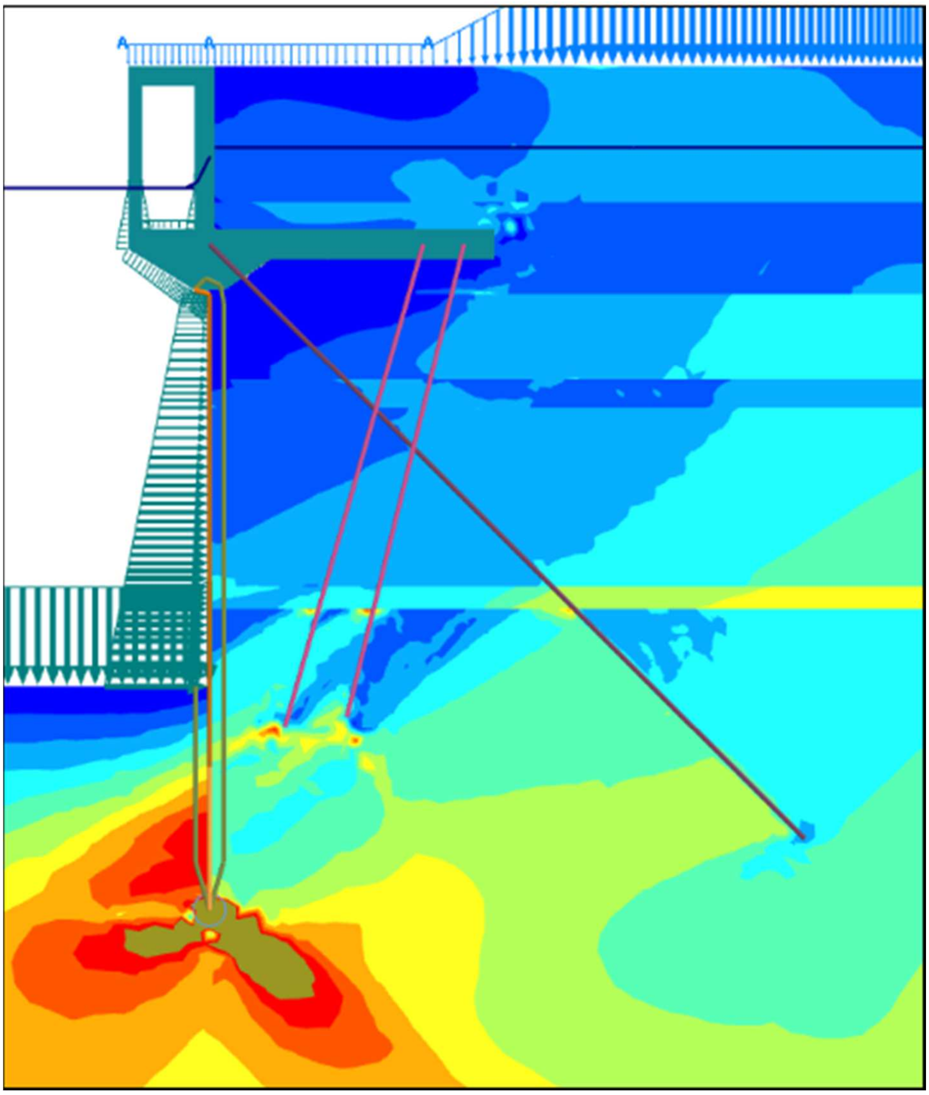
Shielding effect of piles on quay walls

Comparing analytical and finite element methods for quay walls with relieving structures.



Delft, November 2013
A. Galema

Delft University of Technology

Gemeente Rotterdam

Challenge the future

Shielding effect of piles on quay walls

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Preface

This thesis research was conducted to complete the master Civil Engineering with specialization track hydraulic engineering at Delft University of Technology. The research has been done at the company Ingenieursbureau Gemeente Rotterdam. This company provided a place and the tools to conduct this research.

The subject of this master thesis report is about design aspects for quay walls with relieving structures. Experiences and investigations from previous quay wall designs with relieving structures in the Port of Rotterdam illustrate that there is room for improvements. Measurements at quay wall sites do not comply with the expectations from designing. The shielding effect of piles and anchors on quay walls might contribute to this difference and this effect has been investigated in this research.

Graduation committee:

Prof. dr. ir. S.N. Jonkman	*Delft University of Technology, department of hydraulic engineering*
Dr. ir. J.G. de Gijt	*Delft University of Technology, department of hydraulic engineering* *Ingenieursbureau Gemeente Rotterdam (Public Works Rotterdam)*
Dr. ir. K.J. Bakker	*Delft University of Technology, department of hydraulic engineering*
Ing. H.J. Everts	*Delft University of Technology, department of geo engineering*

Contact information:

Delft University of Technology
Faculty of Civil Engineering and Geosciences
Building 23
Stevinweg 1
2628 CN Delft
P.O. Box 5048
2600 GA Delft
The Netherlands
servicepunt.tudelft.nl

Ingenieursbureau Gemeente Rotterdam
Department Stadsontwikkeling
Building Europoint III
Galvanistraat 15
3029 AD Rotterdam
P.O. Box 6575
3002 AN Rotterdam
The Netherlands
stadsontwikkeling@rotterdam.nl

Delft, November 2013
A. Galema

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I would like to express my gratitude to my graduation committee for their guidance, encouragement and useful critiques of this research work. I would also like to thank my colleagues at Ingenieursbureau Gemeente Rotterdam for their help in offering me the resources for conducting this research and for their guidance and support. Finally, I wish to thank my girlfriend Anna Baudoin and my family for their support and encouragement throughout my study.

Delft, 15 November 2013
Arjan Galema

Summary

This thesis has as subject the shielding effect of piles on the front wall of quay walls. The reason for writing this paper is to find an answer to the difference between field measurements and design calculations of stresses and deflections in the front wall of a special type of quay wall in the Port of Rotterdam. The relevance of this issue concerns the optimization of a quay wall with relieving structure to investigate the behavior of this construction and possibly to reduce costs. Focus is on the behavior of prefabricated concrete bearing piles and MV-pile anchors below relieving structures. This thesis has been prepared for obtaining the degree Master of Science in Civil Engineering at the Technical University in Delft.

The purpose of this research is to analyze and quantify the shielding effect of piles on the front retaining wall of quay walls. From measurements on the extension works for the EKOM quay wall in the Port of Rotterdam it can be concluded that the design calculations and field measurements do not comply. The calculated maximum front wall deflections are higher than the measured values and also the maximum calculated bending moments in the front retaining wall are higher than measured. These findings have been used in the design of a more recent quay wall project in the Amazonehaven by applying bending moment reduction factors for the front retaining wall. This research focusses on how the prefabricated concrete bearing piles and MV-pile anchors contribute to lower deformations and bending moments in the front retaining wall.

For this research a literature study is conducted to determine the design aspects of an existing quay wall in the Amazonehaven in the Port of Rotterdam. Furthermore the recommendations of the handbook Quay Walls, design guidelines for horizontally loaded piles, behavior of arc forces in soils and recommendations for shell factors in soils have been studied. For the research an analytical method based on Van IJsseldijk - Loof and Begemann – De Leeuw and also a finite element method (FEM) using Plaxis 2D have been applied.

The results of the analytical method have resulted in an initial input on the shielding effect of piles on the front retaining wall of quay walls. The method makes use of simplified models and ground data. Horizontal soil stress reductions near the combi-wall over the Holocene soil layers of 20% is considered maximum feasible.

The method based on FEM is prepared to make a comparison with the analytical method and to use sophisticated models and soil data for more details. The comparison shows that the horizontal soil stress reductions near the combi-wall corresponds well, but that the bending moments and pile wall deflections differ between both methods. The conclusions reached with the FEM method has led to a shielding effect of the piles on the maximum field bending moments in the combi-wall of 13%. The MV-pile anchors have a small to negligible influence on the shielding effect.

As conclusion it can be said that for the design of quay walls with relieving structures the shielding effect of piles on the front retaining wall is part of the design. Reductions of 13% are achievable according to the FEM analysis.

It is advised to design quay walls with relieving structures with FEM. This provides a better picture of how relieving structures work and the shielding effect of piles can be modeled in more detail. It is also recommended to conduct more field research to determine the shielding effects for different quay walls designs.

Samenvatting

Deze afstudeerscriptie heeft als onderwerp de afschermende werking van palen op de voorwand van een kademuur. De aanleiding tot het schrijven van deze scriptie is om een verklaring te kunnen geven voor het verschil tussen veldmetingen en ontwerpberekeningen van spanningen en doorbuigingen in de voorwand van een kademuur in de haven van Rotterdam. De relevantie van dit onderwerp betreft het optimaliseren van een kademuur met ontlastvloerconstructie om het gedrag van deze constructie te onderzoeken en mogelijkerwijs de kosten te kunnen reduceren. Hierbij ligt de aandacht bij het gedrag van de palenrijen onder de ontlastvloerconstructie. Deze scriptie is opgesteld voor het behalen van het diploma Master of Science in de Civiele Techniek aan de Technische Universiteit te Delft.

Het doel van dit onderzoek is om de afschermende werking van de palenrijen op de voorwand van een kademuur in kaart te brengen en te kwantificeren. Hierbij een verklaring te kunnen geven voor het verschil tussen de veldmetingen en ontwerpberekeningen voor een kadermuur in de haven van Rotterdam. Bij één van deze kademuren is gemeten dat in de voorwand lagere buigende momenten voorkomen dan dat was aangenomen tijdens het ontwerpproces.

Als methode is er een literatuurstudie opgesteld aangaande een bestaande kademuur in de Amazonehaven in de haven van Rotterdam. Tevens zijn de aanbevelingen van het handboek Kademuren, de ontwerprichtlijnen voor horizontaal belaste palen, boogwerking in grond en de aanbevelingen omtrent schelpfactoren in grond toegelicht. Voor het onderzoek is gebruik gemaakt van een analytische methode volgens Van IJsseldijk – Loof en Begemann – De Leeuw en daarnaast van een eindige elementen methode (EEM) met behulp van Plaxis 2D.

De resultaten van de analytische methode hebben geleid tot een eerste inzicht omtrent de afschermende werking van palenrijen op de voorwand van een kademuur. De methode maakt gebruik van vereenvoudigde modellen en grondgegevens. De effectieve horizontale grondspanning voor de combiwand kan met 20% afnemen door de afschermende werking van de palenrijen.

De methode gebaseerd op EEM is opgesteld om een vergelijking te kunnen maken met de analytische methode en om geavanceerde modellen en grondgegevens te kunnen gebruiken. Uit de vergelijking blijkt dat de reductie van de effectieve horizontale grondspanningen voor de combiwand goed overeenkomen, maar dat de buigende momenten en palenrijverplaatsingen afwijken tussen beide methodes. Het onderzoek met het EEM-kalibratiemodel hebben ertoe geleid dat de buigende momenten in de combiwand met 13% kunnen worden gereduceerd. De invloed van de MV-paal ankers op de afschermende werking is klein tot verwaarloosbaar.

Geconcludeerd kan worden dat voor het ontwerpen van kademuren met ontlastvloerconstructies de afschermende werking van palenrijen onderdeel zijn van het ontwerp. Aan de hand van de EEM zijn buigende-momentenreducties tot 13% in de voorwand haalbaar gebleken.

Geadviseerd wordt om kademuren met ontlastvloerconstructies met behulp van EEM methodes te ontwerpen. Deze methodes geven een beter beeld van de werking van ontlastvloerconstructies en van de afschermende werking van palenrijen. Meer veldmetingen worden ook aanbevolen om een beter beeld te krijgen van de reducties die in de praktijk haalbaar zijn.

Table of contents

List of symbols

ϕ	angle of internal friction	[°]
τ	shear friction	[kN/m^2]
c'	effective cohesion	[kN/m^2]
σ_n'	effective soil pressure in normal direction	[kN/m^2]
d	thickness soil layer	[m]
σ_v	vertical soil pressure	[kPa]
σ_v'	effective vertical soil pressure	[kPa]
γ	unsaturated volumetric weight of soil	[kN/m^3]
γ_{sat}	saturated volumetric weight of soil	[kN/m^3]
E_{50}^{ref}	secant stiffness in standard drained triaxial test	[kN/m^2]
E_{oed}^{ref}	oedometer stiffness for reference stress	[kN/m^2]
E_{ur}^{ref}	unloading-reloading soil stiffness	[kN/m^2]
c_u	undrained shear strength	[kN/m^2]
ψ	dilatation angle	[°]
θ_a	angle of active sliding plane	[°]
δ	angle of wall friction	[°]
$k_{x,y}$	soil permeability in x-,y-direction	[m/day]
R_{int}	interface strength ratio ($R_{int} \approx \tan(\delta) / \sin(\phi')$)	[-]
$\gamma_{0,7}$	shear strain (at strain shear modules reduction of 70%)	[-]
G_0	shear modules for very small strains	[kN/m^2]
λ^*	modified compression index	[-]
κ^*	modified swelling index	[-]
μ^*	modified creep index	[-]
α	obliqueness front retaining wall	[°]
β	obliqueness ground level	[°]
K_a	active coefficient of horizontal earth pressure	[-]
K_0	neutral coefficient of horizontal earth pressure	[-]
K_p	passive coefficient of horizontal earth pressure	[-]
σ_{xx}	horizontal soil stress	[kN/m^2]
σ'_{xx}	effective horizontal soil stress	[kN/m^2]
σ_{zz}	vertical soil stress	[kN/m^2]
σ'_{zz}	effective vertical soil stress	[kN/m^2]
M	bending moment	[kNm]
W	section modules	[m^3]
α	kinematic factor for pile stiffness	[°]
β	kinematic factor for pile bending moments	[°]
S	shell factor	[-]
D	diameter or width pile	[m]
$I_{p(ile)}$; I_{beam}	moment of inertia pile	[m^4]
E_{conc}	modules of elasticity of concrete	[N/mm^2]
a	center to center distance piles	[m]
P; q; S	distributed load or surcharge load	[kN/m]
U	water pressure	[kPa]
R	number of pile rows	[-]
β_{supp}	kinematic factor support bending moment	[-]
β_{field}	kinematic factor field bending moment	[-]

u_3	intersection displacement pile and soil	[m]
$\sigma_{3,pile}$	effective horizontal soil stress on pile per meter width	[kPa]
$\sigma_{3,soil}$	effective horizontal soil stress per meter width	[kPa]
q_{pile}	horizontal load on pile ($\sigma_{3,pile}$*D_{pile}*S)	[kN/m]
M_b	pile bending moment at support concrete relieving structure	[kNm]
M_{field}	pile bending moment at field	[kNm]
z	internal leverage arm concrete pile, $z \approx 0{,}9*0{,}9*D_{pile}$	[m]
ρ	concrete reinforcement ratio	[%]
f_{yd}	design tensile strength reinforcement	[N/mm^2]
M_{max}	maximum moment in reinforced concrete pile	[kNm]
$\sigma_{xx,eff}$	remaining effective horizontal soil stress per meter width	[kPa]
Red.	reduction horizontal soil stress	[%]
A_{conc}	area of concrete pile	[m^2]
G; w	weight per length; weight per volume	[kN]
EA	normal stiffness	[kN]
EI	bending stiffness	[kNm^2]
ν	Poisson’s ratio	[-]
$L_{spacing}$	center to center distance piles and anchors	[m]

1 Introduction

1.1 General introduction

Quay walls are built to facilitate the transshipment of cargo. In the Port of Rotterdam they are designed for mooring large seagoing vessels and inland barges. In some cases quay walls also store a substantial amount of cargo near the waterfront, for example large piles of iron ore and coal. These surcharge loads can be so extreme that a specific type of quay wall design is required: quay walls with relieving structures.

During the lifetime of quay walls many different loads act on the structure. The important loads are: earth pressures, water level pressure differences, cranes, storage, berthing forces of vessels, collision forces and dynamic forces such as earthquakes. These can be divided into three categories: constants, variable and accidental loads. Constants loads are considered to act quasi-constant during the lifetime on the structure like earth pressures, water level pressure differences and storage. Variable loads are partially of the lifetime acting on the structure, for instance berthing forces and cranes. Collisions and earthquakes are typical accidental loads, which have a very low frequency of occurrence. During designing accidental loads must be taken into account to minimize the risks of economic damage or the loss of life.

Quay walls in the Port of Rotterdam are mostly designed for a lifetime of at least 50 years and are based on reliability class two. This implies quay walls with retaining heights larger than five meter, dedicated to seagoing vessels and not being part of primary flood defenses. Reliability class two is defined by European regulations and codes when the risk of economic damage is high and the danger to life negligible. It describes how the quay wall should be designed and gives the designer the opportunity to optimize the construction in terms of risks and building costs.

In the Port of Rotterdam quay walls with relieving structures have been built where the surcharge load on the quay is high. Applying relieving structures often result in economically good designs, because the loads acting on the front retaining wall are reduced. A downside of quay walls designs with relieving structures is the increased complexity as compared with traditional quay wall designs. The designs include more structural foundation elements and as a result the complexity will increase. To support relieving structures at the backside of quays prefabricated concrete bearing piles are driven into the subsoil. During the lifetime of quay walls, soil settlements and horizontal soil displacement are expected. As a consequence, complex interactions between the piles and the displaced soil occur. The piles resist the soil displacements and as a result may interact with the forces acting on the front retaining wall. This effect is called the shielding effect and it is not limited to the presence of prefabricated concrete bearing piles; anchors may also be prone to shielding.

From measurements on the extension works for the EKOM quay wall in the Port of Rotterdam it can be concluded that the design calculations and field measurements do not comply (Gijt, J.G. & Winter, W.J. de, 1988). The calculated maximum front wall deflections are higher than the measured value and also the maximum calculated bending moments in the front retaining wall are higher than the measured value. The shielding effect of piles and anchors may explain this difference.

In a more recent quay wall project in the Amazonehaven, also including a relieving structure with prefabricated concrete bearing piles, bending moment reduction factors on the front retaining wall have been applied to incorporate for this difference. The support bending moment in the front retaining wall derived from design calculations was reduced with 10% whilst the maximum field bending moment was reduced with 25%.

The applied solution for the quay wall project in the Amazonehaven is the direct consequence of the experience learned from the extension works on the EKOM quay wall. However, the shielding effect of the prefabricated concrete bearing piles and MV-pile anchors has not been studied yet. In order to understand the shielding effect of piles and anchors on quay walls, first the principles of quay walls with relieving structures is explained in the next paragraph. Then in section 1.3 the problem description is formulated followed by the research question in section 1.4. Finally the scope of the thesis is described in section 1.5.

1.2 Design of quay walls with relieving structure

Quay wall designs without relieving structures can become expensive in the case of high surcharge loads near the waterfront and especially in combination with large earth retaining heights. The forces on the front retaining wall would become high and as a consequence large structural dimensions for the front retaining wall would be required. This results in large material costs and an expensive design. Therefore in some cases it can be beneficial to adapt the design by either reducing the surcharge load or by reducing the earth retaining height.

Quay wall designs with a relieving structure reduces the earth retaining height of the original front retaining wall by creating an additional retaining wall on top; the relieving structure. The structure also lowers the earth pressures acting on the front retaining wall. Relieving structures behave in the same way as an L-shaped gravity wall. The structure is supported by the front retaining wall near the waterside and by prefabricated concrete bearing piles at the backside. These supports relieve the soil body underneath the relieving structure resulting in lower earth pressures on the front retaining wall. The dimensions of the front retaining wall can become smaller resulting in a reduction on the total material costs. This solution is attractive if the extra expenses on the relieving structure and concrete bearing piles are smaller than the cost reductions for the front retaining wall. The influence of quay wall designs with relieving structures is illustrated in Figure 1a to Figure 1f. The figures explain step by step quay wall design aspects. Explanations of the figures are given below:

Explanation figure 1a

Most quay wall designs in the Port of Rotterdam are constructed with (vertical) front retaining walls. The reason to install this wall is to avoid soil slopes which would require much space. The angle soil slopes naturally form depend on the density and shape of the soil material. This angle is called the angle of internal friction ϕ. Dry sand of good quality will have a ϕ around 30° to 35°. Saturated sandy soil slopes are less stable than unsaturated slopes resulting in a ϕ of around 12° (or with a slope of 1:6). The space required for a slope of 1:6 to have enough draught for seagoing vessels is large.

Explanation figure 1b

When a surcharge load is applied, for example a pile of iron ore, the angle of internal friction remains the same. The soil body in principle is stable under a surcharge load. This is valid if internal sliding planes do not occur. These sliding planes occur if the internal shear force due to the surcharge load is larger than the maximum mobilized internal shear friction τ of the soil body. τ is defined as a combination of the strength characteristics of the soil body and the pressures inside the soil body due to the surcharge load:

$$\tau = c' + \sigma_n' * \tan(\varphi) \quad \text{(eq. 1)}$$

with:

ϕ = angle of internal friction [°]
c' = effective cohesion [kPa]
σ_n' = effective pressure perpendicular on plane [kPa]

Equation (eq. 1) is based on Coulombs theory and the sliding planes are modeled as straight planes (Verruijt, 2004). Curved sliding planes or a combination of both also occur.

Explanation figure 1c

In the Port of Rotterdam quay walls with retaining heights of more than thirty meter have been built. With this retaining height, the horizontal distance needed for a natural slope with the angle of internal friction would be around 100 meter. For a quay design, this would take too much space and would require very long crane beams to handle goods. A solution to this problem is the use of a (vertical) earth retaining wall. These walls support the soil body and shortens the distance between the crane and the vessels. They can be constructed from steel sheet pile wall elements and/or steel pipe pile elements. In the case of large retaining heights, a combination (combi-wall) of pipe and sheet elements are used to sustain the soil body. These walls not only sustain the soil body, but sometimes also sustain high surcharge loads on top of the quay. The wall starts to 'feel' the load at an angle ϕ. From this point downward the vertical surcharge load start to act on the wall. At the angle θ the surcharge load is completely acting on the wall. θ is the angle of active earth pressure and is explained in more detail in appendix B. Figure 1c clearly shows the high soil pressures acting on the wall due to the presence of large surcharge loads near the earth retaining wall. To sustain these high pressures, a very thick and expensive combi-wall is required.

To support the combi-wall at the top, anchors are installed. Anchors transmit the horizontal soil pressures acting on the quay wall, back into the subsoil, resulting in an equilibrium stable state. With very stiff anchors, the combi-wall basically can be modeled with a well-known kinematic model: an elastic beam on two supports.

Explanation figure 1d

If the surcharge load is situated at a distance of 100 meter behind the wall, no additional loads from the surcharge load would act on the wall. The quay wall would only retain the soil body. This results in a smaller front retaining wall. However, due to the general lack of space in ports, it is not always desirable to place storage goods further away from the waterfront.

Explanation figure 1e

The use of a relieving structure basically lowers the influence lines of horizontal earth pressures due to surcharge loads. When comparing figure 1e with figure 1c, at figure 1e the loads from the surcharge is acting much lower on the steel front retaining wall than in figure 1c. The presence of the concrete relieving structure at the top results into this positive effect. This structure guides the horizontal forces acting on the top part to the foundation members below, thereby relieving the load on the steel front retaining wall below. As result, the front retaining wall can be designed smaller which will reduce the total construction costs. This solution is most effective in the case of near presences of high surcharge loads.

Explanation figure 1f

In figure 1f design optimizations have been included in comparison with figure 1e. Three optimizations are illustrated. Firstly, the effect of placing the foundation members at an angle lowers the horizontal earth pressures on the front earth retaining wall. Secondly, by placing the front retaining wall slightly to the back, collisions with vessels in the harbor are avoided and forces from crane loads are directly above the foundation members (front retaining wall and anchor). Thirdly, the use of a hollow concrete box at the front reduces the total vertical load on the front retaining wall.

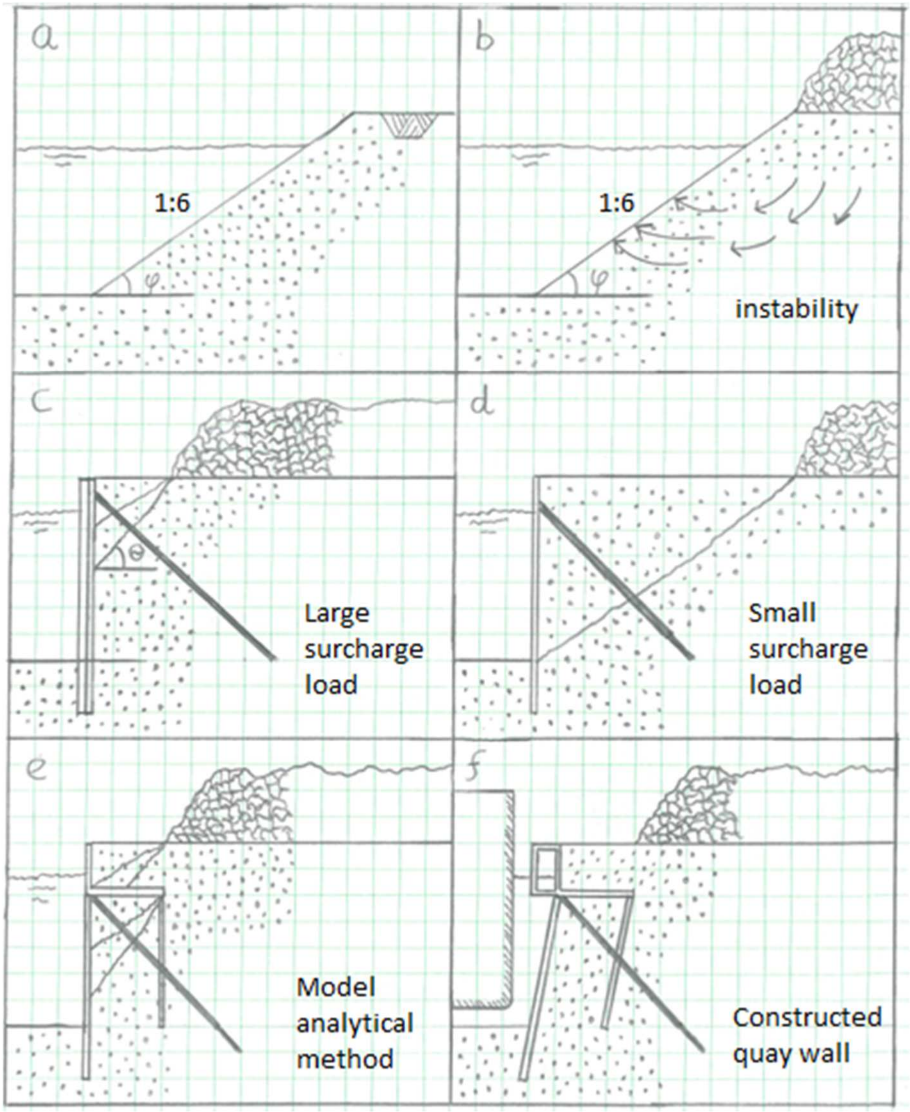


Figure 1 – Effect of a relieving structure in quay wall designs in the case of local high surcharge loads

The quay wall design in Figure 1f optimizes the use of raw construction materials in the case of high surcharge loads near the waterfront and large earth retaining heights.

1.3 Problem description

The thesis research topic is about the shielding effects of piles on quay walls. This effect seems to be noticeable for quay wall designs with relieving structures, but is not clearly understood. The extension works for the EKOM quay wall (executed in 1983) in the Mississippihaven in the Port of Rotterdam is an example where this effect seems to be noticed by measuring the bending moments and deflections in the front retaining wall. The field measurements on the front retaining wall did not comply with the design calculations (Gijt, J.G. & Winter, W.J. de, 1988). The calculated maximum front wall deflections are higher than the measured value and also the maximum calculated bending moments in the front retaining wall are higher than the measured value. The shielding effect of piles and anchors may explain this difference, but this has not been studied.

In 1991 a deep water quay wall with relieving structure has been constructed in the Amazonehaven in the Port of Rotterdam (see annex A for more information). This quay wall (Figure 2) is constructed to facilitate the transshipment of iron ore and coal. These bulk materials are stored on the quay thereby creating large horizontal loads on the front earth retaining wall. Therefore the design incorporated a quay wall with relieving structure to reduce the loads on the front wall.

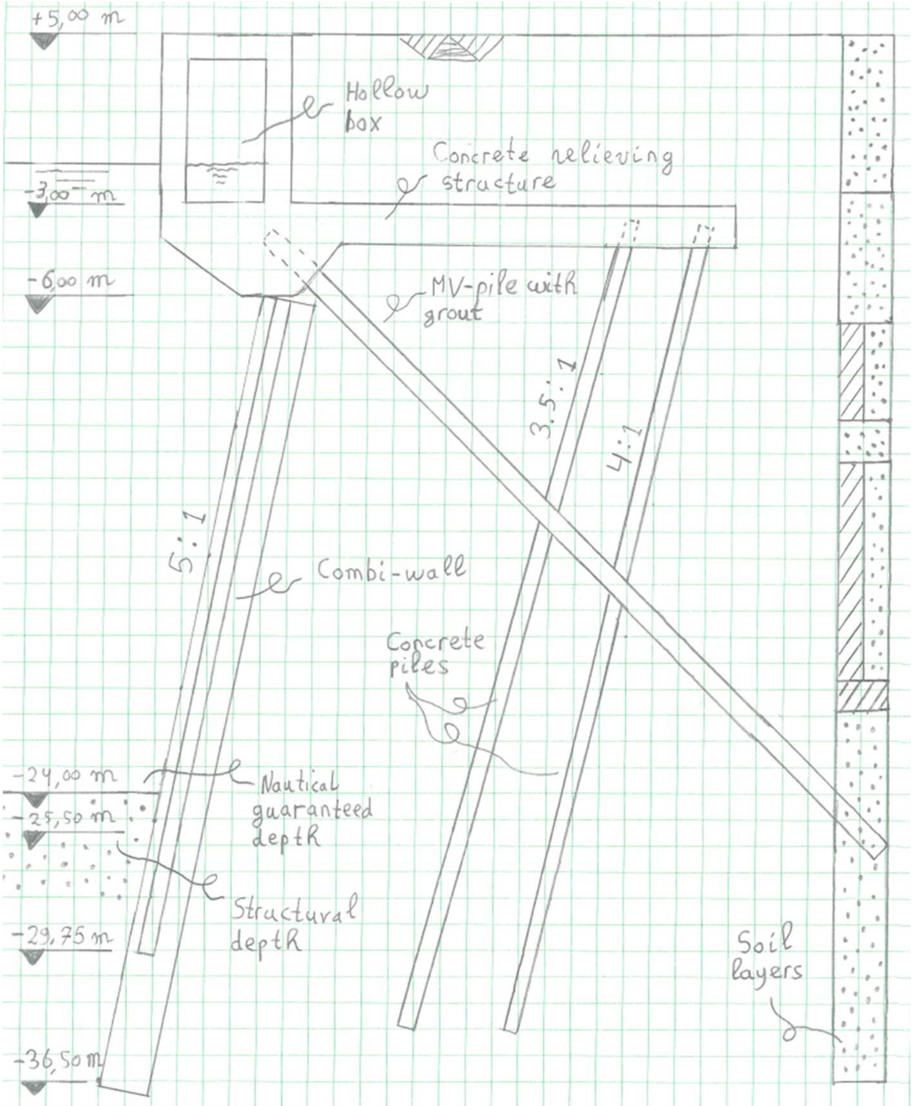


Figure 2 - Cross-section of deep water quay wall in the Amazonehaven (Port of Rotterdam, heights in meter N.A.P.)

The designers of the deep water quay wall project incorporated the previous knowledge from the extension works on the EKOM quay wall. For the design of the front retaining wall bending moment reductions factors have been applied. The support bending moment in the front retaining wall derived from design calculations was reduced with 10% whilst the (maximum) design field bending moment was reduced with 25%. This solution resulted in a more economical design but the shielding effect of the concrete piles and MV-pile anchors was not researched. It is not clear how much shielding occurs and how the dimensions and center to center distances of the piles and anchors influence the amount of shielding. For instance, can the combination of anchors and piles be consider as a secondary earth retaining wall? Is this beneficially or should this be avoided? The next three paragraphs explain in more detail geological effects occurring underneath the relieving structure and the link with design problems. The deep water quay wall project in the Amazonehaven is taken as reference.

1.3.1 Shielding effect of prefabricated concrete bearing piles and MV-pile anchors

A phenomenon in geotechnical engineering is the occurrence of soil arches near constructions in the subsoil. Soil arches can also be present at the deep water quay wall in the Amazonehaven. Especially near the area where the anchors and concrete bearing piles (Figure 2) come together. Near this location the anchors and piles create an obstacle for free flowing soil which contributes to the formation of soil arches. Also above and below this location arching effects can occur on the anchors and piles. If this happens, the combi-wall is then slightly shielded (Figure 3) from a part of the effective horizontal earth pressures. The problem for designing lies in the quantification of the amount of shielding.

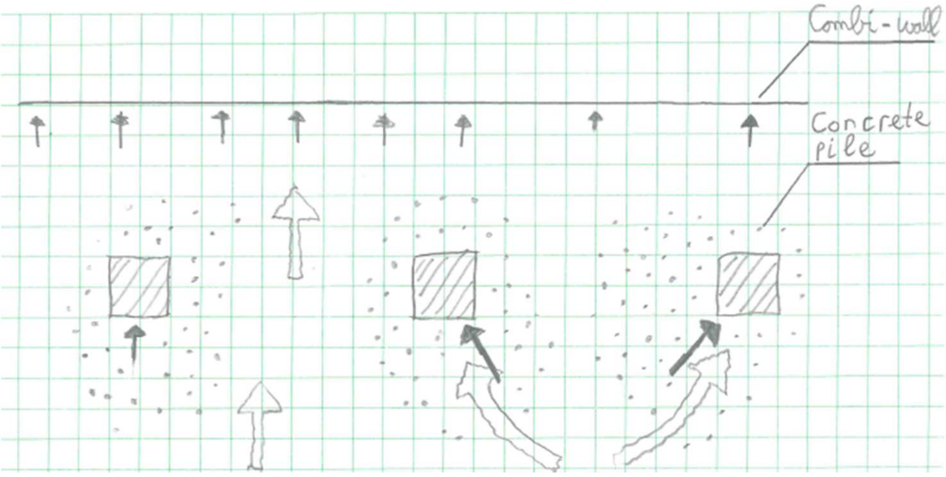


Figure 3 – Top view of the arching effect on the concrete bearing piles which results in a shielding effect on the combi-wall (black arrow = force, hollow arrow = movement)

1.3.2 Horizontal and vertical rearrangement of soil stresses near the combi-wall

Near the combi-wall two types of soil stress rearrangements occur (Figure 4). The left picture illustrates horizontal soil stress rearrangements between the pipe piles which result into reduced loads on the sheet pile elements. The right picture illustrates vertical soil stress rearrangements between the hinge support at the top and the near the bottom area of the combi-wall. This phenomena in technological terms is also known as ‘umlagerung’ (EAU, 2012). Due to horizontal ‘umlagerung’, the horizontal earth pressures on the sheet pile walls are lowered, but this is increased on the steel pipe piles. Therefore the pipe piles are the main structural elements and the sheet piles are placed to create a solid wall.

Due to vertical ‘umlagerung’ a reduction of horizontal soil stresses for the middle part of the combi-wall is realized. The top and bottom part of the combi-wall are loaded with higher horizontal soil stresses. The shifting of soil pressures result in smaller bending moments in the front retaining wall. However, in case of vibrations near the quay wall, this effect can be diminished again.

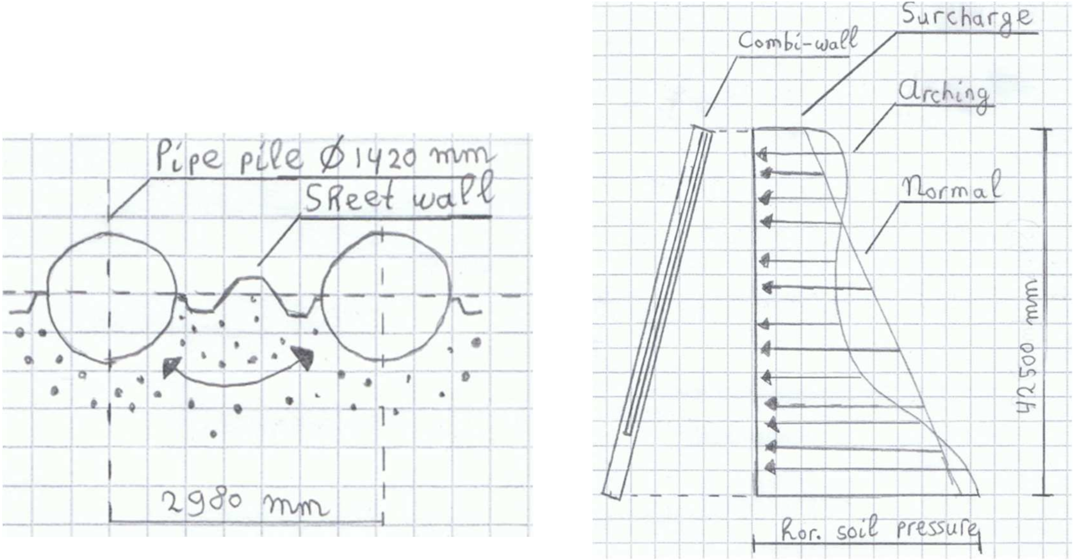


Figure 4 - Left: horizontal ‘umlagerung’ on pipe pile elements shielding the sheet pile wall; right: vertical ‘umlagerung’ on combi-wall which shift the horizontal soil pressures to the ends of the combi-wall

1.3.3 Pile-pipe interactions and tensioning effect of MV-pile anchors

Instead of the shielding effect of the concrete piles and MV-pile anchors, these elements also induce other effects which influence the structural behavior of the quay wall. These are the pile-pipe interactions and the tensioning effect on anchors (see Figure 5).

Pile-pipe interactions result in higher soil pressures near the bottom part of the combi-wall. The piles are laterally loaded with soil stresses and support the relieving structure. The bottom part of the pile is near the combi-wall. Displacements of either the combi-wall or the pile tips results in complex pile-pipe interactions.

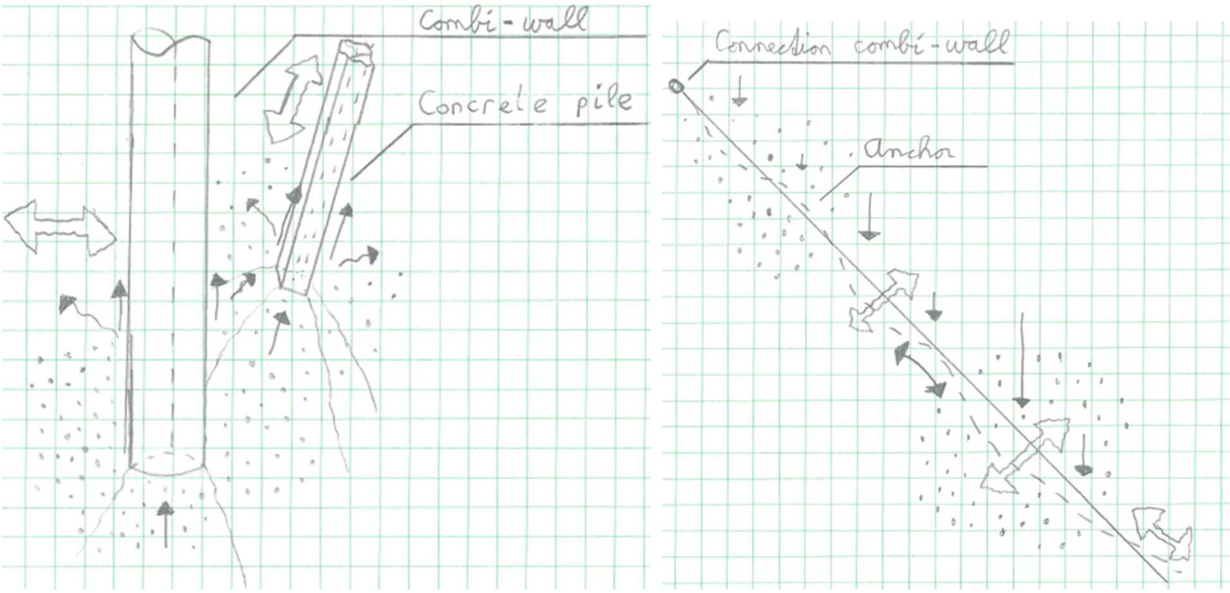


Figure 5 – Left: side-view of pile-pipe interactions; right: tensioning-effect of MV-pile anchors (legend: black arrow = force, hollow arrow = movement)

The tensioning effect is caused by bending of the MV-pile anchors and results in redistribution of forces in the MV-pile anchors. The axial force in the anchors can become larger and the connection with the relieving structure should be strong and flexible enough to resist this extra movement. Settlements of weak clay layers may result in large deformations for MV-pile anchors.

Problem description in brief:
The total shielding effect of the prefabricated concrete bearing piles is a combination of the pipe-pile interactions near the combi-wall and the soil-pile interaction over the length of the concrete piles. For the MV-pile anchors the shielding effect is based on the tensioning effect and the soil-anchor interaction over the length of the anchor. However, the concrete piles and MV-pile anchors also interact with each other increasing the complexity for designing. Finally the displacement of the concrete relieving structure is involved as well. The displacement of the relieving structure depends on the displacement of the combi-wall, MV-pile anchors and the concrete piles.

The total amount of shielding from the piles and anchors is a combination of many factors. In the past there were not any methods available to analyze these complex interactions.

1.4 Research question

The main focus for this thesis is an investigation of the shielding effect which the piles and anchors have on the combi-wall. This effect is being investigated for the deep water quay wall project Frans Swarttouw in the Amazonehaven in the Port of Rotterdam with analytical and finite element methods.

The main thesis question is:

“What is the shielding effect of the piles and anchors on the combi-wall for the deep water quay wall project Frans Swarttouw in the Port of Rotterdam and which design aspects influence this effect?”

The following sub-questions are formulated:

1. Can the group behavior of the piles and anchors be considered as a second earth retaining wall and how large is the shell factor?
2. What is the difference between the available methods to model the shielding effect and how reliable are the results?
3. In what way do the model parameters influence the shielding effect and is a comparison with other quay wall locations and dimensions possible?
4. Does an optimum exist between the dimensions of the combi-wall and the dimensions of the piles and anchors?

1.5 Scope of thesis

The first chapter governs the introduction. It describes the principles of quay walls with relieving structures, the problem definition and the thesis research question. Chapter two explains the plan of approach and the important parameters which determine the shielding effect. The research process is illustrated with a flowchart. The third chapter describes an analytical method to illustrate the shielding effect of piles on quay walls with a relieving structure. Chapter four describes a method based on finite element method analysis to research the effect. With finite element method (FEM) software more sophisticated research can be conducted, for instance the research is not limited to model vertical piles only. The influence of oblique placed piles and the influence of anchors can be consider as well. In addition, more different soil layers can be modeled. In chapter five a comparison has been made between the data from the analytical method with the data of the finite element analysis to compare and verify the results. Based on the findings of the research, design recommendations have been formulated and a cost reduction analysis has been elaborated. The thesis research finalizes with the conclusion and recommendations.

Scope of thesis in brief:

- Chapter 2: Introduction
- Chapter 2: Plan of approach research
- Chapter 3: Research with analytical method
- Chapter 4: Finite element method with option embedded piles
- Chapter 5: Discussion shielding effect and design recommendations
- Chapter 6: Conclusion
- Chapter 7: Recommendations

2 Plan of approach research

2.1 Introduction

The shielding effect of piles on quay walls is the essential part of this study. This effect may explain the unexpected lower bending moments and deflections of the front retaining wall in quay wall designs with relieving structures. This effect can happen because relieving structures, intended to lower the soil pressures on the front retaining wall, are supported by (prefabricated pre-tensioned concrete) piles. The presence of the piles in addition may also lower the soil pressures on the front retaining wall. This effect is called the shielding effect.

The shielding effect is caused by the piles and anchors underneath the relieving structure. The difference in stiffness and strength properties of the piles in comparison with soils allows for a partially transfer of forces to different parts instead of a direct transfer behind the piles. Figure 6 explains this transfer of forces for a abutment structure with a bridge deck. In the figure there is no front retaining wall, but the principles of shielding of a part of the horizontal soil stresses is comparable for quay walls with front retaining walls. The figure illustrates that basically all pile rows in every situation can be horizontally loaded due to local soil movements.

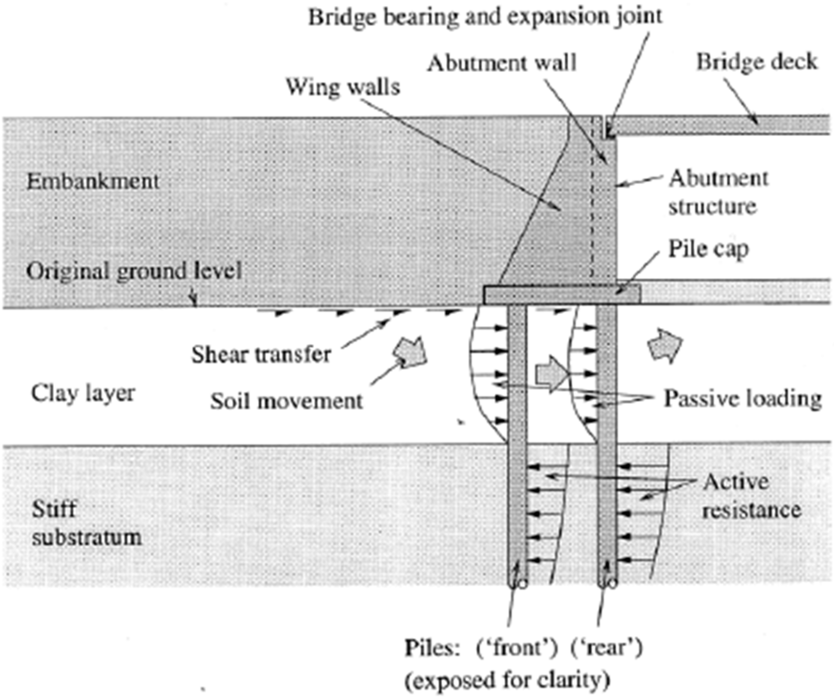


Figure 6 – Passive piles in an embankment subjected to horizontal loading from moving soil past the piles (Ellis, E.A. & Springman, S.M., 2001)

In Figure 6 the clay layer starts to move due to the embankment load. The supporting piles underneath the abutment structure are subjected to horizontal loading from moving soil past the piles. Due to the strength and stiffness properties of the piles, the piles start to resist this soil movement. Soil pressures increases on the 'front' side of the piles over the height of the weaker clay layer. The piles start to shift this increased soil pressure upwards to the abutment structure and downwards to the stiffer substratum thereby effectively lowering the soil pressures at the 'rear' side of the piles in the weaker clay layer. The extra horizontal load in the abutment structure is transferred to the bridged deck. In the stiffer substratum, the horizontal load is dispersed over the top side of the substratum by active resistance of the soil. The piles can only transfers a small amount of soil pressures upwards and downwards depending on the bending moment resistance of the piles. A large part is also directly transferred from the 'front' to the 'rear' side of the piles. It is the portion which is transferred up and downwards which results in reduces soil stresses behind the piles. For more information about passively and actively loaded piles and the principles of soil arching on the piles see appendix C.

To determine the shielding effect of piles on quay walls, a quay wall with relieving structure in the Port of Rotterdam is discussed; the deep water quay wall project in the Amazonehaven (see Figure 7). This quay wall is constructed with a relieving structure, a combi-wall, MV-pile anchors and concrete bearing piles. For more information about these structures see appendix A.

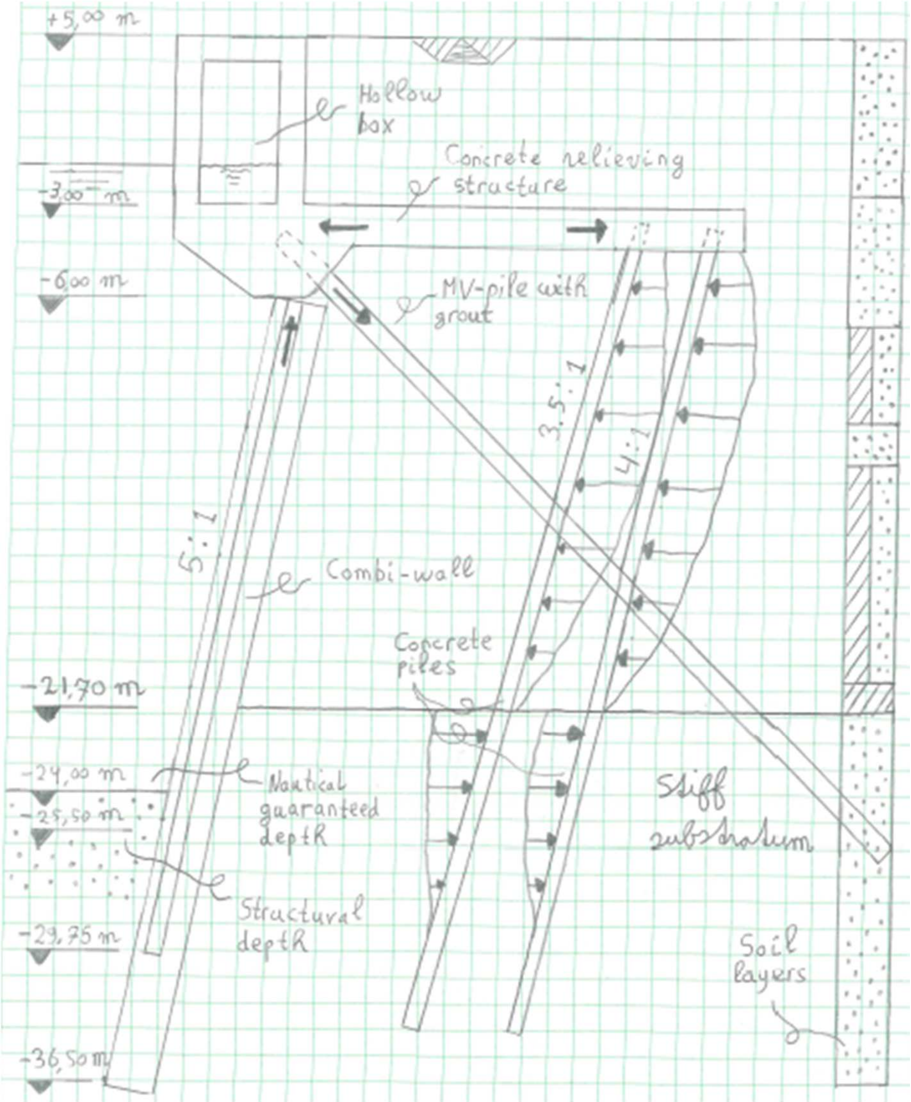


Figure 7 - Cross-section of deep water quay wall in the Amazonehaven with passively loaded piles partially shielding the front retaining wall (Port of Rotterdam, heights in meter N.A.P.)

Due to the self-weight of soils and the surcharge loads on the top level of the quay, the soil pressures in the soil layers are increased. The soil layers start to move seawards. It is expected that the weaker soil layers (Holocene layer) above -21,7 meter N.A.P. slide further outwards than the top part of the stiffer substratum (Pleistocene layer). This creates the same situation as for Figure 6 where the top part of the concrete piles is horizontally loaded from moving soil past the piles. The bottom part of the piles in the stiffer substratum is actively loaded by soil pressures. The piles start to transfer a part of the horizontal soil pressures upwards into the relieving structure and downwards into the stiffer substratum.

The additional compressive force in the relieving structure is transferred to a compressive force in the combi-wall and a tensile force in the MV-pile anchors, see Figure 7.

The concrete piles are not the only structural objects prone to shielding; the MV-pile anchors may also shield the combi-wall. The anchors also have a certain bending moment resistance and therefore can also partially transfer horizontal soil pressures to the relieving structure and the stiffer substratum. However, it is expected that the anchors have a small influence, because the bending moment resistance is about eighteen times smaller in comparison with the concrete bearing piles.

The shielding effect of the piles and the anchors for the deep water quay wall project in the Port of Rotterdam is further researched to qualitatively and quantitatively understand the effects of shielding for future quay wall designs.

2.2 Design aspects determining shielding effect

One approach to determine the shielding effect on the deep water quay wall is to measure the deflections and bending moments in the front retaining wall and concrete bearing piles on the existing quay wall in the field. However this is not an option for this master thesis research because of financial reasons. As stated in the problem description (paragraph 1.3) there is some field data available on the extension works of the EKOM quay wall. These findings will be discussed in this research.

Another approach is to simulate the deep water quay wall with appropriate quay wall models. By applying models and simulations many different types of calculations and design aspects can be considered. On the other hand, models are a representation of the real situation. The results from model calculations should always be properly assessed. To create appropriate quay wall models for the deep water quay wall project, first the important parameters which determine the shielding effect should be analyzed.

The behavior of the prefabricated pre-tensioned square concrete bearing piles is determined with:

- Soil movement along piles and the interface strength between piles and soil
- Soil pressures due to self-weight and surcharge loads
- Stiffness of piles
- Clamped connection in stiff substratum (Pleistocene layer)
- Stiffness of and connection with the relieving structure

Next the behavior of the MV-pile anchors is determined with:

- Soil movement along the anchors
- Soil pressures due to self-weight and surcharge loads
- Interface strength between grout body and soil layers
- Earth mass being mobilized by anchor
- Stiffness of MV-pile anchors
- Stiffness of and connection with the relieving structure

And finally the behavior of the combi-wall is determined with:

- Soil pressures behind wall
- Water pressure difference on wall
- Stiffness of anchorage consisting of MV-pile anchors and concrete piles
- Stiffness of combi-wall
- Clamped connection in stiff substratum (Pleistocene layer)
- Stiffness of and connection with the relieving structure

The concrete relieving structure is relatively large and stiff in comparison with the foundation elements and is of less importance to determine the shielding effect. However, negative skin friction between the concrete and soil can also have an influence in reducing the soil stresses acting on the foundation members.

2.3 Applied methods thesis research

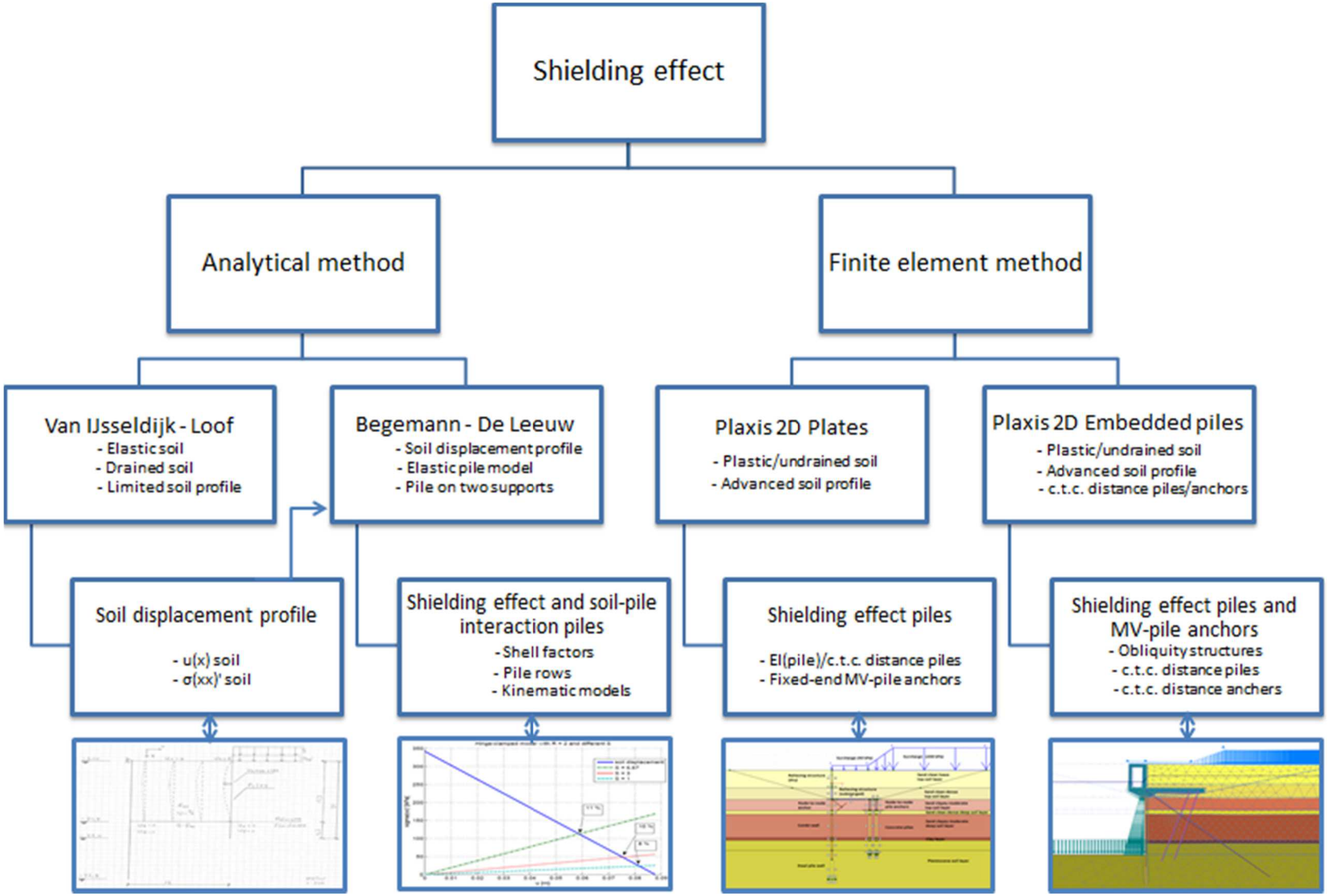
The methods applied for this research are based on analytical and numerical calculations. The idea of applying different methods is to increase the accuracy of the calculation results. In chapter three an analytical method is described. It is based on the method Van IJsseldijk – Loof to determine the soil displacements and the method Begemann – De Leeuw to determine the soil-pile interactions. The analytical method is easy to apply and gives quick results. The downside of this method is that it can only be applied for vertical bearing piles and in the case of elastically soil behavior. Therefor the need arises to apply different methods as well.

In chapter four a numerical method based on finite element method (FEM) analysis is described. The program used for the FEM analysis is Plaxis 2D. With Plaxis also oblique placed piles can be modeled and therefore the influence of the MV-pile anchors can be researched as well. For Plaxis 2D two different methods are available: modeling the piles and anchors with continuous plate elements or modeling it with the embedded piles option. The last option gives the designer more freedom to choose the center to center distances between the anchors and piles. However, this version was initial not available for this research and therefore the first method is also used. Here the influence of the center to center distance between the piles and anchors is more difficult to adjust in the quay wall model. For more information about the two Plaxis methods see appendix D.

The results from Plaxis with the embedded piles option give a better view of the shielding effect than the method based on plate elements. Chapter four illustrates the findings from this method. The results from the previous method can be viewed upon in appendix G. Some adjustments to the quay wall model have been made from the experience with the previous method. The relieving structure is now modeled more accurately. Also the connection between the combi-wall and the relieving structure is made eccentric to reduce the maximum field bending moments in the combi-wall. Finally the soil layer underneath the relieving structure has been adjusted. This so called weak ‘sponge’ layer is present to prevent high pressures building up underneath the relieving structure. From field research it has been concluded that the relieving structure is most of the time not in contact with the soil underneath it. The presence of this weak layer should prevent this.

In paragraph 2.4 the plan of approach and the applied methods are visualized in a flow chart. The images below the flow chart are for clarification for the reader and correspond with the applied methods. The analytical method is discussed in chapter 3 whilst the FEM analysis is discussed in chapter 4 and annex G.

2.4 Flow chart research



3 Research with analytical method

3.1 Introduction

This chapter describes the amount of shielding of piles on the front retaining wall of quay walls by using an analytical method. The deep water quay wall project is used as a reference case. For more details about this project see appendix A.

The methods Van IJsseldijk - Loof and Begemann - De Leeuw are considered in this chapter. The first method describes the interaction between horizontal soil displacements and surcharge loads. The second method describes the interaction between horizontal soil displacements and soil-pile interactions. The methods are elaborated in appendix D.

The applied methods do require simplifications for the deep water quay wall project. First an appropriate model should be set up. Than the soil displacements under the influence of surcharge loads and the soil-pile interactions are considered. The amount of shielding from the piles depends on these interactions. This chapter concludes with the results from this investigation.

3.2 Design approach modeling deep water quay wall

3.2.1 Geometrical and kinematical schematizations

The deep water quay wall project needs to be schematized into a simplified model. This is necessary to avoid complex interactions between the foundation members and to be able to use the analytical method. These foundation members are: the combi-wall, the concrete bearing piles and MV-pile anchors. The dimensions are kept mostly the same as for the deep water quay wall, see Figure 8.

The concrete relieving structure is roller supported at a level of -6,00 meter N.A.P. to exclude the effect of the MV-pile anchors. Without anchors in the proximity, the only foundation members left are the combi-wall and concrete bearing piles. Now the effect these two members have on each other can be fully investigated. The best way to do so is placing these members in a vertical position. This will increase the loads on these members, but makes an analysis according the theory of Begemann - De Leeuw possible. Also a better comparison with other quay wall designs (i.e. quay walls with diaphragm wall) can be made when placing the foundation members in a vertical configuration. The roller support initially is fixed to exclude the movements of the concrete relieving structure. This support can be given a spring stiffness to investigated the behavior in a more realistic manner.

The concrete relieving structure is schematized as a L-wall on two supports: a combi-wall support and a concrete bearing piles support. The concrete hollow box from the relieving structure is modeled as a single vertical wall. The horizontal forces on the relieving structure from earth pressures and surcharge loading are directly transferred to the roller support. The structural dimensions are kept the same as for the deep water quay wall project.

The surcharge loads applied on the quay are chosen such that the governing loading conditions during the lifetime of the quay are met. For the deep water quay wall design the maximum bulk load was set to 450 kPa. This load originated from the bulk storage of iron ore. However, experience from the past has learned that this load is considered to be too high in the first place and that the distance between this load and the front retaining wall was also rather large in the order of one retaining height. Therefore for this research, a lower load but placed nearer to the front retaining wall is chosen. This surcharge load is 40 kPa for the first 11 meter near the waterfront simulating for instance traffic loads. Than over a distance of 7,6 meter the surcharge load rises to 200 kPa and will remain at this level for at least twice the total retaining height to simulate iron ore bulk storages. Considering a steep slope of iron ore with an angle of internal friction of around 40 degree and a volumetric weight of 25 kN/m^3 results in a slope distance of 7,63 meter and eventually in an increased surcharge load from 40 to 200 kPa.

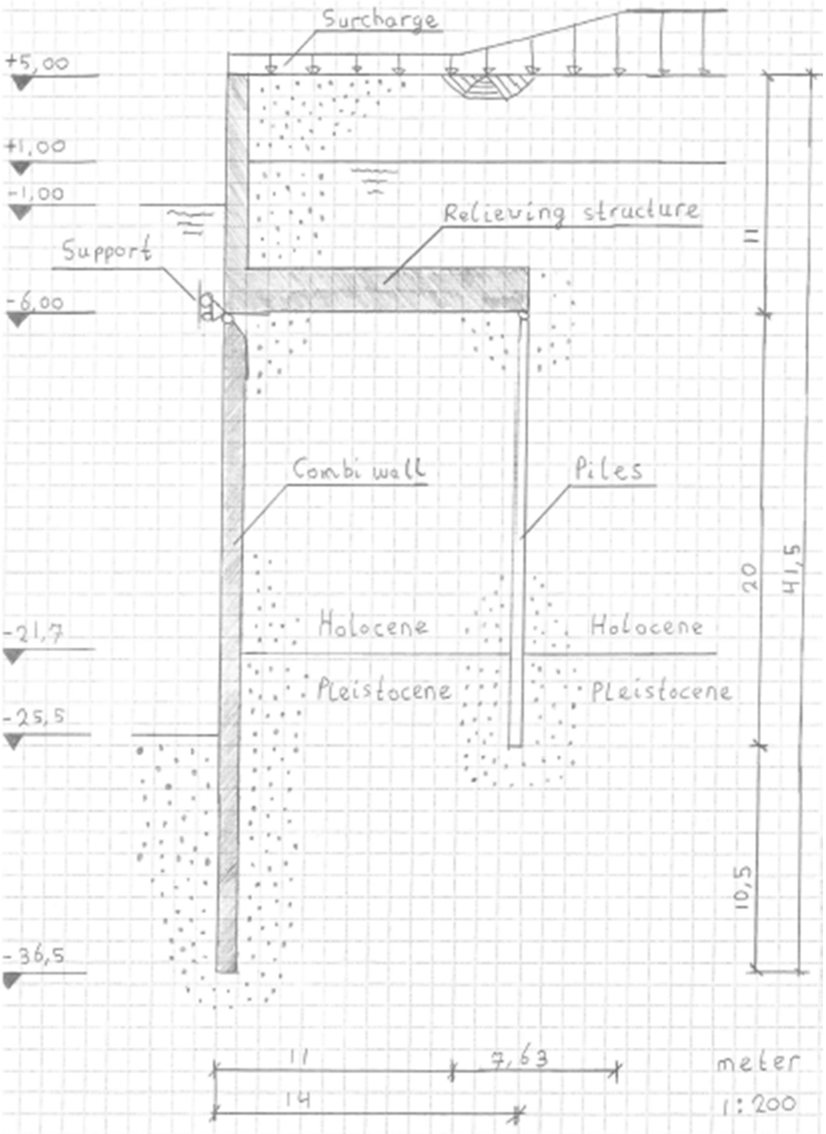


Figure 8 - Cross-section deep water quay wall project model for analytical method to determine the amount of shielding from the piles (heights in meter N.A.P.)

As explain in appendix D the method Begemann - De Leeuw can be considered if the soil body can be schematized into two different soil layers, a soft layer and a stiff layer. The weak clay layer between the Holocene and Pleistocene soil layers in the Port of Rotterdam is an ideal layer to divide the subsurface into two soil bodies with different stiffness characteristics, see appendix A. The Holocene soil layers can be schematized as one soil layer with stiffness E_{50} of 15000 N/mm^2 and the Pleistocene layer as a stiff layer with at least three times the E_{50} of the Holocene layer: E_{50} = 45000 N/mm^2 for the Pleistocene layer.

In Figure 9 the back-view of the quay wall model is shown. The distance (a) between the center lines of the concrete bearing piles is 3 meter. The cross-section of each pile is 450x450 millimeter. The ratio distance over diameter is a/D = 6,67. This ratio is in compliance with the deep water quay wall project. If the shell factor is the same as this ratio, the group behavior of the concrete bearing piles behave like a second earth retaining wall.

From appendix C it can be concluded that the piles should indeed behave more or less like a second earth retaining wall, because the shell factor is between 3 to 5 for a single pile in the case of plastically deformations of soils. During the analysis the effect of different shell factors from 1 to 3 to 6,67 have been researched. Another aspect which has been researched is the influence of the amount of pile rows. A second pile row increases the soil-pile resistance thereby increasing the shell factor. The stiffness of the pile group is also larger for two rows. The shell factor for a double pile row is therefore chosen to be between 5 and 6,67. As stated before the influence of the MV-pile anchor cannot been considered in this analysis and therefore the anchors are not drawn in Figure 9.

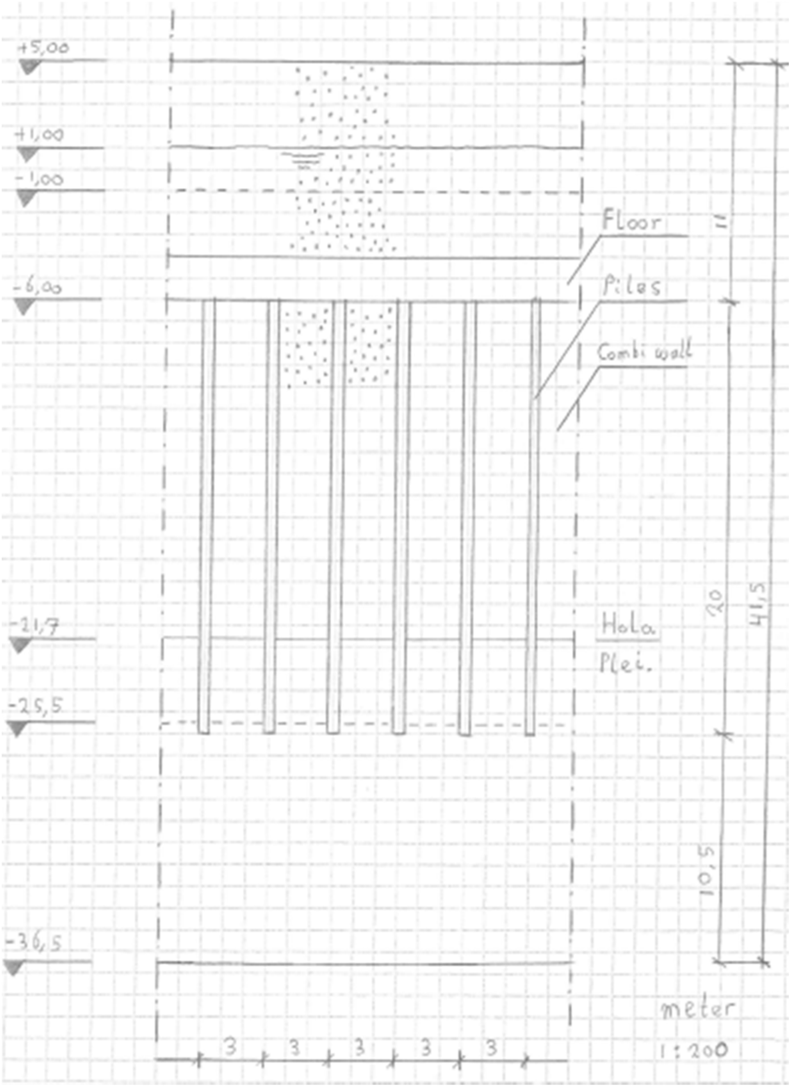


Figure 9 - Back-view quay wall model for analytical method (heights in meter N.A.P.)

3.2.2 Loading and load influence lines schematizations

In order to apply the method of Begemann - De leeuw to determine the soil-pile interactions and the amount of shielding some basic assumptions should be made. First of all is the method only valid for an infinitely long horizontal soil profile. This is clearly not the case for the quay wall model. Secondly the surcharge load should be defined correctly. The load for the deep water quay wall model is 200 kPa. However, in this method the load should be considered at the position of the relieving structure. This includes a soil profile of 11 meter which can be considered as an additional surcharge load. The total surcharge load then amounts to 342 kPa.

In order to determine the influence of the finite horizontal soil profile length for the quay wall model consider Figure 10. In this figure the soil profile starts at the position of the relieving structure at -6,0 meter N.A.P.. This can temporally be considered as the ground level. The (fictive) front retaining wall will bend outwards due to the earth pressure acting on it. Due to this movement, the retained earth also moves outwards. The influence lines for this movement are drawn in the figure. These are the ϕ and θ lines explained in annex B. These influence lines determine the behavior of the concrete bearing piles at the back. Now consider the movement of the front wall. Due to the displacement of the combi-wall the coefficient of earth pressures K will change. The top part hardly moves resulting in a coefficient of K_0, the middle part moves further resulting into K_a until the combi-wall is clamped into the bottom soil layer. At this location the coefficient will be between K_o and K_p. According to the trajectories of the influence lines ϕ and θ the concrete bearing piles hardly "notice" the bending behavior of the combi-wall. Only the top part transverses from the natural coefficient of earth pressure K_0 to an active state K_a. This implies that the piles will remain in place and that the method of Begemann - De Leeuw can be applied.

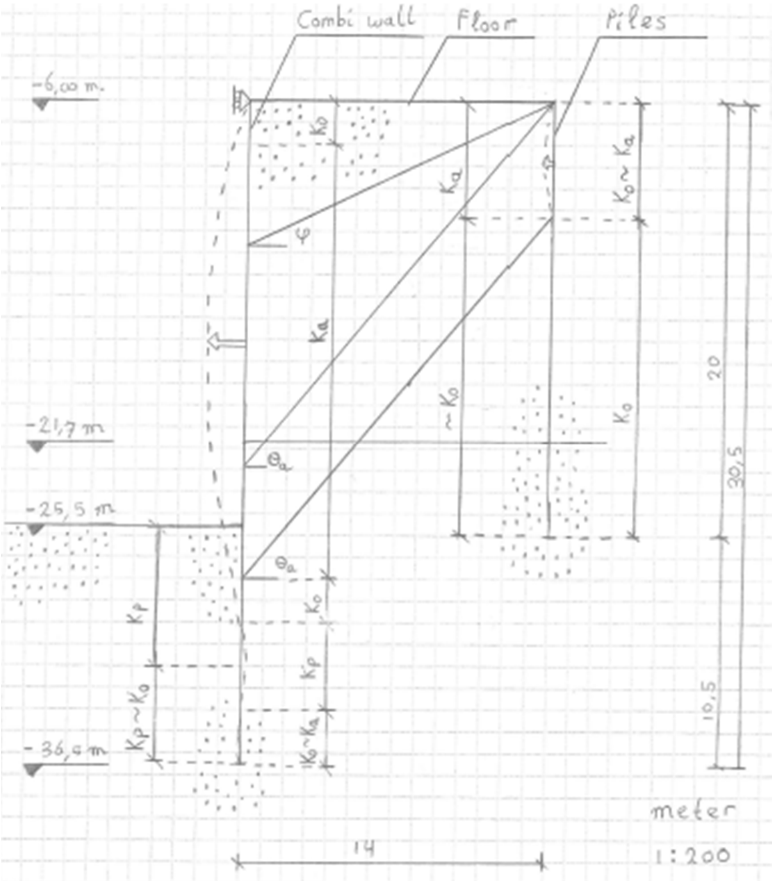


Figure 10 – Comparison coefficients of earth pressures K with method Begemann - De Leeuw

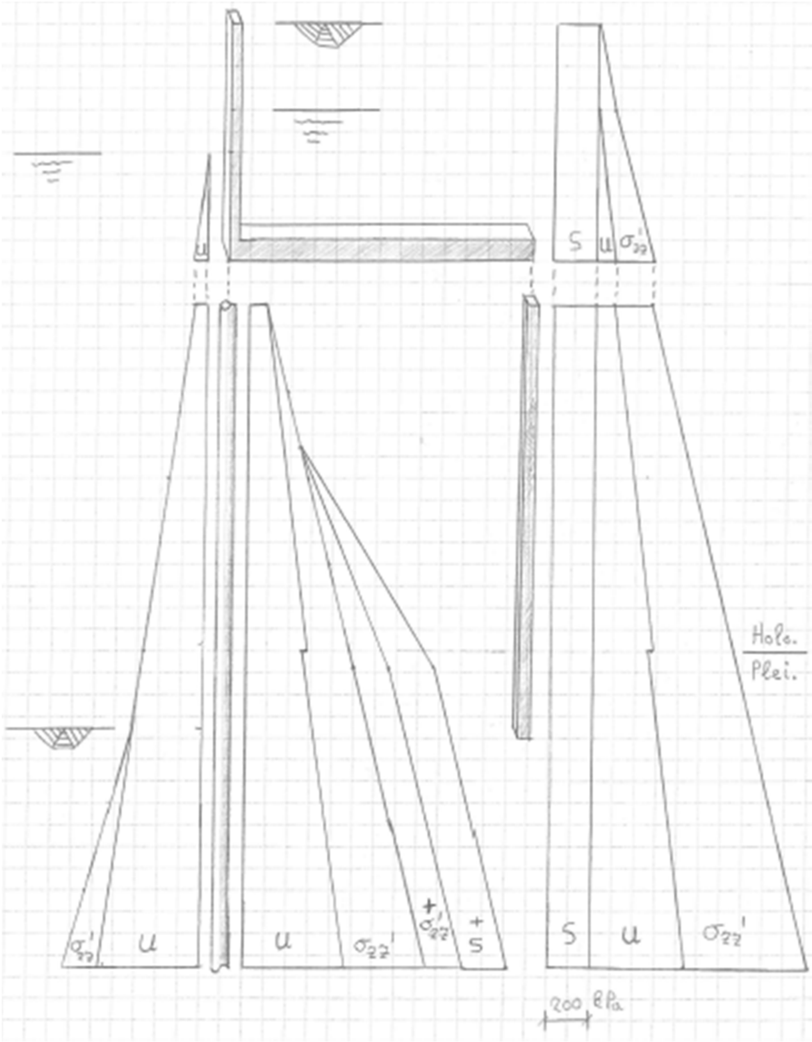


Figure 11 – Illustration vertical pressures acting on quay wall model for analytical method (pressures in kPa)

Figure 11 illustrates the vertical pressures acting on the quay wall model. The water pressures (U), effective vertical earth pressure (σ_{zz}') and surcharge loading (S) from iron ore deposits are drawn in the figure. The total surcharge load of 342 kPa is determined by the summation of S and σ_{zz}' at the bottom level of the relieving structure. These vertical pressures can be translated into horizontal pressures when the effective horizontal soil stresses and water pressures are considered. The effective horizontal soil stresses are times K smaller than the effective vertical soil stresses.

The combi-wall “notices” the surcharge load and earth refill at angles ϕ and θ which are drawn in Figure 10. At an angle of ϕ, additional pressures start to act on the combi-wall. At an angle θ, the total load acting just behind the concrete relieving floor is present (lines +σ_{zz}' and +S). From this point downwards the soil pressures between the part underneath the relieving structure and underneath the earth refill are the same. At this level the influence of the relieving structure to reduce the effective horizontal soil stresses acting on the front retaining wall is gone. The amount of shielding of the piles determines the actual contribution of +σ_{zz}' and +S. Over the height of the Holocene layer the contribution will be smaller due to the transfer of forces to the relieving structure and the stiffer Pleistocene layer.

3.3 Soil-pile interactions

3.3.1 Soil displacement profile quay wall model

With the method Van IJsseldijk – Loof the soil displacement profile at the location of the bearing piles can be calculated. This method translates the horizontal effective soil stresses σ_{2xx}' into a soil displacement profile u_2. Therefore the total vertical surcharge load of 342 kPa at the bottom level of the relieving structure must be converted to an effective horizontal soil pressure. The next step is the translation of the effective horizontal soil pressure into the soil displacement profile. To determine σ_{2xx}' and u_2 the two equations from appendix D.1 are used. This process is illustrated in Figure 12 where the soil profiles at different coordinates have been drawn. The figure illustrates a clear distinction between the Holocene and the Pleistocene layer. It represents the soil displacements profiles if no piles would be present or with piles having zero bending stiffness.

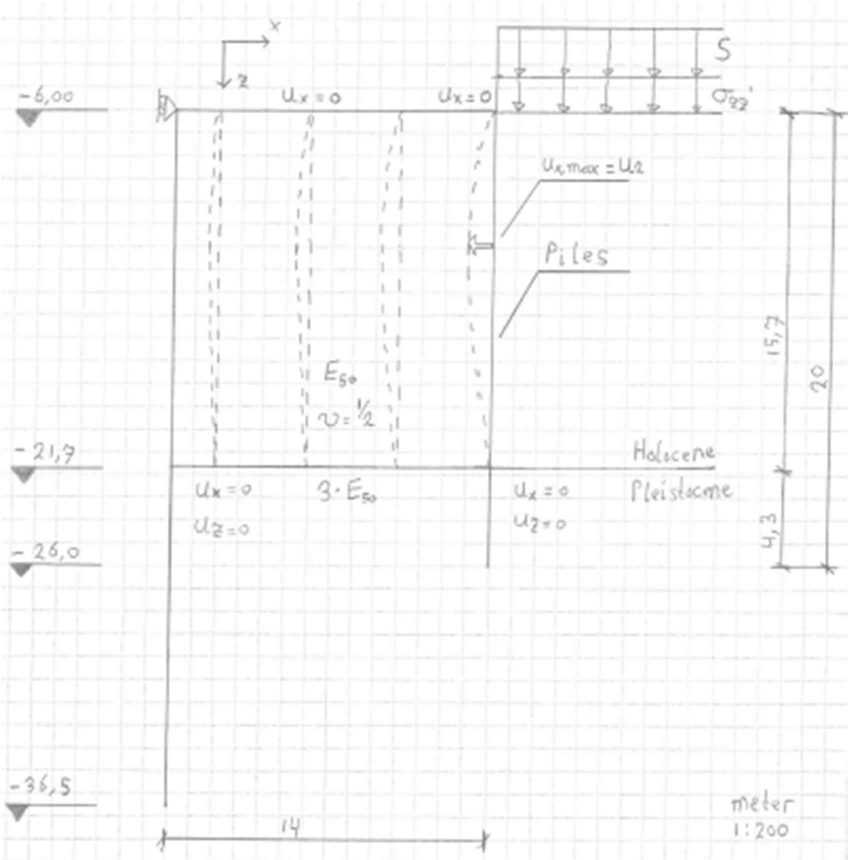


Figure 12 - Soil displacement profiles for quay wall model based on method Van IJsseldijk - Loof

Table 1 - Data and results soil displacement profiles for quay wall model

Data		
x	0	[-]
y	0,4	[-]
q	342	kPa
H	15,7	m
$E_{holocene}$	15	MPa
$E_{concrete}$	30000	N/mm^2
D_{pile}	0,45	m
$L_{apparant}$	15,7	m
a	3	m
I_{beam}	0,00342	m^4
u_2	0,0877	m
$\sigma_{2,xx}$	171	kPa
$2^*\sigma_{2,xx}$	341	kPa

Table 1 illustrates the data applied and the results from this method. The horizontal coordinate of the concrete bearing piles is located at x=0. The maximum soil displacement occurs around 0,4 times the height of the weak Holocene layer seen from the top. This results in a y-coordinate of y = z/h = 0,4. The total amount of surcharge load q (including self-weight of the earth refill) is 342 kPa. The height of the weak layer is 15,7 meter and the modules of elasticity of the Holocene layer $E_{holocene}$ is 15 MPa. This modules is about three times the average cone resistance q_c (see annex A.2). Without piles the effective horizontal soil stress σ_{2xx}' is 171 kPa and the maximum soil displacement u_2 is 0,0877 meter.

3.3.2 Soil-pile interactions quay wall model

The above described soil displacement profile occurs if the bearing piles would not be present. In reality however, the piles are present and start to interact with the soil displacements. To determine the pile displacements the method Begemann – De Leeuw must be used. It is based on the maximum soil displacement u_2 to determine the soil-pile interactions. The method considers two extreme cases: piles with no bending stiffness's and piles having infinite bending stiffness's. In the case of infinite bending stiffness's, the pressure on the piles is two times the effective horizontal soil pressure (see Table 1). Then the soil and pile displacements would be zero.

To determine the soil-pile interactions the soil displacement profile is compared with the load-displacement profile of the piles. This comparison is illustrated in Figure 13 with on the vertical axis the load on a pile and on the horizontal axis the displacement of the pile. A pile with infinite bending stiffness has zero displacement and a pressure of 341 kPa is acting on the pile. A pile with zero bending stiffness has a maximum displacement of u2 which is 0,0877 meter and with effectively zero pressure on the pile. The blue line in Figure 13 illustrates these two extremes.

The next step is to plot the load-displacement profiles of the pile. For this the stiffness of the pile is needed and the load acting on the pile. The stiffness characteristics of the pile can be calculated with the following parameters: the concrete modules of elasticity E_{conc}, the moment of inertia of the pile I_{beam}, the apparent length L_{app} and the kinematic schematization α to model the supports types. Also the amount of pile rows R determine the stiffness of the group of pile rows. The load on the pile depends on: the shell factor S and the center to center distance of the piles (a). Figure 13 illustrates three load-displacement profiles (three dotted lines) of a pile considering two rows of piles combined (R = 2), a hinged-clamped connection (α = 2) with different shell factors S. These factors are set to 1, implying no soil arching, to 3, implying normal soil arching or to 6,67, implying fully soil arching. The shell factor relates the pile width or diameter to the total soil arch width behind the pile, see annex C.3 for more information. For more information about method Begemann - De Leeuw, see annex D.2.

The influence of the amount of pile rows R, the shell factor S and the kinematic schematizations α are considered for the quay wall model to determine the shielding effect of the piles. The parameters R and α change the stiffness behavior of the pile whilst S change the loading on the pile. The shielding effect is calculated by considering the stresses acting on a pile ($\sigma_{3,pile}$*D_{pile}*S) and the stresses acting without a pile (σ_{2xx}*a). The remaining effective horizontal soil stress per meter width is then:

$$\sigma_{(xx,soil,eff)} = (\sigma_{2xx} * a - \sigma_{3,pile} * D_{pile} * S)/a \qquad \text{(eq. 2)}$$

Now the reduction of the effective horizontal soil stress in % can be calculated with the following formula:

$$Reduction = \left(1 - \sigma_{(xx,soil,eff)}/\sigma_{2xx}\right) * 100\% \qquad \text{(eq. 3)}$$

The shielding effect of the piles can only occur if the piles are strong enough to resist the occurring bending moments. To determine this the factor β is used, see annex D.2. A distinction is made between the support bending moments β_{supp} and M_b and the field bending moments β_{field} and M_{field}. In the case of a hinge-clamped schematization (α=2) only field bending moments occur. The maximum allowable bending moment for each pile is calculated according to the following formula (Walraven, J.C. & Braam, C.R., 2011):

$$M_{Rd} = z * f_{yd} * A_{conc} * \rho/100 \qquad \text{(eq. 4)}$$

Influence shell factor S

First the influence of the shell factor is considered. The amount of rows is set to two and the kinematic model is hinge-clamped supported. This implies a hinged connection with the concrete relieving structure and a clamped connection in the Pleistocene soil layer. Table 2 and Figure 13 illustrate the total reduction in horizontal soil stresses due to the shielding effects of the double pile row. The larger the shell factor, the bigger the reduction with a total reduction of 11 % for S = 6,67.

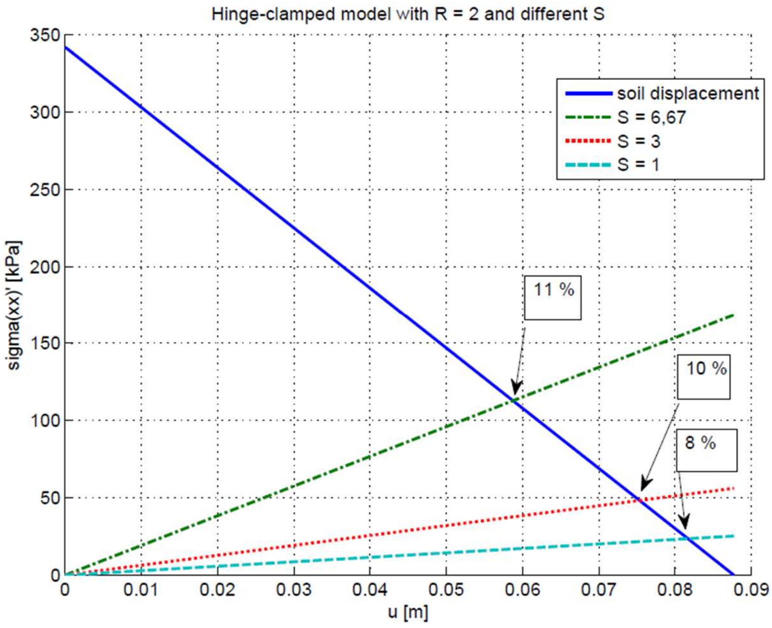


Figure 13 - Illustration soil-pile interactions with hinge-clamped model, R = 2 and different shell factors S

Table 2- Calculation soil-pile interactions with hinge-clamped model, R = 2 and S = 6,67

Input		
α	2	-
S	6,67	-
R	2	-
$\beta_{support}$	0,000	-
β_{field}	0,070	-
Results		
u_3	0,0831	m
$\sigma_{3,pile}$	17,9	kPa
$\sigma_{3,soil}$	17,9	kPa
q_{pile}	53,8	kPa
M_b	0	kNm
M_{field}	467	kNm
z	0,365	m
ρ	2	%
f_{yd}	435	N/mm2
$M_{Rd,pile}$	642	kNm
$\sigma_{xx,eff}$	153	kPa
Reduction	11	%

with

α = kinematic factor (1/384*α) [-]
S = shell factor [-]
R = number of pile rows [-]
β_{supp} = kinematic factor bending moment at support ($q^2 * l_{app} * \beta$) [-]
β_{field} = kinematic factor bending moment at field ($q^2 * l_{app} * \beta$) [-]
u_3 = intersection displacement pile and soil [m]
$\sigma_{3,pile}$ = effective horizontal soil stress on pile per meter width [kPa]
$\sigma_{3,soil}$ = effective horizontal soil stress per meter width [kPa]
q_{pile} = horizontal load on pile ($\sigma_{3,pile} * D_{pile} * S$) [kN/m]
M_b = bending moment at support relieving structure [kNm]
M_{field} = bending moment at field [kNm]
z = internal leverage arm ≈ 0,9*0,9*D_{pile} [m]
ρ = reinforcement ratio [%]
f_{yd} = design tensile strength reinforcement [N/mm^2]
A_{conc} = area concrete pile [m^2]
M_{Rd} = bending moment resistance reinforced concrete pile [kNm]
$\sigma_{xx,eff}$ = remaining effective horizontal soil stress per meter width [kPa]
Red. = reduction effective horizontal soil stress over weak layer [%]

Influence pile rows R

The second step is to consider the influence of the amount of pile rows. For two rows of piles the bending stiffness doubles. The total reduction of horizontal soil stresses per meter width is 5% for a single row of piles and 10% for a double row. The calculation process is similar as for the shell factor calculation and the details can be found in appendix E.

Influence kinematic schematizations α

The last step to consider is the influence of the kinematic schematizations of the piles. The shielding effect of the piles is possible due to the bending stiffness of the piles and the stiffness of the support reactions. The combination of the two is expressed with α:

- $\alpha = 1$: clamped connection with relieving structure and clamped in Pleistocene layer
- $\alpha = 2$: hinged connection with relieving structure and clamped in Pleistocene layer
- $\alpha = 5$: hinged connection with relieving structure and roller support in Pleistocene layer

In appendix E the calculation process is illustrated for the influence of α with shell factors set to 1, 3 and 6,67. The reductions are minimal for $\alpha = 5$: around 4% and almost independently of the shell factor. With $\alpha = 2$ larger reductions occur: 8%, 10% and 11% for respectively $S = 1$, 3 and 6,67. The largest reductions occur for $\alpha = 1$: 13%, 18% and 20% for respectively $S = 1$, 3 and 6,67.

Comparison with deep water quay wall

The deep water quay wall in the Amazonehaven has been constructed with a double row of piles where the connection of the piles and the relieving structure is clamped. Figure 14 and Table 3 illustrate this for the quay wall model with a shell factor of 6,67. The amount of shielding of the piles on the front retaining wall is 20%. This implies that according to the method Begemann - De Leeuw the effective horizontal soil stresses for the deep water quay wall model near the area of the combi-wall and over the height of the weak Holocene layer are reduced by around 20%.

Table 3 - Calculation soil-pile interactions with clamped-clamped model, R = 2 and S = 6,67

Input		
α	1	-
S	6,67	-
R	2	-
$\beta_{support}$	0,083	-
B_{field}	0,042	-
Result		
u_3	0,0789	m
$\sigma_{3,pile}$	34,1	kPa
$\sigma_{3,soil}$	34,1	kPa
q_{pile}	102,3	kPa
M_b	1051	kNm
M_{field}	525	kNm
z	0,365	m
ρ	2	%
f_{yd}	435	N/mm^2
$M_{Rd,pile}$	642	kNm
$\sigma_{xx,eff}$	137	kPa
Reduction	20	%

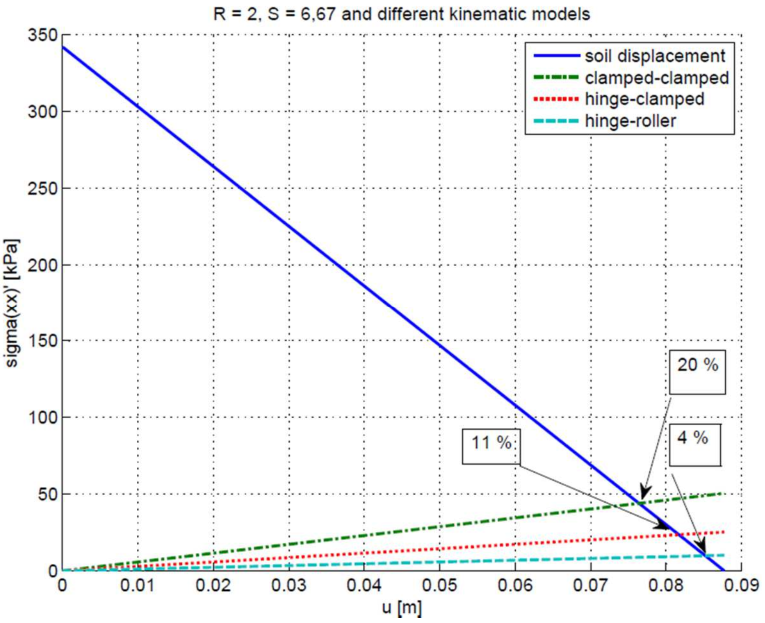


Figure 14 - Illustration soil-pile interactions with R = 2, S = 3 and different kinematic models

Horizontal pile deflections and pile bending moments
The connection between the piles and the relieving structure is supposed to be clamped in the deep water quay wall in the Amazonehaven. However, the concrete piles can become cracked after a while and therefore the hinged connection option has been discussed as well. The difference between a clamped or hinged connection with the relieving structure for the pile deflections u and pile bending moments M are plotted in Figure 15.

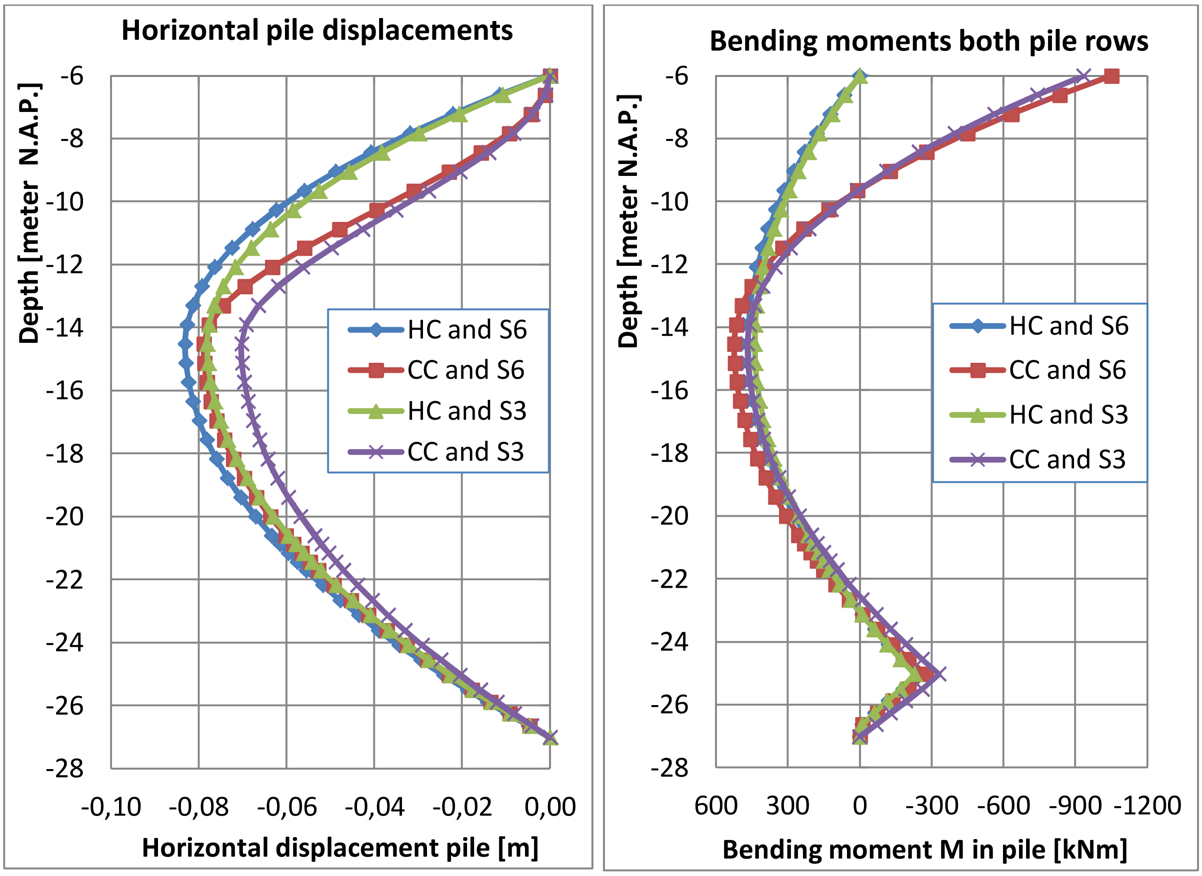


Figure 15 – Horizontal pile displacements (left) and pile bending moments (right) from analytical method

In the figure four lines are plotted corresponding to either a hinged connection (HC) or a clamped connection (CC) with the relieving structure and based on a situation where the piles only partially shield the soil stresses (S3) or fully shield the soil stresses (S6). The S3 factor implies that an individual pile in the pile row shields three times the pile diameter of soil stresses. For the S6 factor the complete distance between the piles is shielded. This distinction is made because it is not clear how large the shell factor will be.

The pile deflections are based on the maximum pile deflection u_3 and the boundary conditions from the method Van IJsseldijk - Loof. It is assumed that the deflections for both pile rows from the deep water quay wall behave in a similarly way. The maximum pile deflections vary between 7 to 9 centimeter.

The data for the pile bending moments are based on the support bending moments M_b at the relieving structure, the field bending moments M_{field} and the soil support bending moments in the Pleistocene layer. The soil support bending moments are considered to be half the field bending moments. The figure illustrates the bending moments for two pile rows combined. The maximum bending moments for one pile row are half the values indicated in the right picture of Figure 15. The maximum field and support bending moments for each pile row are respectively around 230 and -500 kNm.

3.4 Conclusions analytical method

In this chapter the deep water quay wall project Swarttouw in the Port of Rotterdam has been used to investigate the soil-pile interactions for the shielding effects on the front retaining wall. For the analysis the method Van IJsseldijk - Loof to translate soil stresses into soil displacements and the method Begemann - De Leeuw to find the soil-pile interactions have been used.

The soil-pile interactions have been investigated for the following parameters: the number of pile rows, the shell factor and the kinematic configuration of the piles. The total reduction of horizontal soil stresses per meter width depend on these parameters and varies between 4% to 20% in comparison with the situation without piles. A configuration which describes the actual situation accurately is the model with two pile rows, a shell factor of 3 and a clamped-clamped support system with a total reduction of 18%. This system with a shell factor of 6,67 instead of 3 is also likely to occur with a total reduction of 20%. This implies that according to the method Begemann - De Leeuw the effective horizontal soil stresses for the deep water quay wall model near the area of the combi-wall and over the height of the weak Holocene layer are reduced by around 18 to 20%.

Looking at the results of Figure 15 the following conclusions have been made regarding the horizontal pile deflections u and pile bending moments M:

- Higher shell factors result in larger pile deformations.
- A clamped connection between the piles and the relieving structure result in smaller deformations at the top part.
- The field bending moments are practically the same for the HC and CC models.
- High bending moments occur at the level of the relieving structure for the CC models.
- The bending moments resistance for a single pile (or pile row) is 642 kNm. The occurring bending moments are determined with two pile rows installed which basically implies that the maximum allowable bending moment is doubled to around 1284 kNm. The maximum occurring bending moments are 1051 kNm at the level of the relieving structure. Based on this (simplified) analysis the concrete piles can resist the bending moments.

An interesting aspect to consider is the influence of the shell factor S, because this factor in principle doesn’t have a constant value for soils. From the analysis it can be concluded that there is a large difference between a shell factor of 1 and 3, but the difference between a shell factor of 3 and 6,67 is not so large. The actual shell factor is expected to be between 3 and 6,67 and therefore it seems to be that the shell factor is of less importance for the total reduction due to soil-pile interactions. This also implies that the bearing piles indeed partially act as a second earth retaining wall. However, this only counts for this particular configuration. If the distance between the pile rows (b) and the center to center distance between the piles (a) changes, the results may be different. This effect is further researched with the finite element method in the next section.

4 Finite element method with option embedded piles

4.1 Introduction

This chapter describes the amount of shielding of piles on the front retaining wall of quay walls by using an finite element method (FEM). For this the program Plaxis is used with the embedded piles option. The deep water quay wall project is used as a reference case. For more details about this project see appendix A.

The plan of approach to determine the shielding effect with FEM can be explained in a few steps. First the deep water quay wall project has been modelled in Plaxis. Then to determine the shielding effect, different supports types in different quay wall models have been used to illustrate the influence of the concrete piles and MV-pile anchors. To quantify the influence between the different quay wall models, a reference model is used: the calibration model.

First the quay wall model is set up and the model schematizations are described in section 4.2. The soil layer models and material models are defined including the loads acting on the quay wall model. Then the shielding effect is examined. This is done by comparing different quay wall models which each other: the calibration model and different fixed node to node (n-t-n) support models. The calibration model is based on the deep water quay wall model in the Amazonehaven in the Port of Rotterdam. The n-t-n support models replaces the MV-pile anchors and concrete piles by spring supports which should behave in similarly ways, but without the structural elements in the models. This creates the opportunity to compare the results with the calibration model. Finally the influence of design aspects on the shielding effect are considered. For instance the influence of the center to center pile distance and the influence of the surcharge load is researched.

4.2 Schematizations quay wall model

Similarly as for the analytical method an appropriate quay wall model is set up. The finite element method allows for more accurate quay wall modeling, especially oblique installed piles and the analysis of plastically soil deformations are possible. The deep water quay wall is modeled with the plain strain model. This is a two dimensional model with an effective width of one meter. The stresses in the model are calculated with the 15-node option. The mesh size of the model, which is the two dimensional model size area, should be chosen large enough to avoid the influences of mesh boundary conditions. The mesh size of the quay wall model is given in appendix F.

First the soil data models are defined by analyzing the geological soil data from appendix A.2. Secondly the construction material models are set up. For this the information of the deep water quay wall project is used (appendix A.1). Then the loads acting on the quay wall model are defined. These loads should reflect the expected loads acting on the quay during its lifetime. This is however not the same as the design loads acting on the quay. This research focusses on the shielding effect of piles and anchors on quay walls. For this the most likely occurring loads during the lifetime of the quay wall are of more interest.

4.2.1 Soil data and construction material models

Soil data

In Figure 16 the soil layers for the quay wall model are illustrated. This soil layers and data are based on geological investigations in the Amazonehaven. The subsurface can be separated into three parts: the earth fill part from around 0 to +5 meter N.A.P., the Holocene layers from -22 to 0 meter N.A.P. and the Pleistocene part below -22 meter. The earth fill part originates from the port extension works on the Maasvlakte.

The figure also illustrates the soil layers and there identification numbers. The subsoil mostly consist out of sand with formations of clay and silt in the Holocene layers. The boundary between the Holocene and Pleistocene layer is characterized by a weak clay layer with identification number three. Soil layer with ID number two is the so called ‘sponge’ layer underneath the relieving structure (see Figure 17). This layer is based on soil layer with ID number five, but with weaker strength and stiffness properties. This layer should avoid high soil pressures underneath the relieving structure. From field measurements it has been concluded that the relieving structure is practically not in contact with the soil underneath it. The Pleistocene layer consist of relatively strong sand with formations of gravel.

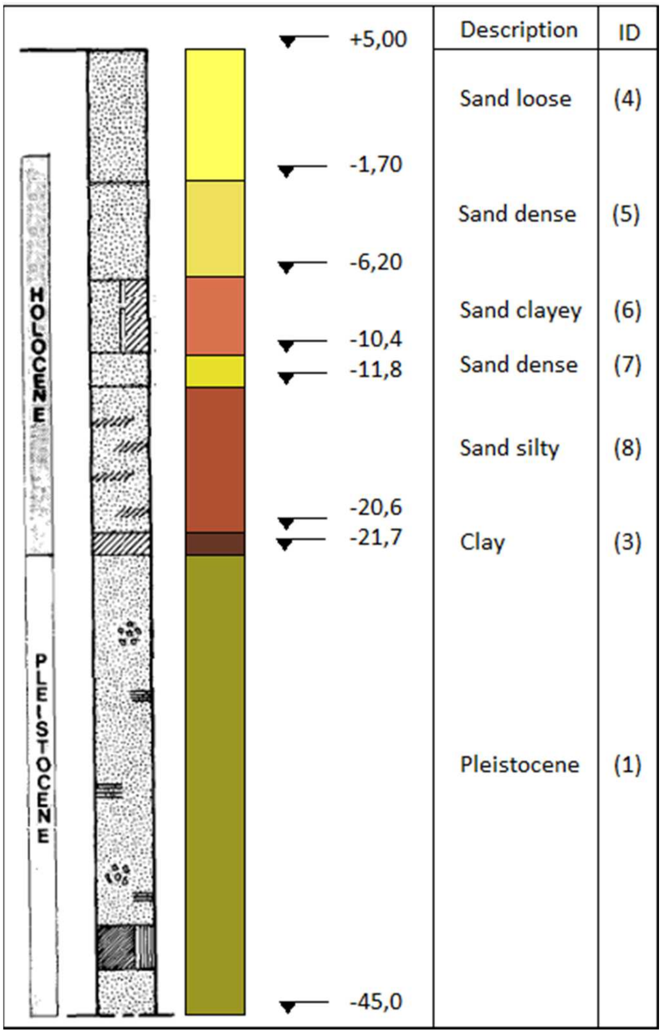


Figure 16 - Soil data quay wall model finite element method

Relieving structure

The relieving structure is modeled as a linear elastic non-porous soil material with the characteristics of concrete. In this way the thickness of the structure is included in the model and then the skin friction interaction between the soil layers and the concrete body can be modeled correctly. Concrete walls and slab can become cracked after a certain time which reduces the stiffness of the structure. This effect is incorporated by reducing the modules of elasticity of the concrete by half. The dimension of the structure are identical to the deep water quay wall in the Amazonehaven (see Figure 34 in annex A). The height of the structure ranges from -6 to +5 meter N.A.P. whilst the total width is around 18 meter. The thickness of the concrete floor is 1,5 meter. The combi-wall, MV-pile anchors and concrete bearing piles are attached to this floor.

Combi-wall

The combi-wall consist out of pipe pile elements with intermediate sheet pile elements to create a solid and stiff earth retaining wall. The pipe piles have a diameter D of 1778 millimeter and thickness t of 20 millimeter. These dimensions are larger than for the deep water quay wall project, which by calibrating was necessary because of the chosen surcharge load and because of the vertical orientation of the combi-wall. The intermediate sheet pile elements are three times GU 16-400 elements. The pipe piles are modeled with a center to center distance (system length) of three meter. The strength and stiffness properties of the combi-wall have been determined for the system length (Arcelor Mittal, 2013). Plaxis however considers structural elements with an effective width of one meter. The strength and stiffness characteristic of the combi-wall thus have also been determined for an unit width of one meter. The combi-wall ranges from -6 to -36,5 meter N.A.P..

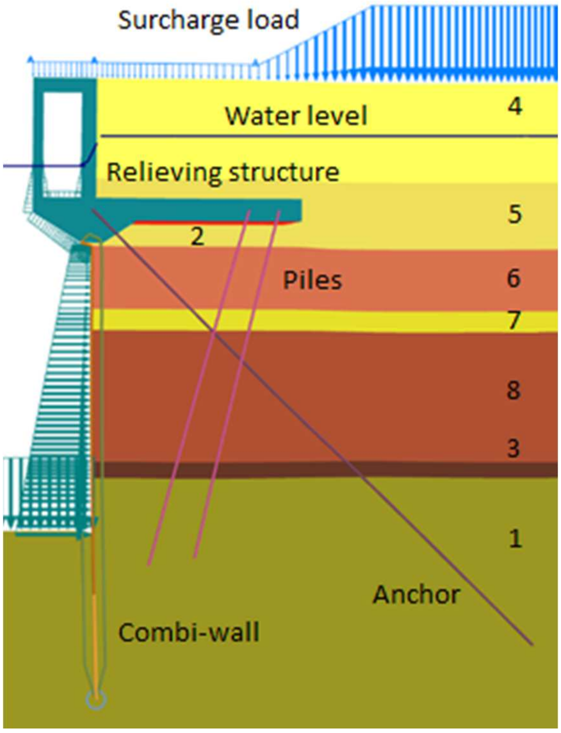


Figure 17 - Quay wall model finite element method

MV-pile anchors and concrete piles

In contrast with the combi-wall, do the anchors and piles require a different modeling approach. These structural elements are installed into the subsoil to respectively anchor the quay wall and support the relieving structure. They are not intended to create a solid earth retaining wall. If this is indeed the case will be discussed later on. The anchors and piles can be modeled in Plaxis with the embedded piles approach.

The MV-pile anchors are of the type PST 370/152 which are steel H-profile elements with special equipment attached to the tip to create a rough grout body around the anchor during installation. The circumference of the grout body is around 1,82 meter. The MV-pile anchor is installed with an angle of 45 degrees with the horizontal and with respect to the system length of about three meter. It ranges from -4,5 to -33 meter N.A.P..

The concrete piles at the backside of the quay wall model are aligned into two rows with center to center distance (b) of two meter between the rows and the piles are spaced three meter apart in the field of the rows (a). They are pre-tensioned to increase the strength of the piles which is particularly necessary during installation. The squared piles have a width of 45 centimeters. To increase the stability of the quay wall the piles are installed under an angle. The first pile row is installed with an angle of 1:3,5 from -4,5 to -27,5 meter N.A.P. whilst the second pile row is installed with angle 1:4 from -4,5 to -27 meter N.A.P..

The strength and stiffness properties of the soil layers and the material properties can be looked up in the tables of annex F.

4.2.2 Load schematizations and simulation lifetime of quay wall

Surcharge load

The deep water quay wall is designed to store large bulk loads of iron ore on the quay. The maximum design load was designed to be a pile of iron ore with a vertical pressure of 450 kPa acting on the quay at 70 meter from the waterfront. It seems to be reasonable to apply this load for the quay wall model. However, it seems to be that the actual loads have never been reached during the relatively short lifetime of about 22 years. Therefore the surcharge load applied for the model is set to 200 kPa and this load is also positioned closer to the waterfront with a distance of about 19 meter (see Figure 17). From the waterfront onwards the surcharge load initially is set to 40 kPa to simulate for instance traffic loads, than after 11 meter the load increases to 200 kPa over a distance of 7,6 meter and remains on 200 kPa afterwards. This simulates a continuous pile of iron ore.

Water level head difference

The water level head difference is based on water level data of the Port of Rotterdam and on groundwater level tables. The water level in the harbor basin is set to -1,00 meter N.A.P. and the groundwater level to +1,00 meter N.A.P.. This results in a water level head difference of two meter. Near the relieving structure drains are installed to lower the head difference. It is expected that the actual head difference therefore is smaller. However drains can become clogged which can result in higher water level head differences. For the calibration model a difference of two meter has been used. A smaller difference is discussed in section 4.4.3.

Staged construction mode quay wall model

To simulate the behavior of the deep water quay wall in a correct way, the quay wall model should also be built up from scratch. This process is called the staged construction mode which is an option available in Plaxis and this is illustrated in Figure 18. After creating the quay model including defining all soil and material data the staged construction mode can be defined. The initial step is to simulate the natural existing situation with a flat surface and with all soil layers activated only being affected by the earth's gravity. Then a trench is dug to -6 meter N.A.P.. Next the combi-wall, MV-pile anchors and concrete piles are installed. On top of these foundation members the concrete relieving structure is casted. If the concrete has hardened enough the initial soil profile behind the quay is restored. The next step is to dig and dredge the harbor basin to the required depth. Now the construction phase is finished and the quay wall can be used.

In the use phase the quay wall model should withstand the frequently and infrequently design loads. The loads defined for this model are the surcharge load and the water level head difference. These load are considered to act constantly on the quay wall during the lifetime.

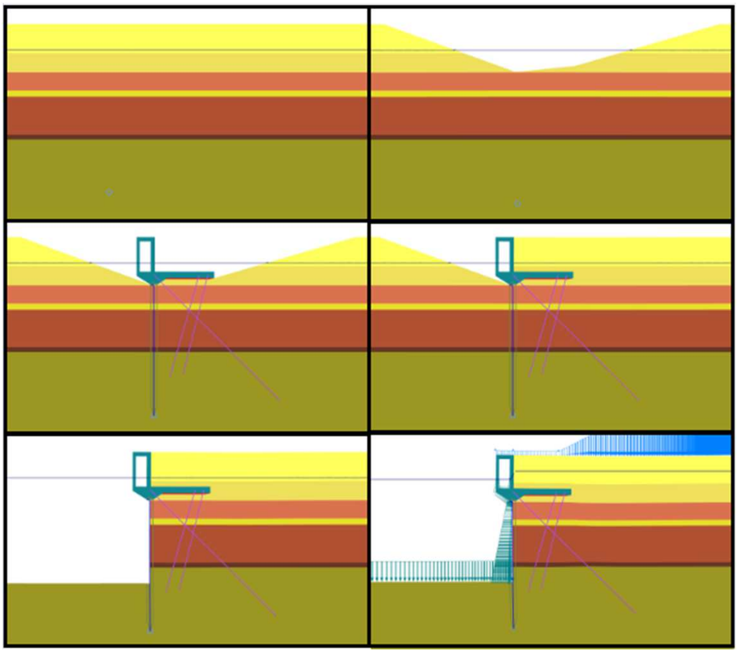


Figure 18 - Staged construction mode quay wall model

4.3 Shielding effect from piles and anchors

In order to understand the shielding effect which the piles and anchors underneath the relieving structure can have on the combi-wall, a calibration model is set up. This model is elaborated to give an insight in the structural forces and deformations of the quay wall model and it used to compare the outcomes of the shielding effect from the piles and from the anchors separately. During the comparison the deformations of the combi-wall and relieving structure should be close guarded. This step is necessary because the shielding effect is likely not only dependent on the presence of the piles and anchors, but also on the displacements of the combi-wall as well. A very stiff combi-wall displaces less under loading than a weaker combi-wall. The soil body thus also displaces less in the case of a stiffer combi-wall. Then the forces on the piles and anchor are expected to be smaller as well. Therefore in order to research the shielding effect the displacements of the combi-wall for all quay wall models should be kept the same as for the calibration model. This should also be the case for the concrete relieving structure.

First the calibration model is discussed, then the shielding effect of the concrete piles. Next the shielding effect of the MV-pile anchors and finally the combination of both.

4.3.1 Deformations and structural forces for the calibration model

Deformed mesh

The calibration model is based on the schematizations described in section 4.2. This implies a system length of three meter for the steel pipe pile elements of the combi-wall, the MV-pile anchors and the concrete piles at the back. The deformed mesh of the quay wall model after applying the loads is illustrated in Figure 19. Due to the surcharge load and water level head difference the relieving structure and combi-wall start to move to the waterside. The anchors retain the quay wall while the piles support the relieving structure. They also move slightly to the waterside as well. This movement is larger over the Holocene soil layer than at the Pleistocene layer. Apparently some shielding of these elements occur. The maximum horizontal displacement of the deformed mesh is 36 centimeter which is near the middle part of the combi-wall whilst the maximum vertical displacement is 50 centimeter which is just underneath the surcharge load at ground level and about 24 meter from the waterfront. The total horizontal u_x and vertical u_y displacements of the calibration model are graphically illustrated in annex F.3.

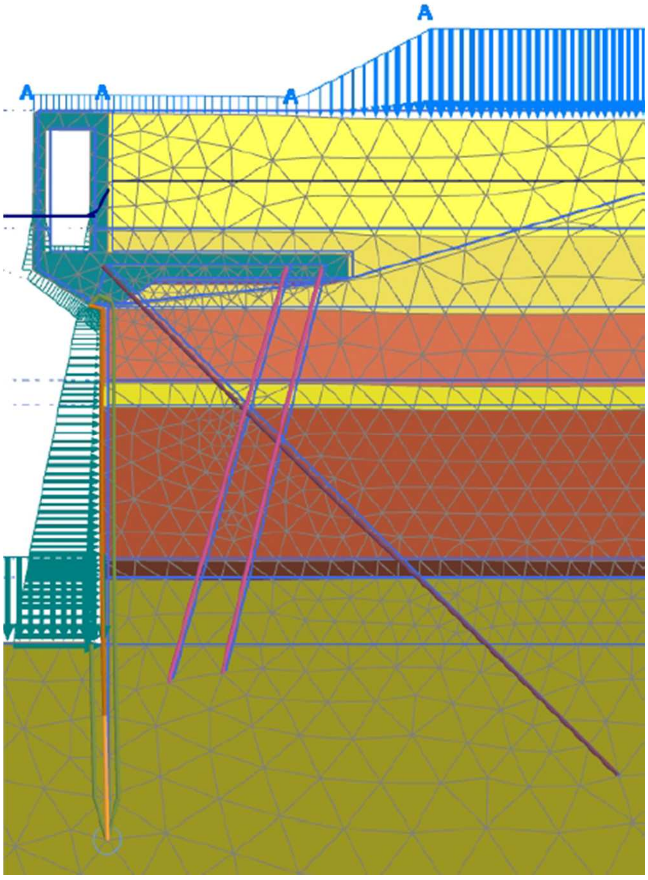


Figure 19 - Deformed mesh calibration model

Maximum forces M,Q and N in combi-wall

The bending moments M, shear forces Q and axial forces N of the combi-wall for the calibration model are given in Figure 20. The forces are given per meter length quay wall. The bending moments at the top part of the combi-wall are created due to the eccentric connection with the relieving structure. As a result the maximum bending moment in the field is lower than without this eccentric connection. The high shear force Q at the top part is also the result of the eccentric connection. The axial force is rather constant over the length of the combi-wall. The small increase downwards is created due to skin friction between the combi-wall and the soil.

Maximum forces M,Q and N in anchors and piles

In Figure 21 and Figure 22 the bending moments M, shear forces Q and axial forces N of the MV-pile anchors and concrete piles are given. The bending moments for both the anchors and the piles illustrate some shielding over the Holocene layer. This can recognized by the changing of the sign from negative bending moments to positive bending moments. This happens near the relieving structure and near the boundary with the Pleistocene layer. If the bending moments swap signs, at those location the shear forces are at its maximum. A part from the horizontal soil stresses acting on the anchors and piles is transferred by shear forces to the top and bottom part of the subsoil. The axial forces in the anchors and the piles are almost constant over the height of the Holocene layers and gradually decrease downwards into the Pleistocene layer. The concrete bearing piles do have pile tip bearing capacities which explains the remaining axial forces at the bottom of the piles.

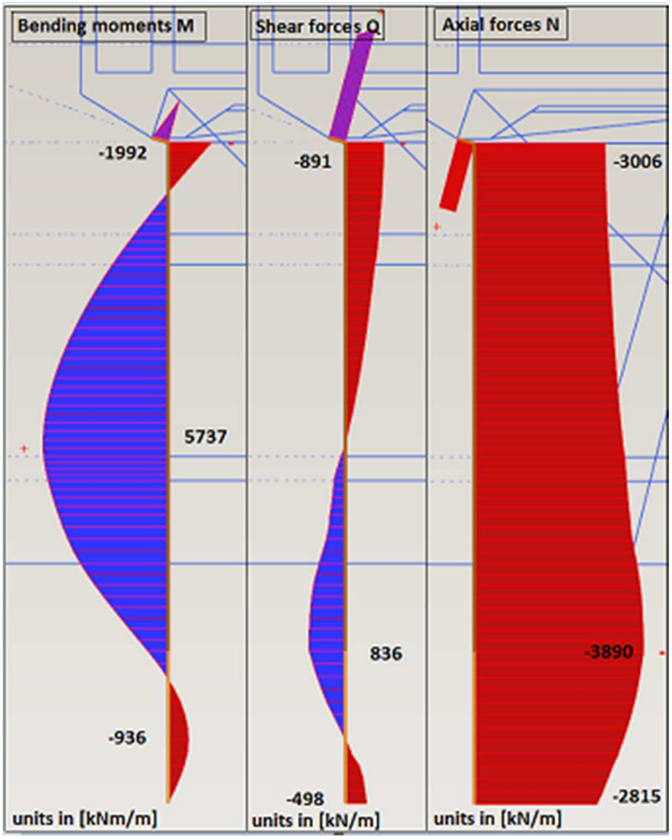


Figure 20 - Structural forces M, Q and N in combi-wall for calibration model per meter length quay wall

Total maximum displacements |u| of the foundation elements

The displacements |u| of the combi-wall, the MV-pile anchors and the concrete piles are in the same order of magnitude (see Figure 21). This make sense because the surrounding soil interacts with the structural elements more or less in the same way. However it is interesting to note the difference between the ends and the middle parts of these foundation elements. Just as with the deformed mesh from Figure 19 the movement is larger over the Holocene soil layers than at the Pleistocene layer.

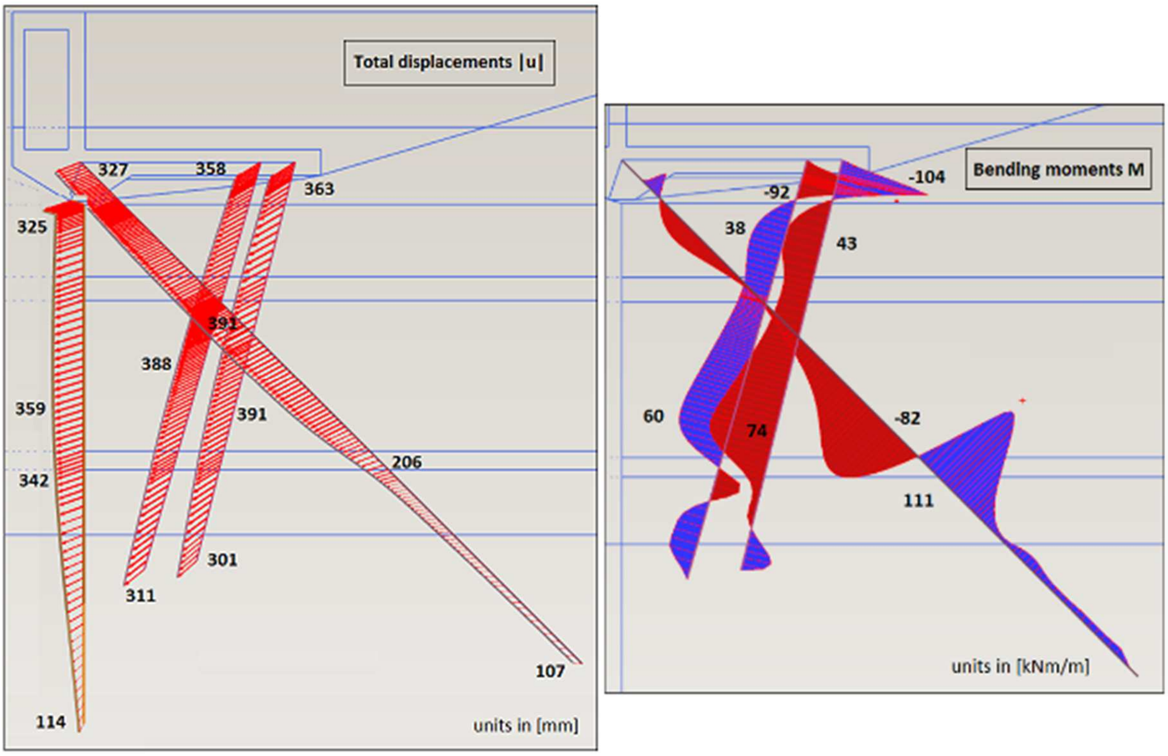


Figure 21 – Total maximum displacement |u| for all foundation members (left) and maximum bending moments in anchors and piles (right) for the calibration model per meter lenght quay wall

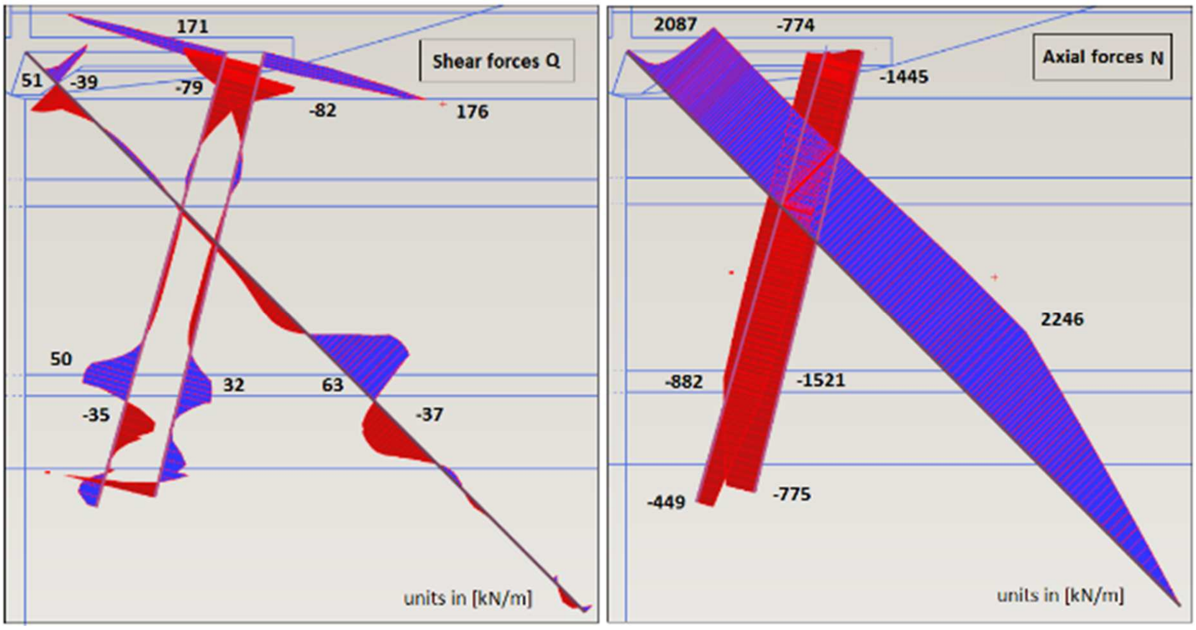


Figure 22 – Shear forces Q (left) and axial forces N (right) in anchors and piles for the calibration model per meter length quay wall

4.3.2 Fixed node to node support models

One way to determine the shielding effect is to compare the results of the calibration quay wall model with slightly different quay wall models based on the calibration model, for instance with the fixed node to node (n-t-n) support models illustrated in Figure 23. In the n-t-n support models either the piles or the anchors or both the piles and anchors have been replaced by spring supports. In the n-t-n support models the soil stresses and displacements underneath the relieving structure are different than for the calibration model. Due to this difference the shielding effect of the piles and anchors can be researched.

The shielding effect can be discussed by comparing the bending moments differences for the combi-wall between all four models from Figure 23. If the soil stresses underneath the relieving structure are smaller, it is expected that the bending moments in the combi-wall are also smaller. However, the soil deformations should also be discussed, because larger soil deformations in general results in larger bending moments in the combi-wall.

Another approach is to qualitatively analyze the differences of the effective horizontal and vertical soil stresses underneath the relieving structure and by this way understand the shielding effect. Finally a vertical cross-section in the subsoil one meter in front of the combi-wall is examined to determine the effective horizontal force over the Holocene layer, the Pleistocene layer and both layers together. In this way the shifting of forces to the relieving structure and deeper laying Pleistocene layer can be analyzed and compared with the results from the analytical method. The n-t-n support models are explained in more detail below.

4.3.2.1 Fixed n-t-n pile model

To examine the shielding effect of the piles an different quay wall model has been set up which replaces the piles with fixed n-t-n pile supports. In this way the influence of the piles can be recognized. The difficulty with replacing the piles by the supports is to keep the deformations of the quay wall model and especially the combi-wall the same. This can be achieved by determining the spring stiffness of both pile rows from the calibration model and apply these to the pile supports. That the supports indeed illustrate the same behavior of the combi-wall and the relieving structure is illustrated in Figure 66 of annex F.3. Here the total displacements |u| for the calibration model and the fixed n-t-n pile support model are compared. The displacements for both models are practically identical especially near the connection between the relieving structure and the combi-wall. However, the combi-wall-concrete piles interactions (see section 1.3.3) at the bottom part of the combi-wall are difficult to model correctly with the pile support model. Nonetheless with this model the shielding effect of the piles can be examined.

4.3.2.2 Fixed n-t-n anchor model

The shielding effect of the MV-pile anchors is analyzed in the same way as for the concrete piles. This implies that the anchors ranging from -4,5 to -33 meter N.A.P. are substituted by fixed n-t-n anchor supports with similarly spring stiffness characteristics. Here also the deformations of the quay wall should be in line with the deformations of the calibration model to avoid the influence of the displacements of the relieving structure on the shielding effect. In Figure 66 of annex F.3 the total displacements |u| of the relieving structure is illustrated for the calibration model and the n-t-n anchor support model. The use of these supports result in similar deformations for the relieving structure which implies that a comparison between both models to determine the shielding effect of anchors can be made.

4.3.2.3 Fixed n-t-n pile & anchor model

It is expected that the combination of piles and anchors in front of the combi-wall results in the largest shielding of effective horizontal soil stresses. In order to determine this combined effect from both structures the fixed n-t-n pile & anchor support model is created. Again the quay wall deformations should be in the same order as for the calibration model (see Figure 66).

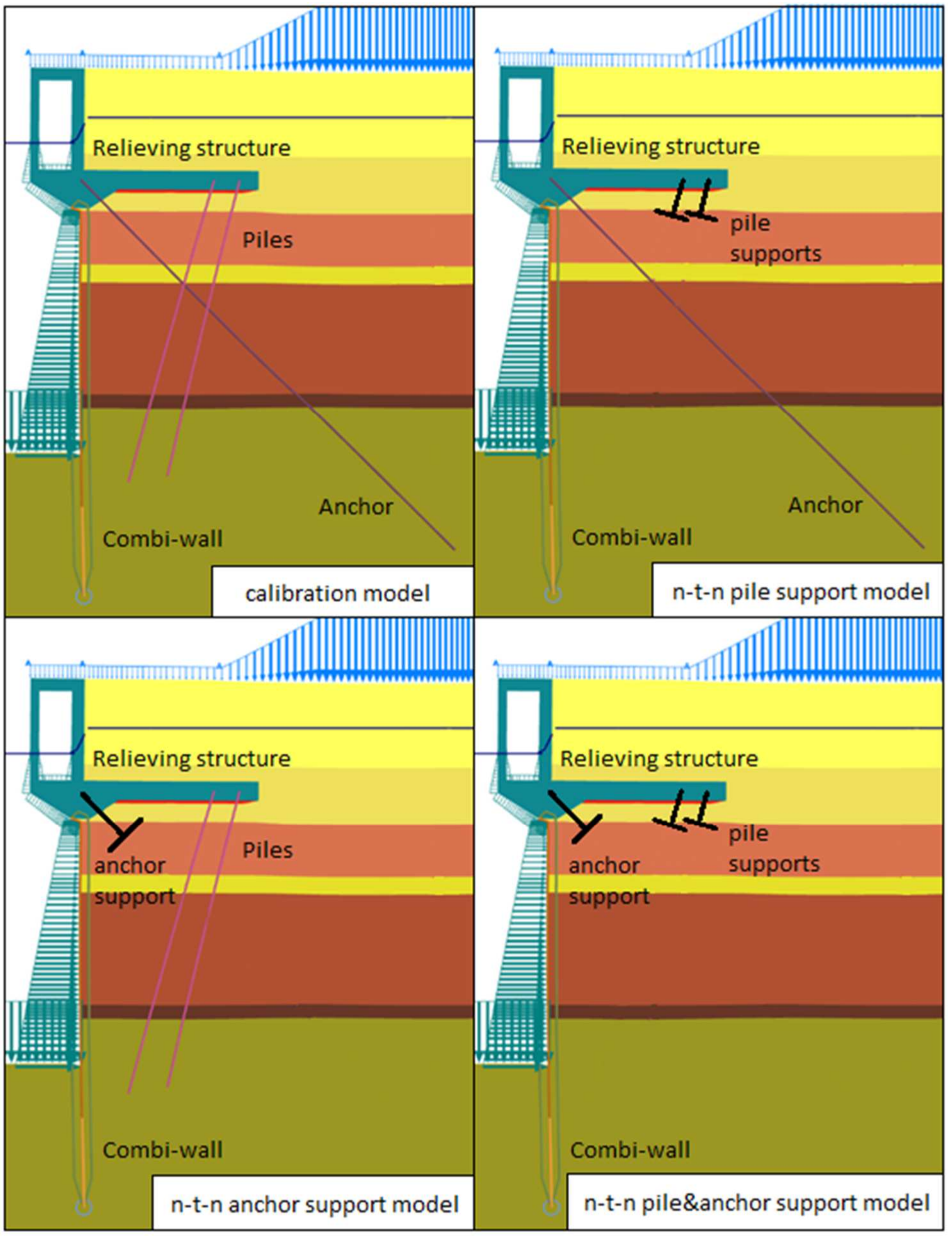


Figure 23 - Calibration model (top left), n-t-n pile support model (top right), n-t-n anchor support model (bottom left) and n-t-n pile & anchor support model (bottom right)

Comparison of effective horizontal soil stresses σ_{xx}'

If the piles and anchors indeed shield the combi-wall, this effect should be recognized by considering the effective horizontal soil stresses underneath the relieving structure. The stresses for the calibration model and the three n-t-n support models are visualized in Figure 24. The n-t-n support models can be recognized by the black anchors inside the figures. The scale of the effective horizontal soil stresses is limited from 0 to -500 kN/m^2. Underneath the combi-wall higher stresses occur, but these stresses are not interesting for this comparison.

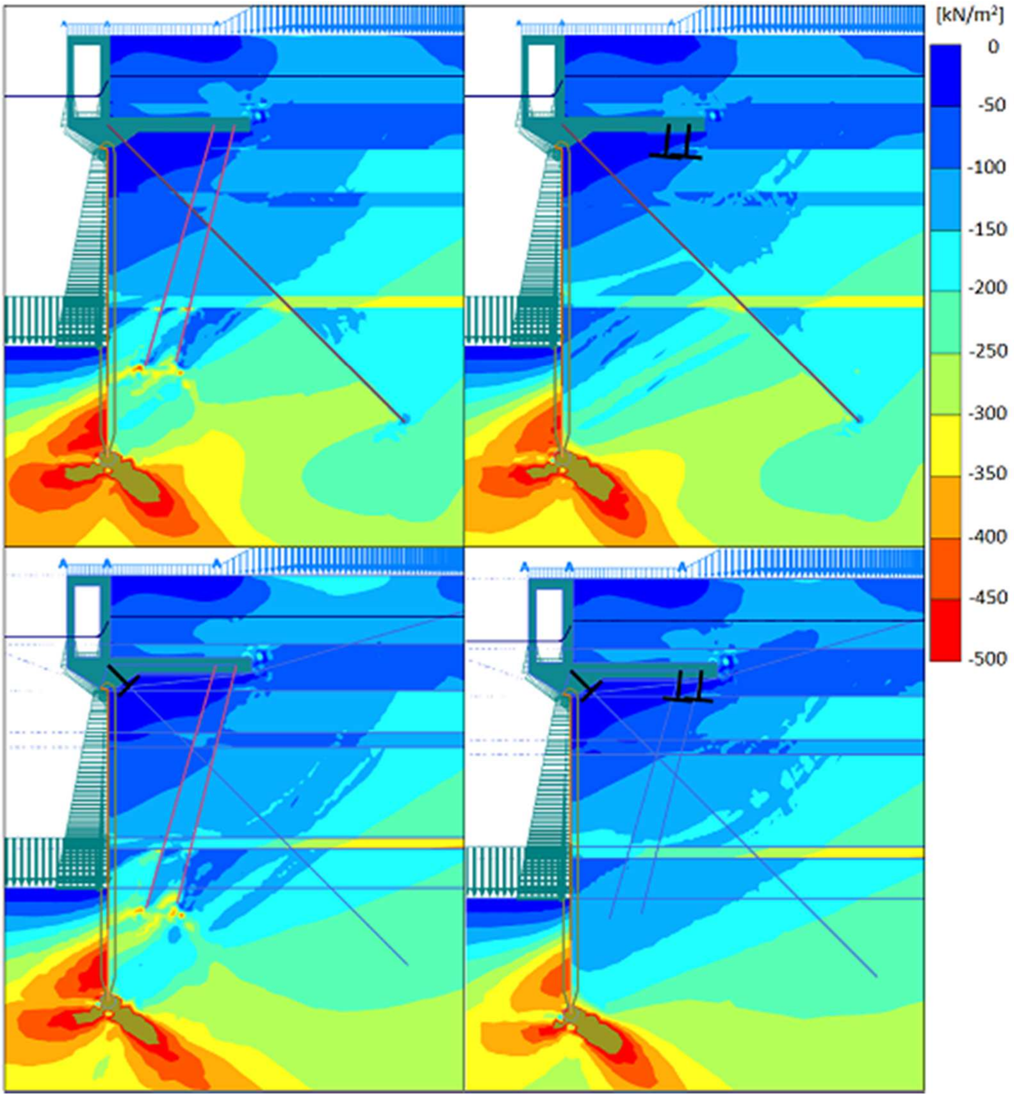


Figure 24 – Effective horizontal soil stresses for the calibration model (top left), the n-t-n pile support model (top right), the n-t-n anchor support model (bottom left) and the n-t-n pile & anchor support model (bottom right)

The important conclusions from considering the effective horizontal soil stresses are:

- The stresses acting on the combi-wall are the largest in the Holocene layer without MV-pile anchors and concrete piles.
- The concrete piles shield more than the MV-pile anchors. The combination of both give the highest reduction of stresses near the combi-wall over the Holocene layer.
- The pipe-pile interactions (combi-wall concrete piles interactions) have a large influence on the behavior of the combi-wall. The stresses on the waterside in the Pleistocene layer are larger when the concrete piles are installed. This will have an influence on the deformations and bending moments of the combi-wall.

- The MV-pile anchors created large stresses in the Pleistocene layer which reach to the bottom part of the combi-wall. This effect may also influence the deformations of the lower part of this wall.

Comparison of effective vertical soil stresses σ_{yy}'

The analysis of the effective vertical soil stresses is based on the same analogy as the effective horizontal soil stresses. The scale to indicate the stresses is limited from 0 to -1000 kN/m^2. However, the stresses underneath the combi-wall are larger than the scale indicates, but this is not important for this comparison. See annex F.3 for more pictures.

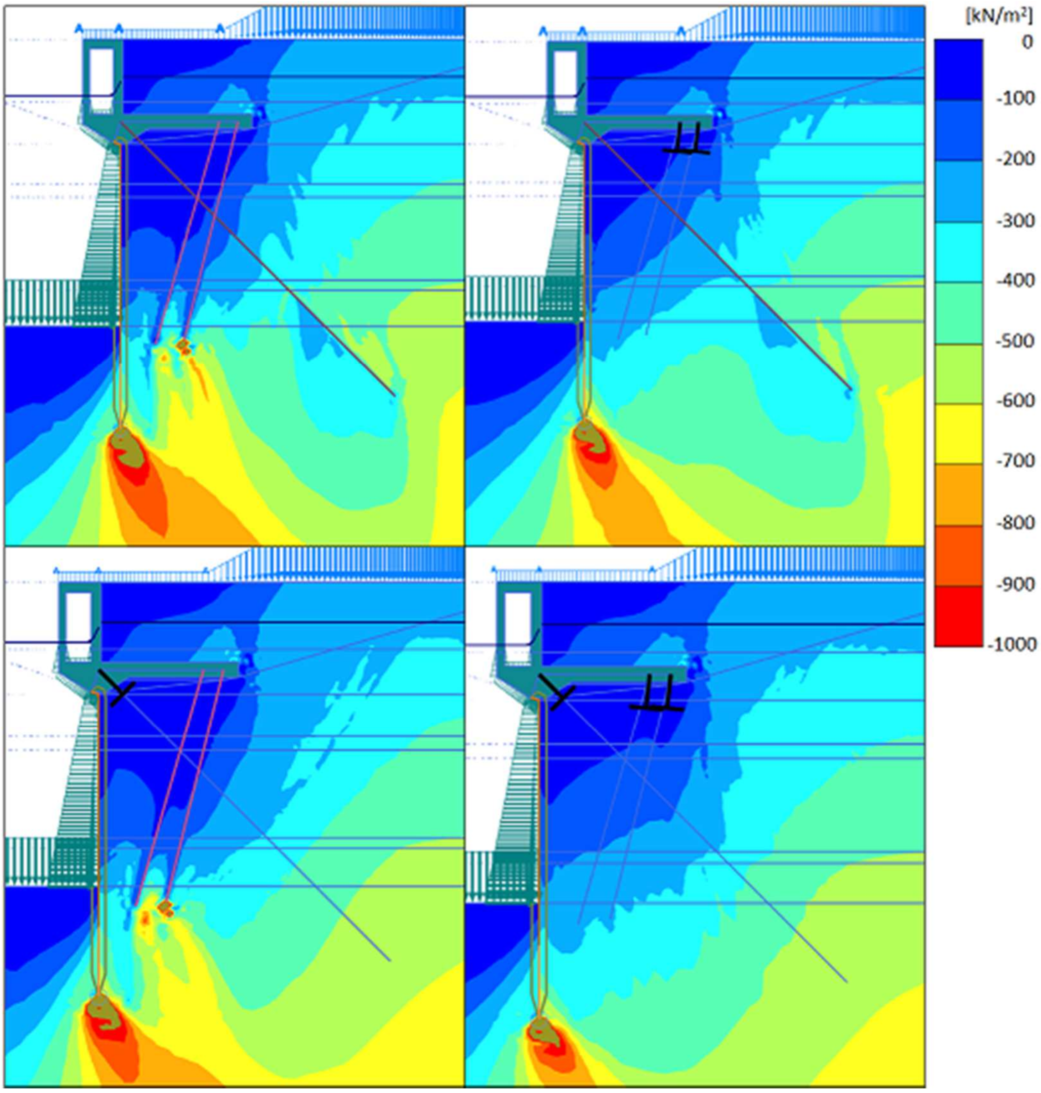


Figure 25 – Effective vertical soil stresses for the calibration model (top left), the n-t-n pile support model (top right), the n-t-n anchor support model (bottom left) and the n-t-n pile & anchor support model (bottom right)

The important conclusions considering the effective vertical soil stresses are:

- The stresses with MV-pile anchors and concrete piles installed are lower underneath the relieving structure. This effect can be explained due to negative skin friction between the concrete piles and the settling soil.
- The MV-pile anchors increases the stresses above the anchor whilst the stresses are lowered underneath the anchor. The anchor wants to be pulled out of the subsoil.
- The concrete piles create large stresses under the pile tips in the Pleistocene layer. This will change the behavior of the bottom part of the combi-wall.

Comparison bending moments M and horizontal deflections u_x of the combi-wall

In Table 4 the comparison between the calibration model and the three n-t-n support models is illustrated. The bending moments near the support connection with the relieving structure are almost identical for all four models. The same accounts for the horizontal deflection at the connection with the relieving structure. The support bending moment is created due to the eccentric connection of the combi-wall with the relieving structure. The field bending moments however differ a lot between the models which is also the case for the maximum horizontal field deflections.

The table also illustrates the bending moment differences between the n-t-n support models and the calibration model expressed in %. For instance, the difference for the field bending moments for the n-t-n pile model is -10% in comparison with the calibration model. The bending moments thus are actual larger if the piles are present in the model. This seems to be contradictory in the first place, because the idea was that the presence of the piles would lower the bending moments in the combi-wall. In the next paragraph this effect is indeed recognized and the piles do indeed shield the combi-wall, but this cannot be concluded based on the comparison between the bending moments.

An explanation for this contradictory results is because the n-t-n support models cannot exist in the real world. Piles and anchors must be installed to stabilize the quay wall. In the calibration model the piles and anchors are installed which will cause larger deformations for the combi-wall. This explains the higher field bending moments in the combi-wall for the calibration model. The reason for the larger deformation is causes by the pipe-pile interactions which do not occur for the n-t-n pile support models. This interaction creates larger deformations for the combi-wall resulting in larger bending moments (see Table 4). And while the piles are not present in the n-t-n pile and n-t-n pile & anchor models, thereby avoiding the pipe-pile interactions, the bending moments in the combi-wall are lower.

For the n-t-n anchor model the anchors are not installed and only the piles are present. Here also large pipe-pile interactions occur. The field bending moment difference in the combi-wall is 5% in comparison with the calibration model which implies that the anchor slightly shields the combi-wall for about 5%.

If the anchors and piles are both not present which is the case in the n-t-n pile & anchor model, the field bending moment difference for the combi-wall is -18%. This result is again contradictory because of the pipe-pile interactions.

The only conclusion which can be formulated from this paragraph is that the presence of the concrete piles combined with the pipe-pile interactions result in higher bending moments in the combi-wall. The shielding effect based on the bending moments in the combi-wall cannot yet be recognized, because the pipe-pile interactions disturb the comparison.

Comparison effective horizontal forces in vertical cross-sections in front of the combi-wall

The analysis of the bending moments in the combi-wall from the previous paragraph did not clarify the shielding effect due to limitations of the n-t-n support models. Therefore the horizontal soil stresses one meter in front of the combi-wall are now examined. To exclude the pipe-pile interactions, the stresses are separated for the Holocene soil layers, the Pleistocene layer and a combination of both. The stresses can be expressed as an equivalent horizontal force (F_{equi}) with Plaxis. The results of this analysis can be found in Table 4.

Now the shielding effect expressed as the reduction of effective horizontal soil stresses due to the concrete piles can be recognized. A shielding effect of 20% over the Holocene layer is reached due to the presence of the piles only. The combination of concrete piles and MV-pile anchors result in a shielding effect of 16%.

Interesting to note is the influence of the MV-pile anchors. In the n-t-n anchor model, the equivalent horizontal force is lower than for the calibration model resulting in a reduction of -4%. This can be explained due to the fact that the anchor is loaded with a tensile force which creates additional horizontal soil stresses acting on the combi-wall. It explains why the equivalent horizontal force over the Holocene layer is larger for the calibration model where the anchor is installed. Another interesting aspect to note is the negative differences over the Pleistocene layer for the n-t-n pile model with -16% and for the n-t-n pile & anchor model with -24%. This is due to the pipe-pile interactions.

The conclusion from this analysis are that the concrete piles indeed shield the combi-wall by about 20%. It should be noted that this reduction is based on the effective horizontal soil stresses in a vertical cross-section one meter in front of the combi-wall and considered over the Holocene layer. The shielding effect of the MV-pile anchors is difficult to recognize, because the anchors create effective horizontal soil stresses themselves. If in Figure 24 the effect of only the MV-pile anchors present (see top right picture) is compared with the model with no anchors present (see bottom right picture), it seems to be that the anchors do not have a large influence on the shielding effect. The bending stiffness's of the anchors are also about eighteen times smaller than for the concrete piles which confirms this conclusion.

Table 4 – Comparison for all quay wall models with the calibration model concerning bending moments M, horizontal deflections Ux and the equivalent horizontal forces F in a vertical cross-section one meter in front of the combi-wall

Results\Model		calibration model	n-t-n pile model	n-t-n anchor model	n-t-n pile & anchor model	small EI pile model
$M_{support}$	[kNm/m]	-1992	-2015	-1998	-2017	-1696
	Δ [%]	-	1	1	1	-15
M_{field}	[kNm/m]	5737	5174	6037	4704	6907
	Δ [%]	-	-10	5	-18	20
$U_{x,support}$	[mm]	-325	-334	-323	-330	-332
	Δ [%]	-	3	0	2	2
$U_{x,field}$	[mm]	-359	-345	-362	-330	-389
	Δ [%]	-	-4	1	-8	8
$F_{equi,Holocene}$	[kN/m]	-1108	-1333	-1064	-1280	-1383
	Δ [%]	-	20	-4	16	25
$F_{equi,Pleistocene}$	[kN/m]	-3271	-2741	-3337	-2496	-3103
	Δ [%]	-	-16	2	-24	-6
$F_{equi,combi}$	[kN/m]	-4390	-4085	-4412	-3787	-4486
	Δ [%]	-	-7	1	-14	2

$M_{support}$ = support bending moments combi-wall [kNm/m]
M_{field} = field bending moments combi-wall [kNm/m]
$U_{x,support}$ = horizontal deflections combi-wall at support [mm]
$U_{x,field}$ = horizontal deflections combi-wall at field [mm]
$F_{equi,Holocene}$ = equivalent horizontal force Holocene cross-section [kN/m]
$F_{equi,Pleistocene}$ = equivalent horizontal force Pleistocene cross-section [kN/m]
$F_{equi,Combi}$ = equivalent horizontal force over height combi-wall [kN/m]
Δ = positive or negative difference with calibration model [%]

4.3.3 Small bending stiffness pile model

The previous analysis indicated the shielding effect of the concrete piles based on the effective horizontal soil stress reductions over the Holocene soil layers in front of the combi-wall. However, no clear answers were given about the field bending moment reductions in the combi-wall due to the complex pipe-pile interactions. Therefore another model is created which includes the pipe-pile interactions and at the same time avoids the shielding effect. This small bending stiffness pile model (small EI pile model) includes the pile bearing capacity, but neglects the bending moments stiffness of the piles. The occurring bending moments in the combi-wall can be compared with the bending moments from the calibration model. The differences between the two models give an indication of the shielding effect.

The effective horizontal soil stresses for both models are indicated in Figure 26. The left picture shows the stresses for the calibration model whilst the right picture shows the stresses for the small EI pile model. Due to the absence of bending moment stiffness's of the piles in the small EI pile model, the deflections of the relieving structure are different than from the calibration model. Therefore an additional horizontal spring support (n-t-n support) is added to keep the deformations of the relieving structure similarly. However, this is difficult to achieve, because the concrete piles behave in a completely different way with a small EI: the piles are very flexible and become shorter than for piles with a normal EI. At the same time it is not possible to support the relieving structure in the vertical direction, because then the pipe-pile interactions are disturbed. Therefore the best way to achieve a comparable situation is to apply a horizontal spring support near the relieving structure (see right picture of Figure 26). The comparison of the total deflections |u| for the relieving structure is illustrated in appendix F.3. Here it can be seen that the total deformations |u| between the calibration model and the small EI pile model are different, but at the location of the connection with the combi-wall they are practically the same.

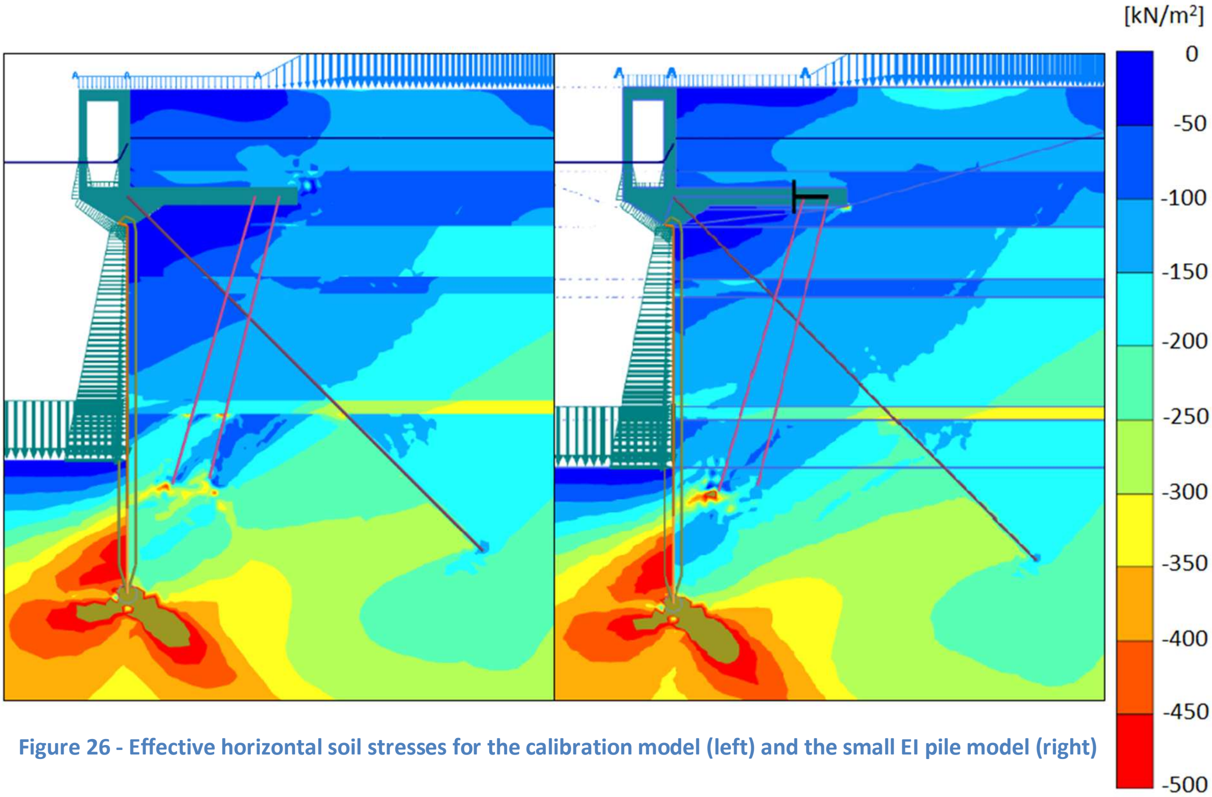


Figure 26 - Effective horizontal soil stresses for the calibration model (left) and the small EI pile model (right)

The effective horizontal soil stresses from the small EI pile model are comparable with the n-t-n pile support model from the previous section (see top right picture of Figure 24). In both cases the piles are either not present or they have small bending stiffness's properties, but the main difference is the influence of the pipe-pile interactions which are still present in the small EI pile model.

The comparison with the calibration model concerning the bending moments in the combi-wall, horizontal deflections and the effective horizontal forces in vertical cross-sections one meter in front of the combi-wall are also indicated in Table 4. The small EI pile model indicates a maximum field bending moment difference of 20% for the combi-wall. However, the support bending moments are also lower for the small EI pile model and therefore this difference is considered to be too high. This reduction can be regarded as an upper bound limit for the field bending moment reductions in the combi-wall.

The equivalent horizontal force over the Holocene layer indicates a difference of 25% which is more than the 16% and 20% of the n-t-n support models. For the equivalent force in the Pleistocene layer the difference is smaller. These differences can be explained due to the pipe-pile interactions. The small EI pile model includes theses interactions, but only for the vertical forces acting on the piles. The horizontal stresses acting on the piles are not shielded, because the piles have no bending stiffness's. Then the pipe-pile interactions are also smaller. Therefor this model has also some downsides to determine the shielding effect based on the field bending moments in the combi-wall.

4.3.4 Comparison of fixed n-t-n support and small EI pile models

The n-t-n support models illustrate the most accurate effective horizontal soil stress reductions in front of the combi-wall whilst the small EI pile model illustrate the most accurate field bending moment reductions in the combi-wall. However both methods do have strong simplifications to determine the shielding effect. In order to determine the possible field bending moment reductions in the combi-wall, the results for both methods should be combined.

The effective horizontal force over the Holocene layer is too high for the small EI pile model, because the deformations of the relieving structure and thereby also the soil deformations in the small EI pile model are larger than for the calibration model. The reduction of the effective horizontal soil stresses of 25% should be in the range of 16 to 20% from the n-t-n support models. The field bending moment reduction in the combi-wall of 20% for the small EI pile model is therefore also too high. The ratio of the effective horizontal soil stresses between both methods is 16%/25% = 0,64. Using this ratio on the field bending moment reduction in the combi-wall of the small EI pile model results in a reduction of 0.64*20% = 13%. This value can be regarded as an lower bound limit for the field bending moment reduction in the combi-wall.

4.4 Design aspects influencing shielding effect

This section focusses on the influence of design choses for quay walls with relieving structures to determine the amount of shielding. For this research the calibration quay wall model from section 4.3 is used as reference case.

The stiffness properties of the piles are first considered. The influence of a varying center to center distance between the piles is research as well as the influence of the different pile widths. The next step is to consider undrained soil behavior and the effect of water level differences. Another point of interest is the obliqueness of the combi-wall. The deep water quay wall is constructed with an oblique installed combi-wall. This may have an influence on the shielding effect as well. Finally the influence of the surcharge load is considered. The tables with calculation results can be looked up in annex F.4.

4.4.1 Influence center to center distances and diameters of the piles

From the previous section it can be concluded that the stiffness properties of the piles have an influence on the shielding effect. To quantify this effect different pile center to center (c-t-c-) distances and pile widths have been modeled in Plaxis. The pile widths are chosen to be 40, 45, 50 and 55 centimeters while the c-t-c distances between the piles are set from 1 to 5 meter with step sizes of one meter. The calibration model consist of piles with a width of 45 cm and a c-t-c pile distance of 3 meter. The data of this analysis is plotted in Figure 27 where the maximum field bending moments of the combi-wall are given for different pile diameters and c-t-c pile distances.

According to the figure the distribution of the bending moments in the combi-wall are close to each other for pile diameters 45, 50 and 55 centimeter. The results for the 40cm pile diameter is different because the deflections of the quay wall model where relatively large. This resulted in larger bending moments in the combi-wall. For all the pile diameter calculations, the shielding effect can be recognized. The larger the c-t-c distance between the piles, the larger the bending moments in the combi-wall. The closer the distance, the stiffer the pile rows behave and the lower the bending moments in the combi-wall.

The maximum horizontal field deflections of the combi-wall are plotted in Figure 28. The displacements for the 40 centimeter pile diameter are relatively large whilst the other three are more in line. Between the 45 and 50 centimeter pile diameters the influence of different c-t-c pile distances gradually increases for the maximum bending moments and the maximum horizontal field deflections of the combi-wall. For the 55 centimeter pile diameter initially the bending moments and deflections of the combi-wall are lower for smaller c-t-c distances whilst this is almost the same as for the other two for larger c-t-c pile distances. Higher pile stiffness's result in a relatively larger shielding effect on smaller c-t-c pile distances.

Comparing the 40 and 55 centimeter pile diameter lines it is clear that the higher the pile stiffness is, the more shielding occurs, because the bending moments in Figure 27 and deflections in Figure 28 of the combi-wall for different c-t-c pile distances are relatively smaller for the first one.

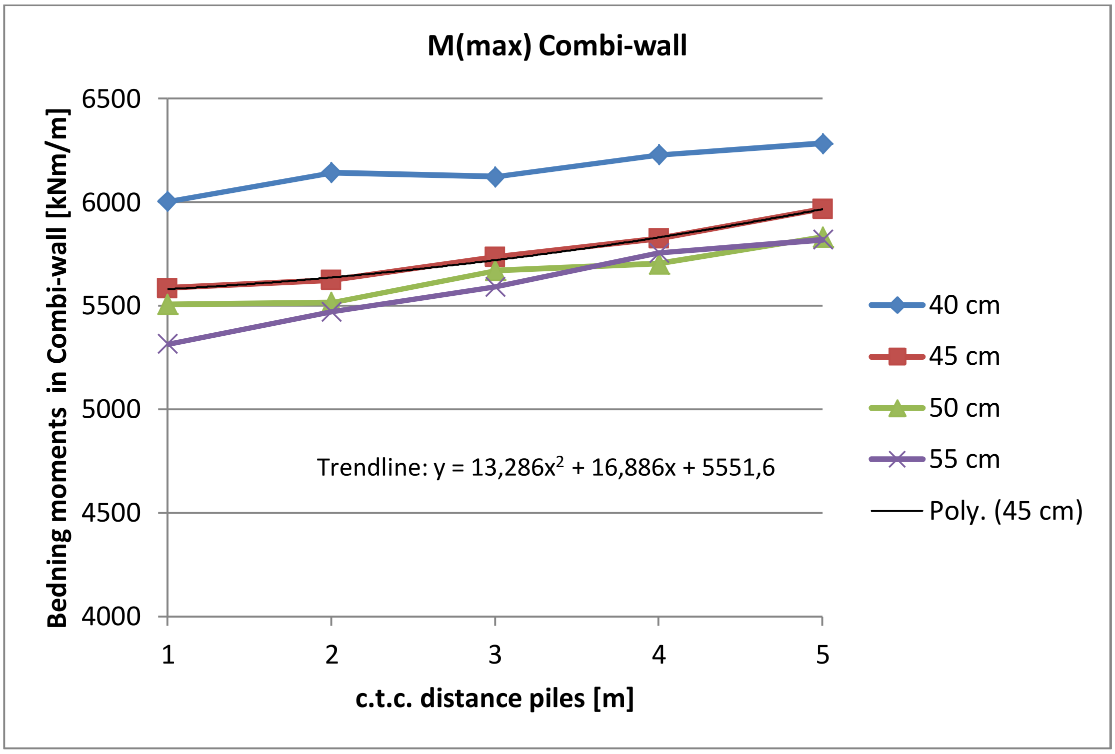


Figure 27 – Influences c-t-c distances and diameters of the piles on the maximum field bending moments of the combi-wall for the calibration model per meter length quay wall

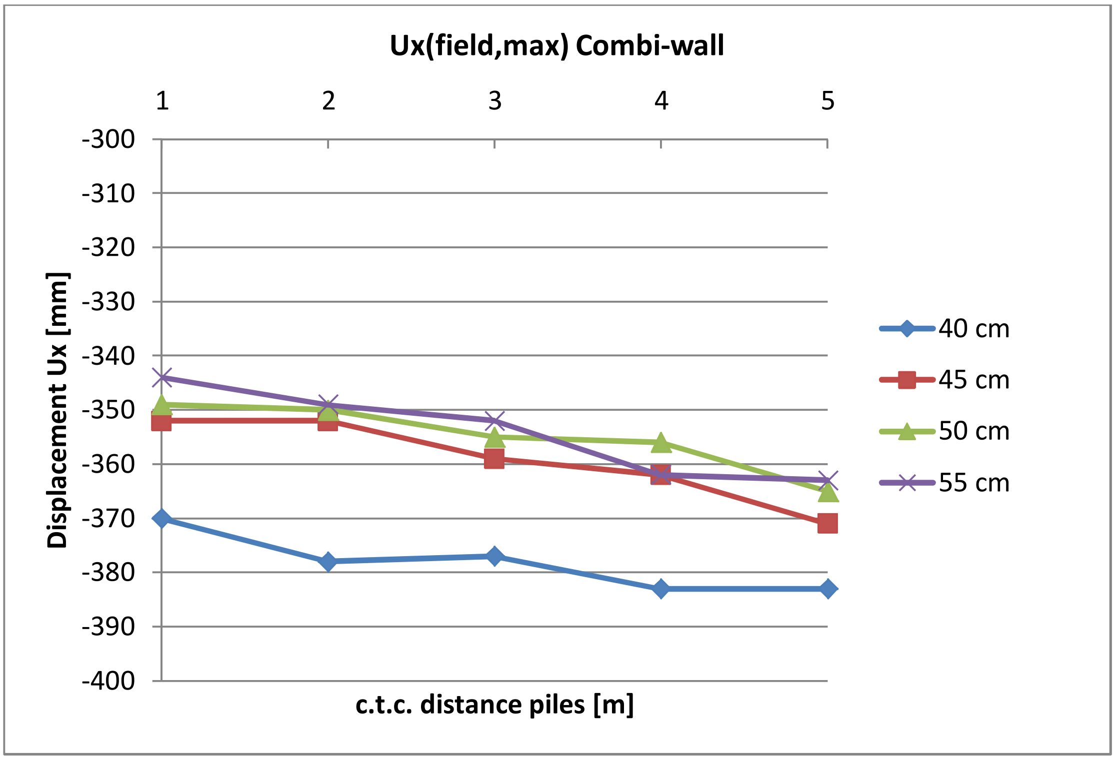


Figure 28 - Influences c-t-c distances and diameters of the piles on the maximum field deflections u_x of the combi-wall for the calibration model

4.4.2 Influence obliqueness combi-wall

A combi-wall which is obliquely installed in the subsurface results in lower bending moments in the combi-wall. This is illustrated in Figure 29 where the Plaxis calculation results are plotted. For this calculation the combi-wall is installed under an angle of 1:5 which is comparable with the deep water quay wall in the Amazonehaven. The figure illustrates the influence of different c-t-c pile distances and pile diameters on the field bending moments in the combi-wall.

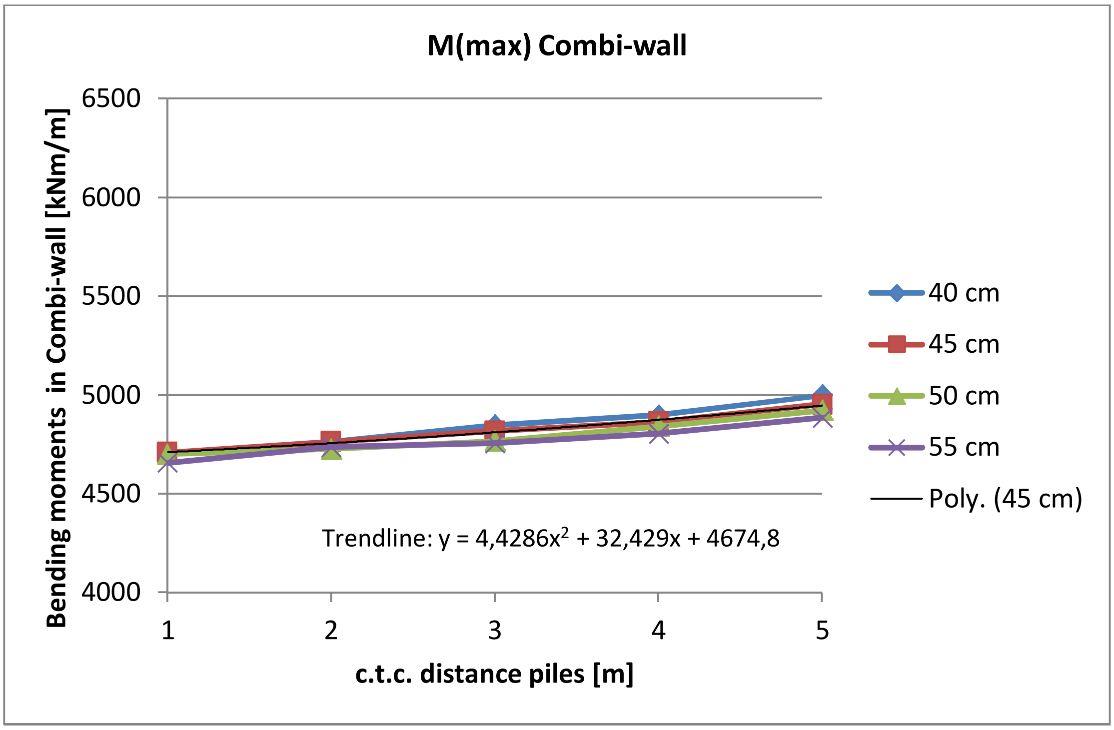


Figure 29 – Influences different c-t-c distances and diameters of the piles on the maximum field bending moments of an oblique installed combi-wall with angle 1:5 and per meter length quay wall

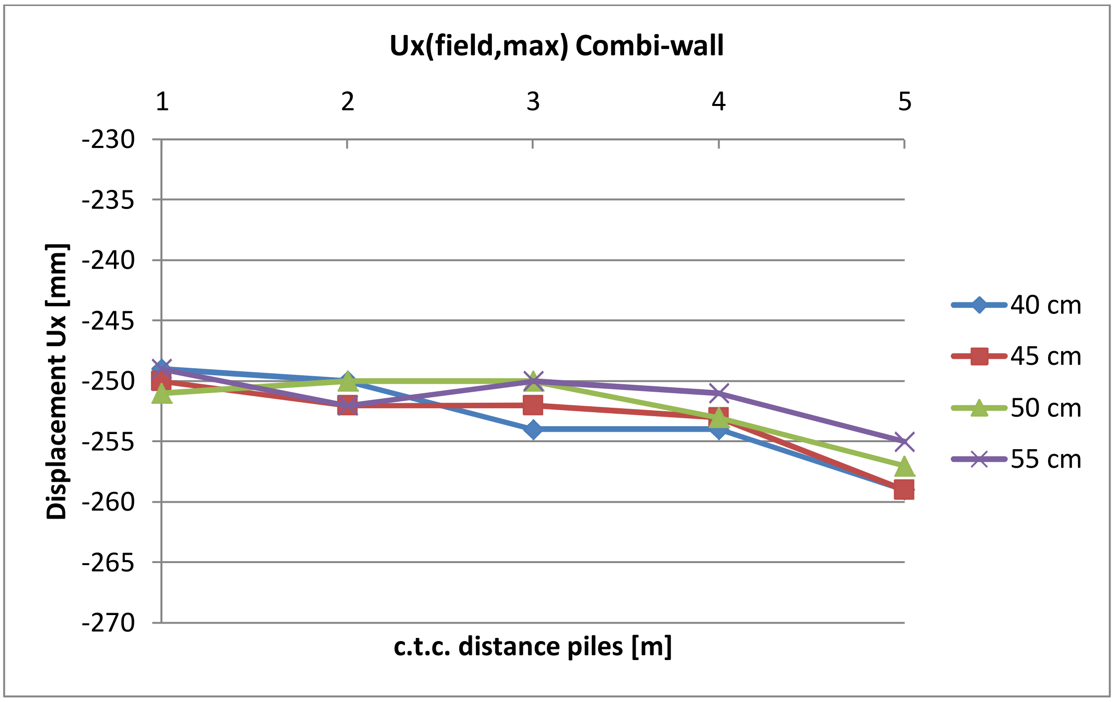


Figure 30 - Influences different c-t-c distances and diameters of the piles on the maximum field deflections u_x of an oblique installed combi-wall with angle 1:5

It can be seen that the maximum bending moments in the oblique installed combi-wall are lower than for the calibration model. The 40 centimeter pile diameter line is no longer much different than from the other three which implies that the deformations of the combi-wall are also more in line (see Figure 30). The maximum deformations occur for the weakest concrete piles.

4.4.3 Influence of undrained soil behavior and of water level differences

Undrained soil behavior has a large influence on the bending moments in the combi-wall. The undrained model is based on the calibration model but with only the clay layer modelled as undrained. The maximum field bending moments in the combi-wall are smaller than for the calibration model. However, if more layers are modelled as undrained, the field bending moments in the combi-wall are much larger. If this is the case the deformations are also larger and then the shielding effect will be higher. If only the clay layer is modelled as undrained, the results are comparable as for the calibration model (see Figure 31) .

For the calibration model a water level head difference of two meter has been modelled. If this difference is lowered to half a meter meter to discuss the influence of water level head differences, the field bending moments in the combi-wall increases, but this also has a small influence on the shielding effect. In Figure 31 this is illustrated for the low water model.

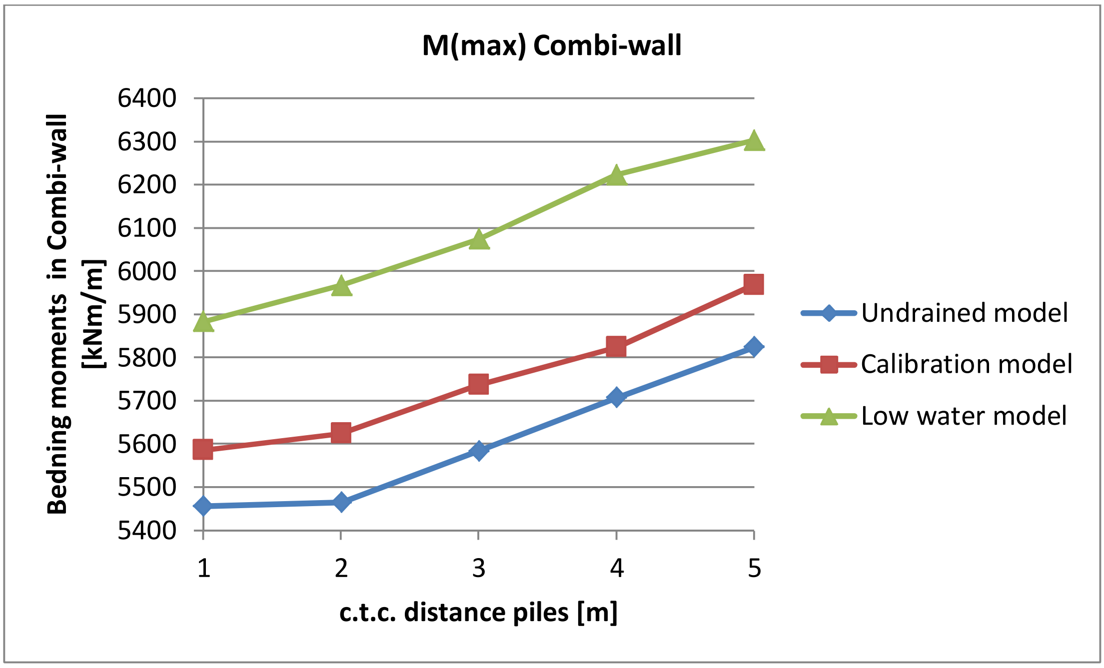


Figure 31 – Influences of undrained soil behavior and of water level head differences with different c-t-c distances of the piles on the maximum field bending moments in the combi-wall and per meter length quay wall

4.4.4 Influence surcharge load

It is expected that the surcharge load has an influence on the shielding effect of the concrete piles. This load causes extra deformations of the subsoil and therefore the loads on the piles are also likely to increase. The influence of the surcharge load is illustrated in Figure 32. The figure illustrates the effect of different c-t-c pile distances on the maximum field bending moments in the combi-wall for the situation without surcharge load, the situation with 50% of the surcharge load and for the calibration model with 100% surcharge load.

The surcharge load has a large influence on the field bending moments in the combi-wall. The influence on the shieling effect of the piles can also be noticed, all thou this difference is smaller. The difference between the c-t-c pile distances of 1 and 5 meter for the no surcharge model is about 4% whilst this is 7% for the calibration model. This implies that larger surcharge loads results in larger shielding effects.

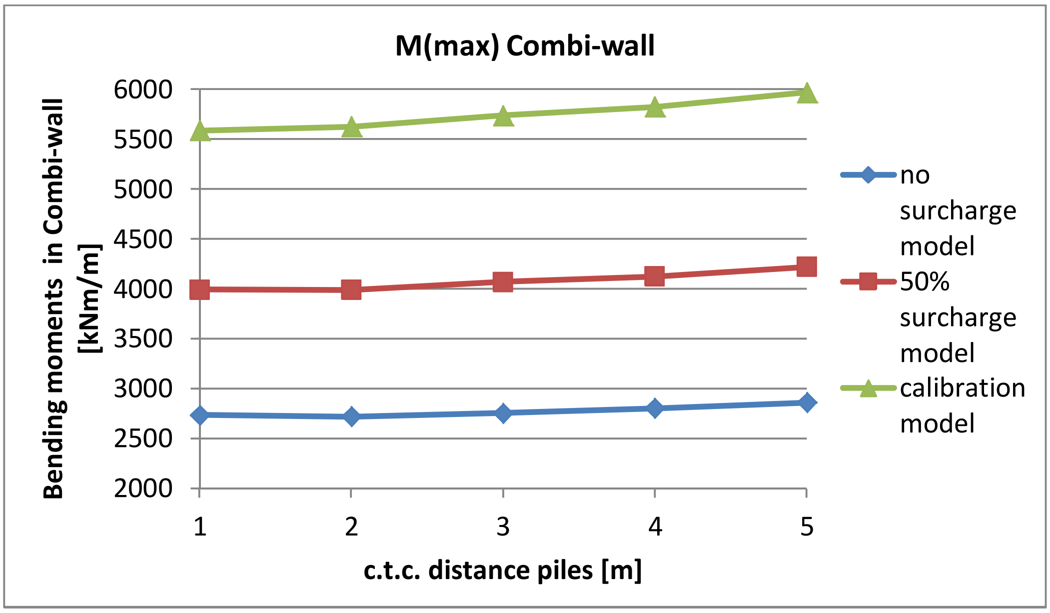


Figure 32 – Influences of surcharge loads with different c-t-c distances of the piles on the maximum field bending moments in the combi-wall per meter length quay wall

4.5 Conclusion finite element method analysis

The shielding effect has been examined with the finite element method and with the use of Plaxis. The effect has been studied for the concrete piles and the MV-pile anchors underneath the relieving structure for the deep water quay wall in the Amazonehaven. An calibration model, different fixed node to node support models and a small EI pile model have been set up to visualize the stresses in the subsoil and the maximum field bending moments in the combi-wall. Finally design aspects have been considered which may influence the shielding effect. The conclusions from the finite element method are summarized below:

- The effective horizontal soil stresses over the Holocene soil layers in front of the combi-wall are a maximum of around 20% lower due to the shielding effect of the concrete piles. The combination of concrete piles and MV-pile anchors result into a reduction of around 16%.
- The reductions of the field bending moments in the combi-wall have a 13% lower bound limit and a 20% upper bound limit due to the shielding effect of the concrete piles.
- The MV-pile anchors have a small to negligible influence on the shielding effect.
- The field bending moments in the combi-wall are sensitive to the displacements of combi-wall. Especially pipe-pile interactions have a large influence on the combi-wall's deflections and bending moments.
- The higher the pile stiffness's and/or the smaller the center to center pile distances the more shielding occurs.
- Undrained soil behavior and higher surcharge loads do have an influence on the amount of shielding from the concrete piles. The bending moments in the combi-wall are very depended on these two design aspects.
- Water level head differences have a small influence on the shielding effect.
- An oblique installed combi-wall lowers the field bending moments in the combi-wall.

5 Discussion shielding effect and design recommendations

5.1 Introduction

In the previous chapters the shielding effect of piles on quay walls have been analyzed based on two methods. Chapter three describes the effect by using an analytical method based on the theory of Begemann - De Leeuw and chapter four is based on a finite element analysis method (Plaxis). Both methods are based on different assumptions and limitations. The analytical method considers linearly elastic soil behavior and is only valid in the case of piles placed in infinite long soil profiles. However, this is not the case for the modelled quay wall. The distance between the front retaining wall and the pile wall at the back is finite and due to the high surcharge load the soil will behave plastically. Using this method for the quay wall model in chapter three is therefore not completely valid. The finite element method is based on much more parameters and the actual situation for the deep water quay wall can be modeled more accurately. However, this method is also based on model schematizations which have been explained in chapter four and therefore the results should be properly assessed.

First the results of the research methods are discussed and secondly the total building costs for the quay wall are explained. In addition, the high shear forces in the concrete piles near the clay layer will also shortly be discussed. The results from the calibration model illustrate that the concrete piles are being loaded with rather large shear forces as well. Especially near the boundary of the weaker Holocene layer and the stiffer Pleistocene layer. It is expected that the weak clay layer causes higher shear forces in the concrete piles.

5.2 Comparison results of research methods

The results for the analytical method are based on the horizontal soil stress reductions near the location where the combi-wall is installed and on the pile bending moments and pile deflections. For the finite element method more data is available to illustrate the shielding effect. The results are based on the structural forces and deflections of the combi-wall, MV-pile anchors and the concrete piles and also on the deformations and stresses of the subsoil.

Comparison effective horizontal soil stresses in front of the combi-wall

The results for the analytical method range from 10 to 11% and 18 to 20% depending on the kinematic schematization. A hinged connection between relieving structure and pile wall results in a reduction of 10% whilst a clamped connection is in the order of 18%.

The results for the finite element method are in the range of 16 to 20% depending on the quay wall model. The small EI pile model resulted into a reduction of 25%, but the deformations of this model where not entirely correct and therefore this value is considered to be too high. The results of the two methods regarding the effective horizontal soil stresses in the Holocene soil layers near the combi-wall agree well.

Comparison bending moments and deflections of the concrete piles

The pile deflections from the analytical method and the finite element method are slightly different. The maximum displacement of the piles for the analytical method are between 7 and 8 centimeter. If the displacement of the piles from the finite element method are normalized, thereby removing the pile tip displacements in the Pleistocene layer and near the relieving structure, the maximum field displacements are around 5 and 6 centimeter.

The bending moment differences are much larger. The results of the analytical method shows five times higher support and field bending moments in the concrete piles than the results of the finite element method. For instance, the support and field bending moments are -525 and 262 kNm/m for the analytical method whilst these are around -100 and 67 kNm/m for the finite element method. The FEM method has the advantage to model soil deformations plastically which result in smaller pile bending moments and smaller deflections.

Comparison bending moment reductions in combi-wall

With the FEM analysis bending moments differences in the combi-wall have been researched. It has been concluded that the field bending moments reduction in the combi-wall are 10 to 12% for the previous Plaxis calculations and around 13% for the Plaxis calculations based on the embedded piles option. This result can be adopted for future quay wall designs based on similar design parameters from the deep water quay wall in the Amazonehaven in the Port of Rotterdam. In addition, the use of finite element methods is very helpful to determine the shielding effects for different quay wall designs. This effect will automatically be included during the design and calculation phases.

5.3 Estimate construction costs quay walls with relieving structures

First an estimate for the total construction costs is discussed. The costs are based on unity prices for the construction materials and on installation costs for the foundation elements. After this estimation of the total construction costs, the shielding effect is introduced. Due to this effect the combi-wall can be designed smaller which reduces the total construction costs. However, note that the reductions due to the shielding effect are implicitly included in the quay wall designs. Therefore the cost reductions are also implicitly included. The comparison therefore can only be made between the 13% field bending moment reduction for the calibration model and for the same quay wall model without this reduction. Without the shielding effect, the field bending moments in the combi-wall would have been 13% higher than for the calibration model. This difference can be expressed into a cost reduction for the combi-wall.

Construction costs calibration quay wall model

From literature it is know that the costs for quay wall designs with relieving structures is about € 1800 per meter retaining height and per meter quay wall length (Gijt, J.G. de, 2010). This estimate is valid for relatively long and high quay walls design in the Port of Rotterdam. The longer the length of the quay wall, the more repetition occurs, the more learning takes place and eventually the smaller the installation costs per meter length quay wall will be.

The deep water quay wall is constructed in 1991 with a total length of 900 meter and a total earth retaining height of about 30 meter. The total construction costs per meter length quay wall is therefore estimated to be around € 1800 times 30 meter is € 54000.

The total construction costs can also be calculated based on the construction material costs and installation costs. The cost analysis for the construction materials is based on the following unity prices (2013) for construction steel and reinforced concrete:

▪ Construction steel for combi-wall and MV-pile anchors:	€ 1200/ton
▪ Reinforced concrete (100kg FeB500) for relieving structure and piles:	€ 400/m3

The installation costs for the foundation elements per meter quay wall length are based on the following unity prices (2013):

▪ Installation costs Combi-wall	€ 5000/m

▪ Installation costs MV-pile anchors	€ 5000/m
▪ Installation costs prefabricated concrete piles	€ 8000/m

The total construction costs per meter quay wall is elaborated in Table 5. The total costs are around € 53200 per meter quay wall. This estimate shows great similarity with the previous estimation of € 54000 per meter quay wall. The material costs for the combi-wall and the relieving structure are the most expensive parts.

Table 5 - Construction costs for calibration quay wall model (prices in € (2013) and per meter length quay wall)

Material costs steel	A [m2]	V [m3/m]	w [ton/m3]	Unit price €/ton	Unit price €/m3	Price/m
Combi-wall	0,043	1,00	7,85	€ 1.200		€ 9.422
Pipe wall	0,036	0,268	2,10	€ 1.200		€ 2.521
MV-pile anchors	0,020	0,262	2,06	€ 1.200		€ 2.468
Material costs concrete						
Relieving structure	48,8	48,8			€ 400	€ 19.510
Concrete piles	0,405	3,20			€ 400	€ 1.280
			Subtotal material costs:			**€ 35.201**
Installation costs						
Combi-wall						€ 5.000
MV-pile anchors						€ 5.000
Concrete piles						€ 8.000
			Subtotal installation costs:			**€ 18.000**
Total costs quay wall						**€ 53.201**

with:

A = area of construction element [m^2]
V = volume of construction element [m^3]
w = mass of construction element [ton/m^3]

Cost reductions due to shielding effect

Due to the shielding effect of the piles on the combi-wall a field bending moment reduction of 13% in the combi-wall can be realized. The combi-wall for the calibration model is constructed with a pipe diameter D of 1778 millimeter and a pipe thickness t of 20 millimeter. The bending moment capacity of these steel pipe piles is 22083 kNm (see Table 20 in annex F). The occurring bending moments in the combi-wall are 5757 kNm/m times the system length of three meter is 17271 kNm. The safety factor or unity check for the maximum bending moments in the combi-wall in relationship with the allowable bending moments is 22083 divided by 17271 is 1,28 for the calibration model. The chosen pipe pile diameter seems to be strong enough for this research, but also not too strong. It should be noted that the unity check is only an indication of the safety, because this research is based on a serviceability limit state (SLS) calculation.

The shielding effect of the concrete piles is implicitly included in quay wall designs. To illustrate the possible costs reductions for the combi-wall, a comparison with and without this effect should be made. Thus with and without the 13% field bending moment reductions for the combi-wall. This comparison is illustrated in Table 6 where eight different combi-walls with different diameters are compared. The configuration applied for the calibration model in chapter four is illustrated in bold.

Table 6 - Design optimization calculation for the combi-wall due to shielding effect (Arcelor Mittal, 2013)

Data and unity checks steel pipe piles									
Diameter pipe	[mm]	1575	1626	1676	1727	**1778**	1829	1880	1930
Diameter pipe	[inch]	62	64	66	68	**70**	72	74	76
Wall thickness	[mm]	20	20	20	20	**20**	20	20	20
D/t	[-]	79	81	84	86	**89**	91	94	97
Weight	[kg/m pipe]	767	792	817	842	**867**	892	917	942
Δ weight	[%]					**0,0**	2,9	5,8	8,7
$M_{el,Rd}$	[kNm]	17253	18410	19582	20841	**22083**	23390	24735	26090
Δ $M_{el,Rd}$	[%]					**0,0**	5,9	12,0	18,1
unity check M	[-]	1,00	1,07	1,13	1,21	**1,28**	1,35	1,43	1,51

with:
D/t = ratio diameter over thickness pipe (<90) [-]
Δ weight = difference weight of combi-wall with calibration model [%]
$M_{el,Rd}$ = bending moment capacity of combi-wall [kNm]
Δ $M_{el,Rd}$ = difference bending moment capacity with calibration model [%]
unity check = safety factor field bending moments in combi-wall [-]

In Table 6 the weight per meter pipe, the bending moment capacities and the unity checks are given. A pipe pile with diameter of 1880 millimeter has a 12% larger bending moment capacity which is almost the same as the 13% reduction. This implies that without the shielding effect this diameter would have been required. The increased weight of the steel pipes is then 5,8%. The prices for the steel pipe piles in the combi-wall are calculated according the weight of the pipes. This percentage can therefore be seen as the cost reduction for the combi-wall due to the shielding effect.

5.4 Additional check influence clay layer on shear forces in concrete piles

The presence of the weak clay layer between the Holocene layer and the Pleistocene layer results into higher local shear forces in the concrete piles. In Figure 22 the shear forces in the piles for the calibration model are illustrated. The maximum shear forces are respectively 50 and 32 kN/m length quay wall at the location of the clay layer. The total shear force in the piles however must be based on the system length of three meter. This results into a shear force of 150 and 96 kN in each pile for respectively the first and second pile row.

If the weak clay layer is substituted by a stronger soil layer, for instance the soil layer above the clay layer, the shear forces in the concrete piles at that location are much smaller. The shear forces will be 60 and 33 kN in each pile for respectively the first and second pile row. The presence of the weak clay layer considerable increases the shear forces in the concrete piles by a factor of about three. This is something designers should be aware of.

6 Conclusion

6.1 Introduction

This thesis research is conducted to investigate the shielding effect of piles on quay walls. For this research the deep water quay wall project Frans Swarttouw in the Amazonehaven in the Port of Rotterdam has been studied and used as a model to investigate the effect. For this research an analytical method, a finite element method and field measurements on an existing quay wall have been discussed.

6.2 Answering the sub-questions

The following sub questions have been formulated:

1. Can the group behavior of the bearing piles be considered as a second retaining wall due to arching principles and how large is the shell factor?
2. What is the difference between the available methods to model the shielding effect of piles and how reliable are the results?
3. In what way do the model parameters influence the shielding effect and is a comparison with other quay wall locations and dimensions possible?
4. Does an optimum exist between the dimensions of the front retaining wall and the dimensions of the bearing piles?

Answer sub-question 1:

The group behavior of the concrete piles can behave as a second earth retaining wall for smaller center to center distances.

According to the findings in the literature study, as a first approximation a shell factor of 5 can be considered. However, this factor in reality is not a constant value and varies over the length of the bearing piles. In this research, values between 1 (minimal) and 6,67 (maximum) have been investigated. From the analysis of the analytical method the difference between a shell factor of 1 and 3 is large, but the difference between a shell factor of 3 and 6,67 is small. Since a factor of 5 is reasonable to consider it can be concluded that the bearing piles basically retain the earth over the system length of three meter. The bearing piles thus act as a secondary earth retaining wall. For larger center to center (c-t-c) distances between the piles, shielding effect will still occur, but they will be smaller.

Answer sub-question 2:

The analytical method can be used for a preliminary outlook, but the finite element method with the embedded piles option is recommended for designing.

According to the literature study, two suitable methods have been chosen for this research. These are method Begemann - De Leeuw and method Plaxis. The first method is based on an analytical analysis and the second method on finite element method analysis. The analytical method is easy to use and takes a small amount of time to work with. It is recommended to apply this method only for a first approximations of the shielding effect, because the method is in principle only valid for linearly elastically soil behavior and infinite long soil profiles. This method can also only be applied in the case of a clear distinction between a weaker soil layer above a stiffer soil layer below.

The results from Plaxis are more accurate than from the method Begemann - De Leeuw. The quay wall models from Plaxis included plastically soil behavior, multiple soil layers and oblique installed concrete bearing piles. The bending moments in the combi-wall could also be determined with Plaxis. These aspects cannot be modelled with the analytical method.

In the previous Plaxis calculations included in annex G, the concrete piles could only be modeled with continuous plate elements. This schematization is valid in the case that a row of piles can be modelled as a continuous earth retaining wall which is the case for c-t-c distance equal to and below three meter and with piles having a diameter of at least 45 centimeter. Therefore it is recommended to make use of the embedded piles option of Plaxis where the c-t-c pile distances are included in the model parameters.

Answer sub-question 3:

The stiffness properties of the concrete piles together with the amount of surcharge load and the possibilities of undrained soil behavior have the highest influence on the amount of shielding. The pipe-pile interactions between the combi-wall and the concrete piles should be minimized if possible.

From the analytical method it can be concluded that the stiffness properties of the bearing piles have a large influence on the shielding effect. The difference between one row of piles and two rows is large. Also the connection with the relieving structure has a large influence on the shielding effect. A clamped connection results in a stiffer pile behavior and a larger shielding effect.

From the FEM analysis it can be concluded that the effective horizontal soil stresses over the Holocene soil layers in front of the combi-wall are a maximum of around 20% lower due to the shielding effect of the concrete piles. The combination of concrete piles and MV-pile anchors result into a reduction of around 16%. The MV-pile anchors have a small to negligible influence on the shielding effect. Furthermore does a higher pile stiffness and/or smaller center to center pile distance result into more shielding.

The field bending moments in the combi-wall are sensitive to the displacements of the combi-wall. Especially pipe-pile interactions have a large influence on the deflections and bending moments in the combi-wall.

Undrained soil behavior and higher surcharge loads do have an influence on the amount of shielding from the concrete piles. Water level head differences have a small influence on the shielding effect.

Answer sub-question 4:

The concrete piles should be designed to support the relieving structure and not for maximizing the shielding effect.

The possibility to optimize the quay wall design by applying more concrete piles and thereby smaller combi-walls is only briefly researched. This seems to be unfavorable because the additional costs for stronger concrete piles and/or smaller c-t-c pile distances are more than the reductions which can be achieved for the combi-wall. The bearing piles must only be designed to support the relieving structure. The resulting shielding effect is an additional benefit.

6.3 Answering the main research question

The main research question is:

“What is the shielding effect of the piles and anchors on the combi-wall for the deep water quay wall project Frans Swarttouw in the Port of Rotterdam and which design aspects influence this effect?”

Answer main research question:
Maximum field bending moment reductions in the combi-wall of 13% seems feasible for the deep water quay wall project Frans Swarttouw in the Port of Rotterdam due to the shielding effect of the concrete piles. This can results in 5,8% material costs reductions for the combi-wall. The MV-pile anchors only have a small to negligible influence on the shielding effect.

7 Recommendations

The following recommendations have been made in response to the research carried out as described in this report:

Field bending moment reductions up to 13% can be applied for combi-walls due to the shielding effect of concrete bearing piles.
It is recommended to consider the shielding effects of bearing piles on quay walls. This effect is depended on the local geological soil data and the dimensions of the quay wall. The field bending moments in the combi-wall are about 13% smaller when the shielding effect is considered. This result is based on the deep water quay wall project from 1991 in the Amazonehaven in the Port of Rotterdam. For other locations the results may be different. It is advised to apply finite element method analysis to determine the actual shielding effect reductions.

Soil displacement effects of piles cannot be modelled in Plaxis.
The concrete piles which have been used for the deep water quay wall in the Amazonehaven have been rammed into the subsoil. During this process the surrounding soil is pushed aside which creates stronger interface strengths between the soil and the piles. In Plaxis the piles are modelled without this preloading effect of the soil. The interface behavior between the actual piles and the pile models are therefore not similar which is important to consider during designing. The negative skin friction and the horizontal soil stresses may be different due to the preloading of the surrounding soil.

Study of the shell factor.
The difference of the shielding effect between higher shell factors are smaller than between lower shell factors. For analytical methods, it is therefore recommended to choose a shell factor of around 5 for a row of piles in densely packed sand layers with possible accumulations of clay. In clay layers lower shell factors must be applied in the order of 1 to 2. The shell factor is difficult to determine and therefore it is recommended to further investigate this factor, especially for weaker soil layers.

Use the analytical method only for preliminary design.
The analytical method Begemann - De Leeuw is useful for preliminary designs. The results for the shielding effect regarding the effective horizontal soil stresses in front of the combi-wall agree to a certain extend with the finite element method. The pile wall displacement profile and the pile wall bending moments are not very accurate. These cannot be used for preliminary designs. The use of FEM to determine the displacements and bending moments in the concrete piles is preferred.

Design quay walls with relieving structure with finite element method.
For the design of quay walls with relieving structures the use of finite element methods is preferred. The benefits of working with Plaxis are manifold. Not only can the shielding effect of piles be researched, the design of all structural elements can be optimized in Plaxis. Other methods have either strong simplifications, for example the method Begemann - De Leeuw, or they are not suitable to model the relieving structure correctly for instance with Dsheet. The ability to make detailed calculations with Plaxis outweighs the extra time required to be able to work with the program.

Large influence of weak clay layer on the shear forces in the concrete piles

Designers should be aware of local high shear forces in the concrete piles in the weak clay layer near the boundary between the Holocene and Pleistocene layer. Due to the presence of the weak clay layer, the shear forces can be about three times larger than in the case of the presence of a stronger soil layer. The piles should be strong enough to resist these local high shear forces.

More field measurement data should be gathered.

Due to the limitations of the applied models and methods it is recommended to further investigate the shielding effects on quay walls with field investigations and measurements. The experience learned for instance for the extension work of the EKOM quay wall are helpful to understand the behavior of quay walls and its foundation elements.

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Appendix