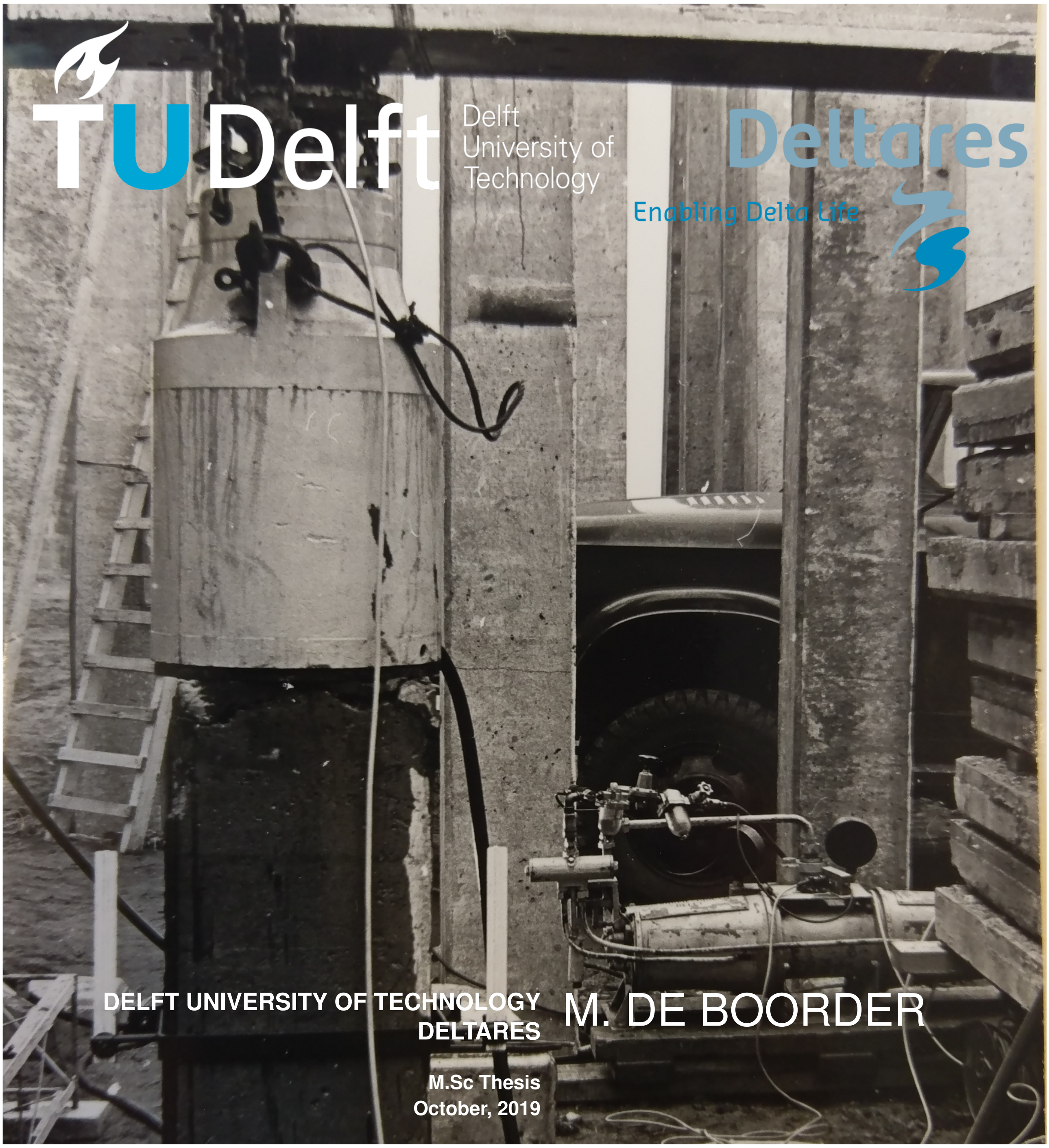

DELFT UNIVERSITY OF TECHNOLOGY
DELTARES

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M.Sc Thesis
October, 2019

Development of a new CPT averaging technique and review of existing CPT based methods for the calculation of total pile capacity

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Development of a new CPT averaging technique and review of existing CPT based methods for the calculation of total pile capacity

by

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to obtain the degree of Master of Science in Geo-Engineering at Delft University of Technology

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Cover picture: Photo of a pile load test by Grondmechanica Delft, 1977

TU Delft

List of symbols

Symbol	Description	Unit
α_p	Reduction factor for base capacity calculations	-
α_s	Reduction factor for shaft capacity calculations	-
δ_f	Interface friction angle	$^\circ$
$\Delta\sigma'_{rd}$	change in radial stress due to loading stress path	kPa
ΔL	Length segment	m
Δr	Dilation	mm
ϕ	Internal friction angle	-
σ'_{v0}	Effective vertical stress	kPa
τ_f	Local ultimate shaft resistance	kPa
a	Cone area ratio	-
A	Area	m^2
D	Diameter	m
D_{CPT}	Diameter of CPT cone	m
D_r	Relative density	-
f	Damping factor	-
f_s	Sleeve friction	kPa
G	Shear modulus	-
h	Height above the pile tip level	m
IFR	Plugging ratio	-
O	Circumference or perimeter	m
p_a	Atmospheric pressure	kPa
p_{ref}	Reference stress	kPa
$q_{b,0.1}$	Bearing capacity at 0.1D displacement	MPa
q_{ult}	Ultimate bearing capacity	MPa
q_c	Cone resistance	MPa
q_{c1N}	Cone resistance normalized by effective vertical stress	-
$q_{c,avg}$	Average value of q_c computed through an averaging technique	MPa
$q_{c,tip}$	Cone resistance present at pile tip	MPa
$q_{c,j}$	Cone resistance at a data point	MPa
q_t	Total end cone resistance correct by pore water pressure	MPa
Q_c	Calculated capacity	kN
Q_m	Measured capacity	kN
Q_b	Pile base capacity	kN
Q_s	Pile shaft capacity	kN
Q_T	Total pile capacity	kN
R	Radius	m
R_i	Internal radius	m
s	Reshapes the weight of related to the stiffness ratio	-
u_2	Pore pressure	kPa
$W_{d,j}$	Weight of one point related to its distance to the pile tip	-
$W_{qc,j}$	Weight of one point related to the stiffness ratio	-
$W_{T,j}$	Total weight ($W_{d,j} \cdot W_{qc,j}$)	-
z	Element depth	m
z'	Depth relative to the cone tip normalized by the cone diameter	-

Preface

This thesis is written in order to obtain the degree of Master of Science in Geo-Engineering at the faculty of Civil Engineering and Geosciences of Delft University of technology. This research was supported in a joint effort by Delft University of Technology and Deltares.

This document relevant for anyone interested in consistency of CPT based pile capacity calculation methods for driven piles in sands. The investigation into the CPT based pile base and shaft calculation methods in this report was carried out due to the presence of certain limitation in these calculation methods. The focus of this report is on the development of a new q_c averaging technique for the calculation of a representative cone resistance value used in the calculation of base capacities for driven piles. The goal of this new averaging technique is to find an accurate and consistent averaging technique, which can be used in combination of a single, constant α_p factor. The new CPT averaging technique was calibrated using high quality CPT laboratory tests and then analysed through comparisons with existing CPT averaging techniques (Koppejan and LCPC) on the Deltares pile load test database. Additionally, existing base and shaft capacity calculation methods for the NEN-9997-1, were analysed for potential improvements.

The completion of this whole thesis would not have been possible without the support and patience of my supervisor at Deltares and committee member, Dirk de Lange. I therefore express my gratitude to his guidance and the valuable knowledge he passed on to me. I would also like to thank the rest of my committee; Professor Kenneth Gavin, Robert Lanzafame and Mandy Korff for their, knowledge, willingness to help, support and time that they have invested into this project. Additionally, the help provided by Ross Boulanger, Han Fei and Faraz Tehrani is greatly appreciated.

Last but not least, I would like to thank my friends and family for their perpetual support and all the motivation you have given me during this thesis.

M. DE BOORDER
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Abstract

Over the last couple years, a number investigations into the α factors used for the cone penetration test (CPT) based calculation methods for the base and shaft capacity of driven piles have been carried out. These investigation, express different concerns and limitation of the currently used CPT based calculation methods. Prompting the need for further investigation into the consistency of these methods.

For the base capacity calculation methods, the α_p factor used is accompanied by a cone resistance (q_c) averaging technique. However, some limitation have been found for the currently used q_c averaging technique, which affects the consistency and accuracy of the base capacity calculations when a constant α_p factor is used, namely, a dependency with penetration depth. Presentiments by Randolph (2003) & White and Bolton (2005), suggest that a single, constant α_p factor can be used when an appropriate averaging technique is applied in combination with the inclusion of the residual loads present in a pile. Hence, a new CPT averaging technique was developed during this study. The goal of this new averaging technique is to find an accurate and consistent averaging technique, which can be used in combination of a single, constant α_p factor. The new CPT averaging technique was calibrated using a series of high quality CPT laboratory tests with varying soil deposits. Comparisons were then made between the new and existing averaging techniques, as well as, base capacity calculation methods by applying the different averaging techniques to the Deltares pile load test database. Lastly, the effect of residual loads were investigated by applying the averaging techniques to 4 well documented pile load tests where distinctions between residual loads were made.

The investigation carried out in this report concluded that the new CPT averaging technique, developed in this study, was more accurate than the other investigated CPT averaging techniques, when comparisons were made with the CPT laboratory tests. This was also the case when comparing the CPT averaging techniques applied to the pile load database. Although the pile load database demonstrated that the new averaging technique had the least spread in results, a dependency with embedment length in the sand bearing layer was still present. This effect was not removed when pile load tests including residual loads were considered, as predicted by Randolph (2003) & White and Bolton (2005). However, the limited number of these tests calls for further research in order to confirm any conclusions.

For the shaft capacity calculation methods, the α_s factor used by the Dutch norm in particular, NEN-9997-1, was investigated. This is because the formulation of the NEN-9997-1 calculation method is believed to be too simplistic (van Tol, Stoevelaar, Bezuijen, Jansen, & Hannink, 2013). Comparisons between a variety of shaft capacity calculation methods were made by applying the calculation methods to the Deltares pile load test database and to the 4 well documented pile load tests where distinctions between residual loads were made, in order to look into the effect of residual load on the accuracy of shaft capacity calculation methods.

Additionally, results from the research carried out in this report into the α_s factor used by the Dutch norm, concluded a presence of a strong correlation between the α_s of sand layers and friction fatigue terms used in other shaft capacity calculation methods. This can be used in the future to improve the current shaft capacity calculation method used by the Dutch norm.

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1 Introduction

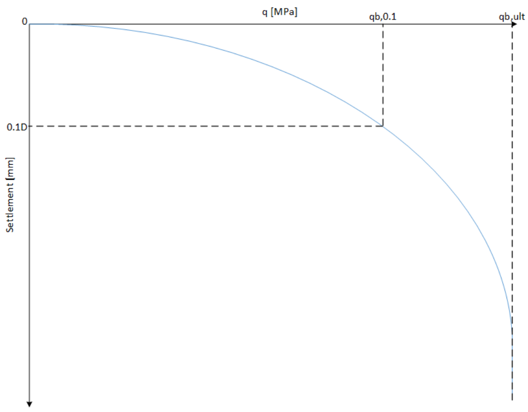
1.1. Background

The purpose of a foundation is to create a stable platform for diverse civil structures, such as, buildings, bridges and tunnels. The main requirement demanded from a foundation is that under a certain prescribed load no or little deformations should occur. These foundations come in all types of shapes and sizes in order to cope with a variety of load directions. In this thesis, a focus will be made on one foundation type loaded in one direction; close-ended driven pile foundations in sand and loaded in compression.

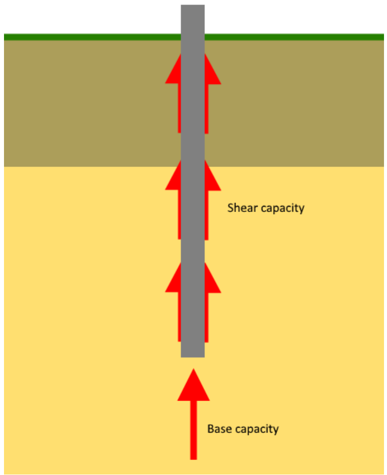
1.1.1. Pile capacity

The capacity of a pile foundation is a consensus of a certain definition when interpreting data from a pile load test. The two main general consensuses are the ultimate bearing capacity, q_{ult} and the 0.1D bearing capacity, $q_{0.1}$ (Figure 1.1a, i.e. $q_{b,0.1}$ base capacity is reached at a settlement equal to 10% of the diameter of the pile). This settlement is either measured at the pile head or pile tip, with the latter having a more physical accurate value as the elasticity of the pile has no influence on the final measured value. This effect can be significant for long piles and piles with lower stiffness where the elastic shortening of the pile can be in the magnitude of several centimetres upon loading of the pile.

The total capacity of a pile loaded in compression can be divided into two components; base capacity and shaft capacity, where the base capacity is the maximum reaction force of the soil acting against the pile tip and the shaft capacity is the maximum total friction force acting between the soil and shaft of the pile (Figure 1.1b). When estimating or calculating the total capacity given by a pile these two components are calculated separately. Early methods of estimation included effective stress and earth pressure design methods. For example, the API-69 based on effective stress and relative density which is still used today, with no significant changes implemented to the formulation since 1969 (recommended practice 2A-WSD, 2014). However, the most commonly used methods are cone penetration test (CPT) based methods, developed as early as 1952 for the estimation of the pile capacity. The CPT-based methods are more accurate compared to the effective stress methods when estimating the pile capacity. Hence, this thesis will focus only on the CPT-based estimation methods.



(a) Illustration of the capacity of a pile



(b) Illustration of the base and shaft capacity of a pile

Figure 1.1

1.1.2. CPT-based methods

A number of different methods have been formulated to calculate the total capacity of a pile based on the data obtained from a CPT, specifically the cone resistance, q_c. This is because a CPT cone behaves similar and experience the same soil failure mechanisms as to a foundation pile being pushed into the ground. The similarities between the two lead to relatively simple relations to predict the final capacity of a pile. However, distinctions are present between the two. For instance, the penetration of a cone from a CPT is displacement controlled, while a pile load test is load controlled through load cycles and allows a (time) window for creep to occur. Additionally, cone used for a CPT are jacked into the ground while the piles considered in this report are driven in the ground. The differences in the loading rate and the penetration speed therefore requires the need of reduction factors, α_p and α_s on the q_c values for a more representative strength of the soil when making calculations for the total capacity of a pile. The α reduction factors take into account different interactions between the pile and its surrounding soil as well as how this differs from a CPT and therefore vary for different types of piles. All the CPT-based methods calculate the pile capacity in slightly different ways. However, a general form can be written as:

$$Q_T = Q_b + Q_s \tag{1.1}$$

$$Q_T = q_{c,avg} \cdot \alpha_p \cdot A + \sum q_c \cdot \alpha_s \cdot O \cdot \Delta L \tag{1.2}$$

Where:

Q_T: is the total pile capacity

Q_b: is the pile base capacity

Q_s: is the pile shaft capacity

α_p and α_s: are correction factors for the base and shaft capacity respectively

$q_{c,avg}$: is an average of q_c values determined using an averaging technique (Figures 2.6 and 2.7)

A: is the surface area of the pile base

O: is the circumference/perimeter

ΔL: is the length segment over which a value of q_c acts

For the CPT-based calculation of the pile base capacity, Q_b, an arithmetic average of q_c is used. The averaging of q_c values is needed to find a representative value for the calculation of the actual end-bearing capacity (White & Bolton, 2005), since a scale effect is present between the soil mobilised by a cone penetrometer and by a foundation pile known. This zone of mobilised soil is known as the influence zone. This influence zone is a function of the diameter and is acknowledged in currently used averaging techniques (Koppejan/4D-8D (van Mierloo & Koppejan, 1952) and LCPC (Bustamante & Gianeselli, 1982)), which take an average over a number of pile diameters above and below the pile tip. This results in a representative value of q_c, which is influenced by the strength of the soil within a certain zone around the pile tip.

1.1.3. Current practice

In the Netherlands, an extensive investigation was carried out into the α factors for driven piles in sand by Deltares for the Civieltechnisch Centrum Uitvoering, Research en Regelgeving (CUR) (Stoevelaar, Van Lottum, & Rietdijk, 2009), which led to a reduction in α_p (1.0 to 0.7) used in the base capacity calculation for the Dutch construction norm, NEN (Normcommissie-351-006-Geotechniek, 2017). Engineering companies have been dissatisfied with the adjustment due to this switch to an α_p of 0.7, which has led to the application of larger piles for constructions and therefore larger costs to construction projects. The dissatisfaction is also present due to no negative incidents having occurred directly linked to the driven piles before the change in α_p. This could be due to the residual loads, which is a force acting against the pile tip developed once the driving force is removed. The residual load leads to a redistribution of the base and the shaft capacity with no change in total capacity of the pile. When the residual load is not considered an underestimation of the base capacity as well as a underestimation of the shaft capacity occurs (Fellenius & Fang, 1991). This could possibly be an explanation for the absence of incidents before the α_p change as pile tests used in this investigation did not consider residual loads. However, Deltares recommends the consideration of residual loads in future investigations. One of the points made in the Deltares report is that there is a correlation between the pile capacity and embedment length and only leads to good estimation for piles with up to 5D - 8D embedment in sand (Stoevelaar et al., 2009), a trend also noticed by Lehane and White (2005). This trend is most likely due to the influence zone considered in the Koppejan averaging technique above the pile tip being too large (8D). Randolph (2003) & White and Bolton (2005) however, both argued that with an appropriate averaging technique and if the effects of residual loads are accounted for a constant α_p can be used independent of penetration depth.

For the Dutch shaft capacity calculation method, doubts are still present with the method believed to be to simplistic to capture all the installation effects of soil displacement piles, such as, driven piles as well as the limiting value being on the low side (van Tol et al., 2013). Even though the Dutch calculation method of the shaft capacity was concluded to be adequate with the application of a limiting value on q_c in the Stoevelaar et al. (2009) report. Lehane in recent publications mentioned that design approaches incorporating a friction fatigue term, h/D, in their formulation provided the best estimation for the shaft capacity e.g. ICP-05 and UWA-05 (Lehane and White (2005) & Lehane, Li, and Williams (2013)).

1.2. Objectives

The focus of this report will be on close-ended driven piles in sand which have been tested in compression, since a range of different α factors apply to different types of piles. The main goal is to develop an averaging technique which can be used with a constant α_p to accurately calculate the base capacity of a pile, when the residual loads are considered. The new averaging technique aims to remove the most significant limitation of the currently used averaging techniques, the limitation being the dependency of the computed α_p factors having a dependency with the embedment length in the sand bearing layer. This results in a certain risk when a constant α_p factor is used with the current averaging techniques. Secondly, another goal of this report is to find out whether there is a need to improve the Dutch CPT-based shaft capacity calculation method. This is due to the recent studies discovering that the CPT-based shaft capacity calculation methods incorporating a friction fatigue term (h/D) provide the most accurate estimations. The Dutch CPT-based shaft capacity calculation method is also simplistic compared to calculation methods which include a friction fatigue term.

1.2.1. Research questions

The main issue investigated in this thesis will be: **Is it possible to find a (more) consistent method to calculate the capacity of a pile?** This research question will be answered with the following sub questions:

- Is an improvement needed in the currently used averaging techniques for the calculation of the base capacity?
- Can the consideration of residual loads help or improve the calculations of the base and shaft capacity?
- How do other shaft capacity calculation methods compare to the conventional $\alpha_s \cdot q_c$ method used in the NEN?
- Is there a need to limit the q_c values for the calculation of the base and shaft capacity as is currently done in the NEN?

1.3. Approach

To answer these research questions, first, a literature study will be conducted, which looks into the soil interactions at work between a driven pile and the soil as well as their effects on the pile capacity. This is followed up by in-depth research into the current CPT-based calculation methods and recent studies done on their accuracy as well as an evaluation of the studies, which concludes the steps and criteria of the pile load tests used in these recent studies (Chapter 2). Next, the methodology will be discussed, which will include the decisions behind the steps made leading up to the development of the new alternative q_c averaging technique. This new averaging technique is based on laboratory CPT calibration tests as well as the analysis (Chapter 3). Subsequently, the new alternative averaging technique will be presented in Chapter 4. Comparisons will be made between the different CPT averaging techniques and the laboratory CPT calibration tests. After that, currently used CPT-based calculation methods as well as the new alternative method will be further reviewed in Chapter 5, where the methods will be applied to selected pile loads from Dutch Deltares pile load test database. This will be a continuation of the previous investigation into the α factors and the limits applied to q_c carried out by Deltares on the Dutch CPT-based methods. An in-depth analysis of the comparisons between the calculated values using the CPT-based methods (Q_c) and the measured values (Q_m) from the selected pile load tests, will determine limitations and trends of the CPT-based methods and their respective α factors. Additionally, an analysis will be made on the effect of residual load on the α factors (Chapter 5). This is due to the Deltares database not including the measurement of residual loads in their pile load tests. Comparisons will be made on the CPT-based methods excluding and including the residual load, to ultimately find the effect on the dependency of α_p with the embedment of the pile. Finally, followed by the conclusion and recommendations which will present the findings of this report as well as answer of the research questions (Chapter 6).

2

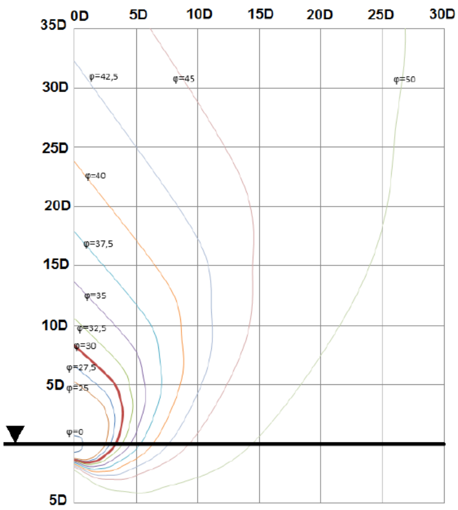
Literature study

2.1. Soil interactions and their effect on pile capacity

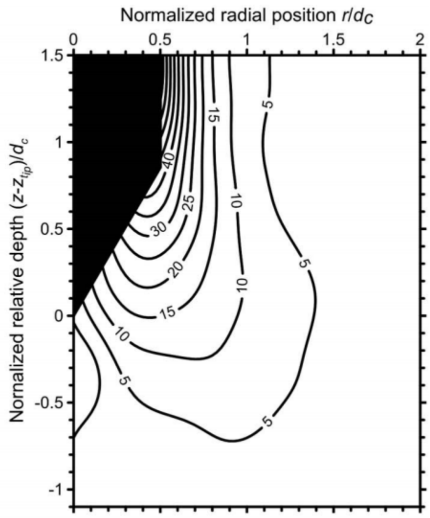
A variety of different soil interactions occur during the lifetime of a pile. The end product of the soil state after these interactions and therefore the capacity of a pile is affected by three main fundamentals: the type of soil (what is the strength of the soil? is it loose or dense?), the installation procedure (how much energy is used? how many load cycles occurred?) and the time aspect (when is the pile loaded after installation? how long has the pile been in the ground?). An overview of these fundamentals, their corresponding interactions and their effect on the capacity of a pile are presented in the next sections.

2.1.1. Soil profile

The capacity of a pile is affected by all the soil layers within the influence zone of the pile. This means that the strength and stiffness parameters of the soil within a cylindrical zone of the pile and within the influence zone near the pile tip are of importance. The basis of the influence zone around the pile tip originates from the theory of Prandtl (1921) on failure surfaces on shallow foundations and was further developed by Terzaghi (1943), Meyerhof (1951) and van Mierloo and Koppejan (1952). The shape and size of this influence zone demonstrates a high dependency on the diameter of the pile. No single definite method has been developed that is agreed by all academics however, the academics do agree that the influence zone which contributes to the base capacity of the pile depends on the diameter of the pile. Hence, the diameter dependency is widely used when deciding on the distance above and below the pile tip to be used by CPT averaging techniques for the calculation of the base capacity. The Koppejan and LCPC, for instance, use a range of $1.5D-8D$ above and below the pile tip (van Mierloo and Koppejan (1952) & Bustamante and Gianeselli (1982)) for the calculation of $q_{c,avg}$. The distance used in van Mierloo and Koppejan (1952) for the influence zone originates from Prandtl (1921), which is dependent on only the strength parameter, ϕ (internal friction angle), and the diameter (see Figure 2.1a). While, Bustamante and Gianeselli (1982) based the distance on data from 197 pile load tests. Over the years laboratory tests have however not been able to reproduce these spiral influence zones and hence discussions have surfaced on the real influence zone around the pile tip (Figure 2.1b, (Tehrani, Arshad, Prezzi, & Salgado, 2018)).



(a) Failure surfaces dependency on internal friction angle according to van Mierloo and Koppejan (1952), (Oomen, 2017)



(b) Contour plot of shear strain around the cone for uniform sand (Tehrani et al., 2018)

Figure 2.1

2.1.2. Installation effects

A number of changes occur in the soil surrounding the pile during installation of a driven pile. The main processes are: cavity expansion, dilation/compaction, grain crushing, friction fatigue and residual loads.

Cavity expansion

During installation the soil around the pile is pushed outwards. Under the pile tip the displacement is in the form of a spherical expansion while along the pile shaft a cylindrical expansion occurs (See Figure 2.2a). The extend of this displacement is dependent on the installation technique as well as the soil strength and stiffness discussed before. This expansion also known as cavity expansion, leads to compression of the soil and to the formation of excess pore pressure in the vicinity of the pile (Randolph, 2003). Therefore, a reduction in the effective horizontal stress occurs and the pile experiences an immediate reduction in shear force during installation.

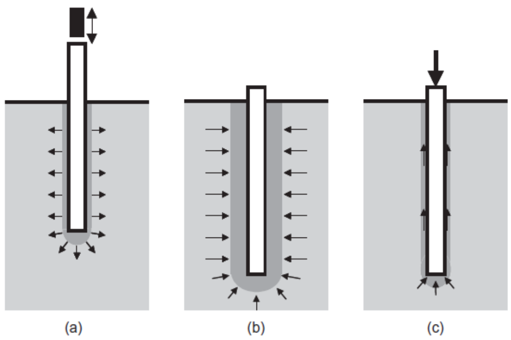


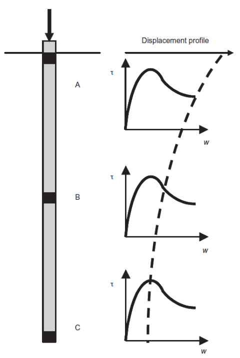
Figure 2.2: The three main phases during and after pile installation (a) Installation (b) Equalisation (c) Loading (Randolph, 2003)

Dilation, compaction and grain crushing

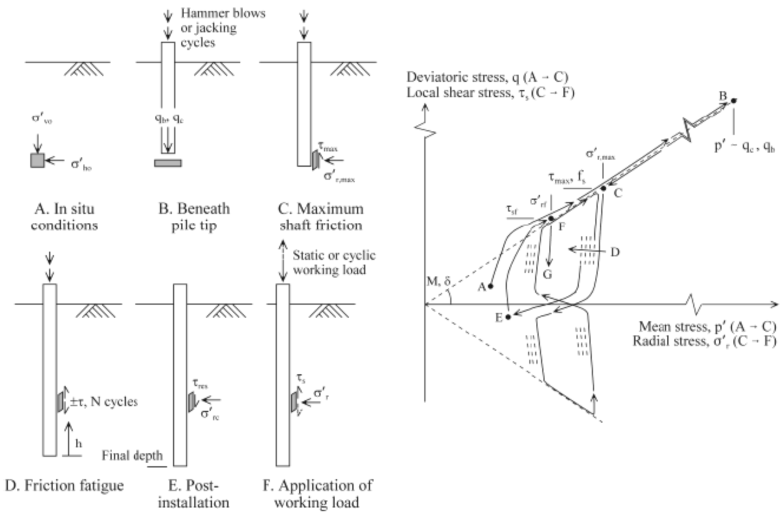
Dilation and compaction are other significant factors to consider that affect the final capacity of the pile. This is especially true for the shaft capacity. Whether the soil will dilate or compact is dependent on two factors; the relative density and the confining stress (Bolton, 1987). In general dilation will lead to a increase in shear strength and compaction will lead to an decrease in shear strength in the close vicinity of the pile. This is dependent on the soil mobilised as well as the shear cycle. At the base of the pile, on the other hand, the high stresses present during driving limit the soil to mostly compaction and grain crushing.

Friciton fatigue

During installation, the soil directly adjacent to pile undergoes severe deformation (Randolph, 2003). This is more severe for soil elements further above the pile tip as these soil elements are disturbed by a greater portion of pile shaft as the pile is installed. Randolph (1983) demonstrated that a progressive failure occurs along the adjacent soil leading to strain-softening behaviour as the number of shear cycles increases on a soil element. This is shown in Figure 2.3a and 2.3b. These shear cycles lead to the phenomenon known as 'friction fatigue' were the horizontal stress acting on the shaft reduces as the distance from the pile tip increases (White & Lehane, 2004). As the number of shear cycles increases, the interface friction angle of the soil reaches a plateau known as the constant volume stress state. This is where the soil does not experience any more volume changes (i.e. no dilation and compaction). Essentially, this involves taking a residual strength of q_c depending on the number of shear cycles experienced. A dependency of the shaft capacity on the embedment length is therefore a big talking point and it may therefore not be possible to find a single α_s factor to correctly predict the shaft capacity from q_c data, as is currently done in existing CPT based methods such as the NEN (Normcommissie-351-006-Geotechniek, 2017).



(a) Strain softening occurring during installation (Randolph, 2003)



(b) Illustration of load history of soil during the different stages of displacement piles and the stress path of a soil element adjacent to a displacement pile (White & Bolton, 2005)

Figure 2.3

Residual loads

The application of stresses leads to the development of residual loads during the installation of the pile. This residual load fully develops once the driving force is removed after the desired penetration of the pile is reached. Upon removal, residual compressive stresses remain at the pile base which are balanced by negative skin friction of the upper part of the pile shaft (Figure 2.4). This 'locked in' compressive load is known as the residual load, $q_{b,res}$ and as a result, leads to incomplete unloading of the pile (Xu, 2007). Typically the residual load, $q_{b,res}$ is in the region of 5% - 25% of $q_{c,avg}$ (Xu, Schneider, & Lehane, 2008). This is based on the UWA base capacity database for closed-ended piles in siliceous sand. Some extreme cases are known, such as the Baghdad site where the residual load was estimated to be 70% of $q_{c,avg}$ (Xu et al., 2008). If this load is not measured during a pile load test, the measured total load on the pile is not affected. However, the base capacity is underestimated and the shaft capacity is overestimated as a result of the residual load not being taken into account (Fellenius & Fang, 1991).

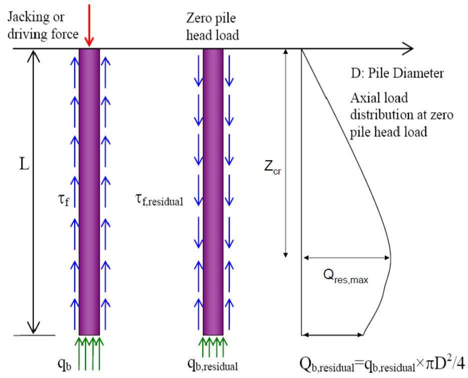


Figure 2.4: (1) Compression force on the pile (hammering/jacking) (2) Resulting residual stresses after removal of compression load on pile head (3) Axial load distribution of situation no.2 (Xu, 2007)

2.1.3. Effects of set up time

Once the pile has been installed time dependent effects start affecting the total bearing capacity. These effects can be categorised into two; dissipation of excess pore pressures and set up time. Dissipation of excess pore pressures that developed during installation will start to radially dissipate immediately after installation, resulting in the consolidation of the soil. This is a process known as equalisation (See in Figure 2.2b). 'Set up' time is a positive gain in pile capacity with time (Gavin, Jardine, Karlsrud, & Lehane, 2015) experienced due to the soil adjacent to the pile experiences a certain amount of recovery of the shear surfaces. In the case of sands, the recovery reaches an asymptote after approximately one year and only applies to the shaft capacity. The base capacity experiences little change over time (Gavin et al., 2015). Experimental data from Bowman and Soga (2005) suggests that loose sands gain strength due to volumetric contraction over time, while dense sands gain strength through a period of contraction followed by dilation. This dilation is accompanied by constrained volume changes resulting in increased radial stress due to more efficient packing over time. Skov and Denver (1988) suggested that this pile capacity increase can be described by logarithmic function of time however, more recent studies based on several pile ageing tests suggest a better fit can be achieved with a tanh curve (Karlsrud, Jensen, Lied, & Nowacki, 2014). Comparison between the two have been made by Karlsrud et al. (2014) demonstrating significant improvements to the logarithmic function by Skov and Denver (1988) (Figure 2.5).

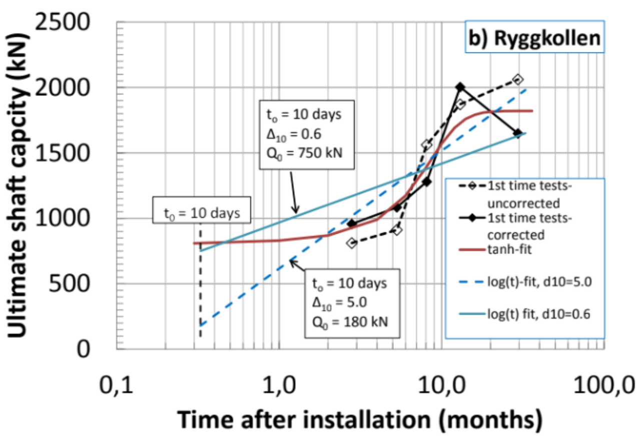


Figure 2.5: The measured shaft capacities in fresh tests at Ryggkolen (Karlsrud et al., 2014)

From the Karlsrud et al. (2014) study, an approximate 200% gain in shaft capacity is observed for piles in sand after a 2 year period with a reference capacity recorded at t = 10days. This demonstrates the importance of when the pile is loaded after installation. In the loading phase a reversal in the shear components occurs adjacent to the shaft of the pile as the 'locked in' residual load is overcome by the load applied on the pile. So that a portion of the negative skin friction developed during installation of the pile is transformed into positive skin friction. During a pile load test, however, the relatively large displacement of the pile results in all the negative skin friction to transform into positive skin friction (LGM-Mededelingen, 1982). This is where the shaft capacity originates from and is known to be dependent on a very narrow zone around the pile as well as the ageing that has occurred in that zone (Randolph, 2003). This tends to demonstrate great variety in results from pile load tests, even in pile tests carried out on the same test site. This is due to the complexity of the changes in stress in the immediate vicinity of the pile occurring during driving. These changes limit the current analytic predictions as the processes occurring during the stress changes are challenging to implement in design calculations.

2.2. Current CPT-based methods

2.2.1. Base capacity

In the Netherlands, the $q_{c,avg}$ used for the calculation of the base capacity is an arithmetic average of the q_c around the pile tip calculated through the Koppejan method also known as the 4D-8D method (van Mierloo & Koppejan, 1952). The maximum upper limit for this $q_{c,avg}$ value is set to 15MPa for driven piles in the Normcommissie-351-006-Geotechniek (2017). This is multiplied by an α_p factor of 0.7 for driven piles, which considers installation effects and set up time in a general form and hence varies for different installation methods as well as different pile type. The Koppejan method returns a $q_{c,avg}$ influenced by 0.7D to 4D under the pile tip and 8D above the pile tip of the original q_c values from a CPT, following the minimum path rule (Figure 2.6). Further correction factors are applied for the shape of the pile (see equation A.1). A more detailed explanation on the Dutch calculation method can be found in Appendix A

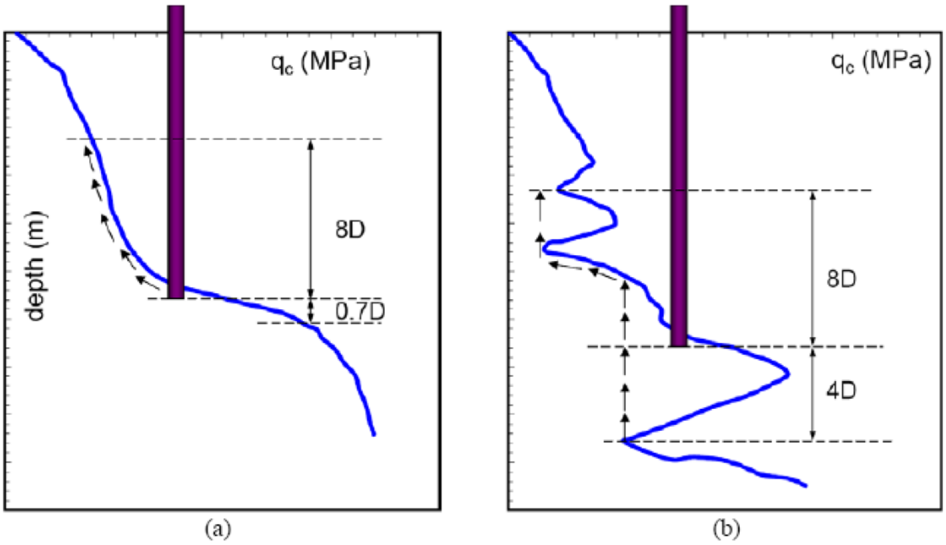


Figure 2.6: Calculation of the $q_{c,avg}$ according to Koppejan (Xu, 2007)

Where:

$q_{c.avg} = 0.5 \cdot [0.5 \cdot (q_{cI} + q_{cII}) + q_{cIII}]$

q_{cI}: arithmetic average of q_c values below the pile tip over depth which may vary between 0.7D to 4D as shown in Figures 2.6(a) and 2.6(b)

q_{cII}: arithmetic average of the q_c values following a minimum path rule recorded below the pile tip over the same depth of 0.7D to 4D

q_{cIII}: arithmetic average of the q_c values following a minimum path rule recorded above the pile tip over a height of 8D

Another arithmetic averaging method to calculate $q_{c,avg}$, is the LCPC method or also known as the French method (Bustamante & Gianeselli, 1982). The value $q_{c,avg}$ evaluates the q_c values from a CPT at 1.5D above and under the pile tip. Peaks in the q_c values are filtered out by limiting values below $0.7q_{c,mean}$ and above $1.3q_{c,mean}$ the pile tip. The steps for the calculation of $q_{c,avg}$ according to the LCPC method can be found below and illustrated in Figure 2.7.

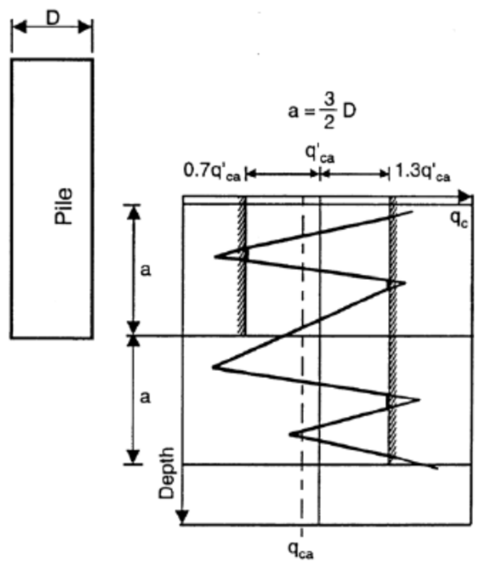


Figure 2.7: Steps for the calculation of the $q_{c,avg} = q_{c,eq}$ according to 1.5D (Robertson & Robertson, 2006)

Steps for the LCPC method:

Calculate the average tip resistance $q_{c,mean}$ as the tip of the pile by averaging q_c values over a zone ranging from 1.5D below and above the pile tip.

Eliminate q_c values in the zones which are higher than 1.3 and those lower than 0.7 multiple of the mean of the con tip resistance $q_{c,mean}$, as shown in Figure2.7

Calculate the equivalent average cone tip resistance $q_{c.eq}$ by averaging the remaining cone tip resistance q_c values over the same zone that were not eliminated.

Most of the CPT-based methods use one of the two arithmetic averaging techniques for the calculation of the base capacity. This ensures that the scale effect between a CPT cone and a pile is included in the formulation. The current state of art CPT-based methods for the calculation of the base capacity for close-ended driven piles which take the form of equation 1.1 are: Fugro-05 (Fugro), ICP-05 (Imperial College), NGI-05 (Norwegian Geotechnical Institute) and UWA-05 (University of Western Australia). The formulation of these current state of the art CPT-based methods have been presented in Table 2.1 with the addition of the Dutch norm equation for comparison. Only the UWA-05 and NEN-9997-1 use a constant value for α_p in their formulation.

Table 2.1: Design equations for the calculation of the base capacity of close ended piles for recent CPT methods (Xu, 2007)

Method	Design equation
Fugro-05	$q_{b0.1}/q_{c,avg} = 8.5 \cdot (p_a/q_{c,avg})^{0.5}$
ICP-05	$q_{b0.1}/q_{c,avg} = max[1 - 0.5log(D/D_{CPT}), 0.3]$
NGI-05	$q_{b0.1}/q_{c,tip} = F_{Dr} = 0.8/(1 + D_r^2)$
UWA-05	$q_{b0.1}/q_{c,avg} = 0.6$
NEN-9997-1	$q_{b0.1}/q_{c,avg}* = 0.7$

$p_a = 100kPa$, $D_{CPT} = 36mm$, $D_r = 0.4 \cdot ln[(q_{c,tip}/(22 \cdot (p_a \cdot \sigma'_{v0})^{0.5}), 0.3]$, $q_{c,avg} = q_c$ averaged over ± 1.5D from the pile tip according to the LCPC averaging method for ICP-05 and Fugro-05, $q_{c,avg} = q_c$ averaged using the Dutch averaging technique for NEN-9997-1 and UWA-05. (*Limited to 15MPa, see Appendix A).

The reduction in the Fugro-05 and NGI-05 calculation is related to $q_{c,avg}$ and the state parameter D_r, respectively. The larger these terms are the more reduction is applied. This is also the case for the ICP-05 calculation method however, in this case the reduction is proportional to the diameter of the pile. In the UWA-05 and NEN-9997-1 calculation methods a constant reduction is applied (α_p) to the $q_{c,avg}$ calculated through the Koppejan averaging technique. In addition to the constant reduction, the NEN-9997-1 also applies a 15MPa limit to $q_{c,avg}$. The Koppejan averaging technique is also used by the Fugro-05, while the ICP-05 uses the LCPC averaging technique. The NGI-05 does not use a $q_{c,avg}$ in its formulation and instead uses the q_c present at the pile tip.

2.2.2. Shaft capacity

For the calculation of the shaft capacity all the q_c values along the shaft are considered. In the Netherlands, the q_c values used are reduced to 12MPa for values above 12MPa, with the exception of soil layers with a minimum thickness of 1m. In this case, the q_c limit is 15MPa. This is multiplied by an α_s factor which captures

Table 2.2: Design equations for the calculation of the shaft capacity of close ended driven compression piles in sand for recent CPT methods (Xu, 2007)

Method	Design equation
Fugro-05	$\tau_f = 0.08 \cdot q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.05} \left(\frac{h}{R^*}\right)^{-0.90}$ (1)
	$\tau_f = 0.08 \cdot q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.05} \cdot 4^{-0.90} \cdot \left(\frac{h}{4R^*}\right)$ (2)
	(1) compression loading for $h/R^* \geq 4$
	(2) compression loading for $h/R^* \leq 4$
ICP-05	$\tau_f = a\left[0.029 \cdot b \cdot q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.13} \left[max\left(\frac{h}{R^*}, 8\right)\right]^{-0.38} + \Delta\sigma'_{rd}\right] tan\delta_f$
	$a = b = 1.0$ for closed-ended piles in compression
NGI-05	$\tau_f = \frac{z}{L} \cdot p_{ref} \cdot F_{Dr} \cdot F_{sig} \cdot F_{tip} \cdot F_{load} \cdot F_{mat} \geq \tau_{min}$
	$F_{Dr} = 2.1(D_r - 0.1)^{1.7}$
	$D_r = 0.4 \cdot ln(q_{c1N}/22)$
	$F_{sig} = (\sigma'_{v0}/p_{ref})^{-0.25}$
	$F_{tip} = 1.6$ for close-ended piles
	$F_{load} = 1.3$ for piles in compression
	$F_{mat} = 1.0$ for steel an 1.2 for concrete
	$\tau_{min} = 0.1\sigma'_{v0}$
UWA-05	$\tau_f = \frac{f_t}{f_c}\left[0.03 \cdot q_c \cdot A_{r.eff}^{0.3}\left[max\left(\frac{h}{D}, 2\right)\right]^{-0.5} + \Delta\sigma'_{rd}\right] tan\delta_f$
	$A_{r,eff} = 1 - IFR(D_i/D)^2$
	$IFR = \Delta L_p/\Delta z$ is equal to 1.0 for close ended piles
	$\frac{f_t}{f_c} = 1.0$ for compression
NEN-9997-1*	$\tau_f = \alpha_s \cdot q_c^*$
	$\alpha_s = 0.01$ for driven piles in sands

τ_f = local ultimate shaft resistance, δ_f = interface friction angle, $R^* = (R^2 - R_i^2)^{0.5}$, $\Delta\sigma'_{rd} = 4G \cdot \Delta r/D$, $G = 185 \cdot q_c \cdot q_{c1N}^{-0.75}$, $q_{c1N} = (q_c/p_a)/(\sigma'_{v0}/p_a)^{0.5}$, $\Delta r = 0.02mm$, $p_{ref} = 100kPa$, L = pile length, z = element depth, h = height above pile tip. (*Limited value, see Appendix A)).

the soil interactions present, such as, friction fatigue, cavity expansion and equalisation in a general form. Therefore, the α_s factor varies per installation technique. The α_s factor also varies per soil type however, this report will only focus on the α_s factor for sand. A more detailed explanation on the Dutch calculation method can be found in Appendix A. The current state of the art equations for CPT-based methods for the calculation of the shaft capacity for close-ended driven piles in sand which take the form of equation 1.1 have been listed in Table 2.2. The offshore equivalents of the ICP (ICP-API) and UWA (UWA-0S) were excluded since their equations are simplified forms of the equations in Table 2.2. In addition, the API-00 (American Petroleum Institute) approach was also excluded since the method is based on earth pressure theory (recommended practice 2A-WSD, 2014). The current Dutch norm equation for the calculation of the shaft capacity was added for comparison (Table 2.2) (Normcommissie-351-006-Geotechniek, 2017).

Most of the state of the art CPT-based shaft capacity calculation method include a reduction term for friction fatigue (h/D or h/R) as well as another term dependent on the depth which includes the effective stress, ${\sigma'}_{v0}$. This effective stress or stress level term is directly related to cavity expansion. Other terms such as dilation and interface friction are also captured in these methods, where the material of the pile is usual normative for the value of the interface friction used. Numerical reduction values are also used for the consideration of other installation effects as well as scale effects between a CPT cone and a pile.

All these CPT-based methods consider different soil interactions in their shaft capacity formulation, in order to make an accurate prediction of the real shaft capacity of the pile. Some methods consider more variables than others. The soil interactions and variables used for all the methods have been summarized in Table 2.3. Some of the CPT-based do not have a specific term for each soil interaction in Table 2.3 however, their effects are instead globally captured by a reduction value, such as α_s in the case of the Dutch method or are captured through a numerical reduction present in the calculation methods. The NGI-05 does not use a single numerical reduction value in its formulation. The reduction is instead applied through other terms such as the stress level. The NEN-9997-1 is the only shaft calculation method which does not use a friction fatigue term. This term tends to have the most influence on the final calculation value for the applicable calculation methods.

Table 2.3: A summary soil interactions incorporated in the formulations of CPT-based methods

Method	Reduction	Friction fatigue	Dilation	Interface friction	Stress level
Fugro-05	0.08	$\left(\frac{h}{R^*}\right)^{-0.90}$	[-]	[-]	$\left(\frac{{\sigma'}_{v0}}{p_{ref}}\right)^{0.05}$
ICP-05	0.029	$\left[max\left(\frac{h}{R^*},8\right)\right]^{-0.38}$	$\Delta{\sigma'}_{rd}$	$tan\delta_f$	$\left(\frac{{\sigma'}_{v0}}{p_{ref}}\right)^{0.13}$
NGI-05	[-]	$\frac{z}{L}$	D_r	F_{mat}	$\left(\frac{{\sigma'}_{v0}}{p_{ref}}\right)^{-0.25}$
UWA-05	0.03	$\left[max\left(\frac{h}{D},2\right)\right]^{-0.5}$	$\Delta{\sigma'}_{rd}$	$tan\delta_f$	[-]
NEN-9997-1	$0.01(\alpha_s)$	[-]	[-]	[-]	[-]

2.3. Review of recent studies on CPT-based methods

Over the past few decades a number of investigations were made into the different CPT based methods for the prediction of the capacity of driven piles, Q_c. The goal of these investigations was to predict the behaviour of these foundation piles more accurately. Typically, the different methods are compared to the measured capacity of a pile, Q_m, which is obtained during a pile load test. The procedure of a pile load test according to the Dutch standard (Normcommissie-351-006-Geotechniek, 2017) has been explained in Appendix A. During pile load tests, the base and shaft capacity can be obtained using strain gauges installed along the pile shaft. The accurate methods have a Q_m/Q_c mean close to 1.0 accompanied by a small coefficient of variation, CoV.

2.3.1. University of Western Australia, UWA

The most recent investigation into the suitability of these CPT based methods is by Lehane et al. (2017). The CPT based methods investigated were; Fugro-05, ICP-05, NGI-05 and UWA-05, ICP-API and UWA-05 as well as the API-00. These methods were compared to an extensive pile load test database (UWA database) which included a total of 71 tests on both open and closed-ended concrete and steel piles filtered out from 287 tests through various criteria. 74 of the 287 tests were used in the formulation for the UWA-05 method in 2005 (Lehane & White, 2005). The measured capacity was either determined by a settlement of 0.1D at the pile tip or pile head. If this value was not reached the extrapolation method of Chin (1970) was used for an estimation of the capacity at 0.1D settlement. However, due to the majority of the 74 tests lacking residual load measurement, it was opted to only evaluate the total capacity. From the findings of this paper, the ICP-05 and UWA-05 methods are one of the most consistent methods with the lowest coefficient of variation, CoV, of the Q_c/Q_m ratio and a mean close to 1.0. This is backed up by several papers (Xu et al. (2008) and Yang, Jardine, Guo, and Chow (2015)) investigating the different CPT techniques on a variety of pile load tests. The API-00 method, on the other hand, tends to over-predict the shaft capacity of long piles in loose sand and under-predict the capacity of short piles in dense sand deposits (Lehane, Schneider, & Xu, 2007). Hence, it was considered one of the least accurate calculation methods.

2.3.2. Deltares

Deltares has also carried out research on pile load tests in the Netherlands (Stoevelaar et al., 2009). A total of 25 pile load tests, which were considered of sufficient quality, were investigated. Similarly, the measured capacity of the pile during a test was determined by a certain settlement at the pile base i.e. 0.1D. This displacement was either directly measured at the pile tip or estimated with the displacement of the pile head and the elastic compression of the pile during load cycles. If this value was not reached during a test, extrapolation was used with the method of van der Veen (1953). The Deltares results showed the limitation of the van der Veen extrapolation method with an error of 10% or less only reached if the test had reached 85% of the total pile capacity at failure. Additionally, the report published by Deltares mentions that the residual loads were not measured for the pile load tests carried out in the Netherlands.

The Deltares report ultimately compares the measured pile capacity to 3 CPT based norms from different countries for the prediction of pile capacity. These three norms are the Dutch, Belgian and French norm. The results conclude that the Q_m/Q_c ratios or α_p factors for the pile base capacity are 0.70, 0.76 and 1.16 for the Dutch, Belgian and French (without the q_c limit) methods respectively. The 0.70 found has been implemented as α_p in the Dutch norm due to this investigation since 2016 (Normcommissie-351-006-Geotechniek, 2017). The ratios for the shaft capacity are 0.57, 0.99 and 1.34 for the Dutch, Belgian and French (without the q_c limit) methods respectively. With the q_c limit, the Dutch method has a Q_m/Q_c ratio of 0.94 for the shaft capacity (with an α_s of 0.01 for sand). No changes were made to the calculation for the shaft capacity in the Dutch norm as the Q_m/Q_c was deemed sufficient (Stoevelaar et al., 2009). The report concludes that for piles with a penetration-depth of up to 5D and 8D into sand, the Dutch method makes a good estimate for the total pile capacity with the use of the q_c limits. For piles with deeper penetration depth, a penetration depth relation is needed for better prediction. The presence of this relation is suggested to be due to the lack of residual load measurements. It is further stated in the report that residual loads should be measured in future pile load tests in order to accurately measure the pile base and shaft capacities (Stoevelaar et al., 2009).

2.4. Evaluation

First of all, the limitations of the two averaging methods are well documented. For instance, the Koppejan method is known to be quite conservative (Lehane & White, 2005), especially since an average is taken over an 8D distance above the pile and has demonstrated dependency with depth due to this large influence zone. This results in high α_p values when the pile tip has shallow embedment in sand underlying a weaker soil layer. The zone from which the average is taken by the LCPC method, on the other hand, is believed to be too small (i.e 1.5D above and below the pile tip). Ideally, the factor α_p is constant for all cases, however, in reality significant variation has been observed. These limitations of the current averaging techniques are why an investigation will be carried into the development of a new averaging method. The goal is to establish a consistent method for calculating a representative strength with a constant α_p for the calculation of the base capacity regardless of penetration depth.

Secondly, a talking point remains for the two different extrapolation methods when 0.1D capacity is not reached in a pile load test; which of these methods is better? and how much percentage of the maximum total pile capacity has to be reached to safely predict the 0.1D capacity threshold with these methods? Comparisons are made between the two in Stoevelaar et al. (2009). In this report, however, the extrapolation methods will not be used and will instead focus only on the measured data.

Both studies did not include residual loads. The measured total load on the pile is not affected by this, however, the base capacity is underestimated and the shaft capacity is overestimated as a result of the residual load not being taken into account (Fellenius & Fang, 1991).

Even though the Dutch calculation method of the shaft capacity was concluded to be adequate, with the application of a limiting value on q_c in the report (Stoevelaar et al., 2009), doubts are still present with the calculation method which is believed to be to simplistic to capture all the installation effects of soil displacement piles, such as, driven piles as well as the limiting value being on the low side (van Tol et al., 2013). This is especially true for soils with high values of q_c (>> 12MPa). Physically there is no meaning to limiting q_c as the limit on q_c serves as a conservative reduction. Hence, a question remains whether it is even possible to calculate the shaft capacity with a single α_s factor for sands, as is the case for the Dutch calculation method. The most accurate shaft capacity calculation methods use a friction fatigue variable in the form of h/D or h/R, which has been proven to work well (Lehane & White, 2005).

Even though these publications compare different approaches to calculate the capacity of a pile (CPT based methods and norms), the same point is always brought forward which is that all the assumptions and test criteria need to be clearly stated when investigating the different CPT based methods. This improves the credibility of the analysis and avoids confusion in future re-evaluations. It has to be noted that criteria for quality control of the pile load tests and assumptions made for the calculation of the bearing capacity can be subjective. Despite the effort for a global consensus, the criteria and assumptions used vary slightly from publication to publication. This is especially true on a local scale as each country develops its own norms. A general overview of all of the criteria used for the Deltares report and in the Lehane et al. (2017) journal article have been listed below (Table 2.4). In the end, the UWA analysis had the luxury to be more critical with its criteria due to the number of pile load tests available to them (287 pile load tests (Lehane et al., 2017)), while the Deltares analysis was based on fewer pile load tests and hence devised more straightforward selection criteria. However, further criteria was specified by Deltares for the pile load test to be included in the α_p and α_s analysis. For each α factor a series of criteria classes were made with their own value of uncertainty. Each α factor has 4 criteria classes with the value of uncertainty being 5% - 20% from classes 1 - 4. The selected criteria classes for the pile load tests can be found in Stoevelaar et al. (2009).

Finally, from the pile load test criteria considered, new criteria have been chosen for the selection of pile load tests from the Deltares database to be used in the analysis carried out in this thesis. The classes used in the previous Deltares analysis will be used as well as some additional criteria, see Table 2.4. In addition, no correction will be made to the load tests for the self-weight of the pile as the resulting force due to the self-weight being negligible compared to the force applied to the pile head during a load test. This report will also not use extrapolation methods when the 0.1D failure criteria is not met during a pile load test and will instead use the load at the maximum displacement reached. For these cases the displacement reached should be at least 80% of 0.1D.

Table 2.4: Criteria for pile load tests in the UWA paper (Lehane et al., 2017), the Deltares report (Stoevelaar et al., 2009) and this report

Criteria	Lehane et al. (2017)	Deltares, Stoevelaar et al. (2009)	This report
Installation type	Driven	Driven	Driven
Pile type	Concrete and steel	Concrete and steel	Concrete and steel
Close +/ open-ended	Close and open-ended	Close-ended	Close-ended
Minimum pile diameter	200mm	250mm	250mm
Minimum pile embedment	5.0m	9.5m	6.8m
Pile shaft	Straight	Straight	Straight
Pile tip only in sand	√	√	√
Shaft capacity component	>75% in sand	[-]	>50% in sand
Siliceous content	>75%	>75%	>75%
Intermediate soils e.g. Silt/peat	Excluded	Excluded	Excluded
CPT data available	√	√	√
First time static load tests only	√	[-]	[-]
Load displacement curves	√	√	√
No limit on number of piles from single site	√	√	√
Piles with extensive ageing	Excluded	Not within selection	Not within selection
Failure criteria	0.1D at the ground level or Chin	0.1D at pile tip or van der Veen	0.1D or max displacement reached at pile tip

[-] = not specified

3
Methodology

The analysis process for the factors α_p and α_s will be discussed in this Chapter. For α_p, the focus will be mainly on arithmetic averaging techniques used for the calculation of $q_{c,avg}$ and the development of a new method for the determination of $q_{c,avg}$. The analysis on the α_s factor will focus on the α_s factors used for sand. For both factors, comparisons will be made between the different pile capacity calculation methods. The methods to be compared are: the Dutch calculation method with a focus on the reduction factors used in the Dutch norms, the ICP-05 method and the UWA-05 method. The ICP-05 and UWA-05 were chosen since these two methods are considered the most accurate methods for the prediction of the total pile capacity in sands, setting a standard for comparisons to Dutch calculation methods. Additionally, the ICP-05 and UWA-05 methods are designed with consideration of residual load in contrast to the Dutch methods. Results from both the base and shaft capacity will then be combined to examine the accuracy of all the methods for the calculation of the total capacity of the pile. The first part of the analysis will only include pile load tests from the Deltares database, which lack residual load measurements. Once this is completed, the study will be extended by looking at the effect of the exclusion and inclusion of residual loads in pile load tests and the changes to the factors α_p and α_s.

3.1. Alpha-p

The main goal of analysing the factor α_p is to find a more consistent averaging technique for the prediction of the base capacity. This could possibly be done by finding a correlation for α_p and penetration depth for existing averaging techniques (Koppejan and LCPC). However, Randolph (2003) & White and Bolton (2005) both argued that with an appropriate averaging technique for the selection of an design value of q_c and the effects of residual loads were accounted for a constant α_p factor can be adopted which is independent of the pile diameter (Gavin, Hicks, Pisano, & Peuchen, 2018). Therefore, a new averaging technique will be developed with a constant α_p and compared to currently used averaging techniques. This will be done by taking the measured base capacity from the selected pile load tests of the Deltares database and an extension looking at a selection of high-quality tests where the residual loads have been measured, which will be divided by the calculated capacity from the averaging techniques (Q_m/Q_c) to return values of α_p. Additionally, any dependency of embedment in the sand-bearing layer, which has been known to be present from previous studies will be analysed. Further, analysis will be made into pile base capacity calculation methods, which use the $q_{c,avg}$ computed from the averaging techniques, to investigate their accuracy (UWA-05, ICP-05 and NEN-9997-1). This includes the Dutch norm (NEN-9997-1) calculation method, which uses a reduction or limitation of 15MPa on the Koppejan $q_{c,avg}$.

3.1.1. Currently used averaging techniques

Averaging techniques for q_c values are needed in order to find a representative value for the calculation of the actual end-bearing capacity. Soil layering can have a major effect on the bearing capacity of the pile especially when there is a high contrast in strength between the layers (i.e. Strong to weak or weak to strong layer of soil). Several laboratory tests have recognised that when a pile/cone tip penetrates the soil, a layer with contrasting strength can be sensed for some distance, regardless of whether the soil layer boundary is below or above the pile (Nauroy and Le Tirant (1985) & Tehrani et al. (2018) & de Lange, van Elk, and Doornhof (2018)). This sensing distance is proportional to the diameter of the pile and related to the failure zone or influence zone described by scholars, such as, Prandtl (1921) and Terzaghi (1943). These surface scale effects highlight the need of averaging techniques (Axelsson, 2000).

The two main averaging techniques used are the Koppejan/4D-8D and LCPC techniques, both having their own limitations. The Koppejan method is known to be conservative when calculating the representative value. This is due to the implementation of minimum path which is especially significant for the zone above the pile tip which has a weight of 0.5. This combined with the large number of diameters used above the pile tip for averaging (8D) generates a $q_{c,avg}$ which is dependent on penetration depth. High values of α_p were discovered for pile tips located close to soft soils and a decrease in α_p was present as the pile tips were located deeper in the sand bearing soil (Stoevelaar et al., 2009). The distance taken for averaging of the LCPC method (1.5D), on the other hand, is too small. The sensing distance and influence zone of a pile has been documented to be much larger. Recent experimental data such as tests carried out by Tehrani et al. (2018) on layered sand suggest a minimum sensing distance of 2.2D. This value would be even larger for layered soil with a much higher contrast in q_c (e.g. Clay layered over/under sand). These limitations indicate the need of a better averaging technique which works well regardless of penetration depths and soil layering. For the development of the new alternative averaging technique, an investigation was made into the filter technique proposed by Boulanger and de Jong (2018).

3.1.2. Boulanger and de Jong

The filtering procedure proposed, was created to predict the behaviour of a CPT. This method is based on the true resistance (q_c) of the soil, which is the q_c value that would be measured if the same soil was free of the influence of overlying/underlying weaker/stronger soil. In reality, a CPT experiences a transition zone in q_c values as the cone moves closer and from soil layer interfaces (Figure 3.1). The thinner the soil layer, the less probable that the q_c will reach its true resistance as can be seen in Figure 3.1. The total depth of the transition zone is dependent on the difference in the true resistance, which is related to the stiffness of the soils.

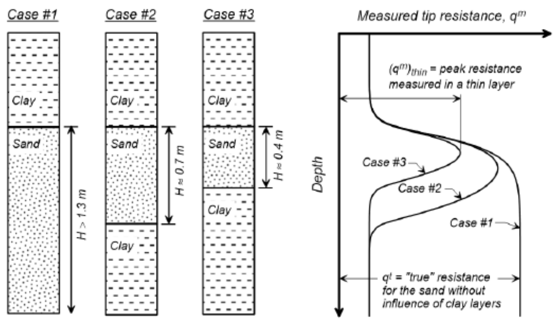


Figure 3.1: Schematic of the soil layer interface and the effect this has on q_c values modified from Robertson and Fear 1995.

In idealized two layer profiles the transition zones can be divided into the following two terms: the sensing and development distance. The sensing distance is the greatest distance between the cone tip and the top of the underlying layer for which q_c is affected by the underlying layer. While the development distance is the greatest distance between the top of the underlying layer and the cone tip for which q_c in the underlying layer is still affected by the upper layer (Boulanger & de Jong, 2018). Results from tests on idealized two layer soil profiles indicate that the sensing distance is greater than the development distance in a strong layer over a weak layer and the development distance is greater than the sensing distance in a weak layer over a strong layer (Tehrani et al., 2018). Essentially, the weak layer is felt over a larger distance by stronger layers of soil than in the reverse case.

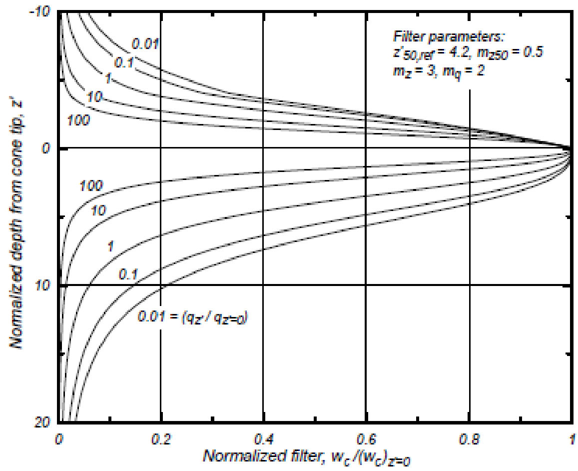


Figure 3.2: Normalized cone penetration filter versus normalized depth from the cone tip with lines for q_z/q_{z-1} = 0.01, 0.1, 1, 10 and 100 (Boulanger & de Jong, 2018)

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The Boulanger and de Jong filter model weighs-in all the q_c values along the CPT profile when looking at a single point. The weight of each CPT point is determined by the distance from the point analysed and by the q_c ratio between those two points. A variety of variables and constants are also applied making this a complex filter model to apply. The total weight factor has been plotted for different q_c ratios in Figure 3.2. However, when this filtering model was used with larger diameters, i.e. pile foundation diameters, large peaks in the q_c values were observed. Most likely due to small errors increasing exponentially with an increase in diameter. Adjustments can be made to the variables, however, the complexity and number of variables of the method makes this a time consuming task. Hence, this method is less suitable to find a representative $q_{c,avg}$. The exact formulas of this method can be found Appendix A.

3.2. Alpha-s

The main goal of analysing of the factor α_s, is to find out whether it is possible to determine a single α_s value for all the sand layers. This will be done by looking at strain gauges located in sand layers from the pile load tests of the Deltares database. From the values obtained by the strain gauge a shear stress, τ can be obtained, which when divided by the q_c value will return an α_s value. Further analysis will be carried out for different shaft capacity calculation methods (ICP-05, UWA-05 and NEN-9997-1) considering the whole shaft capacity of a pile since the calculation methods are designed for the calculation of the whole shaft capacity. Additionally, the limit applied to q_c in the Dutch norm (NEN-9997-1) will be investigated and whether the limit on q_c is needed or not. Doubts have been expressed by several academics over the simplicity of the Dutch pile capacity calculation methods. The paper published by van Tol et al. (2013) suggested that "actual installation effects with soil displacement piles are more complex than in an approach complying with NEN 9997-1". Calculation methods compromising of a friction fatigue term have been found to provide better estimates for the shaft capacity (Lehane & White, 2005). Therefore, the analysis will also look into the accuracy and hence safety of the current Dutch norm. This will be done for both the Deltares database as well as the extension looking at a selection of high-quality tests where the residual loads have been measured.

3.3. Residual load

The effect of the residual load on the pile capacity will be investigated by looking at a selection of high-quality tests where the residual loads have been measured. Comparisons will be made between these tests excluding and including residual loads, in order to observe the changes to the α_p for the averaging techniques as well as the changes in Q_m/Q_c ratios for the calculation methods of the base and shaft capacity. This will allow observation of any improvements in accuracy of the calculation methods when the residual loads are included, as predicted by Randolph (2003) and White and Bolton (2005). This investigation can be found in the extension of the analysis carried out in Chapter 5.

4

New alternative averaging technique

4.1. Development and equations

During early stages of the formulation of this method, it was clear that certain aspects of the Koppejan, LCPC, and Boulanger-de Jong techniques worked well despite their limitations. The goal was to create a method that returns a representative q_c which can be used consistently to predict the base capacity of a pile in combination with a single constant α_p. Although the Boulanger-de Jong method did not work well when calculating a representative q_c value for a pile, the method demonstrated promising results when simulating CPT profiles. Hence, several aspects from the new alternative averaging technique are inspired from the Boulanger-de Jong method. One of those aspects is the q_c stiffness ratio used in the Boulanger-de Jong method, which demonstrated promising results in dealing with the sensing and development distance. In addition to this, a weight was made for averaging values dependent on the distance from the pile tip. The weight associated to the distance is a damped cosine function. This results in values closer to the pile tip having more influence on the representative value than values further away. The distance over which this damped cosine function is applied is a product of the diameter of the pile. This weight dependent on the distance is similar to the Boulanger-de Jong method but uses a simpler function and is not applied over the whole CPT profile, instead the aspect averaging q_c values over certain pile diameters from the pile tip was taken from currently used averaging techniques, such as, the Koppejan and LCPC. Hence, the final formulation of the alternative averaging method is a weighted average dependent on the distance from the pile tip and the stiffness of the soil in comparison to the stiffness present at the pile tip. This formulation is as follows:

$$q_{c,avg} = \sum \left(q_{c,j} \cdot \frac{w_{T,j}}{\sum w_{T,j}} \right) \tag{4.1}$$

$$q_{c,avg} = \sum \left(q_{c,j} \cdot \left[\frac{w_{d,j} \cdot w_{qc,j}}{\sum w_{d,j} \cdot \sum w_{qc,j}} \right] \right) \tag{4.2}$$

$q_{c,j}$: is the cone resistance at one point

$w_{T,j}$: is the total weight of q_c at one point i.e. $w_{d,j} \cdot w_{qc,j}$

$w_{d,j}$: is the weight of one point related to its distance to the pile tip

$w_{qc,j}$: is the weight of one point related to the stiffness ratio $\left(\frac{q_{c,tip}}{q_{c,j}}\right)^s$

s: reshapes the weight related to the stiffness ratio

The cosine function used for the determination of weight related to distance from the pile tip is as follows:

$$y = e^{fx} \cdot cos(0.5\pi x) \tag{4.3}$$

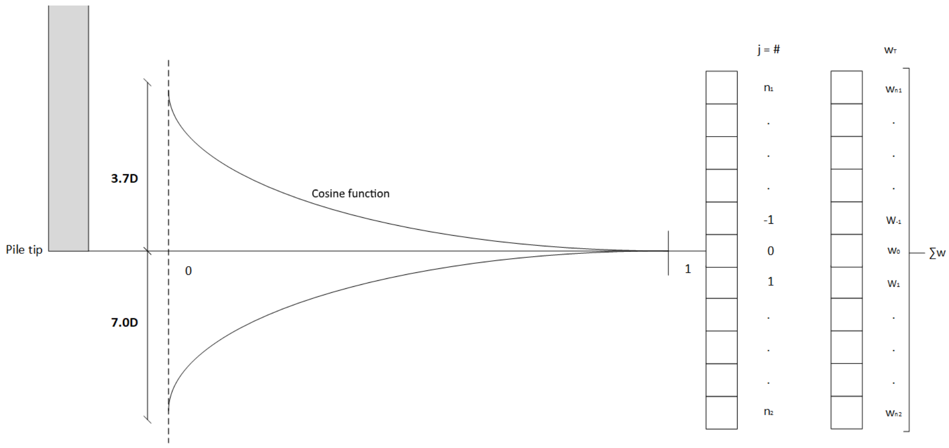

Figure 4.1: An overview of the influence zone and weights used in the alternative method

Where f is a damping factor. An illustration of the application of the new alternative method and an example of the method can be found in Figures 4.1 and Figures 4.2, 4.3 & 4.4, respectively. The example used for Figures 4.2, 4.3 & 4.4 is a simple two layered soil, with a soft soil (clay) overlaying a stronger soil (sand). The weights dependant on the distance (Figure 4.2) and q_c ratio (Figure 4.3) are calculated for two different pile tip positions (-8D and -12D). The first position is located just above the soil layer boundary and the second position just beneath the soil layer boundary. The two different weights in shown Figures 4.2 and 4.3 are combined into a total weight, as shown in Figures 4.4a & 4.4b.

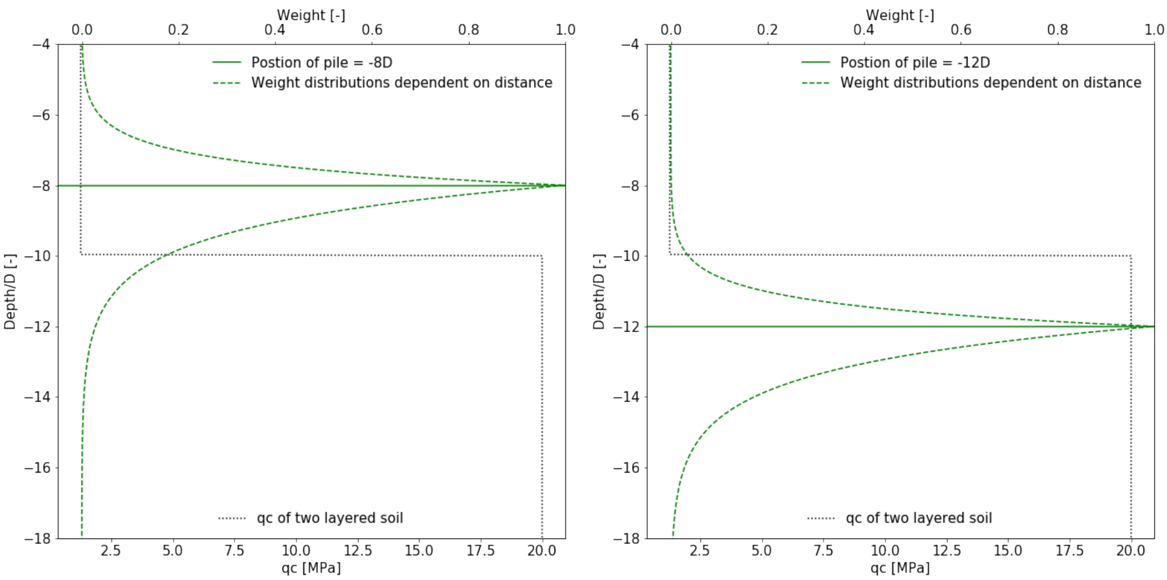

Figure 4.2: (a) Weight dependant on the distance for pile position -8D (b) Weight dependant on the distance for pile position -12D

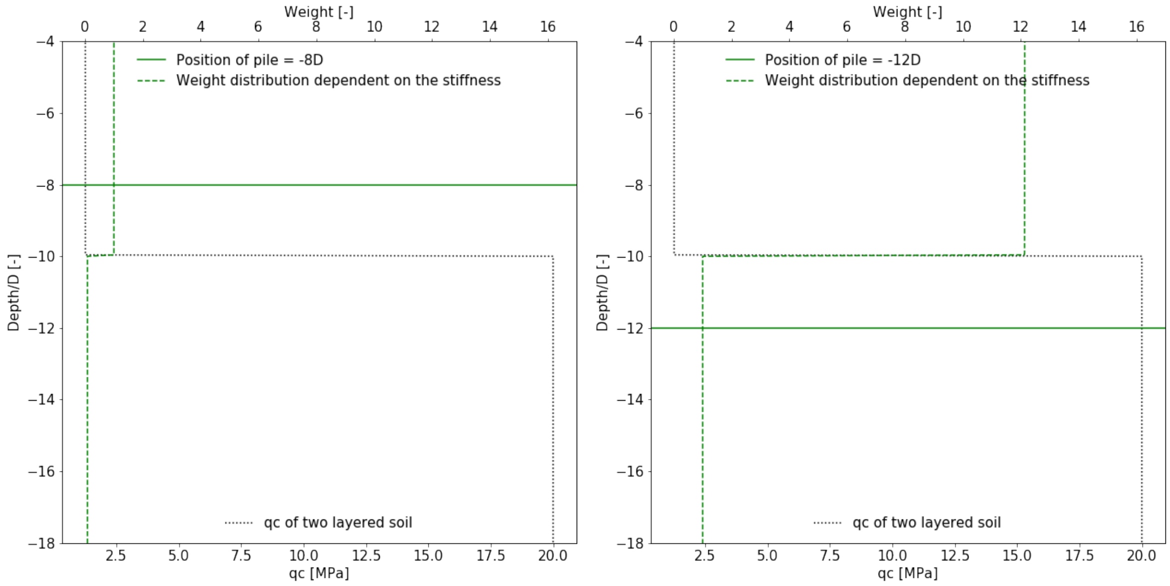

Figure 4.3: (a) Weight dependant on the q_c ratio for pile position -8D (b) Weight dependant on the q_c ratio for pile position -12D

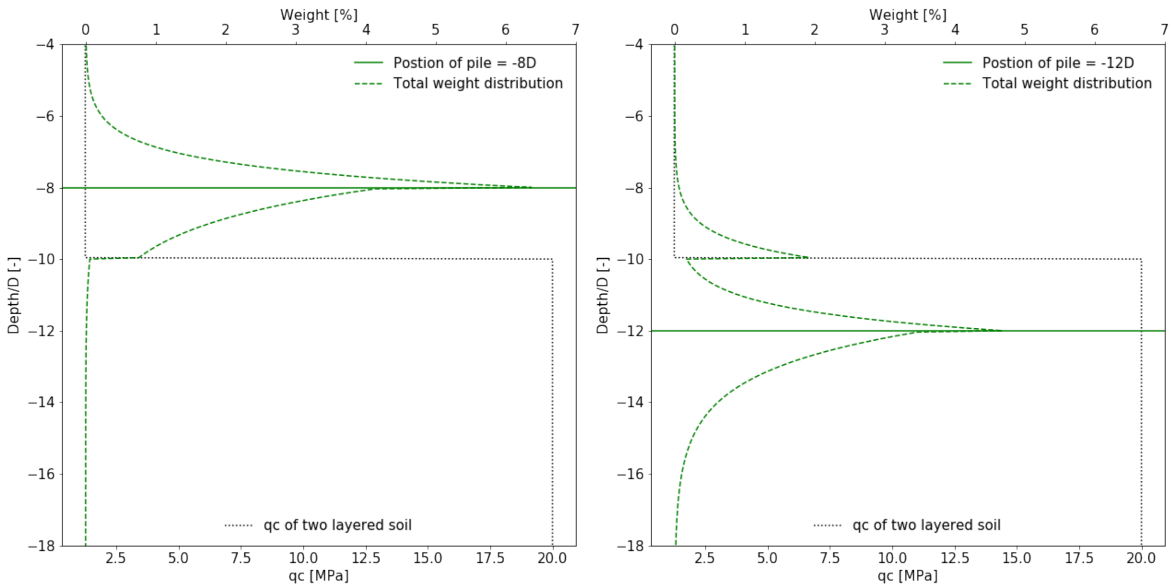

Figure 4.4: (a) Total weight distribution for pile position -8D (b) Total weight distribution for pile position -12D

4.2. Calibration

The influence zones (distance away from the pile tip taken into consideration) and damping factors were determined by applying the new averaging technique to high quality CPT calibration tests in layered soil deposits. The averaging technique was applied to a hypothetical true value of the cone resistance, $q_{c,t}$. This is the value of the cone resistance obtained by a cone penetration test for a uniform soil deposit which is not affected by transition zones. The idea behind this is to adjust the number of diameters over which values are averaged and the damping factor of the alternative method such that as much overlap occurs with the original CPT's carried out in the calibration tests and the line produced by calculating $q_{c,avg}$ for every point using the alternative averaging technique. Therefore, not only overlap through the layered part but also overlap at the transition zones of the layers. This can then be used for piles with larger diameters. For correct simulation a running mean was applied over the height of the cone tip, with the reference point being at half the cone tip height (Figure C.1)(de Lange et al., 2018). The CPT readings start when the point of the cone makes contact with the soil material and therefore had to be modified to the reference point at half of the cone tip height to match the averaging technique data. All the parameters of the alternative averaging technique were then adjusted according to the mean squared error (MSE) between the real CPT measurements and values returned by the alternative averaging technique applied over the CPT profile. Optimal values were found for the amount of diameters above and below the pile to be averaged, the damping factor through a series of iterations as well as the *s* parameter. The mean and standard deviation of the MSE for all the CPT calibration tests was recorded when changing the variables. Variables that would give the lowest mean and standard deviation were deemed as optimal values.

The optimal variables were first determined using 10 CPT calibration tests performed at Deltares. These tests were carried out with different cell pressures on layered sand and clay soil deposits with different thicknesses. The CPT data from these tests is very accurate (0.001m) and created a base for the variables set for the alternative method. The $q_{c,t}$ for the sand layers in these tests was determined using the expression by Lunne and Christofferson (1983) related to the relative density and effective stress. The $q_{c,t}$ for the clay layers on the other hand was determined using a expression dependent on effective stress based on CPT laboratory tests. These tests were a series of CPTs into a single layer of the type of clay used (de Lange et al., 2018). The iterations for the variables based on these tests can be found in Table C.2, Appendix C. Additional CPT calibration tests in layered sand were added to the MSE analysis. In total, 6 tests were added, 5 of which were carried out to investigate the transition zones of the cone resistance in CPT's by Tehrani et al. (2018) and 1 centrifuge test from an investigation of the settlement at the Heinenoord tunnel by Grondmechnica Delft. For these 6 tests the CPT data was not available digitally. The CPT data used was instead obtained by digitizing graphs in Microsoft Excel. Since this data is less accurate and the number of tests is lower than the initial 10 used, a lower factor of contribution was used on this set of data when carrying out the MSE analysis. The 6 sand tests would therefore influence the final result less than the 10 layered sand and clay tests.

Table 4.1: Iterations for the determination of the 4 variables in the alternative averaging method based on the MSE of all the 16 CPT calibration tests

Above	Below	s*	Damping factor	Mean	Standard deviation
7.8	13.5	1.0	13.5	0.2474	0.2205
7.8	14.5	1.0	13.5	0.2402	0.2198
8.0	15.5	1.0	13.5	0.2353	0.2201
8.5	15.5	1.0	13.5	0.2368	0.2206
8.3	15.5	1.0	13.5	0.2351	0.2212
8.3	**15.5**	**0.9**	**13.5**	**0.2230**	**0.2144**
8.3	15.5	0.8	13.5	0.2349	0.2188

* s, is a parameter which reduces the influence of the lower stiffness soils for values of s below 1.0

The final variables obtained through the MSE analysis can be found in Table 4.1 marked in bold. An illustration in Figure 4.1 of the alternative method applied using python demonstrate the workings of the weights in equations 4.1 and 4.2. Although the final variables for the influence zone below and above the pile tip averaged being 8.3D and 15.5D respectively, in reality the areas averaged are much smaller, 1.7D and 3.1D respectively contributing to 95% the total weight. The 99.9% threshold of the total weight is at 3.7D and 7.0D respectively. This is due to the damping ratio, 13.5 and results in values above a certain distance away from the pile tip having close to zero contribution to the calculated $q_{c,avg}$. The damping ratio was kept constant because changing this value stretches the cosine function in a very similar manner to changing the influence zone. The final parameter to be analysed was parameter s, which reshapes the weight according to the stiffness of the soil. A value of 0.9 demonstrated the best fit for the selected data. An observation made during the MSE analysis is that changing the variables for the alternative method resulted in opposite effects. Whenever the MSE for one of the data sets was optimized the MSE of the other data set would become larger. Therefore, a compromise was made and variables were taken in between the two optimums of the data sets. The lower factor for the layered sand data set results in an optimum inclined towards the sand and clay layered data set. Two of the tests are presented in Figures 4.6 and 4.7. All the remaining CPT calibration tests can be found in Appendix C, Figures C.3-C.16.

4.3. Comparison of averaging techniques

An additional investigation was made into the shape of the influence zone and hence the weights dependent on the distance. The MSE analysis on the cosine function, as stated above, was applied to a sine and linear functions. For each different function optimal variables were found leading to unique shapes for the weights dependent on the distance (see Appendix C, Tables C.3 & C.4 and Figures C.2(a) & C.2(b)). All the methods produced similar fits on the CPT calibration tests with minimum variation. Ultimately, the cosine function was chosen due to a better fit in the most accurate data set (the sand and clay layered data set)(Figure 4.5).

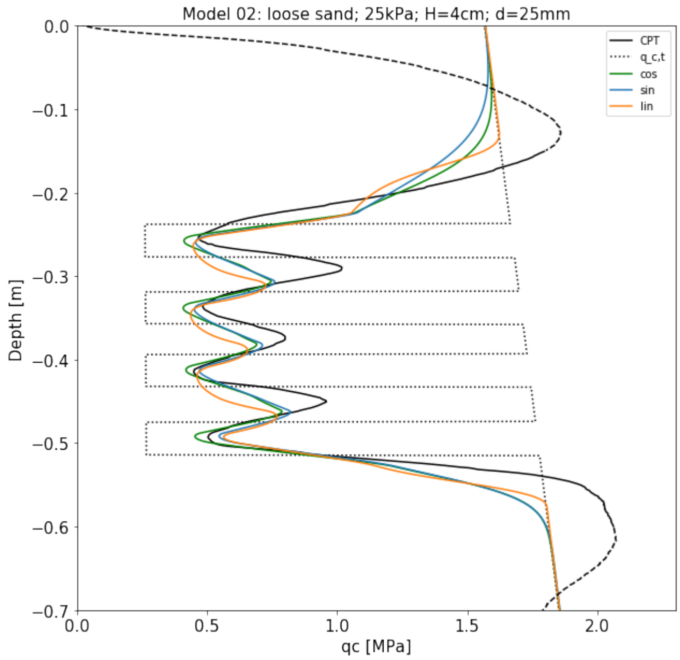

Figure 4.5: Illustration of the fits of cosine, sine and linear weight functions investigated applied to model 03, 25kPa surcharge

A comparison based on the 10 sand and clay layered tests with the other averaging techniques shows the accuracy of the alternative method to the measured cone resistance, with the alternative method having nearly a factor 4 smaller mean and standard deviation over the next most accurate averaging technique (Koppejan)(see Table 4.2). This can also be seen in Figures 4.6 and 4.7 of two different laboratory tests used in the MSE analysis, where the alternative averaging technique has a better fit on the q_c value. Table 4.2 also demonstrates that the cosine function for the weights dependent on the distance in the alternative method yields to the smallest MSE. Additionally, the weight for the values above and below the pile tip was analysed. Different combinations were applied ranging from 55 - 45% to 20 - 80% above and below the pile tip respectively, however, the analysis concluded that the 50 - 50% split was the best combination due to the smallest MSE.

Table 4.2: MSE comparison of the Koppejan, LCPC, Boulanger-de Jong and the alternative method based on the 10 sand and clay layered tests

Method	Mean	Standard deviation
Koppejan	0.4026	0.3533
LCPC	0.7581	1.1050
Boulanger-de Jong	2.4506	4.3978
Alternative linear	**0.1274**	**0.1307**
Alternative sine	**0.1055**	**0.1061**
Alternative cosine	**0.1006**	**0.1043**

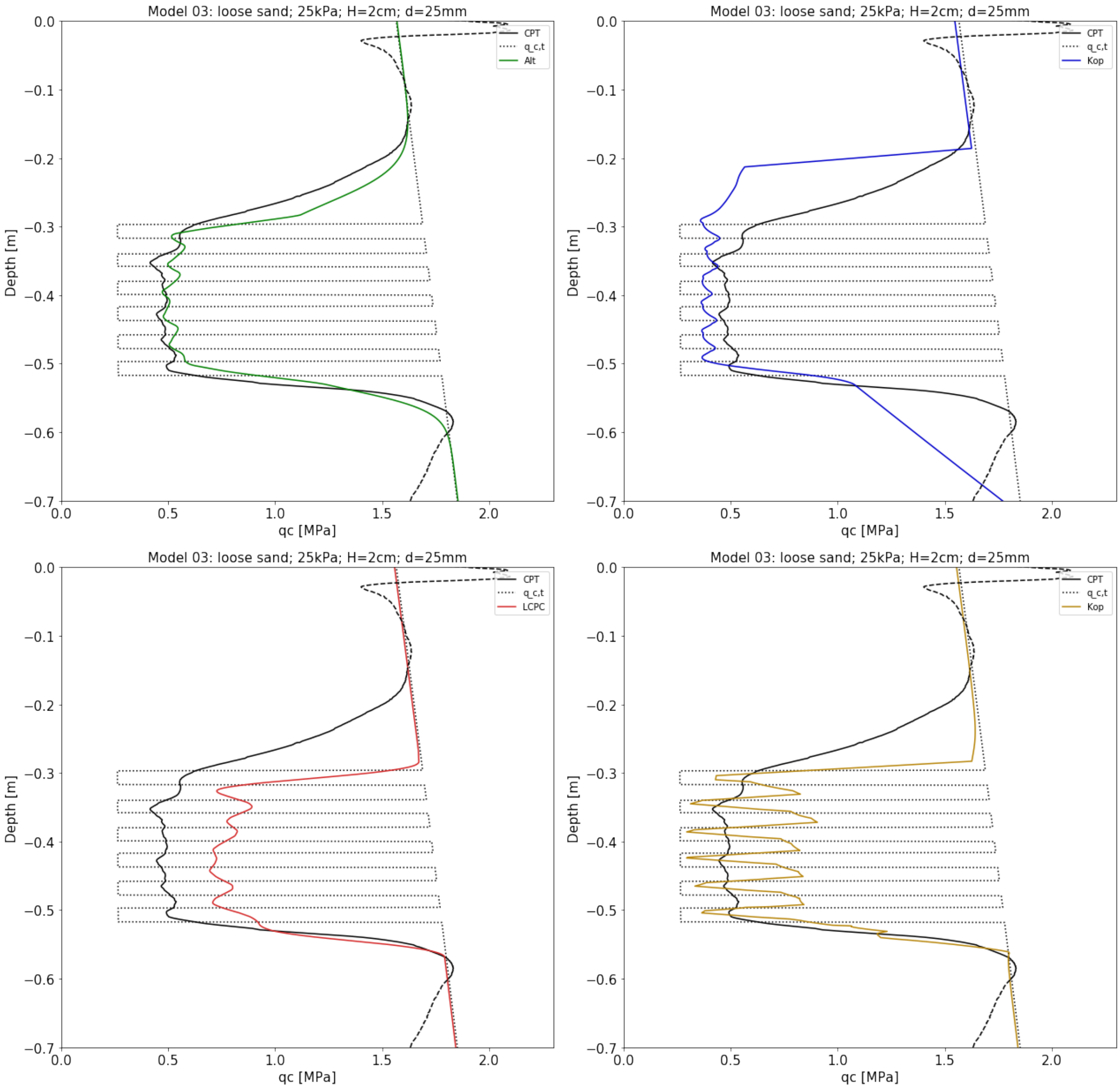

Figure 4.6: Fits for the averaging techniques in test model 03 25kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

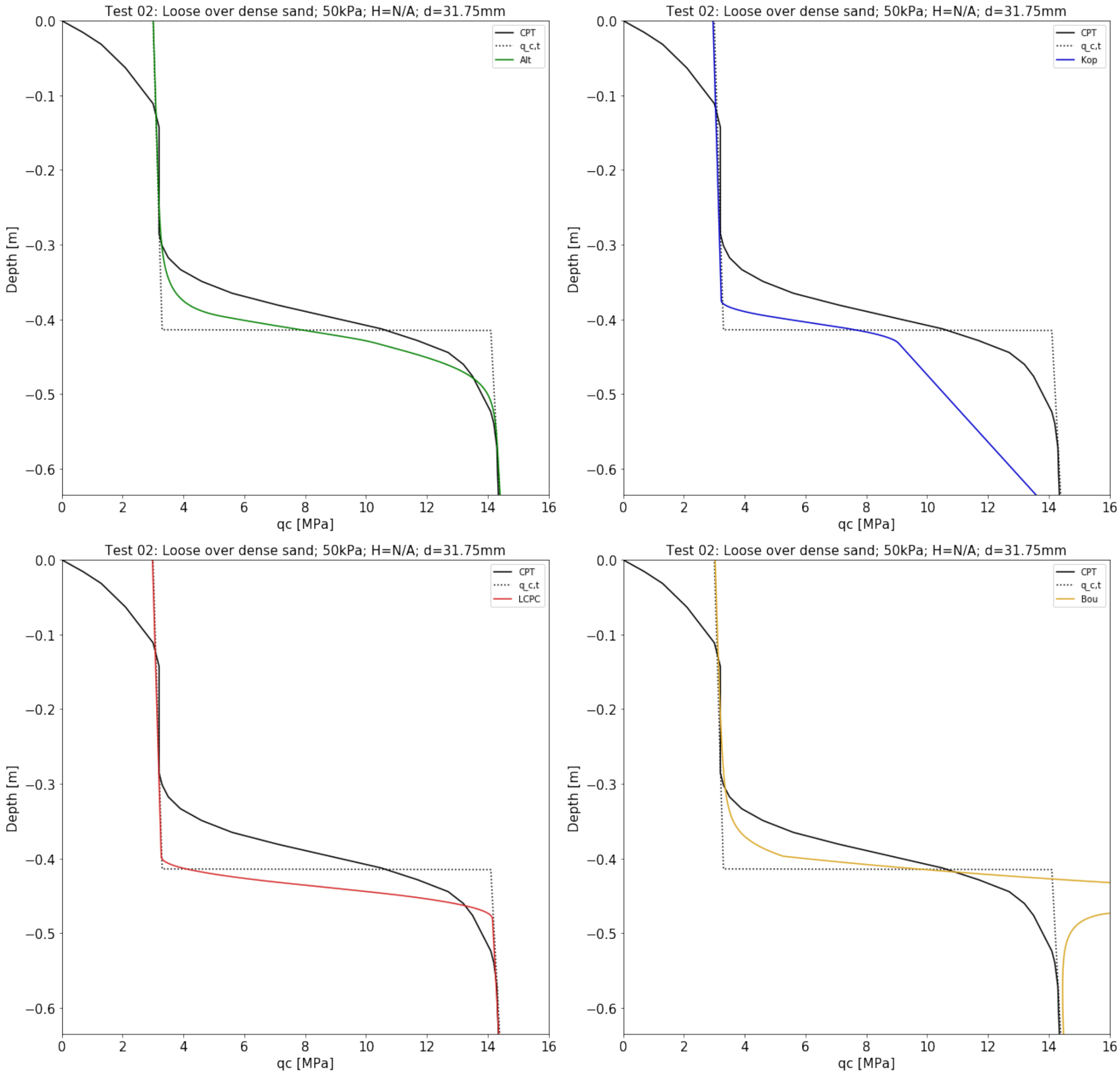

Figure 4.7: Fits for the averaging techniques in test 03 (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

In general, Figures 4.6 and 4.7 show the most accurate fit for the alternative averaging technique (top left). This is the case for all the tests used in the MSE analysis and in comparison to the other averaging techniques an improvement is made in predicting the sensing as well as developing distance (Figures C.3-C.16 in Appendix C). In reality, most pile tips are installed within or near this transition zone. The Koppejan averaging technique (top right) always lays on the conservative side for all the CPT calibration tests used. The effect of the minimum path for 4D under the pile tip can be seen by the jump in the top right of Figure 4.6 and the linear increase due to the 8D minimum path above the pile tip can be seen at the bottom of both Koppejan Figures 4.6 and 4.7. Both of these aspects in the Koppejan averaging technique lead to an underestimation of the soil strength. In the layered sections of the tests, the Koppejan averaging technique produces representative q_c value closer to the measured q_c however, this value is always slightly conservative. The LCPC averaging technique (bottom left) returns a general basic average due to the small influence zone taken into account and can hence, not capture the transition zones. The value calculated for layered sections of the tests overestimates the strength of the soil present. Lastly the Boulanger - de Jong filtering technique (bottom right) had some trouble with frequent changes in q_c with the standard parameters provided in the paper, Boulanger and de Jong (2018). This was most likely due to the presence of very thin layers. Hence, $z_{50,ref}$ (Appendix A) was lowered from 4.0 to 1.5 to provide the fits in Figures 4.6 and 4.7. Although the fit of this method varied from test to test overall, the fit followed the shape of the original CPT with the exception of large peaks at some of the soil layer boundaries. Generally, the results of Boulanger - de Jong technique would lay between the results of the Koppejan and LCPC averaging techniques. However, in order to further improve the fit certain parameters (see Appendix A) would have to be modified for each laboratory test since each test has a different soil layer thickness. This method is therefore not suitable in its current state to calculate a representative value for q_c. The advantages and disadvantages of all the discussed averaging techniques have been summarized in Table 4.3.

Table 4.3: Advantages and disadvantages of the Koppejan, LCPC, Boulanger-de Jong and on the alternative method based the laboratory tests

Method	**Advantage**	**Disadvantage**
Koppejan	- Conservative - Relatively good calculation in layered soils - Representative value in uniform soils	- Only works well when tip is $> 4D$ above weak layer of soil and tip has $> 8D$ embedment in strong soil - Does not capture transition zones
LCPC	- Representative value in uniform soils	- Overestimates strength in layered soils - Does not capture transition zones
Boulanger-de Jong	- Relatively good calculation in two layered soil - Representative value in uniform soils	- Parameters need to be optimised for each test - Produces peaks, especially in thin layered of soil - Does not capture transition
Alternative	- Captures transition zones - Captures thin layers of soil - Representative value in uniform soils	- Less accurate for transition zones between sands

5

Analysis of pile load tests

5.1. Deltares Database

After applying the criteria selected in the Literature study to the Deltares database, the following pile load tests remain:

Table 5.1: Summary of the pile load tests selected from the Deltares database

Test	Material [-]	No./dist of CPT(s) [-/m]	D_{eq} [mm]	L/D [m]	Age [days]	0.1D [-]*
TNO pile 01	Concrete	1/0.0	0.328	12.21	28	M
TNO pile 02	Concrete	1/0.0	0.328	12.21	28	M
Kruithuisweg I	Steel	3/ 5.5	0.355	3.38	34	E
Kruithuisweg II	Steel	3/ 5.5	0.355	4.23	22	E
Kruithuisweg III	Steel	3/ 5.5	0.355	9.86	23	E
Kruithuisweg IV	Steel	3/ 5.5	0.355	15.49	21	E
Kruithuisweg V	Steel	3/ 5.5	0.355	21.13	13	E
ESOPT II**	Concrete	1/2.0	0.283	6.02	31	E
CIAD**	Concrete	1/?	0.452	1.22	?	E

Where L/D is the penetration depth in the sand bearing layer normalised by the diameter. *This column indicates whether the displacement at the pile tip was measured (M) or estimated (E) through elasticity.
**0.1D displacement not reached at the pile tip in these pile load tests.

5.1.1. Limitations of Deltares database

These pile load tests (Table 5.1) are the remaining tests with sufficient quality from the 25 pile load tests used for the investigation by Deltares. Moreover, no additional pile load tests that met the criteria could be extracted from the paper database, confirming the limitation stated in the Deltares report (Stoevelaar et al., 2009). From the selected tests the TNO piles are of the highest quality with no evident limitations. The Kruithuisweg pile load tests have been performed on a single pile, which has been driven further into the ground after each test. These tests could have reduced reliability due to dynamic pile load tests being carried out before the static compression load tests I, II, III and V. This could have had a significant impact on the capacity measured as the soil would be subjected to additional shearing and grain crushing due to dynamic loading. Partial healing of the soil would be absent depending on the time frame between the dynamic and static load test. No information could be found on the exact timing of the dynamic load tests making it difficult to judge the extent of the effect on the pile capacity. Additionally, the 0.1D displacement at the pile tip was estimated through the estimation of the elastic compression of the pile and could hence be off by a couple centimetres. Although the elastic compression of the pile is measured during pile load tests in the

Netherlands, during the final loading phase of 'failure' (0.1D displacement) the elastic compression of the pile can not be measured as it involves loading and reloading the pile. At this stage, the pile would not remain at the same depth as done for the previous loading phases. It should also be noted that test V has a much smaller setup time compared to the rest of the test which may have resulted in an even smaller measured pile shaft capacity. An advantage for the Kruithuisweg pile load tests is the availability of 3 CPTs for the pile. A small investigation was carried out into which combination to use for the calculation of the capacity of the pile in Appendix D. This lead to that the capacity of the pile being calculated for each of the 3 CPTs and averaged since, the 3 CPTs are similar distances away from the test pile. The remaining tests, ESOPT II and CIAD, also have estimated 0.1D displacement at the pile tip. However, full 0.1D displacement was not reached during the pile load test and hence the representative measured load is the maximum reached during the test. This is 84% and 83% of 0.1D for ESOPT II and CIAD, respectively. The CIAD test has an additional drawback as the setup time of the pile is not known. Hence, the extent of ageing effects can not be reviewed in the analysis. Further details of each pile load test and the corresponding CPT profiles can be found in Appendix B. Due to the limitations additional comparisons will be made with the Q_m/Q_c of the pile load tests in Table 5.1 and a high quality test (Pigeon River). In this test the base and shaft capacity have determined with and without residual load (Paik, Salgado, Lee, & Kim, 2003). Further details about this pile load test can also be found in Appendix B.

5.1.2. Base capacity

The first comparisons made were the α_p factors for the CPT averaging techniques, since they form the basis of the CPT based methods for the calculation of the base capacity. The α_p factors were determined by dividing the measured by the calculated base capacity (Q_m/Q_c) for each pile load test. The exact numbers used in the calculation of Q_m/Q_c can be found in Table 5.2.

Table 5.2: The Q_c obtained from the CPT averaging techniques and Q_m used for the determination of α_p for each pile load test

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]		
		Koppejan	LCPC	Alternative
TNO pile 01	573	1118	1371	1359
TNO pile 02	500	998	1182	1182
Kruithuisweg I	254	281	370	439
Kruithuisweg II	770	890	1369	1345
Kruithuisweg III	714	1167	1678	1588
Kruithuisweg IV	712	1307	1523	1619
Kruithuisweg V	644	1079	1434	1411
ESOPT II	586	751	998	1056
CIAD	1618	1716	1656	2236

A comparison was made between the values obtained in this investigation with the values obtained by Deltares for the Koppejan method, to check the reliability of values obtained in Table 5.2. At Deltares calculations were made using D-foundations which is a well-proven software (Stoevelaar et al., 2009), while the values obtained in Table 5.2 were calculated through the use of python code. The comparison validated the python code with differences of up to ±1%. Other variations between these two reports come from the measured values as described in Table 2.4. A graph of the comparison can be seen in Figure D.4, Appendix D. However, the same α_p, 0.70 is computed as in the Stoevelaar et al. (2009) report for the Koppejan averaging method. The α_p for the LCPC is equal to 0.56, slightly higher than the recommended value of 0.50 by Bustamante and Gianeselli (1982) for sands. The α_p for the alternative averaging technique is 0.51 for these tests (Table 5.3). All the values obtained are only relevant for pile load tests without residual load measurements. The results from the Pigeon River pile are consistent with the computed average values except for the Koppejan averaging technique. This is due to a the presence of a soft thin layer of soil located underneath the pile tip which affects the values from the minimum path rule significantly.

Table 5.3: Qm/Qc = α_p of all the pile load tests for the CPT averaging techniques

Pile load test	Qm/Qc [-]		
	Koppejan	LCPC	Alternative
TNO pile 01	0.51	0.42	0.42
TNO pile 02	0.50	0.43	0.42
Kruithuisweg I	0.91	0.69	0.58
Kruithuisweg II	0.86	0.56	0.57
Kruithuisweg III	0.61	0.43	0.45
Kruithuisweg IV	0.55	0.47	0.44
Kruithuisweg V	0.60	0.45	0.46
ESOPT II	0.78	0.59	0.55
CIAD	0.94	0.98	0.72
Mean	**0.70**	**0.56**	**0.51**
Variance	0.028	0.030	0.009
Standard deviation	0.167	0.172	0.096
CoV	**0.240**	**0.312**	**0.187**
Pigeon River	1.26	0.47	0.51

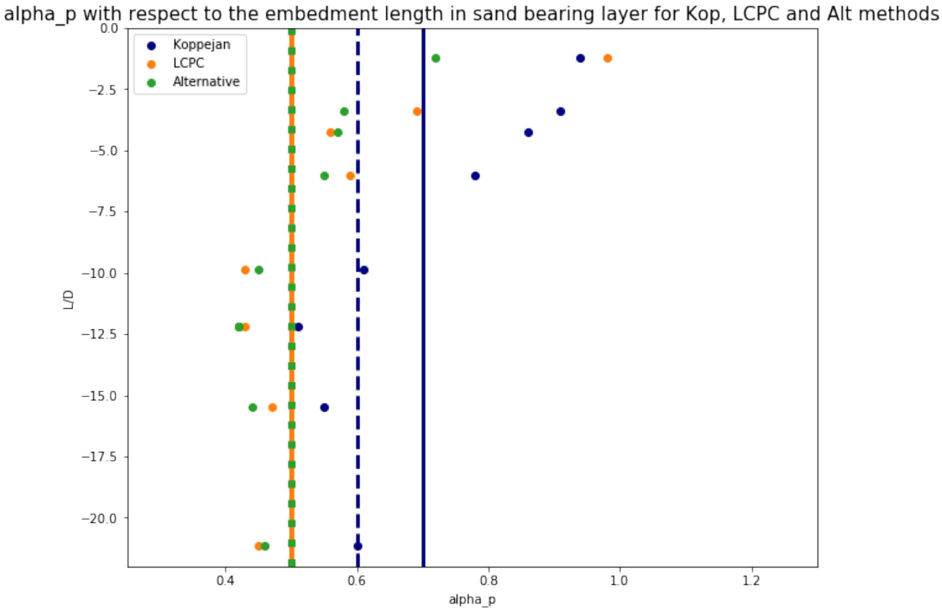

Figure 5.1: Visualisation of the spread in α_p for the CPT averaging techniques compared to the embedment length in sand bearing layer

The importance of the α_p factors obtained in Table 5.3 is not demonstrated by the mean value of each method but by the spread from the mean. The most important factor of the CPT averaging technique is consistency in order to apply a suitable constant value for α_p. By looking at the statistical data such as the variance and standard deviation this consistency can be quantified. The most significant statistic, however, is the coefficient of variation, CoV, which is the standard deviation normalised by the mean. This gives comparisons between the CoVs more relevance than the more commonly used variance and standard deviation. Therefore, the most consistent CPT averaging technique has the lowest CoV, which in this case, is the alternative method. This is further illustrated in Figure 5.1. The Figure shows all the computed α_p values for all 3 averaging techniques and the dependency of embedment length in the sand bearing layer normalised by the diameter. The vertical lines in Figure 5.1 represent the proposed α_p values for the LCPC, the UWA and the Dutch norm (left to right). The Koppejan averaging technique shows the most dependency with the embedment length. This effect, as stated before, is the main disadvantage of the Koppejan method. This dependency is due to the 8D influence zone above the pile tip taken by the averaging technique being too large. At shallow embedding in sand, softer soils have too much influence on the calculated representative value. The same effect of dependency with embedment length in sand is observed for the LCPC averaging technique but for different and opposite reasons. For this technique the influence zone used, 1.5D, is too small for the calculation of the representative value. In contrast, the alternative averaging technique has a smaller spread with changing embedment length in sand, even for pile tests with shallow embedment length in sand where the other averaging techniques fall short. The values of α_p for a pile load test with shallow embedment length in sand can be seen in Table 5.3 (pile load tests Kruithuisweg I, II and CIAD). The manner of transition of the averaging techniques in these shallow embedment zones have been illustrated in Figures 5.2 and 5.3 for pile load tests Kruithuisweg I and CIAD. Both the Koppejan and LCPC representative value recovers slower at the soil layer boundary compared to the alternative averaging technique, which explains the spread in values seen in Figure 5.1. The Pigeon River test has an embedment >8D however, has a very high α_p for the Koppejan method. This is due to the presence of a soft thin layer of soil located <4D underneath the pile tip (Figure B.14). This results in the minimum path rule being activated for the Koppejan method.

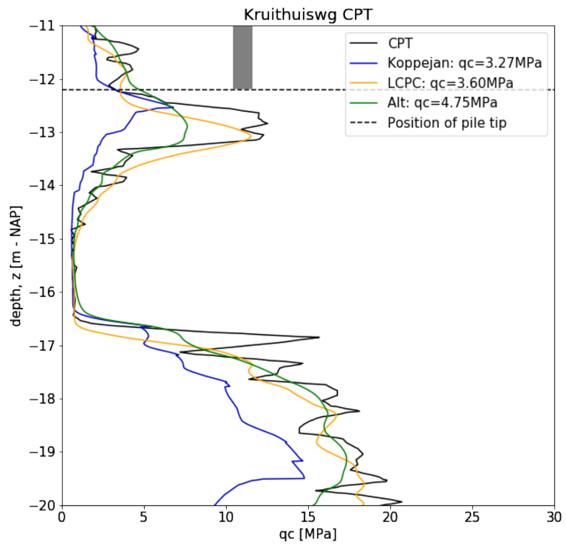

Figure 5.2: Kruithuisweg I: The $q_{c,avg}$ at soil layer boundaries calculated for the whole length of the CPT for the three averaging techniques

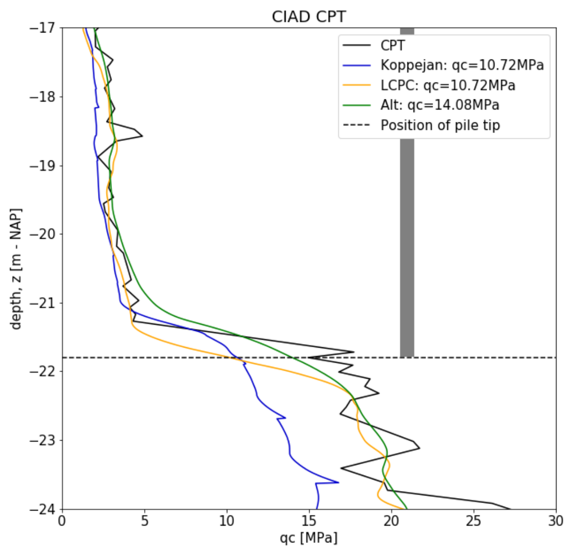

Figure 5.3: CIAD: The $q_{c,avg}$ at soil layer boundaries calculated for the whole length of the CPT for the three averaging techniques

Next, comparisons were made between the calculation methods for the base capacity. The calculation methods compared are the UWA-05, ICP-05, NEN-9997-1 (Dutch norm) as well as the alternative method multiplied by the α_p factor of 0.50 corresponding to a conservative value found in Table 5.3. The equations for these methods can be found in Table 2.1. The UWA-05 and NEN-9997-1 use a α_p factor which is multiplied by the $q_{c,avg}$ obtained from the Koppejan averaging technique corresponding to an α_p equal to 0.6 and 0.7, respectively. It has to be noted that the limitation of 15MPa for the NEN-9997-1 could not be analysed due to all the $q_{c,avg}$ of the selected tests not exceeding this value. The ICP-05, on the other hand, is slightly more complex and involves normalisation of the diameter of the pile by the diameter of the cone used during the CPT. Since, all the pile load tests used are from the Netherlands a representative value of 36mm was used for the diameter of the cone. This is the standard cone-diameter according to Dutch norm (Normcommissie-351-006-Geotechniek, 2017).

Table 5.4: Q_m/Q_c of all the pile load tests for all the considered base calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	Alt*
TNO pile 01	0.85	0.80	0.73	0.84
TNO pile 02	0.83	0.83	0.72	0.85
Kruithuisweg I	1.51	1.37	1.29	1.16
Kruithuisweg II	1.44	1.12	1.24	1.14
Kruithuisweg III	1.02	0.85	0.87	0.90
Kruithuisweg IV	0.91	0.93	0.78	0.88
Kruithuisweg V	0.99	0.89	0.85	0.91
ESOPT II	1.30	1.06	1.11	1.11
CIAD	1.57	2.17	1.35	1.45
Average	**1.16**	**1.11**	**0.92**	**1.03**
Variance	0.066	0.168	0.057	0.037
Standard deviation	0.256	0.409	0.239	0.192
CoV	**0.240**	**0.368**	**0.240**	**0.187**
Pigeon River	2.10	0.94	0.93	1.01

Alt* is the value obtained using the alternative averaging technique times a constant $\alpha_p = 0.5$

The UWA-05 and ICP-05 average underestimates the measured base capacity by more than 10%, although the ICP-05 has the worst CoV of the four methods, heavily skewed by the CIAD pile load test. The Dutch norm, NEN-9997-1, on the other hand overestimates the base capacity by 8% while the alternative method has a value closest to unity and the smallest CoV (0.187) of the four methods. Theoretically, if residual loads were measured the base capacity would be higher. This means that the average of the UWA-05 and ICP-05 would be even further from unity, for these particular pile load tests. When looking more in-depth of the ratios in Table 5.4, the pile load tests skewing the average values above 1.0 are the tests with a shallow embedment in sand-bearing layer (Kruithuisweg I, II, ESOPT II and CIAD) (i.e. embedment of the pile tip is ≤8D). These are affected by the Koppejan averaging technique which tends to return higher α_p values in these conditions. The LCPC averaging technique similarly affects the same pile load tests by underestimating the results of the ICP-05 however, α_p factors are much higher for the tests with the shallowest embedment (i.e. Kruithuisweg I and CIAD) and lower for the tests with slightly deeper embedment (i.e. Kruithuisweg II and ESOPT II). This effect can also not be completely discarded for the alternative averaging technique, according to this data.

5.1.3. Shaft capacity

Due to the focus of this thesis being on foundation piles in sand, the initial investigation was on the shaft friction of the pile in sand layers. From the data of the pile load tests a focus was made on the strain gauges in sand, where the average q_c value is greater than 5MPa between two strain gauges. This is the defined minimum q_c value for sands in the Dutch norm. Hence, the following pile load tests remained:

Table 5.5: Measured α_s for all strain gauges located in sand

Pile load test	Measured friction, tau [kPa]	q_c* [MPa]	α_s [-]
TNO pile 01	76.15	15.44	0.0049
	80.79	14.53	0.0056
TNO pile 02	48.16	18.32	0.0026
	128.15	14.88	0.0086
Kruithuisweg III	62.19	13.37	0.0047
Kruithuisweg IV	59.41	14.26	0.0042
Kruithuisweg V	49.31	14.97	0.0033
CIAD	71.25	12.20	0.0058
		Average	**0.0050**

*The average q_c value in between two strain gauges.

Both pile load tests at TNO had 3 strain gauges in sand (Grondmechanica-Delft, 1993). Therefore, two different α_s values could be obtained. The Kruithuisweg load tests also had 3 strain gauges in sand from test III onwards however, during the testing one of the strain gauges was possibly damaged and therefore measurements from this gauge were discarded (Grondmechanica-Delft, 1982). Lastly, the CIAD pile load test only had 2 strain gauges in the sand-bearing layer. The results of these pile load test establish an average α_s value of 0.005. This is half of the value recommended for sands by Normcommissie-351-006-Geotechniek (2017), 0.01. An equivalent value was recommended by the report published by the TNO pile load tests (Grondmechanica-Delft, 1993). However, no trend could be determined with the position of the strain gauge above the pile tip normalised by the diameter, in relation to friction fatigue (Figure D.6, Appendix D).

A visualisation of the strain gauge data can be seen in Figure 5.4. This figure shows the load measured at each strain gauge in black, the gradient between two strain gauges (i.e the friction, tau) in red. For comparison reasons, the CPT (blue) and the average q_c value between two strain gauges (orange) has been scaled down by a factor 0.005 (α_s). On the right of Figure 5.4, the process applied to strain gauges which were positioned slightly above the sand boundary is demonstrated. The contribution from the softer soil was assumed to be negligible hence, the value obtained by the strain gauge was lowered as seen in Figure 5.4 (right). The formulas from Table 2.2 were then used to calculate the shaft capacity between the stain gauges in sand for comparison (Table 5.6, more Figures containing strain gauge measurements can be found in Appendix B).

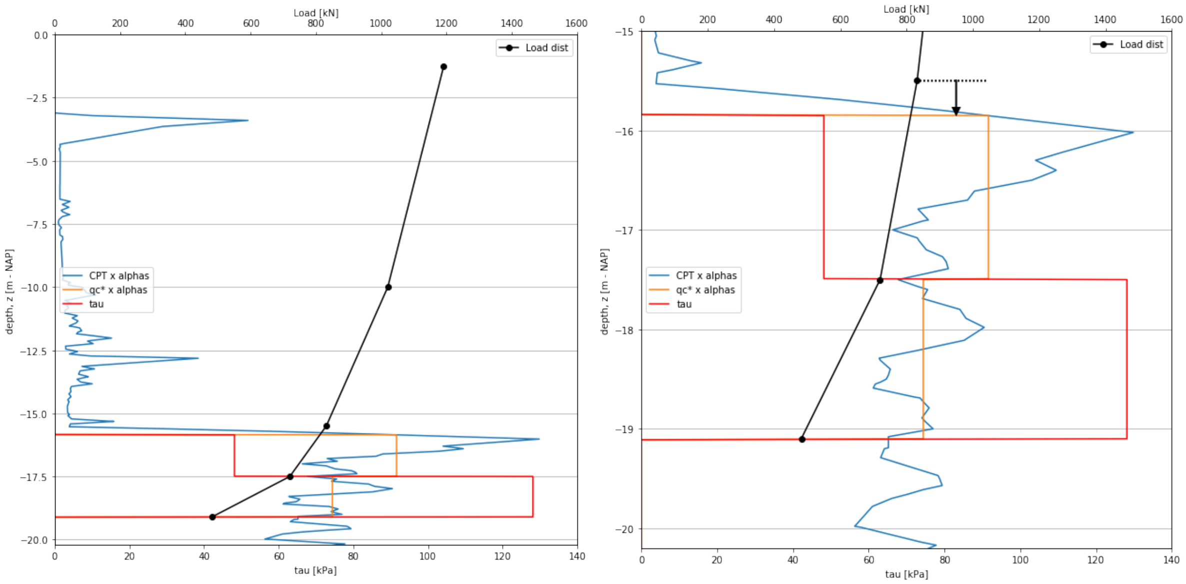

Figure 5.4: Example of strain gauge data acquisition for sand end bearing layer; TNO pile 01

When comparing the measured shaft capacity in the sand layers to the shaft capacity calculated by the ICP-05, UWA-05 and NEN-9997-1 calculation method, low Q_m/Q_c ratios are also obtained (Table 5.6). Similarly, no trend could be deduced from this data. Upon further review, the decision was made to focus on the whole shaft capacity of along the pile. This is due to the calculation methods being designed for the calculation of the shaft capacity along the whole pile. The shaft capacities compared in Table 5.6 are close to the pile tip. In general, within this zone high shaft capacities are measured due to high forces on the tip dispersing radially, of which a portion onto the shaft (Prandtl, 1921). This effect could be overestimated by the calculation methods and could explain the low Q_m/Q_c ratios obtained.

Table 5.6: Q_m/Q_m for the shaft of piles located in sand

Pile load test	Qm/Qc [-]		
	UWA-05	ICP-05	NEN-9997-1
TNO pile 01	0.77	0.83	0.55
TNO pile 02	0.75	0.80	0.53
Kruithuisweg III	0.53	0.62	0.44
Kruithuisweg IV	0.51	0.57	0.40
Kruithuisweg V	0.42	0.46	0.33
CIAD	0.56	0.82	0.66
Average	**0.59**	**0.68**	**0.48**
Variance	0.016	0.020	0.012
Standard deviation	0.126	0.141	0.107
CoV	**0.214**	**0.207**	**0.221**

The values for Q_m and Q_c are located in Table D.6, Appendix D.

For the comparisons between the shaft capacity of the whole pile, only one pile was discarded. This was pile load test Kruithuisweg I. In this pile load test the contribution of ≥ 50% from clay layers is exceeded (Table 2.4). The contribution of clay layers for ICP-05 and UWA-05 was determined using the method developed by Lehane et al. (2013), UWA-13 (see equation 5.1).

$$\tau_f = 0.055 \cdot q_t \left[max\left(\frac{h}{R^*}, 1\right)\right]^{-0.2} \tag{5.1}$$

$$q_t = q_c + (1-a)u_2 \tag{5.2}$$

τ_f: pile shaft friction

h: height above the pile tip level at which τ_f acts

$R^* = (R^2 - R_i^2)^{0.5}$

q_t: total end resistance of the cone

a: cone area ratio (0.70 - 0.85 (Robertson & Robertson, 2006))

u_2: pore pressure acting at the filter stone on the cone

In sands $q_t = q_c$ however, for soft soils under the water table, a correction is necessary for the pore water pressure acting on the cone geometry (Robertson & Robertson, 2006). This correction is applied as shown in equation 5.2. The Soil Behaviour Type Index, I_{SBT} (Robertson, 2010) was used for the classification of clay layers, see equation D.1 in Appendix D.

The Dutch NEN-9997-1 suggests particular α_s factors for different soils (sand/clay/silt/peat) (see Table 7.c and 7.d in Normcommissie-351-006-Geotechniek (2017)). These factors have been used for the calculation of the whole shaft capacity of the pile. The NEN-9997-1* column included the limit on q_c (Table 5.8. However, the calculation Normcommissie-351-006-Geotechniek (2017) normally only takes the resistance in sand as representative capacity for the whole pile capacity (See Appendix A, Figure A.3). This calculation was left out from this report since the calculation method does not look at the shaft capacity along the whole pile. The ICP-05 and UWA-05 formulas in Table 2.2 have been used to obtain the remaining values in Table 5.7. For both these methods, the effective stress is needed. This was done by calculating the total stress using CPT data assigning volumetric weights to soil layers using Table 2.b in the Normcommissie-351-006-Geotechniek (2017) and the I_{SBT}. The effective stress was then computed by taking the total stress minus the pore pressure. However, since the pore pressure was not measured for any of the available CPTs, hydro-static conditions were assumed.

Table 5.7: The measured and calculated shaft capacity of piles for all the considered shaft calculation methods

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]			
		UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
TNO pile 01	676	810	740	883	749
TNO pile 02	629	829	773	988	801
Kruithuisweg I	294	171	150	172	172
Kruithuisweg II	331	488	417	543	512
Kruithuisweg III	529	741	651	897	775
Kruithuisweg IV	592	894	826	1224	1038
Kruithuisweg V	462	1077	1023	1598	1305
ESOPT II	539	318	262	383	366
CIAD	1894	1119	1118	1507	1362

For the Q_m/Q_c calculations of the shaft capacity (Table 5.8), the ICP-05 has an average closest to unity, with only +3% from 1.0. The UWA-05 overestimates the total shaft capacity slightly, with −7% away from 1.0. The NEN-9997-1 α_s factors for the whole shaft overestimate the shaft capacity present, even when the limit on the q_c values is applied. This overestimation is present for all pile load tests except for ESOPT II and CIAD (Table 5.8).

Table 5.8: Q_m/Q_c of all the pile load tests for all the considered shaft calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
TNO pile 01	0.83	0.91	0.77	0.90
TNO pile 02	0.76	0.81	0.64	0.79
Kruithuisweg II	0.68	0.79	0.61	0.65
Kruithuisweg III	0.71	0.81	0.59	0.68
Kruithuisweg IV	0.66	0.72	0.48	0.57
Kruithuisweg V	0.43	0.45	0.29	0.35
ESOPT II	1.69	2.06	1.41	1.48
CIAD	1.69	1.69	1.26	1.39
Average	**0.93**	**1.03**	**0.76**	**0.85**
Variance	0.205	0.262	0.129	0.135
Standard deviation	0.452	0.512	0.359	0.368
CoV	**0.485**	**0.496**	**0.476**	**0.443**
Pigeon River	0.95	1.22	0.80	1.12

All the Kruithuisweg pile load tests produce low Q_m/Q_c ratios, meaning that the calculated shaft capacity is much lower than the measured shaft capacity. This is most likely due to the dynamic tests that were performed on the piles before the static load tests. It is very plausible that the dynamic loading caused further friction fatigue along the shaft. This would explain the low values measured for the shaft capacity. This discrepancy can be observed when comparing the Kruithuisweg results with the Pigeon River pile. The overall effect of these low ratios is lowering the average for all the calculation methods. It should also be noted that for the case of the shaft capacity, the effect of residual load would reduce the measured shaft capacity. This would further lower the Q_m/Q_c ratios and therefore the averages of the calculation methods.

5.1.4. Total capacity

The main advantage of looking at the total of pile capacity of the selected tests is that weather the residual load was measured or not; the total capacity does not change. Hence, calculation methods designed considering the residual load are not skewed by the lack of residual load measurements in these pile load test. One of the first observations from the results in Table 5.9 is that the UWA-05 and the ICP-05 calculation methods produce Q_m/Q_c ratios very closest to unity. The Dutch calculation method significantly overestimate the total capacity of the pile in most cases. The overestimation is on average 5% less when the q_c limit is applied. A 1.00 average was obtained for the combination of the base capacity calculated using the alternative averaging technique with an α_p of 0.5 in combination with the shaft capacity calculated using the ICP-05. Another benefit of this combination is the CoV, which is the lowest out of all the calculation methods (see Table 5.9). However, due to the mentioned limitation of selected pile load tests, it is difficult to judge the meaning between the computed averages and CoV. The reliability of the Kruithuisweg pile load tests is the lowest while ESOPT II and CIAD pile load tests are also doubtful, overall skewing the statistics. Hence, a high-quality pile load test (Pigeon River) was selected for comparison and validation of the values obtained in Table 5.9.

Table 5.9: Q_m/Q_c of the total capacity of piles for all the considered calculation methods

Pile load test	Qm/Qc [-]				
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*	Alt-ICP
TNO pile 01	0.84	0.86	0.75	0.82	0.84
TNO pile 02	0.79	0.82	0.67	0.75	0.80
Kruithuisweg II	1.08	1.00	0.94	0.97	1.01
Kruithuisweg III	0.86	0.83	0.73	0.78	0.86
Kruithuisweg IV	0.78	0.82	0.61	0.67	0.80
Kruithuisweg V	0.64	0.63	0.47	0.54	0.64
ESOPT II	1.46	1.38	1.24	1.26	1.33
CIAD	1.63	1.88	1.30	1.37	1.57
Average	**1.01**	**1.03**	**0.84**	**0.89**	**1.00**
Variance	0.111	0.146	0.077	0.073	0.092
Standard deviation	0.333	0.382	0.278	0.271	0.304
CoV	**0.329**	**0.371**	**0.332**	**0.303**	**0.304**
Pigeon River	1.39	1.04	1.18	1.43	1.10

The Q_m/Q_c ratios from the Pigeon river test (Table 5.9) are underestimated for all the calculation methods. In order to have a more concrete conclusion on the calculation methods, further analysis will be carried out in the extension, focusing mainly on high-quality tests were distinctions were made between residual loads.

5.2. Extension

For the extension of the analysis, 4 additional high-quality pile load tests are analysed (Table 5.10). These test all have residual load measurements, (Appendix B) and hence the base and shaft capacities measured can be redistributed. This will allow comparisons between the 4 tests excluding residual loads and including residual loads, in order to identify the effect of the residual load on existing calculation methods and their corresponding α factors. Additional information about these test can be found in Appendix B.

Table 5.10: Summary of the high quality pile load tests selected

Test	Material [-]	No./dist of CPT(s) [-/m]	D_{eq} [mm]	L/D [m]	Age [days]	0.1D [-]*
Pigeon River	Steel	1/0.0	0.356	10.87	8	M
Port Rotterdam 02	Concrete	3/ 3.1	0.509	0.98	10	M
Port Rotterdam 03	Concrete	3/ 3.1	0.509	0.98	30	M
Marshall County	Steel	1/3.0	0.356	12.64	9	M

L/D is the penetration depth in the sand bearing layer normalised by the diameter. *This column indicates whether the displacement at the pile tip was measured (M) or estimated (E) through elasticity.

An issue about residual load measurements, is that the residual loads for the pile load tests are sensitive to interpretation of the data. The mechanism are well understood in the present day and explained in literature, such as, Fellenius, Harris, and Anderson (2004). However, variation is still present in the interpretation. For the Marshall County and Port of Rotterdam pile load tests the residual loads were estimated through the strain measurements before the static load tests. For the Pigeon river pile load test, on the other hand, the residual loads were estimated through the strain measurements right after pile driving and there are therefore doubts present over the reliability of these residual load measurements (Han, Bisht, Prezzi, & Salgado, 2019). The different interpretation of the residual load measurements can therefore affect this analysis. The different methods of calculating residual loads have been described in Appendix A. Additionally, since 3 CPTs were available for each pile load test for the Port of Rotterdam, the calculated capacities were calculated separately per CPT. The average of the calculated values was then taken as a representative value (see Appendix D, Tables D.4 & D.5 for full calculations).

5.2.1. Base capacity

The following table presents the measured and calculated base capacities of the 4 pile load tests using the three previously used averaging techniques (Table 5.11). Two measured base capacities are provided for each test. These are the measured base capacity excluding and including residual loads. The redistribution of the residual loads accounted for a 21% to 32% increase in the base capacity in these tests.

Table 5.11: The Q_c obtained from the CPT averaging techniques and Q_m including and excluding used for the determination of α_p for each of the high quality pile load test

Pile load test	Measured, Qm [kN]		Calculated, Qc [kN]		
	Excl. Residuals	Incl. Residuals	Koppejan	LCPC	Alternative
Pigeon River	866	1091	688	1860	1706
Port of Rotterdam 02	2022	2977	3000	1957	3655
Port of Rotterdam 03	2355	3310	2680	1986	3478
Marshall County	931	1230	1713	1747	2195

The α_p factors obtained for the Q_m/Q_c in Table 5.11 excluding residuals have been presented in Table 5.12. The 4 tests have a mean for α_p of 0.84, 0.80 and 0.54 for the Koppejan, LCPC and Alternative averaging technique, respectively. The alternative averaging technique again has the lowest CoV of the three averaging techniques and the α_p of 0.54 is similar to the value obtained for the Deltares database.

Previously mentioned effects with depth are producing high α_p values for the Port of Rotterdam pile load test since the embedment length in sand is smaller than 3D. Additionally, the Pigeon River test produces

Table 5.12: Qm/Qc = α_p of all the 4 high quality pile load tests excluding residual loads for the CPT averaging techniques

Pile load test	Qm/Qc [-]		
	Koppejan	LCPC	Alternative
Pigeon River	1.26	0.47	0.51
Port of Rotterdam 02	0.67	1.03	0.55
Port of Rotterdam 03	0.88	1.19	0.68
Marshall County	0.54	0.53	0.42
Mean	**0.84**	**0.80**	**0.54**
Variance	0.073	0.097	0.008
Standard deviation	0.270	0.311	0.091
CoV	**0.322**	**0.386**	**0.169**

a high α_p value for the Koppejan averaging technique, which means a severe underestimation of the base capacity. This is due to the presence of a soft layer of soil underneath the pile tip (Figure B.14). The soft layer leads to the calculation of a small base capacity due to the minimum path rule present in the Koppejan averaging technique. If this value of α_p is ignored, the average α_p of the Koppejan method becomes 0.70. This is the same value obtained from the Deltares database.

In Figure 5.5, the four data points for the high-quality tests have been added to the embedment length against α_p graph. The new data points conform to the previously stated trends. The presence of a relationship with embedment length can therefore not be neglected, even for the alternative method. This trend is, however, more extreme for the Koppejan and LCPC averaging techniques. The dependency with penetration depth is present until a penetration depth of >8D and >1.5 for the Koppejan and LCPC averaging techniques, respectively.

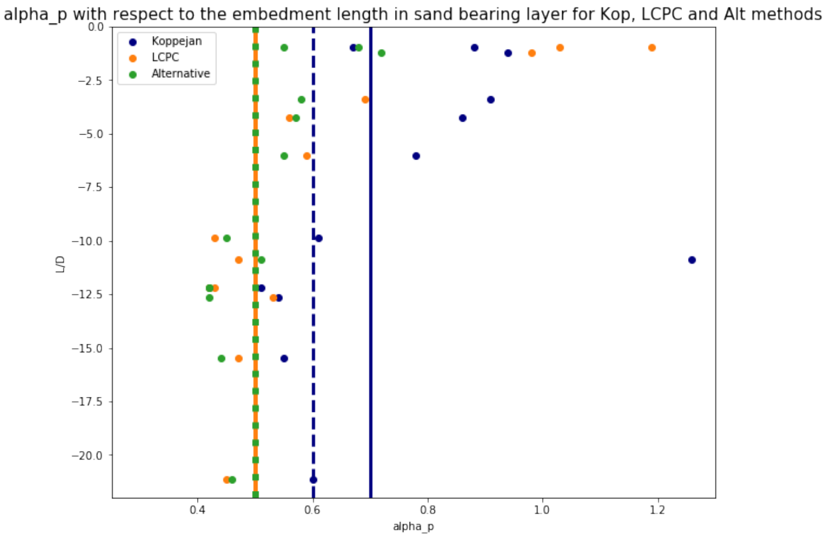

Figure 5.5: Visualisation of the spread in α_p for the CPT averaging techniques compared to the embedment length in sand bearing layer

The α_p factors obtained for the Q_m/Q_c in Table 5.11 excluding residuals have been presented in Table 5.13. The 4 tests have a mean for α_p of 1.13, 1.12 and 0.74 for the Koppejan, LCPC and Alternative averaging technique, respectively. The alternative averaging technique again has the lowest CoV of the three averaging techniques. No improvements are made in the spread of the computed α_p factors when the residual loads are included and the same comments apply concerning the low embedment length in sand of the Port of Rotterdam pile load tests as well as the α_p of the Pigeon River test for the Koppejan averaging technique, which were observed when the residual loads were excluded. In this case, if the Pigeon River test is ignored the average for the Koppejan averaging technique would be 0.98. A visualisation of the different α_p factors

Table 5.13: Qm/Qc = α_p of all the 4 high quality pile load tests excluding residual loads for the CPT averaging techniques

Pile load test	Qm/Qc [-]		
	Koppejan	LCPC	Alternative
Pigeon River	1.59	0.59	0.64
Port of Rotterdam 02	0.99	1.52	0.81
Port of Rotterdam 03	1.24	1.67	0.95
Marshall County	0.72	0.70	0.56
Mean	**1.13**	**1.12**	**0.74**
Variance	0.102	0.229	0.023
Standard deviation	0.319	0.479	0.152
CoV	**0.282**	**0.428**	**0.205**

computed excluding and including residual load for the 4 pile load tests can be seen in Figure 5.6 for the alternative averaging technique. There is a clear shift in the data in Figure 5.6 (increase in α_p), however, the spread does not change as predicted by Randolph (2003) & White and Bolton (2005). The same trend was confirmed for the Koppejan and LCPC averaging techniques.

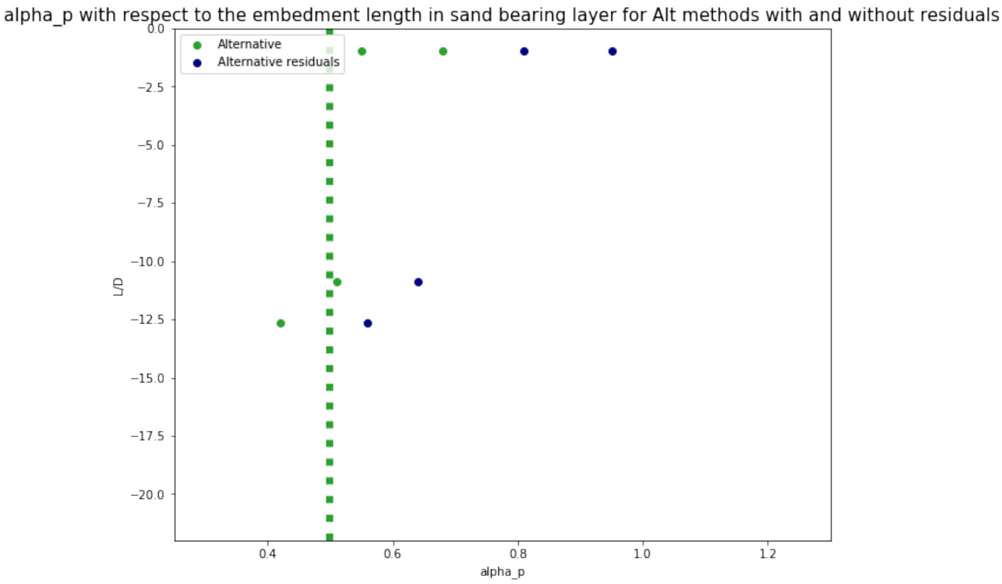

Figure 5.6: Visualisation of the spread in α_p for the alternative averaging technique compared to the embedment length in sand bearing layer

The 4 tests were also compared to the UWA, ICP and NEN-9997-1 existing calculation methods (Appendix D, Tables D.7 & D.8). For both cases (excluding and including residual loads) on average, the methods underestimated the base capacity and no improvements in CoV was present with the inclusion of the residual loads. More severe underestimation was present with the data including residual loads and no analysis was carried out on the limitation of 15MPa for Dutch method due to only one of the piles exceeding this limit value.

5.2.2. Shaft capacity

The following table presents the measured and calculated shaft capacities of the 4 pile load tests using four calculation methods (Table 5.14). Two measured shaft capacities are provided for each test. These are the measured shaft capacity excluding and including residual loads.

Table 5.14: The measured and calculated shaft capacity of all the 4 high quality pile tests for all considered shaft calculation methods

Pile load test	Measured, Qm [kN]		Calculated, Qc [kN]			
	Excl. Residuals	Incl. Residuals	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
Pigeon River	633	408	667	520	787	564
Port of Rotterdam 02	4402	3447	3049	3194	4645	3556
Port of Rotterdam 03	4136	3181	3042	3186	4664	3574
Marshall County	2344	2045	1851	1788	3652	1845

The Q_m/Q_c ratios for the shaft capacities excluding residual loads have been presented in Table 5.15. The average ratios for these 4 test are 1.25, 1.30, 0.82 and 1.20 for the UWA-05, ICP-05, NEN-9997-1 and NEN-9997-1* calculation methods, respectively. The lowest CoV was obtained using the ICP-05, closely followed by NEN-9997-1*. In general, the calculation methods underestimate the shaft capacity when residual loads are excluded, except for the NEN-9997-1 calculation method, which overestimates the shaft capacity for these 4 pile load tests.

Table 5.15: Q_m/Q_c of the 4 high quality pile load tests excluding residual loads for all the considered shaft calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
Pigeon River	0.95	1.22	0.80	1.12
Port of Rotterdam 02	1.44	1.38	0.95	1.24
Port of Rotterdam 03	1.36	1.30	0.89	1.16
Marshall County	1.27	1.31	0.64	1.27
Average	**1.25**	**1.30**	**0.82**	**1.20**
Variance	0.035	0.003	0.013	0.004
Standard deviation	0.187	0.057	0.115	0.059
CoV	**0.149**	**0.044**	**0.140**	**0.050**

The Q_m/Q_c ratios for the shaft capacities including residual loads have been presented in Table 5.15. The average ratios for these 4 test are 0.97, 1.00, 0.63 and 0.92 for the UWA-05, ICP-05, NEN-9997-1 and NEN-9997-1 calculation methods, respectively. The CoVs are similar for all the calculation methods when considering the shaft capacity, including residual loads. The lowest CoV was, however, obtained using the ICP-05. In general, the UWA-05 and ICP-05 shaft capacity calculation methods predict the shaft capacity better with the inclusion of residual loads for these 4 pile loads tests, with Q_m/Q_c ratios close to unity. The NEN-9997 calculation method on the other hand, overestimates the shaft capacity. This overestimation is worse than in the case of residual load exclusion. Limiting the q_c values, such as, in the NEN-9997* method results to calculations closer to the measured values.

Table 5.16: Q_m/Q_c of the 4 high quality pile load tests including residual loads for all the considered shaft calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
Pigeon River	0.61	0.79	0.52	0.72
Port of Rotterdam 02	1.13	1.08	0.74	0.97
Port of Rotterdam 03	1.05	1.00	0.68	0.89
Marshall County	1.10	1.14	0.56	1.11
Average	**0.97**	**1.00**	**0.63**	**0.92**
Variance	0.044	0.018	0.008	0.019
Standard deviation	0.211	0.135	0.090	0.139
CoV	**0.217**	**0.135**	**0.144**	**0.151**

Additionally, due to the availability of strain gauge readings with short intervals along the pile for pile load tests Marshall County and Pigeon River, determination of the measured and calculated shaft capacity per strain gauge interval results a visual aid to analyse the shaft calculation methods (Figure 5.7). For both Marshall County (Figure 5.7(left)) and Pigeon River (Figure 5.7(right)) the residuals are excluded in the measured strain readings. Overall, the UWA-05 and ICP-05 are very similar in both pile load tests, which is due to their similarities in their formulations. The NEN-9997-1 overestimates the majority of the strain intervals along the pile. However, the method is comparable to the UWA-05 and ICP-05 for the clay section at -10.5m for pile load test Marshall County. For the NEN-9997-1*, the limit on q_c is applied in both cases, underestimates the shaft resistance at the bottom of the pile and therefore the limit on q_c in these cases can be considered too low. Lastly, in the case of Marshall County, shaft resistance is also overestimated at near the top of the pile for the NEN methods. This could be due to no friction fatigue term being present within the NEN-9997-1 formulation. Although the effect of friction fatigue is taken into consideration within the α_s factor, the process seems to be too complex to be captured through one parameter in order to accurately calculate the shaft capacity.

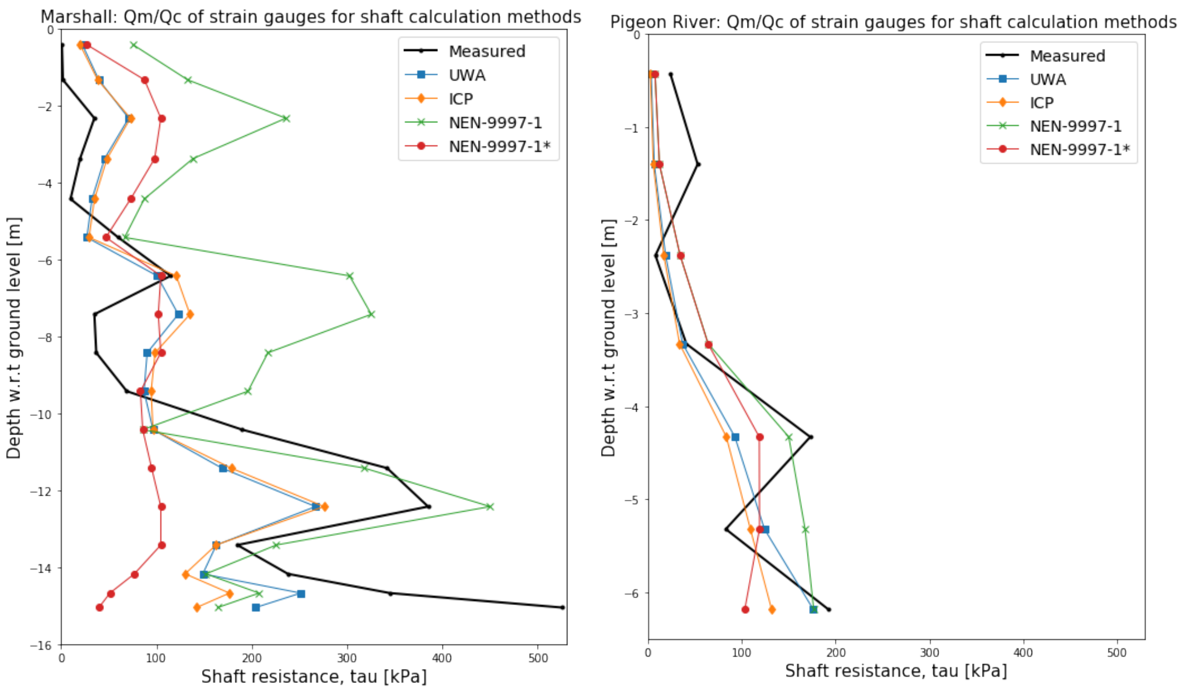

Figure 5.7: Strain gauge measurements compared to calculated values for the UWA-05, ICP-05, NEN-9997-1 and NEN-9997-1* for Marshall county (left) and Pigeon River (right) pile load tests

The α_s factors per strain gauge segment were also investigated, (excluding residual loads) and when combined with the α_s factors obtained from pile load tests from the Deltares database, a mean value of 0.007 was obtained. This mean value, however, has a high CoV of 0.57. It is therefore difficult to consider this a representative α_s value for sands due to the range of values obtained. Due to the additional data points from the new tests, identification of trends in the data can be done more easily. The α_s data was plotted against the distance with respect to the pile tip normalised by the diameter (h/D)(Figure 5.8(a)). An overlay of the friction fatigue term for the UWA-05 was applied over the α_s data (red line in Figure 5.8(a)), which resulted in good fit with limited outliers. A linear relationship between α_s and the friction fatigue term is confirmed when these two variables are plotted (Figure 5.8(b)).

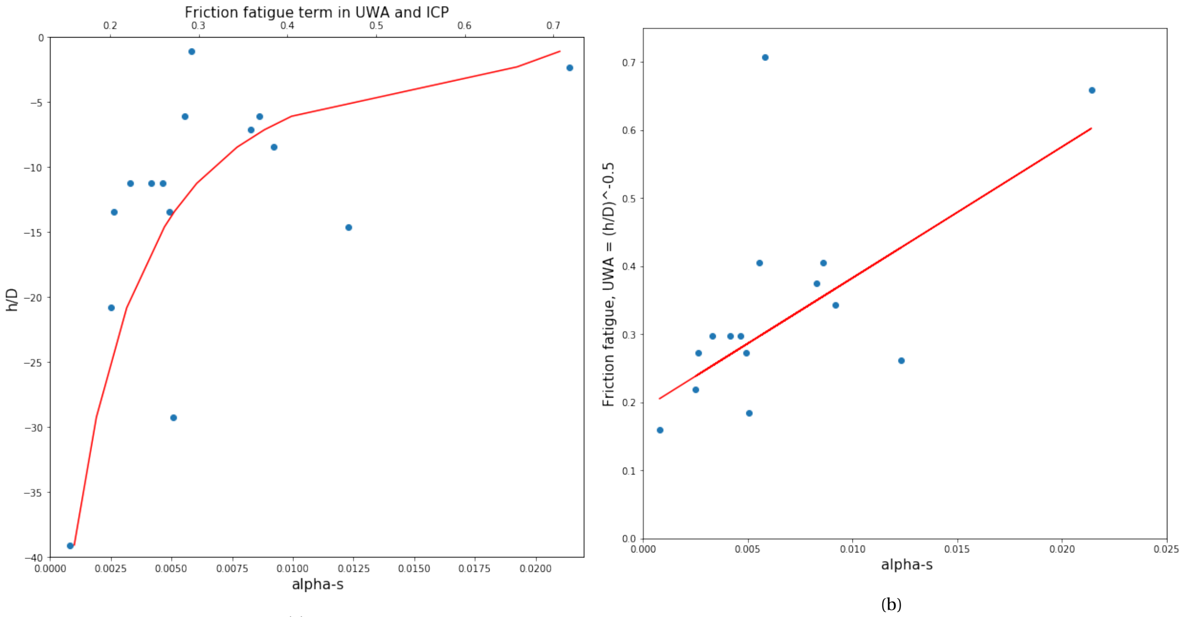

Figure 5.8: (a) Alpha-s against h/D with an overlay of the UWA-05 friction fatigue term (b) Linear relationship between alpha-s and athe UWA-05 friction fatigue term

5.2.3. Total capacity

The Q_m/Q_c ratios for the total capacities of the 4 high-quality tests, using the base and shaft capacities calculated before, have been presented in Table 5.17. It has to be noted that the residual loads have no effect of the total measured capacity. The average ratios for these 4 test are 1.31, 1.37, 0.95, 1.21, and 1.03 for the UWA-05, ICP-05, NEN-9997-1, NEN-9997-1* and Alternative in combination with the ICP-05 calculation methods, respectively. These are quite high compared to the ratios obtained for the Deltares database. This is modtly due to the high ratios obtained by the Koppejan and LCPC averaging techniques. The lowest CoV was obtained by the UWA-05, closely followed by the Alternative in combination with the ICP-05 calculation method. However, the UWA-05 on average underestimates the total capacity by more than 30% while the Alternative in combination with the ICP-05 calculation method on average underestimates the total capacity by only 3%.

Table 5.17: Q_m/Q_c of the total capacity of the 4 high quality pile load tests for all the considered calculation methods

Pile load test	Qm/Qc [-]				
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*	Alt-ICP
Pigeon River	1.39	1.04	1.18	1.43	0.87
Port of Rotterdam 02	1.32	1.60	0.95	1.14	1.12
Port of Rotterdam 03	1.40	1.61	0.99	1.19	1.15
Marshall County	1.14	1.23	0.68	1.08	0.99
Average	**1.31**	**1.37**	**0.95**	**1.21**	**1.03**
Variance	0.011	0.060	0.033	0.019	0.012
Standard deviation	0.104	0.245	0.181	0.139	0.111
CoV	**0.079**	**0.179**	**0.190**	**0.113**	**0.108**

5.3. Final alpha factors and summary

For a final evaluation, a representative database was constructed using pile load tests from the Deltares database as well as the high-quality tests used in the investigation of residual loads. The representative database compromises of the following pile load tests:

- TNO pile 01
- TNO pile 02
- Port of Rotterdam 02
- Port of Rotterdam 03
- Marshall County

The TNO pile tests were included in the representative database due to the high quality of execution for these pile load tests, while the Pigeon River test was removed due to the doubts present over the residual load measurements (Han et al., 2019). The representative database will return representative α_p values for the averaging techniques for piles where the residual loads are excluded. Comparisons will be made with the Deltares database, extension exclusive and extension inclusive residual loads. The comparisons of the base Q_m/Q_c ratios are presented in Table 5.18.

Table 5.18: Comparison of the Q_m/Q_c ratios for the base capacity calculations for the different databases

Database	Qm/Qc [-]			Qm/Qc [-]			
	Koppejan	LCPC	Alternative	UWA-05	ICP-05	NEN-9997-1	Alt*
Deltares	0.70	0.56	0.51	1.16	1.11	0.99	1.03
Extension excl.	0.84	0.80	0.54	1.40	1.80	1.20	1.08
Extension incl.	1.13	1.12	0.74	1.89	2.52	1.62	1.06
Representative	0.62	0.72	0.50	1.04	1.58	0.89	1.00

The α_p factors obtained through the representative database are **0.62**, **0.72** and **0.50**, for the Koppejan, LCPC and Alternative averaging techniques, respectively. The α_p factors for the Koppejan and LCPC method are skewed by the Port of Rotterdam pile load tests. This is due to their embedment length into sand being <1.0D and hence falling within the methods influence zones. The CoVs for the three α_p factors are **0.229**, **0.449** and **0.204**, for the Koppejan, LCPC and Alternative averaging techniques, respectively. For the Q_m/Q_c ratios of the base capacity calculation methods, the UWA-05 has a an average close to unity while the ICP-05 on average underestimates the measured base capacity. The NEN-9997-1 on average overestimates the measured base capacity for the representative database. The NEN-9997-1 calculates a value close to unity for the Deltares database however, the quality of some of the pile load tests in the Deltares database are not guaranteed (ESOPT II & CIAD). The Alt* column used the alternative averaging technique times an $\alpha_p = 0.5$ for all databases, except the extension incl. where an $\alpha_p = 0.7$ was used.

The α_p factors for the averaging techniques with the inclusion of residual loads based on the extension incl. pile load tests are equal to 0.98 for the Koppejan method (removal of Pidgeon River), 0.68 for the LCPC method and 0.74 for the alternative method. Additional pile load test where the residual loads are measured would have to be investigated to confirm these α_p factors since 4 pile load tests are not enough to validate these α_p factors for when residual loads are included. Additionally, the dependency with embedment length in the sand bearing layer was still present.

Next, a comparison was made for the Q_m/Q_c ratios for the shaft capacity calculation methods. The results have been presented in Table 5.19. The most notable values are ratios for the UWA-05 and ICP-05 calculation methods for the extension incl. The values for both calculation methods are very close or equal to unity. The same calculation methods tend to underestimate the shaft capacity when the residuals are not included. The CoVs for the averages obtained by the representative database are 0.248, 0.203, 0.162, 0.180 for the UWA-05, ICP-05, NEN-9997-1 and the NEN-9997-1* calculation methods, respectively. The ICP-05 and NEN-9997-1 in all the databases have one of the lowest CoVs.

Table 5.19: Comparison of the Q_m/Q_c ratios for the shaft capacity calculation methods for the different databases

Database	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
Deltares	0.93	1.03	0.76	0.85
Extension excl.	1.25	1.30	0.82	1.20
Extension incl.	0.97	1.00	0.63	0.92
Representative	1.13	1.14	0.78	1.07

Lastly, the comparison of the total Q_m/Q_c ratios are presented in Table 5.20. For the representative database, the NEN-9997* has a ratio closest to unity followed by the alternative method in combination with the ICP-05 shaft capacity calculation. The CoVs for these two average ratios are 0.177 and 0.185, respectively.

Table 5.20: Comparison of the Q_m/Q_c ratios for the total capacity calculations for the different databases

Database	Qm/Qc [-]				
	UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*	Alt*+ICP-05
Deltares	1.01	1.03	0.84	0.89	1.00
Extension excl. / incl.	1.31	1.37	0.95	1.21	1.03
Representative	1.10	1.22	0.81	0.99	1.09

6
Conclusion and recommendations

6.1. Conclusion

In this report, a new q_c averaging technique is developed to calculated a representative value of the soil strength for the calculation of the base capacity. The new alternative averaging technique is based mainly on the method proposed by Boulanger and de Jong (2018) and was calibrated through a series of CPT calibration tests in thin layered soils and two layered sands.

The accuracy of the new alternative averaging technique on the CPT calibration tests was compared to the accuracy of existing q_c averaging techniques, such as, the Koppejan and LCPC averaging techniques. For these CPT calibration test the Koppejan averaging technique computed values close to the measured q_c for layered sections of the profiles however, the computed values were always on the conservative side. For the transition zones, the Koppejan averaging technique underestimated the strength of the soil present. The LCPC averaging technique overestimated the strength of the soil in layered soils and did not capture the transition zones. Lastly, the alternative averaging technique overall resulted in the best fit for the CPT calibration tests. The transition zones between the different types of soil and layered sections were captured effectively compared to the other averaging techniques, as is demonstrated by the mean squared error analysis.

Further comparisons of the alternative averaging technique and all the considered averaging techniques were made by applying the different averaging techniques to the Deltares database of pile load tests (excluding residual loads). A lower value for the coefficient of variation, CoV, was found for the alternative averaging technique, however, a dependency with embedment length in sand could still be observed. According to Randolph (2003) & White and Bolton (2005) this can be explained through exclusion of residual loads. Hence, the averaging techniques were applied to a number of pile load tests were the residual loads have been measured. However, no decrease in the CoV was observed. The presence of a dependency with embedment length in sand could neither be denied or confirmed with the inclusion of residual loads due to the limited tests available with residual load measurements.

For the shaft capacity calculation, the presentiment by van Tol et al. (2013) over the simplicity of the NEN-9997-1 shaft capacity calculation was confirmed. In the NEN-9997-1, friction fatigue is not captured and the limit applied to q_c overall underestimates the shaft capacity present, especially in the presence of >>12Mpa q_c values. A number of α_s values were computed for different sand layers present in the pile load tests considered. These values showed a linear correlation with the h/D friction fatigue terms used for the ICP-05 and UWA-05. The introduction of a friction fatigue term to the NEN-9997-1 shaft capacity calculation could therefore improve the accuracy of the calculation method.

6.2. Answering of research questions

Is an improvement needed in the currently used averaging techniques for the calculation of the base capacity?

Clear limitations of the Koppejan and LCPC averaging technique have been confirmed in this report. Both demonstrated a high dependency on penetration depth in the sand bearing layer. This dependency is directly linked to the influence zone taken above the pile tip, 8D for the Koppejan and 1.5D for the LCPC averaging technique. The penetration depth dependency is only present if the penetration depth in sand is less than the influence zone taken above the pile tip. Another limitation found for the Koppejan averaging technique, is the minimum path rule exposed through the Pigeon River pile load test. In this pile load test, the Koppejan averaging technique calculated a very small value for the base capacity due to presence of a thin, soft layer of soil under the pile tip activating the minimum path rule. These limitations lead to a relatively large variation in α_p factors when calculating the Q_m/Q_c ratios for the pile load tests. A CoV of 45% for the average α_p was calculated from the representative pile test database for the LCPC averaging. Therefore, a single α_p factor can not be applied in full confidence for the LCPC averaging technique and therefore calls for improvements. On the other hand, a CoV of 23% for the average α_p was calculated from the representative pile test database for the Koppejan method. An single constant α_p, 0.62 (excluding residuals), could possibly be used however, the above limitations for the Koppejan method should be taken into consideration when using this α_p factor.

The alternative new averaging technique was on the other hand, able to make more accurate calculations regarding the base capacity of a pile, with a smaller dependency on the penetration depth in the sand bearing layer. This was done using a relatively simple mathematical formulation, which combined a weight dependent on the distance from the pile tip and a weight dependent on the stiffness of the soil relative to the soil at the pile tip, within a 3.7D above the pile tip and 7.0D underneath the pile tip influence zone. The CoV for this averaging technique is 20% for the average α_p calculated from the representative pile test database, 3% lower variation compared to the Koppejan averaging technique and approximately half of the CoV calculated for LCPC averaging techniques. This allows the possibility for a single constant α_p, 0.50 (excluding residuals), to be used when utilising the alternative averaging technique. Improvements are still needed in order to remove the dependency of the α_p factor on embedment length into the sand bearing layer. However, one needs to remember that the final values obtained in this report are obtained from a limited number of pile load tests. Further credibility would be obtained through the application of the averaging techniques on more pile load tests.

Can the consideration of residual loads help or improve the calculations of the base and shaft capacity?

From a physical point of view, yes, since the consideration of the residual loads leads to the reflection of the true distribution of the loads on the base and shaft of a pile. However, as also stated by Fellenius (2002) residual load measurements is a complex process and requires considerable judgement during the processing of residual load data. Results can therefore be subjective, which makes it difficult to analyse pile load tests. In addition, very limited pile load tests are available at this point of time where the residual loads have been measured. Hence, clear conclusions can not be drawn from this investigation concerning the residual loads and the pile capacity calculations. Further pile load tests are needed to confirm any trends found from the 4 pile load tests investigated in this report.

Based on the 4 pile load tests, the dependency of the embedment length in the sand bearing layer and the α_p factors computed by the averaging techniques, did not diminish as predicted by Randolph (2003) & White and Bolton (2005). This was demonstrated by the recorded CoVs for the averaging techniques including and excluding residual loads showing little change.

Overall, the shaft capacity calculation methods made better estimates of the shaft capacity when the residual loads were considered. The ICP-05 and UWA-05 both having average Q_m/Q_c ratios close to unity. The NEN-9997-1* (Dutch norm including limitation on q_c) shaft capacity calculation method, however, on average overestimated the shaft capacity by 8%.

How do other shaft capacity calculation methods compare to the conventional $\alpha_s \cdot q_c$ method used in the NEN?

In terms of formulation, the NEN uses a single term (α_s) to capture the soil interactions occurring in the vicinity of the pile shaft. All the other shaft calculation methods considered in this report include a term related to friction fatigue. This term reduces the calculated shaft capacity near the top of a pile. The shaft capacity in this area of the pile tends to be overestimated by the NEN due to the lack of this friction fatigue term. Other terms used by the most accurate shaft calculation methods (ICP-05 and UWA-05) are a term related to dilation ($\Delta\sigma'_{rd}$) and a term related to the interface friction ($\tan\delta_f$). On average, the NEN shaft capacity calculation method without any limit on q_c significantly overestimates the measured shaft capacity, as demonstrated by the 22% average overestimation for the representative database.

Is there a need to limit the q_c values for the calculation of the base and shaft capacity as is currently done in the NEN?

The limit applied to q_c values according to the NEN could only be investigated for the shaft capacity because of the q_c value for base capacity calculation only exceeding the 15MPa limit for one pile load test, making the data sample too small. For the shaft capacity, the limiting the q_c values in the majority of the pile load tests improved the Q_m/Q_c ratio (i.e. closer to unity). In comparison to the ICP-05 and UWA-05, the NEN-9997* have similar CoVs. However, limiting the q_c values to 12MPa for q_c values >>12MPa results in a severe underestimation of the shaft capacity, especially near the pile tip where the confining stresses are high. This can for instance be observed in the Marshall County strain gauge data. In the Netherlands, the amount of sand along a pile is limited due to the presence of soft soils. Hence, the limitation of the q_c values and the reduction through α_s globally captures the friction fatigue in the minimal sand present, which leads to adequate estimations in the majority of cases. However, for more accurate and consistent shaft capacity calculations the original q_c values should be maintained, keeping the original physical soil skeleton acting on the shaft of the pile in consideration. Instead a reduction factor should be applied related to penetration depth with respect to the pile tip and therefore introducing a term which takes into account the friction fatigue interaction present between the pile and soil due to the correlation found in this report.

6.3. Recommendations

From the investigation in this report, a number of recommendations can be presented. The recommendations have been categorised into recommendations for the base capacity and shaft capacity calculations.

6.3.1. Recommendations for the base capacity calculations

Considering the α_p factors for the averaging techniques, first of all, the the currently used and new alternative averaging technique at present carries an uncertainty when used for base capacity calculation due to the dependency with embedment length in sand. In contrary to Randolph (2003) & White and Bolton (2005) believes, the incorporation of residual loads had no effect on the this dependency. Their presentiment can however not be ruled out. This is due to the limited pile load tests which included residual loads considered in this investigation. Additionally, the measured residual loads in these tests are sensitive to interpretation and could therefore possibly vary from the true residual load present. Fellenius (2002) states that considerable judgment must be exercised in the analysis and use of the results of the residual loads obtained (Appendix A). Hence, an additional investigation is recommended with additional well documented pile load tests which include residual load measurements and consistency in the interpretation of these tests.

The limitations of the Koppejan averaging technique are understood in the Netherlands, however, its application tends to be on the conservative side. This can therefore not be applied in every situation. The alternative averaging technique has demonstrated to be a more accurate calculation method for the representative q_c values for base capacity calculations of piles due to the low corresponding CoVs. From the pile load tests considered in this report, a wide range of situations were calculated. This showed that the alternative averaging technique can be used accurately for a variety of ground profile conditions, while the Koppejan and LCPC averaging techniques demonstrate more extreme variation under certain conditions.

Ultimately, the alternative averaging technique demonstrated less dependence on the embedment length in sand compared to the Koppejan and LCPC averaging techniques. The alternative averaging technique in its current state is a simple mathematical model although with some modifications this averaging technique could potentially be able to remove the dependency with embedment length in the sand bearing layer. This could be done using a slightly more complex shape for the weights dependant on the distance from the pile tip in combination with the inclusion of residual loads. However, this should not be at the cost of its current simplicity.

6.3.2. Recommendations for the shaft capacity calculations

Currently, the most accurate method is the ICP-05 for the shaft capacity calculation, closely followed by the UWA-05. The NEN-9997-1* is adequate in terms of accuracy but fails to calculate accurate shaft capacities if >>12MPa q_c values are present. In order to improve the accuracy of the NEN-9997-1* method, the limit on q_c should be removed and can be replaced by introducing a friction fatigue term. This would be an effective approach to apply a correct reduction on q_c along the pile shaft. Investigating this option in the future would be beneficial and practical in a more extensive investigation into additional pile load tests where residual loads have been measured. This will determine the suitability of the currently used shaft capacity calculation methods on the calculation of the shaft capacity including residual loads.

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A

Appendix

A.1. Dutch standard pile load test

The approach for pile load tests in the Netherlands are executed in a manner so that the following can be interpreted from the measurements taken during the test: deformation, creep and rebound of the pile. Hence, the design of the pile load tests will consider the load steps, the time for each load step and the application of load cycles (Normcommissie-351-006-Geotechniek, 2017). The load in each load step is maintained until the settlement rate reaches a certain value known at the creep criterion. This value is not specified in the Normcommissie-351-006-Geotechniek (2017) however, around the world, pile load tests normally define the creep criterion at around 0.3-0.5mm/hr (Han, Prezzi, Salgado, and Zaheer (2017), Paik et al. (2003) and Grondmechanica-Delft (1993)). Typically, the following graphs can be produced from a pile load test to determine the desired information about a pile:

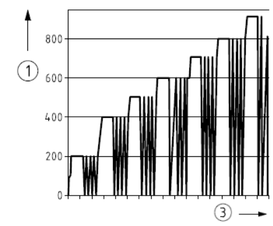
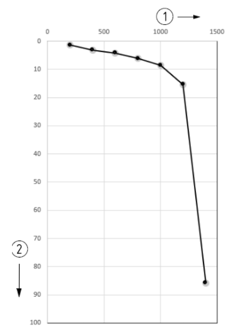
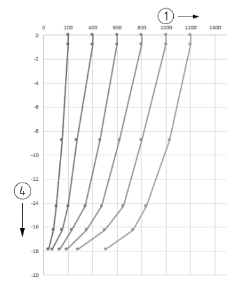


Figure A.1: Load v Time, load steps and load cycles (Normcommissie-351-006-Geotechniek, 2017)



(a) Load displacement curve from TNO test pre-cast pile 2



(b) Load distribution curves for the different load steps from TNO test pre-cast pile 2

Figure A.2: Where; 1 - Load, 2 - Displacement, 3 - Time and 4 - Depth

A.2. Dutch standard pile resistance calculation

The Dutch standard is calculated according to the Normcommissie-351-006-Geotechniek (2017) for the calculation of the pile resistance. This is divided into the base and shaft resistance. The base resistance is calculated as follows:

$$q_{b,max} = \frac{1}{2} \cdot \alpha_p \cdot \beta \cdot s \cdot \left(\frac{q_{c;I;gem} + q_{c;II;gem}}{2} + q_{c;II;gem} \right) \tag{A.1}$$

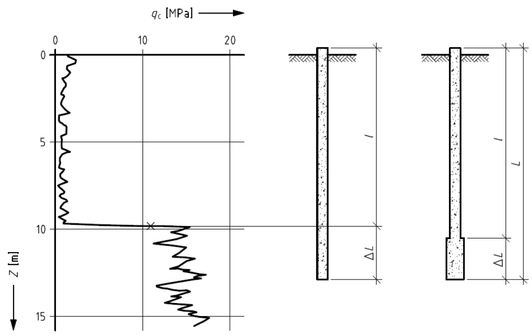
Where $q_{b,max}$ is the maximum base resistance, which is reduced to an upper limit of 15MPa for values above 15MPa.α_p is the reduction factor depending on the pile type. For soil displacement piles α_p = 0.7 (Table 7.c in Normcommissie-351-006-Geotechniek (2017)). s is a shape factor which is equal to 1.0 for symmetric piles. The remaining component of equation A.1 follows the Koppejan method for the calculation of the base resistance (van Mierloo & Koppejan, 1952). $q_{c;I;gem}$ is the minimum average q_c value over the length of at least 0.7D to maximum 4D under the pile tip. $q_{c;II;gem}$ is the average q_c value from the maximum distance below the tip, chosen for the calculation of $q_{c;I;gem}$, to the pile tip using the minimum path method (i.e. the values of q_c taken in account can never be higher than the q_c values below it (see Figure 2.6)). Lastly, $q_{c;III;gem}$ is the average q_c value above the pile tip up till 8D, following the minimum path from the $q_{c;II;gem}$ calculation (Figure 2.6).

The shaft resistance is calculated as follows:

$$q_{s;max;z} = \alpha_s \cdot q_{c;z;a} \tag{A.2}$$

Where $q_{s;max;z}$ is the maximum shaft resistance at a depth z.α_s is the reduction factor dependant on the soil type, see tables 7.c and 7.d in Normcommissie-351-006-Geotechniek (2017). Lastly, $q_{c;z;a}$ is equivalent to q_c reduced to an upper limit of 12MPa for values above 12MPa. An exception is made for soil layers with a thickness of at least 1 m and values ≥15MPa. In this case the upper limit is 15MPa.

The length of the pile which can be included for the calculation of the shaft resistance according to the (Normcommissie-351-006-Geotechniek, 2017) is known as ΔL. This distance varies with the soil profile but generally only takes into account for the resistance from the deep sand layers (Figure A.3).



Figuur 7.g — Verklaring van *L*, Δ*L* en *l*

Figure A.3: Visualisation of pile length ΔL, Figure 7.g from Normcommissie-351-006-Geotechniek (2017)

A.3. Determination of residual loads

According to Fellenius (2002) residual loads measurements can be estimated in three ways: through an effective stress analysis on shaft capacity distributions obtained during a pile load test or CAPWAP analysis on a dynamic pile load test (Figure A.4) or through direct measurement of the residual load. The last option requires working strain measuring devices during installation. The residual load can then be measured right before a pile load test through the strain data. This requires quality strain measuring devices which do not 'drift' from zero.

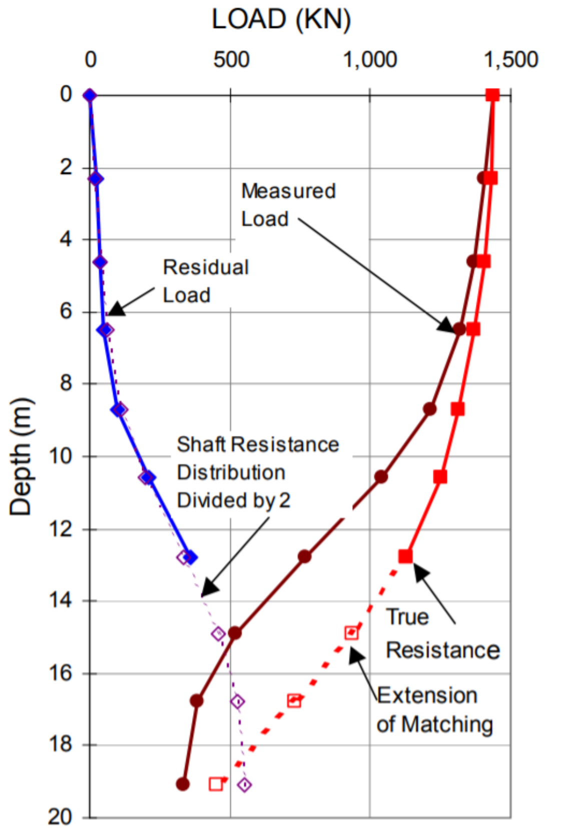


Figure A.4: Example of estimation of the residual loads through effective stress analysis Fellenius (2002)

Quality instrumentation of piles remains a challenge due to damaged strain measuring devices and calibration issues occurring during the pile construction (Fellenius, 2002). Fellenius (2002) therefore warns, that considerable judgment must be exercised in the analysis and use of the results of the residual loads obtained.

A.4. Boulanger - de Jong additional equations and figures

The exact formulas used for the Boulanger - de Jong inverse filtering method can be found below. Additional information about the method can be found in the paper published by Boulanger and de Jong (2018).

$$q^m(z) = q^t(z) \cdot w_c(z) \tag{A.3}$$

$$w_c = \frac{w_1 \cdot w_2}{\sum w_1 \cdot w_2} \tag{A.4}$$

$$w_1 = \frac{C_1}{1 + \left(\frac{z'}{z'_{50}}\right)^{m_z}} \qquad w_2 = \sqrt{\frac{2}{1 + \left(\frac{q^t_{z'}}{q^t_{z'=0}}\right)^{m_q}}} \tag{A.5}$$

$$z' = \frac{z - z_{tip}}{d_c} \tag{A.6}$$

$$z'_{50} = 1 + 2(C_2 \cdot z'_{50,ref} - 1)\left(1 - \frac{1}{1 + \left(\frac{q^t_{z'-0}}{q^t_{z'}}\right)^{m_{50}}}\right) \tag{A.7}$$

Where,

$C_1 = 1$ for $z' \geq 0$: is the unity for the points below the cone tip

$= 1 + \frac{z'}{8}$ for $-4 \leq z' < 0$

$= 0.5$ for $z' < -4$

w_1: weight related to distance from the pile tip

w_2: weight related to the difference in cone resistance

z': depth relative to the cone tip normalized by the cone diameter, d_c

z'_{50}: normalized depth at which $w_1 = 0.5C_1$

m_z: adjusts the variation of w_1 with z' (3.0)

C_2: is the unity for points below the cone tip and less than unity for points above the cone tip.

$z'_{50,ref}$: is the value of z'_{50} for points below the cone tip whenever $q^t_{z'} = q^t$ at the cone tip (1.5)

m_q: adjusts the variation of w_2 with $q^t_{z'} / q^t_{z'=0}$ (2.0)

B
Appendix

B.1. Pile test description

B.1.1. TNO pile 01 and 02 1993

General description
TNO Bouw requested Grondmechanica Delft in October 1992 to perform pile tests on 4 displacement piles. 2 pre-cast concrete driven piles and 2 vibro piles at TNO Delft. The aim of these tests was to compare the measured pile base and shaft capacities to the calculated values according to the NEN 6743 Dutch norm. A total of 15 CPT's were carried out, including one CPT at the exact position of each tested pile (Grondmechanica-Delft, 1993).

Pile information

- Square pre-cast concrete piles, b = 0.29 mm
- Pile length = 18.25 m
- Pile tip = -19.50 m NAP

Strain gauge information
The strain gauges were installed on to the reinforcement bars within the pile. Position of strain gauges:

- 1 – 2 -2.0m NAP
- 3 -10.0m NAP
- 4 – 5 -15.5m NAP
- 6 -17.5m NAP
- 7 – 8 -19.1m NAP

Measured data

- Load on pile head
- Displacement of the pile head
- Displacement of the pile base
- Strain in the strain gauges

Additional information
An α_p = 0.45 - 0.6 and α_s = 0.005 were found according to the conclusion of the official pile load test report.

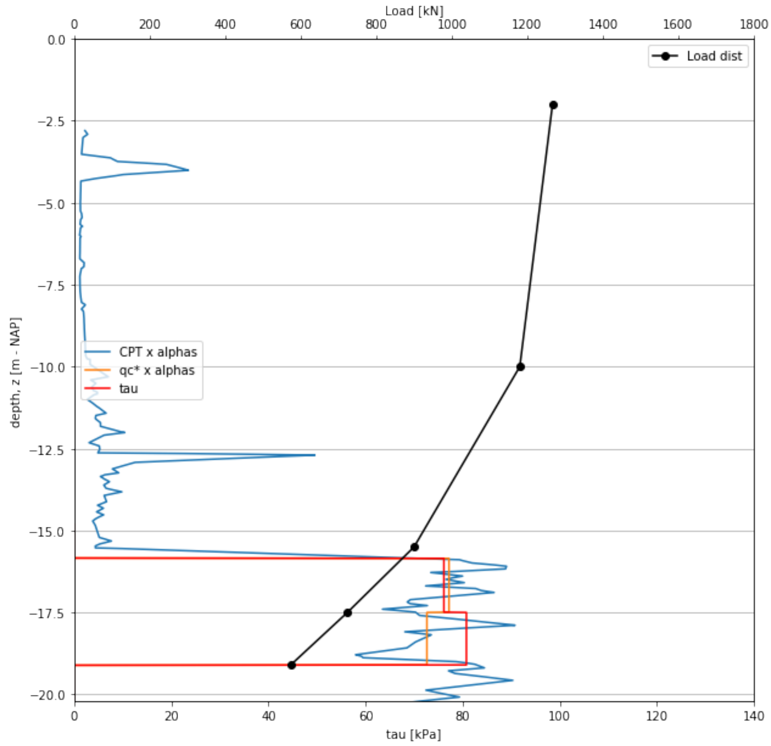


Figure B.1: Strain gauge data for TNO 01 pile load test

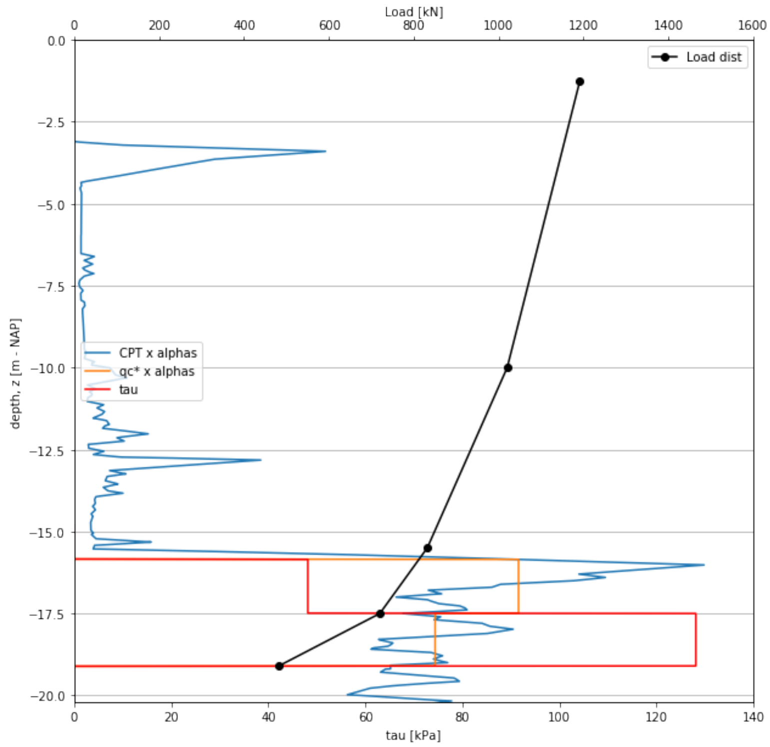


Figure B.2: Strain gauge data for TNO 02 pile load test

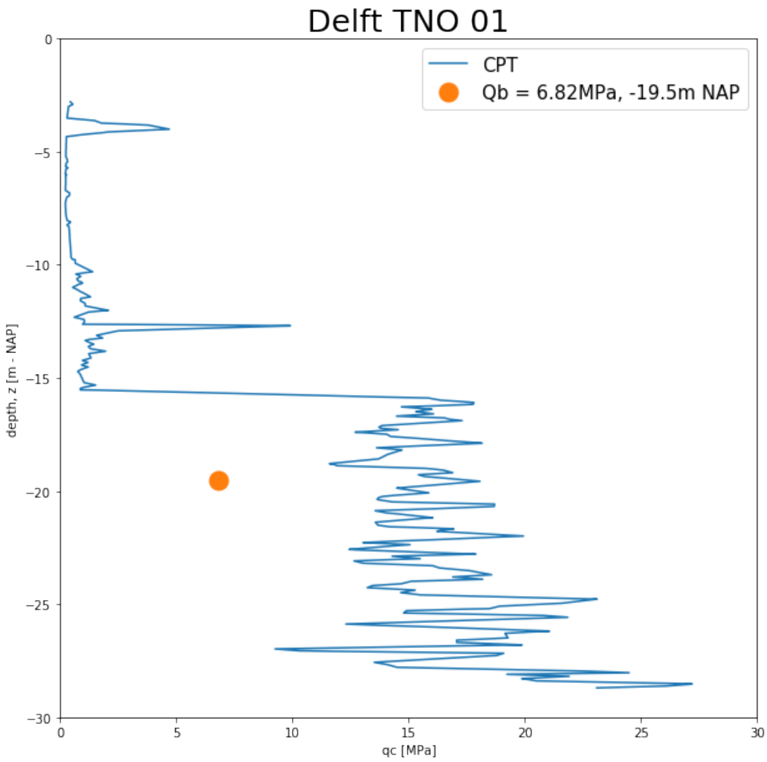


Figure B.3: CPT and measured tip resistance of TNO pile 01

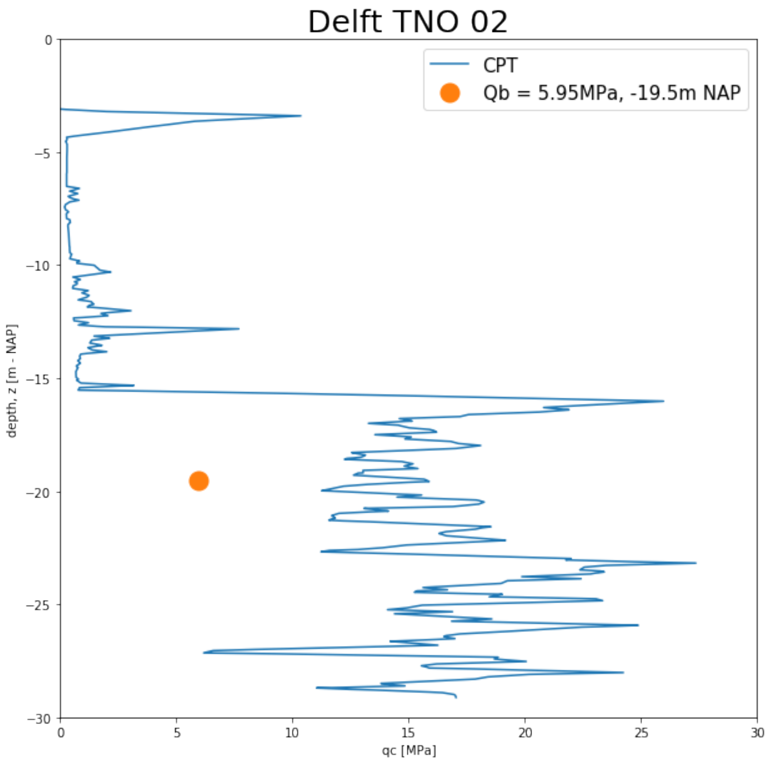


Figure B.4: CPT and measured tip resistance of TNO pile 02

B.1.2. CIAD 1979

General description

The CIAD test was part of an investigation to see the relation between the measured values and calculated values of the pile capacity. The calculation methods compared were CPT, API and L. C. Nottingham methods. The test was carried out in Amsterdam and includes a CPT. The other aim of the CIAD test was to compare the results to dynamic pile load tests done by TNO and IBBC (Grondmechanica-Delft, 1979).

Pile information

- Square pre-cast concrete piles, b = 0.40mm
- Pile length = 22.00m
- Pile tip = -21.80m NAP

Strain gauge information

Position of strain gauges:

- 1 -1.50m NAP
- 2 -16.25m NAP
- 3 -16.75m NAP
- 4 -21.30m NAP
- 5 -21.80m NAP

Measured data

- Load on pile head
- Displacement of the pile head
- Strain in the strain gauges

Additional information

- Pile was damaged on one side; steel reinforcement was visible.
- The connections to the strain gauges were not water tight. The connections were full of water after opening the covers for inspection.
- There was a 2 week break during pile testing due to the lack of sufficient ballast.

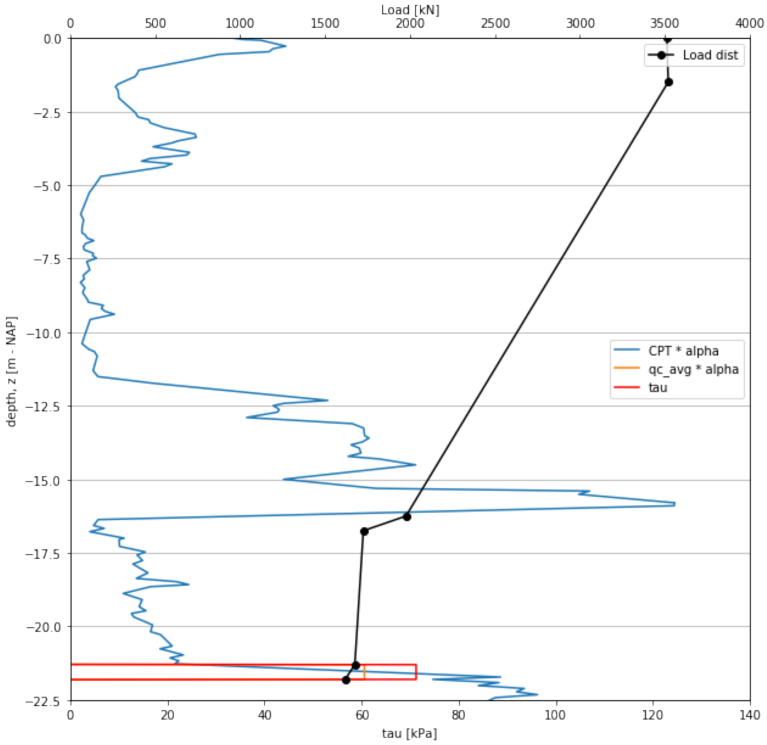


Figure B.5: Strain gauge data for CIAD pile load test

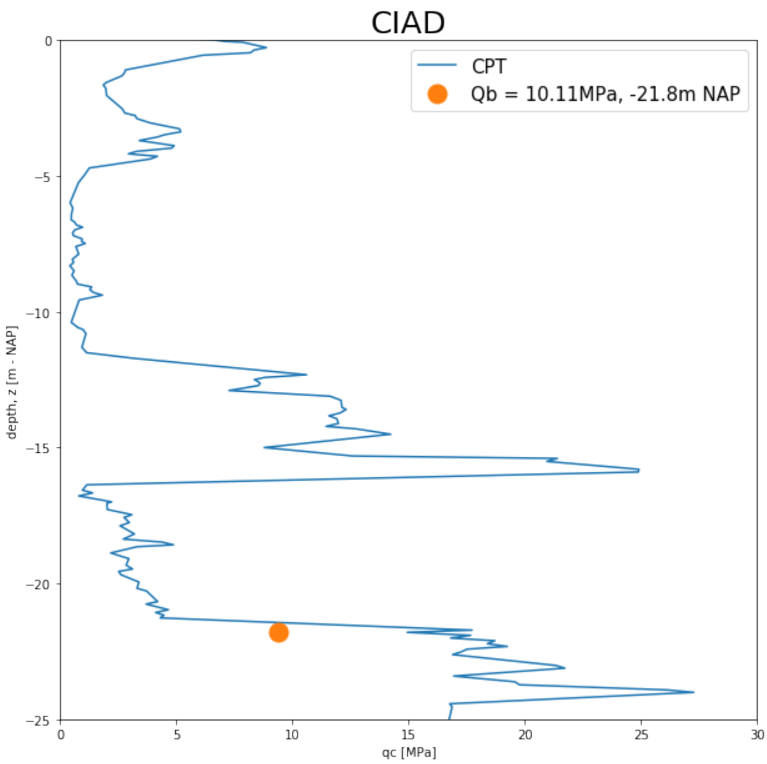


Figure B.6: CPT and measured tip resistance of CIAD pile

B.1.3. ESOPT II 1982

General description

The pile load test was part of ESOPT II symposium the in the RAI-Congress Centre in Amsterdam. The tested pile was located in the Beatrixpark close to the RAI-Congress Centre. Different tests were carried out close to the pile. These include a SPT, DPA, DPB, WST and a CPT which was 2 m from tested pile. The location of the pile tip was chosen due to the fall in q_c values in the sand layer, making it difficult to predict the pile behaviour. Hence, increasing the difficulty for the prediction competition of the bearing capacity of the symposium attendees (LGM-Mededelingen, 1982).

Pile information

- Square pre-cast concrete piles, b = 0.25mm
- Pile length = 15.00m
- Pile tip = -13.00m NAP

Strain gauge information

Position of strain gauges:

- 1 1.5m NAP
- 2 -5.0m NAP
- 3 -8.0m NAP
- 4 -11.0m NAP
- 5 -13.0m NAP

Measured data

- Load on pile head
- Displacement of the pile head
- Displacement of the pile base
- Strain in the strain gauges

Additional information

The concluded ultimate bearing capacities were:

- Qu = 1200kN ± 25kN, ultimate total capacity
- Qs = 500kN ± 50kN, ultimate shaft capacity
- Qb =700kN ± 50kN, ultimate base capacity

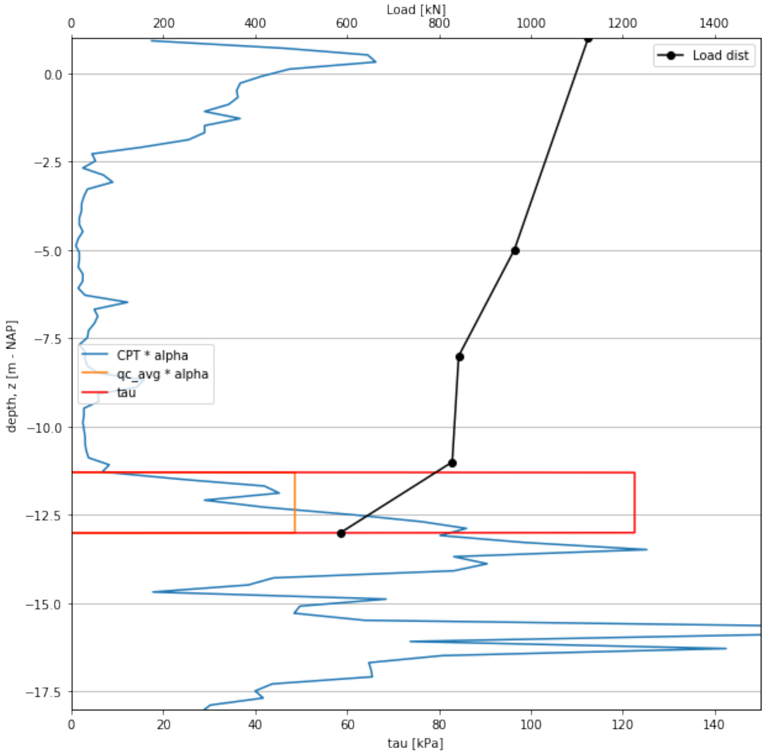


Figure B.7: Strain gauge data for ESOPT II pile load test

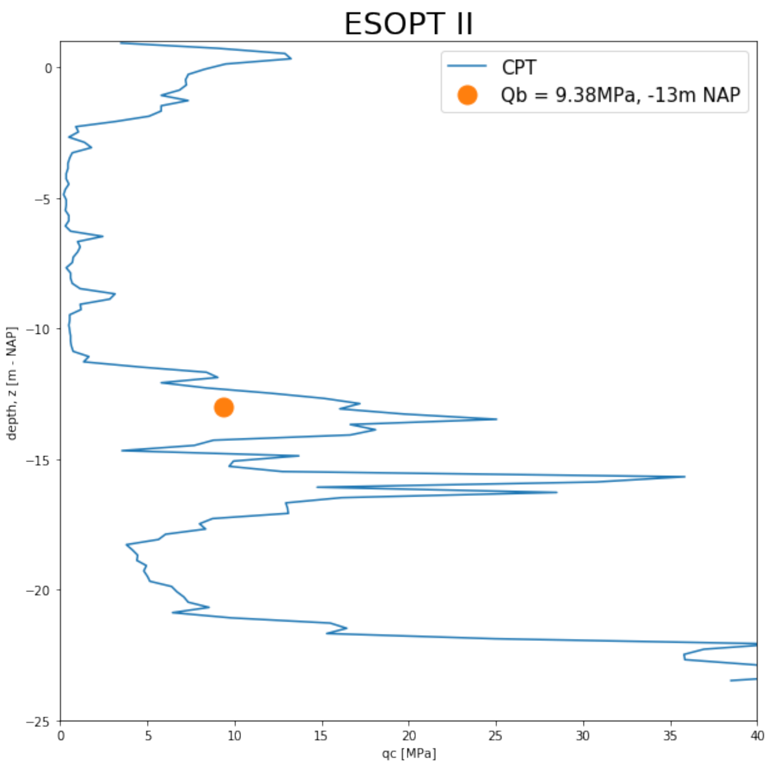


Figure B.8: CPT and measured tip resistance of ESOPT II pile

B.1.4. Kruithuisweg 1982

General description

In 1982, 5 pile load tests were carried out at the salt storage site on Kruithuisweg, Delft. The 5 load test were performed on a single steel pile. In each load the test the pile was loaded till failure and afterwards driven deeper into the soil for the next load test. The tests were performed as part of an investigation into the Koppejan bearing capacity calculation method. Additional comparisons were made on the measured q_c and actual shaft resistance of the pile. Furthermore, another part of the investigation was to explore the difference between different types of CPT's. Hence, 3 mechanical and 3 different electrical CPT cones were compared and used to gather q_c values. It is important to note that dynamic pile load test were carried out before static pile load tests I, II, III and V (Grondmechanica-Delft, 1982).

Pile information

- Circular steel piles, D = 0.355mm
- Pile length = 22.00m
- Pile tip = -12.20 - -24.00m NAP

Strain gauge information

In total 4 strain gauges were used which were installed on 3 levels inside the steel pile. The 3 levels for the positions of strain gauges were: 0.1m from the pile tip, 2.0m from the pile tip and 4.0m from the pile tip.

Measured data

- Load on pile head
- Displacement of the pile head
- Strain in the strain gauges

Additional information

- The Kruithuisweg report suggests that the bottom two strain gauges were damaged during testing and ended up taking an average value for friction between the two strain gauges.
- Dynamic pile load test were carried out before static pile load tests I, II, III and V.

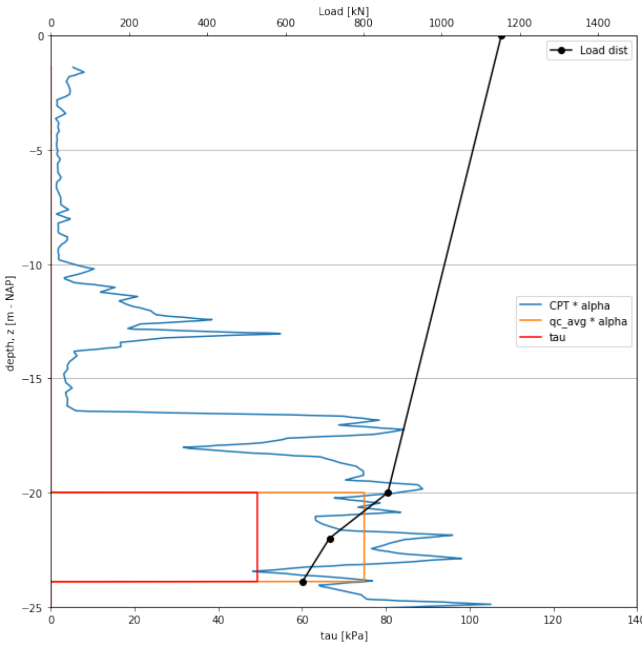


Figure B.9: Strain gauge data for Kruithuisweg V pile load test

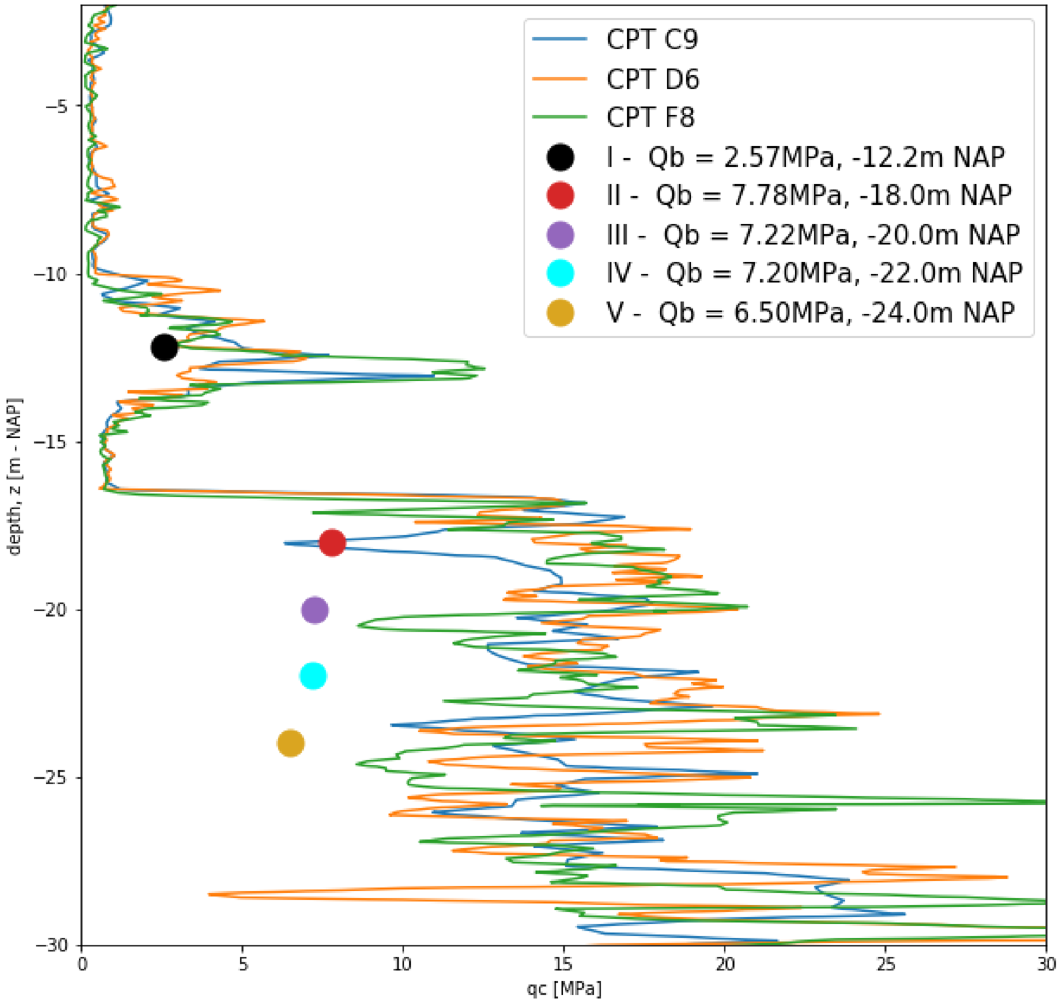


Figure B.10: CPT and measured tip resistance of Kruithuisweg pile

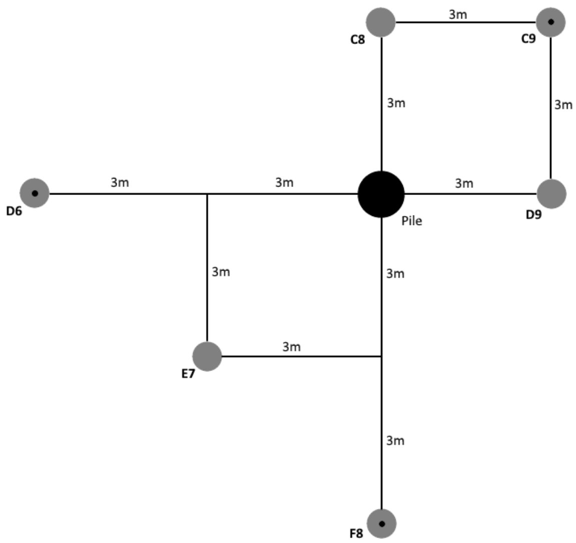


Figure B.11: Diagram of CPT and pile locations for Kruithuisweg pile load tests

B.1.5. Pigeon River 2003

General description

The test site is located South of a bridge construction site over the Pigeon River, on Sate road 9, Indiana. An open and closed ended pile were driven to a depth of roughly 7m as part of a study on load capacity of open-ended piles bearing in sand. CPTs and SPTs were performed both before and after driving as well as Pile Driving Analyzer (PDA) tests during driving. Residual loads were measured after driving the piles into position. Eventually, dynamic load tests were also carried out 8 days after the completion of the static load tests (Paik et al., 2003).

Pile information

- Circular steel piles, D = 0.356mm
- Pile length = 8.24m
- Pile tip = -7.04 (open-ended) and -6.87m (closed-ended) w.r.t ground level

Strain gauge information

In total 18 strain gauges were used which were installed on 9 levels of the steel pile for the closed-ended pile. Strain gauges were placed closer together near the pile base. While for the open-ended pile 20 gauges were used for the inner section of the pile and 18 for the outer section of the section of the pile, see Figure B.12.

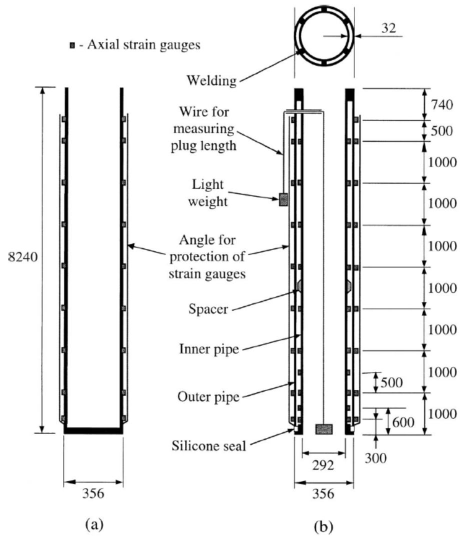


Figure B.12: Dimensions and pile set up for (a) closed-ended and (b) open-ended pile (Paik et al., 2003)

Measured data

- Load on pile head
- Displacement of the pile head
- Strain in the strain gauges
- Position of pile plug in open-ended pile

Additional information

- Residual loads (Table 1, Paik et al. (2003) were measured straight after driving of the pile
- Approximately 2m of fill material was removed before pile driving
- The contribution of the plug to the static pile base capacity is presently not well understood
- The base and shaft capacity of the open-ended pile at 0.1D normalized by the average cone resistances resulted in 30% and 58% lower than the corresponding values for the closed-ended pile

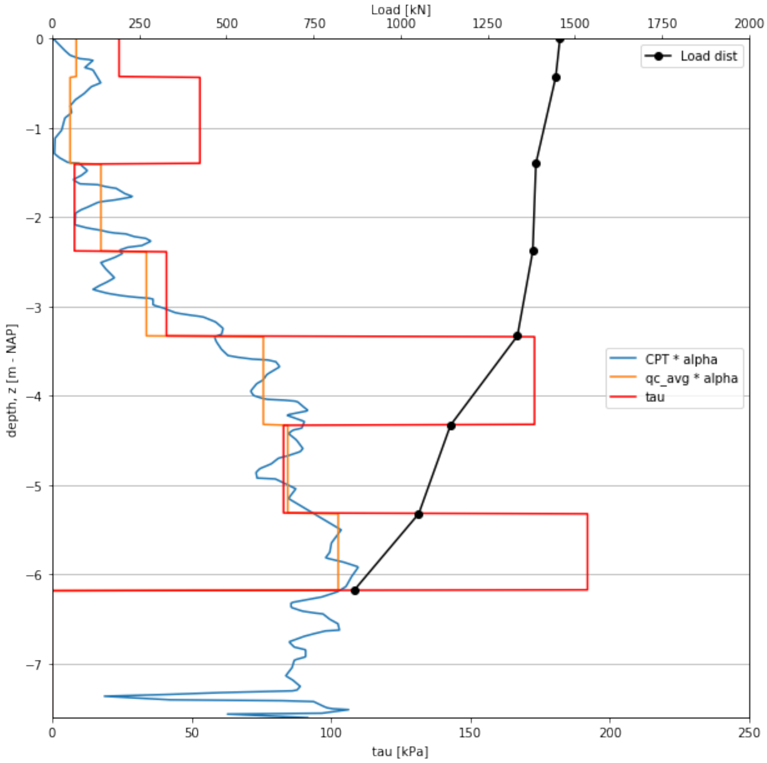


Figure B.13: Strain gauge data for Pigeon River pile load test

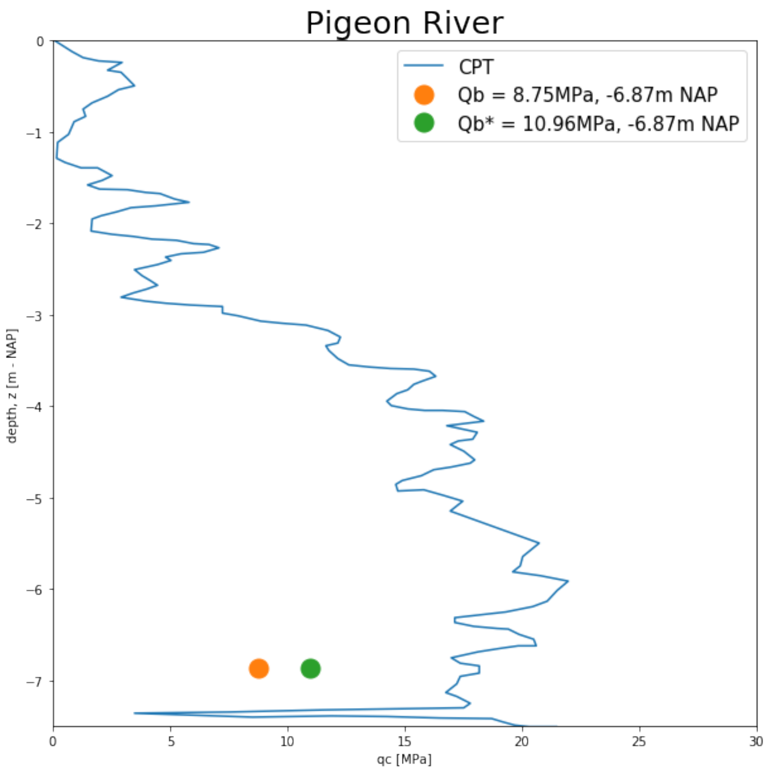


Figure B.14: CPT and measured tip resistance of Pigeon River pile. *Is the measured base resistance measured with residual loads considered

B.1.6. Marshall County pile 2017

General description

The test site is located in Marshall County, Indiana and is part of a construction site of the intersection of the 7th road. A closed-ended pile was driven to a depth of roughly 16m as part of a study on load capacity of piles in multi-layered soils. 2 CPTs and several SPTs were performed before driving. The soil profile consists mainly of layered medium dense to dense silty sand. The pile age during the static load test was 9 days and dynamic tests were carried out after the completion of the static load test (Han et al., 2017).

Pile information

- Circular steel piles, D = 0.356mm
- Pile length = 16.00m
- Pile tip = -15.42m w.r.t ground level

Strain gauge information

In total 20 strain gauges were used which were installed on different levels of the steel pile. Strain gauges were placed closer together near the pile base. See Figure B.15 for a complete overview of the instrumented pile.

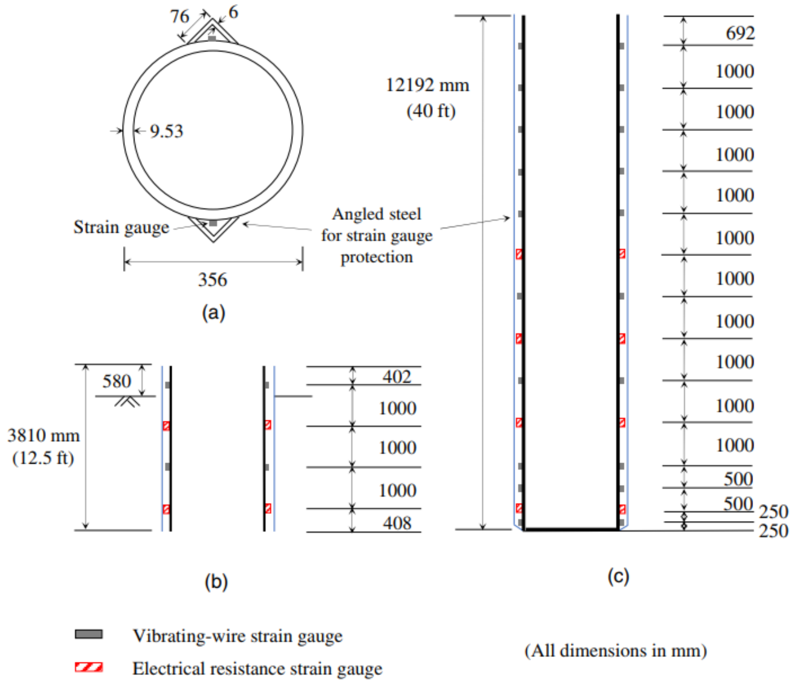


Figure B.15: (a) Cross section of instrumented pile, (b) strain gauge positions in top section of instrumented pile and (c) strain gauge positions in bottom section of instrumented pile (Han et al., 2017)

Measured data

- Load on pile head
- Displacement of the pile head
- Strain in the strain gauges

Additional information

- Pile consisted of two segments
- Residual loads were measured per unit metre (Table 7 in Han et al. (2017))
- Residual loads were measured right before the static load test

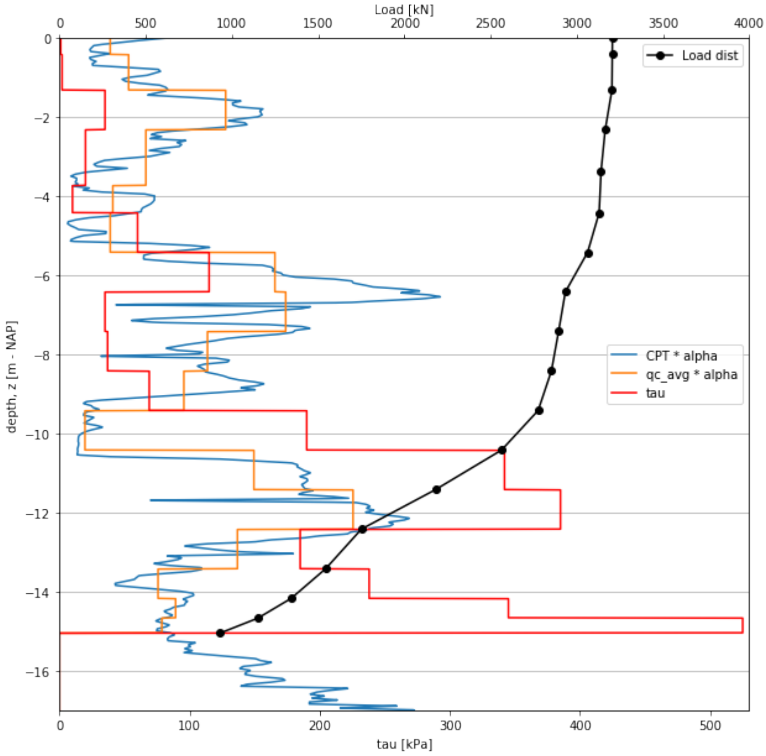


Figure B.16: Strain gauge data for Marshall County pile load test

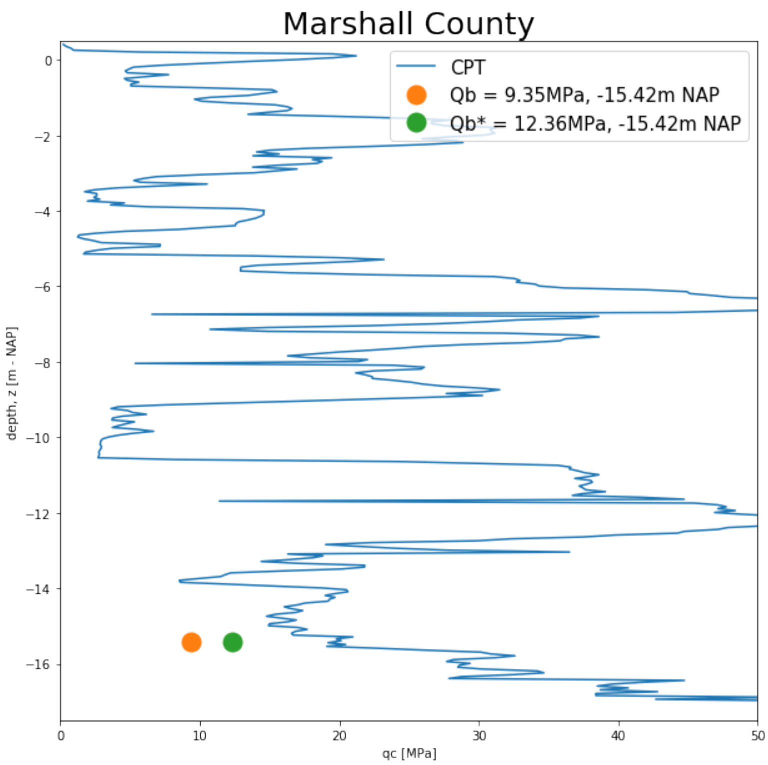


Figure B.17: CPT and measured tip resistance of Marshall County pile. *Is the measured base resistance measured with residual loads considered

B.1.7. Port of Rotterdam 2019

General description

The test site is located in the port of Rotterdam, Netherlands. A total of 4 static load tests were carried out, in order to optimise the design of a quay wall. The piles used were square concrete 36m long piles and a total of 5 CPTs were performed with 3.1m distance between each pile and 3 corresponding CPTs (Figure B.19). The soil profile consists mainly of sand as well as clay and contains none/little intermediate soils. Optical fibres were used for the measured of the capacity of the pile and distinctions were made between the capacity including and excluding residual loads (Matic, de Nijs, de Vos, & Roubos, 2019).

Pile information

- Square pre-cast concrete piles, D = 0.450mm
- Pile length = 36.00m
- Pile tip = -32.00m NAP

Strain gauge information

Two types of optical fibres were used FBG and BOFDA for strain measurements.

Measured data

- Load on pile head
- Displacement of the pile head
- Displacement of the pile base
- Strain in the optical fibres
- Residual load (Gavin, 2019)

Additional information

- Axial stiffness used based on Fellenius
- Residual loads were measured
- An α_p of **0.84** was found for the combined value of the 4 pile load tests
- One of the four pile load tests did not reach the failure criteria 0.1D
- Another pile load test was not considered due to very low capacity in comparison to other 3

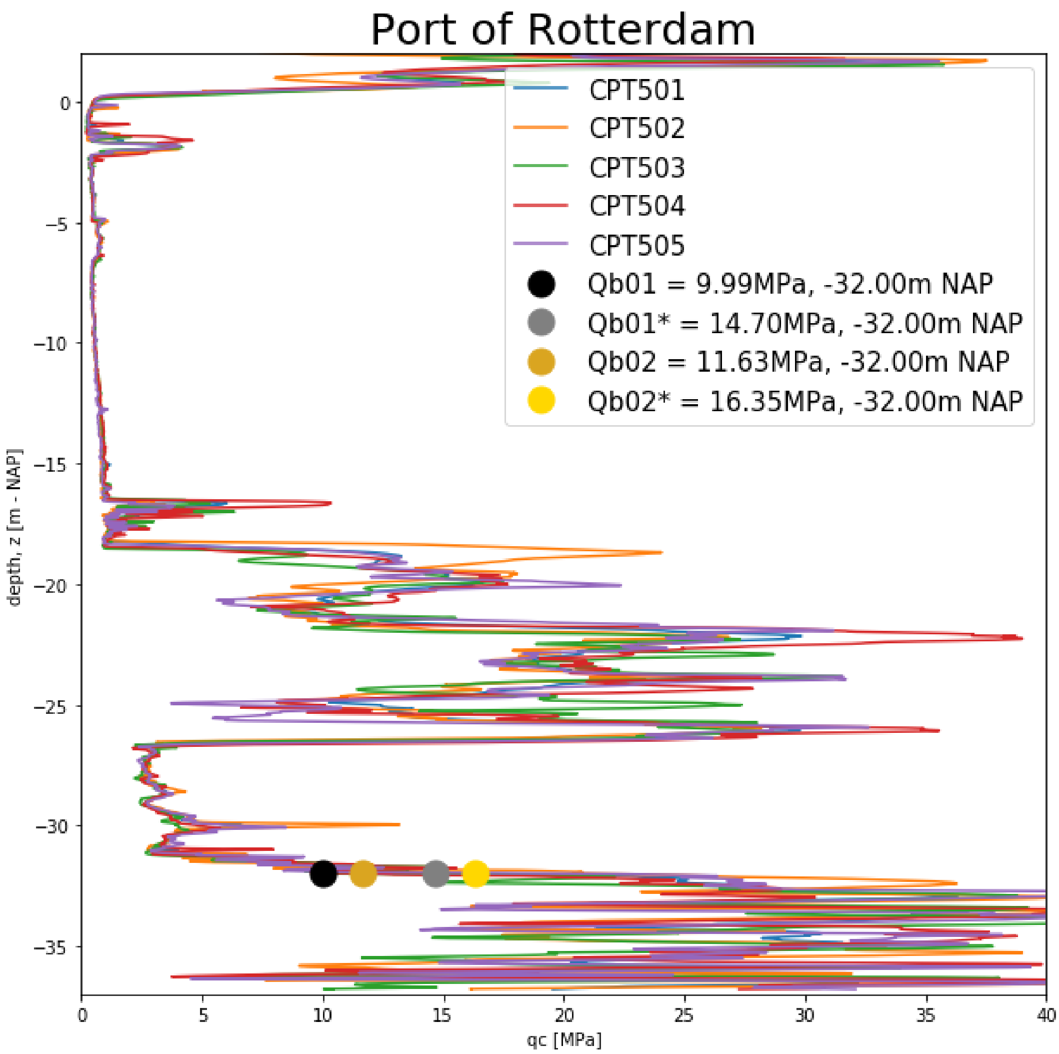


Figure B.18: CPT and measured tip resistance of Port of Rotterdam piles 01 & 02. *Is the measured base resistance measured with residual loads considered

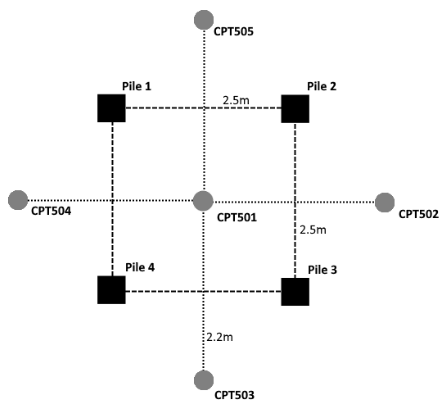


Figure B.19: Diagram of CPT and pile locations for Port of Rotterdam pile load tests

C
Appendix

C.1. CPT calibration tests

In total 16 high quality CPT calibration tests were used for the optimisation of the alternative method through determination of the optimum multiple of the diameter above and below the pile tip to be averaged as well as the damping factor on the cosine function. 10 of those tests were thin layered soil deposits with varying clay layer thickness in sands of different relative density, tested at Deltares. Some of these 10 tests include repeated tests on the same soil test sample with different cell pressures being applied (de Lange et al., 2018). The remaining 6 tests were 2 layered sand deposits with varying relative density. 5 of these sand CPT calibration tests were carried out and published by Tehrani et al. (2018) for an investigation into the transition zones found in the cone resistance measured by a CPT. The remaining test is part of an investigation into the deformation of the Heinenoord tunnel by Grondmechnica Delft, 1995.

Table C.1: Laboratory tests used for the calibration of the variables of the alternative averaging technique in the MSE analysis

Test	Description	Surcharge [kPa]	d_c [mm]	Layer thickness [cm]
Model 02	Loose sand with clay layers	25	25	4
	"	50	25	4
Model 03	Loose sand with clay layers	25	25	2
	"	50	25	2
	"	100	25	2
Model 04	Medium dense sand with clay layers	25	25	4
Model s04	Loose sand with clay layers	50	25	8
Model 07	Loose sand with clay layers	50	36	2
Model 09	Clay with layers of loose sand	50	25	2
Model 09	"	100	25	2
Test 02	Loose over dense sand	50	32	N/A
Test 03	Dense over loose sand	50	32	N/A
Test 04	Dense over loose sand	50	32	N/A
Test 05	Medium dense over dense	50	32	N/A
Test 06	Dense over medium loose sand	50	32	N/A
GM Delft	Loose over medium dense sand	x-g	7	N/A

C.1.1. Running mean

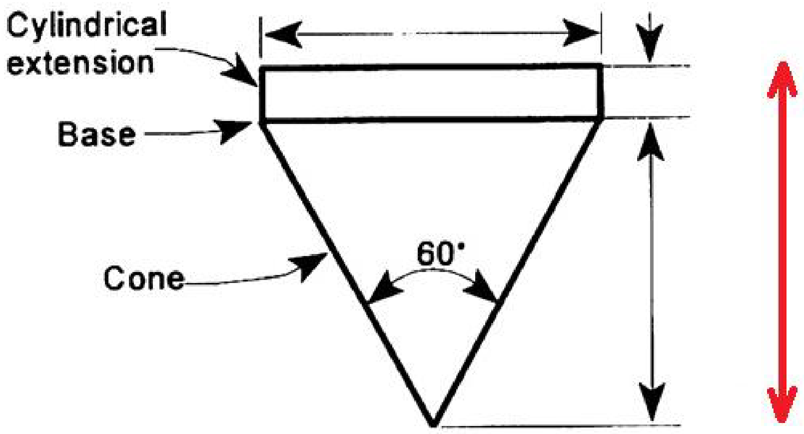


Figure C.1: Height over which the running mean is applied (red) based on the height of the cone tip, de Lange et al. (2018)

C.1.2. Determination of qc,true

Expression of Lunne and Christofferson (1983) for the determination of $q_{c,t}$ in sand:

$$q_{c,t} = 0.061 \cdot \sigma'^{0.71}_{v0} e^{2.91 \cdot D_r} \tag{C.1}$$

Expression used for the determination of $q_{c,t}$ in clay (de Lange et al., 2018):

$$q_{c,t} = (2.0138 \cdot \sigma'_{vo} + 207.26)/1000 \tag{C.2}$$

C.1.3. Additional iterations for the variables

Table C.2: Initial iterations for the determination of the 3 variables in the alternative (cosine) averaging method based on the 10 CPT calibration tests from Deltares

Above	Below	Damping factor	Mean	Standard deviation
11	11	15	0.0944	0.1109
11	12	15	0.0900	0.1047
11	13	15	0.0874	0.1011
11	14	15	0.0863	0.0994
11	14.5	15	0.0861	0.0991
11	15	15	0.0863	0.0992
12	14.5	15	0.9056	0.1040
10	14.5	15	0.0827	0.0962
9	14.5	15	0.0808	0.0961
8	14.5	15	0.0811	0.1002
8.5	14.5	15	0.0806	0.0974
8.5	15	15	0.0806	0.0975
8.5	14	15	0.0809	0.0977
8.5	14.5	14	0.0812	0.0964
8.5	14.5	16	0.0818	0.1007
8.5	14.5	14.5	0.0806	0.0966

The final variable for the alternative sine averaging technique are: Above = 5.5, Below = 13 , f = 8 and s = 0.8.

Table C.3: Iterations for the determination of the 4 variables in the alternative (sine) averaging method based on the MSE of all the 16 CPT calibration tests

Above	Below	s*	Damping factor	Mean	Standard deviation
6.5	11.0	0.9	8.0	0.2253	0.2131
6.5	12.0	0.9	8.0	0.2222	0.2136
6.5	13.0	0.9	8.0	0.2210	0.2144
6.0	13.0	0.9	8.0	0.2185	0.2139
5.5	13.0	0.9	8.0	0.2169	0.2149
5.5	13.0	0.9	8.0	0.2169	0.2149
5.5	13.0	1.0	8.0	0.2308	0.2282
5.5	**13.0**	**0.8**	**8.0**	**0.2130**	**0.2118**
5.5	13.0	0.7	8.0	0.2233	0.2238

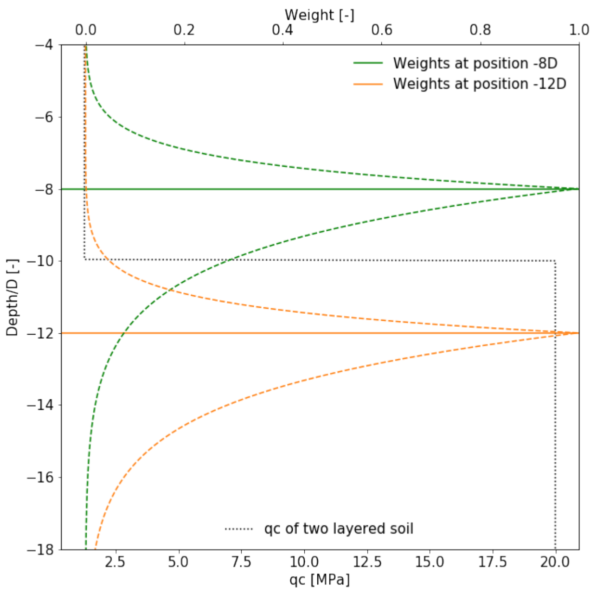
* s, is a parameter which reduces the influence of the lower stiffness soils for values of s below 1.0

Table C.4: Iterations for the determination of the 3 variables in the alternative (linear) averaging method based on the MSE of all the 16 CPT calibration tests

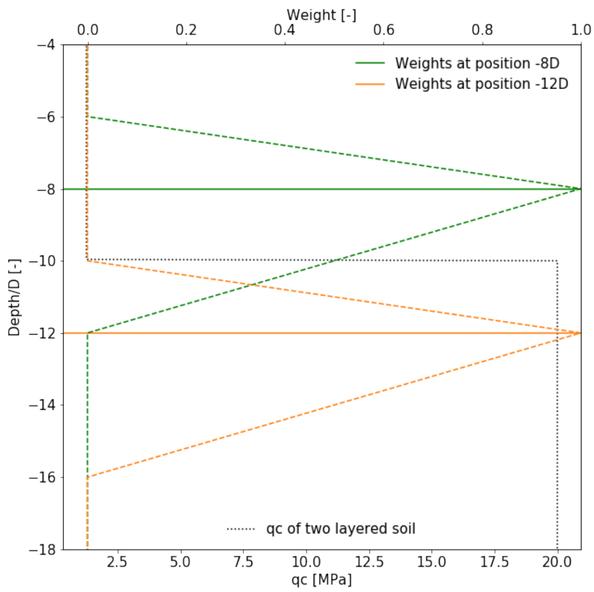
Above	Below	s*	Mean	Standard deviation
2.5	4.5	0.9	0.1988	0.1960
2.0	4.5	0.9	0.1912	0.2037
1.5	4.5	0.9	0.1960	0.2433
2.0	5.0	0.9	0.1958	0.2103
2.0	4.0	0.9	0.1921	0.1966
2.0	4.0	1.0	0.1999	0.2023
2.0	**4.0**	**0.8**	**0.1925**	**0.1990**
2.0	4.0	0.7	0.2045	0.2138

* s, is a parameter which reduces the influence of the lower stiffness soils for values of s below 1.0

The final variable for the alternative linear averaging technique are: Above = 2.0, Below = 4.0 and s = 0.8.



(a)



(b)

Figure C.2: Weights dependent on distance from the pile tip for a two layered soil for the sin function (a) and linear function (b)

C.1.4. The 4 analysed averaging techniques for all laboratory tests used

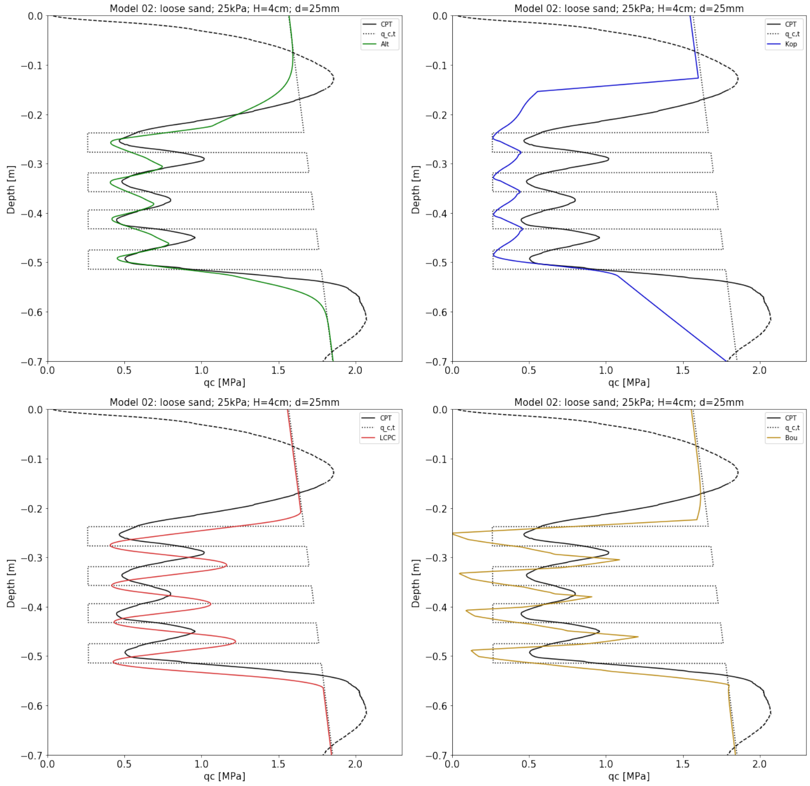


Figure C.3: Fits for the averaging techniques in test model 02 25kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

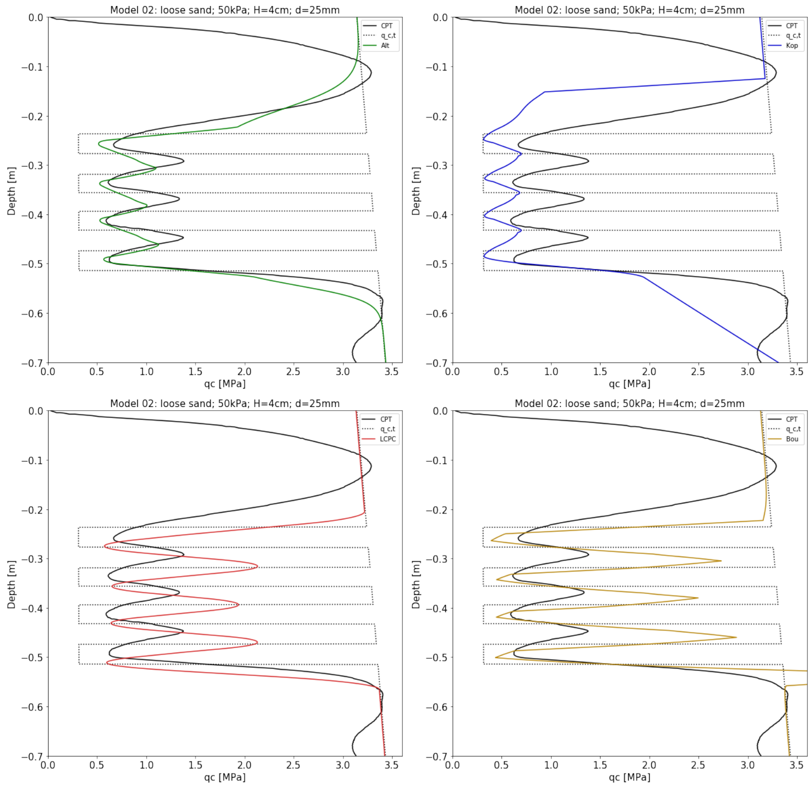


Figure C.4: Fits for the averaging techniques in test model 02 50kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

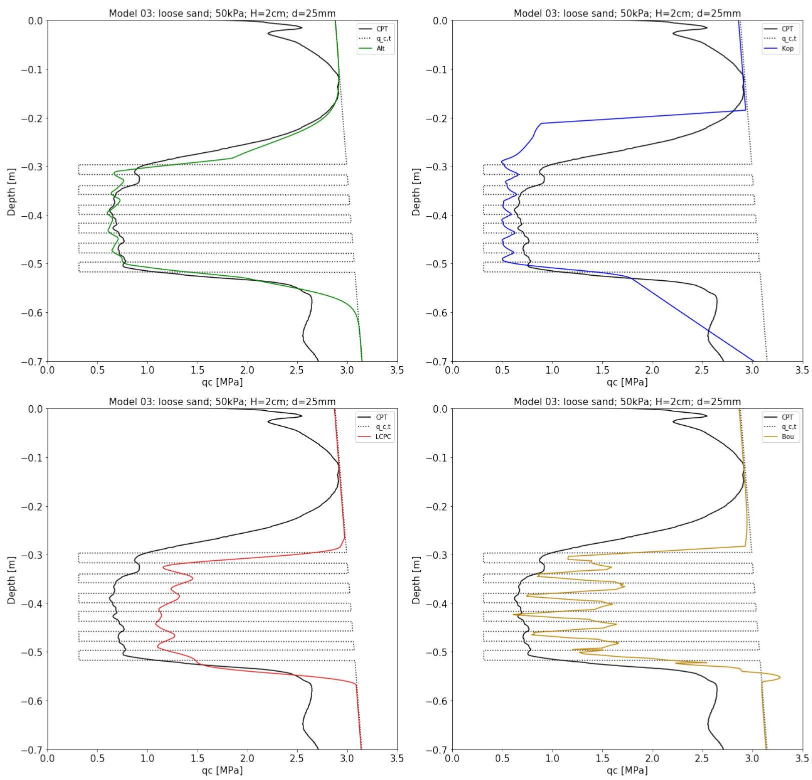


Figure C.5: Fits for the averaging techniques in test model 03 50kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

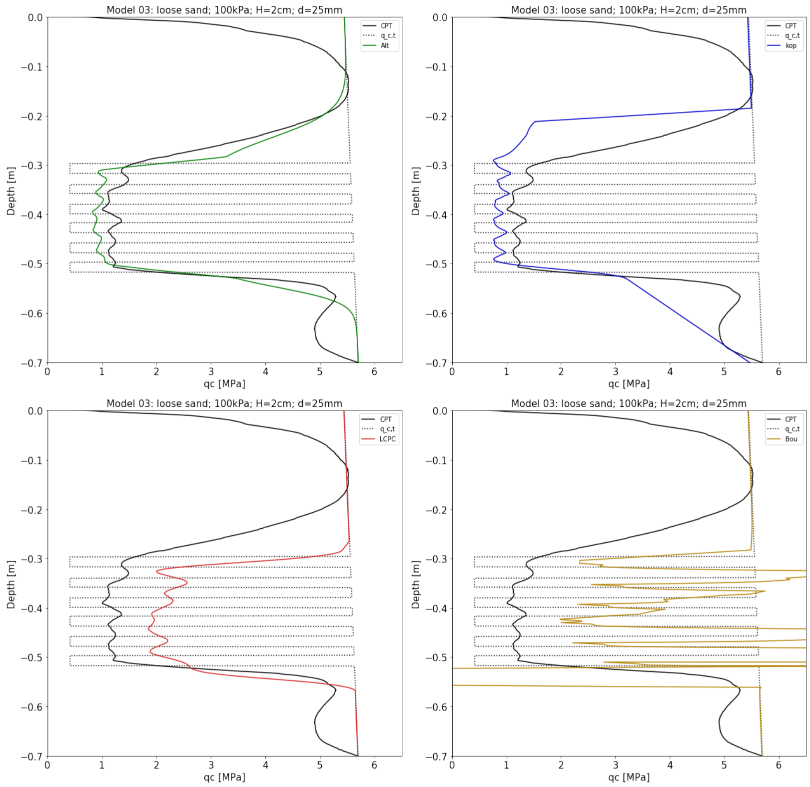


Figure C.6: Fits for the averaging techniques in test model 03 100kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

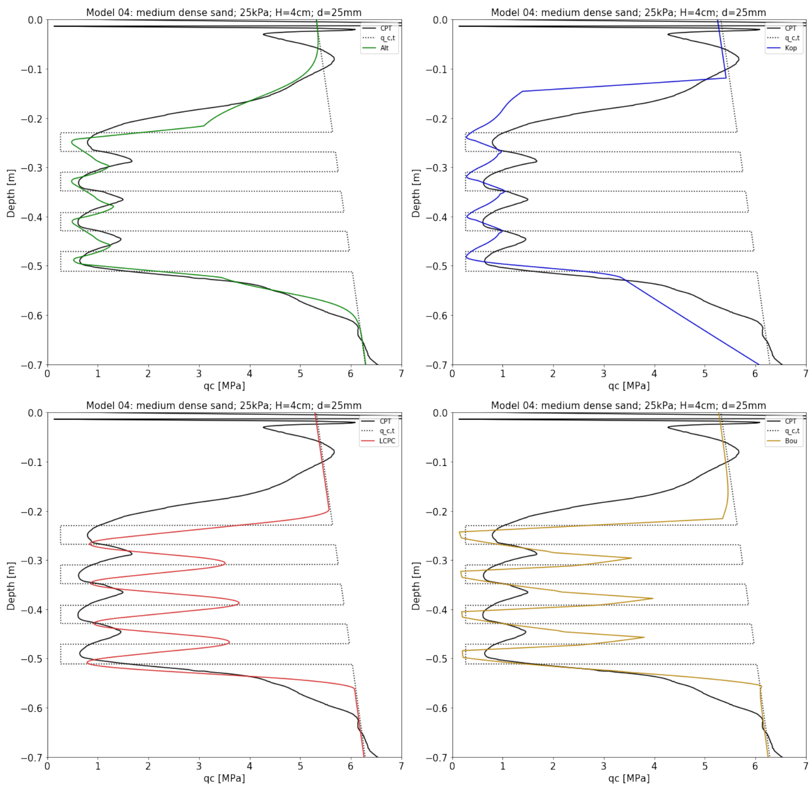


Figure C.7: Fits for the averaging techniques in test model 04 25kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

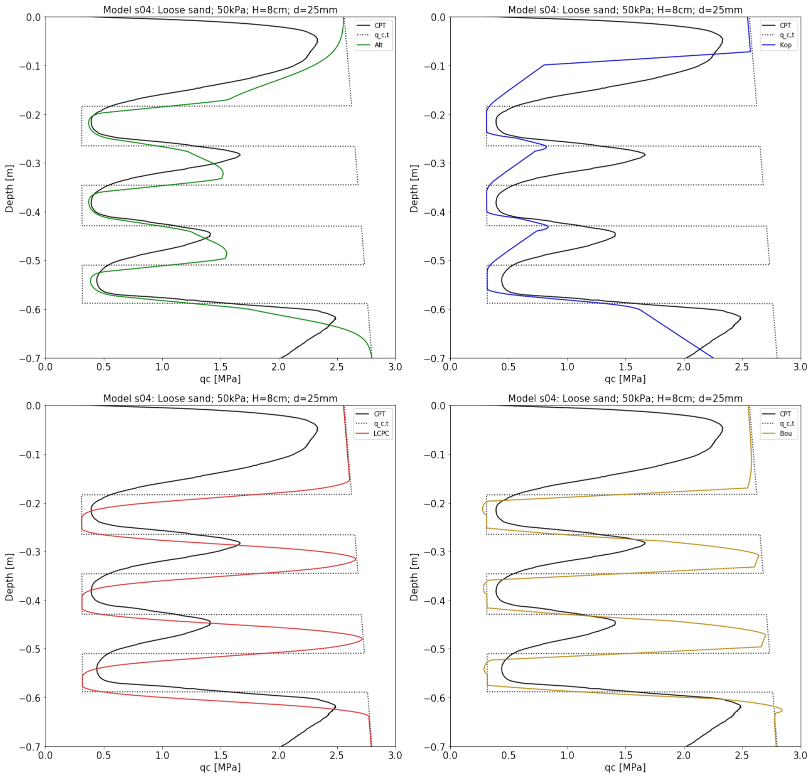


Figure C.8: Fits for the averaging techniques in test model s04 50kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

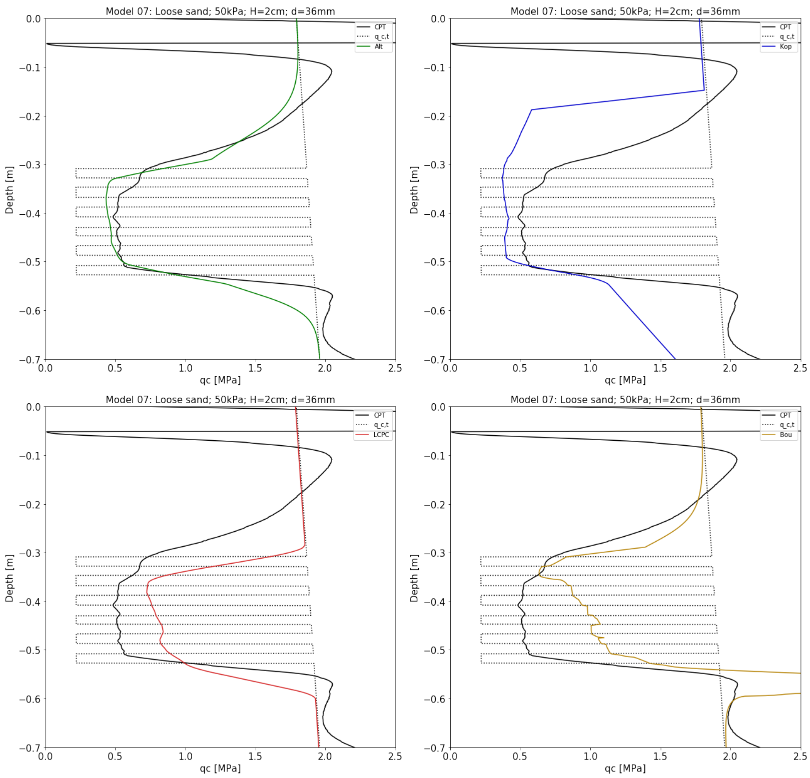


Figure C.9: Fits for the averaging techniques in test model 07 50kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

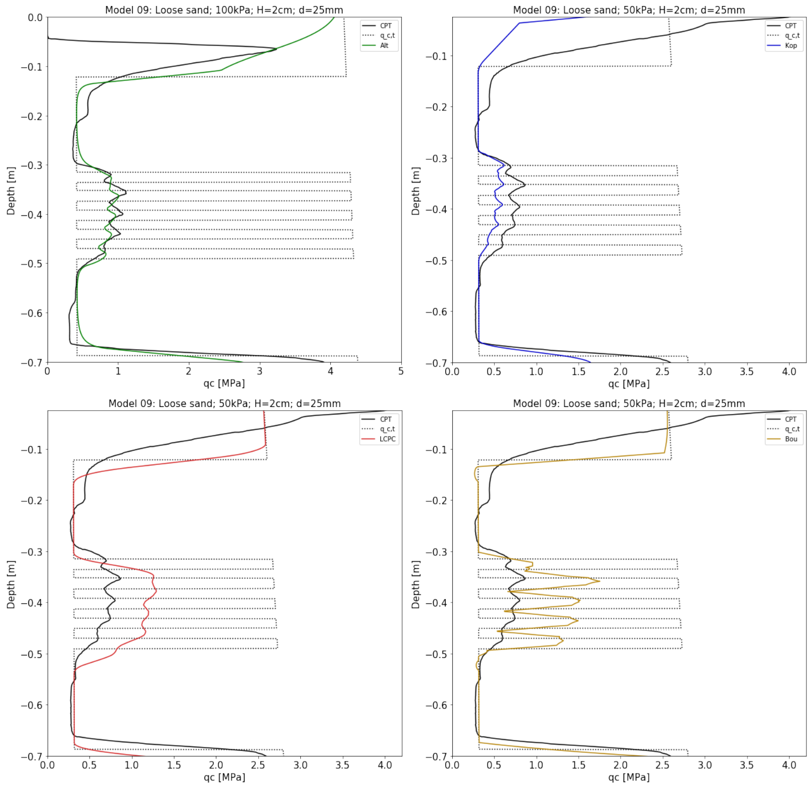


Figure C.10: Fits for the averaging techniques in test model 09 50kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

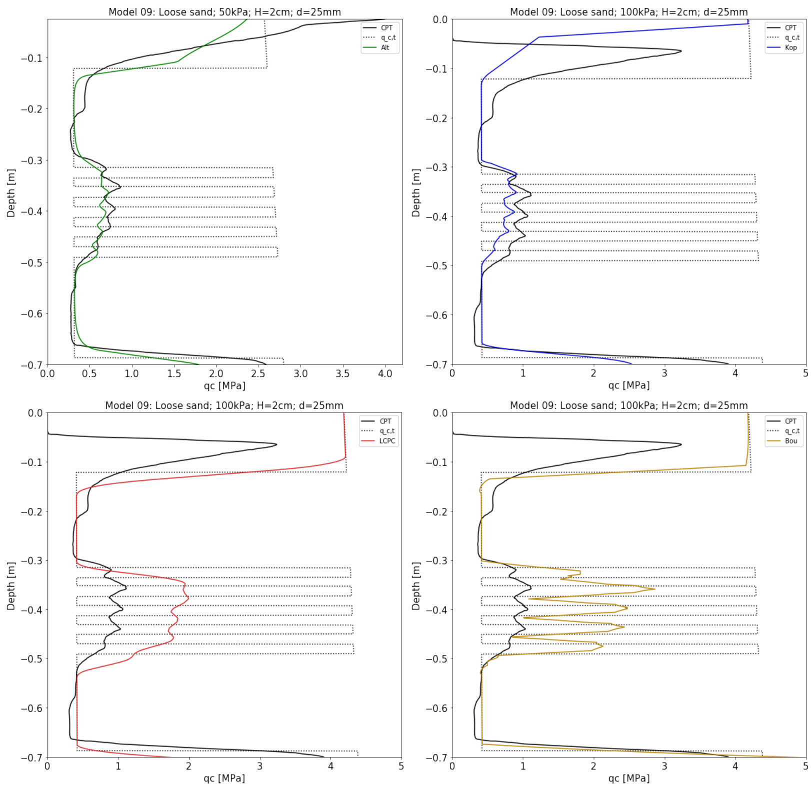


Figure C.11: Fits for the averaging techniques in test model 09 100kPa (sand and clay). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

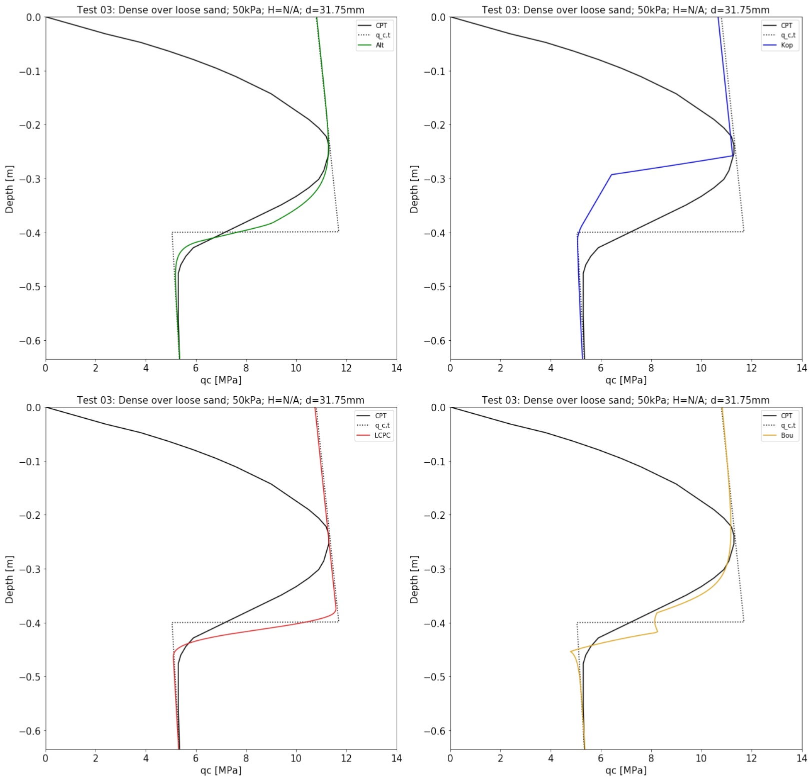


Figure C.12: Fits for the averaging techniques in test 03 50kPa (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

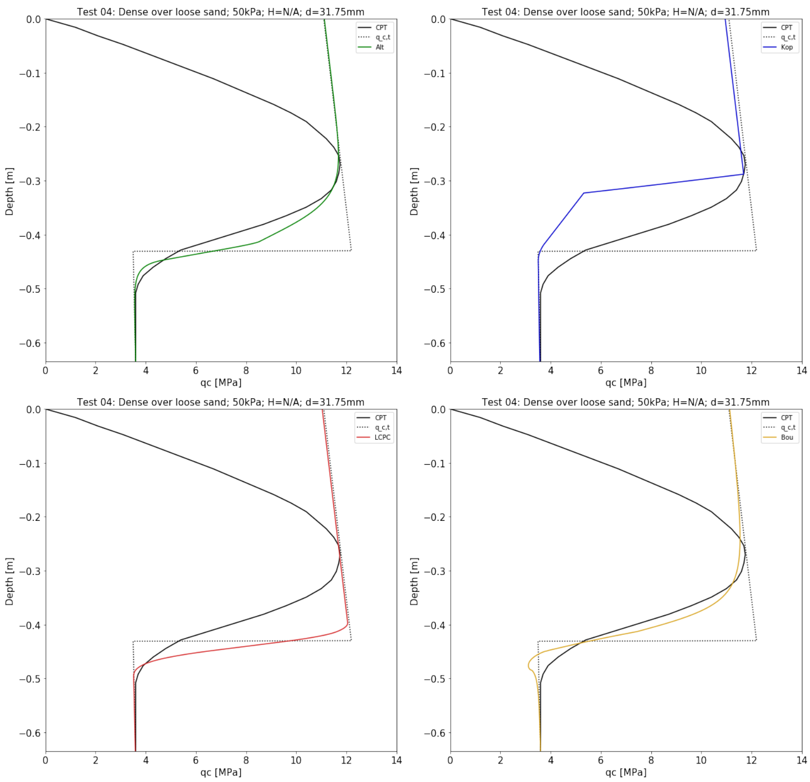


Figure C.13: Fits for the averaging techniques in test 04 50kPa (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

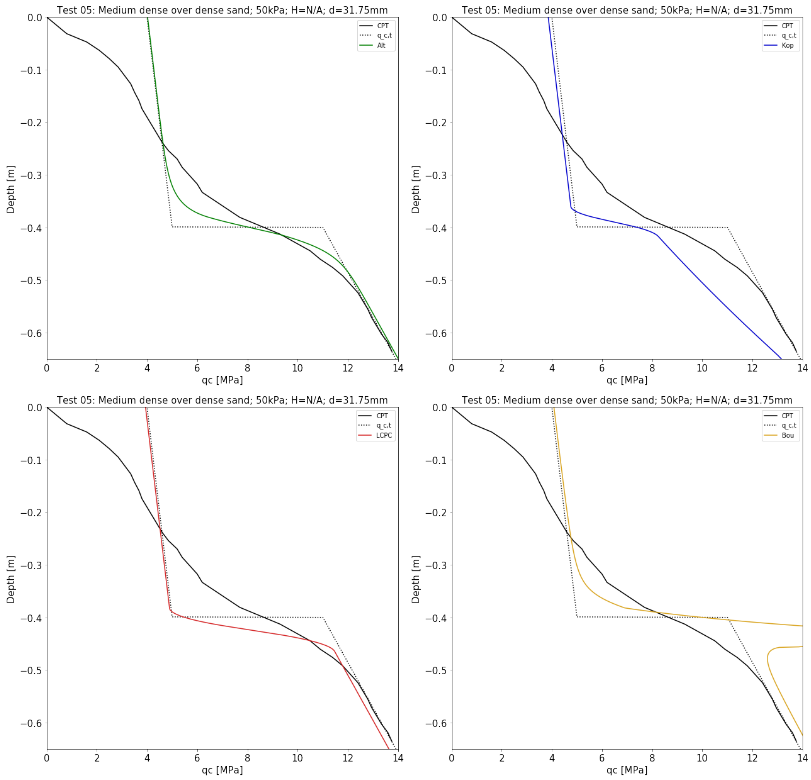


Figure C.14: Fits for the averaging techniques in test 05 50kPa (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

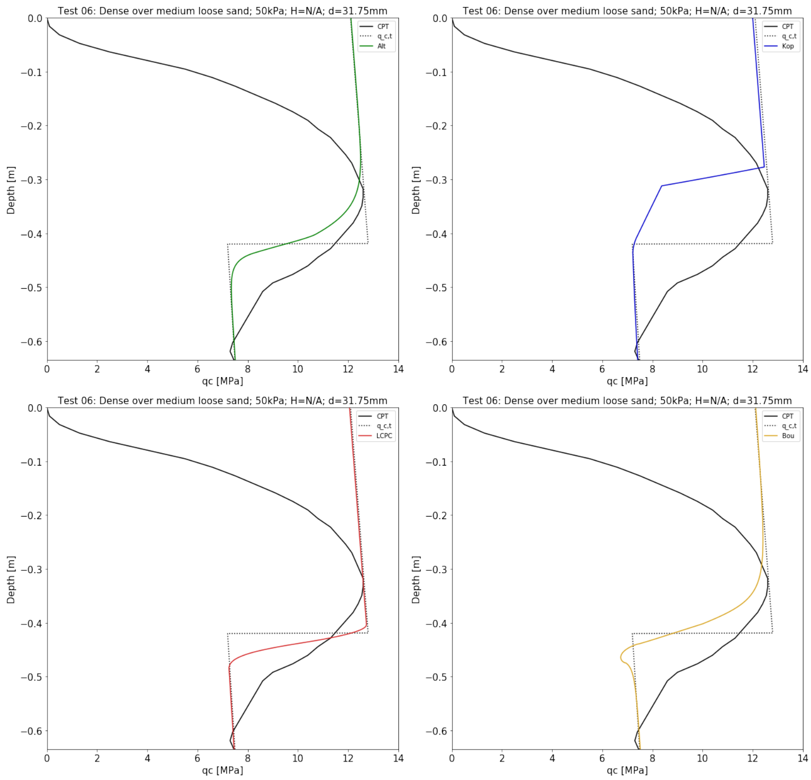


Figure C.15: Fits for the averaging techniques in test 06 50kPa (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

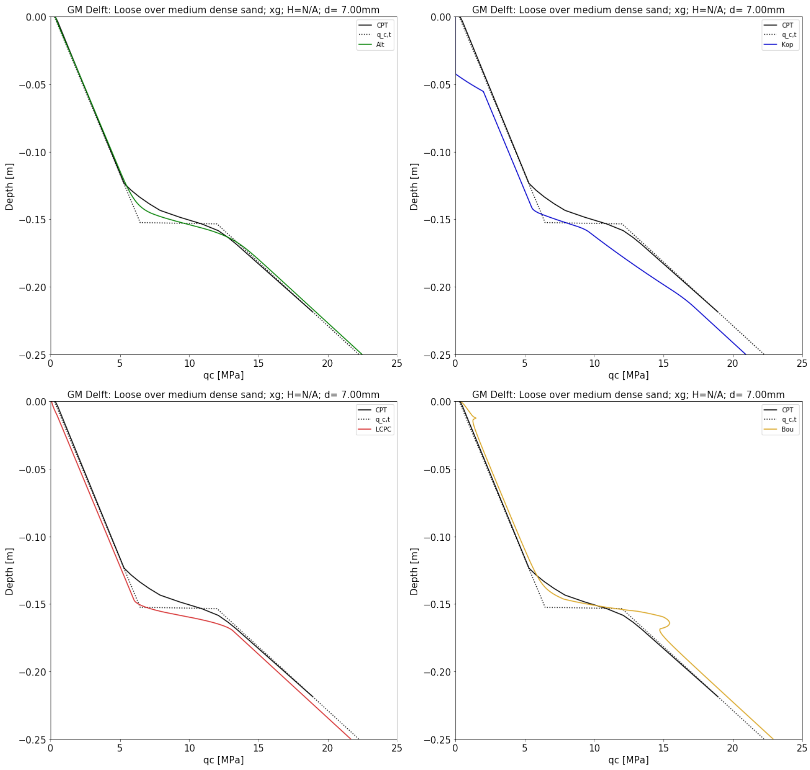


Figure C.16: Fits for the averaging techniques in GM Delft (sand). Top left; Alternative method, top right; Koppejan, bottom left; LCPC, bottom right: Boulanger-de Jong

D

Appendix

D.1. Investigation CPTs Kruithuisweg

First of all 4 different combinations of averaging different CPTs for Kruithuisweg were explored, in order to find representative q_c values for the calculations, which also included 3 mechanical CPTs. The idea of averaging of the different CPTs was later discarded since, it would have a big effect on the minimum path rule in the Koppejan averaging technique due to the peaks being filtered out. Nevertheless, the 4 combinations were: E - 3 electronic CPTs, M - 3 mechanical CPTs, E plus M - 3 electronic and 3 mechanical CPTs and E plus M/0.85 - 3 electronic and 3 mechanical with the mechanical CPTs being divided by 0.85, the value found in Stoevelaar et al. (2009) based the research on mechanical cones by Smits (1982) and Rol (1982). Overall, the mechanical CPT returns the lowest q_c value hence the need for this factor. The lowest q_c values for each pile tip position has be reported in Table D.1. From this comparison the combination of E plus M/0.85 gives a slightly lower value than the E combination however this value is more representative than the even lower values combinations M and E plus M provide.

Table D.1: Different combinations of CPT averaging for Kruithuisweg

Position of pile	q_c - E	q_c - M	q_c - E plus M	q_c - E plus M/0.85
P1 - 12.2m	**3.76**	5.84	4.80	5.32
P2 - 18.0m	**12.55**	**12.55**	**12.55**	13.65
P3 - 20.0m	18.79	**14.33**	16.56	17.83
P4 - 22.0m	16.88	**12.75**	14.81	15.94
P5 - 24.0m	15.21	**11.91**	13.56	14.61

In the end, the capacities were calculated separately of each electronic CPT. The average value of these 3 capacities was then averaged and used in for the Deltares database used in this report (Tables D.2 & D.3). The decision to directly average the q_c of the CPTs was rejected as this would smoothen the CPT profile and hence remove peaks present in the data. The 3 CPTs have the same weight since they have similar distances away from the test pile. The mechanical CPTs were not taken into consideration due to significant lower values near to soil layer interfaces compared to the electronic CPTs. The 3 electronic CPTs can be found in Figure D.2.

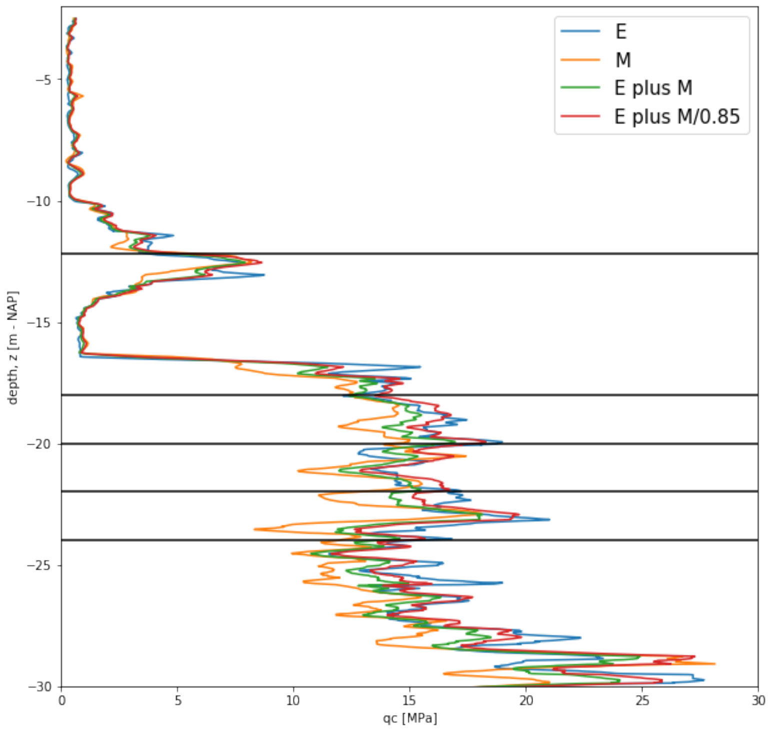


Figure D.1: Visualisation of CPT combinations for Kruithuisweg, Table D.1, and the different positions of the pile tip

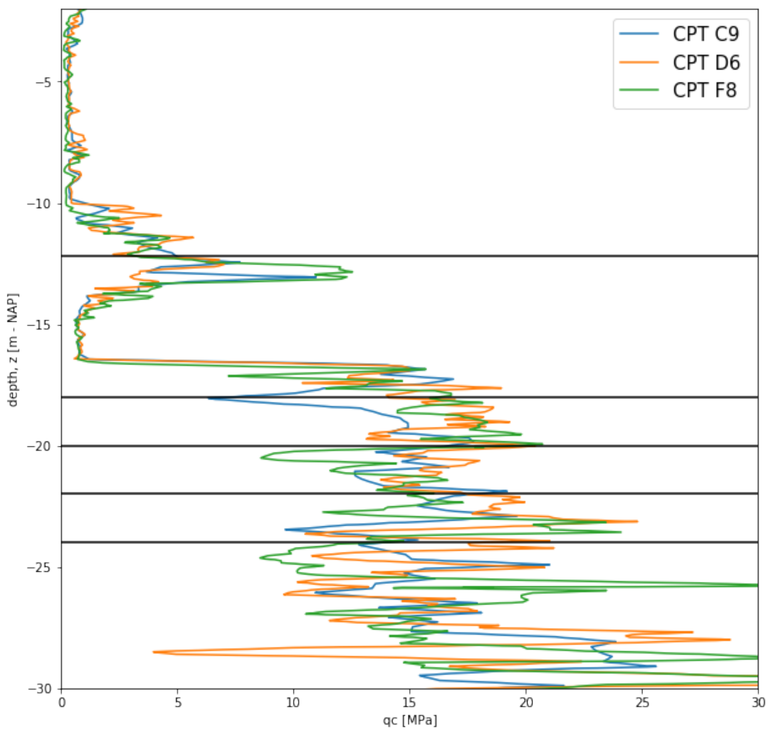


Figure D.2: Visualisation of the 3 electronic CPTs Kruithuisweg and the different positions of the pile tip

Table D.2: The Q_c obtained from the CPT averaging techniques and Q_m used for the determination of α_p for test pile Kruithuisweg

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]		
		Koppejan	LCPC	Alternative
Kruithuisweg I C9	254	315(**0.81**)	420 (**0.60**)	471 (**0.54**)
Kruithuisweg I D6	254	202(**1.26**)	331 (**0.77**)	381 (**0.67**)
Kruithuisweg I F8	254	326(**0.78**)	358(**0.71**)	465 (**0.55**)
Kruithuisweg I	254	281(**0.91**)	370(**0.69**)	439 (**0.58**)
Kruithuisweg II C9	770	559(**1.38**)	1070(**0.72**)	940(**0.82**)
Kruithuisweg II D6	770	1087(**0.71**)	1565(**0.49**)	1537(**0.50**)
Kruithuisweg II F8	770	1023(**0.75**)	1471(**0.52**)	1559(**0.49**)
Kruithuisweg II	770	890(**0.86**)	1369(**0.56**)	1345(**0.57**)
Kruithuisweg III C9	714	1184(**0.60**)	1680(**0.43**)	1586(**0.45**)
Kruithuisweg III D6	714	1393(**0.51**)	1531(**0.47**)	1660 (**0.43**)
Kruithuisweg III F8	714	924(**0.77**)	1824(**0.39**)	1518 (**0.47**)
Kruithuisweg III	714	1167(**0.61**)	1678(**0.43**)	1588(**0.45**)
Kruithuisweg IV C9	712	1178 (**0.60**)	1585(**0.45**)	1635(**0.44**)
Kruithuisweg IV D6	712	1597 (**0.45**)	1522(**0.47**)	1723(**0.41**)
Kruithuisweg IV F8	712	1144 (**0.62**)	1463(**0.49**)	1500(**0.47**)
Kruithuisweg IV	712	1307(**0.55**)	1523(**0.47**)	1619(**0.44**)
Kruithuisweg V C9	644	1160 (**0.55**)	1310 (**0.55**)	1387 (**0.46**)
Kruithuisweg V D6	644	1183 (**0.54**)	1454 (**0.54**)	1580 (**0.41**)
Kruithuisweg V F8	644	893 (**0.72**)	1539 (**0.72**)	1267 (**0.51**)
Kruithuisweg V	644	1079(**0.60**)	1434(**0.45**)	1411(**0.46**)

Table D.3: The measured and calculated shaft capacity of the test pile Kruithuisweg for all the considered shaft calculation methods

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]			
		UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
Kruithuisweg I C9	294	182(**1.61**)	159(**1.85**)	156(**1.89**)	156(**1.89**)
Kruithuisweg I D6	294	197(**1.49**)	173(**1.70**)	240(**1.23**)	240(**1.23**)
Kruithuisweg I F8	294	143(**2.06**)	122(**2.41**)	120(**2.46**)	120(**2.46**)
Kruithuisweg I	294	174(**1.69**)	151(**1.94**)	172(**1.71**)	172(**1.71**)
Kruithuisweg II C9	331	486(**0.68**)	416(**0.80**)	558(**0.59**)	525(**0.63**)
Kruithuisweg II D6	331	510(**0.65**)	434(**0.76**)	546(**0.61**)	510(**0.65**)
Kruithuisweg II F8	331	468(**0.71**)	400(**0.83**)	524(**0.63**)	499(**0.66**)
Kruithuisweg II	331	488(**0.68**)	417(**0.79**)	543(**0.61**)	512(**0.65**)
Kruithuisweg III C9	529	698(**0.76**)	614(**0.86**)	865(**0.61**)	778(**0.68**)
Kruithuisweg III D6	529	764(**0.69**)	677(**0.78**)	914(**0.58**)	779(**0.68**)
Kruithuisweg III F8	529	760 (**0.70**)	662(**0.80**)	911(**0.58**)	768(**0.69**)
Kruithuisweg III	529	741(**0.71**)	651(**0.81**)	897(**0.59**)	775(**0.68**)
Kruithuisweg IV C9	592	870(**0.68**)	802(**0.74**)	1197(**0.49**)	1046(**0.57**)
Kruithuisweg IV D6	592	941(**0.63**)	870(**0.68**)	1272(**0.47**)	1047(**0.56**)
Kruithuisweg IV F8	592	872(**0.68**)	807(**0.73**)	1205(**0.49**)	1022(**0.58**)
Kruithuisweg IV	592	894(**0.66**)	826(**0.72**)	1224(**0.48**)	1038(**0.57**)
Kruithuisweg V C9	462	1017(**0.45**)	973(**0.47**)	1535(**0.30**)	1310(**0.35**)
Kruithuisweg V D6	462	1144(**0.40**)	1086(**0.43**)	1679(**0.28**)	1314(**0.35**)
Kruithuisweg V F8	462	1071(**0.43**)	1009(**0.46**)	1581(**0.29**)	1290(**0.36**)
Kruithuisweg V	462	1077(**0.43**)	1023(**0.45**)	1598(**0.29**)	1305(**0.35**)

Overall, the most variation in Q_c is observed for the Koppejan method across the 3 CPTs. The other averaging techniques tend to have a difference of ± 100kN between the 3 CPTs. The only major deviation recorded is for the pile tip position for Kruithuisweg II, where a values for CPT C9 are approximately 500kN lower compared to CPTs D6 and F8. The was values for this pile position and CPT C9 were still considered for the average taken, in order to take a representative value for the analysis. For the shaft capacity calculations variation across the 3 CPTs is minimal.

D.2. Investigation CPTs Port of Rotterdam

Table D.4: The Q_c obtained from the CPT averaging techniques and Q_m used for the determination of α_p for test pile Port of Rotterdam 02 & 03 (excluding residual loads)

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]		
		Koppejan	LCPC	Alternative
CPT 501	2022	2568(**0.79**)	1934 (**1.05**)	3293 (**0.61**)
CPT 502	2022	3433(**0.59**)	2068 (**0.98**)	3978 (**0.51**)
CPT 505	2022	3000(**0.67**)	1871 (**1.08**)	3693 (**0.55**)
Port of Rotterdam 02	2022	3000 (**0.67**)	1957 (**1.03**)	3655 (**0.55**)
CPT 501	2355	2568(**0.92**)	1934 (**1.22**)	3293 (**0.72**)
CPT 502	2355	3433(**0.69**)	2068 (**1.14**)	3978 (**0.59**)
CPT 503	2355	2040(**1.15**)	1956 (**1.20**)	3162 (**0.74**)
Port of Rotterdam 03	2355	2680 (**0.88**)	1986 (**1.19**)	3478 (**0.68**)

Table D.5: The measured and calculated shaft capacity of the test pile Port of Rotterdam 02 & 03 for all the considered shaft calculation methods (excluding residual loads)

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]			
		UWA-05	ICP-05	NEN-9997-1	NEN-9997-1*
CPT 501	4402	3115(**1.41**)	3259 (**1.35**)	4837 (**0.91**)	3626 (**1.21**)
CPT 502	4402	2995(**1.47**)	3134 (**1.40**)	4475 (**0.98**)	3546 (**1.24**)
CPT 505	4402	3038(**1.45**)	3188 (**1.38**)	4625 (**0.95**)	3497 (**1.26**)
Port of Rotterdam 02	4402	3049 (**1.44**)	3194 (**1.38**)	4645 (**0.95**)	3556 (**1.24**)
CPT 501	4136	3115(**1.33**)	3259 (**1.27**)	4837 (**0.86**)	3626 (**1.14**)
CPT 502	4136	2995(**1.38**)	3134 (**1.32**)	4475 (**0.92**)	3546 (**1.17**)
CPT 503	4136	3016(**1.37**)	3166 (**1.31**)	4682 (**0.88**)	3552 (**1.16**)
Port of Rotterdam 03	4136	3042 (**1.36**)	3186 (**1.30**)	4664 (**0.89**)	3574 (**1.16**)

D.3. Robertson soil classification

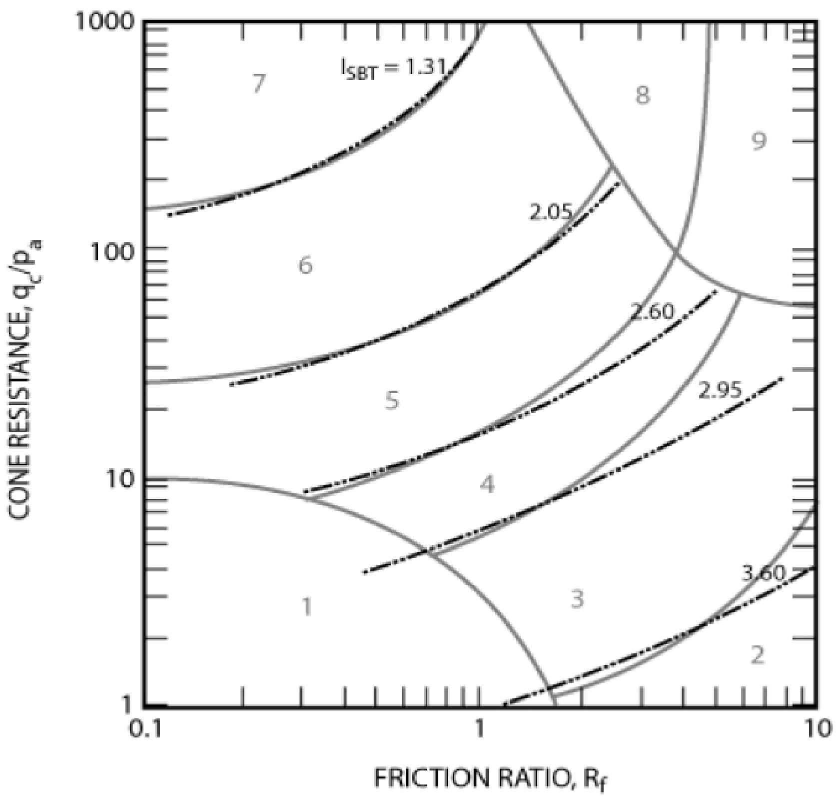
The following equation has been used for the soil classification (Robertson, 2010):

$$I_{SBT} = \left[\left(3.47 - log\left(\frac{q_c}{p_a}\right)\right)^2 + (logR_f + 1.22)^2\right]^{0.5} \tag{D.1}$$

Where:

R_f: friction ratio = $\frac{f_s}{q_c} \cdot 100\%$

The output of this equation is the I_{SBT} value which can be used in combination with the correlation demonstrated in Figure D.3 to identify the soil type.



Zone	Soil Behaviour Type (SBT)
1	*Sensitive fine-grained*
2	*Clay - organic soil*
3	*Clays: clay to silty clay*
4	*Silt mixtures: clayey silt & silty clay*
5	*Sand mixtures: silty sand to sandy silt*
6	*Sands: clean sands to silty sands*
7	*Dense sand to gravelly sand*
8	*Stiff sand to clayey sand**
9	*Stiff fine-grained**

** Overconsolidated or cemented*

Figure D.3: SBT chart based on dimensionless cone resistance and the friction ratio, Robertson (2010)

D.4. Additional figures and tables

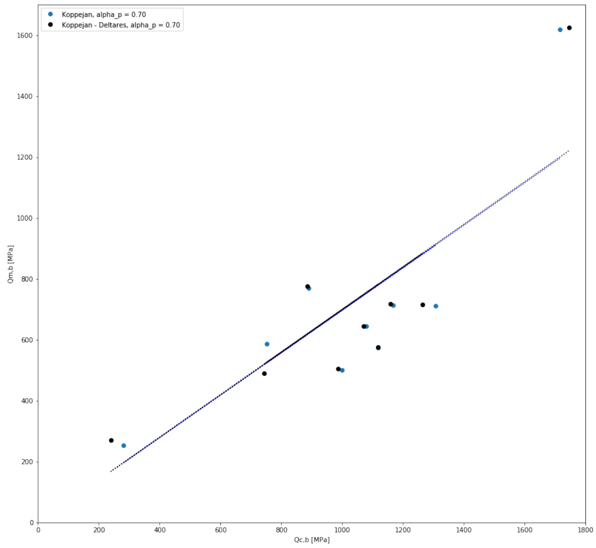


Figure D.4: Deltares comparison graph for the Q_m/Q_c of the Koppejan method

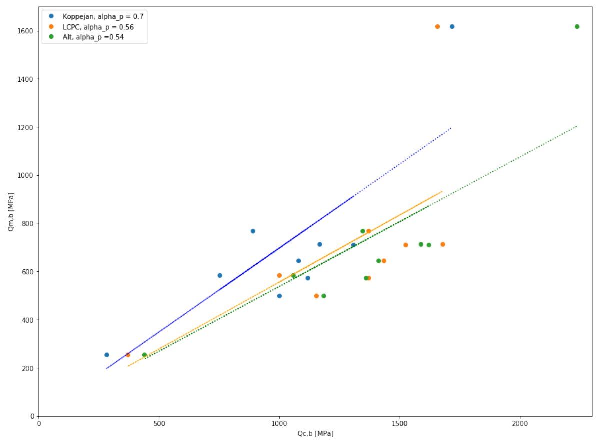


Figure D.5: Comparison graph for the Q_m/Q_c of the all the three averaging techniques

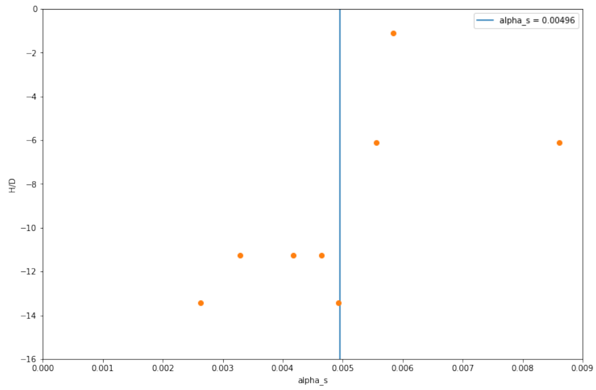


Figure D.6: Graph H/D v α_s for the strain gauges in sand for the Deltares database

Table D.6: The Q_m and Q_c of the shaft resistance in sand layers obtained by the considered calculation methods

Pile load test	Measured, Qm [kN]	Calculated, Qc [kN]		
		UWA-05	ICP-05	NEN-9997-1
TNO pile 01	327	426	396	595
TNO pile 02	350	467	435	666
Kruithuisweg III	236	448	383	534
Kruithuisweg IV	258	507	454	639
Kruithuisweg V	219	515	474	665
CIAD	64	113	113	96

Table D.7: Qm/Qc = α_p of all the four high quality pile load tests excluding residual loads for the base calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	Alt*
Pigeon River	2.10	0.94	1.80	1.01
Port of Rotterdam 02	1.12	2.43	0.96	1.11
Port of Rotterdam 03	1.46	2.79	1.26	1.35
Marshall County	0.91	1.06	0.78	0.85
Mean	**1.40**	**1.80**	**1.20**	**1.08**
Variance	0.203	0.668	0.149	0.033
Standard deviation	0.451	0.817	0.386	0.183
CoV	**0.322**	**0.453**	**0.322**	**0.169**

Alt* is the value obtained using the alternative averaging technique times a constant $\alpha_p = 0.5$

Table D.8: Qm/Qc = α_p of all the four high quality pile load tests including residual loads for the base calculation methods

Pile load test	Qm/Qc [-]			
	UWA-05	ICP-05	NEN-9997-1	Alt*
Pigeon River	2.64	1.18	2.27	0.91
Port of Rotterdam 02	1.65	3.58	1.42	1.16
Port of Rotterdam 03	2.06	3.92	1.76	1.36
Marshall County	1.20	1.40	1.03	0.80
Mean	**1.89**	**2.52**	**1.62**	**1.06**
Variance	0.283	1.534	0.149	0.033
Standard deviation	0.451	1.238	0.386	0.183
CoV	**0.282**	**0.491**	**0.282**	**0.205**

Alt* is the value obtained using the alternative averaging technique times a constant $\alpha_p = 0.7$