Investigation and Development of the Design Process for a Pile Group

with and without Lateral Resistance from the Soil

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Abstract

For bridges in weak ground conditions, piling is an appropriate type of foundation. To manually arrange the piles in a pile group is an iterative, time consuming process where the section forces in the piles must be verified for a large number of load cases. In addition, after installation, piles may have new positions and the capacity must be re-verified.

In this thesis, a software was developed to automatize the design process for pile groups, with the aim to increase the efficiency. The pile groups were optimized according to maximum and minimum normal forces in the piles, by simulating random pile groups using the Monte Carlo method. The randomly simulated pile parameters included the position, inclination and direction of the pile. Interviews with contractors were also carried out to obtain practical requirements, that were implemented in the program. To evaluate the program, four real cases have been studied that resulted in reasonable preliminary pile groups in an hour or less. In some cases, the number of piles could be reduced. The process still requires some manual surveillance, such as calibrating the input data and evaluating the feasibility of the generated pile groups. This is not necessarily negative, as the output always should be verified.

A separate investigation of the concept of the distance between pile center and load center was also done, to see if the concept can be used in optimization of pile groups. Correlation tests between this distance and resulting minimum normal forces were performed, where the method partly was unverified and therefore, little can be stated about the relevance of this concept. Thus, it seems more reasonable to evaluate pile groups directly by their section forces.

Keywords: Pile group optimization, objective function, frame analysis method, lateral resistance of the soil

Sammanfattning

Pålning är en typ av grundläggning lämplig för att grundlägga broar vid svaga markförhållanden. Att manuellt utforma en pålgrupp är en iterativ, tidskrävande process där snittkrafterna i pålarna måste verifieras för ett stort antal lastfall. Efter installationen kan pålarna dessutom ha nya positioner och kapaciteten måste verifieras igen.

I detta projekt utvecklades en mjukvara för att automatisera dimensioneringsprocessen för pålgrupper, i syfte att öka effektiviteten. Utformningen av pålgruppen optimerades med avseende på största och minsta normalkraft i pålarna, genom att simulera slumpmässiga pålgrupper med Monte Carlo-metoden. De simulerade slumpvariablerna var pålens position, lutning och riktning. Intervjuer med entreprenörer genomfördes också för att inhämta praktiska önskemål, som implementerades i programmet. För att utvärdera programmet har fyra verkliga fall studerats som resulterade i rimliga preliminära pålgrupper på en timme eller mindre. I vissa fall kunde antalet pålar minskas. Processen kräver fortfarande viss manuell övervakning, till exempel för att kalibrera indata och utvärdera genomförbarheten för de genererade pålgrupperna. Detta är nödvändigtvis inte negativt, då utdatan alltid bör verifieras.

En separat undersökning utfördes av konceptet för avståndet mellan pålcentrum och lastcentrum, för att se om konceptet kan användas vid optimering av pålgrupper. Korrelationstester utfördes mellan detta avstånd och resulterande minsta normalkrafter, där delar av metoden ej var verifierad och därför är relevansen av detta koncept begränsad. Det verkar således rimligare att utvärdera pålgrupper direkt utifrån deras snittkrafter.

Nyckelord: Pålgrupp, optimering, målfunktion, ramanalys, sidomotstånd från jord

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Notations and Symbols

α	in plane direction of the pile
β	inclination of the pile
Δ_p	displacements for pile
a	design variable
A	area of cross section
A_p	rotation transformation matrix
b	state variable
C_p	translation transformation matrix
D_p	transformation matrix
E	Young's modulus
f_M	maximum moment
F_p	section forces at pile top
f_T	maximum shear force
I	moment of inertia
JG	torsional stiffness
kd	subgrade modulus
K_p	stiffness matrix for pile
L	pile length
L_e	fictitious length for cohesive soil
L_i	characteristic length for friction soil
LC	load center
m	degree of rigidity of connection between pile and pile cap
M_{Ed}	design bending moment in piles
M_{Rd}	bending moment capacity of pile
n	linear subgrade modulus
N_{Ed}	design normal force in piles
N_{Rd}	compression capacity of pile
P_p	forces and moments at the origin of the pile cap
PC	pile center
R	applied forces and moments
S	stiffness matrix for the pile group
U	displacements for the pile cap

Contents

1 Introduction

1.1 Background

Piling is a suitable type of foundation for structures when ground conditions are weak in relation to the loads applied, and bridges are no exception. The purpose of piling is to transfer the loads from the structure down to stronger and stiffer soil or rock.

Piles can be used as single elements or as pile groups. A pile group consists of a number of piles that are connected to each other at their tops by a pile cap. The pile cap is typically a reinforced concrete slab.

In the initial phase of the design of a pile group, a suitable pile type is chosen and the number of piles is estimated from experience or from approximating formulas. The piles are ideally placed in a way that resists the given loads in an efficient way. In general, the designer tries to arrange the pile group to minimize tension in the piles. The overall goal in pile group design is to minimize the number of piles.

A concept that can be used for pile group optimization is to consider the distance between the pile center and the load center. Minimizing this distance generally reduces tension in the piles. This concept will be presented in detail in Section 2.4.1. When optimizing the pile group for one given load, the efficiency of the pile group to resist other loads decreases, which makes the procedure iterative.

It is tempting to treat iterative processes such as the arrangement of pile groups by the use of an automatized optimization process. According to Olsson et al. (1993) this is even necessary for big pile groups. Considering the large amount of load cases, where 20 load cases or more is typical for a bridge pile group in Sweden, the suitability of an automatized process in highlighted.

Relevant regulations for pile groups for bridges can be found in Eurocode 7 and in the regulations *Krav Brobyggande* and *Råd Brobyggande* from the Swedish Transport Administration (Trafikverket). In addition to design regulations, the contractor may have requirements on the arrangement of piles due to practical reasons at the building site. According to *Krav Brobyggande*, pile groups for railway bridges must be designed considering two cases: high and no lateral resistance of the soil (Trafikverket, 2019a). For other types of bridges, pile groups should be designed for high and low lateral resistance. A recent update of these regulations gave rise to this master thesis project, that will have a focus on lateral resistance of the soil in design.

The thesis tries to make an overview of the requirements on a pile group and aims to improve the efficiency in pile group design. This aim is formulated in detail as an objective along with research questions in the following section.

1.2 Objective and Research Questions

The objective of the thesis is to investigate and develop the process for determining a suitable preliminary arrangement of a pile group for a bridge, considering lateral resistance of the soil, and to evaluate this process for a few existing cases. This process should also involve the practical demands for pile group design.

Furthermore, it is of interest to investigate the importance of determining an optimal placement for the pile center, and to evaluate if this can facilitate the process of designing an optimal pile group.

To summarize, the thesis will answer the following questions:

- How should an optimal arrangement of a pile group be done in early design?
- Is it helpful to make use of the concept of pile center and load center? How should this be done?
- What requirements does the contractor have on the arrangement of piles? How should this be considered in design?
- Is the lateral resistance of the soil, or the absence of it, governing the pile group design?

1.3 Approach and Limitations

In this master thesis an optimization software is developed in the programming language Python, that can design a pile group with respect to certain parameters. The structural optimization is carried out using Monte Carlo simulation. The pile groups are modelled using frame analysis assuming end bearing piles. Practical requirements are obtained by interviewing three experienced contractors and are implemented in the software.

Using this program four different case studies are tested. The developed program is also used to investigate the theory that there is a correlation between tension in the piles and the distance between the pile center and load center.

The scope of the project is limited to be able to gain meaningful results within the limits of a master thesis project. Firstly, the pile group is designed with respect to ultimate limit state; i.e. aspects such as serviceability limit state, fatigue and accidental events are not considered. Secondly, the pile group is optimized according to maximum and minimum normal forces. In addition, the randomly simulated pile parameters will be limited to the position, inclination and direction of the pile. This implies that parameters such as pile length, pile type and pile cap dimensions will not be optimized. The designer using the program should define these parameters, as well as the number of piles. The intention is to be able to manually decrease the number of piles, by letting the program arrange them in an efficient way. Finally, this thesis focus on pile groups for bridges; however, most concepts are valid also for other structures.

1.4 Outline of Thesis

The thesis consists of the following sections:

Section 2 describes the functionality of pile groups and provides a general overview of the design process. The frame pile group model is described and the concept of structural optimization is introduced. Section 2 also presents practical requirements based on interviews with contractors.

Section 3 presents the structure of the optimization program and how the program is verified. Four different case studies are presented. The end of section 3 describes the investigation of using load center and pile center as a basis for optimization.

In Section 4, the results are presented. This includes verification of program, the simulated pile groups resulting from the case studies and the pile center-load center investigation.

Section 5 discusses the results and conclusions are drawn which pertain to the research questions.

2 Theory

2.1 Foundations and Types of Soil

Loads from structures are transferred to the ground by foundations. Foundations are usually divided into shallow foundations and deep foundations. Spread footings and slabs are examples of shallow foundations that are suitable for strong ground conditions. When ground conditions are weak, deep foundations are used, where piling is the most common one, see Figure 2.1(c).

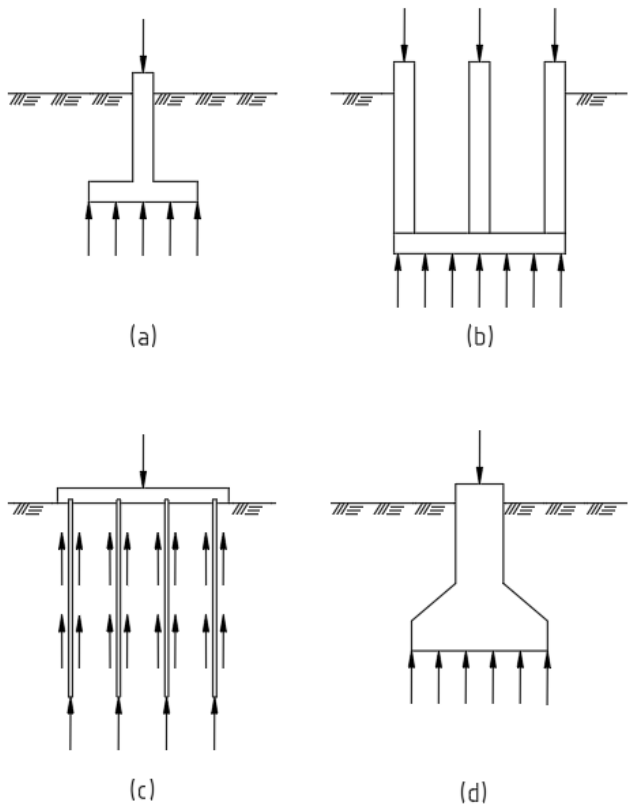


Figure 2.1: Different types of foundations. (a) spread footing; (b) mat foundation; (c) pile foundation; (d) drilled shaft foundation. Inspired by Das (2002).

Soils are divided into cohesive soil, typically clay, and friction soil, for example sand. Cohesive soil and friction soil behave differently and are therefore treated in different ways, meaning that they have different properties describing their strength and stiffness. The parameters describing the strengths are the undrained shear strength c_u for cohesive soil and friction angle ϕ' for friction soil (Sällfors, 2009).

The soil parameter used for deformation calculations is the modulus of subgrade reaction, in short subgrade modulus, kd. The subgrade modulus is a spring stiffness that relates stress to the deformation in the soil and has the unit N/m^2. The spring stiffness can be non-linear. As a simplification, for cohesive soil it is assumed being constant along the depth of the pile. The subgrade modulus for friction soil increases with depth, and the parameter used for stiffness is therefore usually a linear subgrade modulus n with the unit N/m^3 (Bredenberg and Broms, 1978).

2.2 Piles, Pile Groups and their Functionality

Piles may be of various materials such as reinforced concrete, steel or timber. Reinforced concrete piles are the most common type of piles used in Sweden (Olsson et al., 1993). As mentioned in the introduction, a pile cap can be cast to a number of piles and form a pile group. The actions in a single pile is then dependent on the other piles, analogous to columns in a frame. Pile groups are commonly modelled by the use of frame analysis, also called the direct stiffness method.

Piles can be categorized according to how they transmit load, which depends on which type of soil or rock they are surrounded by (Stål, 1984). Figure 2.2 shows conceptual illustrations of an end bearing pile and a shaft bearing pile. With shaft bearing piles, the largest part of the load is transferred to the surrounding soil at the contact surface between the pile and the soil. The shaft bearing pile is also called friction pile, where the word friction describes the friction along the shaft and has nothing to do with friction soil. Both piles in cohesive soil and friction soil can function as shaft bearing piles, but piles in cohesive soil have a more pronounced shaft bearing functionality. Shaft bearing piles in friction soil and cohesive soil are treated differently in design. Unlike shaft bearing piles, the end bearing piles transmit the load mainly via the pile end. This describes a pile resting on bed rock. In practice, piles function by a combination of the two categorizations. However, when calculating the distribution of forces in a pile group using the frame analysis method, this categorisation is not relevant and all piles are assumed to be end bearing, meaning that all resistance is gained at the pile end.

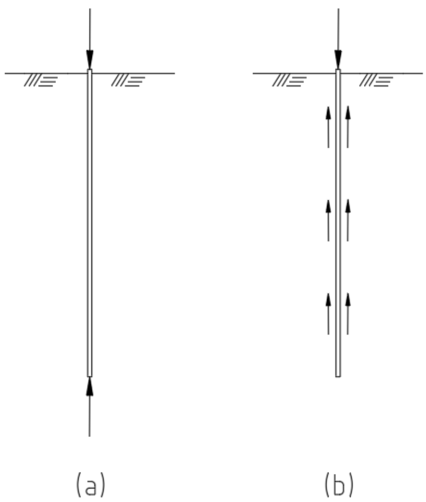


Figure 2.2: The end bearing pile (a) resist load mainly at the end and the shaft bearing pile (b) mainly along the shaft.

In design, the resistance of single piles, and the resistance of the pile group as a whole, must be verified, as well as the resistance of the pile cap (Olsson et al., 1993). Pile groups can fail in two conceptual ways, either by failure of a single pile or by failure of a block of piles. The governing failure mode depends on the distance between the piles. For a relatively small spacing, the piles and the soil enclosed by the piles will act like a rigid body, meaning that the block failure is the governing failure mode, whereas the opposite is true for larger spacing. This is illustrated in Figure 2.3. Since there are requirements on minimum distances between piles, the block failure is normally not governing (Olsson et al., 1993).

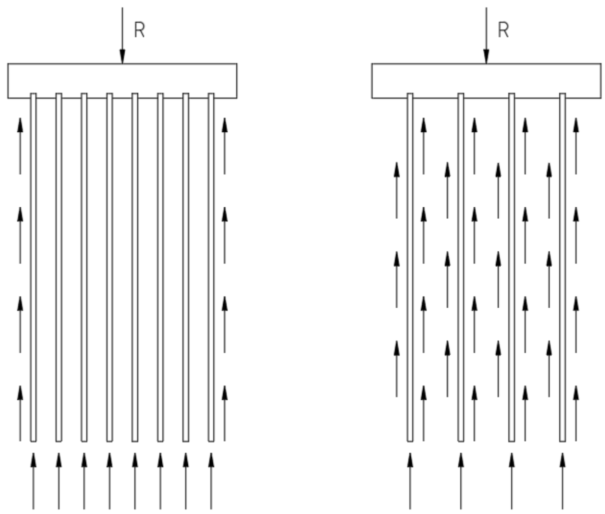


Figure 2.3: Left: Pile groups with small distances fail by block failure. Right: Pile groups with larger distances fail by failure of single piles.

The resistance of single piles should also be based on multiple failure criteria, i.e. failure of the pile itself and failure of the surrounding soil.

A pile group shall be designed so that it can withstand combinations of vertical and horizontal loads as well as bending moments applied to the pile cap. Some general concepts on how to arrange pile groups suitable for different types of loads will be presented in Section 2.4.

2.3 Overview of the Design Process for Pile Groups

The design process for pile groups consists of several steps, summarized in Figure 2.4. The first step is to determine the input data such as loading, geotechnical conditions and geometrical restrictions. Then, the design process includes choosing the number and type of piles, followed by an iterative process to determine an efficient arrangement of the piles. The iteration process considers and calculates the pile capacity in relation to the section forces obtained for a number of different load cases. Once the piles are installed and their final position is surveyed, the designer should confirm that the capacity of the pile group is sufficient by repeating the calculations using the actual positions. If the calculations are not confirmed, an additional pile, followed by a recalculation, is needed.

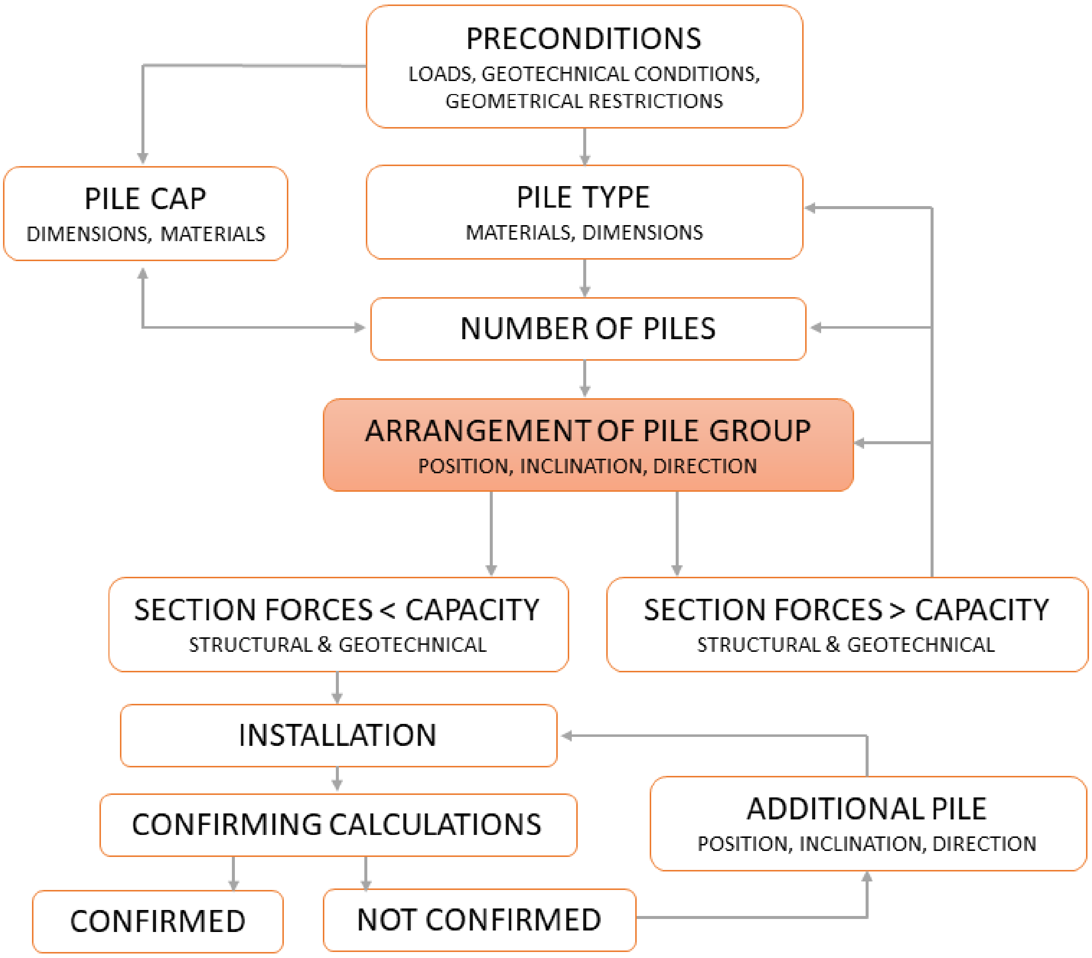


Figure 2.4: Overview of the design process for pile groups describing how different activities are depending on each other. The highlighted activity, arrangement of pile group, is the one being emphasized in this project.

This thesis focuses on the arrangement of the pile group, highlighted in Figure 2.4. The design of the pile group is highly influenced by the type of applied load, i.e. different combinations of vertical and horizontal forces as well as bending and torsion. As the number of load combinations are many, it is usually not obvious which arrangement is the most suitable.

2.3.1 Structural Optimization of Pile Group Arrangements

A structure can be optimal in different aspects, formally referred to as objectives. Objectives may, for example, be to minimize displacements, section forces or the cost of the pile group. The purpose of structural optimization is to find the structure that preforms the task in the best way with respect to the objective (Christensen et al., 2008). To assess the design, an objective function is used. The function evaluates every possible design within constraints of the function. The constraints can be divided into design constraints, behavioral constraints and equilibrium constraints. Design constraints may, for instance, be geometrical limitations of the pile group. The behavioral constraints, on the other hand, represent constraints on the response of the pile group under a certain load condition, such as limitation of displacements and section forces. Finally, the equilibrium constraints demand that the pile group is stable.

There are many types of optimization algorithms that can be used for structural optimization. One way of optimizing a structure considering many possible combinations of design variables is to use Monte Carlo simulation.

2.3.2 Monte Carlo Simulation

The number of possible combinations of the variables used in an optimization can be enormous depending on how they are constrained. Instead of testing all possible combinations, a limited number of combinations from a suitable distribution, gained by a random number generator, can be evaluated. This is called a Monte Carlo simulation (Thomopoulos, 2013). A Monte Carlo simulation runs a model repetitively. Each time random realizations of the input variables are generated, resulting in virtual outcomes for the output variables. The suitability of each virtual outcome can then be evaluated in relation to the objective function. A Monte Carlo simulation can be carried out for input variables that have different types of probability distributions. If all values for a variable have equal probability, the distribution is uniform.

The simulation is particularly useful for predicting outcomes of complex systems, but can also be used in optimization. The Monte Carlo simulation relies on the concept of random number generator. This means that the different trials are independent. Disadvantages of a random generator are that all possible combinations may not be tested and some may be tested more than once, and that the outcome of an analysis varies from one attempt to another.

2.4 Flow of Forces in Pile Groups

As mentioned before, the design of the pile group should reflect the loads.

All piles can be loaded with vertical forces. Horizontal force, on the other hand, can be taken by inclined piles. The more the piles are inclined, the greater is the capacity (Stål, 1984), simply derived from geometry. Horizontal forces can also be taken as moment in both inclined and vertical piles by using the lateral resistance from the surrounding soil. In order to handle bending moment, there must be lateral resistance of the soil or pairs of pile forces in the pile group. These pairs can be formed by piles whose pile force extensions does not intersect at one point. Since piles are long, their bending resistance is low. This is improved by lateral soil resistance.

Suitable design of piles due to different loading is demonstrated in Figure 2.5, assuming no lateral resistance from the soil. The first pile group can only resist forces applied at the pile center (for definition see Section 2.4.1) and the second one is not stable for horizontal force. The third arrangement can handle all types of loads. If lateral resistance of the soil is assumed, all pile groups are be able to handle horizontal force and bending moment.

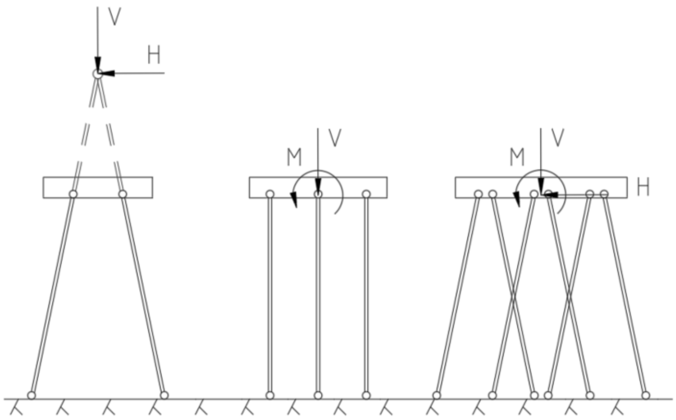


Figure 2.5: Static actions of piles. Figure inspired by Stål (1984).

2.4.1 Load Center and Pile Center

When considering several load cases, a method that makes use of the load center and the pile center is presented in Stål (1984). The magnitude of the generated forces in the piles is said to depend on the relationship between the load center and the pile center. An optimal pile center is defined as the state when external forces acting on the pile center cause only translations and no rotations, as illustrated in Figure 2.6. Thus, the pile center should generally be located as close to the load center as possible to avoid pile forces due to moment acting at the pile center.

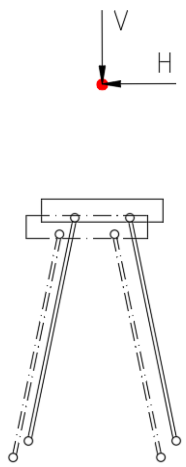


Figure 2.6: External forces acting on the pile center, highlighted in red, cause only translations and no rotations.

The location of the pile center depends on the stiffness of the pile group, i.e. geometric and material properties. The mathematical expression of the pile center is presented in

Section 2.8. If all piles have equal stiffness and the surrounding soil is not considered, the pile center is a theoretical intersection of the extensions of the neutral axes of the piles, as in the left illustration in Figure 2.5. When the stiffness of the soil is also taken into account, the pile center becomes a bit more complicated to determine; this is described more in Section 3.4.

It may not always be optimal to locate the pile center at the exact same position as the load center. This is because it can be advantageous to use the effect of bending moment to reduce pile tension.

In order to reduce the pile forces due to applied moment, the horizontal locations of the piles should be placed as far from the pile center as possible, and thereby increasing the moment of inertia of the pile group (Stål, 1984). However, if the piles are placed far from the applied loads, other problems may appear, such as assuring sufficient stiffness and capacity of the pile cap. A compromise between both phenomena may be preferable.

2.4.2 Symmetrical Pile Group

Forces acting on the piles arise from structural weight and different types of varying loads, such as traffic, wind and thermal effects. One can argue that a bridge is loaded fairly symmetrically when there are traffic loads, braking loads and thermal effects in both directions and wind from both sides. Thus, it is reasonable to design symmetrical pile groups. The symmetry can be around one axis or two, depending on the loading situation. Examples of unsymmetrical loading are centrifugal forces acting on a curved bridge and earth pressure at an abutment.

2.5 Regulative Requirements on Pile Groups

When designing a pile group, several regulations, such as Eurocode and publications from the Swedish Transport Administration, must be considered. In addition to the regulations, there are usually requests for an economical and sustainable structure. The contractor that constructs the pile group may also have requirements on the arrangement of piles, due to practical reasons at the building site. This is treated separately in Section 2.6.

2.5.1 Geometric Requirements

The most important geometric requirements are minimum distance between the piles, minimum distance between the pile and the edge of the pile cap and maximum pile inclination.

There are two main reasons for having a minimum distance between piles – group effect and risk for collision when installing the piles. Group effect is a concept where the piles stand so close to each other that the stress field around a pile affects the pile next to it. Risk for collision is present when piles stand close to each other and their positions deviates during installation. By ensuring sufficient distance between the piles, this is avoided (Svahn and Alén, 2006). The requirements on minimum distances have a significant impact on the size of the pile cap.

The Swedish Transport Administration presents minimum distances in *Råd Brobyggande* (Trafikverket, 2019b). For piles leaning from each other, the minimum distance at the pile cap is 0.8 meter. Parallel piles are more likely to collide and have therefore stricter requirements that depends on the diameter and the length of the pile.

2.5.2 Ultimate Limit State Requirements

The capacity of the pile group is restricted to the minimum of the structural resistance of the individual piles and the geotechnical resistance of the soil. Which failure mode that governs depends on the length of the piles, where the geotechnical capacity governs short piles and structural capacity governs long piles (Svahn and Alén, 2006). Practically most piles can be seen as long piles, which undermine the relevance of the geotechnical capacity. However, for tension forces the geotechnical capacity may be decisive and should be verified. The structural resistance of the piles includes, for instance, moment and normal force capacities and the combination of these.

As Figure 2.7 illustrates, the conceptual behaviour for combined normal force and bending moment is quite different depending on whether the pile is made of steel, timber or reinforced concrete.

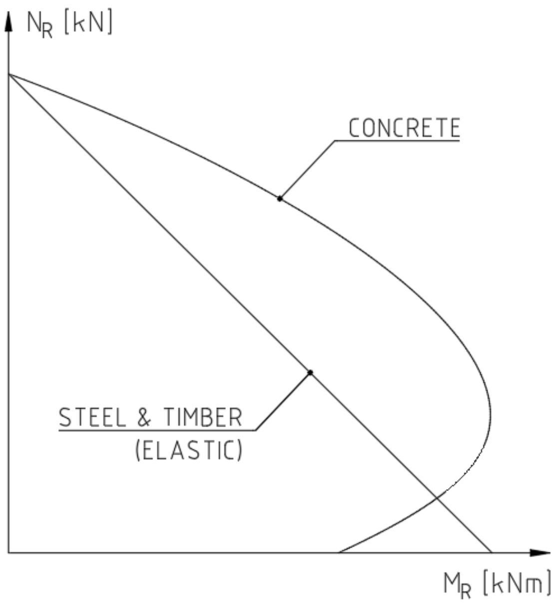


Figure 2.7: Conceptual cross section capacity in ultimate limit state, for piles of different materials. Inspired by Pålkommisionen 96.

For steel and timber piles, a simplified elastic interaction formula expressed as

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \tag{2.1}$$

can be used in preliminary design. The design bending moment is the resultant from the moments M_x and M_y acting around the x- and y-axis. Only the size of the resultant, and not the direction, is considered using

$$M_{Ed} = \sqrt{M_x^2 + M_y^2} \tag{2.2}$$

The capacities $N_{b,Rd}$ and $M_{c,Rd}$ are the compression buckling capacity and the elastic moment capacity. The design moment capacity may be in another direction than the imposed bending moment; a simplification on the safe side. The moment capacity for a quadratic pile is usually lowest for bending around the diagonal. Other effects such as local buckling is not considered in this formula.

For reinforced concrete, the interaction of normal force and moment is not linear. For a given moment, an increase in normal force can be favourable up to a certain point, and thereafter unfavourable.

Irrespective of material, buckling of piles must be considered. For steel and timber, this is done by calculating a buckling reduction factor, and for reinforced concrete by determining second order effects. In both approaches, the stiffness of the soil should be considered.

The tensile capacity can be checked using

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{2.3}$$

There are requirements, other than ultimate limit state, that may have a decisive impact on the pile group. Serviceability limit state, fatigue and accidental events are examples of such requirements. If there are on-going settlements in the area, negative skin friction requires special treatment.

2.6 Practical Requirements from Interviews

To determine an optimal arrangement of the piles, the designer should not only concentrate on the technical aspects but also be aware of the practical part of installing a pile group. By considering the practical aspects at an early stage, the work may be facilitated at the building site and safety for the workers can be assured and both time and money can be saved. Based on interviews with three contractors in Sweden, practical requirements were obtained. The contractors had an average experience of 20 years. The questions asked during the telephone interviews are presented in Appendix A.

The overall intention is to utilize all piles as much as possible, but there are also reasons not to. If the pile group is designed with high extent of utilization, deviations occurring at installation can have a large impact on time and cost. If the piles are either deviating in position, direction or inclination, or are completely eliminated, the pile group may not work and a new pile must be installed. This is both time consuming and expensive, and the contractors argue that it is much more expensive to supplement the pile group by adding a pile afterwards, than to design a robust pile group from the start (Alheid, 2020; Berntsson, 2020; Blomqvist, 2020).

Depending on the type of pile and dimension of the pile, different equipment and machines are needed. If the pile group consists of many different pile sizes, more equipment is needed, which increases the cost. It is also time consuming to change the equipment (Blomqvist, 2020).

It may seem advantageous to install one pile with large dimensions instead of several small piles, but one must not forget that they require different machines. The size, weight and cost of the piling machines increase with pile size. Thus, several aspects need to be considered when choosing quantity and dimensions of the piles (Blomqvist, 2020).

To avoid having to constantly move the machine back and forth, the piles not yet installed are often stored on the ground between the already installed piles (Berntsson, 2020). Therefore, arranging piles in straight rows facilitates the handling of the piles and reduces the risk for hitting the already installed piles when picking up a pile with the crane.

The more the pile is inclined, the more efficiently it handles horizontal forces (Stål, 1984). However, since there are limitations of the pile crane and safety requirements for the working environment, when designing a pile group the piles should not have a larger inclination than 4:1. It is easier to install a vertical pile than an inclined one, as gravity improves the precision of a vertical pile but impairs the precision of an inclined pile. The risk of the machine tipping is a severe safety issue that increases when inclining piles more than 4:1 (Alheid, 2020; Berntsson, 2020).

To summarize, important aspects mentioned by pile contractors is to

- construct robust pile groups
- use same pile dimensions when possible
- arrange piles in ”grids” with straight rows and columns
- limit inclination for safety reasons

2.7 Modelling Aspects for Pile Groups

Choosing an appropriate model is crucial to capture the behaviour of the structure. By the use of frame analysis, displacements and section forces in the piles are easily calculated for one load case at a time. An important assumption for the frame analysis method is that the pile cap is a rigid body. This assumption and other aspects of modelling will be discussed in the upcoming sections.

2.7.1 Rigidity of the Pile Cap

The pile cap is assumed to be infinitely stiff, a so called rigid body. This implies that the pile cap does not deform under loading but will displace as a whole, and this makes displacement calculations easy. As soon as the displacement of the pile cap is known, the displacements of the pile tops can be calculated from geometry. This model is accepted for Swedish bridges according to Trafikverket (2019b), as long as the pile cap is stiff enough. This is evaluated by calculating the stiffness ratio between the piles and the pile cap as described by Bergdahl et al. (1993).

2.7.2 Support Conditions

In the simple model usually applied in pile design, the pile has two supports, one at either end. The upper support represents the connection to the pile cap, and the lower support represents the lateral resistance of the soil.

For a reinforced concrete pile, the hinged connection at the pile top represents the casting of the pile into the pile cap with no extended reinforcement. If the connection is modelled as moment stiff, there must be sufficient reinforcement extending from the pile into the pile cap that ensures transfer of moment, where the relative rotation between the pile cap and the pile is negligible. For other types of piles, there are equivalent solutions for hinged and moment stiff connections to the pile cap.

Modelling the pile end as a hinge implies that the pile does not rely on any lateral resistance of the soil. This is equivalent to assuming that the piles are standing in air. One common way to take lateral resistance of the soil into account, is to model the pile with a fictitious length and a moment stiff connection at the end. Another way is to model the soil as a continuous elastic support along the pile. These methods are further explained in Section 2.8.1.

Traditionally, calculations have been done assuming hinged supports, meaning that the piles only resist load in their axial direction.

2.7.3 Lateral Resistance of the Soil

When the pile displaces it will interact with the surrounding soil. Pile-soil interaction refers to both vertical and horizontal interaction, where the horizontal pile-soil interaction is also called the lateral resistance of the soil.

As mentioned before, pile group designers describe that calculations are traditionally done by not considering the lateral stiffness of the soil. Only if structural integrity cannot be assured without lateral resistance, are such calculations carried out. Omitting the lateral resistance from the soil is often a safe side assumption. However, according to the Swedish Transport Administration, the pile group should be calculated both with high and low lateral resistance from the soil, as there are cases when including lateral resistance of the soil governs the design (Trafikverket, 2019a). For pile group analysis, low and high lateral resistance is decisive with respect to normal force respectively bending moment in the pile member. For design of single piles, the lateral resistance of the soil increases the buckling resistance.

2.8 Frame Pile Group Model

Matrix calculations presented in Bredenberg and Broms (1978) can be used to facilitate the calculations of displacements and pile forces when evaluating a pile group. To use this method, the pile group must be physically stable. This means that no external force can cause a large displacement of the pile cap. The following sections describes the matrix calculations.

A global coordinate system is inserted at the origin of the pile cap, see Figure 2.8. The origin is where the loads are applied, and must not necessarily be the center of the pile cap.

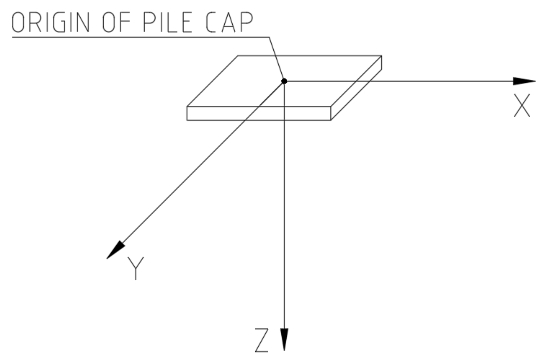


Figure 2.8: Global coordinate system.

A local coordinate system is inserted at the pile top of each individual pile and positive forces and moments are defined as shown in Figure 2.9.

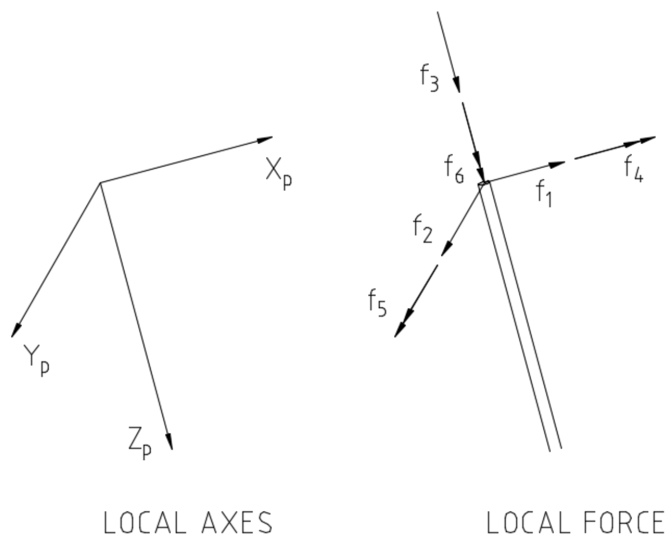


Figure 2.9: Local coordinate system and positive forces and moments.

The relationship between generated local section forces F_p, pile stiffness K_p and displacements Δ_p at the pile top is described as

$$F_p = K_p \Delta_p \tag{2.4}$$

$$F_p = \begin{bmatrix} f_1 \\ f_2 \\ f_3 \\ f_4 \\ f_5 \\ f_6 \end{bmatrix}; \quad K_p = \begin{bmatrix} k_{11} & 0 & 0 & 0 & k_{15} & 0 \\ 0 & k_{22} & 0 & k_{24} & 0 & 0 \\ 0 & 0 & k_{33} & 0 & 0 & 0 \\ 0 & k_{42} & 0 & k_{44} & 0 & 0 \\ k_{51} & 0 & 0 & 0 & k_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & k_{66} \end{bmatrix}; \quad \Delta_p = \begin{bmatrix} \delta_1 \\ \delta_2 \\ \delta_3 \\ \delta_4 \\ \delta_5 \\ \delta_6 \end{bmatrix}$$

The pile constants k_{ij} forming the stiffness matrix K_p are determined according to Section 2.8.1.

The transformation between the local coordinate system of the pile (X_p, Y_p, Z_p) and a local coordinate system parallel to the pile cap $(X_p^{'}, Y_p^{'}, Z_p^{'})$, is done by the use of the transformation matrix A_p, that takes into account the inclination β and the in-plane direction α of the pile. This transformation is illustrated to the right in Figure 2.10.

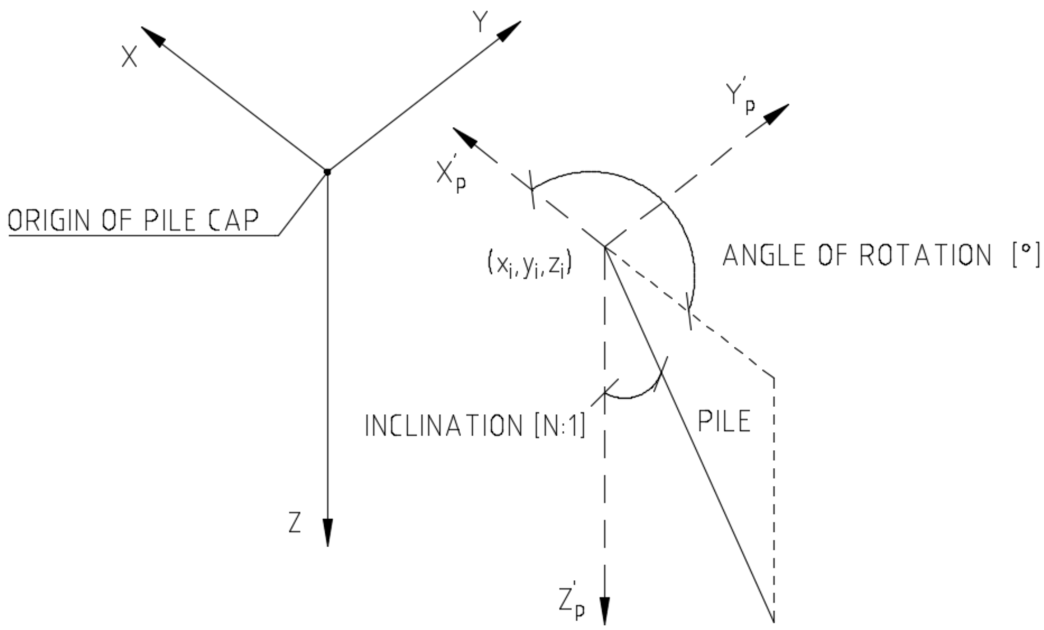


Figure 2.10: The global coordinate system and a local coordinate system parallel to the pile cap. Angle of rotation α and inclination of pile β , for the local systems, are shown in the right figure.

$$A_p = \begin{bmatrix} A' & 0 \\ 0 & A' \end{bmatrix}; A_p' = \begin{bmatrix} \cos\beta\cos\alpha & -\sin\alpha & \sin\beta\cos\alpha \\ \cos\beta\sin\alpha & \cos\alpha & \sin\beta\sin\alpha \\ -\sin\beta\cos\alpha & 0 & \cos\beta \end{bmatrix} \tag{2.5}$$

The total transformation matrix D_p is introduced to take into account the direction, inclination and location of the pile.

$$D_p = C_p A_p \tag{2.6}$$

where C_p takes into account the location and the degree of attachment of the piles. The coordinates of the individual piles in the global system are called x, y and z. These describe the distances between the individual pile and the origin of the pile cap. The degree of attachment between the piles and the pile cap is implemented using $m = 0$ for hinged connection and $m = 1$ for moment stiff connection.

$$C_p = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -z & y & m & 0 & 0 \\ z & 0 & -x & 0 & m & 0 \\ -y & x & 0 & 0 & 0 & m \end{bmatrix} \tag{2.7}$$

Since the applied forces are assumed to be known, the displacements of the pile cap can be calculated using the relationship between the applied forces R and the displacements U;

$$R = SU \tag{2.8}$$

$$U = S^{-1}R \tag{2.9}$$

where S is the symmetrical stiffness matrix (6x6) where element S_{ij} is the force in direction i which will generate a movement of the pile cap in direction j equal to one. The contributions of all piles are summed in a stiffness matrix at the pile cap origin.

$$S = \sum_{p=1}^{n} S'_p \tag{2.10}$$

where

$$S'_p = D_p K_p D_p^T \tag{2.11}$$

When the global displacements are calculated, the displacements of the piles can be determined from

$$\Delta_p = D_p^T U \tag{2.12}$$

Finally, the pile forces and moments F_p are calculated using the relationship between the stiffness of the pile and the displacements (Equation (2.4)). The design pile forces are determined according to Section 2.8.2.

2.8.1 Calculation of Pile Stiffness

The stiffness of the pile group can be calculated both with and without consideration of the lateral resistance from the soil. The interaction between the pile and the soil can be considered by adopting one of the two methods illustrated in Figure 2.11. In Figure a) a model where the pile is moment stiff connected at a fictitious length is shown, based on empirical data for friction soil. Figure b) is a Winkler model where continuous springs provides a horizontal resistance to a semi-infinite long beam. Generally the springs can be both linear and non-linear, and the stiffness can vary along the pile.

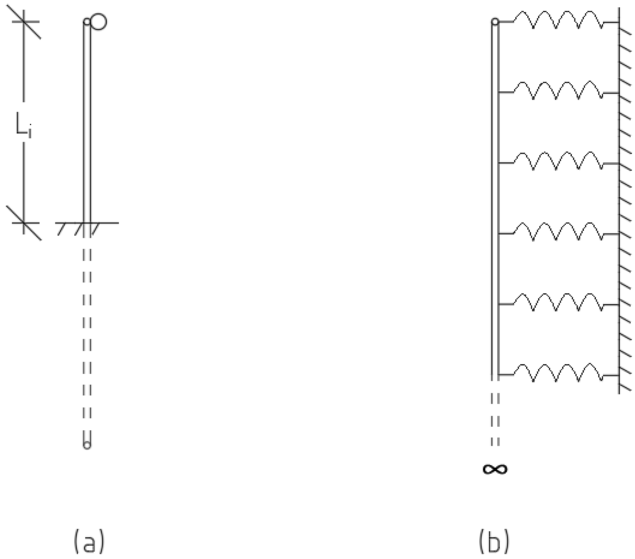


Figure 2.11: Two models for considering lateral resistance of the soil, where a) is a moment stiff end model and b) is the Winkler model.

The two models shown in Figure 2.11 are the basis for the work done by Bredenberg and Broms (1978). The Winkler model with constant linear stiffness along the pile is adopted for cohesive soil, and the moment stiff connection model is used for friction soil. A fictitious length L_i for friction soil and a stiffness parameter, also called characteristic length, L_e for cohesive soil is calculated from the following expressions.

$$L_i = 1.8\sqrt[5]{\frac{EI}{n}} \tag{2.13}$$

$$L_e = \sqrt[4]{\frac{4EI}{kd}} \tag{2.14}$$

where EI is the bending stiffness of the pile, which is assumed to be constant along the pile, n is the linear subgrade modulus for friction soil and kd is the subgrade modulus for cohesive soil. These expressions are valid as long as the pile is longer than $4L_i$ respectively $3L_e$.

The stiffness matrix is calculated differently for cohesive and friction soil, as different models are used. This results in three different pile stiffness matrices K_p (cohesive soil, friction soil and no soil). All pile stiffness matrices presented contains the variable m

that allows for implementing a degree of stiffness of the connection between pile and pile cap where $m = 0$ for hinged connection and $m = 1$ for moment stiff connection.

For friction materials, the pile stiffness coefficients derive from the unit displacements and rotations imposed in Figure 2.12. This is basically from beam theory.

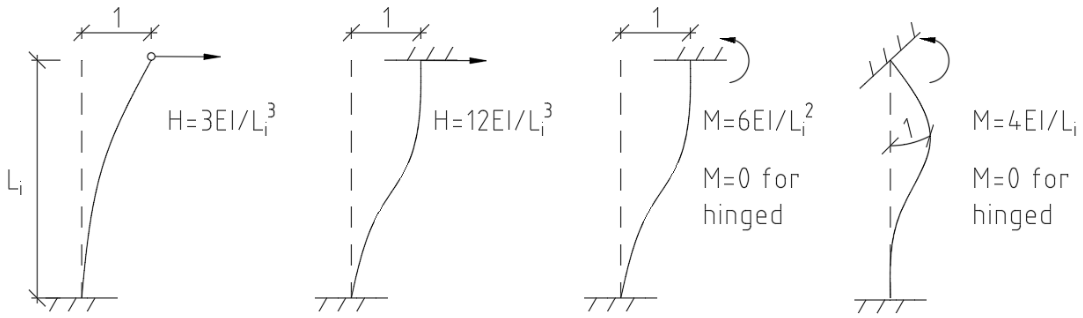


Figure 2.12: The stiffness coefficients for friction soil origin from the resulting forces and moments from unit displacements and rotations on the moment stiff connected beam. Axial displacement and twist are not illustrated in the figure.

The coefficients form the following pile stiffness matrix for friction soil.

$$K_p = \begin{bmatrix} \frac{(3m+1)3EI}{L_i^3} & 0 & 0 & 0 & \frac{m6EI}{L_i^2} & 0 \\ 0 & \frac{(3m+1)3EI}{L_i^3} & 0 & -\frac{m6EI}{L_i^2} & 0 & 0 \\ 0 & 0 & \frac{AE}{L} & 0 & 0 & 0 \\ 0 & -\frac{m6EI}{L_i^2} & 0 & \frac{m4EI}{L_i} & 0 & 0 \\ \frac{m6EI}{L_i^2} & 0 & 0 & 0 & \frac{m4EI}{L_i} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{mJG}{L} \end{bmatrix} \tag{2.15}$$

For cohesive materials, the pile stiffness coefficients are derived analogously for a beam on elastic foundation. The coefficients form the following pile stiffness matrix for cohesive soil.

$$K_p = \begin{bmatrix} \frac{(m+1)2EI}{L_e^3} & 0 & 0 & 0 & \frac{m2EI}{L_e^2} & 0 \\ 0 & \frac{(m+1)2EI}{L_e^3} & 0 & -\frac{m2EI}{L_e^2} & 0 & 0 \\ 0 & 0 & \frac{AE}{L} & 0 & 0 & 0 \\ 0 & -\frac{m2EI}{L_e^2} & 0 & \frac{m2EI}{L_e} & 0 & 0 \\ \frac{m2EI}{L_e^2} & 0 & 0 & 0 & \frac{m2EI}{L_e} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{mJG}{L} \end{bmatrix} \tag{2.16}$$

If the influence of the soil is ignored, the pile stiffness matrix is formed by the well known coefficients for a simply supported beam or a beam with one fixed support and one simple support.

$$K_p = \begin{bmatrix} \frac{m3EI}{L^3} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{m3EI}{L^3} & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{AE}{L} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{m3EI}{L} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{m3EI}{L} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{mJG}{L} \end{bmatrix} \tag{2.17}$$

2.8.2 Calculation of Design Pile Forces

Design section forces are calculated in accordance with Bredenberg and Broms (1978) for each pile so that the resulting stresses can be compared with the permitted values. Maximal shear force f_T occur at the pile top and is calculated as the resultant of the horizontal forces.

$$f_T = \sqrt{f_1^2 + f_2^2} \tag{2.18}$$

The maximal bending moment f_M is assumed to occur at the top of the pile for a moment stiff connection and is calculated as

$$f_M = \sqrt{f_4^2 + f_5^2} \tag{2.19}$$

For a hinged connection, the maximal bending moment is assumed by Bredenberg and Broms (1978) to be

$$f_M = 0.43 f_T L_i \tag{2.20}$$

$$f_M = 0.32 f_T L_e \tag{2.21}$$

The normal force f_3 and the twisting moment f_6 are considered constant at the sections of interest, i.e. where maximum stresses are present.

When verifying the final capacity for the single piles, the location of maximum moment should be determined as the location where the shear force is zero.

2.8.3 Equilibrium Check

An equilibrium check is carried out to conclude that the calculated result is reliable. This is also described in Bredenberg and Broms (1978), and is done by transforming the generated forces F_p into a coordinate system that is parallel to the axis of the pile cap.

$$F'_p = A_p F_p \tag{2.22}$$

The transformed forces and moments at the pile top are then in equilibrium with the force vector P_p in the origin of the pile cap.

$$P_p = C_p F'_p \tag{2.23}$$

The differential between the applied loads at the cap and the sum of all pile forces and moments will show how accurate the calculations are. The difference should be close to zero to ensure that there are no errors in the calculations.

$$R = \sum_{p=1}^{n} P_p \tag{2.24}$$

3 Method

The development of the pile group optimization program is the primary aim of this thesis. This requires formulating desired input data and stating an objective function, as well as choosing design variables and state variables and constraints on them. Some of the constraints were based on the results from the interview study presented in Section 2.6. The program was verified by comparing results with an existing program and by testing the optimization function on simple load cases. Four case studies were carried out to evaluate the program.

An additional study of the often assumed correlation between load center-pile center distance and resulting tension forces in the piles was done, using two of the case studies as reference objects.

3.1 Pile Group Optimization Program

The pile group optimization program was developed in Python. An overview of the structure of the program is presented in Figure 3.1. The number of piles was a manual input. Every pile group randomly generated was analyzed for every load case. If the pile group was stable and the generated section forces were smaller than the capacities, it was assessed according to the objective function. The pile group with lowest objective function was identified as the most optimal pile group. If the optimal pile group had low utilization, the number of piles was reduced manually.

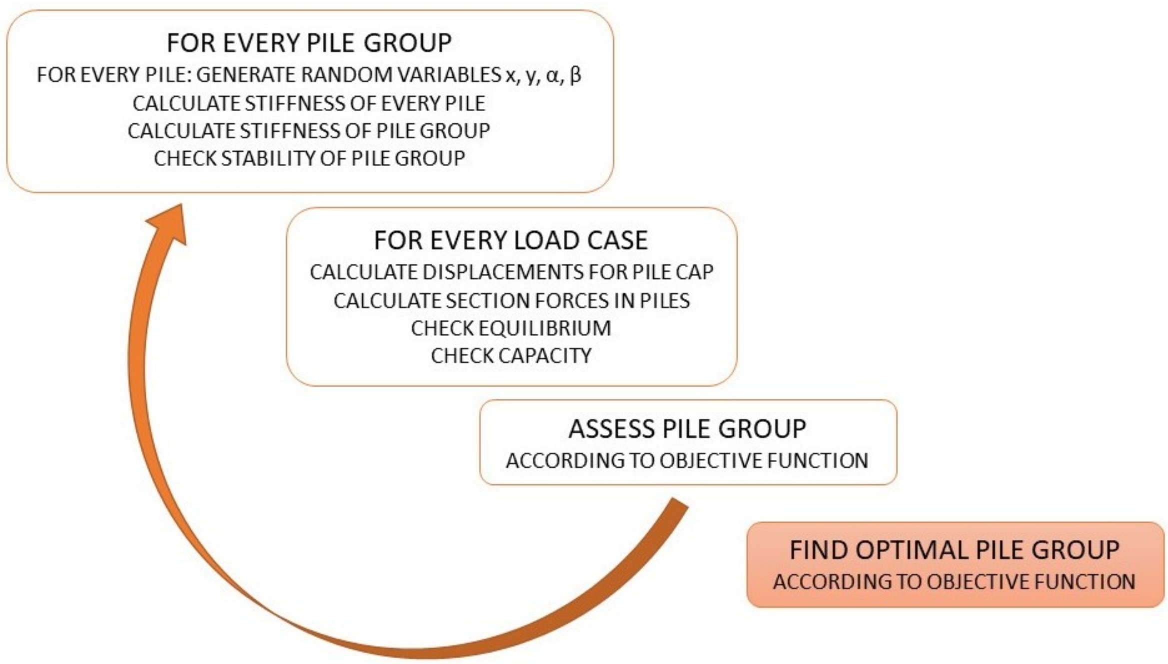


Figure 3.1: Flow chart for optimization program.

3.1.1 Structural Model and Assumptions

In Section 2.7, important modelling aspects such as rigidity of the pile cap, support conditions and lateral resistance of the soil were presented. Here, the chosen model and assumptions for the optimization program is stated.

The pile group calculations were performed both with high and without lateral resistance of the soil, regardless of the type of bridge (not only for railway bridges). This is an assumption on the safe side.

The piles were assumed to be end bearing and the pile cap to be infinitely stiff. For the no soil calculations, the pile end was hinged, whereas for the soil calculations the concept of a moment stiff connection at a fictitious length was used for friction soil and the Winkler model was used for cohesive soil, both described in Section 2.7.3.

Displacements and pile forces were calculated according to the frame analysis method described in Section 2.8.

The cross section was chosen to be the same for all piles in the pile group. This was done due to practical reasons concerning equipment and pile machines described in Section 2.6. Also, the pile length was the same for all piles, which is a limitation.

3.1.2 Input Data

The following input data, that the designer could define, was implemented in the program.

- *Loads:* The loads (3 forces, 3 moments), for each load case, applied to the pile cap origin at the cut-off level of the piles.
- *Number of piles:* Chosen from experience or estimated based on simple calculations.
- *Pile capacity:* The pile capacity was implemented according to ultimate limit state using a simplified method shown in Section 3.1.5.
- *Deformation:* Allowable deformation for the pile cap.
- *Pile top conditions:* Options fixed or hinged.
- *Pile parameters:* Geometrical data such as pile length and dimensions of cross section, and material properties (Young's modulus and shear modulus).
- *Soil parameters:* Options for soil type were cohesive or friction soil, and corresponding soil parameters subgrade modulus or linear subgrade modulus.
- *Pile cap:* The dimensions of the pile cap in both x- and y-direction.
- *Symmetry:* The pile group could be restricted to either double or single symmetry.

3.1.3 Objective Function

Based on the general expression of structural optimization (SO) presented by Christensen et al. (2008), an objective function was formulated.

$$SO = \begin{cases} \text{minimize } f(a, b(a)) \text{ with respect to } a \text{ and } b \\ \text{subject to} \begin{cases} \text{design constraints on } a \\ \text{behavioral constraints on } b \\ \text{equilibrium constraints} \end{cases} \end{cases}$$

where

$f(a, b(a))$ is the objective function
a is a function or vector that describes the design
b is a function or vector that represents the response of the structure for a given a, also called state variables

The objective function f was stated to describe the force difference between the maximum normal force and the minimum normal force in the piles. It measured the distribution of forces in the pile group and was formulated as

$$f = max(N_{Ed,max}) - min(N_{Ed,min}) \tag{3.1}$$

High value of f meant large difference between the maximum and minimum normal forces, and vice versa. The structural optimization intended to decrease the difference between the maximum and minimum normal forces in the pile group. The objective function was evaluated in parallel for conditions with and without lateral resistance from the soil, where the highest objective function was decisive for the pile group.

The variables were the design variables a and the state variables b, which are described in detail in the following section.

3.1.4 Variables

The design variables a had uniform distribution and varied during optimization. They formed a matrix with n rows, where n was the number of piles.

$$a = \begin{bmatrix} a_i \\ \vdots \\ a_n \end{bmatrix}; \quad i = [1, 2, ..., n]$$

Every pile i had four parameters that could vary. With a coordinate system defined as in Section 2.8 the parameters were formulated in a vector a_i as (see Figure 2.10)

$$a_i = \begin{bmatrix} x_i & y_i & \alpha_i & \beta_i \end{bmatrix}$$

where

x_i is the x-coordinate
y_i is the y-coordinate
α_i is the in plane direction of the pile
β_i is the inclination of the pile

The state variables b were chosen to represent the response of the structure, and included section forces b_i for every pile i and deformations U for the total system.

$$b = f(a) = \begin{cases} b_i \\ U \end{cases}$$

where $f(a)$ represent Equation (2.4)-(2.11). The generalized section forces consisted of 3 forces and 3 moments at every pile top.

$$b_i = \begin{bmatrix} N_{i,xEd} & N_{i,yEd} & N_{i,zEd} & M_{i,xEd} & M_{i,yEd} & M_{i,zEd} \end{bmatrix}$$

The deformations were presented as 3 translations and 3 rotations at the pile cap.

$$U = \begin{bmatrix} u_x & u_y & u_z & w_x & w_y & w_z \end{bmatrix}$$

3.1.5 Constraints

The constraints were divided into design constraints, behavioral constraints and equilibrium constraints. The upper and lower bounds were, for some parameters, chosen by the designer and, for others, determined in the program.

As described in Section 3.1.4, the pile had parameters that could vary. Due to geometrical limitations the coordinates (x, y) of the pile could only vary within the limits of the pile cap. The minimum allowable pile distances were also constraining the pile coordinates. A minimum distance of 0.8 meter was used as constraint in the program. However, the pile distance 0.8 meter is only valid for piles leaning away from each other, as described in Section 2.5.1. The check of whether this assumption is true, is left for the user to ensure. Since there are limitations of the pile equipment and safety requirements for the working environment, the inclination β of the piles was limited to 4:1. The in plane direction α of the piles was also limited by only letting the piles lean away from the origin of the pile cap. The number of possible directions α and inclinations β were limited by the user. The more options, the more optimal pile groups can be found, but the simulations take longer time to complete. This flexibility allows for a reasonable number of options for the specific case studied.

The behavioral constraints used in the program were the limitation of allowable section forces due to ultimate limit state and allowable deformations. These parameters depend on selected pile type, cross section and the geotechnical conditions, and should be defined by the user. Interaction of normal force and moment for steel and timber was analyzed elastically as described in Section 2.5.2. The non-linearity of reinforced concrete described in the same section, was approximated with three linear curves between four points, as illustrated in Figure 3.2(b). These points were given by the user. The capacity curve could be approximated in more accurate ways, but this simplification is on the safe side.

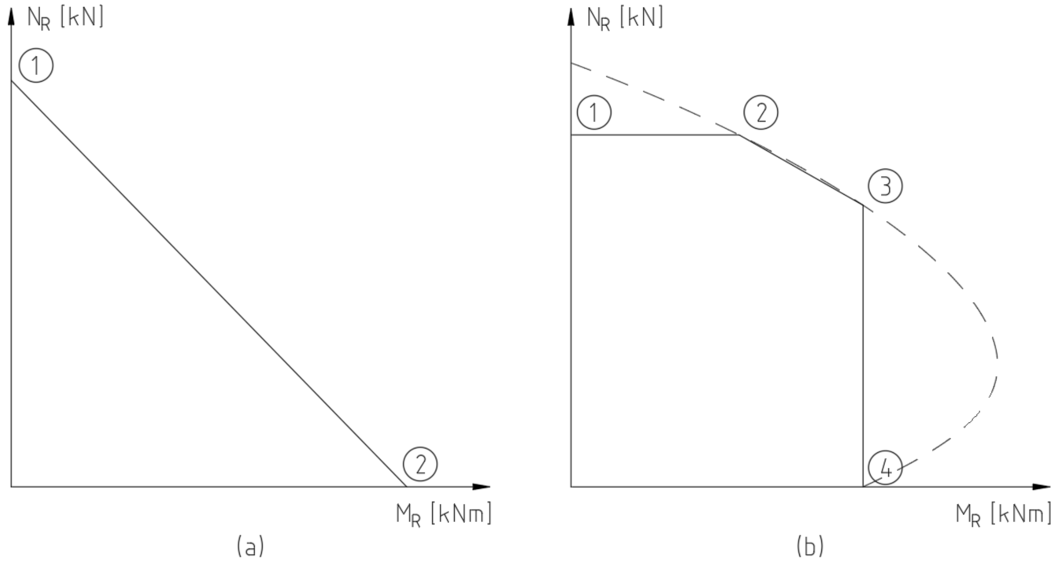


Figure 3.2: (a) Linear interaction diagram for steel and timber with points 1 and 2 provided by the user. (b) Non linear interaction diagram for reinforced concrete with approximated linear curves between points 1-4 provided by the user.

To ensure a stable pile group, an equilibrium constraint was implemented by restricting the determinant of the stiffness matrix S to be non-zero.

A limitation of the program was that the program cannot assure that there are no collisions between the piles below and above ground. However, this is a constraint in reality.

3.2 Verification of Program

Verification of the developed program was done by comparing deformations and section forces for a given pile group with results from the commonly used program Rymdpålgrupp (Eurocode Software AB, 2017).

Furthermore, the optimization functionality was verified by investigating pile groups with a set number of piles, subjected to a unit load. 10 000 random pile groups were generated and the six best pile groups were presented and compared with the theory of optimal pile groups, presented in Section 2.4, for that load. The loads tested, one at a time, were

$$N_z = 100 \text{ kN}, \; N_x = 10 \text{ kN}, \text{ and } M_x = 10 \text{ kNm}$$

These loads are much lower than real loads acting on a pile foundation for a bridge. They are only used to evaluate the conceptual arrangement of the pile group.

3.3 Case Studies

Four case studies were carried out of pile groups already designed, and some of them already built. The cases chosen were mid support and abutment of a pedestrian bridge, mid support of a road bridge and abutment of a railway bridge. The existing designs were shared by the owners of the bridges and were compared to the designs resulting from the random pile group generator.

Input data for all case studies is presented in a summarized table in Section 3.3.4. All input data, including the applied loads, is presented in Appendix C-F. The external forces are in all cases applied at the origin of the pile cap, at the pile cut-off level. All pile groups were designed for load combinations in ultimate limit state. When simulating random pile groups, background information from the existing pile group was used with some exceptions. The most important differences between the assumptions made for the reference pile groups and the assumptions made in this project, is the lateral resistance of the soil and the inclination of piles. In this project, calculations were made for both lateral resistance of the soil and no lateral resistance of the soil and the maximal inclination was set to 4:1. As mentioned before, the angle of rotation of the piles varied with different increments, depending on the complexity of the specific case.

3.3.1 Case I and II - Pedestrian Bridge Mid Support and Abutment

The pedestrian bridge was built in 2019, across Härlövsängaleden in Kristianstad. The substructure of the bridge consists of two abutments and two mid supports, see Figure 3.3.

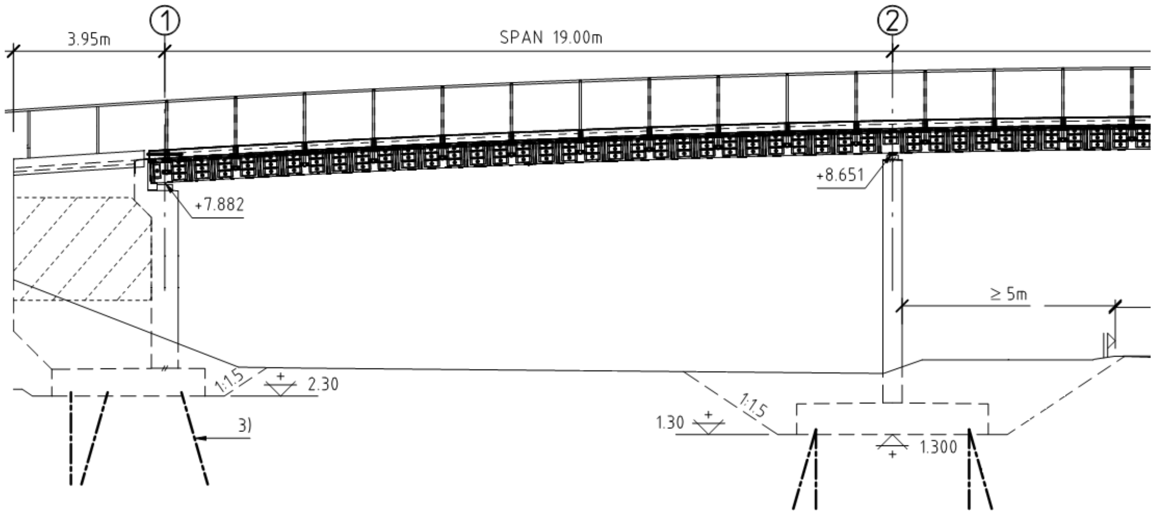


Figure 3.3: Elevation of half of the pedestrian bridge showing abutment (1) and mid support (2). Figure adapted from Kristianstad Municipality (2019).

For Case I, the mid support was investigated. The existing pile group was symmetric around two axes and had 8 reinforced concrete piles, all with the inclination 3.5:1.

For the random pile group generator, 8 piles in a double-symmetric arrangement was used. The inclination options were 4:1, 8:1 and vertical, and the rotation varied in 15°-increments.

The abutment of the same pedestrian bridge as in Case I was used for Case II, see Figure 3.3. The existing pile group was symmetric around the x-axis and had 6 reinforced concrete piles. As for Case I, the piles had the inclination 3.5:1.

For Case II, the random pile group generator produced pile groups with 6 piles symmetric around the x-axis. The inclination options were 4:1, 8:1 and vertical, and the rotation varied in 15°-increments.

3.3.2 Case III - Road Bridge Mid Support

The third case was a road bridge along the E45 road, over the eastern entrance of the Marieholm tunnel in Gothenburg. The structure was built in 2015. An overview of a part of the bridge is given in Figure 3.4.

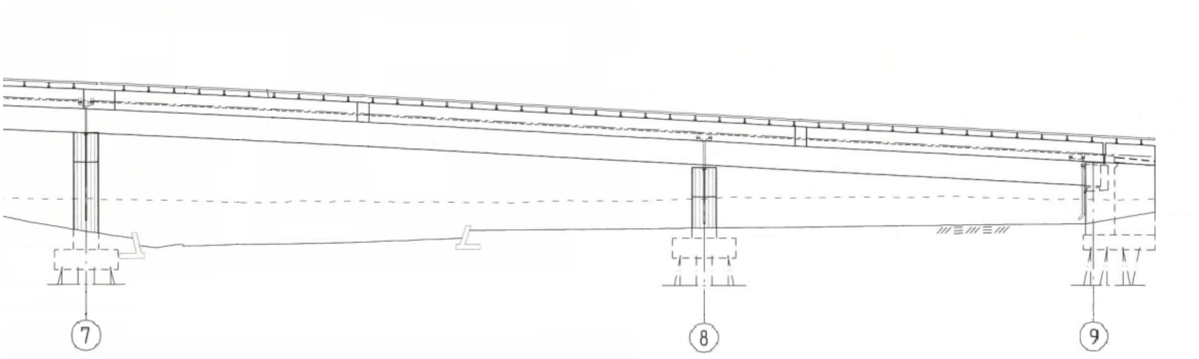


Figure 3.4: Overview of the bridge. Elevation for Case III. The intermediate support (7) is the one investigated in this case study. Figure adapted from Trafikverket (2015).

The existing pile group was symmetric around two axes and had 28 piles with the inclinations 20:1, 12:1 and 8:1.

The random pile group generator was tested with 24 piles, symmetric around two axes. The pile inclination possibilities were 4:1, 8:1 and vertical, and the angle of rotation was free to vary in 45°-increments.

3.3.3 Case IV - Railway Bridge Abutment

The last case was a railway bridge, planned to be a part of Roslagsbanan over Ullna Kvarnväg in Stockholm. An overview of the bridge is presented in Figure 3.5; Figure 3.6 provides a more detailed drawing of the abutment. The pile group was symmetric around the x-axis and had 20 piles, all with the inclination 4:1.

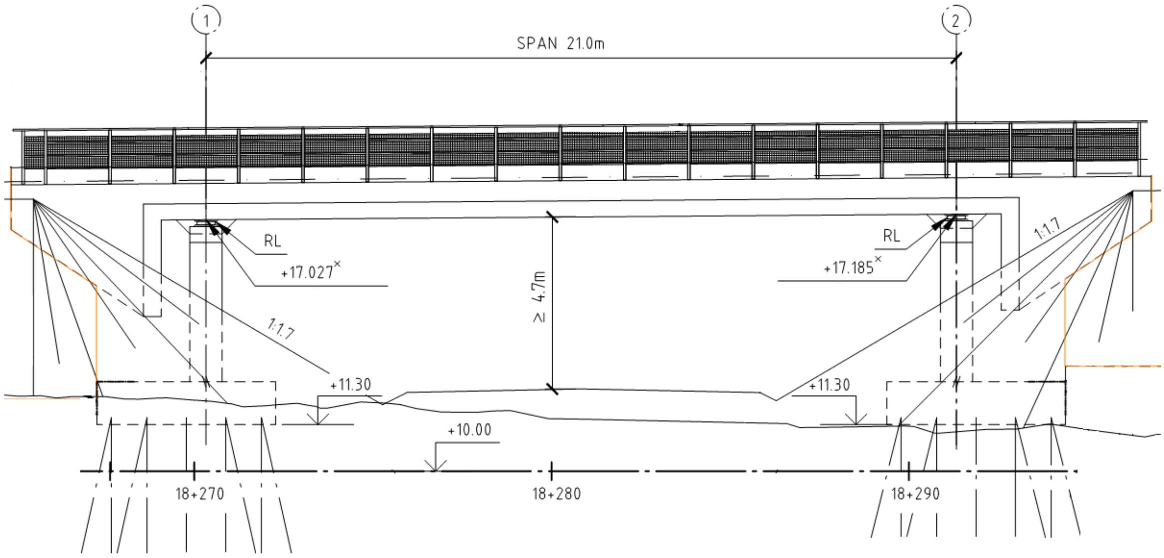


Figure 3.5: Overview of the bridge studied in Case IV. The two abutments are equal but inverted. Abutment (1) is the one illustrated in the upcoming figure. Figure adapted from SL, Stockholm Public Transport (2016).

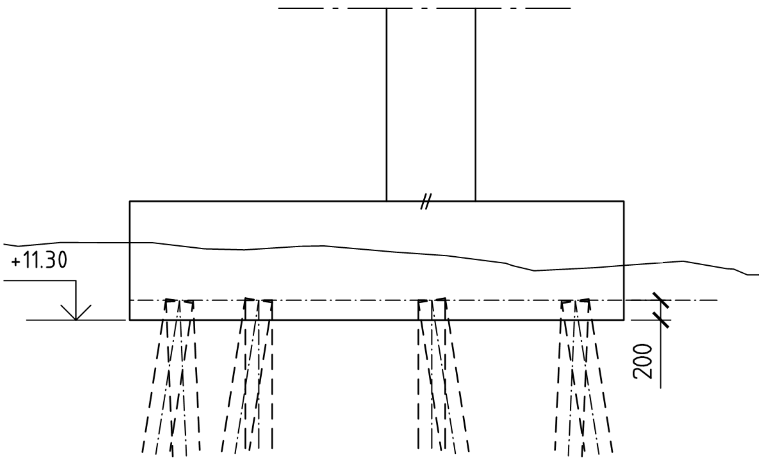


Figure 3.6: Elevation for the abutment. The bridge is spanning from this abutment to the right. Figure adapted from SL, Stockholm Public Transport (2016).

The random pile group generator was tested with 14 piles symmetric around the x-axis. The possible pile inclinations were 4:1, 8:1 and vertical. The angle of rotation of the piles was free to vary in 30°-increments.

3.3.4 Summary Input Case Studies

Input parameters for all reference pile groups are presented in Table 3.1. All values in the table are taken directly from existing calculations, with a few exceptions marked with an asterisk, where no information was available and assumptions were made. The compression capacity and moment capacity for Case I and II were approximated from standard tables from manufacturers, e.g. Hercules Grundläggning (2018). The tensile capacity for Case I and II was approximated from values concerning the geotechnical capacity for piles in friction soil given by Trafikverket (2019c). As mentioned earlier, piles in cohesive soil generally have greater geotechnical tensile capacity than piles in friction soil, which indicates that this is an assumption on the safe side.

For Case II, the design approach was to account for lateral resistance from the soil. The lateral resistance for the topmost three meters was neglected.

For Case III, the capacity was assumed to be zero in tension. Therefore, the piles in tension were decided to be tensile anchored.

The information from the table was used as input data for the random pile group generator, except from inclination, that varied during optimization, and design approach soil/no soil, that both were evaluated.

Table 3.1: Input parameters for all reference pile groups.

Case	*I*	*II*	*III*	*IV*
Pile type	Reinforced concrete	Reinforced concrete	Reinforced concrete	Steel
Pile section	Quadratic	Quadratic	Quadratic	Circular hollow
Pile dimensions [mm]	270x270	270x270	275x275	$\varnothing$=168.3 t=12.5
Pile length [m]	9	13	51	28
Inclination	3.5:1	3.5:1	20:1, 12:1, 8:1	4:1
Pile cap [m x m]	5 x 5	4 x 3.7	5 x 12.5	5 x 7.4
Symmetry	two axes	x-axis	two axes	x-axis
Design	No Soil	Soil	Soil	No Soil
Soil type	Cohesive	Cohesive	Cohesive	Cohesive
Subgrade modulus [kPa]	675	675	533	509
$N_{Rd,max}$ [kN]	800*	800*	1600	1244
$N_{Rd,min}$ [kN]	-50*	-50*	0	-731
$M_{Rd,max}$ [kNm]	30*	30*	50	89

* assumed values

3.4 Investigation of Distance Between Pile Center and Load Center

A correlation study of the distance between pile center and load center and resulting tension forces in the pile group was carried out. This was done to investigate if the position of the pile center could be a reasonable objective function for finding an optimal arrangement of piles. The abutment and mid support for the pedestrian bridge, Case I and II, were chosen to be investigated in this study. They were chosen due to their simplicity and the completeness of presented load cases. 100.000 random pile groups were generated for each case, where the mid support was symmetrical around two axes and the abutment was symmetrical around one axis.

Correlation describes the strength and direction of association between two variables. This was investigated by generating a large number of random pile groups as described in Section 3.3. In this study, the pile group was not optimized and the result included all acceptable pile groups. For every pile group, the distance between the pile center and load center and resulting minimum normal forces of the pile group were calculated. The procedure is illustrated in Figure 3.7.

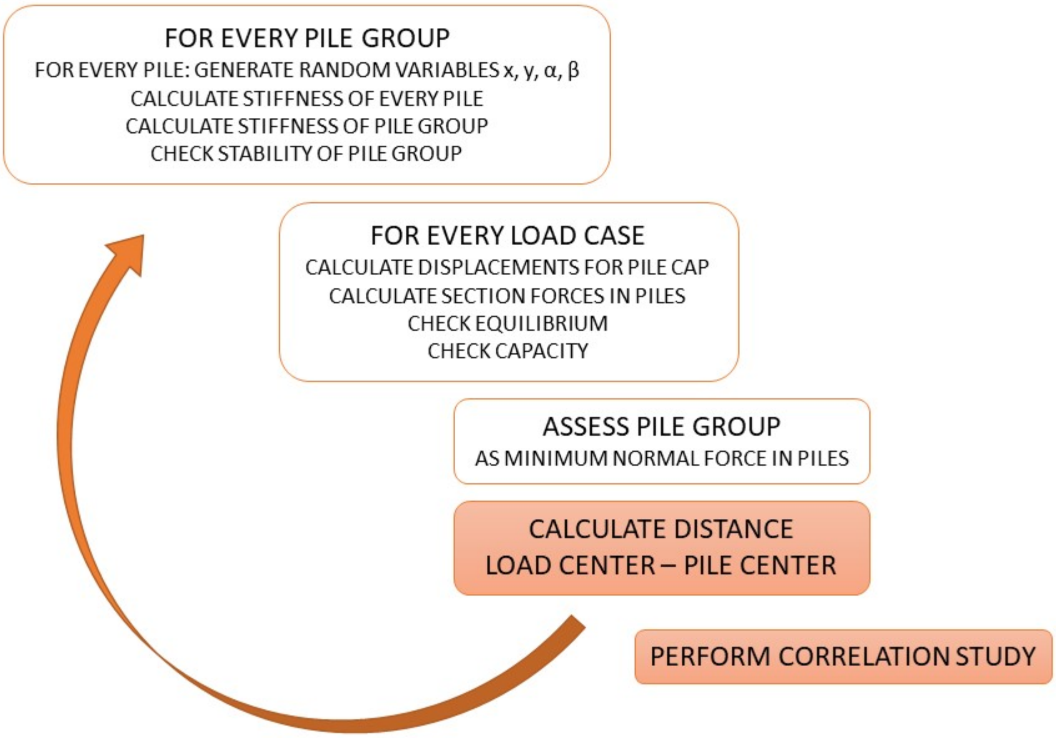


Figure 3.7: Flow chart for correlation study.

Two correlation tests were done; Pearson and Spearman correlation, where the first identify linear correlation and the last assess rank correlation, where the relationship must not necessarily be linear. Both correlation tests result in an R-value and a p-value. The R-value is a correlation coefficient that varies from -1 to 1, where -1 means perfect negative correlation, 1 means perfect positive correlation and 0 means no correlation. The p-value is a measure of statistical significance.

Pile center is a location that depends on the stiffness of the piles and the soil whereas load center is a location that depends on the loads applied. The pile center has a strict mathematical definition that is presented below, but as load center can be interpreted in several ways and little is written in the literature about it, three different methods were tested. All calculations were done in the global coordinate system presented in Section 2.8.

3.4.1 Pile Center

As mentioned in Section 2.4.1, external forces acting on the pile center cause only translation and no rotation. This definition was used to form a mathematical expression for the pile center. The pile cap was given a unit translation U_{PC}, as shown in Figure 3.8, and the generated pile forces F_{PC} were calculated. The coordinates of the pile center were calculated as the eccentricity of the generated forces. Pile center is a two-dimensional location, which means that three pile centers can be calculated for a three-dimensional pile group. In this project, the pile center is only calculated for the two planes orthogonal to the pile cap, i.e. the XZ-plane and the YZ-plane.

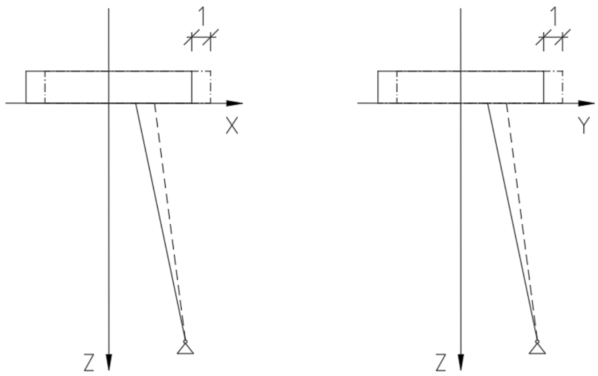


Figure 3.8: A horizontal unit displacement is imposed on the pile cap in x-direction and y-direction.

The generated forces from a unit displacement was calculated by using the inverse to the 3x3 stiffness matrix S_{PC}, analogously with Section 2.8, where the elements S_{ij} were different for the different planes.

$$\begin{aligned} &\text{XZ-plane:} \quad i,j = 0,4,5 \\ &\text{YZ-plane:} \quad i,j = 1,3,5 \end{aligned}$$

The pile center coordinates were calculated analogously for the planes. Thus, the stiffness matrix and the forces due to a unit translation is presented only in the XZ-plane. The stiffness matrix

$$S_{PC} = \begin{bmatrix} S_{0,0} & S_{0,4} & S_{0,5} \\ S_{4,0} & S_{4,4} & S_{4,5} \\ S_{5,0} & S_{5,4} & S_{5,5} \end{bmatrix}$$

The unit displacement

$$U_{PC} = \begin{bmatrix} U_0 \\ U_4 \\ U_5 \end{bmatrix} = \begin{bmatrix} 1 \\ 0 \\ 0 \end{bmatrix}$$

The generated forces were calculated as

$$F_{PC} = \begin{bmatrix} f_0 \\ f_4 \\ f_5 \end{bmatrix} = S_{PC}^{-1} U \tag{3.2}$$

The pile center coordinates (PC_x, PC_{zx}) in the XZ-plane could be calculated as the eccentricity of the generated forces. PC_{zx} is the z-coordinate in the XZ-plane.

$$PC_x = \frac{-f_5}{f_0} \tag{3.3}$$

$$PC_{zx} = \frac{f_4}{f_0} \tag{3.4}$$

This is the coordinates that should be compared to the load center.

3.4.2 Load Center

Three approaches were tested for calculating the load center - the broom method, the mean value method and the center of mass method.

All three methods starts by transforming moments into loads with an eccentricity in the pile cut-off plane. The concept of this transformation is illustrated in Figure 3.9.

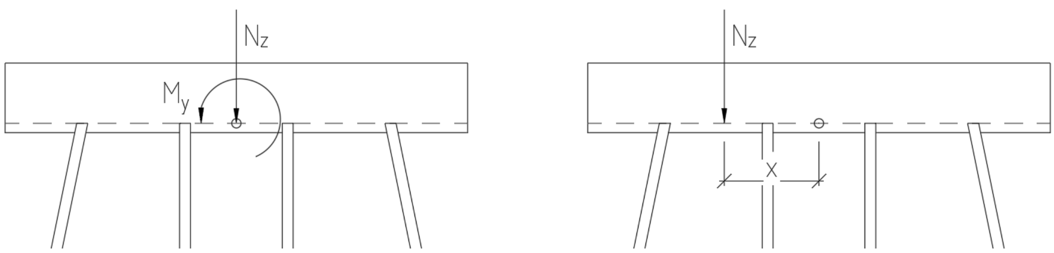


Figure 3.9: Eccentricity concept. The moment is replaced with an eccentricity.

For the broom method, inclined loads were placed at the pile cut-off level, forming a broom. This is a graphical method described in old handbooks, e.g. Stål (1984), that depicts the load center as the smallest part of the broom created by all load cases. The loads were elongated, see Figure 3.10, and the smallest part of the broom was identified visually. A weakness of this method is that the loads are weighted only by their eccentricity and inclination, and not by the magnitude.

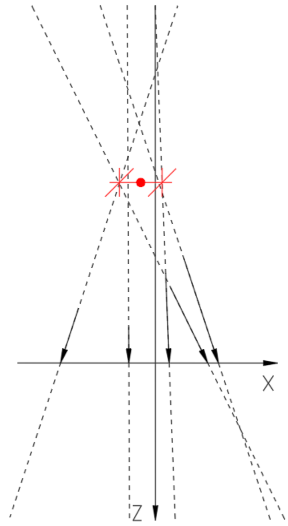


Figure 3.10: The concept for the broom method.

The two other methods both start by transforming moments into eccentric vertical and horizontal loads separately, giving an eccentricity in two directions. This results in two eccentric loads for every load case.

The eccentricities were calculated as follows.

$$\text{XZ-plane:} \quad x = \frac{-M_y}{N_z}, \quad z_x = \frac{M_x}{N_y} \tag{3.5}$$

$$\text{YZ-plane:} \quad y = \frac{M_x}{N_z}, \quad z_y = \frac{-M_y}{N_x} \tag{3.6}$$

Here the two methods differentiate. For the mean value method, the mean values of the coordinates were calculated, meaning that the loads were given equal importance.

$$LC_x = \frac{\sum_{i=1}^{n} x_i}{n} \tag{3.7}$$

In the center of mass method, the magnitude of the force was taken into account by giving the coordinates importance related to their size.

$$LC_x = \frac{\sum_{i=1}^{n} x_i \cdot N_{z,i}}{\sum_{i=1}^{n} N_{z,i}} = \frac{\sum_{i=1}^{n} -M_{y,i}}{\sum_{i=1}^{n} N_{z,i}} \tag{3.8}$$

3.4.3 Distance Between Pile Center and Load Center

The distance between the center of piles and the center of loads was calculated analogously in two planes, XZ and YZ.

$$d_{XZ} = \sqrt{(PC_x - LC_x)^2 + (PC_{zx} - LC_{zx})^2} \tag{3.9}$$

$$d_{YZ} = \sqrt{(PC_y - LC_y)^2 + (PC_{zy} - LC_{zy})^2} \tag{3.10}$$

For every pile group, the total distance was calculated as the sum of these distances as

$$Distance = d_{XZ} + d_{YZ} \tag{3.11}$$

where the total distance was used, together with the resulting tension forces, in the correlation study.

4 Results

In this section the results will be presented. In short, the developed program was successful in producing suitable pile groups for unit loads. It could present adequate pile groups for all four case studies. The correlation study indicates that the distance between the pile center and the load center may not be a good basis for optimization.

4.1 Verification of Program

Pile groups subjected to a specific unit load was generated by the program. The loads were

$$N_z = 100 \text{ kN}, \; N_x = 10 \text{ kN}, \text{ and } M_x = 10 \text{ kNm}$$

For every unit load, the six best pile groups were presented. All results can be found in Appendix B. Here, only one representative pile group for every unit load is presented.

In Figure 4.1, the pile group is subjected to a horizontal force in one direction only, which results in many inclined piles in the direction of the load. The pile plot is a view of the pile cap and the piles from above, where the pile top and pile end (and therefore also the inclination and direction) are shown.

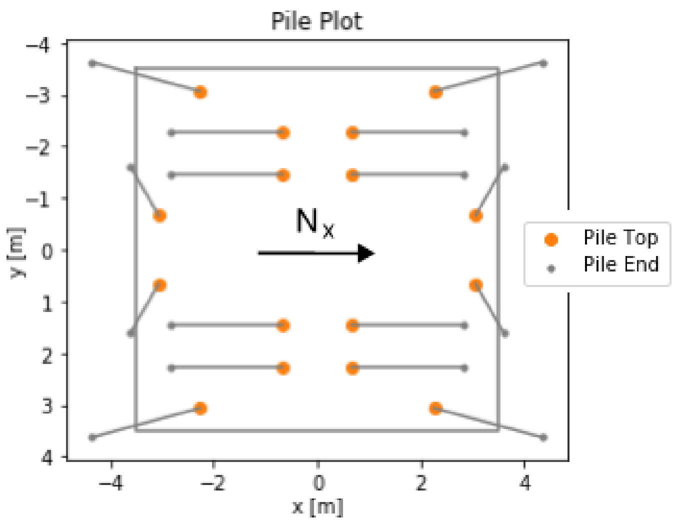


Figure 4.1: $N_x = 10$ kN. Most piles lean in the x-direction.

Figure 4.2 shows a pile group generated from a vertical load. This pile group has a few slightly inclined piles for stability, but most piles are vertical.

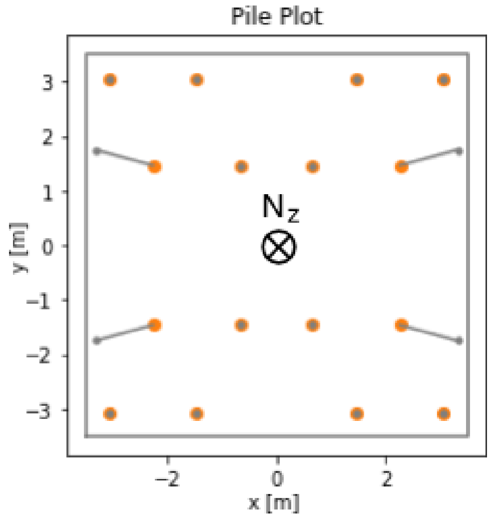


Figure 4.2: $N_z = 100$ kN. Most piles are vertical.

A pile group subjected to a moment around one axis was arranged by the program as seen in Figure 4.3. A moment around the x-axis is handled by a long lever arm between piles in the y-direction.

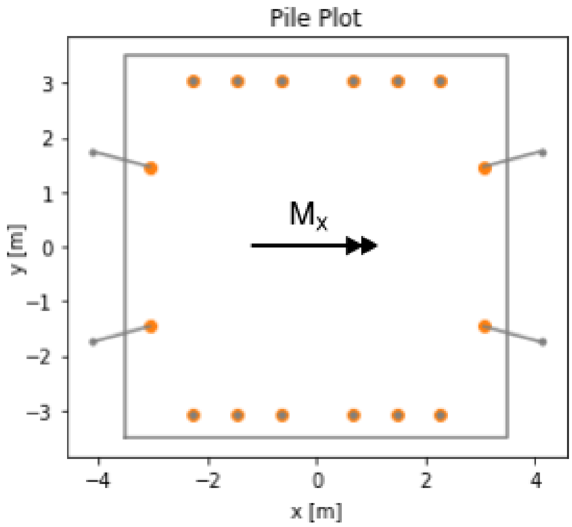


Figure 4.3: $M_x = 10$ kNm. Piles are arranged parallel with a large distance in the y-direction.

The, by the program, generated pile groups for the unit loads are in line with expectations from basic concepts for pile group arrangement.

4.2 Case Studies

The developed program could present adequate pile groups for all four case studies. Section forces for the reference pile groups, both from existing calculations and recalculated using the developed software, and section forces for the simulated pile groups, are presented in tables in the following sections. The values are matching for all cases except Case II. The existing pile groups are denoted with *Ref* in the tables and are followed by the three best simulated pile groups in descending order. The most optimal simulated pile group, due to the objective function, is also presented in figures in the upcoming sections, while the rest are presented in figures in Appendix C-F. The external loads, applied at origin, are found in the same appendices.

4.2.1 Case I - Pedestrian Bridge Mid Support

The eight piles were arranged symmetrical around two axes, which means that there are only two piles that are randomly generated, and the number of possible arrangements is therefore limited. When running the simulation repetitively, few pile groups are acceptable and the same acceptable pile groups reoccur. This means that the restrictions are strict, and according to further investigations of the constraints, the minimum normal force is the variable constraining the simulation.

100 000 pile groups were generated in a simulation that took 1 hour and 5 minutes, of which 78 were acceptable pile groups, where duplicates were detected. The simulation resulted in pile groups similar to the existing one. Figure 4.4 shows the reference pile group, Figure 4.5 shows the best simulated pile group, and Figure 4.6-4.7 shows the resulting section forces for both pile groups along with the capacity curve.

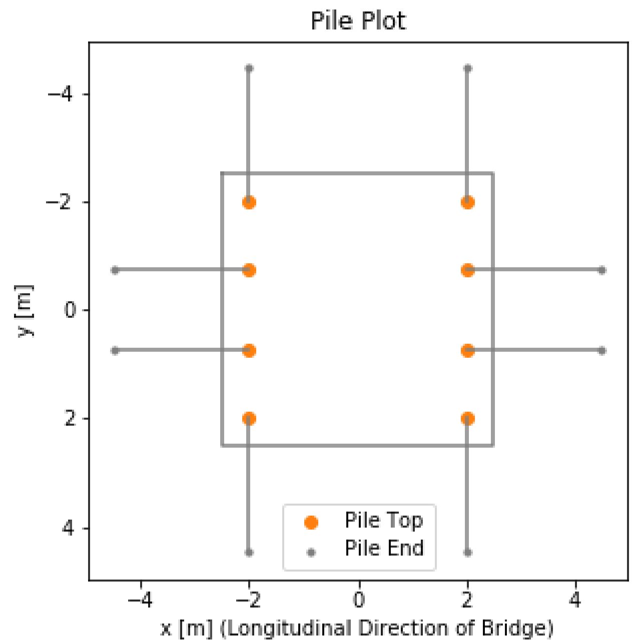


Figure 4.4: Pedestrian bridge mid support. The reference pile group with 8 piles.

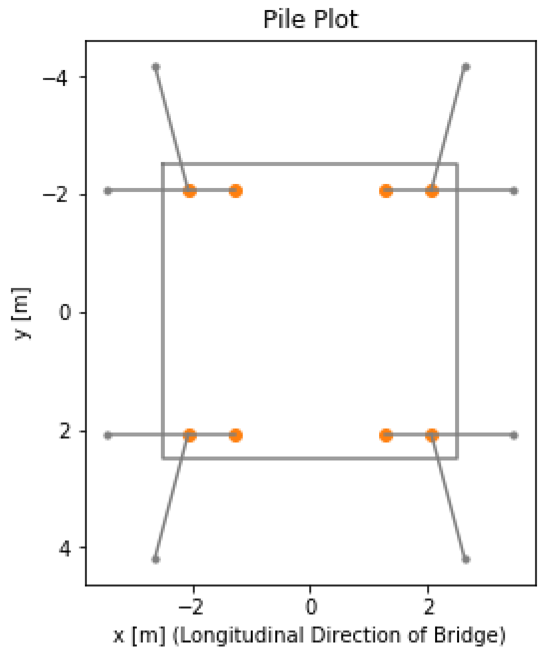


Figure 4.5: Pedestrian bridge mid support. The best simulated pile group with 8 piles.

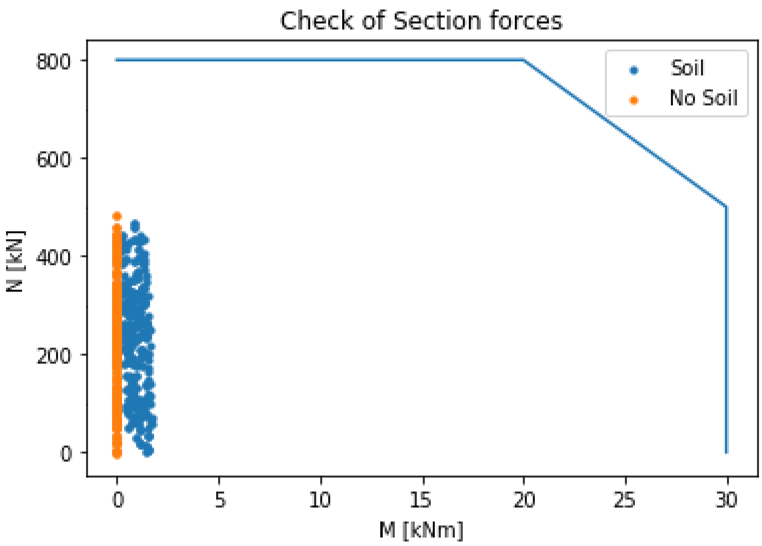


Figure 4.6: Pedestrian bridge mid support. Section forces for the reference pile group. One dot represent one load case and one pile. The blue line is the capacity curve.

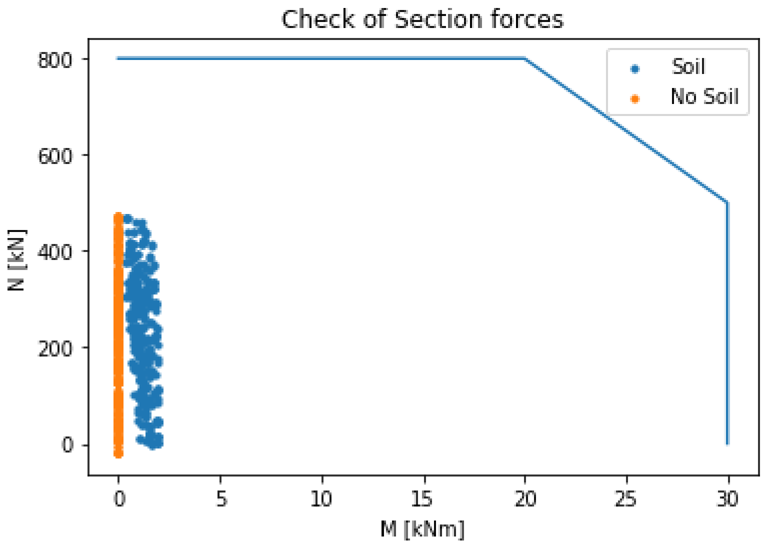


Figure 4.7: Pedestrian bridge mid support. Section forces for the best simulated pile group.

Table 4.1 shows calculated section forces for the reference pile group and the three best simulated pile groups, all with and without lateral resistance of the soil. The maximum normal forces were similar to the reference pile group, whereas the tension forces were larger.

Table 4.1: Calculated section forces.

Pile Group	*Soil/ No Soil*	N_{max} [kN]	N_{min} [kN]	V_{max} [kN]	M_{max} [kNm]	$N_{max} - N_{min}$ [kN]
Values from existing calculations						
Ref	No Soil	481	-3	0	0	N/A
Values from developed software						
Ref	Soil	465	3	1	2	462
Ref	No Soil	481	-3	0	0	484
1	Soil	470	-2	2	2	472
1	No Soil	473	-19	0	0	491
2	Soil	461	-15	2	2	475
2	No Soil	472	-27	0	0	499
3	Soil	459	-13	2	2	471
3	No Soil	479	-30	0	0	508

The reference pile group was designed with pile inclination 3.5:1, while the maximum allowed inclination for the developed software was 4:1, due to safety reasons. Since the ratio between the applied horizontal loads and vertical loads are relatively large for the pedestrian bridge, increased inclination of the piles will reduce tension. In other words, if larger inclination would be allowed in the developed program, tension forces would probably be lower than for the reference pile group.

4.2.2 Case II - Pedestrian Bridge Abutment

100.000 pile groups were generated in the simulation, where 139 pile groups met the requirements. The duration time was 26 minutes. The pile plot for the best simulated pile group is presented, together with the reference pile group, in Figure 4.8. The simulated pile group is unfeasible as there are piles colliding.

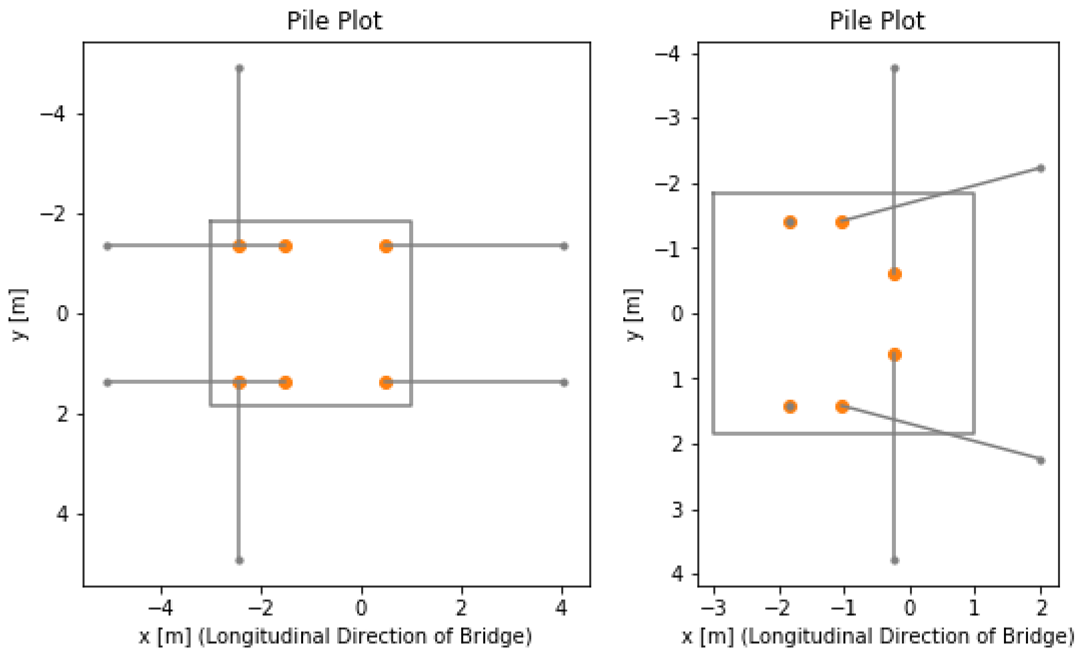


Figure 4.8: Pedestrian bridge abutment. To the left: The reference pile group with 6 piles. To the right: The best simulated pile group with 6 piles.

The section forces for the best pile group, together with the reference pile group, is shown in Figure 4.9.

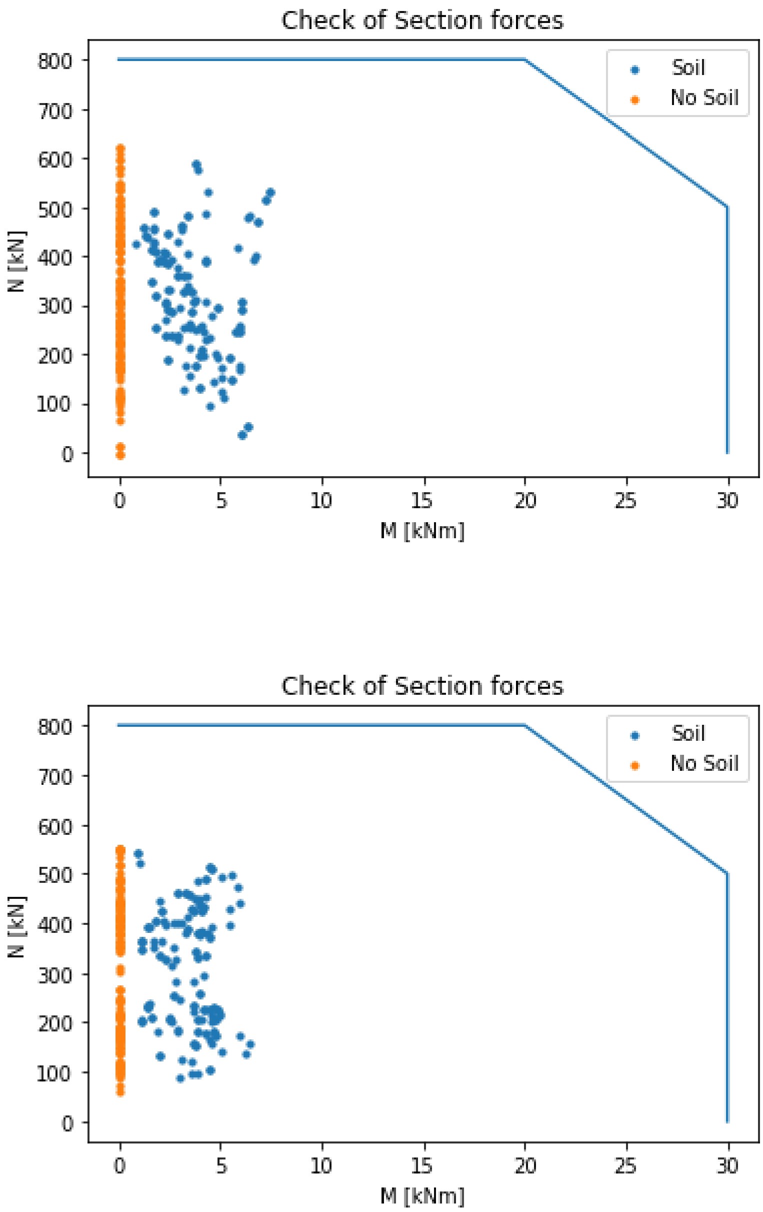


Figure 4.9: Pedestrian bridge abutment. Upper figure: Section forces for the reference pile group. Lower figure: Section forces for the best simulated pile group.

The section forces for the three best pile groups are presented in Table 4.2.

Table 4.2: Calculated section forces.

Pile Group	*Soil/ No Soil*	N_{max} [kN]	N_{min} [kN]	V_{max} [kN]	M_{max} [kNm]	$N_{max} - N_{min}$ [kN]
Values from existing calculations						
Ref	Soil	614	5	2	N/A	N/A
Values from developed software						
Ref	Soil	588	37	7	8	551
Ref	No Soil	621	-3	0	0	624
1	Soil	541	90	6	7	451
1	No Soil	552	61	0	0	491
2	Soil	546	91	7	7	455
2	No Soil	555	47	0	0	508
3	Soil	579	53	5	5	526
3	No Soil	620	52	0	0	568

The simulated pile groups had somewhat smaller maximum normal forces than the reference pile group, and considerably higher minimum normal forces. The risk for having tension in the generated pile groups due to imprecise installation is lower for the simulated pile groups than for the reference pile group, i.e. it is more robust. The minimum normal forces were increased, despite having less inclined piles. This indicates that a smart arrangement of the piles allows for smaller inclination of the piles, which is positive for installation purposes.

As mentioned earlier, the reference values calculated in the program were somewhat different compared to the existing calculations. This is probably due to an assumption of having no lateral soil resistance in the topmost three meters that was made in the original calculations.

4.2.3 Case III - Road Bridge Mid Support

The road bridge of Marieholm was originally calculated with lateral resistance from cohesive soil. However, the section forces for the reference pile group are also calculated without any soil, to be able to compare with the simulation. Generated section forces for the best pile groups with 24 piles are presented in Table 4.3. 100 000 pile groups were generated in a simulation that took 1 hour and 5 minutes, of which 34 pile groups met the requirements.

The existing pile group and the best simulated pile group are shown in Figure 4.10. As can be seen in the figure, the simulated pile group is less organized and there are risks of collisions between some of the piles. The section forces for the existing pile group and the best simulated pile group are illustrated in Figure 4.11.

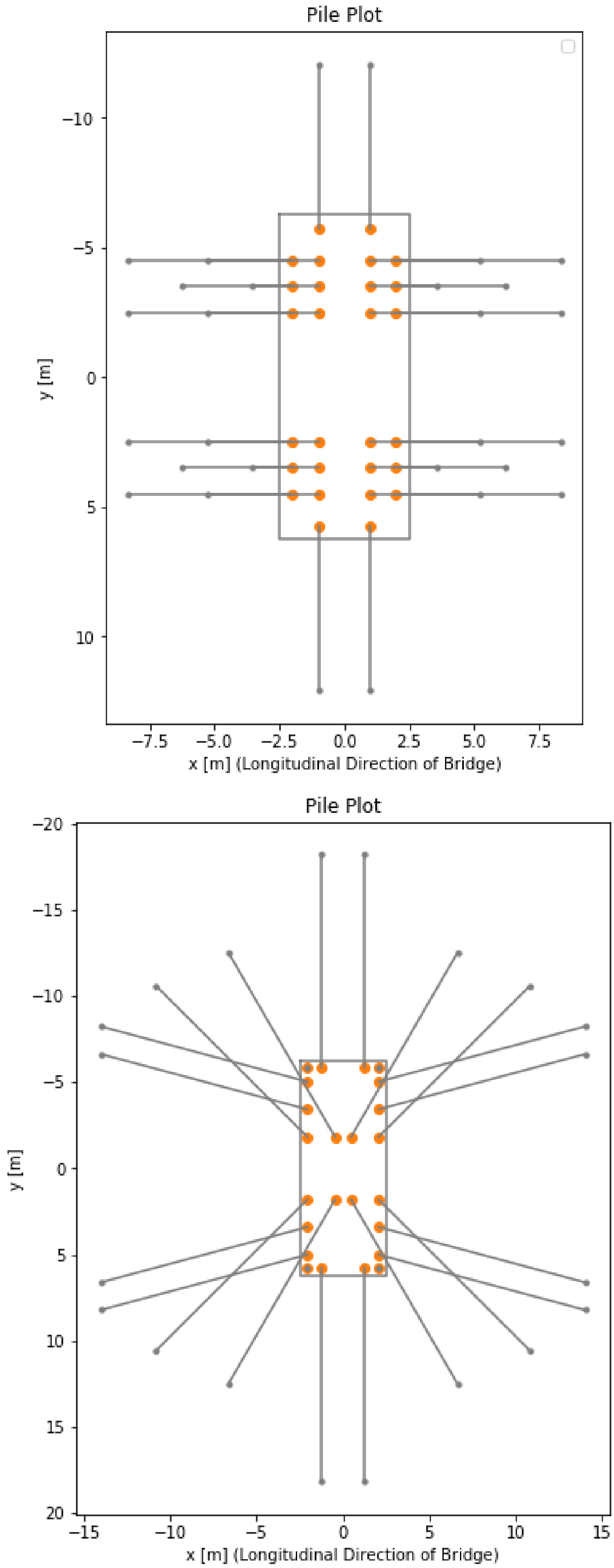
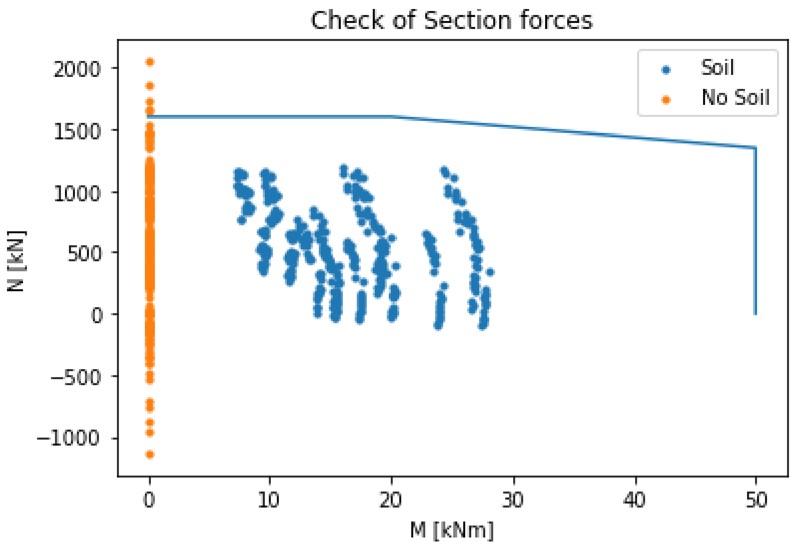


Figure 4.10: Road bridge mid support: Upper figure: The reference pile group with 28 piles. Lower figure: The best simulated pile group with 24 piles.

Figure 4.11 shows that the reference pile group would not have met the requirements for compression and tension if calculations had been performed without respect to the lateral resistance of the soil. According to Trafikverket (2019a), this is not required for a road bridge, as long as both high and low values for the lateral resistance is considered.



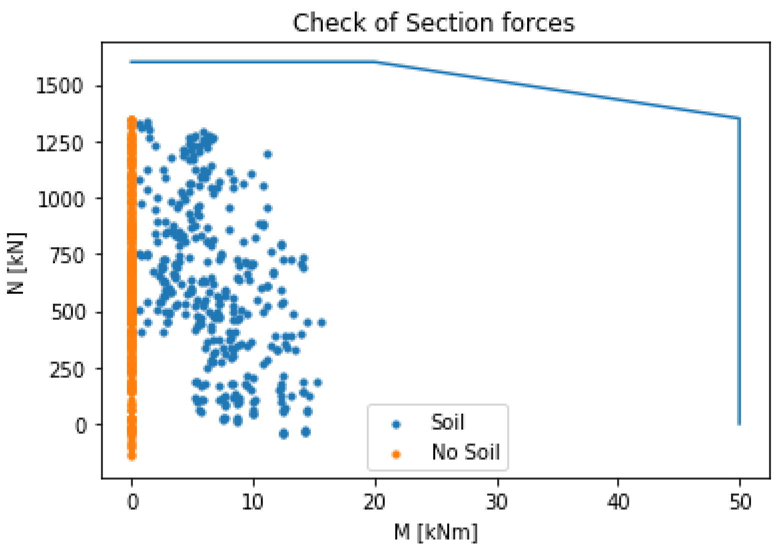


Figure 4.11: Road bridge mid support. Upper figure: Section forces for the reference pile group. Lower figure: Section forces for the best simulated pile group.

Table 4.3: Calculated section forces with 24 piles.

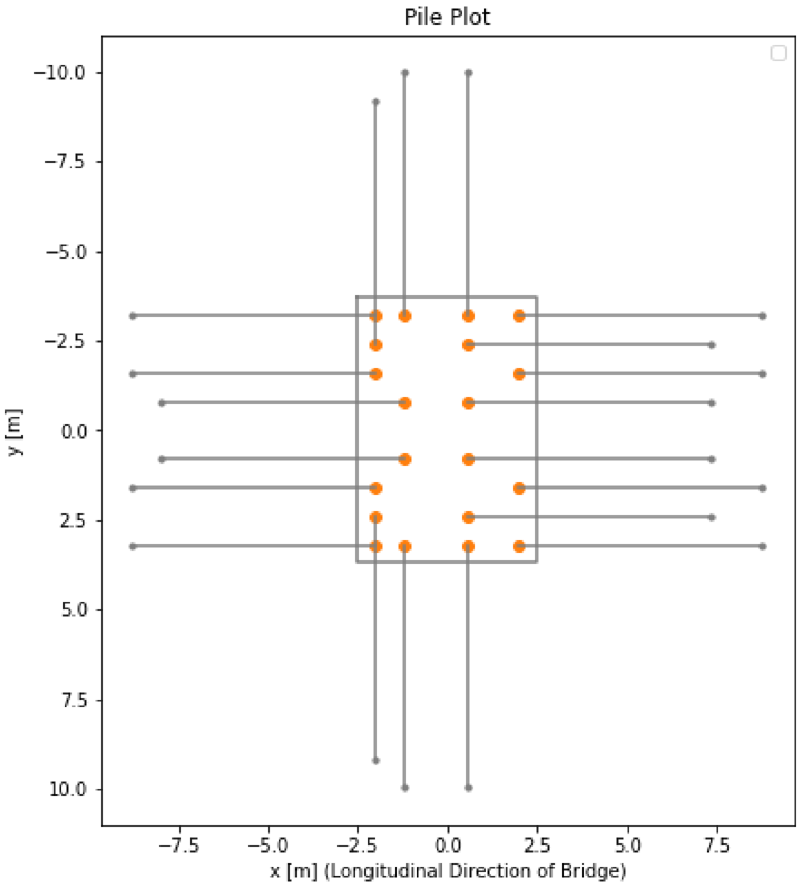
Pile Group	*Soil/ No Soil*	N_{max} [kN]	N_{min} [kN]	V_{max} [kN]	M_{max} [kNm]	$N_{max} - N_{min}$ [kN]
Values from existing calculations						
Ref	Soil	1183	-103	26	28	N/A
Values from developed software						
Ref	Soil	1183	-103	26	28	1286
Ref	No Soil	2056	-1142	0	0	3198
1	Soil	1338	-45	15	16	1382
1	No Soil	1359	-138	0	0	1488
2	Soil	1363	-88	14	15	1450
2	No Soil	1377	-174	0	0	1550
3	Soil	1424	-122	15	16	1546
3	No Soil	1446	-129	0	0	1574

All normal forces calculated without any lateral resistance from the soil are better for the presented simulated pile groups than for the reference pile group. As mentioned before, the reference pile group was originally only calculated with lateral resistance from the soil. For the calculations with soil, the pile group simulation did not manage to generate pile groups with lower compression forces than the reference pile group. However, the simulation performed well for tension forces. The simulation even managed to find solutions with fewer piles as input than the reference pile group.

4.2.4 Case IV - Railway Bridge Abutment

The railway bridge over Ullna Kvarnväg was originally calculated without lateral resistance from the soil. However, the section forces for the reference pile group are also calculated with soil, to be able to compare with the simulation. 10 000 random pile groups were generated in the investigation, of which 843 pile groups met the requirements. The duration of the simulation was less than 5 minutes.

The reference pile group and the best simulated pile group are shown in Figure 4.12. As can be seen in the figure for the best simulated pile group, it is not obvious if some piles will collide or not. The corresponding section forces are presented in Figure 4.13.



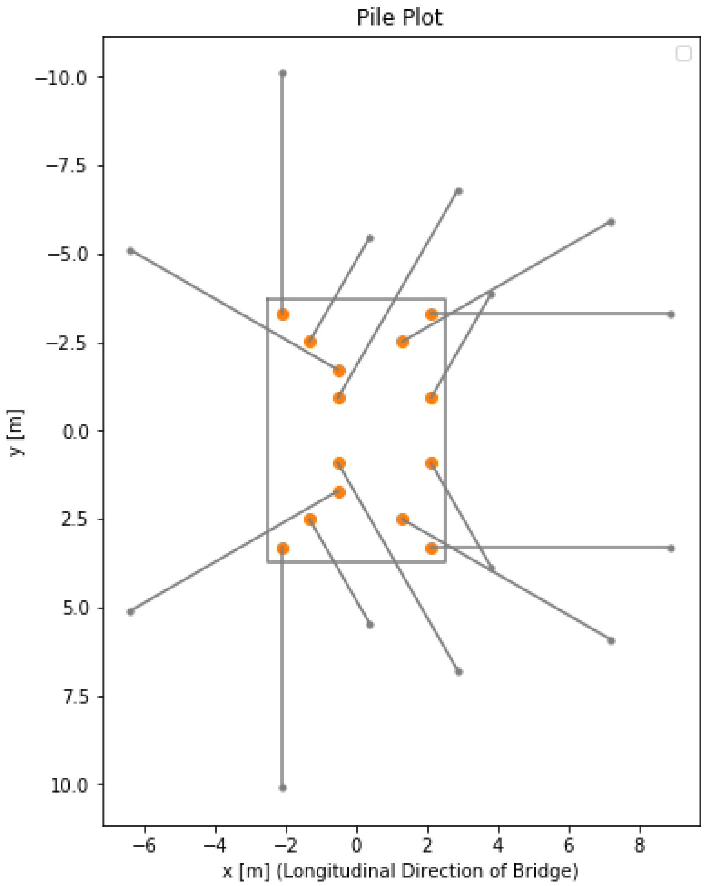
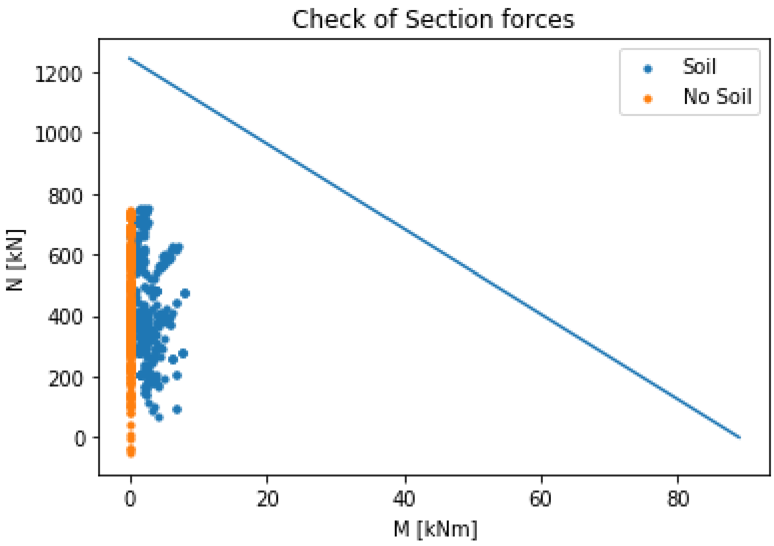


Figure 4.12: Railway bridge abutment. Upper figure: The reference pile group with 20 piles. Lower figure: The best simulated pile group with 14 piles.



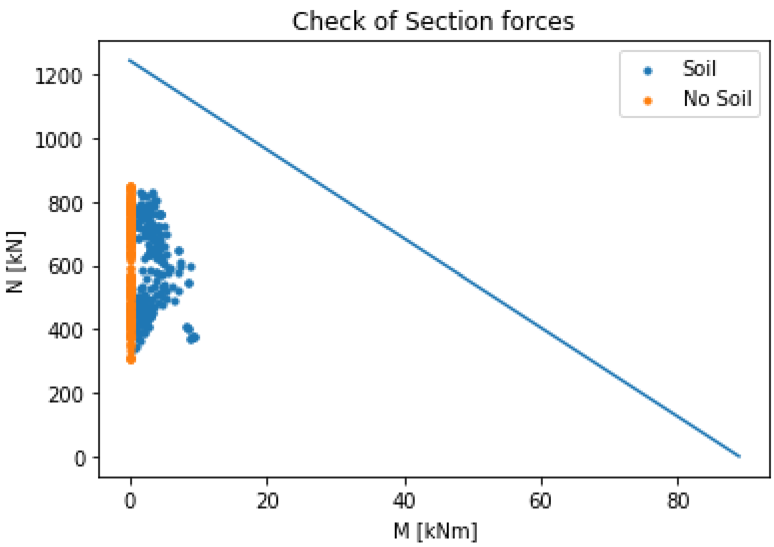


Figure 4.13: Railway bridge abutment. Upper figure: Section forces for the reference pile group. Lower figure: Section forces for the best simulated pile group.

Generated section forces for the reference pile group and for the three best simulated pile groups with 14 piles are presented in Table 4.4.

Table 4.4: Calculated section forces with 14 piles.

Pile Group	*Soil/ No Soil*	N_{max} [kN]	N_{min} [kN]	V_{max} [kN]	M_{max} [kNm]	$N_{max} - N_{min}$ [kN]
Values from existing calculations						
Ref	No Soil	744	-51	0	0	N/A
Values from developed software						
Ref	Soil	754	69	11	8	685
Ref	No Soil	744	-51	0	0	795
1	Soil	828	339	13	9	490
1	No Soil	852	305	0	0	546
2	Soil	834	309	10	8	525
2	No Soil	853	229	0	0	623
3	Soil	926	336	10	8	590
3	No Soil	935	261	0	0	674

The pile group simulation produced pile groups with lower objective function than for the reference pile group, meaning that the distribution of forces between the piles were more even. These results were generated by only 14 piles, i.e. a reduction of six piles.

4.2.5 Summary Case Studies

To make it easier to analyze the results, an overview of the results for all cases is presented in Table 4.5. The values in the table are differences between the most optimal simulated pile groups and the reference pile groups. The difference has been calculated for maximum normal force, minimum normal force, the objective function and the number of reduced piles. If the simulated pile group is an improvement compared with the reference pile group, the difference is negative, and vice versa. The improved values are also highlighted with green color.

Table 4.5: The difference between the most optimal simulated pile groups and the reference pile groups, presented for maximum and minimum normal force and objective function. The number of reduced piles is also presented for every case study.

$Case$	ΔN_{max} [kN]	ΔN_{min} [kN]	$\Delta Obj.function$ [kN]	$\Delta Nr\ of\ Piles$
I Soil	5	5	10	0
I No Soil*	8	16	8	0
II Soil*	-47	-53	-61	0
II No Soil	-69	-64	-133	0
III Soil*	155	-58	96	-4
III No Soil	-697	-1004	-1710	-4
IV Soil	74	-270	-195	-6
IV No Soil*	108	-356	-249	-6

* the reference group was calculated with this assumption about the soil

The existing designs were originally calculated only with or without lateral resistance from the soil, unlike the developed software that took both into account. As described in Section 3.1.3, the objective function was evaluated in parallel for conditions with and without lateral resistance from the soil, where the highest objective function was decisive for the pile group. If the program had performed calculations using only the same assumption of soil used in the existing design, the generated results would have been positively affected.

For Case I, the developed program did not manage to produce pile groups with better section forces than the reference pile group. However, the differences were small.

The arrangement of piles for the simulated pile group in Case II, did both decrease the maximum normal force and increase the minimum normal force.

The simulated pile group in Case III, had a reduction of four piles compared to the reference pile group. Once again, the reference pile group did only consider lateral resistance from the soil. Therefore, the high negative values do not undermine the existing design, but were only used to evaluate the influence of the soil.

For Case IV, the simulated pile group had a reduction of six piles compared with the reference pile group. The minimum normal forces were significantly improved. However, the maximum normal forces were lower for the reference pile group.

4.3 Investigation of Distance Between Pile Center and Load Center

Two case studies were carried out for the correlation tests. For both cases, the calculated load center is presented first. This value does not change during the simulation, in contradiction to the pile center, that is unique for every simulated pile group. Then, correlation plots and values are presented.

4.3.1 Case I - Pedestrian Bridge Mid Support

Coordinates for the load center calculated in three different ways are presented in Table 4.6. The coordinates for the broom method was evaluated visually from Figure 4.14.

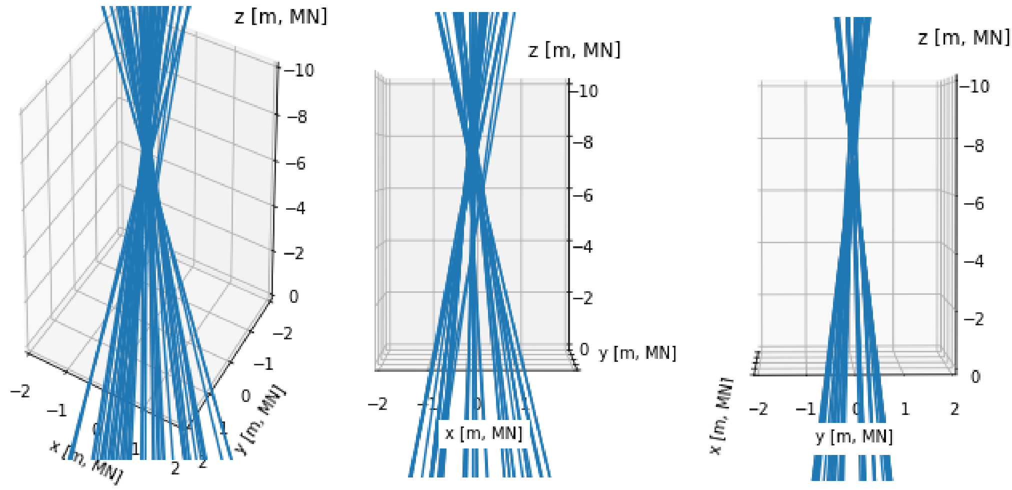


Figure 4.14: Case I: The broom created by all elongated load cases placed at the pile cut-off plane. Load center for the broom method was evaluated visually from these figures.

According to Table 4.6, the calculated coordinates for the load center became similar for all three methods, except for the coordinate LC_{zy}, which differed significantly between the broom method and the other two methods.

Table 4.6: Case I: Coordinates for the load center in XZ- and YZ-plane.

Method	LC_x [m]	LC_{zx} [m]	LC_y [m]	LC_{zy} [m]
The broom	-0.1	-7.1	0	-7.7
The mean value	-0.12	-7.69	-0.02	-23.83
The center of mass	-0.13	-6.27	-0.02	-25.25

10.000 random pile groups were generated in this investigation. 601 of them were acceptable pile groups that were used for the correlation test. The resulting minimum normal force in the pile group and the distance load center - pile center is plotted in Figure 4.15. The result from the correlation tests is presented in Table 4.7, 4.8 and 4.9.

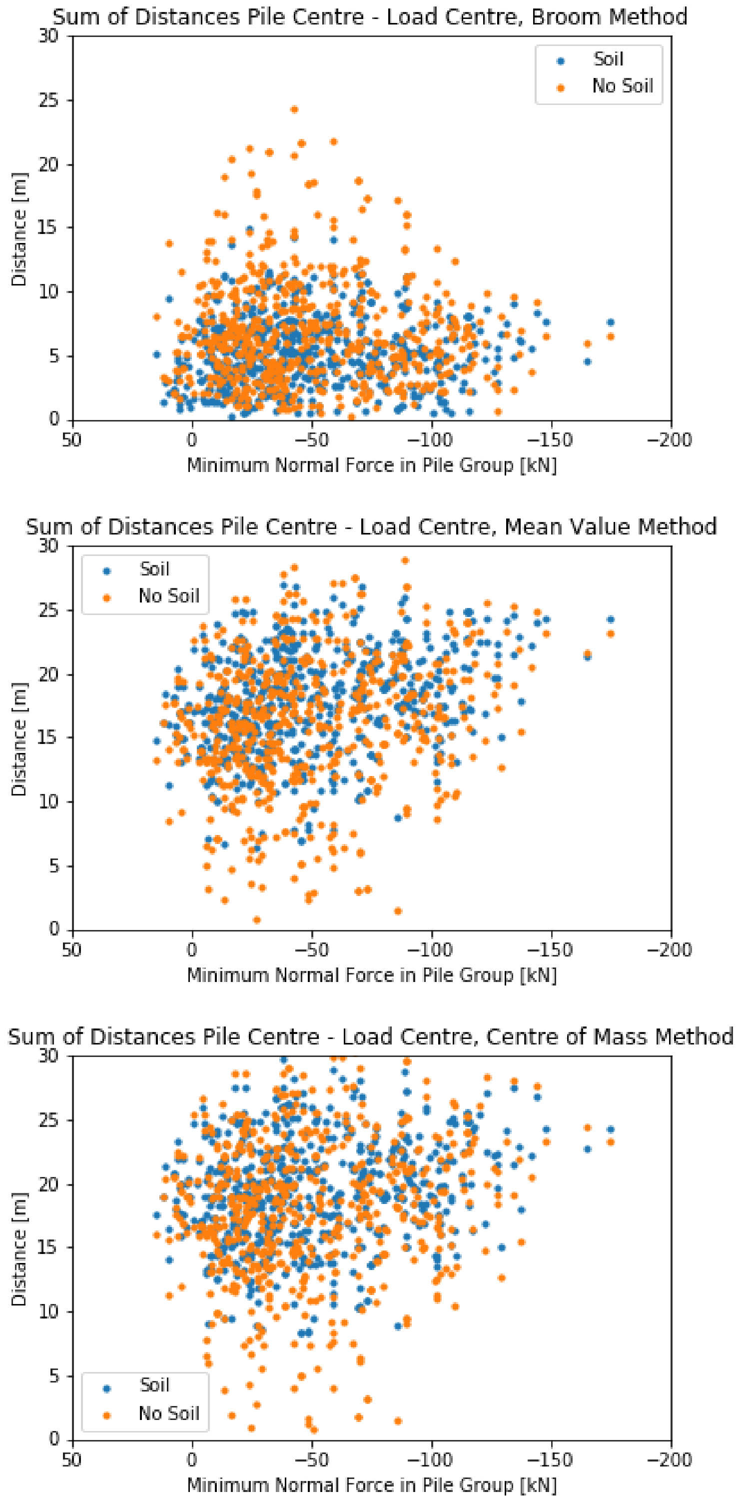


Figure 4.15: Case I: Correlation between resulting minimum normal forces and the distance between pile center and load center, for the three different methods for calculating the load center.

Table 4.7: Case I: Correlation coefficient and two-tailed p-value with the broom method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	0.078	0.056	not significant
Pearson	No Soil	-0.040	0.033	negligible correlation
Spearman	Soil	0.100	0.014	negligible correlation
Spearman	No Soil	-0.012	0.767	not significant

Table 4.8: Case I: Correlation coefficient and two-tailed p-value with the mean value method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	0.288	$6 \cdot 10^{-13}$	low positive correlation
Pearson	No Soil	0.214	$1 \cdot 10^{-7}$	low positive correlation
Spearman	Soil	0.287	$8 \cdot 10^{-13}$	low positive correlation
Spearman	No Soil	0.213	$1 \cdot 10^{-7}$	low positive correlation

Table 4.9: Case I: Correlation coefficient and two-tailed p-value with the center of mass method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	0.193	$2 \cdot 10^{-6}$	negligible correlation
Pearson	No Soil	0.144	0.0004	negligible correlation
Spearman	Soil	0.186	$4 \cdot 10^{-6}$	negligible correlation
Spearman	No Soil	0.140	0.0006	negligible correlation

The results differed between the three methods. The broom method did not show a significant correlation. For the mean value method and the center of mass method, negligible or low positive correlation was found in the investigation of the relationship between pile center and load center, and the resulting minimum normal forces in the pile group.

4.3.2 Case II - Pedestrian Bridge Abutment

Coordinates for the load center calculated in three different ways are presented in Table 4.10. The coordinates for the broom method was evaluated visually from Figure 4.16.

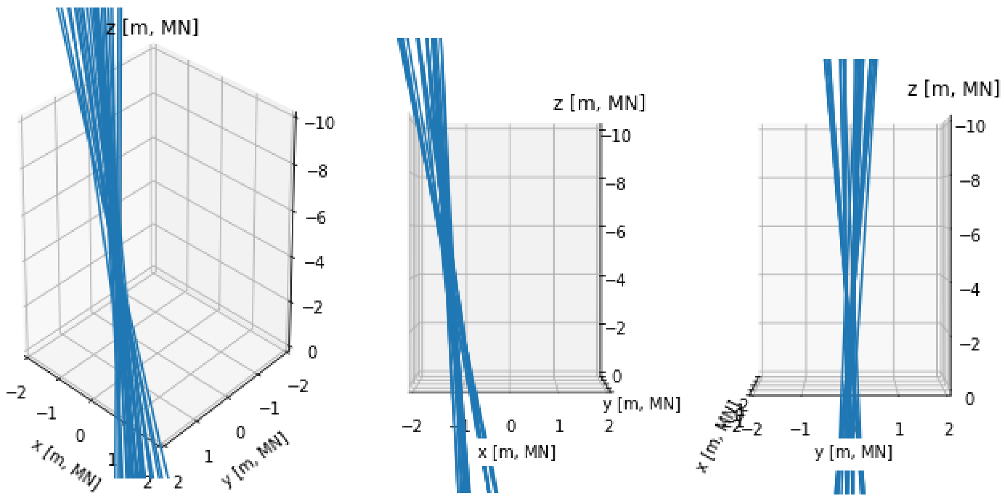


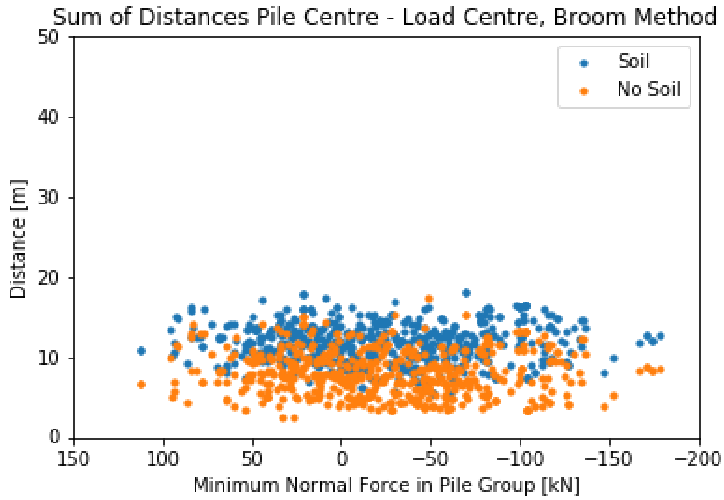
Figure 4.16: Case II. The broom created by all elongated load cases placed at the pile cut-off plane. Load center for the broom method was evaluated visually from these figures.

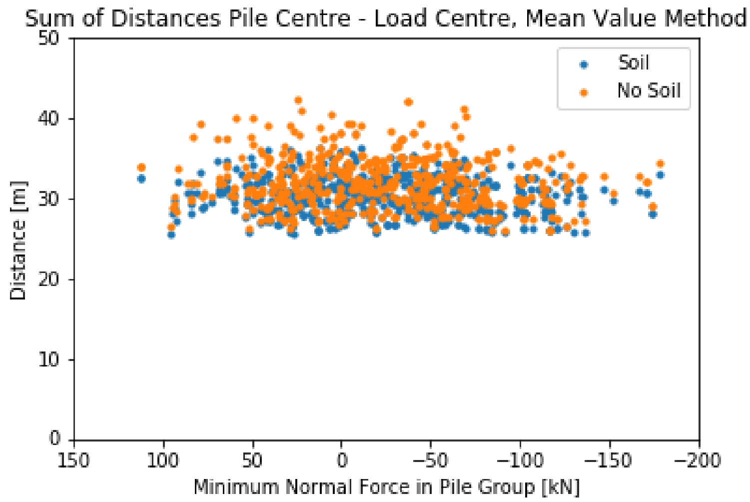
Table 4.10 shows that the coordinates for the load center differed depending on which method was used, especially for the LC_{zy}-coordinate.

Table 4.10: Case II: Coordinates for the load center in XZ- and YZ-plane.

Method	LC_x [m]	LC_{zx} [m]	LC_y [m]	LC_{zy} [m]
The broom	-1.35	-4	0	-1.5
The mean value	-1.05	3.50	0.00	22.51
The center of mass	-1.05	0.45	0.00	17.17

40.000 random pile groups were generated in this investigation. 587 of them were acceptable pile groups that were used for the correlation test. The objective function and distance load center - pile center is plotted in Figure 4.17.





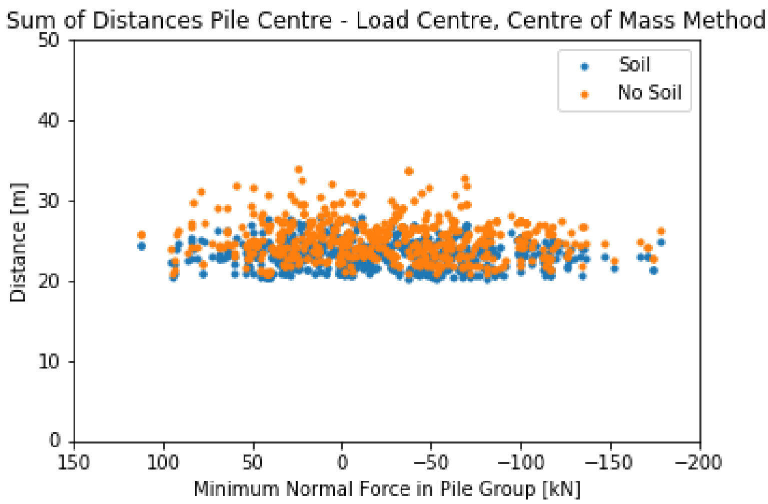


Figure 4.17: Case II: Correlation between resulting minimum normal forces and the distance between pile center and load center.

The result from the correlation tests is presented in Table 4.11, 4.12 and 4.13.

Table 4.11: Case II: Correlation coefficient and two-tailed p-value with the broom method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	-0.018	0.661	not significant
Pearson	No Soil	-0.044	0.284	not significant
Spearman	Soil	-0.023	0.584	not significant
Spearman	No Soil	-0.059	0.153	not significant

Table 4.12: Case II: Correlation coefficient and two-tailed p-value with the mean value method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	-0.084	0.042	negligible correlation
Pearson	No Soil	-0.085	0.041	negligible correlation
Spearman	Soil	-0.066	0.113	not significant
Spearman	No Soil	-0.065	0.116	not significant

Table 4.13: Case II: Correlation coefficient and two-tailed p-value with the center of mass method.

Correlation Type	Soil/No Soil	Correlation Coefficient R	p-value	Classification
Pearson	Soil	-0.028	0.494	not significant
Pearson	No Soil	-0.055	0.181	not significant
Spearman	Soil	-0.011	0.798	not significant
Spearman	No Soil	-0.034	0.415	not significant

For all three methods, no significant correlation was found in the investigation of the relationship between pile center-load center and the resulting minimum normal forces of the pile group, indicating the use of this as an objective function may not be suitable.

5 Discussion and Conclusion

5.1 Discussion

Finding suitable pile groups for bridges is a time consuming task, as different load cases have different optimal designs. The aim of this thesis was to automate parts of the design process for pile groups. A program generating and evaluating random pile groups was developed and tested on four cases.

The method for analyzing the pile groups included many simplifications. Firstly, the soil surrounding the piles was assumed to be homogeneous, i.e. only one type of soil can be implemented in one analysis. It may not be obvious what layer of soil is decisive and how to choose soil parameters that are on the safe side, but not overly conservative. Secondly, the frame analysis method does not predict the behaviour of the piles in an exact manner. It assumes an infinitely stiff pile cap and the connection to the pile cap can only be simply supported or moment stiff, whereas the reality is somewhere in between. The lateral resistance of the soil was considered by the use of two simple beam models, the Winkler model and a moment stiff connection at a fictitious length. However, the simplifications for the soil and the structure are reasonable for the scope of this project - to quickly be able to suggest reasonable, preliminary pile groups.

Calculations with lateral resistance of the soil resulted in less extreme normal forces but larger shear forces and moments, as expected. For the simulated pile groups in the case studies, the normal forces were governing the design, meaning that the calculations without lateral resistance of the soil were most relevant for the result of the pile group arrangements. This agrees with indications by designers. However, the differences between calculations with and without soil were small for all cases except Case III. This indicates that some designs are more susceptible to assumptions concerning the soil than others. There may be cases with high lateral resistance from the soil, that induce large governing bending moments in the piles. None of the cases studied had friction soil, which may give other results.

Evaluating pile groups both with and without lateral resistance, may not have been the best approach to investigate whether a Monte Carlo-simulation can facilitate pile group arrangement. To get more pronounced results the simulation should have been carried out only with the same assumption about the soil as the existing design. The curiosity about the influence of the lateral resistance of the soil and the new requirements from Trafikverket, led to a multifaceted project, but also to results that were hard to interpret.

The developed program does not perform a perfect optimization, but is rather a powerful, simple tool for arranging pile groups in preliminary design. There are many optimization algorithms that may have produced more theoretically optimal pile groups. The drawback of Monte Carlo-simulations is the need for a high number of realizations. In pile groups with few piles, it may be more appropriate to test all possible combinations, instead of simulating the design variables randomly. Even though the program is iterating automatically, there is a need for manual calibration whenever applying the program on a new case. This includes how to restrict the options for rotations and inclinations, if and how to apply symmetry restrictions, how many pile groups to generate, etc. The calibration issues can also be seen as an advantage as the designer can implement her specific requirements. Once the program is calibrated, the simulation takes a few minutes to an hour, depending on the complexity and the number of generated pile groups.

The optimization is driven by an objective function, and the choice of objective function obviously impacts the results. The chosen objective function was to minimize the difference between maximum and minimum normal forces, with the purpose to distribute forces evenly in the pile group. Although the forces became evenly distributed between the piles, the objective function did not assure that the generated section forces had any margins to the capacity of the pile group. From one point of view, it may be positive to utilize the capacity of the piles as much as possible. On the other hand, the pile group should be designed with a safety margin due to installation deviations. Other possible objective functions could be to minimize maximum normal force or to maximize minimum normal force. These objective functions, including the chosen one, evaluate the pile group based on the result for the critical pile in the pile group. This seems reasonable as the pile group is not stronger than its weakest pile. On the other hand, it is a parallel system, meaning that failure of one pile may not lead to overall failure. The weakest link assumption is strictly only relevant for series systems, like a chain with links. For minimum normal forces, it may be interesting to evaluate the number of piles that are in tension, rather than only the critical pile. It is not obvious whether a few piles with large tension forces or many piles with minor tension forces is most problematic. Tension anchoring a pile is time consuming and an economical loss, irrespective of the size of the tension force. The number of tension anchored piles could then be a better indicator of the adequacy of the pile group.

Tension should generally be avoided but in some cases when conditions are favourable, ensuring sufficient tensile capacity is not a problem. This depends on, for example, the type of soil, the self-weight of the piles and what requirements the project owner has. Another approach is to rely on the resistance of the remaining piles, assuming that the pile in tension does not contribute to the resistance, and that the load is redistributed. If the tensile capacity cannot be verified, arranging the pile group unsymmetrical or increasing the size of the pile cap may solve the problem. A bigger pile cap allows for a more flexible arrangement (implemented in the simulation) and results in larger vertical forces from self-weight (must be manually adjusted as input data).

The developed program cannot assure that there are no collisions between the piles below and above ground. This is a general problem for all the case studies. The risk for collisions is in some cases obvious to assess by inspecting the pile plot visually, and require further investigation in other cases. Also, in some of the cases the piles are leaning towards or parallel with each other, meaning that the minimum pile distance of 0.8 meter is not valid. This issue could be interesting for future studies.

For some cases, the random pile group generator produced pile groups with less number of piles than the reference pile groups. There is not only a reduction of piles, the tension forces also seems to be better. An explanation may be that the applied vertical forces are distributed among fewer piles, resulting in larger compression forces. The reduction of piles is a great success from both a material and an economical point of view. However, it is important to keep in mind that the pile group should also be designed according to other limit states and the risk for eliminated piles. A pile group with fewer number of piles may be less redundant.

To make use of the location of the pile center seems in theory like a logical method to design pile groups, but in practice several problems arise. It is not obvious how a mathematical model should be created to reflect the reality accurately. Three calculation methods for the load center have been performed, the broom method, the mean value method and the center of mass method. Only the mean value method for Case I proved low significant correlation for the distance between the pile center-load center and the tension forces in the pile group. All other correlation test showed no significant correlation. The explanation for these results may be found in the methods used for the load center, as they have not been possible to verify.

All three methods require that all load cases are used and that no one has been eliminated due to, for example, symmetry reasons, otherwise the load center is misplaced. The eccentricities are calculated as a ratio between moment and force, and an infinite small force does therefore result in an infinite large eccentricity that may not be relevant. The broom method and the mean value method value all eccentricities equally, which may not be reasonable. None of the methods did identify the critical load cases that generate the worst pile forces. Elaborating with the load center calculations left us with many questions. Is it relevant to calculate a pile center based on only critical load cases? Should all load cases have equal value?

Using non-verified methods makes it hard to draw conclusions from the results. If the methods are accurate, there is no reason to minimize the distance between the load center and pile center, as it does not correlate with the resulting minimum normal forces. Irrespective of the validity of the load center calculations, we argue that evaluating pile groups directly by their section forces is a better way than by using the distance load center-pile center, especially when using an automatic generating pile group program.

5.2 Conclusion

To summarize, the thesis concludes that:

- An optimization program can facilitate the preliminary design of pile groups by automatizing the iterative task to arrange pile groups suitable for a large number of load cases. The process still requires some manual surveillance, for example to calibrate the input data and evaluate the feasibility of the generated pile groups.
- The methods used for investigating the correlation between the distance between pile center and load center and resulting tension forces were non-verified. How-

ever, the results indicate that the concept may not be suitable as an objective function.

- To facilitate the work at the building site, it is important to account for practical aspects at an early stage. Robustness, simplicity and limiting the inclination of the piles, are important aspects according to contractors. For a successful pile group project, communication between the designer and the contractor is encouraged.

- For the cases studied, calculations without lateral resistance of the soil were decisive, which is in line with the approach that traditionally has been common practice for pile group design. Nevertheless, Trafikverket requires calculations with lateral resistance of the soil.

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Appendix A

Interview Questions

The questions asked during the telephone interviews are presented below

- What are the most important characteristics of a pile group (except that it has sufficient bearing capacity)?
- What factors during installation affect the design of the pile group? And does it vary depending on the pile type?
- Does machine constraints have an impact on the design? If yes, how?
- Does safety aspects and working environment affect the design? If yes, how?
- How does the design of the pile group influence time and cost of installation?

Appendix B

Verification with Unit Load

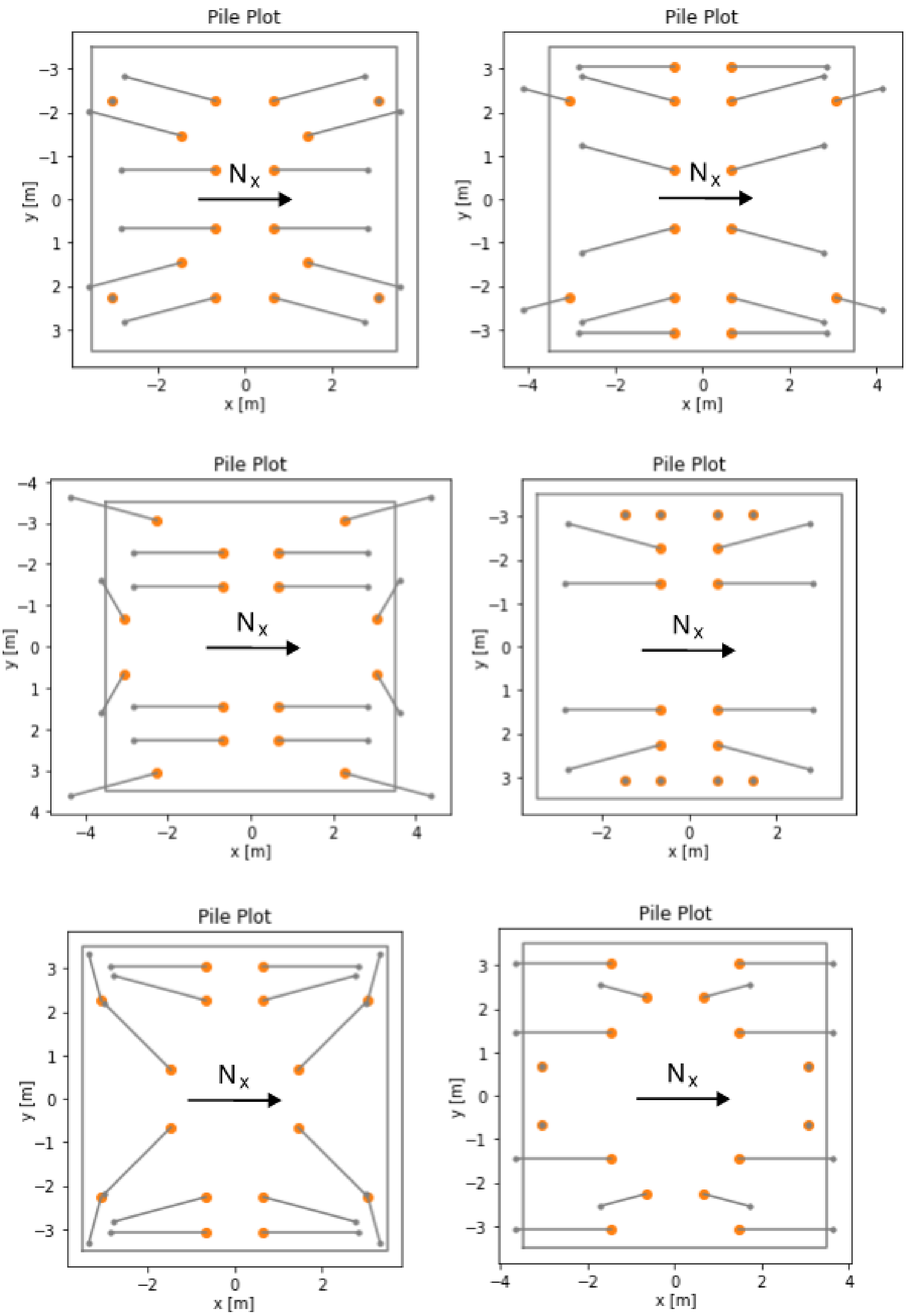


Figure B.1: $N_x = 10$ kN

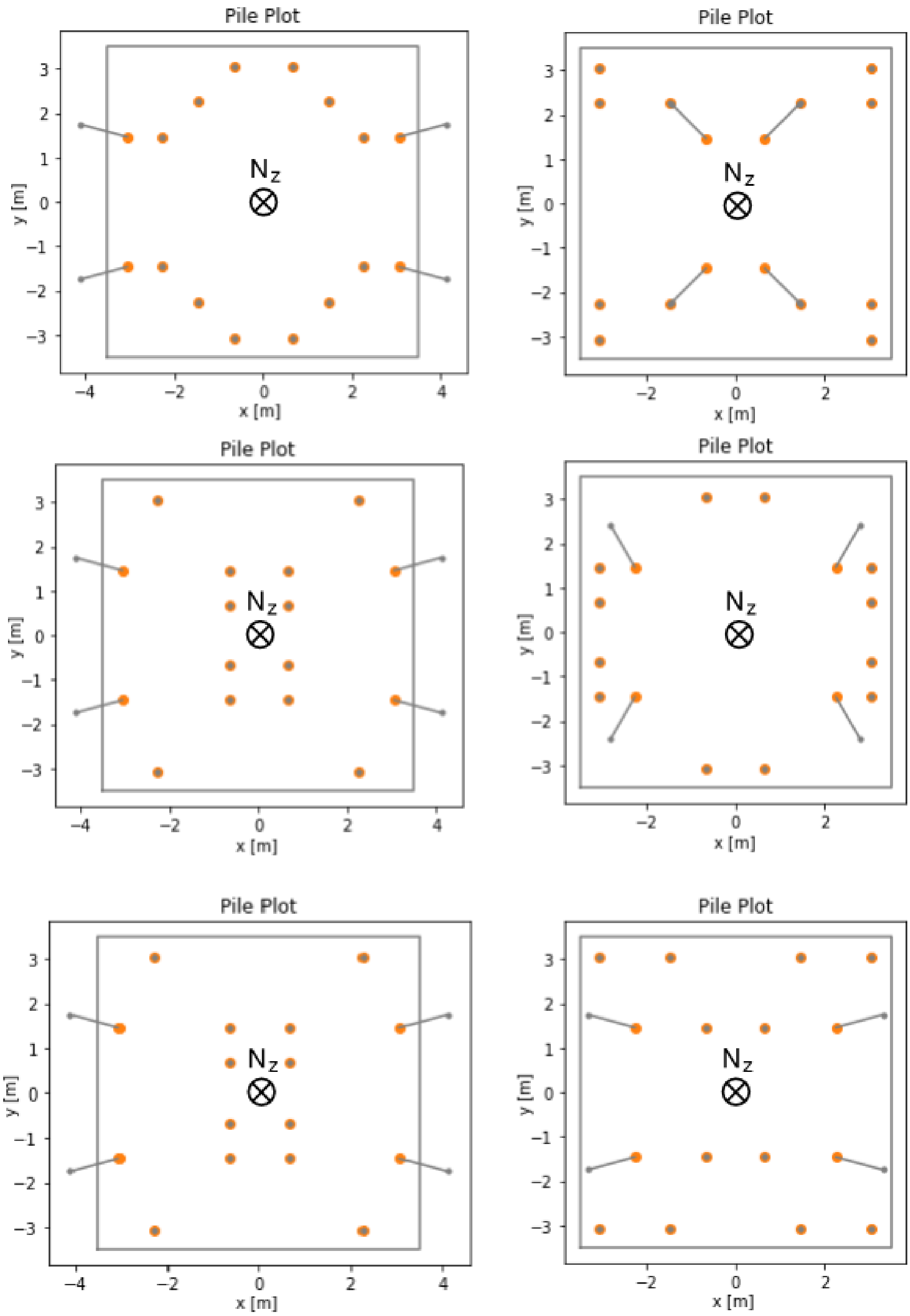


Figure B.2: $N_z = 100$ kN

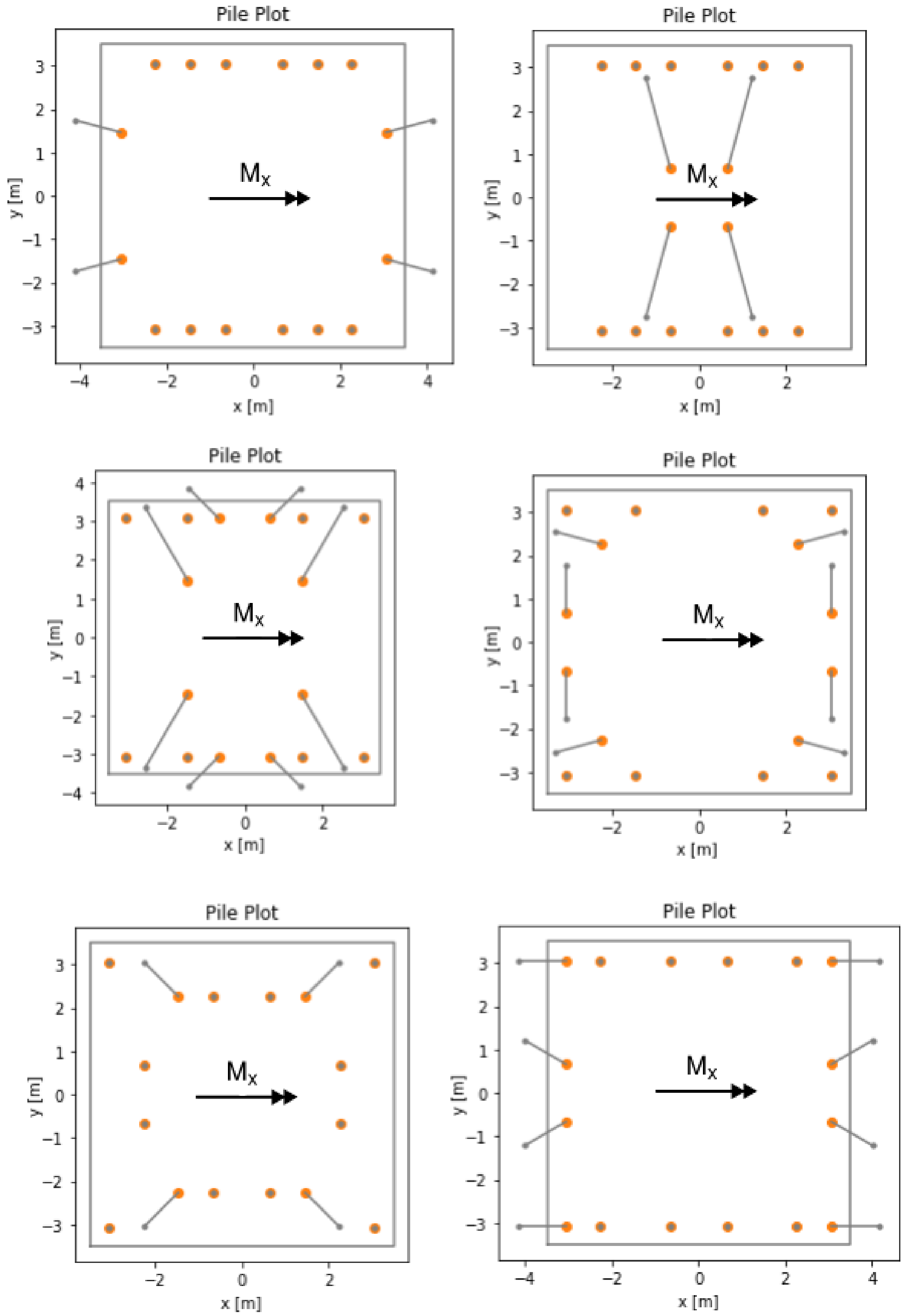


Figure B.3: $M_x = 100$ kN

Appendix C

Case I

Pedestrian Bridge Mid Support

Polaren Pål Report

Version 1.0

Report created 2020-05-08 13:33:34 by Frida Liljefors & Linda Johansson

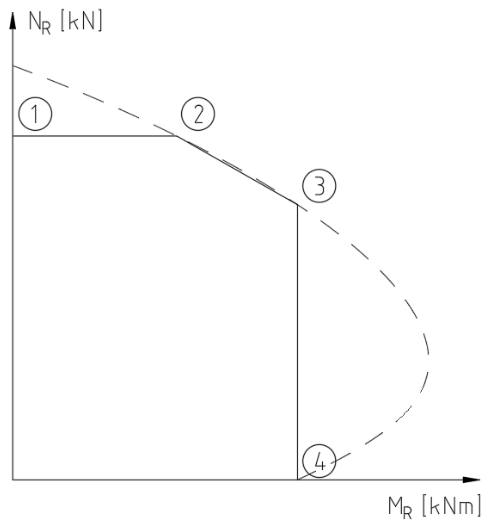
Project Case Study I. Pedestrian Bridge Mid Support.

General input data

Type of analysis	Random generetor: symmetry around two axes	
Number of pile groups	100000	
Pile connection to pile cap	Hinged	
Pile material	Concrete	
Cross section pile	Quadratic	
Diameter/width d	0.27	[m]
Length of pile L	9	[m]
Youngs modulus E	37.3	[GPa]
Shear modulus G	12.0	[GPa]

Pile capacity

Nmax (1)	800.0	[kN]
M (2)	20.0	[kNm]
N (3)	500.0	[kN]
Mmax (4)	30.0	[kNm]
Nmin	-30.0	[kN]
Umax	0.067	[m]



Soil parameters

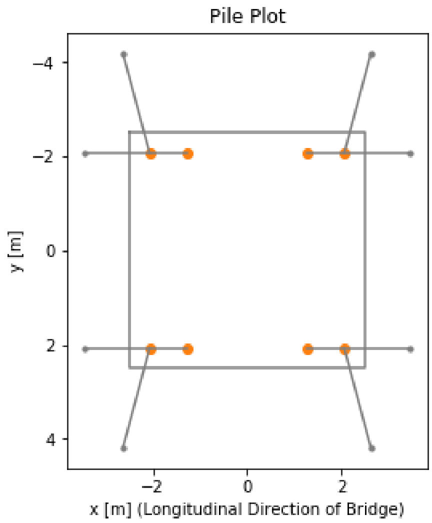
Type of soil	Cohesion	
Subgrade modulus kd	300.0	[kPa]

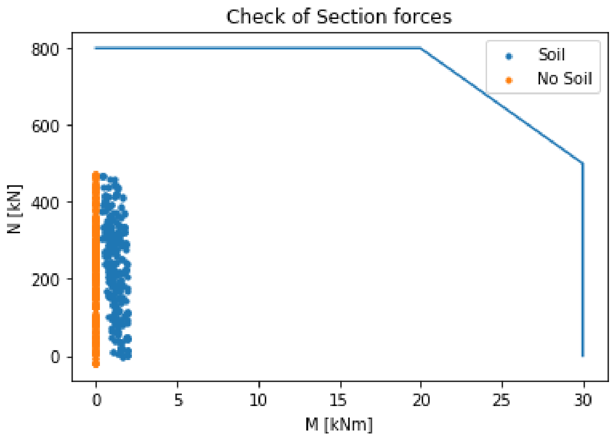
Loads [kN/kNm]

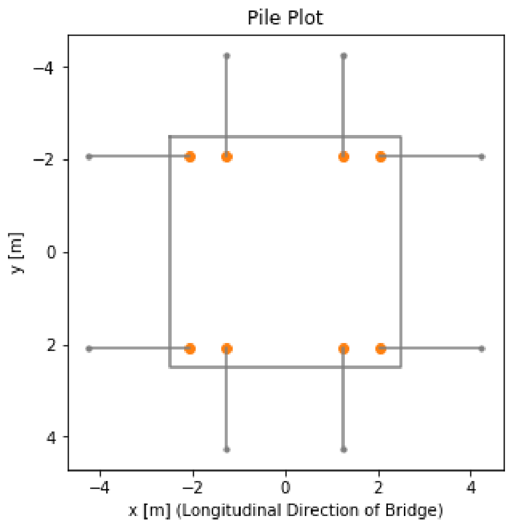
Nx	Ny	Nz	Mx	My	Mz
185.5	-34.9	1681.3	-256.0	-1132.7	0.0
-169.8	-34.9	1608.8	-256.0	812.9	0.0
-116.4	-92.9	1682.5	-671.4	1146.9	0.0
-116.4	92.9	1682.5	671.4	1146.9	0.0
-116.4	-52.8	1285.5	-382.8	1017.8	0.0
-116.4	-52.8	2430.5	-392.5	1283.4	0.0
-116.4	70.3	2106.1	679.4	1236.8	0.0
-116.4	-92.9	2106.1	-679.4	1222.4	0.0
185.5	-34.9	1994.6	-259.0	-1380.5	0.0
-152.9	-89.9	2106.1	-498.9	1432.7	0.0
21.5	-51.9	2430.5	-388.8	96.7	0.0
148.8	-51.9	2430.5	-388.8	-1038.2	0.0
148.8	51.9	2430.5	388.8	-1038.2	0.0
21.5	51.9	2430.5	388.8	96.7	0.0
-0.5	51.9	2430.5	388.8	200.7	0.0
-127.8	51.9	2430.5	388.8	1335.6	0.0
-127.8	-51.9	2430.5	-388.8	1335.6	0.0
-0.5	-51.9	2430.5	-388.8	200.7	0.0
46.8	-89.9	1307.7	-657.2	-79.4	0.0
173.9	-89.9	1307.7	-657.2	-1035.4	0.0
173.9	89.9	1307.7	657.2	-1035.4	0.0
46.8	89.9	1307.7	657.2	-79.4	0.0
-86.2	89.9	1307.7	657.2	878.8	0.0
-158.2	89.9	1307.7	657.2	1170.4	0.0
-158.2	-89.9	1307.7	-657.2	1170.4	0.0
-86.2	-89.9	1307.7	-657.2	878.8	0.0
101.9	-67.3	1875.8	-663.6	-662.6	0.0
185.5	34.9	1994.6	259.0	-1380.5	0.0
101.9	67.3	1875.8	663.6	-662.6	0.0
-25.8	67.3	2106.1	667.2	372.4	0.0
-152.9	67.3	2106.1	667.2	1432.7	0.0
-152.9	-67.3	2106.1	-667.2	1432.7	0.0

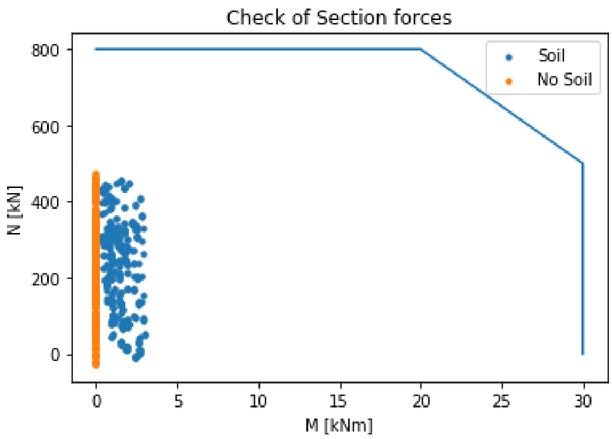
-25.8	-67.3	2106.1	-667.2	372.4	0.0
101.9	-89.9	1380.2	-657.29	-654.7	0.0
185.5	-34.9	1306.5	-255.1	-1227.2	0.0
185.5	34.9	1306.5	255.1	-1227.2	0.0
101.9	89.9	1380.2	657.2	-654.7	0.0
-31.0	89.9	1307.7	657.2	238.1	0.0
-152.9	89.9	1380.2	657.2	1290.7	0.0
-152.9	-89.9	1380.2	-657.2	1290.7	0.0
-31.0	-89.9	1307.7	-657.2	238.1	0.0
185.5	-34.9	2104.9	-259.0	-1252.8	0.0
-169.8	-34.9	1943.8	-258.7	854.3	0.0
-116.4	52.8	2430.5	392.5	1283.4	0.0
173.9	-89.9	1307.7	-657.2	-1053.4	0.0
173.9	89.9	1307.7	657.2	-1053.4	0.0
-158.2	89.9	1307.7	657.2	1194.1	0.0
-158.2	-89.9	1307.7	-657.2	1194.1	0.0

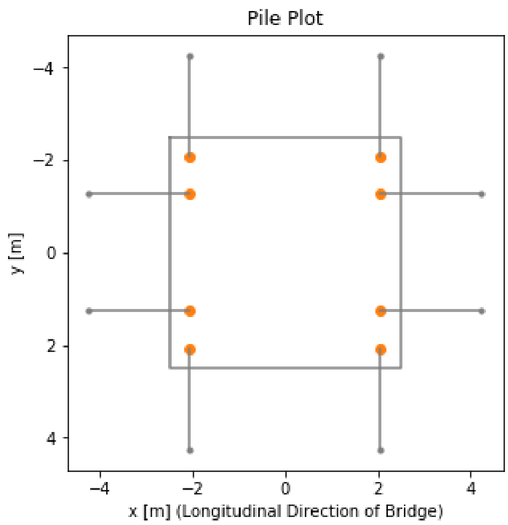
Table C.1: Result Case I. Pile plots and section forces for Pile Group 1, 2 and 3.

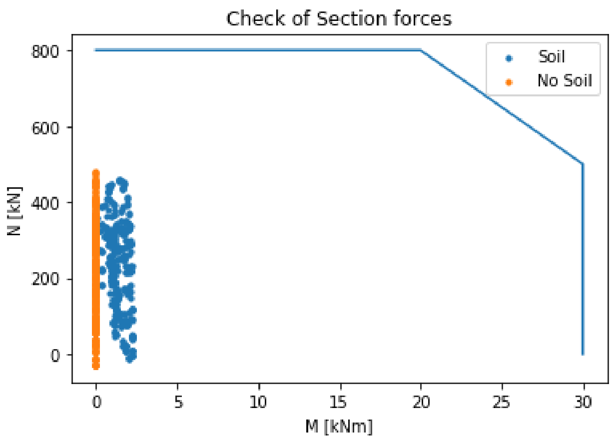












Appendix D

Case II

Pedestrian Bridge Abutment

Polaren Pål Report

Version 1.0

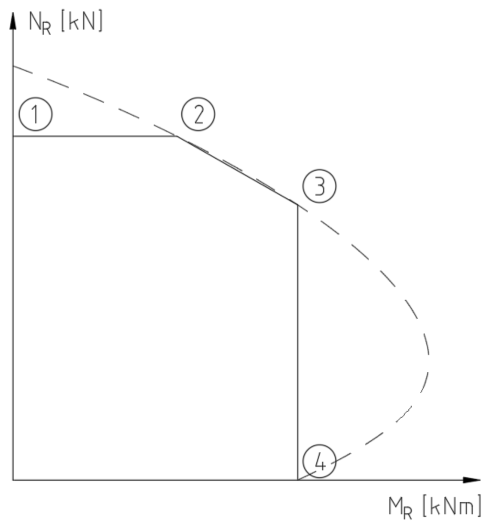
Report created 2020-05-08 13:46:37 by Frida Liljefors & Linda Johansson
Project Case Study II. Pedestrian Bridge Abutment.

General input data

Type of analysis	Random generator: symmetry around one axis	
Number of pile groups	100000	
Pile connection to pile cap	Hinged	
Pile material	Concrete	
Cross section pile	Quadratic	
Diameter/width d	0.27	[m]
Length of pile L	13	[m]
Youngs modulus E	37.3	[GPa]
Shear modulus G	12.0	[GPa]

Pile capacity

Nmax (1)	800.0	[kN]
M (2)	20.0	[kNm]
N (3)	500.0	[kN]
Mmax (4)	30.0	[kNm]
Nmin	-50.0	[kN]
Umax	0.12	[m]



Soil parameters

Type of soil	Cohesion	
Subgrade modulus kd	675.0	[kPa]

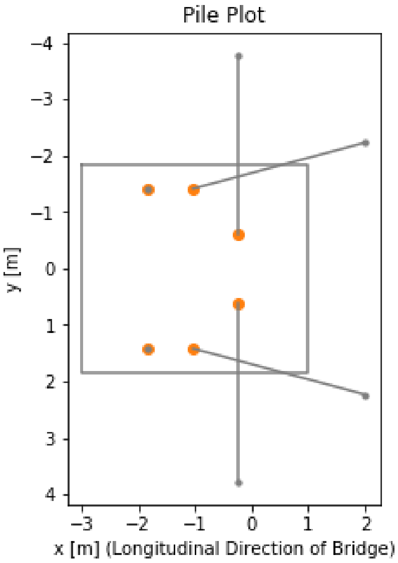
Loads [kN/kNm]

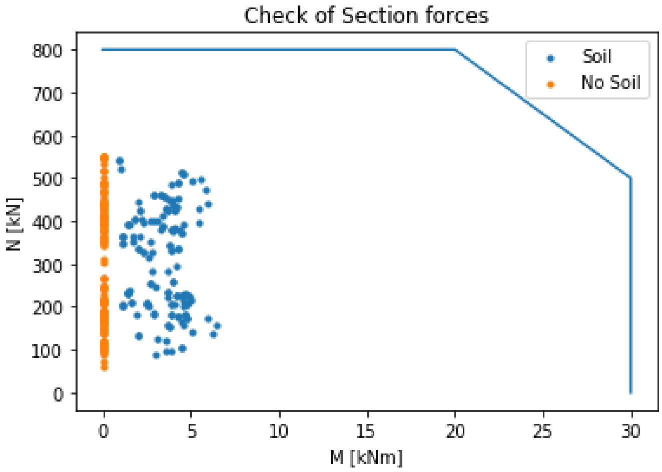
Nx	Ny	Nz	Mx	My	Mz
188.7	-33.9	1878.4	15.4	1656.9	0.0
41.9	-50.2	1811.2	-108.9	2138.6	0.0
91.8	-77.2	1861.6	-146.4	2134.6	0.0
91.8	77.2	1861.6	146.4	2134.6	0.0
91.8	-56.8	1523.2	142.6	1781.4	0.0
91.8	-33.9	2110.4	15.4	2280.1	0.0
91.8	46.6	1861.6	315.9	2134.6	0.0
91.8	-46.6	1861.6	-315.9	2134.6	0.0
188.7	-33.9	1580.8	15.4	1276.1	0.0
76.6	-25.8	2017.7	26.7	2306.2	0.0
68.7	-77.2	1903.9	-146.4	2086.3	0.0
181.0	-33.9	2110.4	15.4	1786.3	0.0
181.0	33.9	2110.4	-15.4	1786.3	0.0
58.4	77.2	1903.9	146.4	2101.1	0.0
68.7	-77.2	1540.4	-146.4	1816.1	0.0
181.0	-77.2	1540.4	-146.4	1287.7	0.0
181.0	77.2	1540.4	146.4	1287.7	0.0
58.4	77.2	1540.4	146.4	1830.9	0.0
76.6	-4.8	2017.7	142.9	2306.2	0.0
76.6	4.8	2017.7	-142.9	2306.2	0.0
53.5	-4.8	1706.5	142.9	2163.0	0.0
53.5	4.8	1706.5	-142.9	2163.0	0.0
181.0	-33.9	2079.0	15.4	1786.3	0.0
91.8	33.9	2110.4	-15.4	2280.1	0.0
181.0	77.2	1708.2	146.4	1586.6	0.0
181.0	-77.2	1708.2	-146.4	1586.6	0.0

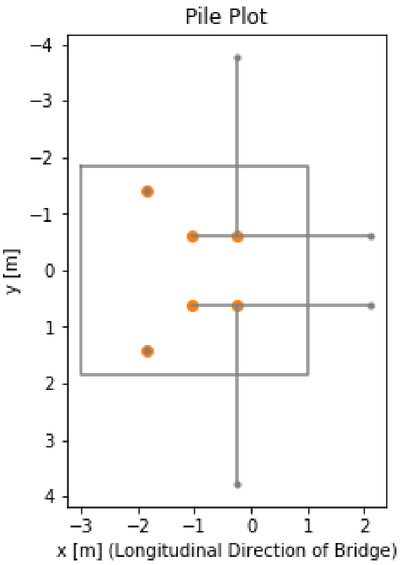
Reference pile group

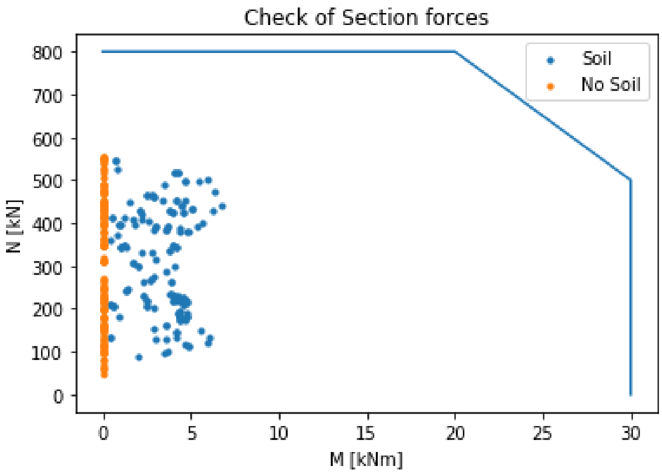
x [m]	y [m]	α [°]	β [1:X]	L [m]
-2.45	-1.35	270.0	4.0	13.0
-1.5	-1.35	180.0	4.0	13.0
0.5	-1.35	0.0	4.0	13.0
-2.45	1.35	90.0	4.0	13.0
-1.5	1.35	180.0	4.0	13.0
0.5	1.35	0.0	4.0	13.0

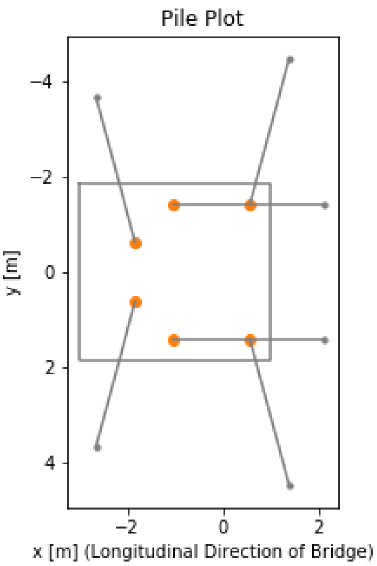
Table D.1: Result Case II. Pile plots and section forces for Pile Group 1, 2 and 3.

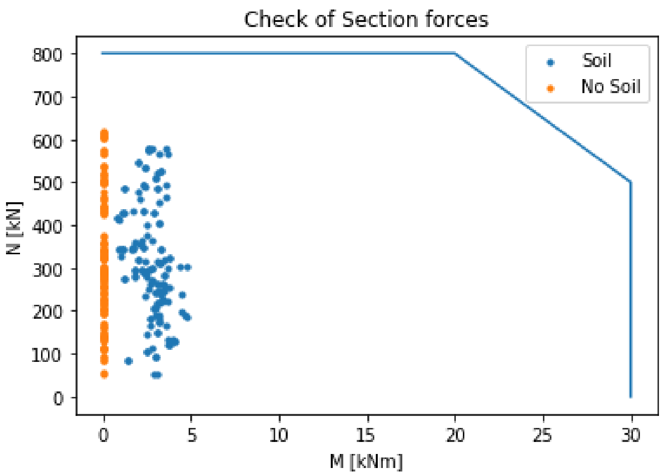












Appendix E

Case III

Road Bridge Mid Support

Polaren Pål Report

Version 1.0

Report created 2020-05-08 13:41:55 by Frida Liljefors & Linda Johansson

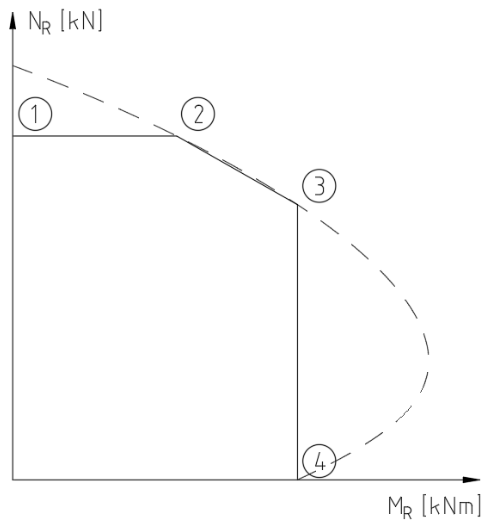
Project Case Study III. Road Bridge Mid Support.

General input data

Type of analysis	Random generetor: symmetry around two axes	
Number of pile groups	100000	
Pile connection to pile cap	Hinged	
Pile material	Concrete	
Cross section pile	Quadratic	
Diameter/width d	0.275	[m]
Length of pile L	51	[m]
Youngs modulus E	35.0	[GPa]
Shear modulus G	14.0	[GPa]

Pile capacity

Nmax (1)	1600.0	[kN]
M (2)	20.0	[kNm]
N (3)	1350.0	[kN]
Mmax (4)	30.0	[kNm]
Nmin	-200.0	[kN]
Umax	0.12	[m]



Soil parameters

Type of soil	Cohesion	
Subgrade modulus kd	533.0	[kPa]

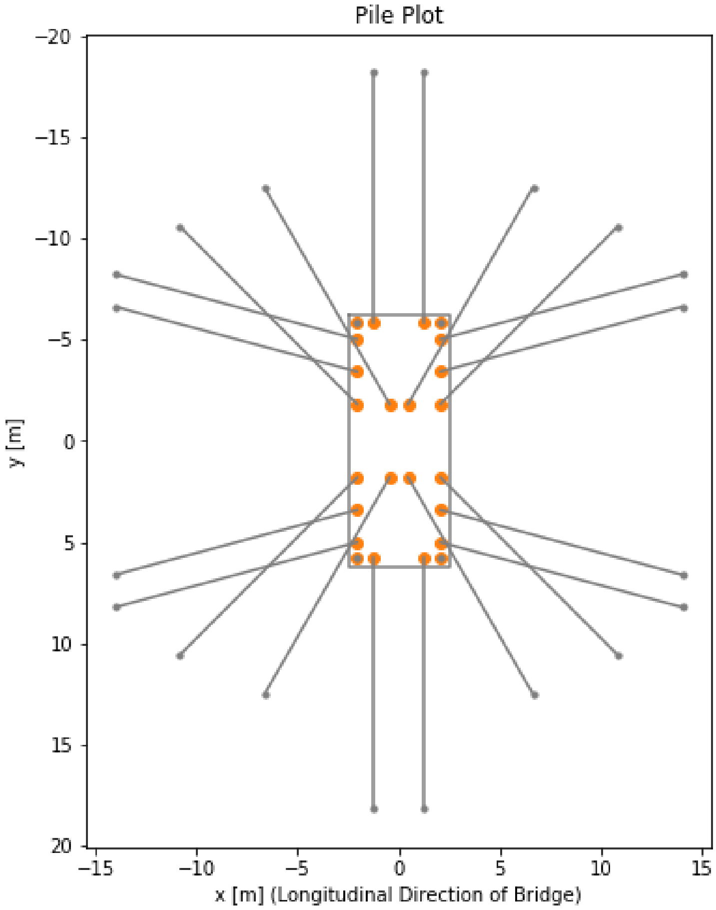
Loads [kN/kNm]

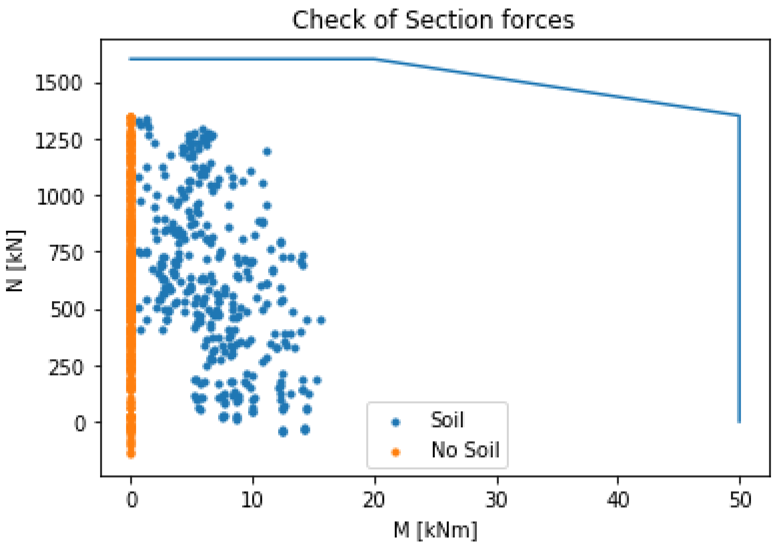
Nx	Ny	Nz	Mx	My	Mz
707.0	243.0	21401.0	8492.0	-10016.0	0.0
707.0	243.0	20924.0	9534.0	-9926.0	0.0
1189.0	243.0	18723.0	9317.0	-13158.0	0.0
807.0	243.0	7261.0	4132.0	-8095.0	0.0
807.0	243.0	10692.0	10819.0	-8628.0	0.0
1111.0	243.0	10003.0	3722.0	-11118.0	0.0
763.0	308.0	20292.0	11140.0	-10253.0	0.0
949.0	484.0	19311.0	10993.0	-11487.0	0.0
763.0	308.0	19656.0	12531.0	-10136.0	0.0
1398.0	484.0	16897.0	11773.0	-14514.0	0.0
863.0	308.0	7684.0	4833.0	-8615.0	0.0
1049.0	484.0	7885.0	6566.0	-10116.0	0.0
863.0	308.0	11777.0	13748.0	-9317.0	0.0
1319.0	484.0	10002.0	6202.0	-12808.0	0.0

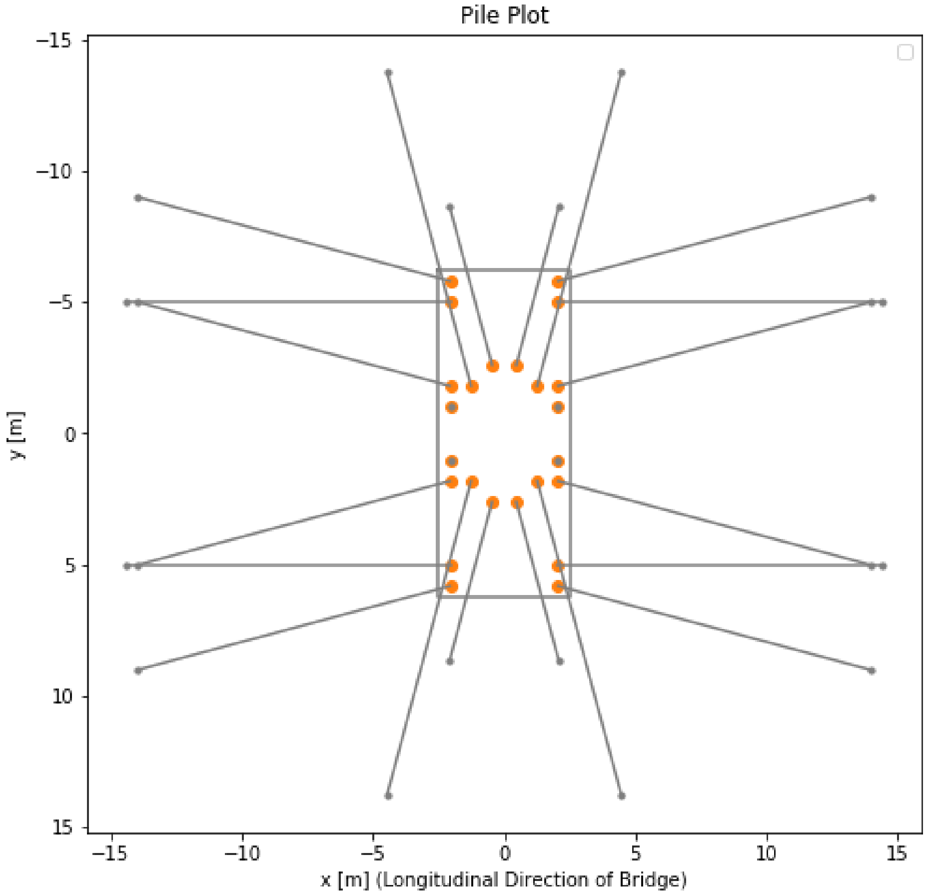
Reference pile group

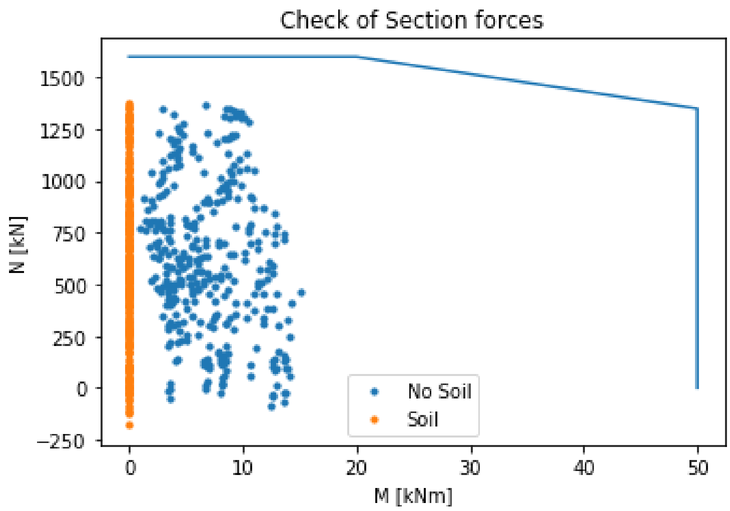
x [m]	y [m]	α [°]	β [1:X]	L [m]
-2.0	-4.5	180.0	8.0	51.0
-2.0	-3.5	180.0	13.0	51.0
-2.0	-2.5	180.0	8.0	51.0
-2.0	2.5	180.0	8.0	51.0
-2.0	3.5	180.0	13.0	51.0
-2.0	4.5	180.0	8.0	51.0
-1.0	-5.75	270.0	8.0	51.0
-1.0	-4.5	180.0	13.0	51.0
-1.0	-3.5	180.0	20.0	51.0
-1.0	-2.5	180.0	13.0	51.0
-1.0	2.5	180.0	13.0	51.0
-1.0	3.5	180.0	20.0	51.0
-1.0	4.5	180.0	13.0	51.0
-1.0	5.75	90.0	8.0	51.0
1.0	-5.75	270.0	8.0	51.0
1.0	-4.5	0.0	13.0	51.0
1.0	-3.5	0.0	20.0	51.0
1.0	-2.5	0.0	13.0	51.0
1.0	2.5	0.0	13.0	51.0
1.0	3.5	0.0	20.0	51.0
1.0	4.5	0.0	13.0	51.0
1.0	5.75	90.0	8.0	51.0
2.0	-4.5	0.0	8.0	51.0
2.0	-3.5	0.0	13.0	51.0
2.0	-2.5	0.0	8.0	51.0
2.0	2.5	0.0	8.0	51.0
2.0	3.5	0.0	13.0	51.0
2.0	4.5	0.0	8.0	51.0

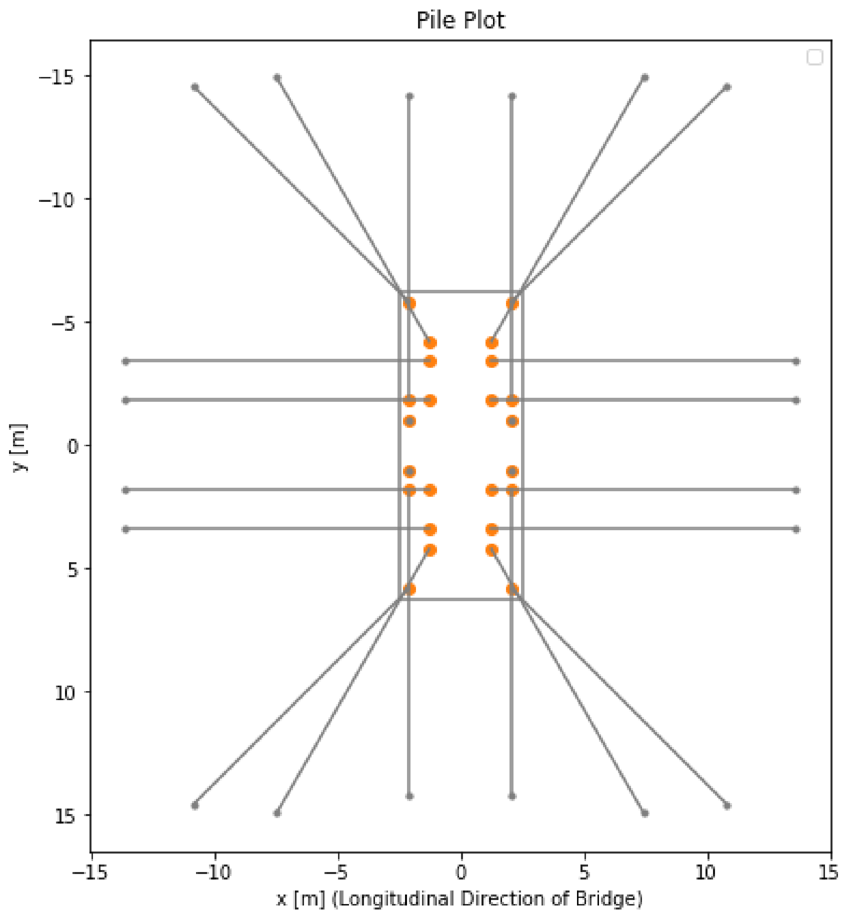
Table E.1: Result Case I. Pile plots and section forces for Pile Group 1, 2 and 3.

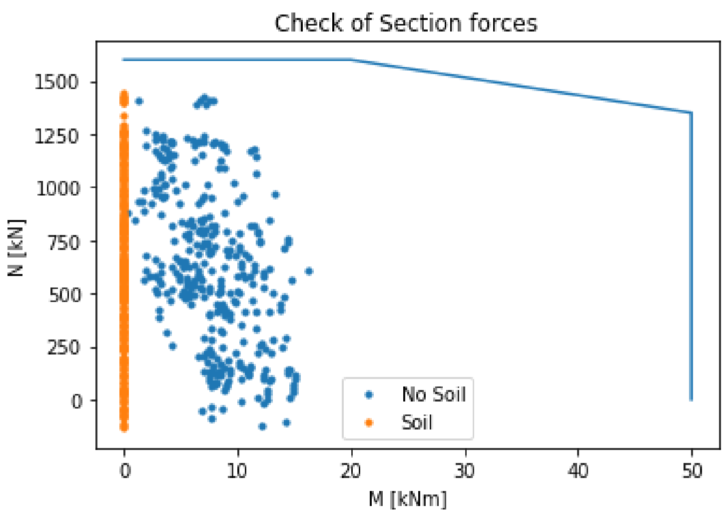





Pile Plot
−15
−10
−5
0
5
10
15
y [m]
−15
−10
−5
0
5
10
15
x [m] (Longitudinal Direction of Bridge)


Check of Section forces
1500
1250
1000
750
500
250
0
−250
N [kN]
0
10
20
30
40
50
M [kNm]
No Soil
Soil


Pile Plot
y [m]
x [m] (Longitudinal Direction of Bridge)


Check of Section forces
N [kN]
M [kNm]
No Soil
Soil

Appendix F

Case IV

Railway Bridge Abutment

Polaren Pål Report

Version 1.0

Report created 2020-05-08 13:42:01 by Frida Liljefors & Linda Johansson

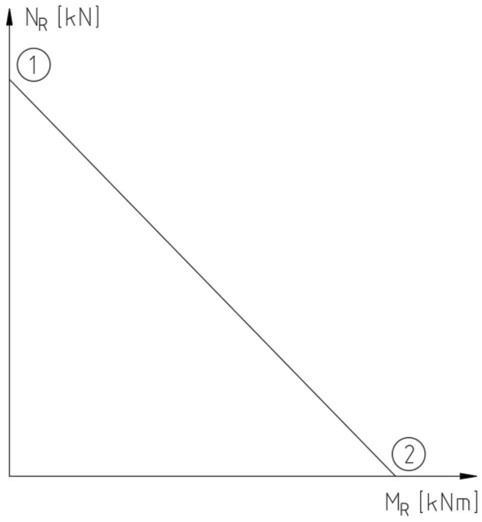
Project: Case Study IV. Railway Bridge Abutment.

General input data

Type of analysis	Random generator: symmetry around one axis	
Number of pile groups	10000	
Pile connection to pile cap	Hinged	
Pile material	Steel	
Cross section pile	Circular Hollow	
Outer diameter d	0.1683	[m]
Thickness t	0.0125	[m]
Length of pile L	28	[m]
Youngs modulus E	210.0	[GPa]
Shear modulus G	80.0	[GPa]

Pile capacity

Nmax (1)	1240.0	[kN]
Mmax (2)	89.0	[kNm]
Nmin	-731.0	[kN]
Umax	0.12	[m]



Soil parameters

Type of soil	Cohesion	
Subgrade modulus kd	509.0	[kPa]

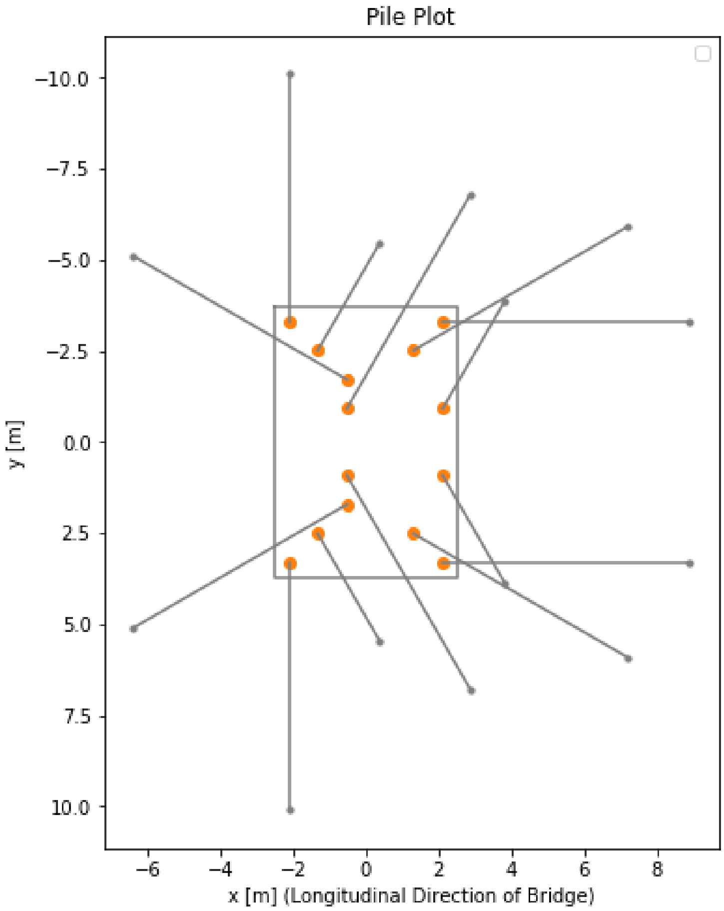
Loads [kN/kNm]

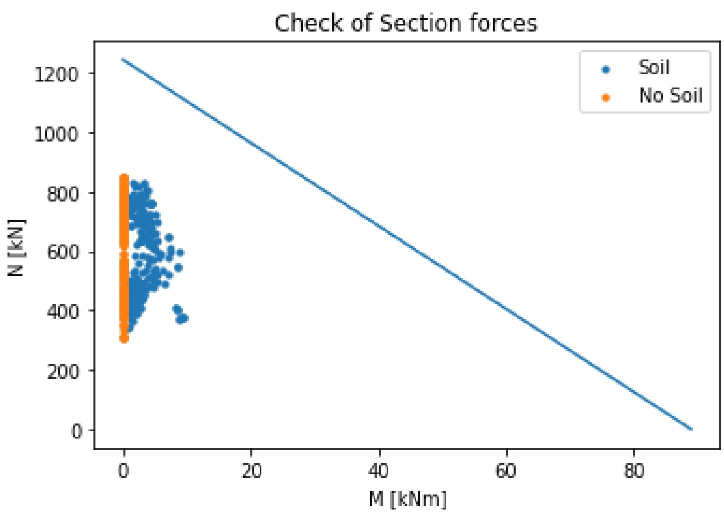
Nx	Ny	Nz	Mx	My	Mz
777.0	-75.0	10109.0	-1316.0	-1436.0	0.0
754.0	-127.0	10109.0	-1641.0	-1399.0	0.0
488.0	-75.0	6964.0	-1316.0	-2261.0	0.0
466.0	-127.0	6964.0	-1641.0	-2225.0	0.0
418.0	-23.0	6123.0	-150.0	-1551.0	0.0
418.0	-23.0	6123.0	-150.0	-1551.0	0.0
731.0	-75.0	10109.0	-1316.0	-1363.0	0.0
746.0	-127.0	10109.0	-1641.0	-1418.0	0.0
746.0	-88.0	9401.0	-1608.0	-1418.0	0.0
717.0	-152.0	9410.0	-2014.0	-1373.0	0.0
470.0	-88.0	7174.0	-1608.0	-2377.0	0.0
441.0	-152.0	7174.0	-2014.0	-2331.0	0.0
381.0	-23.0	6123.0	-150.0	-1488.0	0.0
381.0	-23.0	6123.0	-150.0	-1488.0	0.0
689.0	-88.0	9401.0	-1608.0	-1327.0	0.0
689.0	-88.0	9401.0	-1608.0	-1327.0	0.0
119.0	0.01	6123.0	0.01	-1086.0	0.0

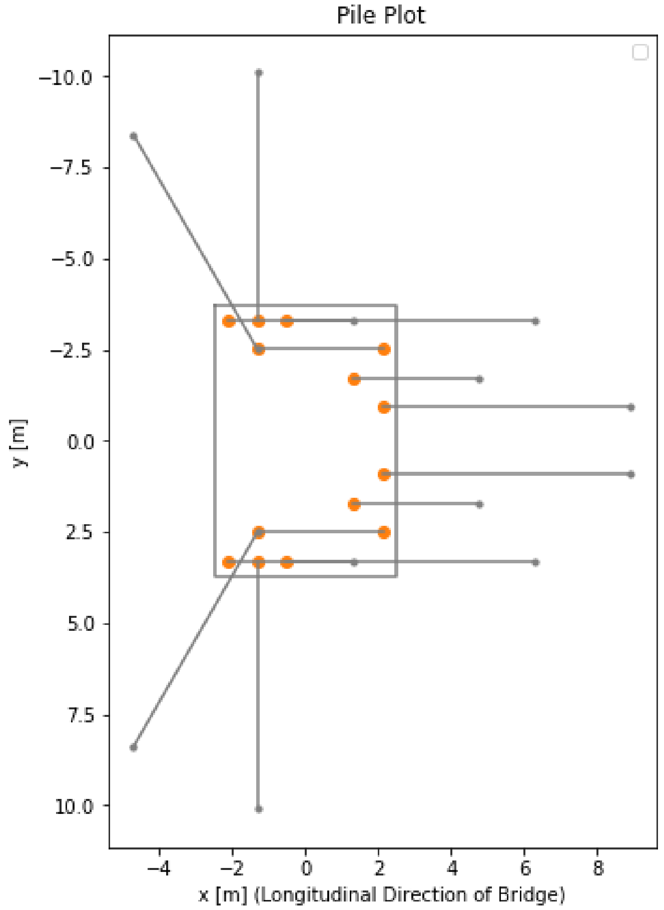
Reference pile group

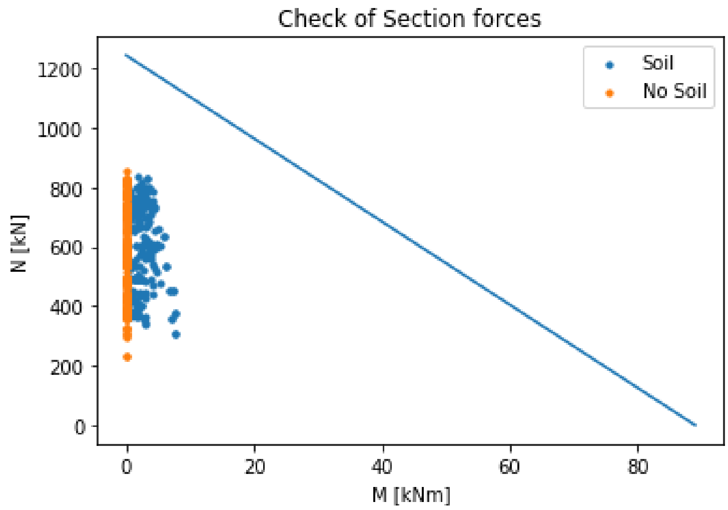
x [m]	y [m]	α [°]	β [1:X]	L [m]
2.0	-3.2	0.0	4.0	28.0
2.0	-1.6	0.0	4.0	28.0
2.0	1.6	0.0	4.0	28.0
2.0	3.2	0.0	4.0	28.0
0.55	-3.2	270.0	4.0	28.0
0.55	-2.4	0.0	4.0	28.0
0.55	-0.8	0.0	4.0	28.0
0.55	0.8	0.0	4.0	28.0
0.55	2.4	0.0	4.0	28.0
0.55	3.2	90.0	4.0	28.0
-2.0	-3.2	180.0	4.0	28.0
-2.0	-2.4	270.0	4.0	28.0
-2.0	-1.6	180.0	4.0	28.0
-2.0	1.6	180.0	4.0	28.0
-2.0	2.4	90.0	4.0	28.0
-2.0	3.2	180.0	4.0	28.0
-1.2	-3.2	270.0	4.0	28.0
-1.2	-0.8	180.0	4.0	28.0
-1.2	0.8	180.0	4.0	28.0
-1.2	3.2	90.0	4.0	28.0

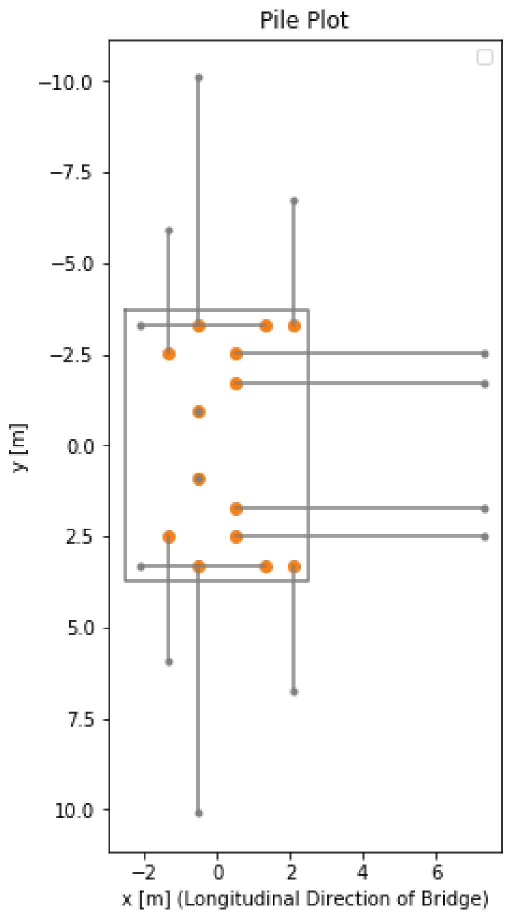
Table F.1: Result Case I. Pile plots and section forces for Pile Group 1, 2 and 3.

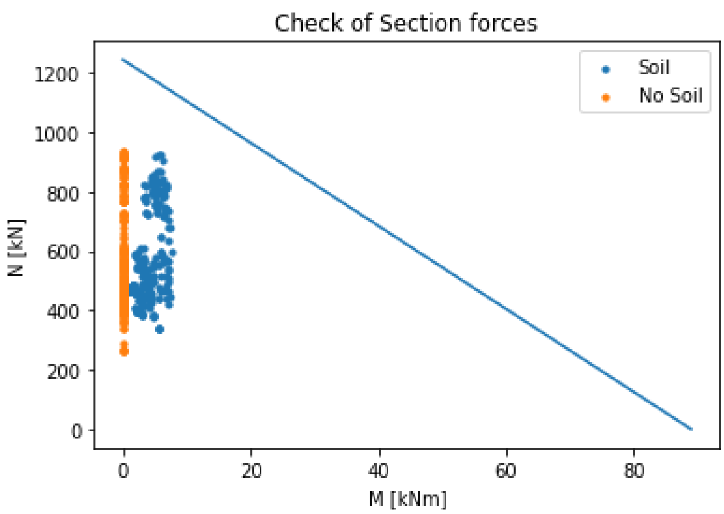





Pile Plot
−10.0
−7.5
−5.0
−2.5
0.0
2.5
5.0
7.5
10.0
y [m]
−4
−2
0
2
4
6
8
x [m] (Longitudinal Direction of Bridge)


Check of Section forces
Soil
No Soil
1200
1000
800
600
400
200
0
N [kN]
0
20
40
60
80
M [kNm]


Pile Plot
−10.0
−7.5
−5.0
−2.5
0.0
2.5
5.0
7.5
10.0
y [m]
−2
0
2
4
6
x [m] (Longitudinal Direction of Bridge)


Check of Section forces
Soil
No Soil
1200
1000
800
600
400
200
0
N [kN]
0
20
40
60
80
M [kNm]