CHALMERS
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Study on improved neutral plane analysis of deep foundation for lightweight buildings on soft soil

Raft foundation on timber piles for 3-5 levels building

Master’s thesis in Master Program Structural Engineering & Building Technology

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Department of Architecture and Civil Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
Master’s Thesis ACEX30-22
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Cover:
Figure shows one of construction sites with a pile machine and timber piles on the front, Kungsbacka Lindenspark

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ABSTRACT

Current building standards and guidelines allow a timber construction to be built with more than two floors, an opening of the market to choose timber as material in load-bearing structures. Because of the weight of the building, this will lead to modification of ground foundation in consideration of low costs and less construction time. Simultaneously, this contributes to a more sustainable and climate-smart society.
The objective of this thesis is to investigate ground foundations for lightweight 3-5 floors residential constructions on soft soil through variety of length of piles, placement and the number of piles and pile material, together with a new approach of calculations on pile settlements for the timber piles. The study is based on an examination of existing building of timber loadbearing construction and soil model of construction side in the middle of Gothenburg, project "The Working Lab".
One of the challenges was to investigate the interaction between piled raft and subsoil, yielding point of disturbed soil under installation of piles and down drag loads. This Master thesis evaluate the settlements and bearing capacity of the piled raft on timber piles based on the calculated assets of the presumed models. For that, the algorithm of piled raft behaviour considering optimization of the design has been created and followed.
The analysis shows a positive result which fulfill Eurocode requirements. It can be used in general for timber construction and constructions with low applied loads. Further studies would be required for a development of the practical model and guidelines together with verification in practice.

Key words: timber piles, neutral plane analysis, soft clay, lightweight timber construction

Undersökning av neutral plan analys av pålkonstruktioner för lätta byggnader på mjuk lera

Examensarbete inom masterprogrammet Structural Engineering and Building Technology

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SAMMANFATTNING

Dagens byggnormer tillåter att bygga trähus mer än två våningar. I sin tur säkrar detta marknaden för användning av trä i stommar och bärandekonstruktioner. Med hänsyn till lätta konstruktioner leder det till en optimering av grundkonstruktioner och lägre kostnader och byggtider. Samtidigt bidrar detta till ett mer hållbart och klimatsmart samhälle.
Syftet av detta examensarbete är att utreda den möjliga optimeringen av bärande grundkonstruktioner för byggnationer med 3–5 våningar byggd på mjuk lera, med hjälp av variation av pållängd, antal och placering av pålar och variation av pålmaterial, genom att utreda en befintlig byggnation av trästomme och jordprofilering gjord på plats i centrala Göteborg inom projektet ”A Working lab”. Den utmanande delen var att granska interaktionen mellan pålade plattan och underjord samtidigt med en parallell granskning utfördes på omrör jord under pålning och påhängd last som uppstår efter en viss tid. Det examinationsarbetet gjordes i syfte att undersöka sättningar och bärande kapacitet av pålad grundplatta med träpålar. Med hjälp av framtagna modeller med förbestämd placering av pålar och längder utvärderas pålplanerna. För detta inrättades en beräknings algoritm och användes för framtagning av resultatet.
Resultaten visade sig vara positiva och uppfylla Eurokodens alla krav samtidigt kan den användas i situation då byggnaden har tillräckliga låga laster.
Dock för framtida användning behövs metoden utvärderas djupare och verifieras från praktiken.

Nyckelord: träpålning, neutral plan analys, mjuk lera, lätta träkonstruktioner

Contents

Preface

In this Master thesis investigation of settlements and bearing capacity of the ground construction for the lightweight building was made. By investigation from the raft and soil interaction to additional timber piles and modification on the design according to adjustments on length, number, and design algorithm has been made.

The project been carried out under collaboration with Professor in Experimental Geomechanics Jelke Dijkstra and Jonatan Isaksson as supervisor. Thank you to believe in me. A big thanks for our input, guidelines, and knowledge you shear with me.

I would also like to thank Johan Hjortbrink, KELLER for his input on timber piles and Toni Lundstedt, Bergman-Hook for his time and co-operation. Also, Aarsleff team, see front page, for a long and meaningful discussion on the installation prosses.

Also, major gratitude I would like to send to my family and my friends for understanding and support through entire project. I could not do it without you.

Onsala May 2022

Tatjana Sundberg

1 Introduction

1.1 Background

Since 1994 regulations in Sweden have outlined fire resistance requirements (BBR, 2015) for the building of timber constructions and allowed to preside timber buildings that are more than two stories in height. This creates an opening in the market to choose timber as a material in load-bearing structures. Softwood timber as a structural material has lower density (characteristic density ρ=300-400kg/m^3) than concrete (approximate density ρ=2400kg/m^3) or steel (ρ=7850kg/m^3). It is benefit timber as material and in its high strength-to-weight ratio. When gives the possibility to build more light-weight structural solutions, which lead to a reduction in foundation load, and reduces costs and construction time.

Timber as a natural and renewable material, requires less energy to produce. Other building and construction materials, such as steel and concrete require energy to prepare. As an excellent ecological material, timber stays within sustainability measures well above CO_2 emissions according to analyses and calculations. Roger Sathre writes in his doctor thesis "...a net reduction of CO_2 emission can be obtained by increasing the proportion of wood-based materials used in building construction, relative to concrete materials." (2007). This means that timber has a more environmentally friendly footprint as a building material and contributes to a more sustainable and environmentally friendly construction design.

Timber piles as a ground stability for residential buildings is not just an attractive alternative due to the ecological aspect but also regarding the economics. According to Borrello, the timber piles have an indefinite service life under ideal conditions (2009). What are the ideal conditions for timber? Timber is sensitive to wetting and drying cyclers, but by placing timber piles under groundwater level it can prevent this problem.

In current traditional guidelines such as international standards as American Society for Testing and Materials (ASTM), timber piles require calculations for concentric compression loading parallel to the grain (D2899). In this case, the bearing capacity of floating piles in soft clay depend on the length and shaft area of the pile and the toe resistance can be neglected recording the $A_{shaft} \gg A_{toe}$. Therefore, the number of piles influence the load-carrying capacity. Nevertheless, Chanda and his team claim that the load-carrying capacity of pile in the group is larger than the sum of single piles due to load-sharing (2020). Thus, it would be reasonable to analyse timber piles strategically placed as a group or as a single pile with centre-to-centre space variation for improvement of a load-bearing capacity and reduction of settlements.

Soft clay is abundant in Western Sweden and Gothenburg is not an exception. A floating piled raft foundation using long and slender concrete piles is most conservative solution in a deep deposit of soft clay in Gothenburg.

The conservative approach on the design of piled raft foundations is that the piles carry most of the load by friction or adhesion without consideration of the raft bearing capacity. Furthermore, the pile length is mainly governed by settlement criteria. In more stiff soils such as stiff clays and sand the raft greatly contributes to the bearing capacity, (e.g.) Elwakil (2015) demonstrates that the percentage of load carried by the raft in the piled raft system is around 39%. Therefore, it is essential to investigate and optimize the structure design for ground foundations of lightweight constructions on soft clay.

1.2 Problem formulation

Many modifications have been done both due to geotechnical and structural part of design of ground foundations. To precede the design which fulfill all requirements many factors are important to take into account such as slenderness ratio (L/D), rigidity of the raft (B/L), load transferring mechanisms, geotechnical profile under the pile, soil-pile interaction, pile-raft interaction, non-linear behaviour of the pile, especially if it is concrete, ground movements, liquefactive effects, connections between pile and raft, etc. One of the leading persons in geotechnical world, Harry Poulos, has said: "The key element in foundation design is recognition of the important issues" (Poulos, 2020).

In 1994 the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) start an investigation and gathering methods on designing of foundations. Previous studies offer several methods, but it is up to knowledge and responsibility of the engineer to achieve the satisfied requirements of the design. The design of building foundations varies quite widely but three stages that can be rationalised:

- the preliminary design over the bearing capacity and settlements of the construction;
- the characterisation of the construction;
- the detailing stage.

In most cases, it is over-dimensioned both in numbers of construction details and volume of material, according to Poulus (2016) and his investigation on tall buildings. Therefore, it is important to continue investigation on the ground foundation and construction detailing. Feasibly it is lead to the ultimate design with a lower cost and less impact on the environment.

1.3 Aim and objectives

This thesis aims to study more accurately the response of piled raft foundations for lightweight structures using timber piles on soft clay combining structural and geotechnical engineering. The complex interaction mechanisms between raft, piles and soil provide a number of issues to be studied therefore this thesis goals are:

- To investigate construction site and estimate soil properties on an example of existing building;
- To identify mechanisms and associated model parameters that influence the bearing capacity and the long-term settlement response of the raft;
- To identify the structural response of timber piles and response for axial and bending stresses in the soil;
- To understand the load transport mechanism for piled raft on soft soil under static conditions;
- To understand the failure mechanism for piled raft with a timber pile;
- To modify designs regarding the ultimate load bearing capacity and maximum settlements of the piled raft.

The expectations are to obtain a deeper understanding in the field of deep foundations by applying new methods and identified from literature. The main aim of the thesis, if it is feasible to replace the conservative concrete pile with less popular timber pile?

1.4 Limitation

The design process going through several parts of investigation. The dialog between structural designers and geotechnical engineers delivers an interactive process to optimize the design of the structure. In this way it takes same time to find the perfect solution which fulfill all the requirements. Because of time limitation and challenging task just parts of the design will be studied.

However, it is important to remember the neglections that have been made in the design procedure and to re-evaluate them in a future resource. In this thesis several neglections has been made for soil profile based on existed construction site in centrum of Gothenburg. Another limitation is that building model is taken from manufacture and assumed just as one element.

The study has been based on assumption that the reader has a basic knowledge of structural and geotechnical principles of mechanics.

1.5 Methodology

Since this Master thesis aims to investigate and optimize ground foundation for lightweight constructions it must present and discuss a several models for evaluation.
The study develops in three main stages: firstly, a literature study over current literature and earlier research on raft and pile design is presented. This background is quite significant to understand how the geometry and there interaction between different parts of the design, influence the stresses of construction, as well as soil and radiuses settlements, see Figure 1.1

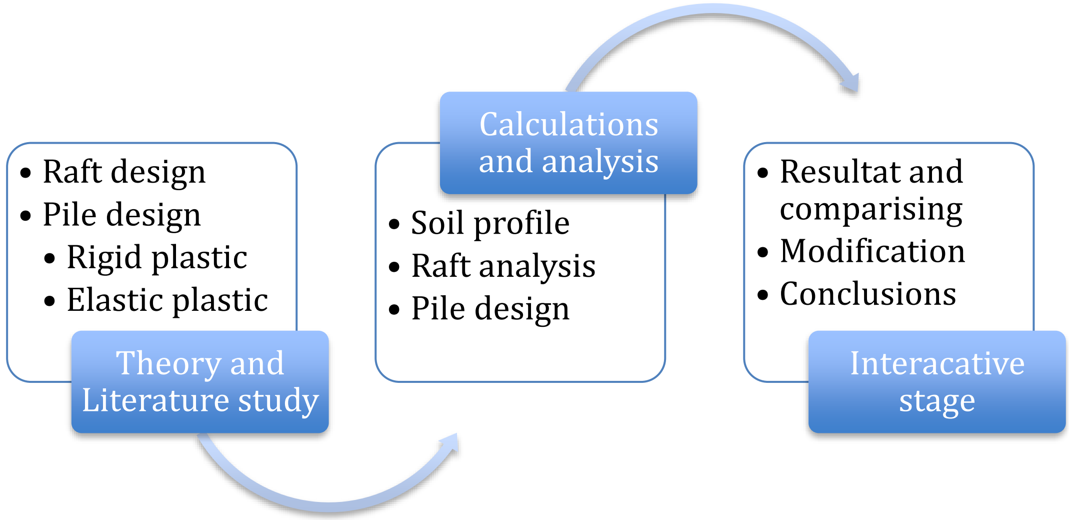


Figure 1.1 Methodology process in this study on ground foundations

Secondly, the study establishes a more complete mathematical model, which will provide a deeper understanding of the theory on piled raft foundation and most focus on failure mechanisms of piled raft foundation and settlements evaluation from the parameters and design perspectives.

Thirdly, the theoretical new approach is tested on an example of an existing lightweight construction on soft clay. The modified Neutral plane analysis has been used for reduction of down drag loads and comparison with the result on the previous calculations.

1.6 Outline of thesis

In the last several years, pile material choice has been on conservative materials such as concrete and steel, but more and more timber piles have been used in the ground constructions. In 2020, 249% more timber piles were used then in 2019 in Sweden (Pile Commission, 2019). Increased numbers can depend on prices and problem with transportations for more conservative choice of piles.

In this thesis, an evaluation of replacement of conservative choice material for deep ground constructions by timber piles, and an investigation of feasibility of this kind of design is described. Outline of evaluations of this thesis is:

- Shallow foundation on soft clay, Chapter 3;
- Raft foundations for lightweight constructions on soft clay, Chapter 3;
- Raft foundations on floating piles such as concrete piles, Chapter 4;
- Timber piled foundations on soft clay for lightweight constructions, Chapter 4;
- Design optimization, Chapter 5;
- Discussion, Chapter 6.

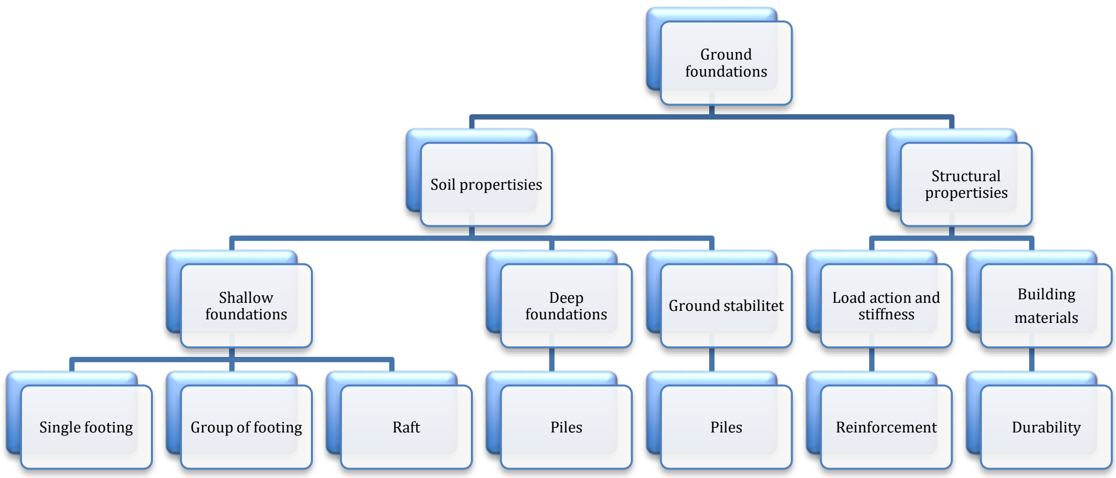


Figure 1.2 Process of establishing of ground foundation

2 Soil response

For safe building design a complete understanding of movements in the ground must be taking into account. How is stress behaviour distributed under the applied load of a building? To answer this question, parameters of soil, geometry and weight of a building should be known.

Relying on the stress distribution in homogenous soil, according Mohr/Coulomb's to strength theory, the soil behaviour can be simplified by an approximated analysis. The analysis is described by vertical load application from a construction with specific geometry, affecting an effective stress of soil linear with a depth. The theory is used widely and most commonly called for the theory of elastic displacement. To obtain theory of elasticity, the assumption has been made that soil behaviour is elastic.

Along with this consideration, the 2:1 method has been established for further calculation simplifications. This means that average stresses under a uniform distributed load depends on the geometry, load magnitude and variety of the depth, see Figure 2.1.

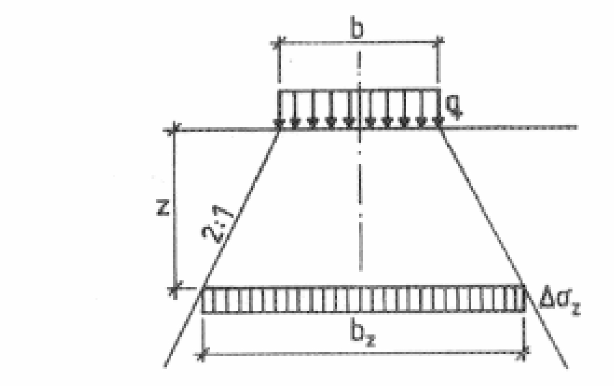


Figure 2.1 Geometry based on assumptions of 2:1 method adapted from Sällfors (2008)

According to Sällfors (2008) the average stress variation can be calculated according to Equation 2.1

$$\Delta\sigma = \frac{blq}{(b+z)(l+z)} \qquad (2.1)$$

On the contrary, the relationship between stress (load) and movements (settlements) in isotropic and homogeneous soil with a linear stress-strain dependents can be regarded by Hook´s law represented by Figure 2.2. This elastic perfectly plastic model is an alternative to elasticity modulus, *E* in soil mechanics used compression modulus, *M*. (Sällfors, 2008). It shows that the maximum load in Ultimate Limit State, ULS is not dependent on the deformation and occur under the yielding point in material behaviour. This means that if the task is to find the soil failure, the elastic part can be ignored (Craig and Knappett, 2012).

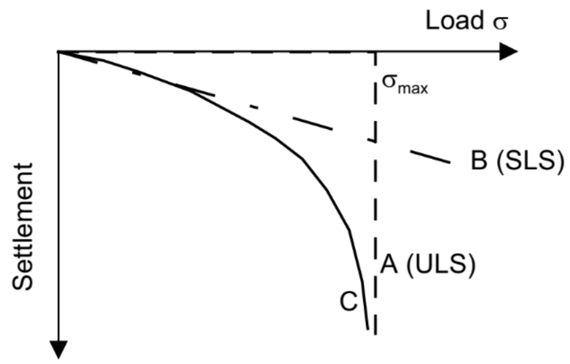


Figure 2.2 Relationship between settlements and maximum load in Service Limit State, SLS (elastic) and Ultimate Limit State, ULS (ideal- plastic) for the raft on soft soil, (Craig and Knappett, 2012)

Consequently, two limitations independent to each other where geotechnical design of construction must fulfill. One is the limitation of bearing capacity (ULS), and the other is settlements (SLS) under serviceability life of the building.
In this study, therefore described elastic rigid plastic behaviour of soil with linear elasticity and homogeneous shear strain distribution for simplicity of analytical evaluations is assumed.

2.1 Failure mechanism

Since the magnitude and the type of movement in soil is an issue of stability, the bearing capacity of the building should be verified with restrictions in guidelines such as European Standards or Eurocode for the critical bearing capacity or failure (ULS). It is two mechanisms of failure that can be observed in ground stability design:

- Failure or deformations in soil for Geotechnical design (GEO);
- Failure in ground construction or part of construction for Structural design (STR).

It is a possible to study a combination of these failures also. However, it is simpler to investigate each failure individually.
The other aspect of a failure can be due to loss of statical equilibrium, for example in global stability (both STR and GEO). This type of failure adequate to presented design will be evaluated in a hand calculation and described in Section 5.1.1.

2.1.1 Failure in soil

Behaviour of soft soil can cause large deformations in volume and shape due to molecular structure of clay. Subsoil stiffness is distinctive very low regarding to water content. In this case clay is perceive to irreversible deformations, so called consolidation.
In this complex model under applied load of the building water easily moves through the clay and generate a flow which indicate parameters of deformation. This phenomenon is called hydraulic conductivity or permeability, *k*.

Permeability of clay is very low compared to other soil types. Since compression loads effecting small clay particles and the minuscule pore space in between them soil will starts to yield and create a compressive region.

The process in the compression zone can be assumed to be isotropic. Therefore, the soil under a foundation responds partially as a plastic. In critical point within the compressive zone, failure mechanism take form. The exposed zone is called a failure zone. To obtain failure in the exposed zone, soil initiate sliding or whole soil section achieves a failure limit. More about this topic in Chapter 3, especial in Section 3.1.1

Failure in soil along a pile occur in just few mm in interaction of a shaft of a pile and depend on an undrained shear resistance of soil. Nevertheless, the failure in soil at the pile-end depends on area of a cross-section movements to generate failure mechanism in soil and required 5-10% of pile diameter. These values have been used in elastic-plastic approach of Neutral plane analysis see Section 4.3.2.

2.1.2 Structural failure

In the case of a flexible building foundation, stiffness of the construction under consideration can lead to structural failure. Usually, an interaction between the ground foundation and soil creates a large distress, developing cracks and deformation between different parts of the construction. This instability lead to differential settlements in a building.

In the case of timbers pile the differential settlements can occur under ground water level under long time scales. It can be observed by dewatering, if the construction sides influence the ground water level negatively.

The differential settlement has been studied by many. Skepton and Mcdonald in 1956 calculated acceptable deferential settlements by using the relative rotation of structural analysis for raft foundations between two points, see Figure 2.3, more about this in Section 2.2.1.

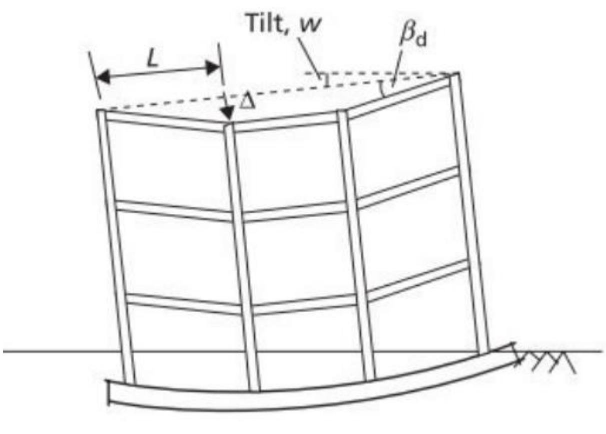


Figure 2.3. Failure mechanism according to differential settlements (Craig and Knappett, 2012)

2.2 Settlements

In this section the focus lays on ground movements under external load, and the conditions under which soil movements occur. Also, the first part will investigate the underground behaviour and the stress distribution in soil under a raft without a pile stabilisation. Following section on time dependent movements in soil and consolidations.

2.2.1 Vertical displacement

In 1974, Grant estimated acceptable settlements for raft foundations on soft clay to be 50 mm. For estimation of settlements of the raft elastic displacement theory can be used. So called settlements (SLS) by vertical deformation appears. Settlements under the raft load should be consider as uniformly distributed load with width and length.

Movements in the soil under the interaction process with the raft, will affect the moment and reaction forces which occur in the building parts. There are usually two types of settlements important to divide up. The total settlements, which are significant for the whole building, and differential settlements, affecting parts of the construction.

If a fully flexibility of construction can be assumed, it is easy to define raft as a chain of nodes where one of the nodes has the largest deflection or maximum settlement as it shown in a Figure 2.4. Therefore, the node B is the most exposed point in the raft and s_{max} is the maximum settlement of the raft and Θ_{max} represents the angular distortion.

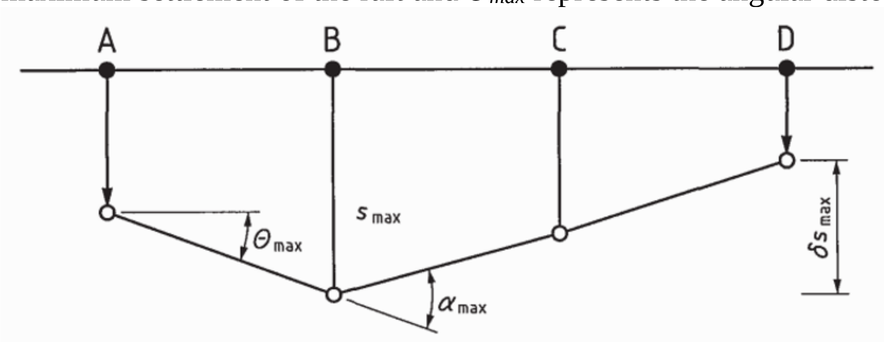


Figure 2.4 Definition of settlements EN 1997-1:2004(E)

According to Craig and Knappett (2012), the relative rotation or angular distortion is equal to differential settlement. As it shows on Figure 2.3 the Δ divided by length between two structural parts, L and is absent under circumstances where Δ/L<1/300.

The differential settlement was inspected for the first time in 1954 by Skepton and Mcdonald as it was mentioned before. In following research, Bejrrum (1963) and Charles and Skinner (2004) continued the investigation on differential settlements, but the part of interest is how differential settlements affect the raft foundation on soft soil? To answer this question, it probably best to look in to Horikoshi and Randolphs work (1997). They determined the differential settlements as a function of the raft and soil stiffness K_{sr}.
For flexible raft foundations the raft-soil stiffness has to be quite low and can lead to superior differential settlements. It is an approximation which can be fulfillled by increasing the number of piles or improving rigidity of the raft.

In 1983 has Randolph presented a new approach for calculation of raft stiffness. The approach based on the pile group stiffness calculations regarding to centre-to-centre spacing as a pile-to-pile interaction. Result of his study shows that the stiffness of combined design is not bigger than a single pile approach more about this in Section 4.2.

2.2.2 Time settlements and estimation

Settlements occur from different causes mostly of external load that create stress distribution in subsoil and are mostly time dependent. In history of soil mechanics, many have investigated stress distribution in soil. Boussinesq proposed elastic-isotropic-half space approach already in 1885. This was improved by Froehlich in 1934.

Usually, settlements are associated with permeability, *k* or hydraulic conductivity as it was mentioned in Section 2.1. Figure 2.5 where the settlements under the service life of the building divided in three categories.

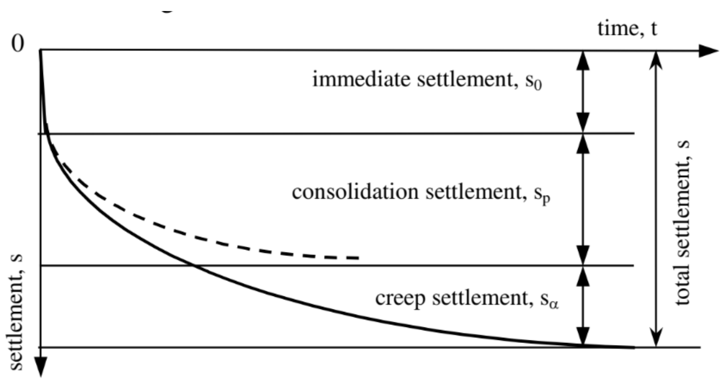


Figure 2.5 Schematic presentation of settlements in time (Eurocode 7, 2022)

The fastest deformation appears almost immediately as a reaction to the external load. Then, it slowly decreases over time. It can take many years before settlements flattens and remain 'constant'. Along with that, creep deformations can be seen.

Additionally, the total deformation is a sum of immediate and consolidation settlements and, if it is relevant, creep deformation is included, equation 2.2. According to guidelines, total settlements up to 50 mm are acceptable (Eurocode 7, Annex H, 2022).

$$s_{tot=}s_0 + s_p + s_\alpha \qquad (2.2)$$

Immediate settlements occur due to shear or distortion of soil. Estimation of immediate settlements can be achieved by establishing the Younge modulus, *E* from laboratory test such as in-situ triaxial tests for undrained model. It is difficult to establish elastic modulus, E_k for cohesion soil, therefore the elasticity module can be assumed as $E_k = 150c_{uk}$ for normal consolidated clay regarding to Craig (2012).
As one of assumptions of elasticity model is the stress variation is linear with depth, therefore the settlements can be estimate regarding equation 2.3. It has to be noted that the correction factors μ_0 and $\mu_1$depend on the shape of ground foundation see diagrams on Figure 2.7

$$s_0 = \mu_0\mu_1 \frac{q_{net}B}{E} \qquad (2.3)$$

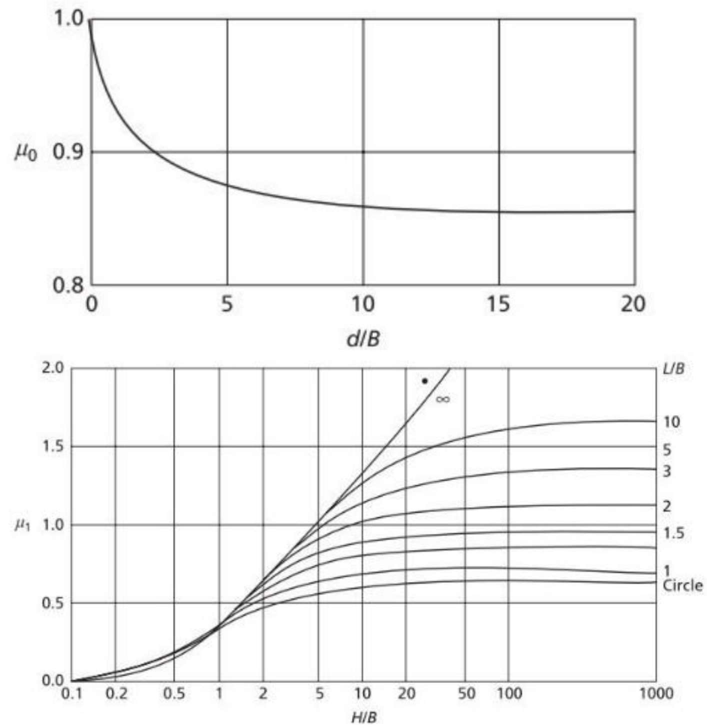


Figure 2.7 Diagrams of coefficients μ_0 and μ_1 under assumption that Poisson´s ration equal with 0,5 and depend on geometry of the ground slab. (Craig and Knappett, 2012)

Coefficients depend on the shape and stiffness of the foundation. Most of modern buildings build as rectangular and rigid ground foundations. The stiffness of the raft depends on reinforcement of concrete raft and relies in detailing of the amplified parts of the slab. The most sensitive zones of the slab are edges and corners and can be determined by calculations of intern forces in the raft. The design of the reinforcement is not included in this study.

2.2.3 Consolidations settlements

The second part of settlements (SLS) are settlements caused by consolidation or compression from applied load and can be assumed as a one-way stress field. This phenomenon happens due to volume change in soil and has a complex approach due to time dependents. A several assumptions have been made for simplification of calculations:

- the soil is homogenous;
- M-modulus is constant;
- permeability, k is constant;
- pure pressure under t=0 is known;
- the pure pressure is constant in time.

Therefore, the estimation of consolidate settlements is feasible according to these estimations and due to equation (2.4)

$$s_p = \mu \sum_i \left[\left(\frac{\sigma'_c - \sigma'_0}{M_{0,i}} + \frac{\sigma'_0 + \Delta\sigma - \sigma'_c}{M_{L,i}} \right) z_i \right] \qquad (2.4)$$

Correction factor, μ for evaluation of settlements depends on the soil property and degree of consolidation regarding the usage of compression modulus regarding Tomlinson (1986)

Consolidation settlements change with depth and time thus the simple hand calculations are not an easy task to achieve. For this reason, it is preferable to do numerical analysis. Regarding Terzaghi´s theory on consolidation, time variation applies in settlement calculation and variation of the initial process in time can be achieved. There is a substantial discrepancy between Terzaghi considerations and the observed deflections. Therefore, the usage of correction factor, μ is necessary.

In this study, by the investigation of soil profile is obtained what the compression modulus is constant in the stress range, therefore a bi-linear modulus is not considered.

3 Building foundations

3.1 Foundations

Building foundations are parts of construction which interact with the ground and transfer the structural loads to the soil. In particular, the building foundations are characterised by two main categories: the shallow foundation and the deep foundation.

Shallow foundations are mostly characterised by construction directly placed on the ground or close to the surface. Constructions such as continuous slab on ground, basement slab or footing of different geometric forms are examples of this kind of structures. All types of shallow foundations on soft soils have high risk for vertical and differential settlements. It is preferable to use slab as ground foundation on soft clay due to load and moment redistribution. The load path can be reduced by several ways. For example, excavated basement reduce stresses in soil and due to those building stresses replace the ground stresses, it can be beneficial for settlements and differential settlements. In this way, the excavated soil partially compensates the bearing capacity of the building. This method is called a compensated raft foundation. However, it is consistently neglected in designs.

In the case of Gothenburg clay, this kind of solution should be watertight. Ground water could have uplift phenomena and slightly resemble a boat or a raft.
The other problem of this design is that the loads from the building should be transmitted through the basement level to the raft and to the soil.

The other category of foundations is deep foundations, which mostly includes all type of piles. The pile is a construction part which takes all loads deep in to the soil or to a soil level with a better characteristic. Types, parameters, and installations techniques of piles will be described in Chapter 4.

3.1.1 Raft foundation and soil bearing capacity

The slab resting directly on the ground, transmitting the load path to uniformly distributed load over entire area, is called raft or mat foundation. The raft foundation transports the load directly into the ground and generating insignificant internal forces. Importantly, the raft foundations are used to support the whole building and designed for applaied loads and load combinations. However, more concentrated loads generate moments and shear force, which can provide increasing tensile stresses and lead to cracking. It is important to consider soil stiffness when evaluating bending moment and shear resistance of the raft.

In Gothenburg, there are many projects built on soft soil and differential settlement is an issue for many ground foundation designs. Rafts are one of the ways to decrease settlements in building design and only accepted settlement will remain.

The bearing capacity of the raft depends on underground properties. Thus, to evaluate the strength of the raft the stiffness of soil must be evaluated.

H. Poulos recommends in his study to check for both lower and upper bound set of soil stiffness for the evaluation of raft behaviour (2001).

Under deeper investigation of soil stiffness under undrained conditions, a combination of lower (static, LB) and upper bound (kinematic, UB) approach lead to:

$$4c_u \leq q_f \leq 6c_u \tag{3.1}$$

Consequently, the equation of the unique solution for collapse load of the raft is

$$q_f = (2+\pi)c_u = 5.14c_u \tag{3.2}$$

In Sweden, the design of the bearing capacity of the raft foundation must follow the regulations of the European Committee for Standards part 7 or simple Eurocode 7 (EC7). The design is simple and based on elasticity theory of LB and UB, where stresses are uniformly distributed under the footing. It can be presented as the ultimate bearing capacity under undrained soil conditions

$$q_f = 5.14v_c c_u + \gamma_D D \tag{3.3}$$

Consequently, for a long-term approach, presumes to be calculated under drain conditions of the soil thus it have to be taking into account too.

According to EC7 recommendations, the bearing capacity for shallow foundations under drained conditions should be done using elastic-plastic solution and can be calculated by:

$$\frac{R}{A'} = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 0{,}5\gamma' B' N_\gamma b_\gamma s_\gamma \tag{3.4}$$

By simplifying the equation and adapting it to the case of the studied raft foundation, the equation appears as:

$$q_f = c'N_c + \gamma D N_q + 0{,}5\gamma' B' N_\gamma \tag{3.5}$$

where bearing capacity factors N_q, N_c, N_γ depend on friction angle ϕ'.

However, under Gothenburg's circumstances, the soil stiffness versus applied load from buildings is not sufficient to reduce settlements to acceptable levels. Therefore, a conservative design of the building foundations is regularly over through the combination of raft and piles.

3.1.2 Rigidity of the raft

Settlement under the raft varies along the length of the raft. This means that the point close to the centre of the raft has different deflection than points closer to the edge. It is the rigidity of the ground construction that determine the stress variation in the soil under the raft.

According to Kany (1974), who improved the original Steinbrenner´s diagram (1934), the stress field in soil under flexible raft can be obtained from equation 3.6.

$$\sigma_z = \frac{q}{2\pi}\sum_{n=1}^{4}\left[\arctan\left(\frac{a_n b_n}{z\sqrt{a_n^2+b_n^2+z^2}}\right) + \frac{a_n b_n z}{\sqrt{a_n^2+b_n^2+z^2}}\left(\frac{1}{a_n^2+z^2}+\frac{1}{b_n^2+z^2}\right)\right] \tag{3.6}$$

Kany determined settlements in specific point at the surface level through dividing raft on four areas and addition of settlements for each layer at the corners of the raft, see Figure 3.1.

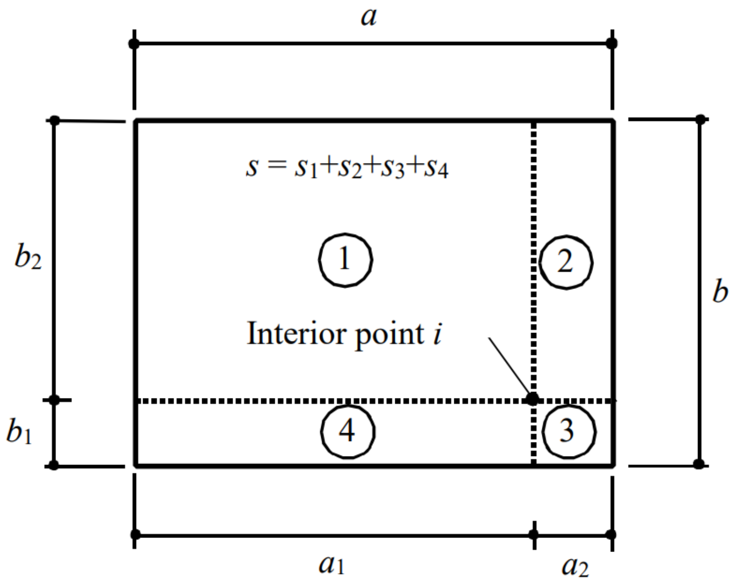


Figure 3.1 Geometry to superposition on settlements evaluation according to Kany (1974).

This evaluation can lead to failure if the foundation will be restricted on the edges and under action of horizontal load according to Burlands evaluation on structural failure. It can be avoided by dividing designs in two categories. If a foundation is assumed as fully flexible, it should have uniform contact with soil pressure. In this case, the settlement would be estimated as non-uniform. Alternatively, a fully rigid structure should be assumed to have nonuniformly distributed soil pressure, giving uniform settlements. As is seen in the Figure 3.1, the soil pressure for the raft in clay has the largest reaction at the edges.

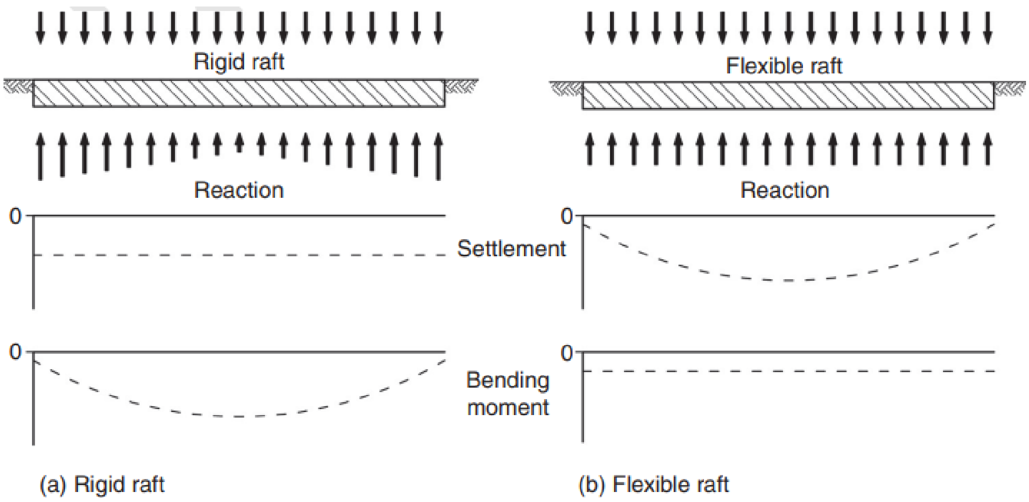


Figure 3.2 Schematic presentation of rigidity of the raft on soft clay (Burland., 2012)

In reality, foundations are neither fully flexible nor fully rigid and their properties are somewhere in between. In this way, it would be acceptable to assume that the raft is a rigid construction but with a rigidity correction factor (μ_r). The correction factor is applied on maximum settlement and has been obtained from investigations that have made over the years (Kempfert al et., 2006), see Table 3.1.

Table 3.1 Correction factor of rigidity of the raft, μr (Plattgrundläggning handbook, 1993)

L/B	H=B	H<<L
1	0.68	0.77
2	0.72	0.78
3, 4, 5	0.79	0.8

To estimate rigidity of the raft, circumstances of interaction between raft and the soil under raft must be taken in consideration. In the case of investigation of soil and raft stiffness, the slenderness ration, λl should be considered. The slenderness ratio of the raft can be estimated from the equation 3.7, (Plattgrundläggning handbook, 1993)

$$\lambda l = l * \sqrt[4]{\frac{k_s b}{4 E_{pl} I_{pl}}} \qquad (3.7)$$

Furthermore, if the stiffness ratio $\lambda l < 1.5$, the raft is assumed as a stiff construction and flexible if $\lambda l > 3$.
To estimate the raft thickness, t especially in the corners of the raft, equation 3.8 should be used.

$$t \geq 0{,}69 \left(\frac{E_{jd}}{E_{pld}}\right)^{\frac{1}{3}} \left(\frac{l^{16}}{b^3}\right)^{1/13} \qquad (3.8)$$

If the raft appears as a rigid construction, then the raft can contribute to bearing capacity and can be sufficient with a number of piles, length of piles or pile type. But many raft designs where the raft is relatively slender and cannot contribute to bearing capacity of the structural load must be improved by deep foundations.

3.1.3 Piled raft foundation

The usage of raft foundations is an option when soil properties have sufficient bearing capacity and settlements is under an acceptable level. In cases when the raft foundation is improved with piles, part of the load will be carried by piles deep to the ground. This kind of commercial design still remain the initial design which was proposed by Burland et al., (1977). Burland recommends a quite simple approach, where one pile under each column, as a bearing part of construction, transfers total load from one part to another. In this case, it leads to massive number of piles in the design. However, it does fulfill the load redistribution into the ground. But can it be that this design is over dimensioned? Can it be possible that the requirements can be fulfillled by even less piles or of different material?

The piled raft foundation is generally assumed to be a continuous slab, supported by columns/piles which do not carry any load. Otherwise, it is the slab placed on the subsoil with an interaction area which can be seen as a construction part with springs along the area as shown in Figure 3.2. Uniformly redistributed load makes the slab react as a beam and a concentrated load makes the slab react as a plate due to the geometry of the slab. (2D / 3D slab reaction).

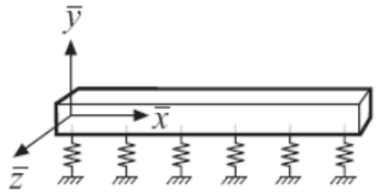


Figure 3.3 Visualisation of the raft-beam on springs, numerical approach.

To evaluate the slab bearing capacity placed on two components, it is feasible to use Timoschenko's beam theory which is a combination of Hook's law and Navier's formulation. In this study, the calculation of slab and slab stiffness is not included.

3.2 Design approach for raft foundations

Many have studied variations of piled raft design: pile number, placement, material of piles, length of piled and so on.

Basically, all these variations lead to the main problem which occur under calculations of axial forces and bending moment. Due to stiffness requirements of the pile-raft connection the length of piles should be bigger than the width of the raft. Thus, Fleming in his book "The Piling engineering" (2009) has categorised design strategy in two sections.

First is where the raft does not contribute to bearing capacity, hence the piles have full capacity of the applying load. Terms for this category is if the width of the raft is smaller or just same length as the pile. While if the width of the raft is greater than the length of the piles (*B/L*>1), the structure can be assumed as a shallow foundation and the raft contributes to load bearing capacity and serviceability.

According to Poulus and Randolf (1996) the raft-pile interaction can be obtained from equation 3.9.

$$K_{pr} = \frac{K_p + K_r(1-2\alpha_{cp})}{1-\alpha_{cp}^2 K_r K_p} \tag{3.9}$$

Where K_{pr}, K_p, K_r stiffnesses respective construction part and α interaction factor. But entire contribution of raft to load bearing capacity can be calculated by equation 3.10

$$\frac{P_{tot}}{P_r} = \frac{K_r(1-\alpha_{cp})}{K_p + K_r(1-\alpha_{cp})} \tag{3.10}$$

3.2.1 Underground constructions design

Two analyses for elastic raft foundation analysis have been introduced: the subgrade reaction coefficient method and the elastic-half-space method.
The subgrade method is based on the assumptions that the permeability of the ground is directly proportional to the settlements and that the activated ground responds as independent compression springs, where the structure can be imitated by beam elements.

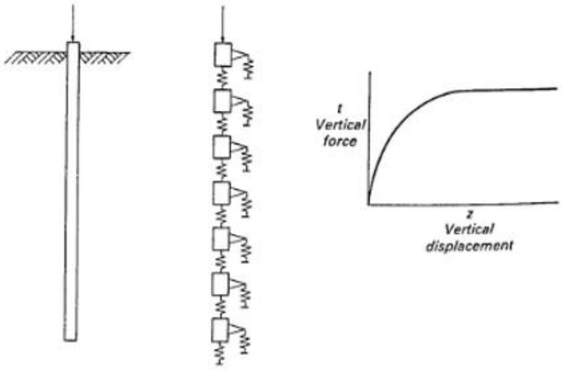


Figure 3.4 Visualisation of t-z model for pile-soil relationship according to Tomlinson (1987).

From a historical point of view, the Winkler in 1867 presented a one parametric model of the interaction of soil and the raft. The approach was used widely until Pasternak improved the model into two-parametric model, where the soil around the foundation is taken in the consideration. The next step in the process was when elastic-half-space analysis was developed by Ohde in 1942, which included physical soil properties as the nonhomogeneous layers of soil, geometric properties of affected soil, Young's module, Poisson's ratio, soil density etc. In 1885 second order approximation by Boussinesq for subgrade modulus was introduced. The elastic-half-space analysis or Boussinesques comprehensive model integrates the settlements and the raft behaviour through the ground, see Figure 3.4.

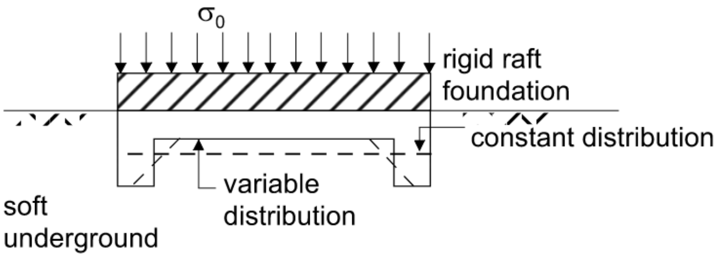


Figure 3.5 Visualisation of Boussinesques comprehensive model. (Kempfert al et., 2006)

Both analyses depend on the load redistribution, rigidity of the raft and the soil parameters. Kempfert and Gebreselassie (2006) made a large amount of research on the estimation of settlements due to raft model on soft soil and gathered conclusions on the topic. Recommendations on the calculations of settlements and bearing capacity of the raft should be:

- based on oedometer test and improved with factor μ, which is the ration correlate from laboratory tests and measurements in the field
- for time-settlements calculation consolidation test should be used and the coefficient of consolidation derived from that, c_v
- non-drainage under the raft or nearby due to load transfer
- the rigid raft model can be verified by applying either the elastic-half-space method or the modified at the edge and corner strips with factors 1.75 and 3.5 the modulus of subgrade reaction (strip model).

3.2.2 Combined piled raft design

The piled raft is a complex system where both raft and the pile contribute to load bearing capacity. Mostly important is a zone of interaction of the structure and the soil, but there are also more things to consider in the design.

The four factors which are important to consider are shown on Figure 3.5. They are:

- the interaction between pile and soil around pile;
- the interaction between pile and raft;
- the interaction between raft and soil;
- the interaction between pile-soil-pile.

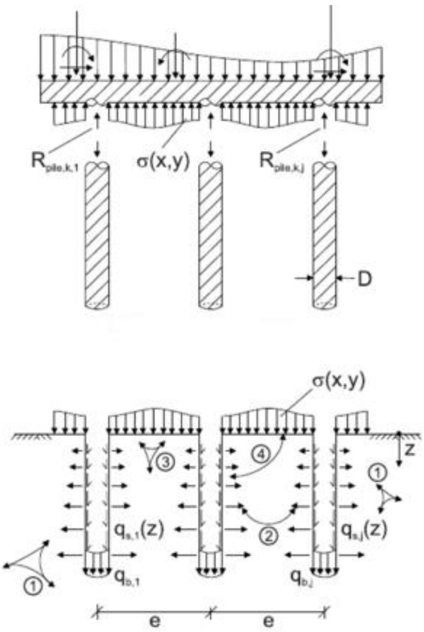


Figure 3.6 Visual presentation of load actions and stress distribution on piled raft

The design requires sufficient input on soil properties for further investigations. Consequently, the first step in the design process is to evaluate soil parameters and compile a profile of subsoil levels.

Load capacity of the raft depends on soil parameters and can be obtained through equation 3.11. Equation 3.12 include the pile shaft and pile toe resistance. And in equation 3.13 shows how both raft and piles contribute to total bearing capacity.

$$R_{raft,k}(z) = \iint \sigma(z,x,y)dxdy \qquad (3.11)$$

$$R_{pile,k,j}(z) = R_{b,k,j}(z) + R_{z,k,j}(z) \qquad (3.12)$$

$$R_{tot,k}(z) = \sum_{j=1}^{n} R_{pile,k,j}(z) + R_{raft,k}(z) \qquad (3.13)$$

In preliminary design it can be assumed that pile raft design follows the diagram in Figure 3.7.
There P_1 is an ultimate raft load and at this point (A) the inclination change due to increasing of stiffness by piles stiffness. At this point (B) the piled raft construction has reached the total

ultimate load capacity, P_u and conclude total failure. In reality a graph is not linear and see mush smother, but it is one of assumptions been made by Poulus.

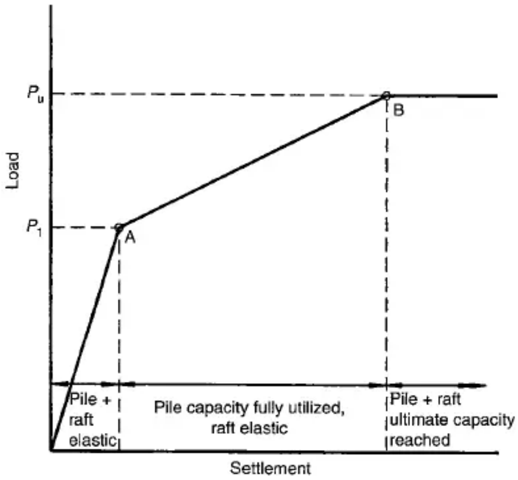


Figure 3.7 Interaction on piled raft behaver under load action when failure mechanisms created and fulfills (Poulus, 2001).

3.3 Load and load combination

There are several load and load combinations that designer must take in consideration for optimal and safe design of the raft or piled raft. It is the engineer responsibility to consider all possible loads and stresses implicated on ground construction due to the weight of the building.

3.3.1 Load indication

One of the assumptions of the elastic-plastic theory is that the raft shape is infinitely long. To define an infinitely long surface for interaction between raft and underground it should be assumed several limitations. Hence it is important to find the effective area of the raft.

However, as a first step a shape of applied load must be defined. The shape of the interaction area can be described as:
Point load: where the applied load impact at one concentrated point.
Linear load: where the load is applied linear form, for example on the edge of the raft.
Distributed load: where the load is distributed uniformly or linearly among entire raft.

At the same time, the load can be applied vertically or horizontally and can be static or dynamic. Mostly considerable load applied on the raft is:

Permanent action, *G*
- self-weight: vertical load from the building it self;
- earth pressure: both horizontal and vertical load transported through walls to the raft;
- differential settlements: shear load in the material of the building, such as façade or walls and slabs;
- shrinkage: shear load in concrete raft;
- creep: shear load as a result of time.

Variable action, *Q*

- wind load: horizontal load affecting the whole building which is transported to the raft through the walls and slab;
- snow load: vertical load applied on the roof, depends on roof shape;
- fatigue load: both horizontal and vertical load depends on cycled reloading;
- dynamic load: transverse horizontal load, a result of seismic activities or other movements such as train or traffic;
- imposed load from activities in the building: vertical load from the people and furniture;
-temperature: stresses in material mostly in concrete parts of the building.

Accident load is irrelevant in this study because of the static model of construction.

3.3.2 Load combination

In the design of the building, the standard routine is to evaluate the time of serviceability. Therefore, the Eurocode offers the partial safety coefficient method to estimate load combination in the building for service time between 50-100 years or quasi-permanents load.

The load combination of the raft design should be checked for

- Combination in Ultimate Limit State (ULS);
- Combination in Serviceability Limit State (SLS);
- Quasi-permanent combination (QP);
- Equilibrium combination (EQU).

Characteristic load combination for ULS applies on both structural design (STR) and geotechnical design (GEO) structural load situations. The combination must be checked for both in favourable and unfavourable actions, and can be calculated by equation 3.14

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i} Q_{k,i} \qquad (3.14)$$

Similarly, for calculations of SLS both STR and GEO load situations are applied except the stage of load combinations neglecting the stress-strain field of soil, and partial factor is assumed is to be 1. The characteristic load combination for SLS should be calculate using equation 3.15

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad (3.15)$$

Quasi-permanent combinations for SLS as long-term effect should be obtained using equation 3.16

$$\sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (3.16)$$

It must be highlighted that the imposed load calculation when applied to an area of several floors should be multiplied with a factor α_{n}.

The static equilibrium limitation applies in stability calculations, such as sliding and tilting. The limitation is divided in two reactions. The first as favourable load combination, equation 3.17

$$\gamma_{Q,1} G_{k,inf} = 0{,}9 * G_{k,inf} \qquad (3.17)$$

and unfavourable load combination determined by equation 3.18

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j} + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\,\psi_{0,i}Q_{k,i} = \gamma_d 1{,}1G_{k,j} + \gamma_d 1{,}5Q_{k,1} + \gamma_d 1{,}5\psi_{0,i}Q_{k,i} \quad (3.18)$$

3.3.3 Structural and geotechnical actions and design approach

Hence, in load evaluation due to load redistribution through the soil, ground water or surroundings, a geotechnical load combination should be represented. Geotechnical load (GEO) or deformation in subsoil and underground can for instance be lateral loading of soil pressure or down drag recording to IEG Report 8:2008, rev 2.

According to Eurocode 7, dimensioning of piles should be managed due to design approach 3 (DA3) for STR and according to design approach 2 (DA2) for GEO. Total model of applied load consist of A1+ M1+R2, and the partial safety factor, γ_d should be applied on both load combinations and ground bearing capacity. These categories reduce the surface failure by coefficient γ_E and shear resistance by coefficient γ_R.

Since this study aim at evaluating only the static model of the building, the estimation of load combination excludes cycled and dynamic load or accidental load. In SLS and ULS, the load combination is presented in SS-EN 1990, Chapter 6 as the method of partial safety coefficient and should fulfill:

$$R_d \leq E_d \quad (3.19)$$

where R_d is the sum of the material weights and applied loads, earth and water pressure and E_d bearing capacity of the construction.

The calculations based on probability theory and included a safety factor. Then the mathematical model limited by assumptions which has many simplifications, the safety factor, F_s can be included for certainty on calculations and safety of design. The factor is based on the probability theory and is established by correlation of possibilities of failure under considered circumstances, for Eurocode is recommended value of 1.5.

4 Deep foundations – Piles

In case of highly concentrated load, soil of low strength or high requirements on settlements it is conventional method is to add deep foundation stability in the design by adding piles. Piles are quite similar to columns and have geometrical properties: cross sectional area per meter, and it can be described as diameter, D and length, L where $L>>D$.

Installation methods for piles can be divergent, for example driven, drilled, vibrated, jacked, or hammered into the soil. They vary in material and prefabrication methods such as prefabricated concrete piles and in-situ lime-cement piles. Also, piles vary in construction material and can be made of concrete, steel, timber or composite materials.

In this Chapter all focus will be on the pile design. Also, I include an elaboration on settlements and analytical calculations of load capacity of the pile through α- and β- methods and a modified Neutral plane analysis.

4.1 Pile types

According to the EC7, piles can be categorised in three overall categories:

- soil displacement piles;
- non-displacement - bored piles;
- micropiles.

In cases where piles are pushed or jacked under installation in a vertical direction, they are called for soil displacement piles. The installation procedure is executed by driving piles into the ground with a drop hammer. Generally, these are pre-cast concrete piles, but they can be timber piles, steel piles or prestressed concrete piles. The process of installation for this kind of piles very noisy and costly.

Under this installation prosses, soil movements are most possible occur. Clay will change in volume and may heave. Because of these properties of the surrounded clay will change and will create a resistance in the several centimetres of clay along the pile shaft, approximately 3-5mm. As a conscience of that phenomenon, a remoulding of clay and change of stress field around the pile occurs. More elaboration on this in Section 4.3.1.

Bored piles are piles where the soil is replaced with a pile before pile installation. While deep mix piles are categorised as bored piles, it is a generally flexible process with possibilities of many risks during installation. The major problem during this type of process is ground water level and soil displacement.

In case when the diameter of the displacement pile is less than 0.15m and the bored pile is less than 0.3m they are classified as micropiles. This type of pile is usually made of steel such as a "root pile" or pipe pile and is used very widely both in building constructions and infrastructures projects. In many of the projects in

Gothenburg, this type of piles are chosen for ground bearing strength achievement.
Hence, different types of pile and choice of installation method depends on soil properties and construction requirements. It is the responsibility of the engineer to decide on the proper analysis and type of pile.

4.1.1 Reinforced concrete piles

The most common pile in the Gothenburg area and whole Sweden is the reinforced concrete pile with a cross section area between 250 -275 mm. In this kind of piles, the amount of reinforcement is pivotal to achieve the required strength. Concrete piles are pre-cast with a length of 12-14 m and can be jointed together with a steel joint.

Mostly the variation in strength depends on the reinforcement of the pile for SP1, SP2 and SP3, which are types of concrete pile used 4 or 8 bars with diameter of 12mm or 16 mm. Concrete quality of C50/60 is a standard but can vary depending on soil properties and exponential classes (Hercules Betonghandboken, 2004).

According to table, adapted from the Hercules Betonghandboken, the strength of the concrete pile has been evaluated and rely on the cohesion of the soil, see Table 4.1

Table 4.1 Represent the strength of the concrete pile (SP1) both in compression and tension, joined, C50/60 (Hercules Betongpålehandbok, 2004)

	(ULS) [kN]		(SLS) [kN]	
C_{ud} [kPa]	**tryck**	**drag**	**tryck**	**drag**
3	537	280	598	231
4	648		657	
6	846		736	
8	977		787	
10	1038		824	
12	1085		853	
15	1136		887	
20	1193		929	

Durability of the concrete pile is high. However, in cases where sulphides occur in ground levels, there can be problems which affect the stiffness of the pile. There are many things that affect the concrete pile under Life Cycle Analysis (LCA), such as frost and water table variance, and chlorides from the traffic, nevertheless, the main issue with concrete piles is the interaction of two materials such as concrete and steel and shrinkage of concrete over time.

4.1.2 Timber piles

In Sweden timber piles are more often used as a shaft resistant pile in cohesion soil. Mainly, they are pinewood piles with a length up to 18 m and as all trees have a conical form. With diameter at one end of 350-400 mm and minimum of 125mm at the other, they should be exceptional straight.

Timber piles have been used widely in road embankments both in Central and North Europe and in Canada. They are one of the optimal solutions for reinforcement of roads and for buildings constructions at the marine environment on soft clay with low stability. Different usage of timber piles can be possible as a contribution and expectantly replacement for concrete and steel piles. Furthermore, the variety of cross-sections of the timber piles, and strength of soft wood, lead to low serves load.

According to Bro 94 (1994) is the bearing capacity of timber pile depends on compression strength parallel to the grain and the characteristic sheer strength of the soil which should not exceed 7kPa. Characteristic stresses should be calculated in compression according to the equation 4.1

$$R_d = f_d A \qquad (4.1)$$

were f_d for unjointed timber pile is accepted as 6 MPa and for the jointed pile is decrease with 17% to 5MPa.

The American Society of Test and Materials (ASTM) properties of the timber piles can be assumed according to Table 4.2. As it seems, the strength of the pile changes with a quality and specie of timber. Thus, timber pile parallel to the grain in compression recommended by American Society can be assumed to be
$f_d = 1200 * 6.9 = 8.28MPa$

*Table 4.2 Properties of timber piles for two common wood species, (1psi is 6,9kPa so E_(t.pile)=1.5*6.9=10.35 GPa)*

Timber species	Compression parallel to grains (psi)	Bending stress (psi)	Modulus of elasticity (psi)
Southern Pine	1200	2400	1.5×10^6
Douglas Fir	1250	2450	1.5×10^6

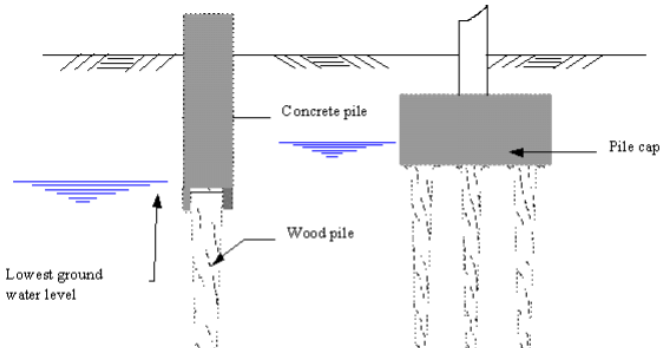
Timber is a natural material and the cross-section of a pile vary with length according to $A = 0.15 + 0.014L$. In the same way, the pile strength will differ along the pile. In consideration of that and regulations of EC7, the calculations on maximum vertical strength can be made at the head of the pile with equation 4.2

$$R_{head,EC} = A_{head} f_d = \pi * (\frac{0.35}{2})^2 m * 5MPa = 481\text{k}Pa \qquad (4.2)$$

$$R_{head,ASTM} = A_{head} f_d = \pi * \left(\frac{0.35}{2}\right)^2 m * 8.28MPa = 797\text{k}Pa \qquad (4.3)$$

Timber piles should be up to 80 % peeled from bark due to ASTM. This treatment helps during the installation of timber piles. Despite this treatment, most failures occur under installation when the hummer splits the grain because of overdriving of the process. Because of that, a steel hat is used on tip of the pile and a special weight of the hummer is chooses, usually around 4 tons (Aarsleff team, 2022).

Ground water level (lowest level) is the main problem under the life cycle of timber piles, which is way preservative treatment should be taken in consideration. In practice, the treatment of the head part of the pile which is exposed to ground water variation is with creosote. However, since it is a carcinogenic substance, the use of treated pile has been minimised in urban areas. In later years, concrete heads are preferred instead of chemical



treatment, see Figure 4.1

Figure 4.1 Concrete protection of timber pile a) concrete head on top of timber pile; b) concrete pile cap on top of timber pile group.

The investigation of concrete protection, and joints between concrete and timber piles, are not included in the study.

4.2 Pile design

As already mentioned, the pile reminds of a column. Besides that, it is necessary to focus on the application point and the limit state. Visualisation of structural failure in the pile and soil failures under ultimate load is shown in Figure 4.2. As shown, there are three major failures of the pile: under compression, under tension and transverse loading.

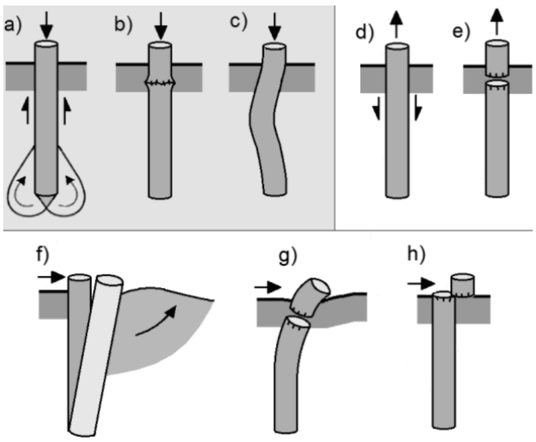


Figure 4.2 Pile failure a)-c) compression d)-e) tension f)-h) transverse loading adopted from

Depending on the rigidity of the pile, it behaves as one unit and sinks where displacement occurs deep in the soil, see Figure 4.2 mode a), where the compression resistance due to shear of the shaft and toe resistance of the pile are shown. The same phenomenon can be detected under installation of the pile.

Next failure described occurs in the pile material see mode b) where the pile in compression or tension as it in the mode e). Also, bending (g) and buckling (c) is failure of the created moment. It is possible to have failure in compression when the buckle in the soil and displacements affect the soil strength. This is followed by a failure of tensile resistance see mode d), so called uplift and at last is a failure in soil under transverse loading mode f).

4.2.1 Design values and analysis establishing

The pile design must meet Swedish standards BFS 2015:6 EKS 10 (Boverket) and Eurocode 7 for criteria of calculations for characteristic value regard to analogy of pile model. In this approach, a model presents a characteristic value for pile resistance reduced by partial factor γ due to equation (4.4)

$$R_d = \frac{1}{\gamma_{Rd}} * \frac{R_k}{\gamma_R} \tag{4.4}$$

where γ_{Rd}=1.7 is the partial resistance factor for cohesion piles

Partial factor (ULS) should be used in any characteristic values (Annex A, EC7):

- On actions (γ_F);
- On soil parameters (γ_M);
- On effect of actions ((γ_E);
- On spread foundations (γ_R);
- On driven and bored piles (γ_R) etc.

Another approach is the load testing approach. This method is based on load testing when characteristic resistance is reduced by a value evaluated from the drag test

$$R_k = \frac{R_{cal}}{\xi} \tag{4.5}$$

Where the correlations factors, ξ are based on earlier test and monitoring of construction behaviour (Trafikverket, TK Geo 13). According to EC7, verification of STR and GEO limits state to obtain characteristic value for pile foundations under axial load. They can be divided in three subcategories: static tests, dynamic tests, and ground tests.

The characteristic value is evaluated from the medium or minimum value of drag tests (ground test) divided with a correlation factor ξ_3 or ξ_4 respectively according to TK Geo 13. The factor is pre-calculated and depends on numbers of geotechnical tests, see Table 4.3, there *n* is quantity of the tests.

Table 4.3. SS-EN 1997-1:2005 correlation factors based on the ground test for estimation of characteristic value. n is the number of testing piles.

ξ for n =	1	2	3	4	5	7	10
ξ_3	1,40	1,35	1,33	1,31	1,29	1,27	1,25
ξ_4	1,40	1,27	1,23	1,20	1,15	1,12	1,08

The next procedure is to calculate resistance. The characteristic resistance has to be reduced by a partial resistance factor $\gamma_{Rd,e}$=1.4. It can be calculated by an empirical method or Semi-Emperical method along with CPT tests. Parameters gained from these calculations decide the α- and β- methods.

4.2.2 α-method

Calculations for axially loaded piles on bearing capacity can be based on adhesion between the pile and soil, which develops under pile friction in clay, the so called α-method.

Many have investigated the approach of α-method and have looked into adequate assumptions. Niazi and Mayne (2013) investigated the α-method 25 times for piles in soft soil and compare them by pile length, failure and stress redistribution.
In 1984, the Semple and Rigden presented the adhesion coefficient as a function of ratio of depth to pile diameter and ratio between average shear strength and average overburden pressure, see Figure 4.3

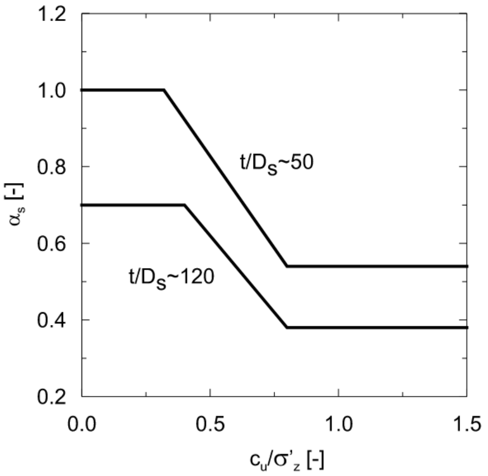


Figure 4.3 Adhesion coefficient α regarding Semple and Rigden, 1984

According to American Petroleum Institute (API), the function for evaluation of α looks like:

$$\alpha_s = 0.5\left(\frac{c_u}{\sigma'_z}\right)^{-0.5} \quad \text{if} \quad \left(\frac{c_u}{\sigma'_z}\right) \leq 1.0 \qquad (4.6)$$

$$\alpha_s = 0.5\left(\frac{c_u}{\sigma'_z}\right)^{-0.25} \quad \text{if} \quad \left(\frac{c_u}{\sigma'_z}\right) > 1.0 \qquad (4.7)$$

In 1992, Fleming et al. established that the adhesion coefficient α depends on the consolidation ratio (OCR) of surrounding soil. In contrast, Randolph at al. proved in 1979 that α does not depend on OCR in total stress analysis. In conclusion, the zone of possible α values has been established, see Figure 4.4

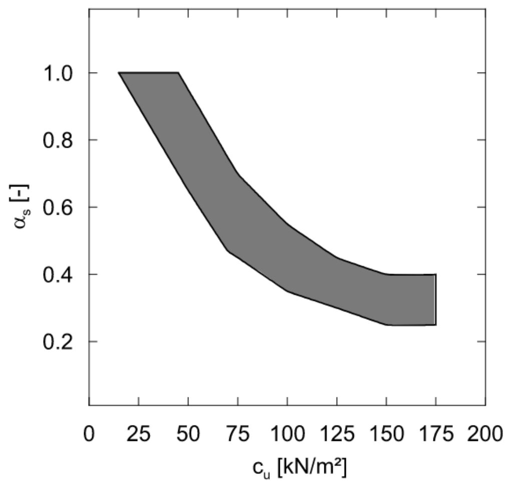


Figure 4.4 The zone of possible value of α. (Kempfert et al., 2006)

The α-method is based on total stress calculations under undrained conditions and symbolizes the short-term conditions in cohesion soil.

4.2.3 β-method

The β-method is similar to the α-method, although the calculations for axially loaded piles on bearing capacity are based on the effective stress analysis at the correct depth and correspond to more suitable drained soil behaviour. It is reminiscent of the concept of low bound approach, LB and upper bound approach, UB methods for shallow foundations. The method is based on the friction of the pile in the soil along the shaft and predict the friction factor β, so called Bejrrum-Burland coefficient.

It seems that the β-method is better description of the long-term response of the bearing capacity of the pile. The method constrains the ratio of the horizontal and vertical stresses which are represented by the earth presser coefficient, K which emerge under pile installation. In this case, the shaft bearing factor β can be estimated according to equation 4.5:

$$\beta = K \cdot \tan{(\phi')} \qquad (4.8)$$

It depends on the penetration depth and soil friction properties. The β factor has been established of the approximate zone of possible values, see Figure 4.5.

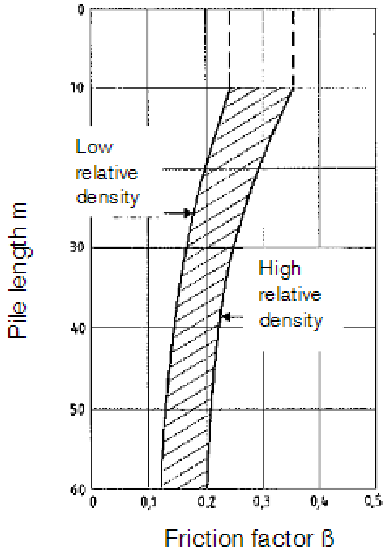


Figure 4.5 Shaft bearing factor based on the friction in cohesion soil, (Alen, 2009)

The β-method is based on effective soil pressure and correlation of the shaft resistance along the penetration depth. Nevertheless, for establishing the soil pressure the method of average pressure coefficient based on Rankine theory.

4.2.4 Installation effect of piles

Displacement piles have a strong influence on the installation process where the stress in soil will be increased by the movement of pile. Many have look into that, for example Randolph (1979) and Clark and Meyerhof (1972) who measured stresses under installation. Many have also created experimental tests for measurements of driven piles, for example Ottolini et al. (2014). He followed the installation process and tested the theoretical assumptions through experimental tests.

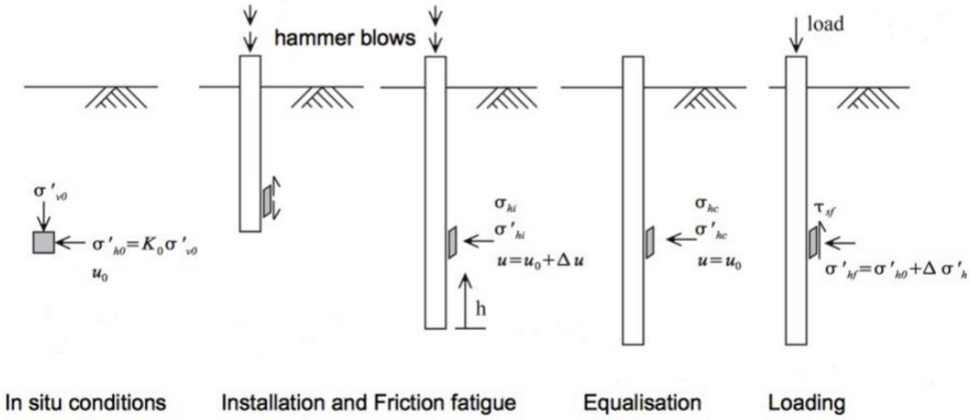


Figure 4.6 Schematic presentation of installation process for the pile in cohesion soil.

The installation process has 5 steps, as shown in Figure 4.6. The untouched clay has effective vertical and horizontal stresses σ'_{v0} and σ'_{h0} respectively. At the same time, the ground water creates water pressure in clay fine pore structure. Then, under installation of the piles, the stress wave occurs because of hammer blows. In this way, stresses are generates due to shear strength along the pile and in the surrounding soil. This process leads to strength redistribution and an increase of water pressure in the region along the pile shaft and the pile toe, or consolidation of surrounding soil. After some time, the soil region around the pile will stabilise and the water pressure come back to a normal stage. However, the total stress will have increased. In conclusion, when the load will be applied on the pile, the developed shear strength will resist the applied load and will change by 20% according to the Coulomb friction law:

$$\tau_{sf} = \sigma'_{hf} tan\delta \quad \text{and} \quad \sigma'_{hf} \approx 0{,}8\sigma'_{hc} \qquad (4.9)$$

In his research, Skempton (1954) pointed out that the excess of pore pressure can be calculated from the expression 4.10

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)) \qquad (4.10)$$

where A is the pore pressure coefficient, and for undrained clay, B can be taken as 1. Method works perfectly in soft clay due to permeability, but it is not recommended in the stiff clay or sand due to quite small changes in effective stress. The method is really useful and works as a base for neutral plane analysis.

During the installation process, the surroundings can be disturbed due to movements in the ground and vibrations. To partially avoid this problem predrilling can be made and part of clay mass can be removed. Usually, this are around the 10-12 meters boreholes with an auger of an area close to pile area. The downside is that this process is costly, and it takes a longer time. The question is if it is worth the time and money?

Another effect of the installation is that during installation the pile is exposed for repeated stress from the hammer. These cycled jabs define a limitation for the geotechnical bearing capacity of the pile and cause fatigue in the material. The effect of the fatigue in the material influences compression capacity in the pile and which must be taken in consideration during the design of piles.

4.2.5 Pile rigidity or concept of structural capacity of a pile

As it was mentioned in Section 4.2.4 piles during installation can deflect due to soil movements and applied load from jacks. However, the pile and soil stiffness determine stresses until pile start to yield and failure mechanism is fully accomplished. Other criteria should be mentioned that a long and short piles behaver is differently. In this study the long piles are the main investigation thus the short pile behaver is not considered.

As it shows in Figure 4.13, axial and the vertical stresses and a bending moment have large effect on a pile head under working loads. Where horizontal forces usual from a wind acting in tension or as compression applied vertical and acting through connections also bending moment from structural tilting has a great reaction on pile behaviour.

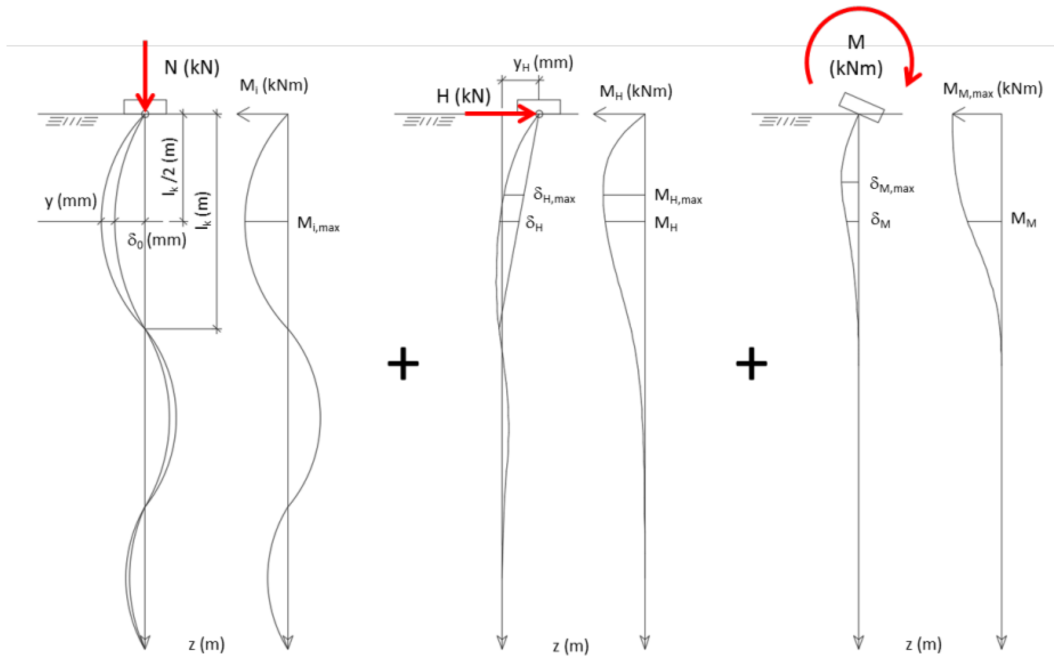


Figure 4.7 Forces affecting the pile at the pile head.

Long slender pile starts to buckle and develop a failure mechanism under applied lateral load from structure above. It creates a bending moment in the pile and contribute to pattern of failure mechanism. There a stiffness of the pile is one major contribution to structural capacity.

In Figure 4.7 mode a) shows a geometrical model of failure mechanisms in compression or tension, with the negative effect, under applied load in the soft soil. In this case, under maximum load the pile starts to deflect and buckle. Pressure from surrounded soil would contribute to resistance of construction.

Figure 4.7 mode b) show second reason of lateral deflection of the pile which is occur under effect of horizontal force. Because of assumption on rigidity of the pile and non-restriction at a bottom the maximum stress will appear in an upper part of the pile. The critical depth can be evaluated, z_{cr} and the critical force, P_{cr} (Report 96-1, Pile Commission, 2014), see Structural Calculations.

To establish equilibrium in the system the horizontal load should reflect the soil shear strength and the soil pressure create the bending moment in the entire system, mode c).

Pile Compendium (Statens Geotekniska Institut, 1993) presents analysis on bearing capacity for the pile. The pile strength should be reduced by factor γ_n which was presented in Section 3.3.3. Also, a material correction factor, μ_m should be applied on reduced pile strength, it can be calculated according equation 4.21

$$R_d = \mu_m f_{red} \frac{A}{\gamma_n} \qquad (4.11)$$

In equation (4.21) for timber piles $\mu_m = 0.6 - 0.9$ and f_{red} usual takes as 11MPa (Statens Geotekniska Institut, 1993).

4.3 Neutral plane analysis

Regarding equation 3.4 and 3.5 from Section 3.1.1, where soil bearing capacity and raft pressure on contact surface is mentioned according to the LB and UB theory, should be appliable on pile geometry for estimation of bearing capacity with pile toe as a base. By utilisation of the LB and UB theory it is feasible to calculate the toe resistance due to equation 3.4.

In this study the shaft and toe bearing capacity of pile has been reviewed. Of course, result of calculations for toe resistance is so small contrary to shaft resistance it can be neglected. This is depending on shaft area, A_s is larger than the area of the toe, A_b and contribute to bearing capacity of the pile in superior. Equation 4.12 include two mathematical parts of pile load-bearing capacity, one is for the shaft capacity and the other one is for toe resistance:

$$Q = Q_b + Q_s = A_b q_b + A_s \bar{\tau}_s \qquad (4.12)$$

where $q_b = N_c c_u$ and N_c is a constant and is between 6 and 9 depending on soil properties and $\bar{\tau}_s = \alpha c_u$ or $\tau_s = \beta \sigma'_v$ is a shear strength according to the chosen method as it was already explained in Section 4.2.1-4.2.3.

4.3.1 Neutral plane

Most interesting fact according pile movements over time is when pile sinks and interaction of pile shaft and subsoil creates resistance from friction of soil material along the pile. That determines stresses in soil which influence on soil consolidation around a pile. Simultaneously the pile displacement goes slower then displacement in compressed subsoil. This phenomenon creates a negative movement along the pile regarding to the subsoil, so called down drag. As it justified, the shaft resistance of the pile happens according to compression from applied load of the pile only, see Figure 4.8. Action effect presenting soil movements and resistance presenting pile shear strength of pile-soil resistance.

These two forces are located in the same plane and have a diverge direction. In this case, equilibrium can be fulfillled and a plane there a down drag is equal to load effect, as it shown on last picture of Figure 4.8. The total load effect increasing with the depth along the pile and resistance of the pile starts with the magnitude of toe resistance at the toe level then continue increasingly towered the top. By arrangement these forces the Neutral plane (NP) can be found, or more accurate Neutral zone, where the $E \leq R$ and can be consider as start of soil loading. Displacement at this level and downwards will be equal to soil displacement and will increase with the depth regarding to 2:1 theory, see Chapter 2.

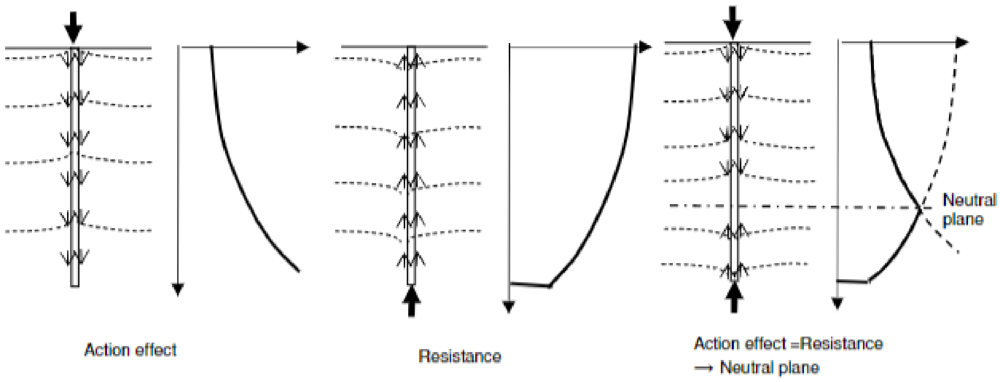


Figure 4.8 Visual presentation of movements in soil along the pile shaft.

Since a friction zone of resistance from NP to the lower part of the pile based on the elastic properties of soil. Upperpart of the pile based on elastic properties of the pile and has significance of the down drag. These criteria of the NP model benefit the settlement redaction since the stability of the lower strata.

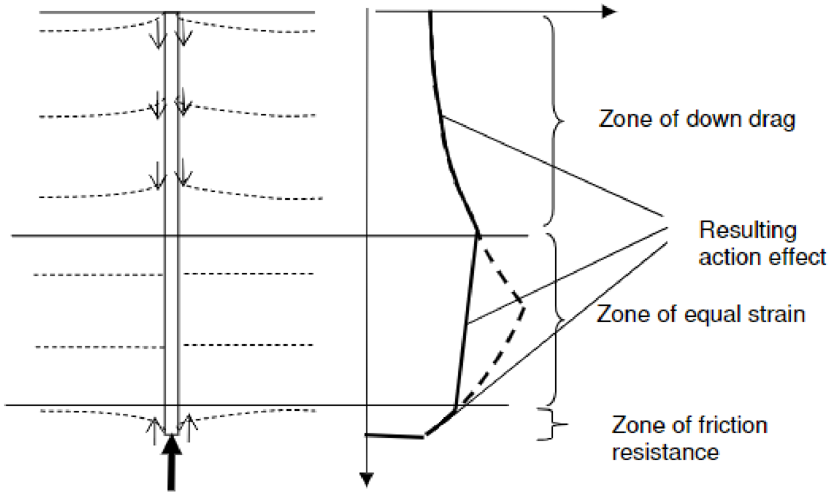


Figure 4.9 Visual explanation of Neutral zone or the zone there forces is close to equal.

For a short-term response NP method can be used for definition of the pile settlements controversial surroundings. Otherwise, establishing settlements for the long-term can be used in the design of foundations. Recommendations are to use the NP for final conditions for evaluation of total settlements.

However, as it was mentioned in the one of the first chapters, see Chapter 2, the failure conditions (ULS) usual does not appear under working loads (SLS). So, in this case the resistance represents the maximum deformation which occur under long time (10 years) in soil under compression and participate as a limitation for design and maximum stress is a stress at NP level.

It is two approaches for evaluate of this level, one is a α-method for undrained soil conditions and second is a β-method mostly used for drained soil properties see Section 4.2.

Furthermore, improvement of bearing capacity in foundation can be achieved by optimal placement of piles or variation of number of piles. Additionally, is to vary length of shaft to increase the shaft area or decrease shaft friction by treating pile shaft with bitumen or colour. (Kempfert el at., 2006)

One of the alternatives is to consider calculation on the group of piles. The group of piles can be reassigned when the piles is situated closer than 5D-8D. It has been noted that the group effect according to American Association of State Highway and Transportation (AASHTO), provides the following guidelines see Table 4.1

Table 4.3 Pile group efficiency according to American Association of state Highway Transportation (AASHTO)

Pile spacing (center to center)	Group efficiency
3*D*	0.67
4*D*	0.78
5*D*	0.89
6*D* or more	1.00
D = diameter of piles	

According to Poulos (1975) and Davis (1980) the Neutral plane analysis was established as rigid pile method and was evaluated both numerical and in a reality by them. The results were sufficient and accomplish conformation of analytical solution. Under these considerations the analysis been agreed of usage on full scale as rigid-plastic analysis of NP estimation and maximum load establishing.

4.3.2 Neutral plane 2.0

According to Matyas and Santamarina (1994) the NP method can be taken to the next level. They claim what the NP method is a unique solution to the many variations of neutral plane mechanism and the traditional method of rigid-plastic calculation is overestimated up to 50 % of a neutral plane depth value. Along with their paper optimization on the NP depth can be made approximative on elastic-plastic behaviour of soil.

As it is mentioned in Chapter 4.3.1 the NP method is a plane of equilibrium, $E \leq R$. The Figure 4.10 shows the rigid-plastic model of NP method. Where Q_{NP} is maximum value of applied load and fulfill $Q_{NP} = Q_d + Q_n$ which represent sum of external load and down-drag force. At the same time the Q_{NP} should not exceed the bearing capacity of the pile R_d see Appendices C.

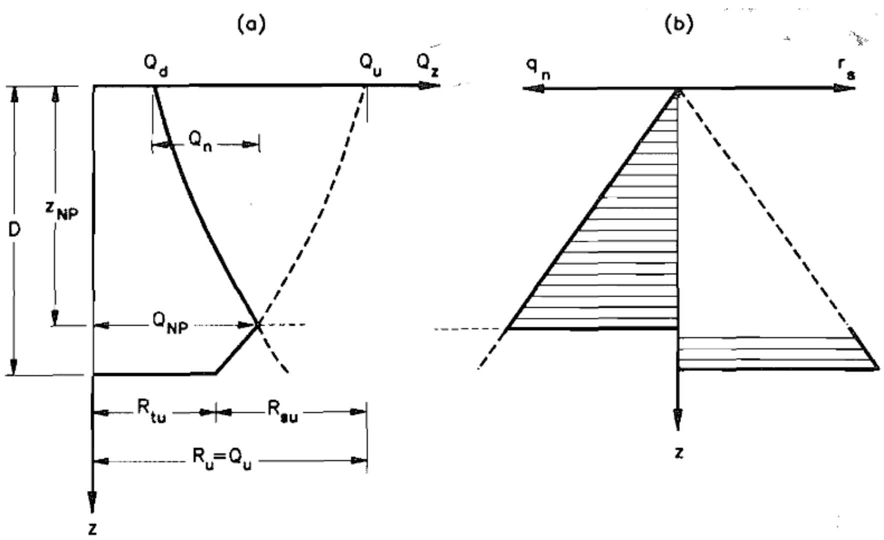


Figure 4.10 Model presentation of rigid-plastic approach shows the load distribution and shaft resistance

The rigid-plastic method based on approximation of linear behaviour of stresses with a depth as unit of shaft bearing capacity, r_s or unit of drag load q_n and can be established by some constant a when $r_s = az$ (Matyas et alt., 1994). So how can constant a be described in mathematical terms?
Let this present as part of elastic-plastic solution based on shaft and toe resistance according to the soil parameters, see Figure 4.11. The pile assumed as a rigid.

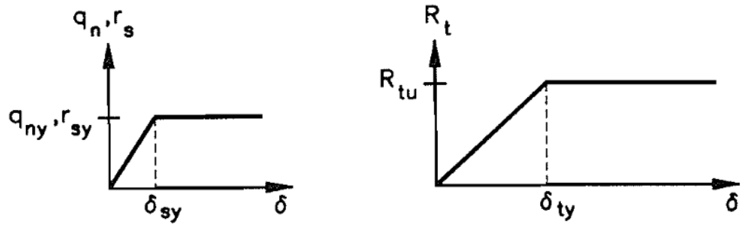


Figure 4.11 The elastic-plastic soil model and behaviour of stresses a) for the shaft b) for the toe, adopted from Matyas et alt., 1994.

The figure shows that displacements remain linear regarding to strength until a yielding point. At this region constant a should be an inclination of a linear line for each part of the pile. In a region after yielding, it became a plastic and r_s and $R_t$will remains constant. It means that constant a is equal to 1.
In this way it can be implemented main description of the modified NP method which is accordantly revolve around the yielding point. Where δ_{sy} and δ_{ty} is yielding displacements for shaft respective toe resistance and became a part of the method. It can be estimated at very initial stage of the design. Nevertheless, these values are already established by experience and field tests and can be assumed according to that see Appendices D.
For establishing the mathematical problem Matyas and Santamarina (1994) presented three dimensionless ratios.

$$\psi = \frac{\delta_{ty}}{S},\ \omega = \frac{\delta_{sy}}{S} \text{ and } \lambda = \frac{\delta_h}{S} \qquad (4.13\text{-}4.15)$$

where S is total settlements of pile and can be evaluated by sum of settlements on the head and the toe, $S = \delta_h + \delta_t$ (4.16)

These three ratios describe evolving of settlements in anticipation of total acceptable settlement S according to equations 4.13-4.16. It can be noted that $\lambda + \psi = 1$ when $\omega = 0$ and this is a criterion for fully evolved mechanism also it can be represented by rigid plastic approach. Though, total displacement profile is assumed as linear (normalised values), see Figure 4.12.

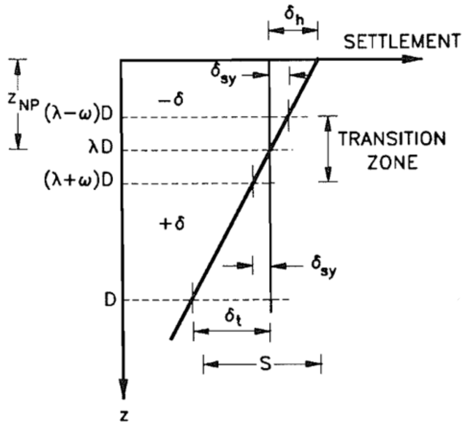


Figure 4.12 Displacement profile according to elastic plastic method with a max displacement at the head of the pile (L=D).

Where a ratio λ can be evaluated from Figure 4.12 as $\lambda = \frac{z_{NP}}{L}$ (4.17)
So, the λ shows placement of NP level along the pile and should vary along the pile length from the pile head to the last point there the rigid-plastic NP occur.

In consideration of presented ratios and Figure 4.12, equilibrium can be established due to equation (4.18).

$$Q_d + \int_0^{(\lambda-\omega)L} A_s\, q_{ny} dz + \int_{(\lambda-\omega)L}^{\lambda L} A_s\, q_{nzm} dz = \int_{\lambda L}^{(\lambda+\omega)L} A_s\, r_{szm} dz + \int_{(\lambda+\omega)L}^{L} A_s\, r_{sy} dz + R_{tm} \quad (4.18)$$

In this equation a right side present the upper part of the pile to NP level and the left side from NP level and down to the toe. Realisation of presenter equations and integration of the equilibrium expression an equation for load at neutral plane have been impressed according to equation 4.19, see Figure (4.12)

$$\frac{Q_{NP}}{R_u} = \frac{1}{\alpha}\left(\lambda^2 - \lambda\omega + \frac{1}{3}\omega^2\right) + \frac{1}{F_s} \quad (4.19)$$

where the R_u is ultimate load and Q_{NP} is maximum load. Also, at the end the Q_{NP}/R_uratio should not exceed 1.

By analysing graphs at Figure 4.13 it is easy to establish that a line of Q_u
will be potential of the pile and line for Q_d shows the reaction of the pile on the external load Q_d. Then with increasing the Q_d the Q_n will increase also and it would lead to rigid-plastic solution of the pile.

Notice what the transition zone is limited by $(\lambda - \omega)L$ and $(\lambda + \omega)L$ and can be estimated by $t_{tr} = 2\omega L$ as it shows Figure 4.12. Also, the maximum load occurs in transition zone and should be checked for joints in this region.

At this point, the transition zone is known and dimensionless ratios ω, ψ and λ also, it is feasible to proceed on establishing of neutral plane by elastic plastic approach and can be accomplished by procedure of evaluation of Q_{NP} and Q_n and settlements from NP level.

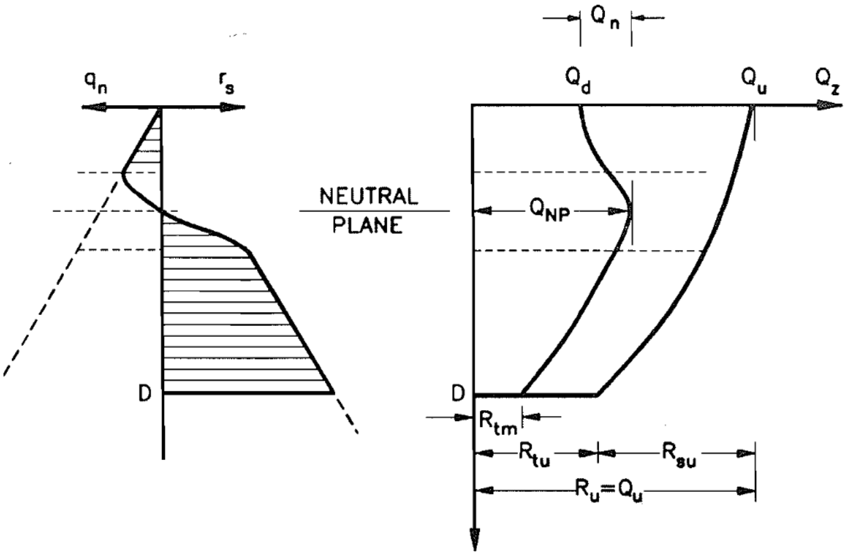


Figure 4.13 Elastic-plastic method visual presentation of soil yielding under NP zone

To accomplish estimation of the Neutral plane according Matyas and Santamarina (1994) the algorithm was established on calculations:

1. Geometry and safety of the model such as penetration depth of pile, L; diameter of pile, d; safety factor F_S, recommended by Eurocode as 1.5-3.
2. Soil properties drained unit weight, γ'; effective stresses along the pile length, σ', resistance at the pile head, R_{tu} and resistance of the shaft R_{su}.
3. Essential settlements values for the yielding determination, such as required movements for soil around shaft for yielding, δ_{sy}, recommended as 3-5mm; displacement at the toe for exceed yielding at this level, δ_{ty}, recommended 5 -10% of the pile cross-section; total acceptable settlements $S = \delta_s + \delta_t$.
4. Estimate dimensionless values ω, ψ and λ according to equation (4.15) and (4.20)

$$\lambda_{NP} = \frac{\sqrt{(\alpha-1)^2+8\psi(\alpha-1)+8\psi^2\left(1-\frac{\alpha}{F_S}-\frac{2\omega^2}{3}\right)}-(\alpha-1)}{4\psi} \qquad (4.20)$$

5. Estimate depth of the Neutral plane, Z_{NP}, maximum load, Q_{NP} and transition zone, t_{trans} according to equations (4.21)

$$Q_{NP} = \left(\frac{1}{\alpha}\left({\lambda_{NP}}^2 - \lambda_{NP}\omega + \frac{1}{3}\,\omega^2\right) + \frac{1}{F_S}\right) R_u \qquad (4.21)$$

$$z_{NP} = \lambda_{NP} L \qquad (4.22)$$

$$t_{trans} = 2\omega L = 2\,\frac{\delta_{sy}}{S}\,L \qquad (4.23)$$

$$Q_n = Q_{NP} - Q_d \qquad (4.24)$$

5 Evaluation and optimization of the design

In this study, the raft model for simulations has been taken from timber housing manufacture. Model of the timber building of 5 floors with a characteristics weight of 10 ton per floor. The concrete raft with a uniformly distributed load of 5x10 tons of self-weight, the accurate calculation on load and load combinations can be found in Calculation.

Raft has been divided according to dimensions of house-module manufacture of volume of one unit and evaluated as a element of the raft. The load and load combinations were applied according to area 4x10 m^2 of the unit fort print and calculated with some adjustments, see Figure 5.1.

To accomplish aim of the study to optimize the piled foundation for lightweight building on soft clay 4 cases was established and investigated with consideration of applied loads and soil behaver

- Modell A has 6 piles and cc=min {4,7m; 3,4m; 5,8m} =3,4m
- Modell B has 8 piles and cc=min {4,7m; 2,9m; 2,77m} =2,9m.
- Modell C has 10 piles and cc= min {2,35m; 2,9m; 3,73m} =2,35m.
- Modell D has 15piles and cc= min {2,35m; 1,45m; 2,77m} =1,45m.

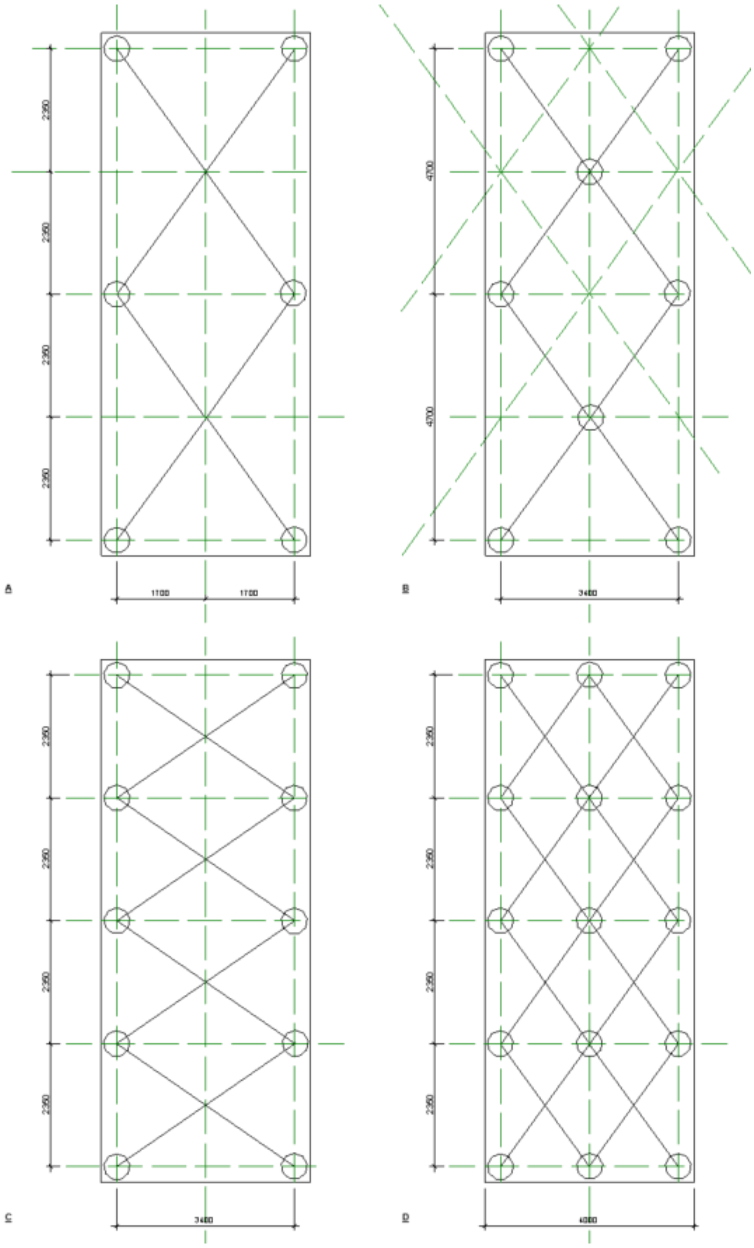


Figure 5.1 Pile plane models for evaluation in this study

5.1 Optimization of design and interaction process

How to optimize design and find the winning concept? It is a process which can be described as three main stages of design algorithm. One is a preliminary design then numbers of piles is roughly estimated and a pile load capacity suggested. Second part is more specific where a type and placement of piles generates. Finale stage of optimization process concludes detailing and improvement on design.
In this way the design process accomplishes several steps and some of them is repeatedly with some common points of iteration process.

Algorithm on design of piled raft foundation:

- Problem visualisation – design criteria
    - Soil properties
    - Ground water level

- Geotechnical category and safety class
    - Load evaluation
    - Acceptable settlements values

- Ultimate Limit State
    - Design soil profile (ULS)
    - Design load and load combination (ULS)
    - Bearing capacity analysis for raft
    - Bearing capacity analysis for single pile and pile- soil interaction
    - Bearing capacity analysis for pile-block
    - Check of global stability
    - Check tilting

- Serviceability Limit State
    - Design soil profile (SLS)
    - Design load and load combination (SLS)
    - Settlements analysis
    - Differential settlements analysis
    - Check dynamical affect

As it shown on Figure 5.2 the iterative process of optimization is repeatedly process which is resulting a winning concept. By improvement of criteria and input data in a model with consideration to environment impact, requirements, costs and time of building process the optimization of the design takes form.

In this process the optimization of a building sector and guidelines can be make same improvements. Regular adaption to a building branch is a main aspect on improvements of codes and guidelines.

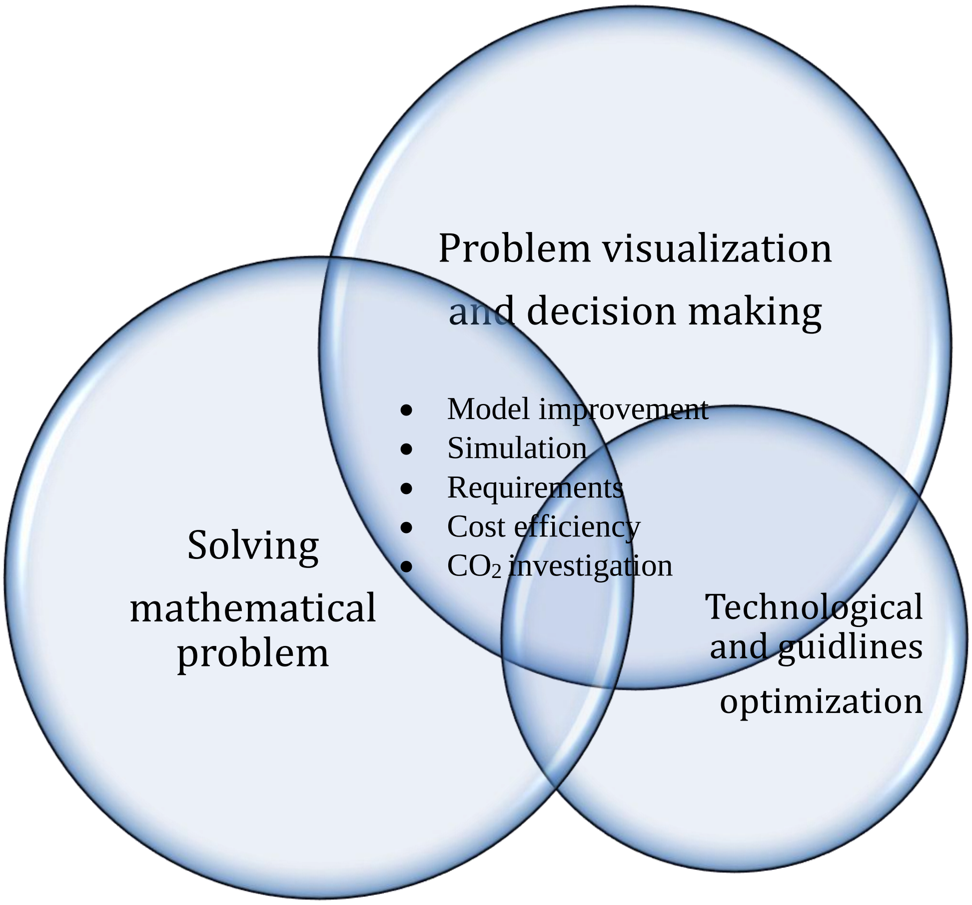


Figure 5.2 Schematic presentation of involving parameters in optimization process of the structural design.

5.2 Optimisation of piled raft with respect to cost

Success of building project depends on time and good planning. Therefore, another part that influences on the building process is money. Costs of the building material, resources, administration etc numbers that focuses over entire building process. Installation of piles is time-consuming and noisy process. It is soil properties and pile material determined how fast and how costly this process will be. Estimation of this costs in generally takes approximative price that relys on previous experience.

One of the problems can be when a soil investigation is not complete, or the soil profile is not made properly or complete because of insufficient soil test. In this case a liner interpolative assumption makes between two bore points. Thus, the soil profile in between can have different layers and can be problem under installation. In this case the pile process can be change under production or the pile type and so own. Relatively this leads to extra expenses and longer contract time.
It worth to notice that load test on piles makes after installation. As it already pointed out in Section 4.2.4 it takes several weeks for pile to rich load capacity due to adhesion of cohesion soil. The process of pile load test is pricey and contribute with one more step in a timetable. Not all of piles must be checked and this happen randomly only 10/100 is assume as a normal estimation. In case of timber piles this process is not appropriate regard to timber tension strength and the safety relies on calculations on load bearing capacity.

More precisely costs are on pile material and production of building materials, and it known from manufactures, then this price is feasible to compare. In Table 5.1 gathered prices from manufacturers on several pile types. Lime/cement pile is here as one additional representant to concrete pile.

Table 5.1 Prices of different types of piles

Pile type	**Length**	**Materials**	**Manufacture**	**Pris**
Concrete pile (SP1)	*14 m*	Steel and concrete and joint	PEAB grundläggning	*900*14+500=13100 kr*
Lime/cement columns, block	*14 m*	Calcium oxide (CaO) and cement (50:50), block	KELLER	*4*(150*14+100)=8800 kr*
Timber pile	*18 m*	Timber and joint	Liljevrå	*1100+350=1450 kr*

Consequently, in further presentation of costs, the timber pile additionally is the cheapest pile and mostly economical concept for the ground construction. It is nearly 6 times less than the pris of lime/cement columns and 9 times less than the concrete pile (SP1).

5.3 Environmental impact from materials production

According to Boverket (2020) Swedish construction sector contribute to environment impact with whole 19 %. Most of that is transportation of materials but largest part is production of building materials: concrete, steel, and timber. To compare this production process and emissions from this process it shows several numbers which was adapted from the Swedish Transport Administration environmental investigation (Trafikverkets klimatkalkyl) and can be seen in Table 5.2.
Obviously, it is many aspects which generate an impact on environment in piling procedure and for that the total Live Cycle Analyse (LCA) must be made. However due to logical aspect it is essential similar for all piles and that's way here just material adjusts. As it shown in Table 5.2 the emissions values from just production of piles respective material.

Table 5.2 Presentation of environments effect from the production of different types of piles

Pile type	**Length**	**Materials**	**Impact**
Concrete pile (SP1)	*6.1m*	Steel and concrete	*26,32 kg CO_2/m*
Lime/cement columns	*6.1 m*	Calcium oxide (CaO) and cement (50:50)	*41,14 kg CO_2/m*
Timber pile	*6.1 m*	Timber	*0,57 kg CO_2/m*

Since the concrete pile is most used piles in Sweden according to pile commission it is important to see the impact ratio for SP1 is *26,32/0,57=46,17* so it is large than 46 times impact on environment. Along with that, production of LCC piles have *41,14/0,57=72,17*, so 72 times larger impact on environment.

Or with other words, 46 timber piles would worth one concrete pile or 72 timber piles compare to one lime/cement column.

5.4 Soil characteristic parameters

For prissily understand of soil behaver, evaluation of soil profile must be made. Constant Rate of Strain (CRS) analysis is based on test where the soil sample exposed for a pressure with constant velocity on one side and electronical device register pore pressure values on the other. Procedure occurs in laboratory environment and registrar deformations, pore pressure and applied pressure on samples. Effective stresses and compression modulus can be plotted in M modulus diagram, see Figure 5.3

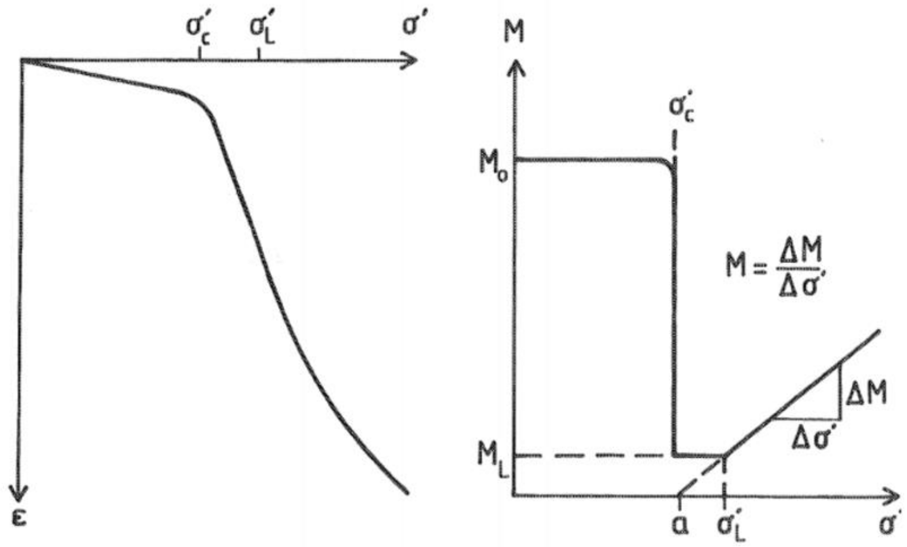


Figure 5.3 Schematical example of CRS result for evaluation of soil effective stresses and M-modulus diagrams

In diagram where the effective stress is not exceeded σ'_c modulus is constant (M_0). Father it is drop down to M_L until effective stress is smaller than σ'_{L}. And into conclusion modulus increase linear regarding to equation 5.1.

$$M = M_L + (\sigma' - \sigma'_L)M' \qquad (5.1)$$

Geotechnic profile can be simplified but oversimplification can lead to wrong estimation of settlement and load affect. Precision and detailing of soil profile is important part of the design and rely completely on responsibility of an engineer.
Complete soil profile evaluation and assumptions on this part is described in Appendices A.

By using the real building site investigation with a bored tests and laboratorial proves and tests brings studies assumptions close to reality. Chosen soil profile is common for Gothenburg and content filled gravel around 1 meter continued with thick layer of clay and the rock on the bottom. Ground water table is around 1,3m and has variations with $\pm$0,2m during the year around. Soil profile of a building site presents in Table 5.3

Table 5.3 Established soil profile from the laboratorial investigations from the A Working Lab construction side

Parameters	**Depth z, (m)**	**Unit weight γ, (kN/m^3)**	**Water content, %**	**Undrained shear strength c_{uk}, (kPa)**	**Friction angle ϕ, ° drain conditions**	**Young's modulus E_s, (kN/m^2)**
Surface (asphalt)	0,05	-	0	20	30	-
Gravel	0,05-1	19	0	20	30	80000
Sandy clay	1-1,3	18	18	20	32	25000
Sandy clay	1,3-3,2	18	31	20	32	25000
Turf	3,2-3,8	12 (γ'=2)	127-479	15	28	5000
Silting clay	3,8-5,2	17	37	15+1,4z	30	4000
Clay	5,2-26 or 5,2-84	18	32	15+1,4z	30	4000

Nevertheless, the soil profile with a clay magnitude less when 40-50 meters end-bearing piles is perfect solution. Beside in this study investigation of a floating pile is of most interest. Hence, why the design will be if the soil profile does not have any rock-bottom and the clay layer is continuous to 65 meters down?
Level above water content with a depth 0-1.3 is a drained soil and absent of cohesion. Usually, undisturbed soils are normal consolidate but in centrum of the big metropolitan is overconsolidated. Clay has unit weight 17 kN/m^3 until +39 and 18kN/m^3 below +39. Tests and laboratorial investigations also detailed soil profile evaluation from CRS test presented in appendix A

6 Results

All details on calculations which has been made can be found in Appendices, see Appendices. All results divided in the 4 cases.

Case1: Investigation of the raft both in ULS and SLS.
Case2: Investigation of single pile in ULS and SLS.
Case3: Investigation of piles by rigid-plastic Neutral plane approach.
Case4: Investigation of piles by elastic-plastic Neutral plane approach.

6.1 Input criteria for analytical investigation

Variation of a geometry in the design where the pile length varies between the joints. Since the timber pile is 18 meters long and concrete is 14 meters it can be suitable to study tree different lengths for each pile model just for avoiding an extra joint. Thus, the length was investigated is 18m, 36m and 54m and 14m, 28m and 42m respectively pile material. Other variety in the investigation is numbers of piles and placement under the raft with variation of c.t.c space. By assuming that external loads from the building is uniformly redistributed along the raft area, for simplification of investigated model piles have been strategical placed in different pattern, as it shows on Figure 5.1.

Other criterion for investigation that the raft assumed is rigid. Also, it already been mentioned in Section 2.2.4 this assumption is not totally correct and has to be corrected with a factor μ_r.

In Chapter 5 discussion on applied loads resulted assumption on loads from the manufacture and it been checked by simple calculations rendered from the EC 1.
Results of load calculations can be obtained in Table 5.4.

Figure 6.1 Load and load combinations

	Self-weight (unf), kN/m^2	Self-weight (f), kN/m^2	Total (unf), kN/m^2	Total (f), kN/m^2
ULS:STR/GEO	22.4	20.1	42.3	39.5
SLS	22.4	22.4	23.5	22.4

Design procedure was followed algorithm which is presented in Section 5.1 for each model and length and pile material.

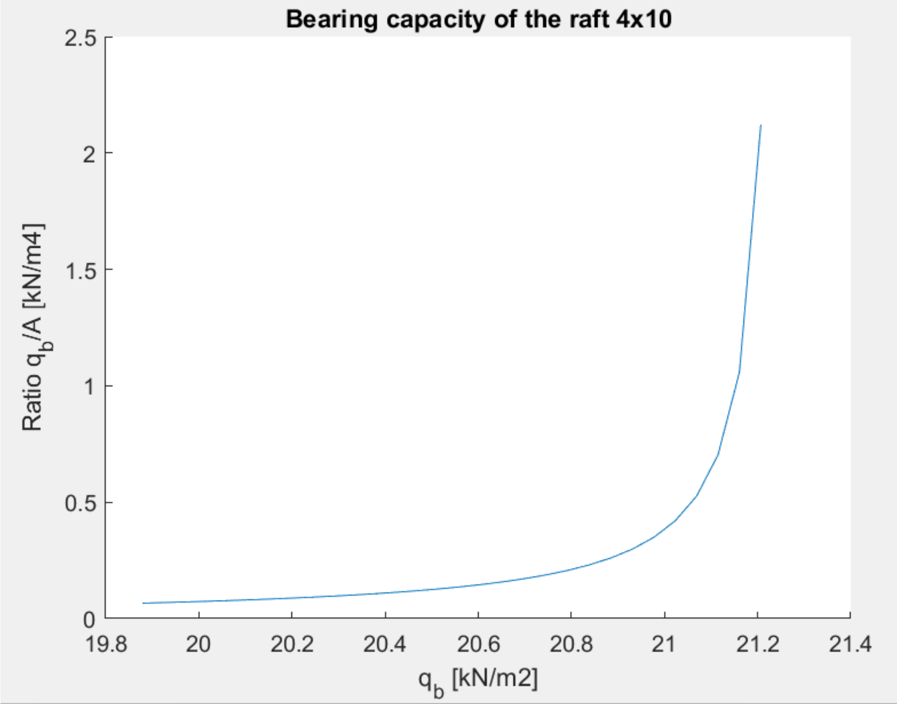
6.2 Outcome from analysis on bearing capacity – ULS

Case 1: In consideration to rafts bearing capacity, the design of bearing construction has partial potential. Also, the raft which is placed directly on ground without any basement has a potential of carrying applied load along entire area. By using presented bearing capacity analyses in Section 3.1.1 result was established that the raft of area 4x10m^2 has capacity of 53.5kN/m^2 placed on the gravel. Since analyses based on surrounded soil property and not consider total soil profile properties the evaluated value of load bearing is not entirely true. By performing same analyses on the raft but with a soil property similar to the soft clay the value of raft capacity change to 20kN/m^2 which is 2.5 times less than the previous result.

In first case where capacity is greater it seems what the raft can take it entire load. Next case is where the raft can take up to 46% of the total load and can be included into design.

1) $53.5kN/m^2 > 43kN/m^2$
2) $20kN/m^2 < 43kN/m^2$

In this case for detailing stage of the design the rigidity of the raft has to be checked.



Case 2: By comparing results of calculations on single pile bearing capacity for existing soil profile for both concrete and timber it is notable differences and similarity on both types. Piles have been chosen for evaluation by one length at the time since the longest possible for concrete is 14 meters and for timber is 18 meters. This choice was made according manufactures as simply as it is due to treat piles under transportation and installation. According to this aspect piles with longer dimension should have joint each manufacture length. Therefore, calculations been made per pile.

For short time calculations only resistance for timber pile with a length of 18 meters is not compatible with the model A, see Figure 5.1.

For the long term just concrete pile of length of 14 meter is not compatible with a model A, see Figure 5.1. As it shows in Table 5.4 of results the ration $R_{d,timber}/R_{d,concrete}$ for one pile is close to 1. This means what the timber pile exceed capacity with 29%

$$\frac{R_{d,timber,1}}{R_{d,concrete,1}} = \frac{318.5}{245.7} = 1.29$$

There variance shows in the longer piles and deeper soil penetration. For the pile with two manufacture length the ratio lead to 43% of increasing in bearing capacity of one pile

$$\frac{R_{d,timber,2}}{R_{d,concrete,2}} = \frac{956}{664.4} = 1.43$$

Finally, longest flouting timber pile has also 43% more capacity than concrete pile
Hence the timber pile is longer in manufacture length which gives superior shaft resistance and greater shaft area. Apart from this, cross-sections of piles distinguish from each other therefore the resistance of timber pile vary non-linier from concrete.
Another aspect should be considered what timber pile has bigger cross-section area at one end so if the pile is turned with the root down the shear stresses develops more rapidly. This phenomenon can be revealed in timber piles of 36 and 54 meters. Even if the average diameter of the timber pile is 0.25m and the concrete one side of the pile 0.235m (SP1) it gives similar results on shaft resistance. At the Table 5.4 shows in column 3 the calculated strength capacity of the single pile according α-method.

Case 3: In columns 4-7 shows calculation on piles depending on the applied load and with interaction with surrounded soil. Comparing by total capacity of the pile shows that the patterns in Models A-D is not fulfill fully capacity of piles. Pile with one length in Model A for both timber and concrete have 100% strength. However, the pile with 3 lengths in Model D have only 50% of pile capacity for both materials.
It can be noticed what concrete pile and timber has similar strength and differ just because of the length and diameter of the pile.

Table 6.4 Result of analysis on single piles bearing capacity for concrete and timber pile by α-method

Pile length	**Material**	***max Rd*** [kN] long term	*max **Rd*** [kN] Model A	*max **Rd*** [kN] Model B	*max **Rd*** **[**kN] Model C	*max **Rd*** [kN] Model D
14m	concrete	245.7	245.7	184.8	184.8	142
28m	concrete	664.4	450	380	381	337
42m	concrete	1264	761	690	660	629
18m	timber	318.5	318.5	222	222	179
36m	timber	956	615	540	540	500
54m	timber	1810	1019	936	937	922

It should be mentioned that shearing bearing capacity between piles as a pile group depend on c.t.c space and can be neglected if it is bigger than 8D, (Randolph, 1994).
According to that the space between piles should not exceed 2m. Along with that and the c.t.c for determined models presented in Chapter 5 it can confirmed what just Model C and Model D which is fulfill these criteria and can be obtained as a pile group.
By including the affected area depended on the c.t.c. dimension in each presented Model A-D and applying the Neutral plane analyses it can be observed different values from the previous number which was obtained from α-method, see Table 5.4.
Initially effected area for each and one model has been done and included in the next evaluations. As it shows on the Figure 5.5 it is some diversity in the applied load area and can be seen as the rectangle in model A, C and D. For model B it is a rhomb nevertheless the area of B and C is almost equal. Therefore, the bearing capacity of these two models is equal too.

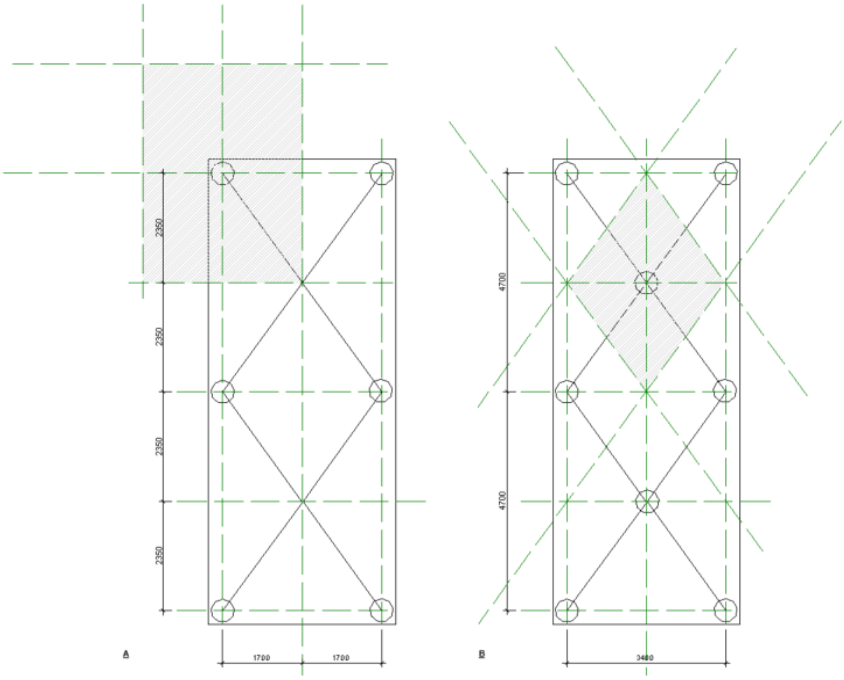


Figure 6.2 Effected area of applied load on single pile (group effect).

Hence entire analysis based on geometry of construction and soil properties where not such a substantial disparity in pile type choice. Therefore, in ULS for bearing capacity of the ground construction the timber pile is more suitable with consideration of cost and environmental aspects.

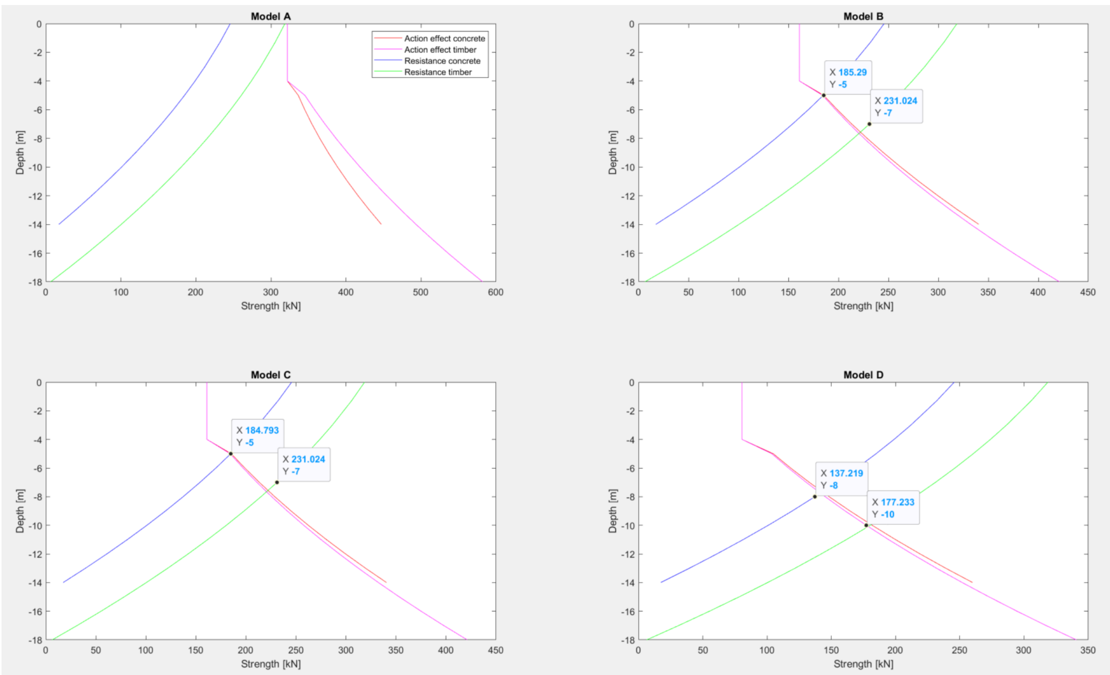


Figure 6.3 Down drag load on pile with a full manufacture length.

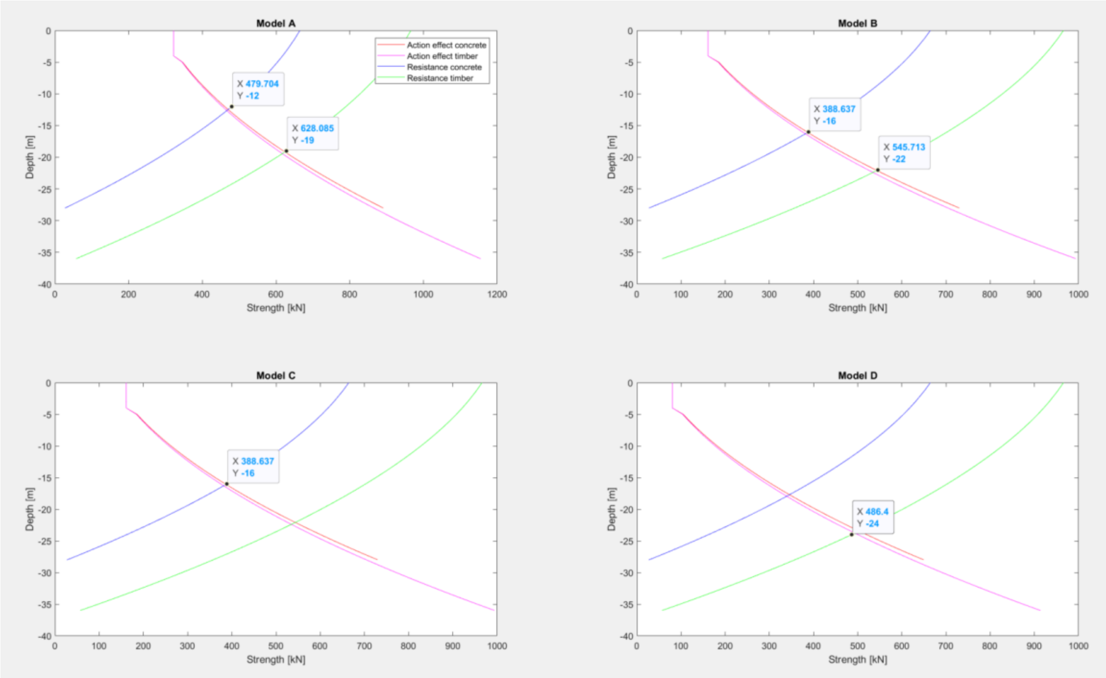


Figure 6.4 Down drag load on pile with a double manufacture length.

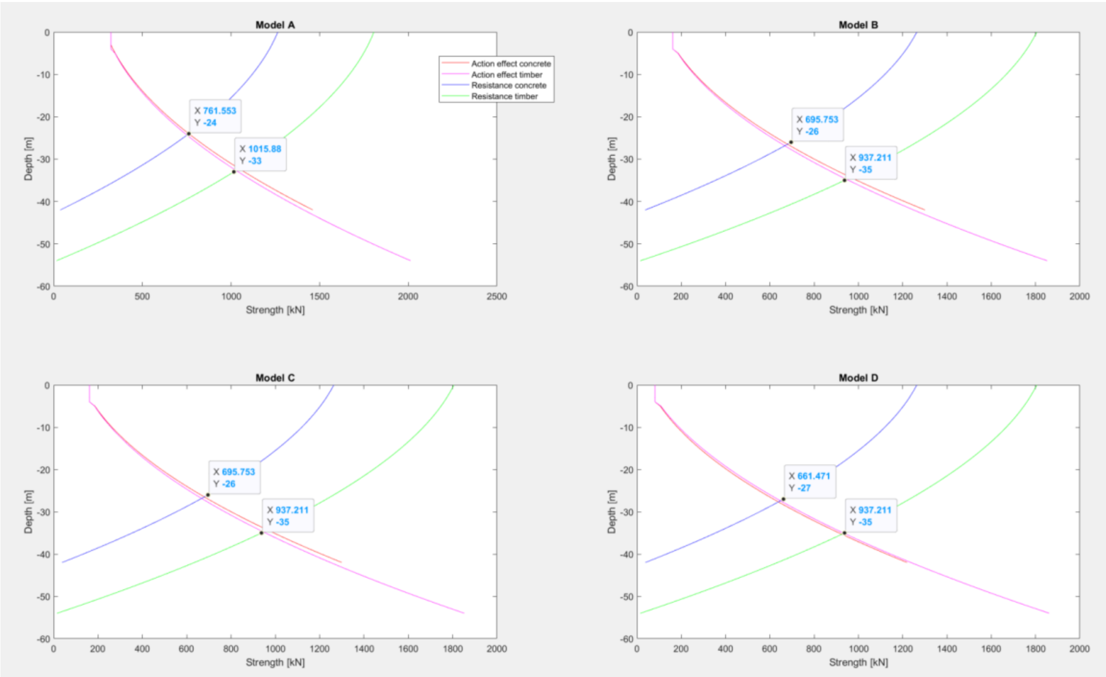


Figure 6.5 Down drag load on pile with a double manufacture length.

Case 4: The algorithm on calculations of the elastic plastic Neutral plane (EPNP) been followed see Section 4.3.2. Result of Neutral plane level, z_{NP} and maximum load, Q_{NP} have been established, see Table 5.5.

Table 6.5 Result of analysis on single piles bearing capacity for concrete and timber pile by α-method

Maximum load and depth of Neutral plane	**Pile length**	**14 m**	**18m**	**28m**	**36m**	**42m**	**54m**
Model A	Q_{NP} [kN]	426	501.4	720	1018	1208	1876
PR	z_{NP} [m]	8.5	10.8	17	21.21	24.65	31.6
Model A	Q_{NP} [kN]	286	338	487.8	691	821	1276
EP	z_{NP} [m]	5.6	7.3	11.2	14.4	16.8	21.6
Model B	Q_{NP} [kN]	263	338	557	855	1044	1713
PR	z_{NP} [m]	8.5	10.8	16.6	21.2	24.6	31.6
Model B	Q_{NP} [kN]	177	228	377	580	710	1165
ER	z_{NP} [m]	5.7	7.2	11.2	14.4	16.8	21.6
Model C	Q_{NP} [kN]	263	338	557	855	1045	1713
PR	z_{NP}[m]	8.5	10.8	16.3	21.2	24.7	31.6
Model C	Q_{NP} [kN]	176.6	228	377.3	580	710.2	1165.5
ER	z_{NP}[m]	5.7	7.25	11.2	14.4	16.8	21.6
Model D	Q_{NP} [kN]	181	257	475.3	773.5	963.3	1631
PR	z_{NP}[m]	8.5	10.8	16.7	21.2	24.7	31.6
Model D	Q_{NP} [kN]	122	173	322	525	655	1110
ER	z_{NP}[m]	5.7	7.25	11.2	14.4	16.8	21.6

In conclusion, result from both methods governed in the table above have been plotted and presented in diagram see Figure 6.6.
Depth of neutral plane is less for elastic-plastic method with 10% for 28meters long pile. Under these circumstances shorter pile can be used without any compromising on strength. It has to be mentioned that elastic-plastic analysis relies on the total accepted settlements. This can be considered in the initial design of ground construction.

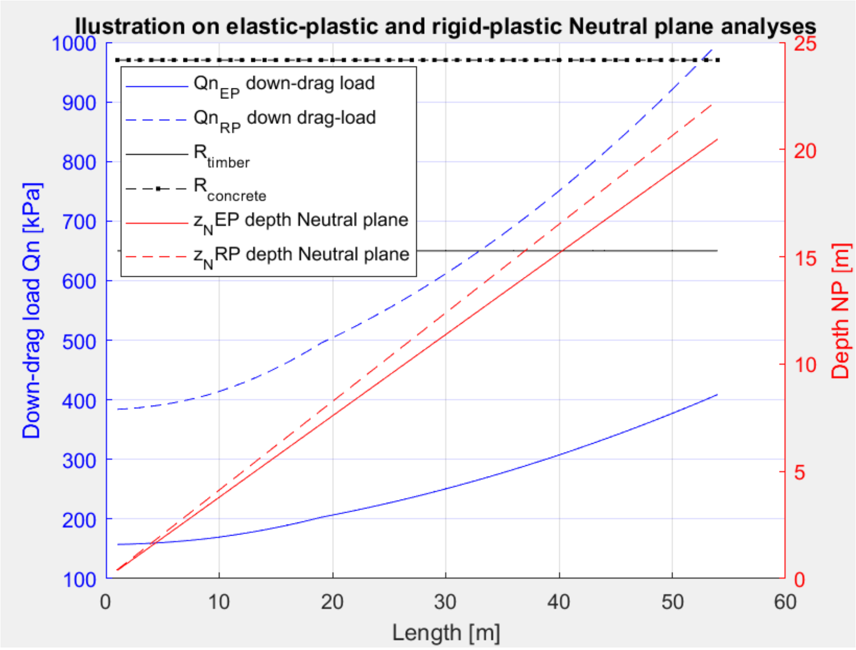


Figure 6.6 Results on elastic-plastic and rigid-plastic neutral plane analysis

6.3 Settlements – SLS

Settlements for each model and pile length is evaluated and presented response by case and pile type on working load action, according to Figure 5.1

Case 1: As it was presented in Section 5.1 the global stability and tilting have to be included into design. By simple calculations on the moment both favourable and unfavourable effecting raft by self-weight loads, it can be confirmed what the house due to light weight has greater tilting when the self-weight.

$M_{tilting} < M_{Rd}$

In similar analysis calculations on sliding was checked and established what the horizontal force was accepted and no sliding in construction is detected.

$H_{wind} < H_{Rd}$

Next step was to check the settlements of the raft. The analysis based on CPT test and assumed on long term. The analysis was evaluated by De Beer in 1965 and used since that. Analysis is including affected stresses which was evaluated under the CPT test, see Appendices B.

Iven the raft has a god bearing capacity on this own it is not the case with settlements. By settlements of the raft for the long term been established as 0.65m, see Figure 5.9 where the accepted settlements according to regulations is maximum 0.5m and it still quite big settlements. Since it is 1.4 times greater than requirements, result on this accusation is to establish more stability in the construction by adding piles in the design.

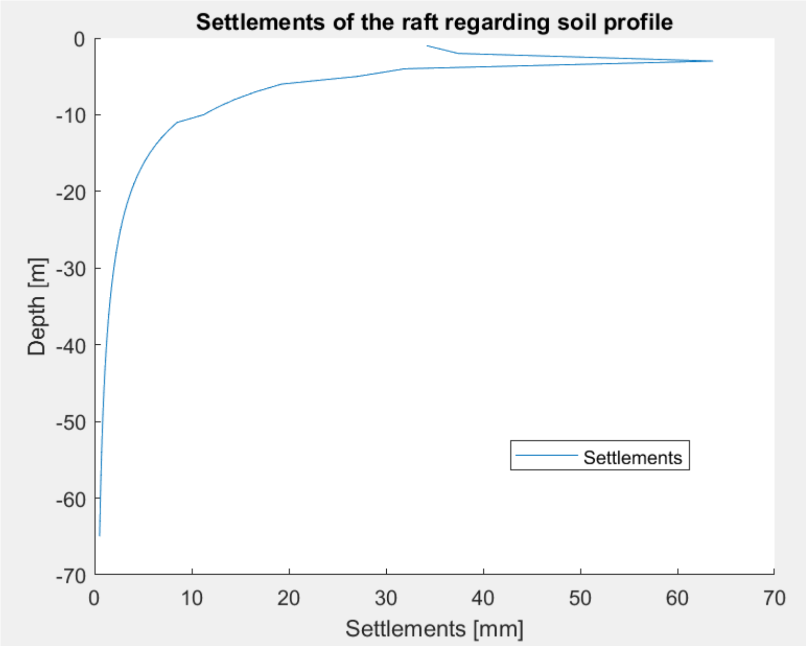


Figure 6.7 Settlements of the raft (raft-soil interaction).

Case 2 and 3:
By evaluating Neutral plane for each pile, it reviles the down drag load though it is repositioned stresses from the surface to the neutral plane depth. In this process regarding stress transformation to the NP the settlements occur on the deeper level of underground. By calculation of stresses from NP and below the total settlements was evaluate and can be obtained in Table 5.6

Table 5.6 Settlements for piled raft with timber and concrete piles and variety in the length

Length\Case	**Case A [m]**	**Case B [m]**	**Case C [m]**	**Case D [m]**
Concrete pile *L*=14m	0,3944	0,2319	0,2319	0,1793
Concrete pile *L=28*m	0,0906	0,0752	0,0746	0,0706
Concrete pile *L=42*m	0,0424	0,0375	0,0375	0,0370
Timber pile, *L=18*m	0,5140	0,1623	0,1623	0,1363
Timber pile, *L*=36m	0,0571	0,0494	0.0509	0,0486
Timber pile, *L*=54m	0.0254	0.0235	0.0236	0.0238

A longer pile where less differences on the settlements

Case 4:
Regarding Matyas and Santamarina (1994) and equation (4.13-4.15) the settlements of pile head can be estimated according to Table 5.7

Table 5.7 Settlements of pile head regarding modified NP approach

Length\Case	***L*=14m**	***L*=18m**	***L*=28m**	***L*=36m**	***L*=42m**	***L*=54m**
Pile settlements as a block	0.2122	0.1491	0.0746	0.0793	0.0722	0.049

Settlements regarding the modified neutral plane approach has bigger settlements then in rigid plastic method. This was expected because the neutral plane is higher.
As it shown in Table 5.7 settlements decreases linear to length of a pile and this was expected also.

7 Discussion and conclusions

- **Soil profile and investigated model**

Soil profile was choused is a typical for pre-coastal environment. Therefore, the design was accomplished according to parameters of clay levels. For design at another construction site should calculations be remade and adjusted to regarding profile. Also, in this study the clay magnitude was chosen to 65meters, and it is affected by settlements for this investigation. Therefore, it should be recalculated for profile with a difference in depth. The soil profile is one of major aspects effecting on construction chose and should not be neglected.
The model was chosen regarding to structural aspects there the applied loads can be taken as uniformly along the entire area of house element. By choosing other types of timber construction the line and point loads should be considered and calculated on the edges and in middle of the raft for biggest deflections.

- **Result evaluation according to material choice:**

Because the study's refined aim is to establish possibility of using timber piles as a part of bearing construction for light weight buildings on soft soil, it seems that the result is quiet positive.

Timber piles is a good contender for replacing the commercial concrete pile both in economical aspect and ecological as it was presented in Sections 5.2 and 5.3. As it was concluded in these sections, the timber pile of 18 meters is 8 times cheaper than the concrete pile (SP1). Also, as it was established in structural calculations the concrete pile strength is only 1.4 times less than the timber pile. Nevertheless, the timber pile has better geometry due to length and cross-section area which results less settlements.

Also, it is a material which is easier to produce and transport to the construction site. In Sweden the timber branch is quite wide-ranging and has a good possibility to accomplish almost any type of timber product. On contrary it is great dilemma on production of a cement and nowadays brings some uncertainties on goals for the future.

The major challenge is to overcome the tendency of concrete and steel piles in construction designs for lightweight buildings. It can spare many economical aspects of building construction branch in the future.

- **Behaviour of floating piles according to Neutral plane analysis and modified Neutral plane analysis**:

According to the results that was gained from the Neutral plane analysis both as α-method and β-method and modified elastic plastic Neutral plane analysis shows decent results.
For the model A the depth of neutral plan has been decreased by 46% and in model D it is up to 49% also a redaction of down drag load occur according to NP level escalation. The results implicate that the Q_n is smaller in the EP analysis when in RP analyses. Therefore, is bearing capacity is increasing regarding to equation (4.24).

In this way it can be possible to consider implementation of more environmental piles such as timber in the design. Nevertheless, the limitation of structural capacity parallel to the grain have to be applied in this design.

After all, the timber piles in comparison to the concrete piles has 35-40% less of bearing capacity. For example, to obtain the same load by timber pile such as one concrete pile accomplish it takes to elongate the pile with almost 11.5meters. Even so the aspects according to economic and environmental effect conclude mush greater value then the other materials.

- **Maintenance of the timber piles under installation and Life Cycle:**

According to the ground construction team from Aarsleff the timber piles is the easiest material to manage under installation. Timber piles is an elastic material and has a higher cracking resistance parallel to the grain then the concrete pile. Therefore, it is easier to transport and uplift the timber pile during installation.

As it was mentioned in section 4.1.2 the main reason is of do not using the timber pile as a commercial alternative for building construction is the sensitivity of the pile to biological attack in saturated soil. The issue can be avoided as it was mentioned in Section 4.1.2 by preservation or replacement of the upper part of the concrete pile, see Figure 4.1. Although the pile can be placed under ground water level in this case it can proved more than 100 years of usage according research and experience.

- **Geometry of the piled raft and detailed design**

Concrete raft could bear 46% of external load and it should be added in the design. In this way the less piles is required and can be placed along the edge of the raft. Deflections in the raft should be avoided and designed reinforced should be adjusted according to the guideline recommendations. In this way the raft will be optimized regarding to economical aspects.
It is difficult to talk about environmental aspects for the entire building but if timber piles will be used even as 2/3 for ground stability amount it can lead to 3 000% less CO_2 emissions just for material choice. Of course, then it has to be calculated emissions on all process is go through during building construction. And still my estimations on more environment friendly design.
All the structural design consists of uncertainties and should be controlled by several methods perhaps numerical analysis. The biggest risk remains then the misunderstanding of parameters or the combination of those do not include in the design.

8 Conclusions – design recommendations

In the current project the feasibility of timber piles for light weight constructions, such as timber buildings, have been investigated for a soft soil profile with background settlements. The results indicate that, when properly designed, timber piles can be used for this soil and these loading conditions without triggering excessive settlements or exceeding the structural capacity of the pile material.

In addition, the research demonstrated that a revised design method for the calculation of the location of the neutral plane that is based on more realistic elastic-plastic soil-structure interaction proved to be more effective in fully utilising the potential of floating piles in soft soils. The latter especially is important for timber piles that typically have lower rated structural capacity. For the case studied and methods employed, timber pile lengths could be increased by up to 10 m within the structural and geotechnical constraints.

Furthermore, timber piles are a renewable and lighter material, that in many cases are more suitable for light weight structures and easier to handle at a construction site.

Finally, at today's price point, timber piles are eight times less costly compared to prefabricated concrete piles and have substantially less environmental impact, i.e. over 46 times less CO_2-eq.

In conclusion timber piles are a viable option that needs further consideration for future Civil Engineering projects.

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Appendices

Appendices table - Results of model calculations:
(based on Section 5.1)

- Soil profile and properties – *Appendices A*
- Ground water level – *Appendices A*
- Geotechnical categorisation and safety class – according EC
- Load evaluation – Load and global stability (MATHCAD)
- Acceptable settlements values – according to EC the total settlements should not exceed 0,5m

- Design soil profile (ULS) - *Appendices A*
- Design load and load combinations (ULS) – *Load and global stability (MATHCAD)*
- Bearing capacity analysis for raft – *Appendices B and Matlab calculations*
- Bearing capacity analysis for single pile and soil-pile interaction – *Matlab "NP calculations"* MATHCAD *"Single pile bearing capacity"*
- Bearing capacity analysis for pile-block – *hand calculations (Matlab)*
- Check of global stability – *Load and global stability* (MATHCAD)
- Check tilting – *Load and global stability* (MATHCAD)

- Design soil profile (SLS) – *Appendices A*
- Design load and load combination (SLS) – *Load and global stability (MATHCAD)*
- Settlements analysis for the raft – *Appendices B and Raft capacity* (MATHCAD)
- Settlements for the pile – *Appendices* C *and Matlab*

- Improved Neutral plane analysis and optimisation of the model - *Appendices C* (MATHCAD)

Appendices A

Soil profile – A Working Lab a building side in centrum of Gothenburg

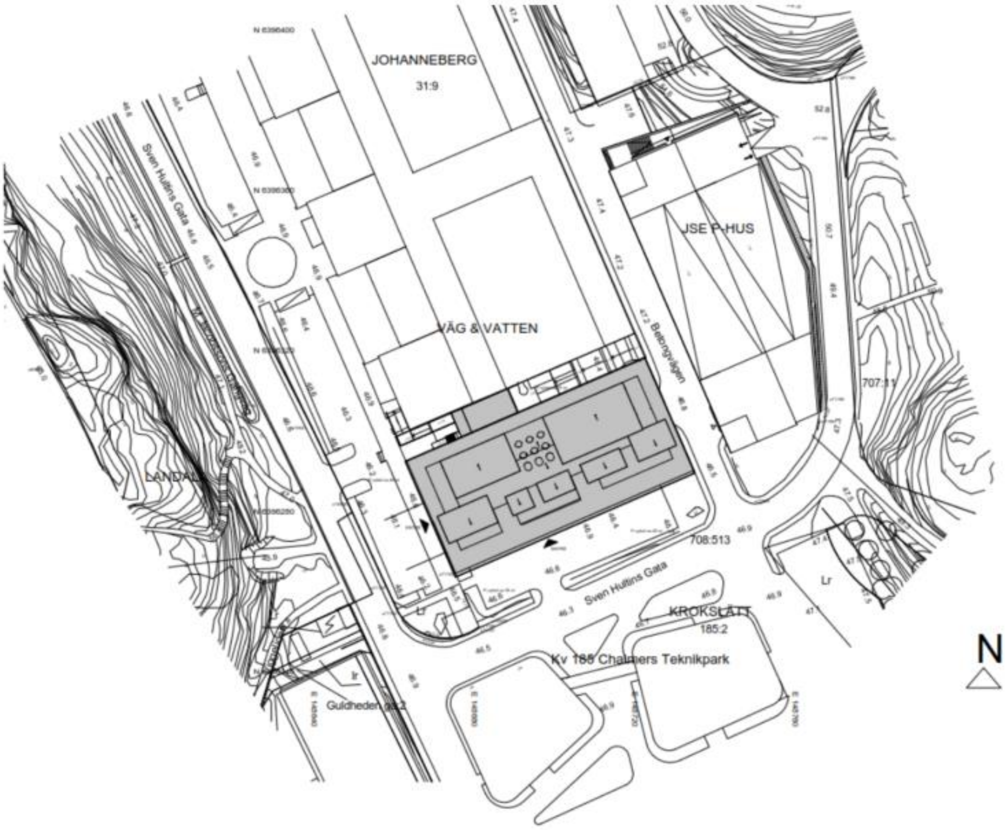

Figure A.1 Site map for Chalmers campus Johanneberg, A Working Lab.

The soil profile evaluates from geotechnical test on side. It was made 8 bore holes and performed 3 JB tests (Soil-Rock Test), 1 In situ test, 2 CPT (Cone Penetration Test) tests, 2 Vane Test, 7 lab tests on the soil layers.

Building sides tests establish that the ground is normal consolidate and ground water table varied on the level of 1,3m with ±0,2 m below the surface.

Parameters was estimate from Cone Penetration Test (CPT) which is shown in Figure A.3. Readings and estimations can be obtained in Table A.1.

All criteria for the existing building are obtained in the column “A Working Lab”. Since the building remains on the ground with a depth of 26 meters to the rock it is normally conclude the end-bearing piles there the stress redistribution will be mainly on the toe of piles. But how about sides where clay magnitude is much bigger and requirements of the bearing capacity of the pile?

Thus, the new dimension for the calculations is chosen as continues clay level and the pile length is chosen as 3 times larger than the previous.

Office building: A working lab	*A Working Lab*	*Estimation*
Length raft, L(m)	10	10
Wight raft, H(m)	4	4
Self-weight of the building, G (kN/m^2)	200	200
Variable load, Q (kN/m^2)	13,8	13,8
Wind load, W (kN/m^2)	12,6	12,6
Raft thickness, t(m)	0,25	0,25
Well-compacted granular, n (m)	0,5	0,5
Ground water, GW (m)	1,3	1,3
Depth to the rock, z (m)	26	65
Water content, w (%)	30-60	30-60
Undrained shear strength dry crust, c_u(kN/m^2)	20	20
Coefficient of consolidation, c_v(m^2/year)	1	1
Consolidation time, t_p(days)	7 & 10^9	7 & 10^9
Undrain shear strength clay, τ_{cu}(kPa)	τ_{cu}=15+1,4z	τ_{cu}=15+1,4z
Correction factor, μ ($c_{u=}\mu\tau_v$)	1	1
Shear strength drain long term analysis, c_k´	0,1c_{uk}	0,1c_{uk}
Geotechnical class, GK	2	2
Density/unit weight of clay over +39, γ(kN/m^3) under +39, γ(kN/m^3)	 17 18	 17 18
Friction angle drained conditions clay ϕ’, ˚	30	30
Modulus M_L (kPa)	900	900
Modulus M´ (kPa)	13	13
Elastisity modulus, E_s	Table A.2	Table A.2

Table A.1 Soil parameters and criteria estimated from “MUR Johanneberg science park etapp 2” and “Geoteknisk PM, Johanneberg science park etapp 2”, all values in the table gives as characteristic value.

CPT test is not the mostly correct, although it is the test which gives approximately strength distribution field with depth and described the soil main parameters for settlements evaluation. And with combination with laboratories studies, it feasible to calculate the undrained shear resistance out of the equation (A.1)

$$\tau_{cu} = \frac{q_T - \sigma_{v0}}{13.4 + 6.65 w_L} \qquad (A.1)$$

Due to this equation the undrained shear strength is calculate to equation τ_{cu}=15+1,4z as variety through the depth z. In the geotechnical report (MUR) soil was represented as normal consolidated. Since of cohesive material the hydraulic conductivity is relatively low and effective stresses occur only under long term situation. In this case the OCR = 1 because of the normal consolidation status in soil and total stress analysis on cohesive material must be done under undrain conditions the proposition $c_{u=}\mu\tau_v$ will be correct, where $\mu=\tan\phi=1$. (Bejrrum, 1972)

BILAGA 4:1

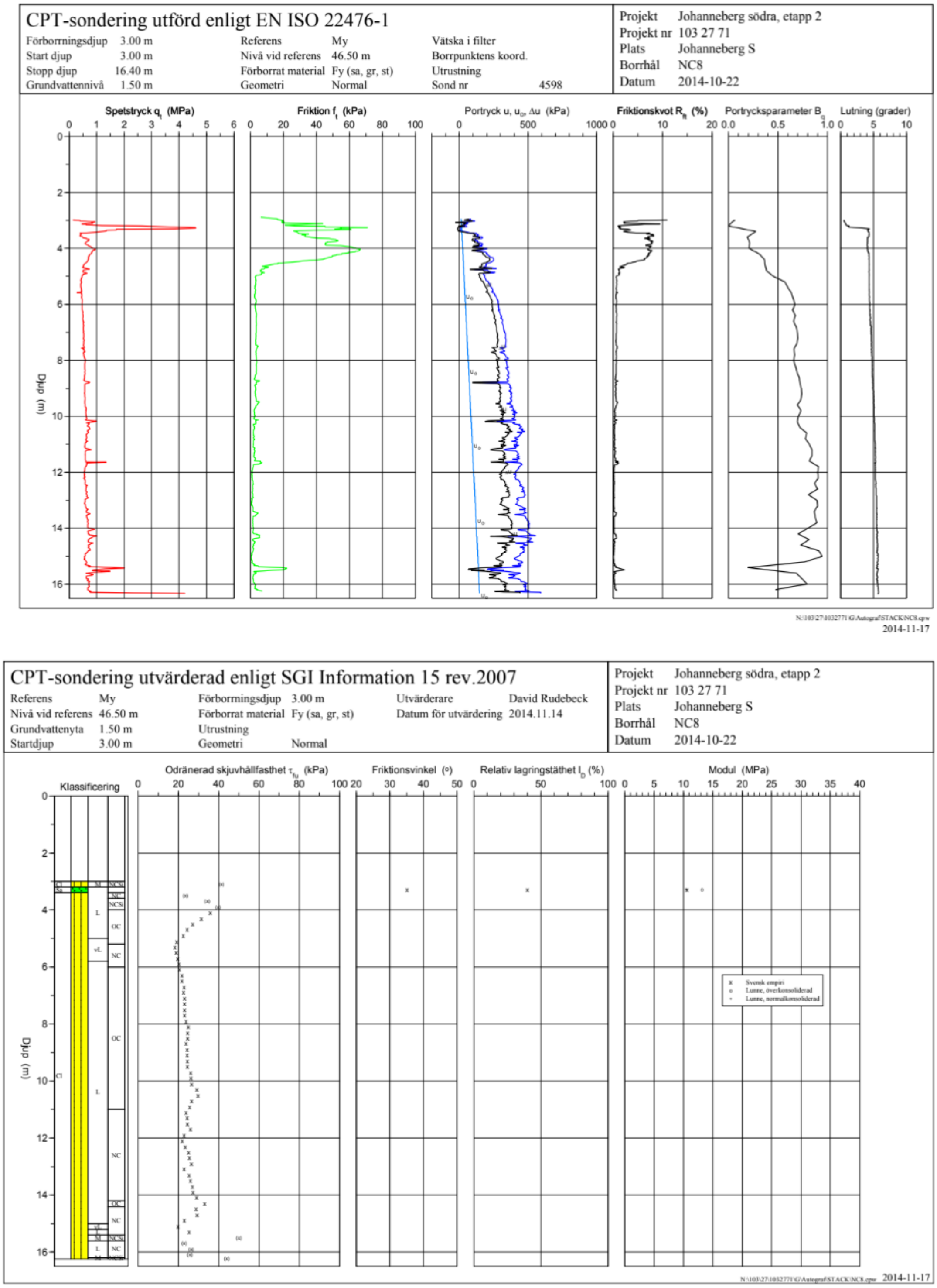

Figure A.2. Cone Penetration Test for evaluation of undrain shear strength made 2014.

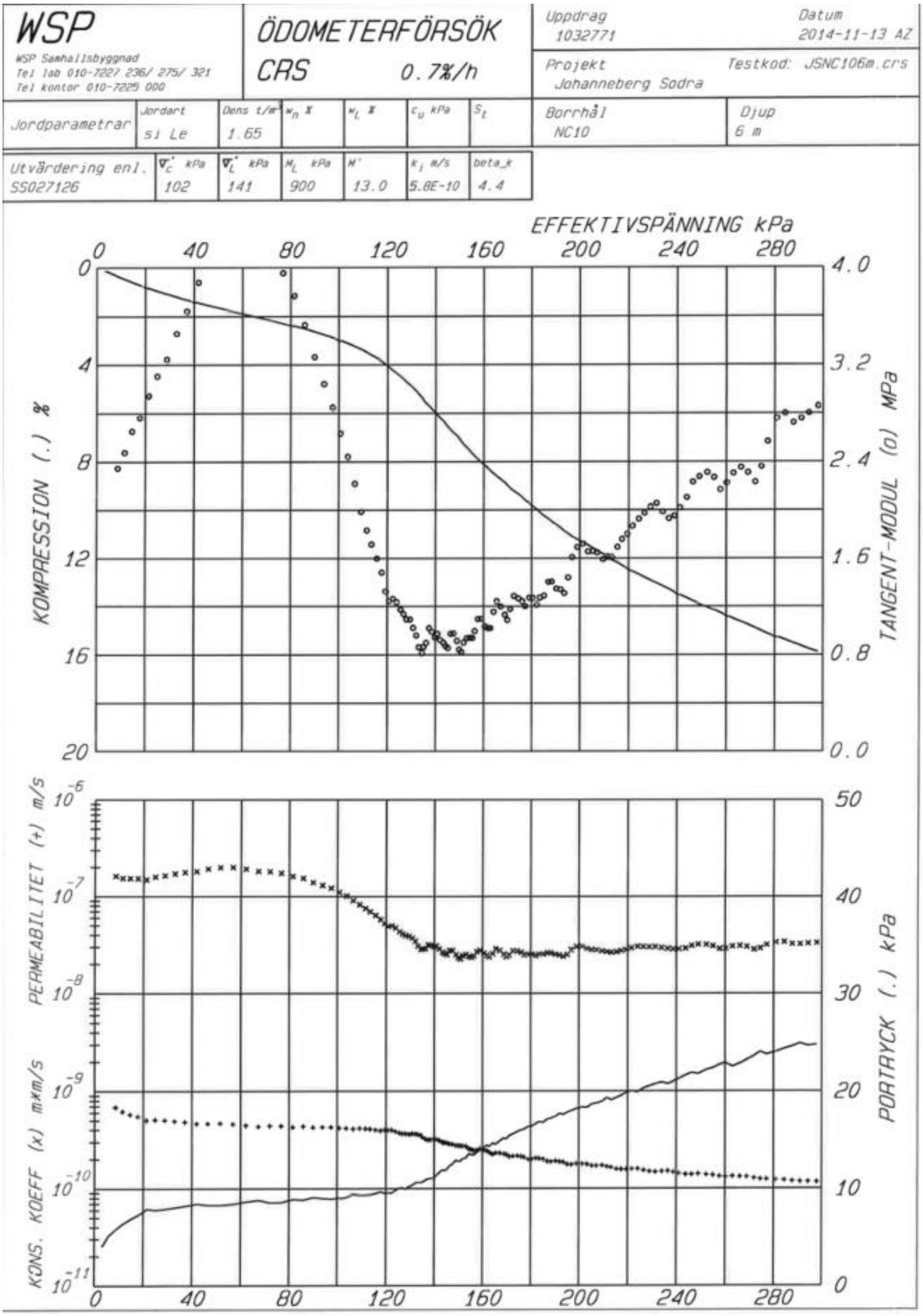

Figure A.3. CPT test and stress distribution based on the abortorium report made in 2014.

The effective stresses of consolidation, permeability, pore pressure and soil compression coefficient are obtained from test and can be seen in Figure A.3.

It is quit challenge to evaluate the initial modulus of consolidation, M_0 and for that it has been the rule to assume as $M_0 = 250c_u$. (SGI, 2008)

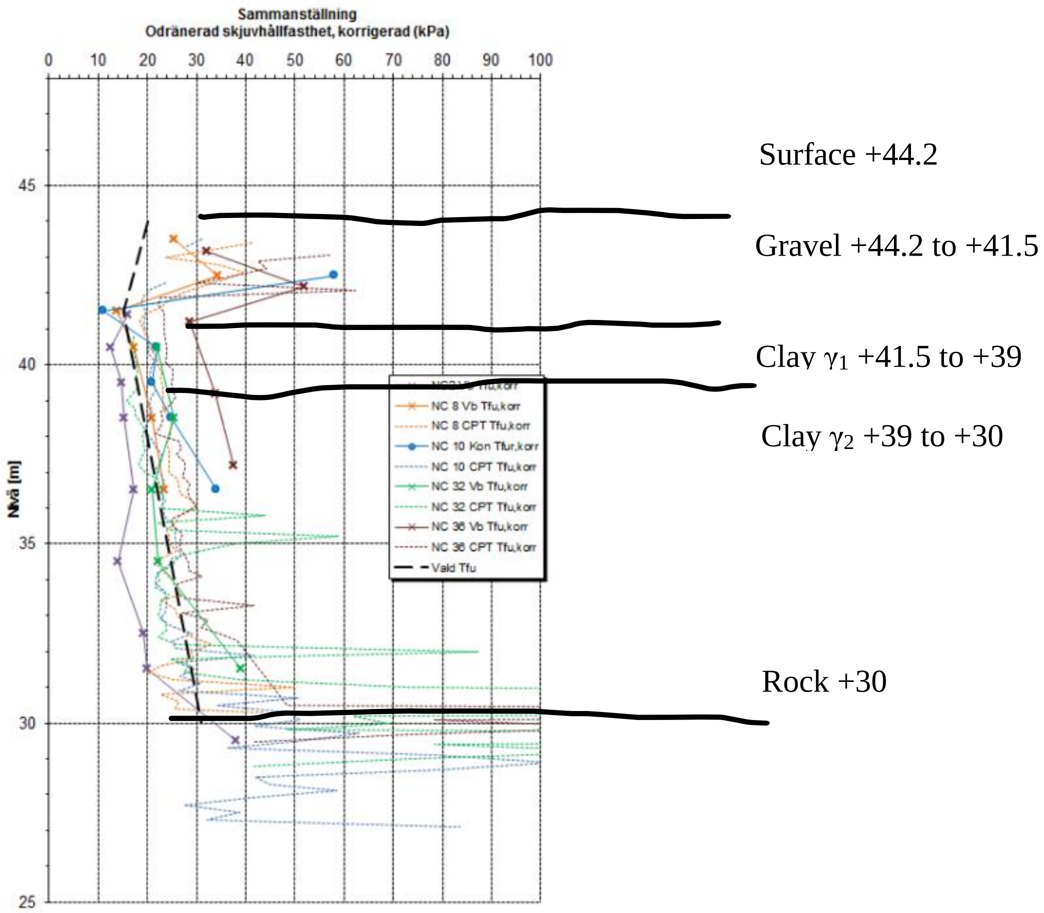

Figure A.4. Soil profile and shear strength through profile due to report made in 2014.

Hence the unit weight of all layers is known then the soil stress is easy to calculate.
In a case to calculations on settlements need to be done several values can be obtained from effect pressure of CPT test diagram. According to geotechnical report (MUR) the testing results of compression modulus is constant and is M_L=900kPa, see Figure A.3

Parameters	**Depth *z*, (m)**	**Unit weight γ, (kN/m^3)**	**Water content, %**	**Undrained shear strength c_{uk}, (kPa)**	**Friction angle ϕ, ° drain conditions**	**Young's modulus E_s, (kN/m$^{2)}$**
Surface (asphalt)	0,05	-	0	20	30	-
Gravel	0,05-1	19	0	20	30	50000
Sandy clay	1-1,3	18	18	20	32	25000
Sandy clay	1,3-3,2	18	31	20	32	25000
Turf	3,2-3,8	12 (γ'=2)	127-479	15	28	5000
Silting clay	3,8-5,2	17	37	15+1,4z	30	4000
Clay	5,2-26 or 5,2-84	18	32	15+1,4z	30	4000

Table A.2 Soil parameter and extension based on Norconsult investigations, 2014

Norconsult

Norconsult Fältgeoteknik AB

Norconsult Fältgeoteknik AB, BOX 8774,
402 76 GÖTEBORG
Telefon 03-50 70 00, Fax 031-50 70 10

LABORATORIEUNDERSÖKNINGAR

WSP Samhällsbyggnad
Box 13033, 402 51 GÖTEBORG
Telefon 010-722 5000

Sammanställning av
Laboratorieundersökningar

Uppdrag

Johanneberg Södra, etapp 2

Provtagnings-metod	PG	Skr	Kv St I	Kv St II X

Uppdragsnummer	**1032771**
Borrhål	**NC10**
Granskning	2014-11-07 Sign AZ

Grundvattenobservation Datum

Djup m	Jordartsbeskrivning[1]	Densitet ρ [2] (t/m³)	Vattenkvot w_N [3] (%)	Konfl.-gräns w_L [4] (%)	Sensitivitet S_t [5] (-)	Skjuvhållfasthet (okorr.) τ_{fu} [5] (kPa)	Skjuvhållfasthet (korrig.) τ_{fu} [5] (kPa)	Skjuvhållfasthet Omrörd τ_r [5] (kPa)	Korrekt. faktor μ [5] (-)	Matrl. typ[6]	Tjälf. klass[6]	Anm.
4,0	grå sandig siltig LERA, siltskikt, enstaka gruskorn, växtdelar	2,02 1,98 1,98	27 28	36	16	54	58	3,48	1,08			
5,0	grå siltig LERA, växtdelar, enstaka skalrester	1,74 1,73 1,69	45 58	59	12	12	11	1,04	0,87			
6,0	grå siltig LERA, skalrester	1,74 1,71 1,70	63 52	47	28	23	22	0,84	0,96			
7,0	grå sulfidfläckig ngt siltig LERA, skalrester	1,66 1,63 1,72	62 68	60	26	25	21	0,96	0,86			
8,0	grå sulfidfläckig siltig LERA, sandskikt, skalrester	1,78 1,78 1,82	37 46	40	51	24	25	0,46	1,03			
10,0	grå sulfidfläckig siltig LERA, sandkörtlar, enstaka gruskorn, rikl. med skal	1,72 1,81 1,64	56 47	33	81	31	34	0,38	1,13			

1) Jordartsbeskrivning i enlighet med SS-EN-ISO 14688 1:2002 & SS-EN-ISO 14688 2:2004 samt BFR T21:1982
2) Skrymdensitet enligt SS 027114, utgåva 2
3) Vattenkvot enligt SS 027116, utgåva 3
4) Konflytgräns enligt SS 027120, utgåva 2
5) Skjuvhållfasthet - konförsök enligt SS 027125, utgåva 1 (avvikelse: lägsta konintrycket för 100 gramskonen är 7 mm enligt SGF:s laboratoriekommittés rekommendationer)
6) Enligt AMA Anläggning 10, Tabell CB/1

Table A.3. Laboratories investigations on soil tests taken on the building side.

Appendices B

1.1 Lower bound approach

Lower bound (LB) theorem based on fulfilment of the equilibrium and yield criteria but no consideration on deformation (Craig eit, 2012). The yield criteria for undrain soil can be described as

$$\tau_f = c_u$$

τ_f is shear stress at failure
c_u is undrained shear strength

To fulfil the equilibrium the simple stress field has been accepted as it presented on the figure XX

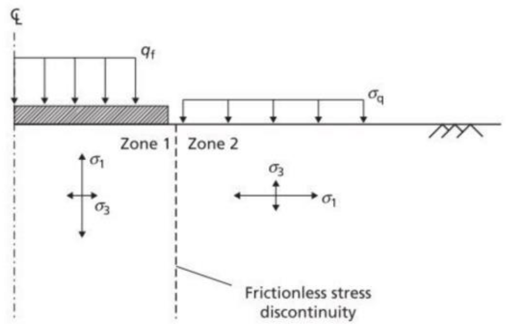

Figure B.1. Lower bound model present in Craig mechanics (2012)

Where it has be divided by amount of stress to two zones. At the zone 1 is the horizontal stress is quite bigger and can be presented as

$$\sigma_1 = q_f + \gamma z$$

q_f is applied load
γ is earth unit weight
z is depth

The first part of equation is a weight applied by the raft and second part is weight of the soil varier with a depth. At the zone 2 the horizontal stresses will be dominant, so the equation is look like this

$$\sigma_3 = \sigma_q + \gamma z$$

σ_q is soil pressure
γ is earth unit weight
z is a depth

By combining the yield criteria and the equilibrium the theorem concludes to the load bearing of the raft equation

$$q_f = 4c_u + \sigma_q$$

c_u is undrained shear strength
σ_q is soil pressure

In this analysis the boundary between zone 1 and zone 2 drown as a simple line but it is not totally true. A more realistic it would be to have a rotation field so called a fan zone and can be represent by friction stresses of the soil.

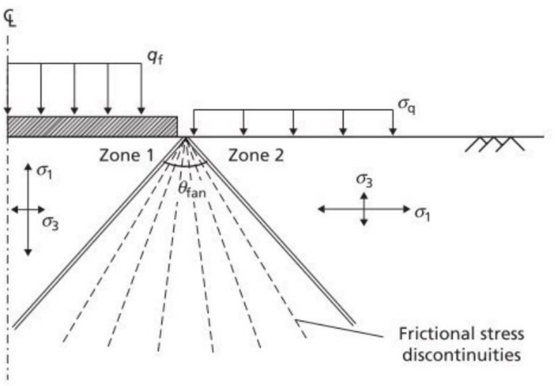

Figure B.2. Lower bound model present in Craig mechanics (2012)

It leads to an evolved equation for calculations of collapse load of the raft

$q_f = (2+\pi)c_u + \sigma_q$
c_u is undrained shear strength
σ_q is soil pressure

1.2 Upper bound approach

The upper bound approach (UB) based on plastic collapse and the external load causing the movements of the soil. Without any considerations of equilibrium, it presents the straightforward mechanism of the three sliding triangles as it shown in figure XX

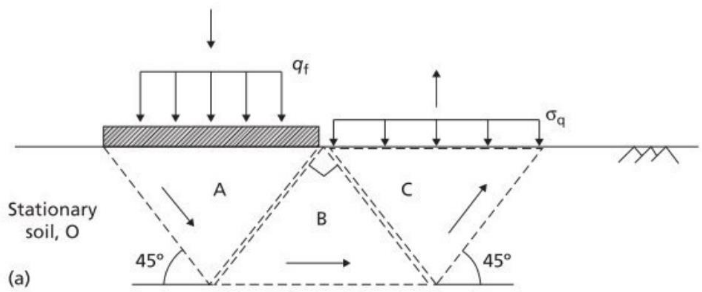

Figure B.3. Upper bound model present in Craig mechanics (2012)

By calculations of all energy usage because of the movements along the edges and summing up them conclusions can be made. What the sum of the energy usage is equal to sum of the work been done by external load. Therefore, the result of this theorem can be present in the simple way
$\sum E = \sum W$, and
$q_f = 6c_u + \sigma_q$
c_u is undrained shear strength
σ_q is soil pressure

Of course, in the same way as in the LB it is to dramatical change on the edges of the triangles and can be improved by replacing the middle triangle by rotation field or more fan zone. The fan mechanism is shown in figure XX more in details.

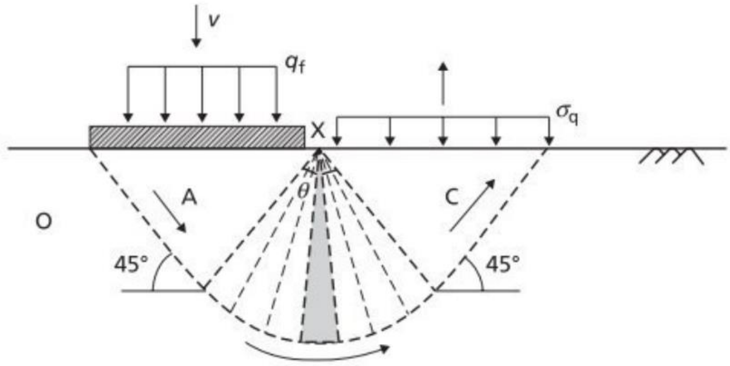

Figure B.4. Upper bound model present in Craig mechanics (2012)

2.1.3 Combination of UB and LB – undrain conditions under short term
The interesting point of view what the resulting equation of this approach gives the same equation as in LB fan mechanism which must be the pretty close to the truth of the collapse load value. Hence it is the most definitely the collapse load result is between the upper and lower bound approach. By consideration of neglecting the soil pressure σ_q, it is easily to accept what the collapse load placed

$4c_u \leq q_f \leq 6c_u$
c_u is undrained shear strength

Therefore, the equation is the unique solution for collapse load of the raft

$q_f = (2 + \pi)c_u = 5.14c_u$
c_u is undrained shear strength

In Sweden to design the bearing capacity of the raft foundation must be following the regulations of the Eurocode 7 (EC7). The design is simple and based on elasticity theory of LB and UB where stresses is uniformly distributed under the footing. It can be present as ultimate bearing capacity under undrained soil conditions

$q_f = 5.14v_c c_u + \gamma_D D$
v_c is shear stress at failure
c_u is undrained shear strength
γ is earth unit weight
D is earth depth
But the conservative design of the building foundations is the combination of the raft and piles.

1.3 Drained conditions – Long term

Hence to EC7 recommendations the bearing capacity for shallow foundations under drained conditions can be calculate as

$$\frac{R}{A'} = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 0{,}5\gamma' B' N_\gamma b_\gamma s_\gamma$$

R=Q represents the applied load on the foundation

A' is effective foundation area
c' is effective cohesion
q' is the external load
γ' is unit weight of soil reliant on ground water
B' is foundation width
N_c, N_q,N_γ is bearing capacity factors reliant on friction angel ϕ'
b_c, b_q, b_γ is inclination of the foundation
s_c, s_q, s_γ is sheep factors of foundation
i_c, i_q, i_γ is inclination factor of the load

By simplifying the equation and adapt to the case of the studied raft foundation the equation appearance as
$q_f = c'N_c + \gamma D N_q + 0{,}5\gamma' B' N_\gamma$
where bearing capacity factors N_q, N_c, N_γ depends on friction angle ϕ'

Many has investigated bearing capacity factors but according to recommendations from EC 7
$N_\gamma = 2(N_q - 1)tan\varphi'$
$N_q = e^{\pi \cdot tan\varphi'} tan^2\left(45 + \frac{\varphi'}{2}\right) = e^{\pi \cdot tan\varphi'}\left(\frac{1 + sin\varphi'}{1 - sin\varphi'}\right)$
$N_c = \frac{(N_q - 1)}{tan\varphi'}$, according to Brinch Hansen and Meyerhof

1.4 Raft model

Chosen raft is a part of the entire concrete slab with a dimensions 30x60 m^2. Calculations on raft bearing capacity in ULS was made and evaluate the rigidity of the ground slab. For cohesive soil conditions is $\phi'=0$ when
$N_c=5.5$,
$N_q=1.0$,
$N_\gamma=0$, but according to ground water level with 1,3m below the surface and the raft is levelled up with the surface. General bearing capacity factors can be calculated, and it depends on the friction angle $\phi'=32^\circ$

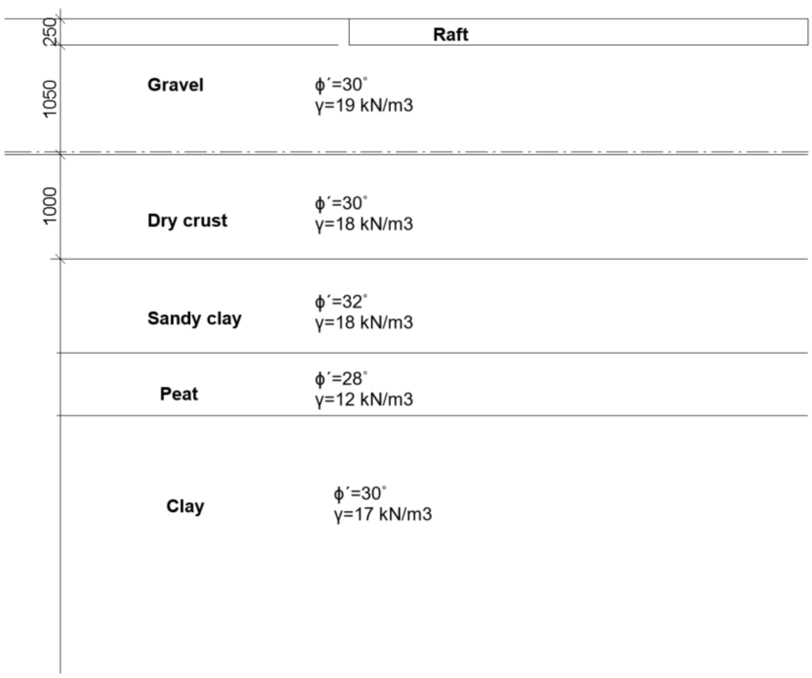

Figure B.5. Soil profile according to Appendixes A for long term response (ULS)

The raft thickness is 0,25m. Regarding to geometry is no eccentricity or changes in depths coefficient is assumed.
Corrections according to ground water been made $\bar{\gamma} = 1{,}54$
Shape of footing correction: s_c=1+0,2*30/60=1.1
s_q=1+0,2*30/60=1.1
s_γ=1-0,4*30/60=0.8

SK 3: γ_n=1.2; γ_m=1.6; γ_{Rd}=1.0 and c'_d=20*0,1/ γ_{m*} γ_n=5.2
$\arctan(\phi'_d) = \frac{\tan(30^\circ)}{1.2*1.6} = 16{,}7^\circ$ → N_c=15.8; N_q=7.07; N_γ=3.36

$q_b = 1{,}04 \cdot 11{,}6 \cdot 1{,}1 \cdot 1 + 11 \cdot 0{,}25 \cdot 4{,}34 \cdot 1{,}1 \cdot 1 + 0{,}5 \cdot 1{,}42 \cdot 1{,}54 \cdot 0{,}8 = 53{,}5kPa$
According to load calculations made in Load and global stability the total applied load regard the self-weight and variable load with value of 46 kPa is quate similar to calculated bearing capacity of the raft.
Regarding Boverket the safety factor has to be for ULS 1.5 so load calculations for maximum design load combination is structural uniformly distributed unfavourable load with value of $1.5 * 2.041MN/40 = 76.5\ kN/m^2$
Sense these calculations the raft can managed the 30 % of the load but it steel need the deep foundation to fulfil total load bearing requirements and regarding settlements redaction.

1.5 Settlements under the raft built on CPT test

According to De Beer (1965) settlements can be estimated due to equation (B.1)

$$s = \gamma_{Rd} \cdot \sum \frac{\mathbf{2.3}}{\boldsymbol{C_d}} \log\left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}\right) * \Delta z \qquad \text{(B.1)}$$

$\gamma_{Rd} = 1.1;\ \gamma_m = 1.2$

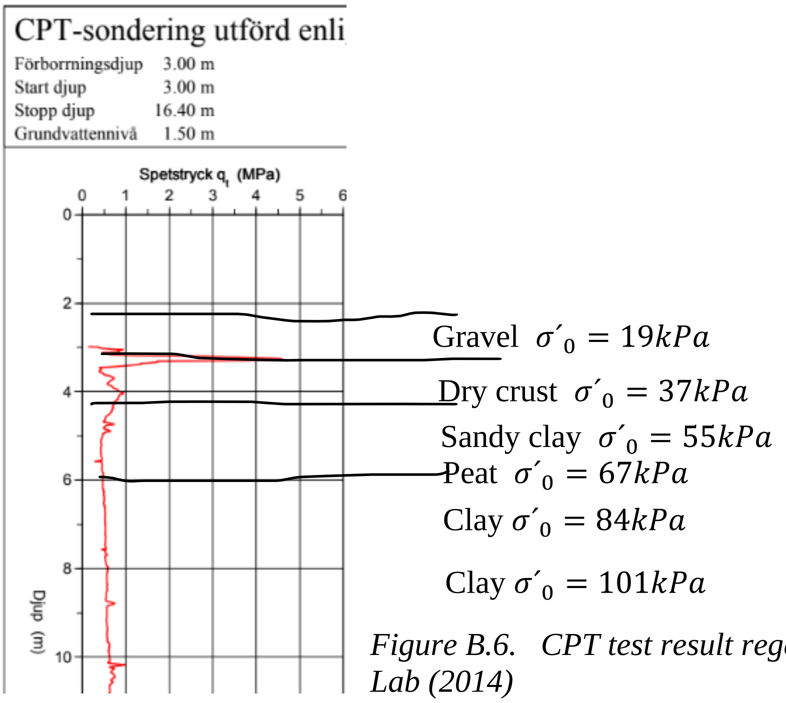

Figure B.6. CPT test result regarding MUR from the A Working Lab (2014)

$$s_{gravel} = 1.1 \cdot \frac{2.3}{46} \log\left(\frac{19+46*30*\frac{60}{(30+1)(60+1)}}{19}\right) 2m = 0.0657;\ \text{1m from the surface}$$

$$s_{crust} = 1.1 \cdot \frac{2.3}{20{,}3} \log\left(\frac{37+46*30*\frac{60}{(30+2)(60+2)}}{37}\right) 2m = 0.0943;\ \text{2m from the surface}$$

$$s_{sandy\ clay} = 1.1 \cdot \frac{2.3}{6{,}8} \log\left(\frac{55+46*30*\frac{60}{(30+3)(60+3)}}{55}\right) 2m = 0.2021;\ \ \text{3m} \ldots.$$

$$s_{peat} = 1.1 \cdot \frac{2.3}{9{,}3} \log\left(\frac{67+46*30*\frac{60}{(30+4)(60+4)}}{67}\right) 2m = 0.122;\ \ \text{4m} \ldots.$$

$$s_{clay} = 1.1 \cdot \frac{2.3}{7{,}4} \log\left(\frac{84+46*30*\frac{60}{(30+5)(60+5)}}{84}\right) 2m = 0.101;\ \text{5m}\ldots$$

and so on …

$S = s_{gravel} + s_{crust} + s_{sandy\ clay} + s_{peat} + s_{clay} \ldots = 3.6m;$ long-term (10 years)

Consequently, the bearing capacity of the raft fulfilled just partially. Generally, it is up to engineer to decide how loads will be redistributed. Here it is possibilities to relay a part of the loads on the raft. Apart the ground stability at this design the raft rigidity has to be checked also.

Under circumstances of the existed soil profile (~26m clay) is already big settlements of 2m and by applying this method on the chosen soil profile (~80m clay) would not make the settlements less.

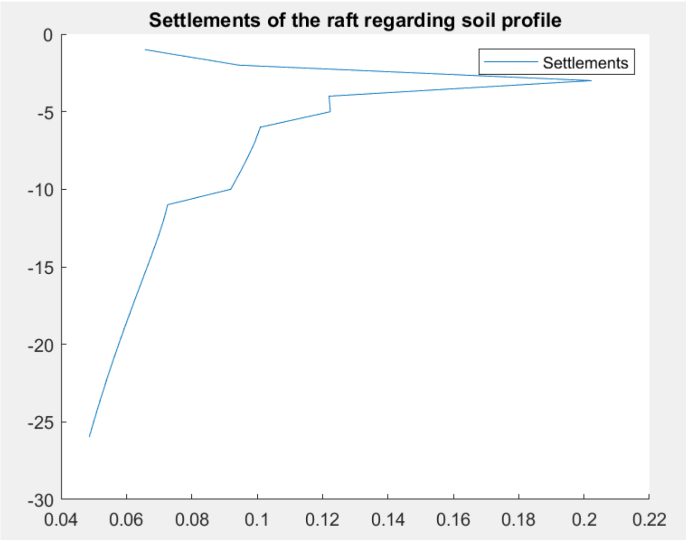

Figure B.7. Settlements for existing soil profile with magnitude of 26 m

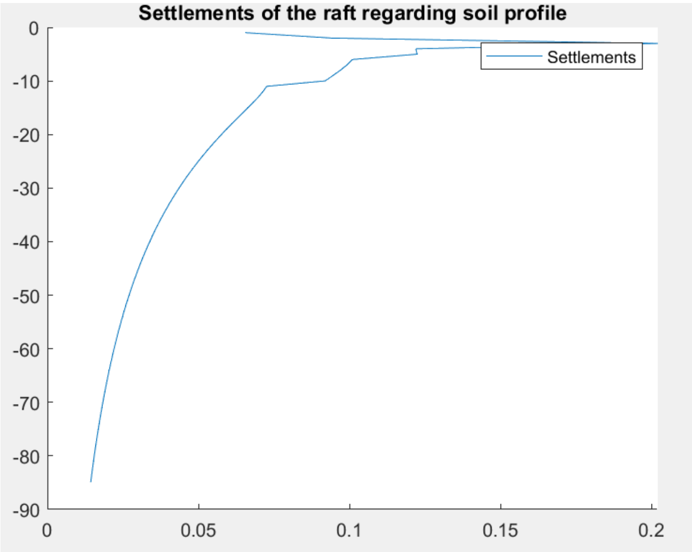

Figure B.8. Settlements of the raft for chosen soil profile with magnitude of 65 m cohesive soil based on CPT test and assumptions

Nevertheless, the settlements under the raft under long-term conditions redirecting the design to more stiff and stable construction where long piles have to be added. At the same time differential settlements analysis has to be made for new design.

All complete calculation been made in MATLAB code “Raft capacity”.

Appendices C

Elastic-plastic approach for NP

The drag load is known as

$$Q_n = \int_0^z A_s \beta \sigma'_z dz \qquad (4.10)$$

where β is Bjerrum-Burland coefficient see Table 4.2 or can be estimated according to

$$\beta = K tan\varphi' \qquad (4.10)$$

Table 4.2. Range of ϕ, β and N_t adapted from Fellenius (1991).

Soil type	ϕ (°)	β	N_t
Clay	25–30	0.25–0.35	3–30
Silt	28–34	0.27–0.50	20–40
Sand	32–40	0.30–0.60	30–150
Gravel	35–45	0.35–0.80	60–300

In this way the bearing capacity of the shaft similar to Q_n calculations can be estimated as

$$R_s = \int_0^L A_s \beta \sigma'_z dz \qquad (4.11)$$

and toe resistance relay on toe area and the effective stress at the toe level

$$R_t = A_t N_t \sigma'_{z=L} \qquad (4.12)$$

TABLE 1. Ranges of ϕ, β, and N_t values

Soil type	ϕ (°)	β	N_t
Clay	25–30	0.25–0.35	3–30
Silt	28–34	0.27–0.50	20–40
Sand	32–40	0.30–0.60	30–150
Gravel	35–45	0.35–0.80	60–300

NOTE: From Ontario Highway Bridge Design Code (1992).

Table D.1.

Regarding Table D.1 and the soil profile it is known:
Clay → $\phi'=28^\circ$ → $\beta = 0,3$ → $N_t = 21$

Due to this assumptions and soil profile knowledge, it is fusible to evaluate

$$\frac{N_t}{\beta} = \frac{21}{0,3} = 70$$

The pile has diameter, B_m with variety 0,15-0,35 so let take middle value of 0,25m and pile length, D variety between 18-54m, max settlements, S = 0,5m according to EC7.

$$\alpha = \frac{N_t/_{2\beta}}{L/_B} + 1 = \frac{21}{2 * 0,3} * \frac{0,25}{54} + 1 = 1,162$$

According included in report diagram it is feasible to establish few values, see Figure XX

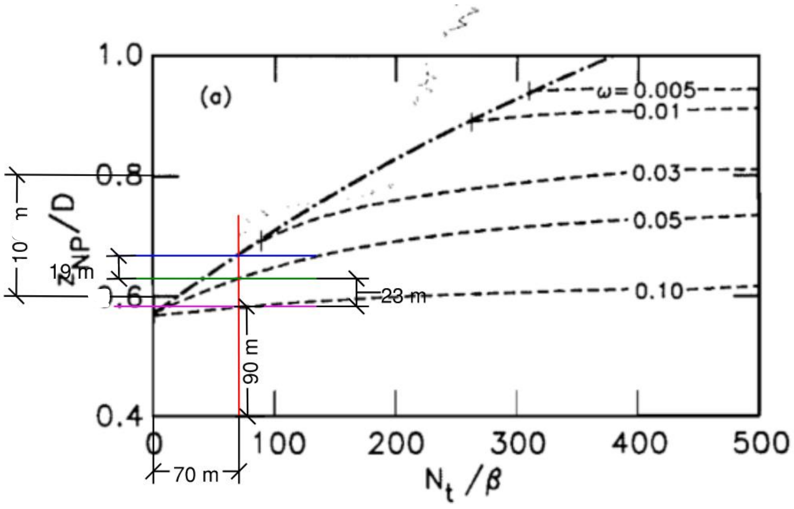

Figure XX.

As it seen on diagram by red line shows the value of $\frac{N_t}{\beta} = 70$. The line cross values of ω

Pink line with value ω=0,1 and $\frac{z_{NP}}{D} = 90 * \frac{0,2}{100} = 0,18$

Green line with value ω=0,05 and $\frac{z_{NP}}{D} = (90 + 23 +) * \frac{0,2}{100} = 0,226$

Bluey line with value ω=0,03 and $\frac{z_{NP}}{D} = (90 + 23 + 19) * \frac{0,2}{100} = 0,264$

In this point the safety factor should be chosen. Alternative 1: F_s=2 and alternative 2: F_s=3. Let make calculations on Alternative 1.

<u>Alternative 1</u>

Assumption is that $F_s = 2 = \frac{R_u}{Q_d}$ and known fact according to calculations and the models

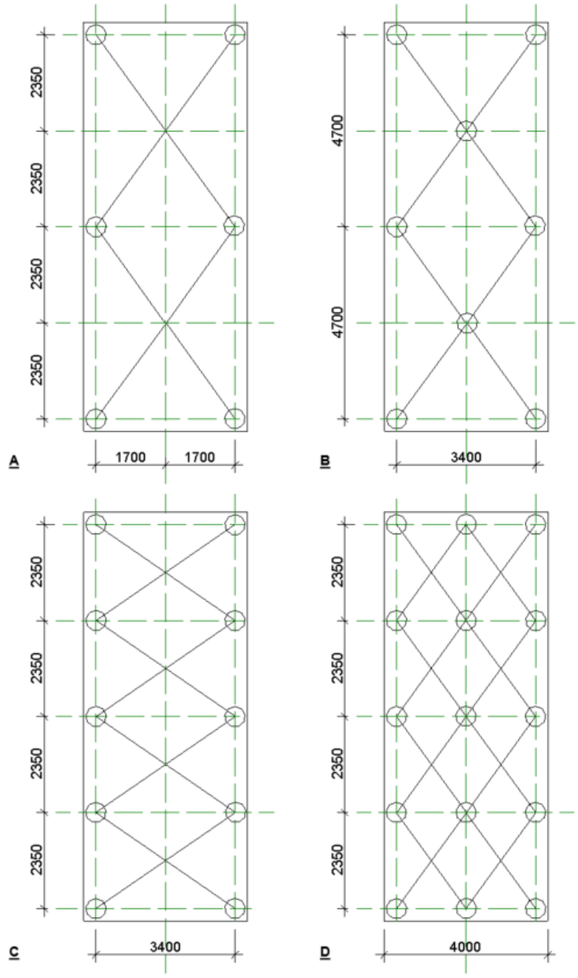

Figure XX. Evaluated models of pile plan A-D

$Q_{d_A} = \frac{46kN}{m^2} * A_A * F_s = 1470\ kN \rightarrow R_{su\_A} = 1708kN$
$Q_{d_B} = \frac{46kN}{m^2} * A_B * F_s = 733\ kN \rightarrow R_{su_B} = 852kN$
$Q_{d_C} = \frac{46kN}{m^2} * A_C * F_s = 735\ kN \rightarrow R_{su\_C} = 854kN$
$Q_{d_D} = \frac{46kN}{m^2} * A_D * F_s = 368\ kN \rightarrow R_{su\_D} = 428kN$

ω=0,1; 0,05; 0,03 → minimum $\frac{z_{NP}}{L}$=0,18 and maximum $\frac{z_{NP}}{L}$=0,264
$D = L = 54m \rightarrow z_{NP} = 9{,}72m$ or z_{NP}=0,18*L
$D = L = 54m \rightarrow z_{NP} = 14{,}256m$ or z_{NP}=0,264*L

Can it be the transition zone between 14,256 and 9,72m for the pile of 54m when the $z_{NP\_av} = \frac{14{,}256+9{,}72}{2} = 12m$ (18 meters in the plastic-rigid method)
and $(\lambda - \omega)L = 9{,}72$; $(\lambda + \omega)L = 14.256$ so

$\omega = 0{,}042$

$$\lambda = \frac{z_{NP}}{L} = \frac{12}{L} = 0{,}222$$

Now the maximum load or applied load at NP can be evaluated

$R_{u\_A} = 1470\ kN \rightarrow Q_{NP\_A} = 786{,}3kN$
$R_{u\_B} = 733\ kN \rightarrow Q_{NP\_B} = 392kN$
$R_{u\_C} = 735\ kN \rightarrow Q_{NP\_C} = 393{,}2kN$
$R_{u\_D} = 368\ kN \rightarrow Q_{NP\_D} = 196{,}85kN$

<u>Alternative 2</u>
$F_s = 3 = \frac{R_u}{Q_d}$ N

When
$Q_{d_A} = \frac{46kN}{m^2} * A_A * F_s = 2013{,}5\ kN \rightarrow R_{su\_A} = 1708kN$
$Q_{d_B} = \frac{46kN}{m^2} * A_B * F_s = 1003\ kN \rightarrow R_{su_B} = 852kN$
$Q_{d_C} = \frac{46kN}{m^2} * A_C * F_s = 1008\ kN \rightarrow R_{su\_C} = 854kN$
$Q_{d_D} = \frac{46kN}{m^2} * A_D * F_s = 504\ kN \rightarrow R_{su\_D} = 428kN$

ω=0,1; 0,05; 0,03 → minimum $\frac{z_{NP}}{D}$=0,18 and maximum $\frac{z_{NP}}{D}$=0,264
$D = L = 18m \rightarrow z_{NP} = 3{,}24m$ or z_{NP}=0,18*D
$D = L = 18m \rightarrow z_{NP} = 4{,}75m$ or z_{NP}=0,264*D
$z_{NP} = \frac{4{,}75+3{,}24}{2} = 4m$ (7 meters in the plastic-rigid method)

See MATLAB program for following calculations.