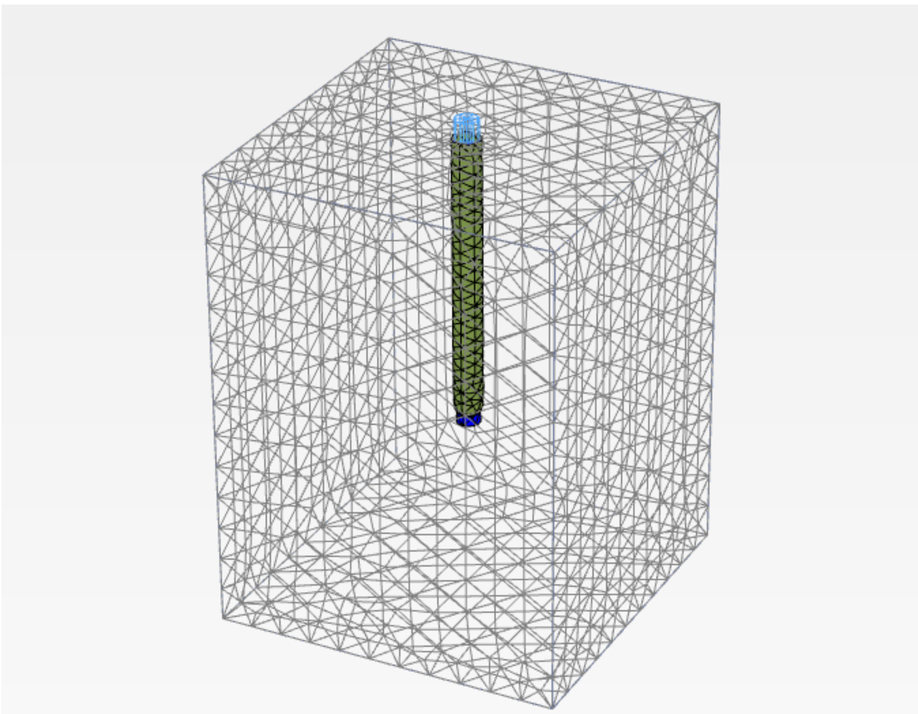


Figure 4.9. 3D dimensional mesh generated after applying load in Plaxis 3D program

4.3.3. Finite Element Analysis Results

In order to compare results provided by experimental work that which presented in former chapters, ultimate bearing capacity load and settlement of the model piles and sand have been chosen in experimental curve and must be computed and analyzed, depending on that result Plaxis 3D curves have been provided .The result of load-displacement field within the model boundaries is displayed in Table 4.4 and 4.5.

Table 4.4. Comparison of the experimental results with Plaxis 3D for steel pile with 2.5 cm diaphragm height

Dr (%)	Load (kN)	Experimental Settlement (mm)	Plaxis 3D Settlement (mm)
10	0.35	27.075	25
40	2	20.35	21.23
65	5	17.57	19
90	10.02	14.2	16.5

Table 4.5. Comparison of the experimental results with Plaxis 3D for steel pile with 5 cm diaphragm height

Dr%	Load (kN)	Experimental Settlement (mm)	Plaxis 3D Settlement (mm)
10	0.37	50.315	48.2
40	2.216	49.15	47.79
65	5.43	38.6	40.3
90	12.5	36.73	43

These diagrams attest for the good behavior of the model, the results are as expected, it is shows that the load is transmitted to the pile and also the piles and the soil interact. The results showed that there is a good agreement of the Plaxis 3D finite element program compared to the experimental results Figure 4.10 and 4.11. Figures 4.12-4.19 show the load-displacement curves of the model piles analysis by Plaxis 3D.

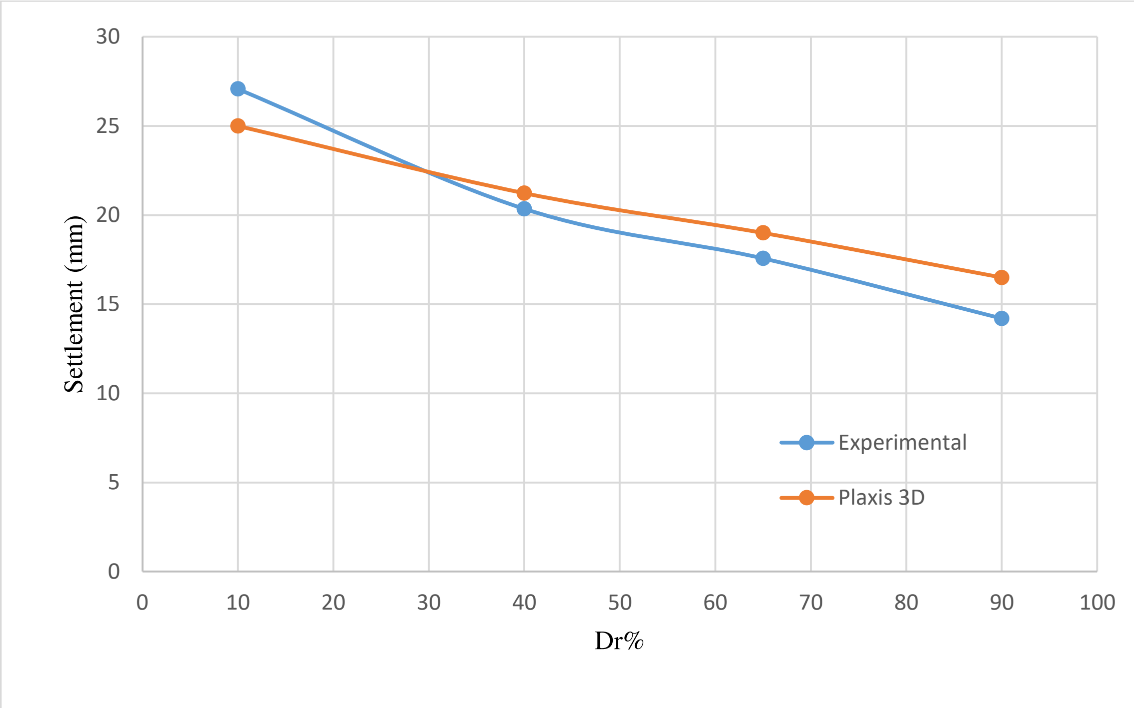


Figure 4.10. Comparison of the experimental results with Paxis 3D For 2.5 cm diaphragm pile

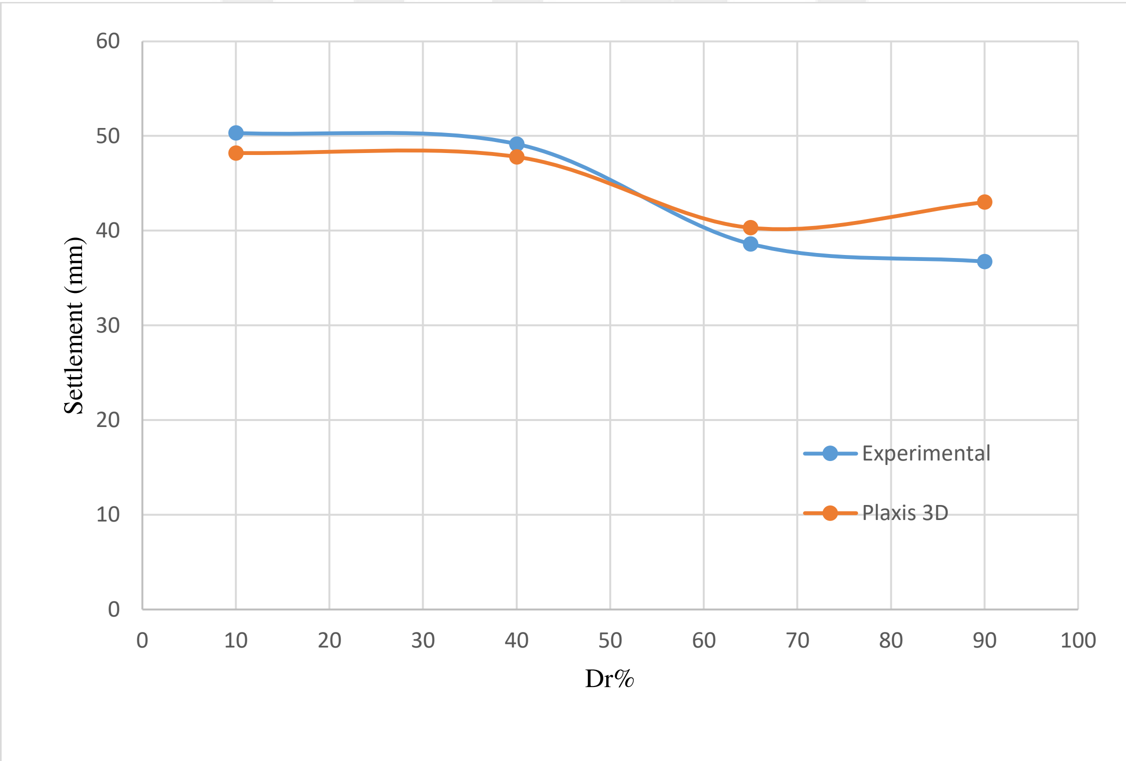


Figure 4.11. Comparison of the experimental results with Paxis 3D for 5 cm diaphragm pile

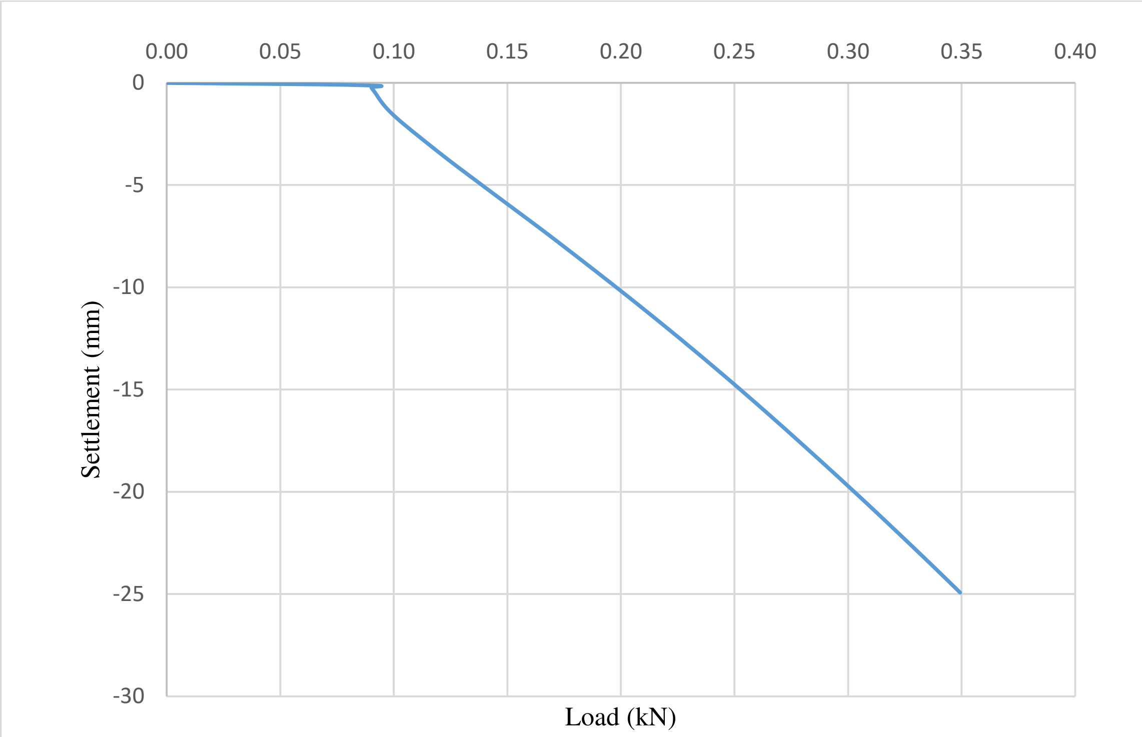


Figure 4.12. Load-Displacement curve of 2.5 cm diaphragm model pile by Plaxis for Dr=10%

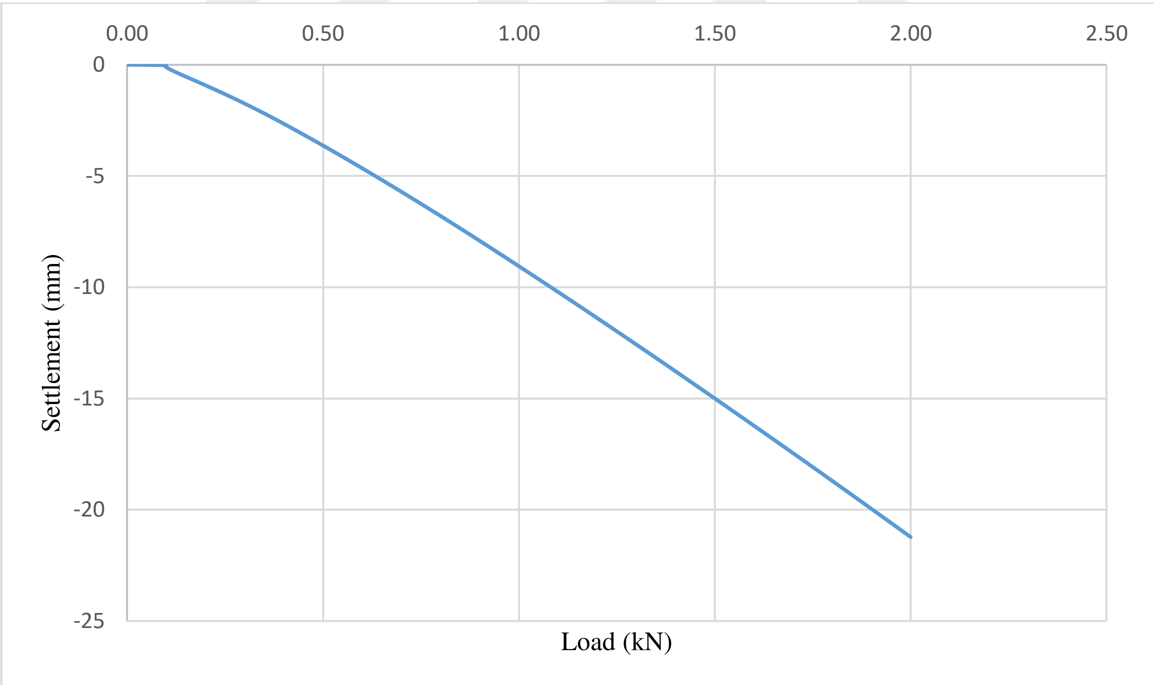


Figure 4.13. Load-Displacement curve of 2.5 cm diaphragm model pile by Plaxis for Dr=40%

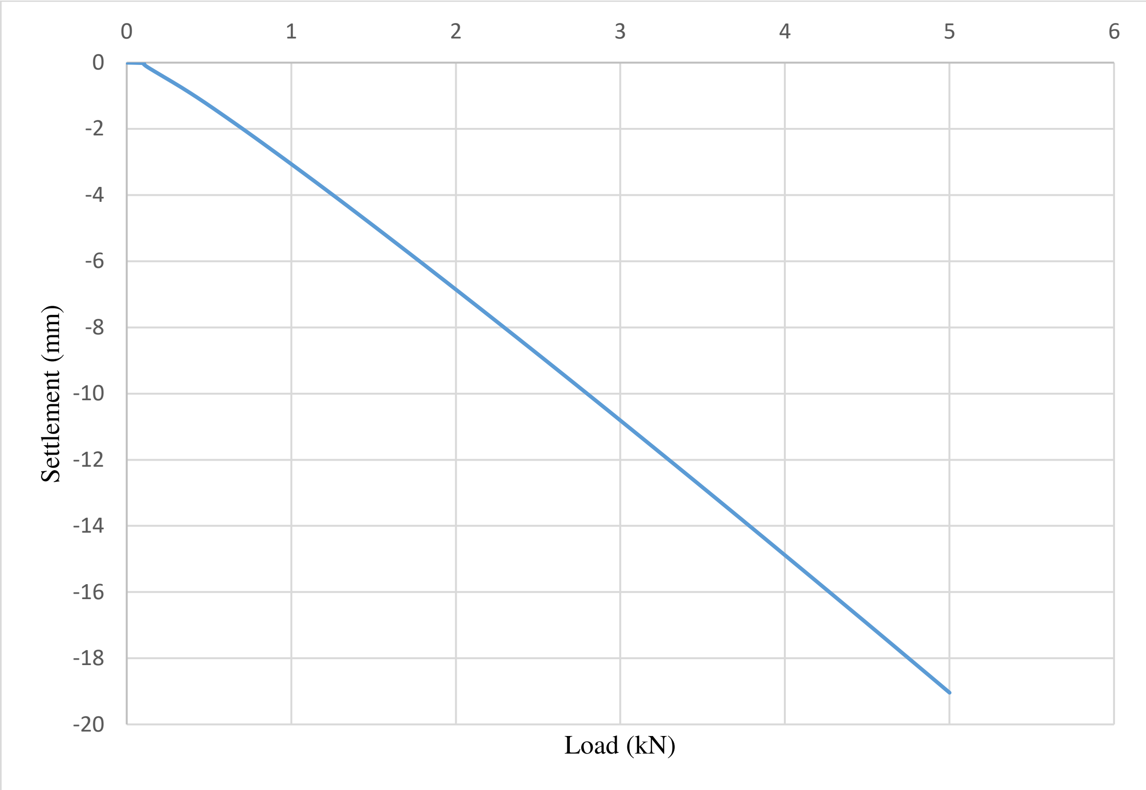


Figure 4.14. Load-Displacement curve of 2.5 cm diaphragm model pile by Plaxis for Dr=65%

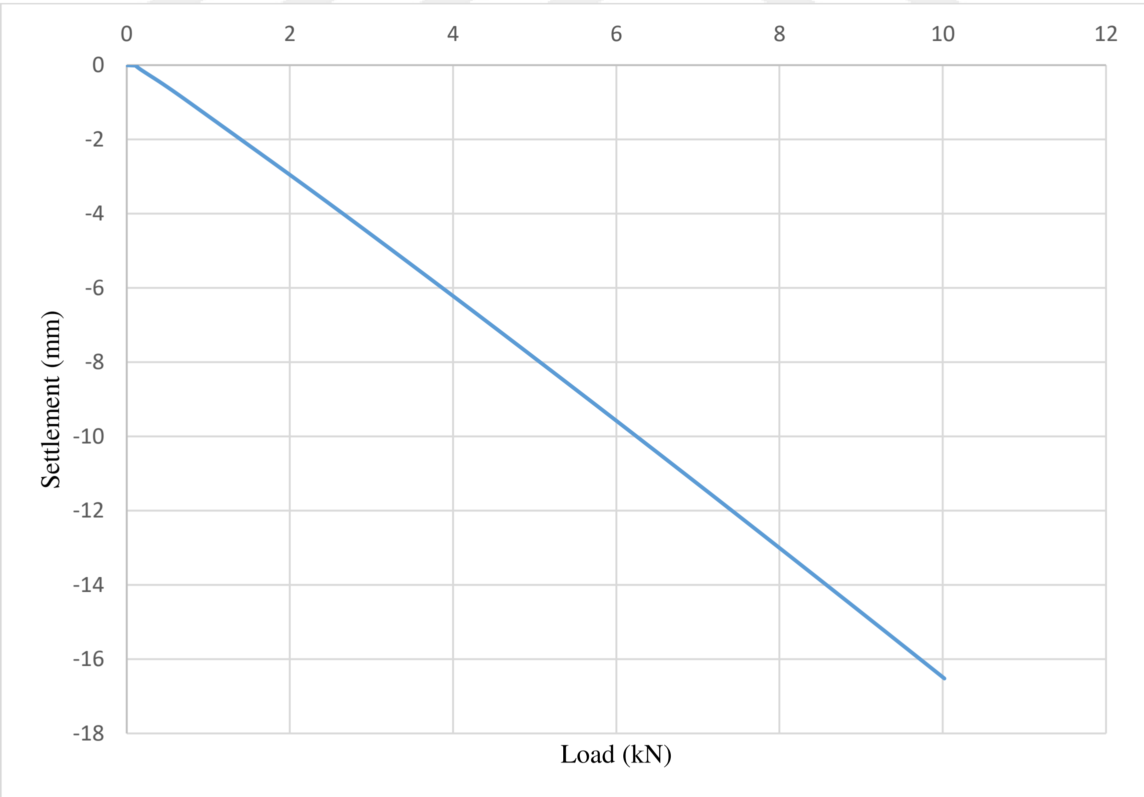


Figure 4.15. Load-Displacement curve of 2.5 cm diaphragm model pile by Plaxis for Dr=90%

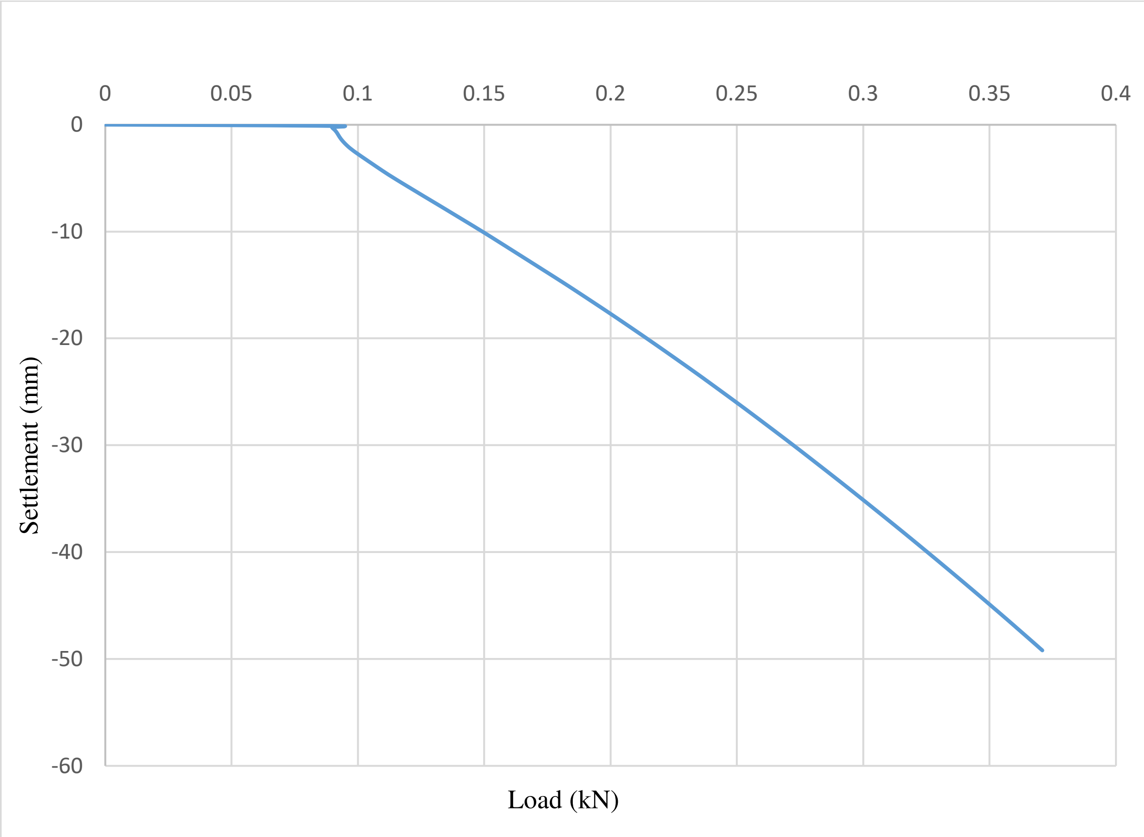


Figure 4.16. Load-Displacement curve of 5 cm diaphragm model pile by Plaxis for Dr=10%

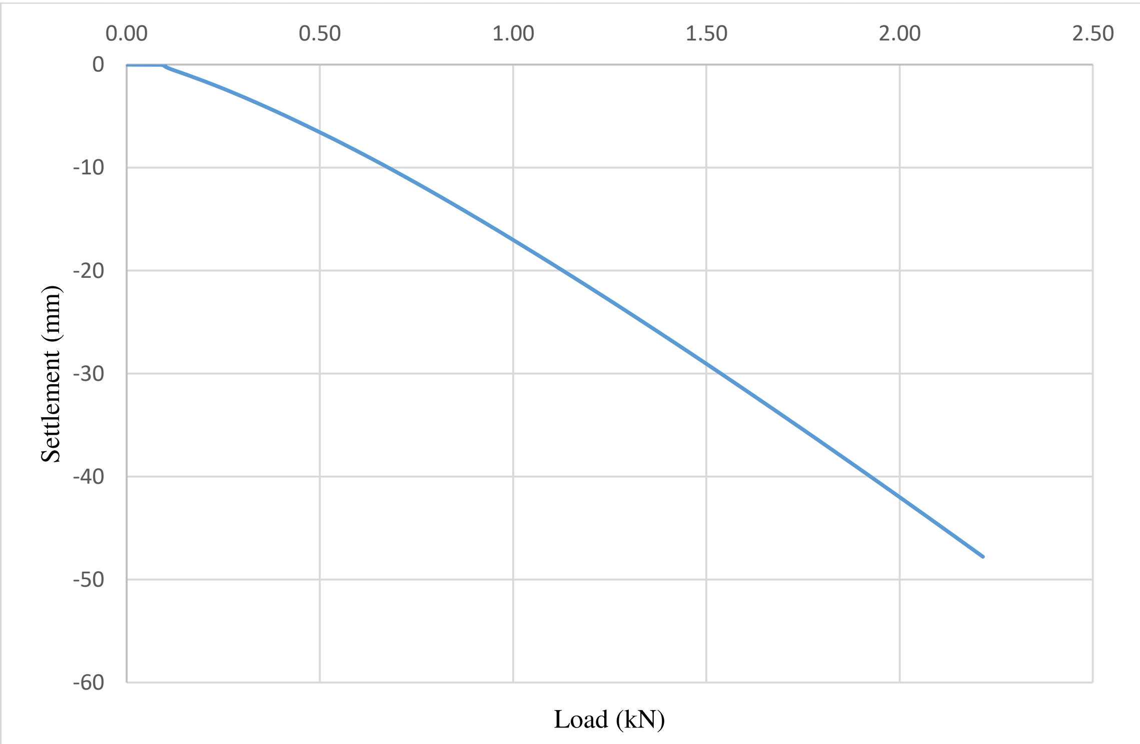


Figure 4.17. Load-Displacement curve of 5 cm diaphragm model pile by Plaxis for Dr=40%

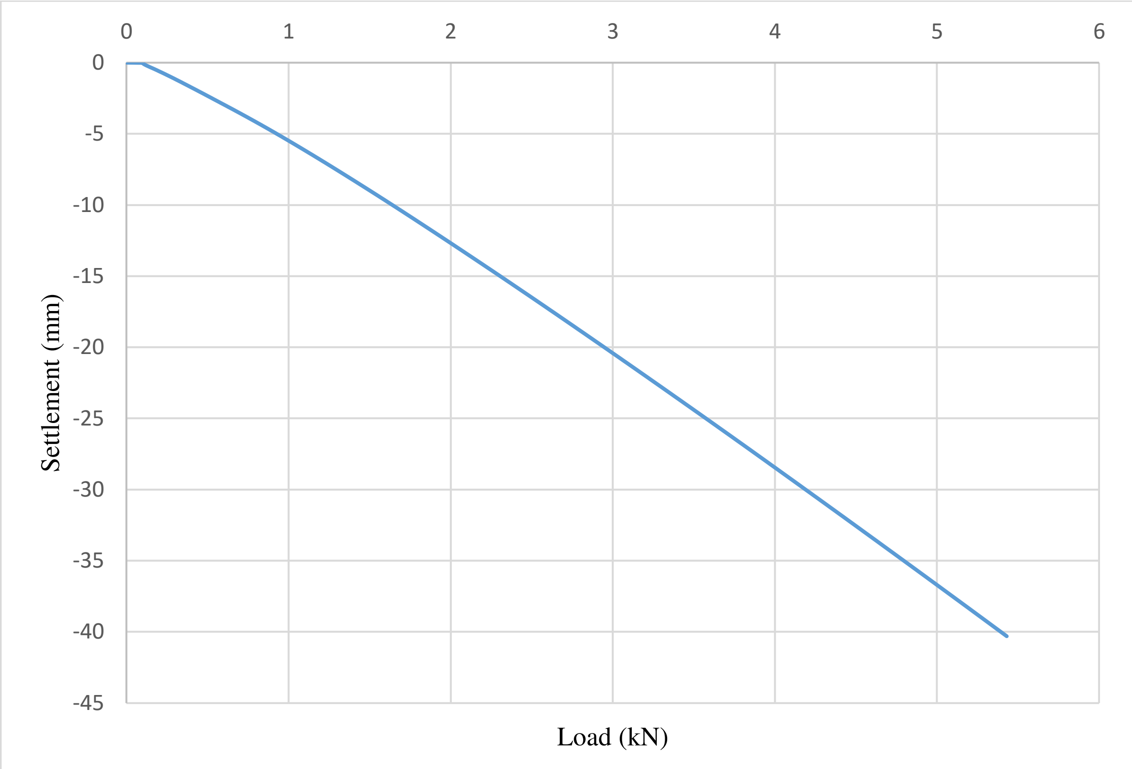


Figure 4.18. Load-Displacement curve of 5 cm diaphragm model pile by Plaxis for Dr=65%

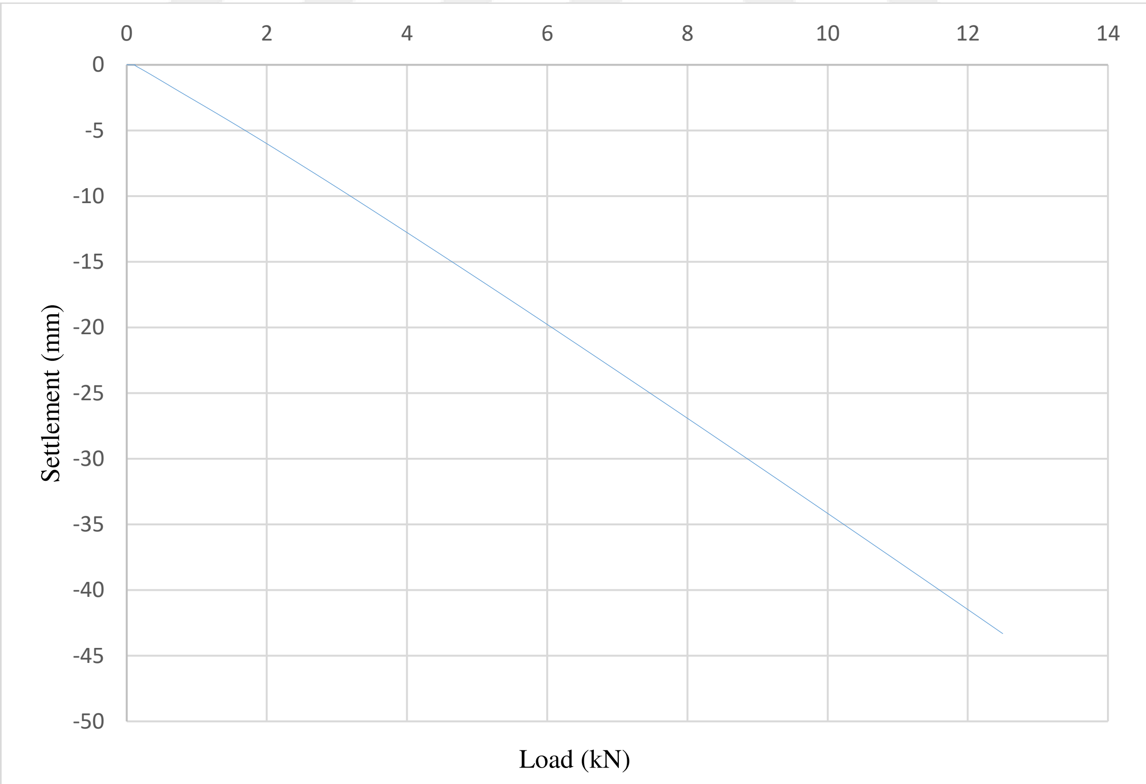


Figure 4.19. Load-Displacement curve of 5 cm diaphragm model pile by Plaxis for Dr=90%

5. CONCLUSIONS

The experimental and finite element studies were conducted to study the behavior of the single pile under axial load in four different relative densities (Dr) 10%, 40%, 65% and 90%. Two steel piles with two different diaphragm height (2.5 and 5cm), 60 cm length and 5 cm diameter have been used for this study. Friction angle between sand and pile and the load-settlement behavior of the pile were used to determine the bearing capacity of the model piles. Also the finite element program Plaxis 3D have been performed to study the behavior of the model piles under axial loading. The major conclusions from this study are:

- The bearing capacity of single piles increased with increasing relative density of the sand in non-linear relationship. It was noticed that the settlement corresponding to the ultimate bearing capacity decreases with increasing relative density.
- As the height of the pile diaphragm increases, bearing capacity of the pile increases
- From the finite element analysis, the bearing capacity of the single piles increases with increasing relative density of the sand. The parameters as the modulus of elasticity and the unit weight of the soil influences the load-settlement behavior of the single piles.
- The load-settlement curves obtained from the Plaxis 3D program and the experimental load-settlement curves are well in agreement.

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APPENDICES

APPENDIX- 1: EXPERIMENTAL TIME-LOAD AND TIME-DISPLACEMENT CURVES OF TOTAL BEARING CAPACITY OF 2.5 CM DIAPHRAGM MODEL PILE

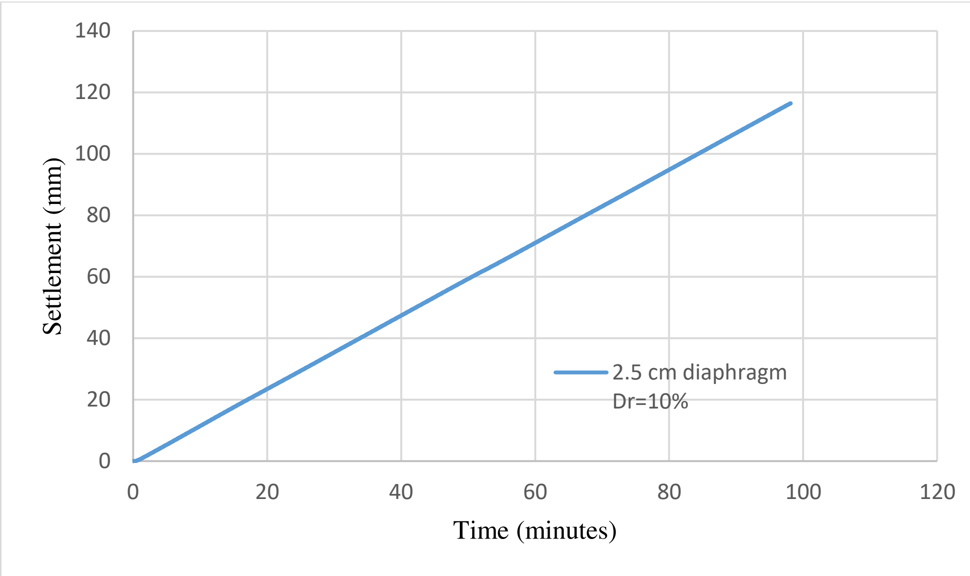

Figure A1. 1. Time-Settlement behavior curve for 2.5 cm diaphragm at Dr=10%.

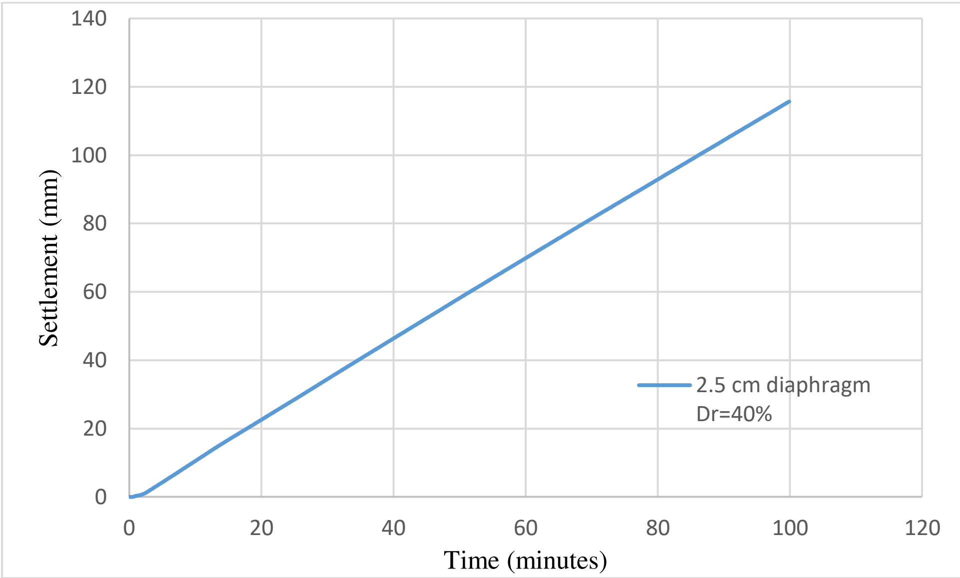

Figure A1. 2. Time-Settlement behavior curve for 2.5 cm diaphragm at Dr=40%.

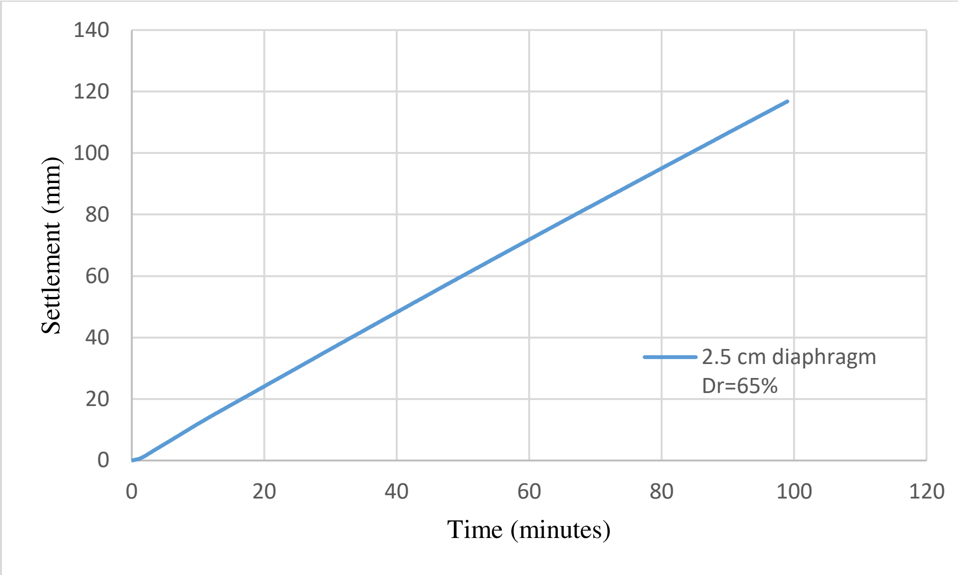

Figure A1. 3. Time-Settlement behavior curve for 2.5 cm diaphragm at Dr=65%.

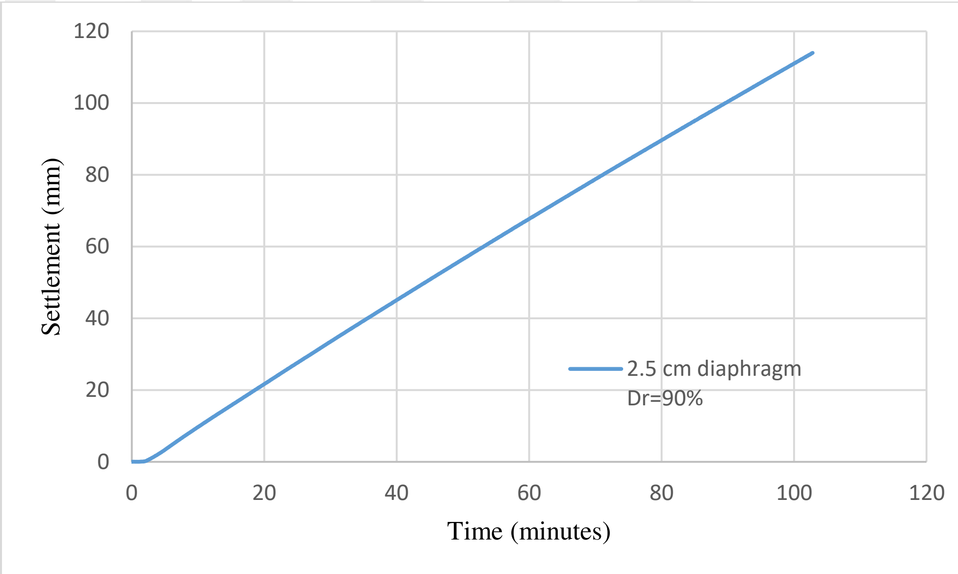

Figure A1. 4. Time-Settlement behavior curve for 2.5 cm diaphragm at Dr=90%.

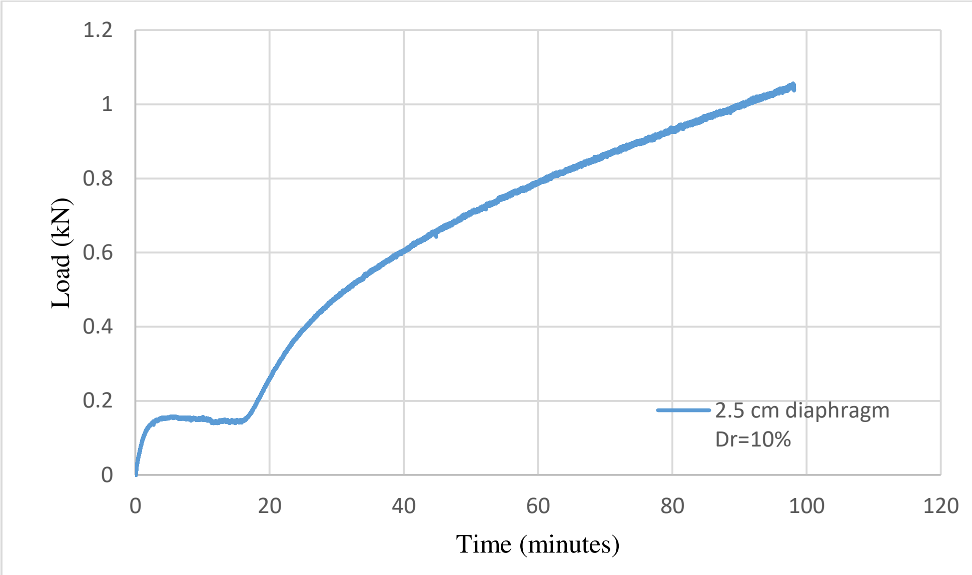

Figure A1. 5. Time-Load behavior curve for 2.5 cm diaphragm at Dr=10%

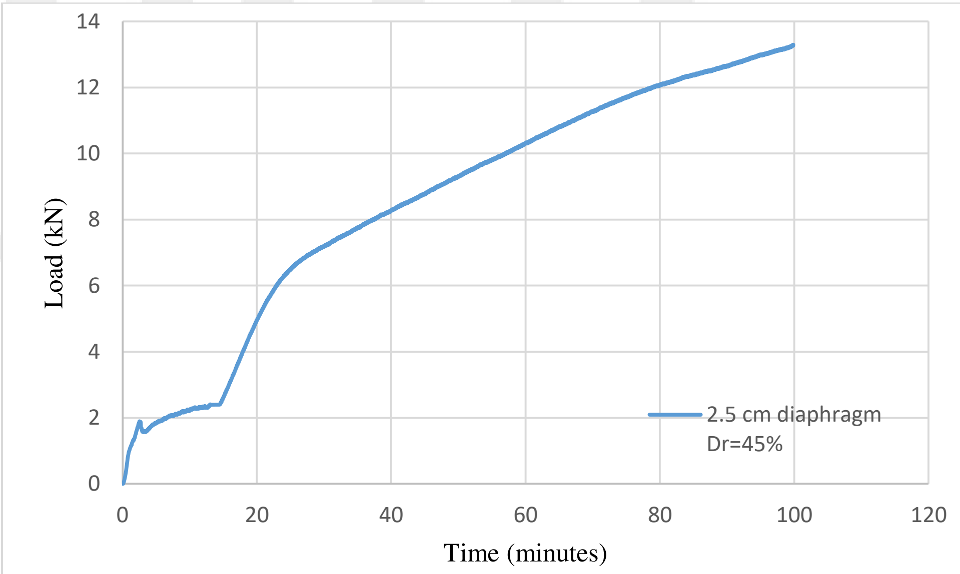

Figure A1. 6. Time-Load behavior curve for 2.5 cm diaphragm at Dr=40%

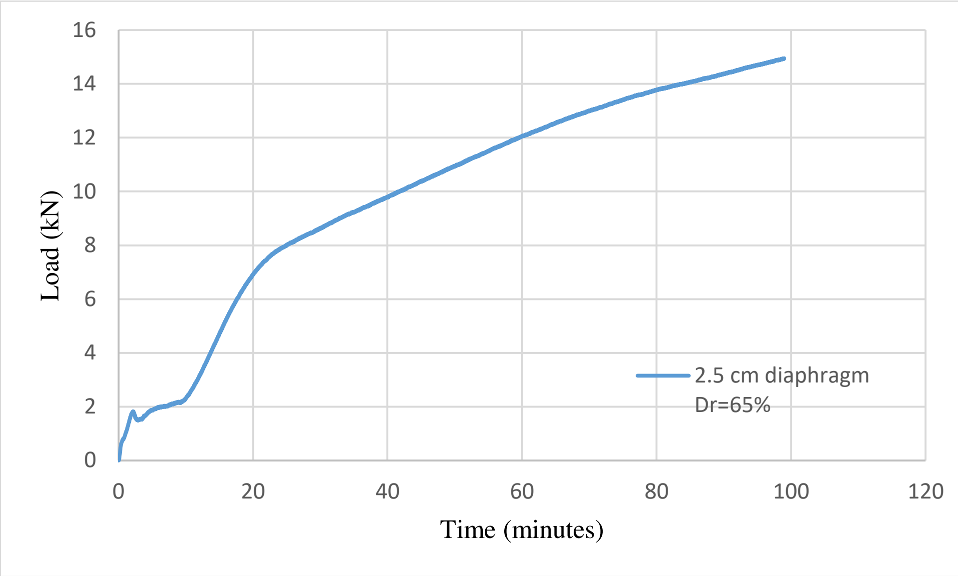

Figure A1. 7. Time-Load behavior curve for 2.5 cm diaphragm at Dr=65%

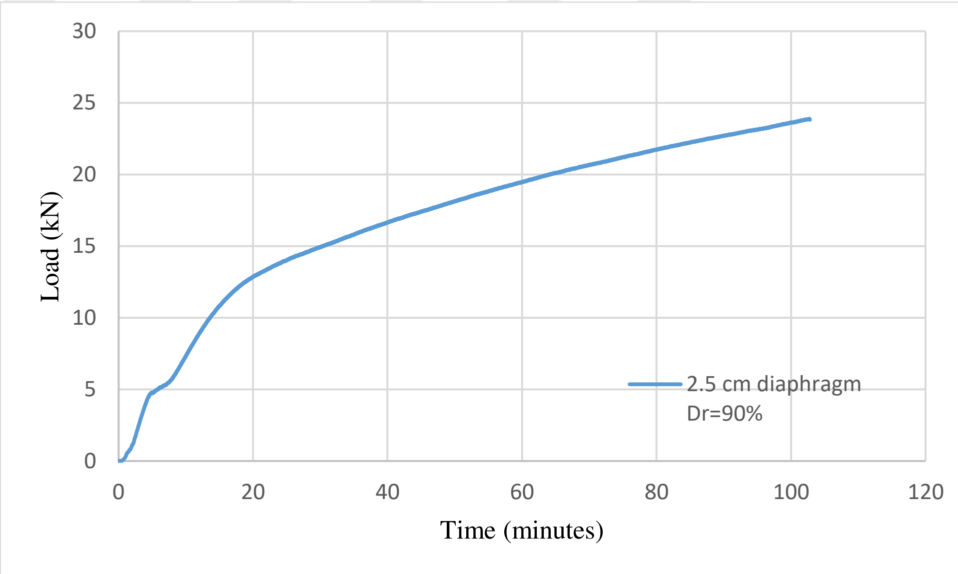

Figure A1. 8. Time-Load behavior curve for 2.5 cm diaphragm at Dr=90%

APPENDIX- 2: EXPERIMENTAL TIME-LOAD AND TIME-DISPLACEMENT CURVES OF TOTAL BEARING CAPACITY OF 5 CM DIAPHRAGM MODEL PILE

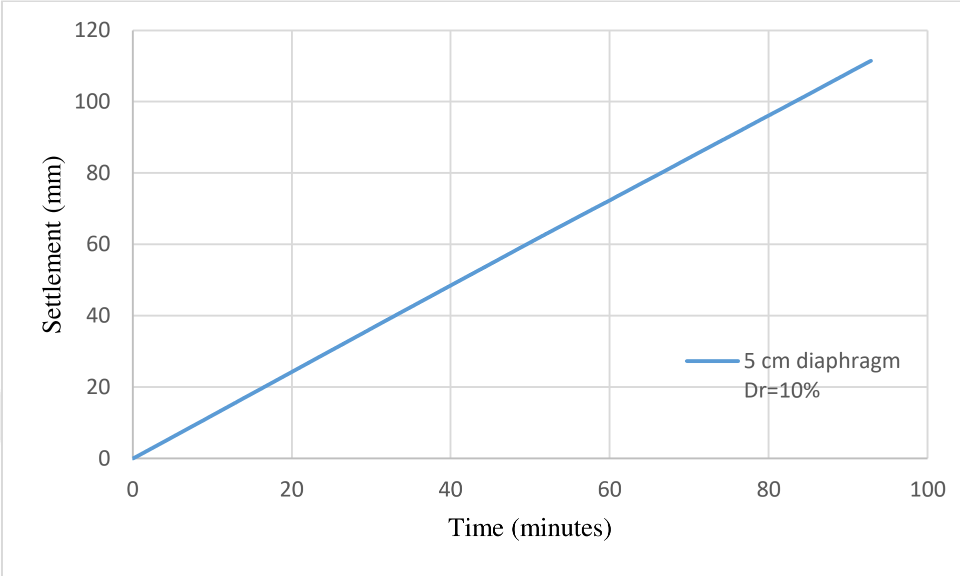

Figure A2. 1. Time-Settlement behavior curve for 5 cm diaphragm at Dr=10%.

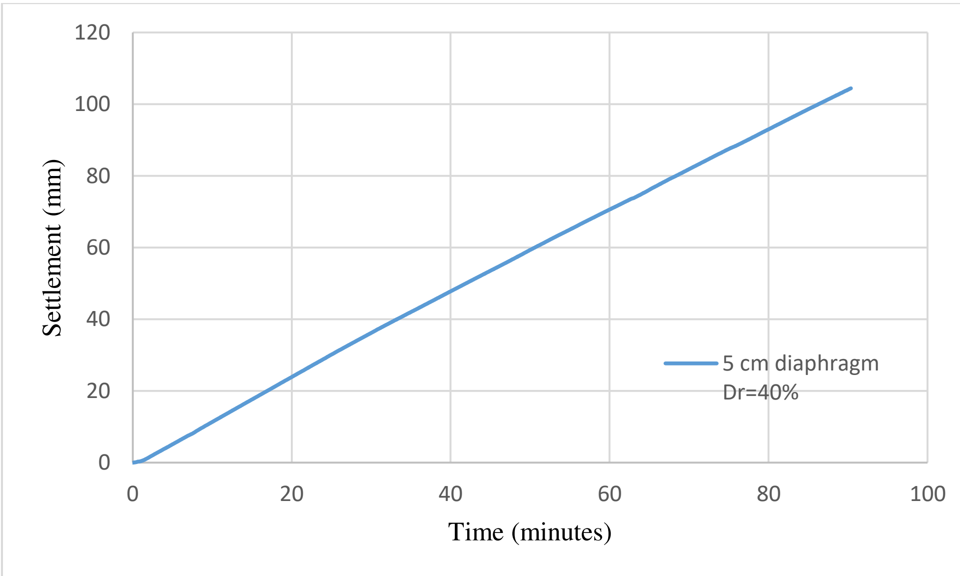

Figure A2. 2. Time-Settlement behavior curve for 5 cm diaphragm at Dr=40%.

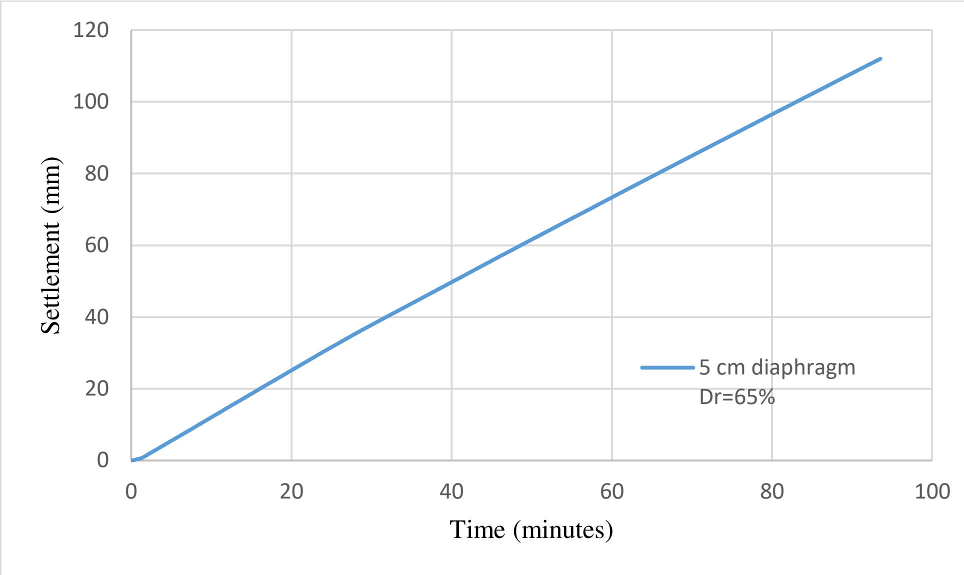

Figure A2. 3. Time-Settlement behavior curve for 5 cm diaphragm at Dr=65%.

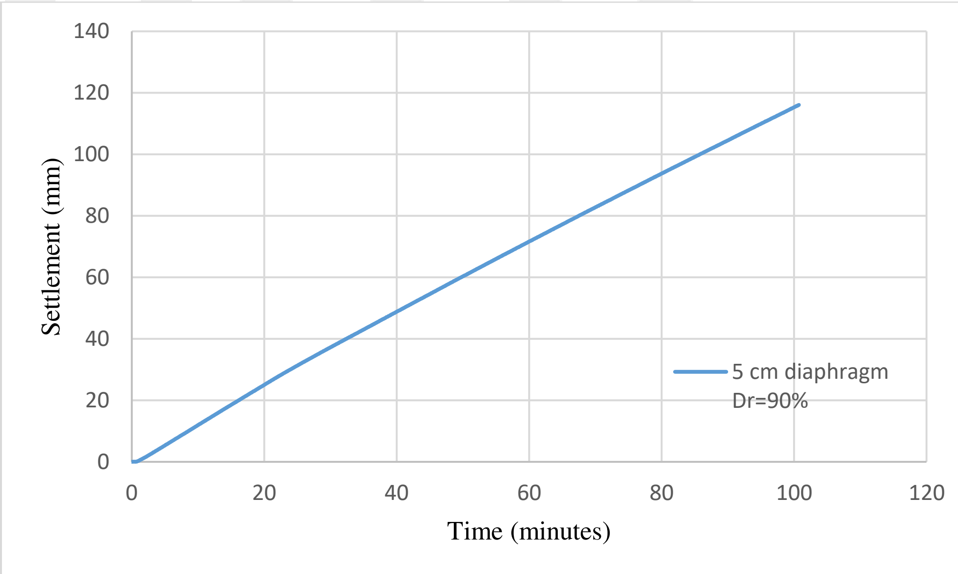

Figure A2. 4. Time-Settlement behavior curve for 5 cm diaphragm at Dr=90%.

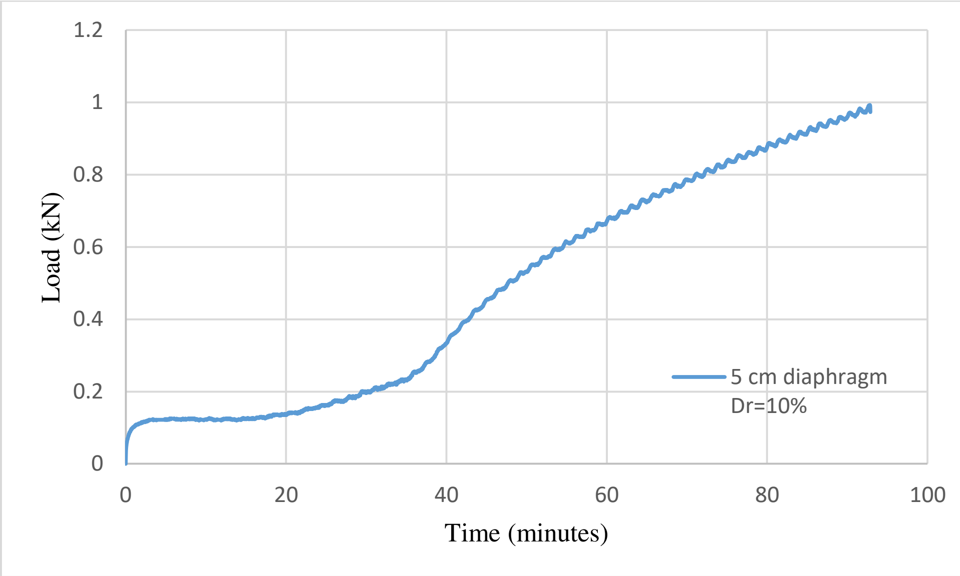

Figure A2. 5. Time-Load behavior curve for 5 cm diaphragm at Dr=10%

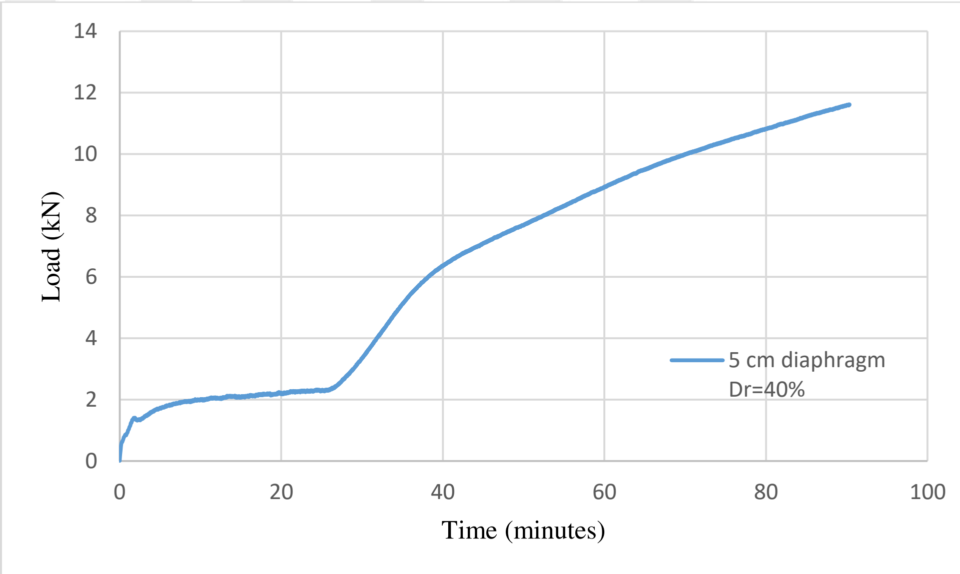

Figure A2. 6. Time-Load behavior curve for 5 cm diaphragm at Dr=40%

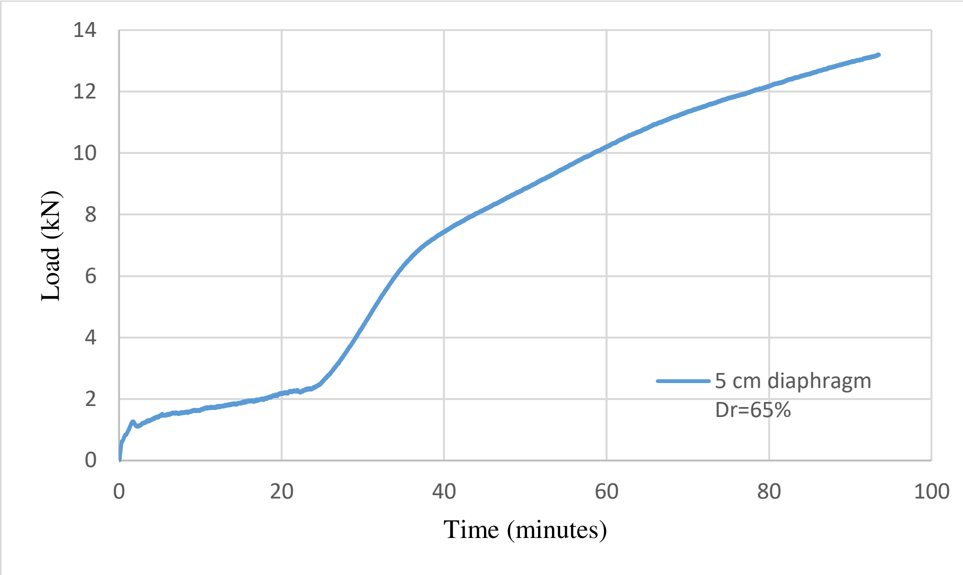

Figure A2. 7. Time-Load behavior curve for 5 cm diaphragm at Dr=65%

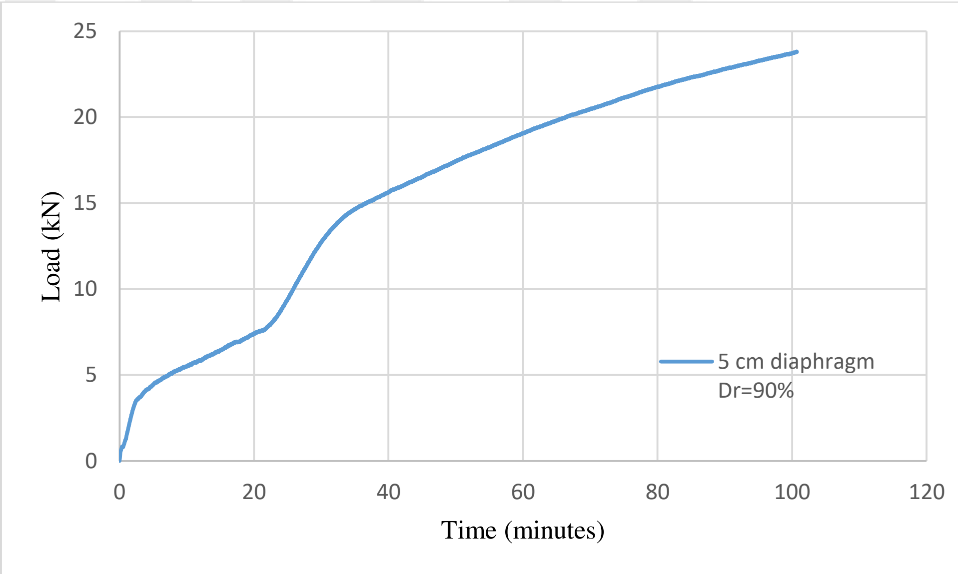

Figure A2. 8. Time-Load behavior curve for 5 cm diaphragm at Dr=90%

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