AMERICAN UNIVERSITY OF BEIRUT

# PILED RAFT SYSTEMS: A COHERENT AND SIMPLIFIED DESIGN APPROACH

by

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# PILED RAFT SYSTEMS AND THE ASSESMENT OF REGIONAL METHODS OF ANALYSIS

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# AN ABSTRACT OF THE THESIS OF

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Piled Raft Systems: A Coherent and Simplified Design Approach

Piled rafts are effective and economical foundation systems for towers and superstructures. They have allowed designers to push the bounds of foundation engineering for tall buildings, in addition to providing more economical solutions than standard pile solutions. The piled-raft system is complex due to the substantial interactions between its structural and base soil/rock components. Nevertheless, given the advantages it presents it is important the Structural and Geotechnical Engineers fully understand the behavior of such systems and evolve appropriate design and implementation approaches. Despite the attractive economical features of piled-raft systems and the fact that they have been used for at least two decades, the fundamental understanding based on research in this field is still lagging. This deficiency is clearest in the inconsistency of local and regional design approaches/practices associated with piled rafts. The fact that such designs require collaboration and interaction/input from both Structural and Geotechnical Engineers has contributed to some of the observed/documented inconsistencies. The central objective of the research presented was to investigate the response of piled-raft systems and to understand the relevant interactions between its various constituents through a series of behavioral analyses carried out using finite element software, Plaxis 3D. This step was followed a critical review of local/regional structural design strategies for piled rafts through a broad and in depth collection of information and meetings with local and regional design firms. The current practice was evaluated and recommendations for enhancement were then made/suggested to allow for streamlining and improving the design processes and outcomes. Part of the suggested improvements was based on the adoption/modification of a simplified approach for design that is anchored in the published literature on the interactions within the piled-raft-soil system. The evolved simplified approach was validated/calibrated with the full three-dimensional solutions obtained from the finite element models in Plaxis 3D. Results from the simplified method were compared to those obtained from Plaxis 3D for a wide range of representative design scenarios for piled rafts. In these comparative analyses, parameters of interest (soil properties, number of piles-pile arrangement, loading, raft thickness, etc.) were varied and the results from both sets of models/approaches analyzed and compared. The simplified approach thus validated is a coded in Matlab and was used to produce design-aid charts and regression models, which can be adopted by structural engineers initial input into their full-scale structural models when representing the foundation system and supporting materials/ground.

**Keywords:** Piled rafts, non-linear simplified methods, finite element analyses, Matlab code, Plaxis 3D.

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# CHAPTER I

# INTRODUCTION

## 1.1 Introduction

The function of a foundation is to transfer the structure's load to the ground safely and economically, and to provide serviceability and reliability of the structure. Although the cost of the foundation of a high-rise building is only a small fraction of the total cost (about 10 to 15%), the foundation is one of the main design elements which affect the behavior of the building as a whole.

In foundation design, it is very common to consider initially the use of a shallow foundation system such as the raft. If this system does not provide adequate support to the structure, a conventional piled foundation in which the piles resist the loads would be adopted. In the past decades, there has been a recognition of the effectiveness of a "combined" piled raft foundation system since it may lead to significant economic benefit without compromising the safety and performance of the foundation. The piled raft foundation acts as a composite system consisting of three load-bearing components: piles, raft, and subsoil.

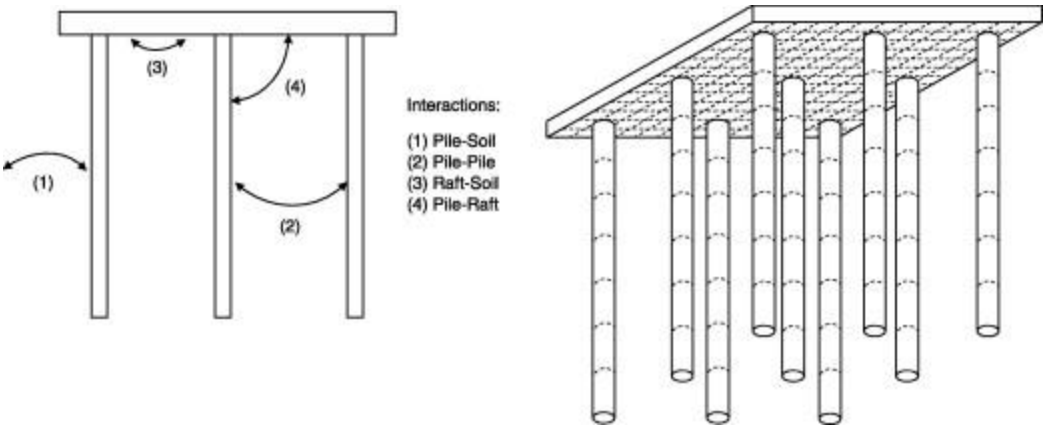


Figure 1. Typical piled raft foundation along with its corresponding interactions

From the literature, it is evident that the piled-raft foundation system was practically implemented about fifty years ago, and the attempt to study its behavior began in the early eighties and intensified in the last few years. However, till date, there is a lack of a universal strategy for designing piled raft foundations. This is due to the complex interactions among the raft, piles and soil, which is three dimensional in nature and cannot be realistically and accurately captured by analytical methods developed so far. Nonetheless, with the advancement of computer technology, researchers are trying to model the complex behavior of the piled raft foundations.

In piled raft systems, piles have been designed to carry the structure's load and the raft provides a rigid platform to connect the piles and distribute the load to the piles and underlying soil. In several design cases, the piles are permitted to yield under the applied design load, while allowing the load capacity of the piles to be exceeded. This will permit the piled raft foundation to support additional loads while ensuring tolerable settlements. This gives rise to the importance of being able to accurately determine the settlement of the foundation as a whole in accordance with the interaction between the foundation's components. As a result, the behavior of piled rafts is determined by complex soil-structure interaction making the understanding of such effects crucial for ensuring a reliable design for piled raft systems.

## 1.2 Thesis Objectives and Significance

### *1.2.1 Objectives and Scope*

The central objective of this research is to investigate the response of piled-raft systems and to understand the relevant interactions between the system's constituents. The main focus will be on studying whether the structural design strategies of piled rafts practiced locally are reliable. This will be done by comparing the local design methods employed at various structural firms with results obtained from realistic and representative finite element analyses carried out on Plaxis 3D.

As a first step in achieving this objective, interviews are conducted with top tier design companies who have had local or regional experience with designing pile-raft raft systems. The interviews will serve the purpose of unveiling the current design practice with regards to the practical design of piled raft. The findings from the interviews will be used as a basis for an investigation whereby the reliability and applicability of adopted design procedures will be assessed by comparison with results from robust finite element analyses conducted using Plaxis 3D. The analysis will shed light on the current state of practice in local design companies and will aid in identifying and recommending revisions or improvements to the current stare of practice to improve its effectiveness in modeling the complex response of pile rafts.

In parallel to the investigation related to the local state of practice, this study aims at assessing the accuracy of published simplified and approximate methods of analysis for piled rafts. Results from these methods will be compared to results obtained from Plaxis 3D for a wide range of representative design scenarios for piled rafts. In this analysis, parameters of interest (soil properties, number of piles, loading type, raft thickness, applied load, etc.) will be varied and the results from simplified methods for analyzing a piled raft will be compared to parametric results obtained from Plaxis 3D. Among the methods that are published in the literature, the main focus will be on tri-linear and non-linear simplified methods for analyzing a piled raft. A Matlab code that includes the formulation of the simplified aforementioned method will be compiled and used to produce results to be compared with the numerical results from Plaxis 3D.

In order to obtain reliable results from the finite element software, it is important to employ suitable constitutive models for the soil. In the case of piled rafts, it is advisable to use the Hardening Soil model which is an advanced model for simulation of soil behavior. The soil stiffness is described precisely in the Hardening Soil model, by using three different input stiffnesses (the triaxial stiffness, $E_{50}^{ref}$; the triaxial unloading/reloading, $E_{ur}^{ref}$; the oedometer stiffness, $E_{oed}^{ref}$). Also, the hardening soil model accounts for stress-dependency of stiffness moduli (i.e. the stiffness increases with pressure), which is not taken into account in the Mohr-Coulomb model (the simplest constitutive law).

Some parameters of the hardening soil model coincide with those of the Mohr-Coulomb model; these parameters are $c, \varphi$ and $\psi$.

### *1.2.2 Significance:*

Piled rafts are effective and economical foundations for towers and superstructures. Not only do piled rafts provide a more economical solution, they also push the envelope for the design of tall buildings. Due to the scarcity of lands and cost constraints, it is very crucial to study and know how to appropriately design and implement such complex foundations. Even though the economic benefits in the literature are well perceived, research in this field is still lagging. Locally, attention needs to be drawn to the structural design of such complex foundation. The work in this thesis is specifically targeted to cater for this need and to improve the current state of design practice for piled rafts. On the basis of the proposed survey outcomes, the present practice will be evaluated so that recommendations for enhancement can be made to attain an improved practice and economical outcome for future projects.

Despite the attractive economical features of piled-raft systems and the fact that they have been used for at least two decades, the fundamental understanding based on research in this field is still lagging. This deficiency, is clearest in the inconsistency of local and regional design approaches/practices associated with piled rafts. The fact that such designs require collaboration and interaction/input from both Structural and Geotechnical Engineers has contributed to some of the observed/documented inconsistencies. . Usually structural engineers are responsible for the analysis and design of superstructures, and they frequently handle the geotechnical part as well. Typically, the design is done through collaboration between the structural and geotechnical engineer. Most companies assign structural engineers to be the main designers and as a result, they are responsible for the pile group analysis and the raft's structural design. The interaction between the structural and the geotechnical engineers is illustrated in Figure 2.

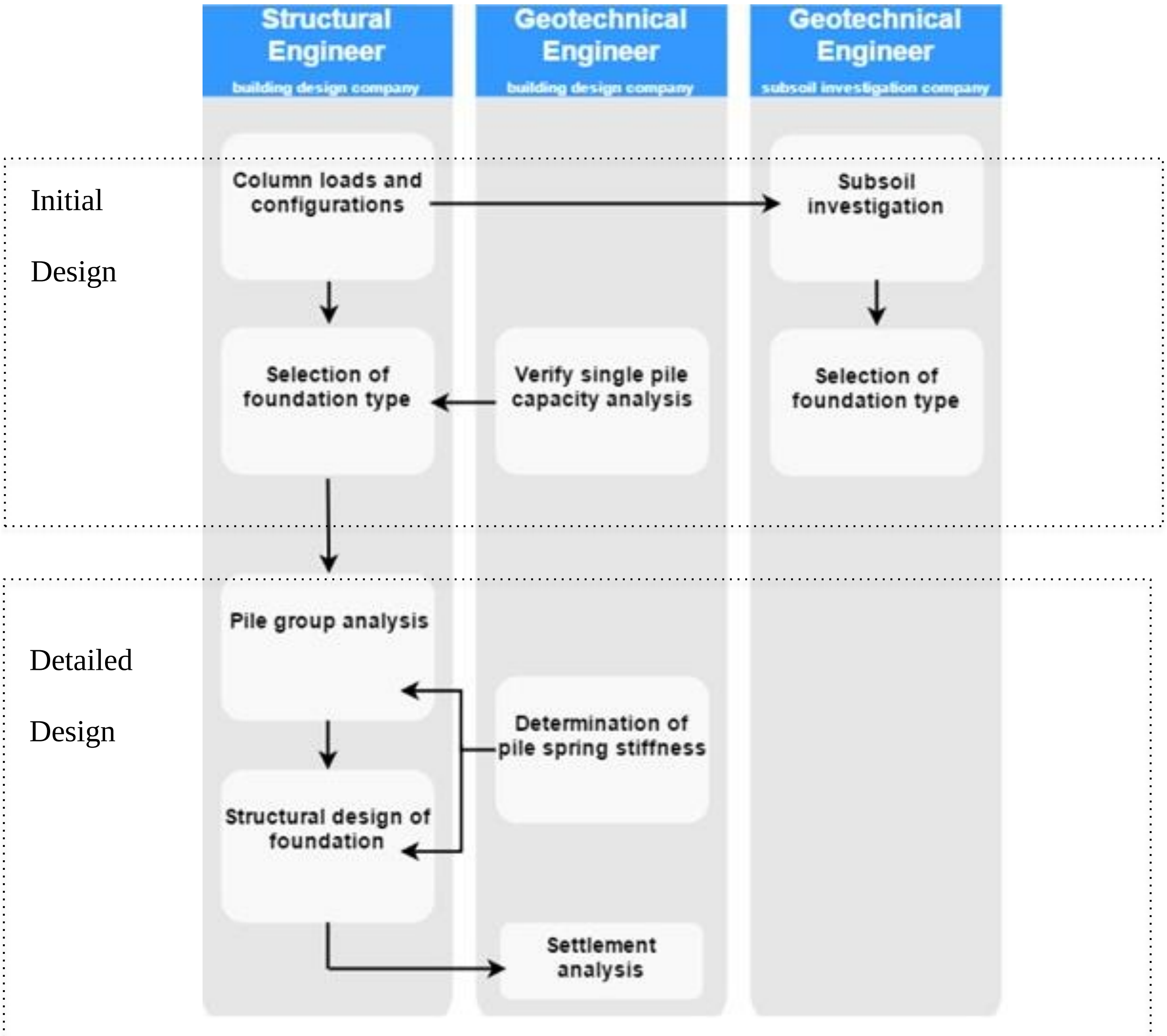


Figure 2. Workflow chart of design between structural and geotechnical groups

## 1.3 Thesis Organization

The Thesis is divided into six chapters. The first chapter is an introduction to the topic and a general overview of the objectives and scope of the research program.

Chapter 2 comprises of a comprehensive background/literature review of the various analytical design models available as well as numerical models and other approaches that are currently used in the design of piled raft foundations. Chapter 2 correspondingly includes some of the attempts to develop approximate numerical models describing the behavior of piled raft foundation as well as the parameters that govern the behavior, in addition to the various approximate design methods proposed to provide a preliminary estimate of the foundation system. A discussion of research campaigns involving finite element and boundary methods are also presented. The advantages of the Finite element method, especially the 3D FEA, relative other methods or solutions are also presented and discussed in this chapter.

Chapter 3 consists of the methodology followed in this thesis: interviews conducted, simplified/modified approach that is suggested.

Chapter 4 consists of a detailed presentation of the FEA tool, Plaxis 3D, with a detailed discussion of the diversity of the input requirements, procedures and output capabilities. In Chapter 5, the 3D finite element model is developed and the results of the different analyses carried out in this parametric study are presented detailing the interaction profiles resulting from varying the different foundation constituent parameters and the interaction. A correlation of the effects the different variables have on the foundation behavior for the various scenarios investigated are presented in Chapter 5, with output correlation graphs including settlement, bending moment profiles as well as raft contribution profiles varying with the foundation constitutive parameters, Es, Np, Lp and tr.

Finally, a summary of the results and a comprehensive discussion of the important findings as well as some recommendations for future development within this research track are presented in the concluding Chapter 6.

# CHAPTER II

# LITERATURE REVIEW

## 2.1 Introduction:

The first attempt to combine shallow and deep foundations started at almost half a century from now. Poulos (2005) reports that Leanardo Zeevaert was the first to recommend the combination of the shallow and deep foundations for the compressible volcanic clay of Mexico City. Also, he should be credited for being the successful introducer of piled raft foundation for the construction of “Tower Latino Americana” in Mexico City in 1957.

Several methods of analysis for piled rafts have been established by various researchers, and until today researchers are trying to develop a suitable model that can simulate the complex raft-soil-pile interaction in a representative way. According to Randolph (1983), Butterfield and Banerjee (1971) were the first to attempt at analyzing this complex soil-structure interaction, but the analysis was done for a small group of piles. Following this attempt, Poulos and Davis (1972) developed an analysis technique for a piled raft foundation, but the most commonly accepted method of analysis is that of Randolph (1994). Furthermore, some relevant works that contributed to the development of piled raft foundations include works by: Hooper (1973), Burland et al (1977), Sommer et al (1985), and Poulos (2002).

Due to the fact that technical design starts with the study of the most suitable choice of the foundation, it is important to know the favorable and unfavorable circumstances for adopting such a foundation. The most effective application of piled rafts takes place when the raft is able to provide adequate load capacity, but the total and differential settlements of the raft exceed the allowable values. Poulos (1991) has examined various idealized soil profiles and concluded that the following situations may be favorable:

a. Soil profiles with relatively stiff clays near the surface

b. Soil profiles consisting of relatively dense sands near the surface

In both of the prior situations, the piles simply ‘boost’ the performance of the foundation rather than provide major means of support.

Contrariwise, there exist situations in which a piled raft foundation is not the best foundation choice, these situations include:

a. Soil profiles containing soft clay near the surface
b. Soil profiles containing loose sands near the surface
c. Soil profiles that contain compressible layers at relatively shallow depths
d. Soil profiles that are most likely to undergo consolidation settlements due to external causes

The raft might not be able to provide significant load capacity and stiffness in the first two unfavorable cases listed above. However, in the third case, the long-term settlement of the compressible underlying layers might decrease the influence of the raft to the long-term stiffness of the foundation system. Furthermore, consolidation settlement may cause a loss of contact between the raft and the soil layer due to consolidation of an active clay layer or due to dewatering. This loss of contact results in an increased load on the piles and an increased settlement of the overall foundation system. In the case of swelling soils, a significant addition of tensile forces may be imposed on the piles because of the action of the swelling soil on the raft. There are various theoretical studies of these situations primarily carried out by Poulos (1993) and Sinha (1997). Finally, it is crucial to note that there exist some situations in which the piled rafts are designed to perform in tension. Such situations include basements as to decrease excavation heave and avoid the foundation uplift.

The subsequent sections of this proposal will highlight the various aspects of piled-raft foundations. Then the literature review proceeds with a design philosophy and a detailed review of all design aspects and considerations.

## 2.2 Design Philosophy and Aspects

### *2.2.1 Design Philosophy*

Randolph (1994) defined three main design philosophies regarding piled raft foundations:

1. Conventional approach: piles are designed to carry most of the loads from the superstructure, and the raft has an insignificant contribution to the load resisting mechanism.
2. Creep piling: piles are designed to work at 70-80% of their ultimate load capacity so as to minimize the raft/soil contact pressures to levels lower than the pre-consolidation pressure of the soil, thus reducing settlements.
3. Differential settlement control: piles are deliberately positioned in predefined locations in order to reduce the differential settlements as opposed to decrease the overall foundation settlements.

Additionally, there exists an extreme implementation of creep piling where the piles are designed to perform at 100% of their ultimate load capacity. These piles are known as settlement-reducing piles and by theory; they do not improve the bearing capacity of the foundation. They are rather used to control maximum displacements as well as internal moments within the raft structure. However, these settlement-reducing piles contribute to both improving the bearing capacity and reducing the settlements of the foundation system.

Various methods of analyzing piled rafts have been developed over the past decade. It is worth noting that the methods below allow any of the above design philosophies to be implemented. Figure 3 conceptually summarizes the load-settlement behavior of piled rafts designed according to the first two philosophies.

In figure 3, curve 0 represents the behavior of the raft acting on its own, which settles excessively under the design load. Curve 1 shows the conventional design philosophy in which the behavior of the foundation system is mainly governed by the pile group behavior. Note that the curve is largely linear at the design load which implies that piles take the majority of the load. The case of creep piling where the piles operate at a lower factor of safety than the conventional case is shown by curve 2 where the raft carries more load than curve 1. Curve 3 illustrates the strategy of using piles as settlement reducers and using the full capacity of piles under the design load. This justifies why curve 3 is non-linear at the design load, but it is important to note that the overall foundation system has an adequate factor of safety and that the settlement criterion is satisfied. To conclude, the design depicted by curve 3 is more likely to be economical than the other design philosophies depicted by curves 1 and 2.

### *2.2.2 Design Considerations*

Poulos (2001) states that the issues that must be studied in the design of a piled raft foundation involve:

1. Ultimate geotechnical capacity under vertical, lateral and moment loading
2. Overall settlement and stiffness
3. Differential settlements and angular rotations
4. Lateral movements and stiffness
5. Structural design of raft and piles

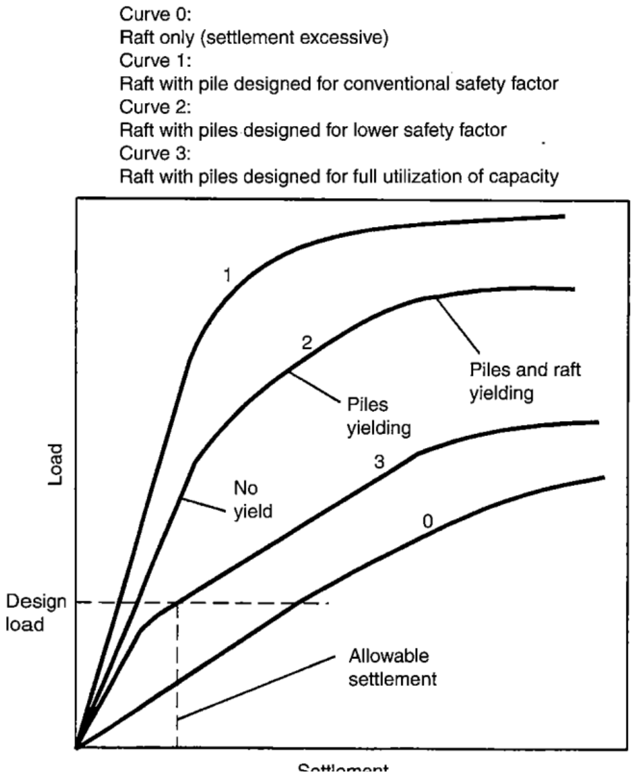


Figure 3. Load-settlement curves for piled rafts according to several design philosophies

The principles for design rely on whether a traditional overall factor of safety method is pursued or whether a limit state design is adopted. In the first situation, the design criterion for the ultimate geotechnical capacity is as follows:

$$R_u = F_s \times F_d \qquad (1)$$

Where, $R_u$= ultimate geotechnical resistance of the foundation system

$F_s$= overall factor of safety (typically ranges from 2-3 for piled rafts)

$F_d$= design loading (overall working load)

For the limit state design, the design criterion for the ultimate limit state is:

$$R_{ud} \geq F_{ud} \qquad (2)$$

Where, $R_{ud}$= ultimate geotechnical resistance of the foundation system

$F_{ud}$= overall loading for the ultimate limit state (usually a combination of factored loadings such as dead, live, earthquake and wind loads).

$R_{ud}$ is generally obtained by factoring down the ultimate geotechnical resistance $R_u$ by a geotechnical reduction factor $\varphi_g$, given by:

$$R_{ud} = \varphi_g \times R_u \qquad (3)$$

The value of $\varphi_g$ ranges from 0.4-0.8 depending on several factors that contribute to uncertainty, comprising of the method of analysis, available geotechnical data, and experiences with similar situations.

For the total settlements, differential settlements and lateral deformations, the design criterion, regardless of whether an overall factor of safety approach or the limit state approach was assumed, is that the maximum movement (i.e. differential movement) has to be equivalent or less than a precise permissible value. This value depends on the nature of the structure.

The structural design of the piled raft foundation system entails an approximation of the following:

- Bending moments and shear forces in the raft
- Axial loads, lateral load, and bending moments in the piles

A complete design method would be capable of tackling all of the above topics in a particular coherent analysis. Such analyses are presented using rigorous three-dimensional numerical analyses, but it is crucial that simple methods be obtainable for both preliminary design reasons and as a validation for the computer-based design techniques.

### *2.2.3 Design Process*

According to Poulos (2001), a reasonable design process for piled rafts comprises of two key stages:

- A preliminary design process to evaluate the feasibility of using a piled raft, and the essential number of piles to fulfill the design requirements.
- A detailed design stage to obtain the optimum number, location, and configuration of the piles in addition to calculating settlement distributions, bending moment and shear forces in the raft, and the pile loads and moments.

In the preliminary stage, it is necessary first to assess the performance of the raft without the piles. If the raft on its own offered only a small proportion of the required load capacity, then it is likely that the foundation will need to be designed with the conventional philosophy. This confirms that the purpose of the raft is merely to lessen the piling requirements marginally. If conversely, the raft has an adequate load capacity, but does not fulfill the total or differential settlement criterion, then the feasible solution is to use the piles as settlement reducers, or to adopt the ‘creep piling’ philosophy. The preliminary analysis involves relatively simple calculations that can be performed without the aid of the computer. However, the detailed stage generally involves the use of a suitable computer program that takes into account the interactions between the soil, piles, and the raft.

Consequently, Poulos (2001) categorized the methods of analysis of a piled raft foundation into three classes:

I. Simplified calculation methods (preliminary design)
II. Approximate computer-based methods
III. More rigorous computer-based methods (detailed design)

#### 2.2.3.1 Simplified Calculation Methods:

The simplified methods include methods by Butterfield and Banerjee (1971), Poulos and Davis (1980), Randolph (1983, 1994) and Burland (1995). All of these simplified methods involve a number of simplifications with regards to modelling the soil profile and the loading conditions on the raft. In this section, the Poulos Davis-Randolph and the Burland’s methods will be discussed.

##### 2.2.3.1.1 The Poulos- Davis- Randolph (PDR) method:

Poulos and Davis (1980) and Randolph (1994) suggested a simple method to assess the vertical bearing capacity of the piled raft foundation as the lesser between:

- Sum of the ultimate capacities of both the raft and piles
- Ultimate capacity of the pile-raft block and the portion of the raft extending beyond the pile periphery

Conventional design approaches can be used to estimate the various required capacities. The load-settlement behavior of the foundation system was estimated through a simple correlation between the individual stiffnesses of the raft and pile group computed using the elastic theory.

A useful extension to this method can be made by estimating the load sharing between the raft and the piles, as outlined by Randolph (1994). The definition of the pile problem considered by Randolph is illustrated in figure 4. The stiffness of the piled raft foundation can be obtained by using his approach as follows:

$$K_{pr} = \frac{K_p + K_r(1-\alpha_{cp})}{1-\alpha_{cp}^2 K_r K_p} \quad (4)$$

Where, $K_{pr}$= stiffness of piled raft; $K_p$= stiffness of the pile group; $K_r$= stiffness of the raft alone; $\alpha_{cp}$= raft-pile interaction factor.

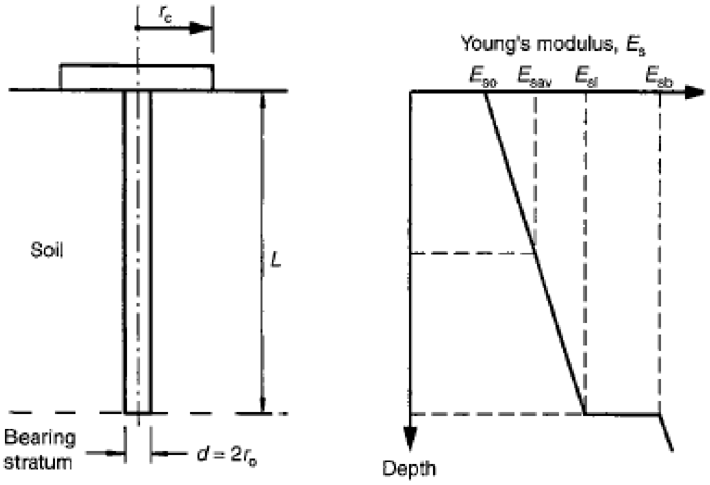


Figure 4. Simplified representation of piled-raft unit

The proportion of the total applied load carried by the raft is:

$$\frac{P_r}{P_t} = \frac{K_r(1-\alpha_{cp})}{K_p + K_r(1-\alpha_{cp})} = X \quad (5)$$

Where $P_r$ = load carried by the raft and $P_t$= total applied load

The raft-pile interaction factor,$\alpha_{cp}$ can be computed as follows:

$$\alpha_{cp} = 1 - \frac{\ln({}^{r_c}/_{r_0})}{\zeta} \quad (6)$$

Where,

$r_c$= average radius of pile cap (correspondes to an area equal to the raft area divided by the number of piles)

$r_0$= radius of pile

$\zeta = \ln({}^{r_m}/_{r_0})$

$r_m$= $\{0.25 + \ \xi\ [2.5\rho(1 - \nu) - 0.25\} \times L\}$

$\xi = E_{sl}/E_{sb}$

$\rho = \ E_{sav}/E_{sl}$

$\nu$= soil's Poisson's ratio

$L$= pile length

$E_{sl}$= soil's Young's modulus at the pile tip

$E_{sb}$= soil's Young's modulus of bearing stratum below pile tip

$E_{sav}$= average soil's Young modulus along the pile shaft

The presented equations are used to develop a tri-linear load-settlement curve as shown in figure 5 below. The stiffness of the piled raft is first computed from equation 1 and this stiffness will remain operative until the pile capacity is fully mobilized. Assuming that the pile load mobilization occurs simultaneously, the total applied load $P_1$ at which the pile capacity is reached is given by the following:

$$P_1 = \frac{P_{up}}{1-X} \quad (7)$$

Where,

$P_{up}$= ultimate load capacity of the piles in the group

$X$= proportion of the load carried by piles

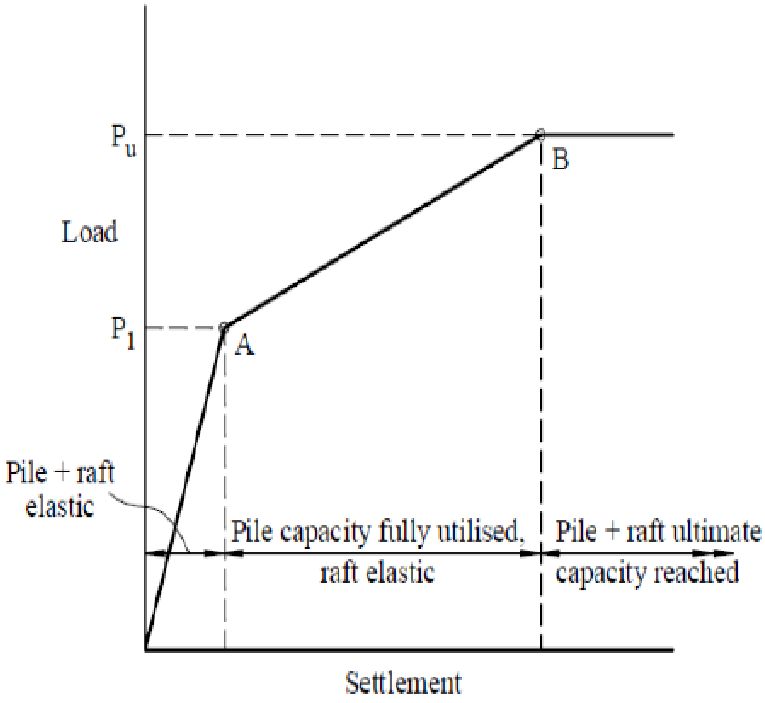


Figure 5. Tri-linear load-settlement curve based on PDR method

Beyond point A in figure 5, the stiffness of the piled raft foundation system is that of the raft alone ($K_r$). This holds until the ultimate load capacity of the foundation system is reached, and is represented by point B in figure 5 after which the load settlement curve becomes horizontal.

The load-settlement curves for a raft with various numbers of piles can be computed with the aid of a computer spreadsheet using a mathematical program such as Matlab or MATHCAD. Despite the fact that these calculations are simplified, they provide rapid means of assessing whether the design philosophies for creep piling or full pile capacity utilization are likely to be feasible.

A method which combines and extends the approaches of Poulos Davis and Randolph includes the following aspects:

- Approximation of load sharing between the raft and the piles, using estimated solutions of Randolph.

- Hyperbolic load-settlement relationships for the piles and for the raft, thus providing a more realistic overall load-settlement response for the piled raft system than the original tri-linear approach that was previously discussed.

Figure 6 shows the load-deflection relationship for the piled raft. As is the case with the tri-linear model, point A represents the point at which the full pile capacity is utilized. Up to that point, both the raft and piles share the load. The calculation process is conducted via a Matlab code.

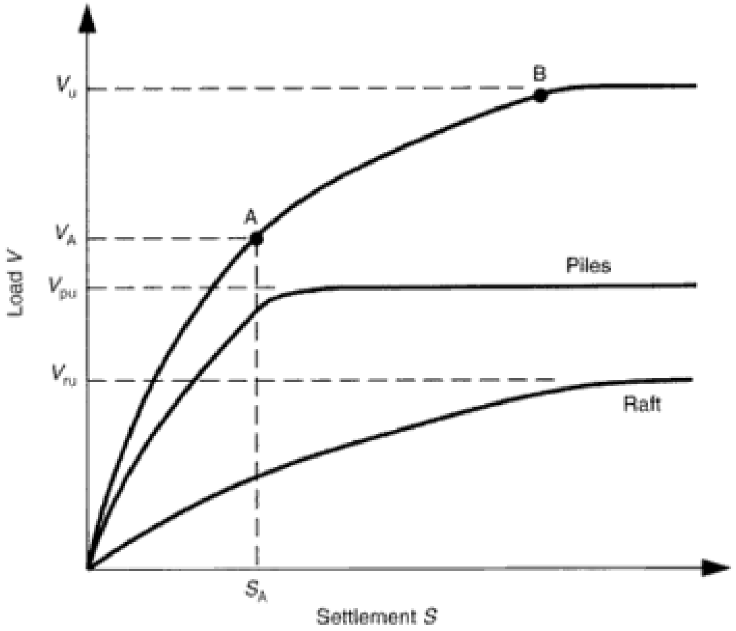


Figure 6. Hyperbolic load-settlement curve

### 2.2.3.1.2 The Burland Method

In 1995, Burland established a simplified design methodology for settlement reducing piles. In the first stage of the analysis, only the raft was considered. The total loads are then applied and finally the resulting settlement is calculated. This first stage is considered the basis of Burland's design process. Subsequently, an acceptable design settlement value is defined and the relevant allowable load capacity is calculated. The difference between the allowable and actual total load applied is transmitted to the underlying piles.

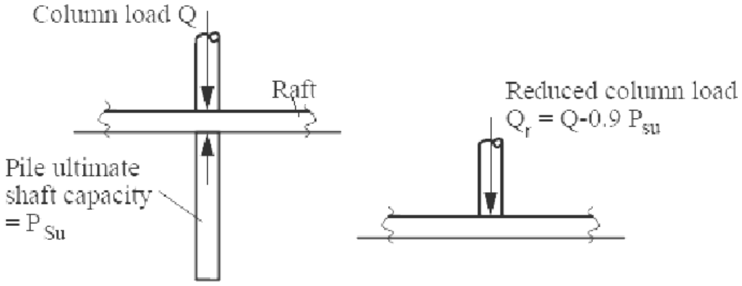


Figure 7. Representation of Burland's method

Finally, settlement of the piled raft foundation and raft bending moments are calculated using an assumed simple raft model with reduced loads applied at the column locations.

### 2.2.3.2 Approximate Computer-Based Methods

The approximate computer-based methods include methods employing a strip-on-springs, plate-on-springs, plate-on beam and springs. These methods are discussed briefly in this section:

#### 2.2.3.2.1 Strip-on-Springs Approach

In the strip-on-springs approach, a section of the raft is represented by a strip, and the surrounding piles are represented by springs. A typical method presented by Poulos (1991) is shown in figure 8. Approximate allowance is made for all components of interaction (pile-raft, raft-pile, raft-raft, and pile-pile). Also, the effects of other parts of the raft outside the strip section are analyzed and taken into consideration by computing the free-field soil movements due to these parts and their interaction with the strip section.

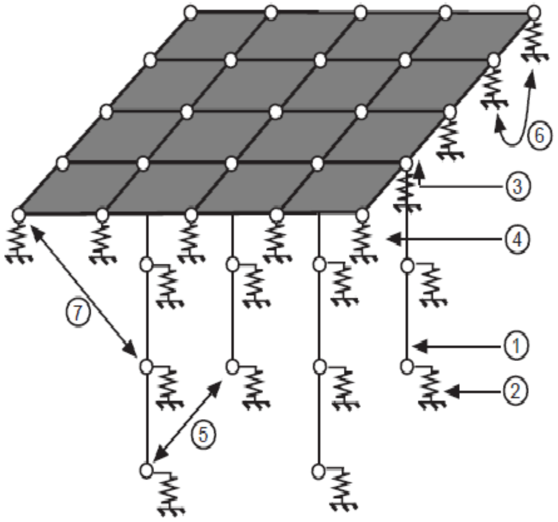


Figure 8. Representation of strip-on-spring approach

This method is adjustable and has shown a relatively reasonable agreement with more comprehensive analyses. Nevertheless, it does have sound limitations, mainly as it is not able to consider torsional moments within the raft, and also because it does not give completely consistent settlements at a specific point if the strip is analyzed in two directions through this point of interest.

This procedure was incorporated into GASP (Geotechnical Analysis of Strip with Piles). Katzenbach (1998) revealed that GASP offered a high degree of conservatism given the great number of assumptions and approximations.

#### 2.2.3.2.2 Plate-on-Springs Approach

A more reasonable representation of the problem would be through adopting the method of a plate-on-springs. In the plate-on-springs approach, the raft is represented by an elastic plate and the piles are modeled as springs supporting the plate. This method is adopted and applied through program GARP (Geotechnical Analysis of Raft with Piles). GARP provides a more realistic and reliable analysis as opposed to the strip on spring approach. One drawback of this method is that the soil response is not appropriately depicted. Similar to the strip-on-spring approach, this approach cannot capture the torsional effects on the overall behavior of the foundation.

Cunha, Poulos and Small (2001) utilized an advanced version of GARP in their parametric study of the feasibility and behavior of piled raft substitutes, according to an optimization process exploring the effect of each of the system's constituent elements on the implementation and cost of the foundation. GARP6 was employed to assess the performance of the rectangular piled raft under vertical loading, while DEFPIG (Deformation Analysis of Pile Groups) was used to estimate the interaction factors between the different piles within the group.

The parametric study was based on the design proposal of Burland (1977) to use settlement-reducing piles. The study examined the effect of the thickness of the raft, number, length and configuration of piles on the overall behavior of the piled raft foundation system. A cost analysis was carried out for each of the systems in order to assess and verify the extent of reduction on the overall cost that can be attained with an enhanced piled raft foundation system when compared to the isolated pile group foundation system. Cunha, Poulos, and Small (2001) testified that the parametric study showed that for a persistent raft thickness, the decrease in the number of piles has a direct influence in decreasing the total cost of the foundation system, while the overall cost of the system leans to increase outstandingly as the thickness of the raft increases.

#### 2.2.3.2.3 Plate-on-Beam and Springs Approach

Kitiyodom and Matsumoto (2003) proposed a simplified analysis method for piled raft foundation system in non-homogenous soils. The approach suggested is based on a development of plate-on-springs approach, where the raft is represented by a thin plate, the piles are modeled as elastic beam elements, whereas the foundation soil is modeled as springs as shown in figure 9. This approach is incorporated into a computer program called PRAB (Piled Raft Foundations with Batter Piles)

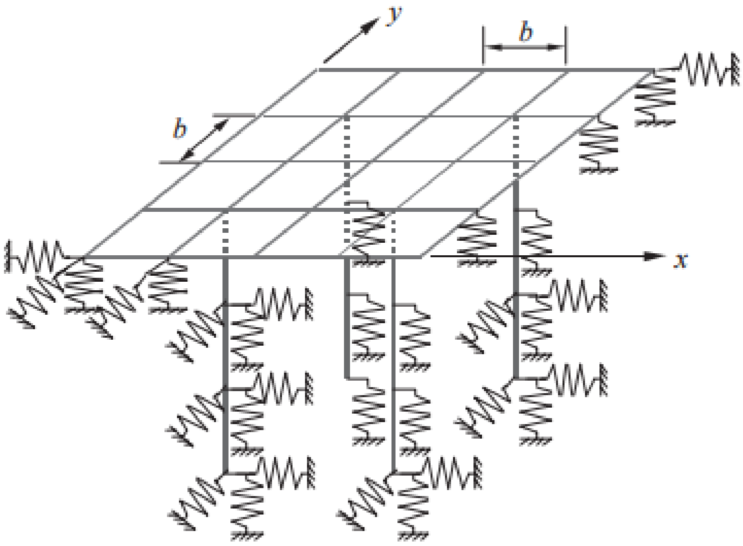


Figure 9. Representation of plate-on-beams and springs approach

A series of estimations were carried out in order to investigate the pile-soil-pile, pile-soil-raft and raft-soil-pile interactions and the structural interaction between the raft and piles. The proposed modeling approach was validated through a series of comparative studies with the results of finite and boundary element analyses reported in the literature. A great agreement with finite element solutions was noticed since PRAB was able to reproduce the behavior obtained from a finite element analysis within an acceptable limit (Kitiyodom and Matsumoto, 2003).

One main characteristic of PRAB is its capability to model different loading conditions while maintaining adequate comparable results to results from finite element analysis. Furthermore, axial, shear and bending moment profiles as well as total and differential settlement profiles are predicted with a great level of certainty when compared with more rigorous computer-based methods (Kitiyodom and Mtsumoto, 2003).

#### 2.2.3.3 Rigorous Computer-Based Methods

This category comprises of methods in which various components of the piled raft foundation systems are modeled in more details than the prior categories. All methods are dependent on computer analysis. Two central numerical techniques have been used: the boundary element method and the finite element methods and sometimes in combination.

#### 2.2.3.3.1 Boundary Element Methods

In this type of method, the raft and each pile within the piled raft foundation system are discretized based on a study done by Kuwabara (1989) for a piled raft in a homogenous elastic soil area. The raft was anticipated to be a rigid element and the compressibility of the piles was considered. It was deduced that the raft was carrying only a small percentage of the load. Furthermore, Poulos (1993) extended Kuwabara’s work to account for the effects of free-field soil movements and for limiting contact pressures between the soil and the raft. However, the limitation of raft rigidity is still a drawback for these types of methods.

#### 2.2.3.3.2 Methods Combining Boundary Element and Finite Element Analyses

In 1978, Hain and Lee studied the analysis of piled raft by representing the raft as a series of thin-plate finite elements, and the pile was represented using boundary element analyses. Furthermore, Sinha (1997) discretized the piles and analyzed them using the boundary element method, whereas the raft was represented by thin-plate finite elements and the soil was assumed to be a homogenous elastic mass. Sinha took into consideration the non-linear behavior by including a limiting raft-soil contact pressures in tension and in compression, and also by specifying limiting stresses between pile shaft and soil, and beneath the pile tip. Additionally, Sinha (1997) was capable of taking the effects of free-field movements into account, thus allowing the effect of consolidation to be studied.

#### 2.2.3.3.3 Simplified Finite Element Analyses

Simplified finite element analyses typically include the representation of the piled raft as a plane strain problem or as an axisymmetric problem. Finite elements are utilized to discretize the raft and the soil and therefore it is relatively simple to take into account the non-linear behavior of the soil and the raft. A verification for the simplified finite element analysis was carried out by Hooper (1973) using an axisymmetric representation on a piled raft foundation in London. The settlement results obtained were similar to the ones observed. However, the main drawback of this simplified approach is that it can analyze only regular loading patterns, and torsional moments for the raft cannot be obtained.

#### 2.2.3.3.4 Three-dimensional Finite Element Analyses

Poulos (2001) states that in terms of the ability to model the real problem, three-dimensional finite element methods are usually considered the ‘ultimate weapon’ to perform a complete analysis. Poulos (2001) adds that the crucial problem of assigning appropriate parameters still remains. Ta and Small (1996) developed a method involving the use of thin-plate finite elements for the raft and a finite layer method for the soil. This method is limited to linear soil behavior, but can efficiently take into account layered soil systems. Wang (1996) carried out a complete analysis. Wang developed a non-linear analysis of vertically loaded piled rafts. The computational effort in three-dimensional analyses is substantial, and results provide benchmark solutions against which simpler methods can be checked. For a rigid raft, it turns out that the results from three-dimensional analyses and from simple elastic methods are in good agreement with each other. However, there are some foundation characteristics that cannot be represented through simple methods such as the lateral response of the pile (even though the loading is vertical), and the non-symmetric distribution of stresses along each pile.

## 2.3 Comparisons of Methods' Capabilities

After getting exposed to several methods of analyses, it is important to compare their capabilities and list their main features. Poulos and Chen (1997) compared the capabilities of these methods by applying them to an idealized hypothetical problem shown in figure 10. The following methods were compared:

- Simplified non-linear method of Poulos and Davis (1980)
- Simplified linear method of Randolph (1994)
- Strip-on-springs analysis using (GASP) and by Poulos (1991)
- Plate on springs, using GARP (Geotechnical Analysis of Raft with Piles), after Poulos (1994)
- Finite element and boundary element approaches by Ta and Small (1996)
- Finite element and boundary element methods by Sinha (1997)

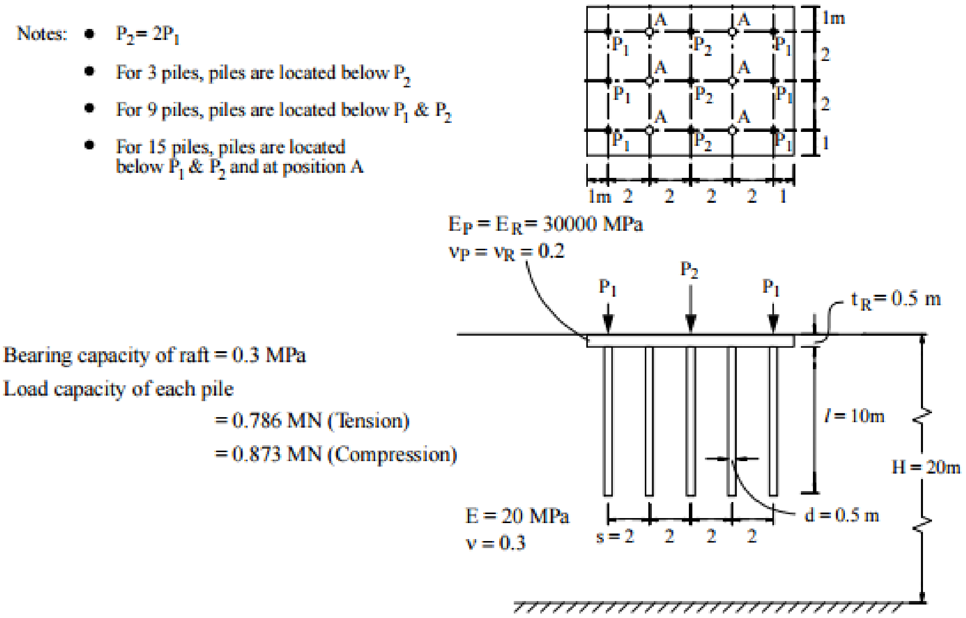


Figure 10. Hypothetical example used by Poulos

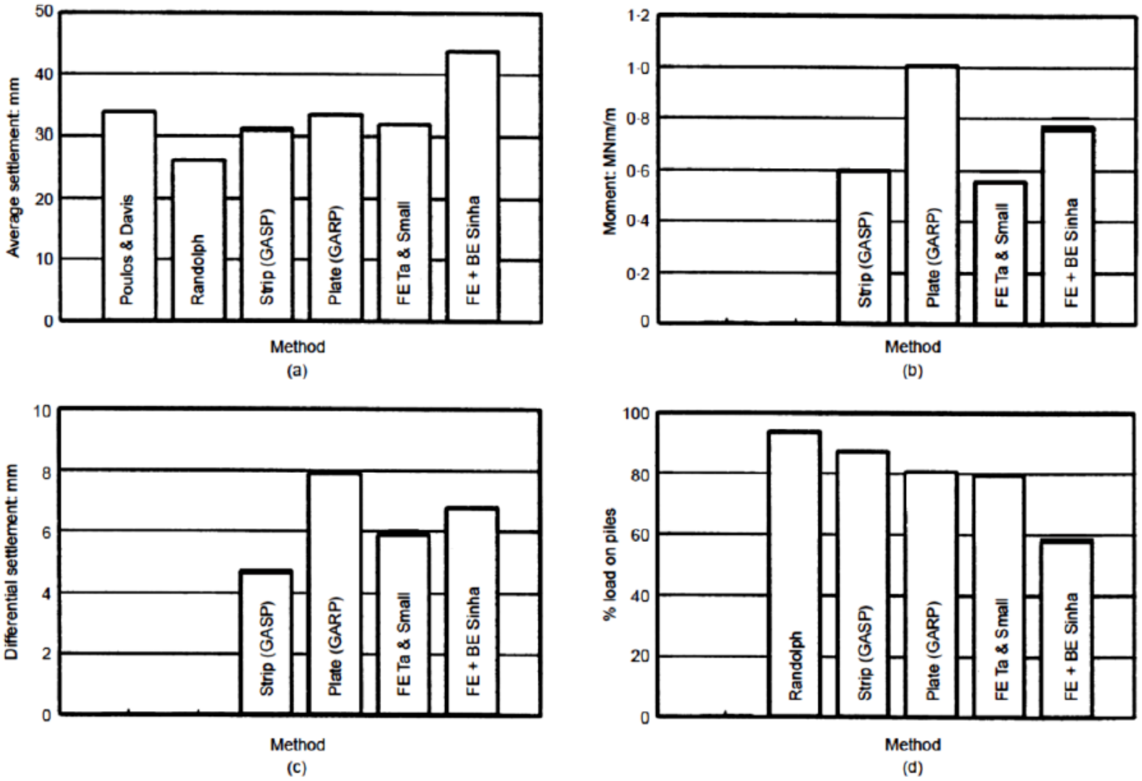


Figure 11. Comparative results for the hypothetical example

Figure 11, shows the comparative results for the hypothetical example of a raft supported by 9 piles, each under a column. The total applied load was 12 MN with $P_1$= 1MN and $P_2$= 2 MN (note: piles labeled A are not present in the analysis). Despite the differences among the various approaches, most of those that include non-linear behavior gave relatively similar results with regards to settlement analysis.

Table 1. Capabilities of methods

| | Response characteristics | | | | | Problem modeling | | | |
|---|---|---|---|---|---|---|---|---|---|
| Method | Total settlement | Differential settlement | Pile load | Raft bending moment | Raft torsional moment | Non-linear soil behavior | Non-linear pile behavior | Non-uniform soil profile | Raft flexibility |
| Poulos&Davis (1980) | ✓ | | | | | | ✓ | | |
| Randolph (1983) | ✓ | | ✓ | | | | | | |
| Van Impe & de Clerq (1994) | ✓ | ✓ | | | | | | | |
| Equivalent raft | ✓ | ✓ | | | | | | | |
| Poulos (1991) | ✓ | ✓ | ✓ | ✓ | | ✓ | ✓ | ✓ | ✓ |
| Brown&Wiesner (1975) | ✓ | ✓ | ✓ | ✓ | | | | | ✓ |
| Clancy&Randolph (1993) | ✓ | ✓ | ✓ | ✓ | ✓ | | | ✓ | ✓ |
| Poulos (1994) | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| Kuwabara (1989) | ✓ | | ✓ | | | | | | |
| Hain&Lee (1978) | ✓ | ✓ | ✓ | ✓ | ✓ | | ✓ | | ✓ |
| Sinha (1997); Frank et. al (1994) | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | | ✓ |
| Hooper (1973) | ✓ | ✓ | ✓ | ✓ | | ✓ | ✓ | ✓ | ✓ |
| Hewitt&Gue (1994) | ✓ | ✓ | ✓ | ✓ | | | | ✓ | ✓ |
| Lee (1993) | ✓ | ✓ | ✓ | ✓ | ✓ | | | | ✓ |
| Ta&Small (1996) | ✓ | ✓ | ✓ | ✓ | ✓ | | | | ✓ |

A more detailed study has been made of the preceding hypothetical case in order to examine various characteristics of piled rafts behavior. Varying the following parameters carried out a parametric study:

- Number of piles
- Raft thickness
- Pile length
- Pile diameter
- Soil characteristics

Poulos (1994) concluded the following results when carrying out the parametric analysis on GARP:

- Increasing the number of piles does not necessarily enhance the foundation performance, and there is an upper limit to a convenient number of piles.
- The raft thickness has a substantial effect on differential settlement and bending moments, but has no significant effect on load sharing and maximum settlement.
- In order to control differential settlement, it is more practical to accommodate in the design a strategic location of relatively small number of piles rather than a large number of piles that are evenly distributed over the raft area.
- The type of the applied load has a greater effect on differential settlement and bending moment as opposed to the maximum settlement or load sharing between the raft and the piles.

Various methods are employed for the analysis of piled raft foundations. These methods range from simple methods to complex three-dimensional finite element analyses. The ability of the methods varies in providing comparable results in terms of differential settlement as opposed to maximum settlement and load sharing between the piles and the raft (which are relatively comparable).

Horikoshi and Randolph (1998) have studied the performance of piled rafts and generated guidelines for economical design. They suggested the following for an ‘optimum’ design of uniformly loaded piled rafts:

- Piles should be placed in the center of the raft (over the central 16-25% of the raft)
- The pile group stiffness should be equal to the axial stiffness of the raft on its own
- The total pile capacity should be designed for almost 40-70% of the design load; this percentage depends on the ratio of the area occupied by the pile group to that of the raft.

As for the case of concentrated loading, some of the guidelines above may not be appropriate, but they are a starting point for design.

The main hindrance to the increased use of piled raft foundations happen to be two-folded: an intrinsic conservatism in foundation design by geotechnical engineers (foundation engineers) and margins imposed by building codes on a minimum factor of safety that needs to be employed. Yet, the increasing number of successful piled raft implementations, and the fact that all preceding studies demonstrated that piled raft foundations have the potential to offer economical foundation systems, under the appropriate geotechnical conditions give rise to the need to have a thorough understanding of the mechanics of piled rafts.

# CHAPTER III

# METHODOLOGY

This chapter encloses a critical review of local/regional structural design strategies for piled rafts through a broad and in depth collection of information and meetings that were held with local and regional design firms. The current practice is evaluated and recommendations for enhancement were then made/suggested to allow for streamlining and improving the design processes and outcomes. Part of the suggested improvements was based on the adoption/modification of a simplified approach for design that is anchored in the published literature on the interactions within the piled-raft-soil system.

## 3.1 Interviews:

Interviews with top structural design companies in Lebanon were performed. The interviews included several questions with regards to designing a piled raft foundation. Some of the questions that were asked included general broad questions such as, “how are piled raft foundations designed at the company?” and “what commercial software is used for the design of such foundations?” also, the questions aimed at critically addressing how such companies deal with the complex soil-structure interaction. Finally, questions regarding the assumptions taken for the piles, soil and raft stiffness’s are posed.

Interviews were held with companies A, B, C and D.

Most of the interviewees were very responsive and answered almost all the posed question.

*Company A’s brief response:*

The foundation is modeled on SAFE, piles are represented by springs with an assigned axial stiffness and the soil subgrade is obtained from a plate load test performed at the field. Iterations are performed in order to finally get a “logical” assumption of the pile stiffness value. The analysis assumes that the piles carry above 80% of the total applied load; this defies the economic purpose of a piled raft foundation.

*Company B’s brief response:*

The foundation is modeled on the commercial program ROBOT. Similar to company A, the piles are represented by assumed values of springs, but the soil’s subgrade modulus is obtained from a geotechnical company’s finite element analysis.

*Company C’s brief response:*

The foundation is modeled on either SAFE or ROBOT. The piles’ stiffness and soil’s subgrade reaction are both assumed based on experience as a function of the pile diameter. Iterations are conducted until suitable values are obtained.

The responses of the interviewed companies show that the design that is practiced locally does not take into consideration the complex interaction of the piled raft foundation’s constituents. Moreover, the design practiced at these companies is rather a “very safe” design for it allows the piles to carry almost more than 80% of the applied load; this eradicates the economic benefit of a piled raft foundation. The current state of design results in a loss of time and money due to the consecutive iterations. It was found that there exists some inconsistency in local and regional design approaches/practices associated with piled rafts. The fact that such designs require collaboration and interaction/input from both Structural and Geotechnical Engineers has contributed to some of the observed/documented inconsistencies. This give rise to the important of finding a suitable method of analysis that the companies may use so as to reduce the loss in terms of money and time.

It is out of question that finite element analysis is the best approach that could be used for design, but since all the structural companies are used to specific commercial programs, and the fact that finite element programs such as Plaxis 3D are very expensive, it deems impossible to change the system and upgrade to using finite element analyses. Nevertheless, it is worth mentioning that only a few geotechnical companies in Lebanon already use the finite element analyses for their design.

The on-going interaction between structural and geotechnical engineers results in delay in the project due to assumptions done regarding the soil's subgrade modulus and regarding the stiffness of a pile. An outcome from the interviews performed would be that mainly all the companies start with a random assumption of the stiffness of a pile. This "out of the blue" assumption can or cannot be valid, but it typically requires at least 3-4 iterations to be able to reach a safe value. The outcome of the interviews conducted raises the importance of performing the study that could aid these companies by providing them with an equivalent pile stiffness ratio obtained from a simplified method that is as close as possible to results from Plaxis 3D. The simplified method studied in this thesis is formulated using a Matlab code. The code would help save the time and money.

## 3.2 The Hyperbolic Poulos-Davis-Randolph Method

The generated code is based on a hyperbolic load-settlement relationship. An extension to the Poulos-Davis and Randolph method is implemented and enhanced in this paper. The method is based on calculation of the total stiffness of the piled raft by means of stiffness of pile group and stiffness of unpiled raft in isolation.

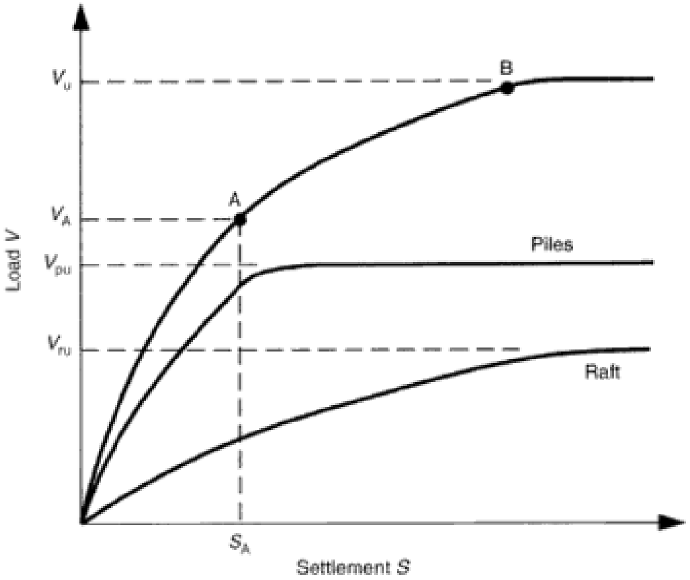


Figure 12. Load-settlement curve based on PDR method

Settlement of the piled raft foundation as whole system is given by:

$$S = \frac{V}{k_{pr}} \qquad (8)$$

Where,

$S$=settlement of the piled raft foundation

$V$=applied load

$k_{pr}$=stiffness of the piled raft system

Beyond point A in the graph shown above, the raft must carry additional load and the settlement would be given by:

$$S = \frac{V_A}{k_{pr}} + \frac{V - V_A}{k_r} \qquad (9)$$

Where,

$V_A$ =Applied load at which pile capacity is mobilized

$k_r$ =Axial stiffness of the raft

$$V_A = \frac{V_{pu}}{\beta_p} \qquad (10)$$

Where,

$V_{pu}$ =Ultimate capacity of piles

$\beta_p$ =Proportion of the load carried by piles

$$k_{pr} = X \times k_p \qquad (11)$$

Where,

$k_p$ =Secant stiffness of pile group

$$X = \frac{1-0.6(k_r/k_p)}{1-0.64(k_r/k_p)} \qquad (12)$$

$$\beta_p = {}^{1}\!/_{(1+\alpha)} \qquad (13)$$

$$\alpha = \frac{0.2}{1-0.8(k_r/k_p)} \times (k_r/k_p) \qquad (14)$$

Load-settlement relationships are assumed to be hyperbolic so the secant stiffness of the piles and raft are the given as:

$$k_p = k_{pi}[1 - R_{fp} \times (V_p/V_{pu})] \qquad (15)$$

$$k_r = k_{ri}[1 - R_{fr} \times (V_r/V_{ru})] \qquad (16)$$

Where,

$k_{pi}$ =Initial tangent stiffness of pile group

$R_{fp}$ =Hyperbolic factor for pile group

$V_p$ =Load carried by the piles

$V_{pu}$ =Ultimate capacity of piles

$k_{ri}$ =Initial tangent stiffness of raft

$R_{fr}$ =Hyperbolic factor for raft

$V_r$ =Load carried by the raft

$V_{ru}$ =Ultimate capacity of the raft

Load carried by the piles is given by:

$$V_p = \beta_p \times V \leq V_{pu} \qquad (17)$$

Load carried by the raft is given by:

$$V_r = V - V_p \qquad (18)$$

By substituting equations (3) and (11) in equations (1) and (2), the following is obtained:

$For\ V \leq V_A$:

$$S = \frac{V}{X \times k_p} = \frac{V}{X \times k_{pi}[1 - R_{fp} \times (V_p/V_{pu})]} \qquad (19)$$

$For\ V > V_A$:

$$S = S_A + \frac{V - V_A}{k_{ri}[1 - R_{fr} \times (V_r/V_{ru})]} = S_A + \frac{V - V_A}{k_{ri}[1 - R_{fr} \times ((V - V_{pu})/V_{ru})]} \qquad (20)$$

Where,

$$S_A = \frac{V}{X \times k_{pi}(1 - R_{fp})} \qquad (21)$$

Basically, the method shown above enables the user to construct a hyperbolic load-settlement curve. The method is based on calculation of the total stiffness of the piled raft by means of the stiffness of pile group and the stiffness of the unpiled raft in isolation. However, this method only considers the interaction between piles and the raft and does not take into account the interaction between a pile and another in a pile group and the interaction between the raft, the soil and the pile. As previously discussed, the problem is much more complicated than what the method above entails. Interaction between a pile and the other and interaction between the pile-soil and raft exist. This paper attempts at trying to take into consideration the additional two interactions so as to have closer results to reality. Using the pile-to-pile and the soil-pile-raft interaction factors obtained from Poulos and Davis curves (1980) could somehow enhance the analysis of the method. Thus modifying the input parameter "initial tangent stiffness" by taking into consideration the additional interactions.

The vertical settlement of a pile is given by:

$$W_{pK} = \sum_{J=1,J\neq K}^{n-1}\left(\delta_{1J}P_{pJ}\alpha_{KJ}\right) + \delta_{1K}P_{pK} \qquad (22)$$

Where,

$W_{pK}$ =Vertical displacement of pile K

$\delta_{1J}$ *and* $\delta_{1K}$ =Displacements due to unit load on piles J and K, respectively

$P_{pJ}$ *and* $P_{pK}$ =Load on piles J and K, respectively

$\alpha_{KJ}$ =Pile –soil-pile interaction factor of pile J on pile K

$n$ =Number of piles

The stiffness of pile spring K is given by:

$$K_p = K_{pK} = \frac{P_{pK}}{W_{pK}} \qquad (23)$$

The interaction factor of pile J on pile K could be obtained through two methods, either by using finite element analyses (Plaxis 3D) or by using the Poulos and Davis curves (1980). Using Plaxis, two separate models are simulated. First model consists of a single pile subjected to load P and then the settlement of the pile is obtained. The second model consists of two separate piles subjected to the same load P with a given spacing and the settlement of each pile is then obtained. The additional settlement of a certain pile due to the presence of another pile near it is simply obtained by subtracting the settlement obtained in model 1 from the settlement obtained in model 2.

The pile soil pile interaction factor $\alpha$, is utilized to compute the additional settlement of a pile due to adjacent piles. In the case at hand, piles K and J, the additional settlement for pile K due to the presence of adjacent pile J is therefore:

$$\Delta w_K = \omega_{1J} Q_J \alpha_{KJ} \qquad (24)$$

So,

$$\alpha_{KJ} = \frac{\Delta w_K}{\omega_{1J} Q_J} \qquad (25)$$

Where,

$\alpha_{KJ}$=Interaction factor of pile J on pile K

$\Delta w_K$=Additional settlement of pile K caused by pile J

$\omega_{1J}$=Settlement due to a unit load of pile J

$Q_J$=Load on pile J

The second method would be to simply use the curves published by Poulos from which one can obtain the interaction factor for a certain length to diameter ratio and spacing to diameter ratio.

*The Raft-Soil-Pile Interaction:*

Also FEM can be employed to construct the pile-soil-raft interaction curve. Two separate models are prepared, the first would be a piled raft and the second model was a raft in isolation

The additional settlement of the raft caused by piles can be similarly computed as the difference between the settlement of both models and beta can be the ratio of additional settlement/displacement of raft alone

$$\beta = \frac{additional\ displacement\ of\ the\ raft\ caused\ by\ piles}{displacement\ of\ unpiled\ raft}$$

Where,

$\beta$=Raft-soil-pile interaction factor

$$w_{rpM} = \sum_{k=1}^{n} (\rho_{1M} Q_M \beta_{KM}) + w_{rM}$$

Where,

$w_{rpM}$= settlement of raft by taking into account its interaction with piles and soil

$Q_M$=applied load on raft

$\beta_{KM}$=raft –soil –pile interaction factor

$w_{rM}$=settlement of unpiled raft

The code generated can compute the load carried by the raft, so the displacement of an unpiled raft was attainable and thus there was no need to construct curves regarding the raft-soil-pile interaction factor.

### 3.2.1 *Program Input:*

There are several input parameters for the program; the diagram below shows an illustration of the input and output parameters:

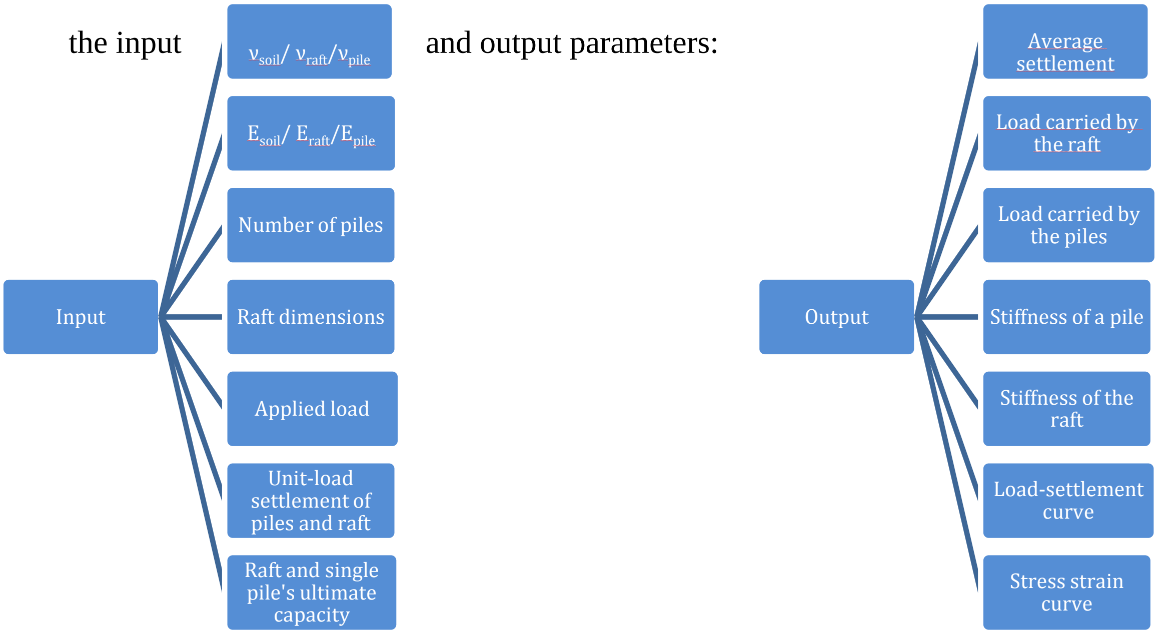


Figure 13. Diagrpam representing the input and output parameters of the formulated code

With regards to the input parameters of the code, it is important to understand how to obtain the ultimate capacity of piles and raft along with their unit load settlement.  The following section will explain several methods from the literature that can be used to obtain the required input:

#### 3.2.1.1 Pile Capacity:

There are mainly three approaches for the calculation of the pile capacity; from fundamental soil properties, from in situ test (SPT, CPT, etc.) results and from full-scale load tests on a prototype pile. The general focus of this study will be on the first one; fundamental soil properties.

The general concept of the evaluation of the ultimate resistance of a pile is based on the shaft friction and base resistance. Thus, the total failure load is written as:

$$Q_u = Q_b + Q_f \qquad (26)$$

Where,

$Q_u$=load applied to the pile at failure

$Q_b$=base resistance

$Q_f$=shaft resistance

The general equation for the base resistance may be formulated as:

$$Q_b = cN_c + q_0'N_q + \frac{1}{2}\gamma dN_\gamma A_b \qquad (27)$$

Where,

$d$=width or diameter of the shaft at the base

$q_0'$=effective overburden pressure at the base

$A_b$=base area of the pile

$c$=cohesion of soil

$\gamma$=effective unit weight of soil

$N_c$, $N_q$, $N_\gamma$= bearing capacity factors that take the shape factor into consideration

In the case of **cohesionless** soils, c=0 causing that the term "$\frac{1}{2}\gamma d N_\gamma$" to become insignificant when compared to the term "$q_0' N_q$" for deep foundations. This results in the following base resistance capacity:

$$Q_b = q_0' N_q A_b \qquad (28)$$

Therefore, the overall pile ultimate capacity can be formulated as:

$$Q_u = q_0' N_q A_b + A_s \bar{q}_0' \bar{K}_s \tan\delta \qquad (29)$$

Where,

$A_s$=surface area of the embedded length of the pile

$\bar{q}_0'$=average effective overburden pressure over the embedded depth of the pile

$\bar{K}_s$=average lateral earth pressure coefficient

$\delta$=angle of wall friction

In the case of cohesive soils, for $\varphi = 0$, $N_q = 1$ and $N_\gamma = 0$

The net ultimate load capacity of the pile is given by:

$$Q_u = c_b N_c A_b + \alpha \bar{c}_u A_s \qquad (30)$$

Where,

$\alpha$=adhesion factor

$\bar{c}_u$=average undrained shear strength of clay along the shaft

$c_b$=undrained shear strength of clay at the base level

$N_c$=bearing capacity factor

Bearing Capacity Factor , $\boldsymbol{N_c}$

The value of the bearing capacity "$N_c$" is 9, as proposed by Skempton (1951) for circular foundations for a $L/B > 4$. Thus, the base capacity of a pile in clayey soils may be expressed as:

$$Q_b = 9c_bA_b \quad (31)$$

*Skin Resistance*

There are generally several methods in the literature that are used to obtain the skin resistance; these methods include the $\alpha$ and $\beta$ methods, Meyerhof 's method (1976) and the method by Viggiani (1993). The methods will be described briefly in this thesis:

### 3.2.1.1.1 The $\alpha$- Method:

Dennis and Olson (1983) developed a curve that gives a relationship between $\alpha$ and undrained shear strength of clay ($\bar{c}_u$). It is important to note that the proposed curve is valid for piles penetrating less than 30 m, but for a case of embedment between 30 to 50 m, a reduction factor is applied linearly from 1.0 to 0.56 (Dennis and Olson, 1983).

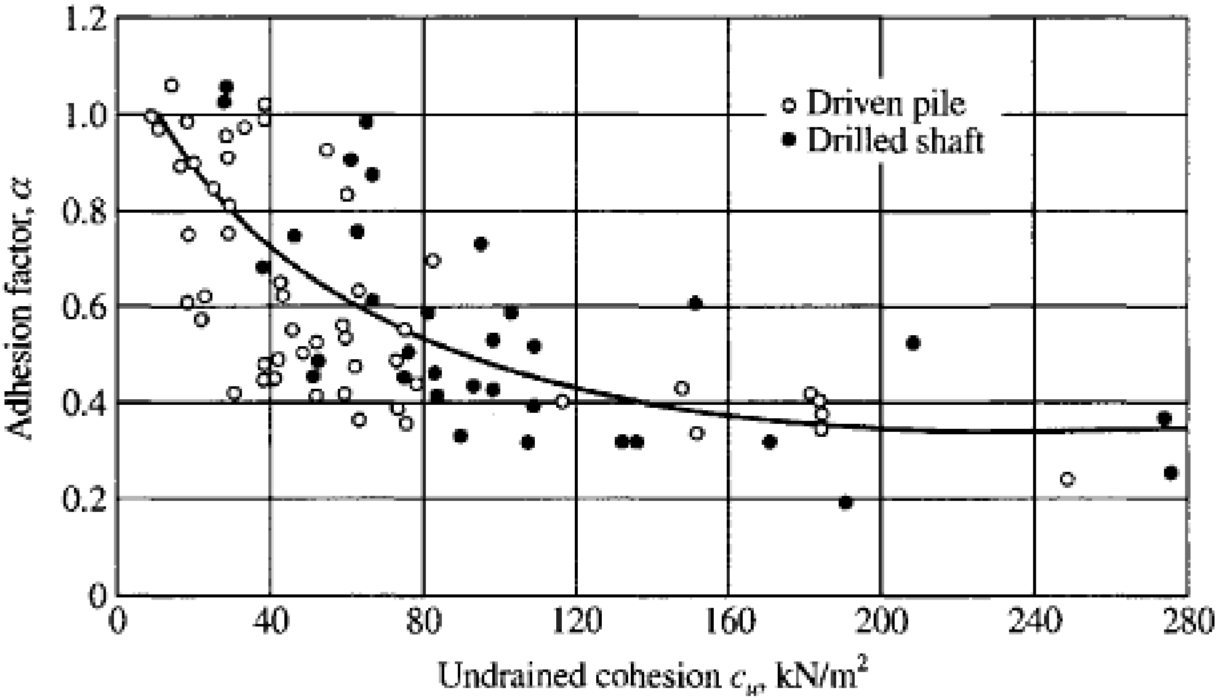


Figure 14: Adhesion factor alpha for piles with embedment lengths less than 50 m in clay (Dennis and Olson, 1983)

### The $\beta$-Method:

The unit skin friction $f_s$ is given by:

$$f_s = \bar{q}'_0 \bar{K}_s \tan\delta = \beta \bar{q}'_0 \quad (32)$$

Meyerhof's Method (1976):

Meyerhof suggested a semi-empirical relationship for estimating skin friction in clays. For bored piles:

$$f_s = c_u \tan\varphi' \quad (33)$$

$c_u$=undrained shear strength of the soil

$\varphi'$=effective angle of internal friction

Viggianni's Method (1993):

Viggiani et al. (2012) presented $\alpha$ values, depending on the pile type and the undrained shear strength; these values are tabulated in table 2:

Table 2: Values of $\alpha$ proposed by Viggiani (2012)

| Type of Pile | $c_u$ | $\alpha$ |
|---|---|---|
| Displacement | $c_u \leq 25$<br>$25 \leq c_u \leq 70$<br>$c_u \geq 70$ | 1<br>1-0.0011$(c_u - 25)$<br>0.5 |
| Replacement | $c_u \leq 25$<br>$25 \leq c_u \leq 70$<br>$c_u \geq 70$ | 0.7<br>0.7-0.008$(c_u - 25)$<br>0.35 |

### *3.2.2 Raft's bearing capacity*

Bearing capacity of a raft can be computed by several methods depending on what type of soil is the raft embedded in. Since the study governs the three cases of soils: drained clay, undrained clay and sands, the following section will explain briefly some methods that could be employed for the purpose of obtaining the raft's ultimate bearing capacity.

### 3.2.2.1 Cohesion-less Soils

Akbas and Kulhawy (2009) suggested the following equation for predicting the bearing capacity for a footing in cohesionless soils:

$$Q_{tcp} = (0.5B\bar{\gamma}N_{\gamma}\zeta_{\gamma s}\zeta_{\gamma d}\zeta_{\gamma r}\zeta_{\gamma i}\zeta_{\gamma t}\zeta_{\gamma g} + \bar{\gamma}DN_{q}\zeta_{qs}\zeta_{qd}\zeta_{qr}\zeta_{qi}\zeta_{qt}\zeta_{qg})A_{f} \quad (34)$$

$$Q_{tcp} = Q^{\gamma}{}_{tcp} + Q^{q}{}_{tcp}$$

Where,

$A_f$=area of the footing

$B$=width of the footing

$D$=depth of the footing

$\bar{\gamma}$=effective soil unit weight

$N_{\gamma}\ and\ N_q$=bearing capacity factors (as previously defined)

The drained capacity factors are given as:

$$N_q = e^{(\pi \tan \bar{\varphi})} \tan^2(45 + \frac{\bar{\varphi}}{2}) \quad (35)$$

$$N_{\gamma} = 2(N_q + 1) \tan \bar{\varphi} \quad (36)$$

$\bar{\varphi}$=effective stress friction angle

$\zeta_{xy}$=modifiers described below

The subscripts of the modifiers in Equation 34 indicate the applicable term $N_q$ or $N_{\gamma}$, along with the modification:

- *r* for soil rigidity
- *s* for foundation shape
- *d* for foundation depth
- *i* for load inclination
- *t* for tilt of foundation base

- $g$ for ground level inclination

Corresponding equations for each of the aforementioned modifiers can be found in Akbas and Kulhawy's paper (2009).

### 3.2.2.2 Cohesive Soils

Terzaghi's well-known expression can be used for the ultimate bearing capacity of raft foundations:

$$q_{ult} = cN_c s_c + qN_q + 0.5B\gamma N_\gamma s_\gamma \quad (37)$$

Where,

$c$=cohesion

$s_c$=shape factor for cohesion

$q$=overburden pressure ($\gamma D$)

$B$=least lateral dimension of the raft

$\gamma$=soil's unit weight

$s_\gamma$=shape factor for soil wedge

$N_c, N_q, N_\gamma$=coefficients of bearing capacity as a function of internal friction angle of soil $\varphi$.

Furthermore, for the case at hand the foundation is of a squared geometry and so the following equation may be used:

$$q_{ult} = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma s_\gamma \quad (38)$$

The hyperbolic method is used as a first "guesstimate" for pile stiffness. By using the proposed method, time, money and effort will be saved and indeed the work will be more efficient. In order to validate the method, a parametric study is carried and will be described in the following section.

### *3.2.3 Pile's Unit-load Settlement*

There are plenty of methods that could be employed for the settlement of single piles. These methods are generally based on empirical correlations or on stress distribution along the pile. In 1995, Meyerhof suggested an empirical formula for the settlement of a single pile in sand, with a factor of safety on ultimate load that exceeds 3:

$$\rho = \frac{d_b}{30 \times FS} \quad (39)$$

Where,

$d_b$=diameter of the pile base

A more general method would be that of Viggiani and Viggiani (2008) (in Viggiani et al. (2012)), where they suggested an empirical formula for the settlement $w_s$ depending on the soil and pile type:

$$w_s = \frac{d}{M}\frac{Q}{Q_{lim}} = \frac{d}{M} \times \frac{1}{FS} \quad (40)$$

Where,

$d$=pile diameter

$Q$=applied load

$Q_{lim}$=bearing capacity of pile

$M$=constant that depends on the soil and pile type (refer to Table 3)

Table 3. M values for obtaining the settlement of a single pile (Viggiani et al. (2012))

| Type of Pile | Type of Soil | M |
|---|---|---|
| Small displacement | Cohesionless<br>Cohesive | 50<br>75 |
| Replacement | Cohesionless<br>Cohesive | 25<br>40 |
| Displacement | Cohesionless<br>Cohesive | 80<br>120 |

Moreover, other methods can be used to obtain the settlement of a pile, for instance the load settlement curves (t-z curves) could be used. The basic idea of this method is that the pile is divided into segments and a linear variation of load in each segment is assumed, then the elastic deformations are calculated.

### *3.2.4 Raft's Unit-Load Settlement*

Fraser and Wardle (1976) suggested the following equation for calculating the raft settlement, $\rho$:

$$\rho = pb\frac{(1-\nu_s{}^2)}{E_s}I \qquad (41)$$

Where,

$p$=applied uniform pressure

$I$=influence factor (refer to Figure X)

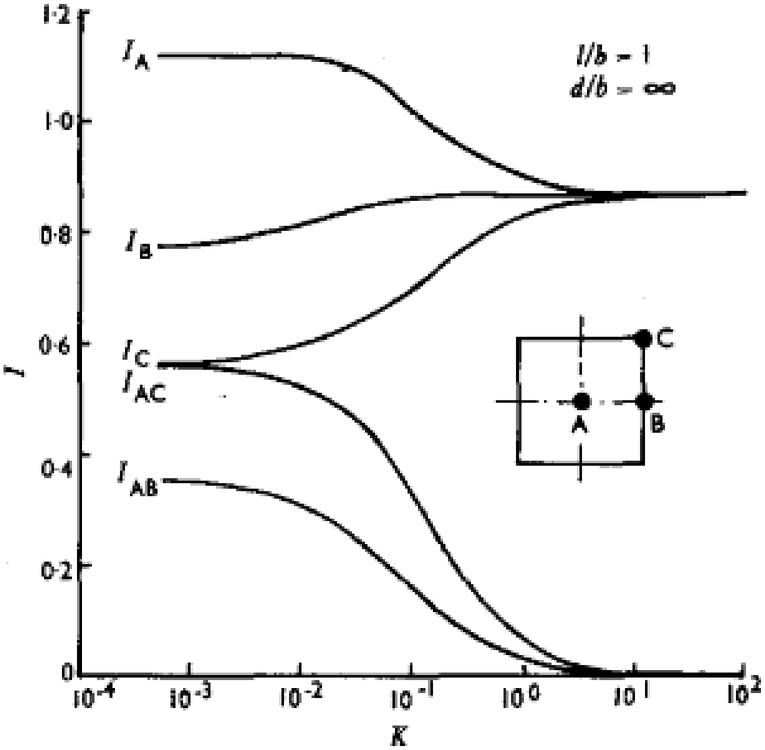


Figure 15. Settlement influence factor I for l/b=1 (Fraser and Wardle (1976))

## 3.3 Parametric Study:

In order to be able to use the generated code for the design of typical piled raft foundations, a parametric study had to be performed. Wide range of representative design scenarios for piled rafts. In these comparative analyses, parameters of interest (soil properties, number of piles-pile arrangement, loading, raft thickness, etc.) A total of 1,344 cases were studied, it is important to mention that all the cases were performed in parallel, using the code and using the finite element software Plaxis 3D for validation. The parametric study includes sands, drained and undrained clays.

### *3.3.1 Formulation of the Problem*

To begin with, the problem was set for having a raft foundation, in which the soil and raft characteristics were chosen for an allowable settlement on the order of 15 cm. Then the piles were introduced and their corresponding parameters were chose so as to have an average settlement of about 5 cm for the piled raft foundation. The following set up of the problem defines the base case of the parametric study. It is worth noting that three base cases exist in this study; for undrained and drained clays and for sands. In order to be able to capture all possible cases, several soil, raft and pile parameters were varied.

The “base case” adopted in the parametric study corresponds to a piled-raft foundation with the following characteristics:

1. Undrained Clay:

- Load Configuration = Uniform
- tr = 1.2 m
- Np = 64
- Lp = 15 m

- Es = 25 MPa
- Area= 40 mx40 m

2. Drained Clay:

- Load Configuration = Uniform
- tr = 1.2 m
- Np = 64
- Lp = 15 m
- Es = 16 MPa
- Area= 40 mx40 m

3. Sand:

- Load Configuration = Uniform
- tr = 1.2 m
- Np = 64
- Lp = 15 m
- Es = 25 MPa
- Area= 40 mx40 m

For each type of soil i.e. drained/undrained clay or sand, a range of representative Young's moduli of elasticity were chosen (7 values for each kind of soil in the aim of covering all possible cases, from weak soils to stiff soils), and each modulus had 4 typical pile lengths in which each pile length had 4 representative raft thicknesses and finally each thickness had 4 distinctive yet typical pile diameters (a clarification of all the cases done for a single Young's modulus is tabulated in Table 4). This leads to a total of 64 runs for a single modulus of elasticity. The Young's moduli of elasticity were chosen so as to cover as much cases possible.

Also, typical pile diameters were chosen to be in the range of [0.8-1.5m] whereas pile lengths ranged between [10-25m]. As for the raft thickness, the typical raft thicknesses range between [0.8-2m]. By taking the aforementioned ranges of pile diameters, lengths and raft thicknesses into consideration, the study would be able to capture almost all the cases of a typical pile raft foundation. It is important to mention that the number of piles along with their configuration was kept constant. Nevertheless, all possible combinations of different number of piles were performed on the average case, and it was concluded that the equivalent stiffness of a single pile does not change and results from the average case are plotted in the following chapter, whereas the average settlement of the pile raft foundation is sensitive for the number of piles.

Table 4. Parameters that are varied for a single soil's Young's modulus

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| L=10 m | $t_r$=0.8 m | D=0.8 m | L=15 m | $t_r$=0.8 m | D=0.8 m | L=20 m | $t_r$=0.8 m | D=0.8 m | L=25 m | $t_r$=0.8 m | D=0.8 m |
| | | D=1 m | | | D=1 m | | | D=1 m | | | D=1 m |
| | | D=1.2 m | | | D=1.2 m | | | D=1.2 m | | | D=1.2 m |
| | | D=1.5 m | | | D=1.5 m | | | D=1.5 m | | | D=1.5 m |
| | $t_r$ =1.2 m | D=0.8 m | | $t_r$ =1.2 m | D=0.8 m | | $t_r$ =1.2 m | D=0.8 m | | $t_r$ =1.2 m | D=0.8 m |
| | | D=1 m | | | D=1 m | | | D=1 m | | | D=1 m |
| | | D=1.2 m | | | D=1.2 m | | | D=1.2 m | | | D=1.2 m |
| | | D=1.5 m | | | D=1.5 m | | | D=1.5 m | | | D=1.5 m |
| | $t_r$ =1.6 m | D=0.8 m | | $t_r$ =1.6 m | D=0.8 m | | $t_r$ =1.6 m | D=0.8 m | | $t_r$ =1.6 m | D=0.8 m |
| | | D=1 m | | | D=1 m | | | D=1 m | | | D=1 m |
| | | D=1.2 m | | | D=1.2 m | | | D=1.2 m | | | D=1.2 m |
| | | D=1.5 m | | | D=1.5 m | | | D=1.5 m | | | D=1.5 m |
| | $t_r$ =2.0 m | D=0.8 m | | $t_r$ =2.0 m | D=0.8 m | | $t_r$ =2.0 m | D=0.8 m | | $t_r$ =2.0 m | D=0.8 m |
| | | D=1 m | | | D=1 m | | | D=1 m | | | D=1 m |
| | | D=1.2 m | | | D=1.2 m | | | D=1.2 m | | | D=1.2 m |
| | | D=1.5 m | | | D=1.5 m | | | D=1.5 m | | | D=1.5 m |

# CHAPTER IV

# FINITE ELEMENT MODELING AND ANALYSIS

The central objective of the research presented was to investigate the response of piled-raft systems and to understand the relevant interactions between its various constituents through a series of behavioral analyses carried out using finite element software, Plaxis 3D. Therefore, finite element analysis was carried out in the aim of understanding the behavior of the piled raft while varying the characteristics of its constituents.

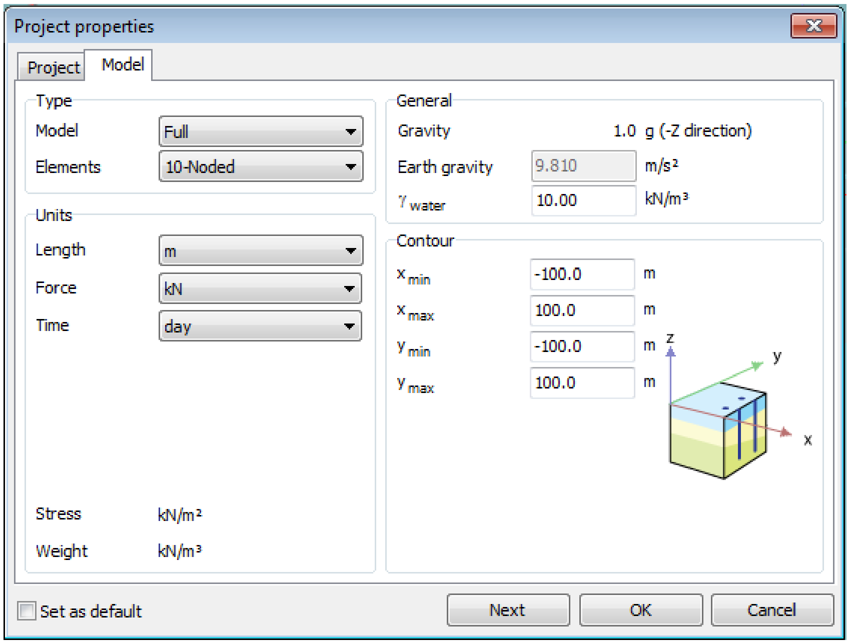
## 4.1 Overview:

Plaxis is a company based in the Netherlands, developing software under the same brand name; Plaxis. The Plaxis 3D program is a three-dimensional finite element program used to make deformation and stability analysis for various types of geotechnical applications (Reference Manual, Plaxis). The user interface of the Plaxis 3D comprises of two sub-programs as Input and Output. Properties of soil and other elements (boreholes, embedded piles, plates etc.) are assigned to the elements by using material data sets by the Input interface. This section includes the Plaxis 3D Foundation features with a thorough explanation of the modeling procedure from setting up the geometry, going through the selection of material models and defining the respective model parameters, to a discussion of the calculation process and output capabilities.

## 4.2 Basic Steps for Plaxis Calculations:

In order to formulate any Plaxis model, the following steps are performed:

1. Defining the geometry



Project properties

Type

Model: Full

Elements: 10-Noded

Units

Length: m

Force: kN

Time: day

Stress: kN/m²

Weight: kN/m³

General

Gravity: 1.0 g (-Z direction)

Earth gravity: 9.810 m/s²

$\gamma_{water}$: 10.00 kN/m³

Contour

$x_{min}$: -100.0 m

$x_{max}$: 100.0 m

$y_{min}$: -100.0 m

$y_{max}$: 100.0 m

☐ Set as default

Figure 16. Geometry definition in Plaxis 3D

2. Creating a borehole:

When creating a geometry model it is recommended to start defining the boreholes and thus the vertical depth of the model. Vertical is defined as the z-direction. The boreholes are divided in layers, which subsequently are assigned different materials (i.e. different soil properties). When multiple boreholes are present in the model, the soil properties are interpolated between the boreholes thus creating non-horizontal soil layers. The pore pressure distribution is defined in the boreholes. The distribution could be entered manually or (if a hydrostatic distribution is expected) be generated from the phreatic level (defined by the user)

3. Assigning boundary conditions: The predefined boundary conditions provide restriction to horizontal displacement and free vertical displacement.

4. Generating the mesh:

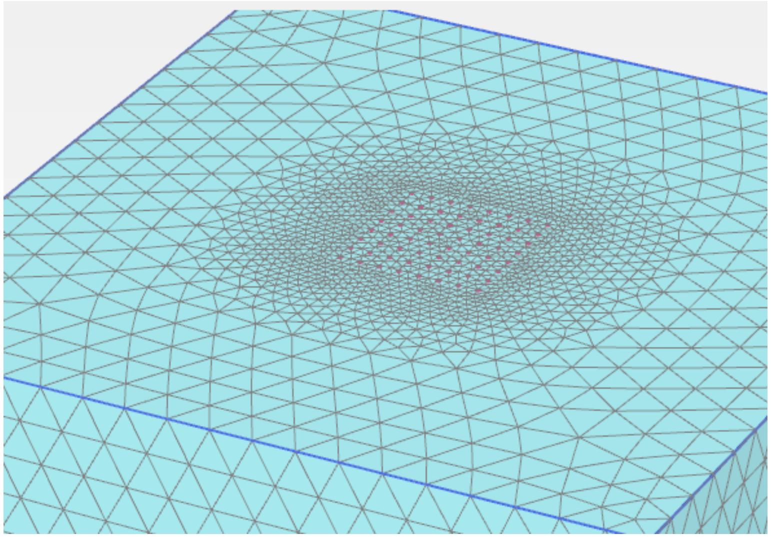


Figure 17. Mesh generation in Plaxis 3D

Note: for the model for the sake of the thesis analysis, fine mesh was used as shown in Figure 17. Also, mesh refinement is performed near the pile so as to capture the actual performance of the piled raft.

5. Assigning the initial conditions
6. Performing the necessary calculations

The last two steps will be thoroughly discussed in the following section.

## 4.3 Modeling Process Overview:

The analyses conducted in this thesis consist of soils performing as materials with a hardening potential. The model geometries are built in the x-y-z space/coordinate system to replicate the actual 3D stress distribution within an in-situ continuum as shown in Figure 18.

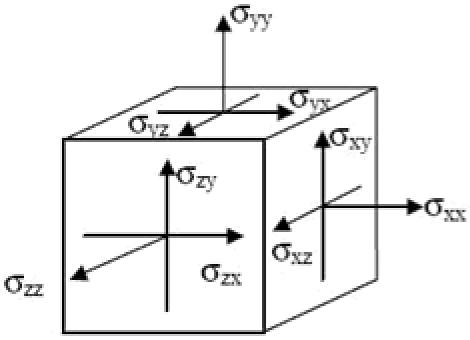


Figure 18. 3D stress distribution on an element in Plaxis 3D

The soil strength parameters are defined for the different soil material sets based on the adopted soil constitutive models. Theses parameters are then assigned to the respective soil layers and the design groundwater levels are set as borehole logs that are to be positioned within the model layout plan. The borehole logs and layout are used to generate, though a series of interpolations, the global 3D soil stratigraphy. Then the relevant work planes are defined at which the structural elements (Raft, Piles, Basement Walls) are defined and inserted in the model.

As listed in the steps, a 3D mesh is generated. The 3D mesh generated is made up of 15-node wedge elements linked together at nodes, which are points in the mesh where values of the primary variables are calculated. For a displacement analysis, the primary variables are values of displacement. The values of nodal displacement are interpolated within the elements in order to give algebraic expressions for displacement and strain throughout the complete mesh.

Forces at the nodes and interfaces are calculated from the stress-strain constitutive behavior governed by the soil constitutive model adopted. The nodal forces are related to the nodal displacements by stiffness equations, which are preset, generated and solved within Plaxis in order to compute values of nodal displacements. This sheds a light to the next section which includes the relevant constitutive models in Plaxis 3D along with a thorough explanation of the chosen constitutive model for this thesis.

## 4.4 Constitutive Models:

In this section, relevant constitutive models in Plaxis 3D are described and the most suitable model was chosen and explained herein.

### *4.4.1 Linear Elastic Model (LE)*

The linear elastic model is based on Hooke's law of isotropic elasticity. It comprises of basic parameters such as Young's modulus E and Poisson's ratio $\nu$. However, the Linear Elastic model is not recommended to model the soil, it may be used to model stiff volumes in the soil, for instance intact rock formation or concrete walls.

### *4.4.2 Mohr-Coulomb Model (MC)*

Mohr-Coulomb's model is a linear elastic perfectly-plastic model that includes five input parameters: Young's modulus E and Poisson's ratio $\nu$ for soil elasticity; the friction angle $\varphi$ and cohesion *c* for soil plasticity and finally the $\psi$ as an angle of dilatancy. It is advisable to use this model for a first analysis of the problem considered since computations tend to be relatively fast. The Mohr-Coulomb model represented a "first-order" approximation of the soil or rock behavior.

### *4.4.3 The Hardening Soil Model (HSM)*

The Hardening Soil Model is an advanced model for simulating the behavior of different types of soils including soft and stiff soils, Schanz (1998). As the Mohr-Coulomb model, limiting states of stress are described by means of friction angle $\varphi$ and cohesion *c* and the angle of dilatancy $\psi$. Nevertheless, the soil stiffness is described in a much more accurate manner, by using three different input sitffnesses: the triaxial loading stiffness E50, the triaxial unloading stiffness Eur and the oedometer loading stiffness Eoed. In contrast to the Mohr-Coulomb model, the Hardening Soil model accounts for stress-dependency of stiffness moduli to reflect the increase in stiffness with pressure. This stress dependency is reflected through three input stiffnesses relating to a reference stress (100KPa).

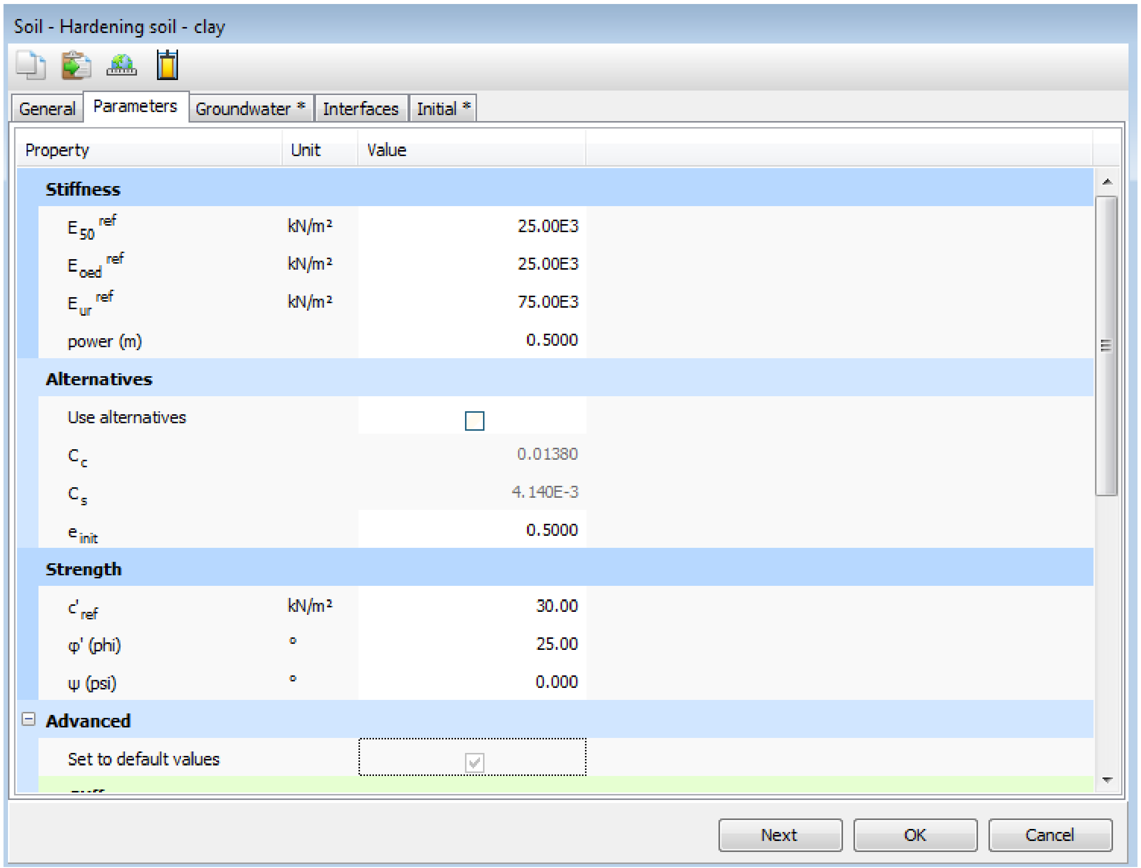
## 4.5 Structural Modeling

In any finite element analysis, delicate care must be taken when dealing with the input and modeling approaches. In this section, a description on how the interface, soils, piles, raft and loads were modeled.

### *4.5.1 Interfaces*

I n general, interface elements are modeled by means of the bilinear Mohr-Coulomb model. Moreover, when a more advanced model is chosen, the interface element will only choose the relevant data for the Mohr-Coulomb model ($c$,$\varphi$,$\psi$,E and $\nu$).

As previously discussed, the HSM is used for the sake of this thesis; Figure 19 summarizes a sample of input parameters for drained clay with a Young’s modulus of 25 MPa:



| Property | Unit | Value |
|---|---|---|
| **Stiffness** | | |
| $E_{50}^{ref}$ | kN/m² | 25.00E3 |
| $E_{oed}^{ref}$ | kN/m² | 25.00E3 |
| $E_{ur}^{ref}$ | kN/m² | 75.00E3 |
| power (m) | | 0.5000 |
| **Alternatives** | | |
| Use alternatives | | ☐ |
| $C_c$ | | 0.01380 |
| $C_s$ | | 4.140E-3 |
| $e_{init}$ | | 0.5000 |
| **Strength** | | |
| $c'_{ref}$ | kN/m² | 30.00 |
| φ' (phi) | ° | 25.00 |
| ψ (psi) | ° | 0.000 |
| **Advanced** | | |
| Set to default values | | ☑ |

Figure 19. Sample on material properties chosen

### *4.5.2 Soil's Modeling: Material's Input*

Tables 5 below presents the input parameters a drained clay case:

Table 5. Soil's input parameters

| Parameter | Symbol | Stiff Clay | Unit |
|---|---|---|---|
| General | | | |
| Material model | Model | Hardening Soil | - |
| Drainage Type | Type | Drained | - |
| Dry unit weight | $\gamma_{unsat}$ | 20 | kN/m$^3$ |
| Saturated unit weight | $\gamma_{sat}$ | 20 | kN/m$^3$ |
| Parameters | | | |
| Secant stiffness for CD triaxial test | $E_{50}^{ref}$ | 25,000 | kN/m$^2$ |
| Tangent oedometer stiffness | $E_{50}^{oed}$ | 25,000 | kN/m$^2$ |
| Unloading/reloading stiffness | $E_{ur}^{ref}$ | 75,000 | kN/m$^2$ |
| Power for stress level dependency | m | 0.5 | - |
| Cohesion | $c'_{ref}$ | 25 | kN/m$^2$ |
| Friction angle | $\varphi$ | 25 | - |
| Poisson's ratio | $\nu_s$ | 0.25 | - |

Note that a similar table to Table 5 is input for the remaining Young's moduli.

### 4.5.3 Pile Modeling

In this study, embedded piles are used to the model the piles. Embedded piles are structural elements developed by Plaxis, which are modeled in a similar manner to beam elements. The main advantage of the embedded piles is the interaction with the continuum as the skin resistance and the foot resistance. The embedded piles can be placed in any direction within the subsoil without any alteration of the mesh. This is attained by crossing through a 10-node tetrahedral soil element while creating virtual nodes (blank grey circles) as shown in Figure 21.

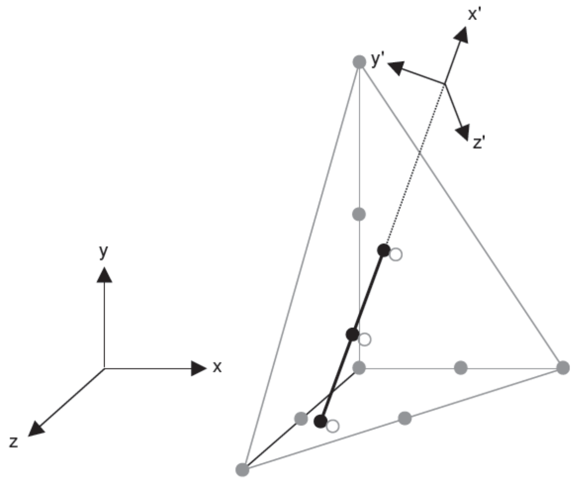


Figure 20. Embedded beam node representation

Embedded beam elements do not have an actual volumes and interfaces. Nevertheless, a virtual elastic zone exists as shown in Figure 20. Assigning an equivalent pile diameter within the material data set of the embedded beam creates the virtual elastic zone. The virtual elastic zone disregards the plastic behavior of the soil within the zone (Reference Manual, Plaxis) and approaches to actual volume pile behavior. Table 6 presents the material and section properties of the piles.

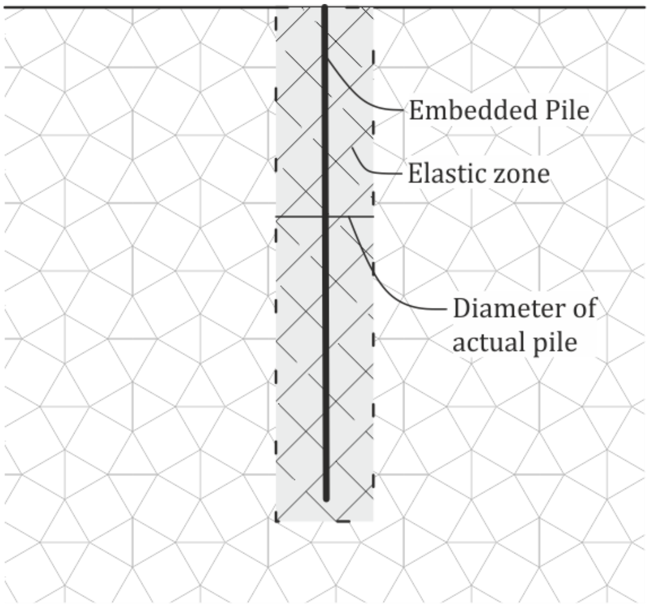


Figure 21. Embedded beam simulation

Table 6. Embedded beam properties

| Parameter | Symbol | Pile | Unit |
|---|---|---|---|
| Property | - | Embedded beam | - |
| Unit weight | $\gamma$ | 25 | $kN/m^3$ |
| Young's Modulus of Elasticity | E | 20,000,000 | $kN/m^2$ |
| Poisson's ratio | $\nu$ | 0.2 | - |
| Diameter | d | 0.8 | m |
| Length | L | 25 | m |

### *4.5.4 Raft Modeling*

According to Plaxis Material model manual, it is recommended to use the linear elastic model to simulate strong massive structures such as rocks or concrete. The drainage type was set to be "non-porous" in the aim of simulating the actual behavior or a reinforced concrete raft. Also, the chosen Young's modulus is a typical modulus for a reinforced concrete raft.

Table 7. Raft properties

| Parameter | Symbol | Raft | Unit |
|---|---|---|---|
| Property | - | Linear elastic | - |
| Unit weight | $\gamma$ | 25 | kN/m$^3$ |
| Young's Modulus of Elasticity | E | 20,000,000 | kN/m$^2$ |
| Poisson's ratio | $\nu$ | 0.2 | - |
| Thickness | t | 1 | m |
| Width-breadth | WxB | 40x40 | m |

Note: the tables above show a specific case only; the pile diameter, length and raft thickness are varying from one case to the other.

### *4.5.5 Loads*

In Plaxis, loads acting on a structure are modeled as either distributed, line or point loads in the geometry model. In the parametric study the uniformly distributed load acting on the raft area is chosen with a value of 480 MN equivalent to 300 kN/m$^2$ (raft area=40x40 m).

Plaxis 3D calculation Procedure

As mentioned earlier, the last two steps are to assign the initial conditions and conduct the necessary calculation. The analysis of the modeled problem is carried out through a series of calculation phases, with an initial phase and a set of phases reflecting the construction stages and different modes of behavior encountered during the process.

Initial phase

Initial stresses are vital in studying geotechnical engineering problems since soil behavior is governed by the state of initial stresses. Initial stresses in a soil body are influenced by the weight of the material and the history of its formation, and is usually characterized by an initial vertical stress $\sigma_{v,0}$ . Initial horizontal stress $\sigma_{h,0}$, is related to the initial vertical stress by the coefficient of lateral earth pressure, $K_0$. In Plaxis 3D, initial stresses may be generated by specifying $K_0$ or by using gravity loading. $K_0$ procedure is used in cases where the surface is horizontal and all soil layers and phreatic levels parallel to the surface, for all other cases gravity loading is used to define the initial state of stress.

### *4.5.6 Staged Construction*

Staged construction is a very useful feature through which construction stages are defined by changing the geometry and load configuration in the different stages of the analysis; therefore enabling a more accurate and realistic simulation of the problem through catering for the effect of the various loading, construction and excavation processes on the general behavior of the geotechnical system.

#### 4.5.6.1 Plastic Calculation

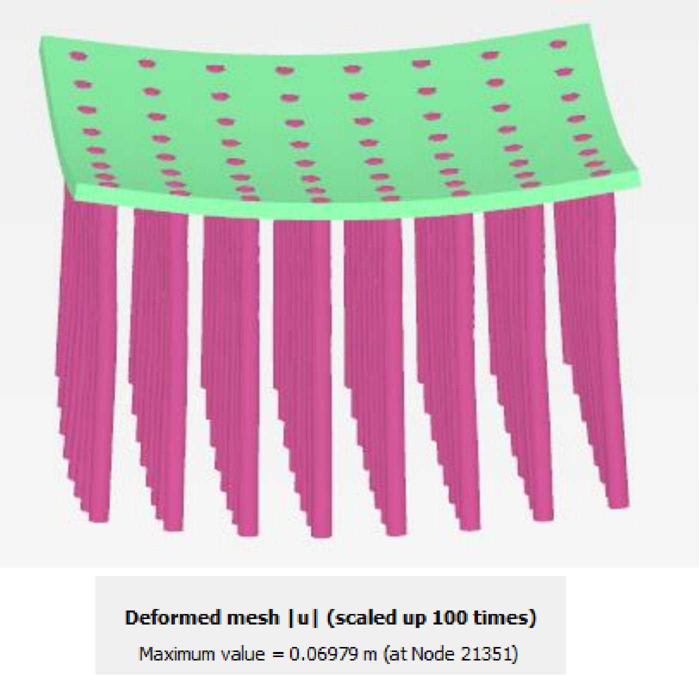
To carry out the deformation analyses, plastic calculations are used according to a small deformation theory in cases where soils are expected to have a soil hardening potential and other soil behaviors. Thus it is appropriate for most practical geotechnical applications. Plaxis 3D provides a broad extent of output options that allow to comprehend the different aspects of behavior of the geotechnical problem modeled. The output is presented in both tabular and graphical format in 2D, 3D or through sections across the model geometry. A sample of graphical output of one of the cases is shown in Figures 22 and 23 in Chapter 5. Nevertheless, the output capabilities include relative, mean, effective and total stresses, pore water pressures and settlements (vertical, horizontal and total), structural loads (axial, shear and bending moments) within structural elements.

# CHAPTER V

# RESULTS AND STATISTICAL MODELING

The parametric study conducted has a dual aim; first to reproduce design aid charts that a structural engineer may use to estimate an equivalent pile stiffness that is as close as possible to a value obtained from finite element analysis. The second aim is to prove that the produced code gives an output that is reasonably close to reality. The model uncertainty of the code is obtained and thorough analyses are conducted and will be discussed.

As discussed in chapter 3, the parametric study conducted aims at covering all typical cases. Due to the large number of charts reproduced, only a few will be presented in the thesis whereas the rest will be in the appendix. The three base cases adopted are explained in more details below. For the sake or representation, the output results of the drained base case are presented in Figures X Y. The output includes settlement and failure contours, in addition to pile load distribution.



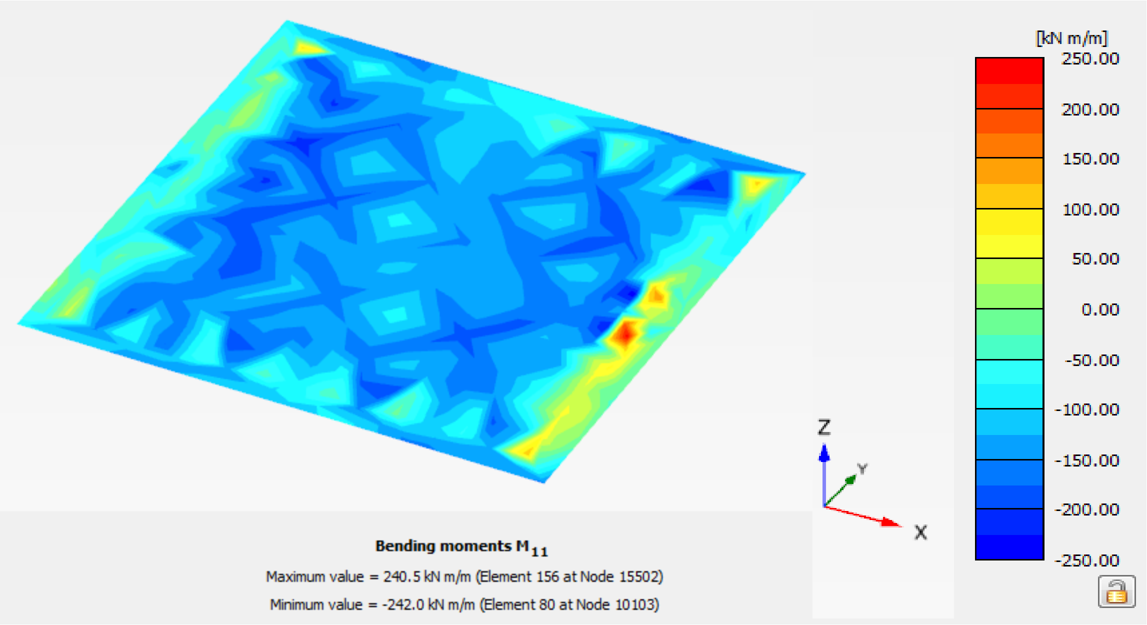
**Deformed mesh |u| (scaled up 100 times)**

Maximum value = 0.06979 m (at Node 21351)

Figure 22. Deformed structure for the average case of an undrained clay

As shown in Figure 22, the deformed shape of the structure shows a dishing effect in the raft. Since the applied load is a uniformly distributed load, the dishing effect is expected and is a function of raft thickness, soil's Young's modulus of elasticity, number and characteristics of piles (diameter and length). The thicker the raft, the stiffer it would be as a resisting structure and thus the dishing effect is less. On the other hand, when dealing with thin rafts, the stiffness of the raft would be less implying that the raft has a tendency to undergo this dishing effect more than a thicker raft. With regards to the soil's Young's modulus of elasticity's effect, the stiffer the soil i.e. the greater the Esoil value is, then the dishing effect will become less. This is due to the fact that the stronger soil will bear more load and thus the raft will deform less, and vice versa.

As the number of piles increase, and the spacing between them decrease, the system (piles+raft) becomes stiffness and dishing effect almost vanishes. However, there is an effect of sagging for the raft between two piles. If the raft is thick enough and is situated on a stiff and has a small pile-to-pile length, the sag seems to decrease. This sag is a function of positive and negative bending moments generated within the raft structure. The resulting positive and negative moment within the raft is analyzed in details and is enclosed within this chapter. A representation of the generated moments within the raft is shown in Figure 23.



**Bending moments $M_{11}$**

Maximum value = 240.5 kN m/m (Element 156 at Node 15502)

Minimum value = -242.0 kN m/m (Element 80 at Node 10103)

Figure 23. Resulting moments in a raft for the average case

In the following sections the effects of varying the different foundation constituents on the equivalent single pile stiffness and the average settlement of the foundation are explored. Moreover, a comparison between the Matlab and Plaxis 3D results will be carried out and finally the Matlab code will be evaluated by studying its model uncertainty.

The number of piles was varied for the average case, the average settlement that resulted varied substantially when compared to the equivalent single pile stiffness. Figure 25 below shows the variation of average settlement as a function of pile length for the average case.

## 5.1 Plaxis Output Results:

This section will include the obtained output from the Plaxis 3D analysis. This section includes output that cannot be obtained using the simplified method, but is interesting to understand so as to have a better idea on how the design of a piled raft system works.

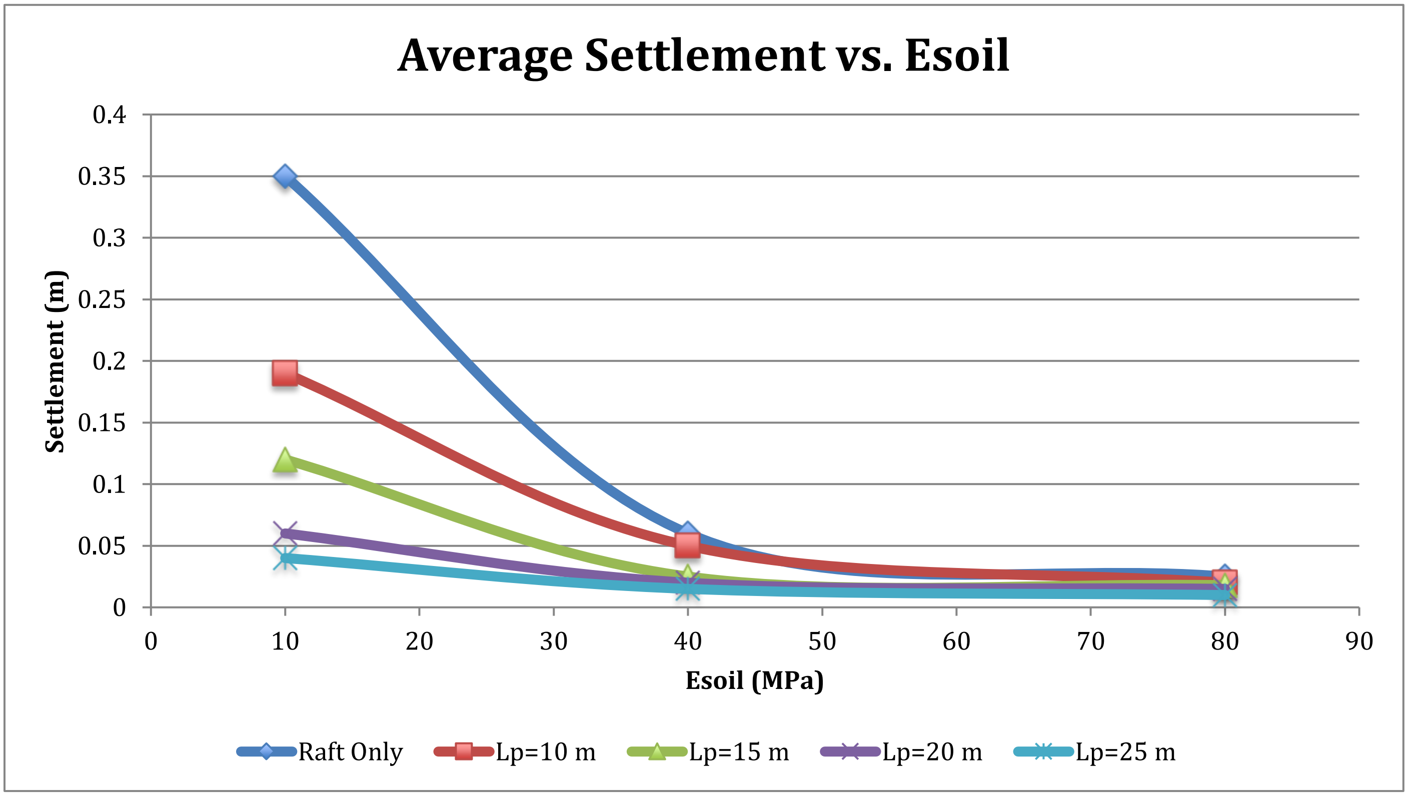


Figure 24. Average settlement vs. Esoil for different pile legnths for the average case

### *5.1.1 Effect of Soil's Young's Modulus on Average Settlement*

As shown in figure 24 the settlement of the raft only option sharply decreases with the increase in soil's Young's modulus, this rate in decrease is much smaller when going from Esoil of 40 to 80 MPa. It is evident that once the piles were introduced, the resulting settlement is significantly reduced. This can be justified by the increase in capacity of the supporting piles as their lengths increase. Even though the steepness of the settlement reduction is relatively similar for all cases, the trend of decreasing overall settlement with the increase in Esoil remains unchanged.

### *5.1.2 Effect of Soil's Young's Modulus on Maximum Positive Bending Moment in the Raft*

Another interesting factor to study would be the bending moment within the raft for the different pile lengths. It was observed that for the case of raft only, no positive moments develop within the raft, and this is expected since there are no piles to cause the development of moments. Moreover, when piles were introduced, positive moments developed, redistribution of the bending moment forces within the raft structure is induced because of the restraints of the piles produced. Figure 25 below details the variation of the positive bending moment as Esoil increases. It can be inferred from the graph that a decrease of up to 60% is observed at higher values of Esoil. Moreover, the increase in pile length results in an increase in the resulting positive bending moments, but this increase stops after reaching a pile length value of 20 m.

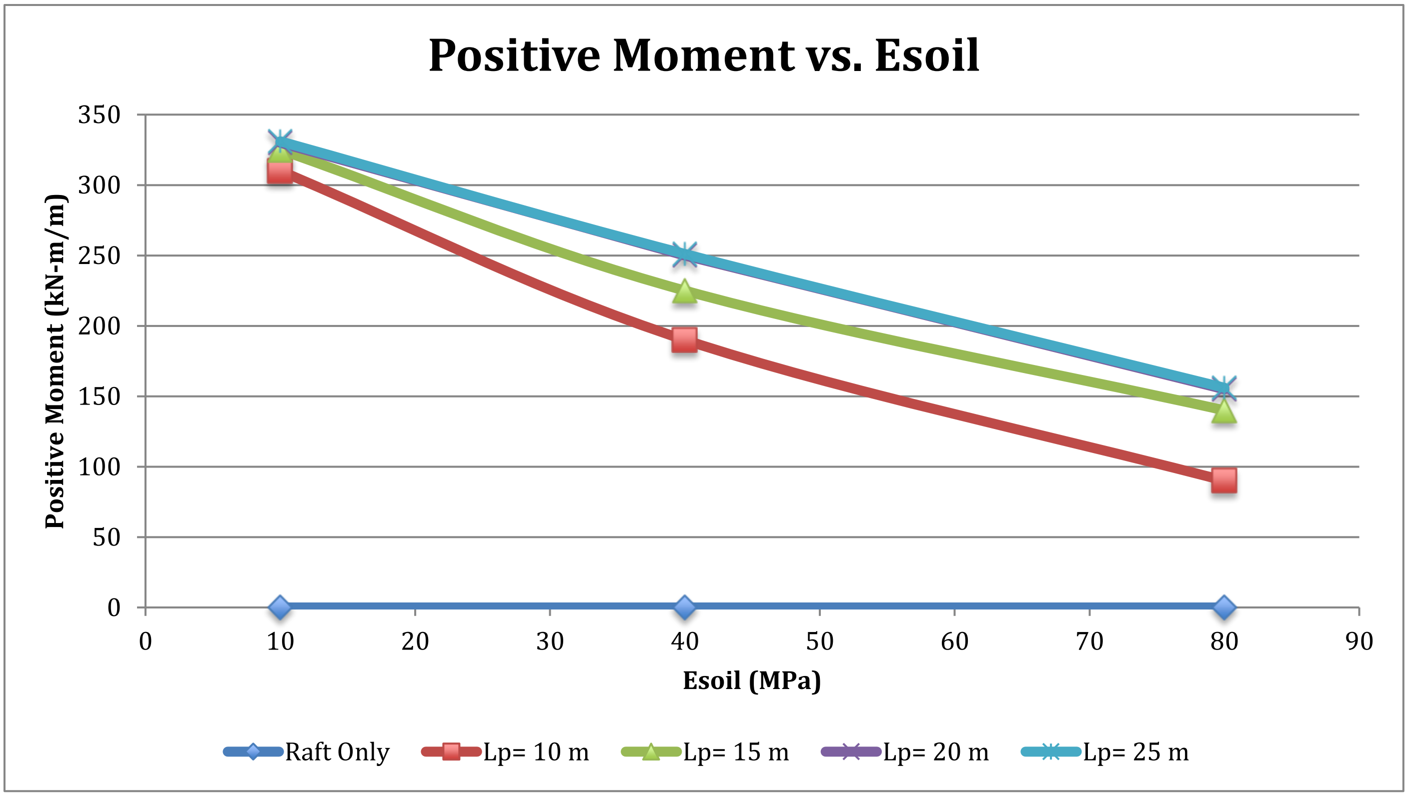


Figure 25. Maximum positive moment vs. Esoil for the average case of undrained clay

### *5.1.3 Effect of Soil's Young's Modulus on Maximum Negative Moment Within the Raft*

With regards to the negative bending moment, the runs showed that for the case of raft only, the negative bending moment is almost persistent for the Esoil ranging between 10 to 40 MPa, then a sudden increase occurs as Esoil approaches 80 MPa. This sudden increase is due to the increase in the resistance capacity of the underlying soil. Once the piles are introduced, a redistribution of moments takes place resulting in both positive and negative bending moments. The presence of piles will aid in controlling the positive and the negative bending moments within the raft structure, this is due to addition restraints provided by the supporting piles. Figure 26 shows a plot of the negative bending moment within a raft vs. Esoil. It can be deduced from the runs that the negative bending moment within the raft is a function of Esoil, pile length and thickness of the raft. For a pile length of 10 and 15 m, the resulting negative bending moments tend to increase with the increase of Esoil. It is important to note that the rate of increase of negative bending moments decreases as Esoil increases to a value beyond 40 MPa. For the cases of pile length = 20 and 25 m, the reverse is true, where a decrease in the value of negative bending moments is observed also the rate of decrease is very minimal as Esoil increases to a value beyond 40 MPa.

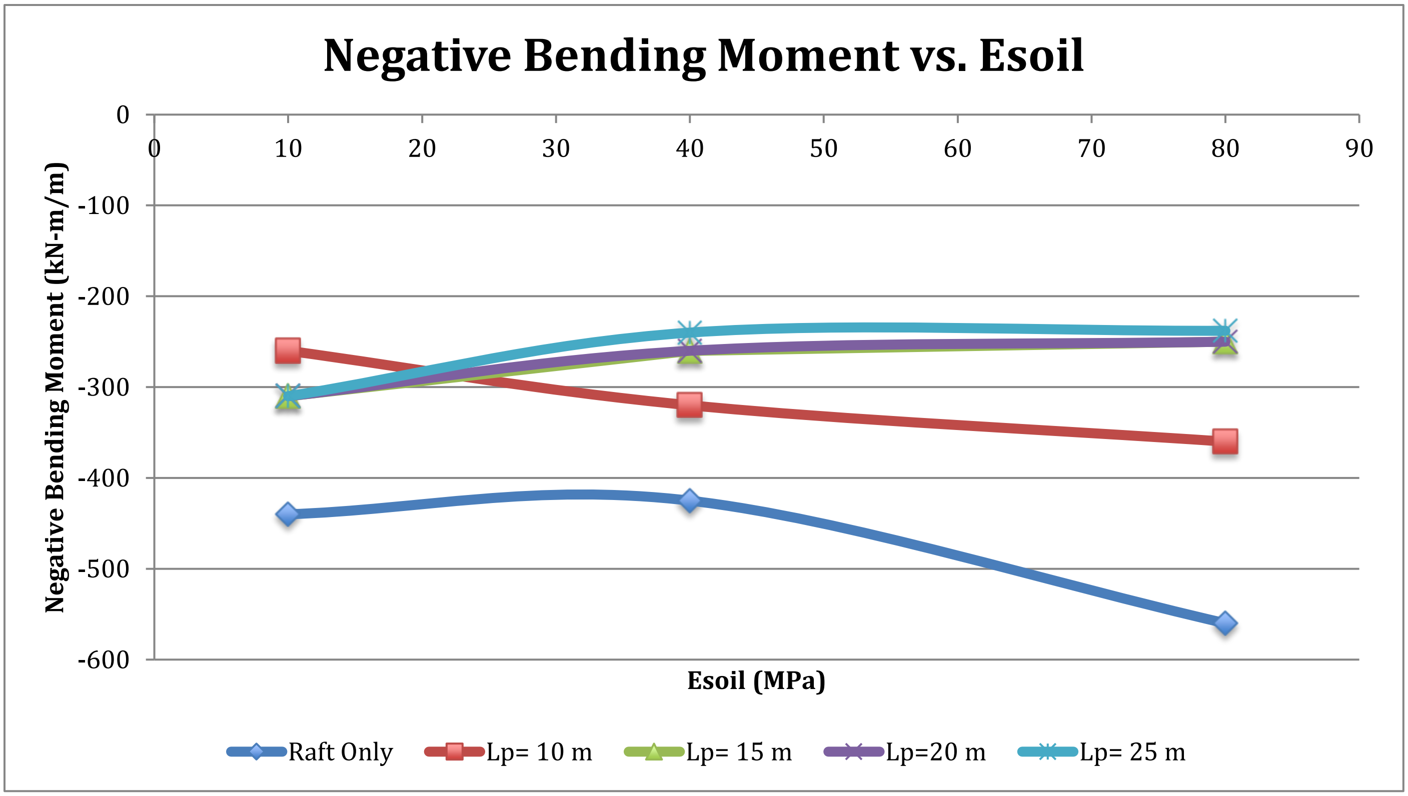


Figure 26. Maximum negative bending moment vs. Esoil for the average case in undrained clay

### *5.1.4 Effect of Raft Thickness*

The thickness of the raft plays a vital role in the performance of the raft and especially with regards to the bending moments generated within the raft. For that reason, an investigation on the effect of raft thickness on the positive and negative bending moment while varying the soil's Young's modulus was held.

### *5.1.5 Effect of Number of Piles on the Maximum Positive Bending Moment in the Raft*

This study was done by also varying the number of piles (36, 64 and 100 piles). These pile numbers were chosen based on the fact that for any number of pile less than 36 piles resulted in zero positive bending moments in the raft. It was concluded from this study that the maximum positive bending moment within the raft tends to decrease as the pile number increases from 36 to 64 piles. However the rate of decrease beyond 64 piles (up to 100 piles) is reduced and somehow reaches an asymptotic value.

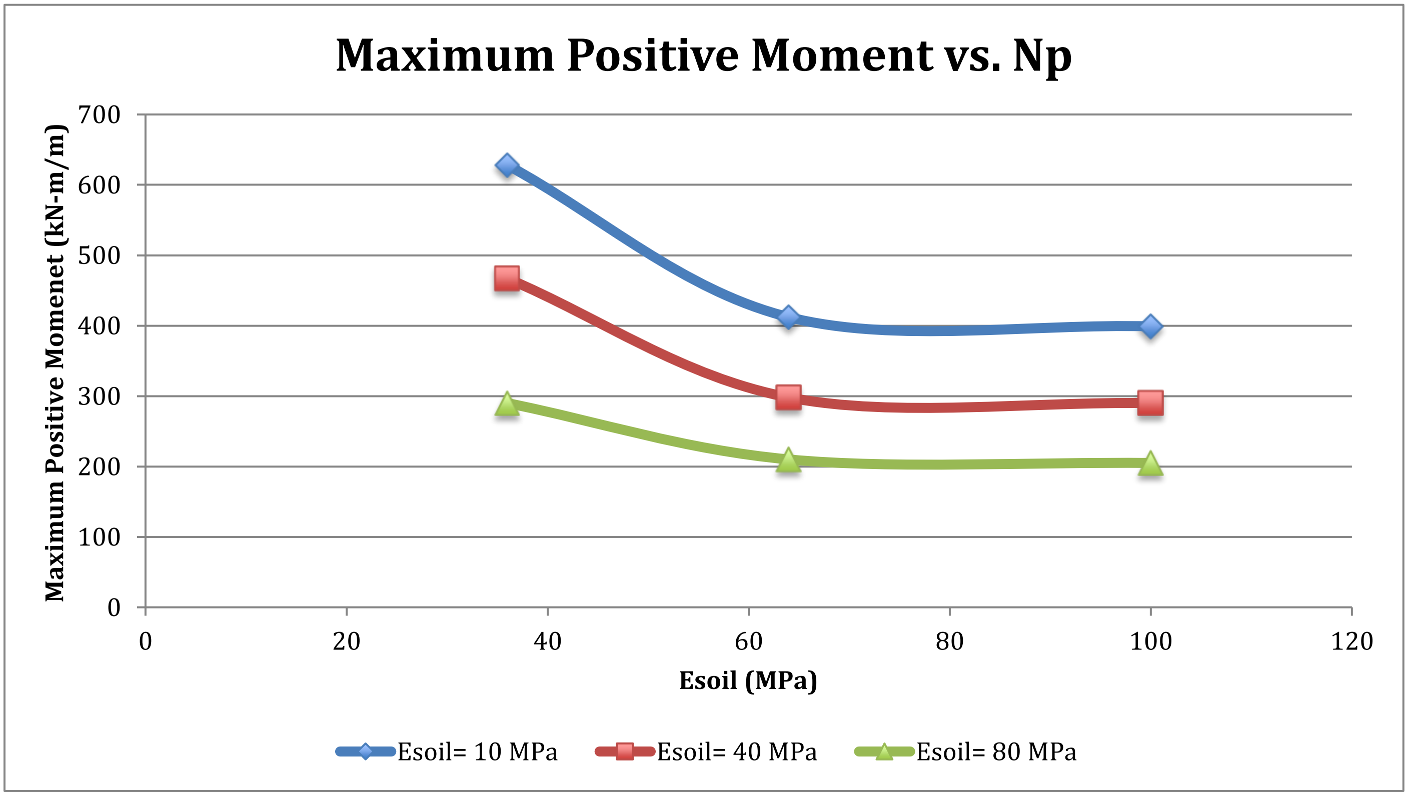


Figure 27. Maximum Positive Moment vs. Np

### *5.1.6 Effect of Number of Piles on the Maximum Negative Bending Moments in the Raft*

For the case of the maximum negative bending moment, an opposite behavior was observed. As the number of piles increased from 36 to 64 piles, an increase in negative bending moment was found, similarly, the rate of increase diminishes as the number of piles increase beyond 64. The behavior is summarized in Figure 28.

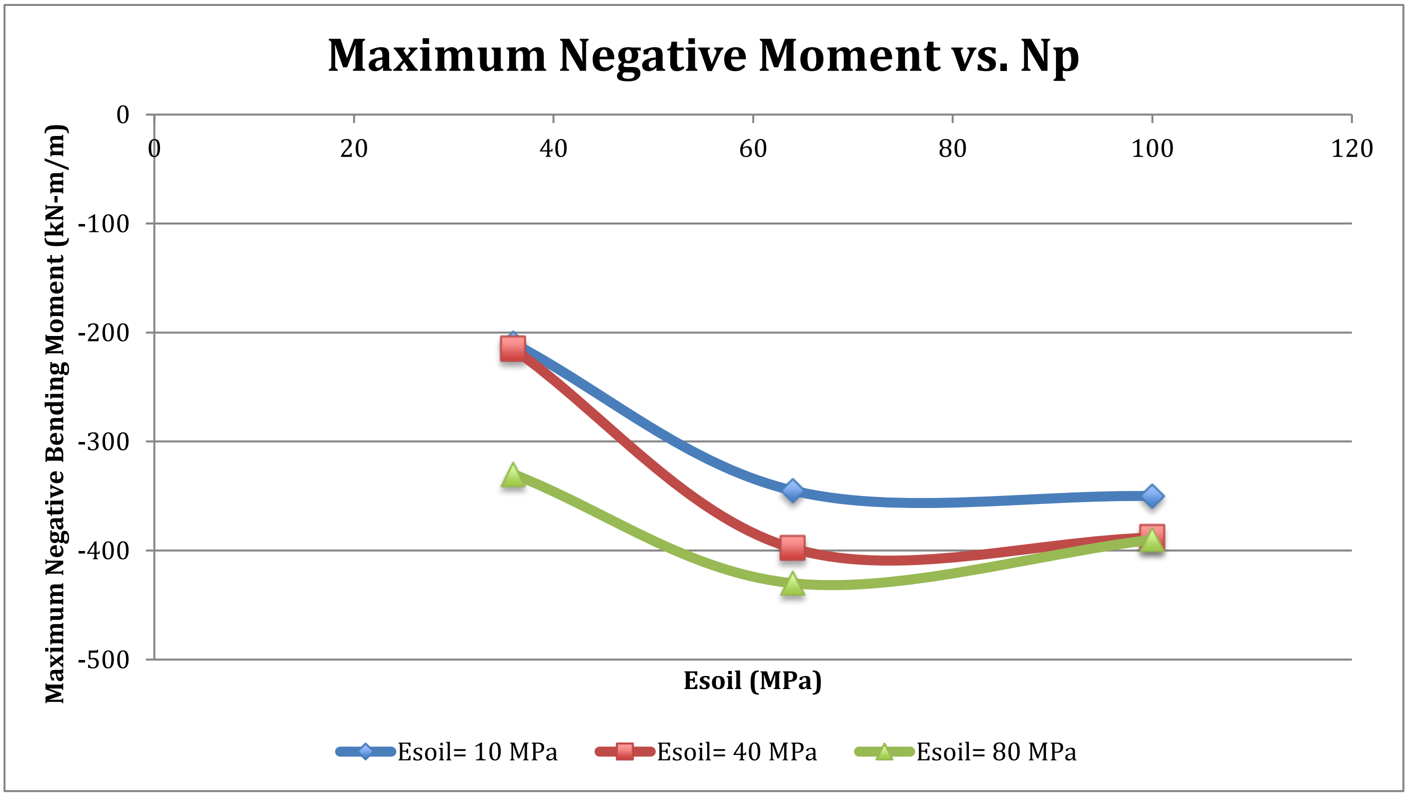


Figure 28. Maximum negative moment vs. Np

## 5.2 Simplified Method Output Results: Design-aid Charts

### *5.2.1 The Effect of the Soil's Young's Modulus:*

As previously discussed and for the purpose of investigating the effect of the stiffness of the soil on the behavior of the piled raft foundation system, the soil elastic modulus Esoil, was varied for a representative range of values. It is evident that as the soil's Young's modulus increases, the average settlement of the piled raft foundation decreases, resulting in an increase in the equivalent pile stiffness value. This increase governs all the cases, and the graphs are almost increasing with a similar slope. Graphs similar to Figure 29 were plotted, each graph corresponds to a certain pile length and a specific raft thickness. Figure 29 shows an average case plot for undrained clay. As shown in Figure 29, the equivalent single pile stiffness increases as the soil's modulus increases. With regards to the different diameters, it is very evident from the plots corresponding to all the diameters that they are reasonably parallel to each other and separated by an intercept. In general and for a specific Esoil, as the pile diameter increases, the pile will be able to carry larger percentage of the load and will therefore settle less, resulting in an increase in equivalent single pile stiffness. For a certain pile diameter, as the Esoil increases, the equivalent single pile stiffness increases as well. This is due to the fact that as the Esoil increases, the soil will be able to carry a higher load and thus the settlement of the system, as a whole will decrease, resulting in an increase in the pile stiffness.

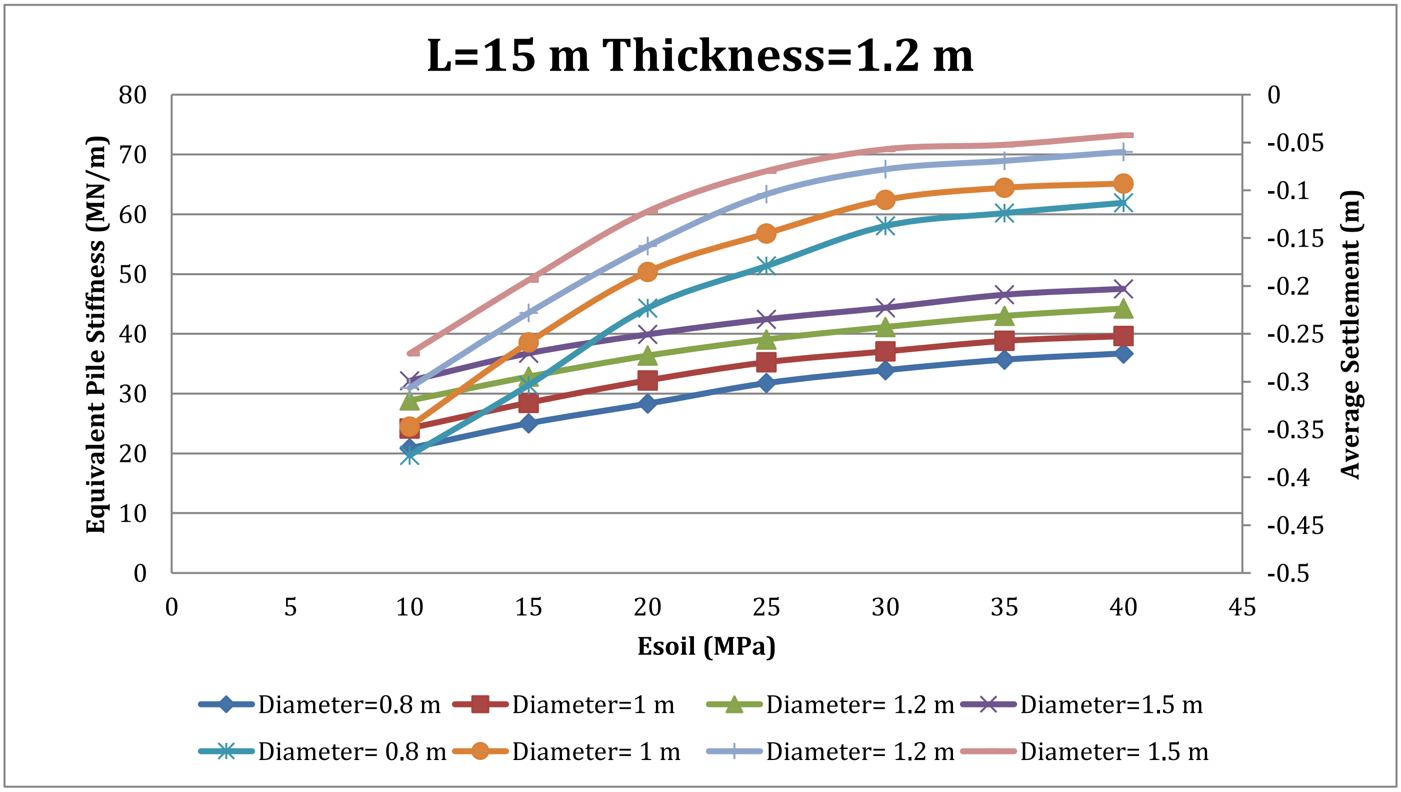


Figure 29. A sample of the design aid chart produced for the average case of undrained clay

In Figure 29, also the plots corresponding to the average settlement of the foundation are presented. As the pile diameter increases for a certain Esoil, the average settlement of the foundation decreases. This behavior is expected since a pile with a large diameter will be able to carry more load thus it will result in a decreased settlement.

Figure 30 shows a graph for the same pile length, but for a raft thickness of 1.6 m. It is very evident that the gap between one diameter and the other is increasing. This behavior is expected because as the thickness of the raft increases, the system as a whole will be able to carry more load and will thus settle less, resulting in an increase in single pile stiffness. The effect of raft thickness on the settlement of the system as a whole is studied by several papers in the literature and all papers come to the same conclusion i.e. as the raft thickness increases, the average settlement of the foundation as a whole decreases.

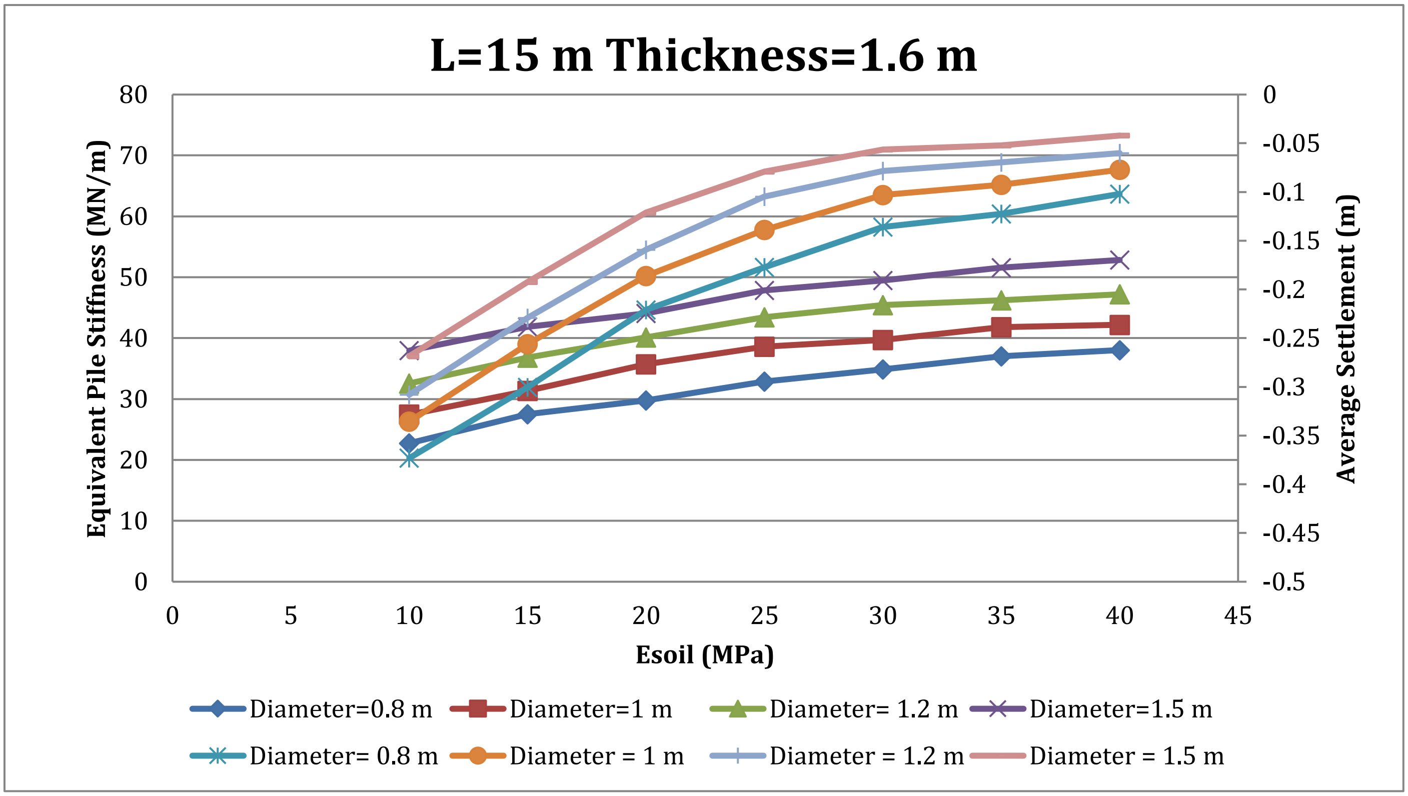


Figure 30. A sample of the design aid charts produced for a pile length= 15 m and a raft thickness= 1.6m

The rate at which the equivalent pile stiffness increases as Esoil increases is constant for all the diameters. A relatively sharp increase in stiffness is witnessed for Esoil increasing from 10-30 MPa, and then the rate of increase becomes much smaller for Esoil going from 30 MPa 50 MPa.

As shown in Figure 30, a similar behavior with regards to the average settlement and the equivalent pile stiffness is witnessed for the cases of drained clays and sands. However, the settlement of drained clay as compared to that of the undrained clay is found to be greater. It is important to mention that the settlement under study is a short-term settlement and thus it is expected to have a higher settlement when dealing with drained clay. In an undrained clay, settlement happens with the presence of water, so as a load is exerted there would be pore pressure build up in the soil, but since the settlement is a short-term settlement, the water pressure cannot dissipate quickly neither can escape, resulting in a lower settlement than that of drained. On the other hand, when dealing with drained clays, when certain load is exerted, it will cause the soil particles to compact and since the soil is drained, no pore pressure build-up results.

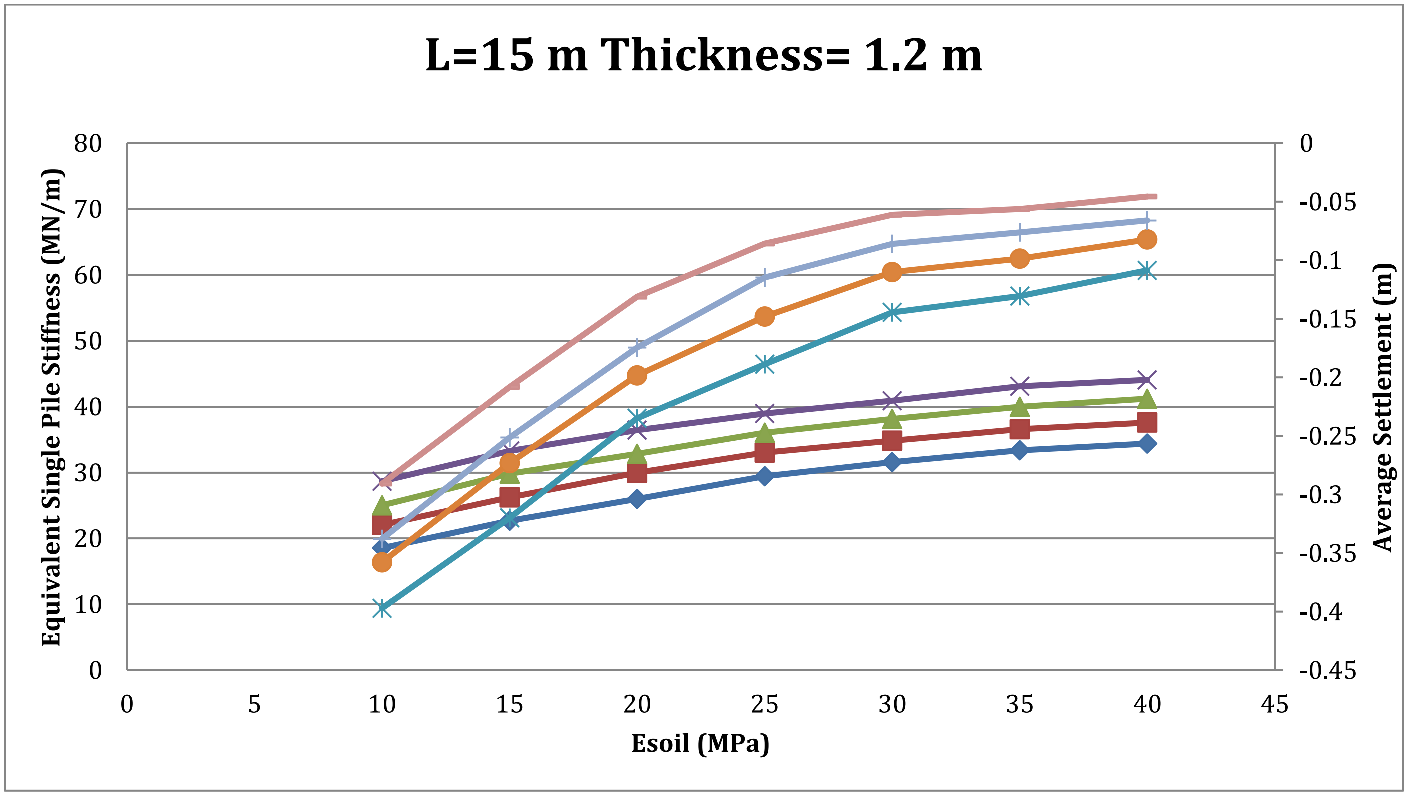


Figure 31. A sample of the design aid chart produced for the average case of drained clay

Similar to the case of the undrained clay, the rate at which the equivalent single pile stiffness increases for all diameters is more or less the same. Another evident issue in both, drained and undrained clay behavior is the rate of settlement decrease as the Esoil increases. As Esoil goes from 10 MPa to 25 MPa, there is a sharp decrease in settlement and then the rate of decrease becomes much lower.

As for the case of the undrained clay, as the thickness of the raft increases, the equivalent pile stiffness also increases. Moreover, the rate of equivalent single pile stiffness increases is similar for all the diameters.

From Figure 33, for a piled raft with a raft thickness of 1.6 m and a specific Esoil, the increase in the equivalent pile stiffness if higher than that of a piled raft with a raft thickness of 1.2 m.

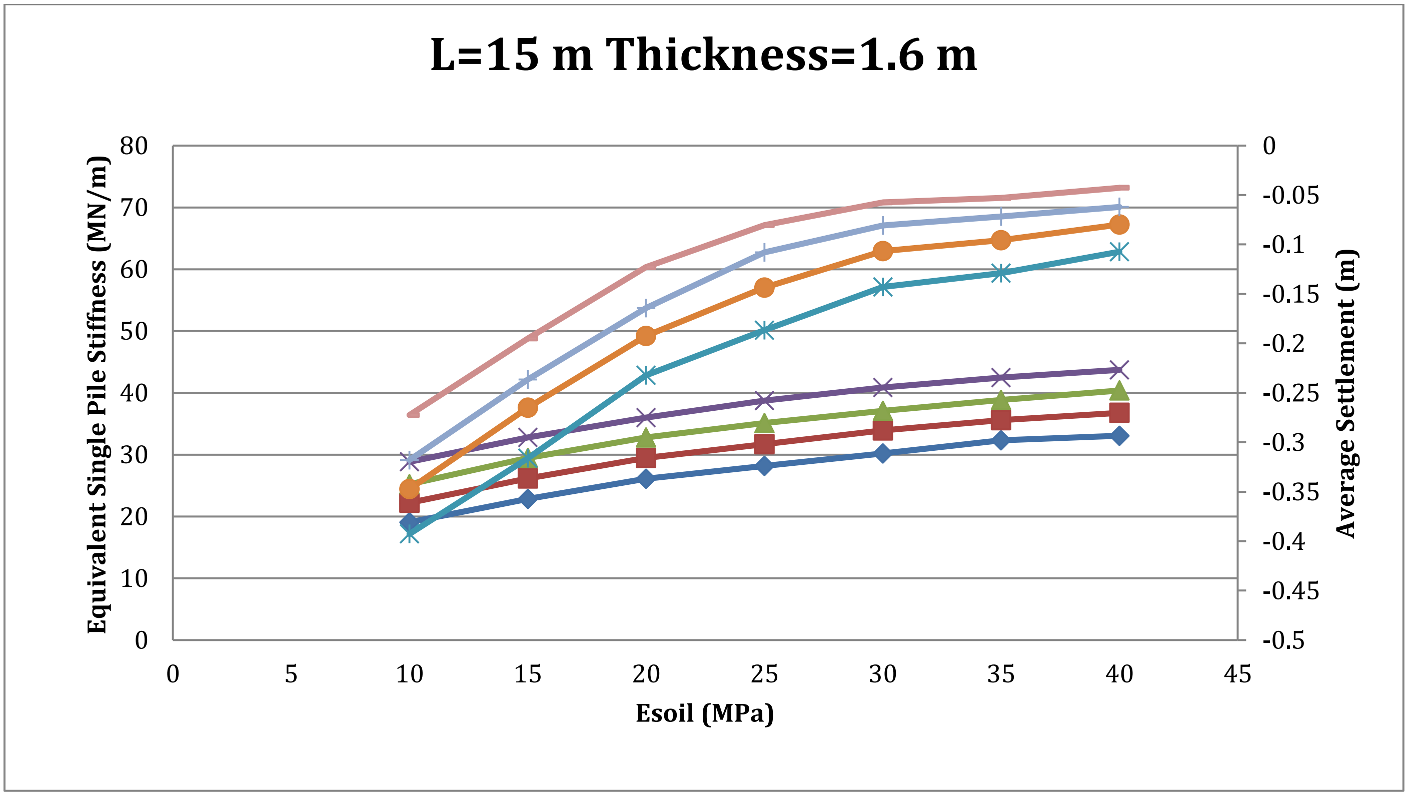


Figure 32. A sample of design aid charts produced for a pile length= 15 m and a raft thickness = 1.6 m

From Figure 32, the rate at which the average settlement of the system as a whole decreases is sharp for relatively small Esoil values and then the decrease becomes minimal for higher values of Esoil.

All the values plotted in the charts above are obtained from the analytical solution code, so in order to verify the results attained, all the cases were done on Plaxis 3D and the results were compared. A thorough analysis for the results is provided in the following section of the thesis. To begin with, it is of interest to see how the settlement obtained from the Matlab code of the total foundation system is compared to the average settlement obtained from the output of Plaxis 3D.

Figure 33 below shows a plot of the obtained average settlement from the analytical solution plotted over the average settlement obtained from finite element analysis. Figure 33 shows the settlement of an undrained clay case with pile length of 15 m and a diameter of 1 m, and a raft thickness of 1.2 m (the average case).

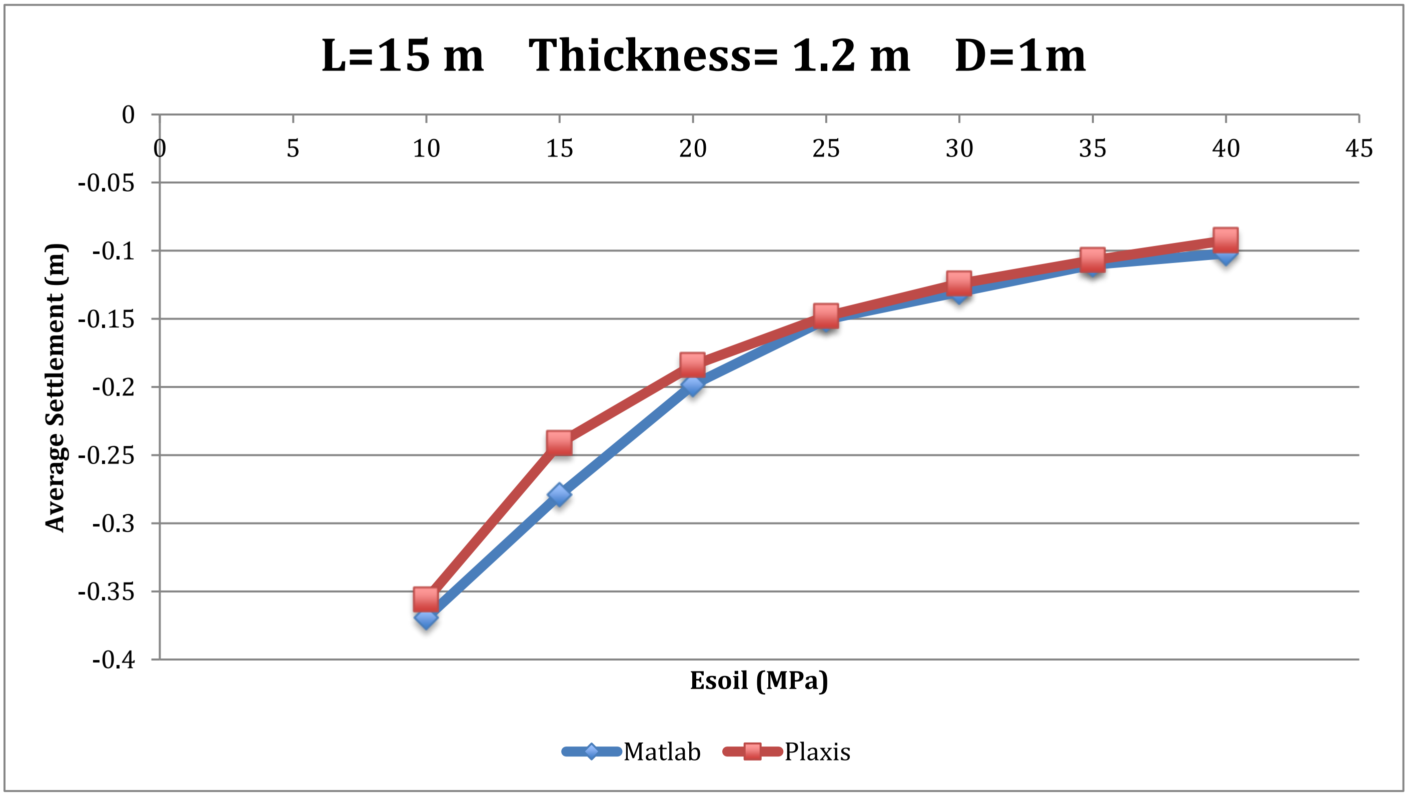


Figure 33. Settlement vs. Esoil for the average case of undrained clay, showing results from the code and from Plaxis 3D

It is evident that both curves relatively overlap. However, there is an overestimation for the settlement in the cases of small Esoil values. This discrepancy is acceptable since the code is overestimating and not underestimating which means that it is somehow accounting for a safety factor. The maximum ratio of predicted settlement to actual (obtained from Plaxis 3D) is not more than 1.2 implying that the design is still in an economic range.

It is interesting to study the boundary cases since the soil will be either very weak or very strong. Figure 34 shows a similar plot to Figure 34 but for the case of a piled raft with a pile length = 10 m and raft thickness=0.8m and pile diameter=0.8 m.

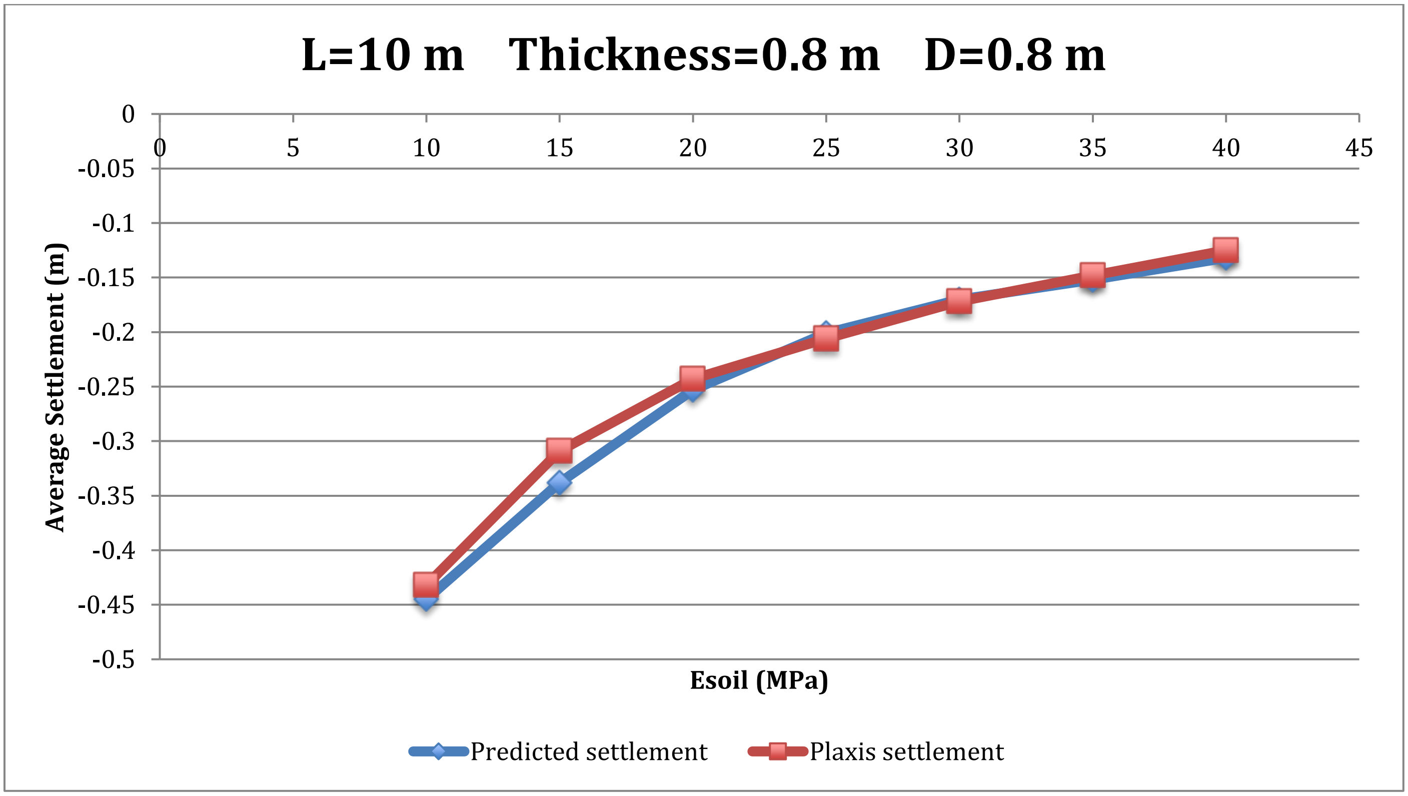


Figure 34. Settlement vs. Esoil for the lower boundary case of undrained clay, showing results from the code and from Plaxis 3D

A similar behavior is observed from the plots above, the lower boundary case and the average case. However, it is interesting to notice that the overestimation in case of the boundary condition is somehow less than that of the average value with a maximum ratio of predicted average settlement to actual average settlement of 1.1.

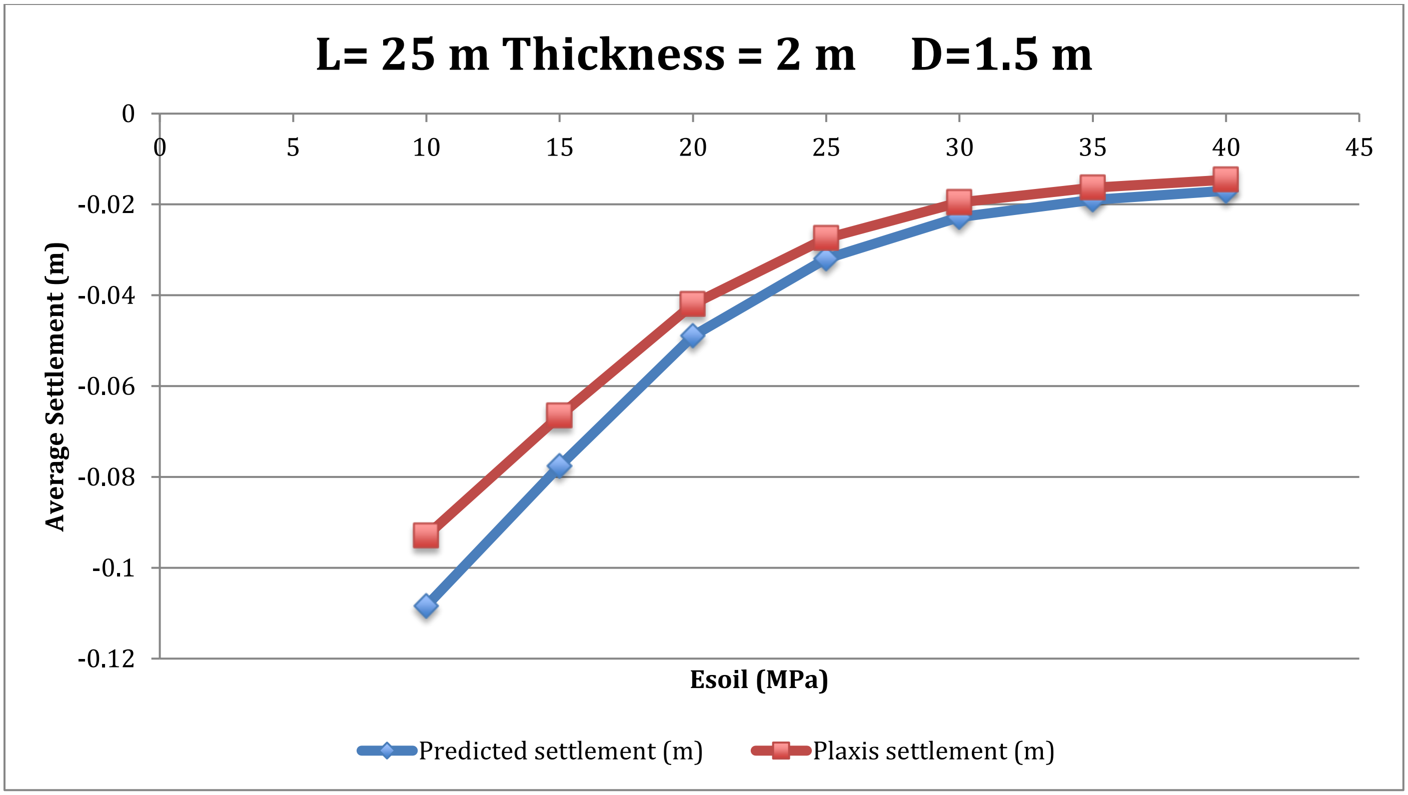


Figure 35. Settlement vs. Esoil for the upper boundary case of undrained clays, showing results from the code and from Plaxis 3D

As mentioned before, the design-aid charts generated are for all representative scenarios with a constant pile number of 64. In order to be able to use them for a varied pile number, the runs performed had to be done once again for all cases. Nevertheless, as an initial step, the average case was studied and analyzed for all possible minimum (25 piles i.e.8D) and maximum (100 piles i.e. 4D) number of piles for the 40 x 40 raft.

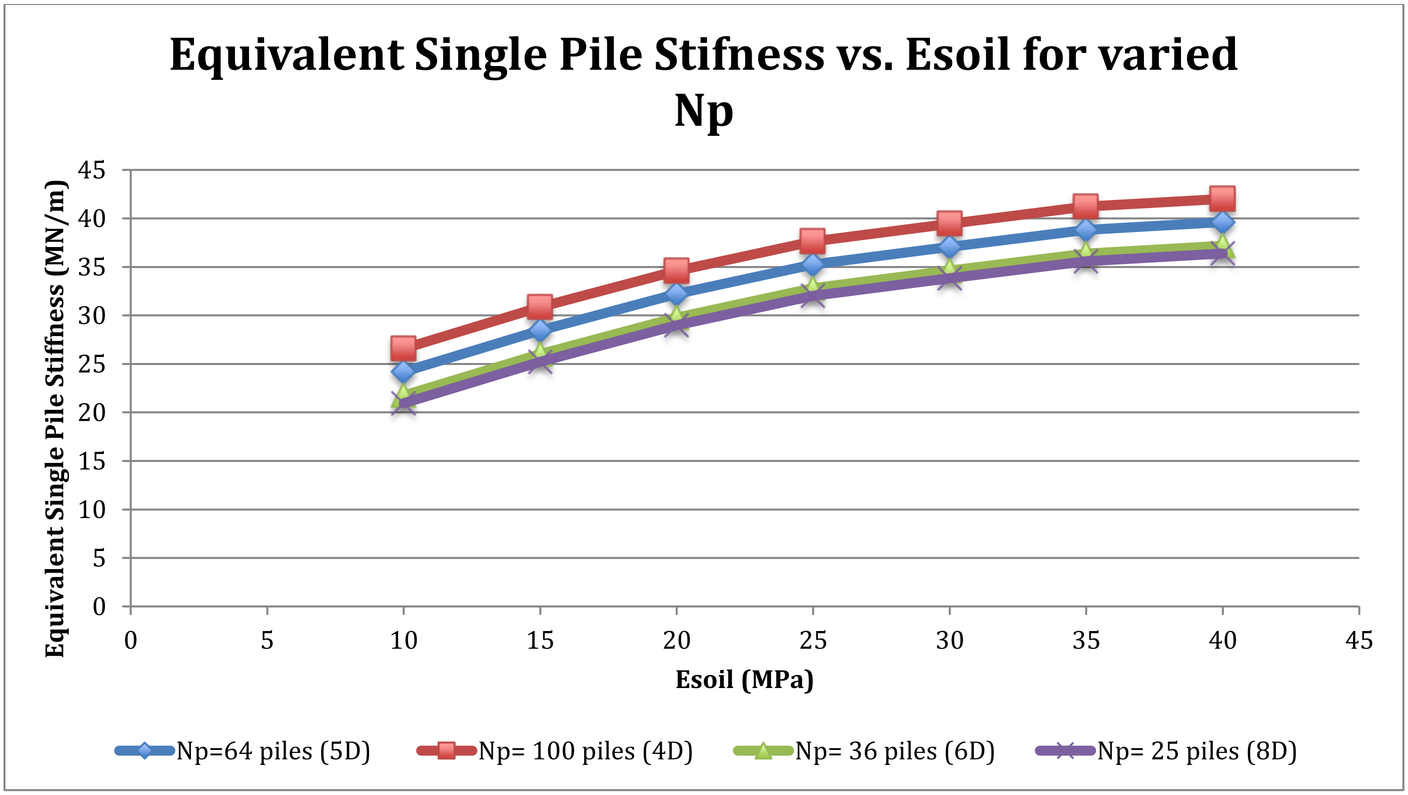


Figure 36. Equivalent single pile stiffness vs. Esoil for a variante number of piles

As shown in Figure 35, as an initial estimate, the stiffness of the piles for any number of piles is relatively close to the one adopted in the design-aid charts (64 piles).

The procedure outlined above will give an estimate of the average settlement. The differential settlements within the foundation largely depend on the distribution of the applied loads and the pile arrangements and the relative rigidity of the raft. Randolph (1994) proposed an approximate method to estimate the maximum differential settlement for a uniformly loaded raft, by relating the ratio of the differential settlement to the average settlement. Charts by Horikoshi and Randolph (1997) for differential settlements can be used, however these charts were not available online and since the cases done in this research were of a uniform pile configuration and uniform load distribution, the differential settlement was not an issue. So in order to prove that, differential settlements from the models constructed using Plaxis 3D and the following table was obtained:

Table 8. Differential settlement

| Minimum Settlement | Average Settlement | Maximum Settlement | Differential Settlement |
|---|---|---|---|
| -0.034 m | -0.0502 m | -0.0634 m | $|0.0294\ m|$ |

So as a first estimate, more or less a differential settlement of 0.0294 seems to be acceptable for the sake of this study.

The main aim of the charts shown in Figures 29-32 is to produce design aid charts that would help the structural engineer choose a single pile stiffness value that is somehow close to reality. Even though the charts are useful, normalizing graphs such as the one shown in Figures 29-32 is a not so simple job for there are several parameters varying and thus it makes things very complicated. Another solution would be to regress the data collected from the analytical solution that was coded on Matlab and then a single equation will result enabling the user to obtain the pile stiffness for a desired pile length and diameter and for a certain raft thickness. The following section will thoroughly explain how the regression was carried out and the obtained results from the regression.

## 5.3 Regression and Statistical Modeling

Statistical modeling focuses on finding a quantitative description on how the variable of interest varies as a function of relevant predictor variables. Statistical modeling in this thesis will be preformed on R. **R** is an open source programming language and software environment for statistical computing and graphics that is supported by the R Foundation for Statistical Computing. The R language is widely used among statisticians and data miners or developing statistical software and data analysis.

### *5.3.1 Statistical Modeling: Multiple Linear Regression*

The simplest statistical model is the simple linear regression, however when there exists more than a single predictor, a multiple linear regression is performed. Multiple regression simultaneously considers the influence of multiple explanatory (raft thickness, soil's Young's modulus, and pile length and diameter) variables on a response variable Y (equivalent single pile stiffness) as shown in Equation 42.

$$y_i = \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + +\beta_k x_{ik} + \epsilon_i, \quad i = 1, \dots, n \text{ (42)}$$

Where,

$x_{i1}, \dots x_{ik}$=observed variables

$\beta_0, \beta_1, \dots \beta_k$=fixed and unknown parameters

$\epsilon_1, \dots \epsilon_n$=i.i.d $\mathbb{N}(0, \sigma^2)$

$\sigma > 0$ is a fixed and unknown parameter

The main aim is to look at the independent effect of each variable while accounting for the influence of potential cofounders:

- The coefficient for each explanatory variable is the predicted change in y for one unit change in x, given the other explanatory variables in the model
- The p-value for each coefficient indicates whether it is a significant predictor of y, given the other explanatory given the other explanatory variables in the model
- If explanatory variables are associated with each other, coefficients and p-values will change depending on what else is included in the model

It is important to test if the regression model produced is significant i.e. whether the model aids in predicting the equivalent single pile stiffness value. Performing an F-test on R tests this.

Testing the Global Utility of the Multiple Regression:

$$H_0: \beta_1 = \beta_2 = \cdots = \beta_k = 0$$

$$H_a: \text{ at least one of } \beta_1, \beta_2, \ldots, \beta_k \neq 0$$

Under $H_0$ the F test statistic has an F distribution with k degrees of freedom in the numerator and [n-(k+1)] degrees of freedom in the denominator. From the upper-tail of the F distribution, if the F statistic becomes large then it will be in the rejection region.

There are several basic steps that need to be done in order to obtain a regression model.

1. Collect the data in a pivot table in an excel file
2. Import the data to R
3. Regress the desired variable that is to be predicted against the predictors by simply using the “lm” function in R.

### 5.3.1.1 Undrained Clay’s Multiple Regression Model:

After performing the aforementioned steps, the following regression model was found for the case of undrained clay.

$$k_p = -40.41042 + 0.44254 \times E_{soil} + 1.83674 \times L + 12.13391 \times Thickness + 16.91568 \times D$$

Where,

$k_p$=equivalent single pile stiffness prediction

$E_{soil}$=Young’s modulus of the soil

$L$=pile length

$Thickness$=raft thickness

$D$=pile diameter

It is evident from the regression that all four predictors have a positive effect on the equivalent single pile stiffness value. However, the slopes of each differ depending on how much each predictor influences the equivalent pile stiffness value. For instance, the influence of the soil's Young's modulus ($E_{soil}$) is very minimal when compared to the influence of the pile diameter on the stiffness value. From the regression equation it can be inferred that the most influential predictor is the pile diameter since it has the highest slope. The regression model does not have a valuable meaning if the data regressed is not following the conditions of a regression. So to be able to judge whether the obtained regression model is meaningful, diagnostic plots are generated using R as well as shown in Figure 36.

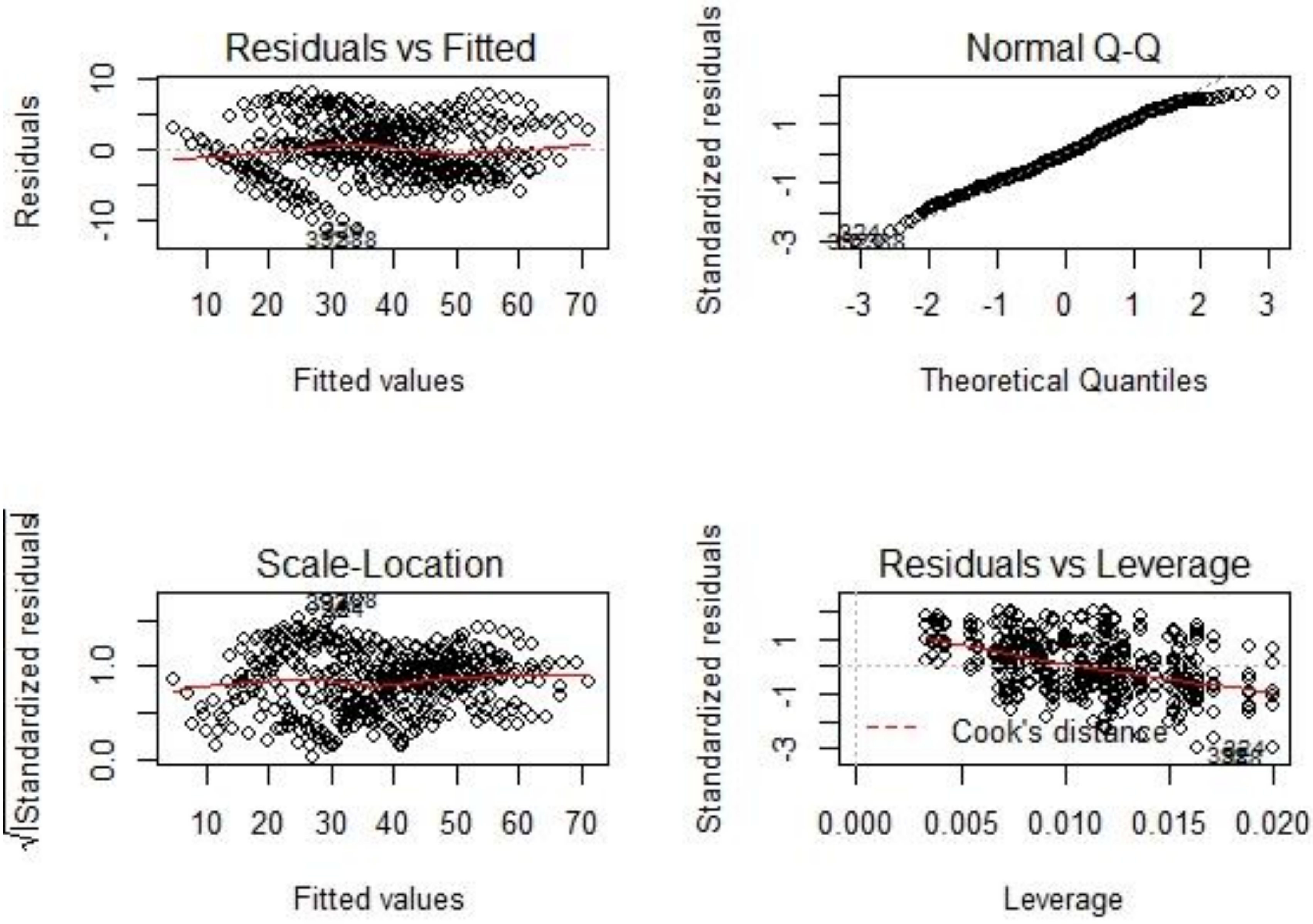


Figure 377. Diagonostic plots for the undrained clay

#### 5.3.1.1.1 Residual vs. Fitted Plot:

This plot shows if residuals have non-linear patterns. There could be a non-linear relationship between predictor variables and an outcome variable and the pattern could show up in this plot if the model doesn't capture the non-linear relationship. Since there is almost an equally spread residuals around a horizontal line without distinct patterns, that is a good indication the data is not bias.

#### 5.3.1.1.2 Normal Q-Q Plot:

This plot shows if residuals are normally distributed. One of the main conditions of applying a regression model is that the residuals have to be normally distributed. As shown in Figure X-b, the residuals lie on the normal Q-Q plot implying that the residuals follow a normal distribution.

#### 5.3.1.1.3 Scale-Location Plot:

It's also called Spread-Location plot. This plot displays if residuals are spread equally along the ranges of predictors. This is the way to check the assumption of equal variance (homoscedasticity). It seems that there is an equal variance for there is a horizontal line with equally (randomly) spread points.

#### 5.3.1.1.4 Residual vs. Leverage Plot:

This plot aids in finding influential cases if any. Not all outliers are influential in regression model analysis. Even though data have extreme values, they might not be influential to determine a regression line. That means, the results wouldn't be much different if the values are either include or exclude them from analysis. They follow the trend in the majority of cases and they don’t really matter; they are not influential. On the other hand, some cases could be very influential even if they look to be within a reasonable range of the values. They could be extreme cases against a regression line and can alter the results if we exclude them from analysis. Another way to put it is that they do not follow the trend that the majority of the cases follow.

### *5.3.2 Drained Clay’s Multiple Regression Model:*

After performing the aforementioned steps, the following regression model was found for the case of undrained clay.

$$k_p = -39.58788 + 0.44323 \times E_{soil} + 1.68257 \times L + 11.49714 \times Thickness + 15.83148 \times D$$

Where,

$k_p$=equivalent single pile stiffness prediction

$E_{soil}$=Young’s modulus of the soil

$L$=pile length

$Thickness$=raft thickness

$D$=pile diameter

The drained clay regression is somehow similar to that of undrained clay, but the slopes of the predictors in the drained clay regression are less than those of the undrained clay. This is relatively expected because as shown in the charts in Chapter 5, the resulting single pile stiffness of drained clay is expected to be less than that of undrained clay for the same piled raft characteristics.

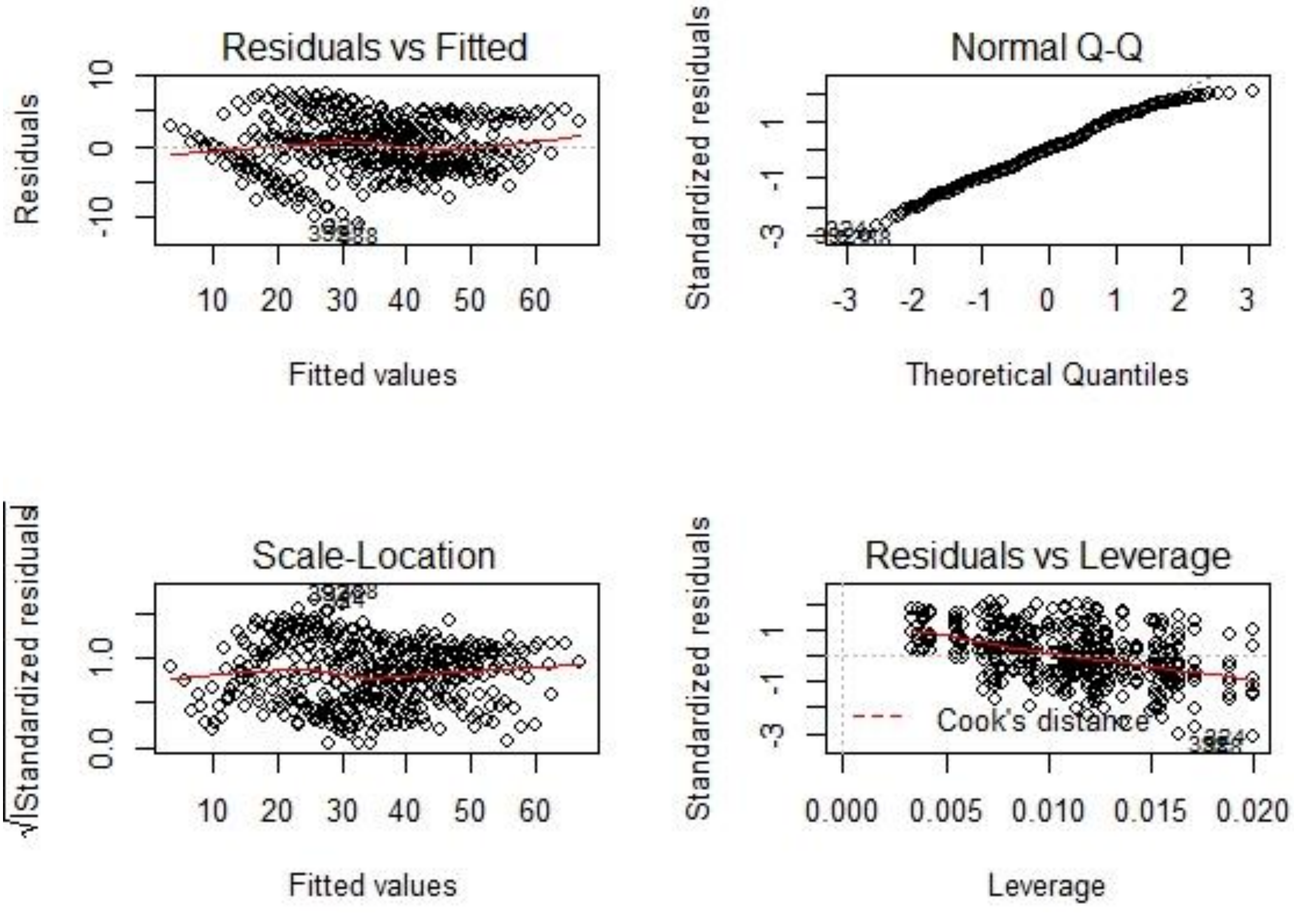


Figure 38. Diagnostic plots for drained clay

From the diagnostic plots the following can be inferred:

1. Residual vs. Fitted: residuals are equally spread
2. Normal Q-Q plot: residuals follow a normal distribution
3. Scale-location plot: no specific pattern, the residuals are randomly spread
4. Residual vs. Leverage plot: no leverage or influential cases

### *5.3.3 Sand's Multiple Regression Model:*

After performing the aforementioned steps, the following regression model was found for the case of undrained clay.

$$k_p = -39.25408 + 0.62536 \times E_{soil} + 1.64038 \times L + 13.44731 \times Thickness + 21.46410 \times D$$

Where,

$k_p$=equivalent single pile stiffness prediction

$E_{soil}$=Young's modulus of the soil

$L$=pile length

$Thickness$=raft thickness

$D$=pile diameter

Contrary to the obtained regression from the drained and undrained clay, the regression in the case of sands has somehow higher slopes than both clay cases. It is important to mention that all three regression models cause an increase in the single equivalent pile stiffness as any of the predictors increases.

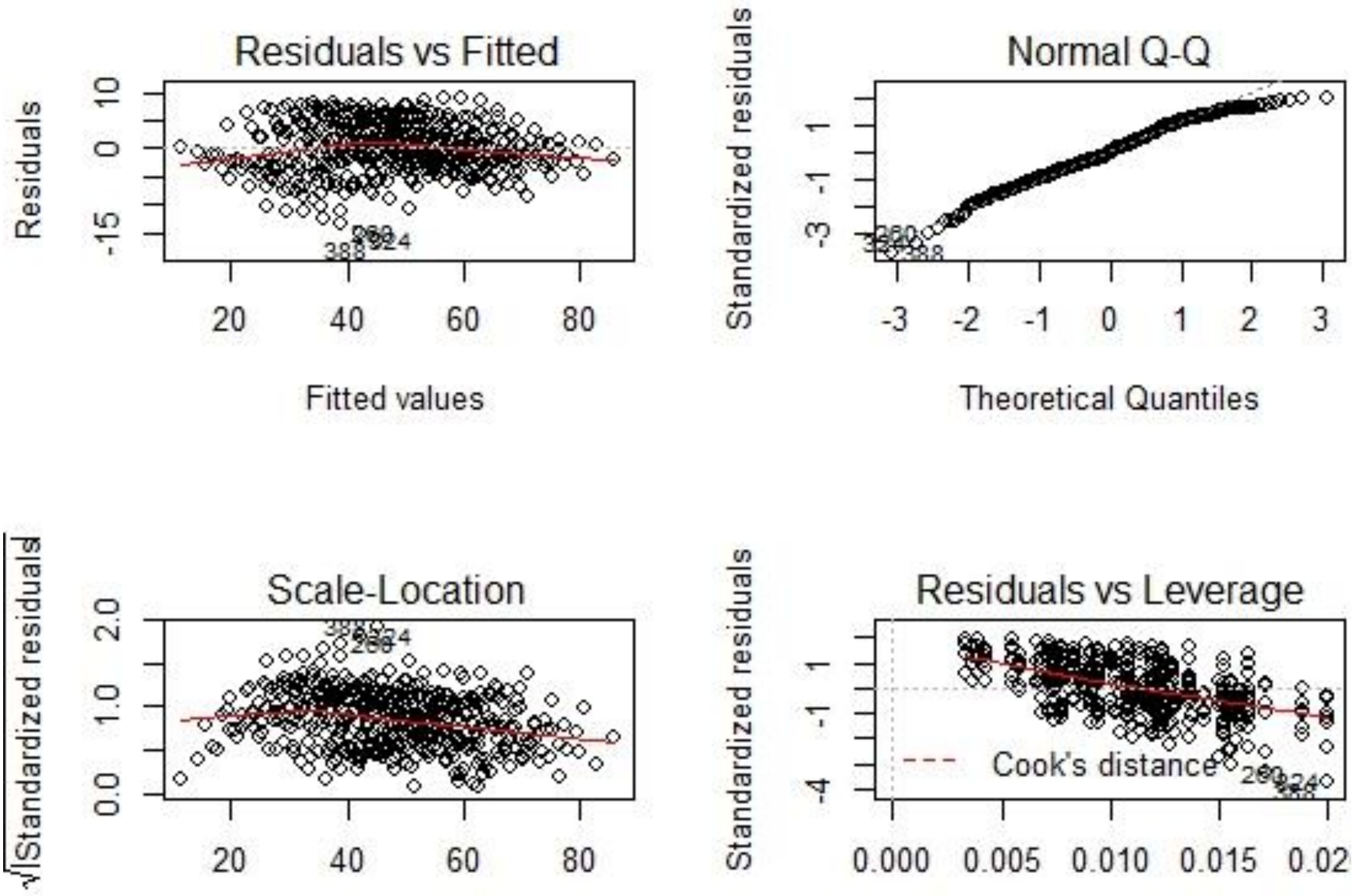


Figure 39. Diagnostic plots for sands

Similarly, from the diagnostic plots the following can be inferred:

1. Residual vs. Fitted: residuals are equally spread.
2. Normal Q-Q plot: residuals follow a normal distribution.
3. Scale-location plot: no specific pattern, the residuals are randomly spread.
4. Residual vs. Leverage plot: no leverage or influential cases.

### 5.4 Model Uncertainty:

After presenting the simplified method and producing regression models that will aid the structural engineer with choosing a corresponding pile stiffness for a certain pile length, diameter and a desired raft thickness, it is important to statistically verify that the model is somewhat close to reality and thus studying its model uncertainty deems a necessary part of the process.

Three model uncertainties were produced for this thesis, one for each kind of soil.

Table 8 below shows the corresponding model uncertainty along with the covariance value for each of the soil type.

The model uncertainty of a certain data is simply the ratio of the predicted data to the actual data, in this thesis the predicted data represents the data obtained from the code or from the regression, and the actual data is the data obtained from Plaxis 3D output.

Table 9. Model Uncertainties and COV

| Soil Type | Model Uncertainty (Code) | Model Uncertainty (Regression) | COV (Code) | COV (Regression) |
|---|---|---|---|---|
| Undrained Clay | 1.14 | 1.34 | 0.129 | 0.22 |
| Drained Clay | 1.12 | 1.21 | 0.133 | 0.32 |
| Sands | 1.17 | 1.42 | 0.129 | 0.36 |

# CHAPTER VI

# SUMMARY OF THE RESULTS AND CONLCUSIONS

A summary of the parametric study results is presented in this chapter. First, the results obtained from the finite element analysis are summarized, after which results from the simplified method are summarized. Finally a summary on the comparison of both methods is summarized and recommendations on the suitability of using the simplified method are provided.

## 6.1 Foundation's Average Settlement:

Settlement is a key feature of the piled raft foundation behavior investigated in this parametric study using the simplified method and finite element analysis; a summary of the results obtained is summarized (the following results are deduced from the simplified method and from Plaxis 3D):

- A decrease in the average foundation settlement is found for an increase in soil's Young's modulus, for all the combinations of raft thickness, pile length, diameter and configuration for all combinations of raft thickness or pile length and configuration.
- The effect of number of piles on the average settlement of the piled raft foundation is substantial when dealing with weak soils. Nevertheless, the number of pile's effect on the average settlement of the foundation is reduced as the soil's Young's modulus increases.
- Irrespective of the piled raft's characteristics, the pile length has a substantial effect on the average settlement of the foundation. When the pile length increases, the overall

settlement decreases, for all combinations of pile diameters, raft thicknesses and soil's Young's modulus.

- The increase in the raft thickness yields a minimal decrease in the total foundation settlement; however this decrease diminishes beyond a raft thickness of 1.6 m. Moreover, the effect of the raft thickness on the overall foundation diminishes for very stiff soils.

## 6.2 Raft's Positive and Negative Bending Moments:

A summary of the variation in positive and negative bending moments within the raft structure with the different constitutive elements of the piled raft foundation investigated in this parametric study via finite element analysis and the following can be inferred:

- An increase in the soil elastic modulus Esoil causes a reduction in the positive bending moments irrespective of raft thickness or pile length or even pile configuration. Reductions of up to 60% were recorded. As for negative bending moments, the behavior is a function of both Esoil and length of piles, where for the lower values of pile length, negative moments increase with the increase in Esoil, and decrease for the higher values of pile length.
- Regardless of Esoil, positive bending moments decrease with the increase in Np, to reach an asymptotic value as Np increases beyond 64 piles; while, negative moments tend to increase with a greater number of piles. Similar to the positive moment, the asymptotic effect in bending moment is witnessed for a pile number above 64.
- For any value of pile length, the increase in Esoil results in a direct decrease in the value of the positive moment computed; whereas, irrespective of Esoil, the increase

in pile length, results in a direct increase in the positive bending moments, while a reduction in negative moments is recorded.

- An increase in raft thickness, tr, seems to have no noticeable effect of positive bending moments, irrespective of Esoil; whereas, negative bending moments undergo a significant increase with the increase in raft thickness.

The resulting bending moments within the raft cannot be obtained from the proposed simplified method. However, the simplified method serves, as an initial step towards design and full design has to take place.

## 6.3 Equivalent Single Pile Stiffness (from the simplified method):

From the design-aid charts the following can be concluded for the equivalent single pile stiffness:

- An increase in Esoil results in an increase in the equivalent pile stiffness value, irrespective of the case, for all pile lengths, diameters and raft thickness, an increase takes place.
- An increase in pile length and diameter results in an increase in the equivalent pile stiffness.
- An increase in raft thickness seems to increase the equivalent pile stiffness.
- An increase in number of piles results in a decrease in average settlement, implying an increase in equivalent pile stiffness value.

The design-aid-charts are presented in the Appendix for all cases. However, it is recommended to use the regression models for it is an easier approach. Statistical proof shows that the proposed regression models are fall in a relatively acceptable range when compared to results obtained from Plaxis 3D.

## 6.4 Conclusions:

A series of conclusions, can be drawn from the study subject of this thesis:

- Piled Raft foundation behavior observed in throughout this parametric study using Plaxis 3D and the proposed simplified method
- Plaxis 3D is an excellent tool to model and analyze complex foundations such as piled rafts
- The local state of design is critically analyzed and a modified/simplified method is proposed on the basis of a parametric study of a wide range of representative cases.
- Design-aid charts for all the cases are produced along with regression models that could be adopted by structural engineers as an initial input to their full-scale structural models when representing the foundation system and supporting materials/ground.

## 6.5 Future Research:

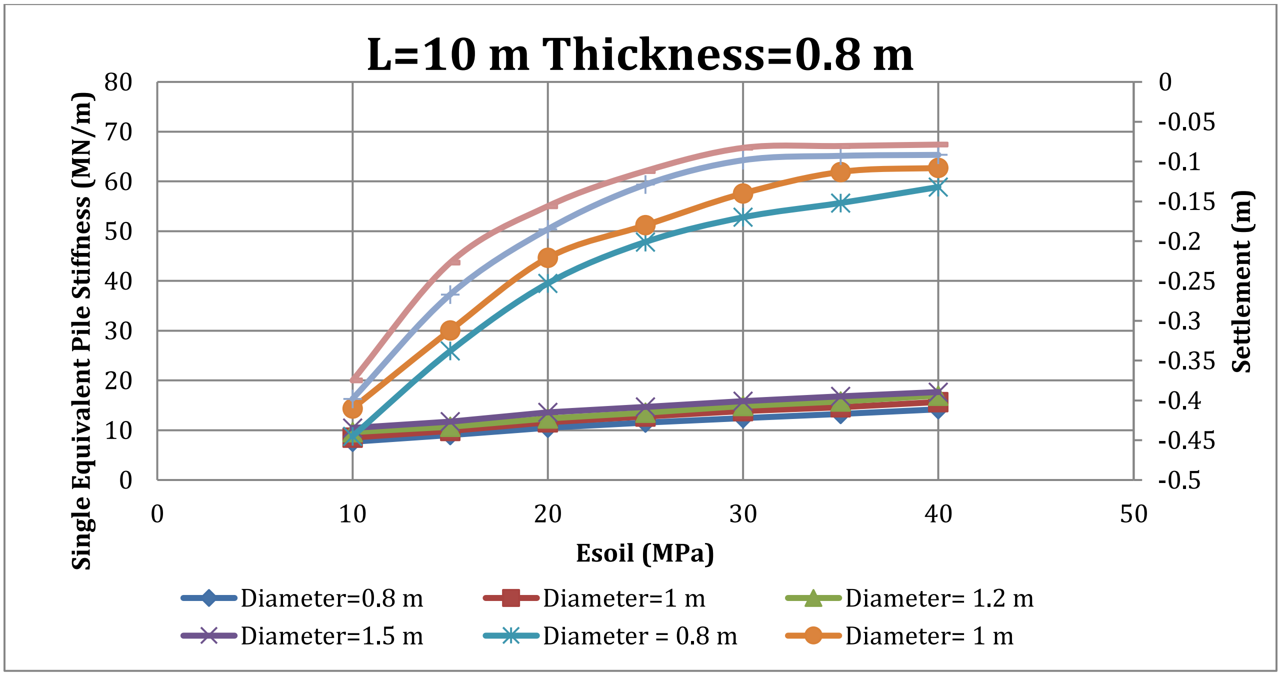
The modes of behavior deduced from the results of the parametric study undertaken are in very good agreement of what is reported in the literature. A future development of this research project would tackle the implementation of the proposed simplified method in generating design-aid charts for piled raft foundations on multiple soil layers and under varied load conditions.

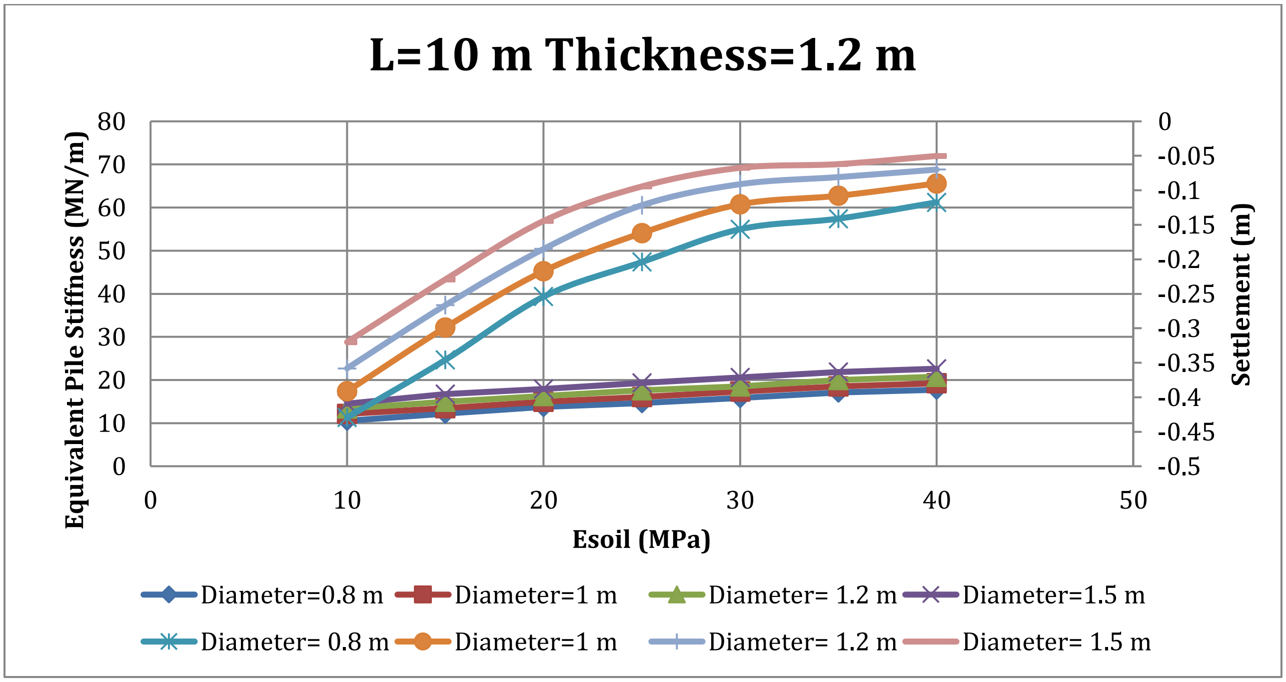
# APPENDIX

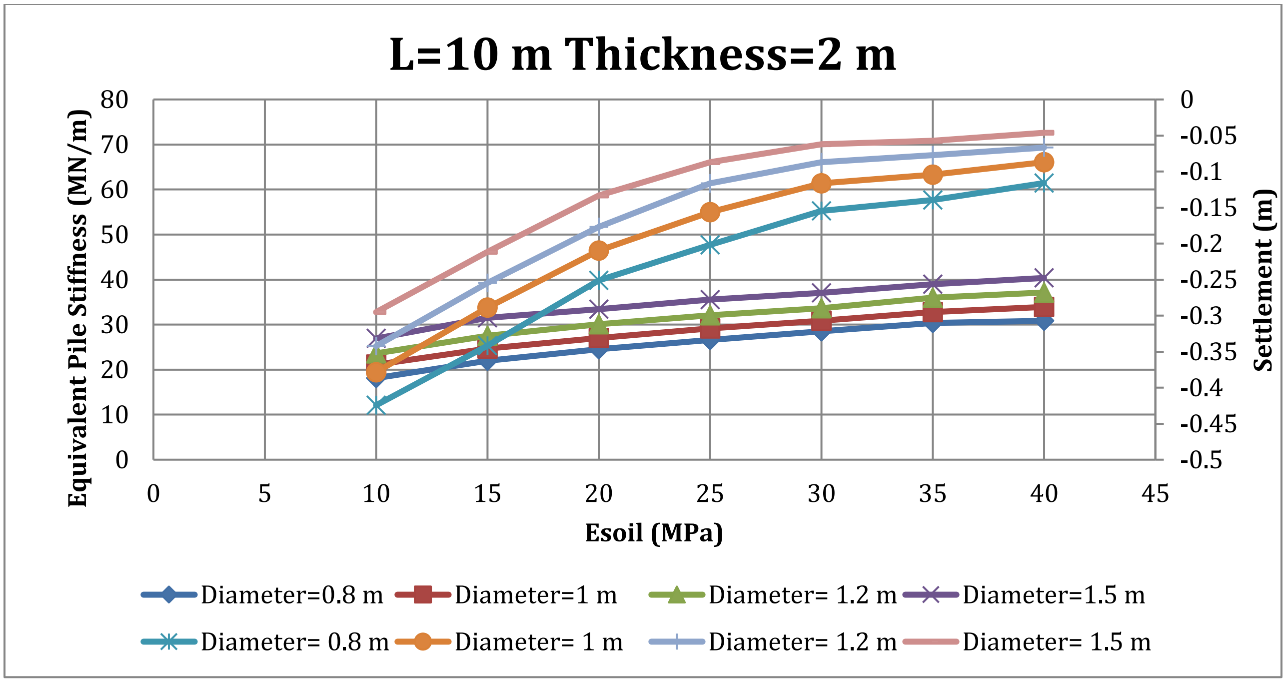
## 7.1 Design Charts

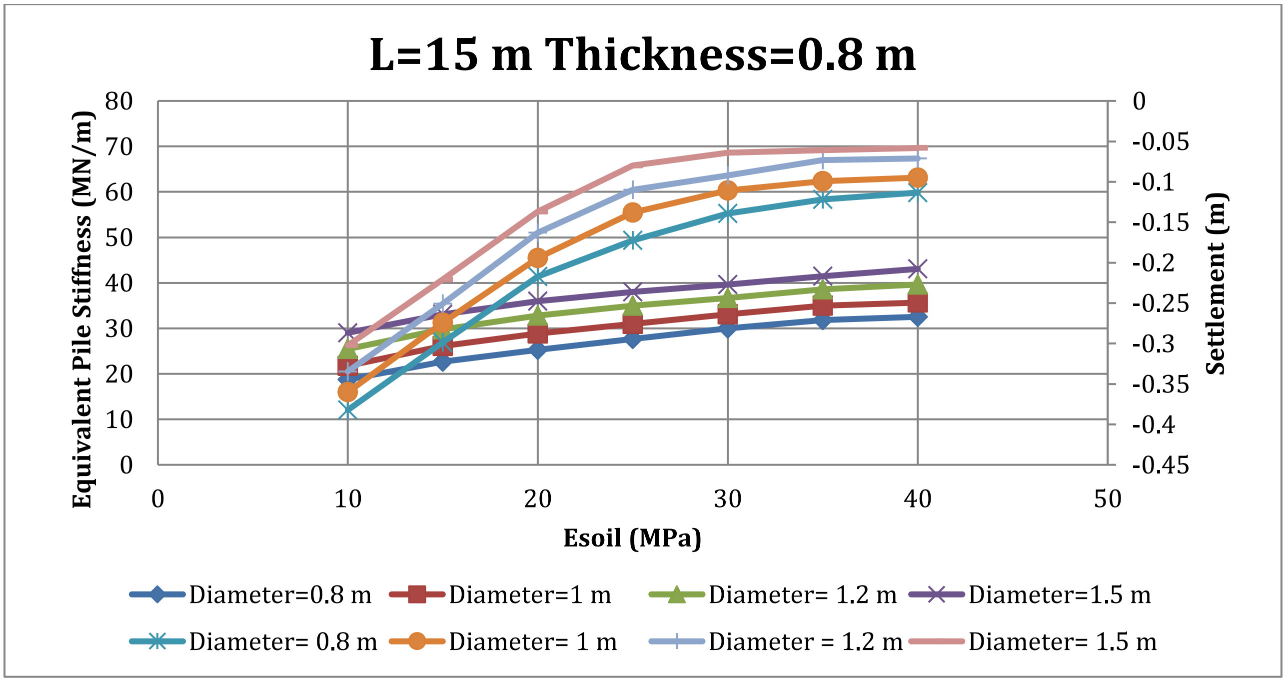
The design charts generated are presented in this section:

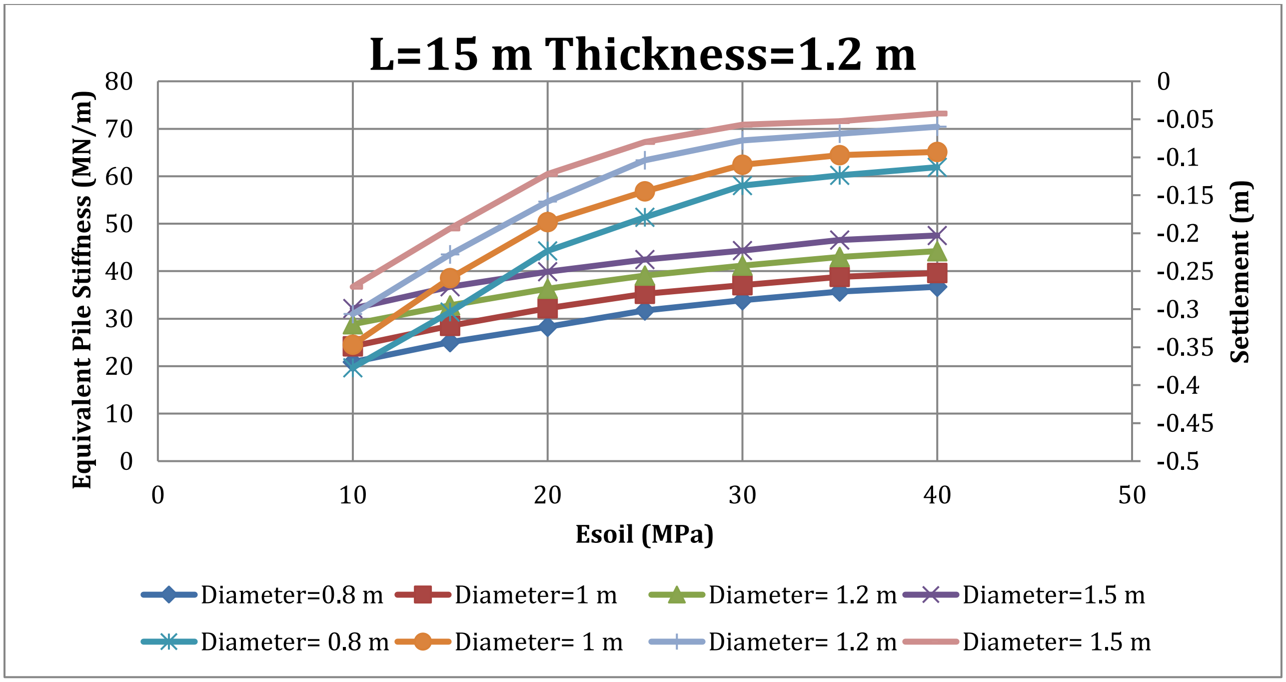
Undrained Clays:

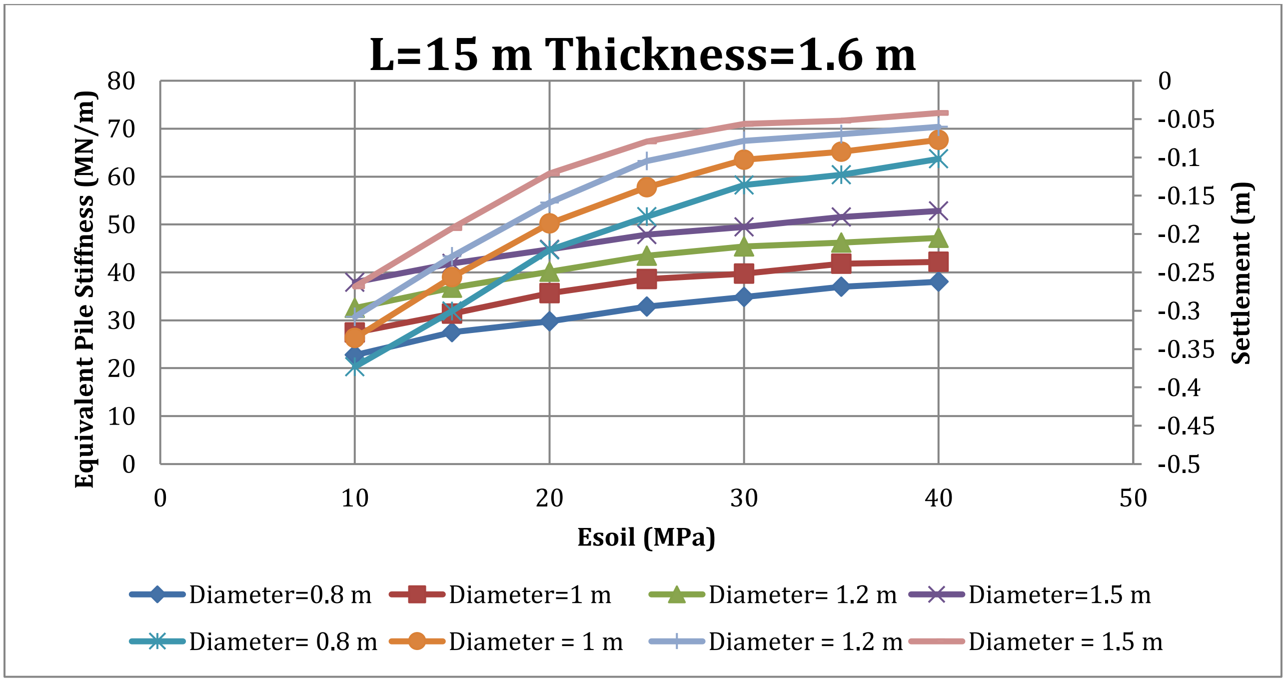


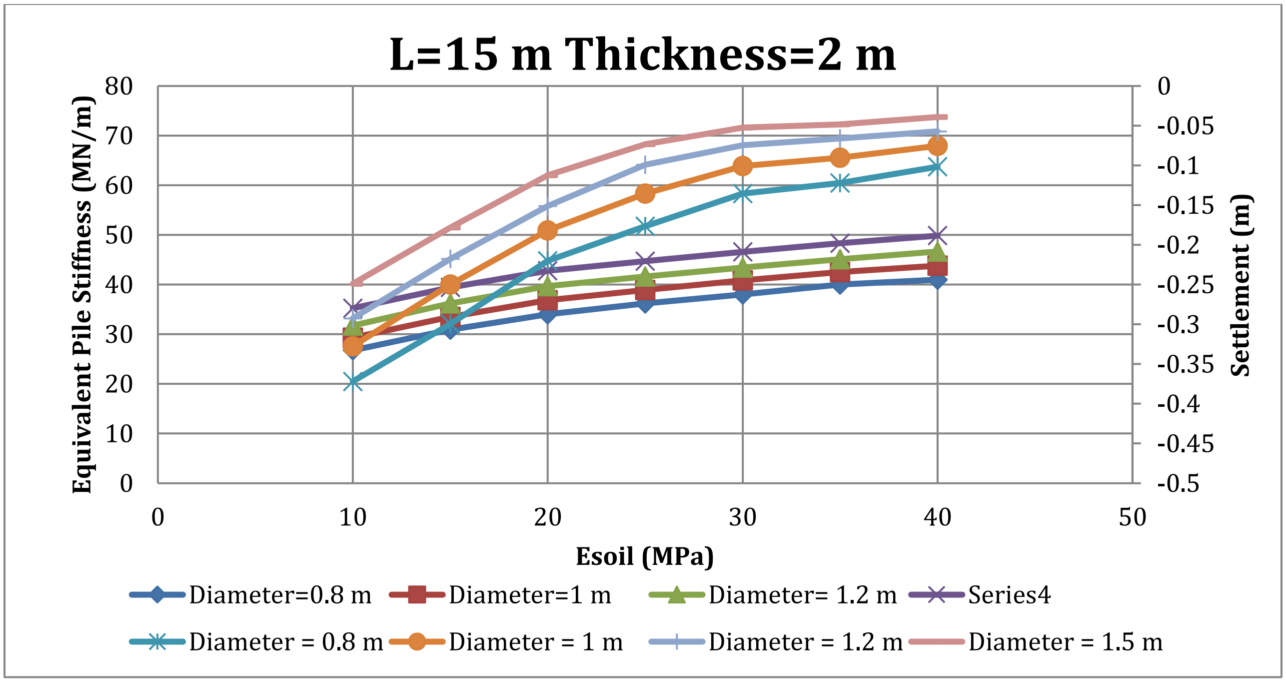


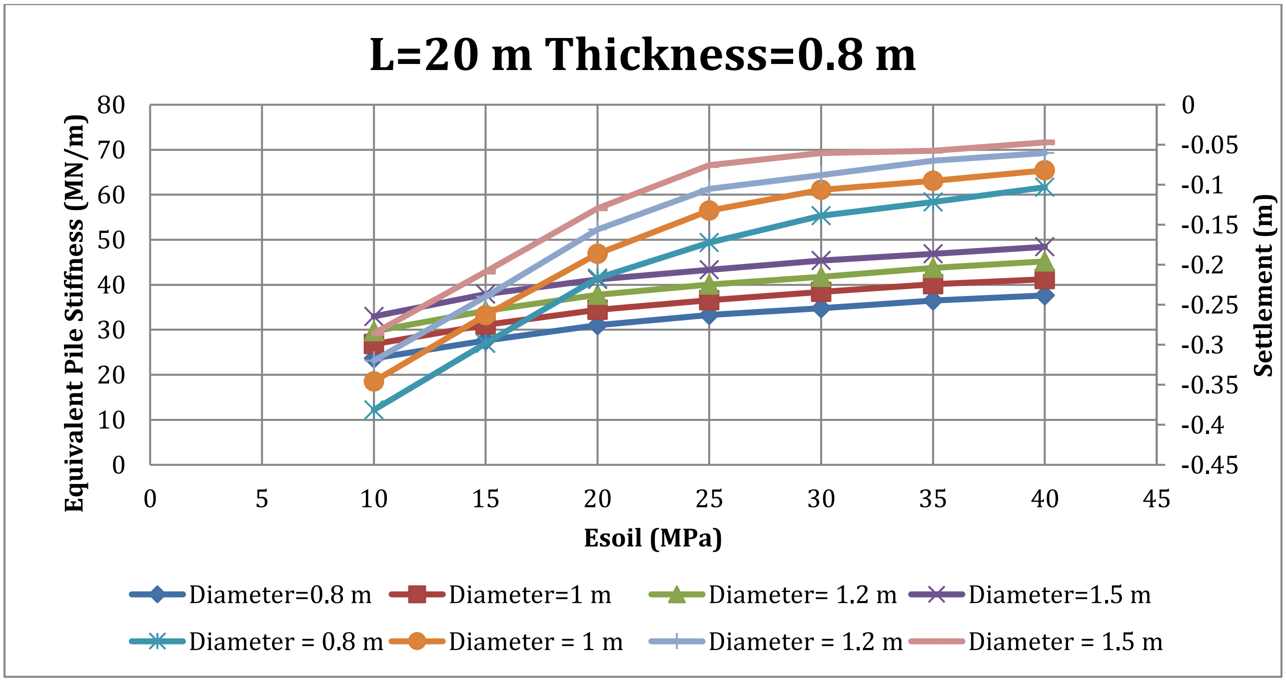

L=10 m Thickness=2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter= 1.2 m
Diameter= 1.5 m


L=15 m Thickness=0.8 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
0
10
20
30
40
50
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter = 1.2 m
Diameter= 1.5 m


L=15 m Thickness=1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter= 1.2 m
Diameter= 1.5 m


L=15 m Thickness=1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m


L=15 m Thickness=2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Series4
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m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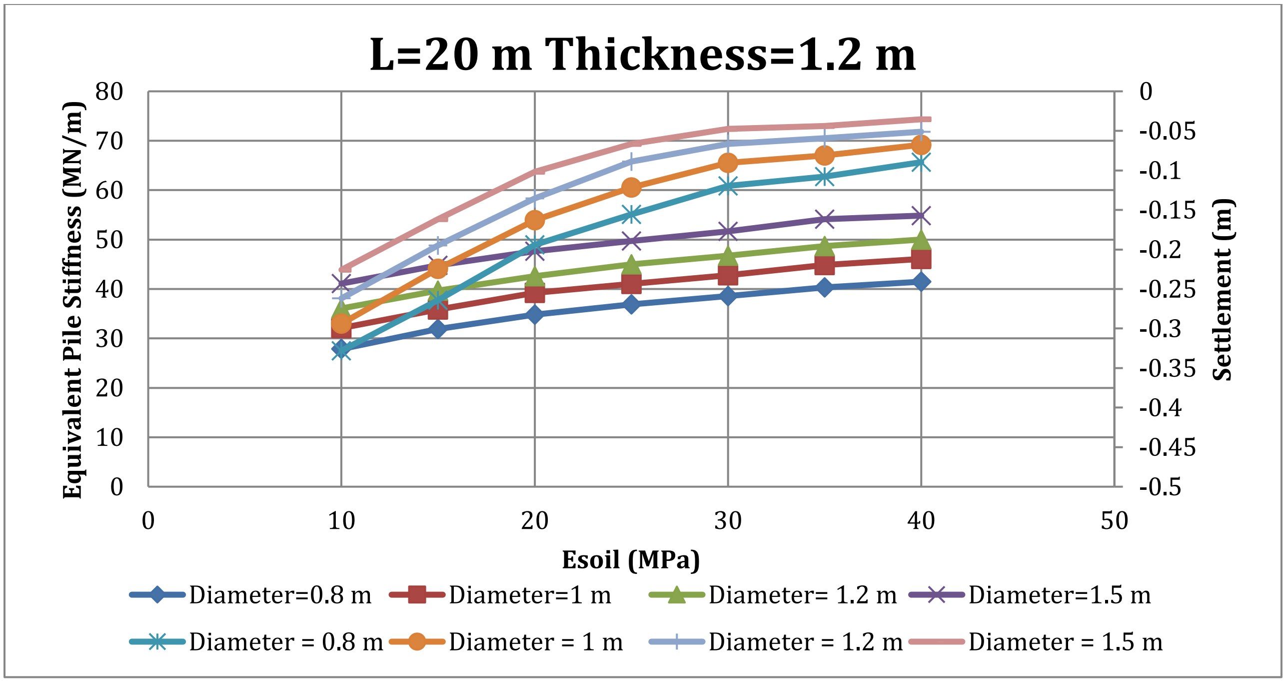
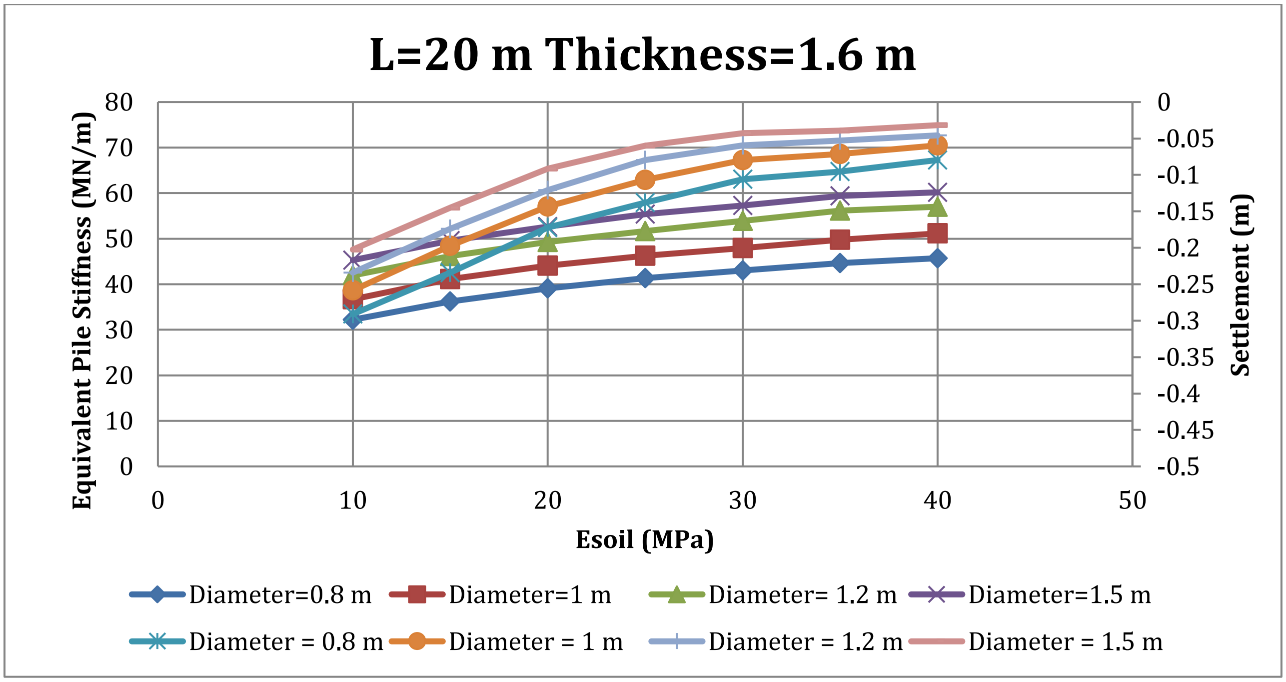
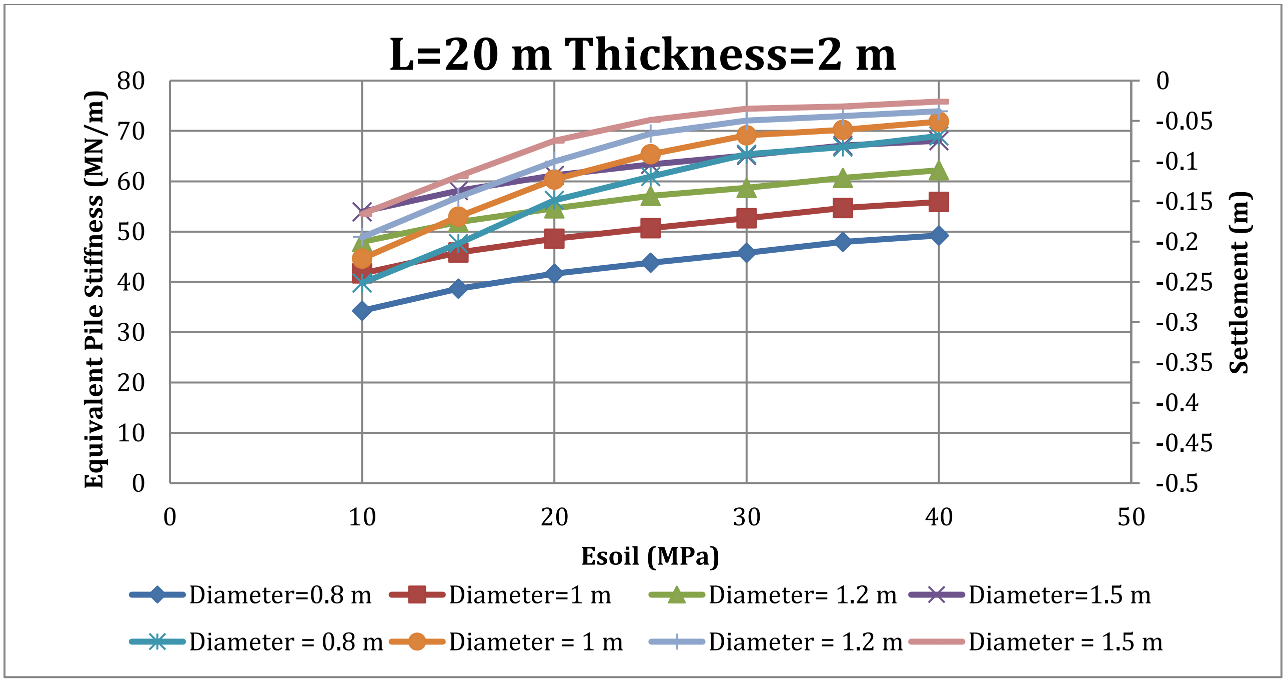
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
0
5
10
15
20
25
30
35
40
45
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m


L=20 m Thickness=1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m


L=20 m Thickness=1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m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Equivalent Pile Stiffness (MN/m)
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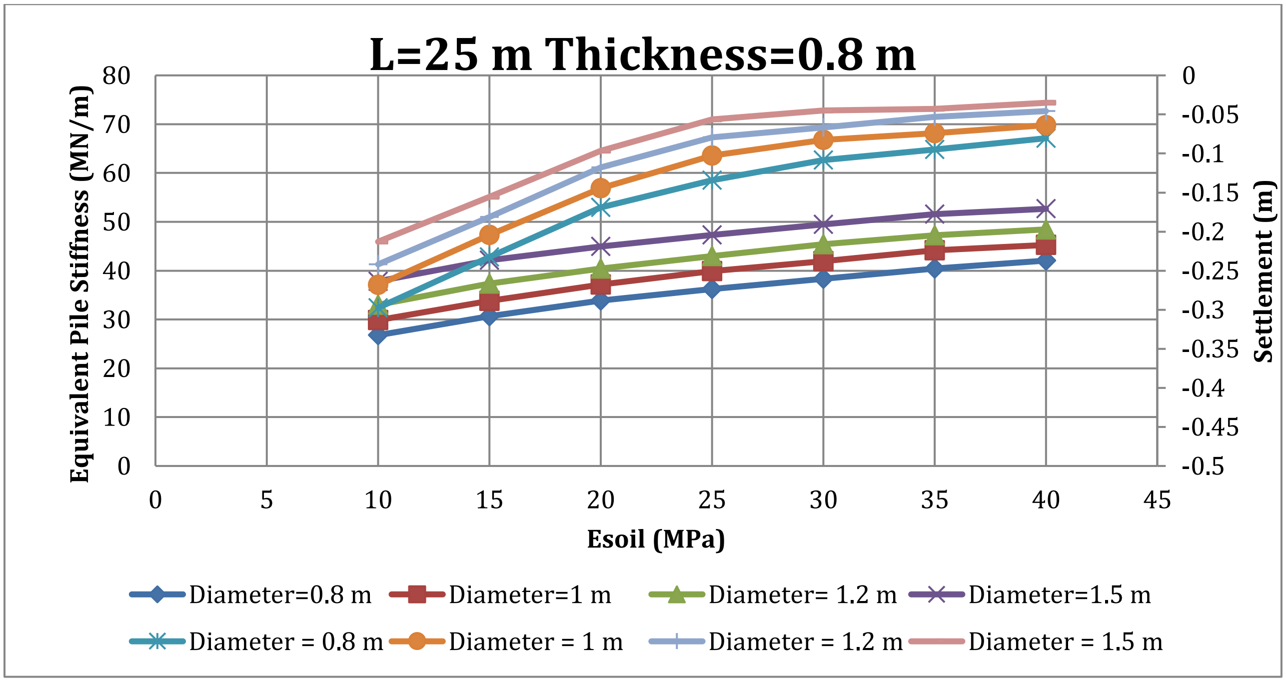
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m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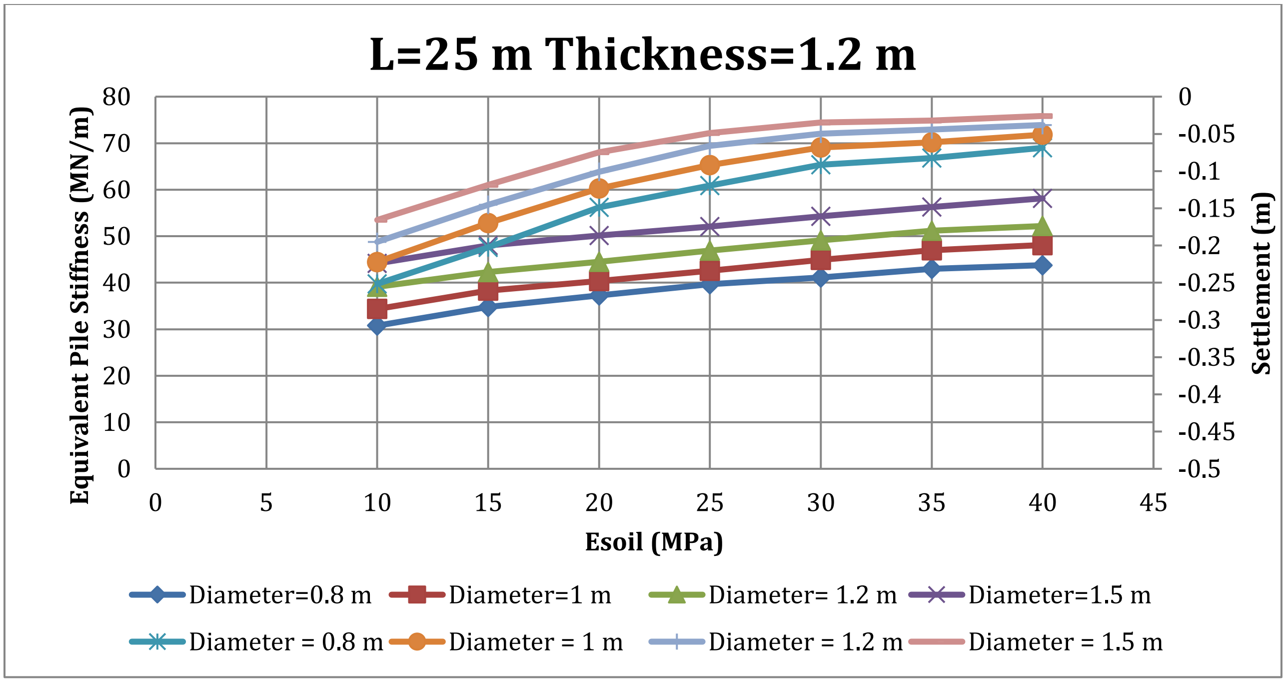
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m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Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
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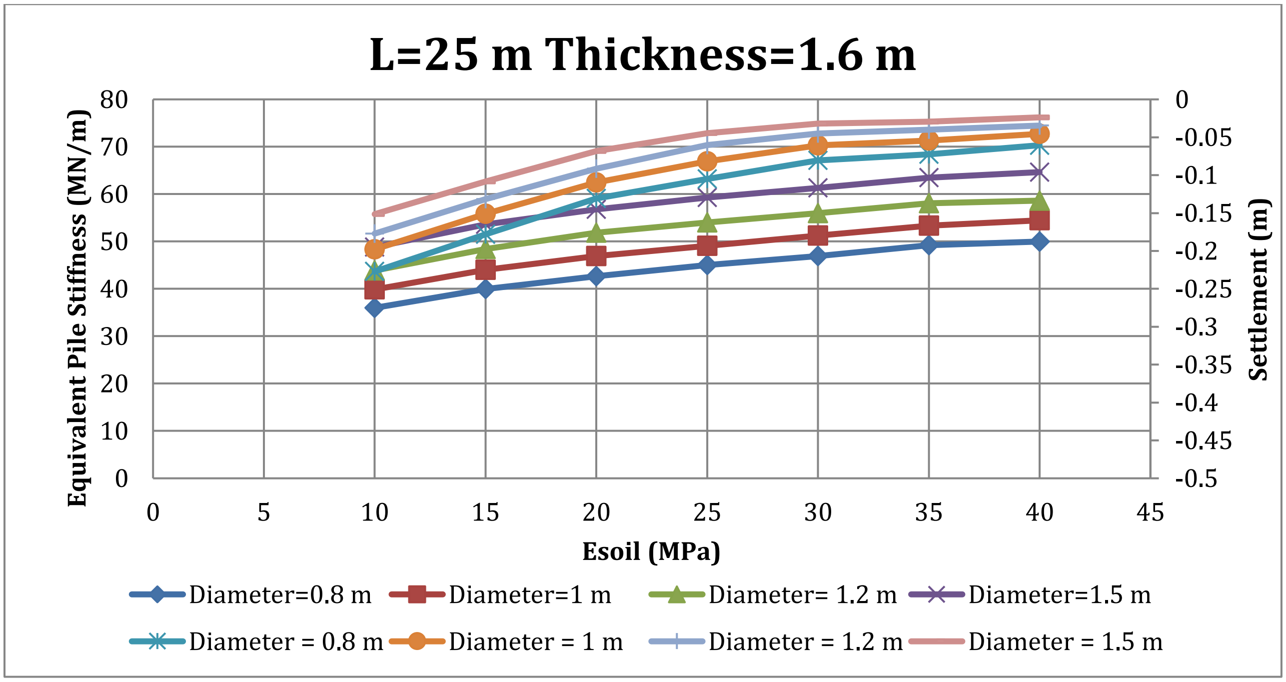
10
20
30
40
50
60
70
80
0
5
10
15
20
25
30
35
40
45
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m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Settlement (m)
Esoil (MPa)
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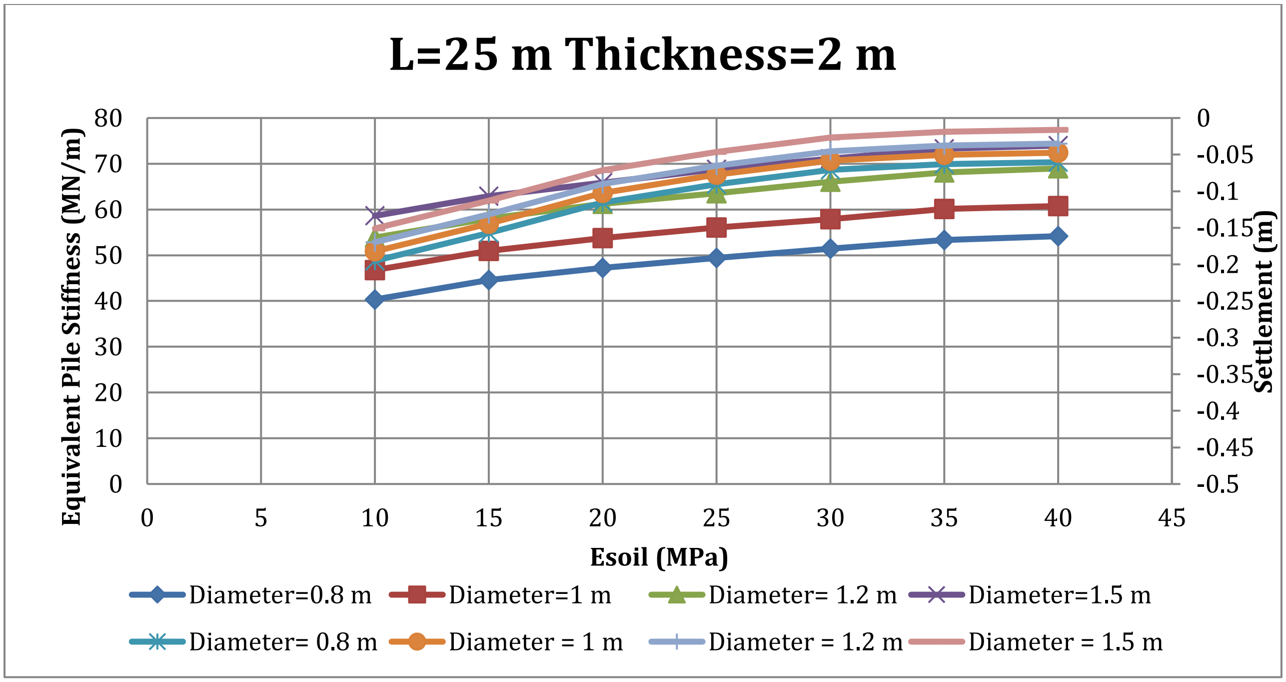
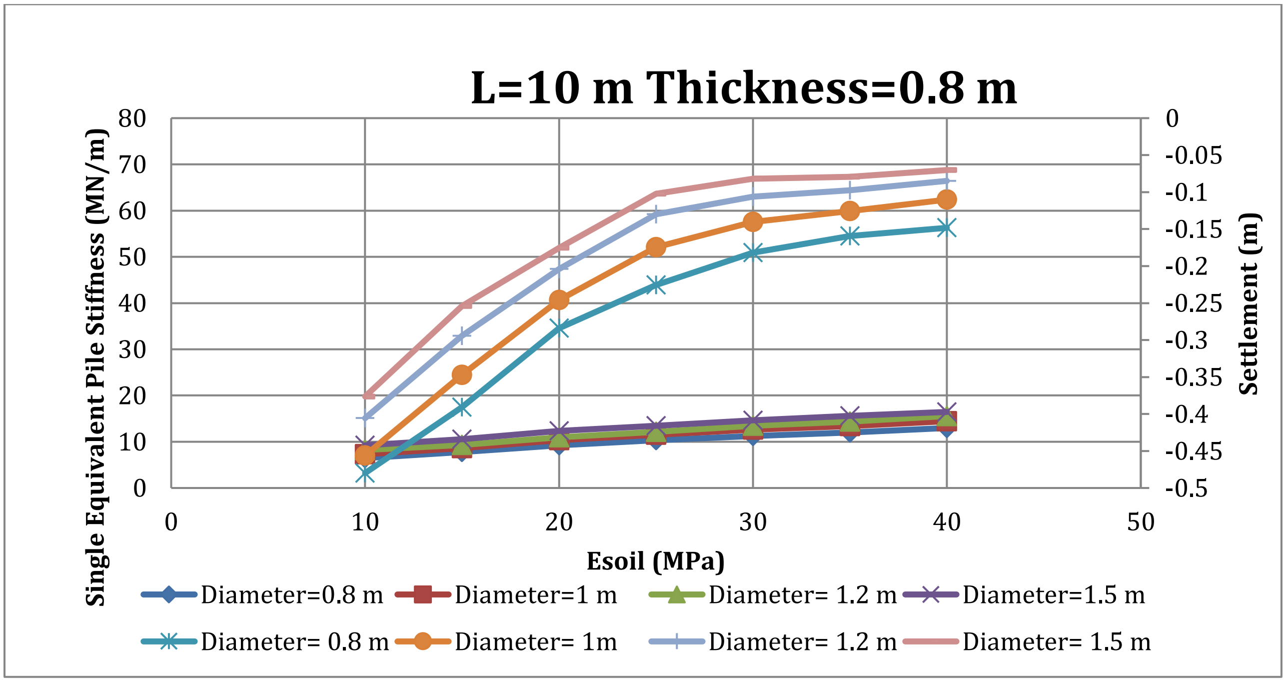
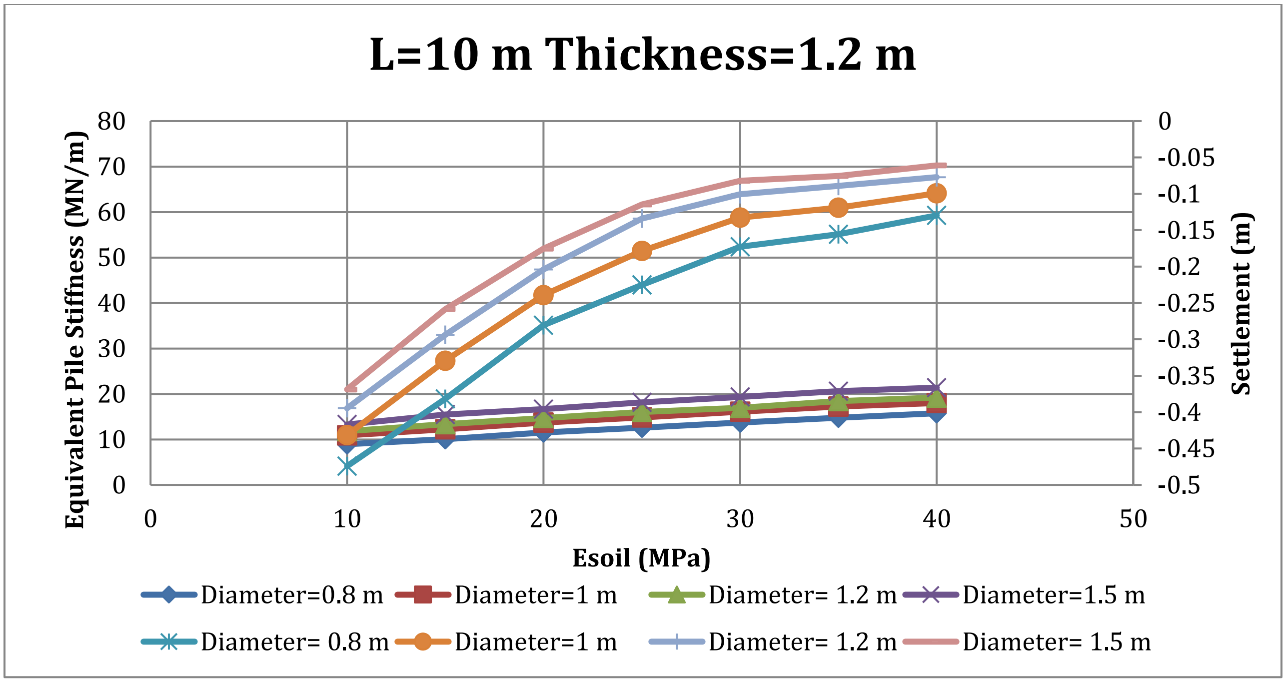
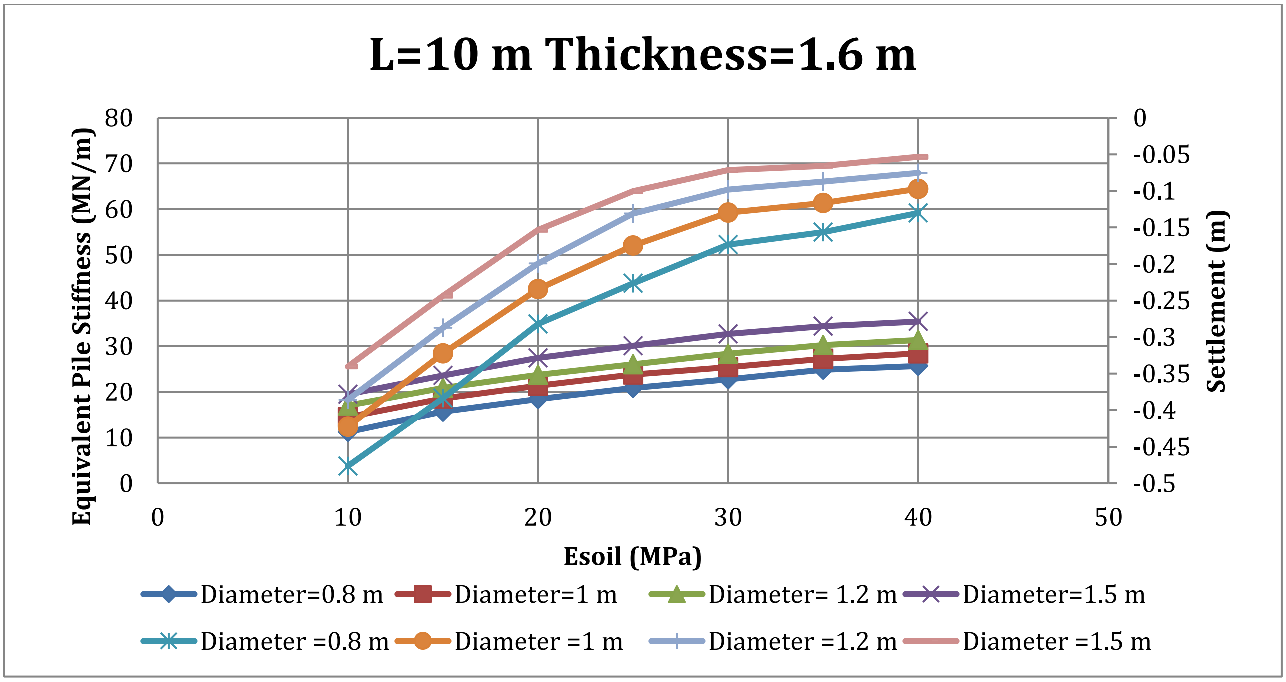
10
20
30
40
50
60
70
80
0
5
10
15
20
25
30
35
40
45
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m

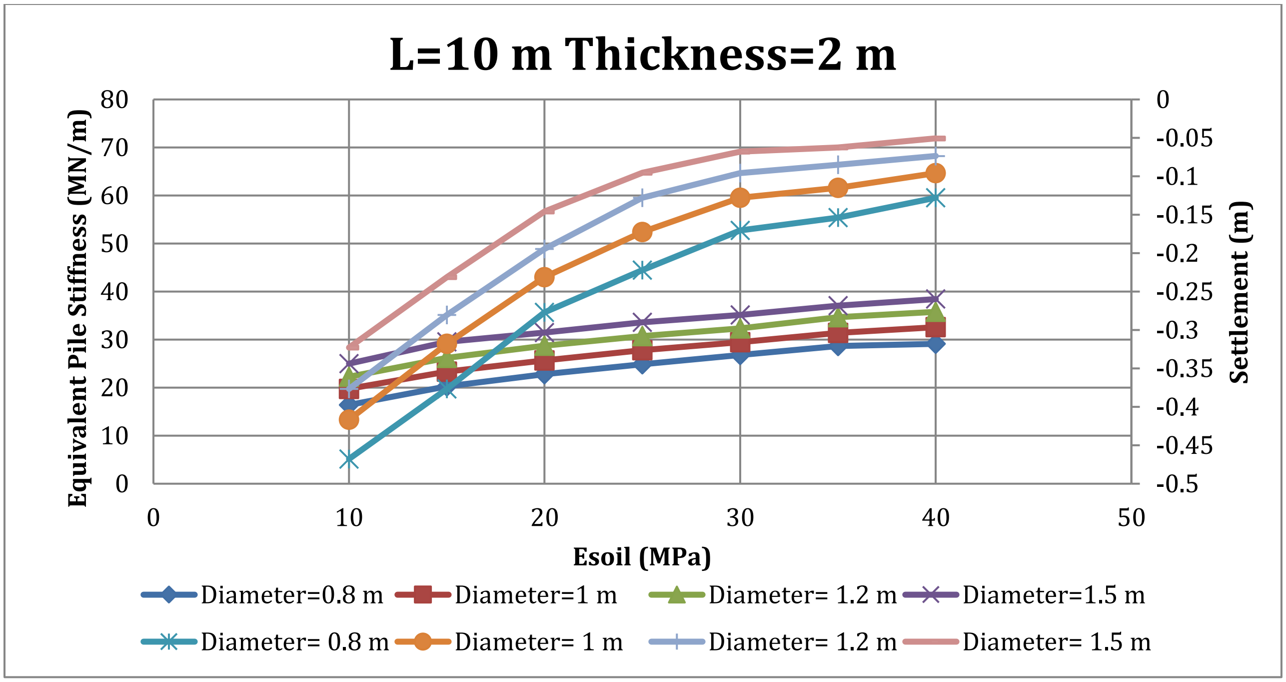


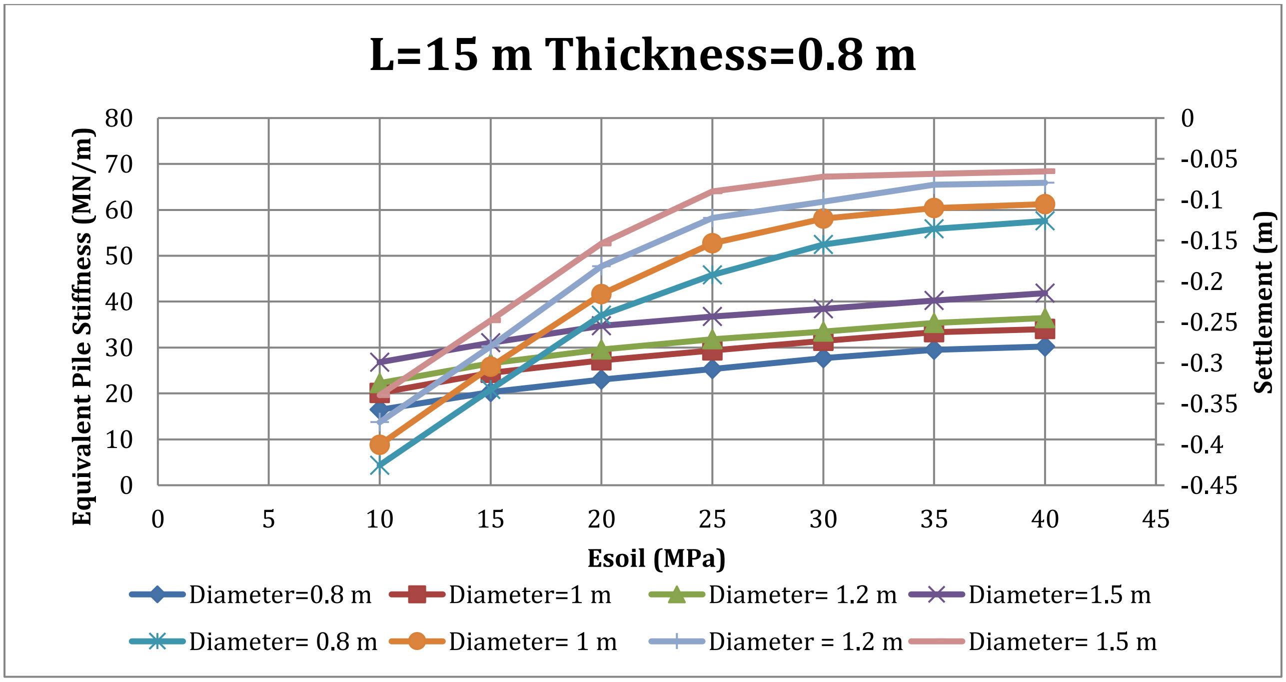
Drained Clays:

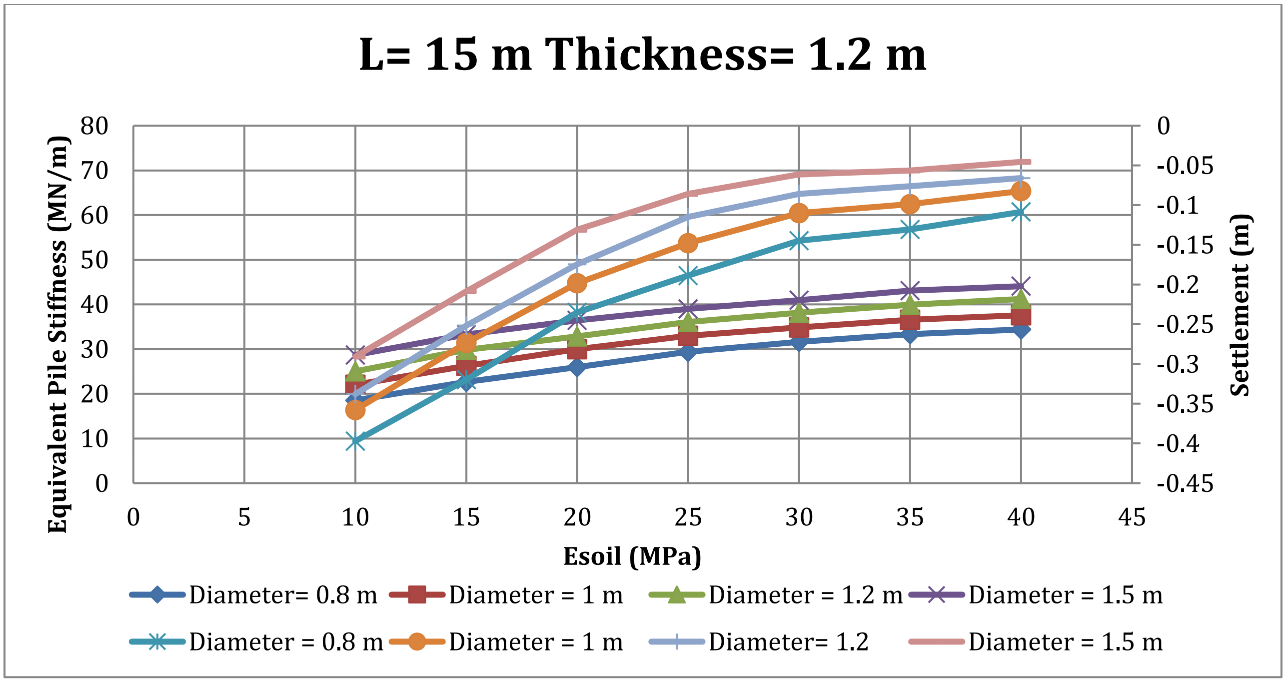


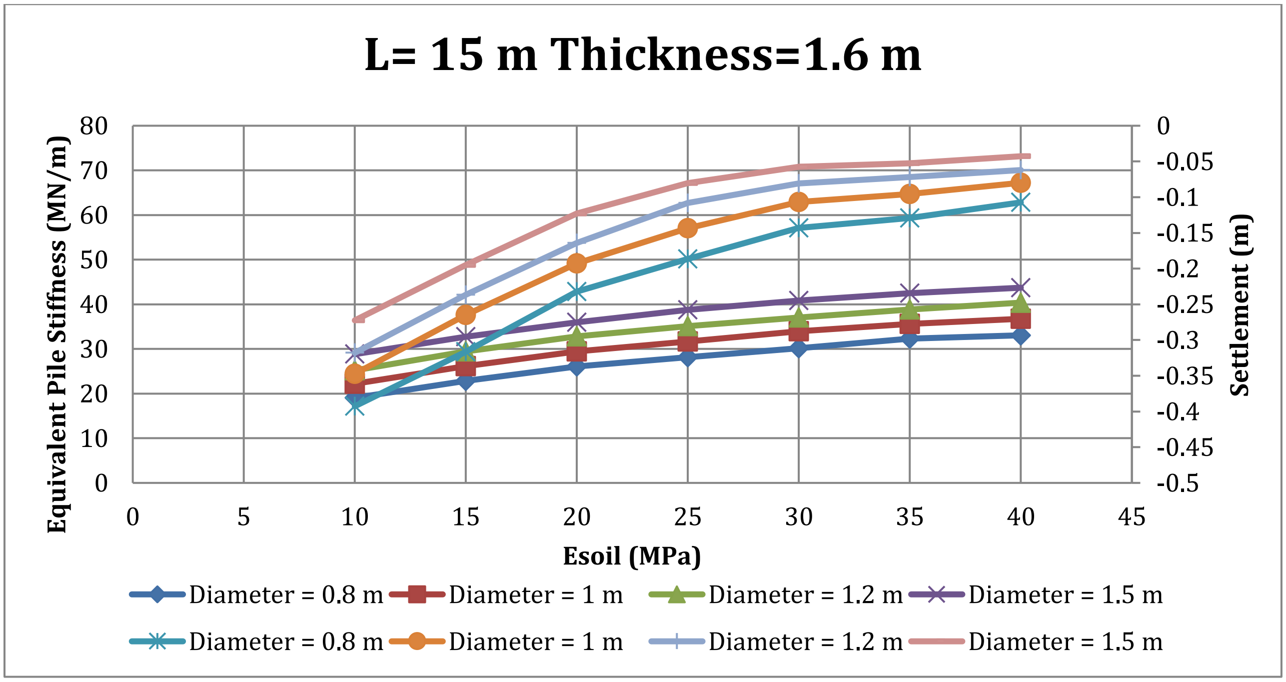

L=10 m Thickness=1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter= 1.5 m

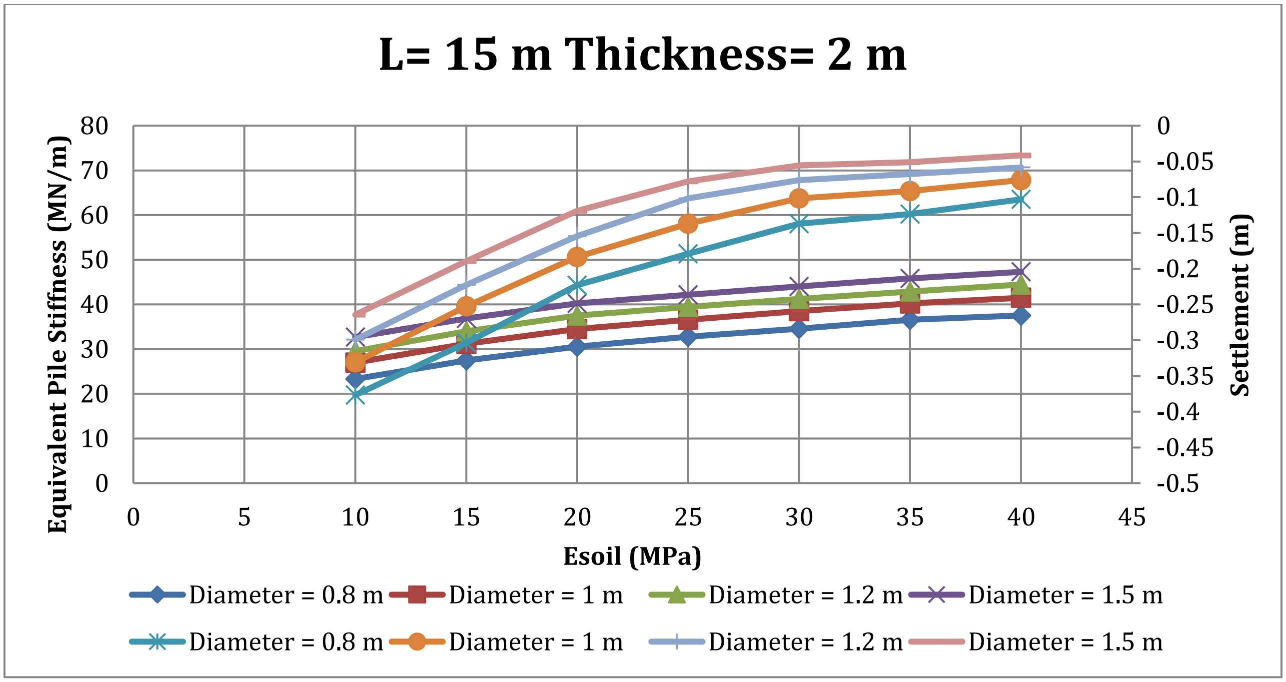

L=10 m Thickness=1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter =0.8 m
Diameter =1 m
Diameter =1.2 m
Diameter =1.5 m

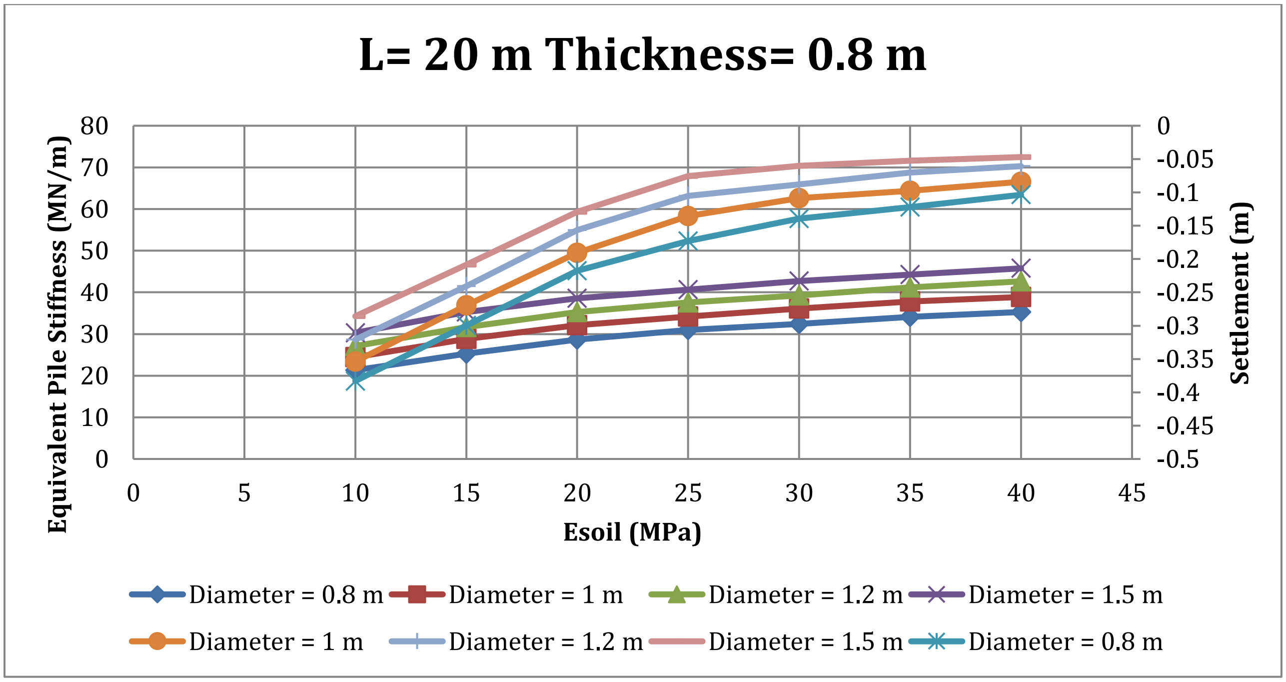

L=10 m Thickness=2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter= 1.2 m
Diameter= 1.5 m

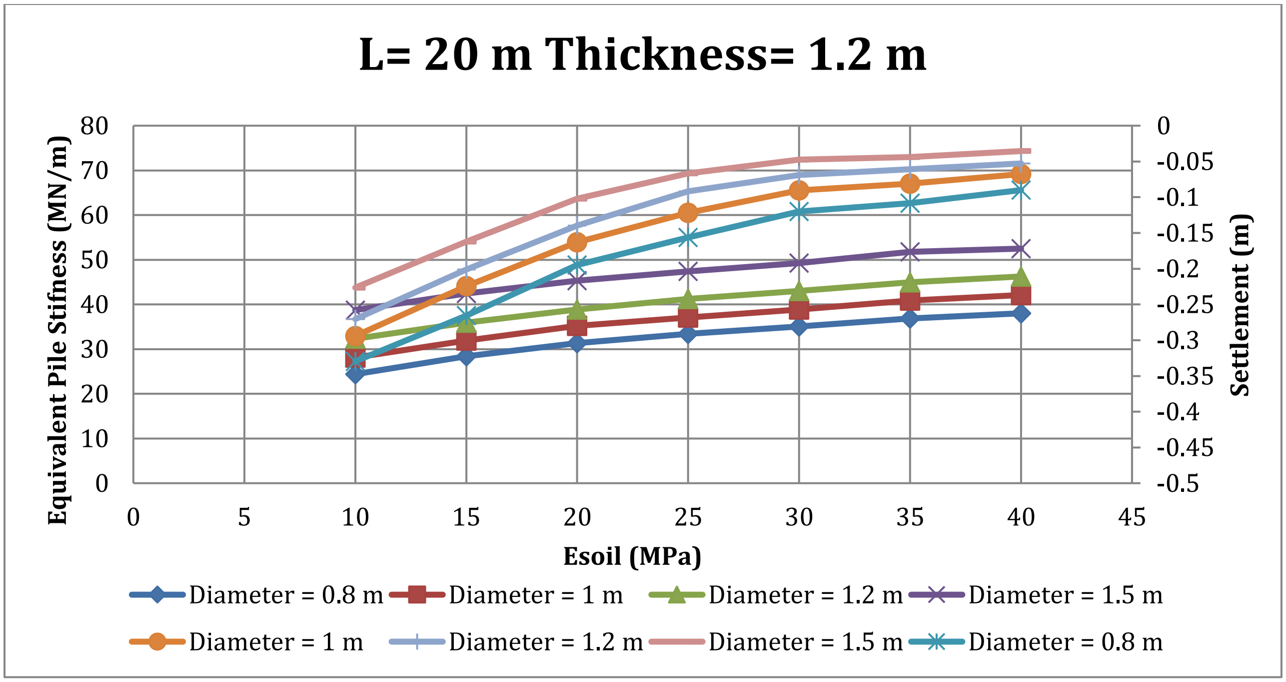

L=15 m Thickness=0.8 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
0
5
10
15
20
25
30
35
40
45
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter = 1.2 m
Diameter= 1.5 m

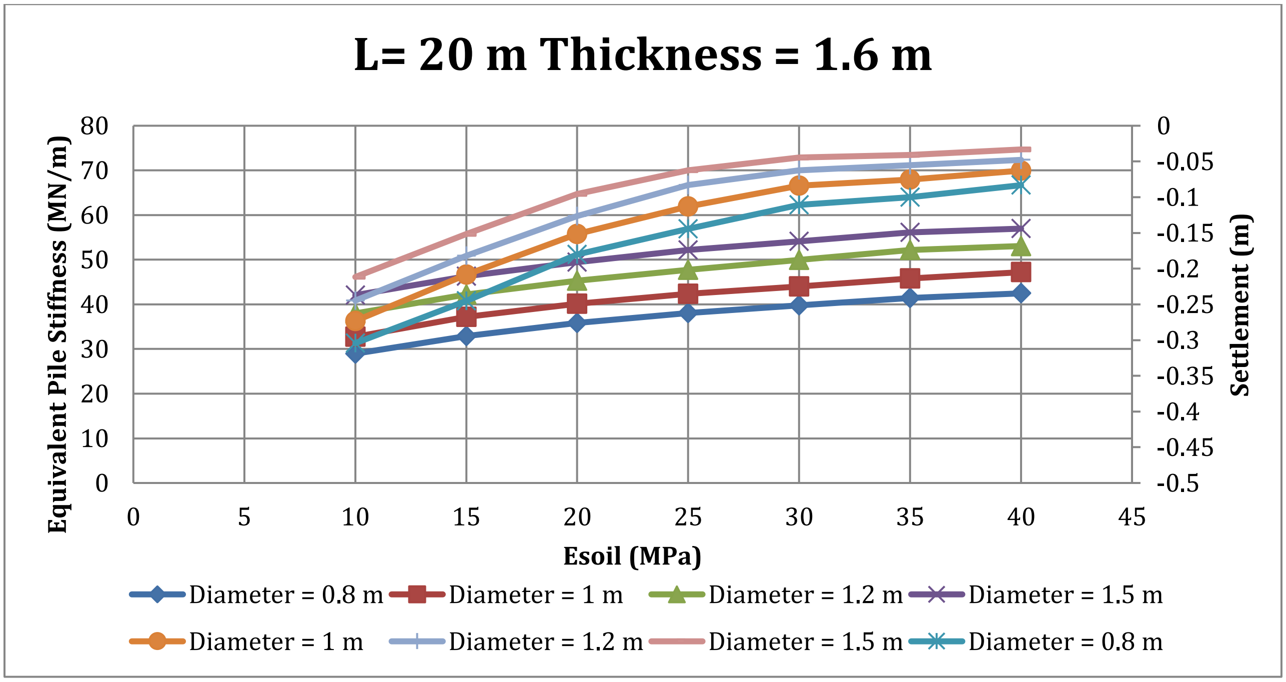

L= 15 m Thickness= 1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
0
5
10
15
20
25
30
35
40
45
Diameter= 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter= 1.2
Diameter = 1.5 m

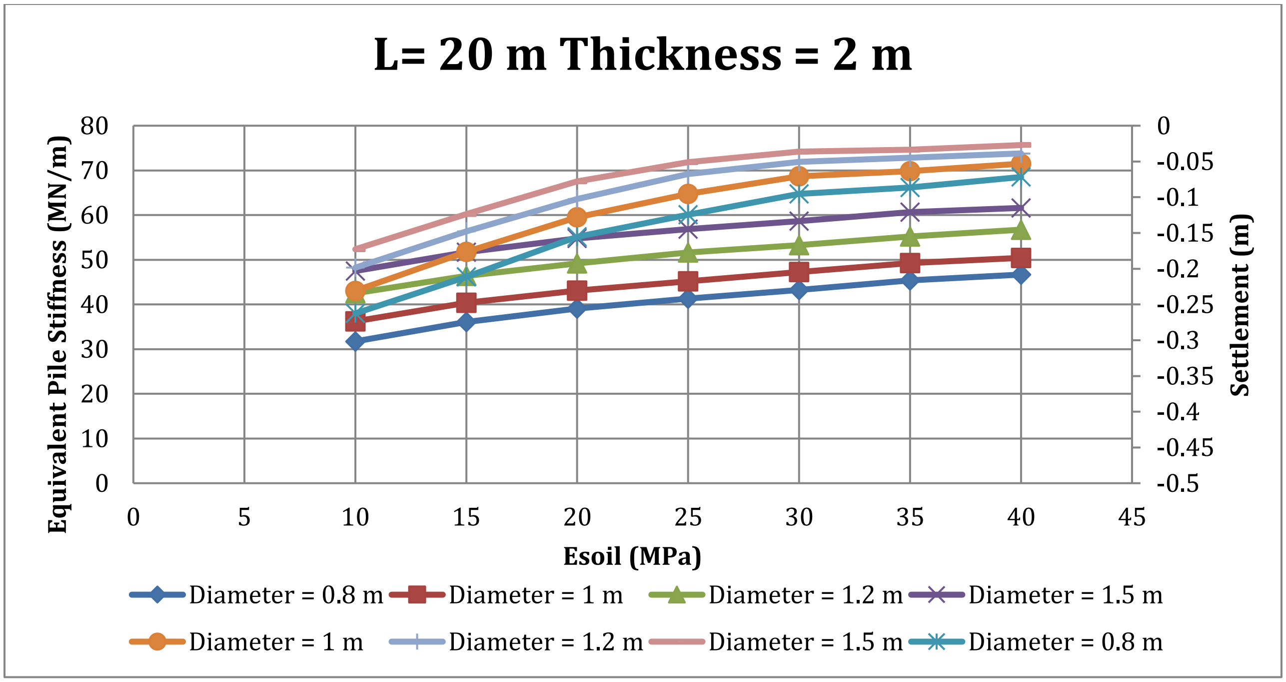

L= 15 m Thickness=1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m

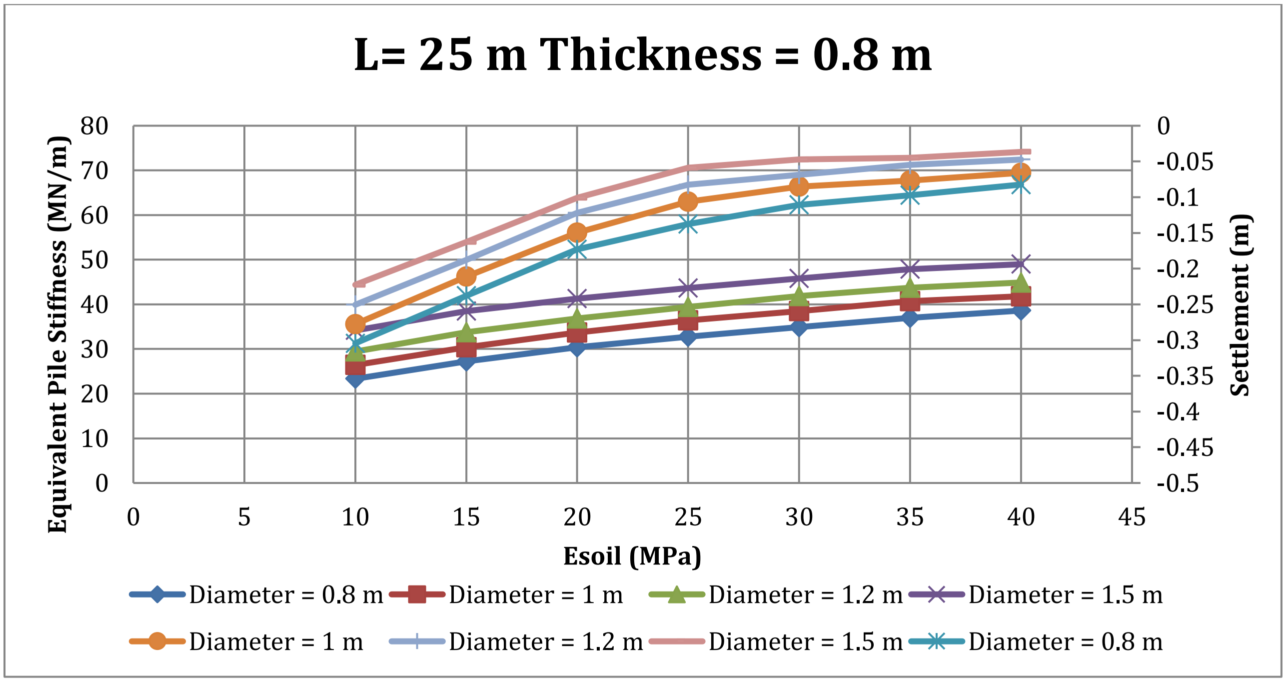

L= 15 m Thickness= 2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m

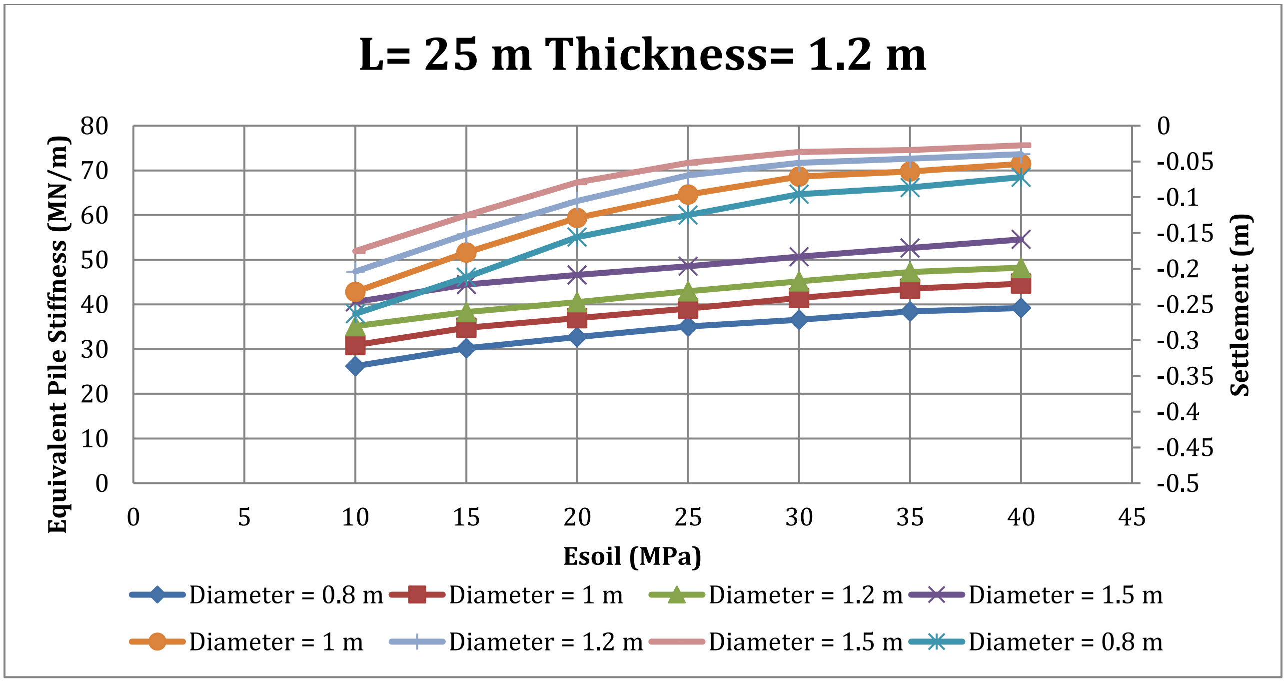

L= 20 m Thickness= 0.8 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m


L= 20 m Thickness= 1.2 m
Equivalent Pile Stifness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

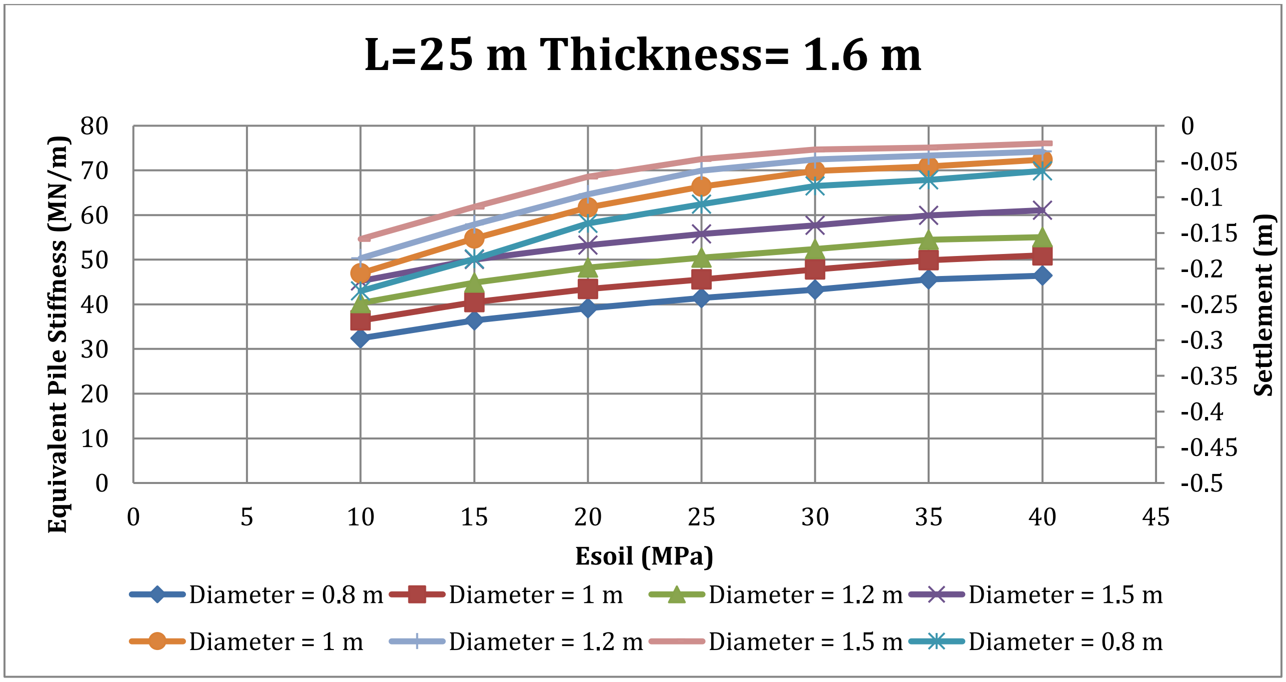

L= 20 m Thickness = 1.6 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m


L= 20 m Thickness = 2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

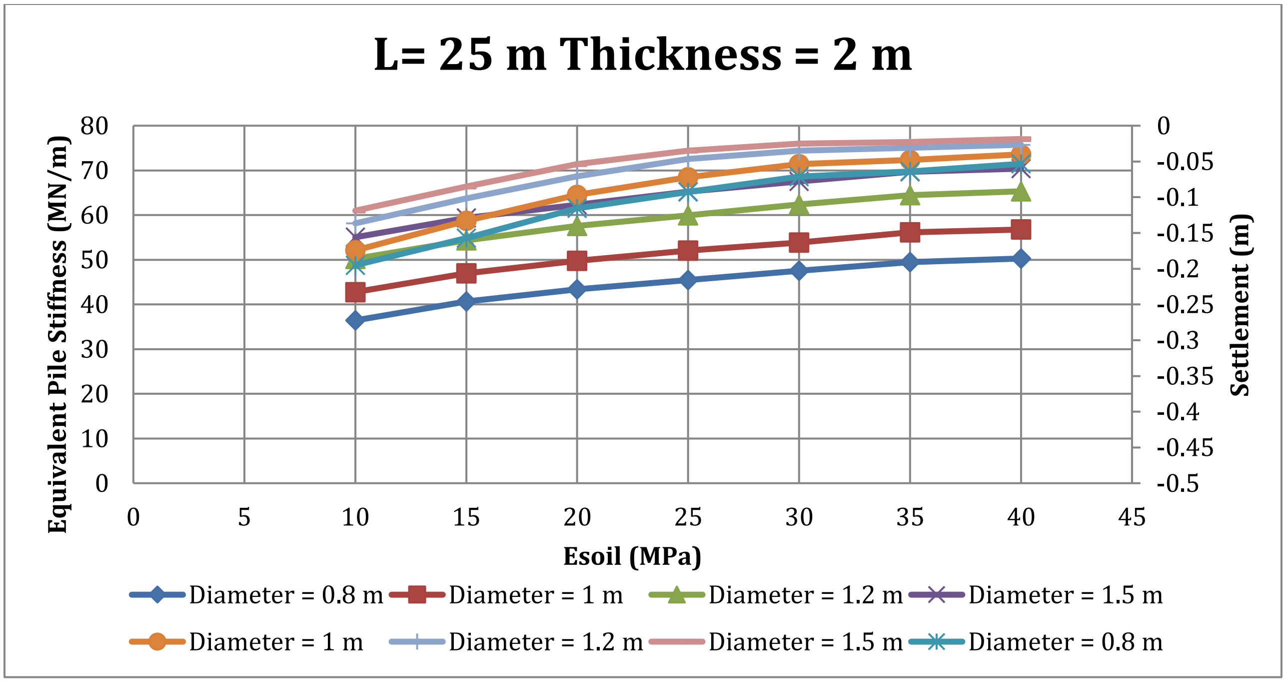

L= 25 m Thickness = 0.8 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m



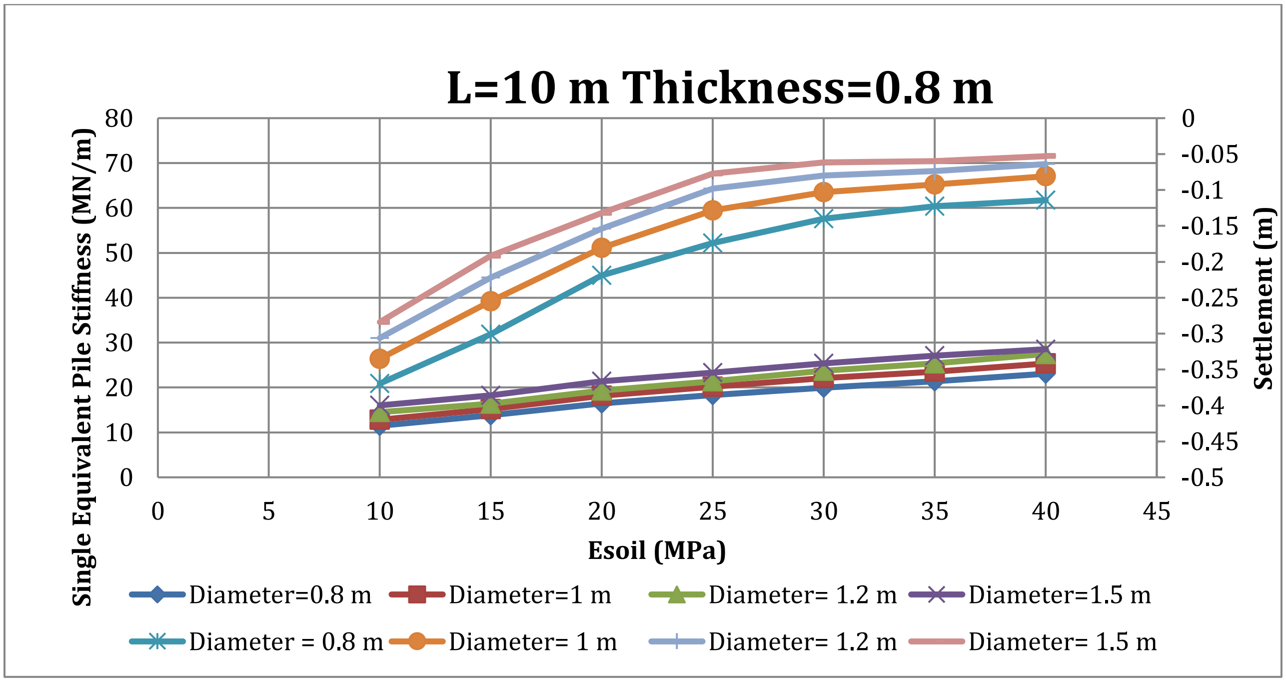
L= 25 m Thickness= 1.2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

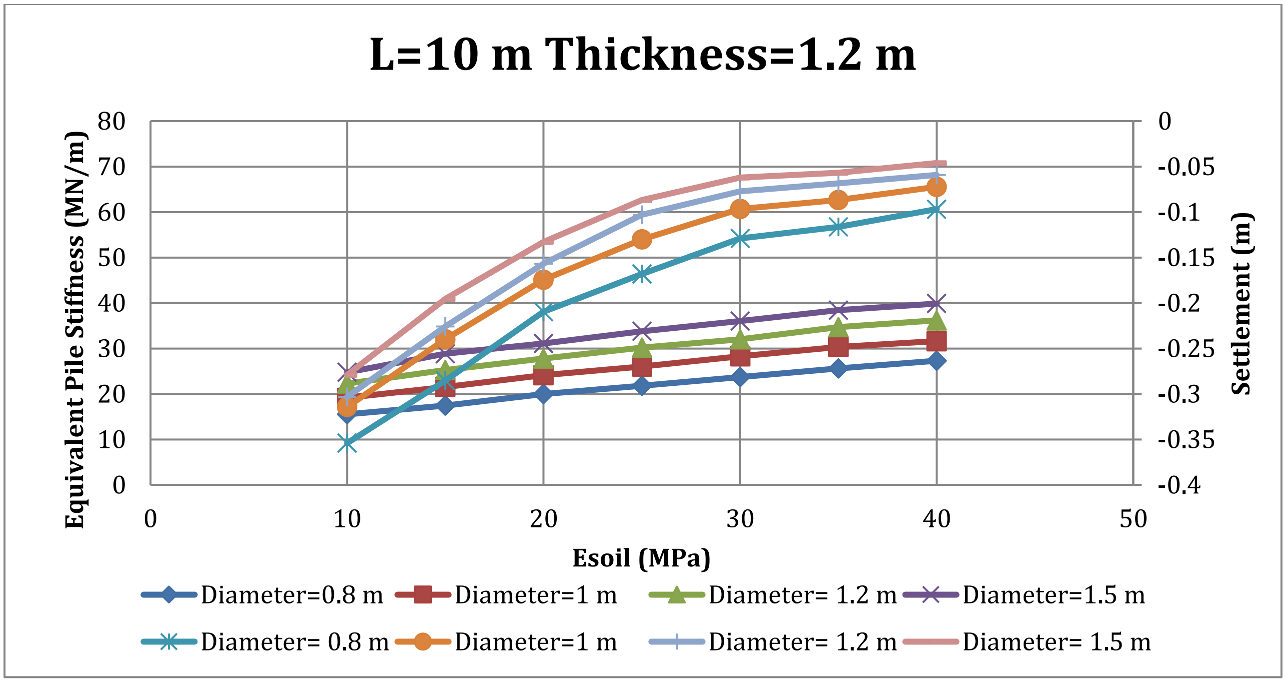


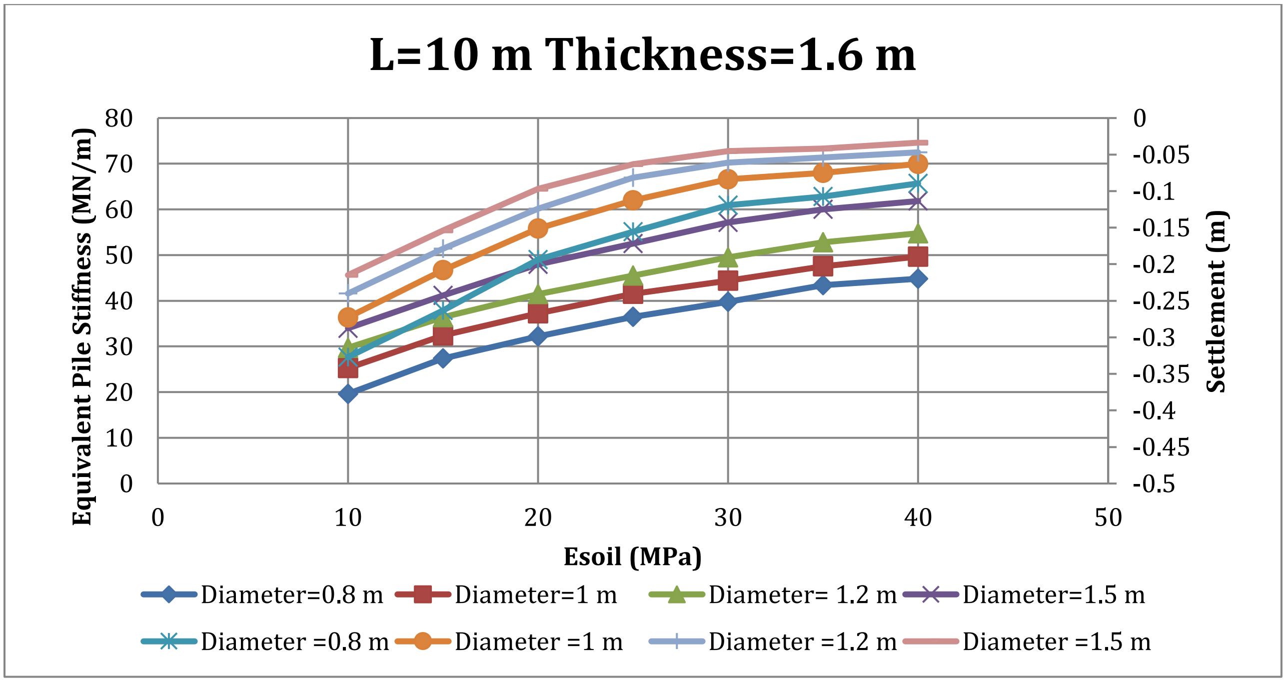
L=25 m Thickness= 1.6 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

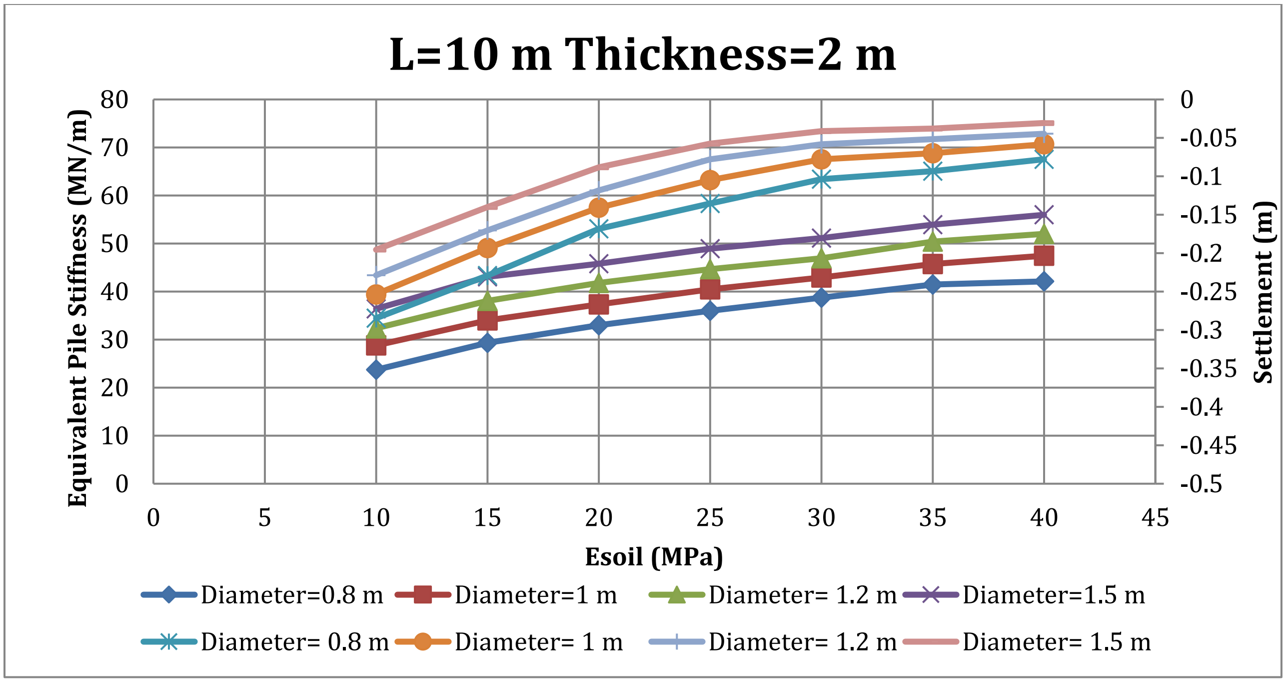

L= 25 m Thickness = 2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

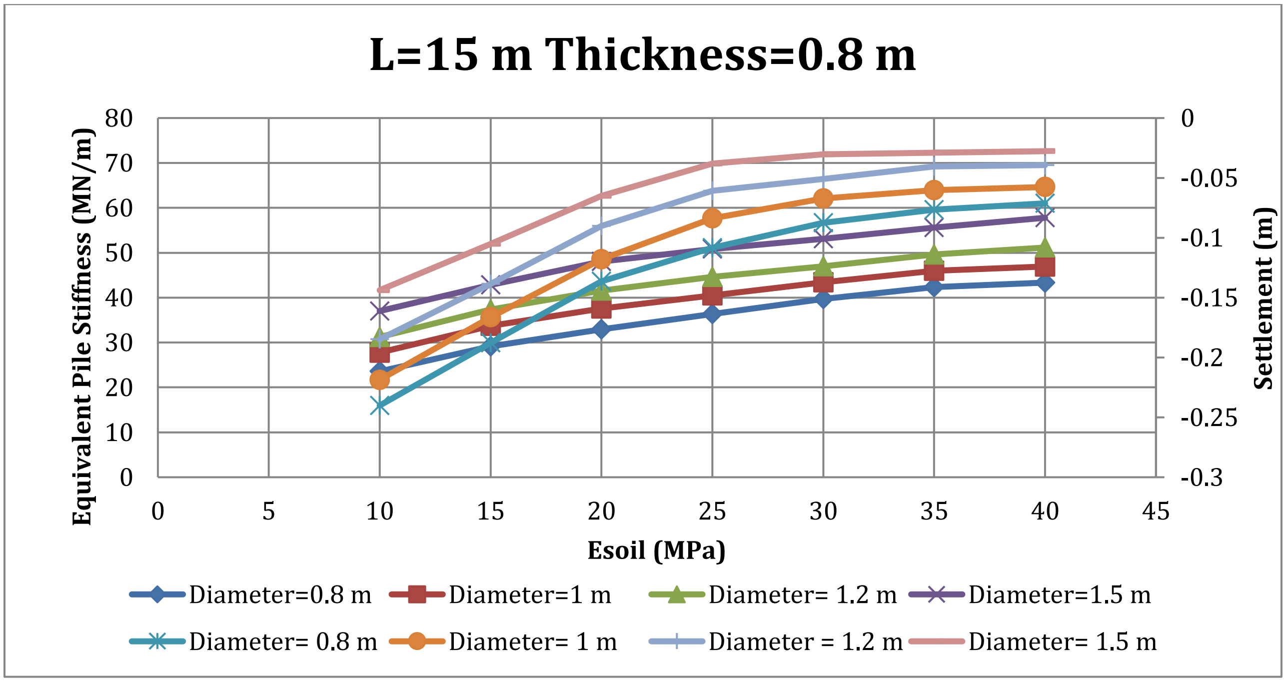
Sands:

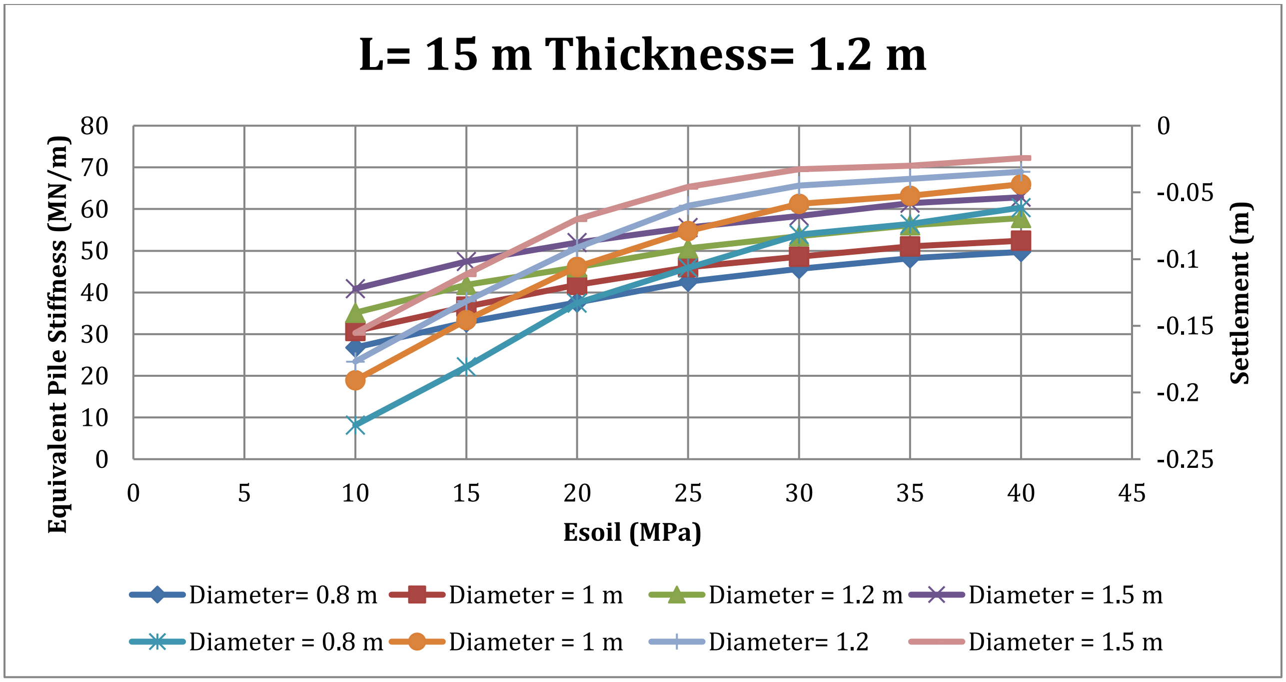

L=10 m Thickness=0.8 m
Single Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter = 0.8 m
Diameter= 1 m
Diameter= 1.2 m
Diameter= 1.5 m

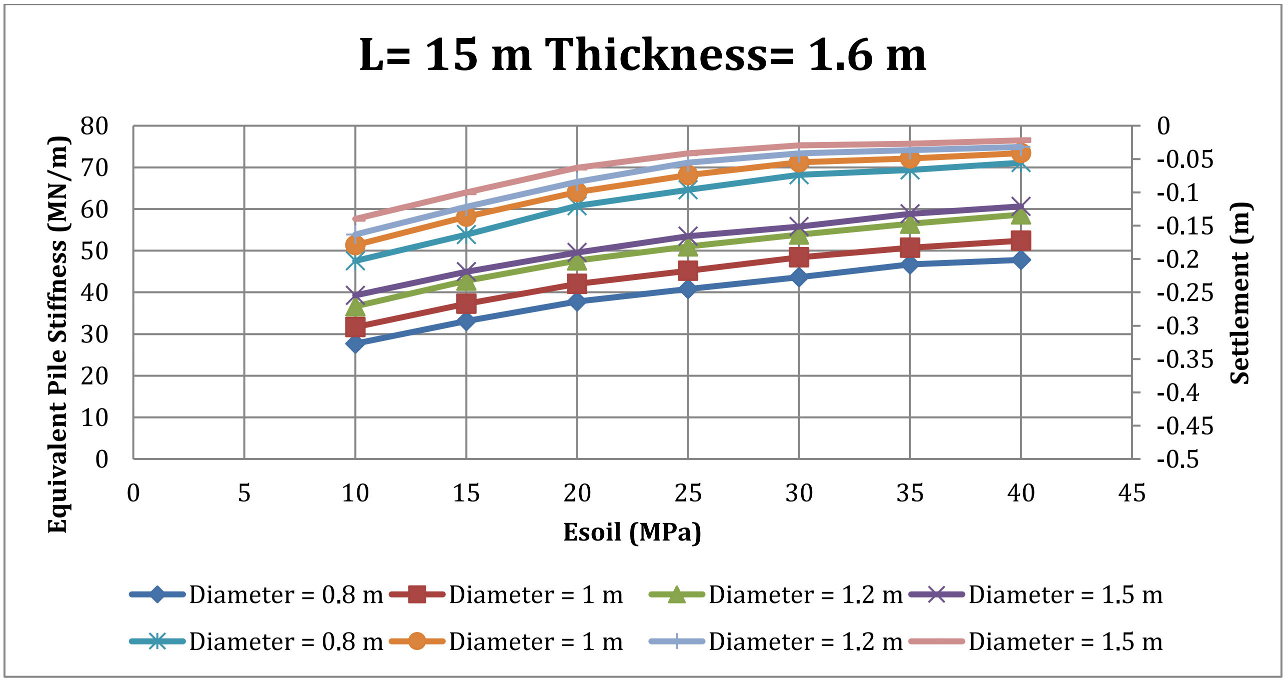

L=10 m Thickness=1.2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
0
10
20
30
40
50
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter= 1.5 m

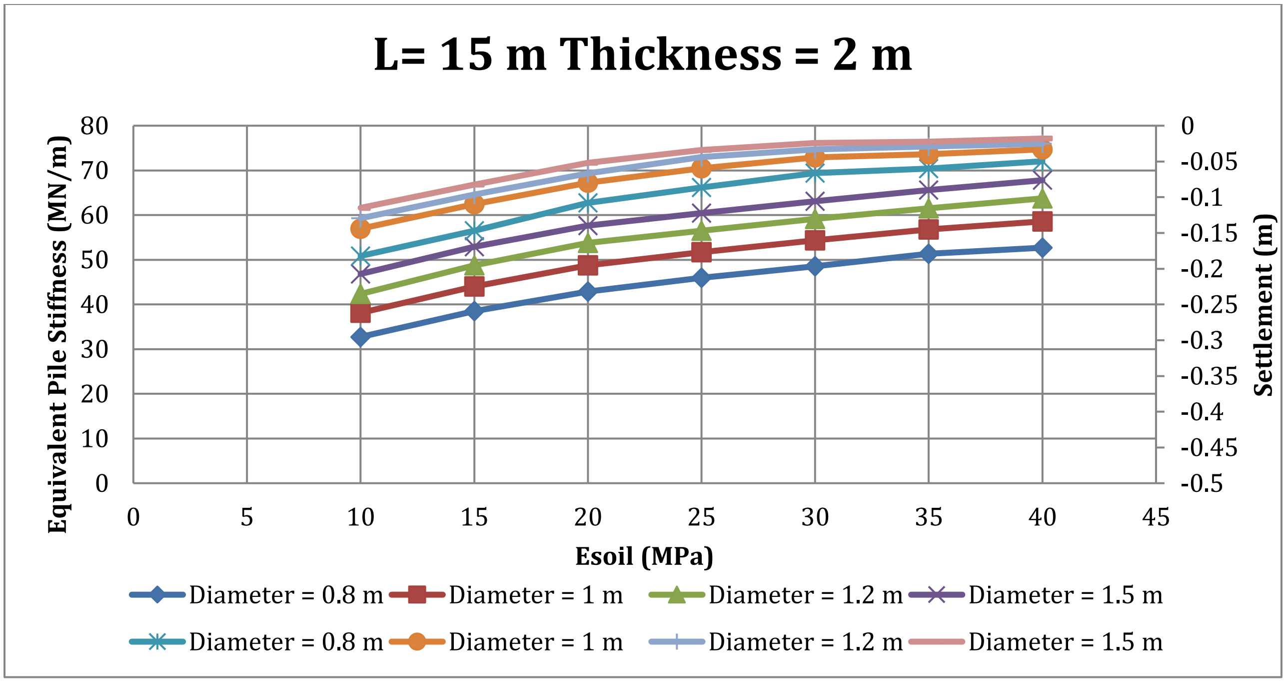

L=10 m Thickness=1.6 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
10
20
30
40
50
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter =0.8 m
Diameter =1 m
Diameter =1.2 m
Diameter =1.5 m

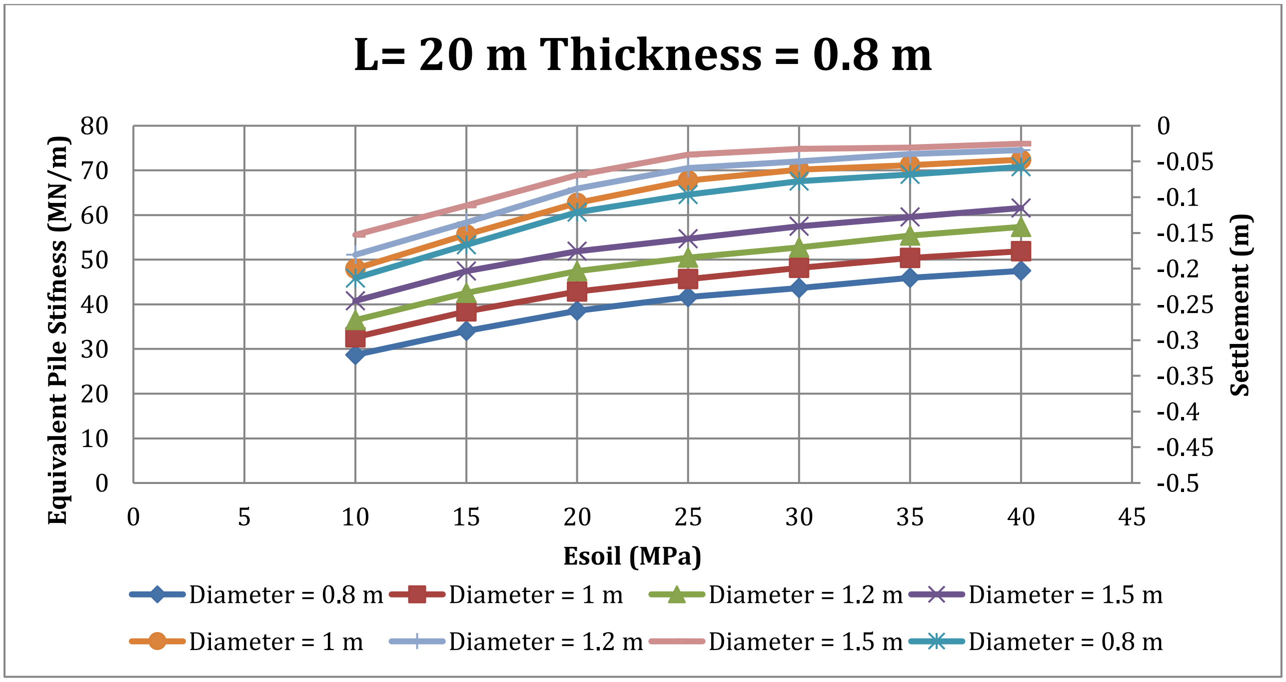

L=10 m Thickness=2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Settlement (m)
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter= 1.2 m
Diameter= 1.5 m

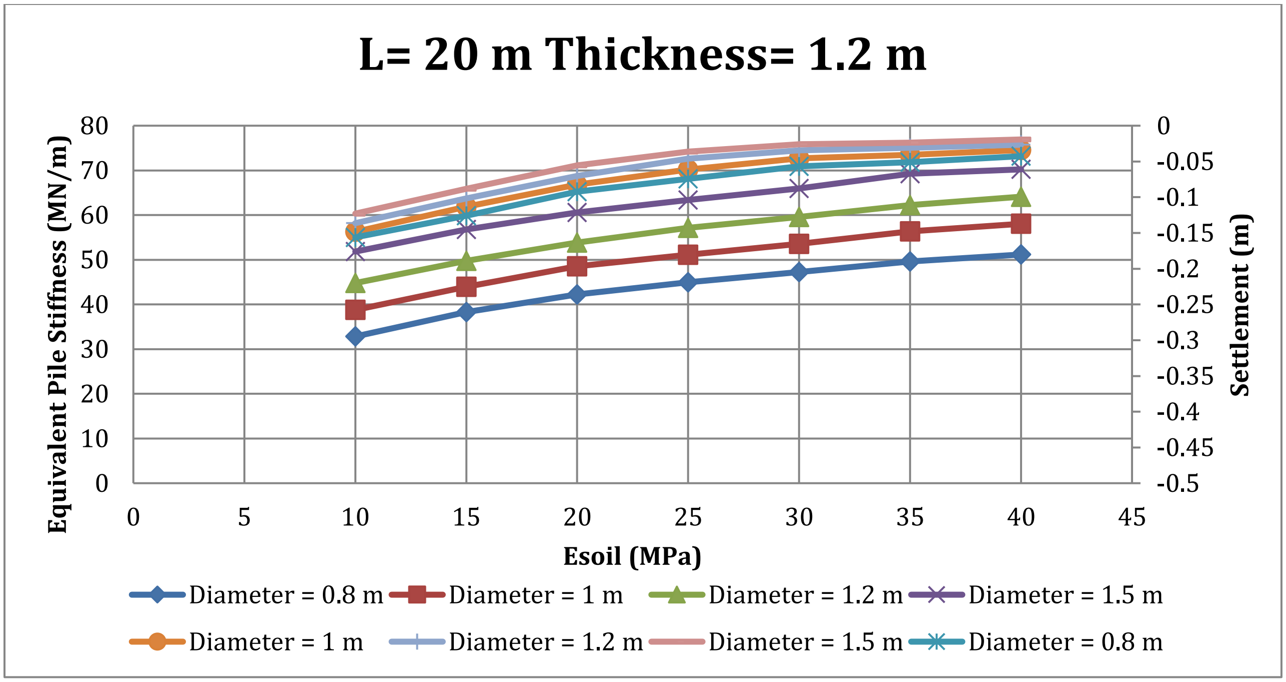

L=15 m Thickness=0.8 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
Settlement (m)
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter=0.8 m
Diameter=1 m
Diameter= 1.2 m
Diameter=1.5 m
Diameter= 0.8 m
Diameter= 1 m
Diameter = 1.2 m
Diameter= 1.5 m

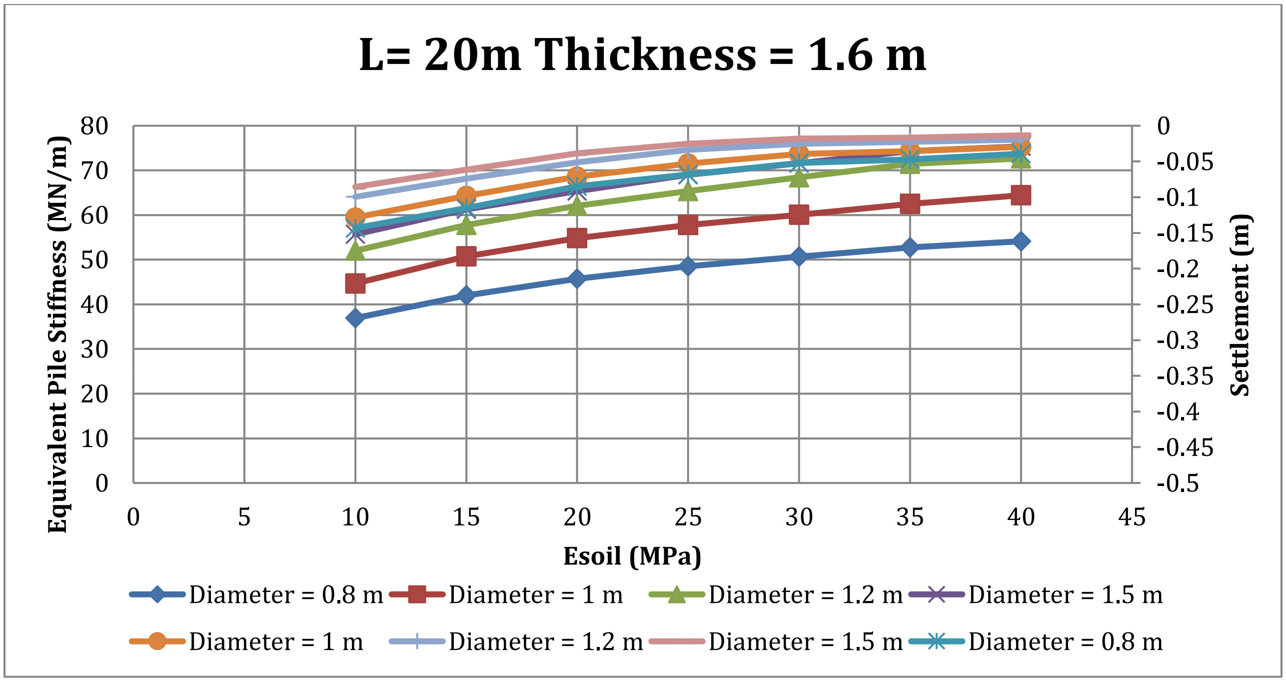

L= 15 m Thickness= 1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
5
10
15
20
25
30
35
40
45
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
Diameter= 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter= 1.2
Diameter = 1.5 m

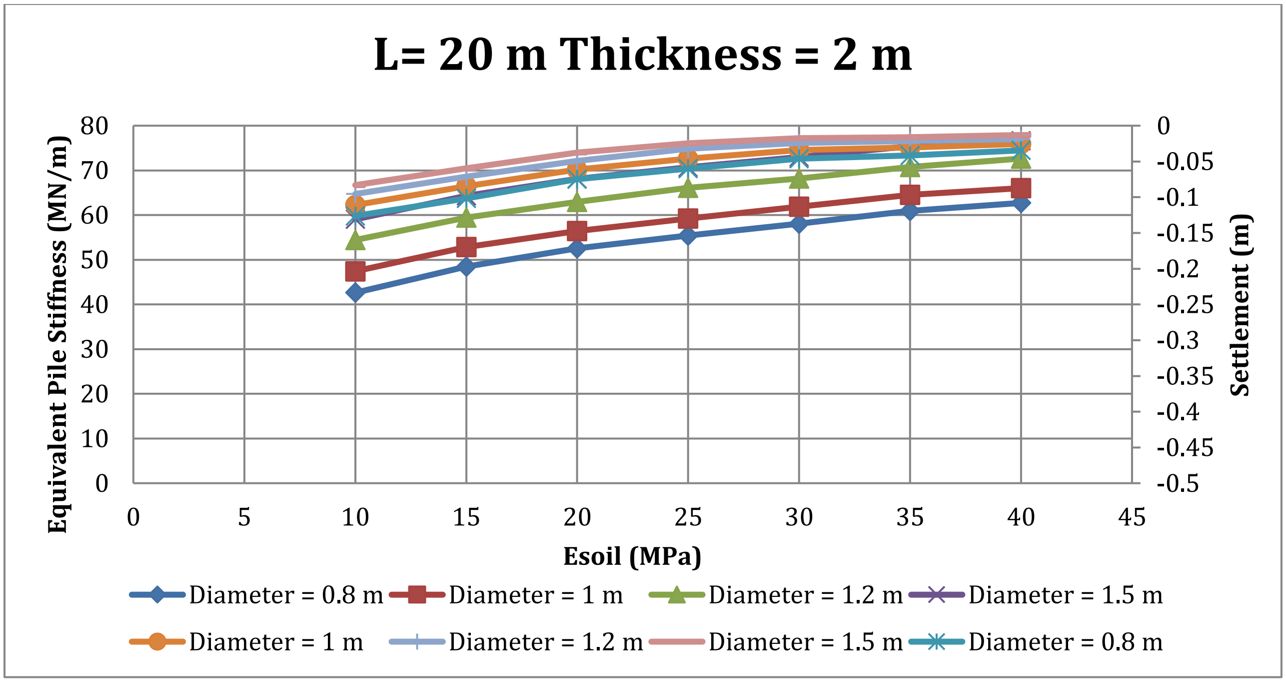

L= 15 m Thickness= 1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
5
10
15
20
25
30
35
40
45
0
10
20
30
40
50
60
70
80
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m

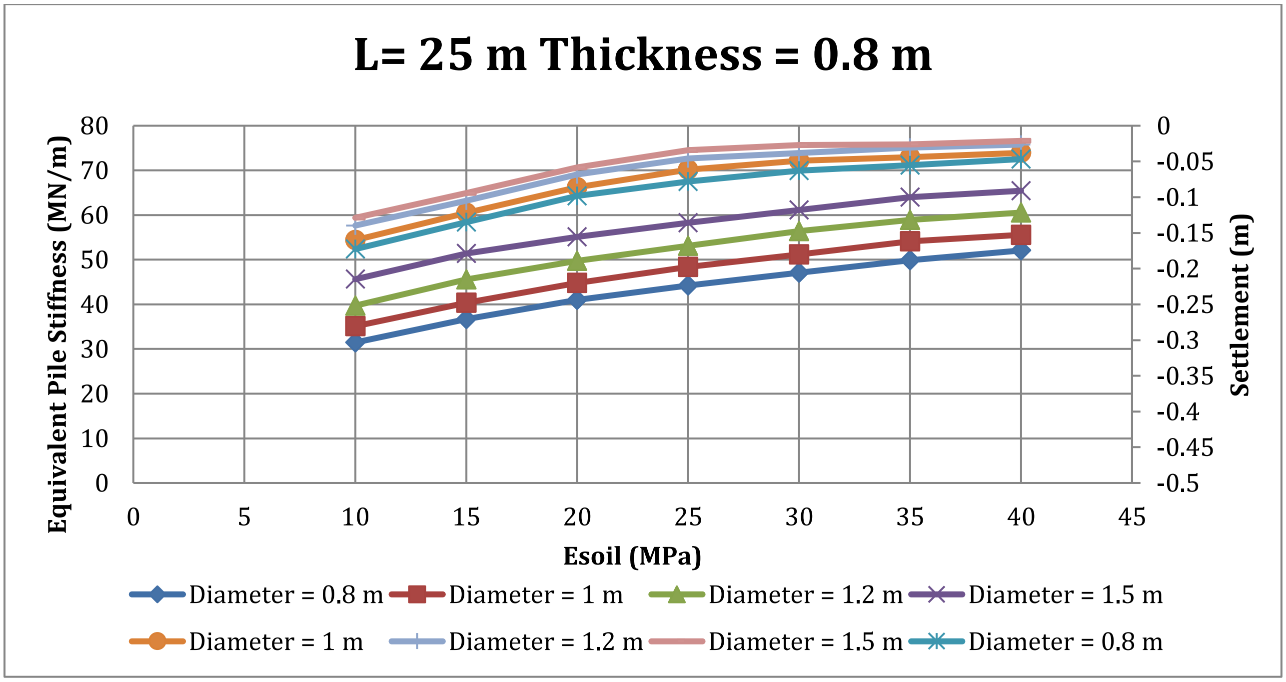

L= 15 m Thickness = 2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m

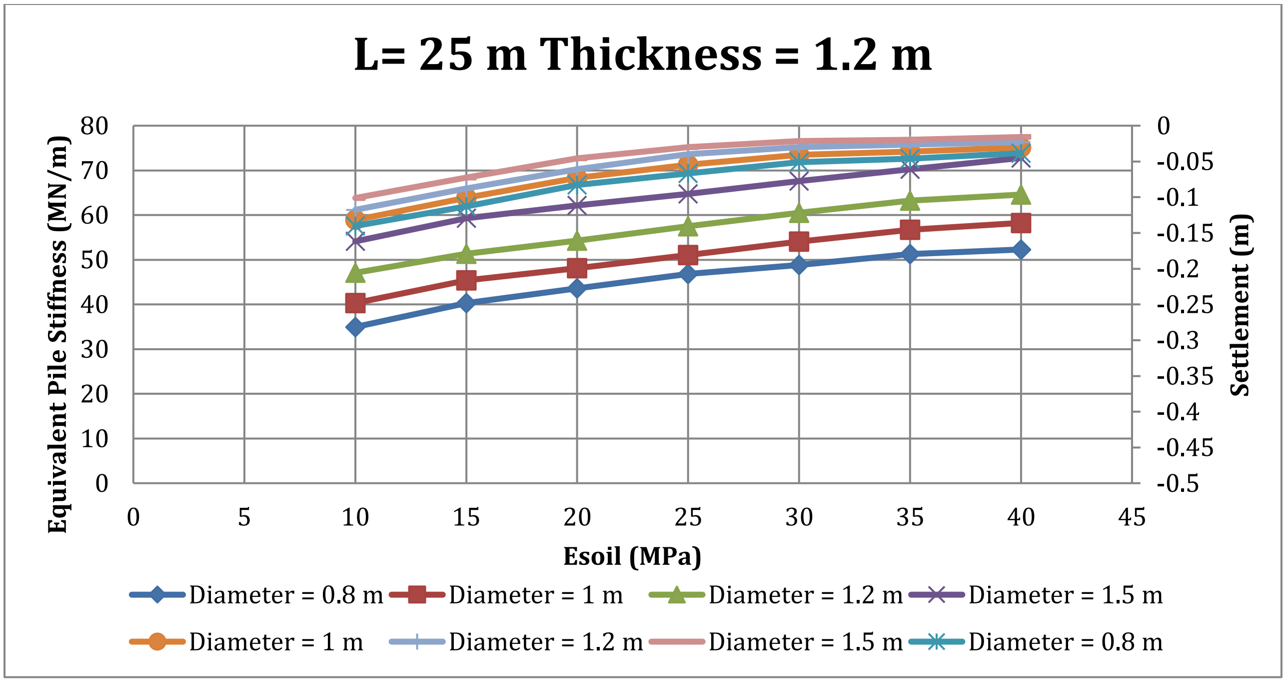

L= 20 m Thickness = 0.8 m
Equivalent Pile Stifness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m


L= 20 m Thickness= 1.2 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
5
10
15
20
25
30
35
40
45
80
70
60
50
40
30
20
10
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

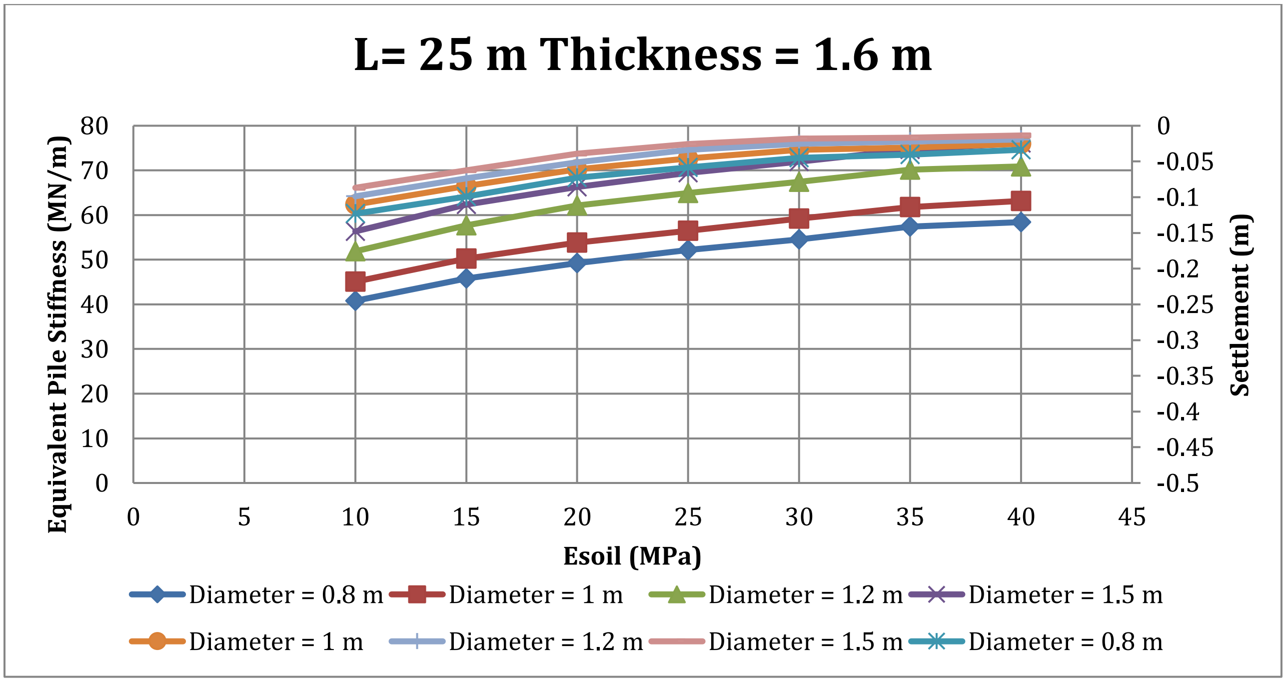

L= 20m Thickness = 1.6 m
Equivalent Pile Stiffness (MN/m)
Settlement (m)
Esoil (MPa)
0
5
10
15
20
25
30
35
40
45
80
70
60
50
40
30
20
10
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m


L= 20 m Thickness = 2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m


L= 25 m Thickness = 0.8 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m



L= 25 m Thickness = 1.2 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m



L= 25 m Thickness = 1.6 m
Equivalent Pile Stiffness (MN/m)
80
70
60
50
40
30
20
10
0
Settlement (m)
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
-0.4
-0.45
-0.5
0
5
10
15
20
25
30
35
40
45
Esoil (MPa)
Diameter = 0.8 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 1 m
Diameter = 1.2 m
Diameter = 1.5 m
Diameter = 0.8 m

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