MASTER THESIS

OPTIMIZATION OF URBAN QUAY WALL DESIGN

"A parametric approach to improve the structural efficiency"

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Thesis report

Optimization of urban quay wall design

A parametric approach to improve the structural efficiency

By

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Preface and Acknowledgements

This research is carried out in order to obtain a master's degree in the field of Civil engineering. The subject was suggested by civil contractor Mobilis, in agreement with the Technical University of Delft. By offering me the space to give substance to the direction of the research, I was able to implement fields of my interests.

At the start of my bachelor in Civil Engineering, I used to think: Why do we need to start from scratch with every structural calculation when someone has already done almost the same? Soon I found out that for each project many conditions and requirements could differ. By working at an engineering office besides finalizing my bachelor study, I learned that usually parts of the calculations are standardized (e.g. spreadsheets) and by linking them manually a uniform story is obtained. Still, the question has always been with me through the study period. At the master's course of Parametric design I was able to dig deeper in options of standardization in design and become inspired by its potential and how it could be deployed for almost unlimited purposes. By looking for a thesis subject my preference went to a parametric nature, which came together with the subject proposed by Mobilis.

Upon finalizing this thesis, I would like to express my gratitude to the graduation committee. At first, thank you Richard for helping me through the process and (together with Nick) for frequently sharing thoughts about the direction of the thesis. I would like to thank the committee members from the Technical University of Delft for their engagement and constructive feedback. Your critical view has been very valuable to me in guiding the direction of my research.

Lastly, I would like to take this opportunity to thank my dear family and friends for their support. In specific my parents, who made it possible for me to get where I am and have been motivating me my whole life. And of course, my special thanks goes out to my wife. Imane, thank you for your patience, for being my daily support and for encouraging me during the last part of my study period.

Sharif Ahmed Mohamed el Hag
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Abstract

Over the last years the critical state of existing quay wall structures have gained a lot of concern. More frequently cases have occurred in which the malfunctioning of quay wall structures came to light. Sinkholes, displaced walls and even collapses of complete quays appeared to be necessary to raise awareness to the current condition of existing quay wall structures. Starting in 2019, the municipality of Amsterdam initiated a program in which a lot of investigation is carried out to gain insight in the condition of Amsterdam's quay walls, and simultaneously investing in the gain of knowledge and stimulating innovation with respect to reinforcement or complete renovation of the existing structures. The biggest challenge for reinforcement or renovation works in a complex urban contexts lies usually in the construction phasing. A lot of restraining factors (e.g. adjacent structures) and stakeholders (concerning the accessibility, houseboats, trees, cables and pipelines) need to be dealt with, which limit the possibilities for both the design and the construction.

The focus in this research is placed at the structural efficiency of the design for renovation projects. The term structural efficiency implies the effectiveness of material use, for which the least amount of material required to carry a certain load provides the highest structural efficiency. By using a parametric approach in the preliminary design phase, a design solution with the lowest use of material can be found. Following from the parametric approach, the governing design aspects become insightful. By taking measures which utilized the capacities for each of the design aspects as much as possible, an optimization of the design can be pursued.

To define the applicability of the design tool presented in this research, a data analysis has been performed based on the scope of the municipality of Amsterdam. From a total data set of about 640 kilometers of embankment managed by the municipality, filters have been applied based on the current structure type, the design space limitations and the loads that can be expected. The scope is narrowed down to the categories in which high loads can be expected and limited space is available, mostly reflecting the quays in the central parts of Amsterdam. By considering all the loads involved with a quay in this context, a comparison is made between the different variable loads and a uniformly distributed load. From this comparison a distributed load of 20 kN/m^2 is reasoned to be a valid upper value covering for the summation of variable load effects.

In a variant study (comparing a variety of solutions which are considered a suitable option in providing the structural function within the limited design space) a design solution with an optional inclined floor is selected as the best variant in line with the main objective; Reducing the material use. The function of a quay wall structure is to overcome a certain retaining height by resisting a difference in horizontal loads. The limited design space does not allow for any horizontal supporting system, requiring the horizontal load difference to be resisted by means of bending moments. The most vulnerable design aspect for

this load action is the connection to the piles, which are assumed to be steel casing piles with a reinforced concrete core. For multiple pile rows an internal leverage arm can be created, decomposing part of the bending moments into tensile and compressive actions. Another way to deal with the horizontal load difference is to adjust the orientation of the structure. By inclining the piles in the same direction as the direction of the resultant load, the eccentricity can be taken out of the structure. Due to an assumed maximum pile inclination of 5:1 the adjustment of the orientation of the structure is also limited. In order to bring the resultant direction of the piles closer to the resultant direction of the loads, the amount of collected vertical loads need to be considered. Depending on the retaining height and the available width of a certain location, the width, the depth or the inclination of the floor can be adjusted in order to take advantage of the amount of horizontal and vertical loads that is being resisted.

A parametric model has been developed using the visual programming environment Grasshopper 3D within Rhinoceros. For performing the structural calculation an interface with the FEA-software RFEM is adopted, allowing for the export of the geometry, support conditions, cross-sections, loads and other model data. The results from the structural analysis are retrieved to the Grasshopper environment to be processed into assessments of the most governing design checks, resulting in a set of unity checks and the amounts of materials for each set of inputs. By automation of the export-import process for a variety of input combinations, all options within a specified range could be calculated. From all results, the combination of input parameters resulting in the lowest value for the material use while satisfying all unity checks is considered the best solution.

It can be stated that for dimensions related to the defined scope, 2 situations can be distinguished. If the available width of the floor is limited, an eccentric load cannot be avoided. For these over-eccentric situations the pile head bending moments turn out to be the governing design aspect. To reduce the eccentricity, the aim is to apply as many piles as possible at the maximum inclination while the deformations are just within limits.

If sufficient width is available to allow for a resultant load direction in the same direction as the resultant direction of the piles, the pile head bending moments are not the governing design aspect. By increasing the floor thickness, large spans could be made. Depending on the soil conditions, the deformations or the axial capacity of the piles become governing. However, focusing on the material use, the optimum is to be found for the situation in which a certain eccentricity is present. In that case the bending capacity of the piles is also being utilized. As the horizontal load can be assumed as constant depending on the retaining height, introducing an eccentricity in the resultant load can be done by reducing its vertical component. This can be achieved by inclining the floor in order to reduce the weight of the soil. However, the same effect (even more effective) can be reached by reducing the width of the floor. From a construction point of view the latter option would always be preferred.

The results presented in this research can be considered as an indicative proposal for a more structural effective solution, as well as a demonstration of the power to be found in a parametric approach. The repetitive character in especially such large-scale assignments as the renovation of Amsterdam's quay walls makes this strategy highly appropriate. Considering the current financial pressure resulting from rising material costs, the intended savings on the material use deserves more attention than ever before.

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Chapter 1

Problem definition

1.1 Introduction

As many of the quay walls in the Netherlands are more than a hundred years old, their current condition is of great concern. Mostly because the current condition is not exactly known, which makes it hard to find out what the residual service life of the structures is. But the fact remains that for some of the quay walls, after being in service for about a hundred years, the end of their lifetime can be expected soon. The consequences of that are becoming visible more frequently with recent cases in Amsterdam[1][2] and Utrecht, where quay walls collapsed due to the lack of maintenance and worsening soil conditions. For the case of the collapse at the Grimburgwal in Amsterdam in 2020, a combination of unfavorable effects have been linked to the traffic in the canal at that specific location causing a local lowering of the canal bed.

While the old quay walls have been designed primarily as simple retaining walls, the loads they have to deal with nowadays are much higher, with for instance heavy delivery trucks passing them on a daily basis. Over a lifetime of increasingly heavy loading, signs of structural failure are logically to be expected. To reduce the risks of failure, the municipality of Amsterdam has already implemented some precautionary measures. The central zone of Amsterdam is appointed as a so-called '7.5-ton zone', for which the access of heavyweight traffic is restricted. Traffic between 7.5 tons and 30 tons are allowed within this zone only by exemption. Traffic above 30 tons can be given an exemption but only by following strict routes to avoid the most critical quays. To guarantee that the quay walls will satisfy the needs of today and the future their residual lifespan should be checked, and the structures should be reinforced or replaced if necessary.

Figure 1.1: Traffic restrictions a quay in Amsterdam

Because of the high uncertainties, much research on quay walls is focused on mapping of the current soil and structural conditions. But as the criticality of this case becomes more and more clear, also much research is dedicated to the most effective way of replacing or reinforcing the quay walls. The biggest challenge that is being considered is the fastest construction method with the least disturbance of traffic (both on the road and inside the canal), minimum CO2-emission and noise pollution. In 2018, the municipality of Amsterdam started a tender called ‘Innovatief Partnerschap Kademuren’ (IPK)[3], a first-time use of this tendering procedure for such a wide-scale project. With this type of tender the municipality encouraged the market to come up with innovative solutions to tackle the problems that are specific for replacement/reinforcement of Amsterdam’s quay walls, but also to stimulate for example multi-functional uses of the quays.

The assignment that the municipality of Amsterdam is facing is already referred to as Amsterdam’s biggest infrastructural challenge. About 900 kilometers of quay is within the district of the municipality, of which about 600 kilometers is in their management. The other part is managed by other authorities or by private owners. From the 600 kilometers about 200 kilometers contains a structure with a foundation. While in the last coalition agreement (Begroting Gemeente Amsterdam 2019[4]) an additional budget of only 22.5 million has been allocated to the maintenance of the bridges and quay walls, today’s prospects have turned into a multi-billion project.

In July 2021 alderman Egbert de Vries (Verkeer Gemeente Amsterdam) mentioned[5] an indicative price of about 35 thousand euros per meter quay wall renovation. Managing a scope of more than 200 kilometers of quay wall, over an expected lifespan of 100 years the total of renovation cost would be 7 billion euros, implying a yearly cost of 70 million euros. On top of that, due to rising costs of construction materials and additional restrictions in construction methods, the estimations about a year later are more than 60 thousand euros per meter quay.

Although the municipality of Amsterdam nowadays is well aware of the risks that are hidden behind these historical structures, the severe conditions have been neglected for a long time. For several cases over the last decades where the critical conditions of quays came to light, the municipality opted for a temporary reinforcing structure inside of the canal to prevent worsening, and monitoring of the most vulnerable structures. Next to the unknown conditions, postponement of works also can be traced to the complexity of the replacement of quay walls and hope for better replacement or reinforcement solutions. To catch up on the backlog that we are in today, a massive up-scaling of maintenance and replacement is required. Investigation of the current conditions of quay walls is still in progress, but the quay walls with the highest priority are mapped and assigned for replacement.

1.2 Functionality

The two main functions of a quay wall structure are (1) to resist the horizontal and vertical loads and (2) to provide a soil-tight barrier between the water body and the soil. By making this barrier vertical, more functional space can be created both on land-side and and inside the canal, serving the accessibility of the area or the effective use of (valuable) space. Especially for inner-city areas, this means the gain of very valuable space.

The barrier structure can be made in several forms, but next to the space on land and in the canal, also the space in the ground is or can be exploited. In urban areas the ground is usually packed with cables and pipelines, and throughout the city several tunnels for road and rail traffic are present. In addition, several other functions of underground space could be thought of, like underground containers for the collection of waste or other type of storage.

For the traffic in the canals the quay wall functions as a mooring facility, for pleasure craft, commercial craft, excursion boats and nearly permanent houseboats. The space on top of the quay is generally used for traffic (cars, busses, trams, bikes and pedestrians) and parking purposes, but could also function for recreational purposes, providing space for trees and park benches. Subsequently, the trees hold multiple functions, like the reduction of heat stress, offering shade, preserving the biodiversity and (for many the most valuable function) their aesthetic value. The iconic scenes of Amsterdam's canals, which attracts many tourists throughout the year, are deemed to be incomplete without the trees. Being part of the UNESCO World heritage, strong demands need to be met regarding the appearance.

1.3 Problem statement

It is obvious that the time to take action has come. The municipality of Amsterdam has several options to deal with the situation. Of course, the preferred solution goes to the preservation of existing quays. But whether and for how many of the quays this is achievable assuring the safety of its daily use is still to be investigated.

As a result of the increasing alerts of defects in the existing structures of both quay walls and bridges, in 2019 the municipality has started the Program of Bridges and Quay walls (PBK). From the steps taken within the program it can be concluded that on every level activities are taking place; in investigation of preservation, (temporary) reinforcement and renovation of the quay walls. To give more of a background story to the issue of quay wall structures, the problems that they entail are pointed out in this section. First the concerns of existing historical quay walls are regarded, indicating why renovations are necessary. Subsequently the points of focus for the traditional quay wall structures are mentioned.

1.3.1 Problems with historical quay walls

The historical quay walls of Amsterdam usually consist of a simple retaining brick wall on a timber floor, supported by timber piles. Obviously, their structural setup is vulnerable compared to today's standards. According to the current calculation models, many of the existing quay walls should already have failed. But still these structures functioned for a long time while dealing with loads much higher than the loads for which they were originally constructed.

The first quay walls date from before the year 1500. During that time they were mainly used for mooring trading vessels, as the canal-side warehouses were owned by the merchants. The heavy loads that quays had to deal with in that time were the goods that were traded and the horse and carriage that passed by. The quay walls were owned by the merchants, meaning they were responsible for the keeping the condition in good order. The quay walls have been subjected to many renovation and reinforcement activities throughout the years. As the quay wall structures of today are sometimes 200 to 300 years old, there is hardly any information available in the archives of the municipality.

The existing historical structures are classified as gravity quay walls. By giving the brick wall enough weight, sliding and tumbling over of the wall as a result of the horizontal loads coming from the soil is prevented. The foundation consists of a simple wooden boards on capping beams, supported by timber piles. The foundation supports the wall vertically, but also have to deal with the horizontal loads coming from the wall. Timber sheets are placed below the floor to prevent erosion of the soil.

However, a common problem of the existing historical quay walls is that their retaining function is not fully sand-tight. Due to displacements within the structure or piping below the structure (through the timber boards/sheets), soil is able to flow out. This can lead to settlements of the ground behind the wall, which for large volumes of flushed soil results in sinkholes. Usually, the bad condition of a quay is the consequence of multiple (mutually worsening) effects. For example: Heavy traffic loads cause horizontal deformations of the structure, resulting in cracks in the wall. During a period of heavy rainfall, soil can drain away through the cracks. Due to piping below the structure the foundation piles become uncovered resulting in higher bending moments in the top of the piles. This leads to additional horizontal deformations which again increases the amount and width of cracks.

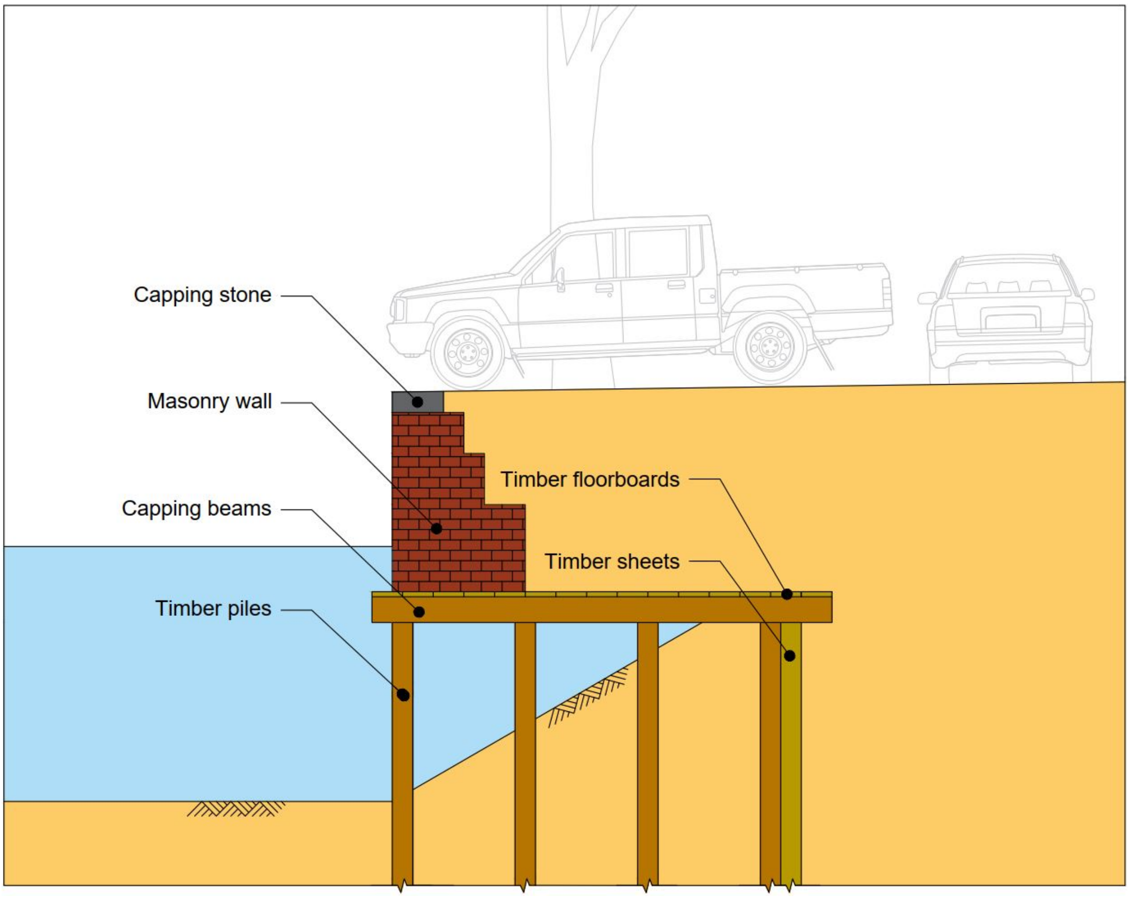

Figure 1.2: Schematic representation of a historical quay wall

1.3.2 Problems with traditional quay walls

Nowadays, after many advancements in general structural knowledge and in specific retaining structures, quay walls in a context such as in Amsterdam are generally made of L-wall structures or combi-walls. An L-shaped RC-element in which the floor functions as a relieving platform and counters bending of the soil load on the wall. This type of structure is especially suitable for Amsterdam, as most of the quays hold roads close to the canal from which high traffic loads can be expected. Anchored quay wall solutions are not possible due to closely adjacent buildings. Next to that, many cables and pipelines are present in the ground. Both below the structure and the canals, and above the slab of the structure (below the road), space should be reserved for cables, pipelines and possible other functionalities.

Considering the construction, it can be stated that the replacement of quay walls using traditional methods is very time-consuming, cause too much nuisance and is quite costly. With this method a construction pit is created around the existing quay wall, which is being demolished to make place for the new structure. The new piles are installed and connected to a cast in situ slab. Next, the wall is being cast in situ and the sheet pile at land-side of the structure is being cut off above the top of the slab. The other sheet piles are taken out. Especially the use of a dry construction pit contributes to the downsides of the traditional construction method. It takes a lot of time to put in place and to be taken out, and in addition it also requires a large space. For both the canal- and land-side the available space is usually very limited.

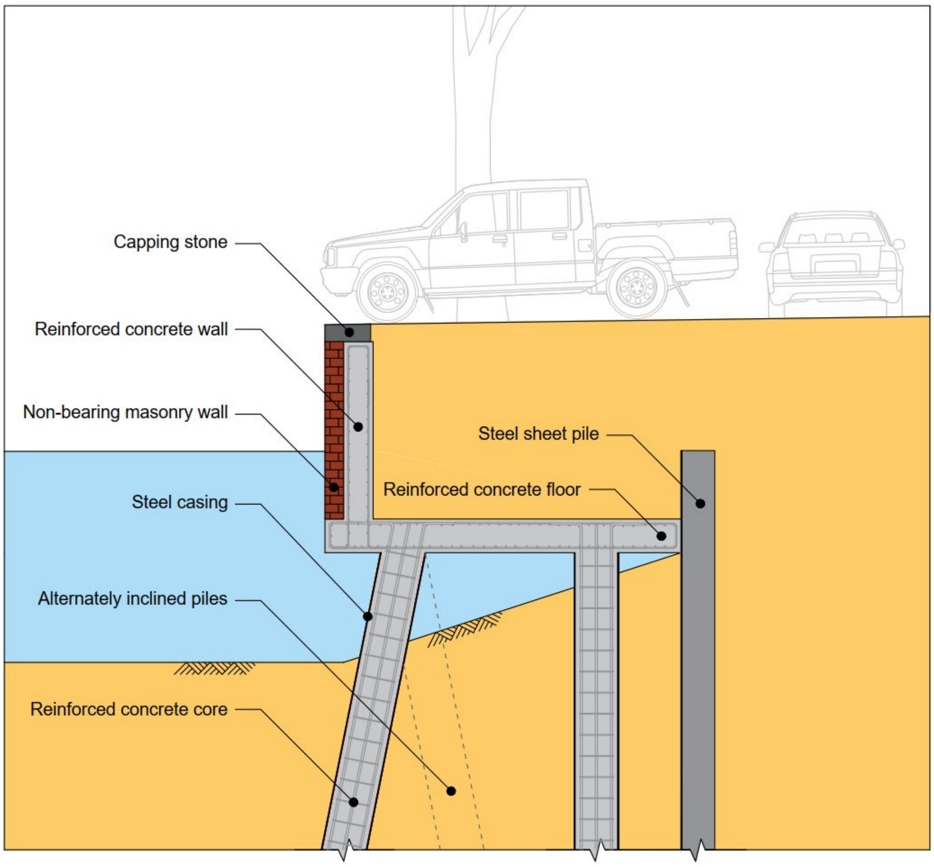

Figure 1.3: Schematic representation of a traditional L-wall

Some of the aspects of traditional L-shaped quay walls that are susceptible for improvements are:

- ◇ The structures still use a reasonable amount of material, as they are usually being overdesigned.
- ◇ The resultant load on the structure is quite predictable, and apart from exceptional load cases (like collision loads from the canal-side) always directed towards the canal. By adjusting the orientation of the piles the eccentricity and thus the bending moments on the piles can be reduced, which means the dimensions of the piles can be reduced and the connection to the floor can be more simple.
- ◇ Large construction pits (often within small working zones) are required, which cause a lot of nuisance for the area and makes the replacement very time-consuming.
- ◇ Trees need to be removed and re-planted if possible, or replaced by new trees.
- ◇ The structure needs to be sand-tight, and for that are also made watertight. To

prevent large unbalances of the water level in the soil and the canal, drainage systems need to be included in the design to make the structure semi-permeable.

- ◇ They are generally made of regular concrete. Implementation of more sustainable types of concrete could be applied to reduce the footprint.
- ◇ Design and reinforcement calculations are done 'manually' and individually. Automation of the design process can save a lot of calculation time, both in the design phase and during the construction phase when unforeseen circumstances require quick adjustments to the design.

For some cases, where the available workspace is very limited or critical due to accessibility requirements on the quay and in the canal, a combi-wall system is preferred over an L-wall. A combi-wall consists of steel piles with sheet piles in between the piles. This solution does not require a construction pit, as the structure can be placed right in front of the existing structure. This solution can be constructed in a relative short period, but it uses large amounts of steel (susceptible for corrosion) and it reduces the width of the canal.

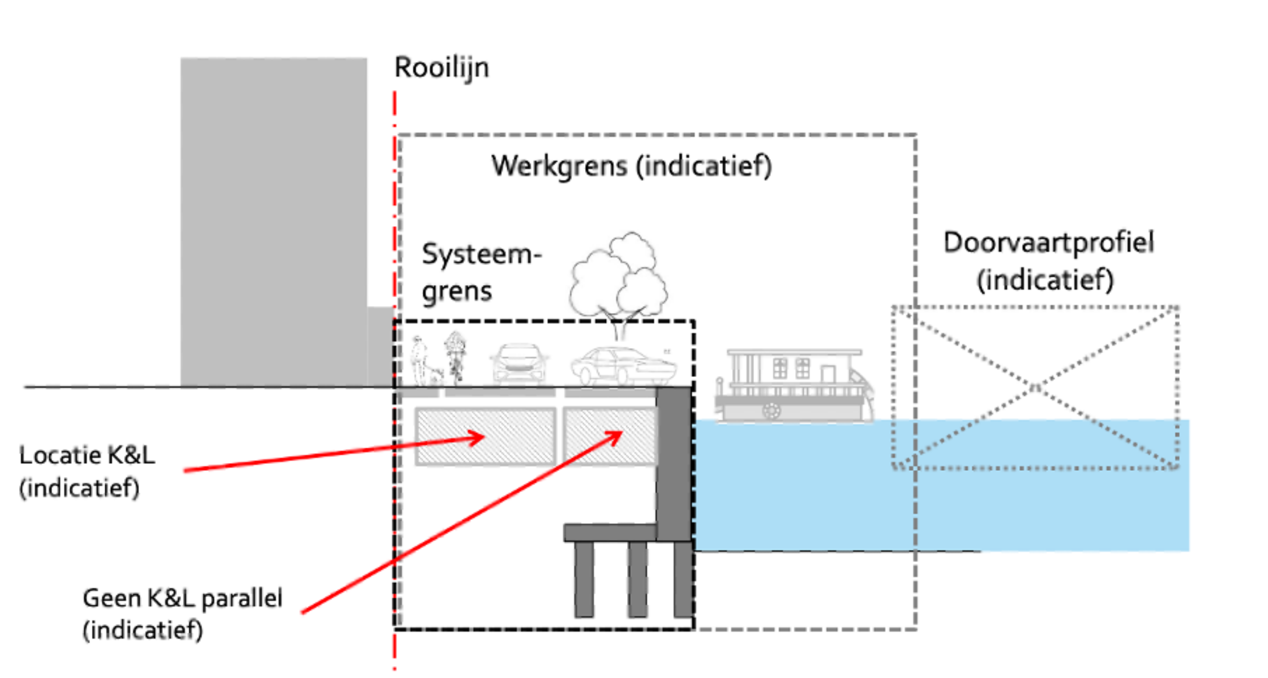

Figure 1.4: Working space (Presentation municipality of Amsterdam, 'Innovatiepartnerschap', 2019[6])

Although the municipality of Amsterdam is stimulating for innovation in design and construction, for now they stick to the traditional designs of an L-wall as the preferred design and a combi-wall as a back-up solution. The governing aspects in the design phase for renovation of quay walls are usually the construction phasing considerations, resulting from the location-specific requirements. These requirements can be of various natures, of which the following can be seen as the largest issues:

- ◇ As both traffic on land and on water can be affected, the accessibility of the location itself and the surrounding area should be carefully assessed.

- ◇ In many canals houseboats are positioned along the full length of the quay. These need to be moved during the construction works.
- ◇ The quay walls are located closely to existing buildings. To avoid harmful effects on surrounding structures, construction activities often need to be constructed vibration-free.
- ◇ Many large, old trees are positioned on the quay, sometimes right next to the quay wall. To make space for a construction pit the trees need to be removed and/or replanted, provoking a lot of resistance from residents.
- ◇ Many cables and pipelines are present in the ground and should be carefully managed during construction works.

Obviously, in a crowded and hectic city like Amsterdam, many other issues and stakeholders can play a role in the construction phasing considerations.

1.4 Reference projects

Due to the increasing awareness of the severity of the problems, the municipality of Amsterdam has started several tracks to deal with problems which are on the table. The program of bridges and quay walls carried by the municipality contains both short-term and long-term objectives. Short term objectives are characterized as temporary life-span extending solutions while long-term objectives focus on the replacement of the complete structure (or at least taking over its function). One of the tracks is the before-mentioned IPK, a result of the open tendering procedure for which sixteen combinations have applied. By the IPK, the municipality offered space for the market to find smart solutions in a very broad sense. For both the quay wall structure and the construction method many advantages can be gained. In the selection phase, in which the innovative potential was part of the selection criteria, this number was brought back to six parties. From the six tenders, the municipality of Amsterdam has awarded[7] three construction partnerships with a contract to develop their innovative concepts into executable solutions. All three concepts are focused on the reduction of nuisance, faster construction, and the reduction of costs. They have been allocated segments of quays with the intention to start a pilot in the autumn of 2022. Below, the concepts and their main characteristics are briefly presented.

1.4.1 G-Kracht

'Giken reaction-based system'

This concept mainly focuses on the limitation of nuisance. By using a pressing machine which moves over the previously installed tubes, no pontoon or additional equipment is required either on land-side or in the canal. A row of tubes with a diameter of about 700 millimeter is being installed at the same location of the existing quay wall. After a small part of the existing wall is being removed, the tubes are being drilled straight through the old timber foundation using a 'toothed' pile tip. During this process a guarantee of stability is claimed. Two out of six tubes are anchored into the second sand layer and four out of six tubes only reaches to the first sand layer. To make the structure fully sand-tight, at land-side smaller tubes are tubes are installed in-between the large tubes. To restore the appearance, a brick wall-supporting element should be placed in front or hanged on top of the tubes.

It provides a very slender solution, which does not affect the soil behind the quay wall. No additional measures are required for the preservation of trees and the replacement of cables/pipelines, except for cables and pipelines crossing below the quay. Similar to the combi-wall system the possible construction speed is very high (about 5 meter per day), but the robustness for the solution is relatively lower. As no relieving platform is used, only lateral support is being provided.

Figure 1.5: Concept G-Kracht[8]

1.4.2 Kade 2.020

'EZ-flow'

Similar to the previous concept, the EZ-flow method only requires a limited part of the quay for construction (about 2 meter for a relieving trench). Construction works are performed from the canal. Only a part of the existing wall is removed, after which a row of piles is being screwed through the remaining wall structure. Inside the canal a second row of piles is installed. Large prefab Z-shaped elements (up to 3 meter in height) are supplied over water, to be placed on the supporting plates of the 2 pile rows, covering the existing quay wall structure. For heights larger than 3 meter, an additional concrete beam is placed below the elements. In the prefab elements openings are present to make grouted connections to the piles, which makes the concept vulnerable for deviations during construction of the piles.

Also for this solution the level of nuisance is limited, although it does take a considerably longer construction time (about 5 meter per week). A slender solution is provided which does not affect the soil behind the quay wall, so no additional measures need to be taken for the trees, cables and pipelines. Without a relieving platform, the structure provides only lateral support. But as 2 pile rows are being used, the lateral effect can partly be decomposed in tensile and compressive action. The robustness of the structure can be classified as higher than the previous concept (or a combi-wall system), but lower than a traditional L-wall system.

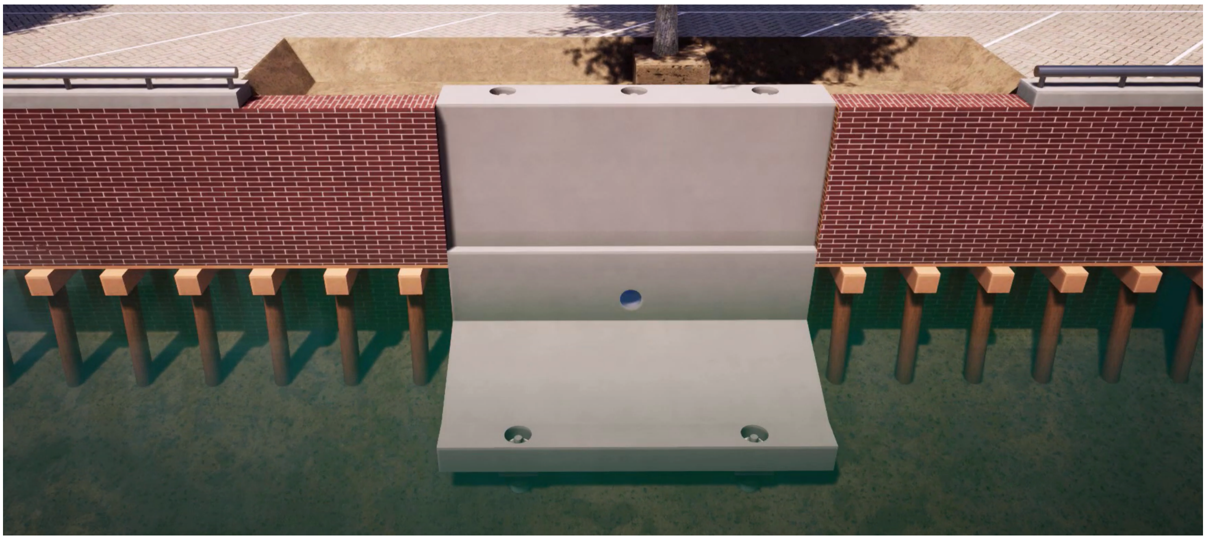

Figure 1.6: Concept Kade 2.020[9]

1.4.3 Koningsgracht

'SAVE method'

The concept Koningsgracht[10] focuses on optimizing the construction process for a traditional L-shaped design solution. To clear the space for constructing the relieving platform, a width of about the size of the parking lane need to be excavated. By using a trench box, the back-soil can be stabilized during excavation of the soil right behind the existing wall structure. In this way a construction pit can be avoided. The trench box is carried by a frame connected to the pontoons. After excavation up to the existing floor, steel casings are being screwed through the floor though which at a later stage the piles are being screwed. Prefab concrete elements with extruding reinforcement are placed on top of the floor, after which the new floor is cast using fiber-reinforced underwater concrete. During casting, the trench box is being lifted to avoid soil deformations. After placing temporary big bags and filling the space behind with sand, the existing wall is removed. Steel anchors are connected to the prefab element of the floor, in order to make a connection to the new wall. The prefab wall elements (with a masonry front cover and anchors at the bottom of the land-side) are placed on top of the pile row installed at the canal-side. The connection between the floor and the wall is made by casting the space in-between. To prevent seepage, prefab seepage elements are placed in front of the wall at the bottom of the canal. The space behind the wall is filled with sand and the big bags are removed.

To deal with the preservation of trees, Koningsgracht provides an alternative solution. In advance, measures are taken to improve the soil conditions below the root system of the trees. By removing the existing wall ground anchors are used to stabilize the soil and a temporary cover sheet is placed to protect the soil during construction. After the pile row is installed and the prefab wall elements are placed, permanent tie-backs (grouted anchors) are installed to the wall element.

Although this solution consists of more construction phases, but due to a parallel working process the promised construction speed is higher than the previous concept (7.5 meter per week). The space required for construction is a bit more, but the result provides a very robust solution completely taking over the function of the old structure. As a result of the large amount of elements and connections it could be assumed that the costs are relatively higher.

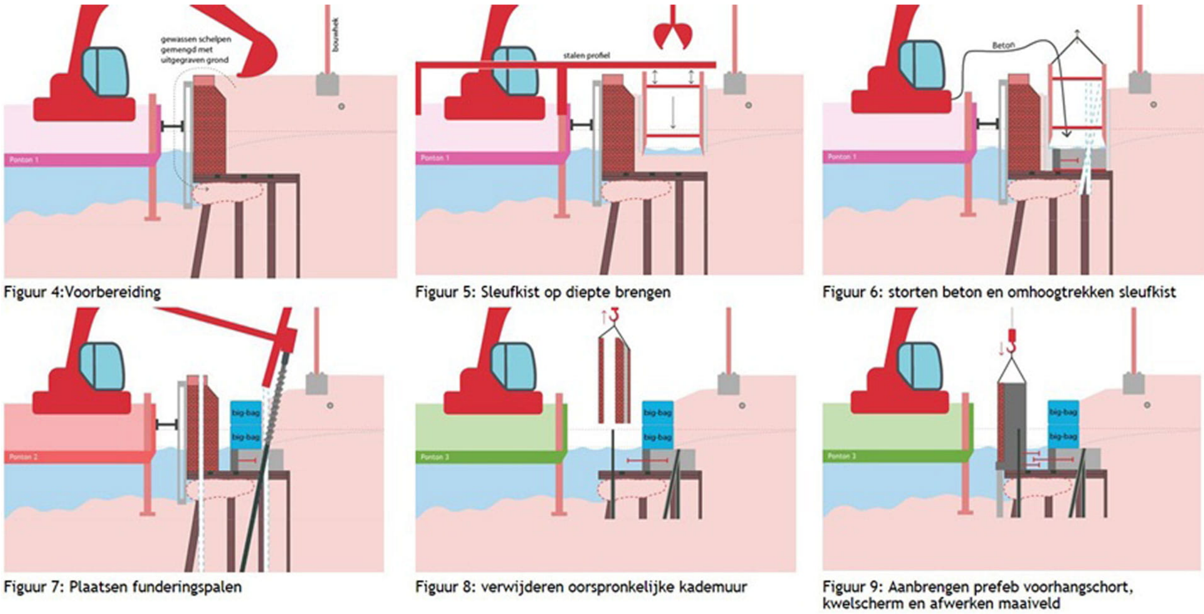

Figure 1.7: Concept Koningsgracht[11]

1.5 Other tracks within Program of Bridges and Quay walls

As mentioned before, within the PBK multiple tracks have been initiated in order to take action on different levels. Part of the program focuses on short-term action (direct renovation or reinforcement for the most critical quays) while other parts focus on gaining knowledge and stimulating innovation for the longer-term aim. Below, some of the awarded agreements within the program are being mentioned.

- SOK Ingenieursdiensten Programma Bruggen en Kademuren
  Next to the three above-mentioned innovative concepts, in July 2021 a collaborative agreement (SOK; Samenwerkingsovereenkomst) has been signed between the engineering office of the municipality of Amsterdam and three engineering firms. The collaboration will be responsible for the engineering services over the full width of the program 'Bridges and Quay Walls'. Next to facilitating innovative concepts, parts of their services are designing, (re)calculating, inspecting, and out-sourcing. Long-term aim in this collaboration is to speed up the activities by a factor 20. To accomplish this, more standardized and serial working methods are required, which are easily linked to parametric design systems.

- ◇ SOK Kademakers
  Whereas SOK Ingenieursdiensten PBK is a process-based assignment, the municipality of Amsterdam started an construction-based collaborative agreement called SOK Kademakers. In January 2021 three parties were awarded the job of repairing and renovating quay walls. Each party is allocated to one of the three clusters in which Amsterdam was divided. In a combination with contractors H. van Steenwijk and Van Gelder, Mobilis is selected for the job in one of the clusters. The purpose of the SOK is to tackle short-term issues. The most critical quays are dealt with within this six-year contract, with the aim of each party taking care of at least 300 meter of renovated quay per year.
- ◇ Lifespan extending innovations
  Apart from the innovative concepts for renovation of the quay walls, also an innovative course[12] has been started for six ideas in which the remaining lifespan of existing quay walls could be extended. Enhancing or reinforcement of the structures is only a temporary solution, but it reduces the pressure of having to renovate a large number of quays at the same time. Currently, when the renovation works for a quay has to be postponed, the temporary safety structure consists of sheet piles and the space in between filled by sand. With other lifespan extending ideas less radical solution (which are also less visible) could be applied.

1.6 Current status

In the latest update of the scope of the Program of Bridges and quay walls[13] the current view and expectations for the coming years (after 4 years of investigation) are being elaborated. An intermediate expectation of about 60 kilometers of quay wall required to be renovated within the next 30 years is being mentioned. However, due to rising costs of construction materials and additional requirements in limiting the level of nuisance, the financial pressure on the program has grown. As a result, the municipality have opted for a down-scaling of the intended renovation works. Instead of the earlier aimed renovation speed of at least 2 kilometers per year, in the current agreements only 1.2 kilometer of quay wall per year will be renovated in the next 2 years. In order to use the financial resources in the most efficient way, the municipality aims for preservation of the existing quays as much as possible, for example by preferring lifespan-extending measures over complete renovations.

1.7 Positioning of the research

The intention for this research is not to compete the above-mentioned partnerships with another innovative concept. The aim is focused at looking for a more sustainable solution in which the material use can be reduced, whereas the aims of the innovative concepts were mainly focusing on construction aspects like reducing nuisance and fast construction. However, as the construction aspects cannot be ignored, some lessons could be learned by comparing the concepts.

In all the three market concepts modular design characteristics can be found, especially in the Kade 2.020 for which a complete prefab module is placed in front of the existing structure. Another note that can be made is that for all three concepts the gain in time and cost is mostly related to working around the (remains of) the old structure. As the old foundation piles and slabs are left in the ground, and the new structure is constructed in front, above or even straight through the remains, construction works can be performed without construction pits. For the concept in which concrete is being cast, in situ underwater concrete is being applied.

From a sustainable point of view, it can also be stated that the level of sustainability in the concepts is not very high. Next to solely using the traditional materials concrete and steel, there is hardly any mentioning (as far as public information reaches) of sustainability aspects, except for the deployment of low-emission equipment.

It can be stated that the objectives for the innovative concepts have been governed by the contextual and construction limitations. By using these limitations as starting points 'the best' solution from a pure design perspective (without limiting the thoughts to the fastest and cheapest construction method with the least disturbance) could be overlooked. On the other side, by starting from a pure design point of view the result will most probably be impossible to become feasible for the context of the project. The approach taken is to start from somewhere in the middle. By starting with design the considerations and linking them to the practical and construction-related limitations that could affect them, the design freedom is kept as large as possible. At some point, assumptions have to be set in order to confine the direction of the research. By keeping the focus on the overall objective of minimizing the material use, the variant which performs best in satisfying the boundary conditions (highest feasibility) is selected to be worked out in the continuation of the research.

Chapter 2

Research outline

In this section the global outline of the project is being presented; a summation of the ins and out of the project which functions as a guideline for the proposed purpose. Along the process the project's main objective remains constant, but the scope is being updated in order to manage the size of the research, to react on the findings and to pin down the assumptions that have to be made.

2.1 Research objectives

Within this research the applicability of parametric design for the replacement of urban quay walls is investigated. The governing objective throughout this research is to minimize the use of materials. For that objective the loads and subsequently the form of the structure will be investigated in order to find a standardized solution for a certain range of application. With a standardized approach advantages can be taken within both the design and construction phase, in either time, cost and quality of the end product. The parametric design model allows for location-specific design alterations, both in the design stage and due to unforeseen circumstances during the construction stage.

The main question that guides the direction of this research is:

How can urban quay walls be parametrically designed in order to reduce the use of materials?

The optimization is aimed at both the product itself and the design process.

→ The main objective in this research is the minimization of material use. In line with that objective, the most effective (minimum material use with respect to the structural requirements) structural form is being proposed for the range of assumptions that are being made.

→ A parametric design model will be developed in which the standardized form can be modified for certain location-dependent input parameters (e.g. retaining height, available width, soil properties, top load)

The main question can be subdivided into the following subquestions:

1. How can the design for urban quay walls structurally be improved?

    a. What structural forms are possible?
    b. Which options are most efficient in carrying the loads?
    c. Which options are most feasible considering construction-related limitations?

2. Can a parametric design tool be developed which delivers an optimized structural solution for each location in the central zones of Amsterdam, taking into account the locational limitations (existing piles, cables, pipelines etc.)?

3. How does the proposed solution perform with respect to the traditional quay wall design?

2.2 Scope

Considering the context of a quay wall replacement the number of dependencies is very large. To make the research manageable a number of starting points, boundary conditions or assumptions need to be predefined. With each of them the accuracy of the result is affected. Whether the starting point, boundary condition or assumption is close to the best actual value defines the deviation from the optimal result. They need to be carefully defined and verified as the quality of the research directly depend on it. However, some of the starting points are qualitatively being adopted.

2.2.1 Starting points

Throughout the research many assumptions are being made. In the chapters Conceptual design and Parametric model the assumptions for each aspect are being mentioned. In the following list the most affecting starting points are presented.

- The project focuses only on urban quay walls. In the data analysis a selection is being made for specific types of quays, based on their geometric and functional properties. Only quays under management of the municipality of Amsterdam are being considered. According to the municipality of Amsterdam, 'dozens of kilometers' of quay walls are private properties. Although the outcome of this study can be applicable, the private owned quays are not part of the scope (in the data analysis).

- The focus will be on minimizing the material use, for which different variants are being assessed in order to choose the variant which promises to be the most efficient in resisting the loads with a minimum use of materials.

- Investigation of soil failure mechanisms will not be part of the scope. A simplified approach will be taken in order to simulate the interaction of the soil on the structure for the verification of the design.

- The research focuses on minimizing the use of the 'traditional' materials concrete, reinforcement and steel. More sustainable materials are not being considered.

- ◇ The optimization will not be done for the type of piles. Only one type of pile is being assumed: A steel casing screwing pile with a reinforced concrete core. Although this type of pile seems contradictory to the main objective, the parametric model can be disconnected from the type of pile that is being used.
- ◇ Due to the monumental value of Amsterdam's canals, the original external appearance should be preserved as much as possible. This means the visible part of the structure (the wall above the water level) should have a brick wall in front of it or a brick wall resembling cladding should be applied.
- ◇ To limit the risk of piping the difference in groundwater table should be minimum, either by using a semi-permeable structure or applying drainage systems. To be conservative, the soil behind the wall is assumed to be fully saturated while the water level in the canal is assumed considerably lower.
- ◇ Following the traditional construction phasing, a sheet pile at land-side is being used to be part of the construction pit and to retain the soil during construction works. Although this sheet pile is part of the permanent situation, only the loads that the structure takes from the sheet pile are taken into account. Since the required dimensions and structural verification are not being considered (and mainly depend on the construction phasing), the sheet pile is not contributing to the material use. For the comparison of different elevations of structure for the same retaining height, this would affect the results. As this comparison is not part of the study a constant height of the sheet pile can be assumed for a certain retaining height, independently of the varied parameters.

In the introduction of the problem many aspects that can play a role in the design considerations have been mentioned. By taking all possible aspects into account for this research, the complexity becomes too high and the possible solutions become extremely limited. In order to not become too restrained in options, some limiting aspects are excluded or only partly involved in the scope. Below the substantiation of the choices for in- and excluding the most important limiting aspects, which define the scope of the project and the applicability of the product, are being explained.

2.2.2 Trees

The trees of Amsterdam are an inseparable part of the characteristic street scene. The existing trees are on spaced about 10 to 15 meter. With an assumed load affected radius of about 2 meter for each tree, about 25 to 40% of the length of quay walls are affected by them, meaning the trees cannot be excluded from the scope. To not be a limiting aspect in the design considerations, the removal and replacement (if possible) of trees is being assumed. The loads, coming from their weight and the wind load, are taken into account. Although the trees do not influence the design considerations, the reservation of space for allowing trees to grow will be taken into account. To be able to grow, the trees require a certain free height and distance from the structure.

2.2.3 Piles

The effect of the piles is of great importance for meeting the requirements of the structure. The piles needs to resist both the vertical weight of the structure and loads above it, as well as the horizontal loads that arise from them. Apart from carrying the loads, their stiffness in interaction with the soil is needed to keep the displacements within limits. Next to the new piles, also the existing piles needs to be considered. As the extraction can cause harmful consequences they are left behind in the soil, which means they should be taken into account for new design. However, as this information becomes usually available only after the old structure is being excavated, the existing piles are being neglected. With the high rate of adaptability provided by the parametric model, this could be taken care of in a later stage.

2.2.4 Cables and pipelines

As the soil of Amsterdam, especially in the residential areas, is packed with cables and pipelines it is inevitable to consider them in the design considerations. But as their presence, locations and the directions of cables and pipelines are different for every quay, it is impossible to come up with a uniform design that does not clash with any of them. For that reason, the presence of cables and pipelines will not be dealt with for the extent of this research. In the case of site-specific locations of the cables and pipelines that could not be (re)moved, in a later stage adjustments could be made to the design by for example relocating the locations of the piles or by adjusting the elevation of the structure.

2.2.5 Traffic on land-side

The municipality of Amsterdam urges for solutions in which all the construction works can be done from the canal, so the roads on the quay do not have to be blocked for traffic during construction. This requirement of course follows from the current construction methods. If new methods can provide a time gain of construction works or limited use of space, the pressure on traffic disruptions becomes lower and might allow for temporary construction works from land-side. For this research works from land-side will not be left out of the scope, but the aim will be to perform as many construction works as possible from the canal to minimize the (time of) road closures.

2.2.6 Traffic in canal

Due to the large amount of traffic movements in the canals of Amsterdam, the aspect of nautical management can be a decisive factor in the project phasing and therefore the design consideration. As this also partially depends on the location and the time pressure due to current construction methods, it will not be taken into account for this project. What will be part of the scope is to aim for minimizing the required space in the canal.

2.2.7 Water level canal

As many structures in Amsterdam are still founded on wooden piles, the groundwater level and thus the water level in the canal needs to be carefully managed. The wooden piles are susceptible to rot, so large fluctuations of the groundwater level should be prevented.

For quay wall structures a high water level generally has a favorable effect, as the water provides a counter pressure from both the canal-side and at the bottom of the floor. The governing load situations depend on the way that the water level is taken into account. A conservative approach is to neglect the effect of water pressure, representing the situation for which the no counter pressure is acting on the structure. However, as the groundwater level is almost constant throughout the year, the presence of the counter pressure from the water in the canal is assumed to be guaranteed, at least for the load combination in which the top load is being considered.

2.2.8 Soil pressure factor

The soil pressure factor K is defined by the relationship between the horizontal and vertical effective soil pressure. The value for K depends on the type and state of the soil that is taking the load. Considering the soil on top of the structure is back-filling sand (for which more accurately soil characteristics can be assumed) and a structure which will deform in the same direction at the resultant lateral load, the soil pressure factor can be assumed within the range of 0.33 up to 0.5. Conservatively, the soil pressure factor is first assumed as 0.5. For the other soil layers (behind the structure) which exert a load on the structure, the soil pressure factor depends on the type of soil. In a simplified geotechnical approach this is taken into account.

2.3 Main parameters

The following parameters are taken as the main variable inputs for the model:

- Available width
- Retaining height
- Height of the wall
- Height of the floor
- Diameter of the piles
- Orientation of the piles

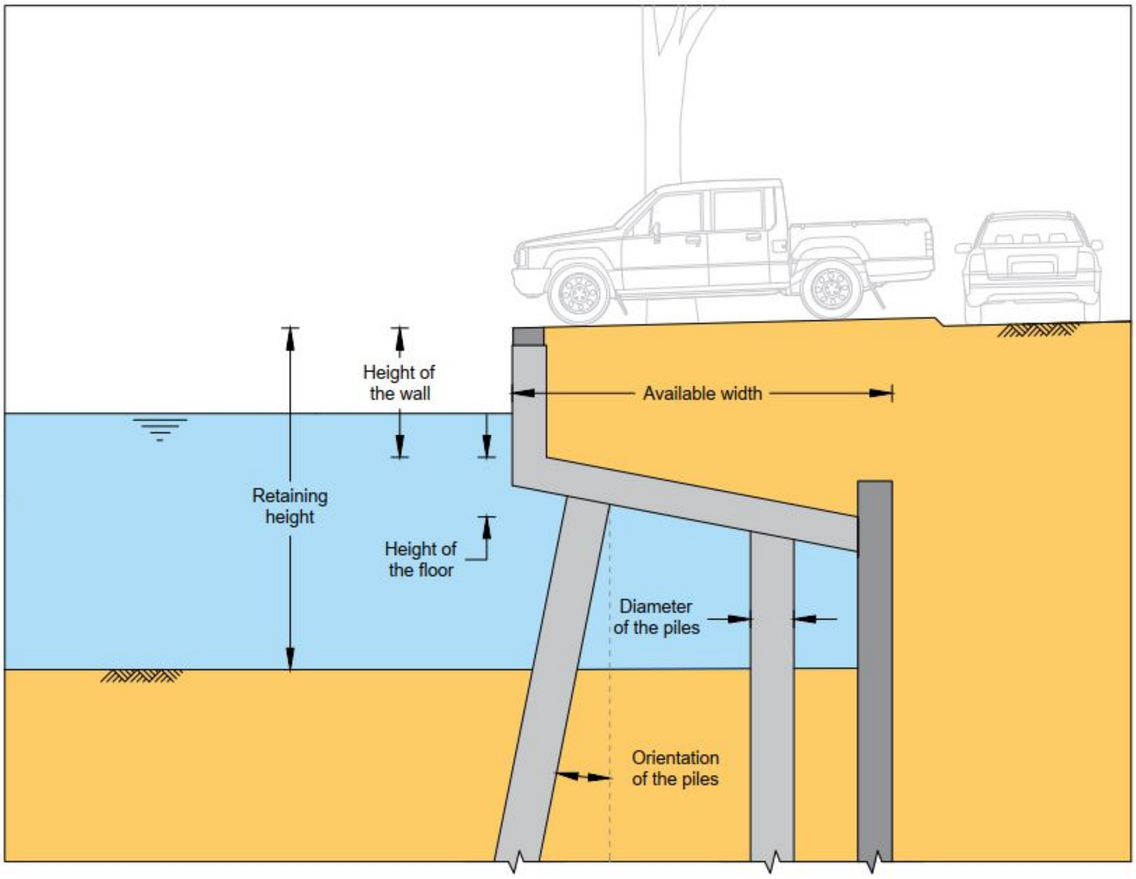

Figure 2.1: Main input parameters

2.4 Software and tools

The parametric model is created using the visual programming environment Grasshopper 3D, which runs within the computer-aided design (CAD) application Rhinoceros 3D (version 7.24). Grasshopper is a tool which is continuous highly in development. Instead of scripting textually, graphical components are used to create an interdependent sequence of instructions (algorithms). Each component consist of a set of algorithms, requiring one or more input values and returning one or more output values. Grasshopper is not an open source tool, meaning the component's underlying scripts can not be retrieved. A large community is built around the tool which provides a large variety of plugins for different applications. Plugins from which components have been used in the model are:

- Parametric FEM Toolbox (v1.4.1)
  Interface to RFEM 5, facilitating the export of model data and the import of calculation results.
- Metahopper (v1.2.4)
  Mainly used for its function to enable or disable objects, especially in the manual iteration tool. When it is being triggered it could activate other components/processes.

◇ Pufferfish (v3.0)
Providing additional functions mainly related to geometrical operations.

◇ TT Toolbox (v2.0.3)
Used mainly for its function to read and write Excel files, allowing for the export of results.

◇ Colibri (v2.0.0)
This plugin is mainly known for its iterative function. As a manual iterative tool is developed (allowing for more customized settings) the tool is only used for the export of CSV files, to be read by Design Explorer (By Thornton Tomasetti) for quick interpretation of the results.

◇ Optimization tool
For the first optimization loop, the evolutionary solver Galapagos (by default in Grasshopper) is used to allow for automated search of the optimum result.

The structural analysis is being performed with the 3D finite element analysis software RFEM (version 5.29). An interface between Grasshopper and RFEM is created (plugin Parametric FEM Toolbox) to export the geometrical input, cross-sections, materials and loads and import the structural analysis results back to the Grasshopper environment.

For comparing the effects on the structure as a result of local loads and uniformly distributed loads, the 3D finite element analysis software SCIA Engineer (version 21.1) has been used.

For finding the capacity of the top cross-section of the piles for different pile diameters and reinforcement configurations, the tool IDEA StatiCa - Reinforced Concrete Sections (version 22.1) is used. The capacity for ultimate limit state and crack with control for different inputs have been listed in order to consider a better value for the capacity in the Grasshopper script.

2.5 Workflow

After performing a preliminary research, including a data analysis and an investigation of the most affecting aspects, the aim is to come up with a standardized design model for which a large part of the quays within the scope can be renovated. To come up with an integral design solution, the starting point is to investigate the loads that need to be resisted. From the assessment of the loads, a standardized design which promises the highest structural efficiency is being chosen. By developing a parametric model, the optimum for the design parameters can be found for different geometrical inputs.

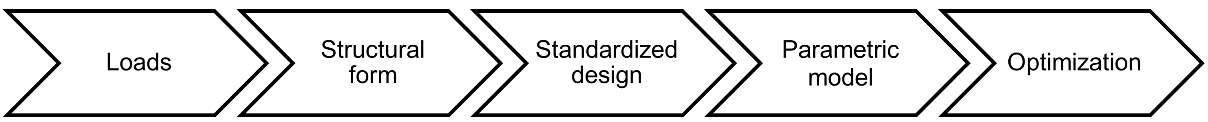

Figure 2.2: Flowchart research

Chapter 3

Preliminary research

The issue that will be encountered by carrying out a literature study, is that representative studies on quay walls are mainly limited to research within the Netherlands. Due to the unique soil conditions in the Netherlands a more complex solution is required in comparison to quay walls outside of the Netherlands, as deeper soil layers should be reached to find enough capacity to transfer the loads to. Even within the Netherlands a distinction needs to be made. Much research on 'quay walls' have been commissioned by the Port of Rotterdam, in search of deep quay walls usually with anchors to the mainland. Although some lessons learned could be applicable for both types of quay walls, the structure deals with different loads due to the much deeper waters in harbours and mooring and collision loads of cargo vessels. This is not representative for urban quay walls at the canals of Amsterdam, The Hague or Utrecht. Urban quay walls usually have lower retaining heights, but less design freedom due to adjacent buildings, cables, and pipelines.

3.1 Data analysis

The definition of the scope requires a number of assumptions, which will be presented in this section. An analysis of the total of quays under the management of the municipality of Amsterdam has been performed using Geo-Connect. Geo-Connect is an application developed within Mobilis, giving access to open data within the GIS environment. A data-set of the total of banks in the municipal district of Amsterdam is reduced by applying several filters.

Filter 1) Only data labeled as a quay wall is considered. Data labeled as sheet pile or bank protection/simple sheeting (beschoeiing) is taken out. The quays with a sheet pile solution are assumed to be functioning for its current use, so for anchored sheet piles an anchored solution is possible and for non-anchored sheet piles relative low loading conditions are present. The considered data consist only of existing quay walls, so new quays are not part of the scope.

Filter 2) Only data of quays in management of 'Verkeer en Openbare Ruimte' (the responsible department within the municipality of Amsterdam) is being considered. The quays which are in private management are left out of the scope, although the result could also be applicable for private quays.

Filter 3) In many districts the quays are not much of interest for the scope of this project. In suburban areas, generally simple solutions are sufficient as enough space is available to keep a distance between the location of the loads (mainly from the roads) and the quay walls. For the quays in harbour areas different types of design (mainly anchored solutions) can be used. The scope is focused on quays in the central and older districts of the city, for which relative limited space is available.

The scope that is formed after applying these three filters contains 114.55 kilometers of quay. The quays in the scope can be divided into three profiles, which are partly derived from the map of waterway profiles[14][15] from the municipality of Amsterdam (appendix A). Profile type 1 covers the canals with a minimum width of 10 to 13 meter, and a minimum water depth of 1.80 to 2.20 meter (corresponding to the municipality waterway profiles B-E). Profile type 2 is related to the larger access canals, which have a minimum width of 24 to 30 meter and a minimum water depth of 2.75 to 3.00 meter (municipality waterway profiles starting with A). Profile type 3 covers all the remaining quays within the scope which allow for harbour types of design solutions.

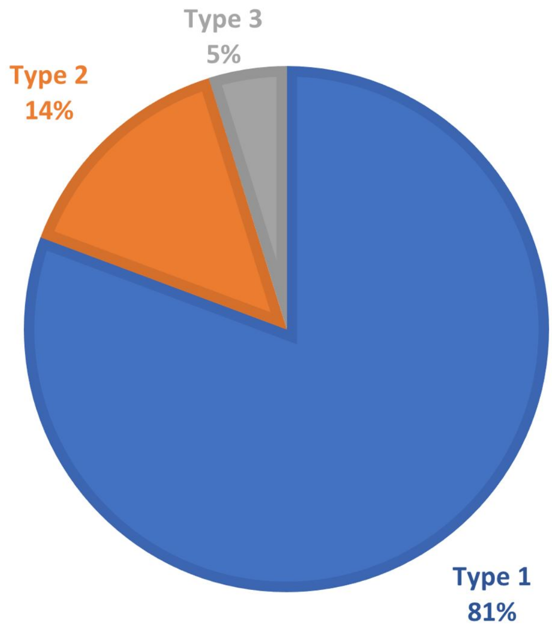

Figure 3.1: Profile type distribution

	Canal width (min)	Water depth (min)	Quay length
Type 1	10 - 13 m	1.80 - 2.20 m	92.44 km
Type 2	24 - 30 m	2.75 - 3.00 m	16.45 km
Type 3	Harbour type		5.66 km

Table 3.1: Profile types

The distribution of profile types within the scope is presented in figure 3.1. Obviously, within the central part of Amsterdam most of the canals fall within profile type 1. In these small canals only pleasure and excursion boats are allowed. Around the central zone some type 2 canals are present allowing bigger ships and commercial vessels to cross the city.

Another division of the scope is done based on the type of quay utilization. 5 utilization categories are defined which indicate the type of top loading on the quay and the available space behind the quay wall. The available space is assumed to be limited to the level of the sidewalk at maximum, so during construction the residents still can reach their houses. However, the aim remains to minimize the required space on land-side and keep as much works as possible being constructed from the canal. Out of profile types 1 and 2 the following utilization categories are defined, for which the distribution is presented in figure 3.7:

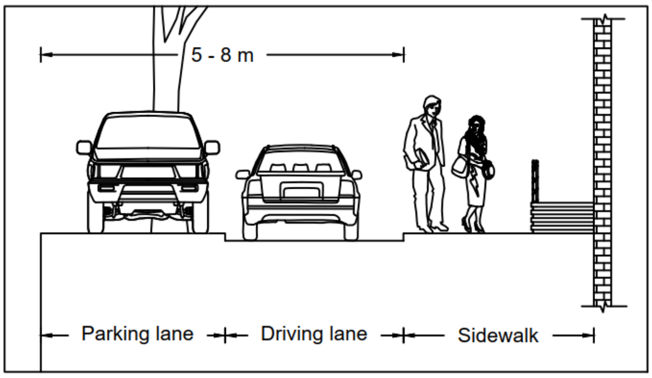

Figure 3.2: Utilization category A

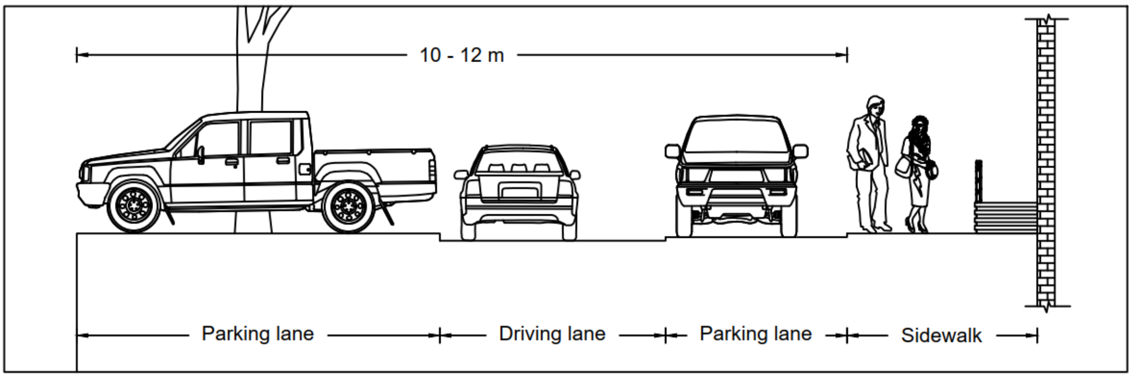

Figure 3.3: Utilization category B

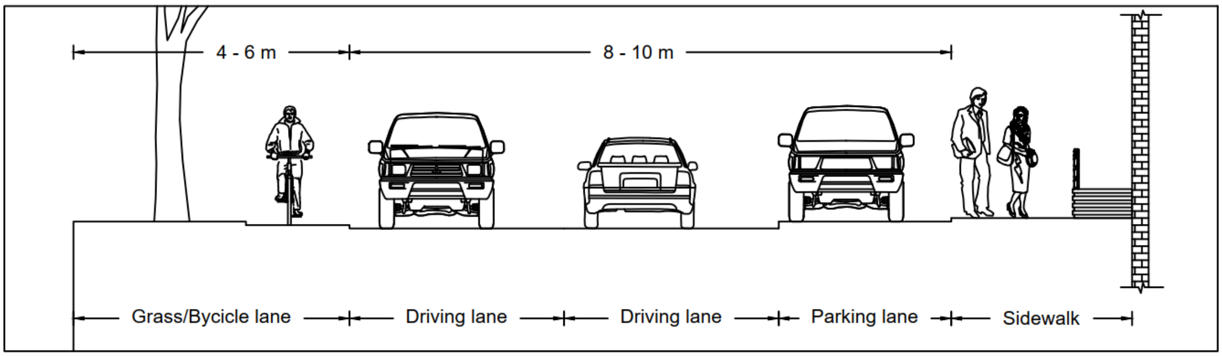

Figure 3.4: Utilization category C

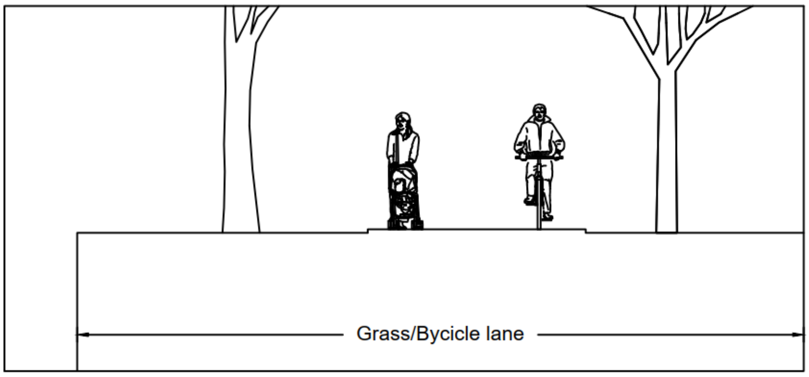

Figure 3.5: Utilization category D

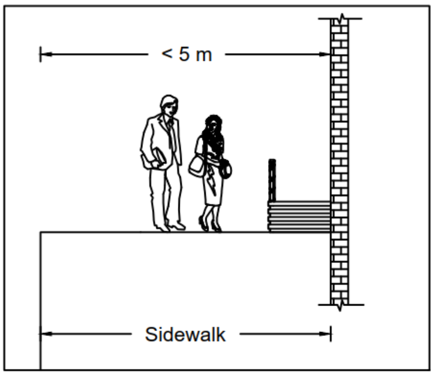

Figure 3.6: Utilization category E: Exceptional cases

For some of the quays which differ from the schematic side views of categories A to D, the category on the conservative side, which closest represent the actual situation is assumed. Category D is valid for locations where traffic loads are outside of the influence zone (about 8+ m). Category E reflects the quay where none of the other categories can be used as a representation of the actual situation. Examples are: Adjacent buildings within 5 meter, locations where other structures are above the quay, limiting the working height and affecting the phasing of construction.

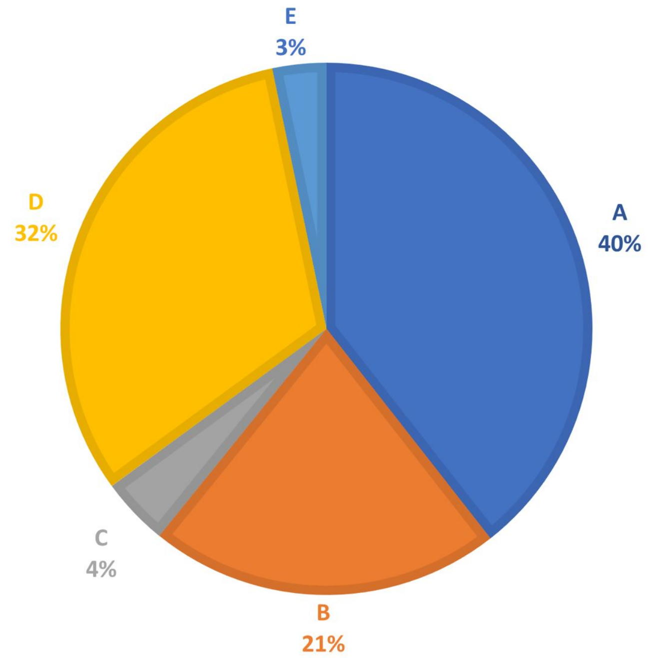

Figure 3.7: Utilization category distribution

The most critical and also the majority of the quays within the scope are in categories A and B. Solutions that could be thought of for these categories are presented in the next table. For category A and B, due to the high loads a stiff structure is required. The limited availability of space for category A demands for a slender solution, while for category B enough space could be available to apply a Mechanically Stabilized Earth wall. However, for this solution it is required that no trees, cables and pipelines are present in the ground within a certain width. The same counts for category C, but as enough space is available between buildings and the quay walls, cables and pipelines are usually not present in the zone next to the quay walls.

Cat.	A	B	C	D	E
1	L-wall/C-W	L-wall/C-W	C-W/SHP	SHP	C-W/SHP
2	L-wall	L-wall	C-W/SHP	SHP	C-W/SHP
Variants traditionally being used for each of the categories (C-W = Combi-wall ; SHP = Sheet pile or simple non-founded sheeting solutions)					

Table 3.2: Traditional design options for the defined categories

3.2 Construction aspects

3.2.1 Accessibility

The central part of Amsterdam knows a lot of critical transit routes, which might get overloaded by traffic in case of specific road closures. To ensure the renovation works do not affect the traffic flows due to traffic diversions, an extensive investigation is required. This counts for both road and nautical traffic.

However, the municipality of Amsterdam strongly adheres to implementing a solution in which the quay does not have to be closed. The aim is trying not to disturb the daily traffic flows, both on the quay and in the canal. The starting point is to construct (as good as) all of the works from the water-side. The working schedule is usually being adapted to the nautical management, making sure the traffic in the canal is not being affected too much. For some canals with a smaller width and a critical traffic flow, this could be governing in determining the type of structure.

As well for the renovation project itself, the accessibility of the construction sites needs to be considered. Due to restrictions for heavy and highly-polluting traffic, making sure the supply of materials and machinery reaches the location is a challenge in itself. Also here, the municipality strongly prefers the supply over water.

3.2.2 Existing piles

A lot of piles are already present in the soil. Most of the quay walls that need to be renovated are the historic quay wall structures, for which usually a lot of timber piles are installed. Extraction of these piles is a very risky action. The hole that it leaves behind causes soil movements which can cause damage to adjacent structures. The change of soil density also has a reducing effect on the soil stiffness for the new piles.

The same counts for other obstructions in the soil, like ancient foundations or boulders. When a pile is being driven and encounters an obstruction, the pile could be damaged. The pile could be replaced (probably resulting in additional piles) or the bearing capacity and the soil characteristics are reduced to compensate for potential damage. Inclining the piles brings an addition risk, as the probability of clashing with other obstacles is higher. Fortunately in the central district of Amsterdam the old existing structures are quite predictable, as they are mainly all made without inclined piles. (Van Leeuwenhoeklezing: "Kwetsbare kades"[16])

3.2.3 New piles

Driving of the new piles can also cause harmful effects on the environment. Apart from noise pollution, the vibrations in the soil that are induced by driving the piles can cause settlements to adjacent structures. For many locations in the central urban areas pile-driving vibrations are not permitted. The requirement for construction is to use vibration-free driving or screwing of the piles.

The diameter of the piles are usually taken as 457 mm or 508 mm. Larger diameters could be applied, but that also would require larger and heavier equipment. For heavier equipment larger loads are expected during the construction phase which could affect the construction phasing, for example when the required equipment is too heavy for a pontoon.

The most common solution for these type of structure, having to resist large bending moments due to lateral loads, are closed end thin-walled steel pipes. At the tip of the tube a steel screw point is installed, which displaces the soil while being brought to depth. By a grout injection installed in the tip a grout body can be constructed around the pile (usually only at the bottom part), increasing the effective diameter and with that the bearing capacity.

Pile inclination

Considering the inclination of the piles, the construction aspects play an important role. Inclined piles require not only a vertical, but also a horizontal working space. Due to safety measures, adjacent buildings and trees the workspace in the urban environment is usually limited. Cutting branches of the trees to make space for installing the piles is only permitted when it is proven that the measures would not affect the health of the tree. For using a pile system which could be applied in most of the locations, the steel casing of the piles are usually installed segment after segment. The construction time is highly affected by using segments, as welding them together at the construction site requires additional safety standards. However, due to the convenience of application, together with the convenience of supply, this method is usually being chosen. In addition, also the required equipment could have a large influence. As the works should be performed from a pontoon, it could be complex for standard equipment to reach certain locations and install the piles at certain inclinations. Due to the large size of the expected renovation works, it could be beneficial to develop customized equipment. At the moment, for the considered urban environment an inclination of 5:1 can be assumed as the limit value.

Corrosion

To take corrosional losses of the steel casing over the lifespan of 100 years into account, the structurally required wall thickness should be increased by a so called 'sacrificial thickness'. In practice piles with a steel tube are often considered to be structurally not active. The minimum required wall thickness for bringing the piles into the soil is considered, while the total of forces are resisted by the concrete core. The steel casing is assumed to be lost, as at the end of the life span a large part or even the complete wall thickness is considered to be corroded. Guidelines for corrosive action are given in CUR166[17] for sheet piles. A new guideline (NEN 6766[18]) is currently drafted for general corrosive action on structural steel elements, which is considered to result in stricter corrosive measures. For this research an indicative value based on CUR166 is considered. For a design life span of 100 years a sacrificial thickness of the steel casing of 4.3 millimeter is given for the case of 'strongly contaminated fresh water', a conservative classification for the part of the piles above the canal bed. For the parts below the bed of the canal, a sacrificial layer of 3.0 millimeter follows for the (also conservative) classification of 'contaminated soil'.

At the top of the pile a large bending moment needs to be transferred, requiring heavy reinforcement. At a lower depth, the bending moment is decreasing and the pile is mainly being axially loaded. By making sure that at a certain depth the bending moment (in combination with the axial loading) can be resisted by only the steel casing, the bottom part of the steel piles can be filled with another material, for example by sand. For this concrete- and reinforcement-saving option it should be guaranteed that the forces are sufficiently transferred from the concrete filling to the steel casing. This can be achieved by welding shearing rings inside the piles. However, taking into account the sacrificial thickness layer a large volume of steel is being used. Whether that would be a better option in terms of total material use should be investigated.

3.2.4 Trees

In Amsterdam many trees have a monumental or (natural/cultural) historical value. Next to these special values the trees have an urban value. Especially the elms are very beloved. With more than 31000 elms Amsterdam announces itself as the 'elm capital of the world'. Particularly the older trees contribute to the capturing of CO_2, provide shading, retain water and contribute to the health and well-being of residents. By cutting down the large trees and planting new smaller trees, these benefits are reduced and the street scene is altered.

Like for private parties, the municipality of Amsterdam is bonded to the urban tree regulation ('Bomenverordening'). Research is being done on the health of these trees and the hindrance they cause for renovation projects. At the moment, the trees located at the quays which are planned to be renovated are being classified by their value (for example a monumental status), health condition and ability to be re-planted. According to the current construction methods, replanting is assumed to be only possible for trees with a trunk diameter smaller than 30 centimeter, which are estimated to be circa 10 to 15% of the trees within the scope. About 75 to 80% of the trees cannot be replaced and need to be cut down and 5 to 10% of the trees possess a monumental value.(*Actieplan bruggen en kademuren 2023-2026*[13])

If after assessment of the trees it is concluded that it could not be guaranteed that the construction works (either taking away and replanting of the tree or construction works in the vicinity of tree) would affect the health of the tree, the construction works could not take place. For a number of cases this has been the reason for postponement of the renovation works. Hoping for prospective innovative solutions in which these trees can be preserved during renovation, temporary supporting structures can be applied.

For a renovated quay wall structure, sufficient space should be available for the tree to grow (both above and below the ground). How much space is required depends largely on the type of tree and the age. As a rough guideline, new trees are generally being planted at about 2.5 meter from the wall of the structure.

3.2.5 Houseboats

About 2800 of houseboats are present along the more than 200 kilometers of quay that is within the scope of PBK. Due to renovation works the houseboats need to be (temporary) replaced. Towing away of them entails a logistic challenge. Specific 'transit locations' are reserved for temporary accommodation of houseboats, but with up-scaling of renovation works the available space for transit locations become insufficient. Of course, a lot of resistance is being encountered from the residents. Having their legal rights, making the quays available for renovation or reinforcement works can become a complex task in itself.

In addition, some of the houseboats are too big to be passing under a bridge. Special 'sink-boats' are required to lower these boats deeper into the canal so they become able to pass below the bridge. In a recent case, the Department of Waterways and Public Works even managed to lower the water level in the canal with 7 centimeters to allow a houseboat to pass below a bridge. For this operation about 2.7 million cubic meter of water has been temporarily discharged into the sea at low tide. As the operation only took 2.5 hours the effect on the groundwater table and the foundations within the affected area are negligible.

3.2.6 Cables and pipelines

The presence of cables and pipelines are essential for the functioning and the quality of life of the society. Over time a lot of utilities have been granted permission to place a network of cables or pipelines in the ground. The ground in Amsterdam is packed with cables and pipelines, which can be running almost at every place and in all directions. Preferably the cables and pipelines should be bundled in a duct running parallel to the quay wall and close to the adjacent buildings, which are the destination for most of the cables and pipelines. Bundling all the cables also makes them more easily accessible, avoiding the road to be closed every time maintenance is required. In addition, the complexity and risks for future works are diminished. However, the replacement of them is a very complex task. Only the mapping of the locations of cables and pipelines and finding their owners already can be a tough job. Subsequently, it involves a large administrative workload which can take a considerable amount of time. Since the law describing the ownership rights of cables and pipelines (in municipal ground) has been adjusted in 2007, the network managers should all be on board with a replacement. And for some of the cables and pipelines, their location requires additional works to be able to reach them.

To avoid the complexity, in practice usually only the cables and pipelines that are directly hindering the works are (temporary or permanently) being replaced. For the case of cables or pipelines below the structure (requiring additional works) or which are not allowed to be temporarily blocked, the design is adapted to safely work around them.

3.3 Soil-structure interaction

3.3.1 Soil resistance

The function of the structure is to transfer the loads towards the soil. Depending on the type of soil, different solutions could be possible. For a soil profile of strong layers the retaining structure could be reduced to a simple gravity wall. By giving the structure enough weight, the gravitational force and the resulting frictional force are enough to prevent the structure from sliding or tumbling over. Both vertical and horizontal loads can directly be transferred to the subsoil.

For a soil profile with softer layers, which is the case for Amsterdam, the vertical and sliding/rotational resistance cannot be found in a direct way. Additional foundation piles are required to be able to transfer the loads to the subsoil. Vertical loads are axially being resisted by (vertical) piles and assumed to be only resisted by the bearing capacity of the tip of the pile. In reality, the resistance also contains a contribution of friction along the shaft of the piles. In this research this is not taken into account.

The resistance of the horizontal loads acting on the piles is delivered by a combined effect of the bending stiffness of the piles and the lateral stiffness of the soil layers. An interaction between both stiffnesses exists, meaning that when the bending stiffness of the piles is low more deformations occur resulting in more lateral support reactions from the soil. The other way around, when stronger soil layers are present the resistance delivered by the soil will limit the deformation of the pile and with that reduces the bending moments in the pile.

3.3.2 Soil loads

For the loads on the structure that are being exerted by the soil being under pressure, a bit more of uncertainty needs to be taken into account. Given the resultant horizontal load that is guaranteed to press the structure in the direction of the canal, the soil at land-side of the structure tends to be in an active soil state. In a conservative approach, the structure does not deform and a neutral soil state can be assumed. The actual situation is somewhere in-between the active and the neutral soil state. Depending on the type of soil, the range for the soil pressure factor can be defined. For the back-fill material, which is generally taken as sand, the soil pressure factor will be approximately in the range of 0.33 up to 0.5. With these values the range of lateral effect as a result of the top load and the self-weight of the back-fill material can be determined.

The actual value depends on the flexibility of the structure. If the retaining structure moves along with the soil, a low value for K can be expected and an active soil pressure is approached. For the case of a very robust structure, which has almost no deformations, the value for a neutral soil pressure is being approached.

In practice usually a conservative value between 0.5 and 1.0 is being assumed, as an interaction of active and passive soil states are finding an equilibrium. For the case of weaker sandy soils values closer to 1.0 and for stronger sandy soils values closer to 0.5 are being assumed. For weaker soils the K value becomes higher, but at the same time the self-weight is lower. However, as the expected direction of the lateral soil pressure on the structure and the expected direction of deformations are well predictable and a high rate of uniformity in the back-fill material can be guaranteed, a soil pressure factor of 0.5 is assumed to be sufficiently conservative.

3.4 Loads

By having discussed the possible functions a quay wall has to fulfill, the loads that emerge from it can be considered. In order to have an adequate understanding of the structural behaviour that is required, the effects of each type of load are being elaborated. Although other type of loads could be thought of, the most relevant types of loads are considered to be:

- Loads coming from the water level
- Loads on top of the quay
  - A uniform load over the full width of the quay
  - The separate contributions following from the traffic load
- Subsoil loads coming from the sheet pile
- Loads coming from the trees
- Loads due to ice-formation
- Loads coming from the mooring of vessels

By getting a feeling of the magnitudes for each type of load, the most significant loads can be pointed out. In order to find a simplified but accurate approach for the to be considered loads, comparisons are being made.

3.4.1 Differences in water level

Both the groundwater level and the water level of the canal fluctuate over time and are interdependent. By integrating drainage systems the structure the permeability could be improved and (together with an undesired flow of water through the soil below the structure) determines the speed at which equilibrium is found. During heavy rainfall, the water level at land-side can increase relative fast. During this period, the groundwater level can be considerably higher than the water level in the canal. With a delay, the water level of the canal also increases until they are equal again.

Due to differences in water level, additional loads could be expected on the structure. Temporarily the top part of the soil can become saturated. On the canal-side, the counter-pressure on the wall becomes higher or lower. As the water level in the canal has a favorable load effect on the wall, the least favorable level should be considered. However, the maximum water level could also be governing for some results, for example the extreme reaction forces of the piles.

3.4.2 Top load

Several loads on top of the quay and in the soil can be thought of. Top loads consist of permanent loads, coming for example from the roads/pavements, trees, benches etc. Next to that, the space between the road and the relieving platform is often used for cables and pipelines. Also other functions can be integrated into the quay, resulting in additional loads, for example garbage containers. In combining all loads due to the possible/expected functions, a maximum value for the loads should be assumed.

In the Eurocode for geotechnical design of structures[19] a variable 'field' load on top of the quay of minimum 10 kN/m^2 is prescribed. This load should be applied over the full length of the structure, and a width of at least 20 meter. For harbor quay walls a load of minimum 20 kN/m^2 should be applied. Whether the 10 kN/m^2 is a representative value for quays in Amsterdam could be discussed. Most of the quay do not have a 'field' width of 20 meter, and a lot of quays have parking spaces close to the canal. The given value for the field load is a representative value. This load is assumed to cover the summation of separate loads. By summing the effect of separate loads in the governing load combinations (predominantly traffic loads), a comparison can be made with the overall field load.

3.4.3 Traffic load

As a result of traffic on top of the quay both static and dynamic loads can be expected, acting in both vertical and horizontal direction. To make sure the quay has enough bearing capacity for the coming 100 years, the possibility of future diversification of loads should be considered without unduly over-designing the quay walls. Although nowadays many quays in Amsterdam are restricted for heavy traffic, and the municipality is aiming to keep heavy (and usually highly polluting) traffic out of the central zone of Amsterdam, at least the possibility of re-entry of heavy traffic should be taken into account. At the other hand, the most direct way to reduce the material use and simultaneously the costs of the design is to reduce the loads. So to conservatively assume extreme load conditions is contradictory to the aim of optimizing the design.

The guidelines for traffic loads up to now are only available in the Eurocode 1 (NEN-EN 1991-2)[20] for bridge design. The model is derived from the former Dutch guideline for steel bridges (VOSB[21]). Over time, the magnitude of the loads, the composition of wheel loads and the dynamic and fatigue loading effect have been incorporated in the load model. As a result, the specified loads of load model 1 contains of a dynamic load factor of about 1.4 to 1.7. In comparison with the fatigue load models, this factor is not hidden within the specified loads.

As no other traffic load model is available, this load model (which is applicable for an ongoing road network) is being applied for every situation with road traffic. According to this traffic load model specified in the Eurocode 1, two axle loads of each 300 kN (corresponding to a 60-ton truck) should be combined with a UDL of 9 kN/m^2 for the most unfavorable lane. For the second lane the axle loads are reduced to each 200 kN and for the third land only 100 kN, both combined with a UDL of 2.5 kN/m^2. The axle load consists of two equal wheel loads, each working on a contact area of 0.4 meter by 0.4 meter. The center-to-center distance between the wheels is 2 meter and the distance between the axes is 1.20 meter.

To adjust the load model for locations with a lower traffic intensity, a reduction factor can be applied. Quays in the central zone of Amsterdam generally only have a limited amount of heavy truck passages per year. This allows for a reduction factor α_q, which follows from the estimated amount of vehicles above 100 kN per year (e.g. moving trucks, garbage trucks, supply trucks, fire trucks). This number should be multiplied by a factor

2 to account for the serviceability lifespan of 100 years. For quays in utilization category A and B (residential areas) an expected maximum number of 1000 heavy truck passages could be argued. This allows for a reduction factor α_q of 0.70. Utilization category C (city roads) can have a larger amount of passing trucks per year. An estimated maximum of 10.000 passages could be assumed, corresponding to a reduction factor α_q of 0.80. To be on the safe site and being able to link number of heavy vehicle passages to each category, for category A and B a reduction factor of 0.70 and for category C a reduction factor of 0.90 is assumed.

Aantal vrachtwagens per jaar per rijstrook voor zwaar verkeer N_{obs} [a]	α_{Q1} en α_{q1}				α_{qr}
	Lengte van de overspanning of invloedslengte (L)				
	20 m	50 m	100 m	≥ 200 m	
≥ 2 000 000	1,0	1,0	1,0	1,0	
200 000	0,97	0,97	0,95	0,95	0,90
20 000	0,95	0,94	0,89	0,88	0,80
2 000	0,91	0,91	0,82	0,81	0,70
200	0,88	0,87	0,75	0,74	0,60
[a] Tussengelegen waarden mogen worden geïnterpoleerd.					

Figure 3.8: Load reduction factors α (Table NB.1 from the National annex of Eurocode 1-2[22])

Load model 2 focuses on the local effects. Only a single axle load of 400 kN needs to be applied at an arbitrary location in the traffic lane. It is not combined with a UDL. The axle load may be reduced by a factor β_q which may be assumed to be equal to α_q. As for quay walls the assumption can be made that enough spreading of concentrated loads is available, local effects of load model 2 can be neglected and are not taken into account in the load combinations.

Load model 3 describes a situation in which special vehicles with exceptional loads are present. It can be assumed that this situation only could occur in utilization category C, but as it requires a situation-specific type of load it will not be considered.

Load model 4 covers the situation in which a crowd of people is present on the quay. As UDL of 5 kN/m^2 is applied over the total width. Although it will not be governing over the field load of 10 or 20 kN/m^2 from the Eurocode for geotechnical design of structures, the crowd load could be used in combination with other loads.

For the parking lanes a UDL of 5 kN/m^2 is assumed according to the Eurocode 1 (NEN-EN 1991-1 NB). This applies for vehicles in the category 25 kN up to 160 kN. Next to the UDL, a concentrated load of 40 kN should be taken into account at a random location (representing a situation in which a vehicle is being jacked). As the jacking load has the same source as the UDL it can be neglected in a global perspective, but locally it could result in larger effects.

Klasse van belaste oppervlakte [b]	q_k kN/m²	Q_k kN
F (lichte voertuigen lichter dan 25 kN)	2	10
G (middelzware voertuigen 25 kN t.m. 160 kN)	5	40
G (voertuigen zwaarder dan 160 kN)	G_v / A_v [a]	2 × maximale krikbelasting

[a] G_v is het gewicht van het voertuig, in kN, en A_v is de oppervlakte ingenomen door het voertuig, in m².

[b] Voor banen en hellingen van parkeergarages moet een extra horizontale remkracht op het wegoppervlak zijn toegepast. Deze belasting moet zijn beschouwd als een statische belasting. Voor voertuigen met een gewicht tot 25 kN moet een horizontale kracht van 10 kN zijn gebruikt. Voor voertuigen met een gewicht groter dan 25 kN moet de horizontale kracht per baan zijn bepaald met $Q_k = m \times a$, in N, waarbij m is de massa van het volledig beladen voertuig, in kg, en a is de vertraging door de remvertraging, in m/s².

Figure 3.9: Load values for parking areas (Table NB.3 from the National annex of Eurocode 1-1[23])

In its own requirements, the municipality of Amsterdam is used to prescribe a load model for only heavy vehicles of emergency services, with a maximum load for fire engine trucks. It consists of 2 axle loads of 100 kN (spaced 1.3 meter) and an axle load of 80 kN (spaced 4.2 meter). Compared to the Eurocode tandem set of 2 times a 300 kN axle load, there is a substantial difference. The municipality follows its own strategy, in which heavy traffic is not allowed in the central zone and an exceptional 28-ton fire truck can be considered as the maximum traffic load. In line with this, it could even be argued if some quays need to be designed for traffic load at all, as for some areas a car-free zone is being discussed. But taking into account that these traffic desires can change over time, Eurocode traffic load model 1 could be considered.

A horizontal load follows from the break force. Breaking of multiple vehicles can become of considerable effect, but for quay walls the effect is in mainly in perpendicular direction and can be neglected. A break force in the direction of the quay wall can occur due to the parking zone close to the wall, but as it is almost impossible that multiple cars are parking at the same time at a considerable speed the effect of break forces on the wall will be neglected.

Some of the quays hold tram tracks. Whether the load of a crowded tram (usually at a certain distance from the quay wall) is governing over the heavy traffic loads at the same location, should be investigated.

3.4.4 Sheet pile load

During the construction phase a sheet pile is used to stabilizes the soil at land-side. To end up with a lighter sheet pile usually a horizontal strut is being used during the construction phase. After the floor is being constructed the strutting force needs to be taken over by the structure. To prevent deformations in the soil behind the sheet pile when this force is taken over, the floor needs to be constructed tightly to the sheet pile. Along with the design verification of the construction phasing, the sheet pile needs to be designed for different load stages. The interaction force that is being transferred in the final situation is dependent on these design choices. In addition, the sheet pile could also be functioning as a seepage screen, which could be governing for the depth of the sheet piles. To make sure the sheet pile is properly designed for all phases an extensive calculation needs to be performed. For the scale of this research a simplified approach is being taken in which only the final situation (after construction) is being considered in order to find the magnitude of the interaction force on the floor. Although this method affects the accuracy of the results (in minimizing the material use), this method has been chosen as a relatively simple and integral (applicable for a range of retaining heights) approach. By performing an extensive geotechnical analysis (with for example Plaxis), more accurate loads can be found resulting in more accurate results in minimizing the material use.

An indicative load calculation can be done by using Blum's method (presented in *Soil Mechanics* by A. Verruijt[24]). Based on only active and passive soil pressures the minimum depth of the sheet pile can be found for which rotation around the anchor point (at floor level) is in equilibrium. Consequently, the magnitude of the horizontal force can be calculated through the equilibrium of horizontal forces.

Surely the horizontal interaction force on the floor is highly dependent on the soil profile and the retaining height. The resistance needs to be found in passive soil pressure at the canal-side. If this layer consist of soft soil material more height needs to be seized by the sheet pile to find enough counter-pressure.

Next to the types of soil, also the depth of the sheet pile affects the interaction force. The calculated minimum depth indicates the depth for which equilibrium can just be found. To lower the risk of failure usually a larger depth is being used. By increasing the depth also the interaction force is reduced. As the bottom of the sheet pile tends to move in opposite direction (to land-side), passive soil pressure can be assumed at that zone. The distribution of soil pressures will find a new equilibrium for which passive soil state is not required anymore over the full height of the sheet pile. According to Blum's method the passive state counter-pressure at the bottom part allows for the sheet pile being assumed as clamped in this region. As the bending moments in the sheet pile are more evenly distributed the thickness of the sheet pile could be reduced compared to the minimum driving depth.

Finally, also the bending stiffness of the sheet pile has an effect on the interaction force. For very stiff sheet piles the lateral deformation of the floor could be more than the deformation of the sheet pile, which means there is no interaction force and the soil is resisted only by the bending stiffness of the clamped sheet pile. However, this would be uneconomical and contact between the floor and sheet pile is required to guarantee a soil-tight connection.

Horizontal interaction force

For this indicative calculation a soil profile for a reference project in Amsterdam is being used, which can be considered as an average soil composition for the central part of Amsterdam (in appendix B sections of the soil profile from the BRO GeoTOP database[25] are shown). Based on this soil profile, an empirical formula is being linked to the magnitude of the horizontal interaction force, depending on the height of the top structure (wall and floor) and the height between the floor and the canal bed (gap height). As stated before, many aspects can have an influence on the force distribution. This approach is a very practical consideration of the situation, for which the force is only being dependent on the two mentioned height parameters.

First the interaction force based on only the soil load is being calculated. Starting point for this approach is that there is no water pressure difference between both sides of the sheet pile. In this way only the effective lateral soil pressure is considered to act on the sheet pile. For different inputs of the height for the top structure and the gap height, the minimum driving depth is being calculated for which the rotation of the sheet pile around the floor level is in equilibrium. For this depth, the corresponding interaction force (to make force equilibrium) is being calculated. By listing the results for different input heights, the dependency of the interaction force on these heights is being estimated. From this dependency a linear formula is being derived which is being used in the parametric model. Within the considered range (height structure = 0 - 3 m, gap height = 0 - 1.8 m) the formula presents a conservative value for the interaction force, as is shown in figures 3.10(a) and 3.10(b). For a gap height of 1.8 meter, the formula of red dotted line is shown.

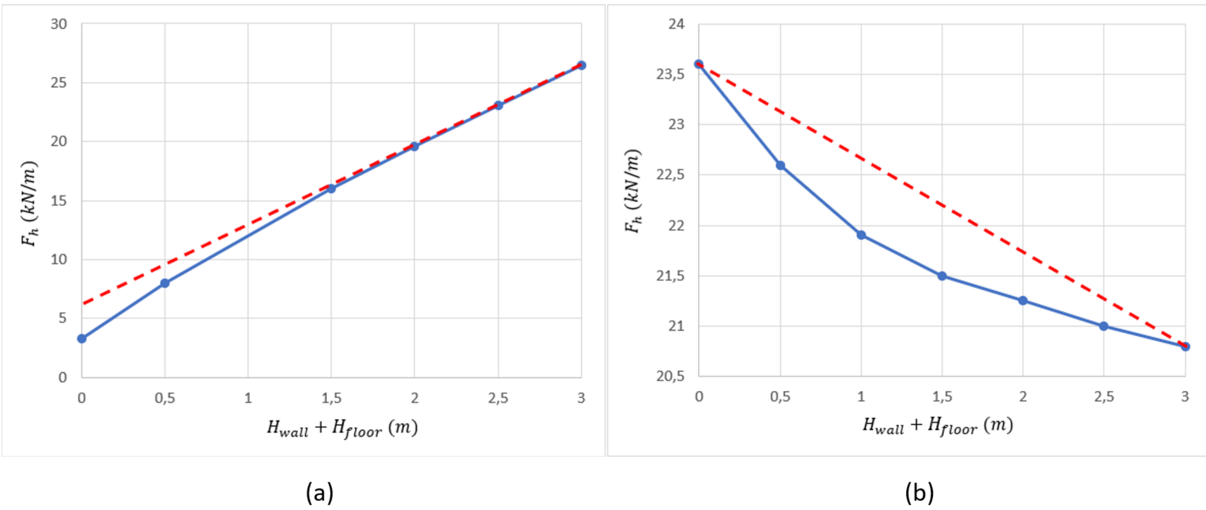

Figure 3.10: Derivation of the formulas from the interaction forces resulting from the soil load (a) and the top load (b).

The derived formula for the soil load is given by:

$$F_{h,s} = 6.1 + 6.8 \cdot (H_{wall} + H_{floor})$$

By making the formula also linear dependent on the gap height, the constant value of 6.1 is divided by the gap height of 1.8. The horizontal interaction force as a result of the soil load becomes:

$$F_{h,s} = 3.39 \cdot H_{gap} + 6.8 \cdot (H_{wall} + H_{floor})$$

In the same way, a formula is being derived for the effect of the top load. The effect of both the soil load and the top load is being listed, and the results are reduced by the results found for only the soil load. For a gap height of 1.8 meter, the formula of red dotted line is:

$$F_{h,tl} = 23.6 + 6.8 \cdot (H_{wall} + H_{floor})$$

In this case, for a gap height and a height of the top structure both being zero, still a horizontal load of 11.4 kN is found. By reducing the constant value of 23.6 by this value, and again divide it by the gap height, the horizontal interaction force as a result of the top load of 20 kN/m^2 becomes:

$$F_{h,tl} = 11.4 + 6.78 \cdot H_{gap} + 0.93 \cdot (H_{wall} + H_{floor})$$

Nevertheless, the approach of only considering the minimum depth in calculating the interaction force is also on the conservative side. As mentioned above, the driving depth is usually taken a bit larger than the minimum depth, which has a decreasing effect on the interaction force. By choosing a larger driving depth than is being required, the material use of the total structure could become less due to both the reduction of bending moments in the sheet pile and a reduction of the pile head bending moments as a result of the horizontal interaction force.

3.4.5 Tree loads

The tree loads on the quay consist of both permanent self-weight and variable wind loads. A guide for this assessment is given in the SBRCUR publication[26] for urban quay walls and the additional erratum[27] for tree loads.

The self-weight depends on the type and age of the tree. SBRCUR provides indicative values for 3 types of trees. For an oak tree with a height of 15 meter, a self-weight of 20 kN may be assumed. For a linden tree of 15 meter and a poplar tree of 20 meter the assumed self-weight is 15 kN. The weight can be equally distributed over the load-bearing part of the root ball (wortelkluit). In SBRCUR a graph is provided which gives the relation between the radius of the trunk and the radius of the root ball R_w. For a trunk radius of 20 cm, the value for R_w is between 0.85 and 2.25 m. For this calculation R_w is assumed to be 1.5 meter.

	Oak tree	Poplar tree
Height	15 m	20 m
Height trunk	5 m	5 m
Trunk diameter	0.4 m	0.4 m
Crown diameter	10 m	10 m
Crown height	10 m	15 m
Self-weight	20 kN	15 kN
Permeability factor c_w	0.25	0.20

Table 3.3: Starting points for wind load calculations on most common tree types (from SBRCUR[26])

The tree itself can be seen as a clamped column, on which the horizontal wind load causes a horizontal reaction force and a bending moment. The wind load is mainly a result of the dimensions of the crown and its transparency. It causes a horizontal load and a local bending moment, which are introduced to the soil through the roots. Tensile loads on one side and compression loads on the other side cause a rotational outbreak (sliding failure) of the lump of roots. As soil cannot take tensile forces, the effect on the quay wall is only a compression force due to the bending moment added to the self-weight. The tensile force is provided by the weight of the soil hanging on the root system at the tensile side.

The most unfavorable effect on the quay wall is when the wind load is directed towards the canal. Although for many cases the wind load in this direction would be lower due to adjacent buildings, this direction of wind load will be assumed in the load combinations.

The total wind load on a tree is calculated by the next formula:

$$Q_w = c_w \; A_{ref} \; {v_{m(z)}}^2 \; \rho_{air}$$

c_w	Permeability factor (assumed to be 0.25 or 0.20 for trees)
A_{ref}	Frontal surface of the crown
$v_{m(z)}$	Mean speed of wind at height z
ρ_{air}	Air density (assumed to be 1.3 kg/m^3)

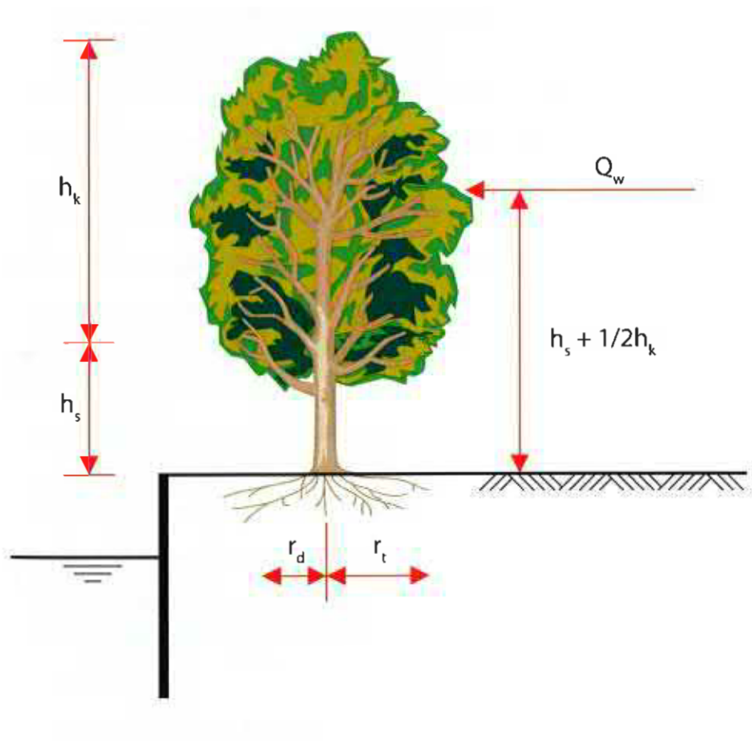

Figure 3.11: Schematic representation of the wind load (from SBRCUR[26])

$$v_{m(z)} = c_r(z) \; c_o(z) \; v_b$$

$c_r(z) = k_r \; \ln(\frac{z}{z_0})$	Roughness factor at height z
$k_r = 0.19 \; (\frac{z_0}{0.05})^{0.07}$	Terrain factor, with $z_0 = 0.5$ (NEN-EN1991-1-4 Table NB.3-4.1)
$c_o(z)$	Orographic factor, can be taken as 1.0

v_b Basic wind speed, can be assumed to be $v_{b,0} = 27.0\, m/s$ (area II)

For oak tree:
$h = 15\, m$ $\quad h_{crown} = 10\, m$ $\quad h_{trunk} = 5\, m$ $\quad d_{crown} = 10\, m$ $\quad c_w = 0.25$

$c_r(z) = k_r\ \ln(\frac{z}{z_0}) = 0.233\ \ln(\frac{15}{0.5}) = 0.76 \qquad v_{m(z)} = 20.52\, m/s$

$Q_w = 0.25 \cdot \frac{\pi}{4} \cdot 10^2 \cdot 20.52^2\ \frac{1.25}{10^{-3}} = 10.33\, kN$

$M_w = Q_w(h_{trunk} + \frac{h_{crown}}{2}) = 103.3\, kNm$

For poplar tree:
$h = 20\, m$ $\quad h_{crown} = 15\, m$ $\quad h_{trunk} = 5\, m$ $\quad d_{crown} = 10\, m$ $\quad c_w = 0.20$

$c_r(z) = k_r\ \ln(\frac{z}{z_0}) = 0.223\ \ln(\frac{20}{0.5}) = 0.82 \qquad v_{m(z)} = 22.23\, m/s$

$Q_w = 0.20 \cdot \frac{\pi}{4}(10 \cdot 15)\ 22.23^2\ \frac{1.25}{10^{-3}} = 14.56\, kN$

$M_w = Q_w(h_{trunk} + \frac{h_{crown}}{2}) = 182.0\, kNm$

The reaction on the quay to resist the bending moment due to wind load, consist of a tensile and a compression force. At both sides this reaction force is assumed to be distributed over a semi-circle with a radius of r_d or r_t, so the resultant compression and tensile force are acting on a distance of:

$x_d = \frac{4r_d}{3\pi} \qquad x_t = \frac{4r_t}{3\pi}$

The tensile force is determined by the weight of soil hanging on the root system, which can be calculated by:

$Q_{v,t} = 0.5\pi\, r_t^2\, \gamma_s\, h_m$

In which: γ_s is the average weight of the soil, taken as 18 kN/m^3
h_m is the contributing width of the soil pack, taken as 1.0 meter

In an iterative calculation, for the oak and poplar tree the following reaction forces can be found:

For oak: $M_w = 103.3 kNm$
$R_w = 2.00m$ $\quad r_d = 1.33m$ $\quad x_d = 0.56m$
$r_t = 1.69m$ $\quad x_t = 0.72m$ $\quad Q_V = 80.6kN$

$q_d = \frac{Q_V}{0.5\pi\, r_d^2} = 29.0 kN/m^2 \qquad q_t = \frac{Q_V}{0.5\pi\, r_t^2} = 18.0 kN/m^2$

For poplar: $M_w = 182.0 kNm$
$R_w = 2.50m$ $\quad r_d = 1.60m$ $\quad x_d = 0.68m$

$$r_t = 2.04m \qquad x_t = 0.87m \qquad Q_V = 117.8kN$$

$$q_d = \frac{Q_V}{0.5\pi\, r_d^2} = 29.3kN/m^2 \qquad q_t = \frac{Q_V}{0.5\pi\, r_t^2} = 18.0kN/m^2$$

By maintaining the same r_d/r_t ratio, the distributed loads are equal for both types of trees. However, the surface at which the load is applied differs in size. The semi-circle surface can be simplified to a squared surface with sides of $0.5\sqrt{\pi\, r}$.

Due to the bending moment coming from the wind load, at one side a tensile force is pulling upwards on the soil while at the other side a compression force is acting. Although the soil cannot take tensile forces, (a part of) the weight of the soil is making equilibrium with these tensile forces. The net vertical load coming from the wind bending moment is zero, but locally the loads could be increased. In comparison to the assumed uniform distributed load, it could be concluded that for the case of the root ball at the compression side being close to the construction, considerably higher local loads could be expected. However, as the total load over the area taken by the tree is lower, in global perspective it will not be governing over the assumed top load of 20 kN/m^2.

Another type of load that originates from the trees close to the wall is the pressing of the tree roots on the structure. The growing roots of nearby standing trees presses the brickwork out of shape. As the magnitude of this load depends on the location and type of the tree, and requires additional investigation, it will not be included. In renovations or constructions of new quay walls it is being recommended to place the trees at a minimum distance of 2.5 meter from the wall to prevent this effect.

3.4.6 Ice load

In case of ice-forming, the top layer of the canal can expand and introduce a horizontal load to the (masonry) wall. At the other side, frost in the soil can introduce expansion loads in the direction of the canal. As they cannot be combined with wave and impact/collision loads, ice loads are generally not governing. However, for local effects on the structure or by causing internal expansion forces, undesired effects could occur.

CUR 166[17] prescribes a design value of 400 kN/m for fresh water, representing an ice layer of 50 cm, a compressive strength of 2.5 MPa and a contact coefficient reduction of 0.33. Looking at the trends, an ice layer of 50 cm thickness is not anymore realistic to be expected. According to KNMI measurements[28], record measurements of ice thicknesses are: more than 40 cm in the winter of 1963, 32 cm thickness in 1997.

In a research publication of the KNMI[29] the expected possibilities of having an *Elfstedentocht* (a Dutch ice-skating event on natural ice, requiring 15 cm of ice thickness) in the 21st century was being analysed. A 15 cm layer of ice is supposed to be formed when the average temperature is below -4.2 ℃ for 15 days or more. If the goals of the Paris agreement can be met and the effect of global warming is kept below an average temperature rise of 2 ℃, the expected possibilities of an *Elfstedentocht* are about 5 to 8% per year.

It could be argued that for urban areas, the heat of the city has a reducing effect on the ice thickness, in comparison to more rural areas in the North of the Netherlands. To be on the conservative side, an ice layer of 200 mm is being assumed. The maximum ice load becomes:

$$q_{ice} = 0.33 \cdot 2.5 \cdot 0.200 = 165kN/m$$

Although this magnitude of load could be acting on the structure, the question remains what form of representation in a structural model would be reasonable. As during the ice formation expansion occurs, the wall could be pressed towards the land-side. In that case, at land-side of the wall a more passive soil state should be considered, countering the effect of the ice load. Next to that, the expansion of the ice layer is also limited. The deformation in the direction of land-side can never become larger than the expansion capacity of the ice layer.

3.4.7 Mooring load

Rijkswaterstaat has developed a classification system[30] which sorts the type of ships based on their sized and required clearance depth. According to the waterways classification[14] of the municipality of Amsterdam, most of the canals are in CEMT class 0, while some of the wider canals are in CEMT class II. Rijkswaterstaat's guideline for waterways (*Richtlijn voor Waterwegen*) prescribes a characteristic hawser load of 150 kN for class I and II.

For most of the urban quay walls this magnitude of load are not valid. Usually the quays only have to deal with smaller recreational or touring vessels, which will induce much lower loads. In a calculation report from the municipality of Amsterdam a mooring load of 40 kN per 5 meter is applied, schematized as a line load of 8 kN per meter. A calculation report for temporary safety structures from the municipality of Amsterdam prescribes a load of 20 kN, while in another quay wall calculation report by Royal HaskoningDHV[31]) a mooring load is set to 5 kN per meter, at a distance of 1 meter above the water level in the canal.

As mooring or hawser loads are not in the same load combination as top/traffic loads, their global effect will not be governing. However, for high concentrated horizontal forces the local effects could affect the design.

3.4.8 Comparison traffic-field load

In today's quay wall design calculations generally the assumption is being made to apply a distributed load of 20 kN/m^2 (following from the Eurocode for geotechnical design of structures[19]; recommendation for harbor type quays) over the full width of the quay. It is considered as a conservative approach, but it is also done for simplicity. This uniformly distributed load is assumed to cover the sum of loads provided in the Eurocode traffic load model[20].

Due to spreading in the sand layer any concentrated load flattens out, so for unshallow sand layers local effects on the floor of the structure are not of high interest. For horizontal effects on the wall, the spreading distance is different meaning some local effects should be considered for concentrated loads near the wall.

The Eurocode traffic load model (LM1)[20] can also be seen as a conservative model. The fact that this model follows from bridge design for the main road network brings up the question whether it could be applied for quay walls in the inner city of Amsterdam. Especially for quays in residential areas (category A and B) the presence of a 60-ton truck is very unlikely. In addition, according to LM1 it should also be combined with a UDL of 9 kN/m^2 (for the first driving lane). Even with reduction factors α_q and α_Q it is still an unlikely high load. With an assumed reduction factor α_q of 0.70 (up to 1000 heavy trucks per year) for categories A and B, the remaining load is equivalent to a 42-ton truck.

As mentioned before, the Eurocode model has to be applied due to the lack of a more representative traffic load model. In the preceding design codes, the VOSB 1995[21] (Old Dutch design regulations for steel bridges) a distinction was made between 3 traffic classes, based on a 60-, 45- and 30-ton vehicle. Also in line with the by the municipality of Amsterdam adopted 28-ton engine truck, a 30-ton traffic class is much more representative, especially for categories A and B. In the VOSB it had to be combined with a UDL of 2 kN/m^2.

For the parking zones, a UDL of 5 kN/m^2 is assumed. For an average parking spot with a width of 2.5 meter and a depth of 5 meter, this UDL represents a vehicle with a weight of: $2.5 \cdot 5 \cdot 5 = 62.5\, kN = 6.25$ ton. It is a quite conservative value, but it gives a reasonable upper value. Assuming the limit weight of 3.5 ton for delivery trucks, the representative UDL could be reduced to $3.5/6.25 \cdot 5 = 2.8\, kN/m^2$. Next to the UDL for parking zones, the Eurocode prescribes a randomly placed concentrated load of 40 kN representing a jacked vehicle. In fact, this concentrated load should be replacing the UDL as it follows from the same vehicle. This load is only of interest for local effect, when the concentrated load is placed near the wall.

For pedestrian zones also a UDL of 5 kN/m^2 is applied. Also this load is quite conservative. Eurocode LM4 prescribes a UDL of 5 kN/m^2 representing a crowd load, but in the appendix a range between 2.5 and 5 kN/m^2 is given. However, for the inner city of Amsterdam the situation of having a quay full of people is well imaginable, so for that reason a UDL of 5 kN/m^2 is a fair value.

Simplified load spreading model

For simplicity, the comparison is based on a wall with a retaining height of 2.0 meter and a soil pressure factor K_0 of 0.5 is used. A difference has to be made in the effect on the wall and the floor of the quay wall. First, the wall is being considered. Assuming a spreading over a 45 degree angle in both vertical and horizontal direction, the following simplified spreading model is applied. Using a spreadsheet for this model, the contributions of each load type is calculated.

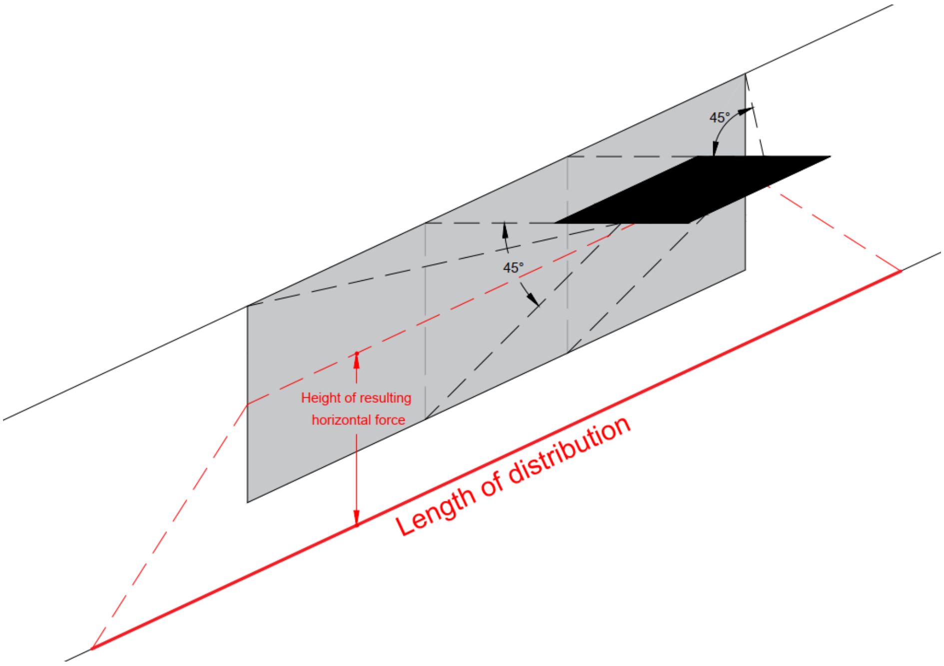

Figure 3.12: figure load spreading 1

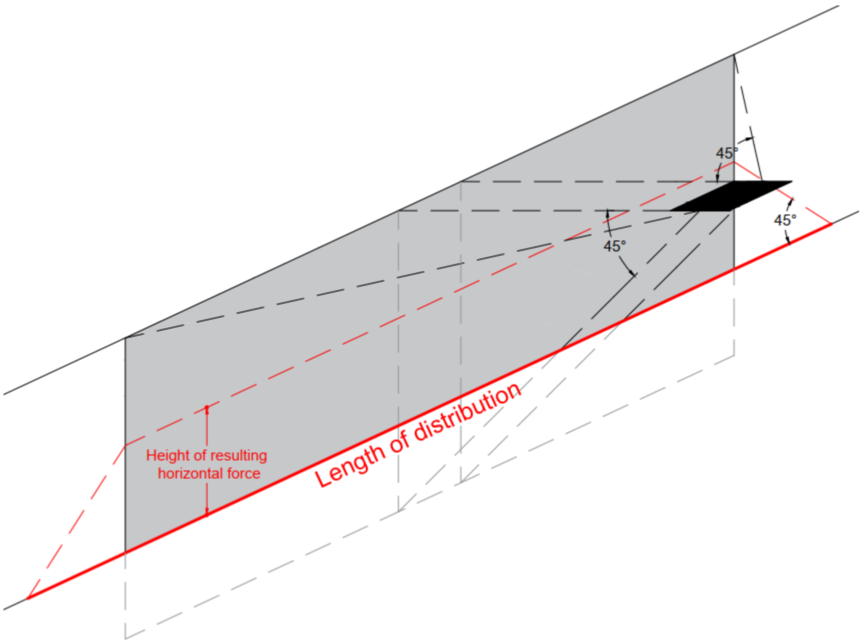

Figure 3.13: figure load spreading 2

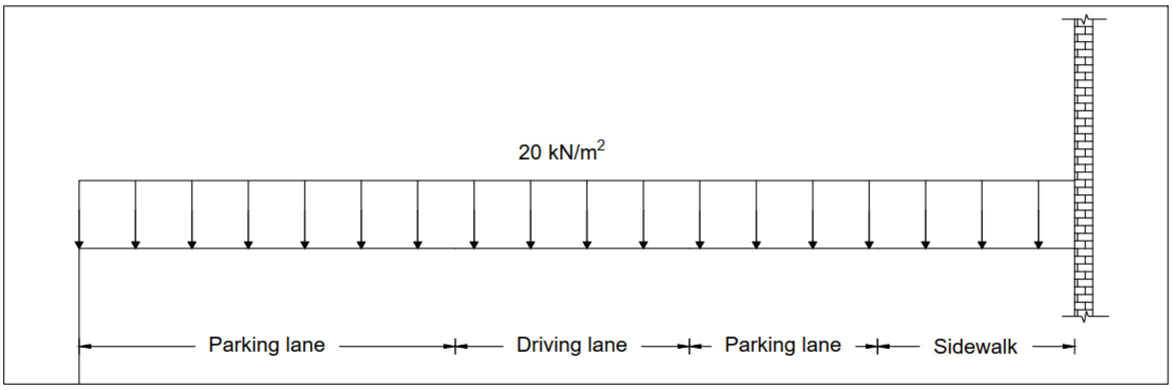

Figure 3.14: Load model CUR

First the load model following from the Eurocode of Geotechnical design is considered. A constant UDL of 20 kN/m^2 results in a bending moment at the bottom of the wall of:

$$m_{UDL} = \frac{1}{2}\, H^2 K_0\, q_{UDL} = \frac{1}{2}\, 2^2 \cdot 0.5 \cdot 20 = 20\; kNm/m$$

This bending moment along the wall-floor connection will be used as a reference value for the comparison with the other traffic load models (with equal height and soil pressure factor). In the next figure a schematic presentation of the loads in LM1 is given. As mentioned above, this model will not be considered as it is reduced to the load model following from VOSB class 30.

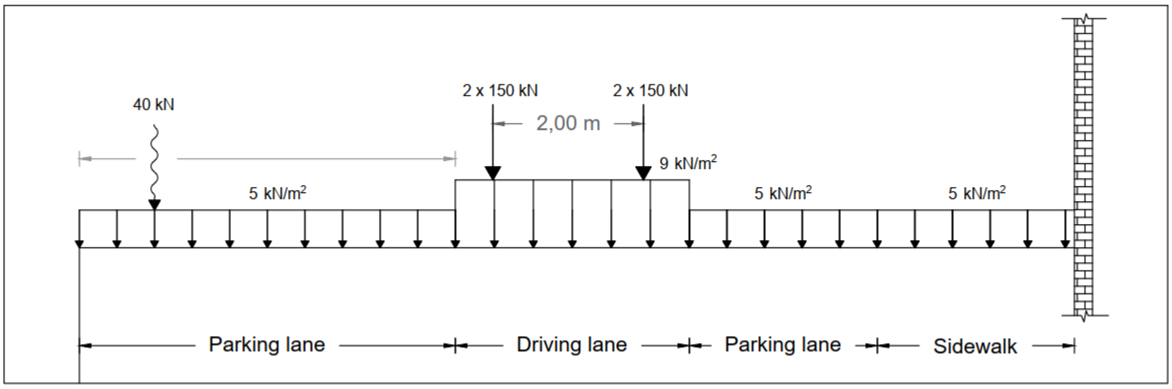

Figure 3.15: Load model EC LM1 (not considered)

In the next load model only the loads in the driving lane are reduced to representing a 30 ton truck.

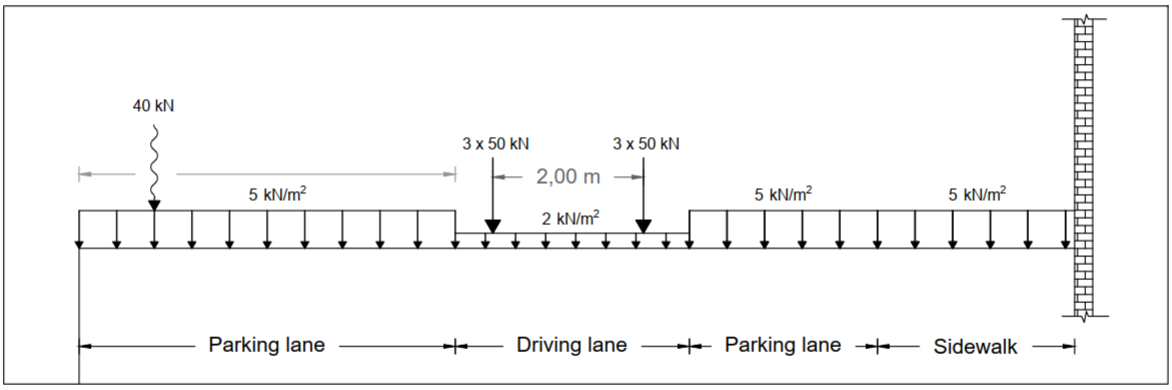

Figure 3.16: Load model VOSB class 30

The jacking load is considered at a distance of 0.5 meter from the wall. Using Boussinesq formula for a concentrated load, the following distribution of lateral load can be found along the height of the wall. The centroidal axis of the load, at which the resulting force acts, is found at a height of $2.0 - 0.956 = 1.044\ m$ from the bottom of the wall. Using a simplified horizontal spreading angle of 45° in the soil and a vertical spreading of 45° in the wall, the spreading length along the bottom of the wall becomes $2 \cdot 0.5 + 2 \cdot 1.044 = 3.09\ m$.

The resulting bending moment along the bottom of the wall is:

$$m_{jack} = 1.044 \cdot F \cdot K_0/(3.09) = 6.76\ kNm/m$$

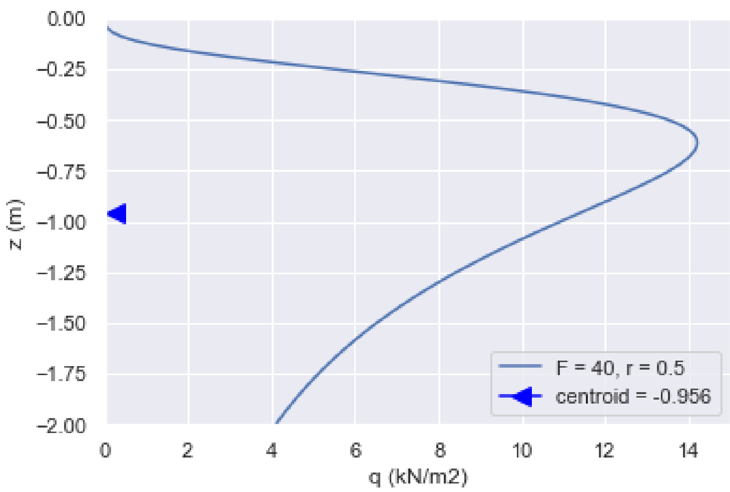

Figure 3.17: Boussinesq load distribution of a 40 kN jacking load

As the UDL following from the parking zone is present over a large width next to the wall, the slightly conservative assumption can be made that a UDL of 5 kN/m^2 results in a bending moment of 5 kNm/m. For the driving lane, the spreadsheet of simplified load spreading is used. The remaining loads have a negligible effect on the wall. By summing the loads over a width of 6.5 meter, the following values are found:

Load type	Load	Moment contribution
Jacking load (0.5 m)	F = 40 kN	6.76 kNm/m
Parking zone	q = 5 kN/m^2	5 kNm/m
Driving lane (TS+UDL)	$F = 6 \cdot 50 + 2(3 \cdot 6.5) = 339\,kN$	3.16 kNm/m
	Total bending moment	14.92 kNm/m

Table 3.4: Bending moment contributions for the load model based on VOSB class 30 with a jacked vehicle at 0.5 meter from the wall

In the next composition of loads, the 30 ton vehicle is placed next to the wall, at a distance of 0.5 meter. The total load is divided over a single surface enclosed by the wheels. In the figure below cross-sectional overviews of the wheel positioning according to the VOSB[21] is shown. A surface area of $2.5 \cdot 5.16$ is used as input, on which the total load of 300 kN is equally distributed.

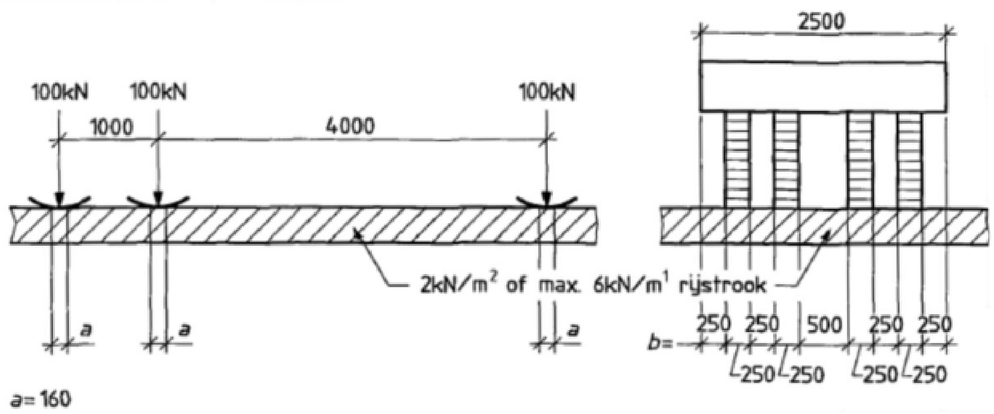

Figure 3.18: Axle and wheel distances from VOSB class 30[21] (dimensions in millimeter)

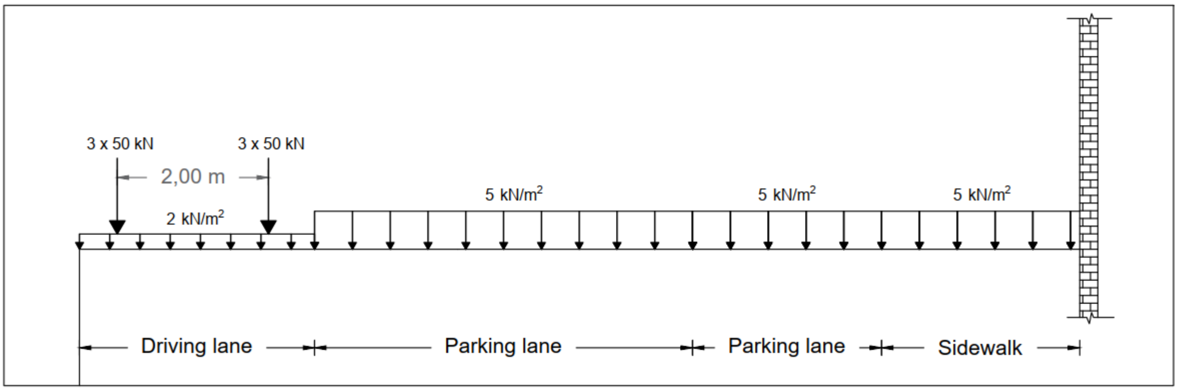

Figure 3.19: Load model VOSB class 30

For the remaining loads (starting at 3 meter from the wall) only a width of 8 meter is considered to have an effect on the bending moment in the wall. A length of 6.5 meter is being considered (length of the vehicle), so a total load of $8 \cdot 6.5 \cdot 5 = 260\,kN$. Again, the simplified spreading model is used to find the contributions of each of the loads, resulting in the following values:

Load type	Load	Moment contribution
30 ton truck	F = 300 kN	15.47 kNm/m
UDL driving lane	q = 2 kN/m^2	2 kNm/m
Remaining space	q = 5 kN/m^2	1.65 kNm/m
	Total bending moment	19.12 kNm/m

Table 3.5: Bending moment contributions considering VOSB class 30

Simplified load spreading for separate wheel loads

In the simplified load spreading model, the total bending moment is assumed to be equally distributed over the total length of spreading. In reality, local effects of separate wheel loads are higher than the above assumed value of 15.47 kNm/m. By adding the overlapping effects of separate wheel loads, using the simplified spreading model, the maximum bending moment can be found. In the following table, the values are given for the situation of a 30 ton truck at 0.5 meter from the wall. The red lines in the wall show the spreading width at the bottom of the wall (including vertical spreading at 45° in the wall). The effects of the outer wheels (for x = 0.5 m) are just not overlapping. A wheel print of 160 mm in length and 2x250 = 500 mm in width is assumed (according to VOSB class 30).

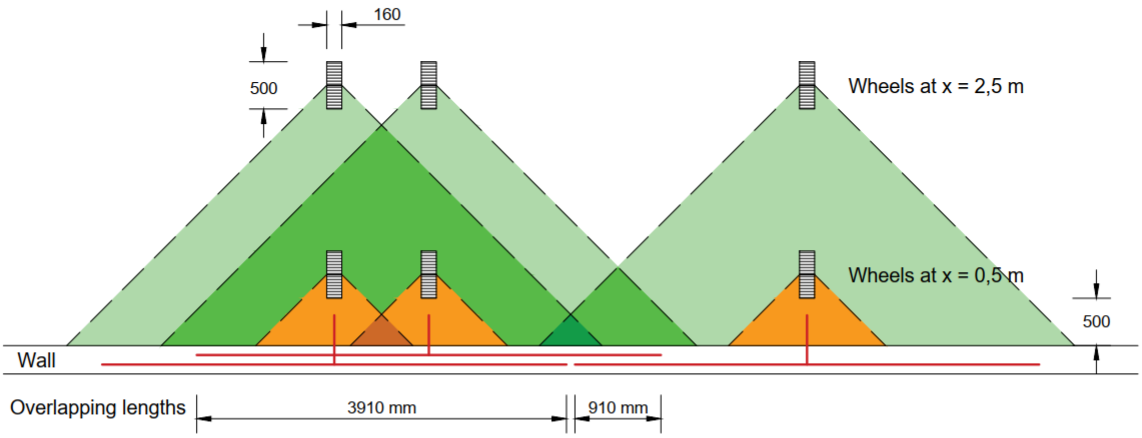

Figure 3.20: Top view of the simplified spreading model

	Wheels at x = 0.5 m	Wheels at x = 2.5 m
Resulting moment	40.62 kNm	18.18 kNm
Distribution length	4.91 m	7.66 m
m-load	8.27 kNm/m	2.37 kNm/m
Overlapping length ($L_y = 1.0m$)	3.91 m	6.66 m
Overlapping length ($L_y = 1.0m$)	0.91 m	3.66 m
Total bending moment =	$(2 \cdot 8.27 + 3 \cdot 2.37) =$	23.65 kNm/m

Table 3.6: Bending moment contributions considering separate wheel loads

The maximum bending moment of 23.65 kNm/m acts in the middle zone over a length of 910 mm, where the effects of 5 out of 6 wheel loads are overlapping. Adding the contributions of load on the driving lane and the remaining space (see table 3.5), the maximum bending moment becomes:

$$m_{max} = 23.65 + 2 + 1.65 = 27.3\, kNm/m$$

Load spreading in SCIA Engineer

Even more local effects can be considered for the wheel load effects on the wall by modeling the loads in SCIA Engineer. Where in the simplified spreading model the bending moment is step-wise changing over the influenced length, in fact the local effects are gradually evolving. The effect of wheel loads (3 loads at 0.5 meter and 3 loads at 2.5 meter) are modeled as block loads on the wall, as is shown in the next figures:

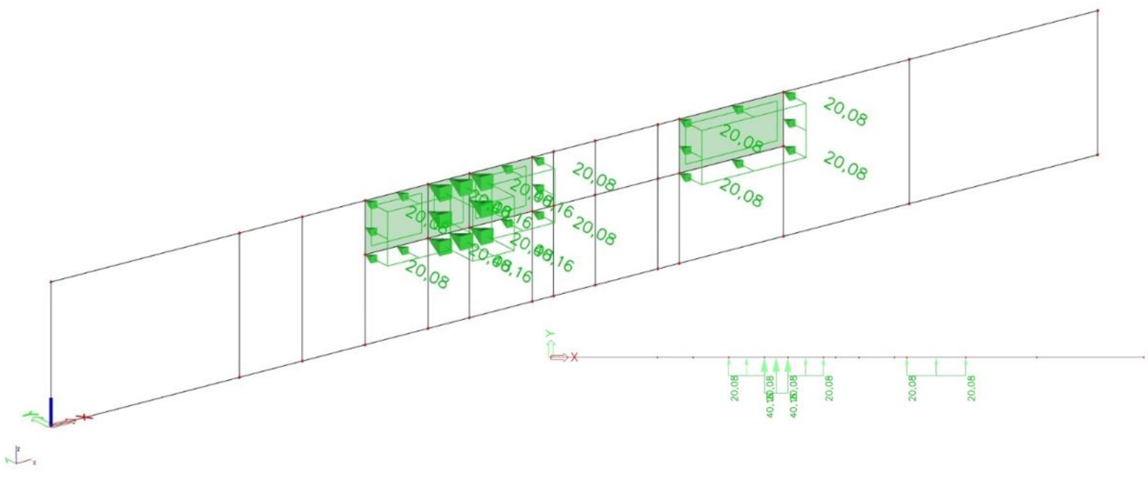

Figure 3.21: Load modeling for wheels at x = 0.5 m

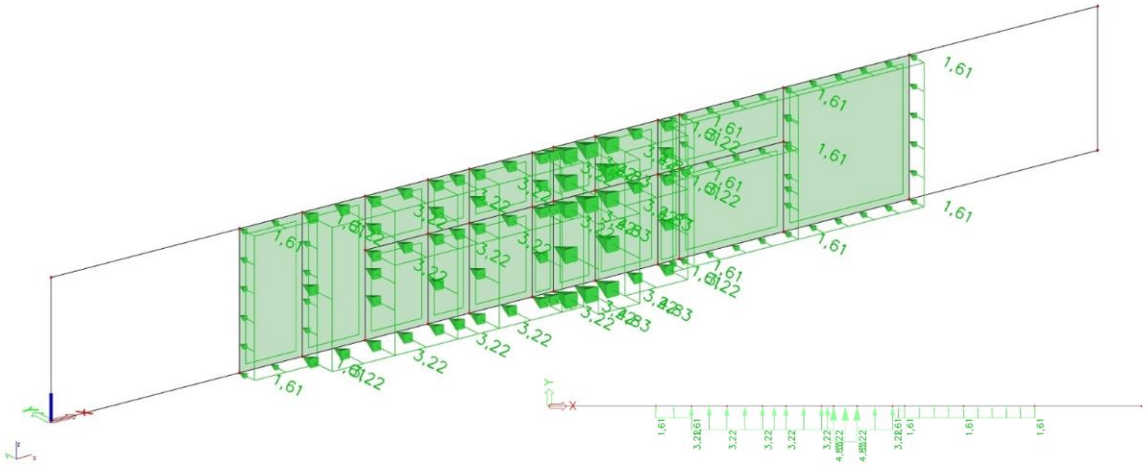

Figure 3.22: Load modeling for wheels at x = 2.5 m

The following bending moments can be found in the wall as a result of the separate wheel loads. The maximum bending moment is 25.59 kNm/m, which is about 8% higher than the maximum bending moment of 23.65 kNm/m for the simplified model. Adding the contributions of load on the driving lane and the remaining space (see table 3.5), the maximum bending moment becomes:

$$m_{max} = 25.59 + 2 + 1.65 = 29.24\, kNm/m$$

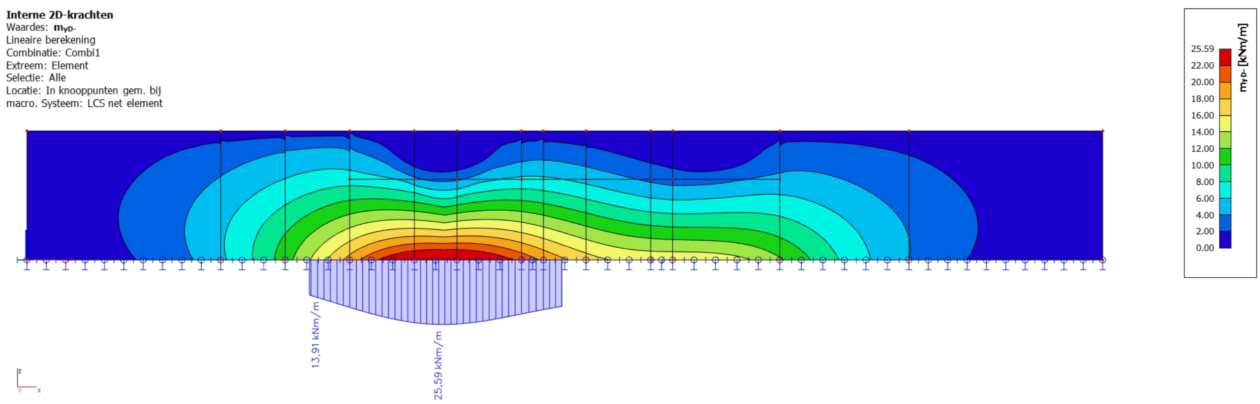

Figure 3.23: Bending moment results from SCIA Engineer

Vertical load spreading

For the horizontal effect on the wall, local effects are more present than for vertical loads. Vertical loads have a high (constant) length of spreading, so concentrated loads can easily be assumed as distributed loads. Considering a 30-ton truck, distributed over 3 axle loads with distances of 1.0 and 4.0 meter, the assumption can be made that the space occupied by the truck has a length $4.0 + 1.0 + 1.0(rear) + 0.5(front) = 6.5$ meter and a width of $2.0 + 0.27(wheel) = 2.27$ meter.

$$A = 6.5 \cdot 2.27 = 14.8\, m^2 \qquad F = 300\, kN \qquad q = 300/14.8 = 20.3\, kN/m^2 \approx 20\, kN/m^2$$

Taking into account 0.5 meter of space free of load on all sides, the equivalent UDL at ground level becomes 15.5 kN/m^2. So even by having 300-ton trucks over the full width of the quay, the equivalent distributed load does not exceed the UDL of 20 kN/m^2.

Conclusion

In this comparison, the effects of the loads are being considered from a global to a local approach for an L-shaped quay wall. The most global model is a UDL of 20 kN/m^2 over the full width. By comparing it to load models with a 30 ton truck next to the wall and subsequently the summed effect of separate wheel loads, more local effects show up.

By giving the wall more height, the local effects are disappearing as spreading over a larger area is possible. In comparing the bending moment in the wall for a 30 ton truck (with a closed surface loading at 0.5 meter from the wall) with the bending moment as a result of a UDL of 20 kN/m^2 for varying heights of the wall, the following graph (based on the simplified spreading model) shows the effect of increased spreading. For smaller heights of the wall the UDL gives a good indication, but as the height increases the UDL-model becomes over-conservative.

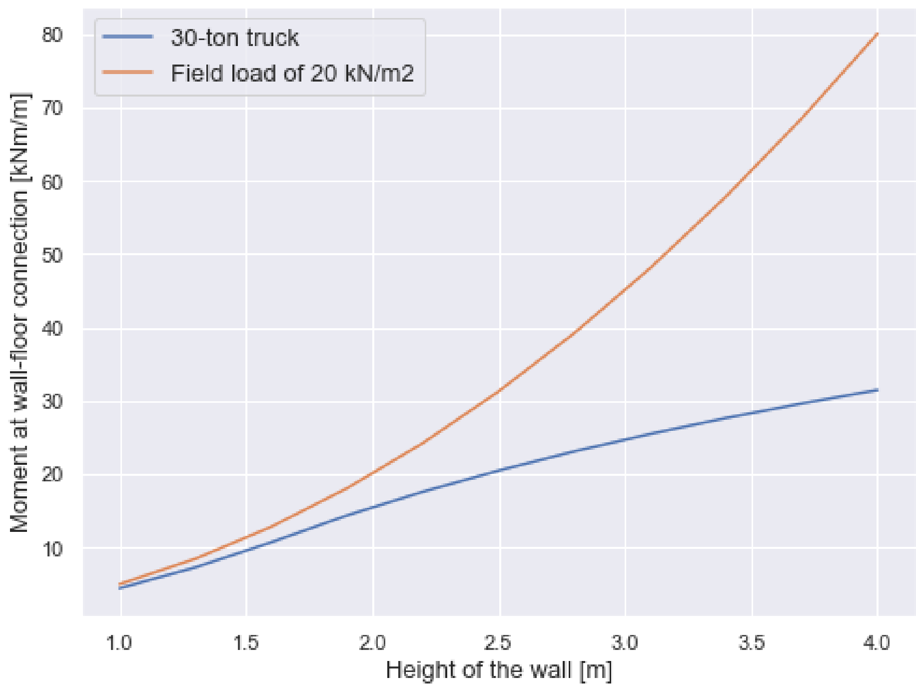

Figure 3.24: Bending moment for different heights of the wall

For the design of the slab under vertical loads, the UDL of 20 kN/m^2 can be assumed as the governing situation. For heights lower than 2 meter, concentrated top loads (for example a 30 ton truck) could locally result in higher effects. For the overall reaction forces on the piles, this assumption is too conservative. By using this UDL over the full length the sum of loads is largely overestimated. Here the traffic load models can be considered for a more accurate representation. As for local load effects the structure still has a capacity for redistribution, the model is being designed based on the UDL. The

UDL of 20 kN/m^2 is assumed to be a good upper value in covering the effects of local loads.In a later design stage the local effects should be considered more carefully, which could have an effect on the wall or higher parts of an inclined floor.

The other considered loads are less critical in global perspective, but their local effect could become critical. The tree loads could only be governing for the case of the (compressive side of the) root ball being very close to the structure. In overall load it will result in a lower equivalent distributed load in comparison to the assumed top load. Other loads, like ice and mooring/hawser loads should be considered in a later design stage to make sure the structure could provide the required local capacity. In the load combinations these loads are not considered due to the corresponding Ψ_2-factors.

3.4.9 Load combinations

For a correct design calculation all possible effects should be taken into account within different load combinations, reflecting the actual situations in which the loads could be acting at the same time. As mentioned above, a distinction is being made between the global effect and the local effect. For simplicity of this research only loads in a global perspective are being assumed.

In developing a parametric model much simplifications and assumptions are required. As will be mentioned later, the assumption is being made to only consider one (assumed to be) load combination which should cover for the most governing aspects. Based on studying reference calculations made for quay wall structures within the same context, the frequent-state load combination can be assumed as governing in the design verification in satisfying the requirement for crack width control. As some design verifications need to be performed other ultimate limit state load combinations, a simplified approach in converting results is being taken. This will be explained in the chapter Parametric model.

It is being assumed that one load (top load of 20 kN/m^2 with a Ψ_1-factor of 0.75) covers for all other loads on top of the quay. Within the frequent-state load combination all other considered local loads can be eliminated due to the corresponding Ψ_2-factors. This entails that in the frequent-state load situation the maximum value for the top load cannot be acting at the same time as the other loads. For the preliminary design stage, and taking into account the magnitude and effects of these loads, this can be considered as a valid assumption.

3.5 Variants

For meeting the functions a quay wall structure need to fulfill, several alternative design options could be considered. In this chapter, the main question to be answered is: What are the options and why not the traditional way?

In the introduction a brief reasoning is given for potential improvement aspects of the traditional design. The main disadvantages are: construction is very time-consuming, cause too much nuisance and are generally not optimized for its material use (both in volume and sustainability). The nuisance consists of both noise and traffic hindrance, caused by the relative large construction pits that are needed for the demolition/construction process. To be able to take out the existing structure the construction pit requires a large width, which occupies space both on the quay and in the canal.

The starting point in finding an effective structural solution for the problem (a difference in ground level requiring for a height to be bridged/retained) is to consider the type of loads that could occur and the possibilities of transferring the loads to deeper soil layers. An enumeration of all the possible loads are presented in the section Loads. In the chapter Conceptual design this path of thinking will be discussed more elaborately. The variants that are being considered in this chapter are the traditional options, two of the innovative concepts coming from the IPK, and some radical different solutions which could satisfy the structural requirements but not (necessarily) fulfills all the functional boundary conditions. As the third innovative concept (G-Kracht) can be placed in the same category as a combi-wall structure, by clamping large steel elements into the soil, this variant is not being considered. The advantage of that concept is more to be found in the promised construction specific improvements.

3.5.1 Design options and motives

Of course the considered variants in this section are only a selection of the possibilities that can be thought of. Many other solutions are possible, think about massive diaphragm walls, U-shaped sections covering the inside of the canal's cross-section, anchored solutions and all other forms which are able to retain a certain height. The selection of variants is based on the most practical and suitable design options given the unavoidable limitations of the context and considering the expected amount of material use.

Traditional L-wall and combi-wall

The first two variants are currently the most used solutions for urban quay wall renovations. In short, the L-wall can be seen as the preferred option due to its robustness. It consists of 2 (or more, but in urban context usually 2) pile rows connected to a relieving platform. At the canal-side a wall is clamped to the floor. As it requires a large construction pit and takes relatively a lot time to be constructed, this option easily conflicts with environmental or construction-related limitations.

When the working space and therewith the design freedom become too limited, the combi-wall is usually being opted. A combi-wall consists of one row of piles, supporting sheet piles in-between them. In front of the system, a prefab brick wall-carrying or brick wall-resembling element is placed. Although this variant does not cause as much hindrance in the use of space, it needs to be placed next to the existing structure, reducing the width of the canal. To make sure the structure is stiff enough relative large dimensions are required. To increase the stiffness sometimes compression piles are being applied on the canal-side of the structure. To not become an obstacle in the canal, this has to be in a very steep angle. The total structure uses a large amount of steel.

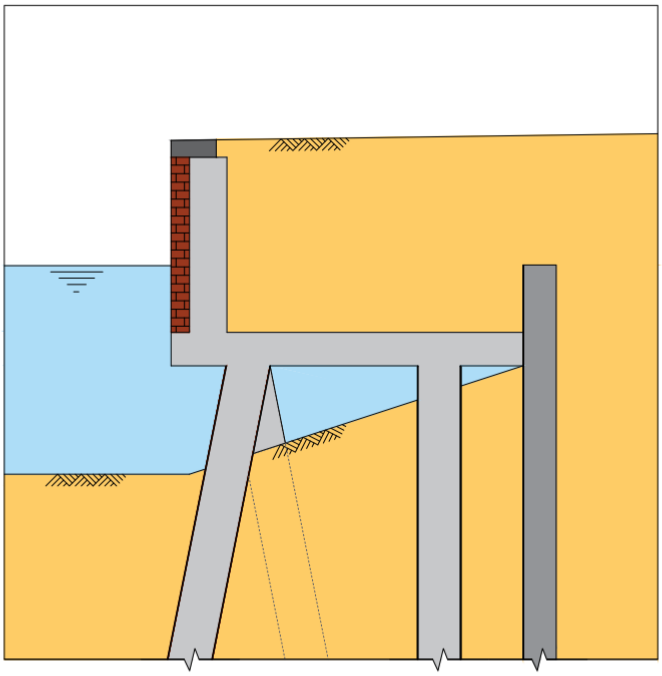

Figure 3.25: Traditional L-wall

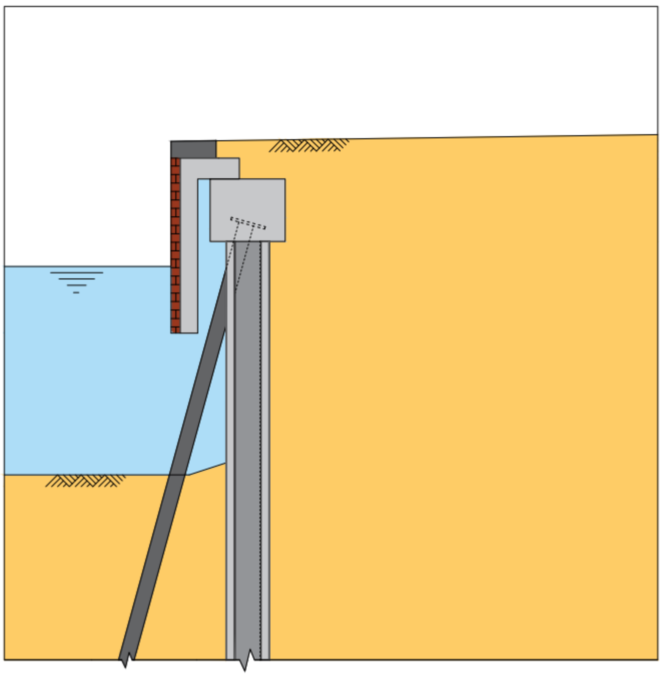

Figure 3.26: Traditional combi-wall

Modular L-wall

A type of solution that could tackle multiple disadvantages of the traditional designs, is a modular type of L-wall. By making openings in the elements in both longitudinal and transverse direction, savings of material could be made. A production technique for such elements could be 3D-printing in concrete (or even other types of materials) or hollow core production methods. Whether the more expensive production process is overriding the savings of the material use needs to be investigated. To ensure the structure is sand-tight, the inside of the structure can be covered by a geotextile layer. In this way excessive rainwater can easily drain of into the canal, so no additional drainage systems are required. The feasibility of this solution is directly depending on whether the construction works can be done without a construction pit. Next to the fabrication method, the main challenge is to make the connection to the foundation piles. To allow for a circular use, a demountable connection is required. If it does not have to be circular, a grouted connection can be made. However, to make this connection underwater and compensating for pile deviations is a complicated task.

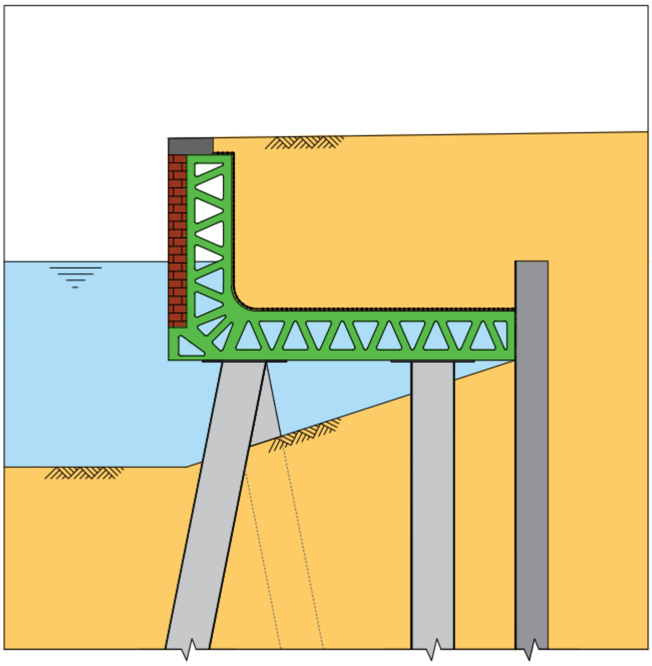

Figure 3.27: Modular L-wall

Kade 2.020

One of the variants from the IPK is Kade 2.020[9]. A reversed L-wall element which is placed on the canal-side of the existing structure. Quite massive prefab elements are placed on piles, without the need for a construction pit. Also for this option, the grouted connection which should be made underwater is a complicated task, taking into account certain pile deviations. It does not include a relieving platform, meaning the existing structure is still bearing vertical loads. As the condition of the existing piles is not known, this could lead to settlements differences behind the wall.

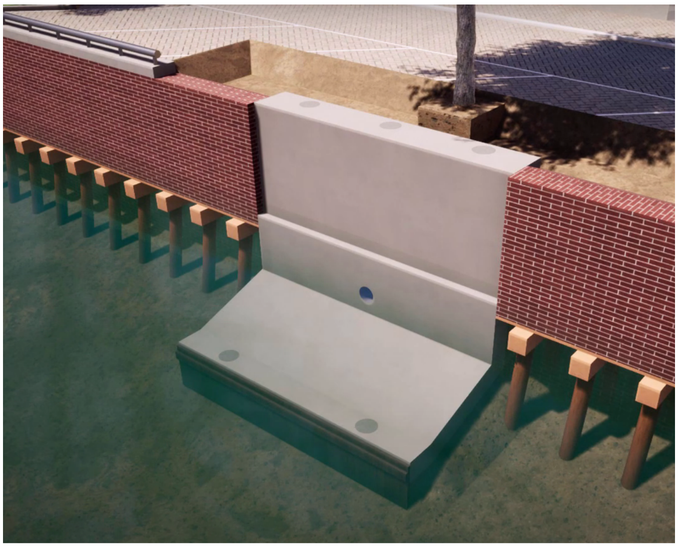

Figure 3.28: Innovative concept by Kade 2.020[9]

Koningsgracht

The variant proposed by consortium Koningsgracht[10] from the IPK offers a renovation method in which the solidity of a traditional L-wall structure can be achieved with a low nuisance level. The construction phasing makes it possible to construct without a construction pit. By using the pontoon for the stabilization of the wall, while a trench box ('sleufkist') is used to excavate soil behind the wall. A prefab element with bars on the land-side and couplers on the canal-side (to make the connection to the wall) is placed inside the excavation. By poured underwater concrete while lifting the trench box the floor is being cast. On top of the new slab big bags are temporary placed to stabilize the soil, while the existing wall and the protruding parts of the existing floor is being removed to make space for a prefab element with a seepage sheet in front of the wall. On the front row of piles a prefab wall is placed, with on the bottom headed bars. By casting the space in-between the wall element and the floor element (in which the couplers from both element are present) with underwater concrete, a clamped connection between the slab and the wall is made. Drainage pipes are present in the prefab element of the slab, and the space in-between the structure and the seepage sheet is filled with gravel to ensure a soil-tight connection.

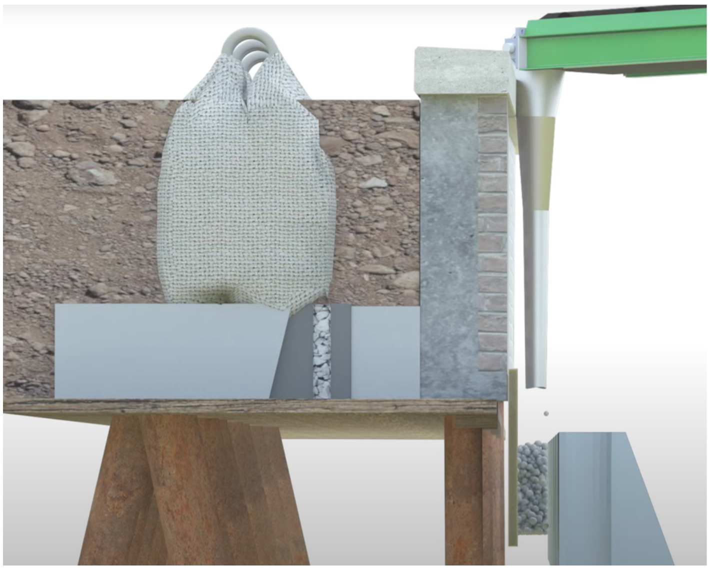

Figure 3.29: Innovative concept by Koningsgracht[10]

Transverse wall

A solution in which both the solidity of an L-wall and a saving of the material use could be achieved is the variant with a transverse wall. In a similar way as in the previous method, the soil can be excavated while the existing wall and the soil behind the structure are being stabilized. By integrating a transverse element, which could be a frame or a concrete cast stiffener, the bending moments in the wall could be reduced, for which a save on material can be made. Additionally, instead of making a costly connection with couplers and headed bars, a more simple connection can be made. The connection does not need to be clamped anymore, but a relative easy connection which only resists tensile or shear forces is required to assure the integrity of the structure. Whether the amount of saved material and costly connections outweigh the extra costs and material for the transverse wall should be investigated. Furthermore, the dimensions of the transverse wall are limited by not becoming an obstacle for the use of space behind the wall or for cables and pipelines.

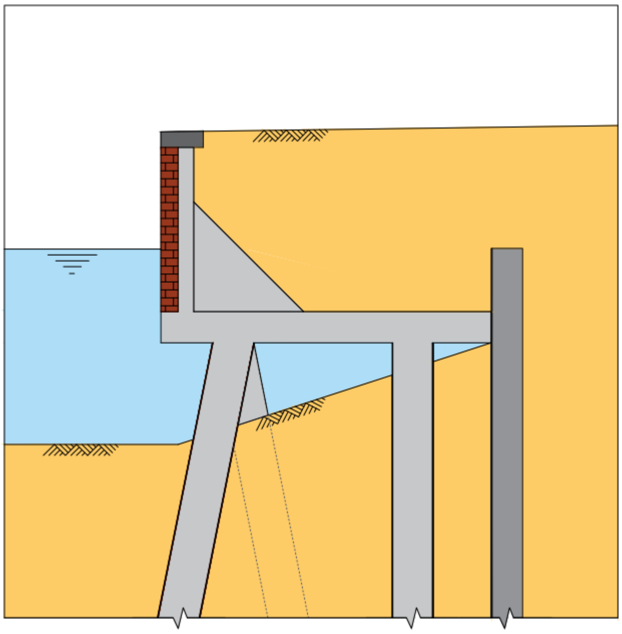

Figure 3.30: L-wall with a transverse stiffener

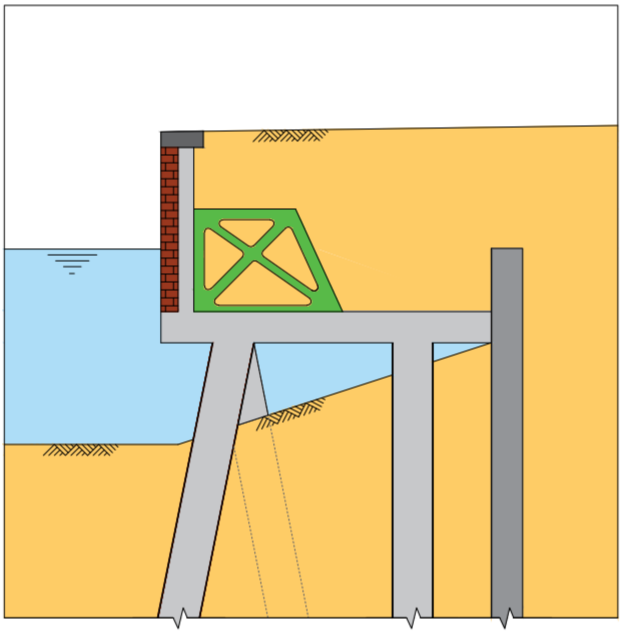

Figure 3.31: L-wall with a transverse frame

Reinforced soil

In this variant the focus is placed at reducing the horizontal loads on the wall. Layers of soil reinforcement (for example geotextile layers) are being applied on top of the floor. To anchor the reinforcement layers into the stable soil, enough anchorage length (approximately 6-8 meter) is required. Within this zone no cables, pipelines or trees could be present. Next to that, the existing structure needs to be removed. For this solution a construction pit is inevitable, which means the construction speed is low and the level of nuisance is high. However, in terms of material use it could save a lot, since the lateral loads are taken out of the structure.

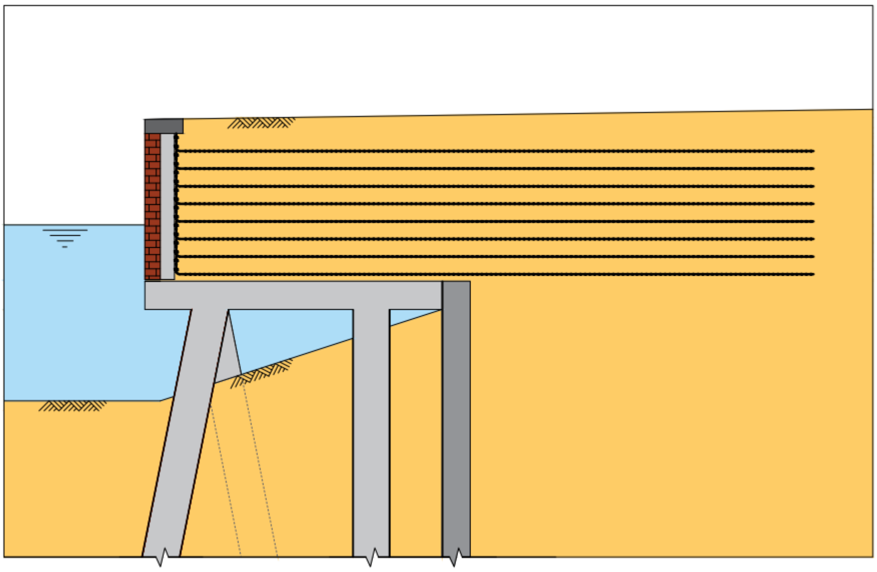

Figure 3.32: Reinforced soil

Big bags

Another solution in reducing the horizontal loads on the wall is by making big bag-type of enclosed soil packs part of the permanent design. In this way the self-weight of the sand in the bag resists the horizontal load from the soil behind it and a large anchorage length can be avoided. When the material of the bag is strong enough to resist the tensile stresses from the loaded sand inside, the stability of the system depends on the frictional resistance against sliding and tumbling over of the bags To prevent sliding a block or strip could be placed in front of the bags. The prefab wall only needs to resist loads coming from the soil in between the bags and the wall. To reduce the bending moment on the wall an anchor could be attached to the big bags.

Although in this variant the wall could be made more slender due to the reduction of bending moments, the horizontal loads are still transferred through the big bags and the floor towards the piles. As the leverage arm of these loads is a bit lower, the bending moments on the piles are reduced, but still cause a large pile head bending moments.

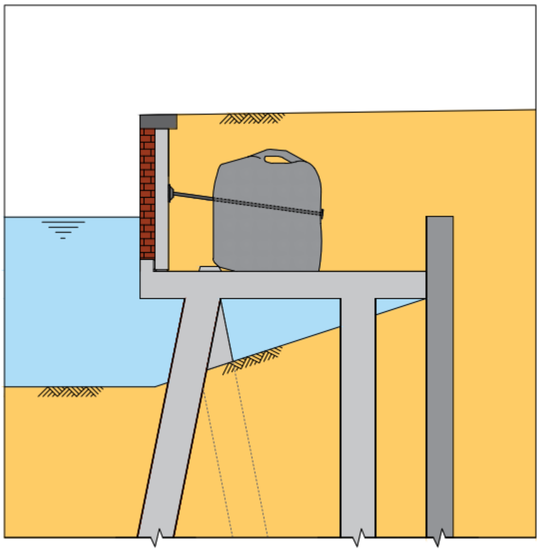

Figure 3.33: Big bag-inspired variant

3.5.2 Comparison of alternatives

To compare the performance of the mentioned alternatives, an assessment model has been used in which the criteria for the speed of construction, the sustainability level, the expected costs, the level of nuisance and the robustness of the structure are marked. In line with the preference for an L-wall structure of the municipality, the score for robustness have been given a triple weight, based on calibrating the performance of a traditional L-wall and a combi-wall. As for the non-traditional variant their performance on construction speed, cost and nuisance depend on the construction phasing that is feasible, their rating for these criteria are set to an average estimated performance. The scores indicate both the best performing concepts as well as the drawbacks and possible opportunities for improvements of the variants.

Criterion	L-wall	Combi-wall	Modular L-wall	Koningsgracht
Execution speed	-2	+2	-1	0
Sustainability	-1	-2	+1	-1
Cost	-1	0	-2	-2
Nuisance	-1	+1	0	0
Robustness (x3)	+2	-1	+1	+2
Score	+1	-2	+1	+3

Criterion	Kade 2.020	Backwall	Reinforced soil	Big bags
Execution speed	+1	0	-1	0
Sustainability	-2	-1	+2	-1
Cost	0	-1	+1	-1
Nuisance	+1	-1	-1	0
Robustness (x3)	+1	+2	-1	+1
Score	+3	+3	-2	+1

Table 3.7: Performance for each of the variants)

3.6 Materials

The main objective in this research is to find minimize the amount of materials use. As different types of materials are used, the starting point is to consider the way of measuring the material use. The overall amount of used materials can be expressed in multiple ways. To compare the material use for multiple designs, the results for each type of material should be expressed in the same unit. The simplest way is multiplying the volume each type of material by its unit cost.

To take the sustainability into account, also a C02-equivalence unit can be used. By considering shadow prices (used for performing life cycle analyses) the material use can be converted into the material's burden. By summing up the burden related to different environmental indicators, the total of environmental impact can be expressed in one unit, environmental cost indicator (ECI, in Dutch *MKI*). In other words: The MKI indicates the magnitude of the overall environmental impact for a certain material.

Of course, also other ways of expressing the material use are available. Which of them are best applicable depends on the application. In this research, the material use is being expressed in the sum of material costs. In a small comparison the difference between both units is demonstrated to be limited.

3.6.1 Used materials

Given the scale of this research the used materials are reduced to the 'traditional' materials. For the concrete in the wall, floor and piles a class C30/37 concrete is being assumed. The casing of the steel piles is taken as class S355 steel. The used reinforcement is class B500B.

With respect to the crack width control, the following environmental classes are being assumed:

XC4	Corrosion induced by carbonation: alternating wet and dry conditions
XD3	Corrosion induced by chlorides: alternating wet and dry conditions
XF4	Attack of freeze/thaw: saturated with water, including de-icing salts
XA1	(chemical attack): weak aggressive conditions

By following the classification in table 4.3N in the Eurocode 2[32], considering a design life of 100 years, a governing environmental class XD3 and a plated geometry, a structural class S5 needs to be considered. This leads to a $c_{min,dur}$ of 45 mm. Including the tolerance of 5 mm the concrete cover is set to 50 mm. For the concrete section in the piles, due to lower environmental classes, a concrete cover of 40 mm is allowed. Following from table 7.1N in the Eurocode 2, a maximum allowed crack width of 0.2 millimeter is prescribed.

3.7 Comparison of units: cost versus MKI values

The shadow prices (presented in table 3.8) involved with these materials are based on MKI-values obtained from the DuboCalc database (version 6.0). As the considered life cycle stages do not take reusing or recycling into account, the considered stages are reduced to LCA-stage A (production and construction). The assumed MKI values are based on 50 years of service lifetime. As the design service life of quay walls is set to 100 years, the MKI values will be lower than assumed. By taking stage C (demolition) into account the values will increase (concrete: +32%, reinforcement: +0.3% and steel: +9%) and if recycling or reusing becomes part of the scope the values will decreased (concrete: -8%, reinforcement: -55% and steel: -17%).

In the next table the assumed MKI-values and the assumed material costs are presented. These MKI-values are not subjected to the 30% margin of uncertainty.

Material type	MKI	Unit	Material cost	Unit
C30/37 concrete	16.47	€/m^3	0.06	€/kg
S355 steel	132.02	€/ton	1.20	€/kg
B500B reinforcement	198.41	€/ton	1.40	€/kg

Table 3.8: Considered values for the MKI and cost for each type of material

By making a small comparison, the effect of choosing the MKI as the expression unit instead of the costs of materials is pointed out. First, the MKI values are converted into a uniform unit, considering specific weights of 2400 kg/m^3 for concrete and 7850 kg/m^3 for steel and reinforcement. By summing the values for each type of expressing unit, the relative contribution for each material can be indicated. Although the value has no meaning, it does say something about the ratio between different materials.

Material type	MKI	Contribution	Unit	Material cost	Unit	Contribution
Concrete	16.47	0.63%	€/m^3	144	€/m^3	0.70%
Steel	1036	39.70%	€/m^3	9420	€/m^3	45.83%
Reinforcement	1556	59.67%	€/m^3	10990	€/m^3	53.47%

Table 3.9: Comparing the relative contributions for each expression unit

From the relative contributions, it can be seen that the effect of using the MKI as the unit to express the material use can cause small shifts of the material contributions. In case a design with a certain use of materials is expressed in an MKI-value, the contribution of the concrete material to the total of material use is almost 10% lower. However, as the largest contribution in both types of expressing the material use remains to be the steel and the reinforcement, that effect is of minor influence.

Considering the relative contributions for the steel and the reinforcement, also a shift can be noticed (-13.4% and +11.6% respectively). This entails that for using the MKI as expressing unit the contribution of the reinforcement to the total material use becomes higher compared to expressing in terms of costs, while the contribution of the steel is reduced.

3.8 Optimization

In large-scale context the term 'optimization' has a volatile nature. The large amount of dependencies create an enormous amount of combinations which could all result in a more optimized solution. As it is almost impossible to create a model which takes all dependencies into account for finding the optimal solution, a number of assumptions or estimations are needed to make the model workable.

3.8.1 Objectives

The main objective that is defined for this research is the minimization of material use. Of course, the parameters that can have an influence on this objective are tremendous. The taken approach to make a workable model is to cover some dependencies by assumptions and to create intermediate objectives. All made concessions lead to an increase of the range of deviation in which the optimal solution can be found.

The main objective requires a certain unit in which the contributions of all materials can be expressed. An obvious unit to use is a monetary value. By multiplying the volume of each material by the cost per volume for that material, one single value can found.

3.8.2 Sequence of optimization

The used optimization sequence can be split into 2 parts: (1) finding the optimal support lines in width and (2) finding the optimal pile locations in longitudinal direction by varying the center-to-center-distance, the floor thickness and the pile diameter.

3.8.3 Tools/software

Within Rhino/Grasshopper a number of plugins are available which can be used to perform an iterative optimization loop. Standard (and the most common) tools are Galapagos and Octopus. Both tools require a set of variable inputs in the form of sliders which are randomly combined in order to find the optimal value for the objective, referred to as the 'fitness'. Whereas Galapagos is limited to a single-objective purpose, Octopus allows for optimizing multi-objective cases.

As mentioned before, the possibilities with Rhino/Grasshopper and its large variety of plugins is continuously in development. A large active community is supporting the application through discussion forums and platforms where anyone can publish their self-developed plugin. This open character makes the application very accessible.

This also entails that for the same purpose, multiple plugins are available. Some are more established and widely supported than others. For the purpose of this research, a number of structural analysis tools are possible. The most known plugin, which can run within the Grasshopper environment is Karamba3D. Karamba3D offers a wide range of optimizations for especially trusses, frames and shells. In combination with form-finding tools Karamba3D is highly suitable. As it runs without an interface to other applications, the calculation time is considerably lower compared to plugins with an interface to external FEM-applications. Since the application area is more directed to other fields of construction, there are some limitations. There is no option for (elastic) line supports, and surface loads cannot be applied in a triangular shape (only constant loads).

Some other, less known structural engineering plugins without an interface show the same limitations. To make use of more extensive structural analysis possibilities, plugins are available which create an interface to more advanced FEM-applications. One of the plugins is Koala. Using Koala components, a structural model containing input for the geometry of the structure, cross-sections, supports, loads and load combinations can be exported to SCIA Engineer through an XML file extension. A structural calculation can be automatically actuated and results may be retrieved into the Grasshopper environment. However, the supporting community behind this plugin is very limited, and the support team from SCIA Engineer seems to be inactive.

A more active and developing tool can be found for structural analysis software RFEM. Within the latest version (RFEM 6) interfaces to Rhino and Grasshopper are implemented by default. A wide range of options is offered, and the tool is still in development. Unfortunately, one of the functions that is still in development is the retrieving of calculation results, which is essential for creating an optimization loop.

The RFEM 6 to Grasshopper interface has been developed in cooperation with the engineering firm Bollinger+Grohmann and responsible developer Diego Apellániz, which have (independently) developed a plugin[33] for RFEM 5 named 'Parametric FEM Toolbox'. Most of the functions available in this plugin are taken over for the RFEM 6 interface, but at the time of this research (and for the purpose of this research) the plugin for RFEM 5 is still more elaborate, mainly due to its function to retrieve calculation results from RFEM 5 back into the Grasshopper model. For this reason it is decided to perform this study with RFEM 5 and the plugin 'Parametric FEM Toolbox'.

Chapter 4

Conceptual design

In the variants study, some radical different forms are compared to the more common design options. By rating their performance based on construction speed, the nuisance for the environment, the sustainable character, the costs and the robustness of the structure, the variants are being weighted. As the municipality strongly prefers a stiff and robust solution, this criterion has been given a higher weight. Based on that preference, the variants which are based on an L-shaped structure are marked as the best performing options. From the ratings it can also be stated that the material use (reflected in the score for sustainability and costs) and the construction phasing (construction speed and nuisance) are the downsides for these options. On both sides room for improvements is available. Within this research, the focus is placed at reducing the material use for L-shaped structures, bearing in mind the boundary conditions from the context of renovation projects within the urban environment of Amsterdam.

By looking deeper into the options and the effect of structural shapes, a more efficient solution (resisting the loads with a reduced amount of materials) is sought for. Starting with boundary conditions and the assumptions that are being made, different loads carrying mechanisms are being considered. The option which promises the most improvement in material use is being chosen to be worked out in the parametric design model.

4.1 Design considerations

In this section the considerations for choosing the variant to be worked out are being presented, followed by an elaboration of that design concept. The main design aspects for this type of structure are being explained, together with the assumptions that are done for working out a structural model in the FEA software RFEM.

4.1.1 Boundary conditions

Due to the density of the urban context, a lot of limitations can play a role in the design considerations. Some of these boundary conditions can be assumed as inviolable, while for others some tolerance is available. To list the most important boundary conditions and grading them in accordance with the tolerance of overstepping, the freedom in the design space can be better defined. With a wider perspective more options become available and better results with respect to the objective could be found.

In table 4.1 the most common boundary conditions are listed and graded by a number from 0 to 3, for which the lowest grade represents an unavoidable boundary condition and the highest grade is given to an aspect with minor impact to the design space freedom. Also boundary conditions related to the construction are mentioned. However, as the focus in this research is placed at the design, these aspects are given a relatively lower grade.

Design-related boundary conditions		
Geometric limitations	0	Limited space behind the wall
	0	Limited space in the canal
	0	Canal depth about 1.5 to 4.0 meter
	0	Retaining height about 3 to 6 meter
Functionality	1	(Vertical) border between land and water
	1	Guarantee of a soil tight barrier
	1	Loads from urban traffic
	1	Monumental value; preservation of the appearance
	1	Presence of trees
	2	Presence of cables and pipelines
	3	Opportunities for other functions of the quay
	3	Presence of houseboats
Construction-related boundary conditions		
Fast construction	1	Using a standardized design
	1	Structural remains of the existing structure
	2	Avoiding the use of a construction pit
Nuisance	2	Limiting the area of construction works
	2	All works constructed from the water
	2	Limitations due to nautical management
Supply	3	All supply over water
	3	Limiting the size of elements and equipment

Table 4.1: List of boundary conditions and the assumed level of tolerance (0 = Unavoidable, 1 = Hardly avoidable, 2 = Some tolerances, 3 = Side considerations)

4.1.2 Workspace

From a simplified perspective, the required solution has to be placed within certain geometrical boundaries. A difference in height of the soil (retaining height) and the requirement to use that same space for other functionalities (mainly traffic, but also additional functions like trees, cables and pipelines) requires for a structure which has the capability to resist the loads and to transfer them to the subsoil.

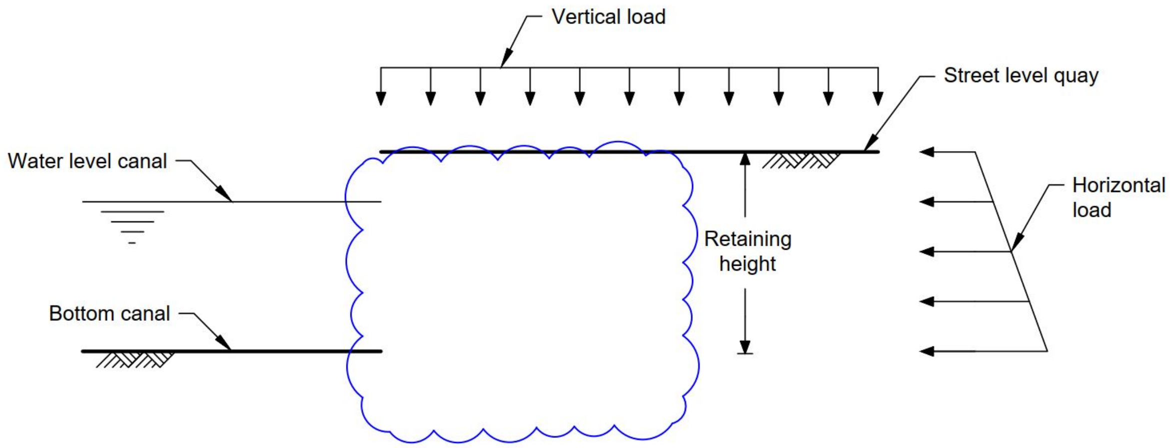

Figure 4.1: Schematic representation of the workspace

4.1.3 Expectation of loads

From the two sources (soil and functions of the quay) of loads, two load cases are being considered. Assuming a uniform soil type, a gradual increase of vertical and horizontal loads results from the soil. The ratio between the vertical weight and the horizontal effect that result from it is defined by the soil pressure factor. As the resultant horizontal load is always acting in the direction of the canal, and the structure will deform in that same direction, the range of the soil pressure factor for the assumed back-fill sand will be between the value for an active soil state (K = 0.33) and a neutral soil pressure state (K = 0.5). Assuming a conservative value of K (0.5), the horizontal effect due to the weight of the soil is for every location in the soil half the vertical soil pressure.

Next to the weight of the soil, a vertical top load is taken into account which is assumed to be a constant uniform distributed load of 20 kN/m^2 (covering for Eurocode: Traffic load model 1 and all other loads on the quay). Assumed as working over the full width of the quay, the soil pressure that result from the top load can be assumed to remain constant over the for every location in the soil. With the assumed soil pressure factor, the lateral soil pressure of 10 kN/m^2 can also be assumed as constant over the depth.

At any height in the soil, the lateral stress remains half of the vertical stress, which means the resultant load has a constant direction. By changing the height and width of the structure, the amount of collected loads can be chosen in such a way that the supports are best able resist them. Regarding the piles, which are the supports of the top structure, the most efficient way of loading them is by axial compression. To achieve that, the resultant (collected) loads should be in the same direction as the piles.

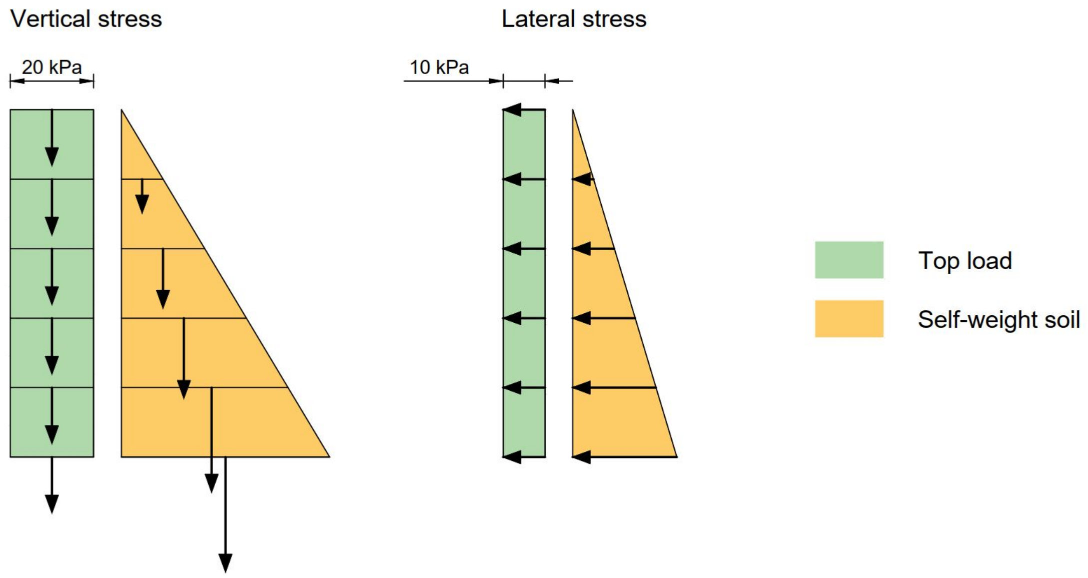

Figure 4.2: Expectations of vertical and lateral stresses

As mentioned before, the maximum inclination of the piles is assumed to be 5:1. By giving the structure a height-to-width ratio of 2:5, the directions of the resultant loads and supports are coinciding. Although the length of the piles are slightly larger (about 2%), the bending moments on the piles due to an eccentric loading are largely reduced which could allow for more slender piles. Although it should be realized that the resultant load direction is dependent on the assumed soil pressure factor, the dimensions of the structure could be chosen in such a way that for a range of K-factors the bending moments on the piles are being within the limits of the pile capacity.

4.1.4 Load carrying mechanisms

Having specified what range of loads the structure needs to be resisting and what design freedom is being offered, different structural mechanisms for resisting loads could be considered. Based on the boundary conditions, some (considered to be) efficient load carrying mechanisms are already being eliminated. For example: The most efficient solution to provide stiffness and to take out a bending moment from the system is to create a horizontal support. This could be an anchored tensile solution at land-side or a compressive support or link at the canal-side. However, due to adjacent buildings an anchored solution is usually not permitted at land-side and a horizontal support at the canal-side will obstruct the passage in the canal. On both sides of the design space, limitations are present which prohibit the use of a lateral support.

Divided into four schematic concepts, different load carrying mechanisms are presented.

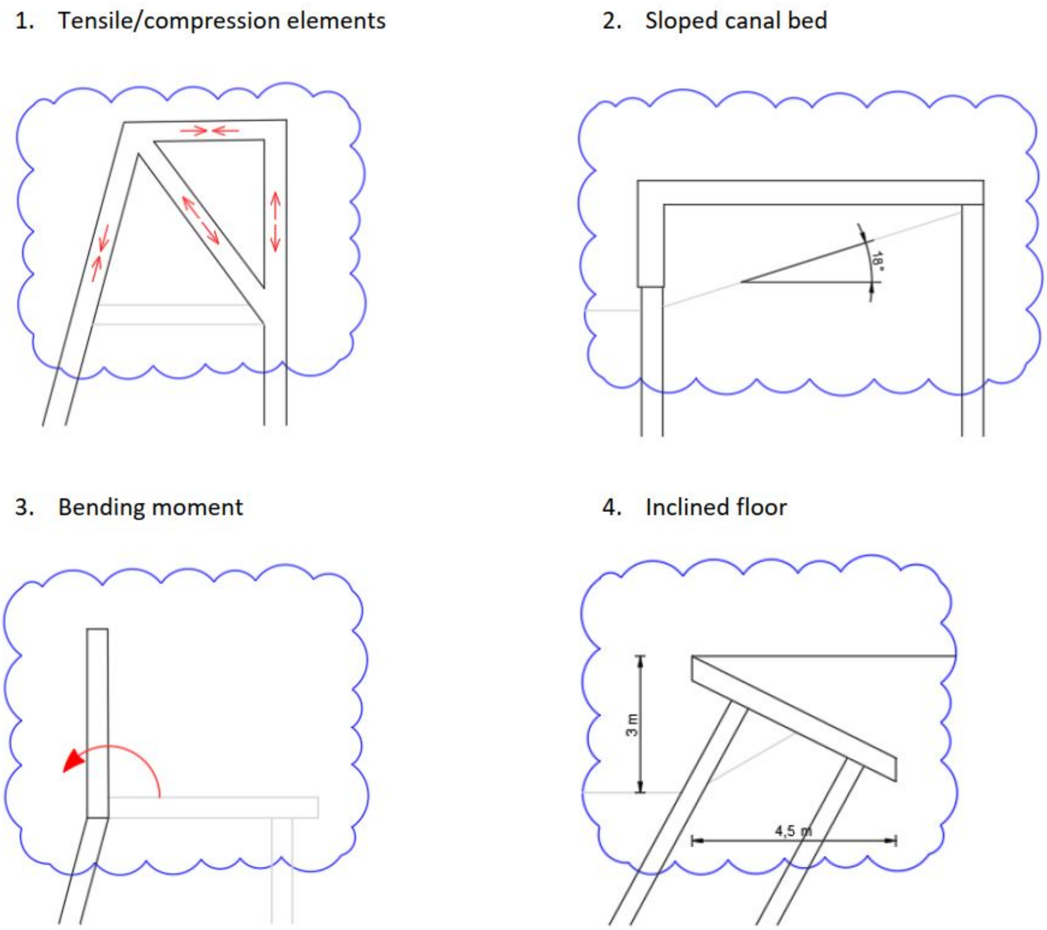

Figure 4.3: Different load carrying mechanisms

1. To avoid bending moments and shear forces, a truss-type of structure could be used. By releasing the rigidity in the connections, the elements can be assumed as only axially loaded. By creating a lever arm between the compressive and tensile elements, the bending moments (due to horizontal loading) can be omitted from the system.

   In the practical context this system would be complex to construct. As the structure should also function as a soil-tight barrier, additional closing elements should be used.

2. A sloped bedding of the canal reduces the horizontal loads that are introduced by retaining the soil. When enough space is available the sloped soil bed could be sufficient to stabilize the quay. But for inner-city locations like in the center of Amsterdam usually limited space is available and large top loads needs to be resisted. A table structure above the sloped bed could take the top loads (only vertical) and directly transfer the loads to the piles, avoiding lateral loads to the structure.

   Still behind the structure a height needs to be retained. Although the sloped bed of the canal reduces the retaining height and the lateral effects of the top load are taken away, some type of closed element (for example sheet piles) should be placed behind the structure to stabilize the soil.

3. The load carrying mechanism used for most of the traditional types of quay wall structure is to resist the lateral loads by bending moments, by using a clamped type of element. For a combi-wall, the vertical elements are directly clamped in the subsoil. With an L-wall, the loads are first collected by the top structure (wall and relieving platform) and subsequently divided over two or more pile rows. The spacing in-between the pile rows create a lever arm, which deconstructs part of the bending moment into a couple (parallel forces of the same magnitude in opposite direction).

   Resisting the loads by using mainly bending elements is not necessarily a desirable mechanism (especially for weak subsoil conditions) in terms of efficiency. An efficient bending element would require a large bending stiffnesses in relation to the cross-sectional area, for example an I-shaped section. However, taking the context into account, that would also require additional closing elements in-between the bending elements ('berliner wall' or 'king post system').

4. The most efficient orientation of the piles is parallel to the direction of the resultant load. In this way the piles are mainly axially loaded and the bending moments on the piles are being reduced. By collecting only the loads that are required for matching the direction of the piles, the floor can be made perpendicular to the direction of the loads, functioning as a wall and a floor at the same time. The inclined floor collects and directly transfers the load to the piles.

   Although the inclined floor efficiently deals with the loads, it required the piles to be very inclined or the floor to have a large width. For both aspects limitations are encountered.

For option 1 and 2 fitting in the mechanisms in the contextual requirements and limitations will be very complex. For option 3 and 4 some variations or improvements could be possible in order to become more structurally efficient.

Option 3

In this option a clamped vertical element (sheet pile, combi-wall, diaphragm wall) retaining the soil is being considered. By resisting horizontal loads large bending moments and shear forces are expected in the structure, meaning the structural element is mainly used for its bending capacity. To reduce the horizontal load on the structure, a relieving platform could be attached. Next to benefiting from the counter effect in terms of rotation resulting from the vertical component of the load in the soil, also the shear load and bending moments in the structure are decreased.

As the platform collects the vertical loads from the soil, the earth pressure below the platform is reset to zero. This trick can be repeated in the form of a wall with multiple relieving shelves. A study[34](Shehata, H.F., 2016) on the influence of relieving shelves shows the effect on the lateral earth pressure, by comparing a vertical cantilevering element with one shelf, two shelves and without a relieving shelf.

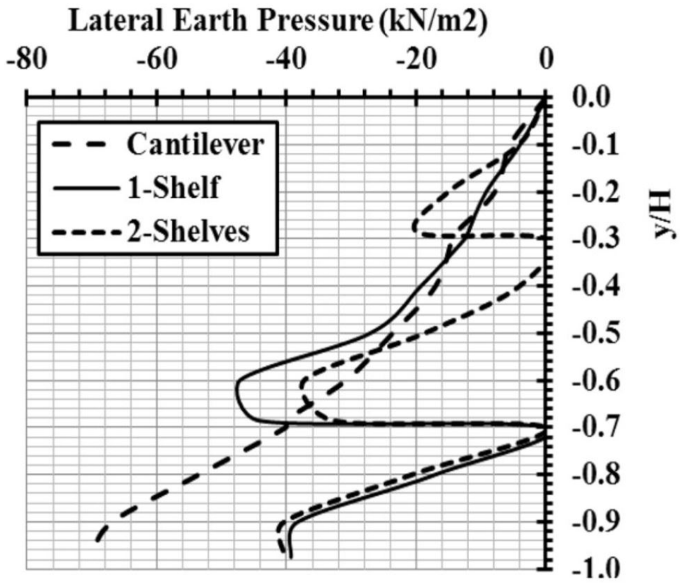

Figure 4.4: Effect on the lateral earth pressure by using relieving shelves[34]

Although the overall lateral pressure on the structure is significantly reduced in comparison to the system without a relieving shelf, the pressure right above the relieving shelves are locally higher. In a study[35](Chauhan, V. and Dasaka, S., 2018) on the performance of relief shelves for a rigid retaining wall a reduction of the lateral stresses up to 23% is indicated, depending on the location and dimensions of the structure. Although with this solution the bending moments can be considerable reduced, the system still needs to resist a large bending moments within one vertical element.

By supporting a relieving platform on (at least) 2 rows of elements, the bending moment resulting from the lateral loads can be decomposed in tensile and compressive reaction forces. In this way, the structural elements are more axially loaded and can be utilize both the axial and bending capacity in a more effective way. Although it does require additional elements, the gain in structural efficiency could result in better solutions in terms of the total material use.

Option 4

In option 4 only 1 element functions both as the wall and the slab. The orientation of the slab is aimed at collecting the loads from the soil and distribute them to the piles. To most effectively bring the load towards the soil, the floor should be oriented perpendicular to the direction of the resultant load. If the piles are subsequently perpendicular to the floor the piles are (almost) exclusively axially loaded, taking out shear forces and bending moments in the top of the piles.

As the resultant load in the soil for the maximum pile inclination is assumed to work at a 1:5 inclination, the horizontal width of the slab should be 2.5 times the retaining height. For urban application this could not be achievable, as a limited width is available and sometimes large heights should be retained. By creating a vertical ending of the floor, more height can be reached. In this way also bending moments are introduced, but as the earth pressure in the top part of the soil are lower this could be effective, as is shown in the figure below (in case of around the maximum retaining height of 5.5 meter).

4.1.5 Selection of design option

By combining all the considerations from the variant study and the assessment of the efficiency (low material use with respect to the required resistance) for different load carrying mechanisms, the preferred solution is being sought in-between the variant with an inclined floor and an L-shaped structure. As mentioned in the variant study, the robustness of the solution has a high weight in the trade-off. Resulting from that criterion, the solution should have at least two rows of elements founded in deeper soil layers. Considering the list of boundary conditions, this option can be seen as the best fit for its purpose, by trading off the possible conflicts with higher rated boundary conditions (predominantly the preservation of trees) with respect to the structural requirements.

Due to the high predictability of the direction of the resulting loads acting on the top structure (the inclined floor, or the floor and wall in case of an L-shaped structure), the structure could be optimized for the expected loads. By adjusting the geometry and orientation of the structure, the amount of weight of the soil can be collected for which the structure (mainly the piles) are most efficiently loaded, implying mainly a better ratio of axial loads and bending moments.

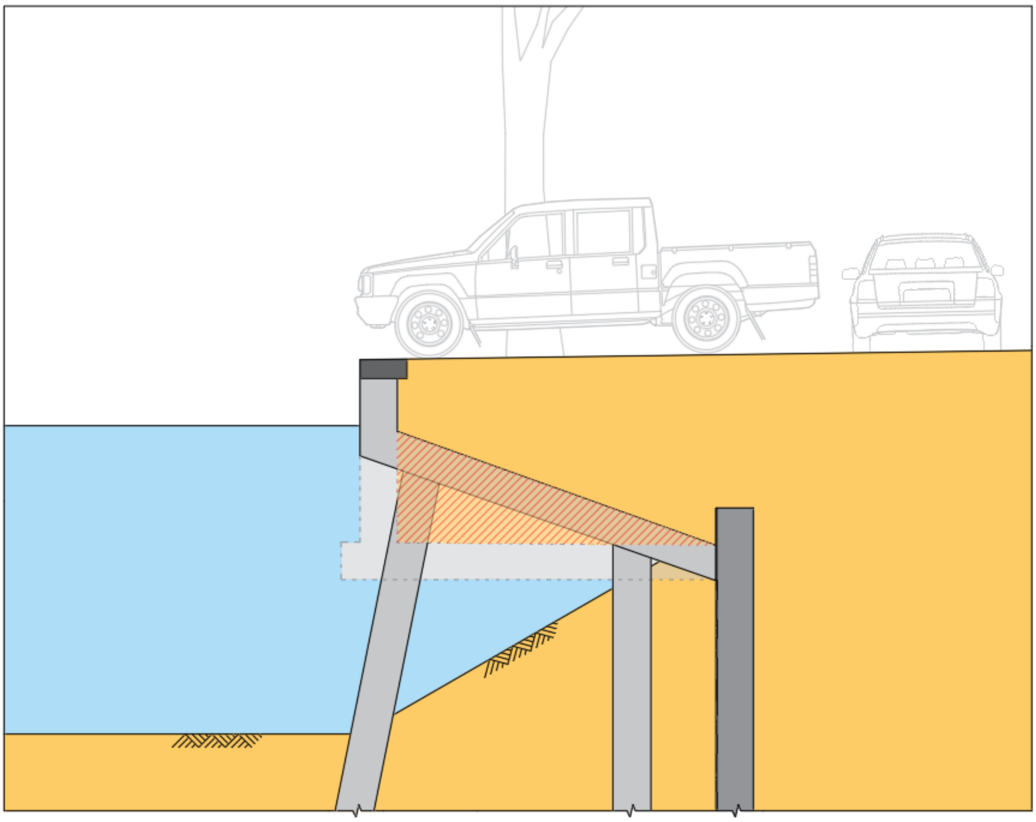

Figure 4.5: Reducing the soil load by inclining the floor

4.2 Working out of the design option

By selecting the preferred design option, the intended structure is more elaborately being studied to map the critical aspects and weak spots of the solution. Along with some hand calculations, more detailed calculations are performed with RFEM. Below, the main design aspects for this type of structure are being explained, together with the assumptions that are done for working out a structural model in the FEA software RFEM.

4.2.1 Schematic representation of the structure

In setting up the representation of the structure in RFEM, a number of assumptions need to be made. The aim has been to keep the structural model as effective as possible (without being too conservative or too optimistic). The structure can be divided in three parts: The wall, the floor and the piles. For each part the assumptions are being presented and the most critical aspects are briefly mentioned. Subsequently, the representation of the considered loads are being elaborated.

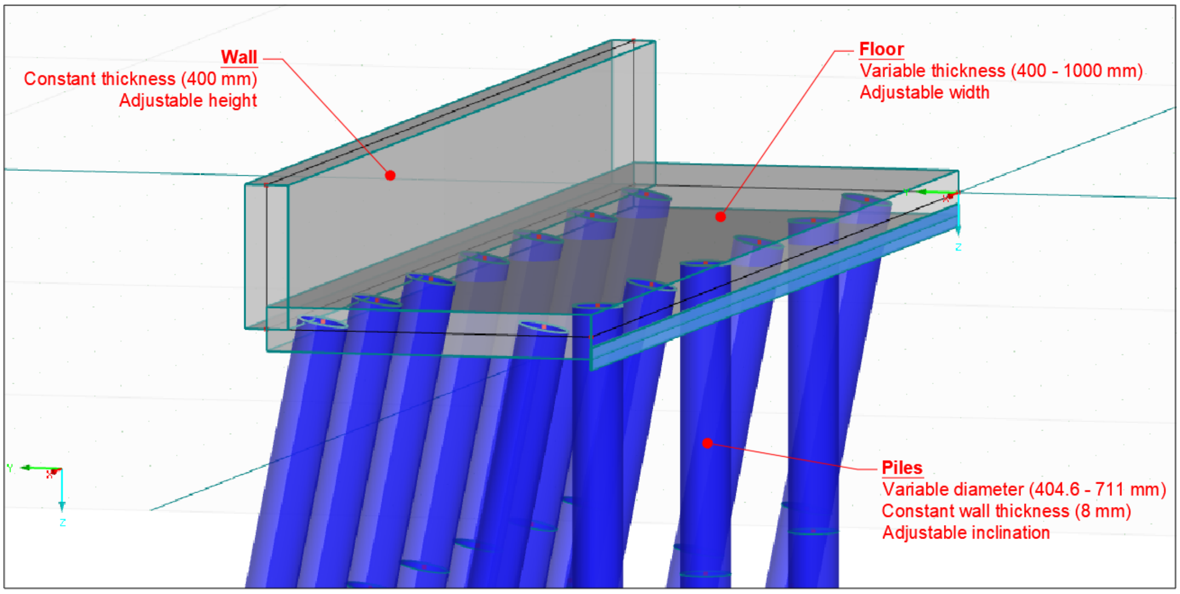

Figure 4.6: Schematic model of the structure in RFEM

4.2.2 Segmentation of the structure

The schematic structure is set up for a segment of the quay wall. In practice, the total quay is usually being split up in segments of about 25 meter. In-between the segments, a small expansion joint is present which allows the structure to freely expand. As it involves an underground structure (without exposure to direct sunlight), expansion due to temperature effects is limited. To guarantee the integrity of adjacent segments, dowels are being applied in-between the walls and the floors. However, in the schematic representation of the structure this additional loading in the case of unequal loading of segments is not taken into account.

Instead of a segment of 25 meter, the structural model considers a segment of 6 spans and 2 end spans. In the longitudinal direction 7 rows of 2 piles are being modeled. The end spans are taken as a factor of the mid spans to generate a more equal distributions of bending moments at the supports. This factor is derived from the formulas (forget-me-nots) in mechanics for continuously supported structures. A span in the middle of the segment can be represented as being clamped at both supports, having a bending moment above the supports of $1/12ql^2$. The end spans have one free end. For approaching to find the same bending moment above the edge support the end span can be represented as a one-sided clamped element, for which the the bending moment above the support is $1/2ql_{end}^2$. By making both bending moments of equal size, the end span factor of 0.408 is found ($l_{end} = 0.408l$). This is only a simplified approach. For finding a distribution of field spans for which the bending moments above the supports are exactly equal, the moment-area theorem could be applied. The considered end span factor appears to be a good estimation for having the support bending moments of about the same magnitude.

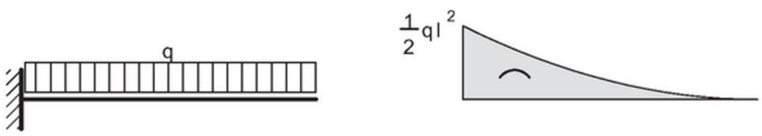

Figure 4.7: Schematic model for one-sided clamped element

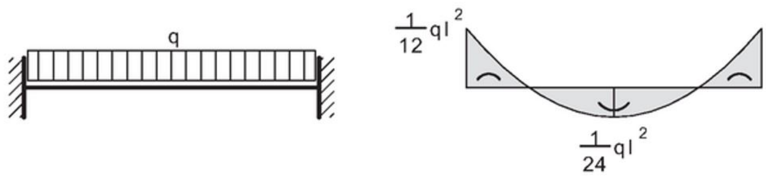

Figure 4.8: Schematic model for two-sided clamped element

4.2.3 Wall

The wall is modeled as a surface which is continuously clamped to the floor, representing the case for a monolithic connection. To approach this behavior in reality the wall reinforcement should be anchored in the floor, meaning protruding reinforcement at the time of casting the floor.

The thickness of the wall is simply taken as 0.4 meter, which appeared to be sufficient for reasonable heights of the wall. Of course when the structure is placed at a deeper level, the wall thickness could be considered. The base reinforcement in both directions is taken as diameter 12 and a spacing of 150 mm. As the largest bending moments are expected around the longitudinal axis ($m_{y,D}-$), the vertical reinforcement is assumed to be in the outer layer.

The assumed base reinforcement is more of a practical function instead of being structurally required. Up to a wall height of about 2 meter the base reinforcement is structurally sufficient to resist the bending moments. However, the most critical structural aspect for the wall are the forces acting in the longitudinal direction ($n_{x,D}$). For smaller heights of the wall it can be considered as a beam. At the locations of the piles the floor acts as a support for the wall, but at mid-span sagging of the floor causes an additional vertical loading on the wall. As a result, the beam action of the wall causes tensile forces in the top part of the wall at the supporting locations. To resist these tensile forces additional reinforcement should be placed in the zone above the piles. For larger heights of the wall or a larger thickness of the floor (higher stiffness, lower sagging deformations) this effect is fading away.

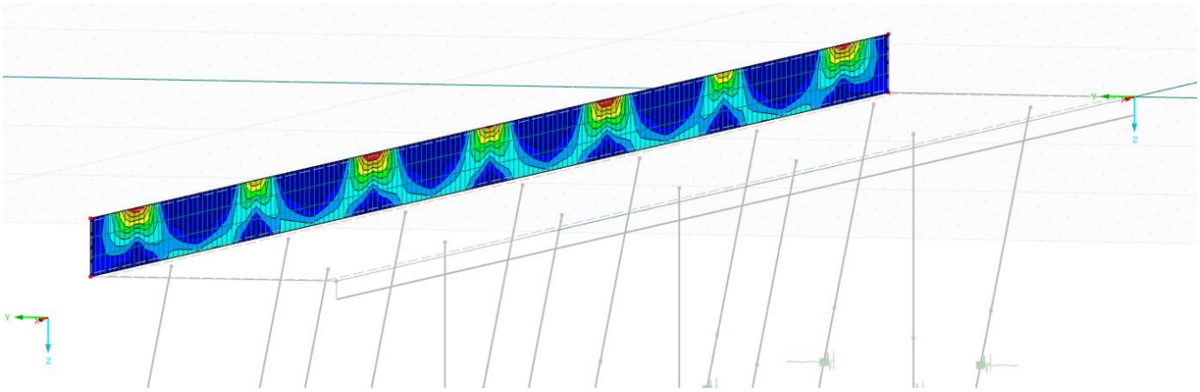

Figure 4.9: Effect of the beam-like behavior of the wall ($n_{x,D}$)

4.2.4 Floor

The floor is modeled as a surface which is at one side continuously connected to the wall. At the locations of the piles fully clamped connections are being assumed. To approach this behavior in reality, the pile reinforcement should be continual and sufficiently anchored in the floor. At construction the reinforcement net should be protruding from the piles so it can be used integrated in the later to be cast floor.

In the schematising of the floor the assumption is being made that the top of the floor remains constant. For increasing the thickness this means the floor only increases from the bottom and the soil layer above the structure is not being affected. The center-line of the floor is considered at the top of the floor.

At land-side the floor is being cast against the sheet pile, from which (after the temporary struts are taken away) a horizontal force is being taken. To model this load an eccentric member is being modeled at a distance of half the floor thickness from the top of the floor. To prevent this member from contributing structurally, it is given a material with negligible low properties.

4.2.5 Piles

The piles are being modeled as hollow circular sections which are clamped connected to the floor. The effect of concrete inside the piles is not taken into account, which is a conservative approach considering the increasing effect on the stiffness of the piles. The self-weight of the concrete filling has a slightly favorable effect on the lateral deformations, but also a slightly negative effect on the vertical displacements.

Inclination of the piles

As the available width is usually limited, the resultant direction of the loads is almost always more inclined than the resultant direction of the supports. To decrease the bending moments on the piles, as much piles as possible should be inclined up to the maximum inclination (assumed to be 5:1). However, too much inclined piles makes the structure vulnerable for deformations. By placing piles in different directions the structure becomes stiffer. The balance needs to be found for which the maximum piles are inclined while the deformations are just within limits. From the interpretation of results for different retaining heights, a good assumption (for which neither the horizontal or the vertical deformations are easily governing) appeared to be a configuration in which all the piles at the canal-side and 4 out of 7 of the piles at land-side are maximally inclined, while the other piles at land-side are vertically placed.

Corrosive losses

Due to corrosion a sacrificial thickness of the steel casing should be taken into account. As mentioned before, different approaches could be taken. To be conservative, a minimum wall thickness for screwing in the piles could be used, after which the steel casing is being assumed as 'lost' and only the concrete core is assumed to be structurally active. The structural contribution is not taken into account as at the end of the lifespan the casing is assumed to be fully corroded.

On the opposite side, when the steel casing is considered to be structural functioning in the end of the life span, the sacrificial thickness layer should be added to the required structural thickness of the casing.

For this research, a middle approach is being assumed. The steel casing of 8 mm thickness is used at the time of construction, which is assumed to be sufficient for construction. Over the lifetime of 100 years, the sacrificial layer of 4.3 mm is assumed to be corroded. The uncorroded thickness of 3.7 mm is assumed as contributing for the bending stiffness of the piles, which is used for the buckling capacity of the piles.

Soil stiffness

The stiffness of the soil follows from site investigation and has an effect on the reaction of the structure. For this research a soil profile of a quay wall construction project in Amsterdam (Nieuwe Herengracht) is being assumed, which can be considered as an average soil composition for the central part of Amsterdam.

For the vertical stiffness an estimation is made by assuming an average stiffness of 125 MN/m. In reality, the to be modelled stiffness is resulting from the deformation of the pile, which depends on the loading of the pile. For large vertical loads a large deformation of the pile toe can be expected, for which the related stiffness value becomes lower. When the stiffness of the piles is much lower relatively to the stiffness of the floor, more redistribution of loads will take place in the floor, for which the internal forces in the floor become higher and the bending moments in the piles are more evenly distributed. The other way around, for a high relative stiffness of the piles the loads are more directly transferred to the piles, for which the pile forces are less evenly distributed and the internal forces in the floor are lower.

The lateral soil stiffness is modelled as springs in both horizontal directions, concentrated at the center of each different soil layer type. Reason for the soil stiffness not being modeled as an elastic line support or a member elastic foundation is that the first option is not available in RFEM (only for lines belonging to a surface or a solid) and the latter option is not provided in the used Grasshopper plugin. To link different stiffnesses to different heights of the piles, the piles have to be segmented in the same heights as the different soil layers. To link the right stiffness with the right segment height a constant order of segment dimensions and stiffnesses properties should be maintained in processing of the data in Grasshopper.

Soil layer type	Level top layer (NAP)	Level top layer (z) (From bottom floor)	Lateral stiffness k_h (kN/m^3)
**Clay*	-1.3	0.0	4.402
Peat	-4.1	-2.8	4.690
Sandy clay	-5.5	-4.2	3.668
Loose sand	-6.9	-5.6	18.976
Peat	-9.3	-8.0	4.690
Sandy clay	-9.8	-8.5	5.135
Peat (basisveen)	-12.3	-11.0	27.356
Sand (first)	-12.7	-11.4	56.929
Sandy clay	-14.3	-13.0	14.672
Fine sand	-15.6	-14.3	56.929
Sand (second)	-20.3	-19.0	83.496
* Top clay layer not included in the model			

Table 4.2: Assumed soil profile and corresponding lateral stiffnesses

The presented lateral stiffness values in table 4.2 are based on a pile diameter of 508 millimeter and determined based on Ménard's method. According to this method, piles with a diameter larger than 600 millimeters are resulting in higher stiffness values. However, this increased stiffness for the larger pile diameters (610 and 711 millimeter) is not taken into account. The k_h-values are multiplied with the related diameter of 508 millimeter and the length of the pile segment in a soil layer, to get the spring stiffness (in kN/m) of a concentrated elastic support at the middle of that soil layer.

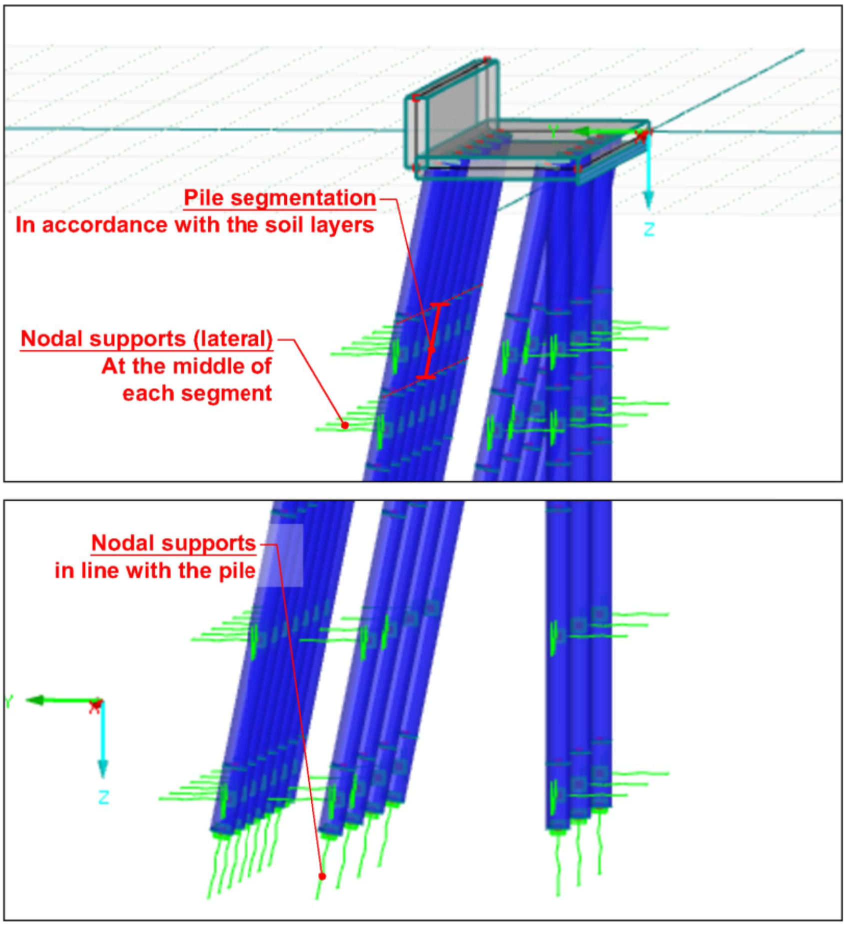

Figure 4.10: Schematization of the supports in RFEM

4.3 Schematic representation loads

For the simplified model only three load cases are being modeled. As mentioned before, the amount of load combinations have a large impact on the calculation time. For that reason only one load combinations is being used for each calculation. The loads in this load combination are entered in characteristic state (without any factors) and divided in three load cases.

The first part of the permanent loads are considered in load case 1. In this load case the self-weight of the structure is automatically generated by RFEM based on the specified materials and cross-sections. In load case 2 the other permanent loads are taken into account. It consists of the weight of the soil, for which the lateral component follows from the assumed soil pressure factor, and the pressure from the water in the canal. Within load case 3 the variable loads are included. As a result of the top load of 20 kN/m^2 both vertical and horizontal surface loads are acting on the structure. As has been reasoned in sectoin 3.4.8, top load is considered to be covering for all separate loads on top of the quay, which helps in reducing the amount of load combinations.

4.3.1 Load case 2

The wall is modelled as a vertical surface at which only lateral loads are acting. The permanent loads are all modeled as triangular surface loads. The load as a result of the soil is zero at the top of the wall and increases by the saturated weight of filling sand, which is taken as 20 kN/m^3. On the opposite side of the wall, a counter-acting triangular surface load is modeled representing the water pressure from the canal. Depending on the input, the load starts acting from a certain height below the top of the wall and gradually increases by the weight of 10 kN/m^3.

The floor is subjected to the vertical weight of the soil. Depending on whether an inclined floor is used, the load is modeled as a block shape (constant magnitude: height of the wall * weight of the soil) for a horizontal floor or a prismatic shape (up to: [height of the wall + height of the floor] * weight of the soil). In upwards direction, the pressure of the water is taken into account. In the same way, the shape of the load depends on whether the floor is inclined, with a minimum value equal to the maximum water pressure at the wall and increasing over the vertical height of the floor (up to: [height wall - water level depth + height floor] * 10 kN/m^3)

For the case of an inclined floor a prismatic shape is expected as lateral loading, which acts on the projected area of the floor in Y-direction. It starting with the value of the lateral loads at the bottom of the wall and gradually increases over the height in Z-direction. The same counts for the lateral load in opposite direction as a result of the water pressure. Depending on the assumed water level (usually above the floor), the prismatic of triangular shape starts from the maximum value of water pressure on the wall and increases over the height in Z-direction. In the same way this load is acting on the projected area of the floor in Y-direction.

The load from the sheet pile, acting on an eccentric beam at land-side of the floor, is modeled as a uniform member load. In section 3.4.4 the substantiation for the assumed load magnitude is given. Based on a simplified approach, a formula has been derived to make the magnitude of the load depending on the height for the top structure (wall and floor) and the height between the floor and the canal bed (gap height). The assumed formula for the sheet pile load as a result of the soil load is:

$$F_h = 3.389 \cdot H_{gap} + 6.8 \cdot (H_{wall} + H_{floor})$$

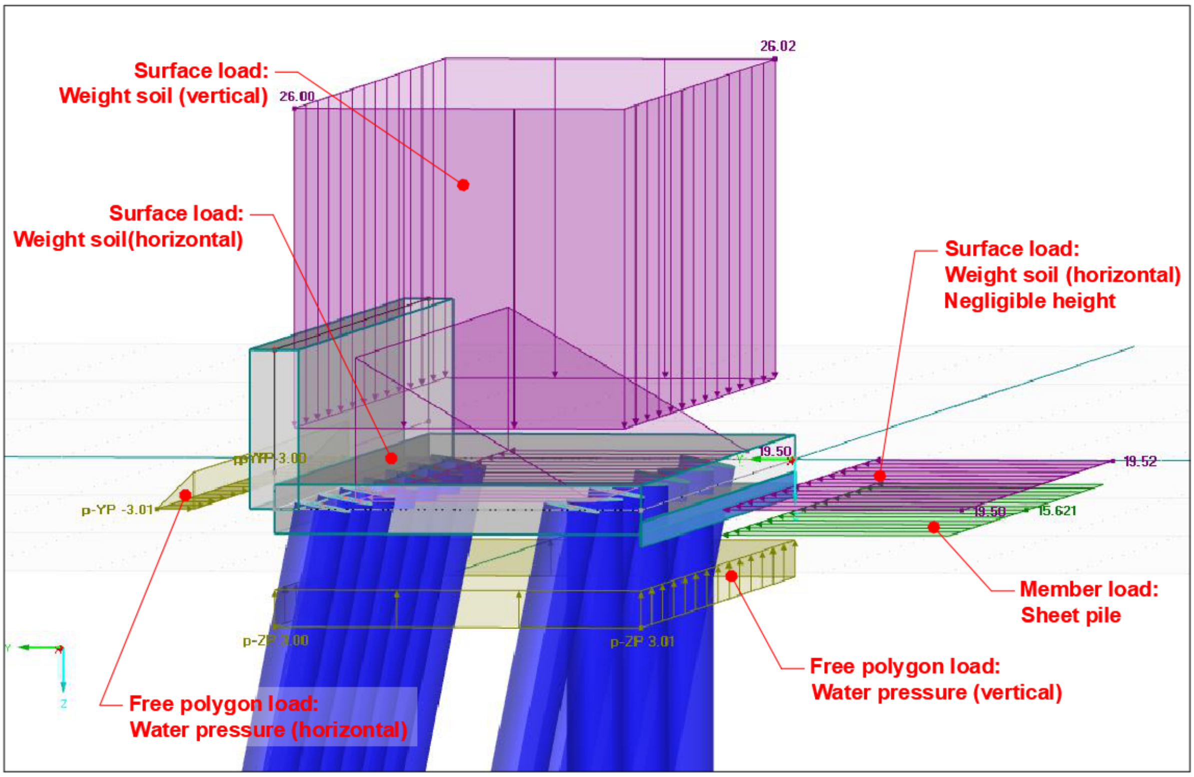

Figure 4.11: Schematic representation of loads in Load case 2 (RFEM)

4.3.2 Load case 3

From the top load, the wall is subjected to a block load with a magnitude of [top load] * [soil pressure factor]. A block load with the same magnitude acts in horizontal direction on the floor in case of an inclined floor. Vertically also a block load is acting on the floor, with a constant magnitude equal to the top load.

As a result of the top load, also a contribution to the interaction force between the sheet pile and the floor is acting. In section 3.4.4 a substantiation of the assumptions are presented. A member load is applied to the eccentric beam with an assumed magnitude given in the next formula:

$$F_h = 11.4 + 6.722 \cdot H_{gap} + 0.933 \cdot (H_{wall} + H_{floor})$$

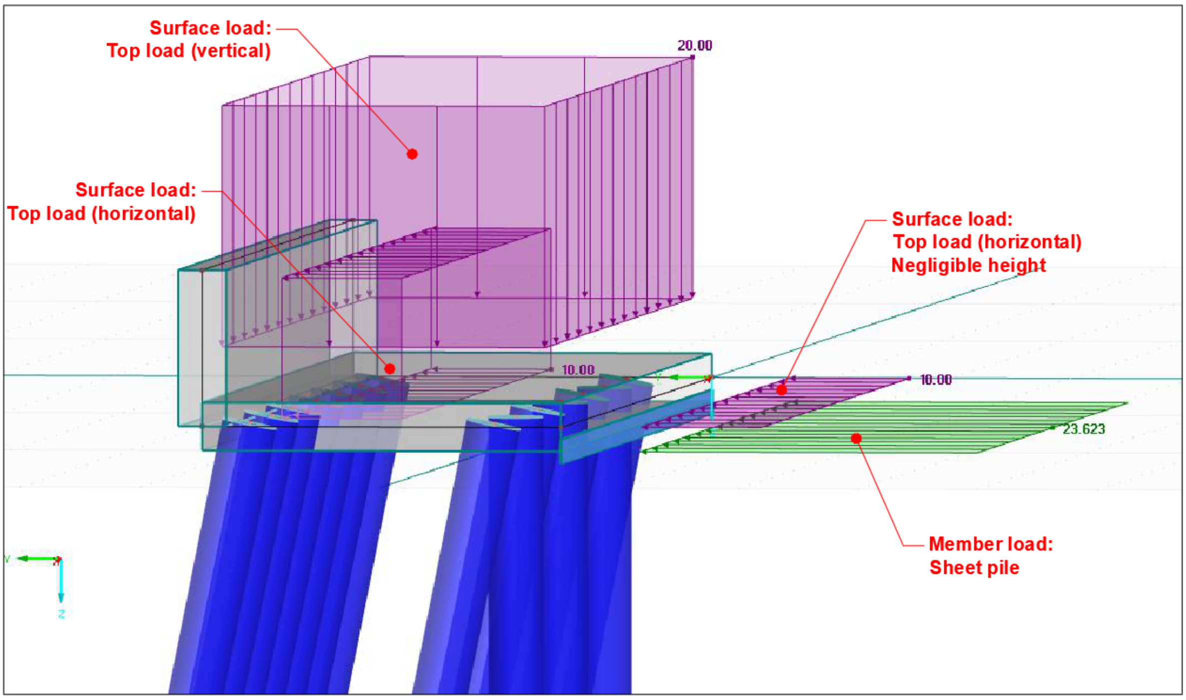

Figure 4.12: Schematic representation of loads in Load case 3 (RFEM)

4.3.3 Mesh size

Within RFEM the target length for the mesh elements should be defined. Common practice is to define the mesh size as large as possible while still generating reliable (meaning accurate) results. Especially for a parametric setup, the definition of the mesh size directly influences the calculation time. For the purpose of this research and the range of dimensions, a target length of 25 centimeter is assumed to provide sufficiently accurate results.

By comparing the results for different mesh sizes, the main effect is found in the peak bending moments in the floor at the locations of the piles. However (as will be described in section 5.4, the bending moments in the floor around the column will be averaged, reducing the effect of the mesh size on the peak values.

Chapter 5

Parametric model

By developing a parametric model different geometrical solutions are being generated which can be compared to find the best option for each locational set of input parameters. To make the model workable a number of assumptions, choices, considerations and limitations are being made. In this section, the outline of the parametric model and all the conditions are being explained.

5.1 Methodology

The effectiveness of this model is most valuable in the preliminary design stage. Based on the location of an intended quay wall structure, the model can run through different geometrical options in order to find the optimal set of variable parameters. The optimal solution is being classified as the set of parameters for which the objective shows the best result. To guarantee the structural feasibility of the solution, a number of structural checks are linked to the found results.

Out of the total amount of results, the combinations for which unity checks above 1.0 are being found can be discarded. The best value for the defined objective can be marked as the optimal solution.

5.1.1 Workflow

Base geometry Setting up the geometry of the structure begins with the start and end points of the floor. A total length of 6.816 times the span (6 mid spans and 2 edge spans with a factor 0.408, substantiated in section 4.2.2) is defining the boundaries of the floor. The nodes at the canal-side are elevated in the case of an inclined floor, and copied for defining the boundaries of the wall. By creating surfaces from the nodes, the geometry of the wall and floor are set. By linking the surfaces with the definition of their thickness and material, it can be connected as input to the exportation component.

Pile locations In dividing the length into 6 spans and 2 end spans, the locations of the piles in longitudinal direction are found. In this direction, the ratio of spans remains constant. In the perpendicular direction of the floor, the locations of the pile rows can be given a certain estimated ratio of the width. Their final locations are given as a result from optimization 1. After the optimal supporting locations in perpendicular direction are found, the nodes of the piles are set.

Pile segmentation By defining the depth and inclination of the piles, the end points of the piles are known from which lines are created. As mentioned in section 4.2.2, the piles need to be segmented in accordance to the layers of the soil profile. The heights of the specified soil layers are used for creating horizontal planes. By splitting the lines of the piles in the at the locations of these planes, the pile segments are created. The order of the segments should be carefully maintained. The total of pile segments are linked to the definition of the cross-section and material type to be used as input to the exportation component.

Supports At the end point of the lowest pile segments, elastic nodal supports (springs) are created which are linked to the definition of the pile tip stiffness. Their orientation is rotated to be in line with the direction of the pile.

The defined lateral stiffnesses (k_h in kN/m^3) of the soil layers are multiplied by the length of the segment in that soil layer and the diameter to find the equivalent spring stiffness (in kN/m). As the supports can only be applied to nodes, the additional nodes need to be defined at the center of each segment. The elastic nodal support with the defined stiffnesses and the center nodes of the segments (in the correct order) are used as input for the exportation component.

Loads The schematization of the loads presented in the section 4.3 is done through surface loads (constant or linear increasing over the depth), linear increasing polygon surface loads (for the loads from the water pressure) and member loads (acting on the eccentric member, representing the load from the sheet pile). The linear evolving loads require input for their minimum and maximum value, a definition of the evolving direction and nodes to specify the location of the minimum and maximum value.

Software conditions Along the development of the parametric model, a number of challenges are encountered. In relation to the interface-plugin for RFEM some detours had to be made. One of them needs explanation, as it follows from a miscommunication between the software.

For each calculation loop a temporary file (named after the load combination) containing the calculation results is created by RFEM. Subsequently this file is being read to import the results to the Grasshopper environment. For the next calculation loop an error occurs, as the temporary file is still in use. To enable an ongoing calculation process, a new temporary file should be created instead of overwriting the previous file. This has been resolved by alternating names for the load combinations.

5.1.2 Objectives

Main objective of this research is to minimize the amount of material used per meter quay length. To express the amount of materials in a single unit value, the amount of material is multiplied by the costs of that material to end up with a total cost per meter length. Of course the results could be changing for different inputs of the costs. When for example concrete becomes more expensive better results could be found for smaller spans with a smaller floor thickness.

As the dimensioning of the sheet pile is generally governed by the construction phasing, it is not being considered in this research. That means the calculated costs of material does not contain the contribution of the sheet pile. As the comparison is made for different retaining heights for which the length of the sheet pile is constant, this does not affect the results. However, for a value of the total structural material use, also a contribution of the sheet pile should be taken into account.

5.1.3 In- and output

The input parameters can be divided into locational-dependent parameters, which are assumed to be constant, and variable parameters which define the geometric option of the structure. Although in essence every input of the model can be assumed as a variable, the following aspect are assumed as the locational-dependent constant input and the variable parameters.

The (intermediate) results obtained from the structural calculation in RFEM are being processes within the Grasshopper environment. By interpreting the results they are being converted into unity checks for different structural verifications. The amount of materials for that set of input parameters is being collected and expressed in a total cost, to be included in the output as the main objective.

Input	Output
Locational-dependent (constant)	*Intermediate results (by RFEM)*
Retaining height	Deformed structure
Available width	Surface forces floor
Water level in the canal	Surface forces wall
Soil profile	Member forces
Variable input	*Final output (by Grasshopper)*
Wall height	Unity check deformations
Wall thickness	Unity check floor reinforcement
Floor height	Unity check wall reinforcement
Floor thickness	Unity check punching capacity
Pile spacing (longitudinal direction)	Unity check axial capacity piles
Pile diameter	Unity check bending capacity piles
Pile inclination	Total material use (in €/meter)

Table 5.1: Input and output parametric model

Schematic workflow

A schematic overview of the steps taken within the grasshopper script are presented in figure 5.1. In the top part, steps within optimization 1 are shown by the green arrows. By defining the optimal locations of the supports in transverse direction, the input is being adjusted and the calculation loop for optimization 2 can be started. All required input for the RFEM model is automatically being defined based on the given input, assembled in the correct manner and exported to RFEM, after which the structural calculation is activated. As the calculation is finished, a temporary file is being saved by RFEM containing the results of the calculation. This file is being recognized by the Grasshopper plugin which automatically retrieves the information back into the Grasshopper environment. Subsequently, the results of interest are being read and processed to the desired conclusions of the results; the unity checks for each of the considered design aspects. At the end of the loop, a change of the results is used as a trigger to store the data and to activate the manual iteration, by adjusting the sliders of the input parameters. By defining the range of each input parameter, and the step sizes by which it should be altered, an automated iterative process can be run. Resulting from the output, the optimal variant can be picked for which the optimal geometry of the structure can be defined.

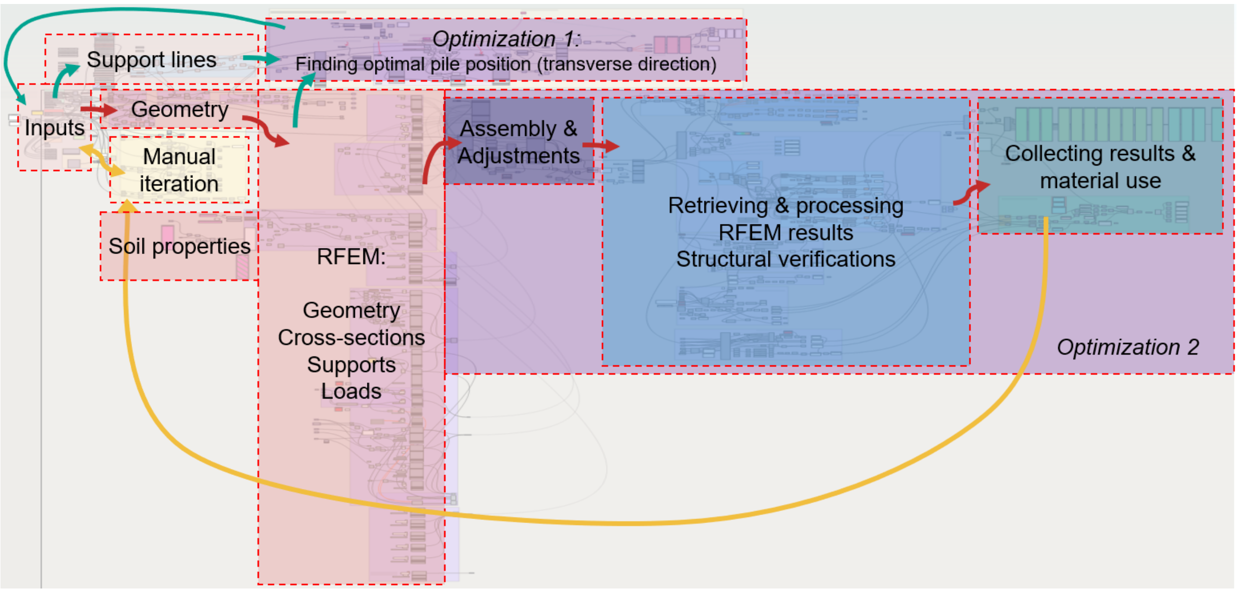

Figure 5.1: Schematic workflow of the Grasshopper model

5.2 Optimization 1

The first step in the optimization process is finding the best locations of the piles in transverse direction of the quay wall structure. Based on hand calculations and investigating the design concept, the pile head bending moments can be considered as the most critical design aspect. To start with optimizing for this critical aspect, other aspects could become governing. That would require optimizing for the other critical aspects until an optimum (in terms of the main objective) is being found and the unity checks are as close to one as possible. However, from the results it appeared that even after optimization the pile head bending moments are still (part of) the most critical design checks. For that reason, focusing the optimization objectives on the pile head bending moments is considered a good approach.

5.2.1 Starting points

The essential part in optimizing is stating a proper objective. With the main objective of this research in mind, the pile head bending moments appear to be the most critical aspect. A logical objective for the optimization of pile locations in the transverse direction of the structure is to search for the support locations at which the bending moments on the piles are being minimized. Next to the minimization of the sum of bending moments, this objective results in an almost equal distribution of bending moments over the two pile rows. Defining this objective for optimization 1 supports the overall objective of reducing the material use (by more efficiently resisting the loads).

However, as a result of eccentric loading conditions this would also result in an unequal distribution of axial loads over the piles. As the scope is focused at quay walls within the dense urban environment, for which a large eccentricity of loads is expected due to a limited width of the structure, the objective of minimizing bending moments is more applicable than minimizing axial loads.

Although it could be considered in a 2-dimensional schematization, for simplicity the geometry and loads in the 3-dimensional model are being used. It is being assumed that the effect of other variable input parameters (mainly the thicknesses of the wall and floor) are negligible for the distribution of bending moments. Along the length of the quay wall segment, two line supports are being modeled which are fixed for bending moments around the longitudinal axis. By defining a range for the estimated locations of the optimal support lines and a mesh size for the division of this range, the Galapagos plugin is used to iterate for different combinations of support lines. For each iteration, the results of the support line bending moment reactions are being retrieved. By converting the result to absolute values and summing all the values, the fitness (objective result) is being found.

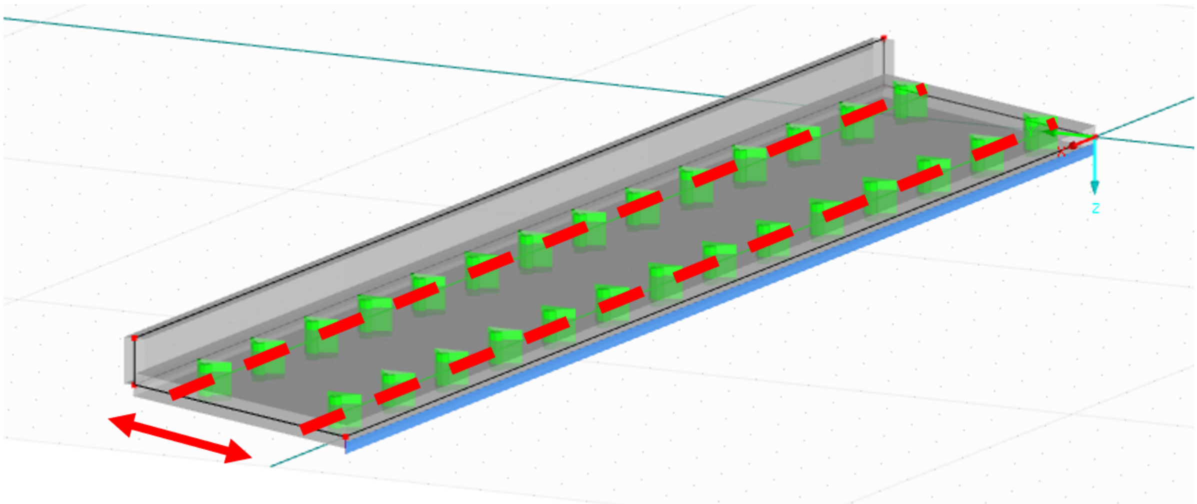

Figure 5.2: Optimization 1; Finding the optimal support lines in transverse direction

5.2.2 Effect

The favorable effect of performing optimization 1 can be seen in the unity checks for pile head bending moments resulting from optimization 2. As the optimal line of support locations is being found for only one specified load combination, different support locations would follow for different load combinations. As the deviation of the resulting direction of loads for different load combinations is assumed to be limited, this approach is considered as a good starting point.

5.3 Optimization 2

After the optimal lines of pile locations in the transverse direction of the structure have been found with optimization 1, the other variable parameters can be considered. For finding the optimal pile locations in longitudinal direction of the structure, all other design aspects (structural verifications) have to be taken into account. Each of them can be considered as an objective.

5.3.1 Starting points

The ultimate objective has been set to minimizing the material use, expressed as total cost of materials per meter length of quay wall structure. As sub-objectives the unity checks for each of the structural verifications are being defined. In general, it could be said that an optimum of material use could be found for the situation in which for different design verifications a unity check as close as possible to 1 is being found. That would mean the structural element is maximally exploited. However, due to practical limitations, a limited set of variable parameters and considering of verifications for stability aspects, this is not achievable but should be considered more as an aim.

Still, this principle holds in the interpretation of the results. Following from the combination of input parameters, a unity check for each considered design verification is being found. By listing all inputs and corresponding results, the input combinations for which unity checks above one are being found can be eliminated, resulting in only the input combinations for which the design is satisfying the structural requirements. The combination for which a minimum material use is being found can be marked as the optimal solution.

5.4 Structural verification

The type of design tool that is being developed in this research is mainly usable in a preliminary design stage. A detailed calculation and verification of each aspect is not yet of interest, but to make sure the design option is within feasible structural limits the most important design checks should be verified by using simplified calculations. Due to the type of loads that needs to be resisted and the design freedom for a structural solution, the expected governing loading are the bending moments. By adjusting the locations of the support lines (as is done within optimization phase 1) the resultant bending moments on the piles could be minimized for a specific load situation. Consequently, in optimization phase 2 the piles are more axially loaded, resulting in a lower unity check for bending and a higher unity check for its axial and buckling capacity. For a different load combination the optimized support lines will also change. For an optimal design option the governing unity checks would be just within the limits for different load combinations. However, as the amount of considered load combinations highly affects the calculation time, the minimum amount of load combinations is being preferred. In the preliminary design stage, a good starting point is to assume only one load combination in frequent state (SLS-frequent) as being governing. The risk in this approach for the assumed type of structure is that the structure becomes vulnerable for deformations. By adjusting the inclination of the piles to the loads for one load combination, the structure could be very prone to a load combination with loads in different directions. However, as the resultant load directions for different load combinations are not much fluctuating due to the largest load contributions being in a constant direction, considering only one load combination is assumed to be a good approach.

Starting point for the structural verification is the assumption that the allowable steel stress is governed by crack width control. Using tables 7.2N and 7.3N of NEN-EN1992-1, a stress limit is defined based on the maximum allowed crack width and the used reinforcement configuration. For the floor (maximum crack width of 0.2 millimeter) a reinforcement option of rebar diameter 16 millimeter and a spacing of 150 mm, the allowed steel stress is taken as 200 MPa. For higher diameters, the allowed steel stress becomes lower. For the top cross-section of the piles, using diameters of 25 and 32 mm, the allowed steel stress would be lower than 160 MPa. However, as this approach appeared to be over-conservative for the top cross-section of the piles, a different approach is taken.

The verification of crack width control needs to be done in frequent load combinations (SLS-frequent). The difference in loads in comparison to the load combination in ULS is the result of the load factors, which are 1.2 for the permanent loads and $1.5/\psi_1$ (with $\psi_1 = 0.75$ for the top load) for the variable loads. Depending on the ratio of permanent and variable loads, the factor is somewhere in-between (later on assumed to be 1.5). Since the allowed steel stress in ULS becomes 435 MPa for the assumed B500 reinforcement, the loads are allowed to increase by a factor 1.8 to 2.18 in order to still have the crack width control as a governing design check.

Based on the results of each calculation, the structure is checked on 6 aspects:

1. Deformations of the structure
2. Reinforcement in the floor
3. Reinforcement in the wall
4. Bending at the pile head
5. Axial capacity of the pile
6. Buckling capacity of the pile
7. Punching capacity of the floor

The first 4 checks are performed in SLS-frequent state. The last 3 checks need to be performed for ULS-based results. For reducing the calculation time, only one load combination is being used in the calculation process. All the checks are using results obtained from the calculation with the SLS-frequent load combination. By multiplying the assumed load factors to the results values, equivalent results in SLS-characteristic and ULS load combinations are being assumed.

As has been mentioned before, within the variable loads are only more global effects are considered. For a complete structural verification, also local loads should be modeled, which could be governing for the dimensions and reinforcement of the wall and the floor. Due to the spreading capacity of the soil and within the structure, local load effects are less of importance for the piles.

5.4.1 Serviceability limit state

Deformations

Too high deformations can have harmful effects on the surrounding buildings. An appropriate upper limit is set to 50 mm, which should be maintained for both the temporary construction phase and for the final situation. For minimizing the calculation time the results of the deformations are read in the frequent load combination. As in the characteristic load combination the top loads are not being reduced by a Ψ_1-factor of 0.75, the deformations are a bit higher (depending on the input, about 5 to 20%) than calculated. To compensate for reading the results in the frequent load combination, the limit value is divided by a factor 1.20.

The second assumption that needs to be mentioned is that the deformations are read at the top of the piles. Almost all of the deformations are coming from the deformations of the piles. It is being assumed that the contribution of the top structure (wall and floor) can be neglected in comparison the the deformations from the piles. Next to that, due to the construction phasing the maximum deformations from the top structure are being induced only after the sand filling is being placed. At this stage, a large part of the deformations have already been introduced to the piles due to the loads coming from the sheet pile. The additional deformations from the wall and the floor will not affect the adjacent buildings, but will only be affecting the final external appearance.

In addition, the top load of 20 kN/m^2 is considered to be an upper value for the uniform load, equivalent to the sum of separate (local) loads on the quay. Assuming this upper value over the full width of the quay is a large over-estimation considering the deformations of the structure. For considering the separate loads (with an equivalent load of 20 kN/m^2) only at a local part of the quay, the deformations become considerably much lower. As local loads are not included as a separate load combination, the results for the unity checks of deformations can be considered conservative.

Lastly, for the deformations in the vertical direction, the presented unity check is a bit more indicative. For the case of constructing a grouted body at the bottom of the pile the effective cross-sectional area could be considerably larger, resulting in a higher vertical soil bearing capacity and lower vertical deformations. Within this research, no grout bodies are being considered.

The plugin retrieves the results of the calculation in the form of a deformed structure following from the specified load case or load combination. The result data consists of a set of mesh point locations. By ordering the mesh points in the same way as is used for the input of the structure and the defined mesh, the distance between the mesh points before and after the calculation can be compared to find the deformations. For a better insight in the results, the unity checks for the maximum deformation in both vertical and horizontal direction are separately stored.

Floor reinforcement

A certain base reinforcement is being assumed for the floor. As mentioned, the crack width control in SLS-frequent state is assumed to be governing over the ULS capacity. For simplicity, this base reinforcement is set to a diameter of 16 mm, spaced 150 mm with a maximum steel stress of 200 MPa, according to tables 7.2N and 7.3N of NEN-EN1992-1. With this reinforcement net, the bending moment capacity for a certain floor thickness can be calculated. The reinforcement in longitudinal direction (x-direction) is assumed to be in the outer layer and reinforcement in transverse direction (y-direction) is assumed to be in the inner reinforcement layer.

Diameter (mm)	Max spacing (mm)	Maximum stress (MPa)
12	100	240
16	150	200
20	175	175

Table 5.2: Base reinforcement options following from tables 7.2N and 7.3N of NEN-EN 1992-1

From the retrieved calculation results, the surface forces from the top and bottom of the floor in both x- and y-direction (m_{xD-}, m_{xD+}, m_{yD-} and m_{yD+}) are being read. The data consist of a set of locations and a set of result values in the same order. By testing all the results to the bending moment capacity in that direction provided by the base reinforcement, the locations at which the capacity is being exceeded are selected.

The field bending moments are generally lower than the capacity of the base reinforcement, but the peaks of the support bending moments are easily larger than the capacity.

For the width defined by the locations where the capacity is being exceeded, the bending moment results are being averaged. The factor of the averaged result divided by the bending moment capacity of the base reinforcement is multiplied by the reinforcement ratio of the base reinforcement to obtain an increased reinforcement ratio. This reinforcement ratio is being applied in a lane over the supports, with the exceeding width increased by the effective height of the floor at both sided.

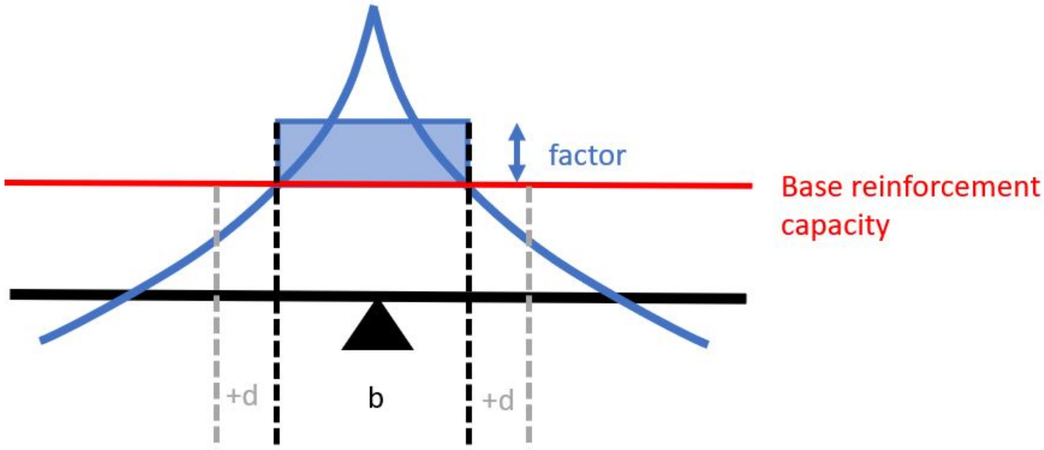

Figure 5.3: Assumed method for averaging peak bending moments

For the case of the field bending moments exceeding the bending moment capacity, the same approach is taken. However, as this generally only occurs for very unpractical base reinforcement configurations or small floor thicknesses, these situations are unlikely to be governing.

Wall reinforcement

In the same way as for the floor, a base reinforcement configuration is being assumed for the wall. By default this is set to a reinforcement diameter of 12 mm with a spacing of 150 mm in both directions. The vertical oriented reinforcement (Z-direction) is assumed in the outer reinforcement layer and the reinforcement in the longitudinal direction is assumed in the inner layer.

The retrieved surface forces in both directions (m_{xD-}, m_{xD+}, m_{yD-} and m_{yD+}) appeared to be lower than the bending capacity of the base reinforcement for wall height up to 2 meter. For higher walls the results for m_{yD-} will exceed the capacity, requiring an increased reinforcement ratio in vertical direction for the bottom part of the wall.

For smaller heights of the wall, another design aspect becomes critical. The wall can be considered to act as a beam, being supported by the floor at the locations of the piles. But as the bending stiffness of the wall is higher than the bending stiffness of the floor the floor will tend to deform more than the wall. As the wall and floor are continuously connected, this effect will cause an additional loading on the wall. As a result tensile forces are acting in the top at the locations of the supports. This tensile force is given in the result for n_{xD}.

A more practical approach is taken for increasing the longitudinal reinforcement in the top of the wall to resist the tensile forces. Assuming a maximum stress of 240 MPa (based on crack width control) the tensile capacity is being defined for the top part of the wall. The top part is being assumed as 2· [effective height], while the tensile effect is practically being assumed to act at 2/3· [height of the wall]. Conservatively assuming that the bending moment is linear increasing from this point to the maximum value at the top of the wall, the tensile forces in the top part (2· [effective height]) are being averaged. The required tensile capacity is divided by the capacity of the base reinforcement to find a factor by which the base reinforcement ratio is multiplied. Based on a maximum reinforcement ratio of 2% the unity check for this effect is determined. The output presents one result for the unity check, which is taken as the maximum result for each of the verifications.

By both increasing the height of the wall or by increasing the thickness of the floor (higher bending stiffness, resulting in less difference in deformation) the tensile forces in the top of the wall could be reduced.

Pile head bending

Bending of the top cross-section of the piles is expected to be the governing design aspect for this structural solution. In the top cross-section the steel casing is not structurally active, which means the connection forces are transferred only through the concrete and its reinforcement. Also for this check the crack width control is assumed to be governing, which is a conservative approach as crack formation at this location is being counter-acted by the presence of the steel casing.

The thickness is taken as 8 mm. It does not help in resisting the loads for this governing cross-section and its thickness only slightly reduces the width of the concrete cross-section and with that the internal leverage arm of the pile reinforcement. Together with a cover of 40 mm and a transverse reinforcement diameter of 10 mm, the internal leverage arm of the reinforcement is determined.

As the capacity of this cross-section is depending on both the axial load and the bending moment, a simplification needs to be made. A very conservative approach is to only consider the bending capacity of the reinforcement and limit the stress in the reinforcement in line with table 7.2N. The effect of a concrete compression zone, resulting in an increased amount of reinforcement loaded in tension, is being neglected in this way. Also the increased bending moment capacity for the cross-section in compression is not taken into account.

As this approach appears to disregard a considerable part of the bending capacity, a more accurate approach is being followed. By considering five different pile diameters with reinforcement configurations having a reinforcement ratio of about 3.0%, the bending moment capacities are being linked to the specific cross-section.

Diameter (mm)	Reinforcement	Ratio
406	12 ∅20	0.0322
457	10 ∅25	0.0323
508	12 ∅25	0.0315
610	10 ∅32	0.0294
711	10 ∅40	0.0335

Table 5.3: Pile diameters and reinforcement ratio

Using IDEA Statica - Reinforced Concrete Sections the maximum bending moment, for which the crack width limit of 0.2 is just being satisfied, is calculated for a varying axial compression force. The bending capacities are listed in table 5.4. By reading the axial member force in the top of the piles, the bending moment capacity is being linear interpolated from the list.

N (kN)	D406 (kNm)	D457 (kNm)	D508 (kNm)	D610 (kNm)	D711 (kNm)
0	56	91	121	193.5	341
-250	77.9	118.5	152	234	390
-500	99	145	163.5	273	437
-750	119.8	159	190.5	310	482
-1000	138.9	181.5	217	346	527
-1250	157.6	204	243	381	571
-1500	177.9	223	268	416	614
-1750	194.5	247	290	450	656

Table 5.4: Bending capacity in SLS-frequent state(crack width control)

5.4.2 Ultimate limit state

Axial capacity

For crack width control, the bending capacity will keep increasing for increasing axial compression. At some point, the cross-section capacity in ultimate limit state for interaction of axial force and bending moments becomes governing. Although it is unlikely for this to be governing, since this magnitude of axial forces can only be expected for very large spans, an upper limit is taken into consideration. To take this upper limit into account, the N-M capacity curve (ULS) is being multiplied by a factor to find an equivalent capacity in SLS-frequent load combination. This factor is somewhere between $1/1.2$ (in case of only permanent loads) and $1/(1.5/\psi_1)$ (in case of only variable loads, with $\psi_1 = 0.75$ for the assumed top load), depending on the contributions of permanent and variable loads. A factor of $1/1.5$ is assumed to be an average value.

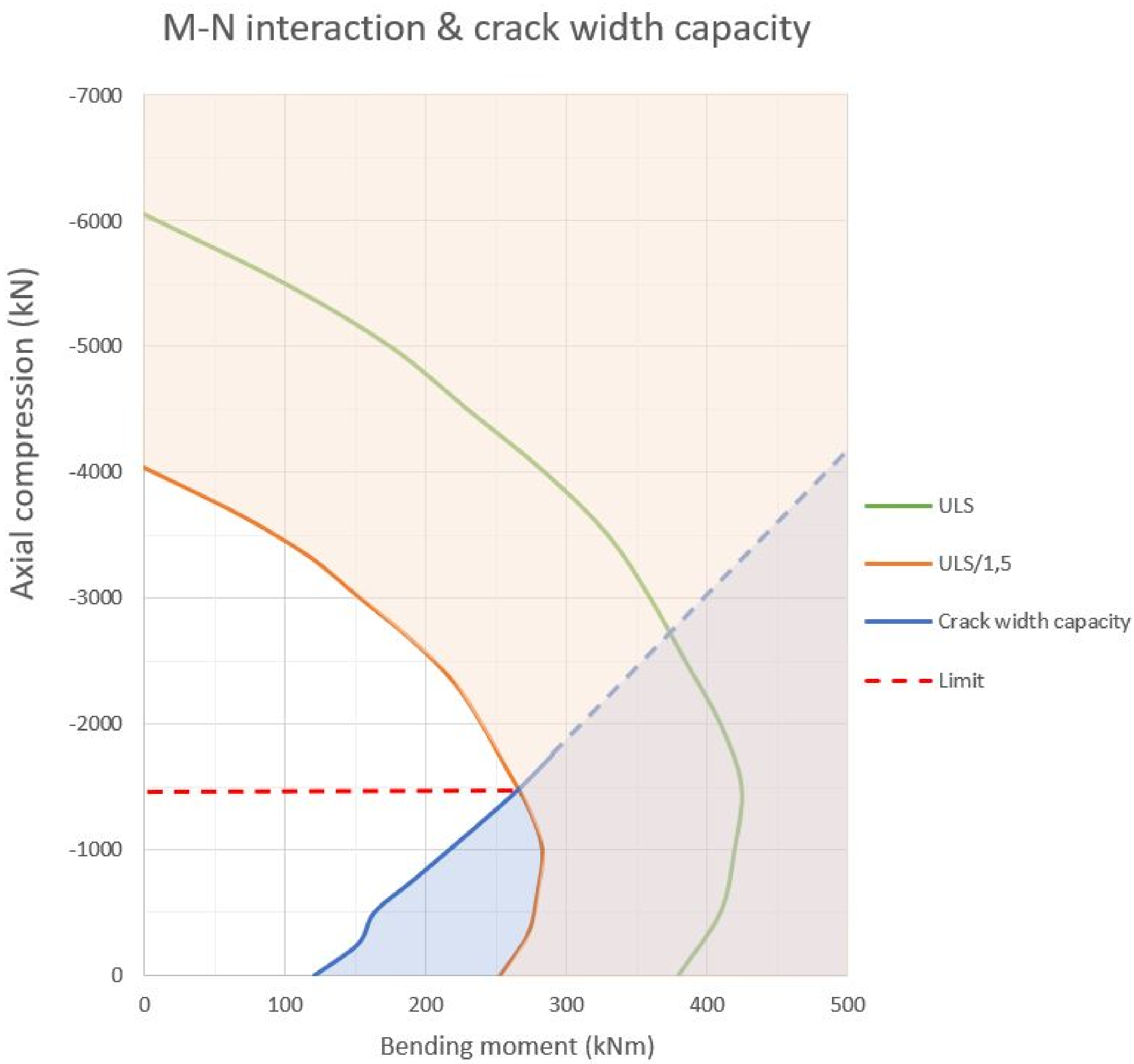

Figure 5.4: Assumed limit values for the interaction of axial compression and bending moments (based on pile with diameter 508 mm)

For each of the presented pile diameter and corresponding reinforcement configurations an upper limit of the N-M capacity is calculated using IDEA Statica, which subsequently is brought down to an equivalent capacity in SLS-frequent state. The limit values for axial compression are presented in table 5.5.

Buckling capacity

Following the shape of the deformed loaded piles, they can be considered as being clamped somewhere in deeper soil layers. For the buckling capacity, the rough estimation is being made that the pile can be considered as clamped around the first sand layer. Due to a lack of counter pressure in lateral direction on the piles in the weaker soil layers above, the risk of buckling should be taken into account. A conservative assumption is made by setting the system length (top of the pile to the first sand layer) to 13 meter (see table 4.2), multiplied by a K-value for the buckling shape of 2.0 for a column clamped at one side. Considering an effective bending stiffness (contributions of the steel casing and concrete), including an correction factor for cracking of 0.6, the buckling capacity can be calculated. For determining the effective bending stiffness, the wall thickness of the steel casing is being reduced by the corroded layer at the end of the life span, which is assumed to be 4.3 mm. This entails that after the life span of 100 years, the buckling capacity of the piles are not guaranteed. In table 5.5 the buckling capacity for a cross-section with a residual wall thickness of 3.7 mm is being presented.

	N-M capacity (kN)		Buckling capacity (kN)	
Pile diameter (mm)	ULS	ULS/1.5	ULS	ULS/1.5
406	-1372	-915	-622	**-415**
457	-1609	-1073	-958	**-639**
508	-1921	-1281	-1413	**-942**
610	-2175	**-1450**	-2779	-1853
711	-2541	**-1694**	-4921	-3281

Table 5.5: Axial capacity of considered piles for N-M interaction and buckling

In the results, the unity checks for N-M interaction and buckling are combined and presented in one unity check for the axial capacity of the piles. The maximum value coming from both checks are being used for the output.

Punching capacity

The punching capacity of the floor is based on the shear capacity of the cone shaped cross-section above the piles. The outer diameter (top of the floor) has a diameter of [top cross-sectional diameter of the pile] + 2 · [effective floor height], for which the effective floor height is the average value of the effective floor heights in both directions. Although the punching capacity could be reached for small thicknesses of the floor, the unity check can be considered as an indicative value. When the capacity of the shear cone is reached, additional reinforcement could be used around the location of the piles to increase the punching capacity. For simplicity, the unity check only considers the capacity of the floor with a base reinforcement configuration.

For the situation in which the direction of the resultant load is more horizontal than the resultant direction of the supports, the maximum axial load is always found at an inclined pile.

Chapter 6

Results

The developed parametric model offers an indicative optimal variant in which the locations of the piles, the height of the wall, height of the floor, thickness of the floor and orientation of the piles could be optimized for the defined locational inputs for the retaining height, the available width, the water level in the canal and the soil profile. As has been mentioned, a lot of assumptions were made prior to and along with developing the presented model. As each of these assumptions can have an influence on the results, the optimum is deliberately mentioned as an indicative result. However, by thoughtfully choosing the assumptions (without being too conservative or too optimistic) the model can be considered as a good indicative tool within the preliminary design stage.

The performance of the optimization model can best be presented by assessing its results in relation to a benchmark. By starting with the structural geometry and pile inclinations of a calculation report for a traditional L-wall structure, the results of the parametric model can be compared to the results of the benchmark calculation. Subsequently, locational-dependent input will be considered through the optimization process to find the effect on the results and ultimately to the main objective: the reduction of the total material use.

6.1 Benchmark

The first step of interpreting the results is to make a valid comparison to the traditional L-wall structure. Based on a calculation report for a quay wall renovation project in Amsterdam (Nieuwe Herengracht) a benchmark has been set. By taking into account the differences in the assumptions that are made within the benchmark calculations and within this research, the results are being compared in order to find the possible improvements for the traditional structure.

By starting with the structural geometry and pile inclinations of the above-mentioned calculation report, the results of the parametric model are being compared. Subsequently, the same geometry and loads are run through optimization 1 for finding the support lines which result in the pile locations for which the least eccentricity is found. In the next step, the piles are being placed a the assumed maximum inclination of 5:1 for all piles at the canal-side and 4 out of 7 piles at land-side. In the next step, the span is being increased up to the value at which one of the unity checks is at the maximum value. By repetitively taking measures which lower the governing design check, an optimization is sought.

6.1.1 Assumptions

To make a valid comparison, the following assumptions are being made:

- ◇ The force coming from the sheet pile in load cases 2 and 3 are set to the values (in SLS-frequent stat) used in the calculation report, which are the result of a geotechnical calculation following the construction phasing. For the sheet pile force as a result of the soil load a value of 14.2 kN/m is assumed, whereas the formula would have given a value of 15.6 kN/m. For the sheet pile force as a result of the top load, a value of 13 kN/m is used in the report, where the formula would have given a value of 23.6 kN/m. Although the formula appears to be highly overestimating the for the force as a result of the top load, in the calculation report the used ULS sheet pile force is taken as a factor 2.1 higher compared to the force in SLS-frequent state. On average, the values from the formulas resulting from a simplified approach (Blum's Method) are considered to be a conservative value.
- ◇ For the design of the calculation report the piles in the row at the canal-side are all being placed at an inclination of 6:1. The piles in the row at land-side are all vertical. This could very likely be the preferred option due to construction considerations. In the optimization process a maximum inclination of 5:1 is being assumed, for which also part of the piles at land-side are being inclined.
- ◇ In the parametric model, the amount of loads and load combinations are reduced to only the permanent loads and a variable top load within one frequent-state load combination. In the calculation report also a load for the masonry wall at the canal-side of the wall is being applied, and a load on top of the wall representing the weight of the topping stone. As these loads are small and only contributing to the vertical load (no horizontal load component), which has a favorable effect on the pile head bending moments, these loads are not taken into account. Next to that, the conservative assumption is made that a mooring load of 8 kN/m over the full length is taken into account, acting within the same load combinations as the top load and without a Ψ reduction factor for combined effects of variable loads.
- ◇ For the design checks from the calculation report the maximum values for the unity checks are within the range of 0.70 to 0.80. One unity check has a value of 0.82, related to the interaction of axial compression and a bending moment at the top of the pile. To make a fair comparison, the obtained unity checks following from the parametric model should be considered within the same range.
- ◇ The cross-section of the piles that are used in the benchmark have a diameter of 508 mm, and the same amount of reinforcement as is assumed within this research (12 ∅25).

As often the designs for quay wall renovation projects are being governed by construction aspects, the conclusions following from comparing the results of the presented model with the benchmark do not say anything about the quality of the designs. This research is only focusing on the structural aspects and the used materials. In practice often trade-offs are being made with respect to the additional costs for construction or its complexity.

6.1.2 Improvements

By running the structural geometry of the benchmark through the model, the results show a design which still has some unused capacity. The maximum unity check (0.762) is found for the bending moment on the pile, which is a bit higher than the result presented in the calculation report (0.70). In the benchmark report, the governing check (0.82) comes from the interaction of forces on the piles in ultimate limit state. The governing result for this check follows from a load combination in which the top load of 20 kN/m^2 (load factor 1.5) is being combined with a mooring load of 8 kN/m (load factor 1.2) and an increased upward water pressure (load factor 1.2). Whether for this situation Ψ-factors should be applied could be discussed. The governing results show a large reduction of the axial compression, while still a large bending moment should be resisted. With this interaction, the increased bending capacity due to axial compression disappears.

However, by having more piles at an inclination and making larger spans, also the possibility for low axial compression forces (or even tensile forces) are being reduced. Instead of placing one row of piles (canal-side) inclined and in extreme compression and the other row of piles (land-side) vertical and in low compression or even tension, the distribution of pile loads is also more equal.

In table 6.1 the intermediate results of unity checks for each of the design aspect are being shown for each improvement step. A green color represents a design aspect in which a lot of capacity is being unused. A red color represents a design check within the critical zone, for which the unity check approaches the limit capacity of 1.0. From the results in the table it can be concluded that for the geometrical input of the benchmark design an unbalance of loads occurs. The piles at land-side are largely eccentric loaded, resulting in high bending moments and much axial capacity not being utilized.

Step	Input adjustments	D (mm)	span (m)	t,fl (m)	UC Y-Def.	UC Z-Def	UC Floor	UC Wall	UC Bending	UC Axial	UC Punch.	Total cost
0	6:1& 50% inclined	508	2	0,4	0,31	0,05	0,31	0,44	0,76	0,28	0,64	€ **4.844**
1	Optimization 1	508	2	0,4	0,31	0,05	0,31	0,43	0,72	0,29	0,66	€ **4.844**
2	5:1& 75% inclined	508	2	0,4	0,24	0,04	0,28	0,41	0,51	0,26	0,59	€ **4.875**
3	1,75 x span	508	3,5	0,4	0,43	0,07	0,31	0,60	0,90	0,45	0,88	€ **2.982**
4	t,floor = 0,7 m	508	3,5	0,7	0,35	0,10	0,16	0,31	0,77	0,46	0,38	€ **3.104**
5	+ 2,0 x span	508	4	0,7	0,41	0,11	0,16	0,33	0,86	0,53	0,41	€ **2.787**

Unity check	Interpretation
1,0	Capacity limit
0,8	Critical zone
0,6	Some capacity unused
0,4	Lot of capacity unused
0,2	Almost no utilization
0	No utilization

Table 6.1: Results for each step of improvement measures

Step 1: Support lines Starting with an improvement for the support lines (optimization 1) the support locations in perpendicular direction of the floor are being found for which the sum of bending moments at the supports are being minimized. For the support line at the canal-side, optimization 1 results in a change of about 9% of the width. For the support line at land-side the change is only 1%. By changing the support lines, the unity check for the bending capacity of the piles shows a small reduction of 5% while the axial capacity is 4% more being utilized.

Step 2: Pile inclinations By inclining the piles in the row at the canal-side at an inclination of 5:1 (instead of the benchmark inclination of 6:1) and incline 4 out of 7 piles of the pile row at land side, a reduction of the unity checks for the pile bending moments can be seen in the results. With this step a reduction of the maximum pile bending moment of almost 30% is found in comparison to the previous situation. As the result of the unity check for axial capacity is also decreased (with the same loads), it can be concluded that the loads are more evenly distributed over the piles. The same conclusion can be drawn from the fact that the deformation in vertical direction also decreases. As less bending moments are acting on the piles also less lateral deformations occur. Each of the considered design checks result in a lower unity check with this step.

Step 3: Increasing the span Since the results from the previous step show a lot of unused capacity, the span can be increased in order to reduce the material use. By multiplying the span in longitudinal direction by a factor 1.75, the unity check for the pile bending capacity has increased by about the same factor. In addition, the unity checks obtained for the punching capacity of the floor have reached the critical zone while the unity check for the wall reinforcement approaches the critical zone (with unity checks of 0.88 and 0.60 respectively). Although the loads and the dimensions of the wall are the same, the internal bending moments become higher as a result of the increased span due to the deformation of the floor. If the floor would have been infinitely stiff, the bending moments are not affected by a changing span. By increasing the floor thickness (increasing its stiffness) this effect can be reduces. As also the unity check for the punching capacity is critical, the next step is to increase the floor.

Step 4: Increasing the floor thickness By increasing the thickness of the floor from 400 mm to 700 mm, the expected effect on the previous governing design aspects are clearly being achieved with reductions of 48% on the unity check for the wall and a reduction of 57% on the utilization of the punching capacity. The increase of the floor thickness has only an increasing effect of 4% with respect to the total material use.

As for this step the self-weight of the structure increases also the direction of the resultant load is affected. This would have an effect on the optimal supporting lines determined in optimization 1. However, the effect from the increased floor thickness is assumed to be low in comparison to the total of loads. The increase of the floor thickness also affects the distribution of loads over the piles, which can be seen in the results for this step. The axial capacity is being governed by the buckling capacity, which is based on the maximum value for axial compression found in the piles. Although most of the additional weight is taken by the pile row at land-side (68%, from which 33% to the 3 vertical piles and 35% to the 4 inclined piles) the governing axial load is still found at the inclined piles at the canal-side. The additional load on these piles (32% of the additional weight divided over 7 piles) result in a small increase of the unity check related to the axial capacity.

As a result of the additional axial load on the vertical piles at land-side, which are governing for the unity check of pile head bending moments as they are experiencing the most eccentricity, the bending moment capacity of the cross-section increases (about 7%) due to the interaction with axial compression. At the same time the global eccentricity is reduced as a result of an increased vertical load contribution, for which the resultant direction of the overall load is closer to the resultant direction of supports (resultant pile inclination). As a result each of the piles is taking less bending moment, for which the governing pile obtains a reduction of about 5% on the bending moment. By both an increased capacity and a decreased bending moment the utilization of the cross-section is reduced by 13% (UC for bending from 0.90 to 0.77).

By the same cause also the deformations are affected. The vertical deformations are increased due to the additional axial loading while the lateral deformations are reduced due to the lower eccentricity.

Step 5: Increasing the span From the results of the last step it can be seen that again some of the capacity is still unused. By again increasing the longitudinal span (the most effective measure in reducing the material use) up to 4 meter the utilization of the bending moment capacity of the piles is still critical (0.86). With this last step the use of material per meter quay wall has been reduced by 42% in comparison to the geometry of the benchmark. By adjusting the diameter of the piles, the width of the floor and/or the orientation of the floor, even more material could be saved.

6.2 Load eccentricity

From the improvements taken in the benchmark, the intended approach and expected results can be found. By following the governing design aspects and taking measures that lower the utilization (unity check) for that aspect, an iterative process is guiding towards an optimum. Following from the starting points of a retaining height of 3.3 meter and a floor width of 3 meter, the results found in the benchmark indicate that after the span and the floor thickness are properly chosen (pile diameter has not been considered) the bending moments (resulting from lateral effects) are governing over more vertical related design aspects, like the axial capacity of the piles or the punching capacity of the floor. In line with the expectations, an over-eccentric load situation is found.

To reduce the gap between the resulting direction of the loads and the piles, the piles could be more inclined. However, as a limit has been given to the inclination of the piles (based on construction limitations) this measure is out of range. To reduce the eccentricity, the loads should be considered by decreasing the ratio of horizontal loads to the vertical loads resisted by the structure. Assuming the horizontal loads are directly depending on the retaining height, which is a constant factor for the location, the horizontal loads are also constant. The eccentricity needs to be reduced by increasing the vertical contribution of the resulting load.

One way that already has been shown to be effective is to increase the self-weight of the floor by increasing its thickness. As is done in the benchmark, the increase of the floor thickness lowers the bending moments on the pile while the axial load becomes higher. For larger spans, this effect also becomes higher. Another measure that can be taken to increase the vertical load is to increase the floor width, for which more vertical load can be collected which reduces the eccentricity of the resulting load. From a certain floor width, the load eccentricity is reduced to a point at which it would be expected that the mentioned vertical-related design aspects become governing over the horizontal-related design aspects (bending moments of the pile or lateral deformations).

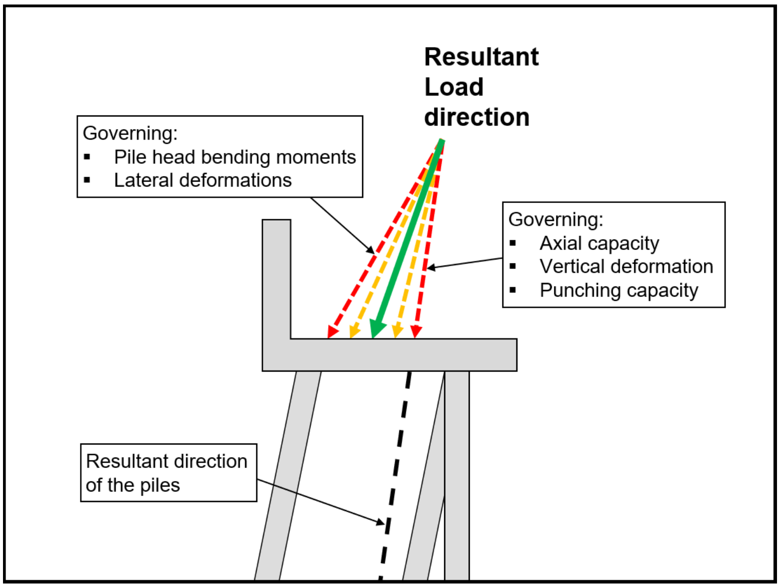

Figure 6.1: Load situations as a result of the resultant direction of the load and the supports

6.2.1 Width of the floor

By considering the same retaining height as is used for the benchmark, the effect of varying the width of the floor is being investigated. To focus on the effect of the floor width and resulting values for the floor thickness and the span, the height of the wall is set to a constant value of 1.3 meter. For each of the considered floor widths, optimization 1 is applied to find the optimal positioning of the piles in transverse direction. Starting from a floor width of 2.5 meter up to a floor width of 4.5 meter for a constant retaining height of 3.3 meter, the optimum solution and the corresponding material use for each of the considered pile diameters is presented. As for the considered cases with a small width-to-retaining height ratio the results are converging to large floor thicknesses, the right side of the tables show the results for which the floor thickness is limited to 0.70 meter. The results for the material use are given a color relative to the highest (red) and the lowest (green) result for all of the considered cases.

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
2,5	3,3	Pile bending	406,4	1,00	1,9	€ 3.981	0,70	1,7	€ 4.270
		Pile bending	457	1,00	3	€ 3.135	0,70	2,5	€ 3.443
W/Hr-ratio		Pile bending	508	1,00	4	€ 2.820	0,70	3,5	€ 3.036
0,76		Pile bending	610	1,00	6	€ 2.567	0,70	5,5	€ 2.658
		Pile bending/wall	711	0,95	10,0	€ 2.220	0,70	6,5	€ 2.851

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
3,0	3,3	Pile bending	406,4	1,00	2,5	€ 3.267	0,65	2	€ 3.758
		Pile bending	457	1,00	4	€ 2.599	0,55	3	€ 3.038
W/Hr-ratio		Pile bending	508	0,90	5	€ 2.430	0,60	4	€ 2.749
0,91		Pile bending	610	0,95	8,5	€ 2.116	0,70	7	€ 2.306
		Axial capacity/wall	711	0,95	10,5	€ 2.216	0,70	9,0	€ 2.356

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
3,5	3,3	Axial/Bending	406,4	0,85	3	€ 2.860	0,65	2,5	€ 3.231
		Axial/Bending	457	0,75	4,5	€ 2.359	0,60	4	€ 2.495
W/Hr-ratio		Axial/Bending	508	0,85	6	€ 2.204	0,70	5,5	€ 2.265
1,06		Axial/Bending/Wall	610	0,75	9,5	€ 2.031	0,65	9	€ 2.056
		Axial capacity/Wall	711	1,00	11	€ 2.260	0,70	9,5	€ 2.345

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
4,0	3,3	Axial/Bending	406,4	0,55	3	€ 2.773	0,55	3	€ 2.773
		Axial/Bending	457	0,45	4,5	€ 2.289	0,45	4,5	€ 2.289
W/Hr-ratio		Axial/Bending	508	0,65	6,5	€ 2.115	0,65	6,5	€ 2.115
1,21		Axial capacity/Wall	610	0,80	10	€ 2.111	0,70	9,5	€ 2.117
		Axial capacity/Wall	711	0,95	11	€ 2.344	0,65	9,5	€ 2.433

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
4,5	3,3	Axial capacity	406,4	0,4	2,5	€ 3.169	0,4	2,5	€ 3.169
		Axial capacity	457	0,45	4,5	€ 2.339	0,45	4,5	€ 2.339
W/Hr-ratio		Axial capacity	508	0,55	6,5	€ 2.137	0,55	6,5	€ 2.137
1,36		Axial capacity/Wall	610	0,65	9,5	€ 2.168	0,65	9,5	€ 2.168
		Axial capacity/Wall	711	0,60	9,5	€ 2.487	0,60	9,5	€ 2.487

Table 6.2: Effect of the width of the floor on the optimum results for each of the pile diameters for a retaining height of 3.3 meter

From these results already a lot of conclusions can be drawn. In section 6.3 conclusions are given for each of the governing aspects and the effects that each parameter has. Focusing on the effect of the floor width, the first thing that can be noticed is that for small floor widths the optimum solution is found for in combination with large floor thicknesses. The cause for that has also been shown in the benchmark, for which the increase of the floor thickness has a larger positive effect in reducing the eccentricity than the negative effect it has on the material use. That also means that from the convergence of the results for different floor widths, an indication about the situation of eccentricity can be given based on the floor thicknesses of the optimum result. For larger widths of the floor the eccentricity becomes lower, which means the vertical related design aspects (mainly the axial capacity of the piles) are becoming governing over the lateral design aspects (mainly bending moment capacity of the piles). The floor thickness in these more centric load situations follows from the minimum thickness required to satisfy the requirements for the punching capacity or the capacity of the wall(governing for large spans).

With respect to the material use, the optimal results are found for the use of piles with a diameter of 508 or 610 millimeter. By going to larger floor widths, a shift towards smaller pile diameters can be noticed. Apart from the scaling effect (larger diameters allow for larger spans), the large pile diameters are mainly desired for their bending capacity. The overall optimum is found for a width-to-retaining height ratio of 1.06 and a piles with a diameter of 610 millimeter. By considering the floor thickness being limited to 0.70 meter, the effect of adjusting the floor width can be indicated. With respect to the benchmark width of 3.0 meter (optimum material use = € 2,306/m), a save on the material use of 10.8% can be obtained by increasing the width of the floor by 0.5 meter (optimum material use = € 2,056/m). On the other side, if the municipality prefers to allocate a bit more space for the accessibility of the quay during construction, the floor width could be reduced by 0.5 meter (optimum material use = € 2,658/m) against an additional cost of 15.3%.

The effect on the material use for increasing the floor width up to width-to-height ratios larger than 1.06 the effect is considerably lower. Considering the context (defined categories A and B from the data analysis) of application, larger ratios are not of interest. By looking at the results for a retaining height of 4.8 meter, about the same trend can be found. The optimum is found around the same ratio, although the axial capacity of the piles becomes a governing design aspect in almost every situation. Although the height of the soil layer above the floor remains constant, due to the large width of the floor more loads are being collected.

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
4,3	4,8	Bending	406,4	0,90	2	€ 4.337	0,70	1,6	€ 5.075
		Bending	457	1,00	3,5	€ 3.283	0,65	2,5	€ 3.999
W/Hr-ratio		Axial/Bending	508	0,90	4	€ 3.271	0,55	3	€ 3.852
0,90		Axial/Bending	610	1,00	8	€ 2.575	0,70	5,5	€ 3.069
		Axial/Bending/Wall	711	0,85	10,0	€ 2.659	0,70	9,0	€ 2.748

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
5,0	4,8	Axial/Bending	406,4	0,6	2	€ 4.247	0,6	2	€ 4.247
		Axial/Bending	457	0,65	3,5	€ 3.167	0,65	3,5	€ 3.167
W/Hr-ratio		Axial/Bending	508	0,80	5	€ 2.848	0,70	4,5	€ 3.002
1,04		Axial/Bending	610	0,65	8	€ 2.523	0,65	8	€ 2.523
		Axial capacity/Wall	711	0,65	9,5	€ 2.800	0,65	9,5	€ 2.800

Width	Retaining height	Governing design aspect	Diameter piles (mm)	Floor thickness (m)	Span (m)	Material use (€/m)	Floor thickness (m)	Span (m)	Material use (€/m)
5,8	4,8	Axial/Bending	406,4	0,4	2	€ 4.188	0,4	2	€ 4.188
		Axial/Bending	457	0,45	3,5	€ 3.124	0,45	3,5	€ 3.124
W/Hr-ratio		Axial/Bending/Punching	508	0,55	5	€ 2.793	0,55	5	€ 2.793
1,21		Axial/Punching	610	0,60	7,5	€ 2.664	0,60	7,5	€ 2.664
		Axial/Punching	711	0,65	9	€ 2.965	0,65	9	€ 2.965

Table 6.3: Effect of the width of the floor on the optimum results for each of the pile diameters for a retaining height of 4.8 meter

By comparing the results for different pile diameters, it can also be noticed that for different pile diameters the optimum can be found at a different width-to-retaining height ratio. As the overall optimum is found at a ratio around 1.06 and the pile diameter of 610 millimeter, piles with a diameter of 457 and 508 millimeter find their optimum at a ratio around 1.21 (for both retaining heights). This can be linked to the capacities defined for each of the cross-sections in section 5.4. The optimum for each of the cross-sections is to be found near the eccentricity for which both the bending capacity and the axial capacity of the cross-section is maximally being utilized. Because the ratio between the bending capacity and the axial capacity of each pile diameter differ, the optimum width-to-height ratio also differs. In section 5.4.2 the axial capacity of each diameter is stated to be governed by either the buckling capacity or the capacity following from the interaction of bending moment and axial compression. For diameters up to 508 millimeter, the buckling capacity is governing. As the ratio between the bending moment capacity to the axial capacity of the pile (indicated by the gradient of the inclined dotted lines in figure 6.2) is higher for a pile with a diameter of 610 millimeter, it will find its optimum at a higher eccentricity in comparison to piles with a diameter of 457 or 508 millimeter, which is the case for a lower width-to-retaining height ratio. The gradients for the diameters 457 and 508 millimeter are almost the same, resulting in almost the same optimum for the width-to-retaining height ratio.

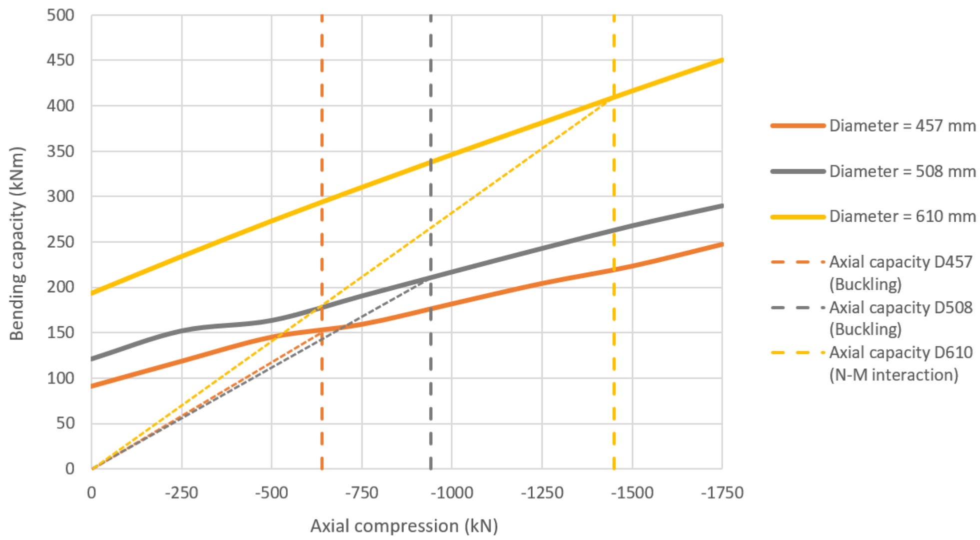

Figure 6.2: Effect of the ratio between the bending moment and axial capacities of pile diameters 457, 508 and 610 millimeter

6.2.2 Concluding graphs for different floor widths

In the graphs presented in appendix D the results are being shown for the 3 load eccentricities, corresponding to a retaining height of 3.3 meter, a floor thickness of 500 millimeter and floor widths of 3.0, 3.5 and 4.0 meters (width-to-retaining height ratios of respectively 0.91, 1.06 and 1.21). The first of the 2 graphs shown for each floor width presents the most critical design aspects, in which the non-governing aspects are made transparent. The dashed lines represent the utilization for a certain design aspect. The continuous lines represent the material use (in €/meter quay). In the second graph, for each of the pile diameters only the governing design aspect is shown. The capacity for this design aspect is reached at the intersection with the red dotted capacity line (UC = 1). By drawing a vertical line to the horizontal axis, the corresponding longitudinal span can be read. By following this dotted vertical line to the intersection with the continuous line of that pile diameter, the material use corresponding to the capacity can be read from the vertical axis at the left side of the graph.

By comparing the graphs for different floor widths, effects in line with the results found in table 6.2 can be seen. As a floor thickness of 500 millimeter is considered, the punching capacity or the capacity of the wall (due to beam-like behavior) is limiting for the extreme spans found in that table. For the smaller pile diameters, the pile head bending moment capacity and the axial capacity of the piles are governing. It can be seen that for these smaller piles the optimum, is found around a floor width of 4.0 meter, as capacity lines for the bending and the axial capacity are crossing the capacity line at almost the same point, indicating a maximum utilization for both design aspects. Instead of a maximum utilization (Unity check ≈ 1.0) in practice a lower utilization is desired for integrating an additional safety margin. This of course affects the results, leading to lower spans and higher values for the material use.

A side note needs to be mentioned with respect to the convergence of the results towards extreme spans for the largest pile diameters. The material use is expressed as the costs of the volume of materials. However, for the larger pile diameters this might not be a valid representation. The costs of constructing such a pile is (at least with the current used methods) relatively more expensive than constructing a pile with a smaller diameter. The question is which part of the costs fall under the material costs, and which are related to the construction method. As mentioned before, for this type of structures within the urban environment the common pile diameters to be used are up to 508 millimeter. The results presented in the tables above proof that in terms of material use, the pile diameters with a diameter of 508 or 610 millimeter are the most efficient.

6.3 Sensitivity of parameters

6.3.1 Structural verifications

From the results of the structural verification in the presented model, trends can be seen for each of the considered aspects. As mentioned, one input parameter can be appointed as the most affecting with respect to the results for the objective (material use expressed in total costs), but also for the results of the unity checks. This input parameter is the span between the piles in longitudinal direction of the quay. Due to the high impact of the piles on the material use, the main part of the aim can be re-specified as the minimization of piles per unit length of quay. By making the spans as large as possible while satisfying the requirements for the structural verification, an optimum can be found. For different pile diameters this optimum is found at different spans.

Deformations

In comparison to the other design checks, the results for the deformations of the structure are less manipulative. They are mainly the result of the soil profile at the specific location. The results for the lateral deformations are following the same trends as the bending moments, both the result of lateral load effects. The vertical deformations are following directly from the axial load on the piles, both mainly depending on the vertical load effects. However, whether the structural aspects (bending capacity and axial capacity) are governing over the deformation capacity (allowed deformation of 50 millimeter) is governing depends on the considered vertical stiffness value. In practice, a range for the vertical stiffness to the pile tip is taken into account.

From the geotechnical advice used in the benchmark calculation, the lower value for the vertical stiffness is given as 22.5 MN/m. With respect to the assumed 125 MN/m the deformations become considerably higher, both in vertical and lateral direction. Considering the optimized case of for the benchmark geometry (with diameter = 508 mm, floor thickness = 700 mm) the effects on the results for both stiffnesses are shown The vertical deformations are increased by a factor of more than 4. With respect to the ratio of the assumed stiffness to the lower stiffness value, it can be concluded that under the same load and geometry of the structure, a redistribution of pile forces occurs.

Vert. stiffness (MN/m)		UC Y-Def.	UC Z-Def.	UC Floor	UC Wall	UC Bending	UC Axial	UC Punching	Total cost
Assumed	125	0,434	0,108	0,162	0,340	0,930	0,540	0,422	**€ 2.787**
Lower limit	22,5	0,651	0,477	0,162	0,305	0,717	0,506	0,395	**€ 2.787**

Figure 6.3: Effect on the results by using a low vertical stiffness value

In the presented results in this research, the focus have been more on the verification of the structural aspects. For applying the lower stiffness value the deformations can become the governing design aspect, especially for the large diameters converging to very large spans. However, the corresponding optimum results are in about the same range as the presented results. By calculating for each input of parameters both the results for a low and a high stiffness, the enveloping results could be found. But still that would be a conservative approach. The truth is somewhere in-between, for which the stiffness of 125 MN/m can be considered as a valid assumption in a preliminary design stage.

Floor reinforcement

The unity check for the floor reinforcement is not governing with the assumed verification process considering floor thicknesses above 400 mm. For thicknesses smaller than 400 mm and large spans it could be resulting in unity checks close to or above 1, but for that case also other design checks become critical. For larger width-to-height ratios the aim is to use the smallest possible floor thickness for which the related design checks are just being satisfied. However, as the floor reinforcement is adjusted based on the results for each set of input parameters, the capacity (which is set to a reinforcement ratio of 2%) is never critical.

Wall reinforcement

The wall reinforcement is mainly dependent on the used height of the wall, which directly affects the results for $n_{x,D}$ indicating the bending moments for beam-like behavior in the wall. For small thicknesses/stiffnesses of the floor, at mid-span (field bending moments) the wall experiences a downwards pulling force from the connection with the floor. This additional load on the wall results in tensile stresses at the top of the wall at the locations of the piles. From the results, it can be seen that this effect is reduced by increasing the floor thickness. By increasing its stiffness (under the same load) the floor is less deforming and with the interaction forces becomes lower. By increasing the height of the wall the bending stiffness of the wall becomes higher, which also means the wall could better resist the additional loads. The other way around, by decreasing the height of the wall, the difference in bending stiffness between the wall and the floor become lower, resulting in a lower additional loading on the wall.

From the results presented in appendix D this effect only appeared to be governing for the case of extreme spans (about 8.5 meters). As the effect is almost independent of the piles, about the same limit capacity is applicable each of the pile diameters. But for only a pile diameter of 711 millimeter, this appeared to be a governing aspect. For this case, an increase of the height of the floor would result in the axial capacity being governing. By increasing the height or the thickness of the wall, the span limit could be made larger.

Pile head bending moments

In line with the expectations, one of the most critical design checks follow from the pile head bending moments. Especially since for the considered scope usually an over-eccentric load situation is unavoidable due to requirements limiting the width of the floor. The limiting factor on the other hand is the maximum inclination of the piles, which has been assumed to be 5:1 with respect to construction aspects.

The overall horizontal loads on the structure are dependent on the retaining height, which can be considered a constant. For lowering the pile head bending moments, the vertical load should be increased. By changing the floor thickness, a trade-off between the utilization of the bending capacity and the axial capacity can be made. For over-eccentric load situations, the floor thickness should be made as large as possible. However, a larger effect is found for increasing the floor width (is allowed with respect to the requirements).

Axial capacity of the piles

In section 5.4.2 the underlying method in finding the unity check for the axial capacity of the piles is explained. Within the unity check, two design verifications are captured: The interaction of axial loads and bending moments and the buckling capacity of the piles. In table 5.5 the capacities for both design checks are presented. From this table it can already be seen that for piles up to a diameter of 508 millimeter, the buckling capacity is reached before the capacity for N-M interaction. Still, very large axial loads need to be acting on the piles in order to reach these capacities. For more centric loading situations and very large spans the axial capacity of the pile becomes governing. But for eccentric load situations the axial capacity does not play a role, unless large floor thicknesses are being used.

Punching capacity of the floor

The punching capacity is a relative easy verification to be satisfy, as an increase of the capacity (by increasing the floor thickness) has only a minor effect on the material use. Only for larger spans and a floor thicknesses close to 0.40 meter the capacity has been reached, but for the range of considered thicknesses (between 0.40 and 1.0 meter) the punching capacity is never the governing design aspect. This design verification functions more as an indication of the minimum floor thickness.

Material use

In reducing the total use of material, the span between the piles in longitudinal direction has the highest impact. As a guideline it could be assumed that the largest span for which the structural verifications (from which the bending capacity or the axial capacity of the piles are generally governing) are satisfied results in the optimal solution. As the contribution of the piles (steel, concrete and reinforcement) to the total material use is in the range of 60-80% for the considered optimum results, the aim is to use the least amount of piles per unit length of the quay wall.

The parameter with the second highest impact on the material use is the diameter of the pile. The results and the graphs in the appendix show a considerable effect on the material use as a result of the used pile diameter. The most governing structural aspects are directly related to the considered diameter. For the considered combinations of the retaining heights and floor widths, in relation to the defined scope, the optimum results indicating the ultimate capacities are found for piles with diameters of 508, 610 or (in extreme) 711 millimeter. Of course, even better results could be found for diameters in-between.

As a third means in reducing the material use, the thickness of the floor can be varied. Depending on the eccentricity, the floor thickness should either be minimized or maximized in order to compensate for the most governing structural aspect. Considering an eccentric load situation is usually applicable within the scope, the maximization of the floor thickness to reduce the eccentricity can be aimed for. Apart from limitations with respect to the construction, the floor width could only be limited by the axial capacity of the piles or the vertical deformations that follow from the increase of self-weight.

6.3.2 Effect of inclining the floor

The intended purpose of inclining the floor is to increase the eccentricity, which is desired for a load situation in which the piles are too centric loaded. The axial capacity is governing while the bending capacity is low-utilized. This is the case for relative large floor widths and a large width-to-retaining height ratio. From table 6.2 the larges considered ratio is for a floor width of 4.5 meter, in relation to a retaining height of 3.3 meter, a wall with a height of 1.3 meter and a non-inclined floor. Considering a pile diameter of 457 millimeter, the table shows the optimum for a span of 4.5 meter and a floor thickness of 0.45 meter, governed by the axial capacity. The results in table 6.4 show the effect that an inclination of the floor can have on the utilization of the capacities and the material use. For each step, the optimal pile locations in transverse direction of the floor (optimization 1) have to be adjusted.

Floor width = 4,5 m	D (mm)	span (m)	t,fl (m)	UC Y-Def.	UC Z-Def	UC Floor	UC Wall	UC Bending	UC Axial	UC Punch.	Total cost
H,floor = 0	457	4,5	0,45	0,38	0,18	0,29	0,14	0,77	1,00	0,94	€ **2.339**
H,floor = 0,5 m	457	4,5	0,45	0,46	0,16	0,30	0,26	0,97	0,96	0,86	€ **2.333**
H,floor = 0,7 m	457	4,5	0,45	0,51	0,14	0,30	0,40	1,11	0,92	0,84	€ **2.343**
Floor width = 4,0 m											
H,floor = 0	457	4,5	0,45	0,47	0,15	0,30	0,15	0,99	0,95	0,91	€ **2.289**

Figure 6.4: Effect on the results by inclining the floor

By giving the floor an inclination with a covering height of 0.5 meter (not to be confused with the thickness of the floor) it can be seen that the reduction of the self-weight of the soil has a decreasing effect on the axial load of the piles. Due to the increasing eccentricity of the resultant load the bending moments on the piles are considerably larger, which results in the capacity being almost reached (UC = 0.97). The effect on the material use is only marginal. By inclining the floor even more (giving it a covering height of 0.7 meter) the bending moment capacity is exceeded while. By this step also the material use increases, which can be related to an increase of reinforcement in the top part of the wall as a result of the beam-like behavior.

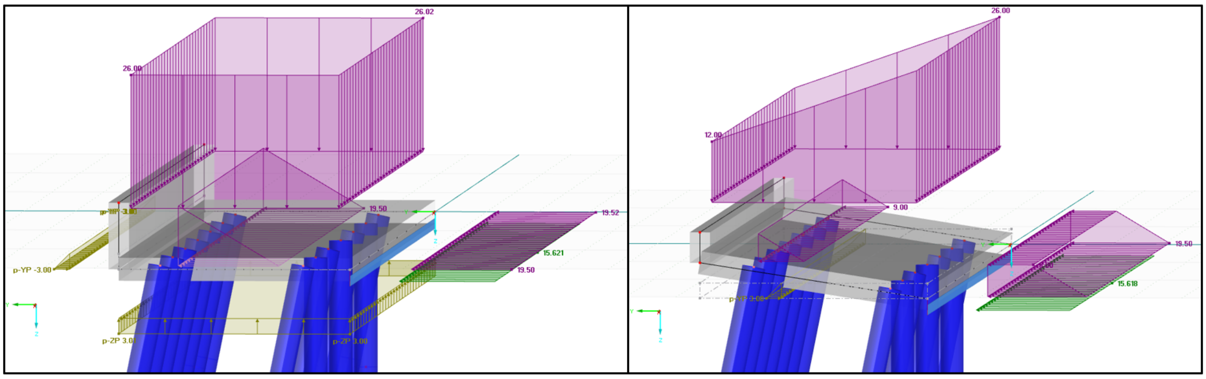

Figure 6.5: Reduction of vertical soil load due to inclination of the floor

As mentioned, a measure that also increases the eccentricity (in the case of the axial capacity being highly governing) is to reduce the width of the floor. By reducing the floor width from 4.5 meter to 4.0 meter, the effect is almost similar to the first step of inclining the floor (0.5 meter). Although for both measures only a small reduction is found for the material use, the effect of decreasing the floor width is much larger in comparison to inclining the floor. In addition, by taking into account the complexity during construction, the width taken during construction and the reduction of space for trees or cables and pipelines, the preference will always be to reduce the floor width instead of inclining the floor.

6.3.3 Soil pressure factor

The conservative assumption has been made to assume a neutral soil state, for which a soil pressure factor of 0.5 (for back-filling sand) is applied. Being conservative means that the horizontal loads that are following from the weight of the soil and the top load are assumed higher than they will be in reality. This applies for only the loads on the top structure, as for the loads coming from the sheet pile different type of soil layers and a different approach is taken into account. As the direction of the soil load and the deformation of the structure are well-predictable both approaches can be assumed as a being on the conservative side.

The actual soil pressure factor is a result of finding equilibrium in the stiffness of the soil and the stiffness of the structure. For assuming lower values of the soil pressure factor, the horizontal component of the resultant load is becoming smaller, which means the eccentricity on the structure also decreases. As the total of horizontal loads to be resisted becomes lower more slender structural elements could be used, reducing the total use of materials.

6.4 Feasibility review

The feasibility of the model is mainly dependent on the construction phasing. The assumptions that are made to allow for the realization of this structure have for some quay wall renovation projects already proven to be governing for the design solution. A list of the most common aspects that could affect the design considerations has been presented in chapter 3. Although each of the mentioned aspects could be a governing condition, the most affecting aspects could be considered as the preservation of trees and the accessibility on both land and water during construction.

The proposed construction does not deal with the preservation of existing trees. During construction the trees need to be removed. If the health of the trees allow for it, they can be replaced after the new construction is placed. However, a lot of large and older trees have a monumental value. For most of them it cannot be guaranteed that their health will not be affected by the construction process. As these trees are legally protected, removing them is not permitted. To deal with local preservation of trees, at a zone along the quay where the tree is located a different structural solution could be used. As the loads in this zone are lower than the rest of the quay (as follows from the

6.4.1 Technological feasibility

For large scale project with a highly repetitive character, the technological feasibility in terms of construction equipment can be approached from a wider angle. As it could be economically favorable to develop equipment which are custom made for its purpose, the options are not necessarily limited to the standard machinery. By saving on for example the material use, a budget can be made available for improvements in the technological efficiency.

6.4.2 Financial feasibility

Within the calculation model, only the material costs resulting from the top structure (wall and floor) and the piles are being considered. The costs for equipment, labor and other costs related to a renovation project are left out of the scope.

In a presentation[36] given at the a conference arranged by the Program of Bridges and Quay walls of the municipality of Amsterdam, a breakdown of the costs for a reference project (Herengracht) has been given. Over a renovation length of 280 meter, the total cost of the project have become € 14.77 million. From the total cost, about 21% was written of as the actual construction of the quay wall (excluded from preparatory works). Placing that with respect to the calculated material use of € 4,800 per meter from the benchmark, and adding an assumed cost contribution of € 900 per meter for the sheet piles, about 51% of the construction costs are coming from the material costs while 49% originates from the construction works (labor and equipment). From the total cost of about € 53,000 per meter quay wall, only about 10% comes from the material costs. Considering a saving of 42% following from the optimization in the benchmark, only about 4% on the total project cost can be saved. However, the implementation of a parametric approach also brings the potential to speed up processes in the multiple aspects of the project. Especially in the engineering much time can be saved, resulting in a saving on engineering costs.

In addition, the promised savings on the material and engineering costs can be mobilised for innovation and development on construction methods. By implementing more standardized construction methods and optimizing for a more effective construction phasing (for example by developing customized equipment), also the costs related to the construction works can be reduced.

6.4.3 Construction feasibility

As mentioned before, the construction aspects are highly affecting the feasibility of a design and could very likely to be governing for design considerations. However, the large scale of the renovation works could allow for more options in terms of construction phasing. With customized machinery or tools the freedom of design could be increased, as well as the construction time.

For the assumed structure, the construction method to be considered applicable and suitable is the traditional method, for which the construction phasing is indicatively presented in appendix C. The biggest downside to this method is the fact that it requires a construction pit, which generally has a large effect on the construction speed. However, as has been shown in the innovative concept Koningsgracht[10] methods could be developed which avoid the use of a construction pit.

Another complexity is found in the construction of the piles. Screwing in the piles requires the machinery to reach certain locations inside the construction pit which could be hard to access. By constructing from the pontoon, the width that can be reached could be limiting the allowed design width of the floor, especially for inclined piles. Also for this aspect, development of customized equipment could simplify the construction and allow for more design options.

6.5 Applicability and coverage

The proposed model is focused on application in utilization categories A and B (and possibly C) of the performed data analysis, with available construction widths starting from about 3 meter and high expected loads. For category C the question is whether a more simple solution is possible, as the high traffic loads are acting at a certain distance from the structure. Considering only categories A and B, this represents 61% of the considered scope of about 115 kilometers, implying this model could be applicable for about 69 kilometers of quays within the scope of the municipality of Amsterdam.

However, in case the preservation of the existing trees are not allowing for taking out and replanting of the trees, locally an alternative solution needs to be proposed. By creating overpasses of retaining elements (for instance a sheet pile or a combi-wall) in the zone of the tree and anchoring these elements to the adjacent quay wall structures, preservation of trees could be made possible.

Chapter 7

Discussion

Due to the fact that this research could not have been done without a large amount of assumptions, the discussion that it entails is also considerable. Important to mention is that the model presented in this research functions only as an indicative tool to be used in an early design stage. For that purpose a lot of assumptions are desired, especially for a parametric approach. However, the quality lies for a large part within the assumptions. To reflect on the quality of the results, the largest aspects that are prone to alternative perspectives are being nominated.

7.1 Accuracy

The accuracy of the results is affected by a lot of aspects. By each of the defined assumptions, simplifications or estimations the found results could be deviated from the actual result. To deal with this risk, the assumptions are carefully being taken by trying to be on the conservative side while not being too conservative. However, some of the assumptions taken to manage the scale of the research can be directly observed in the results. These assumptions are related to the step size of the parameters, especially the assumed cross-sections of the piles. In the research five different cross-sections are considered, for which the (interdependent) bending moment and axial capacities are defined based on a certain reinforcement configuration. Within the considered range of retaining heights and available widths (categories A and B defined in the data analysis) the optimum results are found by using piles with a diameters of 457, 508 or 610 millimeter. For certain combinations of retaining heights and floor widths, considering a cross-section in-between a diameter of 508 and 610 millimeter would have resulted in a lower use of materials.

The step sizes for the floor thickness does not much affect the accuracy of the result, as they taken relatively small (steps of 5 centimeters). Next to that, the effect of the floor thickness on the material use is relatively low.

For the span a step size of 0.5 meter is applied. Although the step size is assumed to be sufficiently small, the effect of the span on the material use is relatively high. For that reason, considering smaller step sizes would also increase the accuracy of the results. Especially for smaller diameters, for which the optimum result (satisfying all the design checks) is to be searched in combinations with smaller spans. A step size of 0.5 meter for this case has a relatively large effect compared to the same step size for large spans (in finding the optimum for larger pile diameters).

Overall, the accuracy of the results can be considered as a good indication for the optimum design dimensions. By expanding the model, a higher accuracy could be obtained. However, in an early design stage it would not be adding much value.

7.2 Construction aspects

This research is only focusing on the structural aspects and the used materials. In practice often trade-offs are made with respect to the additional costs for construction or its complexity. As the optimization of construction phasing is not taken into account within this research, it could be the case that the benefit that is being obtained by saving the use of material will not outweigh the additional costs for the complexity in construction of that design. However, as it could be applicable to a large part of the scope an optimization for the construction phasing based on the measures presented in this research could be considered. By saving a considerable amount of materials over the total scope, a budget could become available for development of machinery that enables the construction phasing. Regarding a follow-up study focusing on the construction phasing, lessons could be learned from the structural performance and the potentials for increasing the structural efficiency.

7.2.1 Type of piles

The assumed type of piles in this research (steel casing with a reinforced concrete core) can be considered as a very expensive pile, for both material use (steel casing) and construction equipment in comparison to alternative pile systems. The pile type is mainly chosen based on construction considerations, but also for the type of loads that the pile need to resist (mostly large bending moments at the top part due to eccentric loading) it can be considered as a logical option.

7.2.2 Sheet piles

The contribution of sheet piles to the material use is not taken into account. In the used simplified calculation method a minimum sheet pile depth is being found. However, this depth is defined based on the final load situation. Depending on the construction phasing (for example a load reduction during construction) the depth or the cross-section of the sheet pile can become larger.

Based on the cross-section used in the benchmark design (AZ20-700) and the minimum depths of the sheet pile resulting from the simplified method, the material costs for the sheet pile are between about € 900/meter (for a retaining height of 3.3 meter) and € 1200/meter (for a retaining height of 4.8 meter), which will be around 30% of the total material costs per meter quay wall.The assumed sheet pile force is a rough estimation, and the magnitude depends on the soil profile, stiffness of the sheet pile and the construction phasing. To validate the assumptions an extensive geotechnical finite element analysis should be performed.

7.3 Expected governing load situation

The applicability of the model as it is presented focuses on the situation for which an eccentric loading is expected. In case the available width does not limit the required amount of collected vertical load to reduce the eccentricity and make the bending moments not anymore the governing design aspect, for combinations satisfying the deformation requirements the governing design aspect turns out to be related to the axial capacity of the piles. The starting point in the first optimization round has been to find the support lines for which the sum of bending moments is minimized, assuming the pile head bending moments would be governing. When the axial capacity becomes governing, a better solution could be found when the objective for the first optimization would be set to more evenly distribute the axial loads over the piles. A real optimum will be found by iterating for both objectives.

7.4 Not taken into account

Wall The effect of the wall on the material use is assumed to be relatively small. For that reason, a constant wall thickness of 400 millimeter with a wall height of 1.3 meter is being assumed. In an extended optimization, the height and thickness of the wall can also be varied. With that, also the sheet pile should be included, as an increase of the wall height results in a decrease of the sheet pile length.

Soil profile The loads on the structure are highly dependent on the type of soil that is present behind the quay. Up to the depth of construction the back-fill material is chosen. The most favorable soil material to be used is homogeneous sand which has a high internal friction angle. The soil pressure factor can be minimized to reduce the horizontal loads on the structure. The soil type of the top layer behind the construction width has a negligible effect on the loads on the top structure (wall and floor).

However, for the soil below the level of the bottom of the floor the soil is not being filled, which means the soil characteristics are location-dependent. The soil profile has an effect on both the resistance through the pile-soil interaction and on the loads through the pressure difference over the length of the sheet pile. Weaker soil profiles in the upper soil layers result in higher lateral load effects on the sheet pile, while less lateral resistance can be found by the piles. Within this research only one soil profile is being considered, which is assumed to be an average profile for the inner city of Amsterdam.

Pile orientation Within the presented design model, for all the piles within each variant one pile diameter is being considered. As a result of optimization 1, the bending moments in the piles (for both rows) are almost equal. The distribution of axial loads over the piles depends on the orientation piles. In this research, the pile row at the canal-side and 4 out of 7 of the piles at land side are assumed to be maximally inclined. Due to the difference in orientation, the ratio of bending moment to axial compression is relatively higher for the vertical piles at land side in comparison to the inclined piles. By considering different cross-sections for vertical or inclined piles, a reduction of the material use can be achieved.

Also the orientation of the piles could be considered. As mentioned before, the amount of inclined piles has been chosen based on the the sensitivity of lateral and vertical deformations. By comparing the governing type of deformation for different retaining heights, a constant configuration of inclined piles has been chosen. Depending on the retaining height and width of the floor, a different configuration could lead to better results.

Piles - steel casing In practice, this type of pile is usually being designed for its core strength. The reinforced concrete core is assumed to resist all the loads and provide the required stiffness. The steel casing is assumed to be 'a loss'. Due to corrosion over the design life span of the structure a considerable thickness of the casing is being reduced.

If the steel casing is assumed to still have a structural contribution of the steel casing at the final stage of the life span, at construction the steel casing should be installed with a very large thickness. In that case, it could be possible to consider only the steel casing as resisting all the loads and providing the required stiffness. To save material the piles could be filled with a different material, for example sand, while only at the top of the pile a concrete filling is used to introduce the loads. By welding ridges at the inside of the casing segments, the loads could be transferred to the casing. As the bending moments decrease towards the bottom of the pile, the wall thickness of the steel casing could be decreasing to save material, as long as the minimum thickness can be guaranteed at the end of the design life.

It could also be made possible to leave the piles empty, and make a direct connection between the top structure and the piles by means of anchor plates inside the floor. As only compression forces are expected, the complexity of making such a connection is much less compared to a connection designed for tensile and compression forces. By providing additional reinforcement in the floor around the pile, the introduction of forces could be made possible.

Local loads For a complete structural verification also local loads should be considered. Within the research, a uniformly distributed load of 20 kN/m^2 is reasoned to be governing over the summation of local load effects within a global perspective. However, for determining the required dimensions and reinforcement of the wall and the floor, the effect of local loads could be governing. For the piles, the spreading of loads in the soil and within the structure reduces the effect of local loads. Considering the piles largest contribution on the material use comes from the piles, the effect of local loads on the results are expected to be limited.

The effect of incidental loads, which can be limited to collision loads coming from the canal, are expected to be the largest local loads. Although collisions are directed in the opposite direction as the permanent horizontal loads, their local effects on the wall, floor or maybe even the piles (under the structure) should be considered. The global effect for such a momentarily load will be negligible as its effect is damped through the structure and the time of loading is limited.

Seepage The sheet pile behind the structure also increases the seepage length and therewith reduces the seepage flow below the structure. An alternative solution to both resist the horizontal loads that is taken care of by the sheet pile and to guarantee flow of soil is being prevented, is to place cladding on top of an inclined slope below the structure. A filter-construction is required to block the flow of sand. To ensure the stability of the inclined canal bed, enough weight should be put in top by for instance heavy boulders. As such a filter-construction allows the flow of water no additional drainage systems need to be integrated in the design.

Cost The calculated costs are only considering the costs of materials. Costs for equipment and labour (and also time) are not taken into account. Due to the complexity in construction the costs for certain decisions could have a much larger effect on the total costs of constructing a meter of quay wall in comparison to the saving of material, at which this research is being aimed.

Construction deviations Since during the construction process the accuracy of the intended design cannot always be complied, possible deviations should be taken into consideration also within the design stage. Especially for a design which is adjusted to an expected resultant load direction, the vulnerability for design deviations is something which should be simulated. Within this research the sensitivity for different aspects are considered, but construction deviations are not taken into account.

Chapter 8

Conclusions and recommendations

Within the stated boundary conditions and assumptions, the model presented in this research can be used as an indicative tool to find an optimum in terms of the material use for the preferred variant of an urban quay wall design. It requires an input for the retaining height, the available width for the floor, and optional inputs for the height and thickness of the wall, the inclination of the piles, an inclination of the floor, the soil profile and the water level in the canal. By starting with optimization 1, the optimal pile positions in transverse direction of the floor can be found. The objective given to the optimization is the minimization of the sum of bending moment reactions over the supports. This objective follows from the assumption that within the defined scope a quay wall structure is expected to be eccentric loaded, for which the bending moments can be expected to be a critical design aspect.

The found optimal pile positions of the two rows of piles becomes input for optimization 2. By defining a range and step size for the span between the piles in longitudinal direction of the floor, for the thickness of the floor and for the considered pile diameters, an automated iterative process can be run. For each set of input combinations the input of the structural geometry, the loads, the supports and all other required model data is generated by within the Grasshopper environment, assembled by the use of the plugin Parametric FEM Toolbox and exported to the FEA software RFEM. The structural calculation is run after which the results are retrieved to the Grasshopper environment. By processing the results through the defined structural verifications (which have been set up on an indicative level) the output is generated, consisting of the utilization (unity checks) for each of the verifications and the material use for that input combination. By obtaining the results, the developed iterative tool automatically adjusts the input parameters according to the defined range and step sizes. The output is stored, from which the optimum result can be found by eliminating the results for which one or more of the structural verifications is exceeding its capacity. The lowest value of the material use from the resulting solutions is stated as the optimum for that combination of retaining height and floor width.

As mentioned, the presented results functions as an indication within the preliminary design stage. A lot of assumptions have been stated in order to manage the workability of this research. That also leaves room for improvements or continuation of the path that has been taken. At the end of this section, recommendations for practice and for further research are being suggested in order to approach the ultimate objective of finding the optimum in both design and execution for the renovation of urban quay walls.

8.1 Eccentricity

In line with the above mentioned hypothesis with respect to the eccentricity of the loads, the results indicate a governing effect of the bending moment capacity of the piles up to a certain width of the floor. Depending on the locational boundary conditions (the retaining height and the available width), the applicability of the presented model can be divided into two situations. In the context of the categories A and B (defined in the data analysis) the allowed width of the floor is usually limited, resulting in a relatively large horizontal component of the resulting load. This is referred to as an over-eccentric load situation. In this situation the results are mainly governed by the lateral effects, which are the bending moment capacity of the piles or the horizontal deformations. As the horizontal loads on the structure follow from the retaining height and can be assumed to stay constant, to reduce the eccentricity the vertical component should be considered. By increasing the vertical component the eccentricity can be reduced, for which the axial capacity of the piles are becoming more utilized and an increased bending moment capacity can be achieved.

To increase the vertical load, effects could be obtained by increasing the self-weight of the floor by using larger floor thicknesses. By this measure, also the stiffness of the floor increases, which means more redistribution of loads within the floor and more equal spreading of loads over the piles. A more effective measure in increasing the vertical loads is taken by increasing the width of the floor. In this way the vertical component is increased by both the additional weight of the concrete and (more effectively) by collecting more loads from the weight of the soil and the top loads.

On the other hand, when the width of the floor is taken relatively large and the direction of the resultant load is close to the resultant direction of the piles, a more centric load situation is present. Instead of the bending moments on the piles being governing, the more vertical-related design verifications (axial capacity of the piles, punching capacity of the floor and vertical deformations) are becoming governing. With respect to the material use, it could be seen that the optimum is also not found in this situation, but is to be found somewhere in-between the over-eccentric and the centric load situation.

To bring the direction of the resultant loads for a centric load situation closer to the optimum direction, the vertical component should be reduced. From the results, it can be seen that the model converges towards the solutions with a minimum floor thickness, for which the punching capacity of the floor and the reinforcement of the wall (mainly as a result of the beam-like behavior) are governing for the minimum floor thickness. To obtain more reduction of the vertical load, a measure that is being considered is to incline the floor to reduce the amount of soil above the structure. Although the expected effect is found in the results, the reduction follows only from the reduction of soil mass while the top loads are still being collected. By using a smaller width of the floor, the reversed effect (mentioned in the over-eccentric situation) is taken. By collecting less loads on the floor, the eccentricity of the resultant load can be most effectively increased. From comparing the results for both measures, the effect on the material use appeared to be insignificant. From a construction point of view, the preferred option would always be to reduce the width of the floor, as an inclined floor entails additional challenges considering the casting of the floor.

The results show that the optimum for ratio of the floor width to the retaining height is to be found somewhere in-between the over-eccentric and the centric load situation, at which the structural capacities for both lateral- and vertical-related design verifications are being well-utilized. Of course, when strict demands are given for the allowed width of the floor due to the accessibility of the quay during the construction, an over-eccentric load situation is unavoidable. Still, the model can be deployed for seeking the most efficient way to deal with this over-eccentric load. By indicating the effect on the material use by taking a larger or even smaller floor width, the demands for the floor width can be reconsidered. In an example, the effect of increasing the floor width from 3.0 to 3.5 meter shows a decrease of the material use by 10.8%. For the same case, a reduction of the floor width from 3.0 to 2.5 meter increases the material use by 15.3%. The effect of inclining the floor in order to reduce the eccentricity has been shown to be less effective than reducing the width of the floor. However, if the requirements would demand a certain width of the floor which is not allowed to be reduced, an inclination of the floor could be applied to obtain the desired eccentricity in order to increase the structural efficiency.

8.2 Parametric approach

Apart from the results showing a considerable potential gain in the structural efficiency, this study contributes to the proof of the beneficial effect that can be found in a parametric approach. By deconstructing a design with a certain repetitive character into a set of algorithms as a function of a set of variable input parameters, a standardization of (a part of) the design process can be made. Especially for large scale projects and/or projects with a high level of repetition much added value can be expected by using a parametric approach.

The context of Amsterdam's quay wall renovations is highly appropriate for implementing a parametric approach, allowing for the speeding up of design processes in an early stage and quick adaptation at a later stage while finding the solution with the lowest use of materials. Even though the construction phasing of renovation projects usually has the largest impact on the opted design, the parametric approach is still very well applicable. The pursuit should be to innovate for both the design aspects and the construction phasing, for which the findings should be working complementary in order to find an optimal design solution.

8.3 Improving the structural efficiency

Resulting from the most affecting parameters, a roadmap of recommended measures can be presented in order to find an optimum. In general, it can be concluded that as a result of the large contribution of the piles to the total material use, the aim is to make the largest spans in order to use the lowest amount of materials. The largest spans can only be made when the piles are maximally being utilized. As the structural verification of the piles is based on the bending capacity and the axial capacity, the utilization of the piles directly follows from the eccentricity at which they are loaded.

The inclination of the piles are the most important in dealing with the eccentric load. As the inclination is also limited due to the required space or other construction limitations, a maximum inclination of 5:1 has been assumed. The aim is to find a resultant support direction (the resultant direction of the piles) in such a way that under the resultant load the eccentricity is at a level for which both the bending capacity and the axial capacity of the piles are maximally being utilized. Since the load direction the structure has to deal with is well predictable, most of the piles can be used to increase the capacity of eccentric loading by being placed at a maximum inclination. To prevent the structure being too vulnerable for lateral deformations, a minimum of piles should be placed in a vertical orientation.

If the reachable resultant support direction still results in an over-eccentric loading, other measures should be considered. The most effective measure with respect to the material use is to increase the width of the floor. By collecting more vertical loads the resultant load direction is becoming more vertical, reducing the eccentricity while the piles are more axially loaded. Depending on the retaining height an optimum for the width of the floor can be indicated. The optimum width-to-retaining height ratio for the retaining heights of 3.3 meter and 4.8 meter is found around 1.05. However, for many cases the optimum width is not being allowed due to requirements limiting the width of the floor, for which an over-eccentric situation is considered.

The last considered measure to increase the structural efficiency of the piles, is to increase the floor width. Also in this way the vertical load is becoming larger, reducing the eccentricity. Although it is less effective than for the width of the floor, by adjusting the floor thickness some improvements can be achieved. For lower width-to-retaining height ratios relative larger floor thicknesses could be used in order to allow for larger spans. However, the difference with respect to more practical-preferred floor thicknesses are relatively small.

8.4 Recommendations

For extending the path followed in this research, the focus should be primarily on the piles and the connection to the top structure. The largest contribution to the objective as well as the governing design aspects are to be found in this part of the structure. Assumptions made within this research are approaching to a structurally efficient solution, but by refinement of the assumptions and the calculation models that are following from them, a considerable potential of efficiency is probably still to be found.

Parallel to pursuing the objective of this research, another focus of research is also (presumably more) important for the improvement of urban quay wall design. As the design options are still largely depending on the available construction methods, research should be dedicated to the construction aspects of quay wall renovations. Especially with respect to the large scale that it entails, both a lot of potential benefit is to be found as well as a lot of space that could be offered for innovation.

8.4.1 Recommendations for practice

Parametric efficiency

The parametric model developed in this research can be optimized for its own efficiency. The current average calculation time is about 32 seconds per round of (1) adjusting the input parameters, (2) export of the model alterations, (3) calculating the used load combination, (4) retrieving the calculation results and (5) processing the calculation results to the output results. Main reason is of course the interface between 2 programs and the calculation within RFEM. For requiring the full set of results with step sizes of 0.05 meter for the floor thickness between 0.4 and 1.0 meter, steps of 0.5 meter for the span between 2.0 meter and 9.5 meter, and the 5 considered diameters of the piles, a total of 975 results is given in about 8.5 hours. To improve the efficiency, an optimization tool(like Galapagos and Octupus) could be used which entails a learning character (Galapagos has been used for optimization 1). By comparing the found results, the tool can converge towards an optimum without considering all combinations.

Chasing the critical design aspects

Following the optimization approach, the governing aspects should be identified and further elaborated to find even better solutions. The piles are considered as the most critical part of the design. Either its bending capacity, axial capacity or stiffness is governing for the complete design, and also has the most effect on the material use. Based on a number of assumptions, of which mostly conservative, the capacities are being defined. To what extent these assumptions are conservative should be further investigated, as they are directly affecting the results. Especially the pile-to-floor connection has potential to be optimized. As is being explained, the top cross-section of the piles is used in a crack-width related calculation. If instead of the crack-width the ultimate capacity becomes governing, a considerable effect on the permissible load could be achieved, for which larger spans and with that a lower material use could become possible. An option could be to place the top of the casing inside the floor, preventing the cracks to evolve.

Pile type

In this research only one type of pile is being considered, a steel casing pile with a reinforced concrete core. Considering the material use this type can be assumed as not the most logical option. It is mainly chosen for executional reasons. Whether other types of piles could be used and will result in better results should be investigated. Especially considering the corrosive losses, the current pile capacity is based on only the cross-section of the concrete core (apart from the buckling capacity, for which a residual wall thickness of the pile after 100 years is taken into account). As the steel of the pile is structurally neglected, it could be beneficial to use a pile type for which the steel casing is being lifted during casting of the concrete. However, for such solutions also the effect on the soil conditions and risk for the adjacent structures should be considered. By adjusting the capacities of the pile type, the model could be functioning independently of the type of pile.

Pile offset

An option that could also be considered is to position the piles at an offset. In this research the pile configuration is considered as the piles being in the same rows (in longitudinal direction). A more stiffer solution could be possible when the piles are placed at an offset, meaning the piles in one row positioned at mid-span of the piles in the other row. In this way torsional effects are being introduced in the floor, but for larger floor thicknesses this would probably not cause issues. With respect to the construction it is more desired to position the piles without an offset. The existing piles are generally placed without an offset, and new piles should be placed in the space between them. By using an offset for the new piles the likelihood of clashing with the existing piles become larger.

Wall dimension

Within the considered inputs, the wall has been assumed as a constant with a relative low effect on the material use with respect to mainly the piles but also the floor. By increasing the height of the wall, the contribution becomes higher as the thickness and the reinforcement will also increase. As the amount of horizontal load on the global structure remains the same, increasing the height of the wall is assumed not to lead towards better results in terms of the material use. On the wall, the horizontal loads are causing mainly a large bending moment, which is taken over by the floor and transferred to the piles. By reducing the height of the wall, less of the horizontal loads are acting on the wall but more of the horizontal loads is taken by the sheet pile. As the sheet pile transfers the loads not by bending moments, but as an embedded element strutted by the floor, only part of the horizontal loads is acting on the floor and transferred to the piles while the other part is directed to the subsoil. In conclusion: to reduce the eccentricity of the loads on the structure the height of the wall (and depth of the floor) should be taken as low as possible (limited by the requirements of the free height, mainly allowing for trees, cables and pipelines), which means the sheet pile needs to be covering as much as possible of the retaining height.

8.4.2 Recommendations for research

Adjusted solution for trees

At the location where the trees are not allowed to be removed or replanted, the presented design is not possible. To make the design applicable an adjusted solution is required to locally support the soil in a different way. A bridging retaining structure could be supported by the adjacent founded quay wall structures or a self-supporting solution (like a combi-wall) could be used. In order to ensure the integrity of the different types of structures, some kind of connection is required. In addition, a soil-tight barrier between land-side and the canal should be guaranteed, which entails a lot of complexity with respect to both the design and the construction aspects.

Modular solution

Some thoughts have been given to converting the structure into a modular solution. For a design life of 100 years, the use of demountable connections is not necessarily a large added value. But apart from being in line with the objective (by allowing for the reusing of elements), the largest benefit that can be achieved is related to the construction process. In the traditional construction method the time-consumption is largely affected by the amount of construction phases. By prefabricating the top structure, the application of the formwork, placing of the reinforcement and casting the concrete at the construction site can be avoided. Even more importantly, a dry construction pit could be avoided. Only one strutted sheet pile at land-side is required to retain the soil of the quay during construction, or a different (temporary) retaining solution could be developed (for example the use of trench boxes like has been presented in the innovative concept *Koningsgracht*).

With respect to the material use, the same design solutions could be applicable. However, the complexity for a modular solution lies in the connection between the piles and the floor. By making rigid joints, some kind of expensive bolted connection is required. Another option could be to create connections which are partly demountable, by creating openings in the slab at which a grouted connection could be made. However, to be able to reuse the elements still a complex dismantling is required. And in addition, considering deviation of pile locations the elements can only be prefabricated after the definite pile locations are known.

Releasing the demand of making (fully) rigid joints, and taking into account the piles are mainly axially loaded under a very predictable resultant load direction, some kind of mortise-tenon joint could be possible. Especially when large floor thicknesses are used, the floor of the prefab element could be made with openings at the bottom, in which the piles are inserted at the construction site. In this way axial forces and shear forces can be transferred, but the piles are free to rotate, which means it should be schematized as a pinned connection. The deformation capacity of the structure will be negatively affected. However, it could be interesting to investigate the gain in construction speed in relation to a more material-efficient customized design of the prefabricated elements.

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Appendix A

Vaarwegenkaarten gemeente Amsterdam

A.1 Doorvaartprofielen gemeente Amsterdam

A.2 Doorvaartprofielen binnenstad Amsterdam

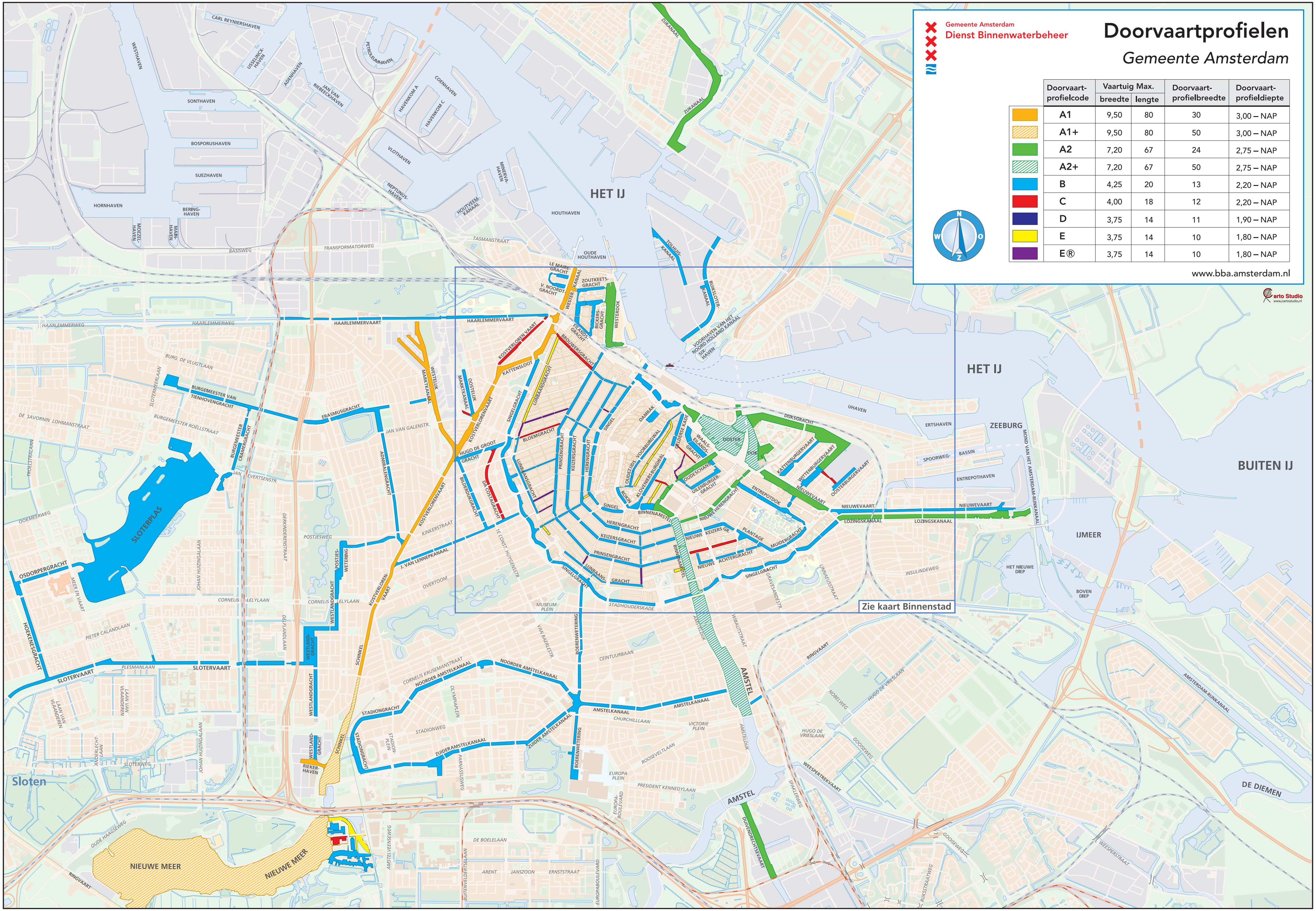
Gemeente Amsterdam
Dienst Binnenwaterbeheer

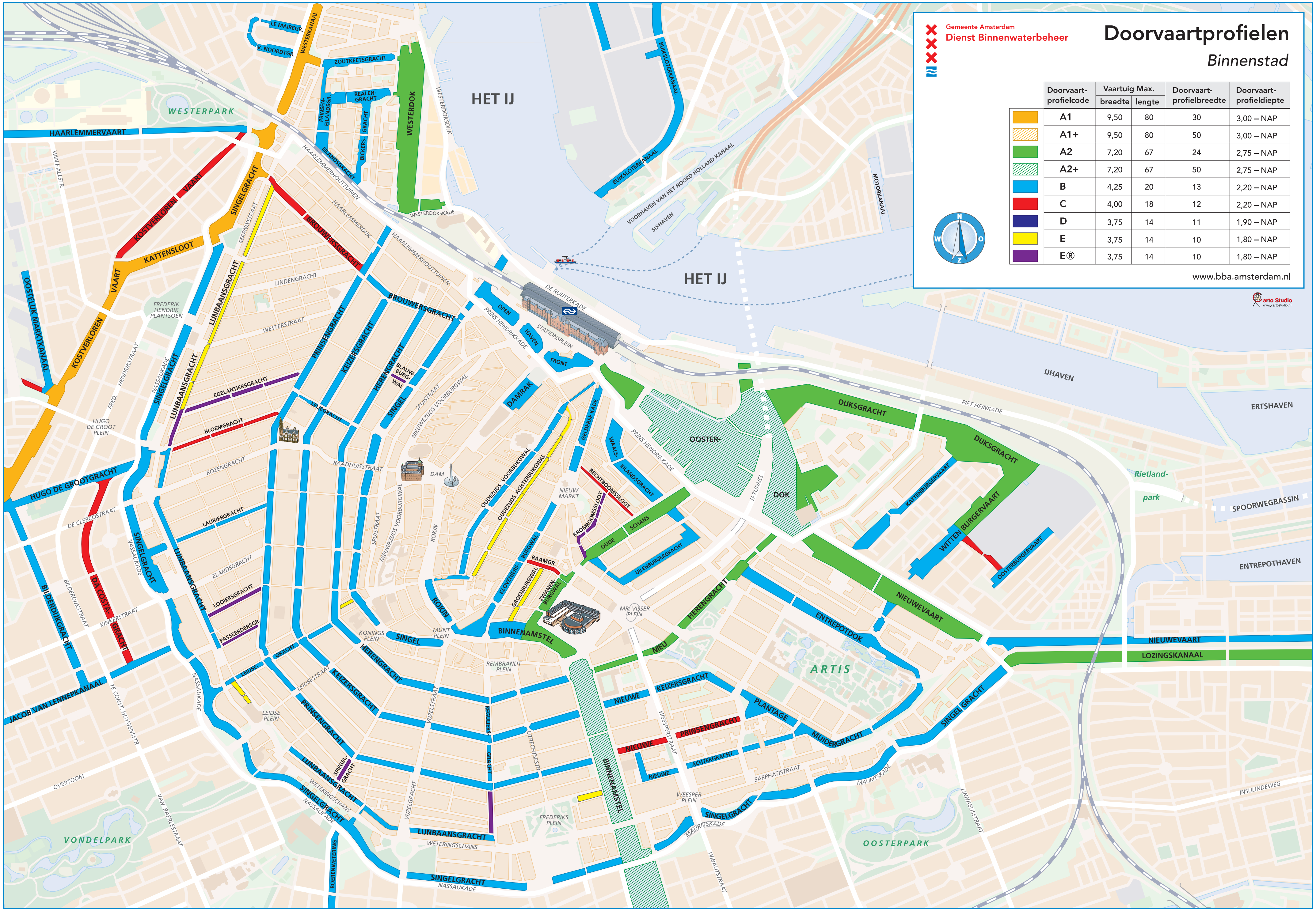
Doorvaartprofielen

Gemeente Amsterdam

Doorvaart-profielcode	Vaartuig Max. breedte	Vaartuig Max. lengte	Doorvaart-profielbreedte	Doorvaart-profieldiepte
A1	9,50	80	30	3,00 – NAP
A1+	9,50	80	50	3,00 – NAP
A2	7,20	67	24	2,75 – NAP
A2+	7,20	67	50	2,75 – NAP
B	4,25	20	13	2,20 – NAP
C	4,00	18	12	2,20 – NAP
D	3,75	14	11	1,90 – NAP
E	3,75	14	10	1,80 – NAP
E®	3,75	14	10	1,80 – NAP

www.bba.amsterdam.nl





	Doorvaart-profielcode	Vaartuig Max. breedte	Vaartuig Max. lengte	Doorvaart-profielbreedte	Doorvaart-profieldiepte
	A1	9,50	80	30	3,00 – NAP
	A1+	9,50	80	50	3,00 – NAP
	A2	7,20	67	24	2,75 – NAP
	A2+	7,20	67	50	2,75 – NAP
	B	4,25	20	13	2,20 – NAP
	C	4,00	18	12	2,20 – NAP
	D	3,75	14	11	1,90 – NAP
	E	3,75	14	10	1,80 – NAP
	E®	3,75	14	10	1,80 – NAP

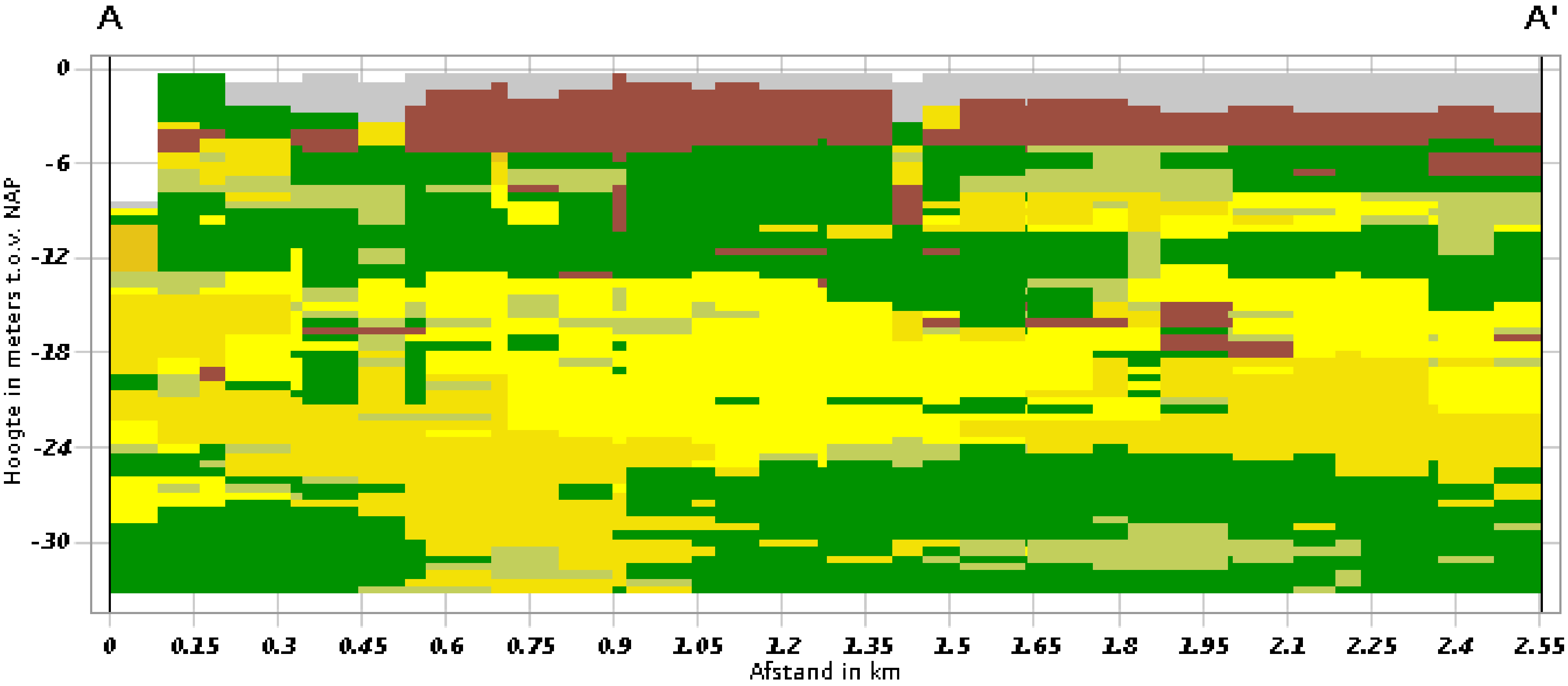
Appendix B

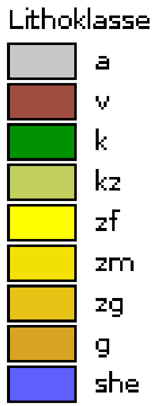
Sections soil profile Amsterdam

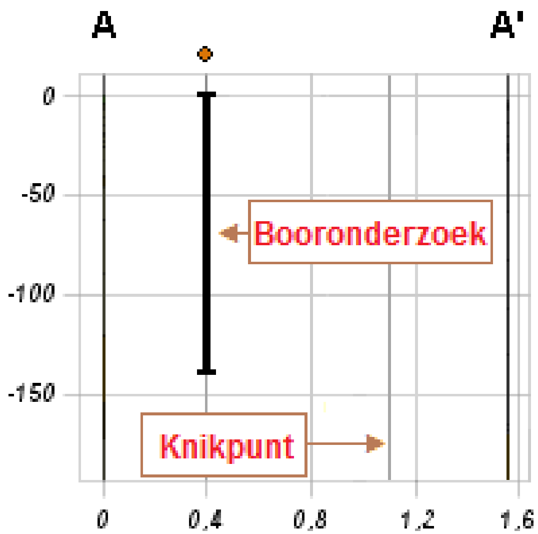
B.1 Soil profile: Section North-South central part of Amsterdam

B.2 Soil profile: Section East-West central part of Amsterdam

Verticale Doorsnede BRO GeoTOP v1.4

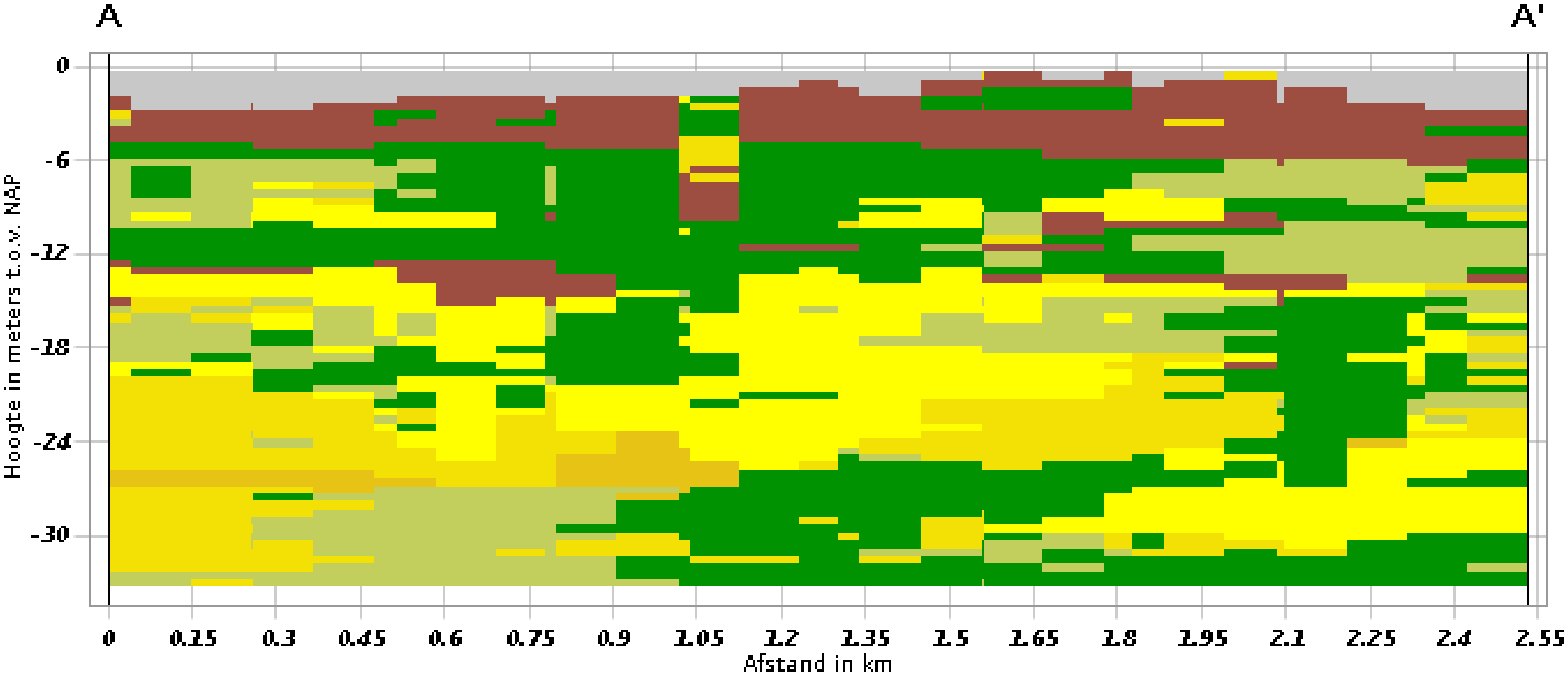


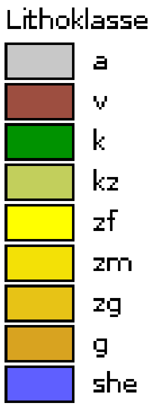

Lithoklasse
a
v
k
kz
zf
zm
zg
g
she

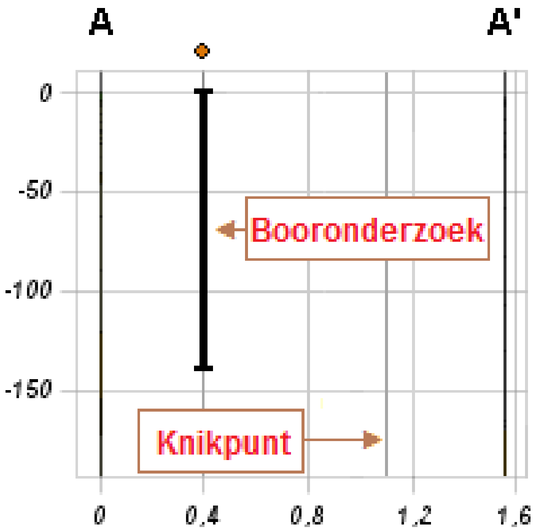

A
A'
0
-50
-100
-150
Booronderzoek
Knikpunt
0
0,4
0,8
1,2
1,6


A
A'
Droogbak
Piet Heinkade
Prins Hendrikkade
Centrum
Amsterdam
Oud-Oo

Verticale Doorsnede BRO GeoTOP v1.4



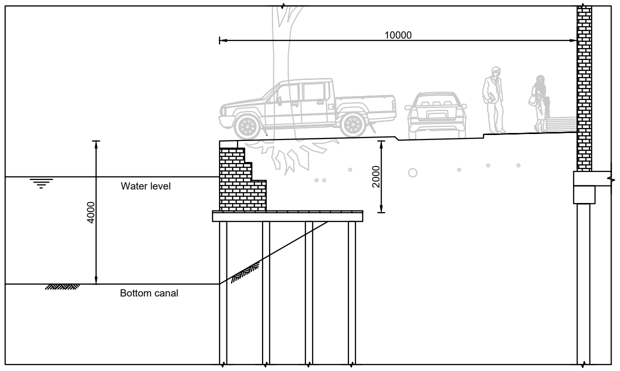

Lithoklasse
a
v
k
kz
zf
zm
zg
g
she

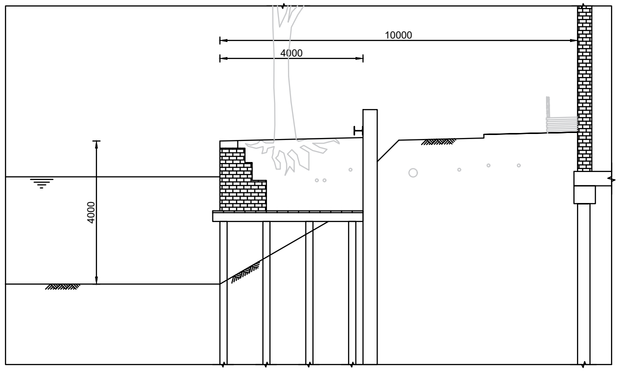

A
A'
0
-50
-100
-150
Booronderzoek
Knikpunt
0
0,4
0,8
1,2
1,6


Droogbak
Piet Heinkade
A
Centrum
Prins Hendrikkade
msterdam
A'
Oud-O

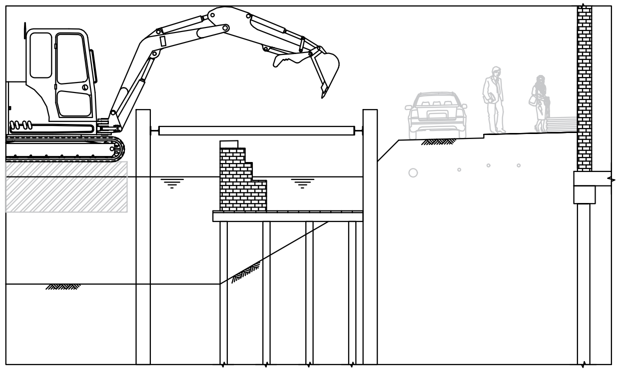
Appendix C

Indicative presentation construction phasing (Traditional method)

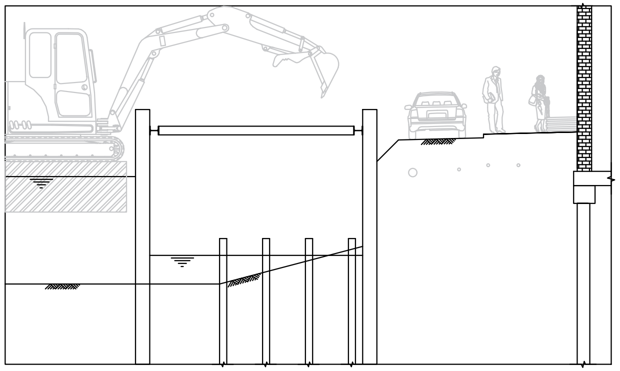




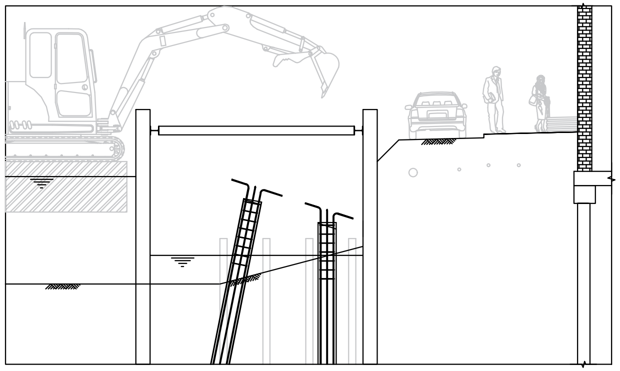
1. Driving in permanent sheet pile behind the existing structure
2. Small relieving trench at land side
3. Attach beam for temporary strutting of the sheet pile



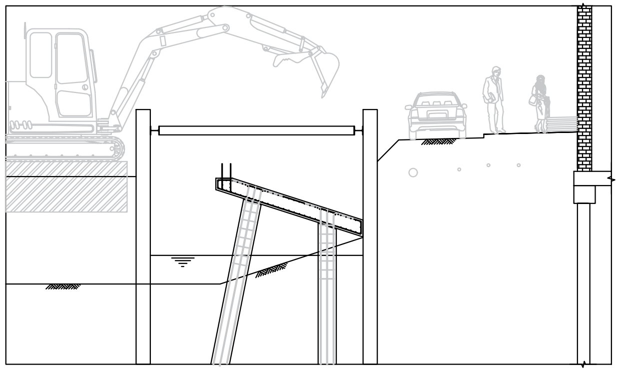
1. Extract tree from the soil if possible (for replacement)
2. Excavate soil behind the wall, replace cables/pipelines in the construction zone (temporary or permanent)



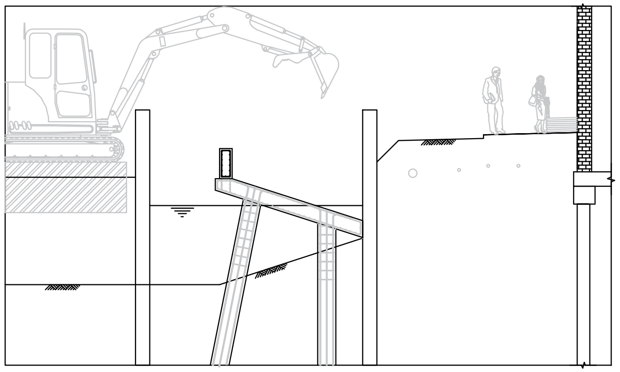
1. Temporarily lower the water table inside the construction pit
2. Removal of old structure (brick wall, floor, cutting off piles below required net construction depth



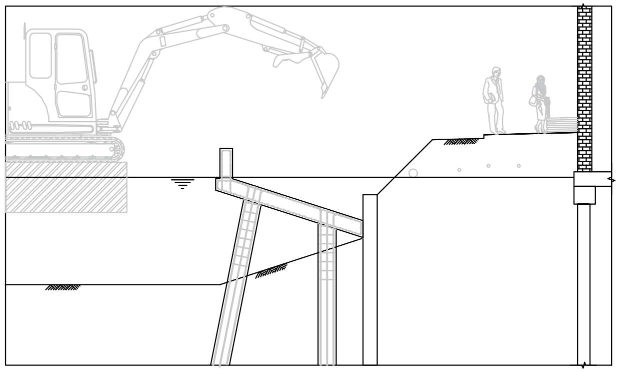
1. Driving in the new piles
2. Filling of the piles with concrete (extruding pile head reinforcement)



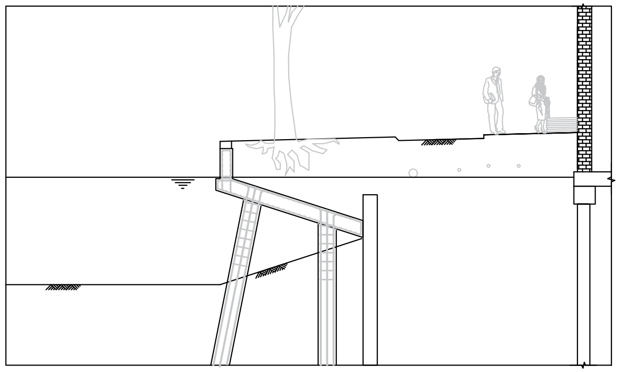
1. Placing the formwork for the floor
2. Placing the floor reinforcement
3. Casting the floor (extruding wall reinforcement)



1. Temporarily reduce the loads on the quay
2. Removing the temporary strut
3. Placing the formwork for the wall
4. Placing the wall reinforcement
5. Casting the wall



1. Cutting off the sheet pile at land-side above the floor level
2. Removing the temporary sheet pile (canal-side)



1. Place the topping stone on top of the wall
2. (Re)placing the trees
3. Restoring the level of the quay using back-fill sand

Appendix D

Indicative results for a retaining height of 3.3 meters

D.1 Floor width = 3.0 meter (Most critical design aspects)

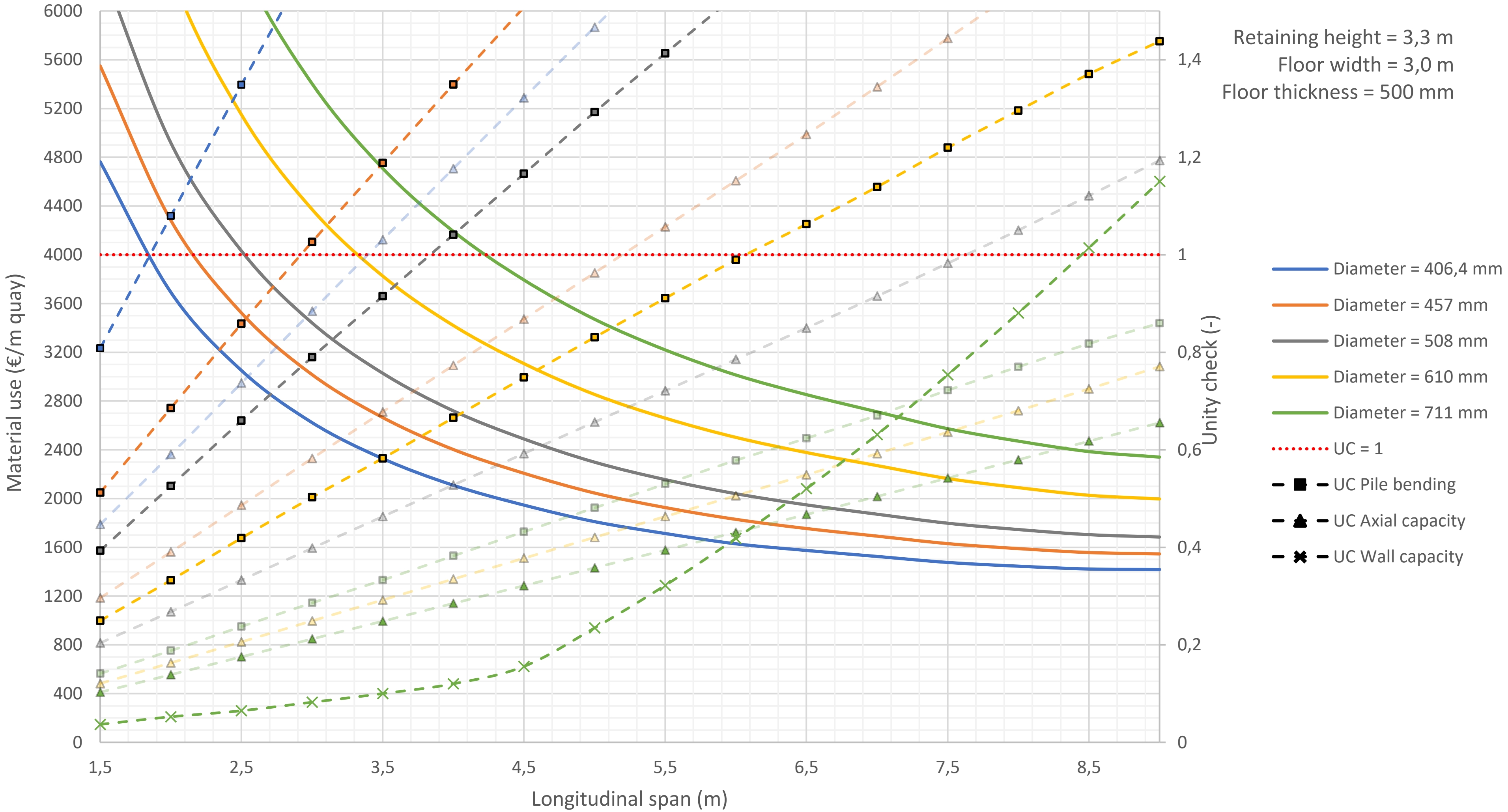
D.2 Floor width = 3.0 meter (Optimal results for each pile diameter)

D.3 Floor width = 3.5 meter (Most critical design aspects)

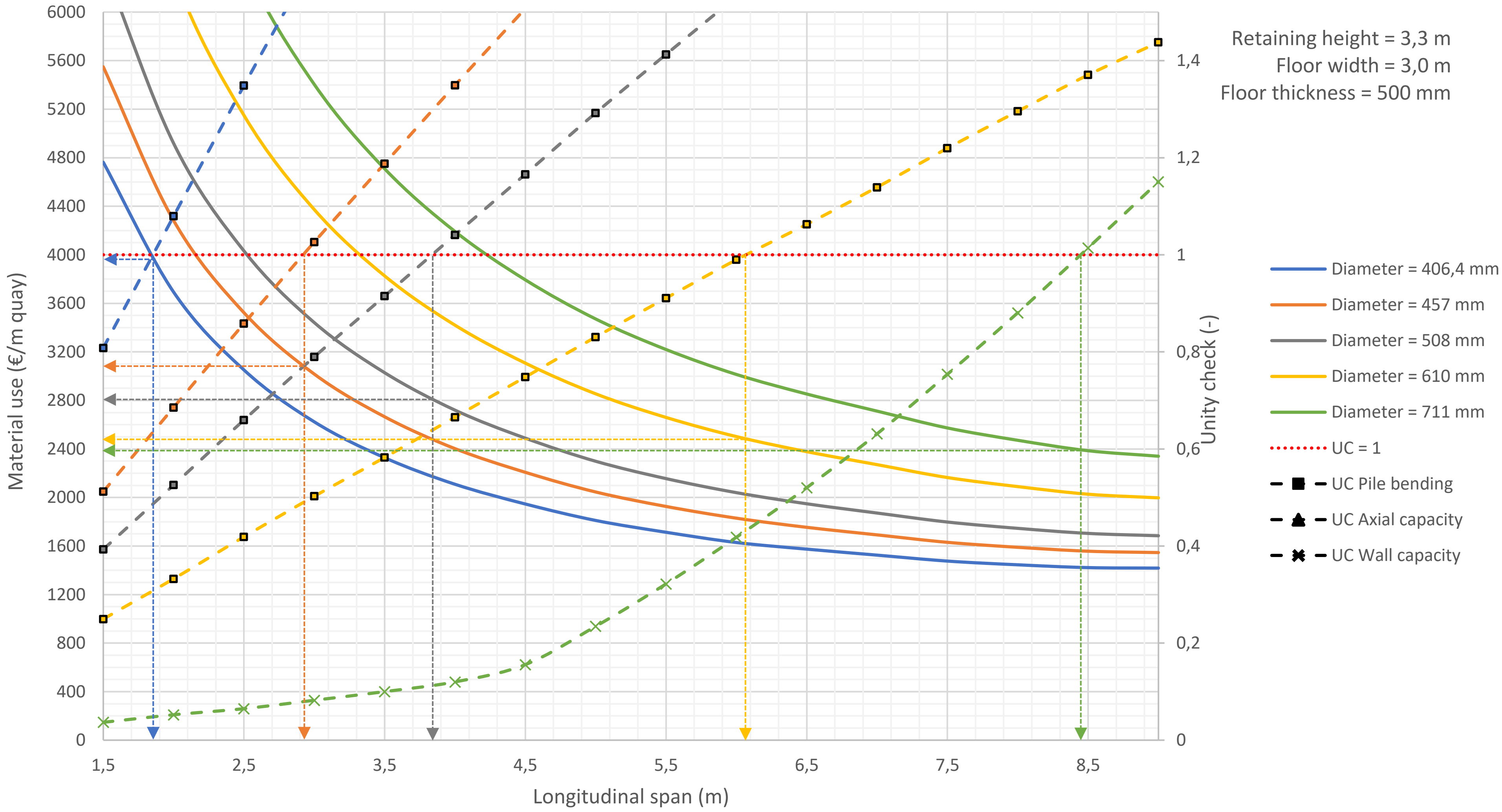
D.4 Floor width = 3.5 meter (Optimal results for each pile diameter)

D.5 Floor width = 4.0 meter (Most critical design aspects)

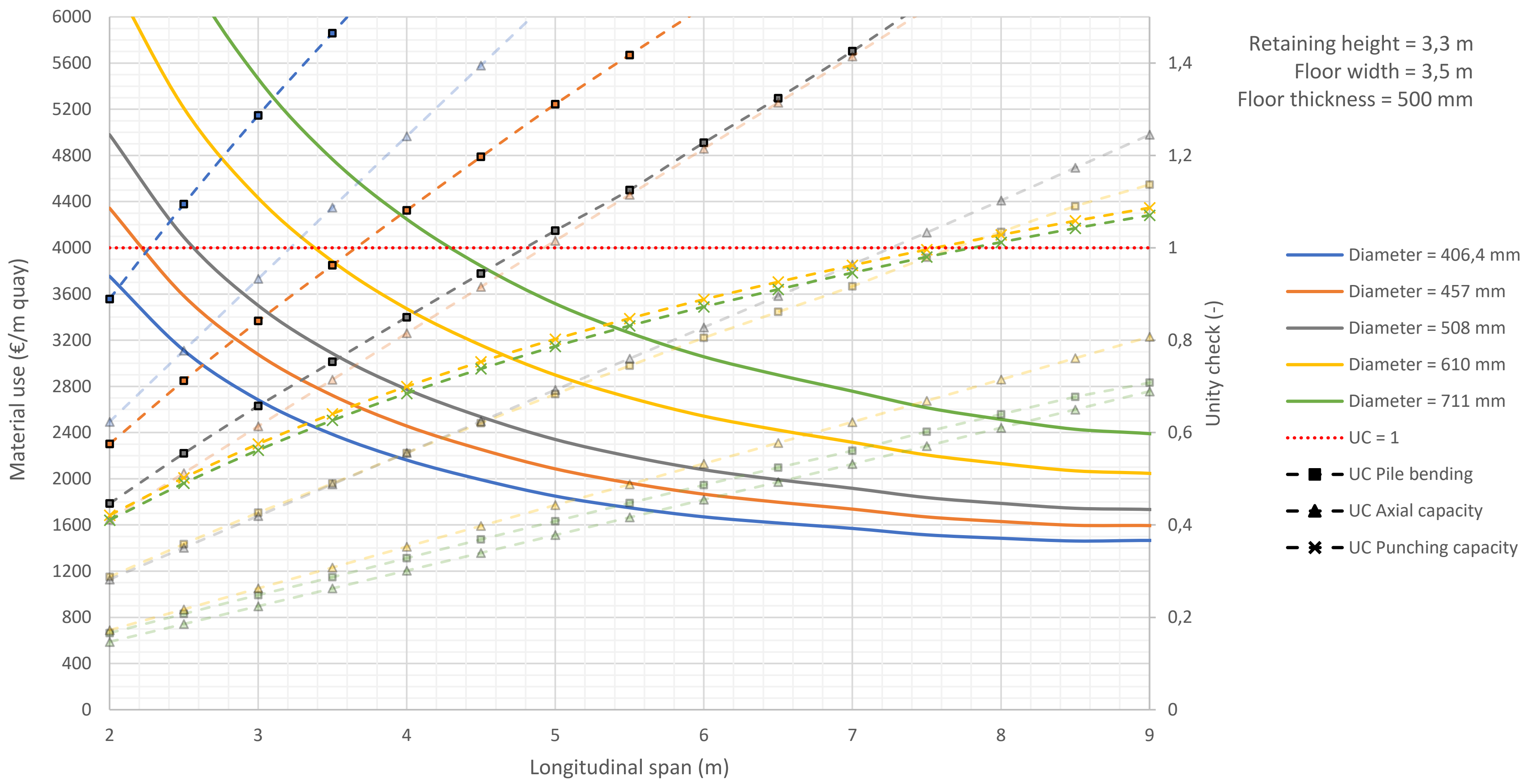
D.6 Floor width = 4.0 meter (Optimal results for each pile diameter)



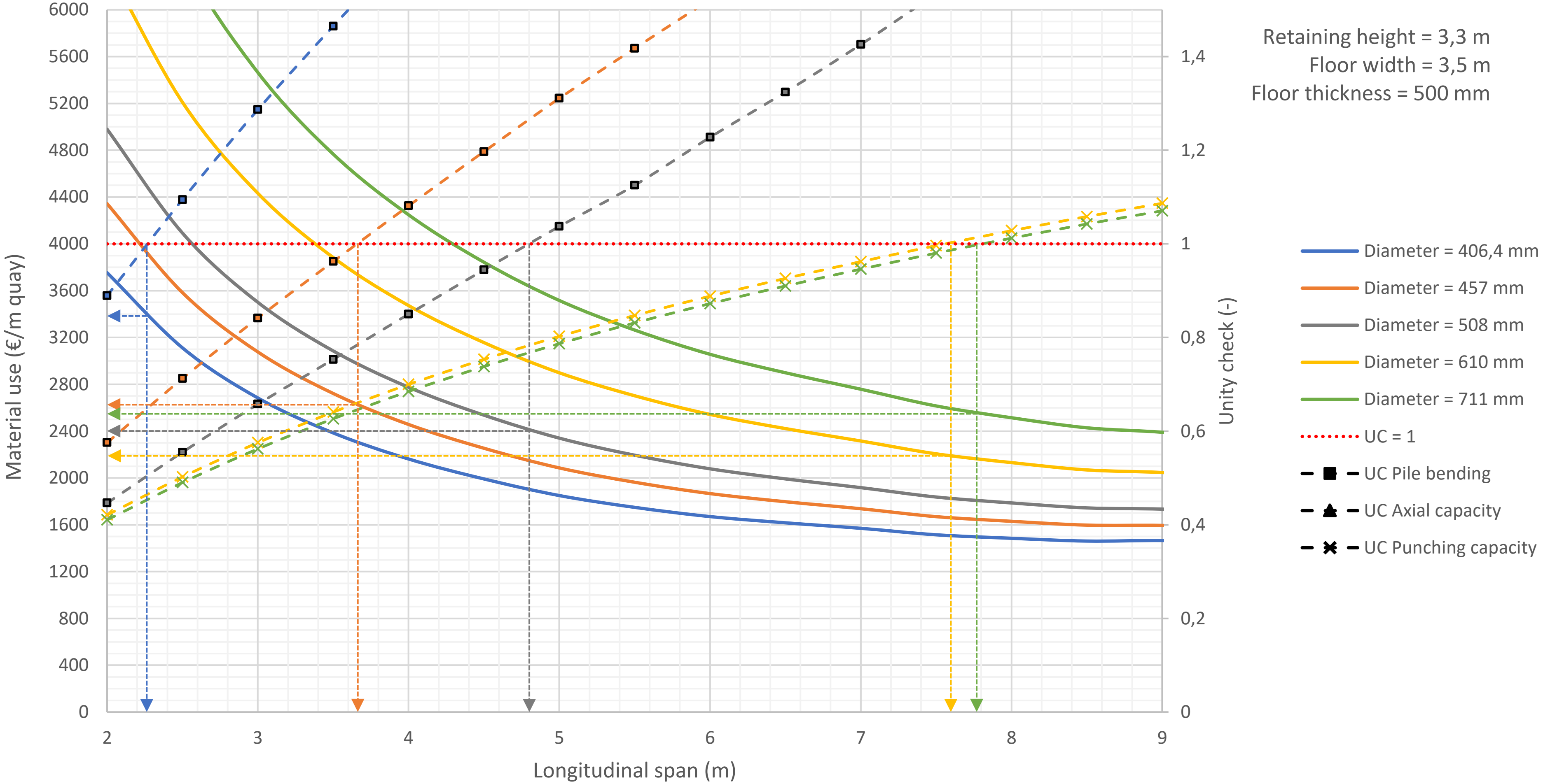
Retaining height = 3,3 m
Floor width = 3,0 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
6000
5600
5200
4800
4400
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
1,4
1,2
1
0,8
0,6
0,4
0,2
0
1,5
2,5
3,5
4,5
5,5
6,5
7,5
8,5
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Wall capacity

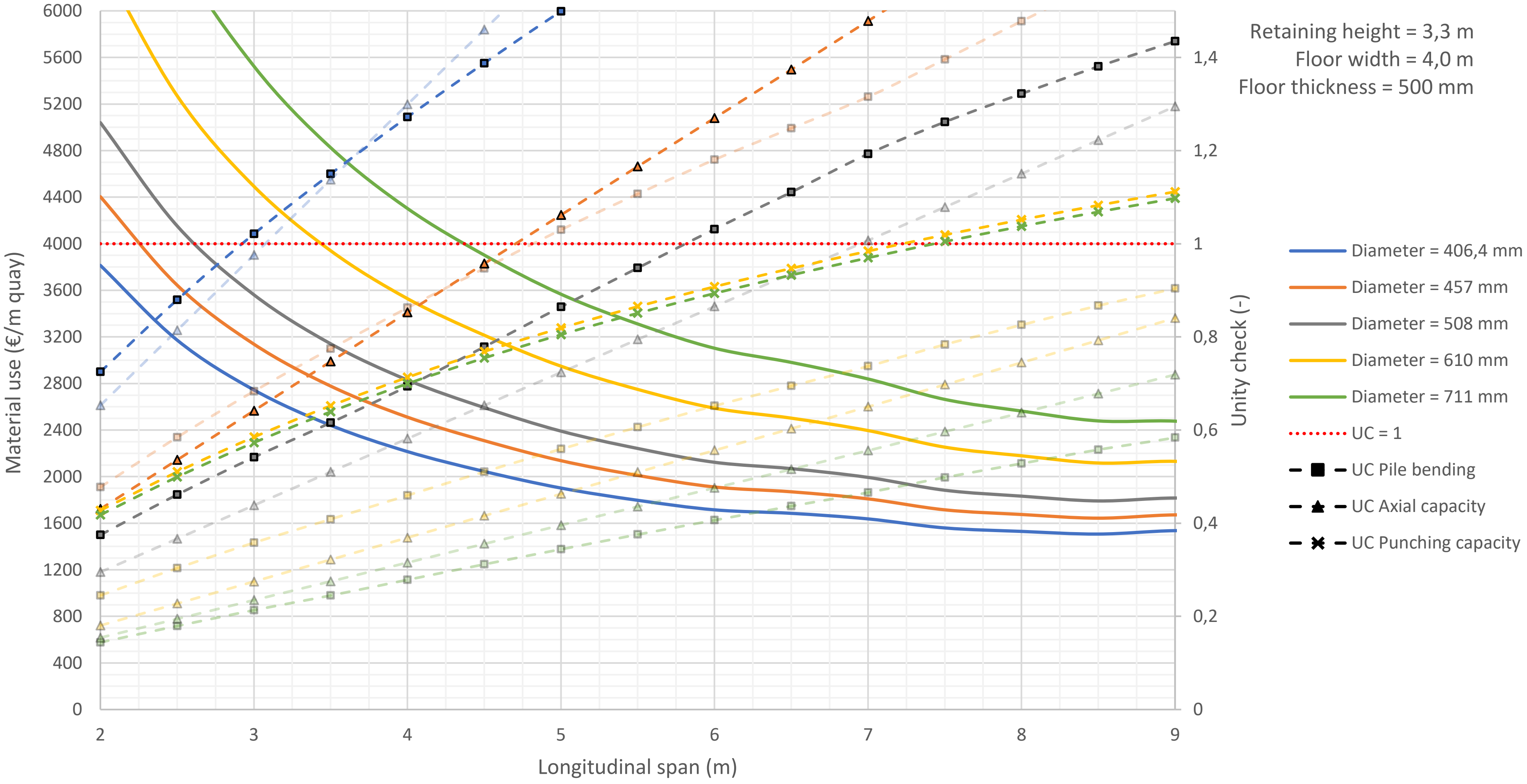


Retaining height = 3,3 m
Floor width = 3,0 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Wall capacity

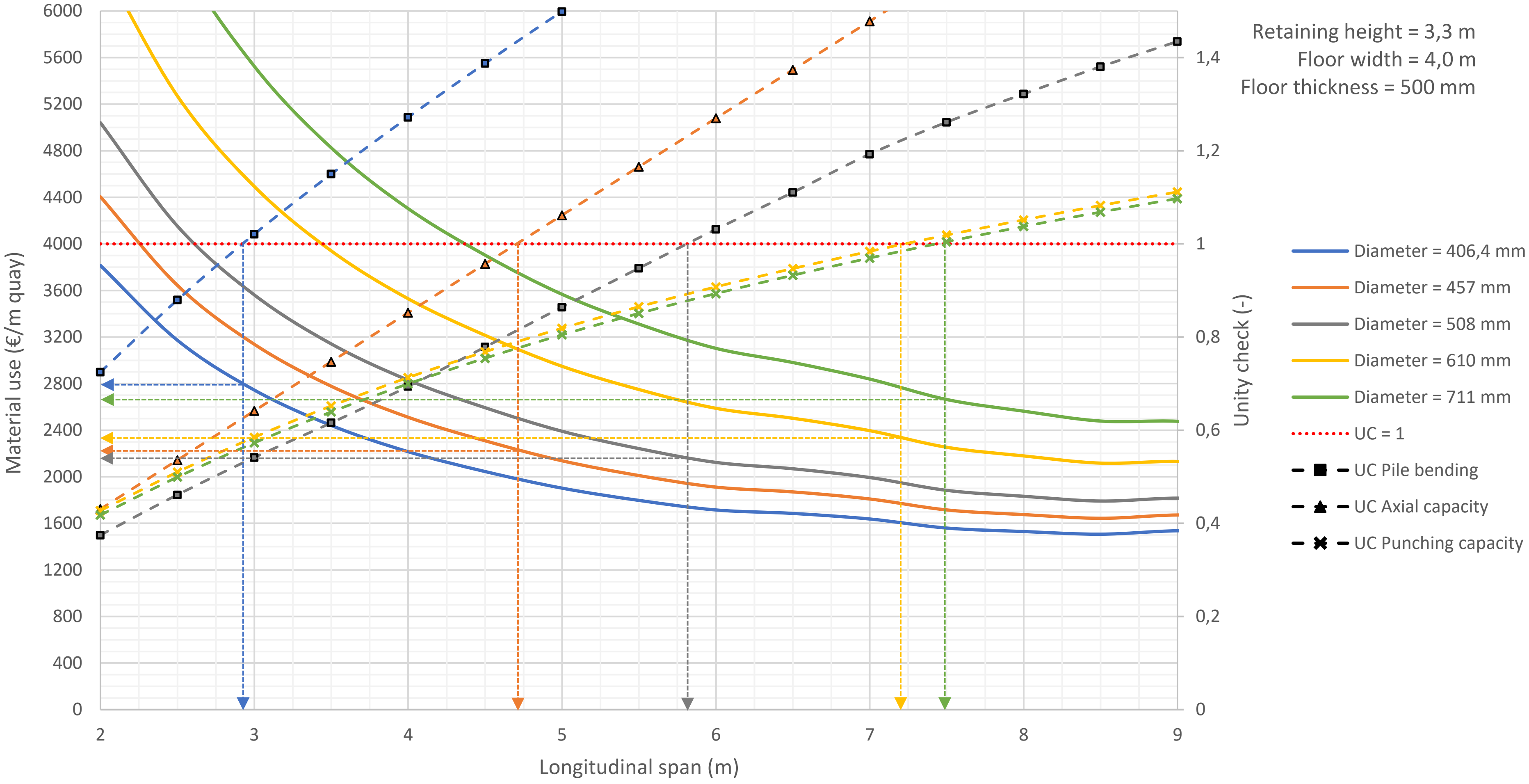


Retaining height = 3,3 m
Floor width = 3,5 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity


Retaining height = 3,3 m
Floor width = 3,5 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity



Retaining height = 3,3 m
Floor width = 4,0 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity


Retaining height = 3,3 m
Floor width = 4,0 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity

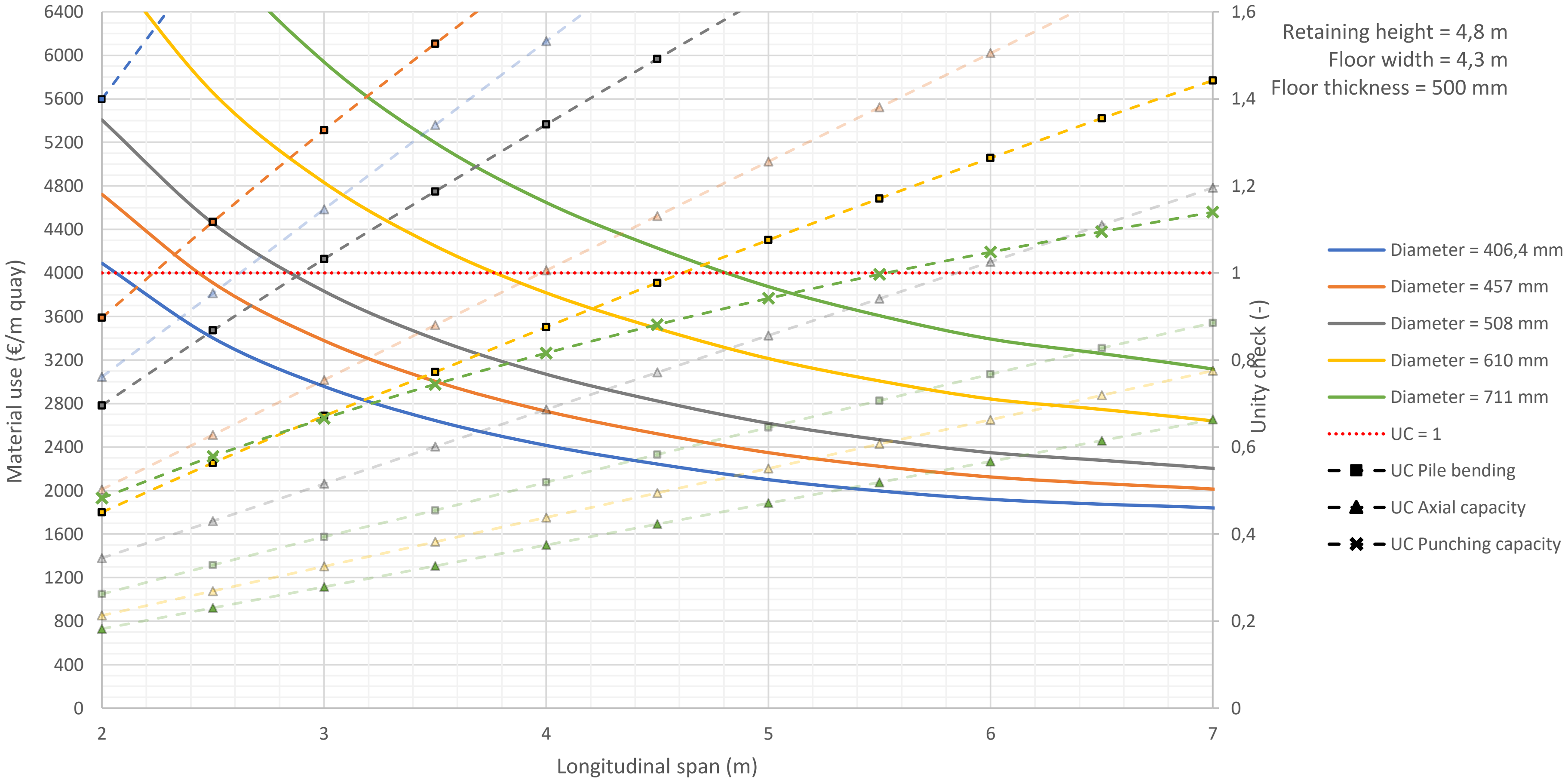
Appendix E

Indicative results for a retaining height of 4.8 meters

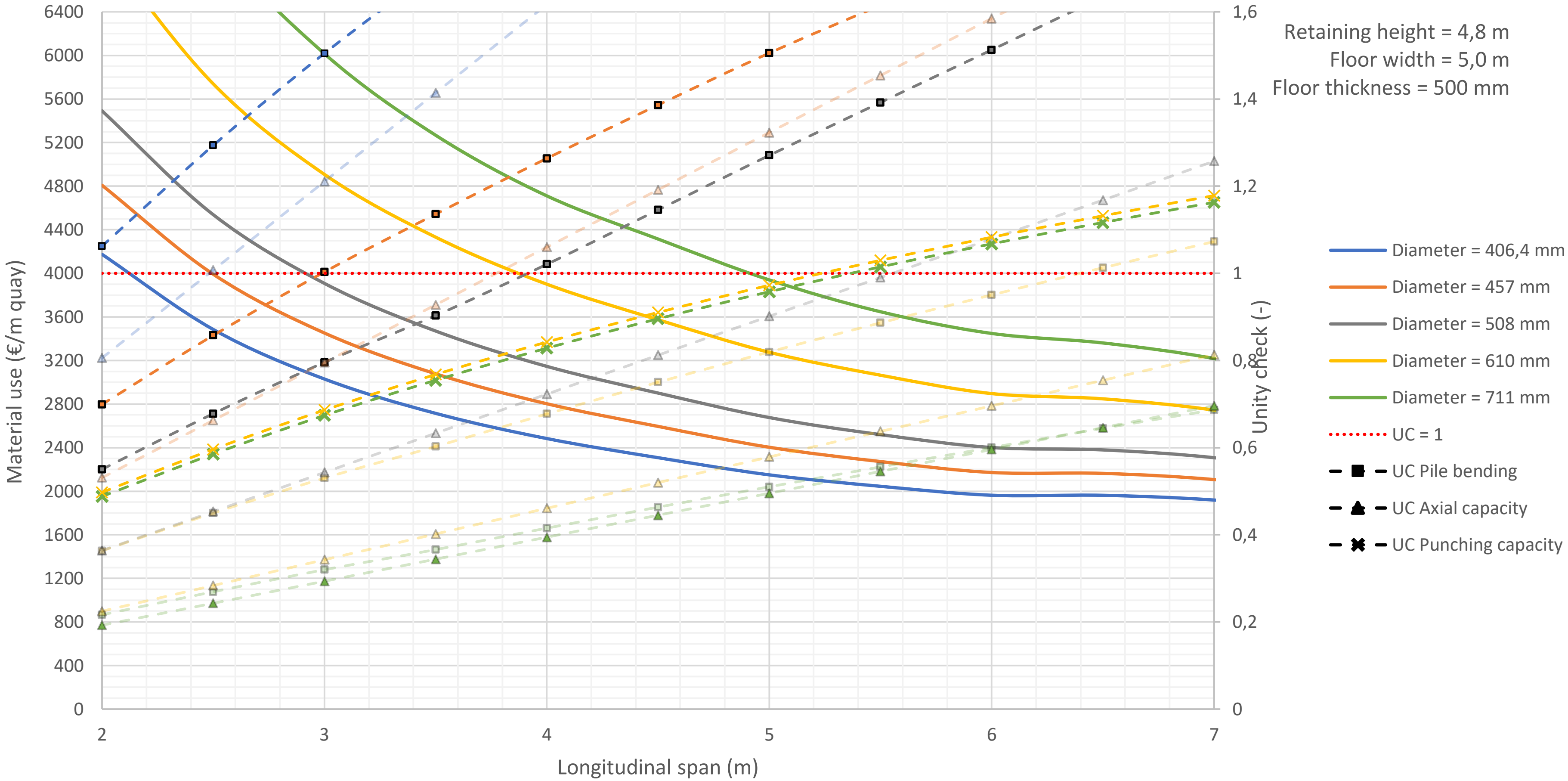
E.1 Floor width = 4.3 meter (Most critical design aspects)

E.2 Floor width = 5.0 meter (Most critical design aspects)

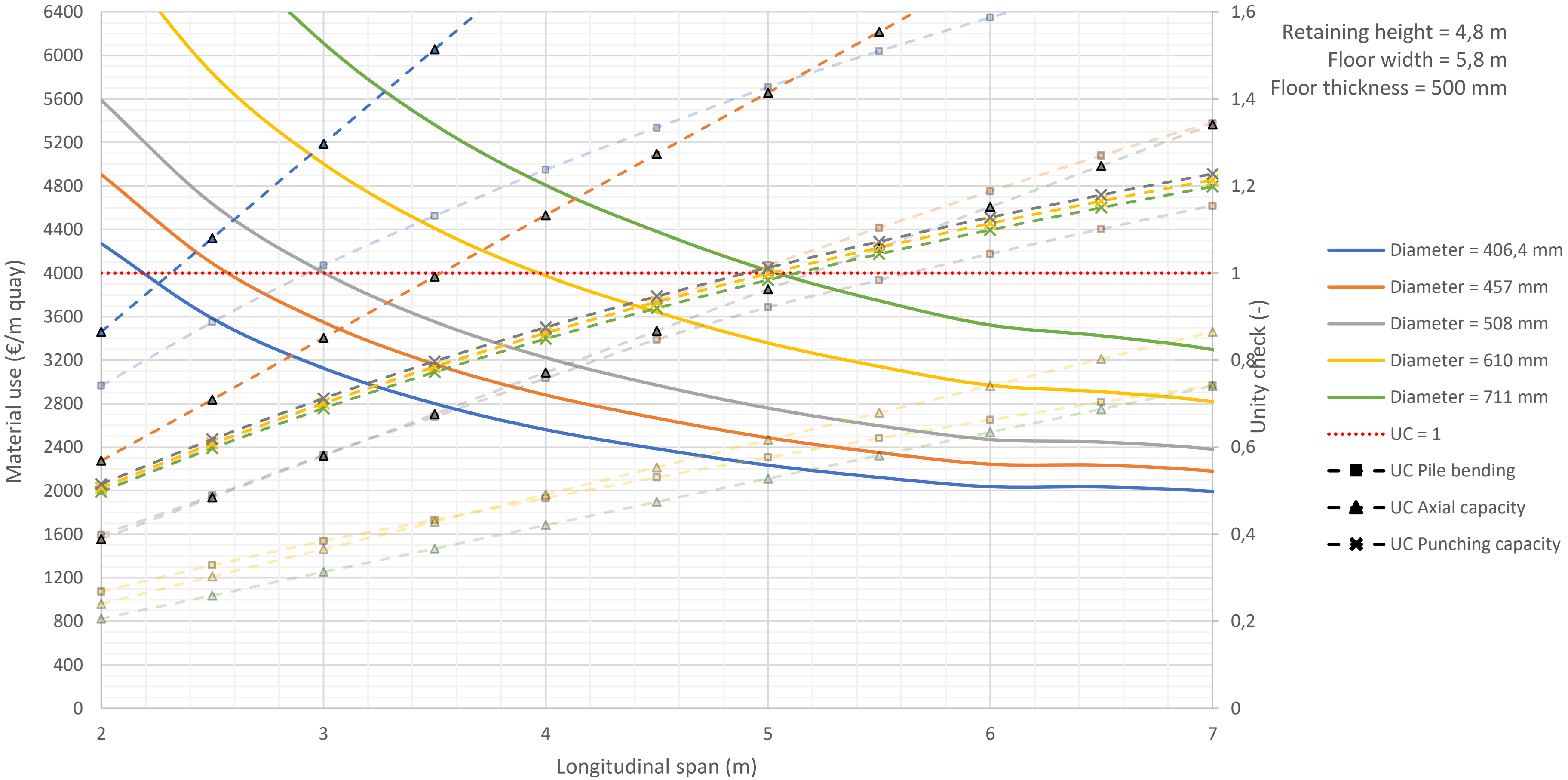
E.3 Floor width = 5.8 meter (Most critical design aspects)



Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Retaining height = 4,8 m
Floor width = 4,3 m
Floor thickness = 500 mm
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity



Retaining height = 4,8 m
Floor width = 5,0 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity



Retaining height = 4,8 m
Floor width = 5,8 m
Floor thickness = 500 mm
Material use (€/m quay)
Unity check (-)
Longitudinal span (m)
Diameter = 406,4 mm
Diameter = 457 mm
Diameter = 508 mm
Diameter = 610 mm
Diameter = 711 mm
UC = 1
UC Pile bending
UC Axial capacity
UC Punching capacity