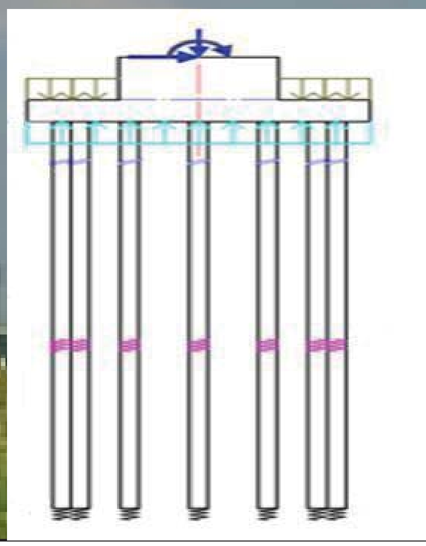
Optimization of modelling pile foundations

Final Report v1.1

G.J.C. van Gorp

MSc Graduation Thesis



Faculty of Civil Engineering and Geosciences
Section of Geo-Engineering

Heijmans Integrale Projecten BV
Ontwerp en Advies
Constructies en Geotechniek

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Faculty of Civil Engineering and Geosciences (CEG) · Delft University of Technology
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Preface

This report is the result of my research at Heijmans Integrale Projecten B.V. to obtain the Master of Science degree in Civil Engineering with the specialisation Geo-Engineering at Delft University of Technology. Topic of this research is the interaction between foundations and structures and the models which can be used for this. Although a lot of research is done to this topic in engineering practice it is still subject of a lot of discussions.

I would like to thank some people for their contribution to this work. First I want to thank Heijmans Integrale Projecten BV, the engineering firm of the contractor Heijmans, for introducing me to the topic and providing me a nice place to work. I would like to thank the people out there for the help during my work and the fun we also had. Especially I want to thank my daily supervisors Léon Tiggelman and Leo Molenbroek for the useful input on my work and the discussions we had˙

Furthermore I want to thank Mark Post of Plaxis B.V. and Jaap Bijnagte of Deltares for the information they gave me about the numerical models developed by the companies where they are employed.

Of course I also want to thank the other members of my graduation committee Paul Lagendijk, Bert Everts and Frits van Tol. Their input and feedback on my work and discussions during committee meetings and individual sessions were very useful.

Also I want to thank my friends for the nice study time we had and all the support they gave which helped me to finish my study. Finally I want to thank my parents for their support in all kind of ways during the many years of study I enjoyed.

This report is an adjusted version meant for publishing on repository.tudelft.nl. This version does not contain the analyze of the current work process of Heijmans and no Appendices.

Rosmalen, November 5, 2014

Abstract

To find the most efficient way to model the behaviour of foundation structures in this thesis three different numerical models are considered. Hereby both design efforts and economical design are taken into account. To asses this models first a literature study is done to the effects which play a role in the design of pile foundations and how the Dutch standards deal with these phenomena. In table 1 is the comparison shown between these aspects and the possibilities in the different models.

Table 1: Assessment of the different numerical models on different aspects applicable to pile foundations

	D-Pile	Scia Engineer 3D	Plaxis 3D
Single Pile			
Bearing capacity by compression	+/-	+/-	+/-
Settlement	+/-	+/-	+
Tensile bearing capacity	+	+/-	+
Axial spring stiffness	+	+/-	+
Horizontal resistance	+	+/-	+
Pile group (cap of ground)			
Bearing capacity by compression	-	-	+
Settlement	+	+/-	+
Tensile bearing capacity	+/-	+/-	+
Axial spring stiffness	+/-	+/-	+
Load distribution in cap	-	+	+
Horizontal resistance	+	+/-	+
Pile-raft foundation			
Cap-soil interaction	-	+	+
Cap-soil-pile interaction	-	-	+

$+$ possible, $+/-$ possible with workaround, $-$ not possible

After this qualitative consideration a quantitative comparison is done between a single pile in the different models which is subjected to loads in three directions.

- The load-settlement behaviour of a pile loaded by axial compression in D-Pile and Scia Engineer is as expected from the behaviour according to Eurocode 7. The behaviour of a pile in Plaxis deviates from this, which is caused by the weak behaviour around the pile tip. This because installation effects are not taken into account by this model. In the empirical relationships in the other models these effects are taken into account.
- The behaviour of piles loaded by tension is in line with CUR 77 which is at the moment the only guideline about the load-settlement behaviour of prefabricated displacement piles. However, the behaviour of D-Pile is linear which is caused by the applied vertical shaft springs in this model. The behaviour of Scia Engineer and Plaxis is non-linear.
- The behaviour of a pile loaded by an external horizontal force on the top differs significant. The numerical models D-Pile and Plaxis approach the real soil behaviour more than the chosen method in Scia Engineer. This because they give a stiffer lateral support at small displacements and plasticity of the soil is included in these methods. The method of Ménard, which is used for the horizontal subgrade reaction in Scia Engineer, gives a linear support which is less stiff at small displacements and does not include failure at large displacements.

The fact that the axial behaviour of displacement piles in Plaxis is without additional measurement not as expected according to Eurocode 7 has lead to the conclusion that this model is not taken into account in the further research of this thesis. However for complex geotechnical structures it can be beneficial to take this measures, because a finite element model like Plaxis is needed to deal with the interaction between soil and structures, which is not possible with spring models.

After the study the behaviour of single pile the research is enhanced for pile groups, which are also subject to the same loads.

- The additional settlement and the different in stiffness between the piles in a group loaded by axial compression can be taken into account by D-Pile. However the bending stiffness of the cap is not taken in account in this model, because the displacement of all pile tops is the same. The numerical model Scia Engineer can deal with the stiffness of the cap which influences the distribution of forces on the cap. However with the chosen way of modeling the group effects which are taken into account by D-Pile cannot be taken into account by Scia Engineer.
- In case of tensional loading the behaviour of piles in a group can differ significant from single piles. Destressing of the soil due to loading of the group can cause a decrease in bearing capacity of closely placed pile. On the other hand can the tensile bearing capacity of closely spaced displacement piles increase due to densification of the soil. Both these effects has to be taken into account manually in D-Pile and Scia Engineer.
- For a horizontally loaded group the effect of the different springs used in the models is clearly visible. The stiffer behaviour of the P-Y curves at small strains used in D-Pile in comparison to the Ménard springs in Scia will lead to less horizontal deflection of a horizontally loaded foundation structure in D-Pile.

Besides the different models in this thesis also the influence of the rake of the piles is investigated. Adjusting the rake of the piles on the direction of forces can have a positive effect on the deformation of the structure and the forces in the different piles.

In this thesis the effects which play a role by pile foundations and the ability of the different models are considered. Furthermore from the investigation followed which effects are normative for the design of the power pylon in the considered soil profile. Combining all these knowledge leads to the conclusion that Scia Engineer seems to be the best model for the design of the foundation structure of the power pylon. Hereby both the efficiency of the design process and the structure are taken into account. The reassons for this are:

- Besides D-Foundation which is a tool to determine the bearing capacity only one model is needed to analyse the behaviour of the piles and the foundation structure. This model is also used by the structural engineer to determine the distribution of forces and the reinforcement in the structure.
- The load-displacement behaviour of (tension) piles is governed by springs which are easy to adjust and it is easy to check if the behaviour is in line with the applicable standards. This is especially the case for tension piles, for which the bearing capacity is dependent on the pile-to-pile distance and the alternation between tensile and compressive loads.
- Uncertainties about the properties in the soil can be taken into account by vary the applied springs which represent the soil behaviour.

Glossary

List of Symbols

Symbol	*Description*	*Dimensions*	*Units*
α	Pile class factor in D-Pile	$-$	$-$
α_p	Pile class factor for the pile base	$-$	$-$
α_s	Pile class factor for the pile shaft	$-$	$-$
α_t	Pile class factor for tension piles	$-$	$-$
β	Factor which takes into account the shape of the pile base	$-$	$-$
ϵ_{50}	Strain at 50% failure	$-$	$-$
η	Efficiency factor of a pile group	$-$	$-$
$\gamma_{0.7}$	Shear strain at which $G_s = 0.722G_0$	$-$	$-$
γ_b	Partial factor for the base resistance of a pile	$-$	$-$
γ_{dry}	Dry weight density soil	M/L^3	kN/m^3
$\gamma_{m;var;qc}$	Factor which takes into account the effects of alternating loads	$-$	$-$
$\gamma_{s;t}$	Partial safety factor for tension piles	$-$	$-$
γ_{sat}	Saturated weight density soil	M/L^3	kN/m^3

γ_s	Partial factor for shaft resistance of a pile	$-$	$-$
ν	Poisson 's ration	$-$	$-$
ν_{ur}	Poisson 's ratio for unloading/reloading	$-$	$-$
ψ	Angle of dilatancy	$-$	$-$
σ'_v	Effective vertical stress	M/L^2	kN/m^2
φ'	angle of shearing resistance in terms of effective stress		$\circ$
ξ_3, ξ_4	Correlation factors to derive the pile resistance from ground investigation results, not being pile load tests	$-$	$-$

Symbol	*Description*	*Dimensions*	*Units*
ΔL	Length of the part of the pile where the shaft friction may be taken into account for the bearing capacity	L	m
A	Cross section of the pile	L^2	m^2
c'	cohesion intercept in terms of effective stress	M/L^2	kN/m^2
$C'_{p;j}$	Pimary settlement constant of the layer according to Eurocode 7	$-$	$-$
c_u	undrained shear strength	M/L^2	kN/m^2
C_{qc}	Conversion factor for the cone resistance	$-$	$-$
c_v	Consolidaton coefficient	L^2/T	m^s/s
$E'_{50,rep,\sigma'_v=100}$	Effective Young 's modulus at a vertical effective stress of 100 kN/m^2	M/L^2	kN/m^2
E^{ref}_{oed}	Tangent stiffness for primairy oedometer loading	M/L^2	kN/m^2
E^{ref}_{ur}	Unloading/reloading stiffness	M/L^2	kN/m^2
E_{pile}	Young 's modulus of the pile	M/L^2	kN/m^2
E_pI_p	Flexurale stiffness of the pile	ML^2	kNm^2
f_1	Factor for the effect of densification in the pile group for ground displacement piles in sand	$-$	$-$

f_2	Factor for decrease in effective pressure in the sand layers where the piles determines its bearing capacity caused by loading the pile group	$-$	$-$
F_{fund}	Sum of the loads on the piles in the group	M	kN
G_0^{ref}	Reference shear modulus at very small strains	M/L^2	kN/m^2
K_0	Neutral soil pressure factor	$-$	$-$
k_{axial}	Axial stiffness of a pile loaded by tension	M/L	kN/m
$k_{elastic}$	stiffness due to the elastic extension of pile	M/L	kN/m
k_h	Horizontal subgrade reaction	M/L^3	kN/m^3
$k_{shaft,rep}$	stiffness with respect to the part of the mobilization of the skin friction	M/L	kN/m
m	Power for stress-level dependency of stiffness	$-$	$-$
m^*	Factor accoding to Eurocode 7 to take into account the shape of the loaded surface	$-$	$-$
$O_{s:\Delta L;gem}$	Mean circumference of the pile shaft	L	m
p^{ref}	Reference stress for stiffness	M/L^2	kN/m^2
p_{ud}	Actual lateral soil resistance	M/L^2	kN/m^2
p_{ud}	Ultimate lateral soil resistance at deep depth	M/L^2	kN/m^2
p_{us}	Ultimate lateral soil resistance at shallow depth	M/L^2	kN/m^2
q_c	Cone resistance	M/L^2	MPa
$q_{b;max}$	Maximum base resistance which may not exceed 15 MPa	M/L^2	MPa
$R_{b;d}$	Design value of the base resistance of a pile	M	kN
$R_{c;d}$	Compressive resistance of the ground against a pile, at the ultimate limit state	M	kN
$R_{s;d}$	Design value of the shaft resistance of a pile	M	kN

$R_{t;d}$	Design value of the tensile resistance of a pile or pile group	M	kN
s_b	Settlement of the pile base	L	mm
s_2	Settlement caused by compression of the layers more than 4D below pile tip level	L	m
s_{el}	Elastic deformation of the pile	L	m
T	Mobilized total skin resistance in D-Pile	M	kN
T_{max}	Maximum total skin friction	M	kN
y	Actual lateral deflection	L	m
d	Pile shaft diameter	L	m
f	Skin friction	M/L^2	kN/m^2
s	Centre-to-centre distance piles	L	m

Contents

Chapter 1

Introduction

In current engineering practice contractors are responsible for the design, construct and sometimes even maintenance of infrastructure. In order to gain new projects and keep workload on level, it is vital that tenders are carefully worked out. For this purpose, structural engineers need to make accurate and reliable predictions of the structural behavior. On the other hand, the tender costs that contractors make mostly are not reimbursed. Therefore it is also important to have a suitable method to execute a quick structural design in tender phase.

An example of a modern design and construct project is the foundation of power pylons as shown in figure 1-1. Building power pylons means a lot of repetition. So it is very important to have an efficient design procedure. Having these is beneficial for the contractor which benefits being competitive, as well society because of more value for money. Important is that a reliable electricity network often is referred to as critical infrastructure which is essential for our economy. So design and construction procedures needs to be safe at all time.

1-1 Research background

One of the things, where in practice many discussions are about, is the interface between geotechnical- and structural engineers. Structural engineers use schemes to predict behavior of structures. These consist of springs and subgrade reactions to take into account the behavior of soil. The spring constants and the modulus of subgrade reaction mostly are determined by the geotechnical engineer. The challenge hereby is that these springs are not constant, because of the non-linear behavior of soil. Another challenge is the spatial variability of soil.

Besides the numerical models that structural engineers use, there are also numerical models for geotechnical engineers. These models can predict the behaviour of additional soil or structure on top of the soil. These models vary from relative simple spring models which needs a few parameters, to complex finite element methods with a lot of parameters.

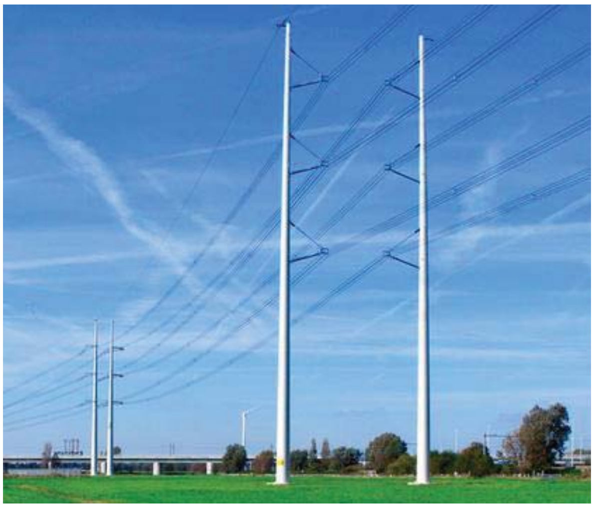


Figure 1-1: Windtrack power pylon

In this thesis different numerical models that are available in the Dutch engineering practice will be analysed. Main goal is to conclude how these models deal with soul-structure interaction. Hereby it will be taken in account what the possibilities of these models are, and their potential to design safe, efficient, and economical. The considered numerical models are:

- Scia Engineer 3D
- D-Pile group
- Plaxis 3D

SCIA Engineer is a structural program, D-pile and plaxis 3D are geotechnical. Furthermore, D-foundations will be used because of its outcome that is necessary as input in the models described above.

1-2 Research objectives

The main question which should to be answered in this research is:

What is the most efficient way to model a foundation structure, wereby both the efficiency of the design as the building costs of the structure will be taken into account?

To answer this question it is divided into several subquestions:

1. What are the aspects which play a role by modelling of single piles and pile groups, like for example the load-deformation behaviour and which methods can be used to analyze these effects?

2. What is the background of the different numerical models which will be considered and how do they deal with the aspects which play a role in the design of pile foundations?
3. What is the difference by modeling single in different models (D-series, Plaxis 3D and Scia Engineer)and how are these differences caused?
4. What is the difference by modeling pile groups in the different models and what is the effect of different pile configurations and how are these differences caused?
5. How can the foundation structure of a power pylon be designed the most efficient?
6. How is the current design process at Heijmans dealing with the interaction between the construction and foundation? What can they gain by implementing the methods described above?

1-3 Limitations

To make sure that enough focus on the topic can be reached the follwing limitations will be taken into account:

- Only pile foundation with a cap on top will be taken into account and no sheetpile walls, raft foundations etc.
- Only three-dimensional models will be taken into account, because the behaviour of the soil is typical a three-dimensional phenomen. Especially related to laterally loaded piles.
- For the pile type driven prefabricated concrete piles will be taken into account.
- The bearing capacity of piles will be determined by calculation methods described in Eurocode 7 and not based on pile test methods.
- In this thesis dynamic loads will not be taken into account.
- The focus will be on design in Dutch soil conditions.
- For piles only loading by external loads will be taken into account and not loading by soil movements.

1-4 Research approach

In figure 1-2 is shown how this research is set up. Every block corresponds with a chapter of this report. These steps will be taken to answer the main question in the conclusion.

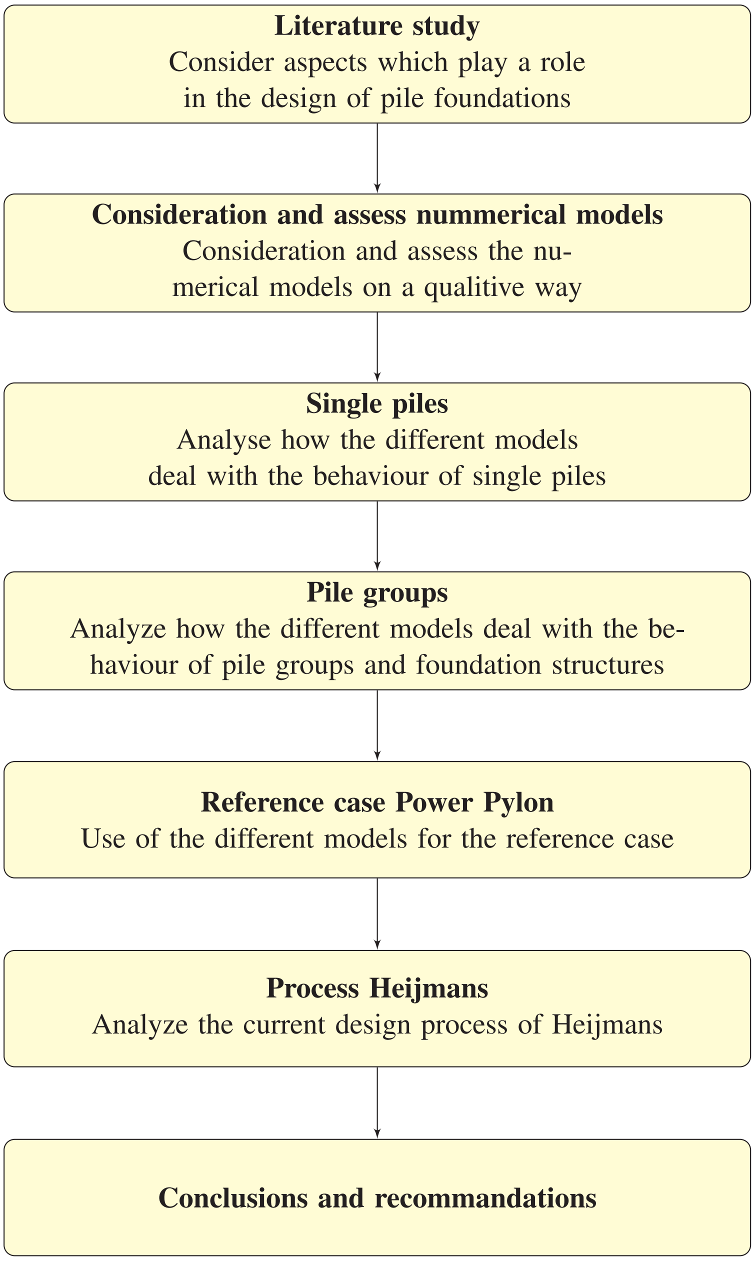


Figure 1-2: Thesis outline

Chapter 2

Literature study

Before one can consider the different numerical models which will be done in chapter 3 one first has to know which aspects play a role in the design of pile foundations. Hereby also different methods which can be used for the analyze of pile foundations will be considered. Furthermore in this chapter will be considered how the current standards (Eurocode), guidelines and recommendations (CUR) prescribed in the Dutch engineering practice deal with these aspects.

There are multiple methods to consider the behaviour of piles. These methods can be defined in three different categories, which are described by Poulos (27):

1. Empirical methods, which are commonly used for the analyze of axial loaded single piles.
2. The use of simple computational methods or design charts based on a theoretical basis. These are used for the analyze of horizontal loaded piles and pile groups. These analyzes can often be done by hand calculations
3. Category 3 methods involve the use of site-specific analysis based on relatively advanced analytical or numerical techniques such as finite element methods or boundary element methods. For this category a computer is always needed.

These methods are described more extensively in Appendix A.

When one wants to choose a method for a practical problem the factors that need to be considered according to Poulos (27) are:

- The significance and scale of the problem.
- The available budget for the foundation design.
- The geotechnical data availabe.
- The complexity of both the geotechnical profile and the design loading conditions.

- The stage of the design process (i.e. whether a feasibility, preliminary or final design is being carried out).
- The expierence of the designer with the methods considered.

Poulos (27) illustrated in his paper that the method of analyze frequently is of much less importance than the geotechnical parameters which are selected and the way in which the geotechnical profile is idealized.

2-1 Single Piles

2-1-1 Axial loaded piles

When one want to analyze the behaviour of a single pile one has to know how the bearing capacity of the pile is mobilized and how much displacements is required for that. This because a pile has to settle a bit to activate its shaft and/or base bearing capacity. There are different ways to analyze this load-deformation behaviour.

Load-deformation relationship

- Load-transfer method: In this method the pile is modelled as a member supported by (non)linear springs which represent the soil in skin friction and endbearing. This method is also called the T-Z method.
- Boundary element method: In this method the soil is modelled as an elastic halfspace. The pile is divided in segments which are loaded by skin friction from the surrounding soil.
- Finite elements method: In this method the behaviour of pile and soil is simulated by elements which represent the characteristics of these materials.

These methods and there pros and cons are further more described in Appendix A-1-1.

Bearing capacity

The bearing capacity of single piles consists out of the bearing capacity at the base of the pile and the bearing capacity along the shaft (positive skin friction). In the Dutch Engineering practice this maximum base bearing capacity is determined by the method of Koppejan, which is described in Eurocode 7 (7). This method belongs to category 1 as described in the beginnning of this chapter. This emperical method is based on the results of a Cone Penetration Test (CPT). In the Eurocode the maximum shaft resistance is also determined by a method based on the cone resistance.
To mobilize the bearing capacity of a pile some displacement is needed. To activate the shaft friction less displament is needed than for activating the base bearing capacity. An example of this is given in Appendix A-1-2.

Furthermore to determine the bearing capacity of a pile the negative skin friction has to be taken into account. The negative skin friction works as an extra load on the pile and is caused by settlement of soil layers. In Eurocode 7 is also described how this negative skin friction can be determined. This is done by the so called Slip-method.

Settlement

Although foundation piles are considered as stiff elements, settlement due to loading has to be taken into account. To determine the settlement of a single pile the following two effects have to be taken into account

- Settlement of the pile base
- Elastic deformation of the pile

The settlement of a single pile can be determined from test loads or with one of the load-deformation relationships described above. In the Dutch Engineering practice the load-transfer method is mostly used. In Appendix A-1-3 this method is described.

Influence of plate on single pile behaviour

In Germany a lot of research is done to Combined Pile-Raft Foundations (CPRF 's). These are foundations which derive their bearing capacity both on the pile and raft (plate). Hanisch et al (20) stated that:

- Combined Pile-Raft Foundations are only applicable in homogeneous soil masses. When the bearing stratum is much stiffer the foundation will behave like a conventional pile foundation.
- The raft has a significant influence on the shaft bearing capacity. When there is little settlement the shaft bearing capacity of a CPRF is less than the shaft bearing capacity of a conventional pile foundation. This because there is less differential settlement between the pile and soil, because the plate pushes the soil downwards. When there is more settlement the soil stress along the pile shaft increases and the shaft bearing capacity also increases.

These two effects are described in Appendix A-1-4.

Tension Piles

A pile can be subjected to tensional forces for example by constructions below the water table, or a foundation which is subjected to a large bending moment. It is important to know the capacity and the stiffness of this tenion pile. For the bearing capacity of tension piles two effects have to be checked.

- Pull out resitance of a tension pile out of the soil mass
- Rise of the tension pile together with the accompanying soil mass. This effect is more a group effect and will therefore be discussed in paragraph 2-2-4.

The resistance against pull-out of the soil is caused by friction between the pile and soil. This shaft resistance can be mobilized by relative deformation between the pile and soil. In clay there is more deformation needed to mobilize this than in sand. Therefore it is generally recommended to obtain this bearing capacity from sand layers or deep stiff clay layers. In CUR 2001-4 (2) different methods are described to deal with the pull out of piles. These are also described in Appendix A-1-5. In Eurocode 7 (7) the q_c method is prescribed to determine the maximum resistance of a pile against pulling out. This method is based on the results of a CPT. Dependent on the type of soil and pile a certain percentage of the cone resistance may be expected as skin friction. The method prescribed in Eurocode 7 is based on the load-transfer method as described in paragraph 2-1-1. When a pile is subjected to alternating loads this will lead to a decrease of the friction between the pile and soil. So this will lead to a decrease in tensile bearing capacity. In CUR 2001-4 there is also a description given of the modelling of a tension pile in the numerical model Plaxis. In section 3-4 the different ways to model tension piles will be described in more detail.

For the total behaviour of the construction it is important to know how the tension pile reacts under loading. In CUR 77 (1) a method is described how the axial stiffness from the top of a tension pile can be determined. The equation to determine this spring stiffness is given in Appendix A-1-5.

Reflection on Eurocode 7

Besides methods with test loads, which will not be taken into account in this thesis, the determination of the load-deformation behaviour of single piles has to be done with a load-transfer method. In this method the pile- and shaft bearing capacity can be determined with a method based on the result of a CPT. The settlement of the pile can also be determined with emperical relationships based on the results of a CPT and pile tests.

In Eurocode 7 is set that the provisions made in the chapter Pile Foundations may not plainly be used for Combined Pile Raft foundations. According tho Eurocode 7 it can be necessary to make aan interaction calculation between the construction, pile foundation and the soil to prove that the demands from the ultimate limite state are satisfied. For relavite simple constructions from Geotechnical category 1 and 2 (7, p.38) the load has to be beared completely by the pile foundation.

For tension piles Eurocode 7 also gives a method to determine the tensional bearing capacity of a single pile. This is also based on a load-deformation method based on the results of a CPT. Hereby also the effect of alternating is taken into account.
When one wants to determine the stiffness of a tension pile this can be done with a method given in CUR 77 (1).

2-1-2 Horizontal loaded piles

In this thesis only piles loaded by external horizontal force will be considered. So horizontal loading through soil will not taken into account. There are multiple ways to model the horizontal resistance of a pile in the soil. In Appendix A-2 different methods to deal with horizontal loaded piles are given. In general the methods which can be distinguished are:

Ultimate soil resistance

There are different methods which can determine the ultimate resistance of a pile in the soil against horizontal loading. Examples of these are the methods of Broms and Brinch Hansen. The method of Brich Hansen gives an ultimate resistance of the soil and considers the three-dimensional behaviouor of the soil around the pile. With this method the maximum horizontal force on a pile, or minimum required installation depth can be determined. Furthermore the bending moments and shearforces in the pile can be determined. This method can be used for both free or fixed head piles. The method does not take into account the displacement needed to reach this maximum resistance. Therefore the method can not be used to determine the deflection of the pile. A more extensive description is given in Appendix A-2-1.

Subgrade-reaction analysis

Just as for axial loaded piles also for horizontal loaded piles a load-transfer method can be used. For this method one has to define springs which represent the horizontal support of the soil against the pile.

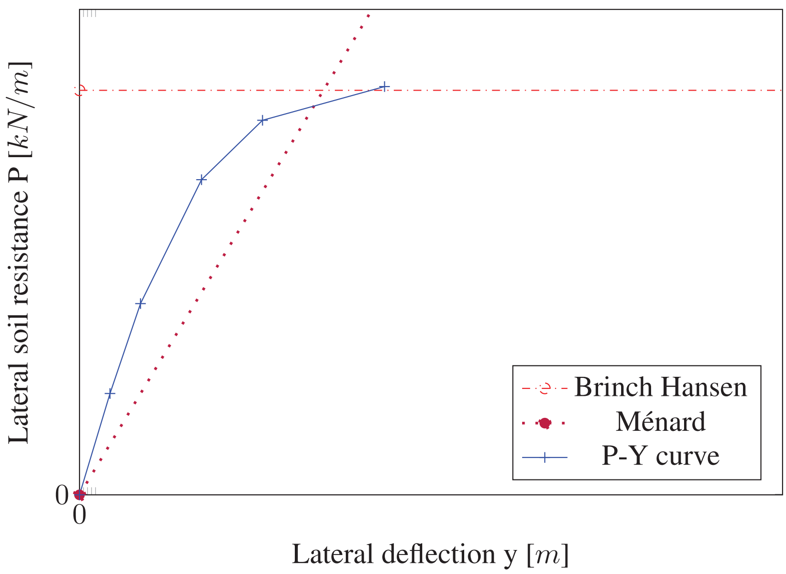


Figure 2-1: Qualitative relationship between different methods of horizontal subgrade reaction

- Linear modulus of subgrade reaction
  With the method of Ménard the horizontal coefficient of subgrade reaction can be determined based on the results of a pressiometer test or CPT. This coefficient depends on the type of soil and diameter of the pile. The method takes into account the arching effect of the soil around the pile. With this method the stiffness of the soil does not depend on the deflection of the pile.
- Non-linear modulus of subgrade reaction
  In reality the soil does not behave perfectly elastic as described in the method of Ménard. When the deflection of the piles increases the soil will start to yield and will behave less stiff until the

ultimate resistance is reached. A method which describes this non-linear behaviour is called a 'P-Y curve'. This method is originally developed in the offshore industry where pile are subjected to high horizontal loads.

These two methods are explained in more detail in Appendix A-2-2. In figure 2-1 a qualitative comparison between the methods described above is given. Here one can see that the method of Brinch Hansen gives a constant resistance. In the P-Y curve one can see that the resistance of the soil is increasing by increasing deflection of the pile till the maximum resistance of the soil is reached (plasticity). The method of Ménard gives a constant increase of the soil resistance by an increasing deflection of the pile.

Boundary element method

Like for axial loaded piles horizontal loaded piles can also be considered with the boundary element method. In these models the horizontally loaded pile is modelled as an infinite thin strip, with the flexural stiffness of the pile (E_pI_p). The soil is modelled as an elastic medium. One can distinguish floating piles (see figure2-2a) and socketed piles (see figure 2-2b). One can also distuingish piles which are fixed at the top and free-head piles.

(a) Floating pile **(b)** Socketed pile

Figure 2-2: Different situations for a horizontal loaded pile

This analysis is also extended with plasticity of the soil. In Appendix A-2-3 a more extensive explanation of this method is given.

Finite element analysis

Finite element methods like previously described can also be used to determine the behaviour of horizontally loaded piles. They can take into account non-linearity of the soil-behaviour and also time-dependent effects as consolidation. In finite elements methods different soil layers can be inserted. A further description of piles in finite element models is given in 3-4.

Structural analysis of the pile

When one analyzes the behaviour of a pile in a model also the structural behaviour of the pile has to be considered. A horizontal force or bending moment on the top of the pile causes a bending moment in the pile. When this bending moment exceeds the cracking moment of the concrete the bending stiffness of the pile in that part reduces. This will lead to increasing deflection of the pile.

Reflection on Eurocode 7

In Chapter 7 of Eurocode 7 (7) is stated that horizontal loaded piles can be checked by field test, which will not be taken into account further, or by validated empirical or analytical methods. In general for single piles one of the following failure mechanisms has to be taken into account.

- Short piles: translation or rotation of the pile as a stiff body.
- Long slender piles: Failure on bending of the pile in combination with local failure of the soil around the pile.

Hereby no definiion of short and long slender piles is given. Furthermore for the bearing capacity in horizontal direction the following aspects have to be taken into account:

- Structural failure of the pile has to be checked.
- For a long slender pile, the pile may be schematized as a beam with a load on one end which is supported by a horizontal modulus of subgrade reaction.
- By determing the bearing capacity the degree of freedom for the connection between the pile and construction has to be taken into account.

For the horizontal displacement of the piles the aspects that have to be taken into account for single piles are:

- Stiffness of the soil and variation of this with the level of strain.
- Bending stiffness of the individual piles.
- The fixed-end moment of piles connected to a construction.
- The effect of changing and cyclic loads.

In Eurocode 7 no real design method is prescribed for horizontally loaded piles. In CUR 228 (4) different design methods are given for the analysis of horizontally loaded piles.

2-2 Pile Groups

Piles in a group behave differently than single piles. This applies for both pile groups subjected to vertical and horizontal loading. According to Poulos and Davis (29) by analysing pile groups it is important to distinguish two different types of groups:

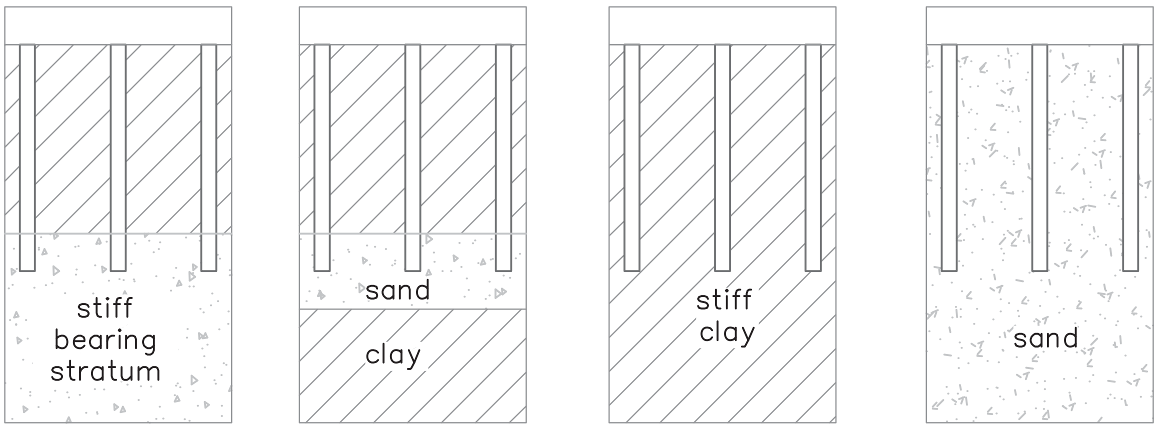
1. A free-standing group, in which the pile cap is not in contact with the underlying soil.
2. A 'piled foundation' in which the pile cap is in contact with the underlying soil.

2-2-1 Bearing capacity of pile groups loaded by compression

By analyzing the bearing capacity of a pile group Poulos and Davis (29) used the The efficiency factor (η) of the group which can be determined with equation 2-1.

$$\eta = \frac{ultimate\ load\ capacity\ of\ the\ group}{sum\ of\ ultimate\ load\ capacities\ of\ individual\ piles} \tag{2-1}$$

For endbearing piles where there is no compressible layer below the bearing stratum (see figure 2-3a) η can be considered as one. When there is a soft layer present below the bearing stratum (see figure 2-3b) or when the shaft friction has a big contribution to the bearing capacity of the piles (for example in stiff clay, see figure 2-3c), then η can decrease significantly.



(a) $\eta\ \approx 1$ **(b)** $\eta\ \leq\approx 1$ **(c)** $\eta\ \leq 1$ **(d)** $\eta\ > 1$

Figure 2-3: Pile foundations in different soil conditions with their efficiency factor (η)

Pile groups in clay

One of the most used methods to determine the pile group capacity is given by Terzaghi and Peck (29). In this method the bearing capacity of the group is the lesser of:

- Sum of ultimate load capacity of individual piles
- The bearing capacity for block failure of the group

The block failure defined by Tomlinsom (33) can be determined by analyzing the block as a burried raft (see Appendix B-1-1).

According to Poulos and Davis (29) the efficiency factors of pile groups can increased by taking the following measures:

- Decrease the length-to-diameter ratio of the pile, this because when the diameter of a single pile decreases, the bearing capacity of a single pile decreases.

- Increase the spacing between the piles
- Decrease the number of piles in the group

This is concluded from tests and calculations on pile groups in homogeneous clay done by Withaker in 1957. Prove of this is shown in Appendix B-1-1.

Pile groups in sand

For pile groups in sand (see figure 2-3d) the efficiency (η) is quite often higher than 1. This is shown in measurements of Vesic which are shown in Appendix B-1-2. The maximum efficiency is reached for pile spacings of 2 to 3 times the pile diameter. Hereby the increase in shaft bearing is larger than the increase in base bearing. When the effect of the cap resting on the soil is also taken into account the efficiency increases even more. This is because a part of the load is transferred directly from the cap into the soil. But in this case for the mobilization of the shaft bearing capacity more displacement of the cap is needed.

Reflection on Eurocode 7

In Eurocode 7 for the bearing capacity of pile group also has to full fill two failure mechanisms. These are more or less similar as the method given by Terzaghi and Peck. The lowest value of the following to failure mechanisms is guiding

- Failure on pressure of individual piles.
- Failure on pressure of the piles and soil between as a collaborating block.

This second criterium must be checked by taken the block as one big pile with an equivalent diameter and determine the bearing capacity.

2-2-2 Settlement of pile groups

Piles in a group do not behave the same as single piles. When piles are placed close together they will influence each other. This can lead to additional settlement of the piles. There are different methods to estimate the settlement of the pile group and or individual piles in the group. These are described below. The effects which are present at the settlement of pile groups are also discussed below.

Methods to estimate settlement

The settlement of a pile group can differ significantly from that of a single pile. Approaches that are commonly used for this are described by Poulos (28). Below a short overview of these methods is given. They are described more extensively in Appendix B-2-1

- Interaction factors and the principle of superposition
  With interaction factors the additional settlement of a pile due to another pile can be defined. These factors are the result of a large number of analyses on simplified soil stratigraphies. The first factors were only for homogeneous elastic soil with a constant Young 's modulus. In time improvements on this are made, like taking into account a stiffer bearing stratum and plasticity of the soil around the pile.

- The settlement ratio method
  With this method the settlement of a single pile at the average load level is multiplied by a group settlement ratio. This ratio reflects the effects of the group interaction.

- Equivalent raft method
  In this method the pile group is represented by an equivalent raft at some characteristic depth along the pile. The settlement can be calculated as the settlement of a burried raft foundation.

- The equivalent pier method
  Here the piles and the soil between them are represented by a single pier. This pier is treated as a single pile of equivalent stiffness in order to compute the avarage settlement of the group.

- Numerical methods
  Numerical methods such as the boundary element method and the finite element method can be used to estimate the settlement behaviour of pile groups. In earlier work this were most two-dimensional analyses, but full three-dimensional methods are getting more common in the engineering practice.

Effects by group settlement

There are different effects which play a role for the settlement of a pile group and the influence of the settlement of a pile to the settlement of another pile. The influence of one pile to another can be described with interaction factors as discussed above. These different effects are shortley described below and more extensively in Appendix B-2-2.

- Pile distance
  When the distance between the piles increases the interaction between the piles decreases.

- Stiffness of the piles
  The relative stiffness of the pile is the ratio of the pile and soil stiffness. The larger the relative stiffness of the pile is, the larger the interacton between adjacent piles.

- Slenderness of the pile
  The slenderness of the pile is the ratio of the length over diameter of the pile. When the slenderness of the pile increases, the interaction between the piles also becomes larger.

- Homogenity of the soil mass
  When piles are in a homogeneous soil mass and their bearing capacity is mainly derived from skin friction the interaction between adjacent piles is bigger than for endbearing piles which are founded on a stiffer bearing stratum (non-homogeneous). When there is a soft layer present beneath the bearing stratum this has a significant influence on the settlement of the pile group.

- Consolidation
  For single piles almost the whole settlement occurs immediately after loading. For pile groups the time-dependent settlement can also play a role.

Reflection on Eurocode 7

For pile groups both the settlement of individual piles (s_1) as the settlement caused by the group effect (s_2) must be taken into account. The total settlement is the sum of these two. The group effect has to be taken into account when the center-to-center distance of the piles is smaller than ten times the smallest width of the pile base. s_2 may be determined in two different ways.

- Considering the pile group as a raft foundation and determine the settlement on the same manner as a raft foundation, which is described in Eurocode 7.

- Determing settlement by dividing the pressure below the pile tip by the Young 's modulus of the soil more than four timss the pile diameter below the pile tip.

Eurocode 7 does not take into account interaction between the piles from 4d below the pile tip till the top of the pile by determing the additional settlement of a group.

2-2-3 Horizontal loaded pile groups

Piles in groups loaded by external lateral forces behave different than single piles. A general trend found by the different researchers is that for in-line piles (see figure 2-4) the horizontal resistance of the piles is the same as for single piles when the center-to-center distance of the piles is more than 8 times the pile diameter. For side-by-side piles the behaviour is the same as for single piles when the distance (s) between the piles is more than three times the pile diameter (d). In figure 2-5 is shown that less passive resistance can be activated when piles are closely placed together. For piles placed closer together the horizontal resistance will reduce. Hereby the efficiency of the trailing piles (see figure 2-4) will be less than the leading piles. This because the trailing piles are 'in the shadow' of the leading piles.

Like for laterally loaded single piles, pile groups can also be analyzed in different ways. They are described in Appendix B-3 and briefly described below.

Subgrade-reaction analysis

There are different ways to reduce the horizontal subgrade reactions for pile groups. These methods are described by Reese & Van Impe (30). The most sophisticated method described in this book is developed by Wang and Reese. In this method efficiency factors (e) are developed to adjust the non-linear modulus of subgrade reaction (P-Y curve) of single piles. These are made by curvefitting the results of different experiments. In this method no differentiation is made with respect to soil, diameter of piles and way of penetration. There are efficiency factors for side-by-side piles, leading piles and trailing piles. These have to be combined when there are more than 2 piles in a group.

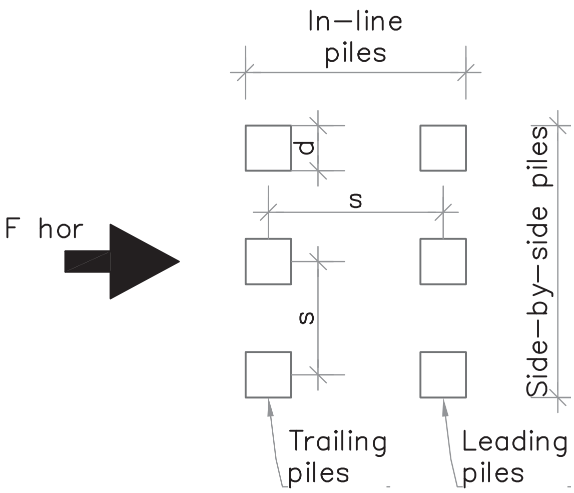
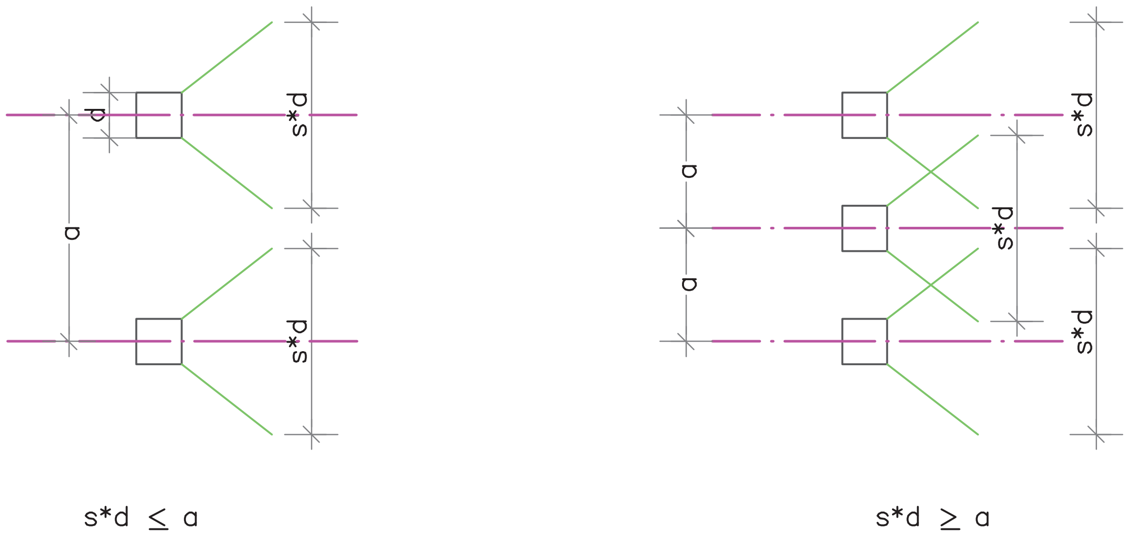


Figure 2-4: Schematization of a laterally loaded pile group



(a) $s \cdot d \leq a$

(b) $s \cdot d \geq a$

Figure 2-5: Schematization of laterally loaded piles with passive resistance (5, p.37)

Boundary element methods

Like for single pile behaviour and group settlement Poulos & Davis also analyzed the behaviour of horizontally loaded pile groups with the boundary element method. For quick analysis they also derived interaction factors (α) for different conditions of loading and head fixity

Trends in the interaction behaviour in horizontal loaded pile groups noted by Poulos & Davis (29) are:

- The interaction between piles increases by increasing slenderness of the piles.
- The interaction generally increases, when the relative stiffness between the pile and soil in-

creases

- For a free-head pile, the interaction factors for bending moments are less than those for horizontal loading.
- For a free-head pile, the deflection-interaction factors are larger than the corresponding rotation factors.
- The outer piles carry the largest loads and the center piles the least.
- The load distribution becomes more uniform as spacing between the piles increases.
- The nonuniformity of load distribution generally becomes more pronounced as the relative stiffness and slenderness of the piles increase.

Boundary element methods can give a good estimation of the pile behaviour, but there are restrictions because the amount of soil layers that can be included is limited and the non-linear behaviour of the soil is not taken into account.

Numerical methods

The behaviour of piles can be analyzed by finite element methods. In this mehod one can include plasticity of the soil and as many layers as needed can be imported. Also different kind of structural elements can be be included like: piles, plates and beams. And they can be connected in different ways (flexible, stiff).

Reflection on Eurocode 7

In paragraph 2-1-2 the regulations for horizontal loaded single piles are set. Additional rules for horizontal loaded piles in groups are:

For the bearing capacity:

- The group effect must be taken into account.
- One has to take into account that a horizontal load on a pile group can result in a combination of push, tension and horizontal forces in the individual piles.

For the deflection of the piles in a group also the group effect has to be taken into account.

2-2-4 Tension piles in groups

In section 2-1 the two failure mechanisms of tension piles are already discussed. The total uplift of the soil mass including piles can be the guiding failure mechanism for a group of piles loaded by tension. The resistance of against pulling out in a group is effected by the next three effects:

- Densification of the soil due to installation of closely spaced piles which causes an increase between in the friction between the pile and soil. So this has a positive effect on the tensional bearing capacity of the piles.
- Decrease of the effective vertical stress due to an excavation. This will decrease the friction between the pile and soil and so has a negative effect on the bearing capacity of the of the pile.
- Decrease of the vertical effective stress in the subsoil due to tensional loads on the piles. This will decrease the friction between the pile and soil and so has a negative effect on the bearing capacity. This effect is the largest for closely spaced piles and descreases for increasing pile distances.

Reflection on Eurocode 7

In Eurocode 7 is prescribed how the two different failure mechanisms described above are taken into account.

- The densification of the soil due to installation is taken into account by increasing the cone resistance with a factor f_1. When one wants to take this into account after installation one has to check or this densification really has occured.
- The effect of an excavtion is taken into account by decreasing the cone resistance with a factor which depends on the decrease of vertical effective stress.
- The effect of loading in a group is taken into account by adjusting the cone resistance with a factor f_2.

2-2-5 Distribution of vertical loads on foundation structures

The distribution of forces depends on both the stiffness of the super strucutre and the stiffness of the foundation. This effects works on both pile and shallow foundations.

Furthermore the distribution of loads depends on the stiffness of the pile cap, as discuseed by Poulos and Davis (29). For the settlement of pile groups Poulos and Davis (29) considered two different cases (see Appendix B-2).

- Piles with a flexible cap where the load is divided equal on the piles.
- Piles with a rigid cap where all piles settles equal.

From the interaction factors of Poulos one can derive that the piles in the centre of a group are influenced the most by adjecent piles. So in a group with a flexible cap the centre piles will settle most.
For the second case the corner piles will behave stiffer, because they are influenced by less other piles, than the centre piles. And when a pile is stiffer it will attract more load. Heinen et al (21) divided buldings in three categories depending on their relative stiffness (Δk). In Appendix B-5 is stated how this can be determined.

- Category 1: Flexible
  In this category there is hardly no redistribution of forces. The forces on the foundation follow from the weight calculation. This category corresponds to the first case described by Poulos as noted above.

- Category 2: Not flexible, not stiff
  In this category the load distribution depends on the stiffness of the suppports (piles). The stiffness of the supports depends on the load it has to carry. Therefore one has to make an interaction calculation. Below the steps one has to take for such a calculation are listed.

- Category 3: Stiff
  In this category the structure can be defined as infinite stiff and all elements will settle equally. However the load will not be divided equally. The outer piles will carrry more load than the centre piles.

For the interaction calculation which is needed for category 2 according to Van Tol (35, p.2-10) the following steps has to be taken:

1. Make a model of the structure with an as accurate as possible approximation of the stiffness
2. Determine the maximum bearing capacity of the foundation element and its load-settlement curve.
3. Determine the forces on each foundation element, assume that the building is flexible (so every element bears a corresponding part of the load).
4. Determine the settlement for the in step 3 calculated force and the accompying spring constant.
5. Calculate the forces in the foundation using the model from step 1 and the spring constants from step 4.
6. Determine again the spring constants, with the forces obtained in step 5.
7. repeat step 5, using the spring constants derived in step 6.

Step 5,6 and 7 must be repeated untill the spring constants do not change much anymore and the forces in the structure stay constant.

Reflection on Eurocode 7

In Eurocode 7 is stated that for the bearing capacity of a pile foundation the stiffness and strength of structure which connects the piles has to be taken into account. For a stiff construction (category 3) individual piles may faill as long as the total load can be redistributed on the total construction. When piles support a flexible construction (category 1) the resistance to pressure of the weakest pile is normative for the arise of an ultimate limite state.

2-2-6 Combined Pile-Raft Foundations

Combined Pile-Raft Foundations (CPRF 's) are mainly used for raft foundations where large settlements are expected. The piles are in this case used to reduce the settlement and placed at tactical locations where high loads are expected. The application of CPRF 's is limited to homogenous soil masses, because otherwise the foundation will behave as a conventional pile foundation as described in 2-1. When one wants to model a CPRF and take into account all interactions as shown in figure 2-6 analytical or empirical methods are not sufficient and numerical methods, like Finite Element Methods must be applied.

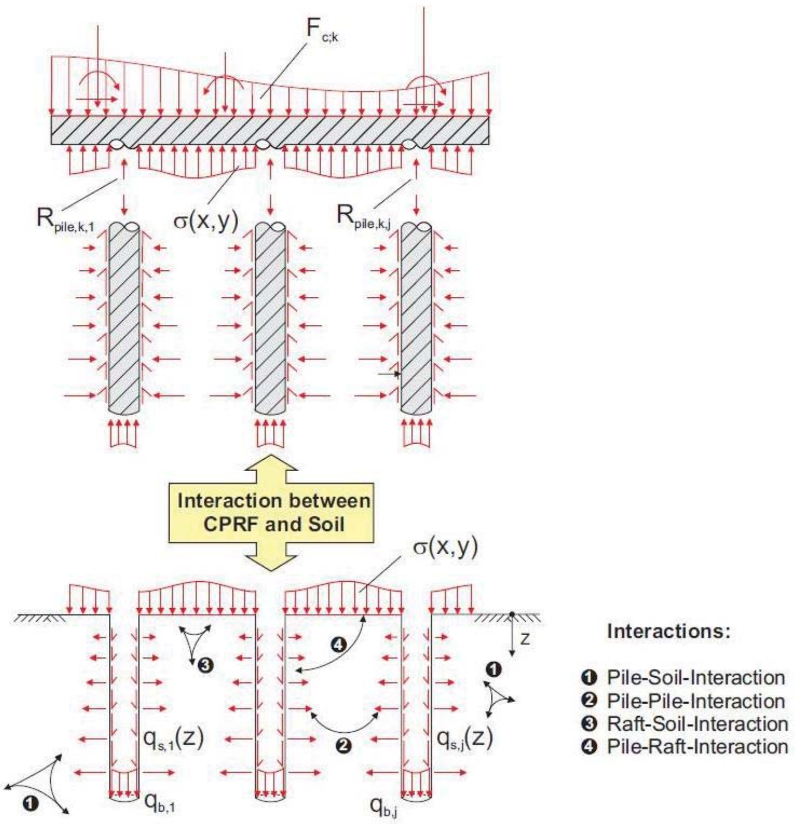


Figure 2-6: Schemtic sketch of a Combined Pile Raft Foundation (CPRF) and the interactions linking to the bearing behaviour (22, p.588)

Like for conventional pile groups, piles of a CPRF 's also influence eachother. This effect is shown in Appendix B-7. But this effect is less than for conventional pile foundations. Other effects noticed by Hanisch et al (20) with respect to CPRF 's are that:

- The ultimate shaft resistance of the piles of a CPRF is not reached. With increasing settlement the shaft resistance keeps increasing.
- The raft of a CPRF causes a significant reduction of the pile spring stiffness. This is in particular the case for small settlements. Here the capacity of the pile is less as discussed in paragraph 2-1-1, which has an influence on the spring stiffness of the pile.

These effects are also desribed more extensively in Appendix B-7.

Reflection on Eurocode 7

As stated in paragraph 2-1 Conbined Pile Raft Foundation may only be applied to constructions in Geotechnical category 3.

2-3 Conclusions

In this chapter the aspects which play a role in the modeling of pile foundations are considered. Hereby the following situations are distuingished:

- Single piles
  - Axial loaded by compression and tension
  - Horizontally loaded
- Pile groups
  - Axial loaded by compression and tension
  - Horizontally loaded
  - Distribution of loads through a foundation structure
  - Pile-Raft foundations

In general there are three different methods distuingished for the analyse of pile foundations:

- Empirical methods
- Simple computational methods based on design charts
- Numerical methods like the boundary element method and finite element methods.

In chapter 3 different numerical models will be considered and there will be checked or the different models can deal with the aspects discussed in this chapter.
As a reflection on Eurocode 7 one can say that for axial loaded piles design methods are prescribed. However the influence of pile-soil-pile interaction on the mobilizaton of the bearing capacity of piles loaded by axial compression is not taken into account by this method. For horizontally loaded piles different aspects are set which has to be checked, but there are no methods prescribed for this like for axially loaded piles.

Chapter 3

Considering different numerical models

In Chapter 2 the aspects which play a role by the design of pile foundations are considered also different calculation methods are taken into account. In this chapter the different models are considered and assessed qualitative on the aspects which were found in chapter 2.

In this thesis three numerical models will be taken into account as described in the introduction. A fourth numerical model will also be briefly considered. This is D-foundation, which is a tool to determine the bearing capacity and settlement of a single pile. This bearing capacity and settlement are needed as input in the other numerical models.

3-1 D-foundation

D-foundation is a numerical model that can determine the vertical bearing capacity and settlement of a foundation pile. These are based on the results of Cone Penetration Tests (CPT), which can be entered in the model. In the model also multiple piles and CPT 's can be entered. The numerical contains four modules (11)

- Design and verification of bearing piles according to the Dutch Eurocode 7 (EC7-NL).
- Design and verification of bearing piles according to the Belgian Eurocode 7 (EC7-BE).
- Design of tension piles according to Dutch Eurocode 7 (EC7-NL).
- Design and verification of shallow foundations according to the Dutch Eurocode 7 (EC7-NL).

For this thesis only the bearing and tension piles according to the Dutch Eurocode will be considered, because these are relevant for this thesis. Furhermore this model cannot determine the horizontal support of the soil against the pile.

3-1-1 Bearing piles according to Eurocode 7

Bearing capacity

With this option the base and shaft bearing capacity of a pile can be defined. This is done by the method which is prescribed in Eurocode 7. This method is based on the results of Cone Penetration Tests (CPT 's) as described in 2-1-1. The bearing capacity of a pile can be reduced by negative skin friction along the pile. This can also be determined by the numerical model. For this one has to define the level where the soft layers, which cause negative skin friction, ends. In the Eurocode the negative skin friction is determined with the slip method. For this calculations the angle of internal friction (φ) and the volumetric weight (γ) of the different layers need to be known. These values can be determined by the numerical model by correlation of the CPT according to different interpretation methods or can be defined manually. The numerical model can work on two different ways.

- Determine the penetration depth for a pile with a defined load. This is for preliminary design.
- Determine the bearing capacity capacity of a pile on a defined pile tip level.

Settlement

The settlement of a single pile can also be determined by D-foundation. These settlement is determined by a relationship between the load and settlement of a pile as described in 2-1-1.

When there are multiple piles entered in the model D-foundation also determines the additional settlement (s_2) of the layers more than four times the pile diameter below the pile tip. This is done according to the method described in Eurocode 7.

3-1-2 Tension piles according to Eurocode 7

The numerical model D-foundation can also determine the tensile bearing capacity of piles. This is done on the method described by Eurocode 7 which is the q_c-method. Hereby all the aspects which has to be taken into account according to Eurocode 7. These aspects are already decribed in section 2-1-1 (single piles) (single piles) and section 2-2-4 (pile groups).

The numerical model D-Foundation does not determine the displacement of a pile loaded by tension. This because Eurocode 7 does not provide a method for for this.

3-2 D-pile

D-pile is a numerical model developed by Deltares which can model the three-dimensional behaviour of single piles and pile groups. The numerical model contains different models which are suitable for different kinds of problems and levels of accuracy vs. efficiency. In general the model can deal with the following options:

- Analysis of all pile types

- Pile sections with different properties
- Options for inclined piles
- Pile head fixed or clamped in the pile cap
- The piles are modelled as linear elastic
- A flexible or stiff pile cap
- Sand and clay layers, drained or undrained with common input properties
- Influence of the pile tip resistance
- Load on the cap by moments, horizontal and vertical forces, rotations and displacements
- Load by horizontal and vertical soil displacements
- Effect of surcharge
- Monotonous increasing loads, load reversal and repeated (cyclic loads)
- Multiple models for the interaction between piles via the soil

In figure 3-1 a lot of different options of the numerical model are shown. The different models in D-pile are briefly described below and more extensive in Appendix 3-2.

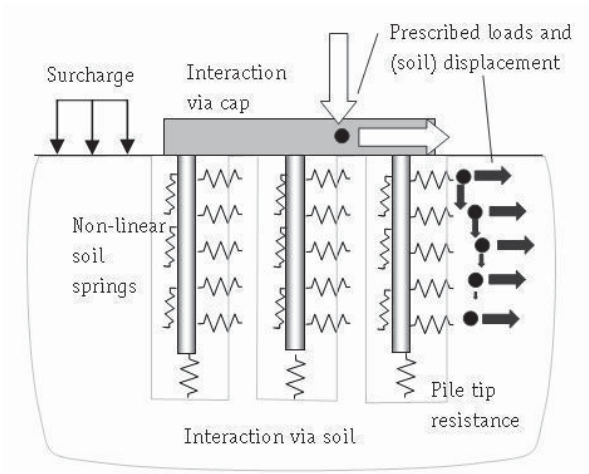


Figure 3-1: Features in D-pile (10, p.17)

- Cap model
  In this model the pile-soil interaction is modelled with elasto-plastic springs. P-Y curves for the horizontal interaction and T-Z curves for the vertical resistance. With this model the negative skin friction on a pile can also be determined by giving a prescribed displacement to the soil. The model does not take into account pile-soil-pile interaction. The model is described more extensively in Appendix C-2-1.

- Poulos model
  In this model the boundary element method as described in chapter 2 is used. The model takes into account pile-soil-pile interaction in the loading direction. Two layers of soil can be inserted and plasticity is not taken into account. See also Appendix C-2-2.

- Plasti-Poulos model
  This model is an extension of the Poulos model which also takes into account local plasticity of the soil around the pile (see also Appendix C-2-3).

- Cap soil interaction model
  This model is an extension of the cap model which also takes into account pile-soil-pile interaction in all directions. In the model two different soil layers can be entered (see also Appendix C-2-4 for a more extensive description).

- Cap layered soil interaction model
  The Cap layered soil interaction model is the most sophisticated model in D-pile, which is an extension of the Cap soil interaction model. In this model more layers can be entered. (see also Appendix C-2-5).

- Dynamic model
  This model is made to model the collision of a ship against a pile or pile cap. Hereby one can define the mass speed and direction of the ship. This model will furthermore not be taken into account, because it is not relevant for this thesis.

3-2-1 results

The result which can be gained from an analysis with D-pile are:

- Bending moments, shear forces and normal forces in the piles.
- Reactions of the soil against the pile.
- Displacement of the piles in all directions
- Displacement and rotations of the pile cap

3-2-2 Conclusion

In table 3-1 the possibilties of the different models described below are summmarized. Here one can see that the Cap layered soil interaction model is the most complete model. This because it contains:

- Pile-soil-pile interaction
- Layered soil
- Plasticity of the soil
- Option to make inclined piles

Table 3-1: Availability of options for the different models in D-Pile (10, p.18)

Model	Soil plasticity	Layered soil	Pile-Soil-Pile interaction	Dynamics	Displacement load	Surface loads	Inclined piles	Max. nr. of piles
Poulos	No	No	Yes (2 layers)*	No	No	No	No	200
Plasti-Poulos	Yes	Yes	Yes (2 layers)*	No	No	No	No	200
Cap	Yes	Yes	No	No	Yes	No	Yes	≈ 75
Cap soil interaction	Yes	Yes	Yes (1 layer)	No	No	Yes	Yes	≈ 25
Cap layered soil interaction	Yes	Yes	Yes (layered)	No	No	No	Yes	≈ 25
Dynamic	Yes	Yes	No	Yes	No	No	Yes	≈ 75

*: The soil system for pile-soil-pile interaction consists of 1 homogenous layer over the length of the pile and one layer below the pile tip.

For this reasons the Cap layered soil interaction model will be taken into account for further research in this thesis.
This method can deal quite well with interactions between piles in the soil. But installation effects like densification of the soil around the pile caused by pile driving cannot be taken into account. However increased shaft and base bearing capacity can be taken into account by taking the right pile class factor (α) according to Eurocode 7.
This model can also deal with the settlement of soil layers more than 4 times the pile diameter below the pile tip as described in 2-2-2. Analysis of foundation structures of category 2 (a stucture which is not flexible and not stiff) as described in 2-2-5 are not possible. This because one cannot define a bending stiffness (EI) of the pile cap. So only the two different cases which were also considered by Poulos and Davis as desribed in 2-2-5 can be taken into account.

3-3 Scia Engineer

3-3-1 general

This numerical model is used by structural engineers to design structures from concrete, steel or wood. It is based on finite elements . In the three-dimensional module different structural elements can be defined:

Materials

In this code different materials can be defined. One can select steel, wood or concrete from a database with material sets with parameters with code from different countries. One can also define general materials.
Scia Engineer distuingishes basic materials as linear elastic for which one has to define:

- Mass

- Modulus of elasticity
- Poisson 's ratio

One can also define advanced materials.

3-3-2 Elements

In Scia Engineer one can define different kind of elements as listed below. These elements are further more described in Appendix C-3-2

- Rod (one-dimensional)
- Plate (two-dimensional)
- Shell (two-dimesional)

Connections

These different elements can be connected to each other in different ways. In a three-dimensional model a connection between two elements has 6 degrees of freedom (Rotation and displacement in the x,y and z direction). There are also ways between this like a flexible connection with a hinge with a certain stiffness. Also restrictions to movement can be set.

Supports

The elements can be supported by different kind of supports too, with different kinds of degree of freedom. The different connections present in Scia Engineer are listed below and described more extensive in Appendix C-3-2

- Flexibel
- Only rigid in pressure
- Only flexible in pressure
- Non-linear
- Friction:

When one wants to model a beam or plate supported by soil this can be done with a modululus of subgrade reaction. This can give support in horizontal or vertical direction. This represents the resistance of the soil against the element.

Loads

In this numerical model a lot of different loads can de defined on the structure:

- Own weight
- Force and bending moment
- Thermal loads
- Climatic loads
- Displacement of specific points

All these loads can be combined to different load combinations.

3-3-3 Pile foundation

When one wants to model a pile foundation in Scia Engineer one can model the pile as a column with a defined cross section and elasticity modulus (E). The cap on top can be modelled as a plate with a defined thickness and stiffness. This is shown in figure 3-2.

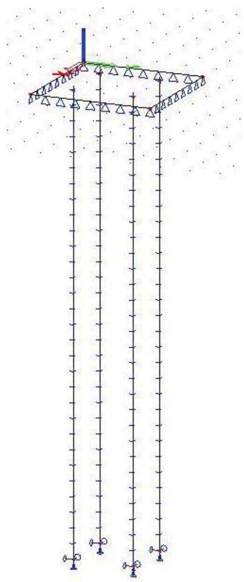


Figure 3-2: Example of a capped pile foundation in Scia Engineer

Vertical support

To model the tip resistance of the pile one can model a spring below the pile tip. The positive skin friction can be modelled with springs along the shaft. These springs can be derived from the load-settlement behaviour as described in 2-1-1

When one wants to model a raft foundation this can be done by placing a subgrade reaction below the plate, like shown in figure 3-2. Normally this is not done for pile foundations.

Horizontal support

The horizontal support of the piles can also be represented by springs. How these springs or subgrade reactions can be determined is described in 2-1-2. One a linear subgrade reaction is chosen a linear spring can be inserted. When one wants to insert a non-linear curve like a P-Y curve a non-linear spring has to be implemented.

3-3-4 Results

The results which can be gained from an analysis with Scia Engineer with respect to a piled cap are:

- Bending moments, shear forces and normal forces in the piles.
- Displacement and deformation of the piles in all directions
- Bending moments, shear forces and normal forces in the cap
- Displacement and deformations of the pile cap.

3-3-5 Conclusion

Scia Engineer gives good insight in the structual behaviour of the foundation. Also a lot of different loads and load combinations can be. made. For single piles or piles with a large centre-to-centre distance the behaviour of the soil around the pile can be modelled with springs. The use of non-linear springs is not always likely by stuctural engineers because it can give conflicts with their automatically generated load combinations. Choosing a stiffness by hand will make te process interactive.
All kind of structures named in 2-2-5 can be modelled in Scia Engineer, because one can define the bending stiffness (EI) of all structural elements.
The interaction pile-soil-pile is not taken into account by Scia Engineer. This has to be done by adjusting the subgrade reaction with for example the interaction factors of Poulos (see 2-2).

3-4 Plaxis 3D

This numerical model is used by geotechnical engineers. The code is based on the finite element method. There are two- and three dimensional versions of the code. In this thesis the three-dimensional version is taken into account.

3-4-1 Material models

In Plaxis different kind of constitutive models can be used to describe the behaviour of the soil and other elements in the model. The models that can be relevant for this thesis are briefly described below and described more extensive in Appendix C-4-1.

- Linear Elastic
  This model is used for the structural elements in a model, like concrete or steel beams or plates. In this model there is no failure criterium included.

- Mohr-Coulomb
  This is a relative simple model which contains linear elastic perfectly-plastic soil hehaviour. The model represents a first order approximation of the behaviour of soil or rock. Plaxis recommend to use this model for a preliminary analysis of a problem.

- Hardening Soil
  This is a more advanced model than the Mohr-Coulomb model. The stiffness of the soil is described more accuratly by using different stiffness parameters. In contrast to the Mohr-Coulomb model this model takes into account stress-dependency of the stiffness. This means that the stiffness increases with pressure.

- Hardening Soil with small strain stiffness
  This model is a modifation of the Hardening Soil model. This model takes into account increased stiffness of soils at small strains. This will make that this model gives more reliable displacements. The parameters for the harding soil with small-strain model are the same as for the hardening soil model, but it needs two additional parameters. This model is recommended by Plaxis as the best model to model pile foundations.

- Soft Soil
  A model which is meant to model the primairy compression of primairy compression of clay type soils. The Hardening Soil model has general better modelling capacities than this model. This model is especially good for the modelling of soft soils.

- Soft Soil Creep
  The soft Soil Creep model is an extension of the soft soil model. This model can also take into account viscous effects like creep and stress relaxation. These problems are mainly dominant in soft soils like clay, silts and peat. This model is primarily developed to model the settlement of foundations, embankments etc.

3-4-2 structural elements

In Plaxis 3D various structural elements can be defined. These are described in Appendix C-4-2. The important elements with repect to this research are briefly described below.

- Beams
  Like in Scia Engineer in Plaxis one can also define beams. These are one-dimensional elements with the properties of a certain cross section. They are modelled as linear elastic materials.

- Plates
  The plates are modelled as two-dimensional elements for which one has to define the thickness. They are also modelled as a linear elastic material.

- Anchors In Plaxis two different kind of anchors can be modelled.

  - Node-to-node anchors
    These anchors can link different elements. There stiffness is defined by the Young 's modulus (E) and the cross sectional area (A). Elastic-plastic behaviour can be taken into account by imposing a maximum force (F_{max}). These anchors can also be prestressed.
  - Fixed-end anchors These anchors are like node-to-node anchors but they are fixed one side. These can be used to model struts.

- Volume piles
  When one wants to make volume piles one creates volume elements which has the material properties of the piles. The interaction between the soil and pile is represented by interface elements. This is a lot of work especially for a big pile group. Also it is very hard to get all the forces out of the volume pile. This because the pile contains out of a lot of elements. The volume pile will therefore not furtermore be taken into account.

- Embedded piles
  To make the modelling of piles more easy Plaxis has developed the embedded pile. In this option the pile is assumed as a line element (see figure 3-3). This element can cross through all the soil elements at any arbitrary position and with an arbitrary inclination. To model the bearing capacity of the pile two special interfaces are made for the pile shaft and base.

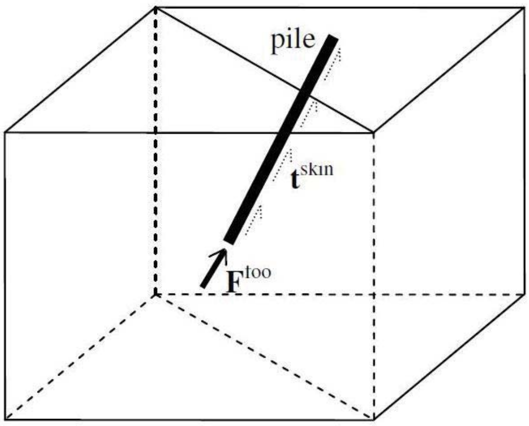


Figure 3-3: Schematization of single embedded pile in a soil mesh (32, p.377)

The pile soil contact can be respresented by the so called skin traction **t** in kN/m. This is the force in the pile (kN) per circumference pile (m). This skin traction is activated by the relative displacement between the pile and the soil ($\mathbf{u}_{rel}$). In Plaxis there are different traction models:

  - A constant or linear relationship between the allowable traction and the depth.
  - Multilinear diagram by means of a set of values of allowable traction values and the corresponding depth. This option can be used to model non-linear skin forces profile which may obtain from pile tests.
  - Layer dependent traction relates the allowable traction with the adjacent soil layers. Here depends the traction on the normall stresses along the pile.

These models are describe more extensive by Septanika et al. (32).

The interaction at the foot of the pile is described by a special-purposed spring element to represent the foot stiffness against the relative movement at the foot of the pile. This behaviour is modelled as linear elastic perfectly-plastic. The maximum allowable force at the foot of the pile has to be defined.

In Appendix C-4-2 a validation example of the embedded pile is shown.

3-4-3 Interface elements

To model the specific behaviour between soil and structure there are special interface elements in Plaxis. They can for example take into account the collaboration between the soil and a wall. This differs between a smouth and rough wall.

3-4-4 Results

In Plaxis 3D one can get information about the behaviour of the structural elements and the soil. The information one can get from the model is:

- Bending moments, shear forces and normal forces in the piles.
- Displacement of the piles in all directions
- Bending moments, shear forces and normal forces in the cap
- Displacement of the pile cap.
- Stresses in all structural elements
- Stresses and strains in the soil
- (Time-dependent) settlement of the soil.
- Stresses and strains in the soil due to installation effects are not visible. This because the installation process is very hard to model.

3-4-5 conclusion

In Plaxis 3D one can also model different kind of structures of all categories as defined in 2-2-5. The following interactions can be taken into account:

- Plate-soil interaction
- Pile-soil interaction
- Pile-soil-pile interaction
- Plate-soil-pile interaction

Further more time-dependent effects like creep can be taken into account.

A con of the numerical model is that the different models need a lot of parameters which may be hard to determine and or need a lot of soil investigation and experience. Also it is not possible to take into account installation effects, like densification of the soil around the pile.

3-5 Conclusion

The three different models which will be considered in this investigation all have their pros and cons. In table 3-2 an overview is given how this models can deal with the different aspects of pile foundations as discussed in chapter 2. The numerical model D-Foundation is not taken into account in this comparison because it is only used as a tool to determine the bearing capacity and settlement which is needed as input for the other models. When the numerical model can deal with the aspect a '+' is given. When a model can not deal with a certain problem a '-' is given. When it is possible to deal with the aspect by an extra workaround a '+/-' is given.

For single piles follows that:

- The base bearing capacity has to be given as input in all three different models.
- In Scia Engineer one also has to define the shaft bearing capacity. In D-Pile this is determined with the undrained shear strength or cone resistance which has to given as input. In Plaxis the shaft bearing capacity can be inserted manually or it can be determined by the model based on the angle of internal friction, cohesion and neutral soil pressure.
- In D-Pile one has to define the amount of settlement needed to mobilize the base bearing capacity. The amount of settlement needed to mobilize the shaft bearing capacity is deternined by the input parameters. In Scia Engineer all these relations has to be defined. In Plaxis the settlement of the pile is governed by the input parameters.
- The horizontal support of the soil against the pile is in D-Pile and Scia determined by the given input parameters. In Scia one has to define the horizontal subgrade reaction manually.

For pile groups where the interaction between the cap and soil is not taken into account one has know that:

- The failure of the total soil mass around the pile can only be taken into account by Plaxis.
- The difference between the settlement of a single piles and piles in groups can be taken into account by D-Pile and Plaxis, because they contains finite elements meshes which respresents the total soil mass.
- For piles loaded by tension the failure mechanism uplift of the total soil mass can only be taken into account by Plaxis. Also the decrease in bearing capacity due to destressing of the soil of a pile group loaded by tension can only be taken into account by Plaxis.
- In the numerical model D-Pile the bending stiffness of the cap is not taken into account so this model cannot take into account the load distribution through the cap.

Table 3-2: Assessment of the different numerical models on different aspects according to pile foundations

	D-Pile	Scia Engineer 3D	Plaxis 3D
Single Pile			
Axially loaded by compression			
Bearing capacity	+/-	+/-	+/-
Settlement	+/-	+/-	+
Axially loaded by tension			
Tensile bearing capacity	+	+/-	+
Axial spring stiffness	+	+/-	+
Laterally loaded			
Resistance	+	+/-	+
Pile group (cap of ground)			
Axially loaded by compression			
Bearing capacity	-	-	+
Settlement	+	+/-	+
Axially loaded by tension			
Tensile bearing capacity	+/-	+/-	+
Axial spring stiffness	+/-	+/-	+
Load distribution in cap	-	+	+
Laterally loaded			
Resistance	+	+/-	+
Pile-raft foundation			
Cap-soil interaction	-	+	+
Cap-soil-pile interaction	-	-	+

- The influence of closely spaced piles on the lateral resistance is taken into account by the models D-Pile and Plaxis. In Scia one has to adjust the horizontal subgrade reaction manually for closely spaced piles.

For pile-raft foundations follows that:

- The numerical model D-Pile cannot take into account the support of the soil against the cap. In Scia Engineer one can model this by putting a vertical subgrade reaction below the plate. Plaxis can also take into account this effect.

- The cap-soil-pile interaction can only be taken into account by Plaxis.

As one can see none of the numerical models is perfect and can deal with all phenomena related to (capped) pile groups. From this analysis Plaxis seems to be the 'best' numerical model. But in the comparison in table 3-2 things that are not taken into account are:

- What parameters are needed as input in the model and how can they be derived (field test lab test etc.).
- The amount of work it takes to build such a model.
- How user friendly the model is and if it requires much expierence to work with.

The different constitutive models in Plaxis makes that one needs to one needs an 'expierenced' modeller for it and enough soil data to get reliable results. In the following part of this thesis research will be done to the three different numerical models described. Hereby the aspects described in the literature study will be taken into account.

Chapter 4

Investigation of numerical models for single piles

In this chapter the numerical models as discussed in chapter 3 will be investigated on how they deal with the behaviour of single piles. After this in Chapter 5 the investigation will be extended for pile groups. For single piles the aspects that will be taken into account are:

- How does the pile behave under axial compression loading?
- How does the pile behave under axial tension loading?
- How does the pile behave under external horizontal loading at the pile head?

The goal is to see what the differences are in results between the different models and what causes these differences.

As already mentioned in the introduction the reference case for this investigation is the foundation structure of a power pylon. Therefore the investigation of these different numerical models the soil profile from this project will be taken. This soil profile is discussed in section 4-1.

For the investigation of these models a prefabricated concrete pile is used, this because it is one of the most used pile types in the Dutch building practice. The pile tip level is chosen at the same level as in the reference project. In figure 4-1 the pile is shown. The pile top level is app. 2. m. below surface level, because the foundation structure is buried below surface level. During this investigation the self weight of the pile will not be taken into account. This because this investigation is about the mechanisms in the different models.

In designs in the building practice one has to consider the limit state GEO for the ultimate limte state and the serviceability limte state. For this investigation only the serviceability limit (SLS) state will be taken into account.

The investigation of the numerical models will start with D-Foundation. This because this model is used to determine the bearing capacity (in compression and tension) and the load-settlement behaviour when the pile is loaded by compression. This data will be needed partley as input for the other models and also as benchmark to verify the outcome of the different models. Hereafter the models D-Pile, Scia and Plaxis will be taken into account.

4-1 Soil stratification and characteristics

Heijmans has made foundation structures for power pylons in the western part of the Netherlands near Bleiswijk between Rotterdam and The Hague. This region is known from the soft top layers. For the foundation structure considered the following soil investigation data is available:

- 2 Cone Penetration Tests (CPT) (see figures D-38- D-41 in Appendix D-7)
- 1 Ball Penetration Test (BPT), this is a special kind of CPT which can also measure the undrained shear strength (C_u) (see figure D-42 in Appendix D-7).
- Mechanical drilling from which 16 disturbed and 9 undisturbed samples are taken. From this samples the soil type is determined and the volumetric weight of the soil is determined.

In table 4-1 the different layers of this profile are given and their general soil properties are given. The derivation of these parameters is done by correlation with table 2b from Eurocode 7 (7). In Appendix D-1 the derivation of the different parameters is described. The water table in this profile is 1.10 m below ground level so at -5.88 m NAP.

Table 4-1: General soil properties for the considered soil profile near Bleiswijk

	Soil type	**Middle [m. NAP]**	q_c $[MPa]$	γ_{dry} $[kN/m^3]$	γ_{sat} $[kN/m^3]$	φ' [°]	c' $[kN/m^2]$	c_u $[kN/m^2]$	$E'_{50,rep,\sigma'_v=100}$ $[MPa]$
1	Silty clay	-4.78	0.5	17	17	20	5	35	1.5
2	Peat	-11.00	0.7	12	12	15	2.5	45	0.5
3	Sand medium packed	-14.00	8.0	17	19	31	0	-	30
4	Sand dense packed	-20.00	15.0	18	20	31	0	-	30
5	clay dense	-38.50	5.0	21	21	22.5	13	100	4

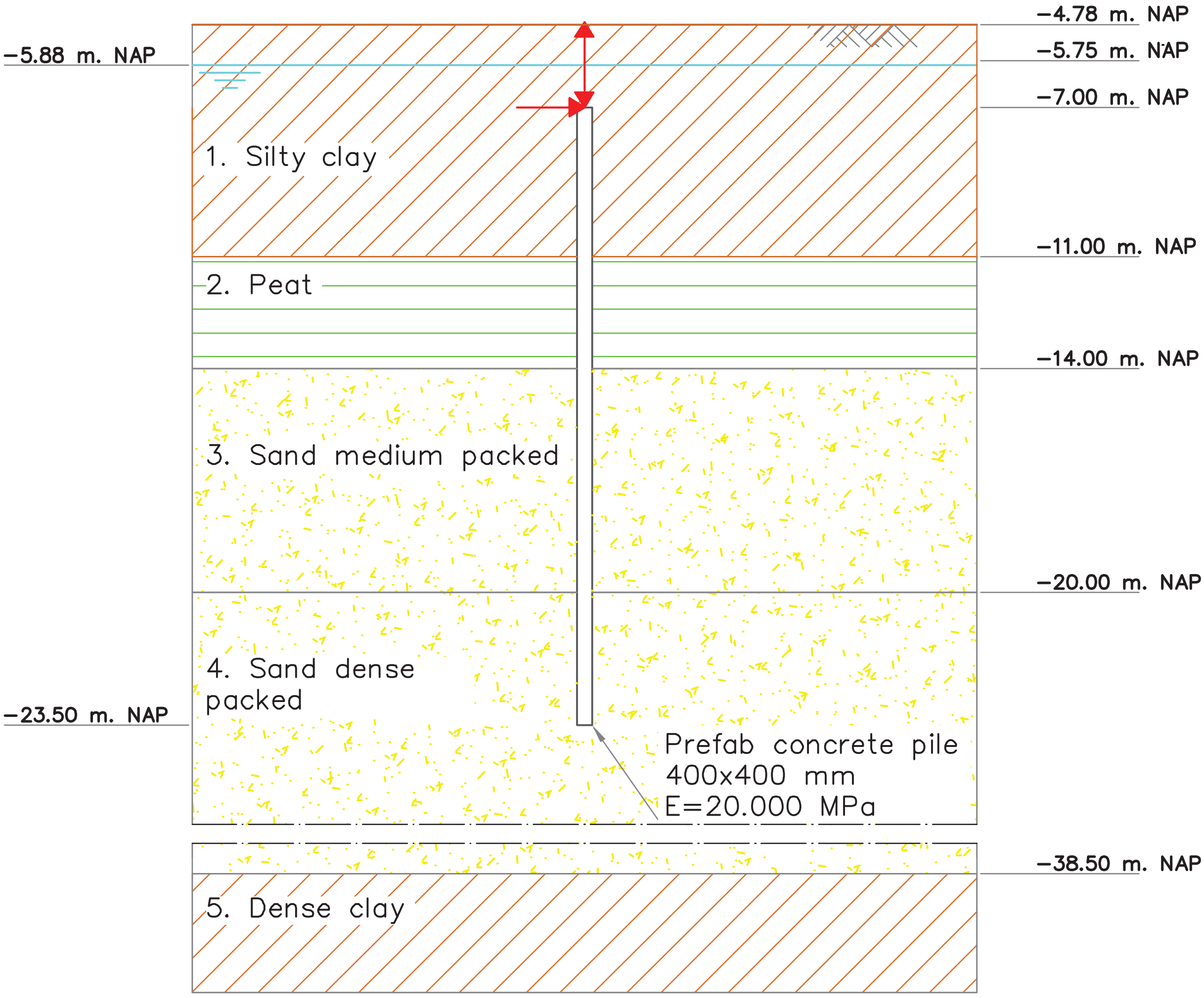


Figure 4-1: Considered pile for the investigation of the numerical models

4-2 D-foundation

This numerical model is used to determine the bearing capacity for both loading by compression and tension.

4-2-1 Bearing capacity

In table 4-2 the bearing capacity of the pile is given in both the serviceability state (SLS) as the limit state GEO also know as the ultimate limit state. For the calculation of this bearing capacity 2 CPT 's are used. In Appendix D-2-1 the formula 's used in this numerical model are explained.

D-foundation also determines the load caused by negative skin friction on the pile, which is caused by settlement of soft layers. This is done with the slip method which is explained in in Appendix D-2-1. The force on the pile cause by negative skin friction is 58 kN.

Table 4-2: The bearing capacity of shaft and base at the serviceability limit state (SLS) and the ultimate limit state GEO determined by D-Foundations

Results		SLS	GEO
Shaft bearing capacity	$R_{s;d}$ $[kN]$	1154	962
Base bearing capacity	$R_{b;d}$ $[kN]$	1050	875
Total bearing capacity	$R_{c;d}$ $[kN]$	2204	1837

4-2-2 Settlement

D-Foundation determines the settlement of the pile by using the emperical relationship as given in Eurocode 7 (7) between the bearing capacity and the settlement of the pile base. In table 4-3 the relationship between the pile base settlement (s_b) and base and shaft bearing capacity ($R_{b;d}$ and $R_{s;d}$) is given. Furthermore the model has also determined the elastic deformation of the pile itself. In figure 4-3 in paragraph 4-6-1 the load-settlement behaviour of the pile top is shown.

Table 4-3: Relation between the pile bearing capacity (pile and base) and the pile base settlement (s_b) in the serviceability limit state determined by D-Foundation

%	$R_{b;d}$[kN]	s_b $[mm]$	%	$R_{s;d}$ $[kN]$	s_b $[mm]$
0	0	0	0	0	0
25	263	-1.12	25	299	-0.27
50	525	-6.48	50	577	-1.43
75	788	-16.8	75	866	-3.70
100	1050	-48.00	100	1154	-10.60

4-2-3 Tensional bearing capacity

The numerical model D-foundation is also used to determine the tensional bearing capacity of the pile. For a single pile pulling the pile out of the soil mass is the guiding failure mechanism. This leads to a tensional bearing capacity ($R_{t;d}$) of 826 kN in the serviceability state and 612 kN in the limit state GEO. In Appendix D-2-3 is explained how this bearing capacity is determined in this model. For this bearing capacity calculation is taken into account that there are no alternating loads. When there are alternating loads this will lead to a reduction of the tensional bearing capacity of the pile.

4-3 D-pile

As described in section 3-2 D-Pile has different models. In this research only two models will be taken into account:

- The Cap model meant for single piles or piles at a large distance from eachother. It is a good model to understand the behaviour of single piles.

- The Cap layered soil interaction model, which also takes into account pile-soil-pile interaction. This is the most sophisticated model in this numerical model.

In table D-12 and table D-13 in Appendix D-3. the parameters for the D-Pile model are given. These are partley directly taken from the general soil properties as shown in table 4-1 and partley specially derived. An explanation of these derivations can be found in Appendix D-3. In these tables the Young 's modulus and poisson 's ratio are only for the Cap layered soil interaction model.

4-3-1 Bearing capacity

The base bearing capacity in this model has to be defined by the user so the result of the D-foundation calculations can be used for this. The shaft bearing capacity in this model is based on the skin friction (f) which for sand layers can be derived with equation 4-1, this equation is based on the pile class factor (α) and the cone resistance (q_c).

$$f = \alpha \cdot q_c \tag{4-1}$$

For clay layers the skin friction (f) is determined with equation 4-2, so it depends on the undrained shear strength (C_u).

$$f = \alpha \cdot C_u \tag{4-2}$$

To get the total shaft bearing capacity (T_{max}) of a layer the model mulitplies the skin friction with the circumference ($O_{s;\Delta L;gem}$) and integrates it over the height of the layer. So in fact this is similar to the determination of the shaft bearing capacity in D-Foundation. In the calculation of the bearing capacity is taken into account that only shaft bearing capacity is derived from the sand layers. Therefor the pile class factor (α) is set to the minimum value (0,000001) for the cohesive layers. Furhermore the model does not take into account factors of safety and correlation factors. So when one wants to approach the bearing capacity according to Eurocode 7 one also has to take into account this factors by adjusting the cone resistance (q_c) or pile class factor (α_s). The effect of this is shown in Appendix D-3-1. For this bearing capacity there is no difference between the Cap model and the Cap layered soil interaction model.

4-3-2 Settlement

The load-settlement behaviour of the pile in the Cap model is determined by a base spring and T-Z springs along the shaft. The base spring has to be added manually by giving a load-settlement relationship. Therefore the outcome of D-foundation is used which is shown in table 4-3. The T-Z curves give a linear relationship between the mobilized shaft friction and the shaft friction (see also figure C-3 in Appendix C-2-1). The stiffness of the spring is defined by dz at 100% which is 0.0025 m for the sand layers. This is the recommended value by the manual of D-Pile.

When the Cap layered soil interaction model is used the elastic medium around the pile also causes additional settlement as shown in Appendix D-3-2. Therefore the Young 's modulus used in the model is not the Young 's modulus at 50% strain of a drained triaxial test is taken, but a Young 's modulus which is 10 times stiffer.

4-3-3 Tensile bearing capacity

The tensile bearing capacity in this model is defined with a similar way as the shaft bearing capacity for a pile loaded by compression. The base bearing capacity works only for loading by compression. So the tensile bearing capacity is determined by the q_c method. To approach the shaft bearing capacity determined with D-foundation as best as possible one has to adjust the pile class factor (α) in the sand layers from 0.01, which is the value for concrete piles loaded by compression, to 0.007 which is the value for concrete piles loaded by tension. Furthermore one also has to to adjust the cone resistance as best as possible and taken into account safety factors. A description of this is given in Appendix D-3-4.

4-3-4 Axial spring stiffness

The axial spring stiffness in the Cap model is only determined by the behaviour of the T-Z curves and the elastic extension of the pile itself. To get a realistic spring stiffness it is good to approach the bearing capacity as precise as possible, because the T-Z curves are also dependent on this bearing capacity.

When the Cap layered soil interaction model is used the spring stiffness is also influenced by the elastic medium around the pile. When the stiffness is made 10 times stiffer, as already discussed for pile loaded by compression, the results for a single pile are not influenced significant as also shown in Appendix D-3-4.

4-3-5 Horizontal resistance

In the Cap model the horizontal resistance of the soil against the pile is determined with P-Y curves. These P-Y curves are generated by the numerical model on the manner as prescribed by the API (8). Hereby a difference is made between P-Y curves for sand and clay. How these P-Y curves are determined is described in Appendix D-3-5. The main parameters which determines the properties of the P-Y curves are listed below.

- P-Y curves for clay depends on the undrained shear stength (C_u), the effective volumetric weight of the soil (γ'), the depth below surface (H) and the strain at 50% of a an undrained triaxial test (ϵ_{50})
- P-Y curves for sand depends on the depth below surface, the effective volumetric weight (γ') and the angle of internal friction (φ). All other parameters are derived from the angle of internal friction.

In the Cap layered soil interaction model the elastic medium between the pile also has infuence on the horizontal resistance of the soil against the pile. This because the P-Y curves are in this model not fixed, but placed against an elastic medium. As shown in Appendix D-3-5 multiplying the normal Young 's modulus with a factor 10 as already stated for vertical piles gives significant less deflection of the pile in comparison with the situation with the original Young 's modulus. The deflection in the situation with a ten times stiffer Young 's modulus is more in line with the results of the cap model.

The example pile considered is placed ca. 2 m. below the surface level. In Appendix D-3-5 is shown that it has a quite a big effect on the P-Y curves or an excavation is modeled (surface level at

-7 m. NAP) or that original surface level (-4.78 m. NAP) is modeled. In the last case the reaction of the soil against the pile is significant stiffer.

4-4 Scia Engineer

In Scia Engineer the pile is modeled as a column and the behaviour of the soil against the pile is modeled by springs. So these springs take into account both the bearing capacity and the load-displacement behaviour of the pile. The goal of these springs is to simulate the pile-soil interaction.

4-4-1 Vertical support

For the vertical behaviour non-linear springs are defined. These springs are placed at discrete nodes along the pile shaft on the part of the pile where positive shaft friction works (see figure 4-2a). The spring placed on each individual node along the shaft is shown in figure 4-2b. For the compression part the spring is defined by taking the relationship between the shaft bearing capacity and pile base displacement as shown in table 4-3 and divide this by the amount of nodes.

For tension piles is stated in CUR 77 (1) that the mobilization of shaft friction works on the similar way as for shaft bearing in compression. Therefore the load-displacement behaviour for a pile loaded by tension is divided by the amount of nodes and added to the shaft spring.

So in fact the spring which defines the mobilization of the shaft bearing capacity is replaced by 10 springs which are a factor 10 less stiff.

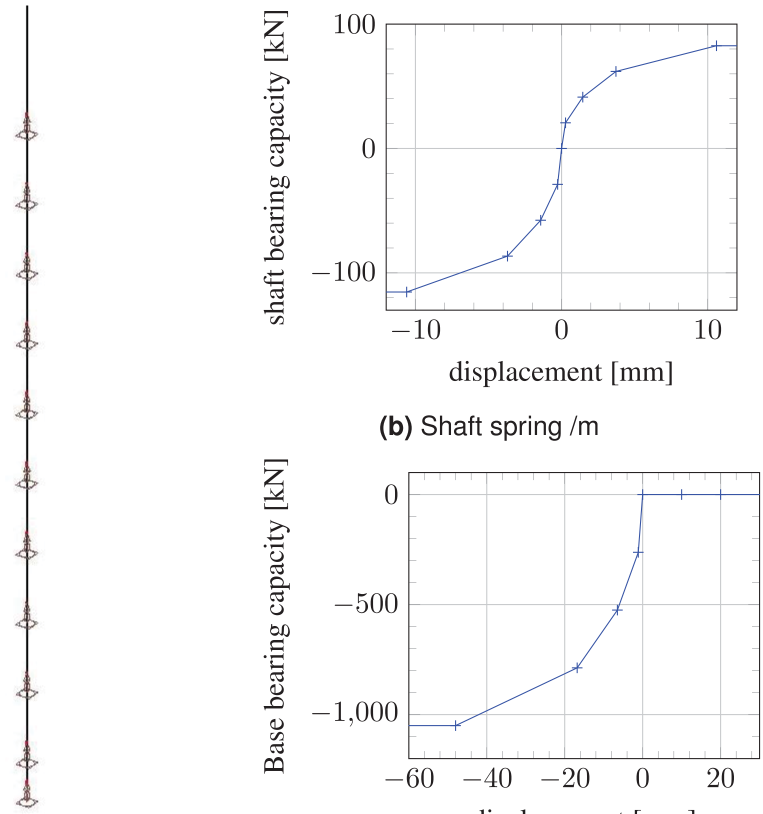
The base spring is directly taken from the results of D-foundation as shown in table 4-3 and is placed at the pile tip. A detailled explanation of the derivation of these different springs can be found in Appendix D-4

4-4-2 Horizontal support

The horizontal support of the soil against the pile is modeled with the method of Ménard. This method gives a linear subgrade reaction against the pile for every defined layer. This subgrade reaction depends on the cone resistance, the soil type and the diameter of the pile (see also Appendix A-2-2). The derivation of these subgrade reactions are described extensively in Appendix D-4-5. Here also the values for the horizontal subgrade reaction as imported in Scia are given.

4-5 Plaxis 3D

In Plaxis different constitutive models can be used to model the behaviour of soil. In this thesis the Hardening Soil model with small strain stiffness (HS small) will be used, because according to the material model of Plaxis (14) it is the most suitable model for foundations. This because the model containes stress-depency of stiffness, a hyperbolc stress-strain relationship instead of a bi-linear relationship in the Mohr-Coulomb model, difference between virgin and unloading / reloading stiffness



(a) Discrete nodes with vertical supports

(b) Shaft spring /m

(c) Base spring

Figure 4-2: Vertical supports and associated springs in Scia Engineer

and small strain stiffness for the stiff behaviour at small displacements. The benefit of this small strain stiffness is that the soil a bit further away from the pile acts stiffer, because the strains are very small. This will ensure that the total displacements in the model will get as realistic possible.

In table D-21 and D-22 in Appendix D-5-1 the parameters for the different layers are given. These parameters are partly the general soil properties as shown in table 4-1 and the other parameters are derived using CUR 2003-7 (3) and the Material manual of Plaxis (14). A detailled explanation of the parameter determination can be found in Appendix D-5. The upper peat layer is the only layer modelled with another model (Mohr-Coulomb). This because this layer is not in contact with the pile and does not influence the stress-strain behaviour of the other layers, because it is on top.

In Plaxis the there are different drainage types (drained and undrained), which are explained in Appendix D-5-1. Sand layers are always set to drained, because excess pore pressure can dissipate very fast from these permeable layers. Cohesive layers can act undrained in the case of short term loading. This because the excess pore pressure has no time to dissipate, due to the low permeability of the soil. In Appendix D-5-8 one can clearly see that when the cohesive layers are set to undrained the horizontal deflection of an horizontally loaded pile is much larger. Because in the reference project of the power pylon the loading is expected to be long term in the rest of this thesis all the drainage type of all layers will be set to drained. In general one can say that undrained soil acts stiffer, but is less strong and drained soil acts less stiff, but is stronger.

4-5-1 Phasing

Plaxis calculations contains different calculation stages. These are extensively described in Appendix D-5-2, but are shortly described below.

- Initial phase: All soil layers and water conditions are defined.
- Phase 1 Excavation: To make the foundation structure an excavation till -7 m. NAP will be made.
- Phase 2 Pile installation: The embedded pile will be activated in the model.
- Loading Phases: The pile will be loaded axially (compression and tension) and horizontally. All this phases will start from phase 2 to prevent that results are influenced by a previous loading phase.

4-5-2 Bearing capacity

In the embedded pile module the bearing capacity has to be given as input. The base resistance as determined with D-foundation is given as input (see table 4-2). For the shaft bearing capacity there are different options available as described in section C-4-2. The skin resistance is set to multi-linear. In the bearing capacity calculation of the pile is stated that the pile derives shaft bearing from the part of the pile where positive skin friction is taken into account. Therefore the skin friction on the embedded pile (T_{skin}) can be determined by equation 4-3.

$$T_{skin} = \frac{R_{s;d}}{\Delta L} \tag{4-3}$$

Where:

$R_{s;d}$	$[kN]$	Shaft bearing capacity
ΔL	$[m]$	Part of the pile over which the positive skin friction is taken into account

The shaft bearing capacity of the pile can be simulated even more precisely when one divides the skin resistance in multiple parts and look with an hand calculation how the shaft bearing capacity is developed over the pile length.

4-5-3 Load-settlement behaviour

For the embedded pile the load-settlement behaviour along the pile shaft is guided by special interface elements. In these elements the mobilization of shaft bearing capacity depends on the relative displacement between the pile and soil. So the load-settlement behaviour along the pile shaft is guided by the behaviour of the constitutive model. The load-settlement behaviour at the pile base is governed by a special purposed spring element and also depends on the relative displacement between the pile and soil. These elements are generated by the numerical model itself and described in more detail in Appendix D-5-5.

The load-displacement behaviour for the first 50% of the pile capacity is in good agreement with the behaviour of D-Foundations. The biggest part op the load is than carried by shaft bearing. This seems to be good however it is remarkable. This because the installation effects are not taken into account in this model. According to the relations between the shaft bearing capacity and pile settlement prescribed in Eurocode 7 as also shown in figure A-5 b in Appendix A-1-3 for the mobilization of the bearing capacity of a bored pile is more settlement needed than for the mobilization of the shaft bearing capacity of a ground displacement pile. This may be caused by the fact that the stiffness parameters choosen are a bit progressive which make the relative settlement between the pile and soil low and a bit conservative assumptions in the relationships prescribed in Eurocode 7 on the other hand.

When the pile is loaded till its maximum capacity and the foot resistance is also mobilized the settlement in Plaxis 3D is significant higher. Reasons for this are:

- The soil parameters derived from Eurocode 7 are conservative. When a higher strength is entered this will lead to less soil failure around the pile foot and less relative settlement.
- The installation effects of a prefabricated concrete pile are not taken into account by Plaxis. So the additional pressure caused by the soil displacement is not present in the model. A way to simulate this is by applying a volume strain around the pile tip.

4-5-4 Tensional bearing capacity

The tensile bearing capacity of the pile is defined by the shaft friction. The embedded pile element works in a certain way that skin friction works for both pressure and tensional loading and the foot resistance only works for loading by pressure. This means that tensional bearing capacity is equal to the defined shaft bearing capacity of the pile. To approximate the tensional bearing capacity as determined with D-foundation the skin resistance of the embedded pile as given in table D-23 has to be adjusted. The shaft bearing capacity of a bearing pile is defined by using a pile class factor (α_s) of 0.01. For tension piles one has to use a pile class factor (α_t) of 0.007. So according to these factors the shaft bearing capacity of a tension pile is 70% of a pile loaded by pressure. Therefore the skin friction is adjusted on the way shown in equation 4-4.

$$T_{skin} = \frac{\alpha_t}{\alpha_s} \cdot T_{skin;compression} \tag{4-4}$$

4-5-5 Axial spring stiffness

As already mentioned in subsection D-5-5 The load settlement behaviour is governed by special interface springs in the embedded pile element. When one adjust the shaft bearing capacity this does not have effect at the first part of the tensile bearing capacity. But for a tension load higher than 75% of the tensional capacity one can clearly see that this has effect on the load-displacement behaviour.

4-5-6 Horizontal resistance

The horizontal resistance of the soil against the pile is governed by the interaction between the pile and soil. Because of the applied horizontal load at the top, the soil will give a horizontal reaction against

the pile. This will cause deformation of the soil elements according to the constitutive behaviour of the applied soil model. These soil elements can sustain stress untill the maximum stress-strain relationship is reached.
However the embedded piles are imported as one-dimensional elements the elements in the cross section area of the pile behaves linear elastic, according to the defined Young 's modulus of the pile. So the soil cannot yield in this area.

For the embedded pile there is no slip between the shaft and soil. This is no problem for piles which has a rough shaft surface. For piles with a smooth surface (like prefabricated concrete piles) This will gives an overestimation of the lateral resistance especially for large horizontal displacement as shown by Dao (16), so the result of an prefabricated embbed pile are only reliable for small strains.

4-6 Comparison of the result of the different models

4-6-1 Loaded by axial compression

The load-settlement behaviour of the single pile in the different models is included in Appendix D. In this section the behviour of the pile in all different models is compared with the load-settlement behaviour of a pile in D-foundation. This because this numerical model gives the load-settlement behaviour according to Eurocode 7 and is therefore used as benchmark to verify the other models. In figure 4-3 the load-settlement behaviour of all different models is shown. Hereby the following assumptions are made:

- Only D-foundations distracts the load caused by negative skin friction from the results. For the other models this is subtracted manually from the load on top. This because these models do not take into account negative skin friction.
- For D-Pile the results of the Cap interaction model are given. This because this model will also be used in the next steps.

In figure 4-3 one can see that the results of Scia Engineer and D-Pile are in good agreement with the load settlement behaviour given by D-foundation. For the results of Scia Engineer this is a logical result of the fact that the results of load-settlement relations in D-Foundation are used to create the springs in Scia Engineer. That the results of D-Pile give good agreement is also logical. The load-transfer method at the shaft is a bit different because they are linear (T-Z curves) in stead on non-linear in Scia. However the load-displacement relationships at the pile base are exactly the same.

The load-settlement behaviour of Plaxis is in good agreement with the other models for the first 50% of the pile capacity. In this case the major part of the mobilized bearing capacity is shaft bearing capacity. When the pile base is loaded more the settlement increases significantly. The strength parameters of the soil around the pile determined with Eurocode 7 are conservative. When one adjust these the stiffness increases. The main reason that the pile base acts less stiff is that the installation effects of the pile are not taken into account in Plaxis. These cause additional pressure in the soil which makes the pile base behave stiffer. The other numerical models use empirical relationship based on field test. So they take into account this densification around the pile base. This effect is also visible

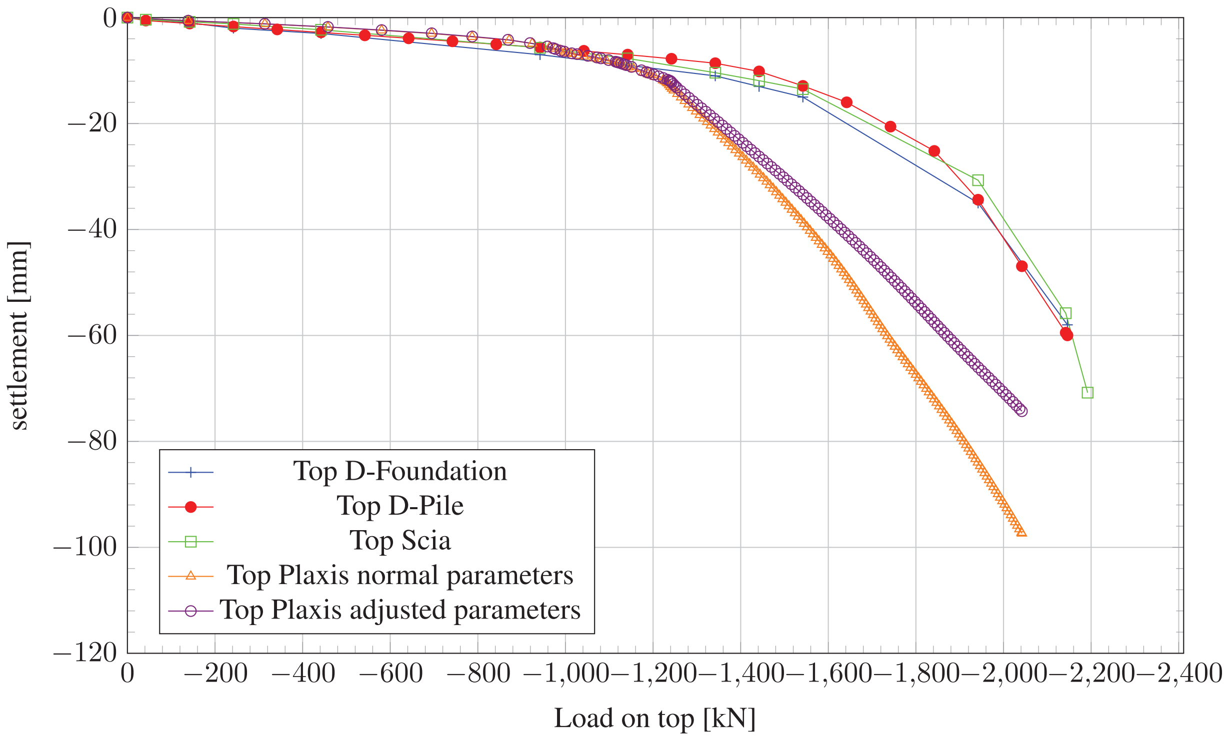


Figure 4-3: Load-settlement behaviour top pile in the different models

in the load-settlement relation for the pile base as given in Eurocode 7 (see figure A-5 in Appendix A-1-3). Here one can see that ground displacement piles act significantly stiffer than bored piles. For piles which determine their bearing capacity only from the base the deviation of the results will be even larger and the pile will behave weaker.

4-6-2 Loaded by an axial tension force

The load-displacement behaviour of the pile loaded by tension in the different models is shown in figure 4-4. Hereby the spring stiffness according CUR 77 (1) is used as benchmark. This spring stiffness contains the elastic extension of the pile and the mobilization of the skin friction. The derivation of this spring stiffness is shown in Appendix D-2-4. All the load-displacements relations shown in figure 4-4 are from the models where the shaft bearing capacity is reduced to the tension bearing capacity as described above.

In figure 4-4 one can see that the load-displacement behaviour of D-Pile is in good agreement with the result of CUR 77. The load-displacement behaviour is also almost linear. This is caused by the linear behaviour of the T-Z curves which regulate the mobilization of the shaft bearing capacity in D-Pile.
In Scia Engineer for the first 50% of the tensional loading the displacement of the top is less than according the method of CUR 77. This is caused by the non-linear shaft springs. That the lines cross at ca. 50% of the capacity is explicable because the stiffness of the mobilization of the shaft friction is determined by taking the secant stiffness at 50% of the capacity from the load-settlement relationship of the pile shaft. This relationship is also shown in figure A-5 in Appendix A-1-3.
The tension pile in Plaxis acts significant stiffer than the spring stiffness according CUR 77. This

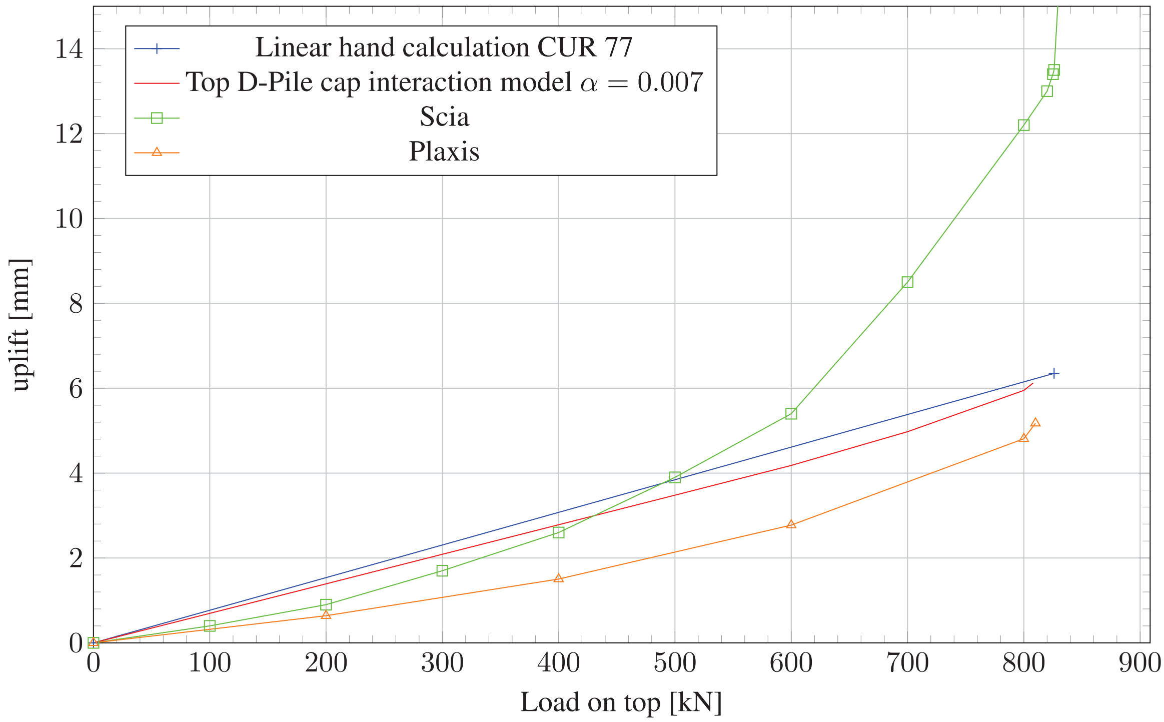


Figure 4-4: Load-displacement behaviour of pile loaded by tension in the different models

is due to the effect that less displacement is needed to reach the maximum defined shaft bearing capacity. The effect was also noticed for piles loaded by axial compression as already discussed in paragraph 4-5-3.

4-6-3 Pile loaded by a horizontal force on top

As discussed there are different situations taken into account to look at the behaviour of a horizontal loaded pile. In D-Pile is looked to a pile in an excavation (surface level at -7.0 m. NAP) and a pile top at -7 and a surface level at -4.78 m. NAP. In Scia Engineer only the original surface level is taken into account. In Plaxis three different situations are taken into account.
For the comparison of the models the situation will be taken into account with the surface level at -4.78 m. NAP and the pile top at -7.0 m. NAP. This situation is chosen because the method of Ménard is dependent of the cone resistance. This cone resistance is both stress and stiffness dependent. The cone resistance is not corrected for relaxation of an excavation. Therefore this situation is the most 'equal' for a comparison.

In figure 4-5 the load-displacement behaviour of the pile top of different models are shown. Here one clearly see the linear behaviour of the method of Ménard. For small horizontal displacements this method is less stiff than the other methods. But these methods contain the non-linear behaviour of the soil. So therefore the pile top displacement increases non-linear in these models.

For the given parameters the pile in Plaxis behaves stiffer than the pile in D-Pile. This is caused by the different parameters in these models. Another explanation of this stiffer behaviour can be found in the fact that the embedded piles in Plaxis does not take into account slip between the pile and

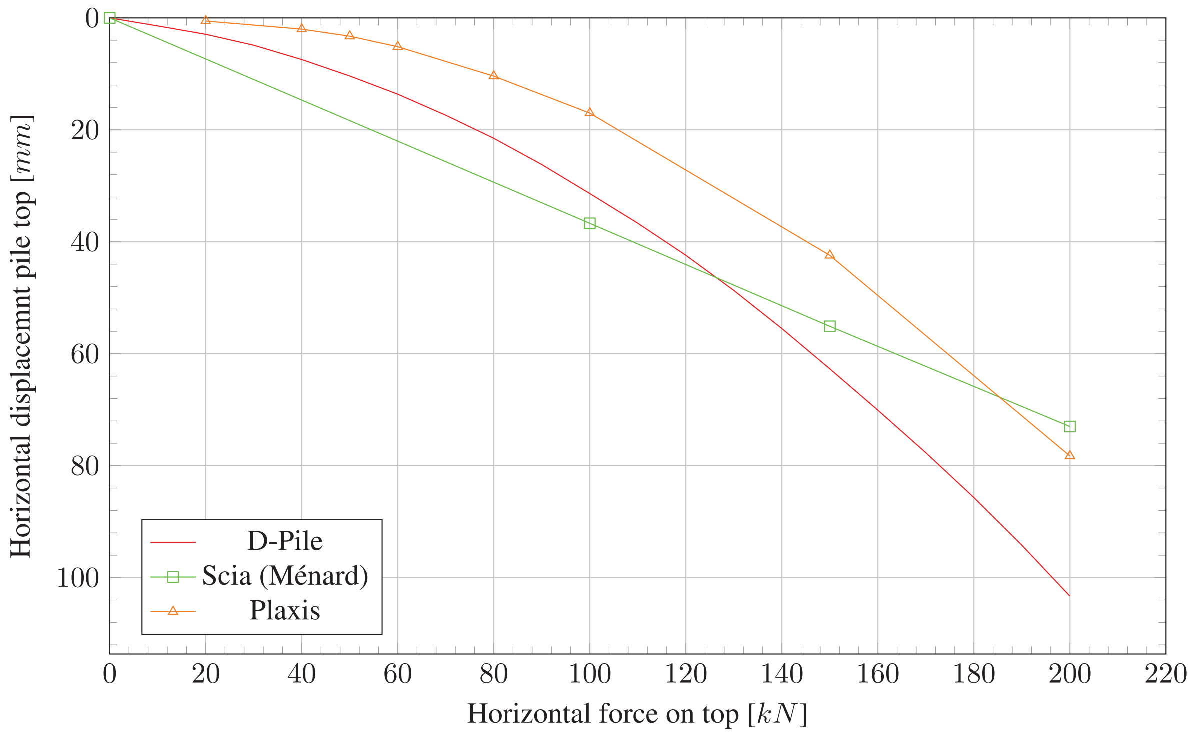


Figure 4-5: Horizontal displacement of the pile top for the different models for the situation where the surface level is at -4.78 m. NAP and the pile top is at -7.00 m. NAP

soil. In the empirical method of the P-Y curves this is the case.

In figure 4-6 the relation of the deflection of a different points at the pile shaft and the pressure of the soil against the pile is shown. This pressure of the soil against the pile can be derived by taking the derivative of the shear force. In Appendix D-6-1 is described how this is done. In figure 4-6 one can clearly see that in Scia the pressure against the pile increases linear with increasing deflection. In the model for the whole clay layer from (-5.75 m. till -11 m. NAP) the cone resistance is constant. Therefore the horizontal subgrade reaction,determined with Ménard, against the pile is constant over this whole clay layer.

In the other models one can see the non-linear behaviour of the soil. When the displacements becomes big the soil cannot give additional pressure to the pile because the soil is failed. Some of these lines are not in line with the trends one expect. This can be caused by numerical inaccuracies in the determination of the pressures. Furthermore one can see that in both models the resistance of the soil increases with depth. Furthermore one can see that the pressure in Plaxis is bigger. This explains that the pile top in Plaxis has less deflection than in D-Pile.

In figure 4-7 the relation between the maximum bending moment in the pile and the horizontal force on top is shown. Here one can see the same trend as for the deflection of the pile, which is the logical result of the fact that the bending moment is the second order derivative of the deflection. The relationships are given in Appendix D-6-1. For the chosen pile type the maximum allowable bending

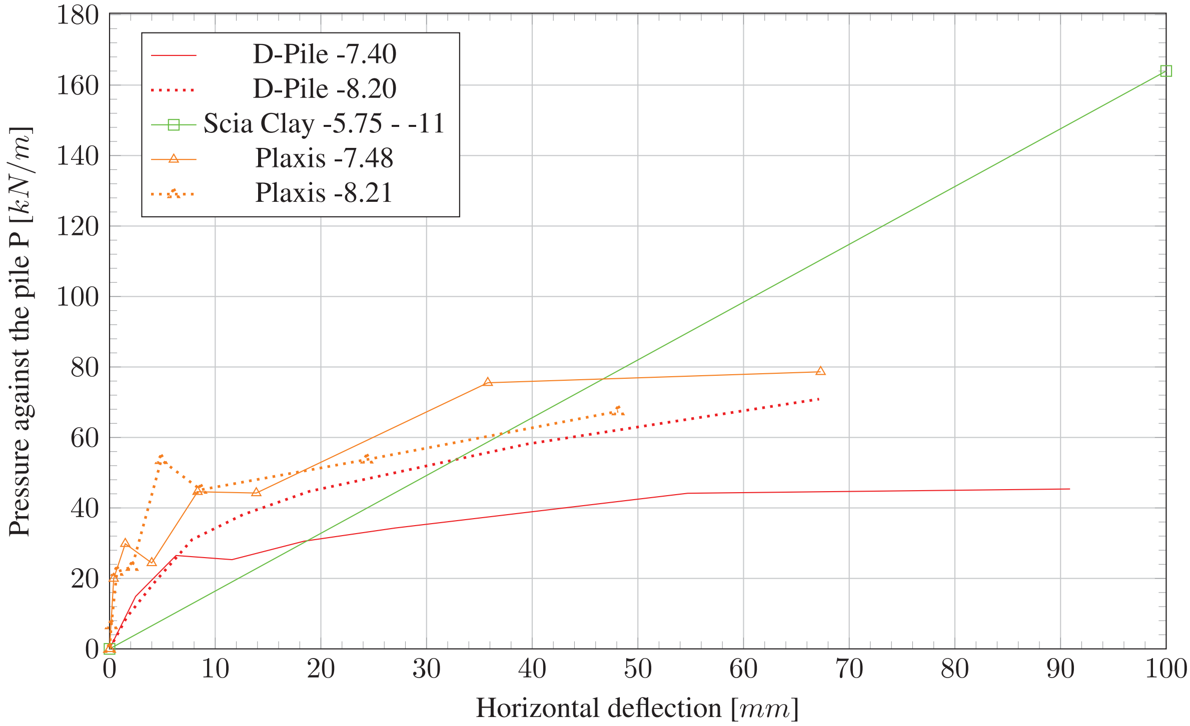


Figure 4-6: Pressure displacement relations for the different models at various heights

moment in the ultimate limite state (ULS) and serviceability limite state (SLS) are also shown in this graph. This is for a pile with a pre-stress of 3.5 N/mm^2 and the minimum amount of reinforcement. This is described in Appendix D-6-2.

For relative low horizontal force the bending moment will be the larger in Scia Engineer, which can be explained by the lower subgrade reaction at small displacements. However the pile will not fullfill both the limite states for the lowest horizontal load for the D-Pile model. The pile can resist the biggest horizontal force in Scia Engineer, but it is doubtful or this is realistic, because this model does not take into account failure of the soil.

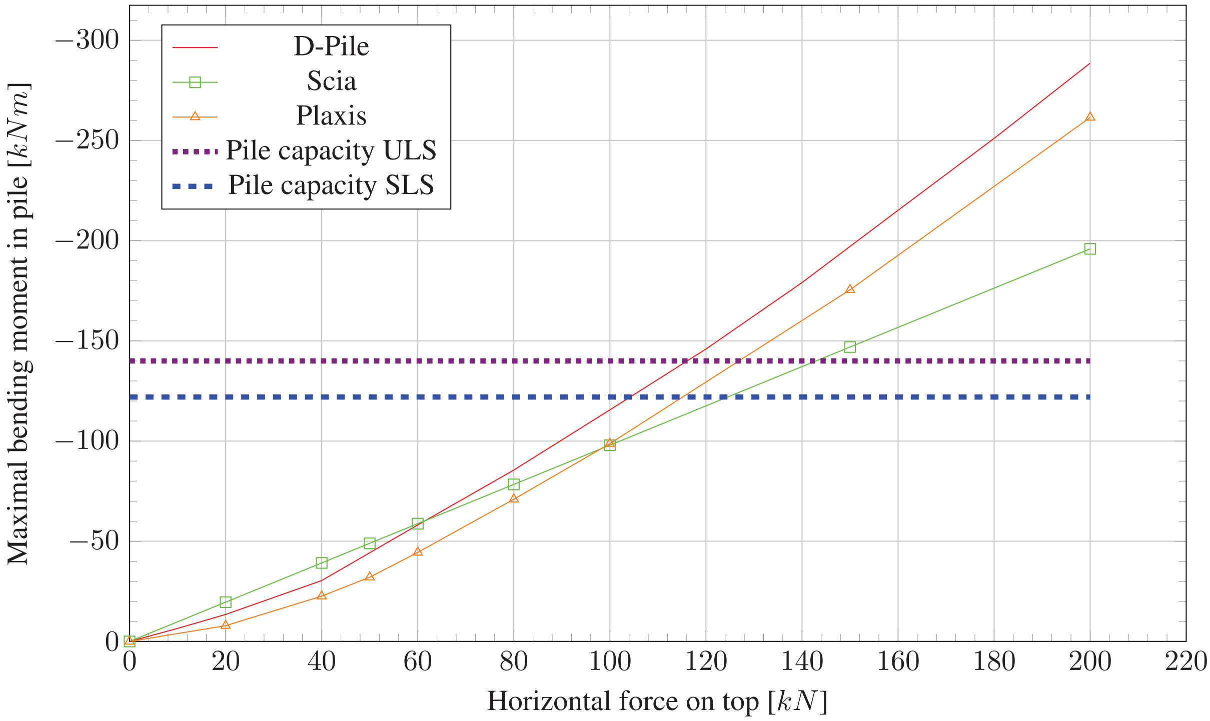


Figure 4-7: Maximum bending moment in the pile for the different models

4-7 Conclusion

To assess the load-settlement behaviour of displacement piles loaded by axial compression in the different models the result of a load-settlement calculation according Eurocode 7 (7) is used as benchark. Hereby the following conclusions can be made:

- The results of Scia Engineer and D-Pile are in good agreement with Eurocode 7. This because these models use springs based on empirical relationships. For these springs it is important to approach the bearing capacity as accurate as possible, because the stiffness is based on a relationship with the bearing capacity.
- The load-settlement behaviour for a ground displacement pile modelled with an embedded pile in Plaxis differs significantly from Eurocode 7. This because the displacement and densification effects around the pile base are not taken into account in this model. This makes this model less suitable for model constructions with ground displacements piles which are sensitive for base the spring stiffness of the piles.
- However the soil displacements effects along the pile shaft are not taken into account by the model the mobilization of the shaft bearing capacity has more or less the same stiffness as the other models with the empirical relationships.
- The load-displacement behaviour of axial loaded piles in Plaxis can be improved by applying measures like placing a volume strain around the pile base, or place an additional spring or plate below the pile base. However this are complicated measurements which may not always desirable in the daily engineering practice.

To judge the load-displacement behaviour of a pile loaded by axial tension the results of the different models are compared with the method given in CUR 77 (1). The conclusions which can be made by this comparison are:

- The load-displacement behaviour according to D-Pile is in good agreement with CUR 77. This also acts almost linear. This is caused by the bi-linear T-Z springs which govern the mobilization of the shaft bearing capacity in this model.
- The behaviour in Scia Engineer is more in line with the non-linear behaviour which is needed to mobilize the shaft bearing capacity. This is caused by the non-linear springs imported in the model which govern the mobilization of the shaft bearing capacity according to the relationhip given in Eurocode 7.
- The load-displacement behaviour in Plaxis is also non-linear, but significant stiffer. This effects was already seen by piles loaded by axial compression.

The non-linear behaviour in Scia Engineer and Plaxis can be benificial if one wants a stiff behaviour of the tension pile. For example when one have very strict deformation demands for a structure.

In the different models the pile is loaded by an external horizontal load on the pile top. The important things noticed in this comparison are:

- When the pile top is below the ground level the resistance of the soil against the pile is dependent on the fact or the soil is modelled from the original surface level or an excavation is taken into account. This can be taken into account by the models D-Pile and Plaxis.
- The numerical models D-Pile and Plaxis take into account the non-linear behaviour of the soil. This means that they give a stiff behaviour for small displacements and include failure of the soil at large horizontal displacements.
- In Scia Engineer the method of Ménard is used to represent the horizontal support of the soil against the pile. This method gives a constant horizontal subgrade reaction against the pile. This subgrade reaction is less stiff than the methods used in the other models for small displacents and gives an overestimation of the horizontal support for large deformations, because failure of the soil is not included in this method.

These things are important to keep in mind when a pile and / or the construction on top is sensitive for the horizontal support of the soil against the pile or subjected to a very large horizontal force on top.

The fact that it is without additional effort not possible to model a reliable load-displacement behaviour for ground displacement piles leads to the choice that Plaxis will not taken into account furhermore in this thesis. Modelling of pile foundation structures in Plaxis can be benificial for special situations like complex geotechnical constructions with complex load compbinations or strong interaction with neighbouring structures as also mentioned by Broere and Van Tol (15).

Chapter 5

Investigation of numerical models for pile groups

Before one can start with the design of the foundation structure of a power pylon in the different numerical models one also has to know how these models deal with the behaviour of pile groups. Therefore the investigation of the models for single piles is expanded to pile groups. This investigation will be done for piles in the same soil conditions as taken into account the investigation of the models for single piles. In this investigation the models taken into account are:

- D-Foundation
  This model will be used to determine the bearing capacity of the different piles in a group axially loaded by pressure and tension. Furthermore it will be used to determine the settlement of a group loaded by axial compression which can be used as benchmark for the other models.
- D-Pile
- Scia Engineer

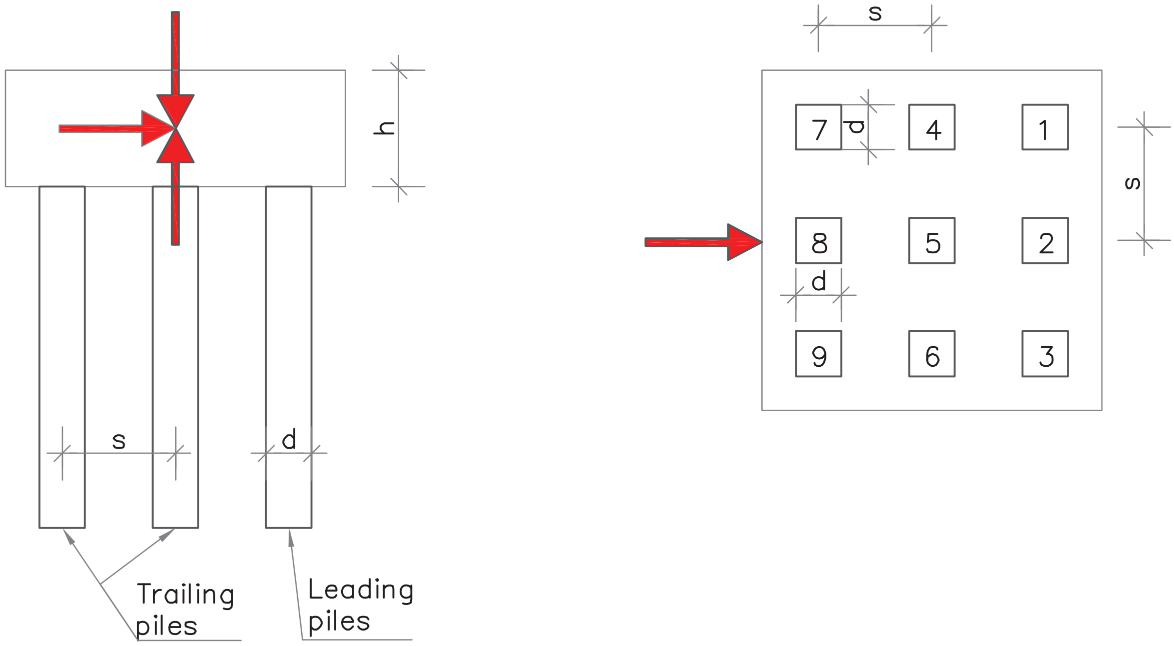
The numerical model Plaxis will not be taken into account. This because in the investigation of the single piles is found that it was not possible to simulate the load-settlement behaviour of a soil displacement pile on a proper way without doing additional efforts.

For pile groups research will be done to a group of 3 x 3 piles with a cap on top, which will be loaded axially and laterally in the centre of the cap. This foundation structure is shown in figure 5-1. Hereby will be looked to the behaviour of the group. Hereby the following variations in geometry will be made:

- What is the effect of changing the the pile-to-pile distance (s/d)

- What is the effect of changing the cap thickness

The goal of this investigation is to see what the differences are in the distribution of forces in the piles and cap. Furthermore the differences in deformations of the piles and cap will be considered. Again here it is important to see how these diffences are caused. When the structure is loaded axial one can distuingish three different piles. At a horizontally loaded structure one can distuingish 6 piles with different behaviour (these are shown in table 5-1).



(a) Cross section **(b)** Top view

Figure 5-1: Schematization of the considered foundation construction, see table 5-1 for classification of the different piles

Table 5-1: Distinction between different piles for axial and lateral loading

Axial loaded		Lateral loaded	
pile nr.	Category	pile nr.	Category
1, 3 ,7, 9	Corner piles	1, 3	Corner piles (leading row)
2,4,6, 8	Edge piles	2	Middle pile (leading row)
5	Centre pile	4, 6	Edge pile middle row (trailing)
		5	centre pile (trailing)
		7, 9	Corner piles back row (trailing)
		8	Middle pile back row (trailing)

At the end of this chapter also some variations are made in rakeness of the piles to see how this influences the behaviour of the foundation structure. Hereby the deformation, rotation and forces in the foundation structure will be taken into account.

5-1 D-foundation

D-foundation is used to determine the bearing capacity of piles loaded by pressure and tension. Furthermore it is used to determine the settlement of pile groups.

5-1-1 Bearing capacity

The total bearing capacity of the pile group is equal to the amount of piles multiplied with the capacity of the piles. The check on failing of the pile group as one collaborating block is not incorporated in the numerical model.

5-1-2 Settlement

The settlement in D-foundation is determined according to the method given in Eurocode 7 (7). This settlement contains out of:

- Elastic deformation of the pile
- Settlement of the pile base
- Additional settlement caused by the group effect (s_2)

The first two effects are already discussed in section 4-2. The additional settlement in a group may according to Eurocode 7 be determined on two different ways. In the first method, given in Chapter 7 of Euorocode 7, the deformation of the soil below the pile tips is based on the Young 's modulus below the pile tip. This is also the method incorporated in D-Foundation. The second method which is given in Chapter 6 of Eurocode 7 is the method of Koppejan where one determines the consolodation and creep of the layers based on the method of Koppejan. In this method the settlement in cohesive layers is time-dependent. Both methods are described in Appendix E-1-2.

In figure 5-2 the pile top settlement for groups with different pile-to-pile distances for both methods is shown. Here one can clearly see that for the chosen parameters the settlement is significant higher for the method where the settlement is determined based on the Young 's modulus. This can be caused by the fact that the Young 's modulus determined by Eurocode 7 is very conservative. Moreover is the settlement determined by the method Koppejan in figure 5-2 the final settlement. For short term loading the settlement will be even less. This because the consolidation and creep in clay are time-dependent. This is shown in figure E-2 in Appendix E-1-2.

In figure 5-2 one can see that the additional settlement decreases when the pile-to-pile distance increases. But this effect is stronger by the method based on the Young 's modulus than by the method of Koppejan.

The mobilization of the shaft bearing capacity depends on the relative displacement between the pile and shaft. This numerical model (and Eurocode 7) does not taken into account the behaviour of the soil between the piles. When a group settles this can have influence on the mobilization of the shaft bearing capacity of the piles, because the group settles in total and some piles maybe need to settle a bit more to have enough relative settlement to mobilize their shaft bearing capacity.

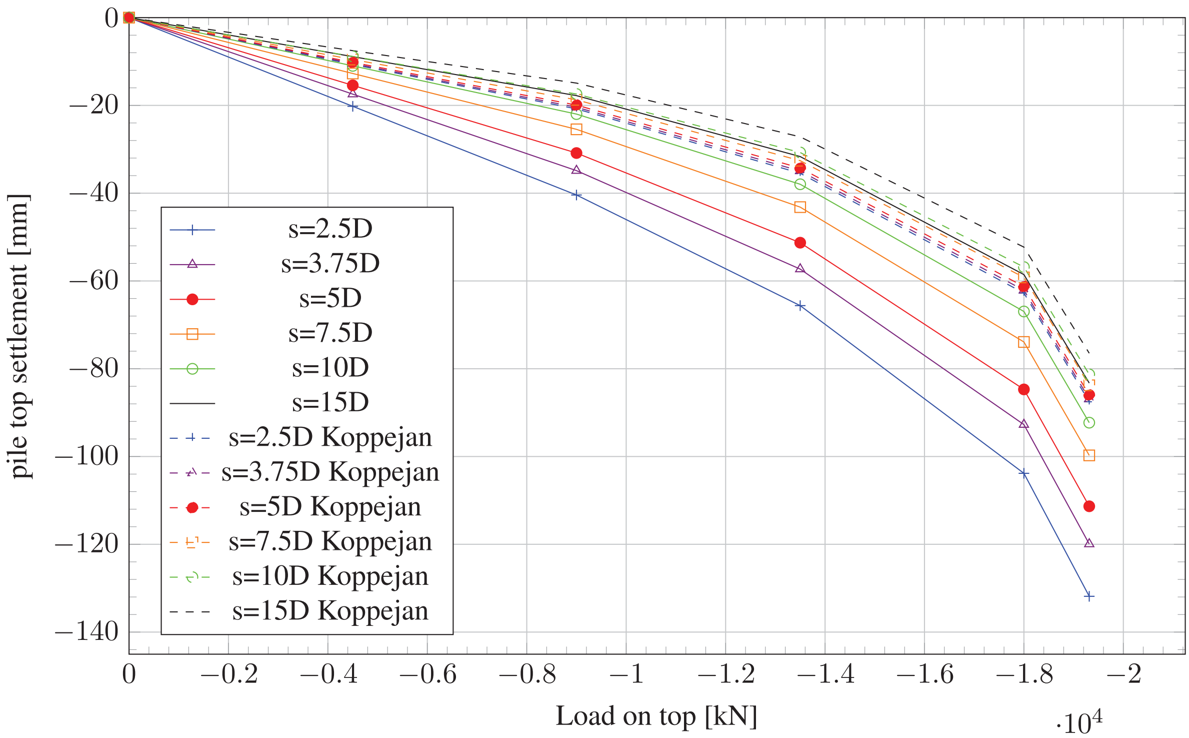


Figure 5-2: Load-settlement behaviour of a pile group (3 x 3) in D-Foundations for different pile distances (s/D) Additional settlement determined based on the stiffnes below the tip and the method of Koppejan

5-1-3 Tensile bearing capacity

The tensile bearing capacity of piles in groups is different than for single piles as already discussed in section 2-2-4. In figure 5-3 the tensile bearing capacity of different piles in groups is shown. Hereby one can see that the pile distance (s/d) has a significant influence on the tensile bearing capacity. This is caused by the reduction of the effective stress due to loading of the pile group (f_2) which depends on the distance of one pile to another. Here one can clearly see that this effect is the largest for piles with the most piles close by (centre pile) and is less big when they are less influenced by other piles (edge and corner piles).

Densification due to pile installation may according to Eurocode 7 taken into account to a maximum distance of 6 times the pile diameter ($s = 6d$). One can see that for closely spaced piles it has a significant positive effect on the tensile bearing capacity. For the centre piles it has no effect because here uplift of the total soil mass around the pile becomes the normative failure mechanism. For the centre pile in a group with $s = 2.5D$ this also the failure mechanism when densification is not taken in to account. For the other piles in the group this is not the guiding failure mechanism.

5-2 D-Pile

In section 4-3 the modelling of a single pile in D-Pile is already described. Here is stated that the behaviour in bot vertical and horizontal direction is very sensitive on the local springs (T-Z, base and

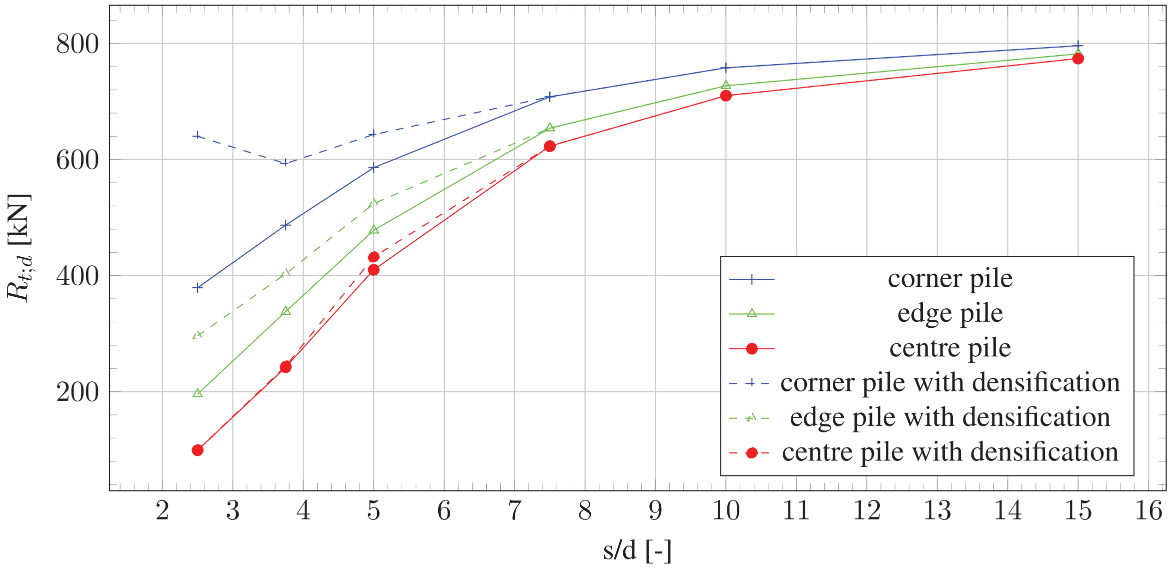


Figure 5-3: Tensile bearing capacity ($R_{t;d}$) in the Serviceability limite state (SLS) for the different piles in a 3 x 3 pile goup for different pile distances (s/d) (Alternating loads and pile mass not taken into account)

P-Y curves) and the chosen stiffness of the elastic medium of the Cap layered soil interaction model. Especially for the horizontal displacement it is very important to adjust the stiffness of the medium to $10 \cdot E'_{50}$ to not get to large displacements. In fact in real soil the behaviour is dependent on the local effects along the pile shaft and pile base and the behaviour of the whole soil continuum.

5-2-1 Axial loaded by compression

To see how the properties of the elastic medium and local springs influences the behviour of total group for a group of 3 x 3 pile group where these proporties are varied. Herefore the following combinations are taken into account:

- Soil with a ten times bigger Young 's modulus ($E = 10 \cdot E'_{50}$), this to prevent a to large displacements.
- Soil with the original Young 's modulus ($E = 1 \cdot E'_{50}$), this because the pile is in this original medium.
- Soil with the original Young 's modulus ($E = 1 \cdot E'_{50}$) and stiffer T-Z and base springs This to decrease the settlement of the local springs and take into account the original stiffnes of the soil continuum.

These different approaches has a significant influence on the behaviour of the pile group, as shown in figure 5-4. When the original stiffness is taken the settlement of the pile tops at the maximum capacity of the group is 130% higher. But this settlement is in line with the settlement as determined according to the method based on the Young 's modulus in Eurocode 7. When the additional settlement is determined with the method of Koppejan the settlemnt of the pile tops is in between the results with the stiff and weak Young 's modulus. Making the T-Z and base springs stiffer has an effect, but this is not very large, the piles reacts a bit stiffer.

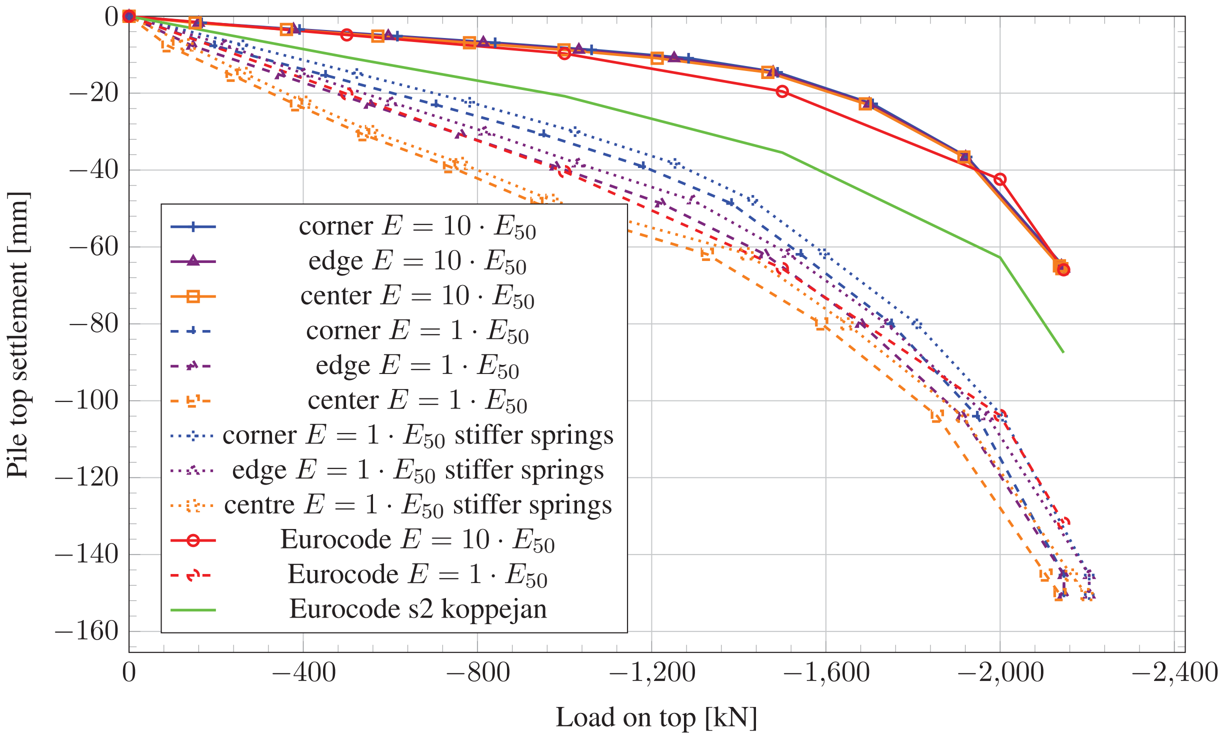


Figure 5-4: Load-settlement behaviour D-Pile for $s/d = 2.5$ for different stiffnesses of the elastic medium

The stiffness of the elastic medium also has a big influence how the load is divided over the different piles in the group. The capacity of all the piles is the same, but some piles act stiffer. The corner piles are influenced less by the other piles. They need less settlement to have enough relative settlement to mobilize their bearing capacity. So they behave stiffer. When the stiffness of the elstic medium is 10 times the original Young 's modulus this effect is negligible as shown in figure 5-4. When the original Young 's modulus is taken into account there is quite some deviation in stiffness between the different piles. This is also shownin figure 5-5, here is shown how much the load on the different piles in the group differs from the average pile load.

When the pile-to-pile distance increases the total settlement of the group decreases (see figure 5-5). But when the original Young 's modulus is taken into account in the model the deviation in forces on the different piles is still significant. Even when the piles are placed at a centre-to-centre distance of 4 m ($s = 10d$) the differences between the centre and corner piles is significant. When the stiffer local springs are taken into account this effect is even larger. Therefore the approach of the stiffer local springs seems to be not a good solution, because it causes a lot of interaction.

The method with the stiffer local springs may overestimate the influence of the different piles on eachother. Even for large pile-to-pile distances ($s = 10d$) the deviation of the load on the different piles in the group is significant.

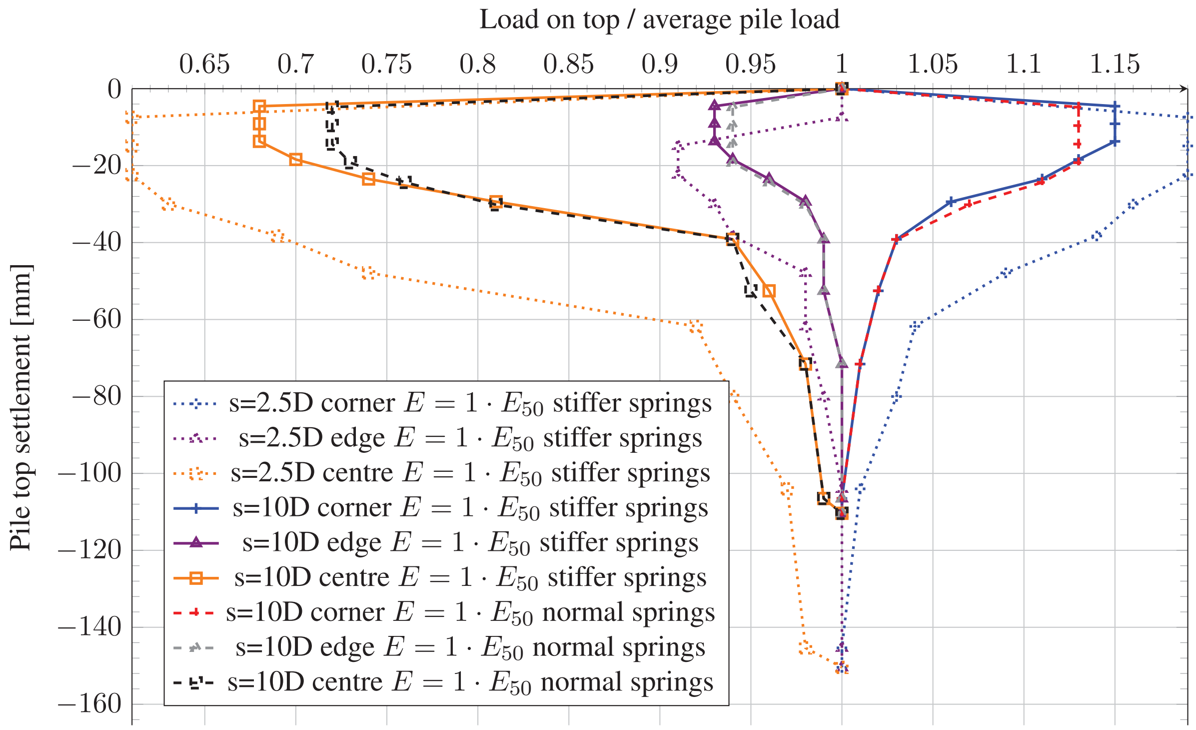


Figure 5-5: Deviation of pile loads for $s/d = 2.5$ and $s/d = 10$ for different stiffness of the local springs

5-2-2 Axial loaded by tension

As shown in Paragraph 5-1-3 the tensile bearing capacity is dependent on the pile-to-pile distance. In Eurocode 7 this effect is taken into account by adjusting the cone resistance as shown in equation 5-1. D-foundation which determines the bearing capacity according to Eurocode 7 also uses this equation.

$$q_{s;z;d} = f_1 \cdot f_2 \cdot \alpha_t \cdot q_{c;z;d} \tag{5-1}$$

Where:

$q_{s;z;d}$	$[kN]$	Design value of the shaft friction in the pile group
f_1	$[-]$	Factor for the effect of densification in the pile group for ground displacement piles in sand
f_2	$[-]$	Factor for decrease in effective pressure in the sand layers where the piles determines its bearing capacity caused by loading the pile group. This factor is for sand. For cohesive soil $f_2 = 1.0$.
α_t	$[-]$	Pile class factor according to Eurocode 7, which takes into account the way of pile installation
$q_{c;z;d}$	kN	Design value of the cone resistance at depth z as described in Appendix D-2-3

As already discussed in paragraph 4-3-1 and 4-3-3 is already stated that the (tensional) shaft bearing capacity in this model is based on the cone resistance (sandy soils) or undrained shear strength (cohesive soils). Furhermore is stated in paragraph 4-3-4 that when one wants to adjust the axial spring stiffness according to CUR 77 (1) it is important to approach the bearing capacity as accurate possible. So when one wants have an accurate behaviour of the tension piles in a group according to the methods prescribed by Eurocode 7 and CUR 77 it is important to adjust the cone resistance (q_c) for the group effects like shown in equation 5-1. However this is a laborious process which has to be done manually.

5-2-3 Lateral loaded

When the cap in D-Pile is loaded by a horizontal load the cap will move equal. When the group effect is not taken into account the pressure of the soil against the pile will be equal for all piles. However when the Cap layered soil interaction model is used the pressure of the soil against the different piles in the group will be influenced by the pile-soil-pile interaction. This can be seen in the deviation of shear forces in the different piles of the piles. As already stated the pressure againt the pile is the derivative of the shearforce. So when one knows how the shear forces are divided over the different piles one also knows the relative difference between the soil pressure against the different piles. This effect is shown in Appendix E-2-2.

The loaded pile group is shown in figure 5-1. The trends one can see by a horizontal loaded group in D-Pile is that all the corner piles takes the same load, furthermore the middle piles in the front and back row (2 and 8) and the edge piles in the middle row (4 and 6) also has the same load. So there are 4 different loaded piles.

When the distance between the piles (s/d) in the group increases the difference in shear force and bending moment between the piles decreases. This is shown in Appendix E-2-2.

The interaction between the piles also depends on the rate of loading. For relative small horizontal forces the P-Y curves act stiff. So a relative large part of the horizontal forces goes into the elastic medium and there is a lot of interaction to the other piles. When the horizontal force is relative large the soil around the pile will yield and a part of the deformation of the pile is local. In this case there will be relative less interaction between the different piles. This is shown in Appendix E-2-2 and in figure 5-12 in Section 5-4.

5-3 Scia Engineer

5-3-1 Schematization

In Chapter 4 is already described how a single pile is modelled in Scia Engineer. This pile is also used in the pile groups. The piles are conected rigid to a plate on top. As already discussed in section 2-2 does closely spaced piles in groups behave different than single piles. These effects are taken into account with the following methods:

- The tensional bearing capacity of a pile is dependent of the position in the group and the distance to the other piles as shown in figure 5-3. As already mentioned in chapter 4 there is a direct relationship between the pile capacity and the pile stiffness. In figure 5-6 an example is shown of a spring for which the tension part is adjusted because of the group effect.

- The horizontal subgrade reaction for closely spaced piles is different than for single piles. This reduction is modelled with the method given by Reese en Van Impe (30) as described in Appendix B-3-1. In figure 5-7 these reduction factor are shown for the different piles in a 3 x 3 pile group.

The different behaviour of piles in groups loaded by an axial compressive load is not taken into account in the chosen way of modelling.

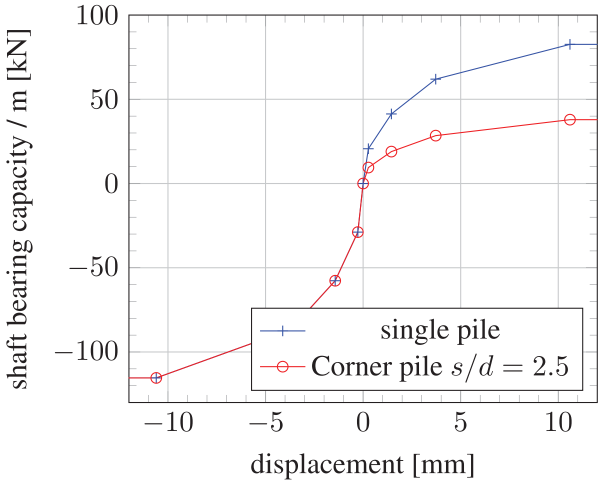


Figure 5-6: Shaft spring /m in Scia Engineer

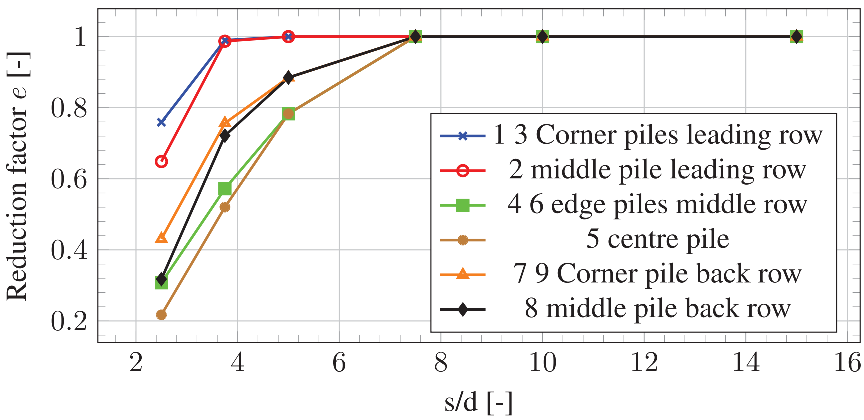


Figure 5-7: Reduction factors on the horizontal subgrade reaction at the different piles in a 3x3 pile group for different pile distances (s/d)

5-3-2 Axial loaded

The modelling of the piles as described in section 4-4 is done with uncoupled springs. These different springs does not influence eachother. Also the addditonal settlement of a group of the layers below the pile base (s_2) is not taken into account by the chosen modelling.

The bending stiffness of the plate and the position of the piles relative to each other has a significant influence on the distribution of loads over the different piles and the settlement of the different piles. This effect is investigated for a 3 x 3 pile group (as shown in figure 5-1) where the pile-to-pile distance (s/d) and the plate thickness (h) are varied.

The results of this investigation are shown in figure 5-8 and 5-9 The bending stiffness of the plate is here represented by the thickness (h) of the plate. The pile-to-pile distance is taken into account by the factor (s/d). So when the ratio of the thickness to the relative pile-to-pile distance ($\frac{h}{s/d}$) is very low the structure is not able to redistribute the vertical load. So the centre pile will collapse, because the force on top is bigger than its capacity. The settlement in this is so large that settlement of this pile is even not visible in figure 5-9.

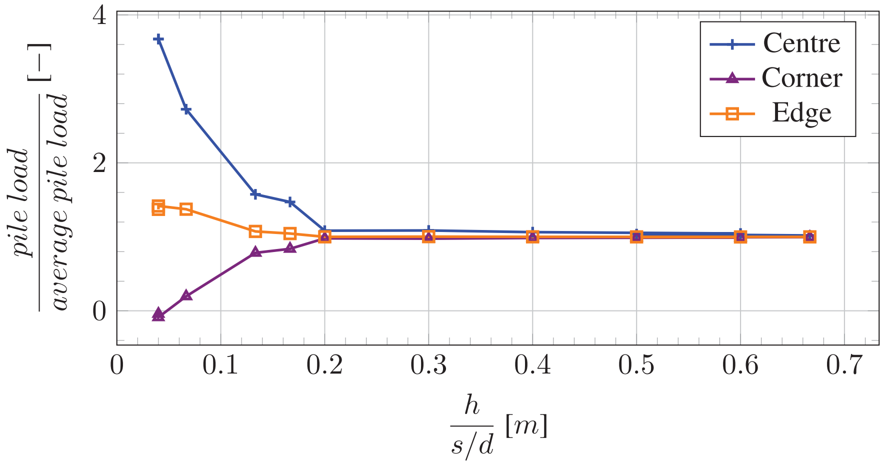


Figure 5-8: Relationship between the plate thickness (h), pile-to-pile distanc (s/d) and load distrubtion over the different piles of 3 x 3 group which is loaded by 27% of its capacity

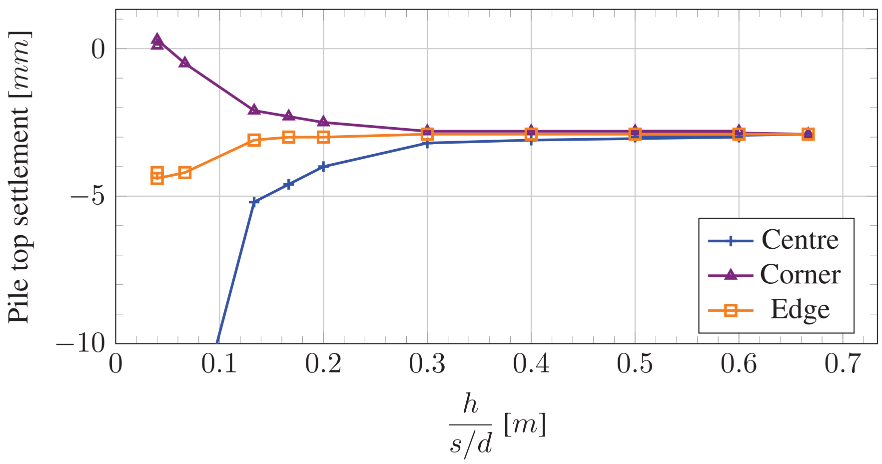


Figure 5-9: Relationship between the plate thickness (h), pile-to-pile distanc (s/d) and the pile top settlement of the different piles of 3 x 3 group which is loaded by 27% of its capacity

When $\frac{h}{s/d}$ is between 0.13 and 0.3 the structure can redistribute the forces but the load is not divided equal over the different piles in the group. Also the settlement of the different piles is not equal.

When the ratio between the plate thickness and pile-to-pile distance is larger than 0.3, for example a plate of 3 m. thick and a relative pile-to-pile distance of 10, the vertical loads will be divided equal over the different piles in the group and all piles will settle equal. In fact this is how D-Pile takes into account the pile foundation.

5-3-3 Laterally loaded

The disbribution of forces in the piles of a horizontal loaded pile group in the defined model in Scia Engineer is dependent on the cap thickness and the pile-to-pile distance. The first effect is mainly for a very thin cap. In Appendix E-3-3 is for a pile group with $s/d = 2.5$ shown that when the cap becomes thicker than 0.5 m. the distribution of forces does not changes significantly anymore. When the horizontal subgrade reaction is not adjusted for the pile-to-pile distance the bending moment and shear force in all piles are equal. When the horizontal subgrade reaction is adjusted with the factors as shown in figure 5-7 the piles with the highest subgrade reaction will have the largest bending moments and shear forces. This is shown in Appendix E-3-3. This in contradiction to a free head pile where the bending moments increases when the subgrade reaction decreases. The forces in the pile get spread more equal over the different piles when the pile-to-pile distance increases.

5-4 Comparison of the results of the different models

5-4-1 Loaded by axial compression

With the chosen way of modelling in Scia Engineer only the behaviour of single piles can be taken into account. The group effect cannot be taken into account this bcause there is chosen to work with uncouppled springs. In D-Pile the group effect is taken into account by the elastic medium between the piles. As shown in paragraph 5-2-1 the amount of interaction between the piles and the total settlement of the group is very sensitive for the chosen Young 's modulus between the piles.

When one wants to take into account the specific behaviour of a group in Scia Engineer, so the additional settlement and the deviation of stiffness between the different piles in the group. A method one can use is implementing a spring which represents the behaviour of the pile top. This spring can be derived from D-Pile. When one choose this method one only takes into acount the top of the pile and not the pile as structural element.

5-4-2 Loaded by axial tension

From the determination of the tensile bearing capacity with D-Foundation (see figure 5-3) one knows that this bearing capaciy is very dependent on the pile-to-pile distance. In both numerical models the failure mechanism uplift of the total soil mass cannot be taken into account. The failure mechanism pull-out of the pile can be simulated with both models. Hereby the effect of closely spaced piles has to be taken into account manually. In the numerical model D-Pile this has to be taken into account by adjusting the cone resistance (q_c) or eventually by adjusting the pile class factor (α) for each individual pile in the group.

In the numerical model Scia Engineer the group effect can be taken into account by adjusting the tension part of the springs which represent the shaft bearing capacity as shown in figure 5-6 in paragraph 5-3-1.

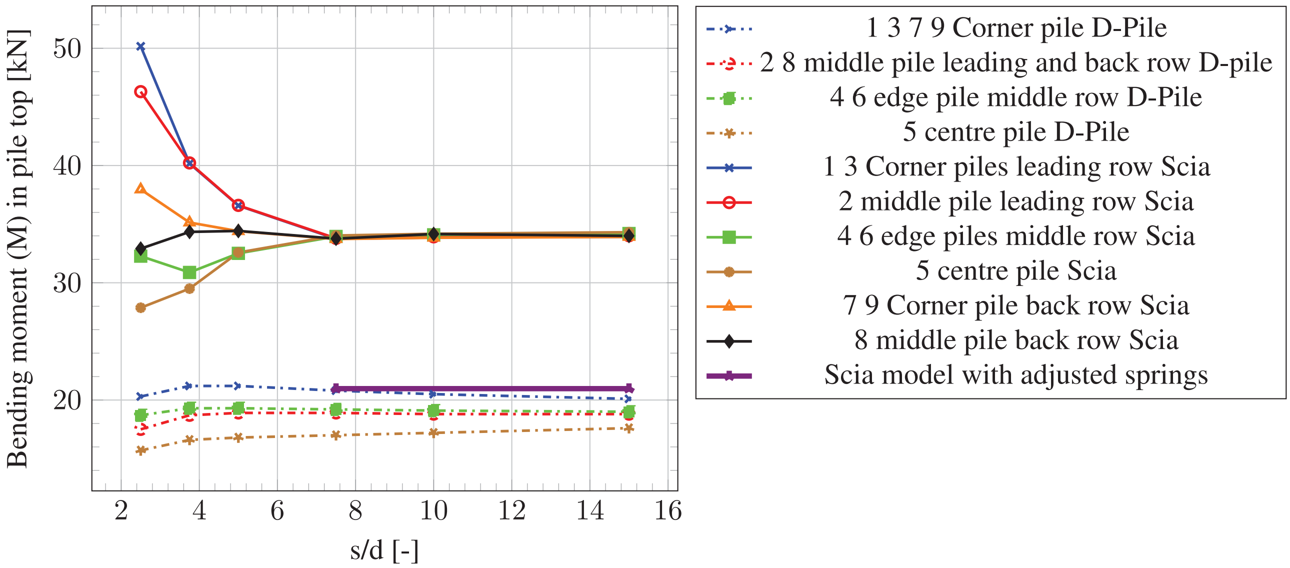
5-4-3 Laterally loaded

In figure 5-10 are the bending moments in the pile heads of the different piles in the group shown. Here one can see that in general there is a large difference between Scia Engineer and D-Pile. This is caused by the applied horizontal subgrade reaction in the different models.

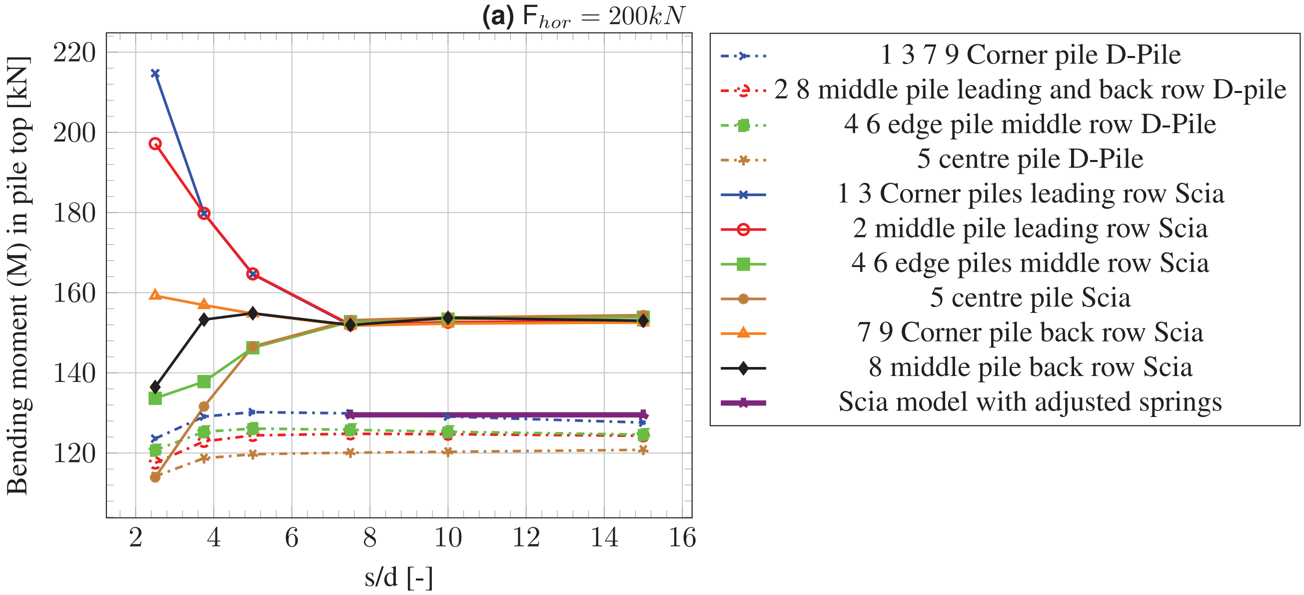
In the numerical model D-Pile the P-Y curves used at different heights along the pile are given. Furthermore the deflection of the pile is known. With these data one can back calculate the actual pressure of the soil against the pile in the model. When the pressure and displacement are knonwn one can determine the actual spring. This method is shown in Appendix E-4-1. In figure 5-11a an example is shown of an determined actual spring. Here one can see that this spring is much stiffer than the spring determined with the method of Ménard. When one implement these newly defined springs in Scia Engineer the bending moment in the pile tops is in the same order of magnitude as in D-Pile as shown in figure 5-10. Also the horizontal displacement of the cap in Scia Engineer is more in line with D-Pile with this adjusted springs. This is shown in figure 5-11b.

Furthermore in figure 5-10 one can see that the deviation of the forces in the pile tops differs significant for small pile distances. These differences are shown more clear in figure 5-12 where the difference in forces is shown between the corner and centre pile. Here one can clearly see that for the method of Ménard this deviation is not dependent of the rate of loading. For D-Pile the amount of interaction is dependent on the rate of loading as already discussed in paragraph 5-2-3.

Also in D-Pile the difference in forces in the pile is observable for larger pile-to-pile distances. In the methode of Reese en Van Impe there is no reduction on the subgrade reactions anymore when the pile-to-pile distance is larger than $s/d = 7.0$. In D-pile at $s/d = 10$ there is still 20% difference in the bending moment and shear force between the centre and corner pile.

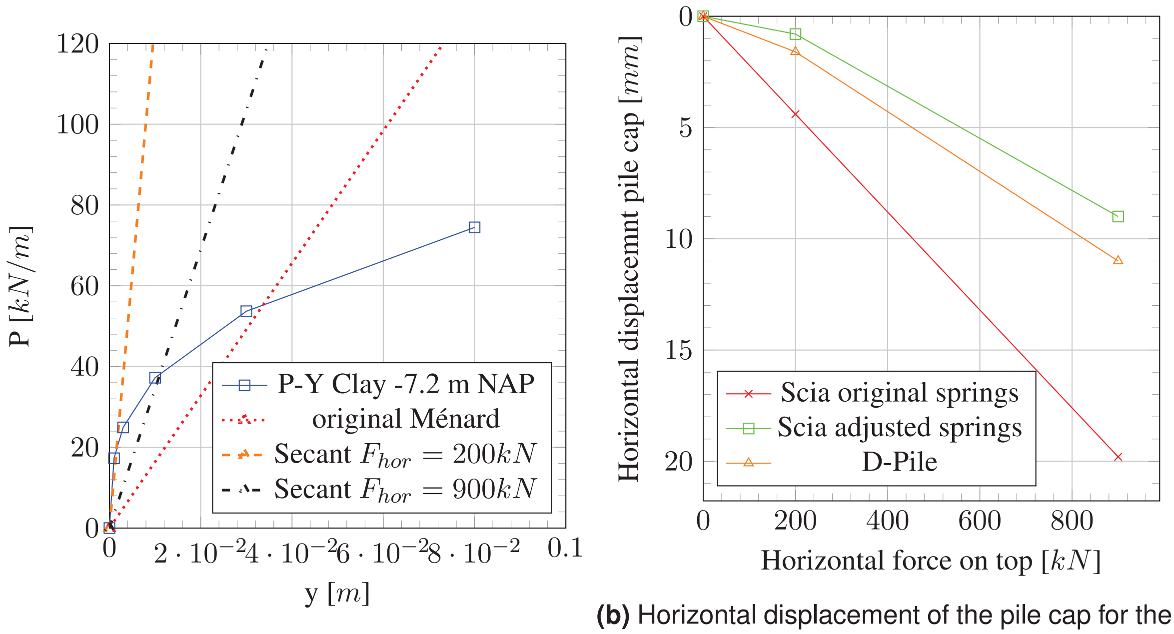


(a) $\mathrm{F}_{hor} = 200kN$



(b) $\mathrm{F}_{hor} = 900kN$

Figure 5-10: Bending moment (M) in the top of the different piles in a 3 x 3 pile goup for different pile distances (s/d)



(a) Springs in the different models at -7.2 m. NAP

(b) Horizontal displacement of the pile cap for the different springs in Scia and D-Pile

Figure 5-11: Adjusting springs in Scia Engineer

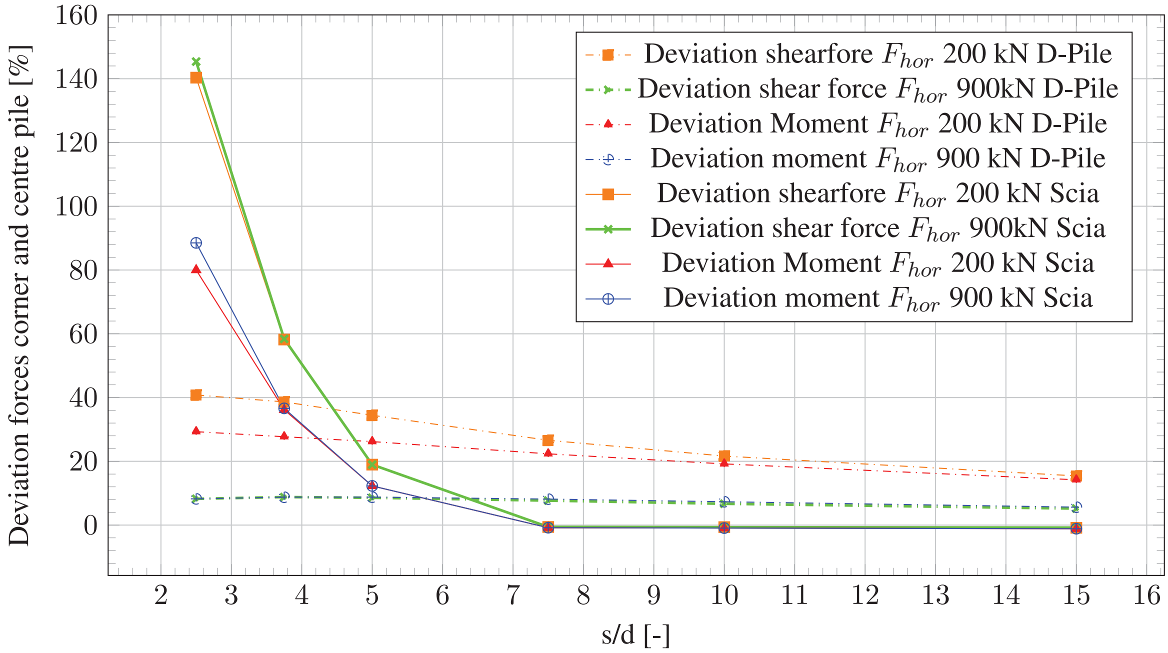
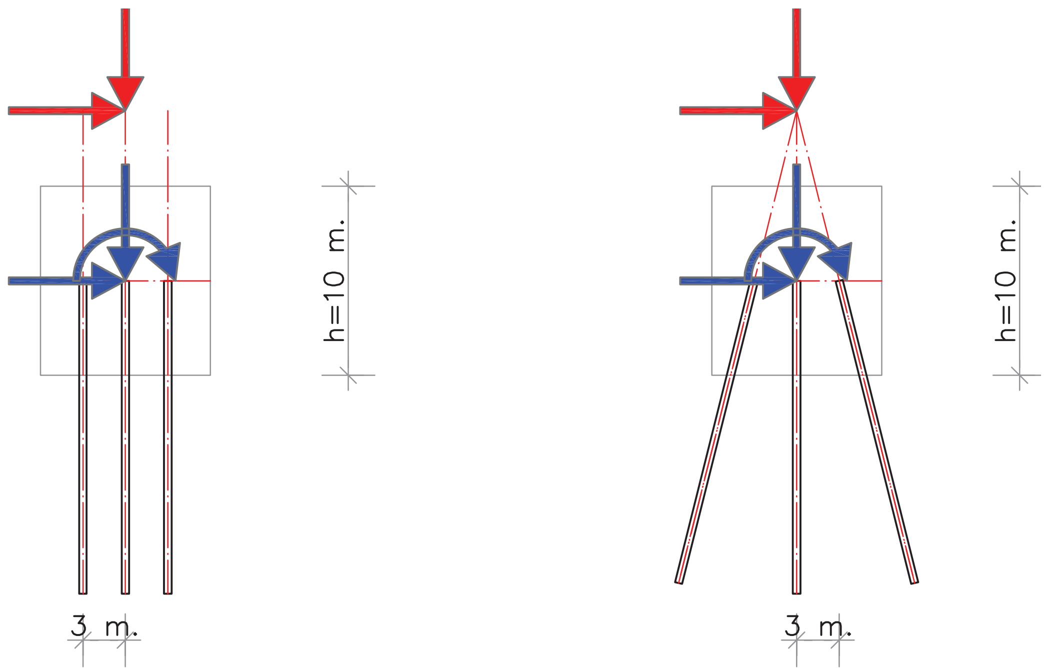


Figure 5-12: Deviation of the forces in pile top betwee a corner and centre pile in a 3 x 3 pile goup for different pile distances (s/d)

5-5 Different pile configurations

Till so far only foundation structures with vertical piles are taken into account. However the rake of the piles can also have a significant influence on the behaviour of the structure. Therefore with the numerical model Scia Engineer two different foundation structures with 3 x 3 piles will be considered: a structure with vertical piles and a structure with rake piles. These structures are shown in figure 5-13.



(a) Group with vertical piles

(b) Group with rake piles

Figure 5-13: Structures with an eccentric horizontal load (red), where the resulting forces on the structure are also given (blue)

In the consideration of these structures the following assumptions are made:

- Pile-to-pile distance 3 m. ($s/d = 7.5$), so according to the method of Reese en Van Impe one does not have to reduce the horizontal subgrade reaction.
- Cap thickness 10 m, so the structure will behave infinite stiff like in the numerical model D-Pile.
- The rake of the pile is chosen 4:1
- The tensional bearing capacity is kept equal for both pile group. The effect that the rake piles have a larger bearing capacity because they are placed further away of each other in the sand layer where they derive their bearing capacity.

The structure is loaded by a vertical force, a horizontal force and a bending moment. This bending moment is caused by an eccentric horizontal load. These different loads are also shown in figure 5-13.

The investigation of these different structures is shown in Appendix E-5. The conclusions of these investigation are given in this section

5-5-1 Vertical loaded

For a vertical loaded structure things that are important to take into account are:

- The springs which represents the vertical support of the pile has to be placed under the same angle as the rake of the pile.
- The horizontal subgrade reaction has to be set on the global coordinates of the model. When one set it to local they will be perpendicular to the pile. When one decompose this to the global directions one also can have a significant vertical support which can influence the load-settlement behaviour of the pile significant.

In the numerical model D-Pile the vertical support is als in line with the direction of the pile. Furthermore the horizontal support in this model does not influence the vertical bearing capacity of the pile.

5-5-2 Horizontal loaded

When a foundation structure is loaded by only a horizontal force the rake of the pile has a significant influence on the behaviour of the structure. A structure with vertical piles will have a larger horizontal displacement than a structure with rake piles. This because a part of the horizontal force will go into the pile as a normal force and will be carried by the vertical bearing capacity of the piles. For the vertical the total resitance has to be given by the horizontal support of the soil against the piles.

Furthermore the structure with vertical piles will rotate in the same direction as the horizontal force. A structure with rake piles will rotate the other way around.

5-5-3 Loaded by bending moments

When the foundation structure is loaded by a bending moment the structure with rake piles deforms more than a structure with rake piles. This because the bending moment causes normal force on the pile tops. In the structure with vertical piles this will cause only a normal force in the piles. However in the rake piles also a bending moment is introduced. This will bend the piles so the structure deforms more. The structure with rake piles is also more sensitive for the applied horizontal subgrade reaction. When this increases the deformatation and bending moment in the pile head decreases. The behaviour of the structure with vertical piles is not dependent on the horizontal subgrade reaction.

5-5-4 Loaded by an eccentric horizontal load

The bending moment in a pile cap is very often caused by a eccentric load. This also the case for the power pylon in the reference project. Here the bending moment is cause by the wind against the pylon and cables. An example of this is shown in figure 5-13. Play around with the rake of the piles can have a positive effect on the behaviour of the structure. This is the case when the center lines of the rake piles intersect eachother in the point where the horizontal force grabs on at the structure as shown in figure 5-13b. This because the horizontal force can be totally taken by a normal force in the piles. This has will make the horizontal displacement and rotation significant less than a structure with vertical piles. Also the bending moments in pile heads are significant lower than for vertical structures. Furthermore the hehaviour of the structure with rake piles is less sensitive for the applied horizontal subgrade reaction.

5-6 Conclusion

In this chapter reseach is done how the behaviour of an axially and horizontally loaded pile group depends on the centre-to-centre distance of the piles and the cap thickness. Hereby is investigated how the models D-Pile and Scia Engineer take into account this changes in geometry. For piles loaded by axial compression the following conclusions can be made:

- In the numerical model D-Pile the additional settlement of a group in comparison with a single pile can be taken into account. Furthermore the deviation in stiffness of the different piles in the group (corner, edge or centre) is taken into account by this model. However this stiffness is dependent on the stiffness of the elastic medium. The original Young 's modulus as determined according to Eurocode 7 give large settlements and relative big stiffness differences between the piles. When one determines the additional settlemeent of a group with the method of Koppejan the settlement of the group is more than 50% less than by a D-Pile calculation with the Young 's modulus of the elastic medium according to Eurocode 7. When one increase this stiffness the settlement and difference in stiffness will decrease.
- With the chosen way of modeling in Scia Engineer it is not possible to take into account the additional settlement and stiffness differences which may occur for pile groups. When one wants to implement this behaviour in the model of structural engineers one can choose to model the top of the pile with springs derived in D-Pile.
- In Scia Engineer the bending stiffness of the pile cap can be taken into account. When the thickness of the cap is thin relative to the centre-to-centre distance the influence on the distribution of forces over the different piles will be significant. This effect cannot be taken into account by D-Pile.

The bearing capacity and load-displacement behaviour of piles in groups loaded by tension depends on the centre-to-centre distance of the piles. In this chapter these effect are considered and there is investigated how the different models deal with this.

- The failure mechanism uplift of the soil cannot be taken into account by both models. The failure mechanism pull-out of the pile can be taken into account by both models.
- When one in D-Pile wants to have a tensile bearing capacity and load-displacement behaviour which are in line with the behaviour determined according to Eurocode 7 and CUR 77 one has to adjust the cone resistance (q_c) or pile class factor (α) for the different piles in the model. This to take into account the decrease in tensile bearing capacity due to destressing of the soil.
- In the chosen way of modeling in Scia Engineer one has to adjust the shaft spring by adjusting the tensile bearing capacity in the springs which represent the shaft bearing capacity.

The behaviour of a horizontal loaded structure differs significant in the different models. This is caused by the different horizontal subgrade reactions used in the models

- In D-Pile the horizontal support of the soil is guided by the P-Y curves and the elastic medium between the piles. This medium takes into account the pile-soil-pile interaction. The interaction between the different piles is dependent on the rate of loading and decreases when the rate of loading increases.
- In Scia Engineer the horizontal support is governed with springs determined with the method of Ménard . To take into account the group effect a reduction on the subgrade reaction is applied with the method of Reese en Van Impe. This reduction is independent on the rate of loading.
- In general the lateral support in D-Pile is stiffer which leads to smaller bending moments in the pile heads and a smaller deformation of the structure. However when one derives horizontal springs from the results of the D-Pile model and implements these in Scia Engineer the order of magnitude of the bending moment and rotation of the structure are in line in the different models.
- The stiff horizontal behaviour of the D-Pile model can be beneficial for structures subjected to large horizontal loads.

In this chapter a small investigation is done to the influence of the rake of the piles on the behaviour of a foundation structure. The main conclusions hereby are:

- A foundation structure with vertical piles horizontal loaded a the top wil displace more than a structure with vertical piles. Also a structure with rake piles will rotate the other way around.
- When a foundation structure is loaded by a bending moment a structure with rake piles will rotate more and is more sensitive for the horizontal subgrade reaction than a structure with vertical piles.
- A bending moment is often caused by a eccentric horizontal load. When the center lines of the rake piles intersect in the point where this horizontal load grabs on one can get a very efficient distribution of forces. This leads to less deformation of the structure and smaller bending moments in the pile head.

Chapter 6

Optimization foundation structure

In this chapter a case from the engineering practice of Heijmans will be taken into account .This to see how the obtained knowledge about the modelling of foundations structures can be used in the engineering practice. The case which will be used for this optimization is a foundation structure of a power pylon (see figure 6-2 and 6-3). Besides modelling in this chapter also different pile configurations (vertical and rake) will be taken into account. This to couple the theoretical knowledge gathered in section 5-5 to a real case.

The optimization of the foundation structure will be done according to the procedure shown in figure 6-1. This process will be run seperately for the different pile configurations.

In Chapter 4 is already stated that in Plaxis it is not suitable to model a reliable load-displacement behaviour for ground displacement piles without taking additional measures. Therefore it is not taken into account furthermore in this thesis. For this optimization also D-Pile will not be taken into account because:

- When one wants to have a reliable load-displacement behaviour one has to approach the bearing capacity of the piles very accurate including the correlation factors ξ_3 and ξ_4, which is a laborious process. When one wants to take into account the behaviour of tension piles it gets even more complicated, because then also the effect of destressing (f_2) and possibly densification (f_1) has to be taken into account. Furthermore D-Pile does not take into account the effect of alternating loads on the tensile bearing capacity (γ_{var}).
- The numerical model D-Pile does determine the forces and deflection of the piles and not the forces in the plate on top. So one needs to make a link between D-Pile and a structural model. This is possible by giving load-displacement relations at the pile top. But these relations are not easily to get from the results provided by D-Pile.

Reasons to still use the D-Pile model in the design process can be:

- The structure is loaded to a large horizontal force and a stiff horizontal support will have a positive effect on het behaviour of the structure.

- The presence of soft layers below the pile tip. These layers can influence the load-settlement relation of the different piles is a group. So then it can be good to determine these different relationships with this model and import them in a structural model.

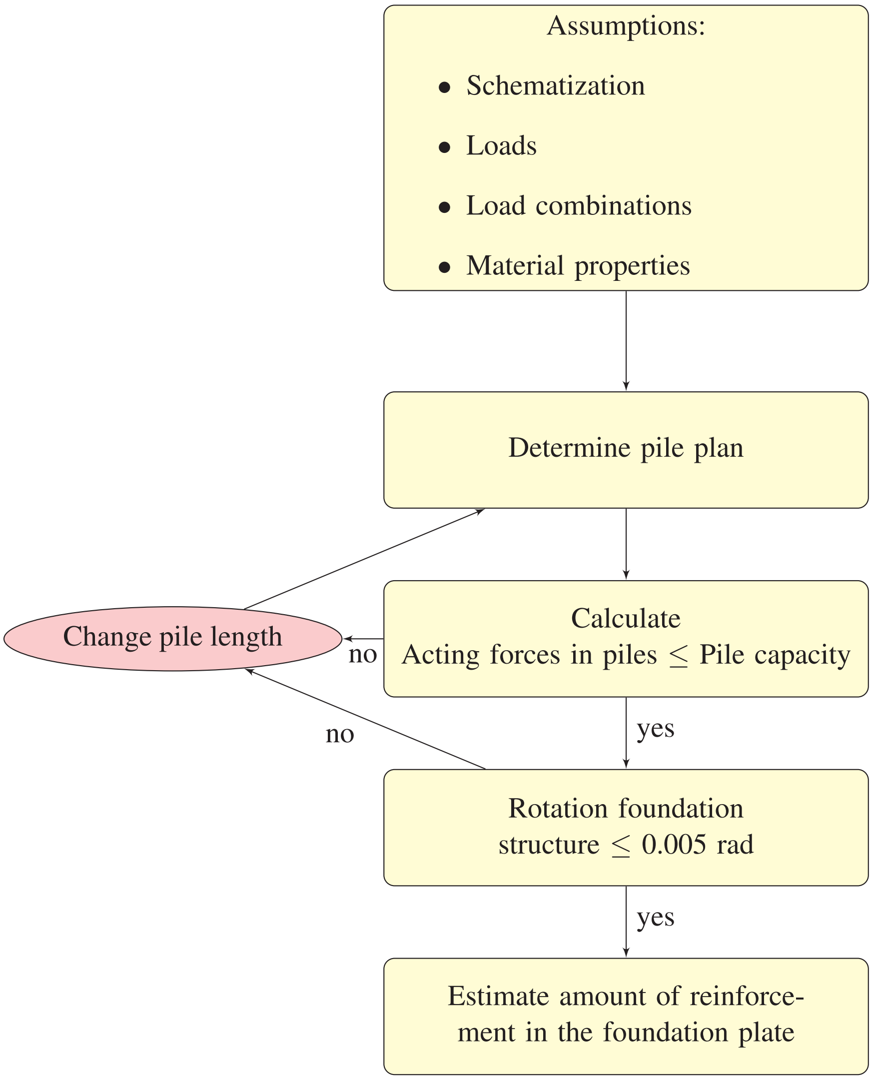


Figure 6-1: Flow chart optimization foundation structure

In the design proces as shown in figure 6-1 the variables that are set fixed are:

- Dimensions foundation structure, these are prescribed by the client.
- Amount of piles.
- The pile type used will be prefabricated concrete piles (400 x 400 mm) as also used in the research in the previous chapters.

So in the optimization process the focus will be on the pile length and the amount of reinforcement in the foundation structure.

6-1 Assumptions

In a route of a overground high-voltage connection of Tennet there are different types of power pylons:

- Bearing pylons which are in line with other pylons
- Corner Pylons
- End pylons, where the power cables go below ground level

The kind of power pylon that will be taken in account in this consideration is a bearing pylon. In figure 6-2 this power pylon is shown. This pylon is placed on a circular foundation structure whose cross section is shown in figure 6-3.

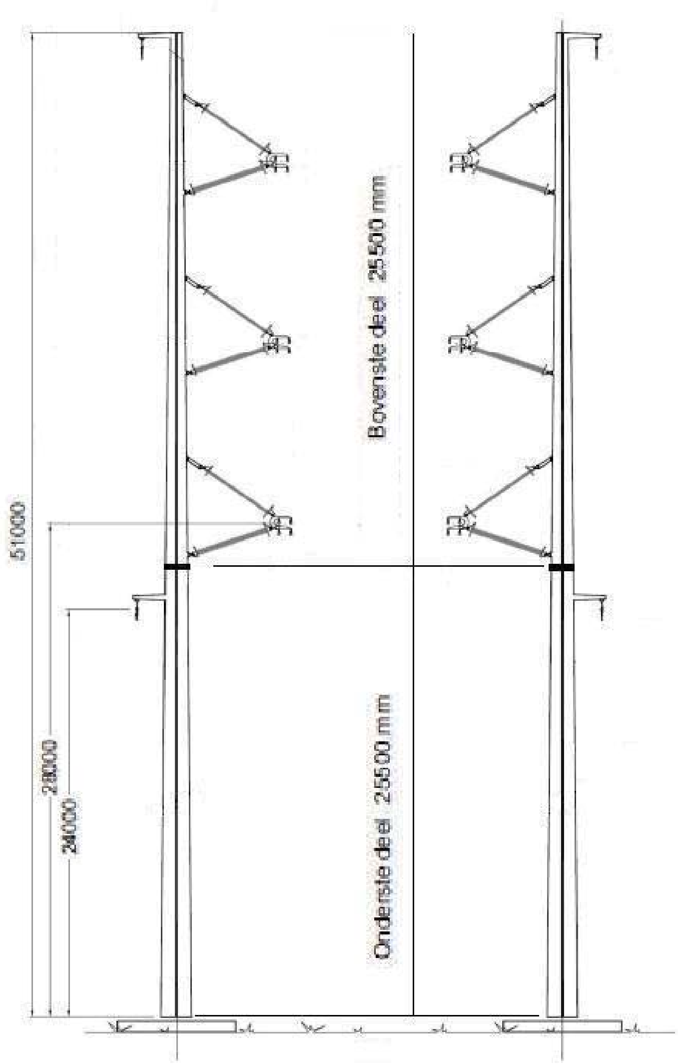


Figure 6-2: Side view of the Win track power pylon type W2S400

For the design the same soil profile will be taken into account as also used for the investigation of the models in chapter 4 and 5.

6-1-1 Schematization

In figure 6-3 a schematization of the foundation structure is shown. The foundation structure consist out of foundation plate with a diamter of 7.5 meter and a height of 0.9 m, which is supported by prefabricated concrete piles. On this plate a plinth is made on which the pylon is placed. This plinth has a thickness of 1.8 m and an diameter of 3.5 m. The forces out of the power pylon grab on at the top of this plinth as shown in figure 6-3. Other loads which works on this foundation structure are a water load, because the bottom of the foundation structure is below the water table. And a load caused by soil on top of the foundation structure.

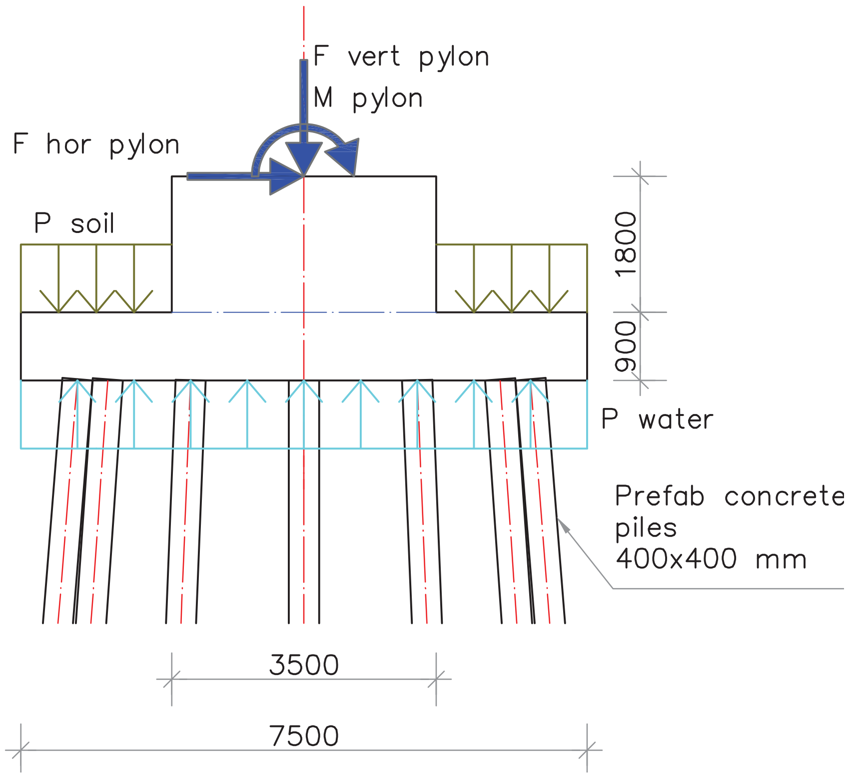


Figure 6-3: Cross section of the foundation structure with acting loads which are taken into account

6-1-2 Loads

For the design of the foundation structure the loads that will be taken into account are:

- Mass Foundation structure
- Soil load on the foundation structure
- Water loads: caused by the ground water
- Negative skin friction: downward force caused by the negative skin friction.

- Loads from the power pylon: a vertical load, a horizontal load and a bending moment.

These loads are described extensively in Appendix F-1-1.

6-1-3 Load-combinations

The foundation will be checked on different load-combinations in the ultimite limit state and serviceability limit state. These combinations are described extensively in Appendix F-1-2 and shortley described below.

Ultimate limit state

In the ultimate limit state two different situation will be taken into account. One where the foundation structure is loaded to maximum pressure. Here all the loads (own weight, structure and soil) will be multiplied with a factor 1.2 and there is no water pressure at the bottom of the foundation structure taken into account.

In the other combination foundation is loaded by a maximum tensile force. Here the soil on top is not taken into account, the mass of the foundation structure is mulitplied wih a factor 0.9 and the water pressure is multiplied with a factor 1.2.

Serviceability limit state

In the serviceability limit state only the situation in normal service life will be taken into account. So all the loads as listed in paragraph 6-1-2. In the serviceability limite state (SLS) all safety factors (γ_G and γ_Q) are set to 1.0.

6-1-4 Deformation demands

The top of the foundation structure may have a maxiumum rotation of 0.005 rad in the servicebility limite state. This means for a foundation structure where the top plinth has a width of 3.5 m, that the difference in height may be:

$$0.005 \cdot 3500 = 17.5mm$$

6-1-5 Materials

The foundation structure is made of concrete. For the calculations it will be assumed that this concrete has a Young 's modulus (E) of 15.000 MPa. Hereby is assumed that the concrete structure is cracked. There is chosen for prefabricated concrete piles (400 x 400 mm.) as also used in the investigation of the numerical models. For them a Young 's modulus (E) of 20.000 MPa will be taken into account.

6-2 Pile plan

As already stated three different pile foundations will be considedred during this optimization

- A foundation structure with vertical piles (see figure 6-4a)
- A foundation structure with rake piles (8:1) as in the original project (see figure 6-4b)
- A foundation structure where the center lines of the pile cross in the point where the resulting horizontal wind force will grap on at the pylon as shown in figure 6-4c. This point is determined by dividing the bending moment on the foundation structure by the horizontal force. These forces are provided by the supplier of the power pylon.

These different pile foundations are shown in figure 6-4

In the design proces as shown in figure 6-1 the variables that are set fixed are:

- Dimenions foundation structure: this is prescribed by the client.
- Amount of piles: in the original design the foundation slab is supported by 12 piles. This is also taken into account for this investigation.
- Pile type: the prefabricated concrete piles (400 x 400 mm) as also used in the research in the previous chapters will be used in this design.

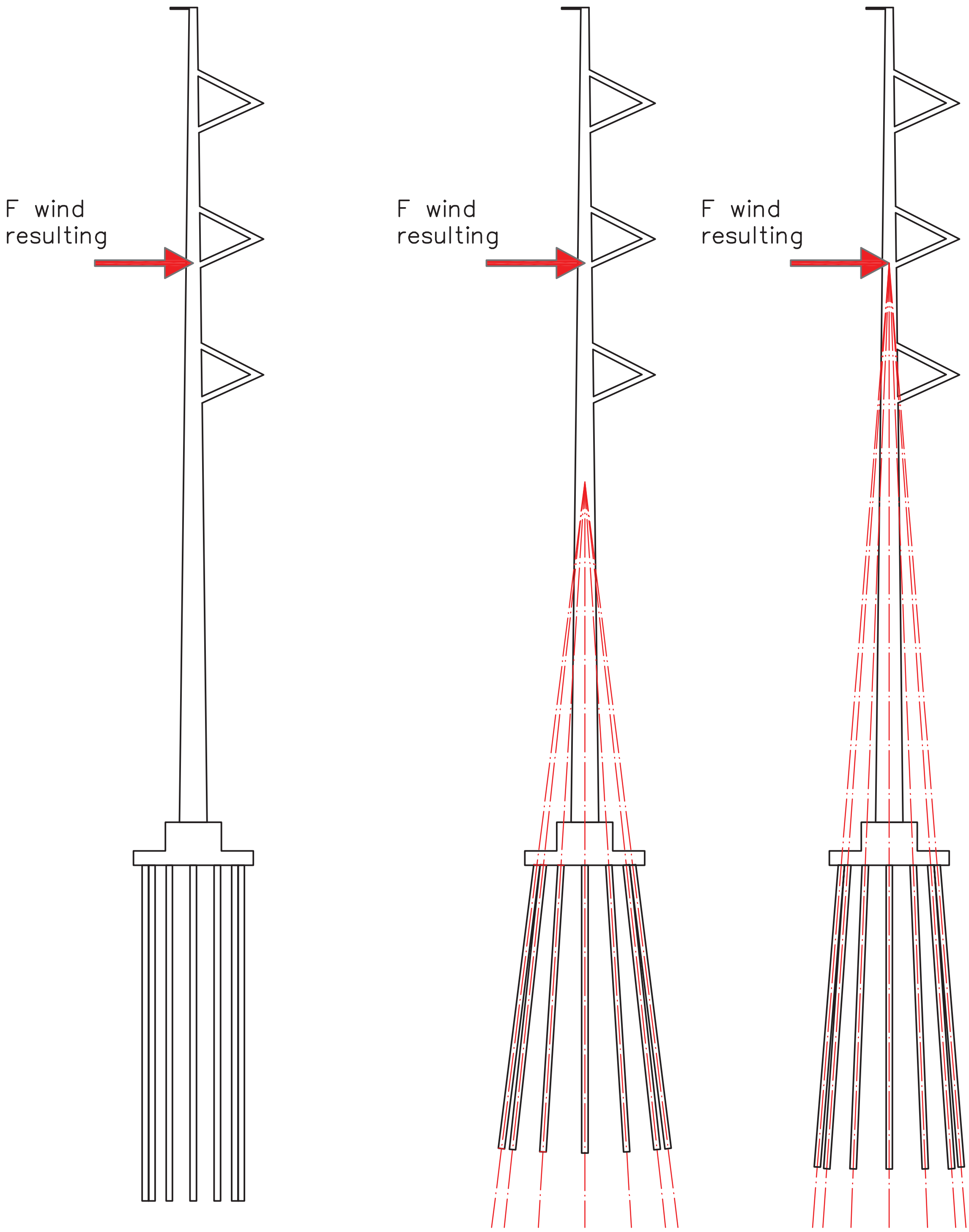
The optimization will start with the original pile tip level of the original project (-23.56 m. NAP) which is also used in the investigation of the numerical models.

6-2-1 Vertical support

The vertical support of the soil against pile will be done with springs which are based on the bearing capacity of the pile in the serviceability limit state. The base spring is placed at the pile tip as already described in paragraph 4-4-1. In the investigation of the numerical models in chapter 4 and 5 the shaft bearing capacity was modelled by seperate springs which are placed at discrete nodes along the pile shaft which where placed at every meter along the pile shaft on the part where positive skin friction is taken into account. This is a quite laborious method especcialliy as one wants to change the pile lengt. Therefore in this proces the shaft friction is represented by one spring which is placed at a height along the pile shaft where half of the shaft bearing capacity is mobilized. This method will make the modelling and interpreting of the result significant easier.

6-2-2 Horizontal support

The horizontal support of the piles is done with the method of Ménard and the reduction of the subgrade reaction because of the group effect is taken into account by the method of Reese and van Impe.



(a) Group with vertical piles **(b)** Rake piles 8:1 **(c)** Rake piles 12.4:1

Figure 6-4: Three different considerend pile groups

6-2-3 Results

From the check on the bearing capacity followed that the outer pile loaded by tension is the normative pile. This is the case for the load-combination where the tensional force is maximal in the ultimate limite state. In figure 6-5 one can see that the original pile length (15.56 m.) does not fullfill the requirements, because the occuring tensile force ($R_{s;d}$) on the outer pile is bigger than the tensile bearing capacity ($R_{t;d}$) in the ultimate limite state. This is the case for all the pile groups. The bearing capacity for the rake piles is higher, because the centre-to-centre distance of these piles is bigger. The decrease of bearing capacity of tension piles in groups is caused by destressing of the soil caused by tensional load of the group (f_2). The decrease in bearing capacity due to this effect decreases when the pile-to-pile distance increases.

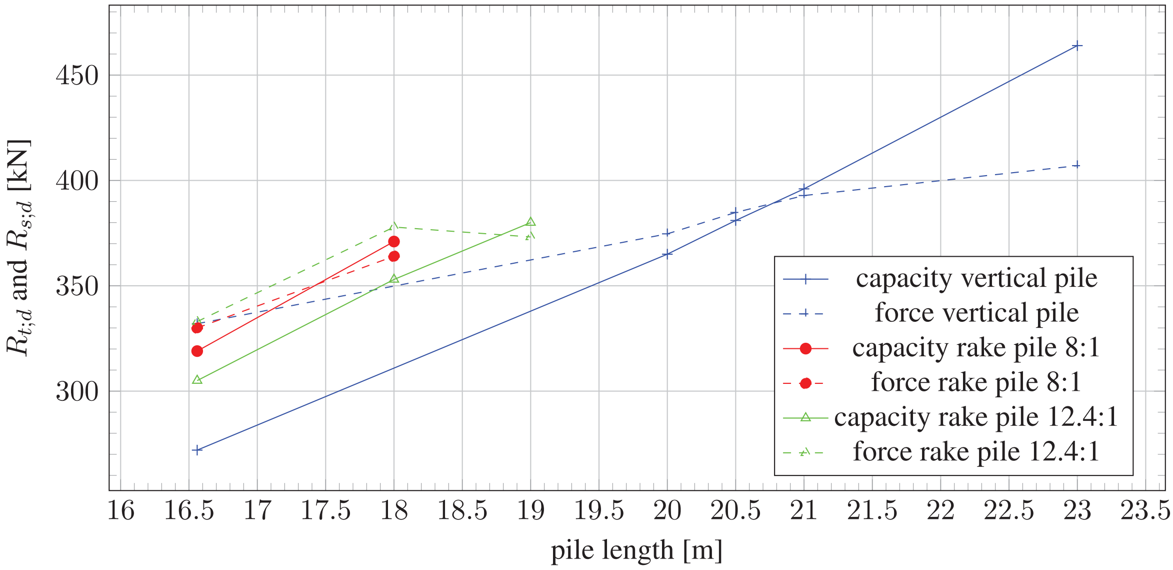


Figure 6-5: Tensile bearing capacity ($R_{t;d}$) and acting tensil force ($R_{t;d}$) in the Ultimite limit state (SLS) for the pile which is loaded by tension the most

When one makes piles longer and calculate a pile with a tensile bearing capacity larger than occuring tensile load of the original pile plan this will not always lead to a sufficient pile plan as one can see in figure 6-5. This because when one makes the piles longer to increase the bearing capacity also the stiffness of the pile increases. And a stiffer pile can attract more load. so therefore it can be possible that one need a few more iterations to get a sufficient efficient pile plan. In Appendix F-2 the tensile bearing capacity for the different piles and the load they attract is given. Here also the springs which mobilized the tensile bearing capacity are given.

In table 6-1 the pile lengths are given which are required to fullfill the bearing capacity (compression and tensional) in the serviceability and ultimate limit state for the different pile configurations. Here one can see that the structure with vertical piles needs the longest piles. The larger the rake of the piles the shorter the piles may be. This is mainly caused by increasing tensile bearing capacity of the rake piles. On the other hand the normal force in the pile is also bigger due to the distribution of forces in the foundation structure.

Table 6-1: Comparion of the different foundation structures

	Vertical piles	rake 8:1	rake 12.4:1
Pile length $[m]$	21.0	18.0	19.0
Rotation cap $[rad]$	0,00087	0,00080	0,00076
U.C. Rotation	0,17	0,16	0,15
Horizontal displacement $[mm]$	5.9	-0.5	1.6
Reinforcement upper layer	$\phi\, 16 - 120$	$\phi\, 16 - 155$	$\phi\, 16 - 155$
Reinforcement bottom layer	$\phi\, 16 - 110$	$\phi\, 16 - 105$	$\phi\, 16 - 105$
Mass reinforcement foundation plate [ton]	2.41	2.21	2.21
Bending moment pile top leading pile ULS [kNm]	37	33	13
Bending moment pile top back pile ULS [kNm]	32	42	18

6-3 Rotation

The rotation demands in the serviceability limit state are not hard to fullfill as shown in table 6-1. This can partly been caused by the fact that a situation where the soil on top of the slap is removed is not taken into account. One can see that the structure with vertical piles rotates the most. The foundation structure which the centre lines of the piles cross the point where the resulting horizontal forces grab on (rake 12.4:1) will have the least rotation. This is in line with the findings in section 5-5. In all the different variants the rotation of the cap is in the same direction. This because the bending moment coming from the pylon is the dominant load for the deformation of the foundation structure.

6-4 Reinforcement

6-4-1 Foundation slab

To have an idea how the different pile configurations influences the foundation structure an estimation is made of the reinforcement in the different variations. The amount of reinforcement is only estimated in the foundation plate. Only the main reinforcement in the top and bottom layer is taken into acount. Shear reinforcement and other kinds of practical reinforcement are not taken into account in this estimation. In Appendix F-3 is shown how the reinforcement is determined.

The needed amount of reinforcement in the foundation plate is also shown in table 6-1. Here one can see that the amount of reinforcement needed in plate is the same for the structure with rake piles. For the foundation structure with vertical piles a the amount of reinforcement needed is ca. 9% larger.

6-4-2 Forces in the pile heads

In table 6-1 also the bending moments in the pile heads are shown. The bending moments are for the leading pile which is subjected to the maximum compression force and the trailing pile which is subjected to the maximum tensional force. In paragraph 4-6-3 is already shown that the prefabricated pile has a capacity of 140 kNm in the ULS and a capacity of 122 kNm in the SLS. So the occuring bending moments in the pile can be resist easily by the piles. However it should be noted that in the piles loaded by tension the capacity to resist bending moments will decrease a bit.

Furthermore one can see that the bending moments are quite dependent on the rakeness of the piles. The structure with rake piles 12.4 : 1 has a significant lower bending moments in the pile heads.

6-5 Conclusion

In this chapter the foundation structure of a power pylon is considered. This to see how the gathered knowledge of this thesis can be used in a case from the engineering practice. The foundation structure is only considered with the numerical model Scia Engineer, because:

- Besides D-Foundation, which will be used to determine the (tensile) bearing capacity only one model is needed for foundation and construction calculations and no interaction between different models is needed.
- In this model the springs which represent the (tensile) bearing capacity of the soil are easy to adjust to take into account the folling aspect wich are prescribed in Eurocode 7:
  - Increase in tensile bearing capacity due to installation effects (f_1).
  - Decrease in tensile bearing capacity due to tensional loading of a pile group (f_2).
  - Decrease in tensile bearing capacity due to alternating loads (γ_{var})

  When one wants take into account these effects in the numerical model D-Pile one has to make seperate soil profiles for the different piles in the group.

In this optimization process different pile configurations are considered. During this process the tensile bearing capacity in the ultimate limit state showed up to be normative.

The different pile configurations has a significant influence on some of the demands which the foundation structure has to fullfill

- In a structure with rake piles one can save ca. 17% on pile length due to increasing tensile bearing capacity and a different distribution of forces through the foundation structure.
- The influence of the rake of the piles on the rotation of the structure is negligible. The rotation was also not normative for the load-combinations taken into account.
- The amount of reinforcement in the foundation plate for the structures with rake piles was ca. 8% less than for the structures with vertical piles.

- The bending moments in the pile heads are not hard to resist by the piles. Hower a smart choice of the rakeness of the piles has a positive effect on the bending moments in the pile heads.

For this reference case the pile plan where the centre lines through the piles intersect at the point where the resulting horizontal force grab on at the structure are very efficient, because:

- The bending moments in the piles are significant lower.
- The rotation is a bit less than for other structures.
- Although it is not a requirement for the structure the horizontal displcement is less than for a structure with vertical piles.

However this type of foundation has also some disadvantages:

- The centre-to-centre distance of the piles is closer by than for rake 8:1 so the tensile bearing capacity of the piles is a bit less, which leads to piles 6% longer piles.
- It is questionable or it is possible to realize the desired rake of the piles exactly in the field.

Chapter 7

Conclusions and recommendations

7-1 Conclusions

For the modelling of displacement piles the numerical models D-Pile, Scia Engineer and Plaxis are compared with each other and the behaviour as described in Eurocode 7 and CUR 77. Hereby the following assumptions are made:

- In the numerical model D-Pile the mobilization of the shaft bearing capacity is governed by the standard b-linear relations in this model. The mobilization of the base bearing capacity is governed by empirical relationships which are user defined. Herefore the relationships as prescribed in Eurocode 7 are used. The horizontal support is governed by non-linear P-Y curves.
- In Scia Engineer the mobilization of the bearing capacity is governed by non-linear springs. Herefore the relationships as given in Eurocode 7 are used. The horizontal support in this model is regulated by linear springs according to the method of Ménard.
- In Plaxis 3D the piles are modelled with the embedded pile function. The soil behaviour is governed by the Hardening Soil model with small strain stiffness.

For the behaviour of single piles the main findings are:

- In the numerical model Plaxis it is difficult to get a reliable load-displacement behaviour according to the method of Eurocode 7 for compression piles. This because the installation effects, densification and stressing of the soil around the pile, are not taken into account in this model. Furthermore the soil properties derived from table 2b in Eurocode 7 are conservative. The models D-Pile and Scia Engineer can properly take these effects into account because their behaviour is based on empirical relationships.
- The load-displacement behaviour of tension piles can be analysed properly with all three different models. The different results of Plaxis for compression piles is caused by the behaviour around the tip which is not applicable for tension piles. However there also some differences.

The load-displacement in D-Pile is linear. In Scia Engineer and Plaxis this behaviour is non-linear. The load-displacement-behaviour in Plaxis is also significant stiffer than the other models and the behaviour according to CUR 77.

- For the analysis of single piles loaded by external horizontal forces the result of D-Pile and Plaxis will approach the real soil behaviour more than the method used in Scia Engineer. This because they give a stiffer lateral support at small displacements and plasticity of the soil is included in these methods. In Scia Engineer there is chosen to model the lateral support with the method of Ménard. This method gives a linear support which is less stiff than the methods in the other methods at small displacements and does not include failure at large displacements.

The fact that the load-settlement behaviour of ground displacement piles in Plaxis 3D deviates significant from the behaviour as can be expected from the empirical relationships in Eurocode 7, has lead to the choice that Plaxis is not furthermore considered in this thesis. Besides the behaviour of single piles the study is expanded for pile groups. The behaviour of pile groups is dependent on the centre-to-centre distance of the piles and the thickness of the plate on top. For a group loaded by an axial force applies:

- In case of an axial compression load the additional settlement, relative to the settlement of a single pile, of the group is dependent on the centre-to-centre distance of the piles. This is also the case for the load-settlement relation of the different piles in the group. However, both these phenomena can best be considered using the numerical model D-Pile. The magnitude of the results depends strongly on the chosen Young 's modulus of the subsoil below the pile tip. The Young 's modulus as derived from table 2b of Eurocode 7 will give larger settlements and a bigger difference in stiffness than one can expect. When the settlement of the group is derived by the method of Koppejan, which combines the methods of Terzaghi and Keverling Buisman, the settlement is more than 50% less.
- The ratio between the thickness of a pile cap and the centre-to-centre distance of the piles also influences the deviation of forces in a structure and the settlement of the individual piles. This effect cannot be taken into account by D-Pile, however structural models such as Scia Engineer can. When the ratio between the cap thickness (h) and the centre-to-centre distance of the pile is smaller than 0.3 is found that the assumption of an infinite stiff cap (as in D-Pile) does not give the right distribution of forces.
- The bearing capacity of piles loaded by tension depends on the centre-to-centre distance of the piles. This effect is not taken into account by the numerical model D-Pile so it has to be taken into account manually. In Scia Engineer this effect can be taken into account by adjusting the springs which represent the shaft bearing capacity. The stiffness of these springs depends on the tensile bearing capacity, which has to be determined manually.

For a horizontally loaded pile group the effect of the different springs used in D-Pile and Scia Engineer is clearly visible in the results. The stiffer behaviour of the P-Y curves at small strains used in D-Pile in comparison to the Ménard springs in Scia will lead to less horizontal deflection of a horizontally loaded foundation structure in D-Pile. This also leads to smaller bending moments in the pile heads. When the springs according to the P-Y curves are implemented in Scia Engineer the results of D-Pile and Scia Engineer have the same order of magnitude.

Besides the chosen model also adjusting the rake of the piles can have positive effect on the structure:

- When the piles are placed rake their tensile bearing capacity increases, because their centre-to-centre distance increases. In this case it may be possible to apply shorter piles.
- When the centre lines of the piles cross each other in the point where the resulting horizontal force acts at the structure this has a very positive effect on the deformation of the structure and the bending moments in the piles.

In this thesis the effects which play a role by pile foundations and the ability of the different models are considered. Furthermore from the investigation followed which effects are normative for the design of the power pylon in the considered soil profile. This proved to be the tensile bearing capacity of the piles. Combining all these knowledge leads to the conclusion that Scia Engineer seems to be the best model for the design of the foundation structure of the power pylon. Hereby both the efficiency of the design process and the structure are taken into account. The reasons for this are:

- Besides D-Foundation which is a tool to determine the bearing capacity only one model is needed to analyse the behaviour of the piles and the foundation structure. This model is also used by the structural engineer to determine the distribution of forces and the reinforcement in the structure.
- The load-displacement behaviour of (tension) piles is governed by springs which are easy to adjust and it is easy to check if the behaviour is in line with the applicable standards. This is especially the case for tension piles, for which the bearing capacity is dependent on the pile-to-pile distance and the alternation between tensile and compressive loads.
- Uncertainties about the properties in the soil can be taken into account by varying the applied springs which represent the soil behaviour.

7-2 Recommendations

For a structure which is sensitive for the load-displacement behaviour of the foundation it is important to have this behaviour incorporated on the right way in the numerical model. This behaviour can be based on relationships given in standards or the results of tests (laboratory or field). This ideal behaviour can be reached by curve fitting the results of the model on the results one wants to approach. Furthermore it is important to look in the model which parameters guide the load-displacement behaviour and adjust these on the standard which is applicable.

When there are soft layers present below the pile tip this can have a signicant influence on the load-settlement behaviour of the group and the stiffness of the individual pile in the group. In such a case D-Pile can be a good tool to analyse the behaviour of the different piles in the group. With this model springs can be determined which can be used by a structural engineer.

When a structure is subjected to a large horizontal load and this leads to deformations in the structure or forces in the piles which exceeds the requirements it can be beneficial to use other springs, for the representation of the horizontal subgrade reaction, instead the springs determined with the method of Ménard. Besides using the numerical model D-Pile to model the whole foundation structure it, also can be used to determine the P-Y curves, which are also given as output by the model, and use them in the model of a structural engineer.

In this thesis the numerical model Plaxis is only briefly taken into account, because it was not possible to model a load-displacement behaviour of the pile as expected from the standards without additional measueres, like applying a volume expansion around the pile or placing an additional spring at the pile tip. This extra effort can be nessecary or beneficial by complex geotechnical problems. As for example a pile-raft foundation where the plate-soil-pile interaction plays a role which can only taken into account by a finite element model like Plaxis.

In figure 7-1 a flowchart is shown which can be used to determine or for the design of the foundation an additional model can be necessary. Hereby the effects as discussed in this research are taken into account.

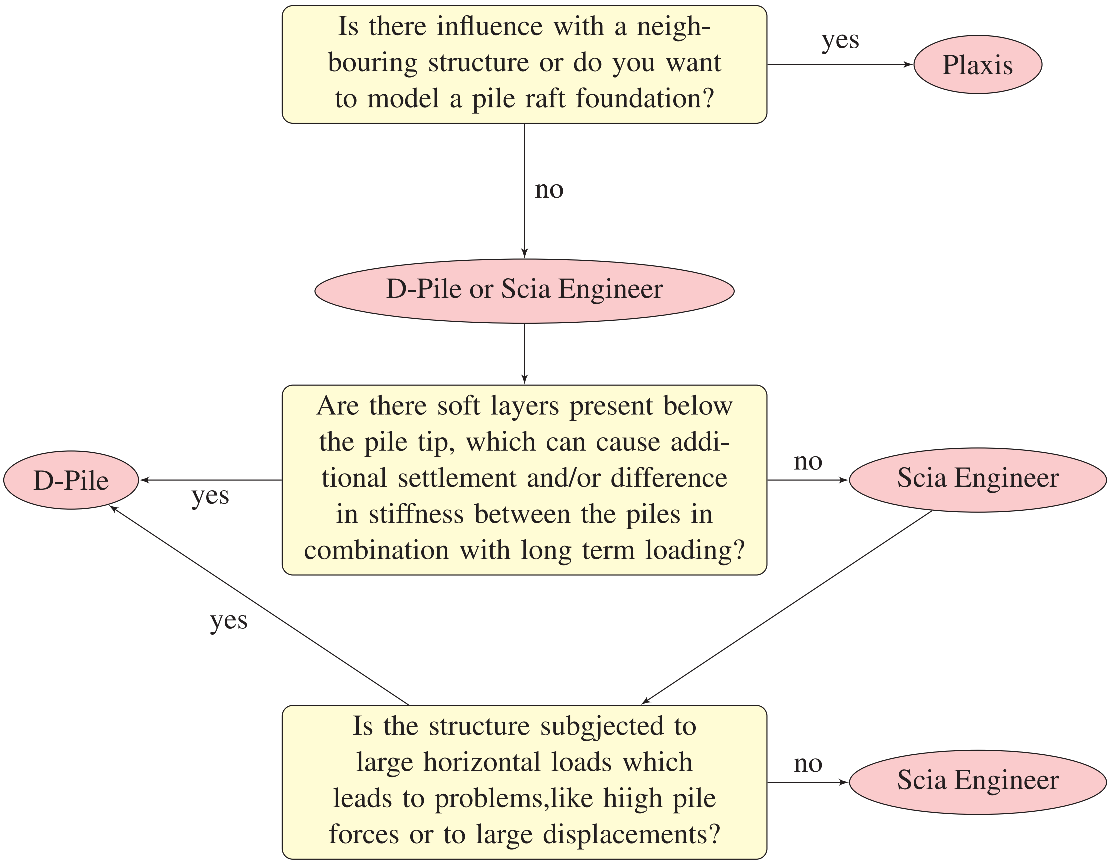


Figure 7-1: Flow chart when to choose the model D-Pile

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