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Analytical and numerical analysis of laterally loaded pile group-cap foundation system

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ANALYTICAL AND NUMERICAL ANALYSIS OF LATERALLY LOADED PILE GROUP-CAP FOUNDATION SYSTEM

by

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A THESIS

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ABSTRACT
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Title Analytical and Numerical Analysis of Laterally Loaded Pile Group-Cap Foundation System

The lateral resistance taken by the pile cap is very important and cannot be neglected because of its large contribution to the lateral resistance of the pile group-cap foundation system. Analytical and numerical analysis have been performed in this thesis in order to determine the lateral resistance taken by the pile cap in sand and clay soils for free and fixed head pile-cap connections. The results from the analytical Strain Wedge Model and numerical finite element program Plaxis 3D Foundation have been compared with field and lab test results. It was found that the lateral resistance of the pile cap could reach up to 50 percent of applied lateral loads for clay and approximately 40 percent for sand. Also, the pile cap in the pile group-cap foundation system with a free head pile-cap connection is absorbing more loads at the same deflection compared to the pile group-cap foundation system with a fixed head pile-cap connection.

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To all of you,

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Background

It is a well known fact that soil's strength increases with the depth below ground surface. Due to this fact, engineers realized that the deep foundations would be the best fit when designing a structure that will put a large amount of load on the soil underneath. One type of deep foundations is pile group-cap system, which is a group of piles that are connected with a massive concrete pile cap. Most of the research in the past has been done on the vertical load carried by the pile group-cap system because it is the most dominant type of load taken by this type of deep foundation. On the other hand, lateral load (such as wind load, earthquake load, wave load, structural expansion, etc.) taken by the pile cap has been usually neglected. In order to safely design a foundation, lateral load taken by the pile group-cap foundation system must be incorporated. By performing a wide literature search, it is noticed that the pile cap lateral resistance in some cases is as big as a half of the total load taken by the pile group-cap foundation system.

1.2 Objectives

One objective of this thesis is to develop an analytical model that determines pile cap lateral resistance in the pile group when loaded laterally, including the effect of pile group-cap connection, soil stiffness and lateral loading magnitudes of the pile group-cap system in sand and clay. Another aim of this thesis is to compare results predicted in an analytical analysis using the Strain Wedge Model program and numerical analysis program Plaxis 3D Foundation, in order to get a better understanding of the kind of approach for solving problem each program is using.

1.3 Scope of research

In addition to the introduction chapter, Chapter 2 presents extensive literature research collected on analytical and numerical analysis of the pile group-cap system. Literature that was collected had publication dates ranging from 1970 to present. Parameters that could affect lateral resistance of the pile cap, such as pile spacing, depth of the pile cap embedded into the ground, type of pile head fixity, etc., have been covered and conclusions have been summarized.

Chapter 3 contains numerical analysis and provides detailed information regarding finite element program methods for solving problems. Plaxis 3D Foundation is a finite element program used in this study and its two material models, Mohr-Coulomb Model and Linear Elastic Model have been described in detail. Also, for the purpose of validation, two tests conducted in the past have been used to validate results predicted in Plaxis 3D Foundation. The first test is a laterally loaded single pile, while the second test is a 4x4 pile group-cap system that is also being loaded laterally.

Chapter 4 presents the analytical model that is developed to deal with a pile group-cap system under lateral loads. Described in this chapter are definitions and principals that the analytical model is employed in the Strain Wedge Model computer program.

Chapter 5 presents a comparison of results obtained from the Strain Wedge Model for sand and clay. Pile cap with free and fixed head pile-cap connection have been laterally loaded in sand and clay, and results have been recorded. Also, percentages of lateral resistance taken by the pile cap in respect to total lateral load applied have been recorded.

Chapter 6 displays results from some of the most recent case studies (experimental data) performed on a pile group-cap system that is laterally loaded. These results are compared with the Strain Wedge Model analysis as well as finite element program Plaxis 3D Foundation.

Chapter 7 presents a summary and conclusion of the results found in this research.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In the past half century, deep foundations have been one of the most interesting topics when it comes to geotechnical research. A great amount of research has been conducted on vertical load carried by the piles and pile caps, but most of the research has neglected the lateral loads taken by the pile cap. In order to fully understand how piles and pile caps work together, lateral loads carried by piles and pile caps need to be understood. A pile cap is a concrete pad that connects steel, concrete or timber piles together. By doing so, resistance taken by the pile group-cap foundation system is distributed between piles and the pile cap. In the past three decades, researchers have realized the lateral load on pile caps cannot be neglected and a more in depth look into lateral loading was started. Some of this research has been conducted on a full scale lateral load pile group-cap foundation system (Mokwa and Duncan 2001, Kim and Singh 1974, Zafir and Vanderpool 1998, Kim et al.1979, etc.). Other research has been conducted only in laboratories using small scale pile group-cap systems (McVay et al. 2000; El-Garhy at al. 2009; etc.). Furthermore, Dewi and Tjie-Liong (2011), Nath et al. (2011), and others have performed a series of tests using finite element programs to determine the lateral resistance that is taken by the cap.

2.2 Previously completed tests

Beatty (1970) performed a lateral load test on a six piles group connected with a pile cap. He was hired by the Ford Motor Company to analyze piles and pile groups that can take high lateral loads due to poor soil conditions and high water table at the project site in Dearland, Michigan. Analyzed piles, in the pile group, were made out of two parts. The first part of the pile was at the bottom spanning 7.9 meters. The outside diameter for this portion of the pile was 0.273 meters with a 7 millimeters wall thickness. The second part of the pile came on top of the first part with a length of 17.4 meters. In order to balance the pile, step-taper shells with a diameter of 0.38 meters were manually driven into the ground. After that, concrete with 28-day strengths of 20700 KN/m^2 was poured in. Piles were reinforced with six No.8 bars that were 8.5 meters long. Applied cyclic loads did not exceed a maximum load of 1075 KN or 180 KN load per pile. Beatty (1970) found that the maximum gross deflection for this pile cap was approximately 1.24 millimeters while the net deflection was around 0.5 millimeters due to elastic recovery of the pile group. This can be seen in the Figure 2.1 below.

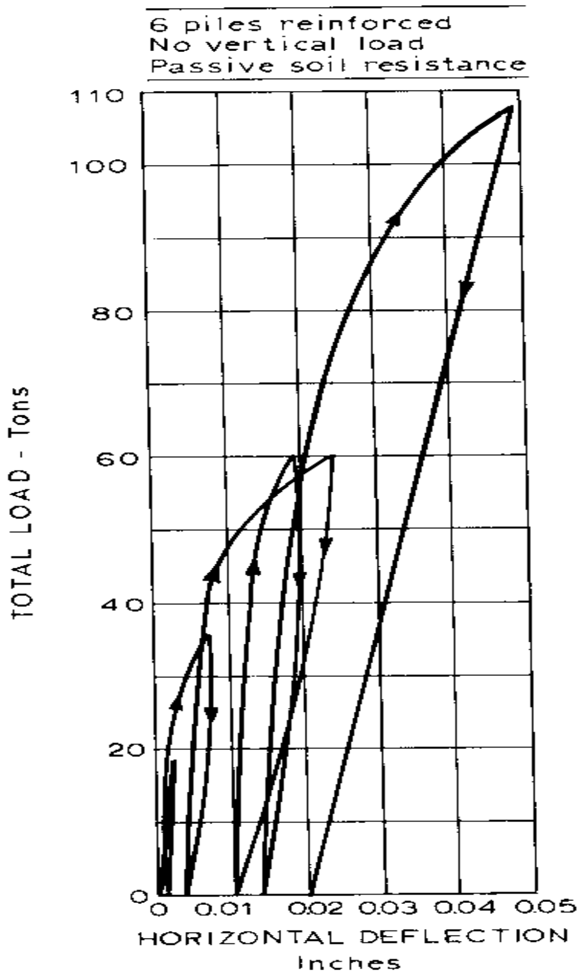


Figure 2.1 Total load versus pile cap deflection (Beatty 1970)

After comparing this test with the pile group without the pile cap, Beatty (1970) came to conclusion that the pile cap substantially impacts total pile group deflection. Also, it was noted that in order to better understand the relationship of piles and the pile cap, additional research is needed. This was one of the leading lateral loading tests performed on the pile group with a pile cap. The disadvantage of this test was that only one test was performed on one pile group.

McVay et al. (2000), just like Barton (1984) and Adachi et al. (1994) performed a centrifuge test. This test was conducted at the University of Florida on model piles in the laboratory. A detailed explanation of the centrifuge test is provided by Schofield (1980). McVay et al. (2000) tested 3x3 and 4x4 pile groups, in two types of soil: loose and medium dense sand. The length of the pile under the pile cap was 9.7 meters with a 1.2 meter thick pile cap. The lateral load applied to the pile cap was at the 0.55 meters beneath ground surface. The test setup can be seen in Figure 2.2 below.

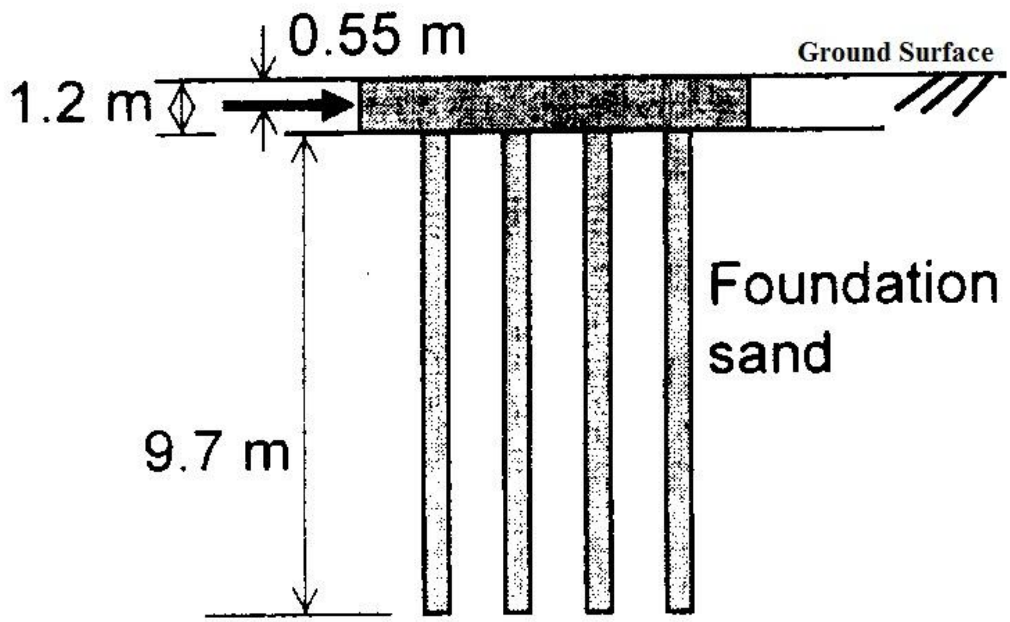


Figure 2.2 Pile group dimensions (McVay et al. 2000)

In most cases that have been conducted before, deflections are usually small. With this being said, McVay et al. (2000) performed four different tests where he moved the location of the pile caps in order to see if it was going to affect pile cap deflection. As expected, the deflection decreased as the pile cap was lowered down into sand. It was also noticed that the lateral resistance of studied groups increased as they were lowered down. The problem with using model testing is not as accurate as testing performed in the

field on full scale piles. The biggest advantage of using small scale piles in the laboratory is the ability to lower pile groups into the ground without changing any soil, densities, or group layout properties. By using a centrifuge model test by McVay et al. (2000), a significant cost savings can be noticed compared to setting up this test in the field. Results that were conducted in this test were double checked in the beam on elastic foundation program FLPIER (Hoit et al. 1997), which uses previously formulated p-y curves. As shown in Figure 2.3 below, results from the model test and FLPIER (Hoit et al. 1997) results are almost identical.

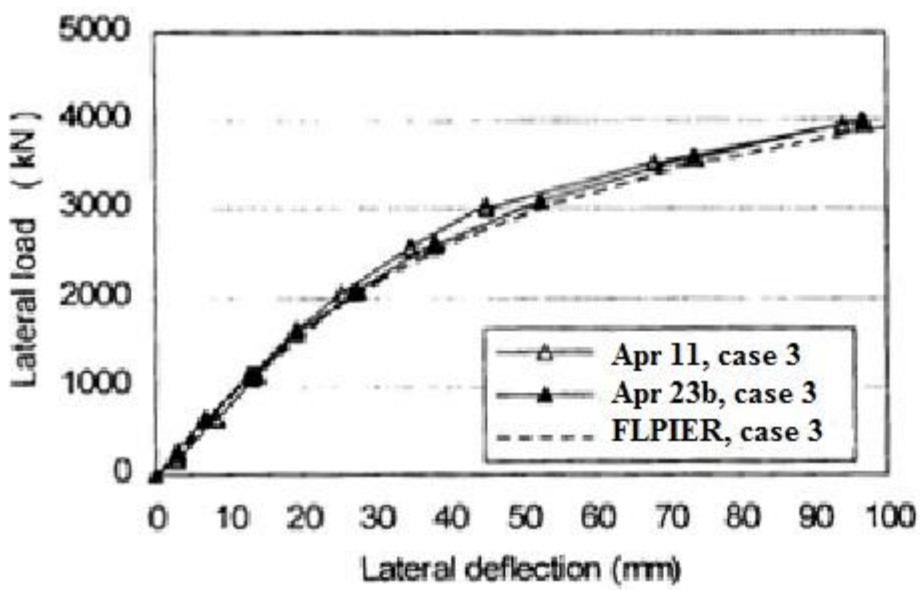


Figure 2.3 Measure and predicted lateral load versus lateral deflection for 4x4 pile group (McVay et al. 2000)

Mokwa and Duncan (2001) performed a series of tests to get a better understanding of the lateral load resistance of pile caps. These tests were performed in 1998 at Virginia Polytechnic Institute close to Blacksburg, Virginia. Three different 2x2 pile groups were analyzed at the northwest, northeast and southeast corner. All piles

selected were HP 10x42 steel piles. The pile length for the northeast and northwest pile groups were 5.8 meters while the southeast pile length was 3.0 meters. The thickness of the pile cap for the northeast and southeast pile groups were 0.91 meters and for northwest pile group, 0.46 meters. Tests were performed for the undisturbed ground, compacted sand, uncompacted sand, and gravel backfill. A more detailed explanation was provided by Mokwa (1999) in his dissertation. The test setup of this experiment is provided in Figure 2.4 below.

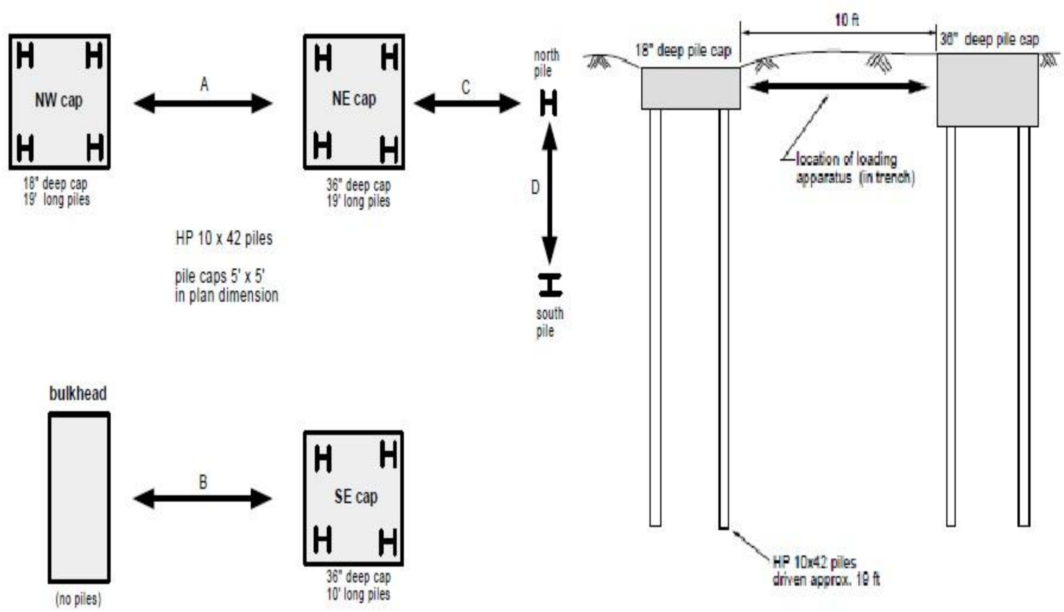


Figure 2.4 Pile group test setup (Mokwa 1999)

After these tests were performed, Mokwa and Duncan (2001) came to the conclusion that for natural soil found at the site, approximately 50 % of the total lateral load applied to the pile group was taken by the pile cap. This observation can be seen in Figure 2.5. Rollins et al. (1997) also found that the lateral load taken by the pile cap is greater than the load taken by the piles.

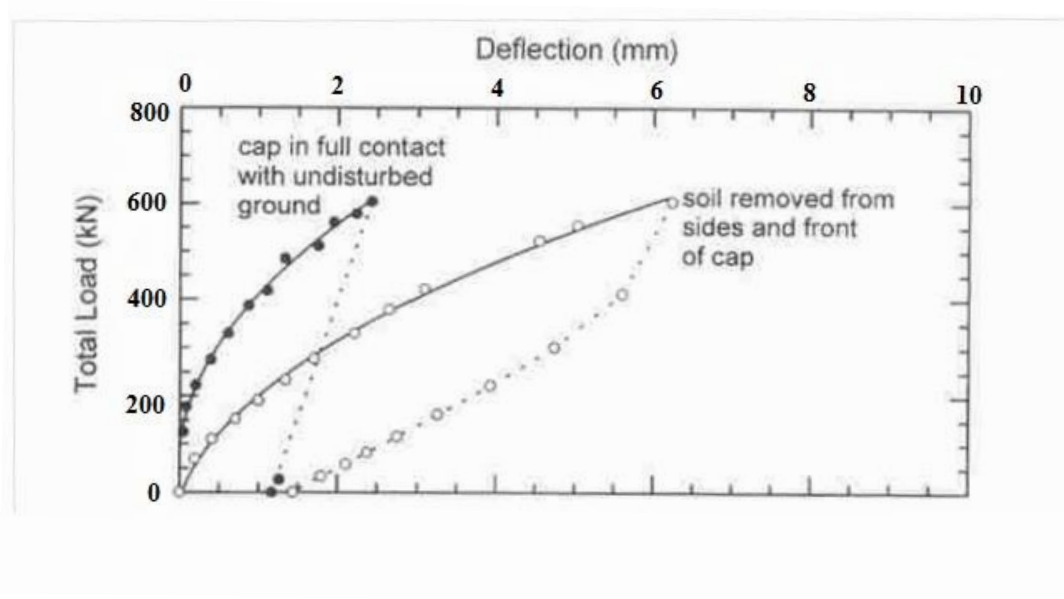


Figure 2.5 Total load applied versus pile cap deflection (Mokwa and Duncan 2001)

This observation was found by other researchers such as Beatty (1970), Rollins and Cole (2006), etc. Also, Mokwa and Duncan (2001) realized that the soil stiffness and depth of the pile cap embedment are two major factors that influence the lateral resistance of the pile group. Tests performed by Mokwa and Duncan (2001) are very helpful because three different pile groups with different pile lengths and cap embedment were tested. Mokwa and Duncan (2001) test was limited to a 2x2 pile group, and it was not compared with larger pile groups such as a 3x3, 4x4, etc.

Shama and Mander (2004) performed a study from both theoretical and experimental aspects on timber pile to concrete pile cap connection under lateral loading. This experiment was performed on two full scale timber piles that were embedded into a concrete pile cap at two different depths. The main objective was to analyze the ductility and strength of the timber pile in connection with a concrete pile cap. Two timber piles with the length above the pile cap of 1828 mm were selected to be analyzed. Pile one was

embedded one time its own diameter. On the other hand, pile two was embedded one and a half times its own diameter. Diameters of piles one and two were 229 mm and 238 mm, respectively. Both piles were embedded into one concrete pile cap with dimensions of 2724 mm long, 914 mm wide and 914 mm thick. Figure 2.6 presents a test setup for the two timber piles embedded into a concrete pile cap.

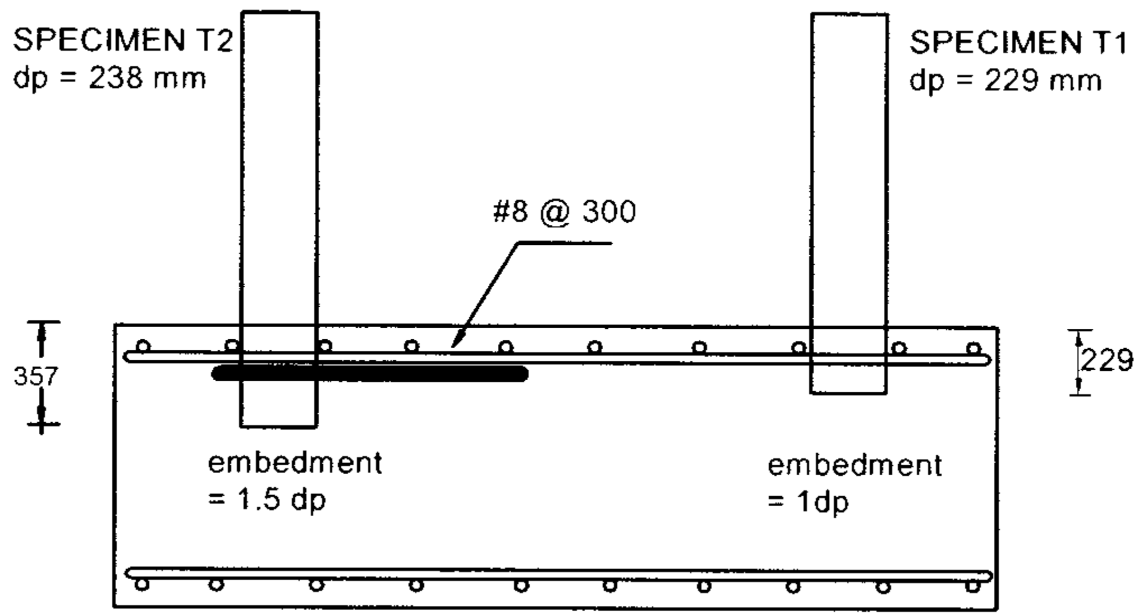


Figure 2.6 Pile group dimensions (Shama and Mander 2004)

Shama and Mander (2004) came to the conclusion that by increasing the embedded length, the connection lateral resistance will also increase. By adjusting the embedment of pile-cap connection, it will prove to have a more ductile connection. According to Shama and Mander (2004), the pile- cap connection with the embedment equal to one size its diameter performed satisfactory. The advantage of this experiment is it was done on a full scale setup. On the other hand, even though two piles were set in the concrete pile cap, only one was analyzed due to the other pile failing.

Rollins and Cole (2006) performed a field test in Salt Lake City on pile to pile cap connection that is laterally loaded by hydraulic jacks. A series of cyclic lateral loads was used on a 4x3 pile group that was connected with a pile cap. Four tests were performed with the pile group in soil, two tests without soil surrounding the pile cap and one test with the pile cap 0.3 meters away from the soil. Figure 2.7 presents the test setup performed by Rollins and Cole (2006).

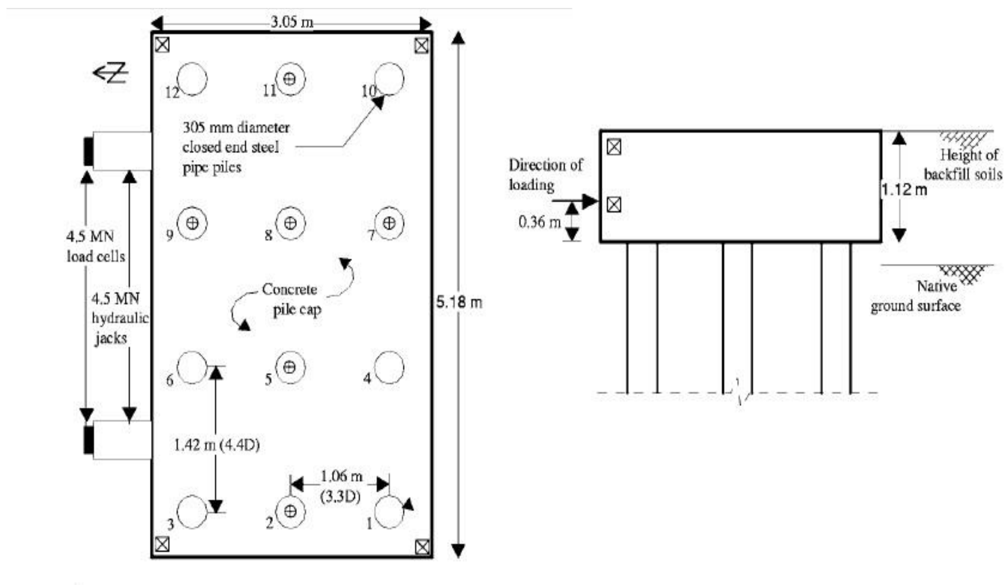


Figure 2.7 Elevation view and overview of the pile group (Rollins and Cole 2006)

It was noticed, after testing was complete, that cap with no soil surrounding the pile group had less lateral resistance then the cap with surrounding soil. Also, two tests that were performed with no soil surrounding the cap and one test with a 0.3 meter trench gave almost the same results. Lastly, Rollins and Cole (2006) realized that approximately 33% - 47 % of the lateral resistance was taking by the cap. This percentage confirms some of the results that were found later by Nath et al. (2011) that the pile cap

contributed about 40% of the lateral resistance. Also Rollins and Sparks (2002) performed a series of full scale tests on a pile cap that was backfilled with gravel. They came to the conclusion that the passive resistance of gravel that was compacted behind a pile cap contributed approximately 40 % of the total resistance. Various downsides of Rollins and Cole (2006) research deal with the pile cap bottom and native ground not being in touch. Due to this phenomenon, adhesion and base friction did not contribute to the lateral resistance of the pile cap. This problem was caused by previous loadings. This testing was helpful because it was done on full scale piles and pile caps.

El-Garhy et al. (2009) performed a series of lateral load pile cap elevation tests on a pile cap in sand. The purpose of this experiment was to show the effect of pile cap elevations on the deflection of the pile cap. The pile cap was placed on five different locations:

1. Pile cap bottom in a contact with a ground surface
2. Pile cap surface in a contact with a ground surface
3. Pile cap bottom buried at the depth of $D_f/t=2$
4. Pile cap bottom buried at the depth of $D_f/t=3.33$
5. Pile cap bottom buried at the depth of $D_f/t=6.67$

Where:

D_f = Depth of pile cap below ground surface

t = Thickness of pile cap

Also, it should be noted that El-Garhy et al. (2009) used three different types of pile groups (2x2, 3x3, 4x4) at pile spacing's 3D and 5D. Piles used for this test were steel hollow piles. El-Garhy et al. (2009) came to same conclusion, similar to McVay et al. (2000) that lateral capacity increases as the depth of the cap increases. This can be seen in Figure 2.8. Also El-Garhy et al. (2009) recognized the same phenomenon for the incensement of the pile spacing but noted that the lateral capacity due to the increase of depth is much larger then due to pile spacing. The advantages of this experiment are the cost and easiness to set up. On the other hand, the disadvantage that can be concluded in this test is the less accuracy of the results due to not using a full scale test.

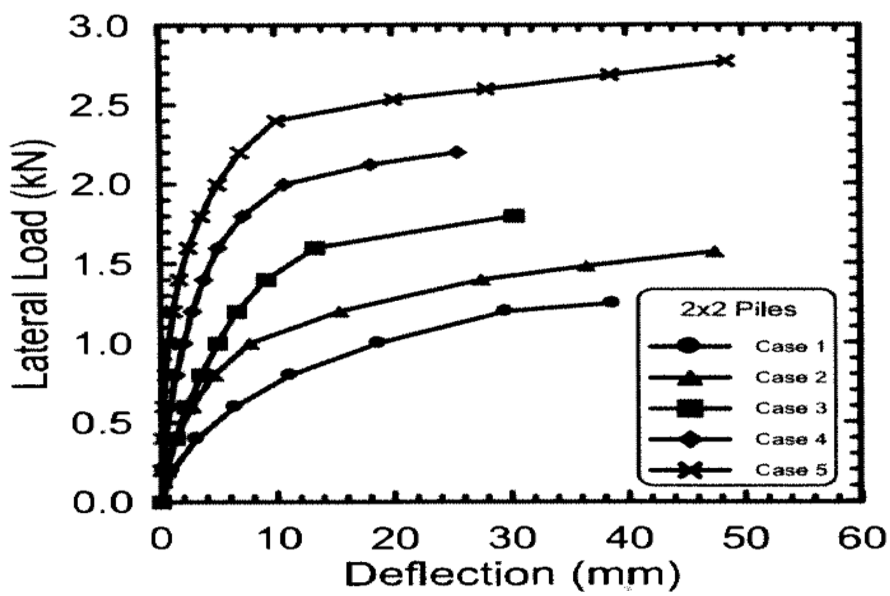


Figure 2.8 Lateral load vs. deflection for 2x2 pile group at different depths
(El-Garhy et al. 2009)

Nath et al. (2011) used finite element program Plaxis 2D to analyze the effect of lateral loading on the pile cap. He wanted to see if lowering the pile cap underneath the surface, pile spacing in the pile group, and increasing the number of piles in a pile group is going to have any effect on lateral resistance of the pile cap. Three different pile groups 2x1, 3x1, 4x1, with pile spacing's of 3D, 5D, 9D (where D is the size of diameter of pile) and elevations of pile cap 0, 0.1 and 0.2 meters below ground were used. Also, the lengths of these piles were 10 and 15 meters. Nath et al. (2011) came up with the following conclusions:

- Lateral resistance of the pile cap increases as the length of the pile increases
- As the spacing increases, lateral resistance of the pile cap increases
- Lateral resistance increases as the number of piles in the group increase

Nath et al. (2011) used only finite element analysis to come to the conclusions listed above. The time spent performing this experiment and cost are the two biggest advantages found. However, using only finite element analysis and not validating results in the field is a disadvantage of this experiment.

PYCAPSI is an analytical analysis EXCEL designed spreadsheet developed by Mokwa and Duncan (2000). The main purpose for developing this spreadsheet is to be able to estimate the pile cap resistance. The resistance of the pile cap provided by PYCAPSI is offered by p-y curves. P-y curves are curves that deal with a force applied to the soil and the deflection of the soil. It should be noted that for the passive earth pressure used in Mokwa and Duncan (2000) study, the log spiral earth pressure theory

was used. This is due to the fact that already existing earth pressure theories such as Coulomb's and Rankine's are satisfactory only for simple states of stress. For designing p-y curves, Mokwa and Duncan (2000) used the same equation used as Duncan and Chang (1970). This equation presents a stress-strain relationship for soil,

$$P = \frac{y}{\frac{1}{Kmax} + Rf * \frac{y}{Pult}} \tag{2.1}$$

Where P represents a load, y represents a deflection, R_f represents failure ratio, K_{max} represents initial stiffness and P_{ult} represents ultimate passive force. It should be noted that the initial stiffness K_{max} have units of force divided by length. Mokwa and Duncan (2000) analyzed a pile group-cap system used by Mokwa and Duncan (2001) with a pile cap embedded 0.91 meters for the following four different types of soil:

1. Natural Soil
2. Compacted Gravel
3. Compacted Sand
4. Loose Sand

Parameters used in this study can be seen in the Figure 2.9 and lateral resistance calculated using PYCAPSI in Figure 2.10 below. In should be noted that PYCAPSI is very useful for designing p-y curves of the embedded pile caps but will need further development.

Parameter	Natural soil	Compacted gravel	Compacted sand	Loose sand
ϕ (deg)	38	50	46	37
δ (deg)	30	25	23	18.5
c (kN/m^2)	48	0	0	0
α	1	0	0	0
γ_m (kN/m^3)	19.3	21.1	16.3	14.5
E_i (kN/m^2)	42,600	36,400	67,000	34,500
ν	0.33	0.30	0.30	0.30

Figure 2.9 Parameters used to calculate P-Y curve (Mokwa and Duncan 2000)

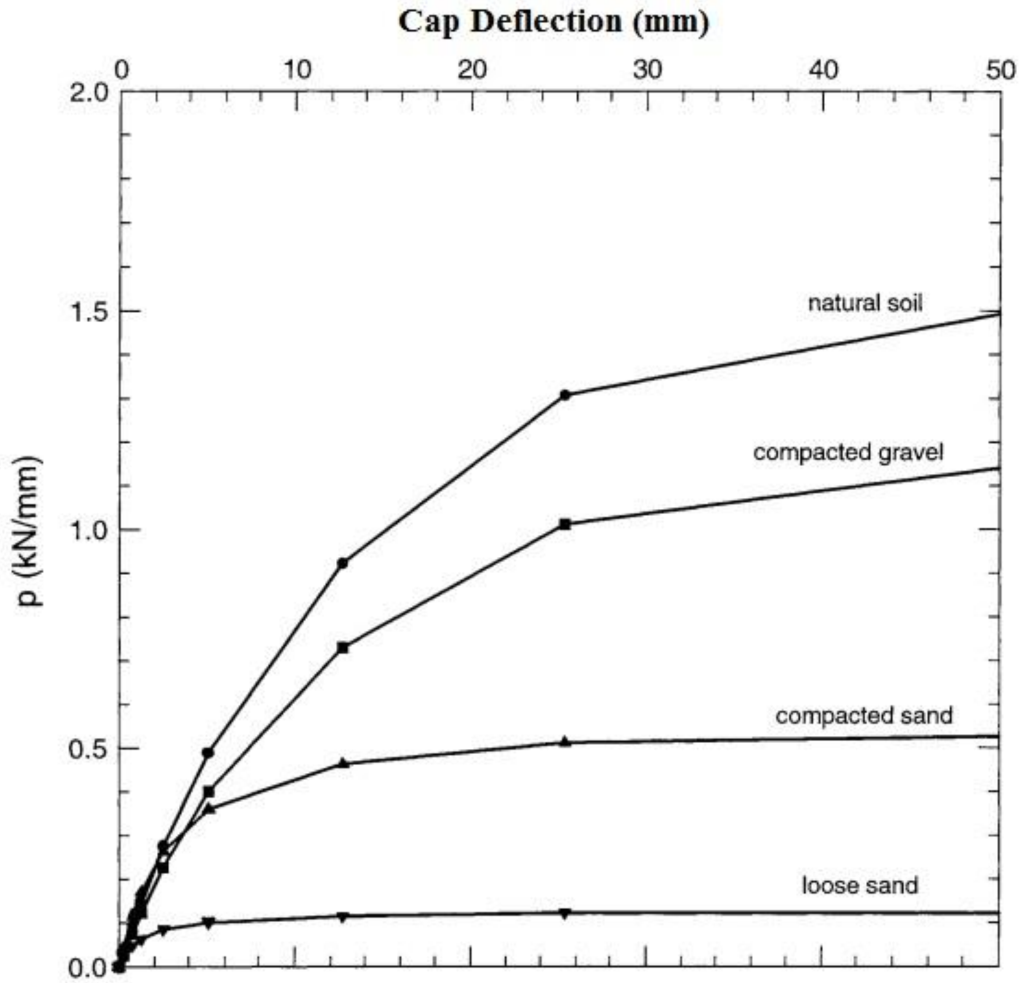


Figure 2.10 P-Y curve calculated in PYCAPSI in four different types of soil (Mokwa and Duncan 2000)

CHAPTER 3

NUMERICAL ANALYSIS

3.1 Introduction

Numerical analysis is a part of mathematics that uses numerical methods to solve problems. Some problems are easy to solve because they have linear elastic behaviors while others that don't have linear elastic behaviors are much more complex and more difficult to solve. In order to solve problems that don't have linear elastic behaviors such as soils, it is required to divide a structure that is being analyzed into a finite number of elements. These elements are connected with nodes. This process is known as finite element method (FEM). Plaxis is one of the finite element method software packages that is being used for two and three dimensional analyses.

3.2 Plaxis

In 1986, Technical University of Delft in Netherland together with Dutch Authorities came up with a program that could deal with axi-symmetric problems and gave it a name of Plaxis. This software has significantly improved over the past two decades and today it is one of the leading finite element method software packages used by geotechnical engineers. Plaxis is a multipurpose program that can analyze stability of soil, deformation, groundwater, heat flow in tunnels, foundations, excavations etc. There

are a couple of versions of Plaxis, such as Plaxis 2D, Plaxis 3D Tunnel, Plaxis 3D Foundation but only Plaxis 3D Foundation was used in this numerical analysis.

3.3 Elements

As it can be seen from the Figure 3.1, 3D finite element mesh of the soil is made of 15 node wedge elements. This 3D finite element mesh is made of 6 node triangle elements in the horizontal direction that are used when designing 2D finite element mesh and 8 node quadrilateral in the vertical direction. It should be noted that 6 node triangle elements are the basis for the floor elements and it has second order interpolation of displacements. Also, 8 node quadrilateral elements are basis for the wall elements and it also has second order interpolation of displacements. In the case when there are no horizontal soil layers, these 15 node wedge elements can be transformed to 13 node pyramid elements or 10 node tetrahedral elements. It can also be noticed from Figure 3.1, a 15 node wedge has six stress points.

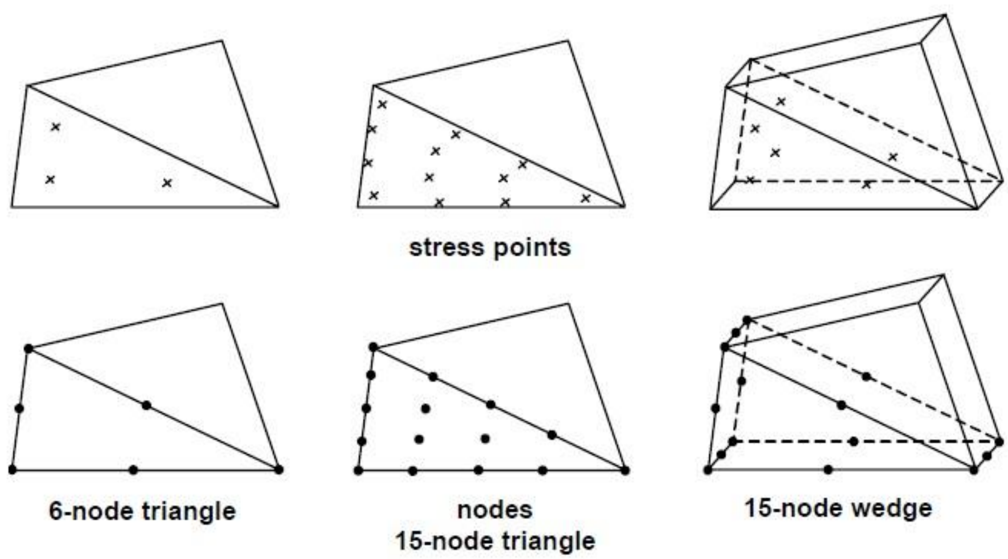


Figure 3.1 Comparison of 2D and 3D soil elements (Brinkgreve 2007)

3.4 Material properties

Soil Properties and Material Properties of structures can be found in Plaxis under the main menu tab. There are seven different modeling elements:

1. Soil and Interfaces
2. Beams
3. Floors
4. Walls
5. Embedded Piles
6. Ground Anchors
7. Springs

Out of the seven elements, the soil property, such as soil and interfaces can be assigned to the soil cluster while other six modeling elements can be assigned to the structure.

3.5 Material models

There are six different types of material models that are offered in Plaxis 3D Foundation. The six material models are: Linear Elastic Model, Mohr-Coulomb Model, Hardening Soil Model, Hardening Soil Model with small-strain stiffness, Soft Soil Creep Model and User Defined Soil Model. Some of these material models are complex and require many input parameters while others, such as Liner Elastic Model only required two input parameters. In this research, only two types of material models were used, which are Mohr-Coulomb Model and Linear Elastic Model.

3.5.1 Mohr-Coulomb model

According to Sejnoha 2009, Mohr-Coulomb failure criterion is made out of three different parts that are put together. These three parts are Mohr's circle, plane strain transformation equation and Coulomb failure criterion. Mohr-Coulomb Model is the most commonly used material model. This is considered to be a first approximation of the soil behavior and it is made of five input parameters: Young's Modulus (E), Poisson's Ratio (υ), Cohesion (c), the Friction Angle (φ) and the Dilatancy Angle (ψ). It should be noted that increments of the Young's modulus and cohesion can be controlled with the respect of depth under advanced parameters. The input of these parameters in Plaxis 3D Foundation can be seen in Figure 3.2.

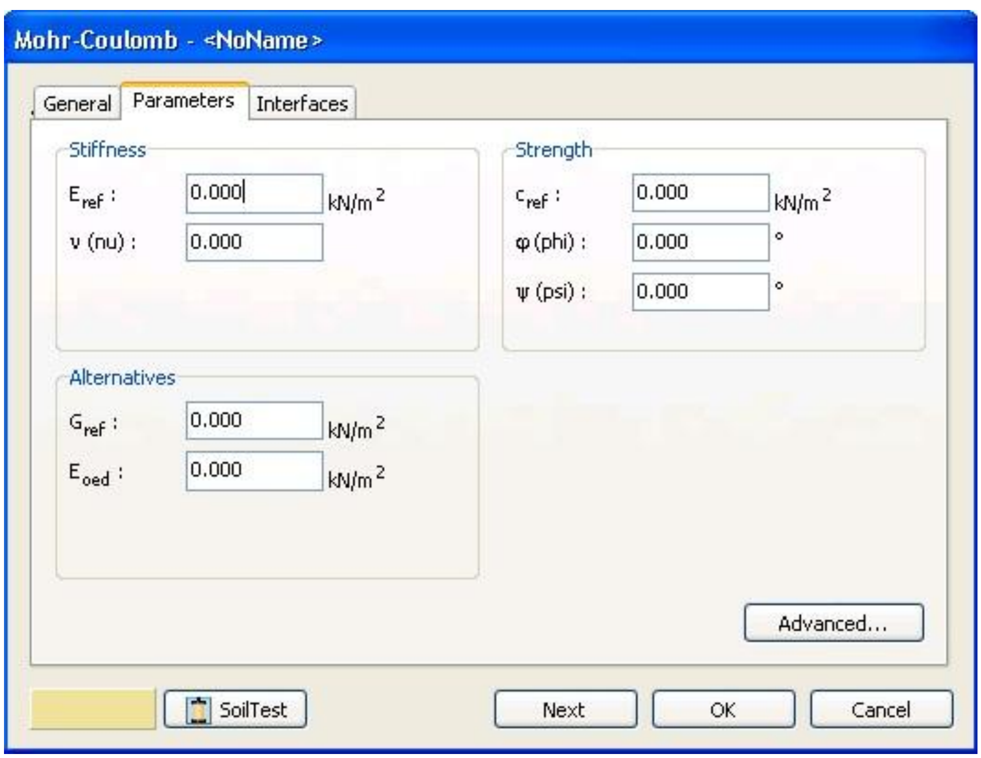


Figure 3.2 Mohr-Coulomb parameters input for Plaxis 3D Foundation (Brinkgreve 2007)

Also, yield function *f* is introduced in this soil model to determine if plasticity is accounted for in the calculation part. This yield function is a function of stress and strain. It is important to know this because plasticity deals with irreversible strains. The Mohr-Coulomb yield conditions are made of six yield functions. All of these six yield functions include cohesion and friction angle and take make up a hexagonal cone in principal stress space and can be seen in Figure 3.3.

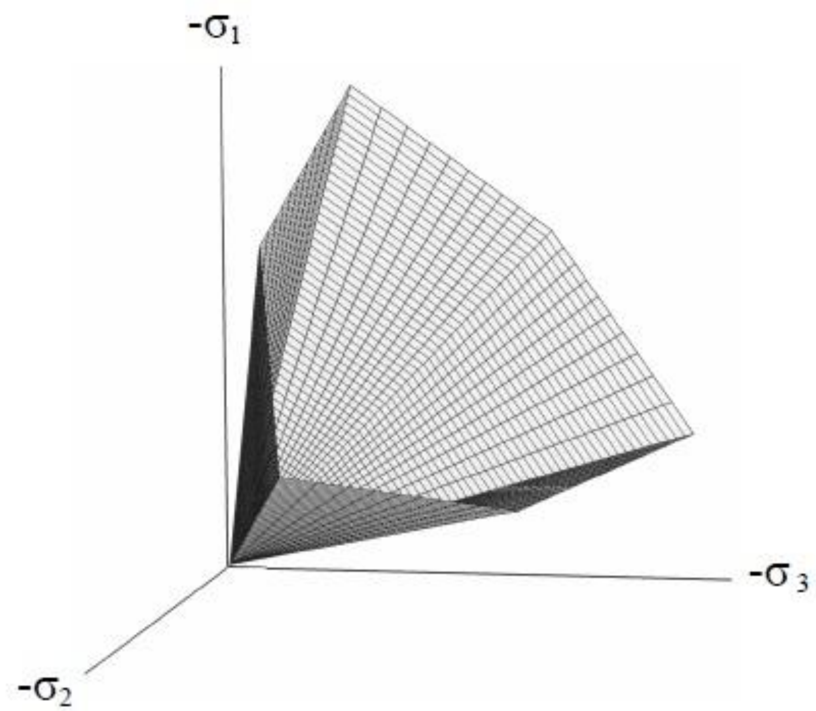


Figure 3.3 Hexagonal cone in principal stress space (Brinkgreve 2007)

Besides yield functions, potential plastic functions are included in Mohr-Coulomb model. Besides having a cohesion and friction angle, the potential plastic function also has the dilatancy angle. The function of the dilatancy angle for the dense soils is to model increments of positive plastic volumetric strain.

3.5.2 Linear elastic model

Linear elastic model is the simplest model that Plaxis 3D Foundation offers. This model is made of only two input parameters, Young's Modulus (E) and Void Ratio (υ). Usually this model is not used for soil but for stiff structures that interact with soil such as piles and pile caps. Just like Mohr-Coulomb model has an option of increasing Young's modulus with depth, so does the linear elastic model in advanced parameter tab. This tab can be seen in Figure 3.4.

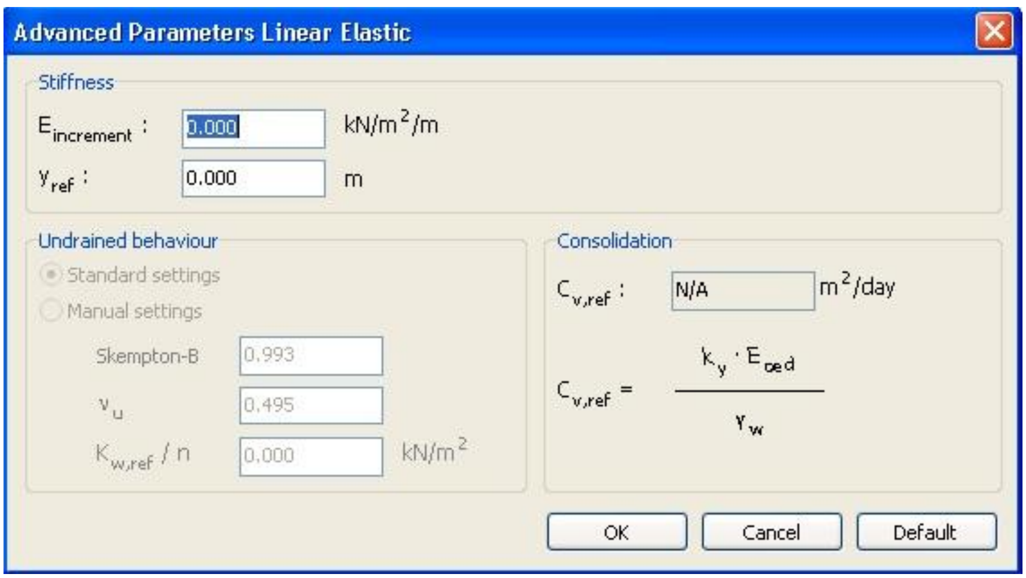


Figure 3.4 Advanced parameters for linear elastic model in Plaxis 3D Foundation (Brinkgreve 2007)

3.6 Mustang Island full scale lateral load test (validation for finite element analysis)

Reese et al. (1974) performed a series of full scale lateral load tests on two piles. These tests were performed at Mustang Island close to Corpus Christi, Texas. Static loads were applied to one of the piles while the other pile was subjected to cyclic loads. Presented below is a verification of field results and results that are projected using finite element program Plaxis 3D Foundation.

3.6.1 Pile configuration and soil properties

As reported by Reese et al. (1974), two piles with an outer diameter of 0.61 meters and length of 21 meters were subject to lateral loading. Lateral loading was applied approximately 0.3 meters above ground surface and it should be noted that the pile heads were free to rotate. The thickness of the pile wall was recorded to be 9.5 millimeters. Table 3.1 presents these input parameters while Figure 3.5 shows a cross section view of the pile that is being laterally loaded.

Table 3.1 Pile properties used in Plaxis 3D Foundation for Mustang Island test

Type of Pile	Pile Diameter (d), m	Pile Length (L), m	Pile Wall Thickness (t), mm	Type of Fixity	Young's Modulus (E), KN/m^2
Steel Pipe Pile	0.61	21	9.5	Free Head	$2x10^8$

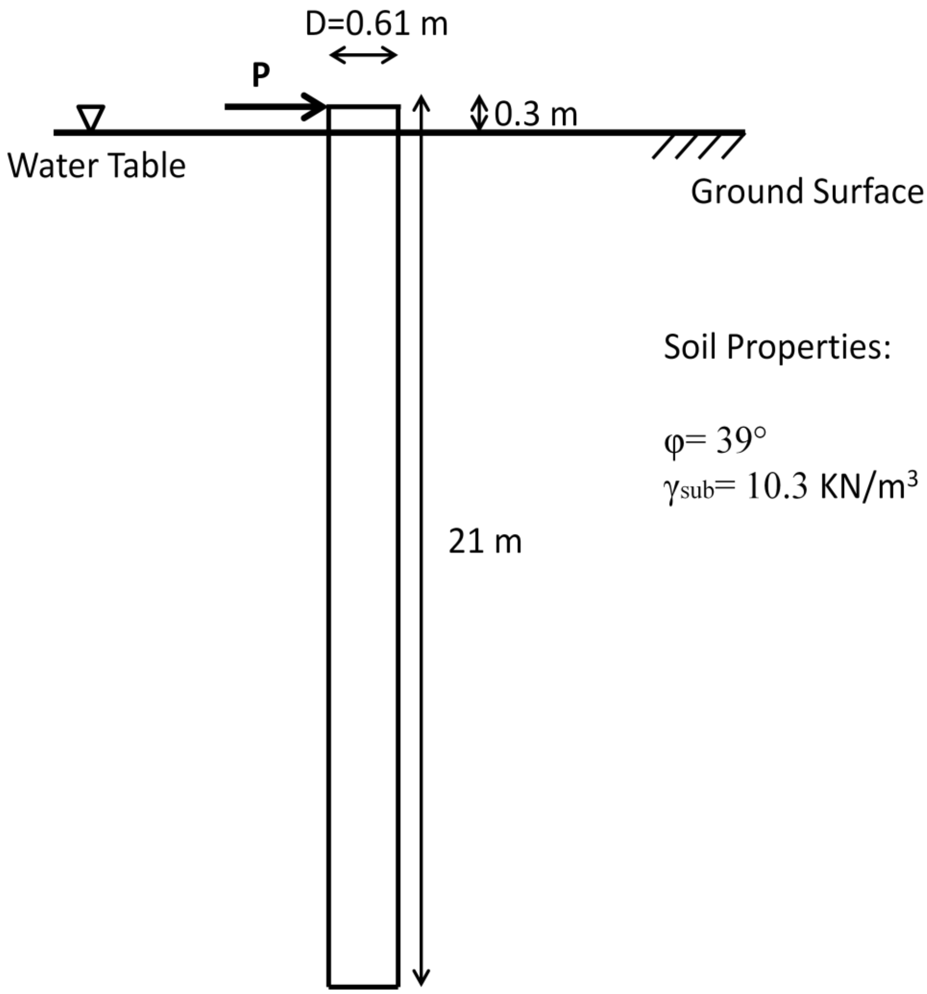


Figure 3.5 Cross sectional overview of the pile used at Mustang Island

Global coarseness selected for horizontal and vertical elements was coarse. Around the pile that was observed, four geometry lines were placed in order to make a rectangle. After forming the rectangle, this cluster was refined to a very fine coarseness. This process was done in order to save the time that Plaxis 3D Foundation takes to finish calculations and also to give as precise results as possible. Many trials were performed to understand how much this cluster should be extended for the single pile. It was concluded that in this case, properties of the rectangle should be approximately 4 meters in length

and 2 meters in width. Figure 3.6 below presents a two dimensional mesh overview of the soil used in this case.

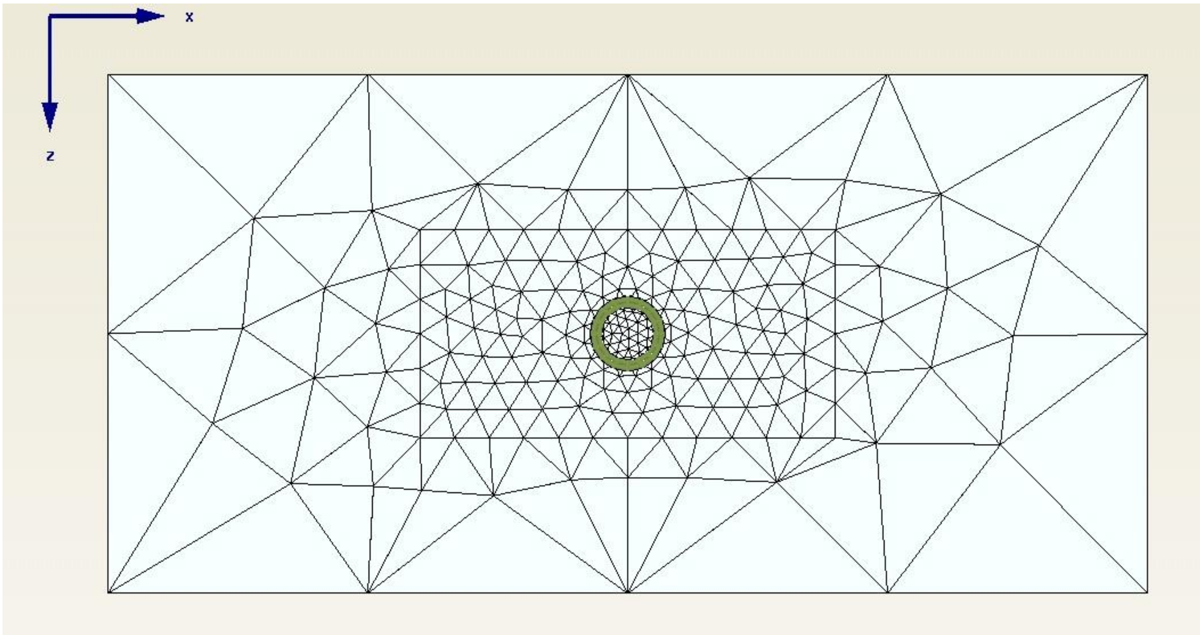


Figure 3.6 Overview of the mesh created in Plaxis 3D Foundation for Mustang Island test

The soil that was found at the site was clean fine sand to silty fine sand with the water table above tested surface. After running a soil sample in the lab, it was found that the friction angle was 39° and submerged unit weight of the sand came to be 10.3 KN/m^3. These parameters can be seen in Table 3.2 below.

Table 3.2 Soil properties used in Plaxis 3D Foundation for Mustang Island test

Type of Soil	**Submerged Unit Weight (γ_{sub}), KN/m^3**	**Friction Angle (φ), deg.**	**Material Model**
Medium Dense Sand	10.3	39	Mohr-Coulomb Model

3.6.2 Field test results compared with Plaxis 3D Foundation

Figure 3.7 presents a comparison of field results calculated by Reese, et al. (1974) and predicted results using finite element program Plaxis 3D Foundation. As it can be seen, results found in the field and finite element results are almost identical. The difference between the acquired and predicted results, at the point where there is the largest difference between them, is approximately 7 percent. This verifies for us that results predicted in Plaxis 3D Foundation are in good agreement with field test results. Also presented in Figure 3.8 is the finite element three dimensional mesh as well as a three dimensional view of the pile used in this case.

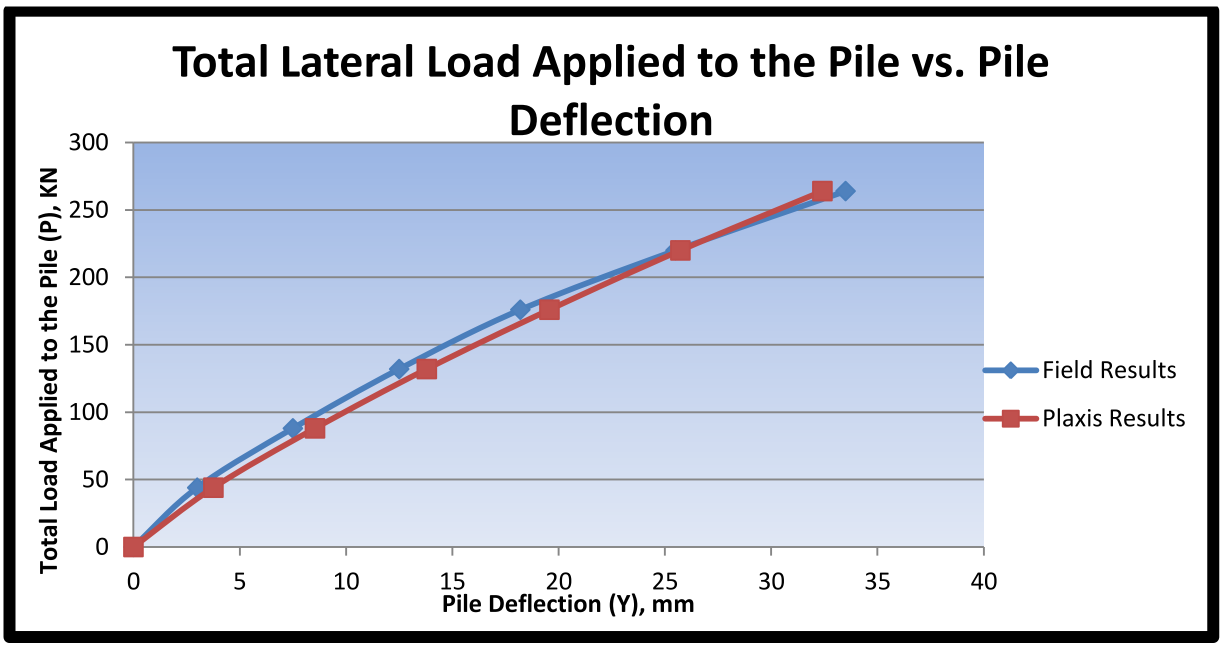


Figure 3.7 Field test results compared with Plaxis 3D Foundation (Reese et al.1974)

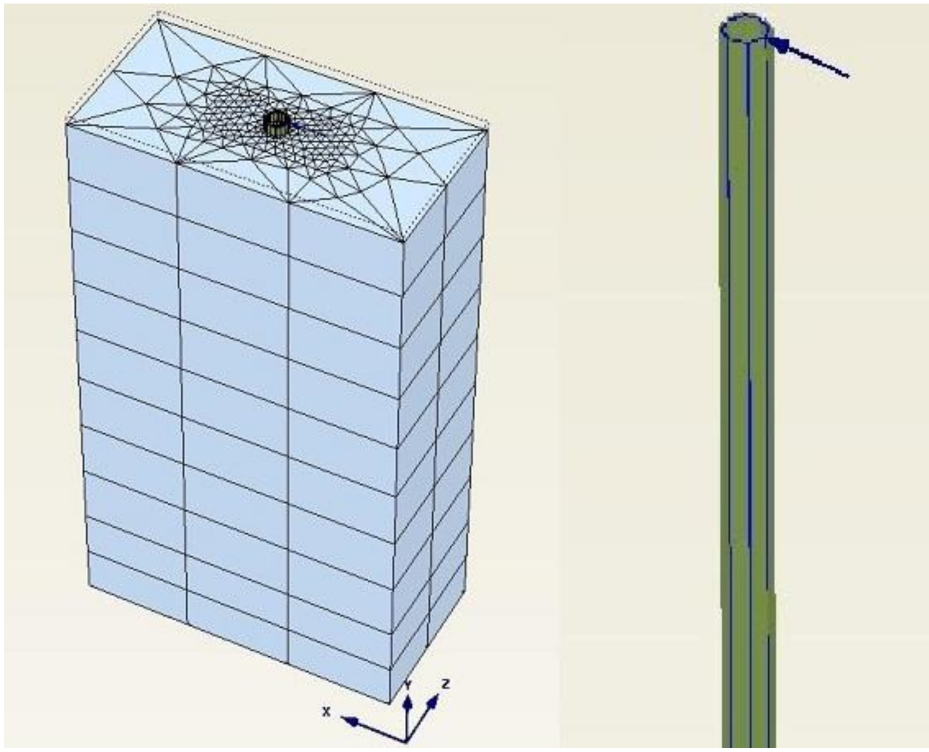


Figure 3.8 Three dimensional mesh and pile constructed in Plaxis 3D Foundation for the Mustang Island test

3.7 Experimental and numerical study of laterally loaded pile groups with pile caps at various elevations (validation for finite element analysis)

As described in Chapter 6, McVay et al. (2000) performed a series of centrifugal lateral load tests on 3x3 and 4x4 pile group-cap systems in sand. His goal was to analyze the effect of pile group elevation subjected to lateral loading. In contrast to the case analyzed in Chapter 6 where the surface of the pile cap was located at the ground surface, pile cap surface is located 1.16 m below ground surface for the validation of these results.

3.7.1 Pile configuration and soil properties

McVay et al. (2000) used square aluminum piles with the width of 0.43 and 8.6 m in length. Pile cap selected was an aluminum block and piles were drilled into the cap. Pile cap dimensions were the same as the ones used by McVay et al. (2000) in Chapter 6. These dimensions are 5.16 m in width and length. It should be noted that the pile cap thickness selected was 1.14 m. Pile spacing of 3D was used in the results validation. Test setup and pile group dimensions can be seen in Figure 3.9 while pile properties are presented in Table 3.3.

Table 3.3 Pile properties used by McVay for case 4

Pile Type	Length (L), m	Width (b), m	Head Fixity	Young's Modulus (E), KN/m^2
Aluminum Pile	8.6	0.43	Fixed-head	3.42×10^7

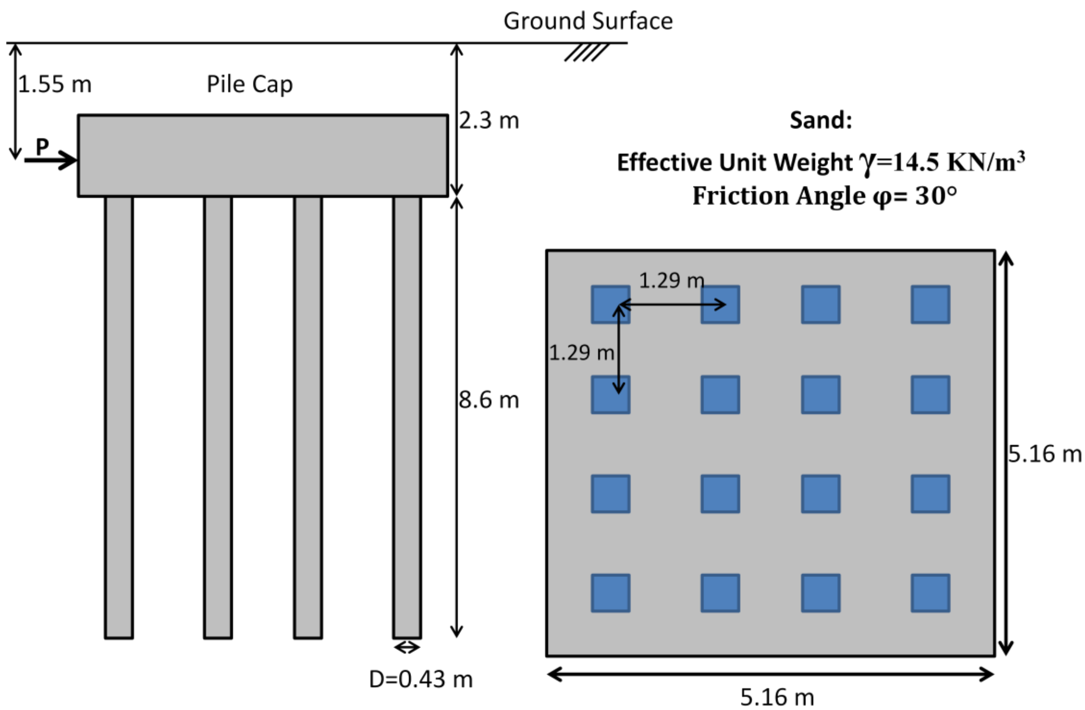


Figure 3.9 Test setup and pile cap overview on McVay's test

Soil properties used in this case are the same as the properties used by McVay et al. (2000) in Chapter 6. For further information about soil properties, refer to Table 6.7.

3.7.2 Lab test results compared with Plaxis 3D Foundation

Presented in Figure 3.10 are the centrifugal tests acquired in the lab by McVay et al. (2000) and predicted results found by finite element program Plaxis 3D Foundation. The same pattern can be found for the pile group as it was found for the single pile in the previous case. Plaxis 3D Foundation is giving a stiffer response in the beginning of the loading and evens out in later stages. The difference between the acquired and predicted results, at the point where there is the largest difference between them, is approximately 9 percent. These differences are considered small and it is concluded that the results are in a good agreement. Figure 3.11 below is representing the three dimensional overview of

the mesh created in Plaxis 3D Foundation as well as the pile group setup, while Figure 3.12 presents deformation direction. In addition to the three dimensional mesh, the two dimensional mesh presented in Figure 3.13 gives a good overview of the mesh.

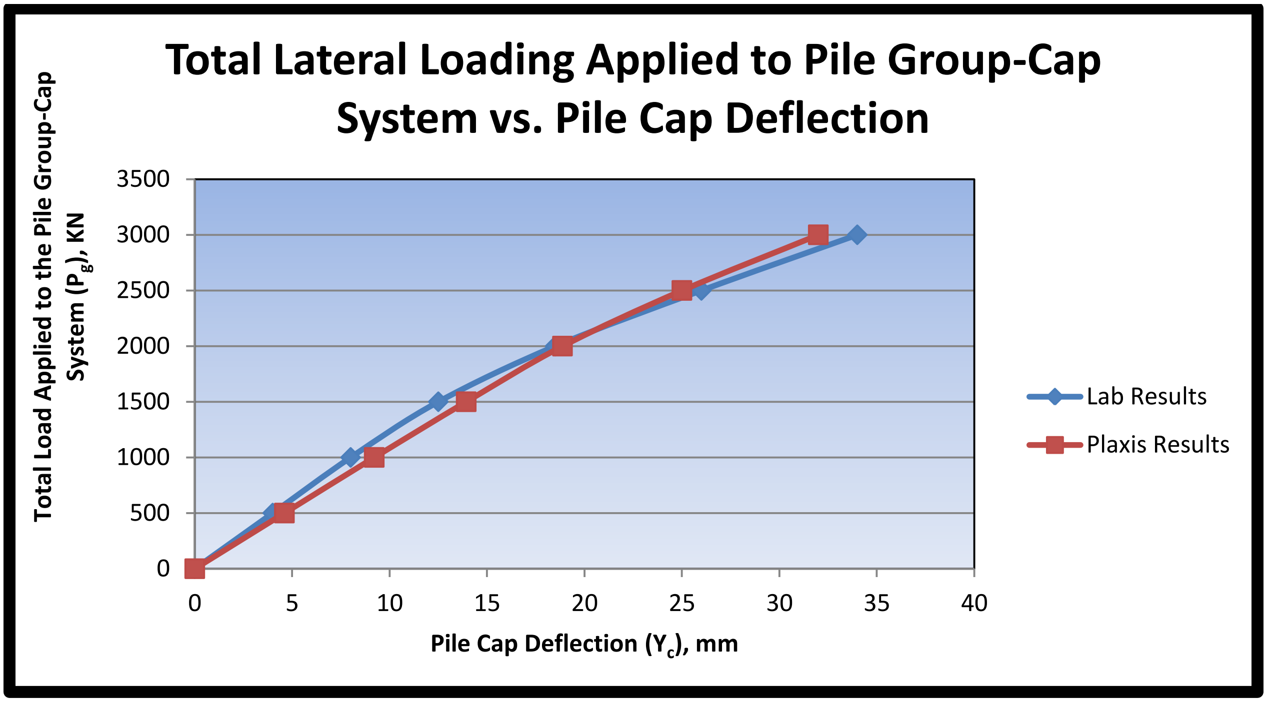


Figure 3.10 Lab results compared with Plaxis 3D Foundation (McVay et al. 2000)

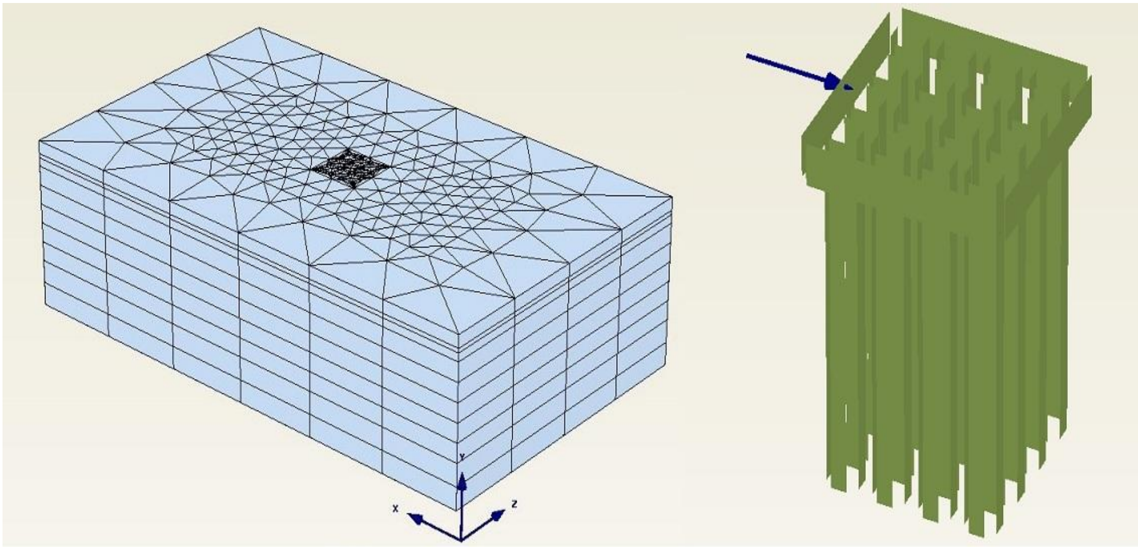


Figure 3.11 Three dimensional mesh and pile group constructed in Plaxis 3D Foundation for McVay test

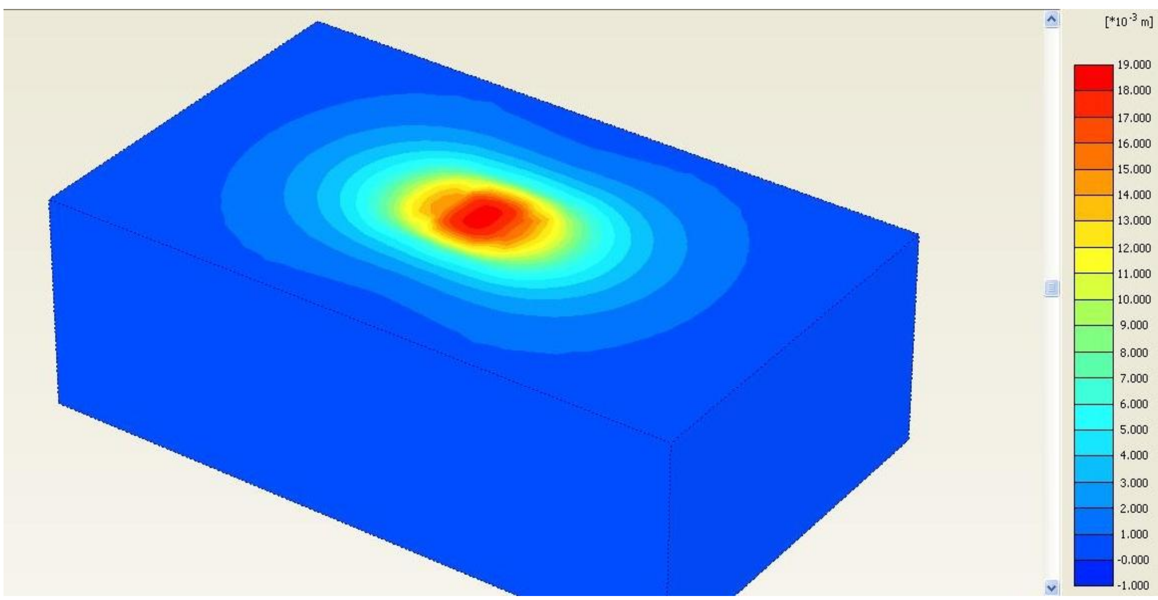


Figure 3.12 Pile group deformation interface lines for the test performed by McVay

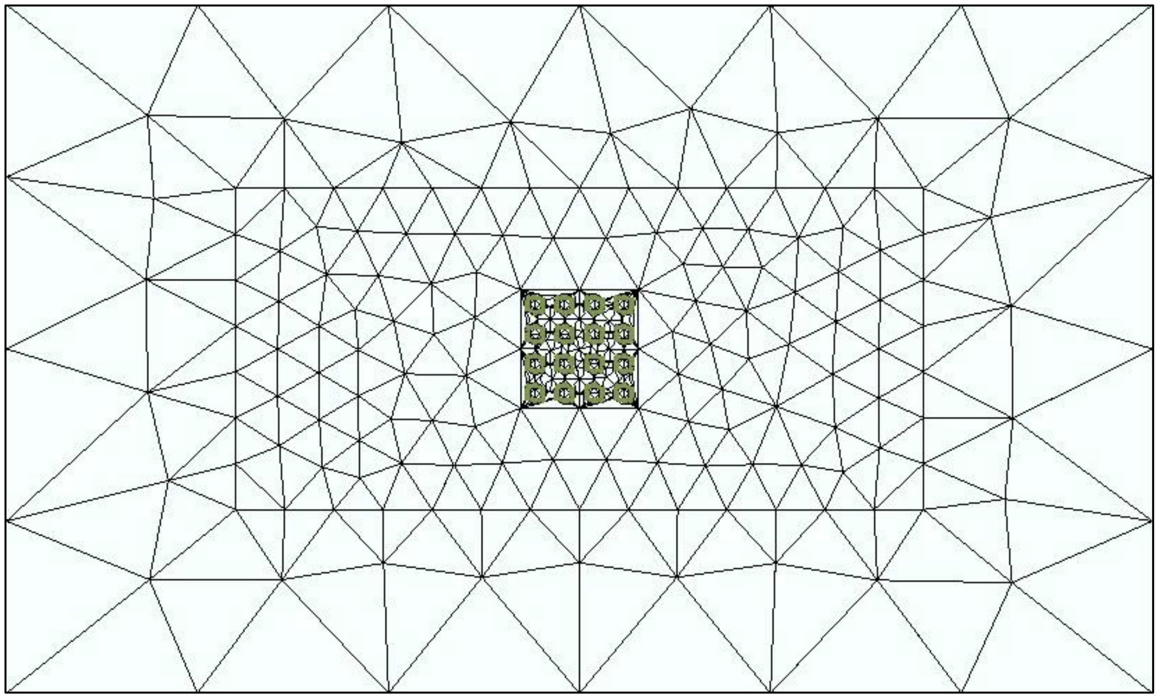


Figure 3.13 Two dimensional overview of the mesh for McVay's test

CHAPTER 4

ANALYTICAL MODEL

4.1 Incorporation of the pile cap in the Strain Wedge (SW) model

A pile group is typically tied together by a pile cap. The piles within the group are embedded some distance into the cap, and are either fixed against rotation (fixed-head condition) or allowed to rotate some amount (free-head condition). Since the cap must move in addition to the pile heads, the pile cap will contribute to the overall working load of the pile group-cap structure. Depending on the size of the pile cap and its depth of embedment below the surface, a pile cap may contribute a significant amount of lateral load capacity to pile group-cap foundation system.

The inclusion of pile cap capacity as a portion of the overall pile group-cap structure capacity is typically excluded in working load design calculations. This may be appropriate for pile caps at the ground surface. However, as the depth of the pile cap below finished grade increases the contribution of the pile cap increases and may need to be considered in design calculations.

In order to predict the response of a pile cap to lateral loading using the strain wedge model, the pile group model by Ashour et al. (2004) is modified to analyze square or rectangular pile caps taking into account the simultaneous development of a passive wedge associated with the pile group and the pile cap (Figure 4.1). One would intuitively

expect that the greater the pile cap width and depth of embedment, the greater the working load contribution from the pile cap to the pile group-cap structure for a given deflection value.

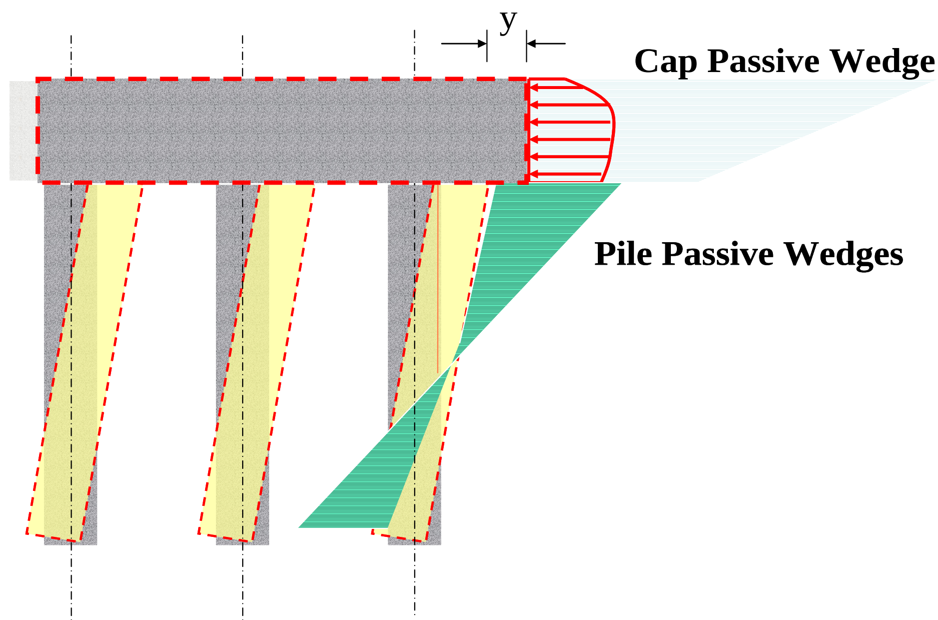


Figure 4.1 Foundation system of soil-pile group with cap

4.1.1 Definitions and assumptions used in developing the SW model to characterize the response of a pile cap to lateral loading

In developing the strain wedge model to account for a pile cap, a number of simplifying assumptions were required. Before identifying the assumptions used in developing the model, the following lists some definitions of terms used in the following sections.

- The pile cap is assumed to only moves laterally, that is no rotation of the pile cap takes place.
- The piles will be connected to the pile cap by either fixed-head or free-head connections with varying vertical and lateral stiffness's (K_v and K_H) as shown in Figure 4.2. The lateral behavior/resistance of cap is controlled by K_v, K_H and the soil mobilized passive resistance K_c.
- The pile cap and its soil passive wedge is connected with the other pile passive wedges as a part of the deep foundation system by sharing the same pivot point (i.e. same point for the toes of all passive wedges) as seen in Figure 4.3.
- The contribution of shear resistance at the base of the pile cap will be negligible.
- The contribution of shear resistance along the sides of pile cap parallel to the direction of loading could be significant (Figure 4.4).
- The pile cap is a rigid structure that will not experience any internal/flexible deformation when loaded laterally.

- The soil profile is uniform within the depth of the cap.

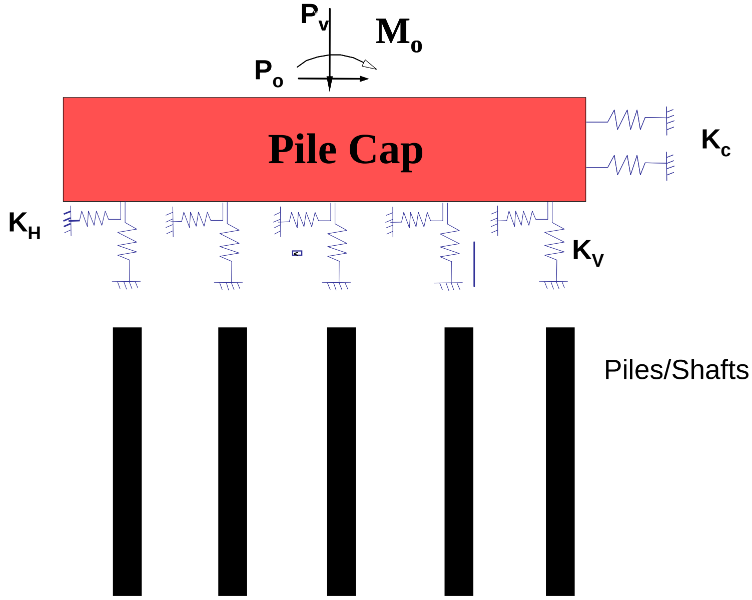


Figure 4.2 Soil-pile group-cap modeling under lateral loads

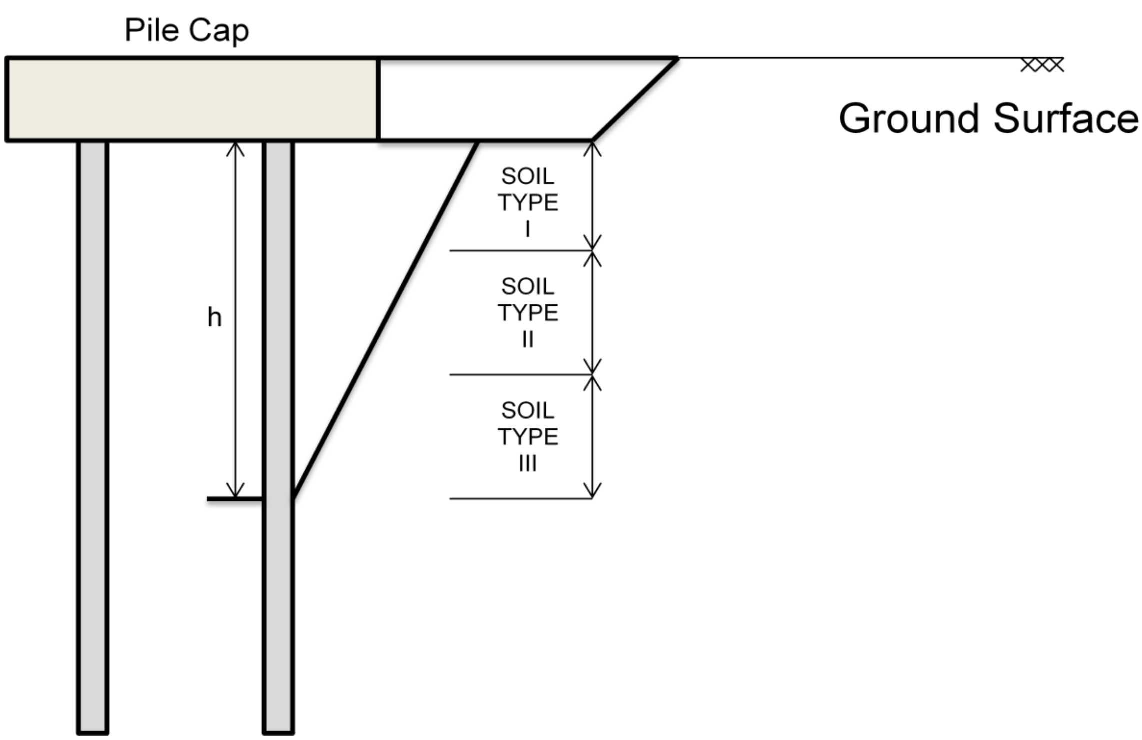


Figure 4.3 Passive wedges in front of the pile and cap with the same zero-deflection point

4.2 Deflection of a pile cap subjected to lateral loading

The SW model as developed to accommodate pile groups under lateral loading assumes that all the pile heads within the group are connected by a pile cap that will maintain a uniform pile head deflection for each pile in the group when the group is subjected to a lateral load. The SW model assumes the pile cap will displace the same amount of the pile group

$$Y_G = Y_C \tag{4.1}$$

Where Y_C is the deflection of a pile cap for a corresponding working load and Y_G is the deflection of the pile group under the same loading conditions.

4.3 Working load of a pile cap subjected to lateral loading as characterized by the Strain Wedge Model

The SW model evaluates the working load associated with a pile cap subjected to lateral loading by developing an equivalent subgrade modulus profile within the cap region. Since the deflection of the cap is known as described above in section, the working load of the cap associated with the input strain value can be determined. In addition, the contribution of side shear along the sides of the cap parallel to the direction of loading can be added to the passive contribution to determine a total pile cap working load.

The current section describes the development of mobilized passive wedges and soil resistance at the face of the wedge (Figure 4.4) within the pile cap region. The side

shear resistance along the pile cap (τ_{cap}) and the working load of the pile cap (P_{cap}) are determined based on the level of loading, stress-strain level and soil properties.

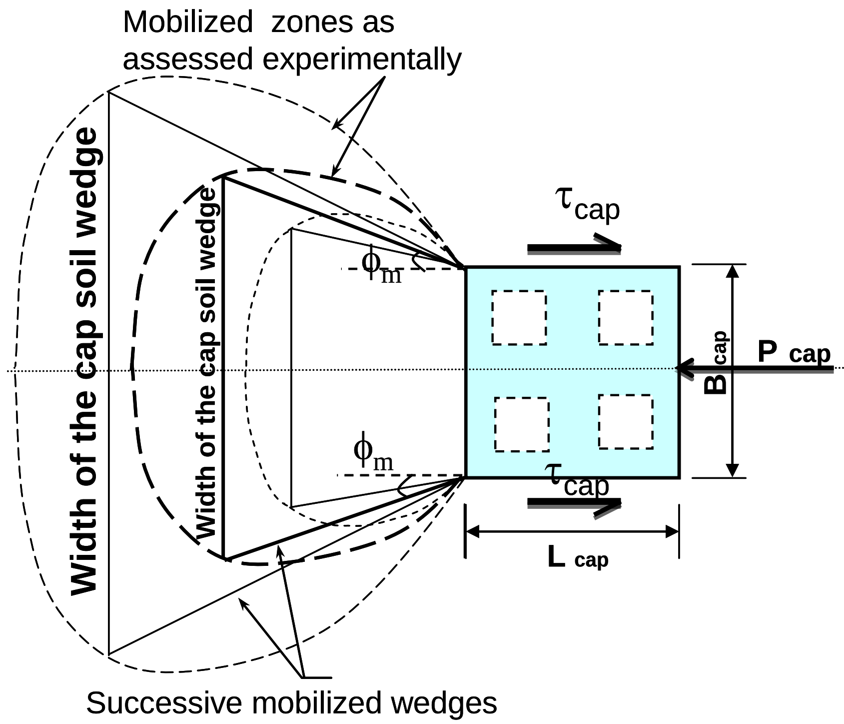


Figure 4.4 Mobilized passive wedge of soil in front of the cap and related forces

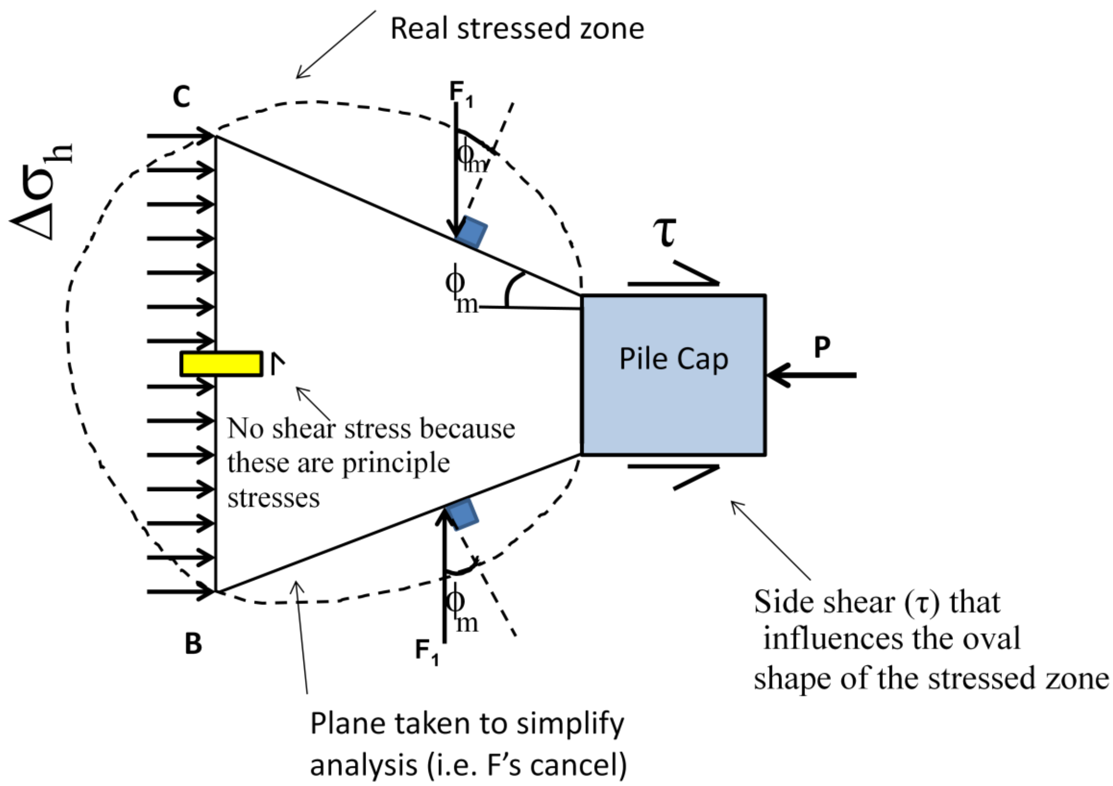


Figure 4.5 Characterization of the mobilized passive wedge of the pile cap and related stresses based on the triaxial load test

The strain in the soil in front of the cap (ε_{Cap}) at a particular step of loading is calculated as,

$$\varepsilon_{Cap} = \frac{Y_G}{B_{Cap}} \tag{4.2}$$

The wedge base angle β (Figure 4.3) is given by

$$(\beta_m)_{Cap} = 45 + \frac{(\bar{\varphi}_m)_{Cap}}{2} \tag{4.3}$$

From the concepts of the triaxial test, the stress level (SL) in soil at the face of the soil-cap wedge (Figure 4.5) is a function of the variation of the effective friction angle $\bar{\varphi}$.

$$SL = \left(\frac{\Delta \sigma_h}{\Delta \sigma_{hf}} \right)_{Cap} = \left(\frac{\tan^2 \left(45 + \bar{\varphi}_m / 2 \right) - 1}{\tan^2 \left(45 + \bar{\varphi} / 2 \right) - 1} \right)_{Cap} \tag{4.4}$$

where $\Delta\sigma_h$ and $\Delta\sigma_{hf}$ are the current and failure deviatoric stress in soil.

$$\tan^2 \left(45 + \frac{\bar{\varphi}_m}{2} \right) = \frac{\left(\bar{\sigma}_{vo} + \Delta \sigma_h - \Delta u \right)_i}{\left(\bar{\sigma}_{vo} - \Delta u \right)_i} \tag{4.5}$$

and

$$\Delta \sigma_{hf} = 2\, S_u \quad \textit{for clay} \tag{4.6}$$

Δu is the clay porewater pressure as determined in detail by Ashour et al (1998).

From the geometry of the passive wedge (Figures 4.3 and 4.5) the width of the face of the passive wedge BC is calculated as,

$$\overline{BC}_{Cap} = B_{Cap} + 2\, (h + 0.5\, D_{Cap})\, \left(\tan \beta_m \right)_{Cap} \left(\tan \varphi_m \right)_{Cap} \tag{4.7}$$

Since the depth to the pile-cap interface and the thickness of the pile cap is known, the available side shear resisting force along the two sides of the cap is given as

$$p_{Cap} = \Delta\sigma_h\, \overline{BC}_{Cap} + 2\, \tau_{Cap}\, L_{Cap} \tag{4.8}$$

L_{Cap} is the length of pile cap parallel to the direction of loading and D_{Cap} is the thickness of the pile cap. τ_{Cap} is determined as illustrated in the next section.

4.3.1 Side shear resistance on the sides of the pile cap

Depending on the construction methods associated with the pile group and pile cap and the depth of embedment of the pile cap below the ground surface, the development of side shear along the length of the pile cap parallel to the direction of loading may be of significance.

Where τ is the mobilized pile side shear stress at the pile-cap interface (i.e. unit side shear resistance), SL_t is the stress level along the sides of the pile at the pile-cap interface; and τ_{ult} is the ultimate value of side shear stress at the pile-cap interface, and is equal to the undrained shear strength of the clay at this location.

The development of mobilized side shear along the sides of a pile cap (τ_s) as characterized by the strain wedge model is dependent on the shear stress level (SL_t) at the soil-pile cap side interface associated with group deflection.

In the case of sand, SL_t is a function of the effective overburden pressure at the depth (σ_{vo}) under consideration and the mobilized angle of internal friction as described in Equation 4.9.

$$\tau_{Cap} = (\bar{\sigma}_{vo})_{Cap} \tan(\varphi_s)_{Cap}; \tag{4.9}$$

where

$$\tan(\varphi_s)_{Cap} = 2\tan(\bar{\varphi}_m)_{Capi} \tag{4.10}$$

φ_s is the mobilized angle of friction angle on sand-pile cap side interface.

In case of clay soil, τ_{Cap} is a function of the undrained shear strength of the clay (i.e. shear stress level SL_t), as presented by Ashour et al. (1998). The normalized shear stress load transfer curves is determined using the following equations,

$$SL_t = 12.9\, Y_C \,/\, B_{Cap} - 40.5\, Y_C^2 \,/\, B_{Cap}^2 \qquad (4.11)$$

Y_C is the cap deflection

$$\tau_{Cap} = SL_t\ \tau_{ult} = SL_t\ S_u \qquad (4.12)$$

Since the deflection of the pile cap is known as described above, the working load of the pile cap associated with passive resistance for a given input strain is then given by the following equation,

$$P_{Cap} = p_{Cap}\ D_{Cap} \qquad (4.13)$$

Where P_{Cap} is the lateral load associated with mobilized passive and side resistance against lateral movement of the pile cap. The working load associated with passive resistance against the pile cap under lateral loading can also be easily computed, as

outlined above, for a pile cap embedded in sand, normally consolidated clay, and layered soils.

When evaluating the working load due to side shear of a pile cap subjected to lateral loading, one must give strong consideration to whether or not side shear is applicable to the particular problem being studied. If a pile is constructed in material that may slough or otherwise be significantly disturbed during construction, or if the pile will be located at or very near the ground surface, the contribution of side shear will most likely not be significant. If, however, the pile cap is located at depth below the ground surface and in a material that is not significantly disturbed during construction, the contribution of side shear to the overall working load of the pile cap for a given deflection may be significant.

CHAPTER 5

PARAMETRIC STUDY

5.1 Introduction

A parametric study is performed using the Strain Wedge Model program on a 3x3 pile group. Two different types of soil, sand and clay, were considered. The total of six different cases was taken into consideration. Variables of pile head fixity and density of the soil were manipulated in order to understand how pile cap behaves under different conditions. Results received from this program are presented in the following chapter.

5.2 Parameter variation

Several parameters used in Strain Wedge Model program for sand and clay were kept as constants. These parameters such as length of the pile, depth of the pile cap below ground surface, effective unit weight for both types of soil, diameter of pile, flexural stiffness and the type of the pile did not change. Variables that were changed while dealing with sand were the fixity of the pile head (free and fixed) and friction angle. Two variables were changed as well while dealing with clay soil. These variables are the cohesion of soil and fixity of pile head. Dimensions of the piles and the pile cap can be

seen in Table 5.1 and Table 5.2, respectively, as well as in Figure 5.1. Also, sand and clay soil properties are given in Table 5.3.

Table 5.1 Pile dimensions used in Strain Wedge Model for sand and clay

Type of Pile	Reinforced Concrete
Pile Length	7.0 m
Pile Diameter	0.4 m
Pile Head Fixity	Free, Fixed
Flexural Stiffness (EI), KN-m^2	42981.7

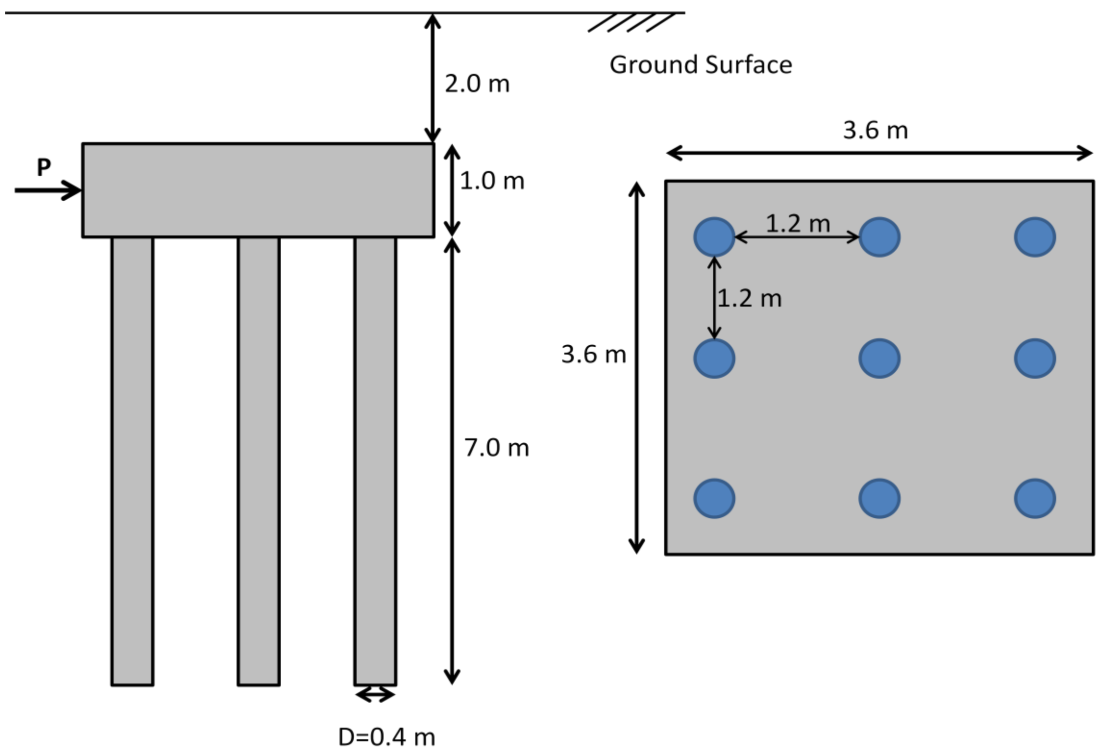


Figure 5.1 Test setup and pile group dimensions of a 3x3 pile group-cap system

Table 5.2 Pile cap dimensions used in Strain Wedge Model for sand and clay

Depth of Pile Cap BGS*, m	2.0
Length of Pile Cap (B), m	3.6
Width of Pile Cap (W), m	3.6
Pile Cap Thickness (t), m	1.0
Pile Spacing	3D
Young's Modulus (E), KN/m^2	3.15×10^7

* BGS-Below Ground Surface

Table 5.3 Soil properties used in Strain Wedge Model for sand and clay

Soil Type	**Sand**	**Clay**
Effective Unit Weight (γ), KN/m^3	10.0	10.0
Cohesion (S_u), KN/m^2	0	30, 80, 130
Friction Angle (φ), deg.	30, 35, 40	0

5.3 Comparison of results using Strain Wedge Model and Plaxis 3D Foundation

Parametric study employs both Strain Wedge Model and Plaxis 3D Foundation. One type of soil, loose sand, is selected to be performed for free and fixed head pile-cap

connection. These results are presented in Figure 5.2 below. From this figure it can be noticed that for Strain Wedge Model, the pile cap is taking more loads at the same deflection then Plaxis 3D Foundation for free and fixed pile-cap connection. This is noticeable for the first part of the curve. However, towards the end of the graph, the pile cap for Strain Wedge Model is taking fewer loads at the same defection then Plaxis 3D Foundation, for free and fixed pile-cap connection. Results compared between these two programs are not exactly the same due to a fact that Strain Wedge Model is using non linear soil and pile properties and Plaxis 3D Foundation is using linear soil and pile properties.

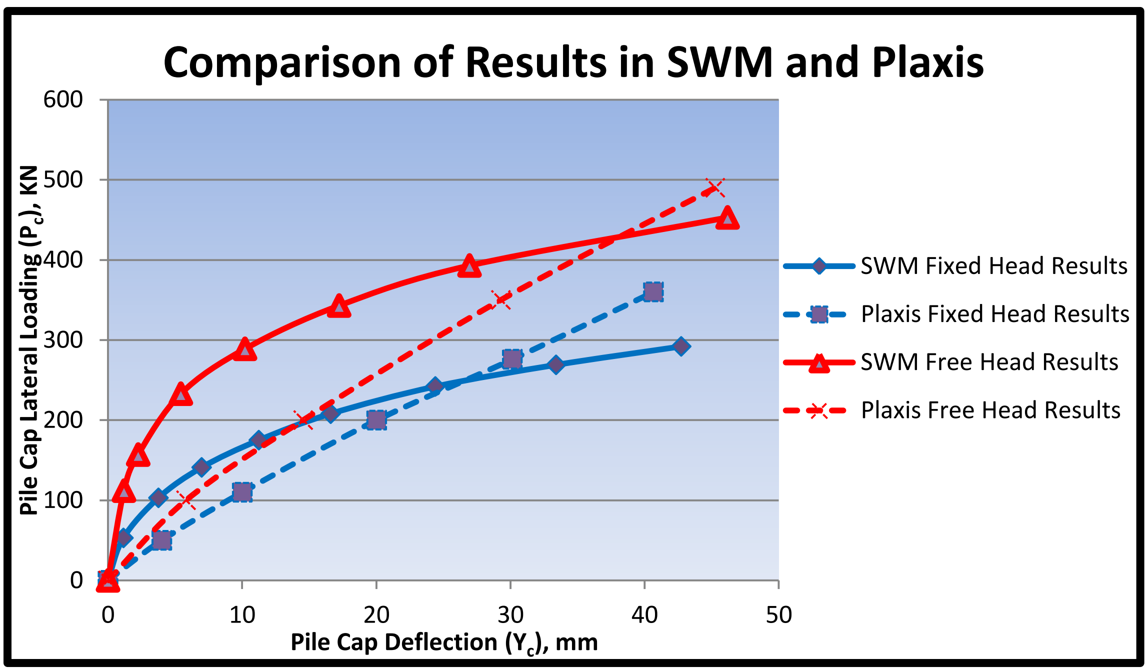


Figure 5.2 Comparison of results in Strain Wedge Model and Plaxis for the 3x3 pile group-cap system

In Figure 5.3, presented are three dimensional mesh and pile group created in Plaxis 3D Foundation, while in Figure 5.4 shows the deformation direction of the pile

group. In addition to these two figures, Figures 5.5 and 5.6 present pile group with cap finite element deformation image for fixed and free pile head conditions. Both figures include a pile group-cap system with the interface elements as well as just a pile group. The reason for including both images is for a better visual recognition of a free and fixed pile-cap connection case.

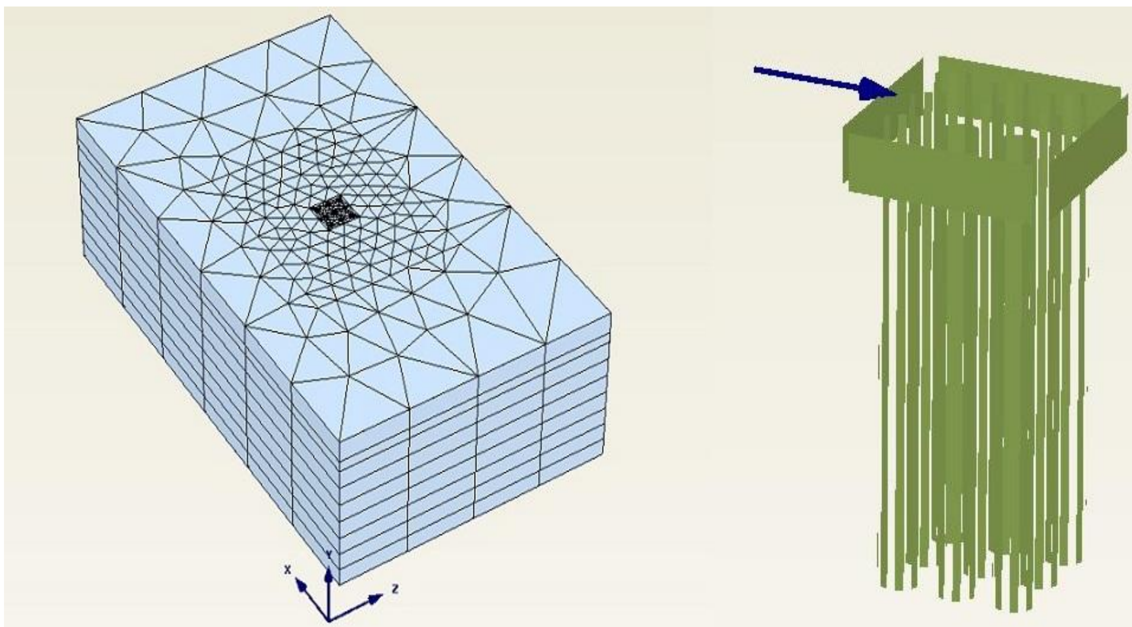


Figure 5.3 Three dimensional overview of the mesh and pile group setup of a 3x3 pile group-cap system

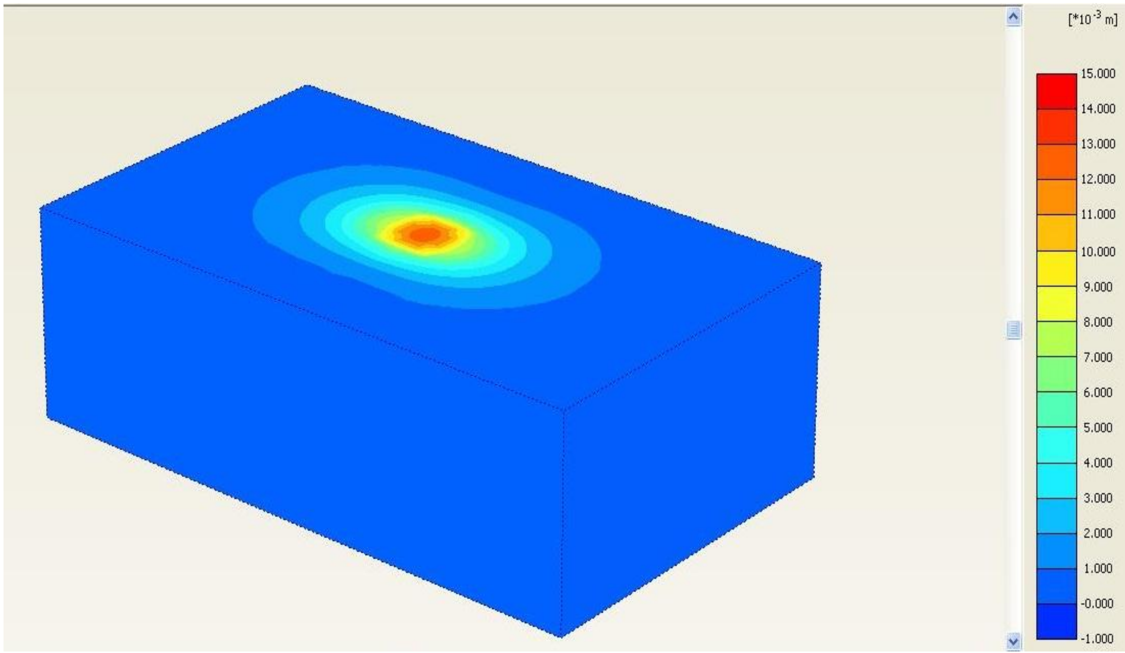
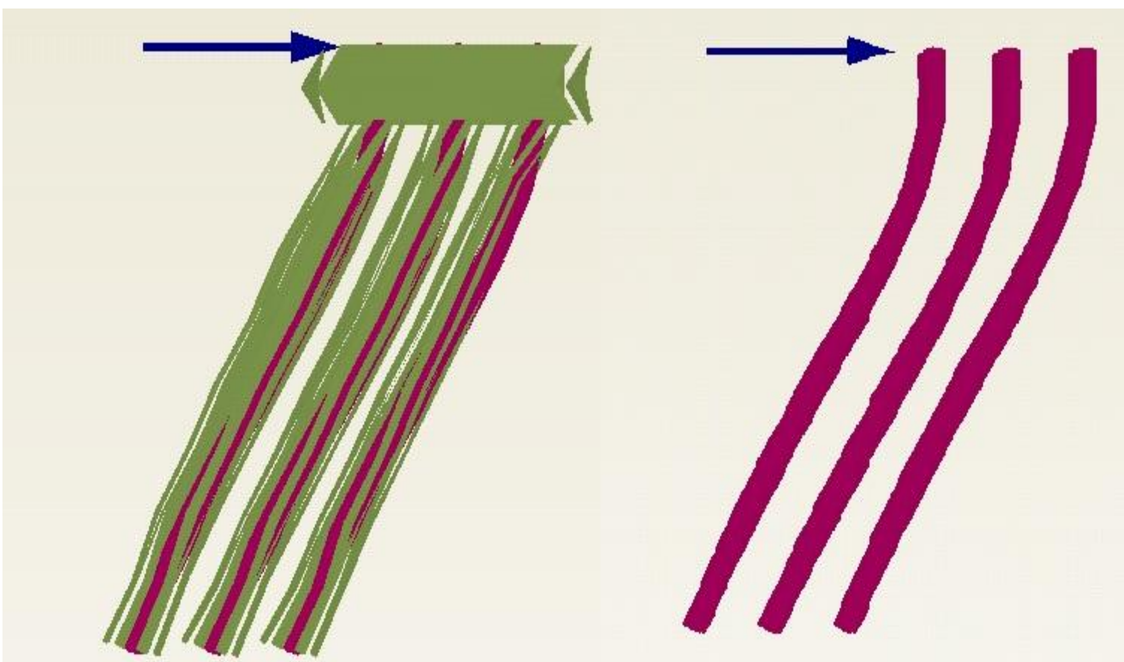
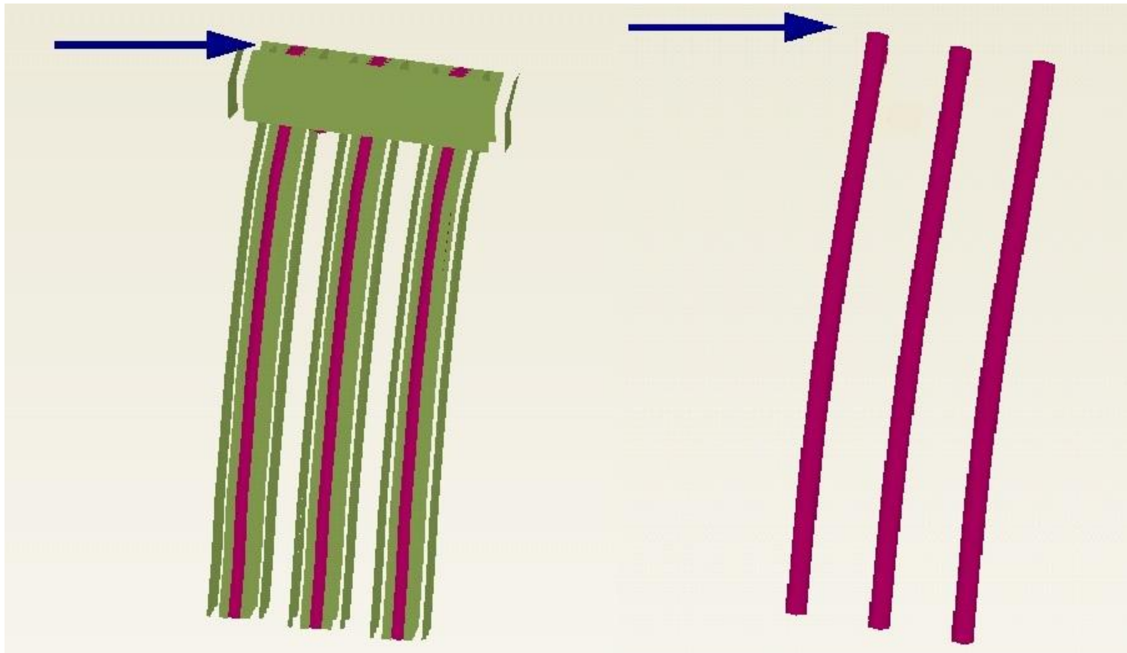


Figure 5.4 Deformation deflection of the 3x3 pile group-cap system



5.5 Finite element deformation image of the pile group with cap for the fixed head pile-cap connection



5.6 Finite element deformation image of the pile group with cap for the free head pile-cap connection

5.4 Effect of soil type & pile head fixity on pile cap resistance

Each of the following six cases is presented by two curves, one for free head and the other for fixed head piles. The first three cases are dealing with three types of sand: loose, medium dense and dense sand. The other three cases are dealing with three types of clay: soft, medium stiff and stiff clay. It can be noted that for all six cases, the highest deflection that the free and fixed head go to is approximately around 80 mm. Presented in Figure 5.7 is the outlook of the pile cap that is being loaded laterally for the free and fixed pile-cap connection.

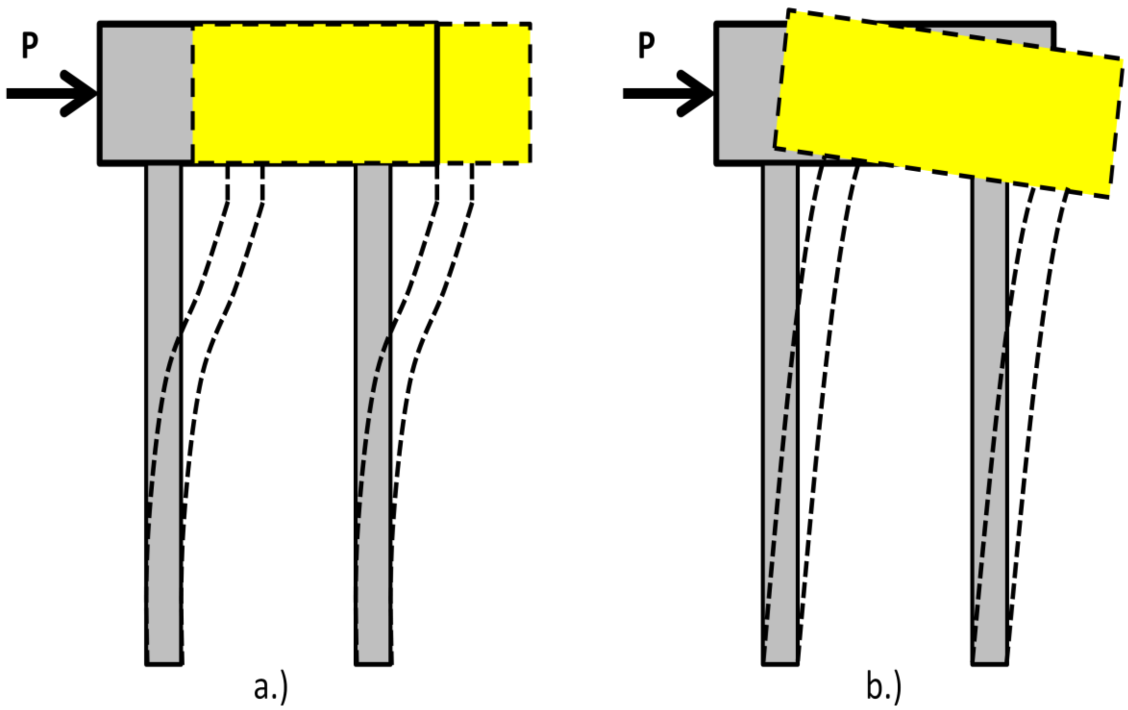


Figure 5.7 Pile cap deflection a.) Fixed pile-cap connection b.) Free pile-cap connection

5.4.1 Free and fixed head piles in loose sand

Ten different loads in the range from 250 KN to 2750 KN are laterally applied on a 3x3 pile group. The friction angle used for loose sand is 30°. Figure 5.8 shows the plot of the load carried by the pile cap versus the deflection of the pile cap for both free and fixed heads. It should be noted that the pile-cap connection plays an important role in lateral resistance taken by the pile cap. As it can be seen from the graph, free pile head connection at the same deflection carries more load than the fixed-head one. The difference of cap loads varies from 50 KN to 150 KN. At the maximum deflection of 80mm, the pile-cap with fixed head connection carries approximately 350 KN. At the same deflection, the pile-cap with free head connection carries around 500 KN. These

results are expected due to a fact that deeper the piles are embedded into the pile cap the more lateral loading they are going to take and less lateral loading is going to be taken by the pile cap.

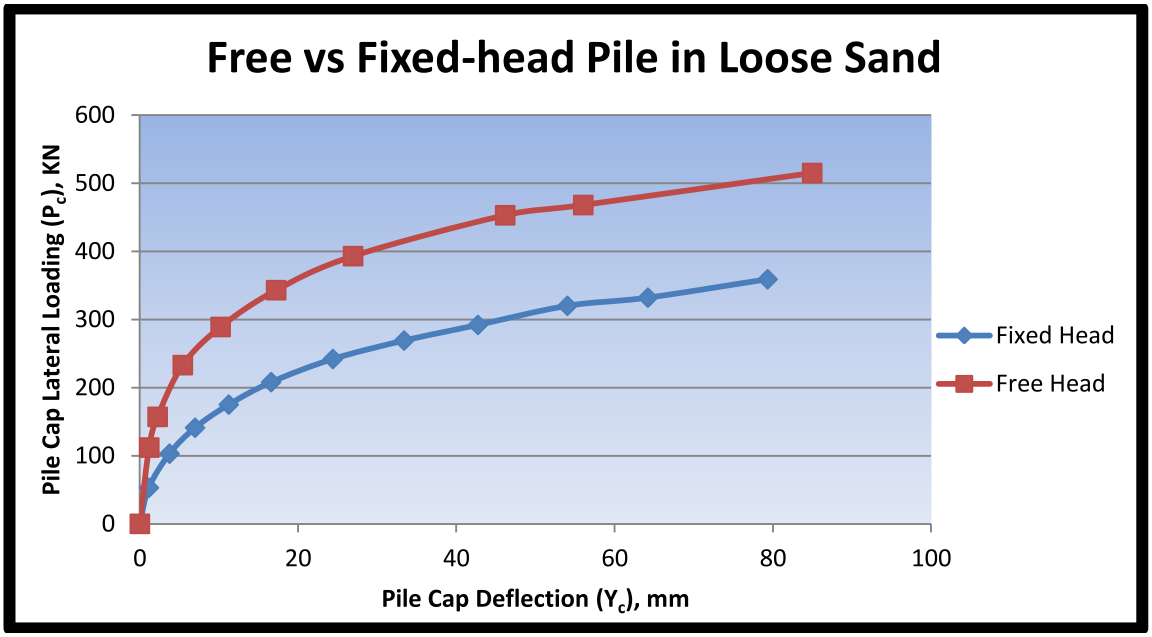


Figure 5.8 Effect of pile-head fixity on the pile cap load in loose sand

5.4.2 Free and fixed head piles in medium dense sand

The next soil tested is medium dense sand. The friction angle used for this soil is 35°. Total lateral load applied on this 3x3 pile group ranges from 500 KN to 4100 KN. In Figure 5.9 the correlation between loads taken by the pile cap and the pile cap deflection for free and fixed head curves can be seen. The difference of the loads taken by the pile cap for a free and fixed head at the same deflection varies between 50 KN to 200 KN. Approximately 650 KN is the load taken by the pile cap with a fixed head pile-cap connection, while at the same deflection this load is around 850 KN for the free head

pile-cap connection. Just like for the loose sand, it is expected to have less load taken by the pile cap than the piles with fixed head pile-cap connection.

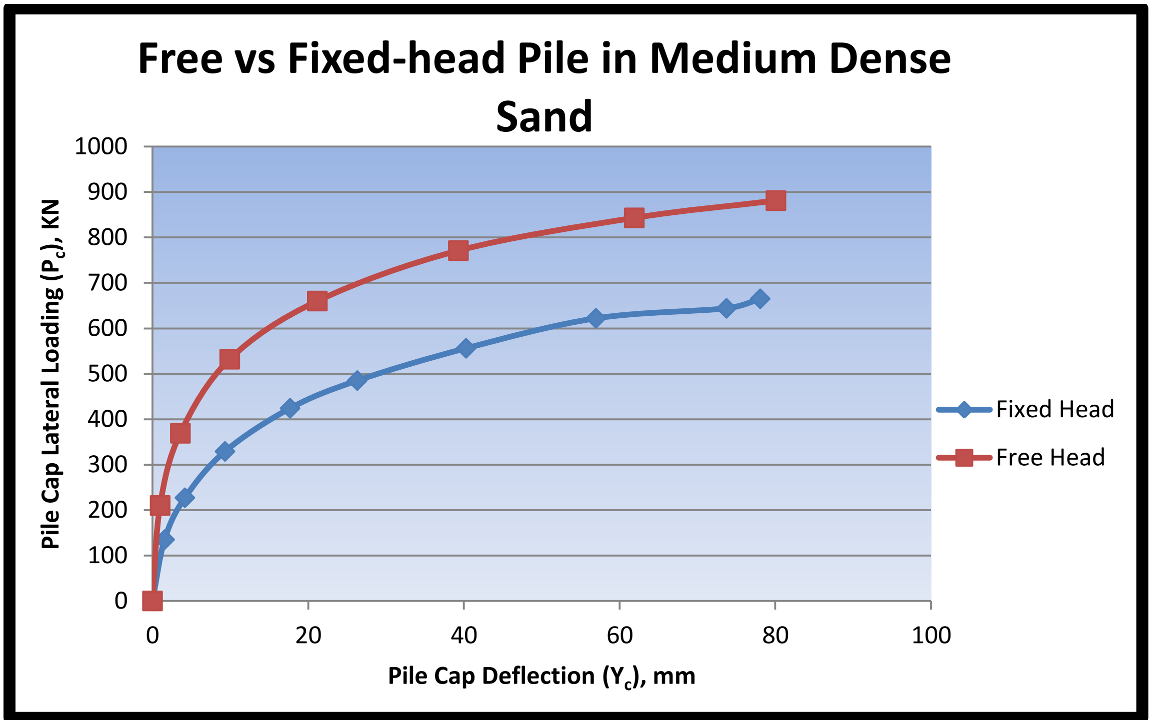


Figure 5.9 Effect of pile-head fixity on the pile cap load in medium dense sand

5.4.3 Free and fixed head piles in dense sand

The last tested sand soil is dense sand with a friction angle of 40°. The range of total lateral load applied to the pile cap is between 500 KN and 6000 KN. As it can be noticed in Figure 5.10, the same deflection load carried by the pile cap is less for the fixed head compared to free head connection. This difference is roughly between 50 KN and 300 KN. At the maximum deflection of 80 mm, the load carried by the pile cap for the free head pile-cap connection is around 1500 KN and approximately 1200 KN for the fixed head pile-cap connection. It can be noticed that for all three types of sand, the pile

cap with the fixed head pile-cap connection is taking more load than the pile cap with free head pile-cap connection.

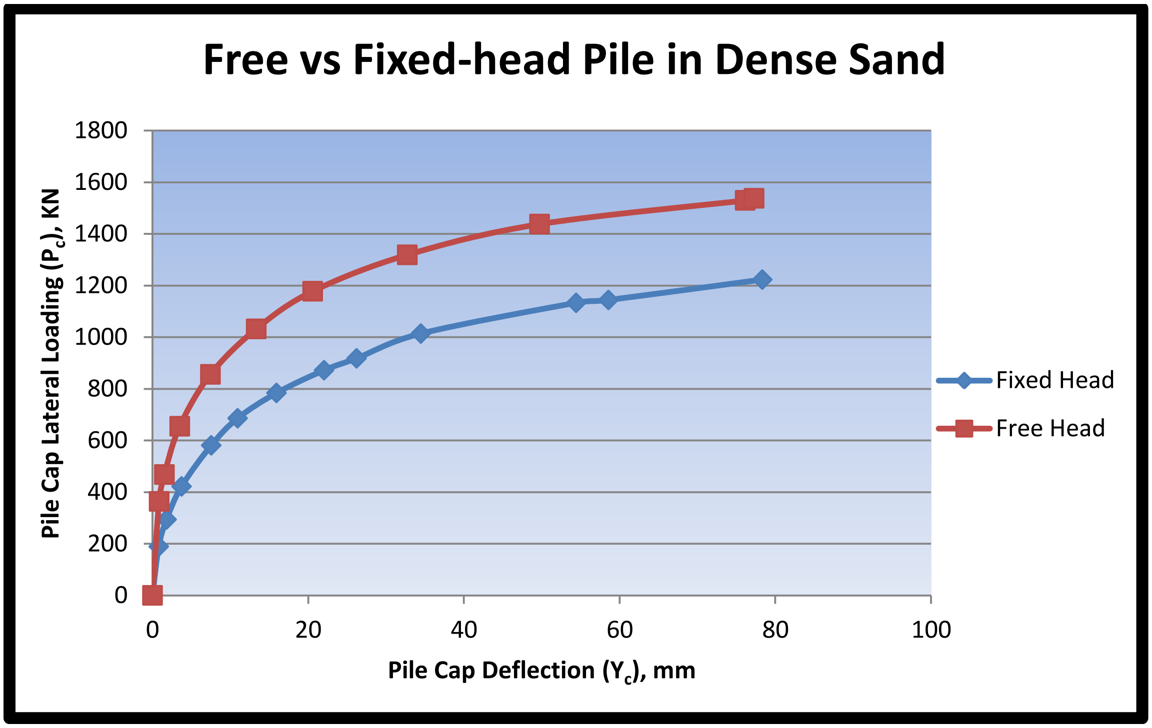


Figure 5.10 Effect of pile-head fixity on the pile cap load in dense sand

Figure 5.11 shows that for the loose sand soil, the percentage of the load that is taken by the pile cap ranges between 21 and 13 percent of the total lateral load carried by the pile group-cap system for the fixed head pile-cap connection. In addition, these numbers range from 23 to 17 percent for the medium dense sand and between 28 and 21 percent for the dense sand. It can be noticed in Figure 5.11 that the percent of the total load of the pile group-cap system taken by the pile cap is decreasing constantly. The highest percentage is recorded at the low deflection of the pile cap while the lowest percentage is recorded at the maximum deflection of 80 mm.

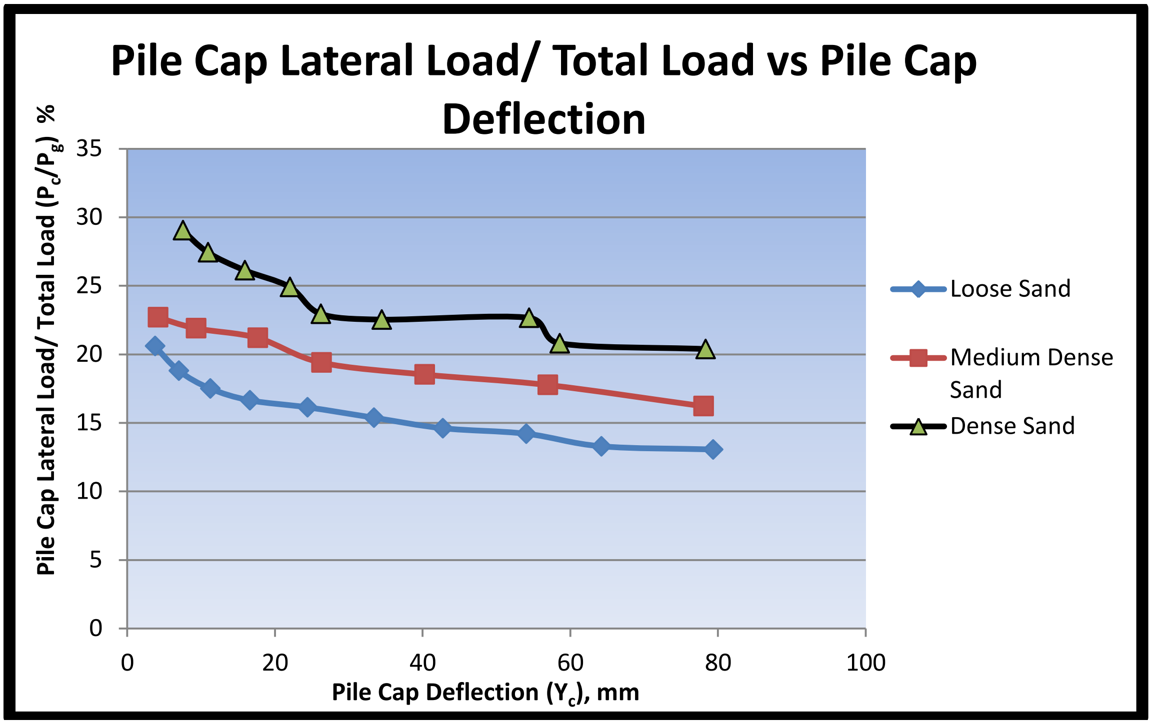


Figure 5.11 Percent of the total load taken by the pile cap vs. pile cap deflection for the fixed head in three types of sand

Figure 5.12 shows the percentage of the total load of the pile group-cap system taken by the pile cap versus pile cap deflection in loose, medium dense and dense sand for the free head pile-cap connection. The percent of the total load applied to the pile group-cap system taken by the pile cap varies between 22 and 31 percent for the loose sand, between 26 and 37 percent for the medium dense sand, and between 33 and 43 percent for the dense sand. The same pattern can be noticed for the free and fixed head pile-cap connections, where the biggest percentage is at the lowest pile cap deflection and vise versa.

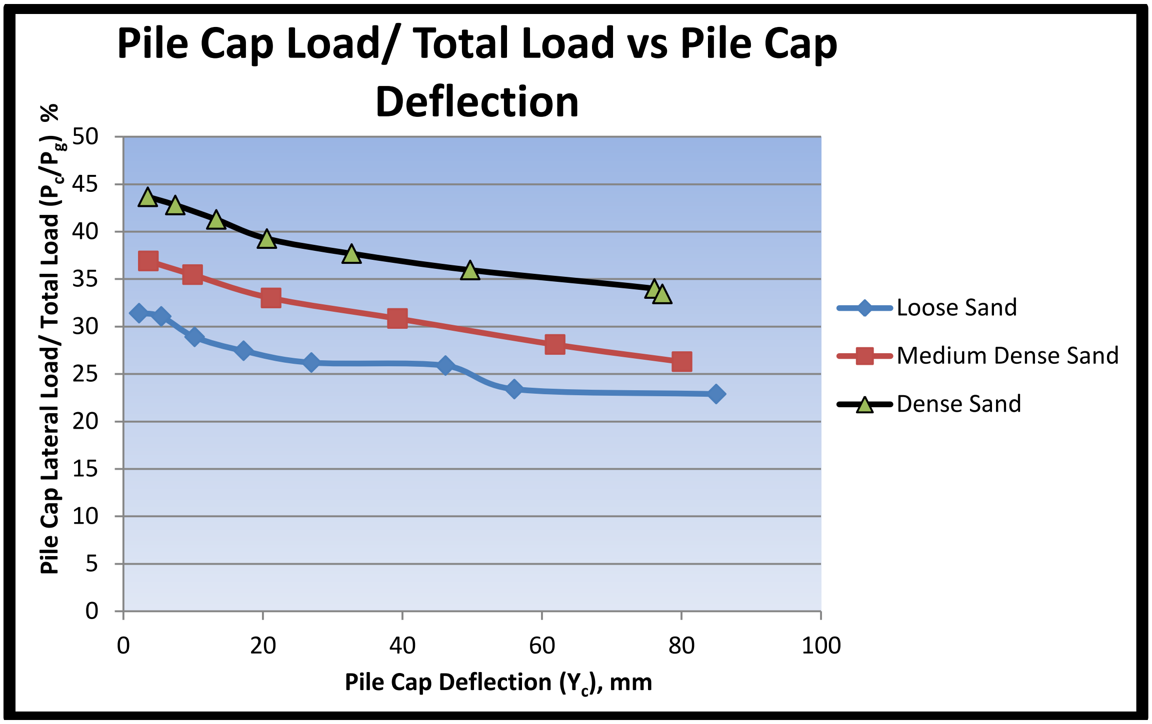


Figure 5.12 Percent of the total load taken by the pile cap vs. pile cap deflection for the free head in three types of sand

5.4.4 Comparison of load taken by the pile cap versus pile cap deflection for sand

As mentioned above, three different types of sand have been used around 3x3 pile group that was loaded laterally. This was done in order to get a better understanding of how the pile cap behaves in different soil conditions. Loose, medium dense and dense sand were used with friction angles of 30°, 35° and 40° respectively. Figure 5.13 shows that for the fixed head fixity at the maximum deflection of 80 mm pile cap that is surrounded by the loose sand is taking the least load. On the other hand, dense sand is taking most of the load.

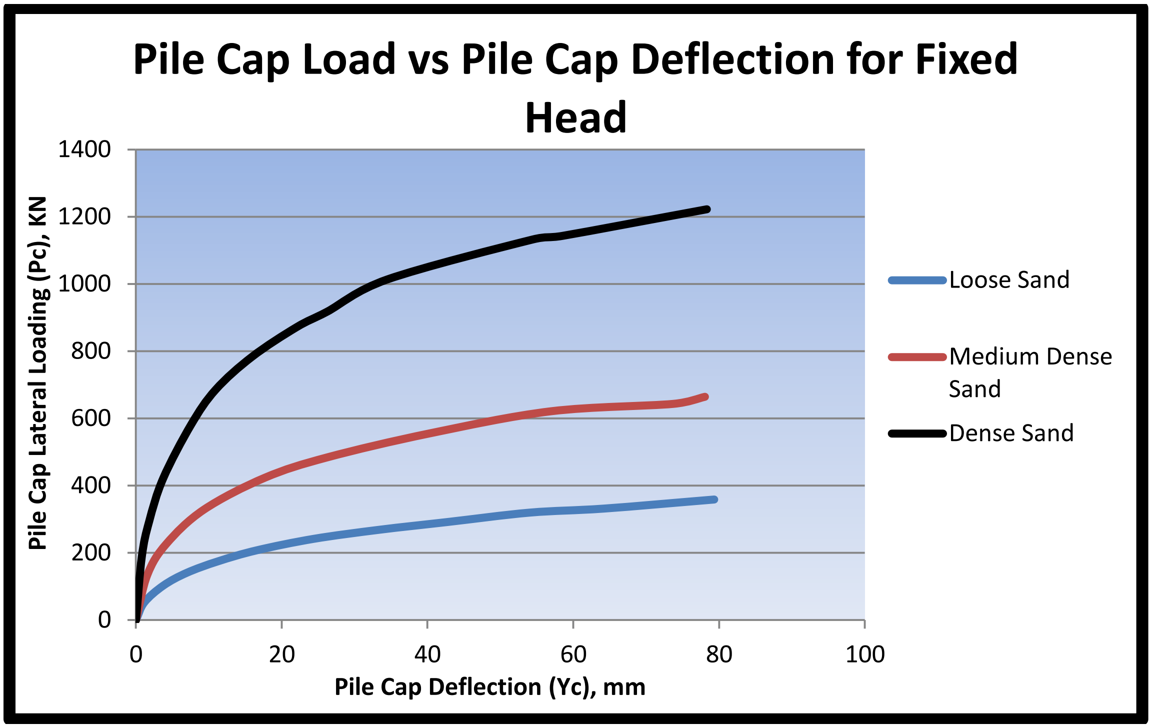


Figure 5.13 Loads taken by the pile cap versus pile cap deflection for fixed-head pile group in three types of sand

The same pattern can be also seen in Figure 5.14 for free head fixity. The difference in load taken by the pile cap, between loose and medium dense sand for fixed head fixity, at the maximum deflection of 80 mm is around 300 KN. This difference is higher for the free head fixity and it is approximately 400 KN. Difference in load taken by the pile cap, between medium dense and dense sand for fixed head fixity, at the maximum deflection of 80 mm is approximately 500 KN. For the free head fixity, this difference is around 650 KN.

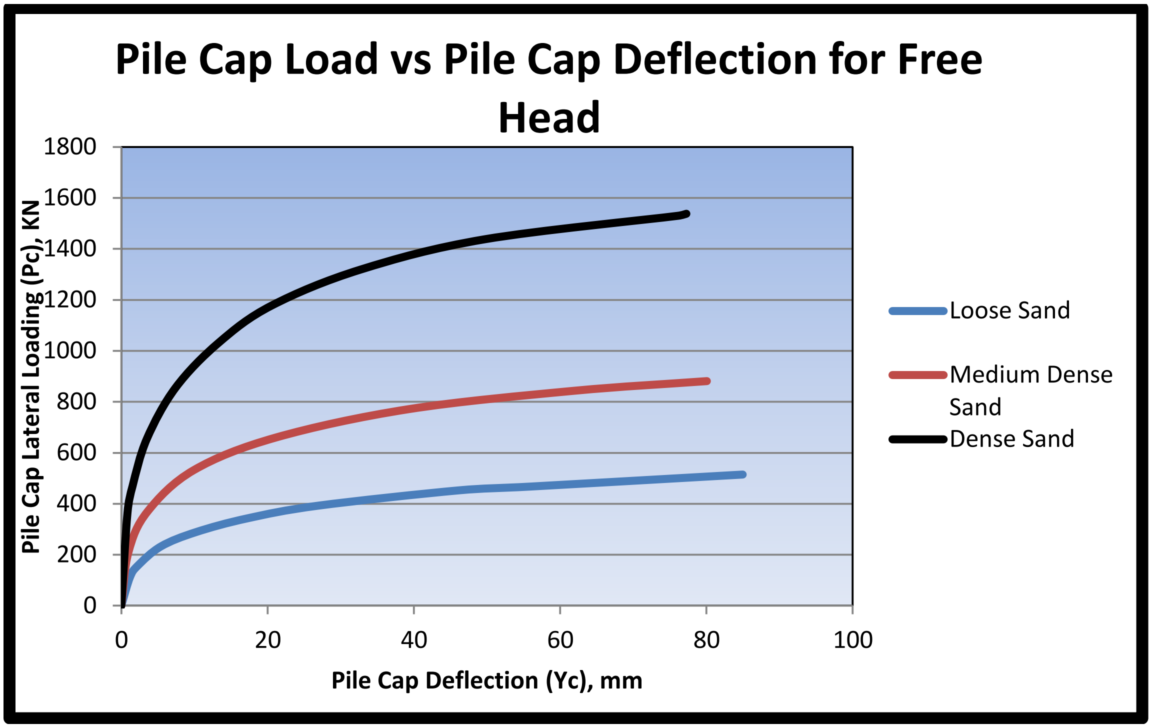


Figure 5.14 Loads taken by the pile cap versus pile cap deflection for free-head pile group in three types of sand

5.4.5 Free and fixed head piles in soft clay

Lateral load increments in the range from 100 KN to 1050 KN are applied on a 3x3 pile group in soft clay soil. As given in Table 5.3 for this soil, cohesion is 30 KN/m^2. Figure 5.15 presents the load carried by the pile cap versus pile cap deflection for free and fixed pile-cap connections. It can be noted that the load taken by the pile cap is larger for the free head than for the fixed head. At the same deflection, the total load carried by the pile cap for free and fixed head pile-cap connection varies between 50 KN and 100 KN. At the maximum deflection of 80 mm, the load taken by the pile cap for the free

head pile-cap connection is around 400 KN while this number is approximately 300 KN for the fixed head pile-cap connection.

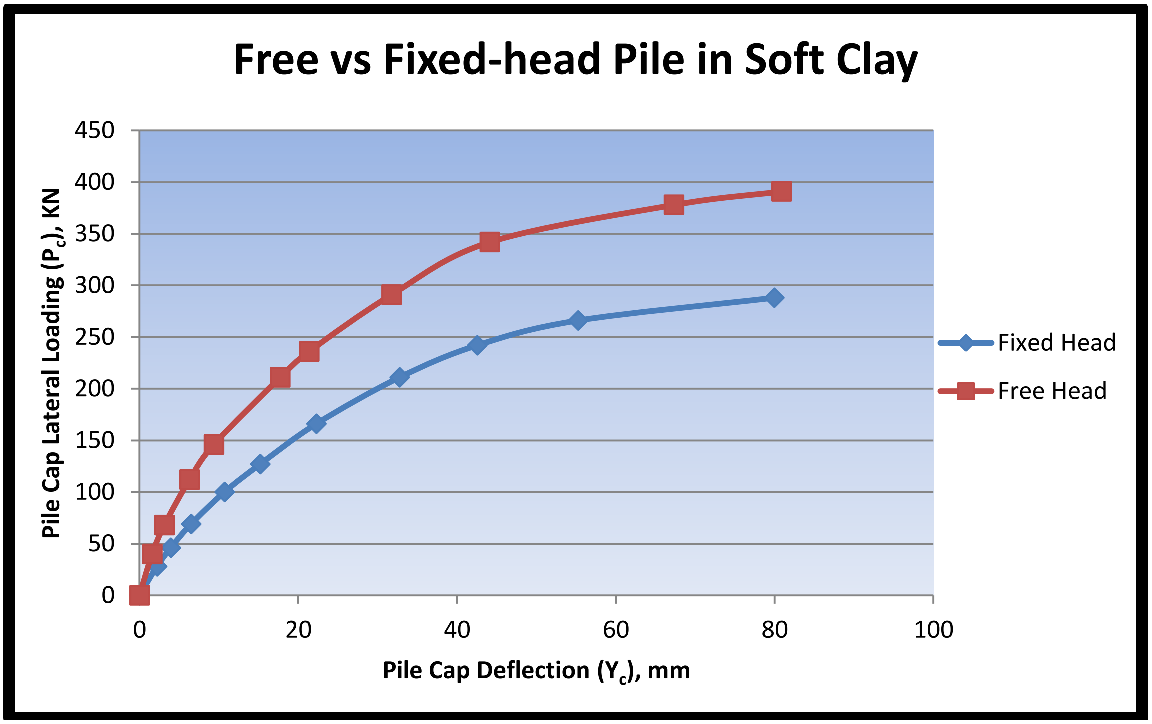


Figure 5.15 Effect of pile-head fixity on the pile cap load in soft clay

5.4.6 Free and fixed head piles in medium stiff clay

Medium stiff clay soil is used in this parametric study. The selected cohesion for this type of soil is 80 KN/m^2. The total lateral load applied to the 3x3 pile group is between 500 KN and 3750 KN. In Figure 5.16, free and fixed head pile-cap connection curve lines present pile cap load versus pile cap deflection. It can be noticed that the difference between free and fixed head pile-cap connections range from 100KN to 550 KN for the medium stiff clay soil. Approximately 1500 KN is the maximum load taken

by the pile cap for the free head pile-cap connection at the maximum deflection of 80 mm. At the same deflection, around 1000 KN is the load taken by the pile cap for the fixed head pile-cap deflection.

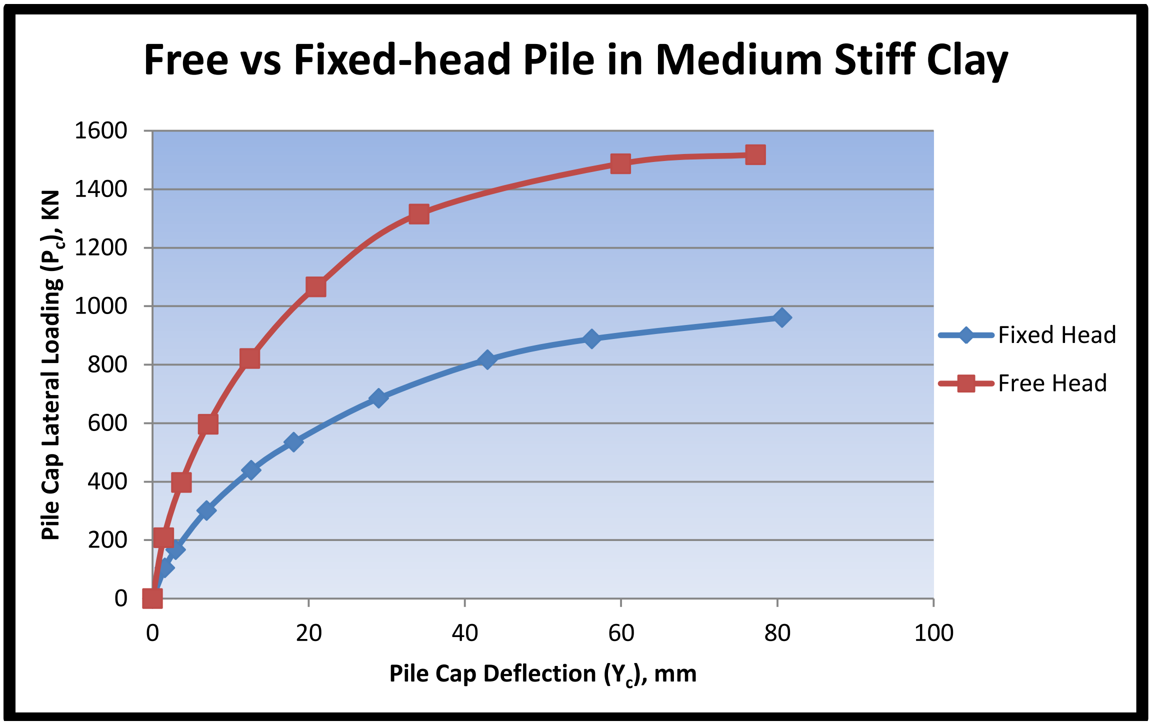


Figure 5.16 Effect of pile-head fixity on the pile cap load in medium stiff clay

5.4.7 Free and fixed head piles in stiff clay

The third clay tested in this parametric study is stiff clay with cohesion of 130 KN/m^2. Figure 5.17 presents the load carried by the pile cap versus pile cap deflection for free and fixed head. The difference at the same deflection for free and fixed head pile-cap connection is large and it ranges between 100 KN and 1200 KN. At the maximum deflection of 80 mm, loads taken by the pile cap for the fixed and free head pile-cap

connection are 1800 KN and 2900 KN, respectively. It can be noticed that for all three types of clay, the load taken by the pile cap for the free head pile-cap connection is larger than the load taken by the fixed head pile-cap connection at the same deflection.

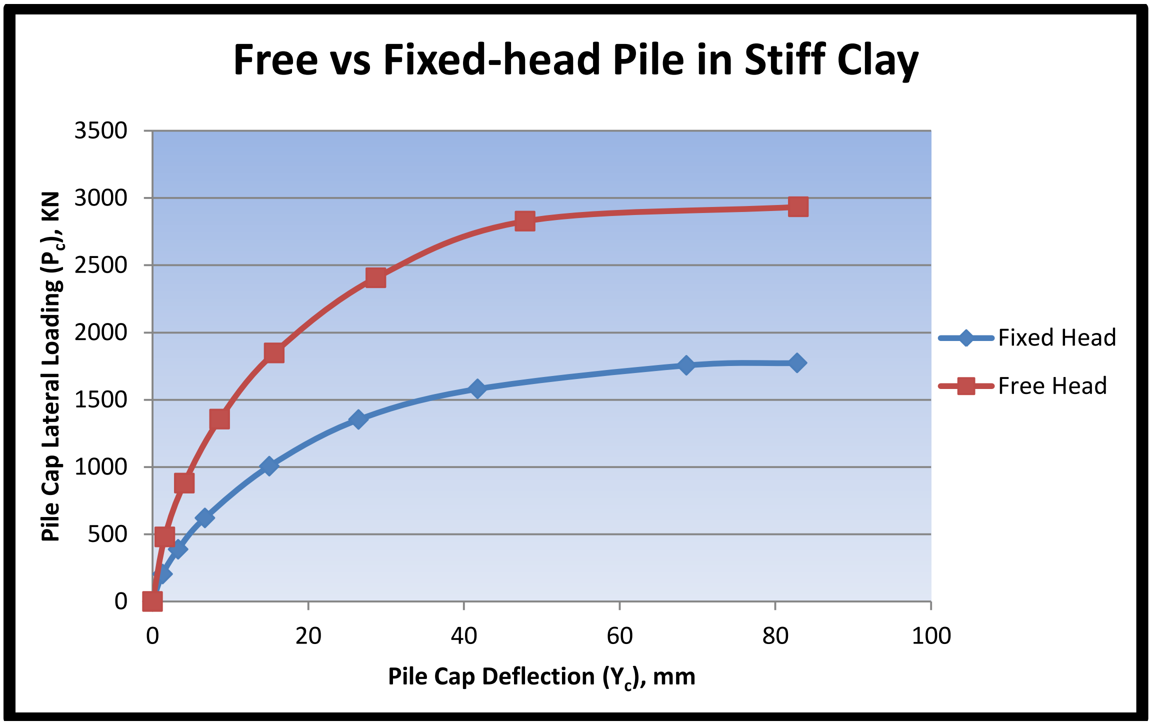


Figure 5.17 Fixed effect of pile-head fixity on the pile cap load in for stiff clay

Shown in Figure 5.18 is the percentage of the total load applied to the pile group-cap system taken by the pile cap versus pile cap deflection for the fixed head pile-cap connection. This percentage ranges from 23 to 31 percent in soft clay, between 38 and 48 percent in medium stiff clay, and between 42 and 53 percent in stiff clay. All three curves have parabolic shape concaved down. Approximately at the 40 mm pile cap deflection, the curve for each type of clay reaches its peak.

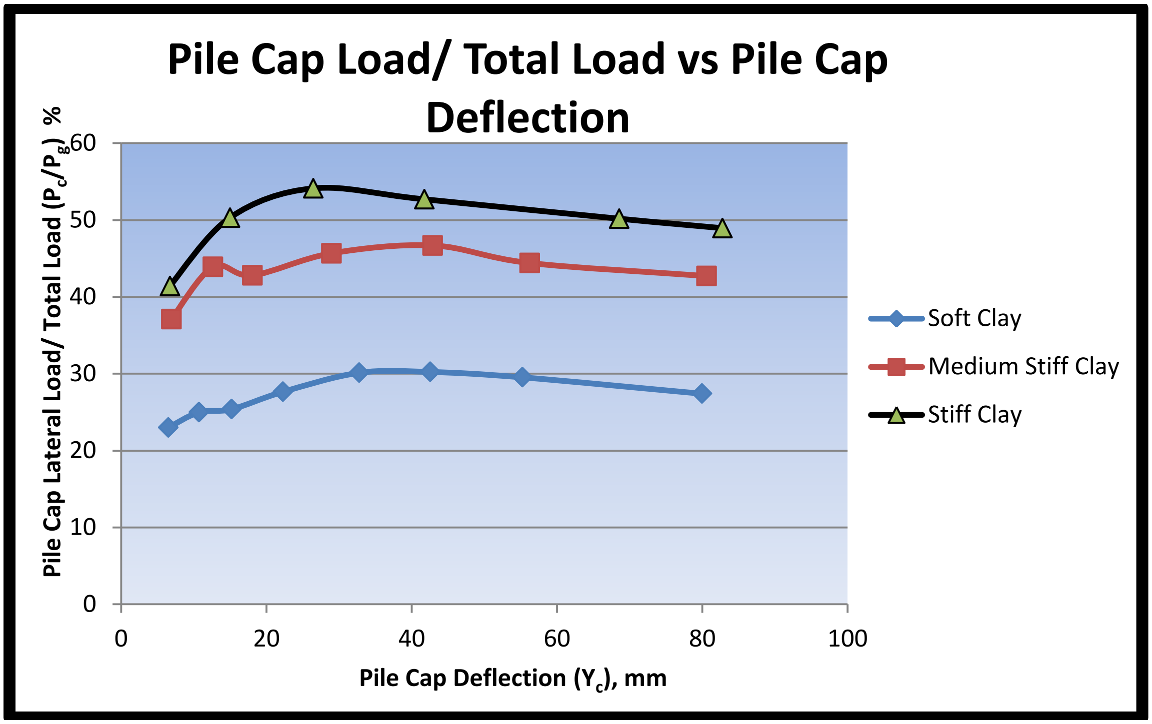


Figure 5.18 Percent of the total load taken by the pile cap vs. pile cap deflection for the fixed head in three types of clay

Figure 5.19 presents the percentage of the total load applied to the pile group-cap system taken by the pile cap versus pile cap deflection for the free head pile-cap connection. It can be noticed that for the free head pile-cap connection, curves that represent all three types of clay are closer to each other more than ones for the fixed head pile-cap connection. Also, in clay, the pile cap takes more lateral resistance for the fixed head pile-cap connection than for the free head pile-cap connection. The percentage of the total load applied to the pile group-cap system taken by the pile cap for soft clay varies from 36 to 42 percent. This percentage ranges from 40 to 44 percent for the medium stiff clay and between 44 and 49 for the stiff clay. Just like in Figure 5.18, curves that represent three types of clay soil have parabolic shape concaved down. The peak of these curves for the free head pile-cap connection is approximately at 30 mm.

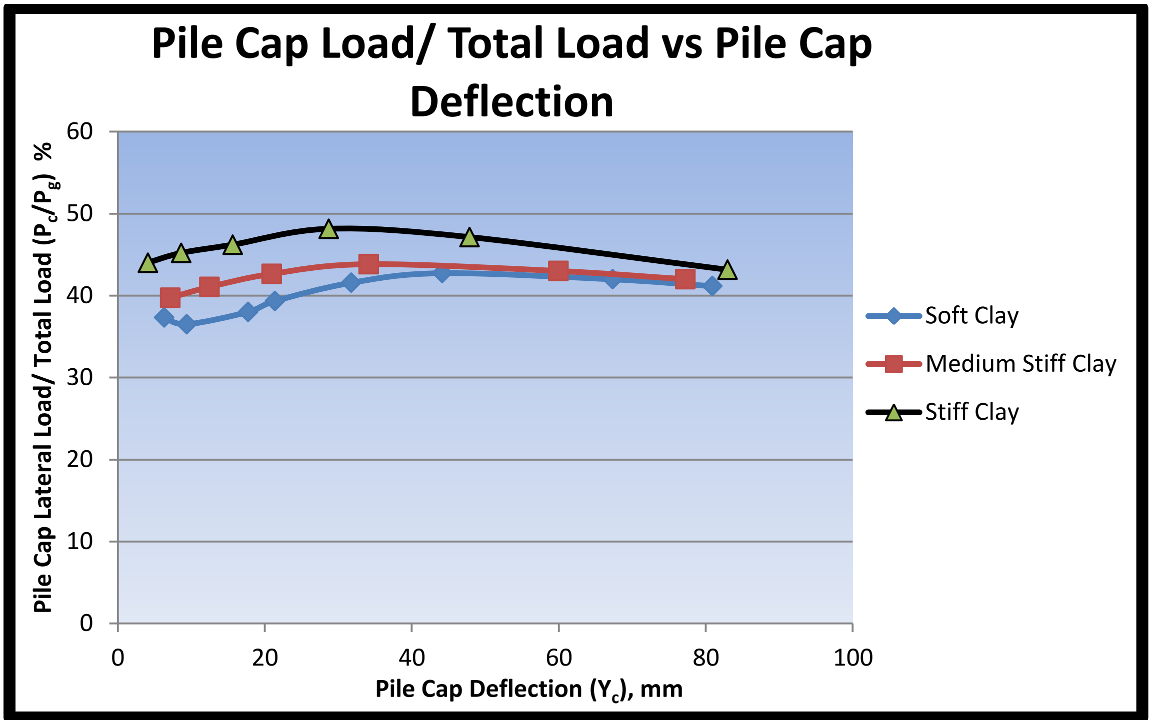


Figure 5.19 Percent of the total load taken by the pile cap vs. pile cap deflection for the free head in three types of clay

5.4.8 Comparison of load taken by the pile cap versus pile cap deflection for clay

Three different types of clay were used as well in order to get a better understanding how the pile cap behaves while being loaded laterally. The difference in these three types of clay is their stiffness. Soft clay had a cohesion of 30, medium stiff clay of 80 and stiff clay of 130 KN/m^2. Figure 5.20, as expected, shows that at the same deflection load taken by the pile cap in stiff clay is going to be higher than for the soft clay for the fixed head fixity. The mechanism of pile group interaction (i.e. passive wedge overlap) in the case of sand is different from the pile group interaction in clay soil. The development of the water pressure in clay soil also affects the way the stresses at the face of the wedge are generated (Ashour and Ardalan 2011).

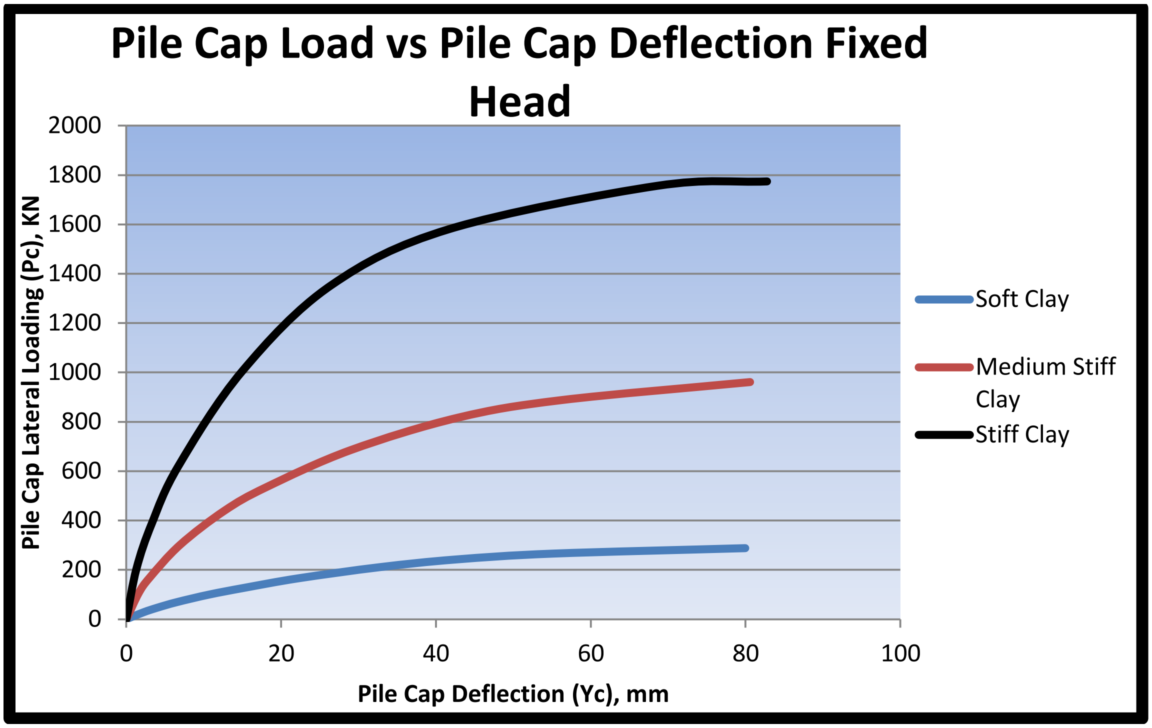


Figure 5.20 Loads taken by the pile cap versus pile cap deflection for fixed-head pile group in three types of clay

The same observation can be made for free head fixity and is presented in Figure 5.21. The difference in the load taken by the pile cap between soft and medium stiff clay for fixed head fixity at the maximum deflection of 80 mm is around 600 KN. For free head fixity, this difference is around 1100 KN. When comparing medium stiff clay and stiff clay, it can be noticed that at the same maximum deflection of 80 mm difference in load taken by the pile cap for fixed head fixity is around 800 KN. For free head fixity, this difference is approximately 1400 KN.

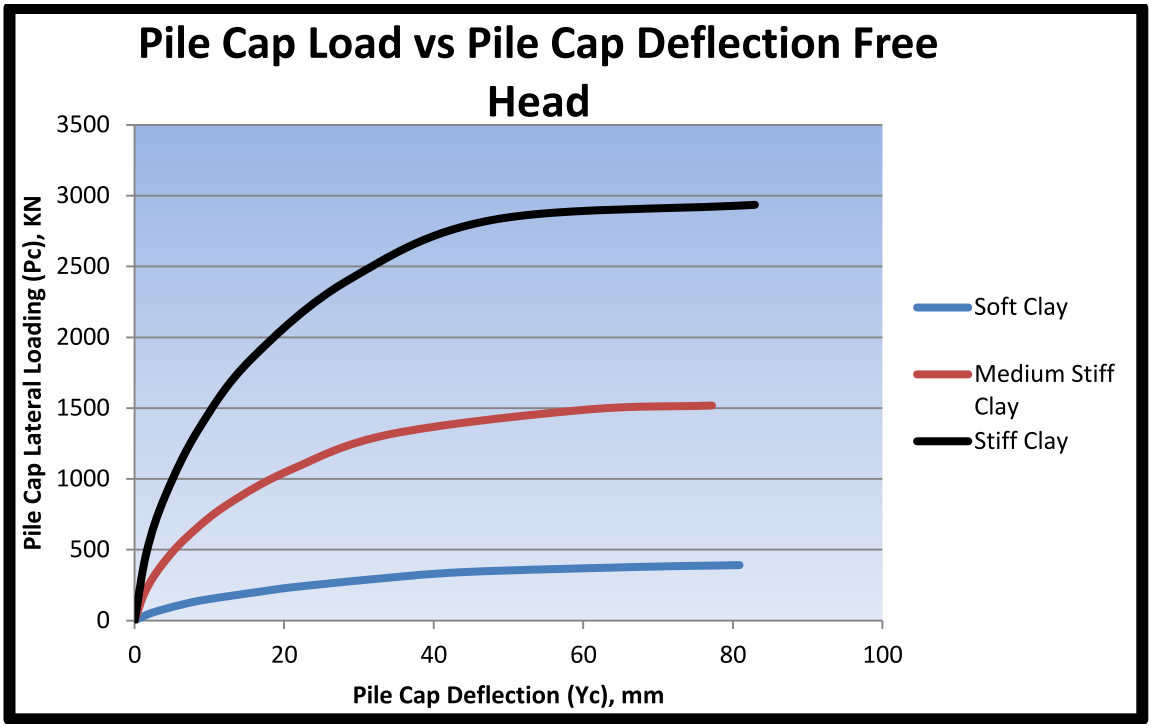


Figure 5.21 Loads taken by the pile cap versus pile cap deflection for free-head pile group in three types of clay

CHAPTER 6

CASE OF STUDIES

6.1 Introduction

The following laterally loaded field test results are used to compare Strain Wedge Model analysis and finite element program Plaxis 3D Foundation. Both techniques are capable of calculating the deflection, shear force and moment for a single pile or a pile group as well as load taken by the pile cap when loaded laterally.

6.2 Cyclic lateral load behavior of a pile cap and backfill (Rollins and Cole 2006)

Rollins and Cole (2006) performed a series of cyclic lateral load tests in Salt Lake City in the period from August of 2000 and August of 2002. All of these tests were performed on full scale pile groups. To get a better understanding of how a pile cap behaves while being laterally loaded, Rollins and Cole (2006) analyzed three different soil conditions. First, a lateral load was applied and two tests were conducted while there was no backfill around the pile cap. Second, four tests were conducted with a different soil surrounding the pile cap. Lastly, Rollins and Cole (2006) performed one test where a narrow trench was excavated around the cap and a lateral load was applied.

6.2.1 Pile configuration and soil properties

A group of twelve steel pipe piles with an outside diameter of 324 mm were used in this case. These piles had a wall thickness of 9.5 mm and were driven 12.2 m into the ground. Spacing used for this pile group was 3.3D (1.06 m) in the direction of lateral loading (x-x) and 4.4D (1.42m) in the opposite direction of the loading (y-y). Piles were filled with concrete and to insure good connection with the pile cap, two different types of reinforced bars (No.8 and No.6) extended 1.06 m into the pile cap. A concrete pile cap used by Rollins and Cole (2006) was 5.18 m wide, 3.05 m long and 1.12 m thick. In order to apply lateral loading to the pile cap, two 4.45 MN hydraulic jacks were used. Described pile properties can be seen in Table 6.1, while test setup and pile group distribution can be seen in Figures 6.1 and 6.2 below.

Table 6.1 Pile properties used by Rollins

Pile Type	**Length (L), m**	**Diameter (D), mm**	**Thickness (t), mm**	**Head Fixity**	**Young's Modulus (E), KN/m^2**
Steel Pipe Pile	12.2	324	9.5	Fixed Head	3.65×10^7

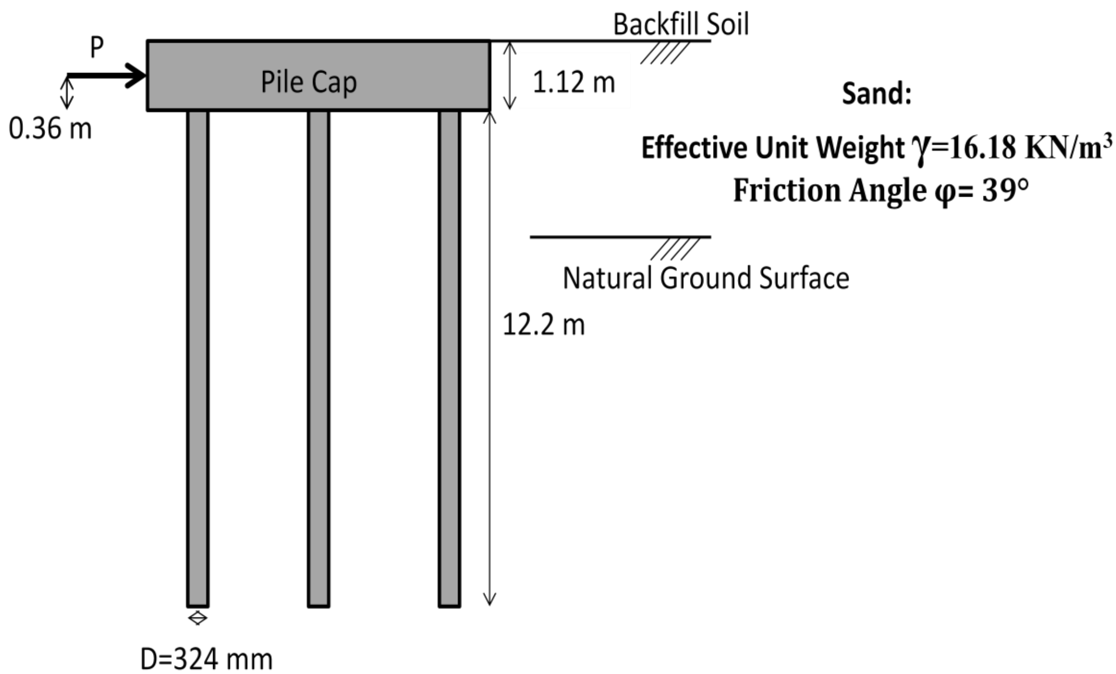


Figure 6.1 Test setup for Rollins test

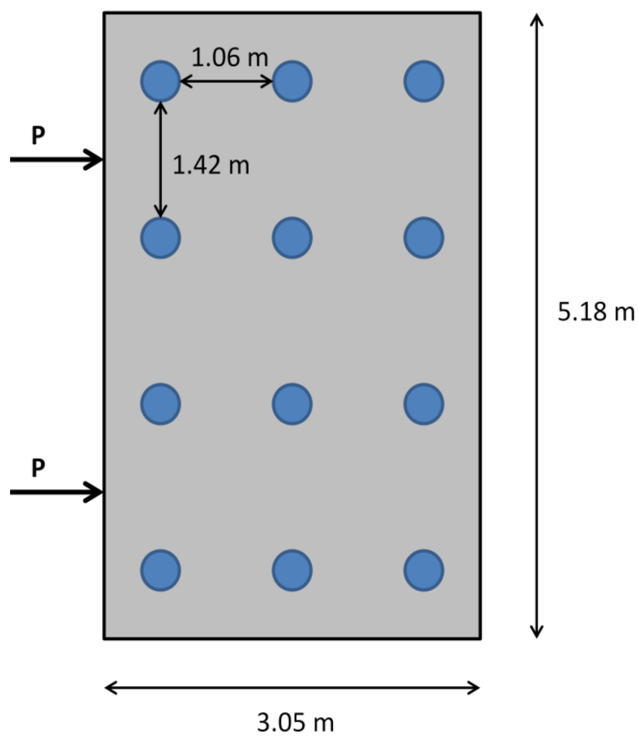


Figure 6.2 Pile group distribution for Rollins test

As mentioned above, Rollins and Cole (2006) removed soil surrounding the cap and replaced it with four different types of soil, but before the soil was removed around the cap, a few tests were conducted in order to analyze natural soil. Properties of this natural soil can be seen in Figure 6.3 below. Four backfill soils that were used were clean sand, silty sand, fine gravel and course gravel. Properties of these soils are listed in Table 6.2.

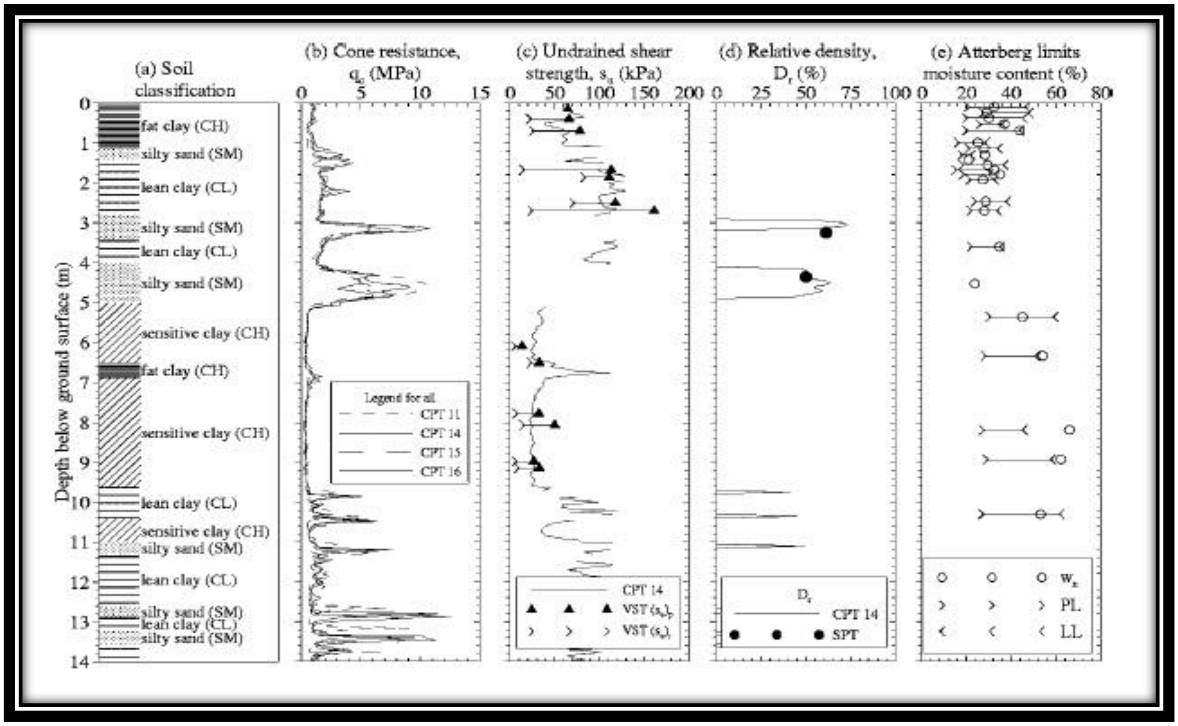


Figure 6.3 Natural soil properties (Rollins and Cole 2006)

Table 6.2 Backfill soil properties used by Rollins

Backfill type	Moisture content, %	Dry unit weight (γ_d), KN/m^3	Friction angle (φ), deg.	Cohesion (S_u), KPa
Clean sand	13.4	16.18	39	0.0
Silty sand	14.6	16.57	27	27.3
Fine gravel	5.5	19.68	34	3.8
Course gravel	4.0	22.23	40	7.2

6.2.2 Field test results compared with Strain Wedge Model and Plaxis 3D Foundation

Figure 6.4 presents a good estimate of results calculated in Strain Wedge Model and Plaxis 3D Foundation compared to results that were acquired from the field tests. It can be noted that the pile cap deflection difference between three curves is larger at the beginning and decreases as the total load applied increases. The total load applied to the pile group represents the load applied to the piles and the pile cap, while the pile cap deflection is equal to the deflection of the pile head. Strain Wedge Model curve presented in the Figure 6.4 is showing a stiffer response for the same load then Plaxis 3D Foundation. This phenomenon is noticed for the first half of the curve. The second part of the curve shows almost identical results for the field tests, Strain Wedge Model and Plaxis 3D Foundation.

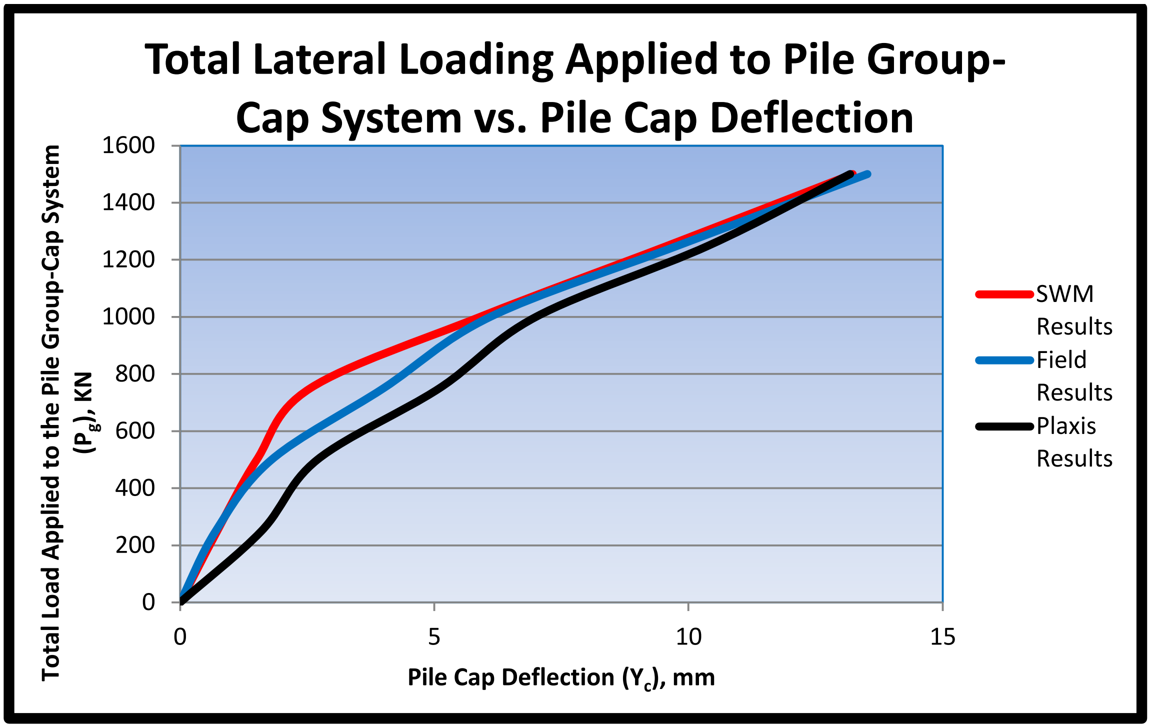


Figure 6.4 Measured and predicted total lateral load applied vs. pile cap deflection (Rollins and Cole 2006)

Figure 6.5 presents total load carried by the pile cap versus total deflection of the pile cap. It can be noticed that the Strain Wedge Model predicted results in the first half of the curve are almost the same as the results acquired in the field. Plaxis 3D Foundation gave softer response but still in a reasonable range. For the second part of the curve, field test results and Plaxis 3D Foundation results are in a good agreement. Strain Wedge Model provided a stiffer response for the second part of the curve but still acceptable results. By performing these tests, we can conclude that the field test results compared to Strain Wedge Model and Plaxis 3D Foundation predictions are in good agreement. The difference between the acquired and predicted results, at the point where there is the largest difference between them, is approximately 15 percent for the load taken by the pile group-cap system and around 20 percent for load taken by the pile cap.

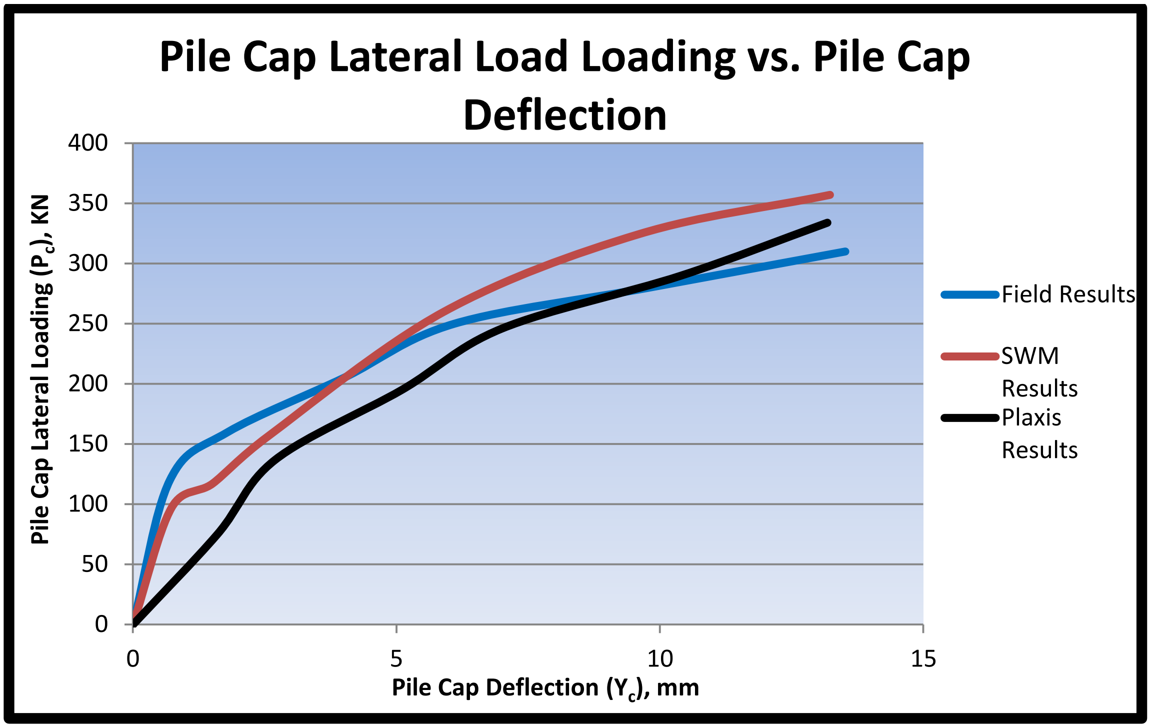


Figure 6.5 Measured and predicted load carried by the pile cap vs. pile cap deflection (Rollins and Cole 2006)

6.3 Experimental evaluation of lateral-load resistance of pile caps (Mokwa and Duncan 2001)

As reported by Mokwa and Duncan (2001), a series of lateral load tests were investigated in order to analyze lateral resistance of pile caps. A special testing facility, constructed just for this occasion was built near Blacksburg, Virginia where Virginia Tech University is located. Observed in this test were three pile groups made of four piles, a concrete wall without pile embedded and two single piles. A total of thirty one lateral load tests were performed at this location from June to October 1998.

6.3.1 Pile configuration and soil properties

Piles used for this case were HP 10 x 42. In order to drive these piles down into ground, a diesel pile hammer with rated energy of 30.5 m·KN at stroke of 2.3 m was used. One of the difficulties that Mokwa and Duncan (2001) came across was an increase of drilling resistance once the piles came in contact with the cemented silt. For two single piles that were drilled into the ground, drilling resistance highly increases at the depth of 5.8 m. However, while drilling piles for the northeast pile group, the drilling resistance increased at the depth of 5 m. The pile groups consisted of four piles that had a center to center spacing of 4D (1.0 m). Piles were driven down 3 m beneath southeast cap and 5.8 m beneath northeast cap. The dimensions of the pile caps were 1.5 m wide, 1.5 m high, with thicknesses of 0.46 and 0.91 m for the northeast and southeast cap, respectively. Test setup and pile group dimensions can be seen in Figure 6.6 and Figure 6.7 below.

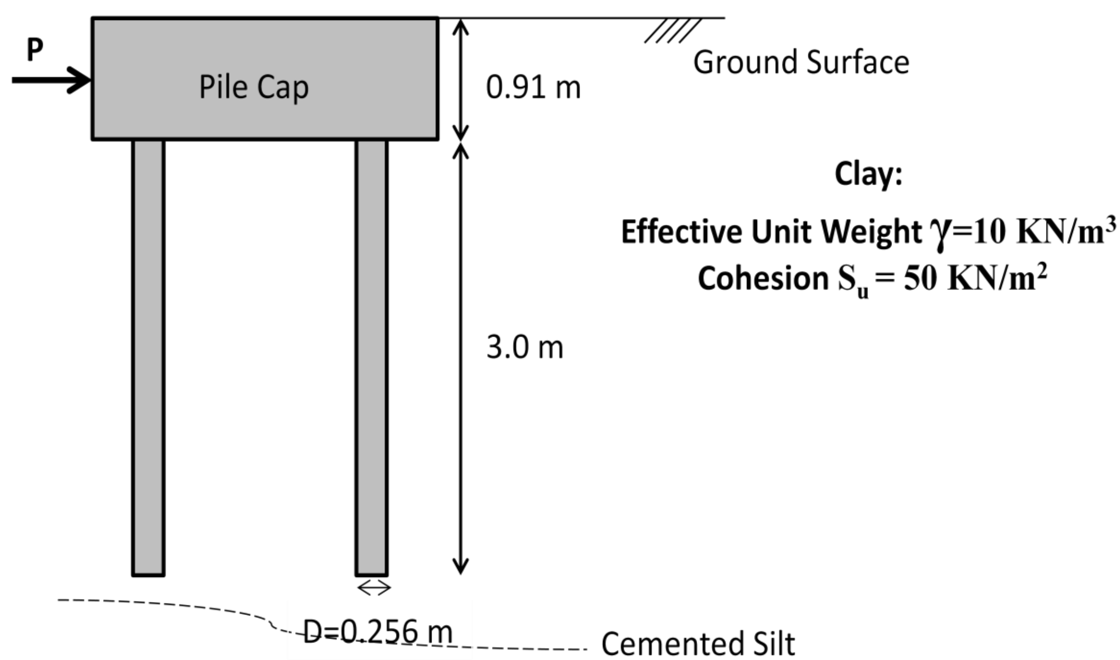


Figure 6.6 Test setup performed by Mokwa

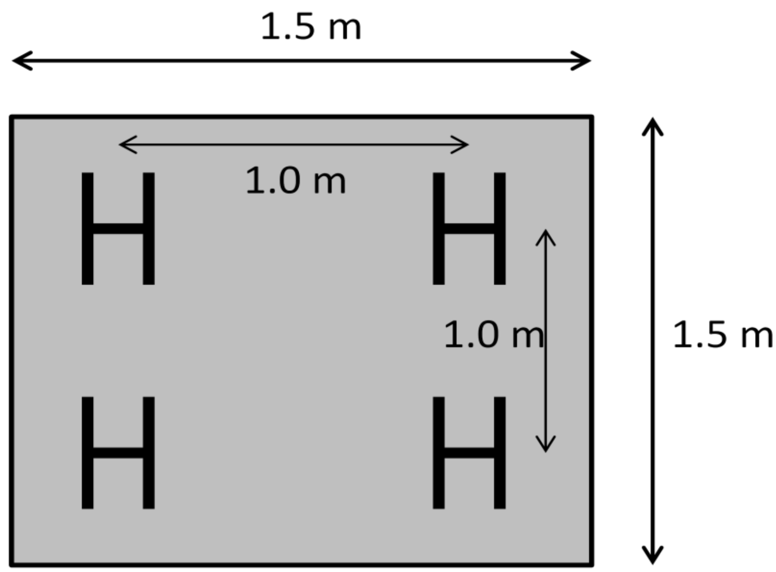


Figure 6.7 Pile group distribution used by Mokwa

It should be mentioned that concrete used for pile caps had an average 28-day compressive strength of 32,900 kPa. Pile caps were reinforced at the top and the bottom of the face for a stable connection with the piles. Properties of the pile used in this test can be seen in Table 6.3.

Table 6.3 Pile properties used by Mokwa

Pile Type	**Length (L), m**	**Width (b), m**	**Thickness (t), mm**	**Head Fixity**	**Young's Modulus (E), KN/m^2**
Steel H Pile	3 and 5.8	0.256	10.7	Fixed-head	4.88 x 10^7

Pile cap resistance was observed by Mokwa and Duncan (2001) in three different conditions: with soil removed from the sides and front of the cap, with natural soil surrounding the cap and with backfill that was filled around the pile cap. Results acquired

from the field test were compared to predicted results calculated in Strain Wedge Model and Plaxis 3D Foundation. Only natural soil surrounding the pile cap was observed in this case. Description of this soil can be seen in Table 6.4 below.

Table 6.4 Natural soil description in Mokwa's test

Depth below ground surface (m)	Soil description
0 – 1.1	Brown silty sand and sandy lean clay with fine sands and frequent small roots
1.1 – 2.7	Dark brown, moist sandy lean clay with occasional gravel
2.7 – 4.0	Brown moist sandy silt with lenses of silty sand
4.0 – 5.2	Brown, moist sandy silt and silty sand
5.2 – 9.0	Light brown sandy lean clay and sandy silt with trace of gravel

6.3.2 Field test results compared with Strain Wedge Model and Plaxis 3D Foundation

Presented in Figure 6.8 is the total load carried by the piles and pile cap versus pile cap deflection. It should be noted that the pile head deflection is equal to the pile cap deflection. Predicted results from Strain Wedge Model and Plaxis 3D Foundation are in a reasonable range with measured results acquired from the field test. Three curves representing the results from the field test, Strain Wedge Model and Plaxis 3D Foundation are almost identical which proves these results are in good agreement.

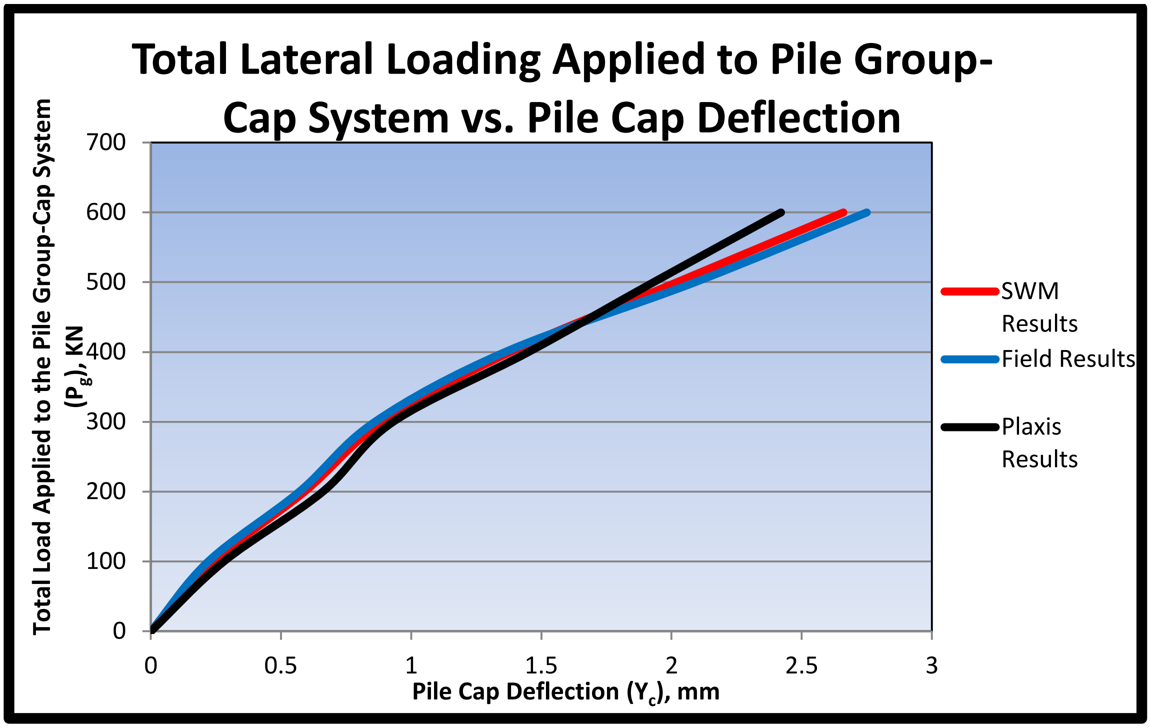


Figure 6.8 Measured and predicted total lateral load applied vs. pile cap deflection (Mokwa and Duncan 2001)

Following is Figure 6.9, which presents load carried by the pile cap versus pile cap deflection for the measured as well as the predicted results. It can be noticed that the pile cap deflection and load carried by the pile cap are almost the same for the first half of the curves for the field test, Strain Wedge Model and Plaxis 3D Foundation. In contrast, for the second part of the curves, Strain Wedge Model and Plaxis 3D Foundation are giving a stiffer response than the field test. Even though these results are not exact, they are still close enough to give a level of confidence that they are in a good agreement.

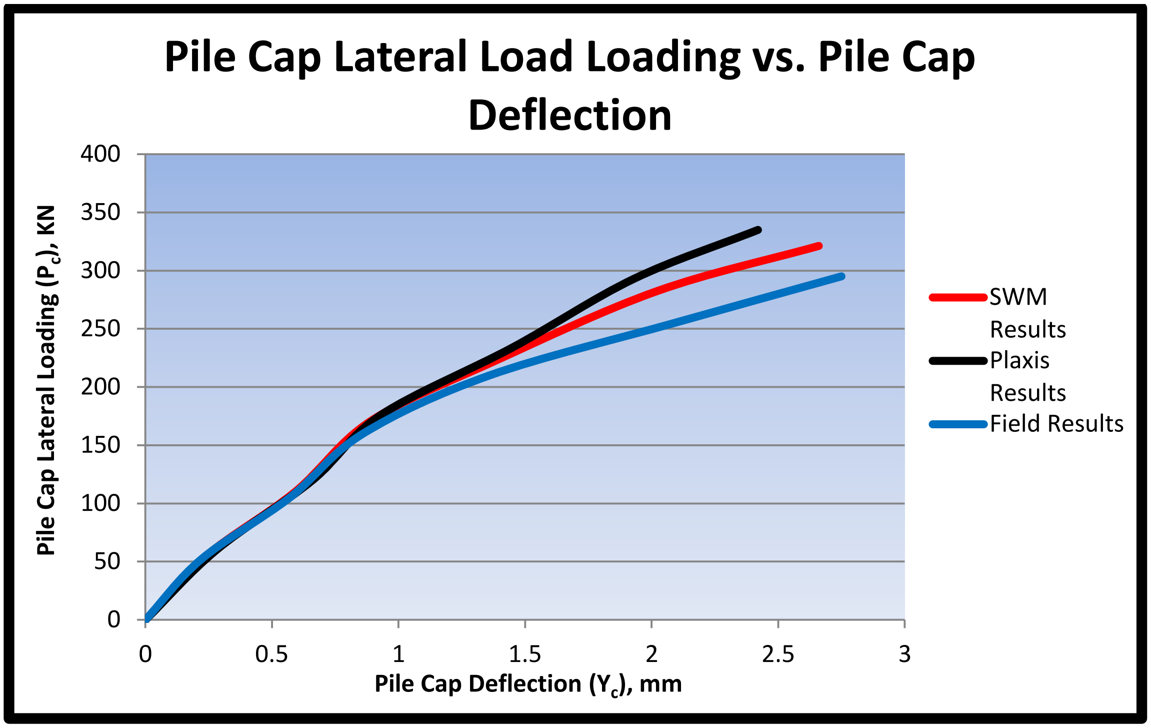


Figure 6.9 Measured and predicted load carried by the pile cap vs. pile cap deflection (Mokwa and Duncan 2001)

6.4 Experimental and numerical study of laterally loaded pile groups with pile caps at variable elevations (McVay et al. 2000)

McVay et al. (2000) performed a series of lateral load tests on the 3x3 and 4x4 pile group-cap systems in loose and medium dense sand in the centrifuge. The purpose of this test was to see how pile cap elevation influences the total lateral resistance of the pile group. Four different cases, with pile caps at the different locations were analyzed. First, the pile group-cap system was not it contact with the soil. Second, the bottom of the pile cap was in contact with the soil. Third, the pile cap was buried into the soil with its top in contact with the soil. Lastly, the pile cap was buried deep into the soil. In this case of

study, case three was analyzed for the 4x4 pile group and its results were verified in the Strain Wedge Model and Plaxis 3D Foundation programs.

6.4.1 Pile configuration and soil properties

Piles used by McVay et al. (2000) were square aluminum piles with a width of 0.43 m and length of 9.7 m. Each pile was drilled into the pile cap, which was made of aluminum as well. The pile cap used by McVay et al. (2000) had dimensions of 5.16 m in length, 5.16 m in width and 1.2 m in thickness. The pile spacing selected was three times the width of the pile. It should be noted that the properties of the piles and pile cap are prototype properties since tests were performed in the centrifuge. Pile properties and pile cap properties can be seen in Table 6.5 and Table 6.6. Also, test setup is presented in Figure 6.10 below.

Table 6.5 Pile properties used by McVay for case 3

Pile Type	Length (L), m	Width (b), m	Head Fixity	Young's Modulus (E), KN/m^2
Aluminum Pile	9.7	0.43	Fixed-head	3.42×10^7

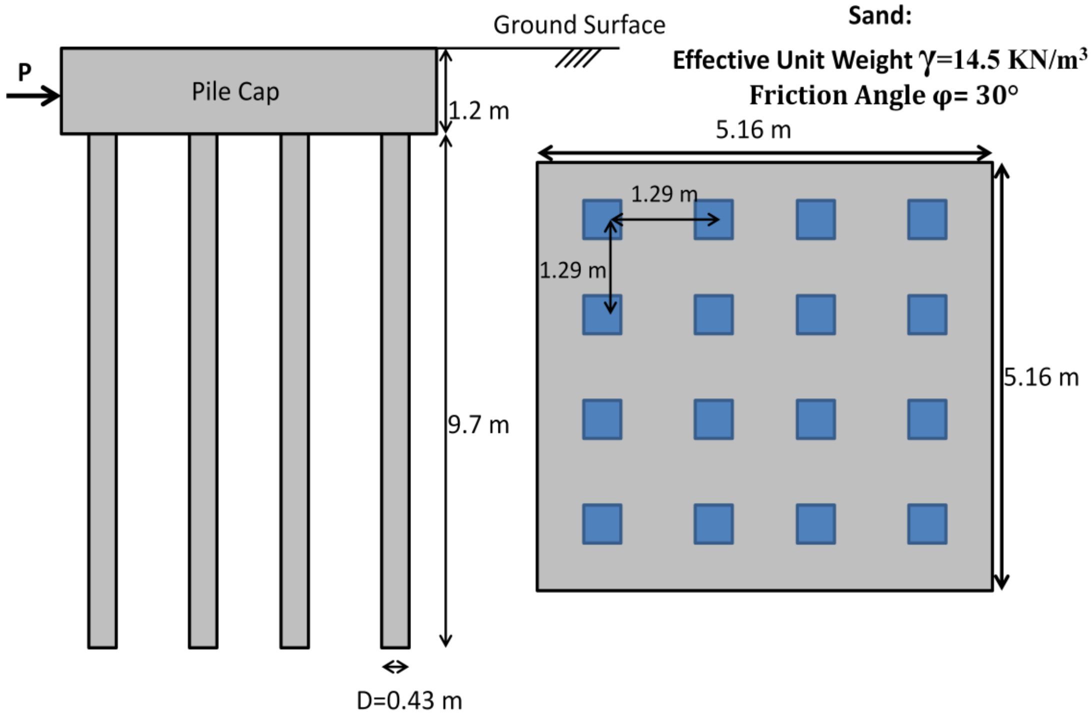


Figure 6.10 Test setup and pile group distribution used by McVay

Table 6.6 Pile cap properties used by McVay for case 3

Pile Cap Type	Width (B), m	Thickness (t), m	Spacing (S), m	Young's Modulus (E), KN/m^2
Aluminum	5.16	1.2	1.29	3.42×10^7

Two types of sand, loose and medium dense were used by McVay et al. (2000) in order to study pile cap resistance. In this case of study, the pile group-cap system was analyzed only in loose sand. Dry unit weight for the loose sand was selected to be 14 KN/m^3 while the friction angle was chosen to be 30.5°. Properties of the loose sand that was placed around the pile cap can be seen in Table 6.7 below.

Table 6.7 Soil properties used by McVay

Soil Type	Dry Unit Weight (γ_d), KN/m^3	Friction Angle (φ), deg
Loose Sand	14	30.5

6.4.2 Lab test results compared with Strain Wedge Model and Plaxis 3D Foundation

Figure 6.11 shows the total load applied to the pile group-cap system versus pile cap deflection. It should be noted that the pile cap deflection is equal to the pile head deflection. As it can be seen in Figure 6.11, Strain Wedge Model is giving a stiffer response than results found in the lab. This is the case for the first part of the graph, while for the second part, these results are almost identical. On the other hand, the Plaxis 3D Foundation curve, at the same pile cap deflection is taking less load then Strain Wedge Model results and results found in the lab. The difference between the acquired and predicted results, at the point where there is the largest difference between them, is approximately 12 percent. Even though these results are slightly off, we can conclude that they are still in a good agreement.

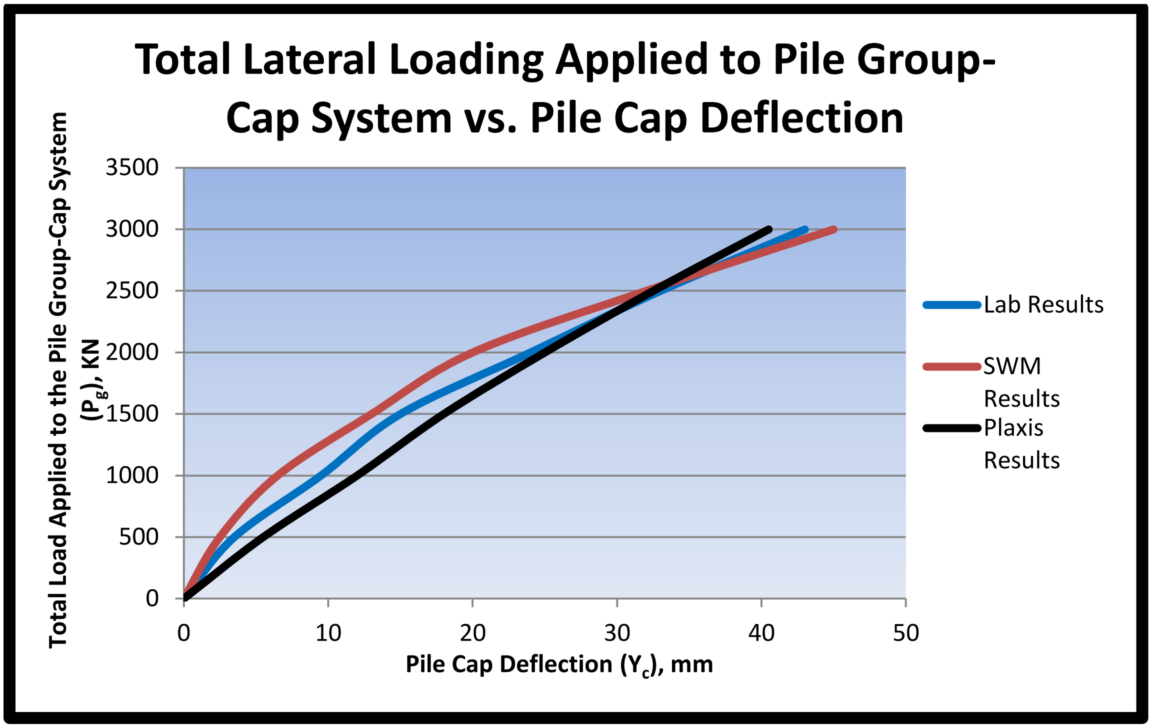


Figure 6.11 Measured and predicted total lateral load applied vs. pile cap deflection (McVay et al. 2000)

CHAPTER 7

SUMMARY AND CONCLUSION

Analytical and numerical analysis have been carried out to determine the effect soil stiffness, pile-cap connection and loading magnitudes have on a pile cap. It was noticed that the pile cap lateral resistance is significant and cannot be neglected. Also, the pile head fixity cannot be ignored since the pile group-cap system with free head and fixed head pile-cap connections is giving different response. The following conclusions are made from this study:

- Pile cap resistance for the free head pile-cap connection is higher than pile cap resistance for the fixed head pile-cap connection at the same deflection in sand
- Pile cap resistance for the free head pile-cap connection is less than pile cap resistance for the fixed head pile-cap connection at the same deflection in clay
- Pile cap resistance for both free head and fixed head pile-cap connection increases as the stiffness of the soil surrounding it increases
- In sand soil, the pile cap with a fixed head pile-cap connection is taking up about 28% of the total load applied to the pile group-cap system
- In sand soil, the pile cap with a free head pile-cap connection is taking up about 43 % of the total load applied to the pile group-cap system

- In sand soil, the percentage of the total load applied to the pile group-cap system that is taken by the pile cap is gradually decreasing as the pile cap deflection increases
- In clay soil, the pile cap with a fixed head pile-cap connection takes up about 53% of the total load applied to the pile group-cap system
- In clay soil, the pile cap with a free head pile-cap connection takes up about 49% of the total load applied to the pile group-cap system
- In clay soil, the percentage of the total load applied to the pile group-cap system that is taken by the pile cap increases at first, then decreaes as the deflection of the pile cap increases

REFERENCES

[1.] Adachi, T., Kimura, M., and Kobayashi, H. (1994). “Behavior of laterally loaded pile groups in dense sand.” *Intl. Conf. Centrifuge 94,* Singapore, 509-514.

[2.] Ashour, M., Norris, G., and Pilling, P. (1998). “Lateral loading of a pile in layered soil using the Strain Wedge Model.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 124, No. 4, pp. 303-315.

[3.] Ashour, M. and Ardalan, H. (2011). ”Employment of the P-Multiplier in Pile-Group Analysis.” *Journal of Bridge Engineering*, 16(5), 612–623.

[4.] Ashour, M., Pilling, P., and Norris, G. M. (2004). “Lateral behavior of pile groups in layered soils.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130, No. 6, pp. 580-592.

[5.] Barton, Y. O. (1984). “Response of pile groups to lateral loading in the centrifuge.”Proceeding of a Symposium on the Application of Centrifuge Modeling to Geotechnical Design, A.A. Balkema, Rotterdam, The Netherlands.

[6.] Beatty, C. I. (1970). “Lateral test on pile groups.” *Foundation Facts*, VI(1), 18- 21.

[7.] Brinkgreve, R.B.J. (2007). Plaxis: 3D Foundation- version 2, Netherlands: Delft University.

[8.] Dewi, S., and Tjie-Liong, G. (2011). “Analysis on laterally loaded group piles by Plaxis 3D Foundation.” Indonesia: Bina Nusantara University.

[9.] Duncan and Chang (1970). “Nonlinear analysis of stress and strain in soils”. *Journal of the Soil Mechanics and Foundations Division*, 96(SM5), 1629-1653.

[10.] El-Garhy, B., El-Nemr, M., and Shalaby, I. (2009). “Effect of pile cap elevation below ground surface on lateral resistance of pile groups-experimental study.” *International Journal of Geotechnical Engineering*, 3(1), 21-28.

[11.] Hoit, M., C. Hays, and M. McVay (1997). The Florida Pier Analysis Program: Methods and Models for Pier Analysis and Design. *Transportation Research Record* 1569, TRB, National Research Council, Washington, D.C., pp. 1-7.

[12.] Kim, J. B., and Singh, L. P. (1974). “Effect of pile cap - soil interaction on lateral capacity of pile groups.” Master of Science, Bucknell University, Lewisburg, PA.

[13.] Kim, J. B., Singh, L. P., and Brungraber, R. J. (1979). “Pile cap soil interaction from full-scale lateral load tests.” *Journal of Geotechnical Engineering*, ASCE, 105(5), 643-665.

[14.] McVay, M.C., Zhang, L., Han, S., and Lai, P. (2000). “Experimental and numerical study of laterally loaded pile groups with pile caps at variable elevations.” *Transportation Research Record*, (1736), 12-18.

[15.] Mokwa, R. L. (1999). “Investigation of the resistance of pile caps to lateral loading.” Ph.D. Dissertation, Virginia Polytechnic Institute and State University, Civil and Environmental Engineering Department.

[16.] Mokwa, R.L. and Duncan J.M. (2000). “Investigation of the resistance of the pile caps and integral abutments to lateral loading” *Report,* Virginia Transportation Research Council, Charlottesville, VA.

[17.] Mokwa, R.L. and Duncan J.M. (2001). “Experimental evaluation of lateral load-resistance of pile caps.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(2), 185-192.

[18.] Nath, U.K., Hazarika, P.J., Giri, V., and Tesfaye, A.M. (2011). “Study of lateral resistance of pile cap using finite element analysis.” *International Journal of emerging trends in engineering and development*, Issue 1, Volume 1.

[19.] Reese, L.C., Cox, W. R., and Koch, F.D. (1974), “Analysis of laterally loaded piles in sand.” Proceedings, Offshore Technology Conference, Houston, TX, Vol. II, Paper No. 2080, pp. 473-484.

[20.] Rollins, M., K., Weaver, T. J., and Peterson, K. T. (1997). “Statnamic lateral load testing of a full-scale fixed-head pile group.” *Report*, Utah Department of Transportation, FHWA.

[21.] Rollins, K.M., and Sparks, A.E. (2002), “Lateral resistance of full-scale pile cap with gravel backfill.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 128(9), 711-723.

[22.] Rollins, K.M., and Cole, R.T. (2006). “Cyclic lateral load behavior of a pile cap and backfill.”*Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 132(9), 1143-1153.

[23.] Schofield, A. N. (1980). “Cambridge geotechnical centrifuge operations.” Geotechnique, 30(3), 227-268.

[24.] Shama, A.A., and Mander J.B., (2004) "Behavior of timber pile-to-cap connections under cyclic lateral loading." *Journal of Structural Engineering*, ASCE, 130 (8), 1252-1262.

[25.] Sejnoha, M. (2009). “GEO FEM – Theoretical manual: A computer program for nonlinear finite element analysis of geotechnical problems.” FINE Ltd.

[26.] Zafir, Z., and Vanderpool, W. E. (1998). “Lateral response of large diameter drilled shafts: I-15/US 95 load test program.” Proceedings of the 33rd Engineering Geology and Geotechnical Engineering Symposium, University of Nevada, Reno, 161-176.