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Master of Science Thesis

Design of Quay Walls using the Finite Element Method

The importance of relieving structures in quay walls

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Design of Quay Walls using the Finite Element Method

A parametric study in the port of Rotterdam to assess the importance of relieving structures in quay walls.

For obtaining the degree of Master of Science in Civil Engineering at Delft University of Technology

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“Ama y haz lo que quieras. Si callas, callarás con amor. Si gritas, gritáras con amor. Si corriges, corregirás con amor. Si perdonas, perdonaras con amor”

San Agustin

Voor mijn vader…“Waar een wil is, is een weg”

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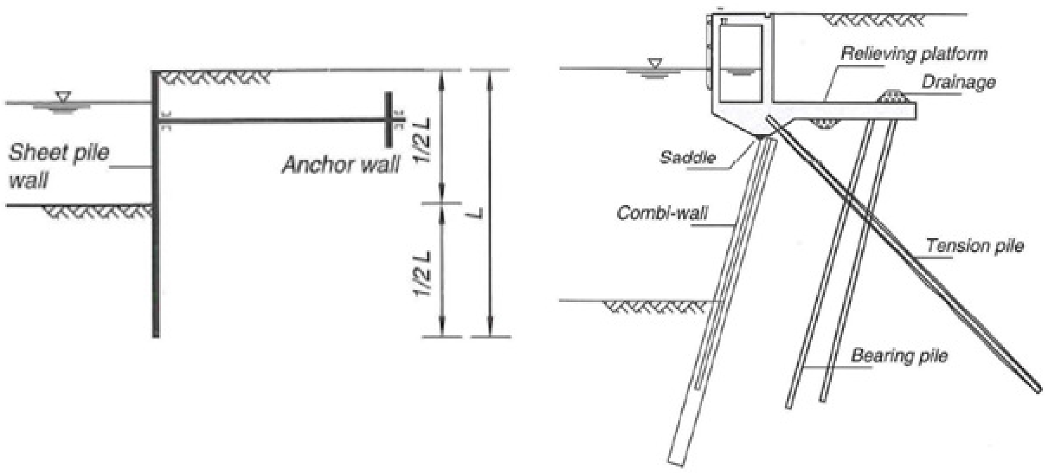
SUMMARY

From ancient times, transport and trade of goods has been a major activity over the world. The mankind has developed ways to transport goods on land, air and water. The biggest share of all intercontinental cargo goes by sea, via seaports, to make this possible several types of quay wall structures where developed. This MSc thesis focuses on two types of quay walls, namely the anchored quay wall without relieving structure and the quay wall with relieving structure, as shown in the next figure.



Anchored quay wall (left) and quay wall with relieving structure (right)

In the case of high retaining heights the anchored quay wall mostly consists of a combined sheet pile wall, which means that the wall is composed by tubular steel members that are connected by means of sheet piles. This combination leads to high bending stiffness that gives the possibility to withstand high loads due to the high retaining height. When the retaining height is combined with high surface loads a relieving structure is applied.

A relieving structure consists of a concrete structure (a concrete slab or L shaped structure), a combined sheet pile wall, tension anchor (horizontal anchors, tension piles or ground anchors) and a bearing piles system. The function of the relieving structure is to reduce the pressure on the retaining wall caused by the surface load and soil on the relieving structure.

A relieving structure leads to a reduction in the internal forces in the elements of the quay wall, but is more labour intensive. This has impact in the costs of the quay wall. The main objective of this research is to assess the importance of the relieving structure from an engineering and economical point of view. The engineering aspects that are compared are the displacement of the wall, moment of the wall and the anchor forces, the economical aspect are the direct costs (delivery of elements and installation of those). This objective is accomplished by means of a parametric analysis of several designs with and without relieving structure, by varying the anchor level, anchor type and the dimensions of the relieving structure.

To extend the investigation, two calculation methods were compared. The beam on elastic foundation method, which models the soil as discrete springs and estimates the internal forces in the wall purely by the horizontal pressure acting on it. This is done with calculation software based on this method the so called D-sheet package. The disadvantage of this method is that it is not possible to include the relieving structure explicitly in the model. This means that the relieving structure needs to be modelled and calculated separately as a framework.

The other method that was evaluated is the Finite Element method which calculates the internal forces in the quay wall by calculating the displacements and stresses of the surrounding soil and elements. The software used is the so called PLAXIS package, which is based in the Finite Element Method. This method gives the possibility to work in a wider field of modelling and calculations. Also it is possible to include the relieving structure explicitly in the model and more effects as the vertical displacements and the displacement of the anchor are taken into account.

The results of the calculation showed that from an engineering point of view, the relieving structure is more favourable. The displacements of the retaining wall when a relieving structure is present are smaller than the ones calculated when the relieving structure is absent (up to 70%). Regarding the moments, a deep lying relieving structure reduces de moment in the retaining wall in some cases up to 75%, which result in smaller primary elements (piles of the combined wall) that are much easier to install without the need of special equipment. The same is concluded about the anchor force. Due to the pressure reduction, the anchor forces are in the range of 30% to 40% lower than when the relieving structure is absent. This leads to smaller cross-section of the anchors, shorter anchors or smaller centre to centre distance between the anchors. Also in the case of the failure of one anchor a relieving structure gives a better redistribution of forces due to the high stiffness of the slab of the relieving structure.

The cost estimation performed in this thesis lead to the conclusion that a quay wall with relieving structure is more expensive that a quay wall without it. The main contribution to the costs of the analysed quay walls come from the steel combined wall. The price of the steel primary members (steel tubes) depends on the way they are fabricated. When the tubes exceed certain limit in dimensions (1.8 m diameter and 25 mm thickness) the fabrication costs increase with a factor 1.5 to 2. In this thesis, the calculated elements of all variants exceed this limit, which lead to high costs of the fabrication of steel tubes for both, the quay wall with and without relieving structure.

During decision making the parties involved would have to conider whether the reduction of the costs by using a quay wall without relieving structure overrun the aforemetioned engineering aspects. An important aspect is that when the relieving structure is absent the dimensions of the primary elements of the combined wall increase and are considered extraordinary in onshore projects. The companies that can install such elements are limited and even offshore equipment would be necessary, which could increase the total costs of the project.

Regarding the calculation methods some remarkable differences where found, specially in the calculation of the anchor forces. The moments in the retaining structure for both cases with and without relieving structure calculated with D-sheet vary sligtly from the ones calculated with PLAXIS, a maximum difference of 20% is found.

When the relieving structure is absent the anchor forces calculated with both methods differ in a range of 20%. However, when a relieving structure is present D-sheet estimates forces that are 30% to 35% lower than PLAXIS when the relieving structure has a length of 10 meters. When the relieving structure length increases to 20 meters, D-sheet calculates anchor forces that are 20% to 60% lower than PLAXIS. Literature review showed that the method to estimate horizontal pressures due to strip loads used by D-sheet underestimates the resultant force acting on the wall, ehich leads to an under estimation of anchor forces.

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First of all, I would like to express my deep gratitude to Dr. ir. J. De Gijt and Dr. ir. K.J. Bakker, my research supervisors, for their patience, guidance, encouragement and useful critiques during this research work. I would also like to thank the other members of the committee for their critical advice and useful recommendations that made this research possible.

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During this period I established a family in The Netherlands. It is not possible to name them all, but special thanks to the Bleumers, Lopez uit Nieuwerkerk, Maroniers and “de Poltjes”. Thanks to all my friends in the world, especially Arjan, Aliya and Eri. Each one of you contributed to a piece of what I am now.

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A	Cross-sectional area	[mm²]
A_s	Area reinforcement steel in cross-section	[mm²]
c	Cohesion/concrete cover	[kN/m²]/[mm]
e	Eccentricity	[mm]
E	Young´s modulus	[kN/m²]
E_{kin}	Berthing energy of the ship	[J]
ε_{sm}	Mean steel strain	[-]
ε_{cm}	Mean concrete strain	[-]
f_{ck}	Characteristic concrete compressive strength	[N/mm²]
f_{cd}	Design concrete compressive strength	[N/mm²]
f_u	Steel ultimate strength	[MPa]
f_y	Steel yield strength	[MPa]
f_{ctm}	Average concrete tensile strength	[MPa]
h_{eff}	Effective concrete height	[mm]
γ	Weight density	[kN/m³]
$\gamma_{\varphi'}$	Partial factor angle of internal friction	[-]
$\gamma_{c'}$	Partial factor cohesion	[-]
γ_{cu}	Partial factor undrained shear strength	[-]
γ_{γ}	Partial factor weight density	[-]
$\gamma_{f;g}$	Load factor for permanent loads	[-]
$\gamma_{f;q}$	Load factor for variable loads	[-]
I	Moment of inertia	[m⁴]
k_h	Modulus of subgrade reaction	[kN/m³]
K_a	Active earth pressure coefficient	[-]
K_p	Passive earth pressure coefficient	[-]
K_0	Neutral earth pressure coefficient	[-]
M_{Sd}	Design moment	[kNm/m]
M_{Rd}	Moment of resistance	[kNm/m]
N_{Sd}	Design normal force	[kN/m]
N_{Rd}	Normal force capacity	[kN/m]
O	Perimeter of the contact surface of soil and grout	[m]
q_c	Cone resistance	[MPa]
$Q_{f;0}$	Pile shaft friction	[kN]
Q_{eb}	Ultimate end bearing capacity	[kN]

ρ_{eff}	Effective reinforcement percentage	[-]
R_a	Anchor holding capacity	[kN/m]
$S_{r;\max}$	Maximum crack width spacing	[mm]
σ_v	Vertical soil pressure	[kN/m²]
σ_S	Steel stress	[MPa]
δ	Angle of wall friction	[°]
φ	Angle of internal friction	[°]
Ψ	Combination factor for load combinations	[-]
τ	Shear stress of the soil	[kN/m²]
w_k	Crack width	[mm]
z	Depth/internal level arm	[m]/[mm]

1. INTRODUCTION

The port of Rotterdam is one of the main ports of Europe. This port is the gateway to the European market for more than 350 million consumers and is the most important hub of good flows over the world. The port of Rotterdam started its functioning around the year 1400 and it continues its expansion nowadays (Figure 1.1).

Figure 1.1 Development of the port of Rotterdam [1]

During the lifetime of the Port of Rotterdam, several types of quay walls were constructed (see section 2.1). The most popular type is the sheet pile wall with and without relieving structure (Figure 1.2).

The sheet pile wall without relieving structure (Figure 1.2, left) consists of a retaining structure made of (combined) sheet piling and an anchor (to limit the sheet pile embedding depth). High bending moments arise in the sheet pile wall when high retaining heights need to be established. Even higher internal forces act on the wall when this is in combination with high surfaces loads. When the quay wall is subjected to high surface loads, a relieving structure is constructed (Figure 1.2, right).

A relieving structure consists of a concrete structure (a concrete slab or L shaped structure), a combined sheet pile wall, tension anchor (horizontal anchors, tension piles or ground anchors) and a bearing piles system. The function of the relieving structure is to reduce the pressure on the retaining wall caused by the surface load and soil on the relieving structure.

Depending on the retaining height and functional requirements a decision has to be made whether to apply a relieving structure in the quay wall or not. Factors that guide to this decision are: the reduction of the displacement due to a stiffer structure and the saving of costs in sheet pile length and thickness.

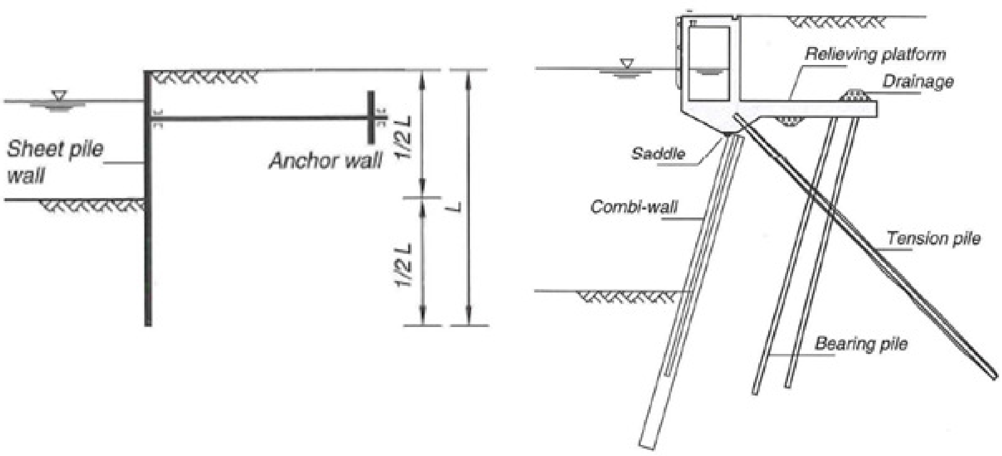

Figure 1.2 Anchored sheet pile wall (left) and sheet pile wall with relieving structure (right) [2]

In the Port of Rotterdam, quay walls with a relieving platform have been chosen many times when high retaining heights need to be reached. However, few investigation has been made of whether this structure is more efficient (structurally and economically) compared to the traditional sheet pile wall. When a relieving structure is not applied, bigger cross-sections need to be used, which can lead to problems during construction and may even not be possible to realize.

Several design tools to determine the internal forces in the retaining wall and anchor are available. In chapter 3 a description of the three most used methods to calculate the internal forces in retaining structures is given. These methods are:

- BLUM method
- Beam of elastic foundation
- Finite Element Method (FEM)

The BLUM method can be used to have a rough estimate of the internal forces. With knowledge of simple statics this method can be solved analytically. The beam on elastic foundation method and the finite element method are more sophisticated methods that are solved with the aid of computer software. The results of these methods are more accurate and are commonly used in the design of retaining structures. In the Public Works of Rotterdam, D-sheet (software based on the beam on elastic foundation method) and PLAXIS (FEM method) are used to design quay walls, these two programs will also be used in this master thesis.

1.1 Problem description

In the Port of Rotterdam quay walls without relieving platform are discarded when high retaining heights (in combination with high surface loads) need to be reached. It is known that without applying a relieving structure big and long cross-sections need to be constructed. These cross-sections lead to the need of using heavy machinery and finally to high construction costs.

The Public Works of Rotterdam is interested in investigating to what extent a quay wall design with relieving structure is more convenient than one without a relieving structure, from the engineering and economical point of view.

To be able to assess the problem from an engineering point of view the following criteria have to be evaluated: Anchor forces, displacements and bending moments in the wall. It is known that the anchor stiffness has influence in the force distribution in a retaining structure. To extend this investigation two anchor types are considered.

The design tool is of great importance in the design of a structure. This determines in what degree forces are estimated and the design is optimized. As mentioned before, D-sheet and PLAXIS will be used in this thesis. The results of these two methods will be compared and a recommendation for the design using these methods will be given.

1.2 Research questions

As mentioned before, in this master thesis the importance of the relieving platform is investigated. This is translated in a main research question:

“Is a quay wall with a relieving structure more favourable than a traditional quay wall (sheet pile wall) with anchoring, from an engineering and economical point of view?”

A set of sub questions are made to extend this investigation, these are:

"What is the influence of the anchor type and level in the design of a quay wall?"

"What are the differences in the results using the beam on elastic foundation method and the finite element method? How does this affects in the design of quay walls?"

"What is the influence of surcharge modelling with the finite element method?"

1.3 Main objective

The main objective of this thesis work is to perform a comparison as to in what extent a quay wall design with reliving structure is more effective than one without relieving structure. The comparison will be made from an engineering and economical point of view.

It is expected that based on the results of this study a conclusion can be made whether the design of quay walls with relieving structure is more favourable or not. Also a recommendation regarding the use of a beam on elastic foundation method (D-sheet) or the Finite Element Method (PLAXIS) will be given.

1.4 Layout of this report

The layout of the report is as follows. After the introductory chapter some theoretical background in the subject is given (Chapter 2). Here the types of quay walls and anchors are explained along with the design guidelines, verification of strength and stability. Subsequently Chapter 3 gives an explanation of the existing calculation methods that are available to design quay walls.

In Chapter 4 the starting points are discussed, e.g. site conditions, geometry, materials, loads, design philosophy and cases considered in the investigation. In Chapter 5 the hand calculation methods along with their exactitude are evaluated. In Chapter 6 the quay wall without relieving structure is presented and in Chapter 7 the quay wall with relieving structure, in both chapters first information about the calculation method is given, followed by the results obtained in the calculation with the Beam on Elastic Foundation method and the Finite Element method.

Finally, in Chapter 8 the results of the cost assessment for a quay wall with and without relieving structure are presented and in Chapter 9 the conclusions and recommendations of this investigation are given.

2. THEORETICAL FRAMEWORK

2.1 Types of quay walls

In the history of ports, several types of quay walls were developed. The main goal of this development was to ensure handling of cargo as quick as possible, satisfying current and possible future developments, like:

- Demand arising from local conditions;
- Demands of future users;
- Nautical demands;
- Anticipated developments in navigation, dimensioning of ships, transhipment and freight storage.

Some aspects that have to be taken into account to satisfy those demands are:

- Assessment of water levels, tidal conditions and soil properties;
- Enough draught for present and future vessels to berth;
- Enough storage area for the handling of freight.

In addition to providing berth facilities for ships, a quay wall must satisfy the following functions:

- Retaining function: The structure must retain soil and water. The retaining height follows from aspects like ship draught, possible erosion and minimum anticipated water level;
- Bearing function: The structure must be able to bear the loads generated by cranes (own weight and movement), storage and transhipment of freight;
- Mooring function: Enough mooring facilities (bollards) are needed to ensure that cargo is loaded and unloaded efficiently;
- Protecting function: The quay wall can be also seen as a barrier against high water.

To fulfil these functions several types of quay walls are developed, the main categories are explained in this section.

2.1.1 Gravity walls

The soil retaining function is derived from the self-weight of the wall, where due to the high weight resistance to shearing is developed in the soil. Gravity structures are used when the soil is not suitable for a sheet pile wall (rock or very firm sand) and when the soil has enough bearing capacity. Important points to take into account are:

- Gravity walls often consist of prefabricated elements, this allow continuous construction and repetitions. This can be attractive for the construction of long quays when the one-off costs of the building of a construction dock, formwork, transport and placing of the elements can be written off a large number of elements;
- A good foundation is needed to ensure stability of the wall;
- Silt layers have to be avoided underneath the wall because they can act as a slip plane;
- Drains need to be applied to prevent built up of excess pore pressure;
- The toe needs to be protected against bed erosion to ensure stability.

The choice of the type of gravity wall depends on the local structure of the subsoil and on the relation between the costs of materials and labour. There are several types of gravity wall (Figure 2.1):

Block wall: Most simple type of gravity wall, which consists of blocks of concrete or natural stone piled on top of each other. The blocks are founded on gravel or crushed stone. The superstructure consists of reinforced concrete and is cast on site. Little labour is required but much building material is used, because of the weight of the wall good bearing subsoil is required. Block walls have long horizontal and vertical joints, which give good drainage so that water overpressure behind the wall is limited. Because of this, a good filter behind the wall is needed to prevent leakage of soil.

L-wall: These types of structures owe their stability to the concrete structure and the soil that rests on them. The concrete floor assures the development of shear resistance with the soil. This structure is used when the soil is not suitable for a block wall or material wants to be saved.

Caisson wall: A caisson wall made of large hollow cellular concrete elements. This is mostly built in a dry dock and floated to position. Once in position it is sunk and the hollow spaces are filled with ill material to have enough weight to resist horizontal pressures. Weak soil needs to be replaced with a good bearing soil to assure stability of the wall. The top of the caisson lies usually above water, which give dry conditions to finish the superstructure. Caissons are economical in material use, but labour intensive.

Cellular wall: Cellular walls are constructed by driving straight web profiles to form cylindrical or partially cylindrical cells that are kinked to each other. Usually they rest on the bottom of the harbour or extend only little below this level. The cells are constructed in land or water and filled with sand or other material, the structure consists of soil enclosed by steel rings where only tensile stresses occur. Relative little material is needed for this structure. A disadvantage is that it is thin and vulnerable for collision and corrosion.

Reinforced earth wall: This structure usually consists of vertical revetment panels made of concrete, which are connected to horizontal tension elements, e.g. steel strips, rods or polymers (geo-grid). The horizontal force is transferred by friction between the contact surfaces of the horizontal elements and soil. During the backfill process the facing panels and the strips are placed in layers until the desired height is reached. In this type of structure the sealing of the joints is critical.

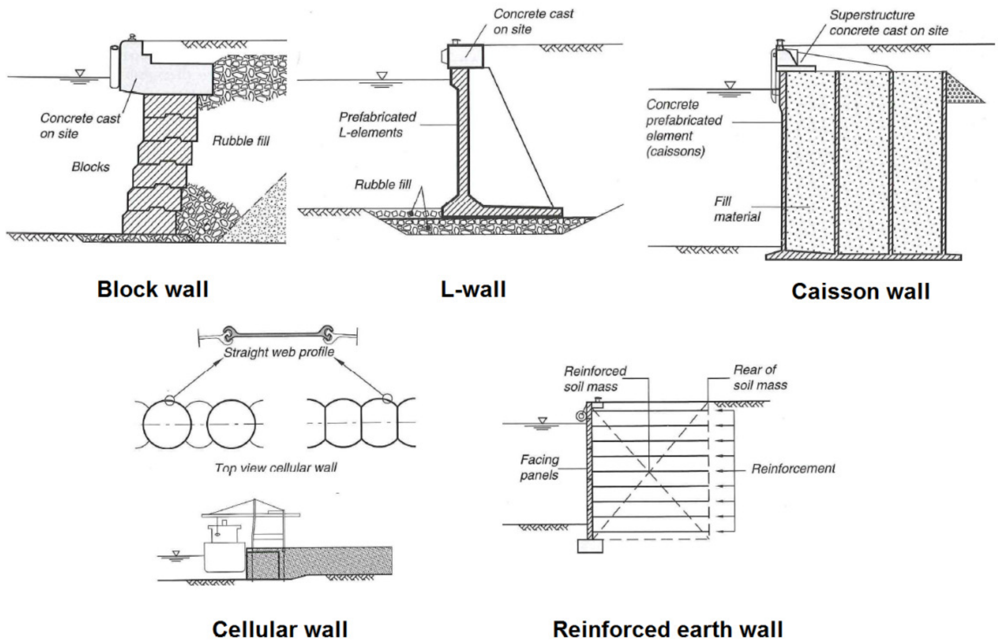

Figure 2.1 Principles of several types of gravity wall [2]

2.1.2 Sheet pile walls

Sheet pile wall structures derive their soil retaining function and stability from the fixation capacity of the soil, sometimes in combination with anchors (Figure 2.2). This type of wall is used in soil with poor bearing capacity that is easy penetrable. The sheet piling is connected to each other by means of an interlock. In quay walls, the elements are connected at the top by means of a superstructure that has also berthing and mooring facilities for ships (bollards and fenders). The bottom in front of the wall needs to be protected against erosion, also drainage behind the wall needs to be placed, this to prevent excessive pore pressure (due to rain water) behind the wall. There are two types of sheet pile wall, each one is described briefly:

Free standing sheet pile wall: If a sheet pile wall is not anchored, it acts like a cantilever beam to transfer soil pressures to the subsoil. The supporting pressure that is necessary to gain equilibrium is leant by the passive earth pressure at the toe. This kind of sheet piling is used for low depths.

Anchored sheet pile walls: Anchors are used when one wants to reach a high depth. The anchor takes the horizontal forces and the sheet piling behaves like a beam on two supports, a hinge on the top and a partially or fully fixed support at the bottom (the types of anchors are explained in section 2.2).

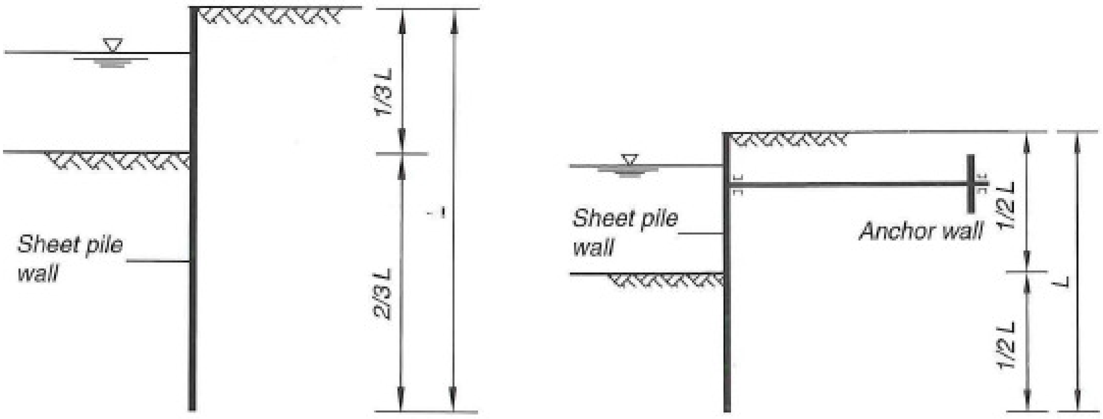

Figure 2.2 Principles of the free and anchored sheet pile wall [2]

2.1.3 Sheet pile walls with relieving structure

A method to reduce the horizontal loads on the retaining structure is the relieving structure. A relieving structure is basically a concrete structure founded on piles that transmits the loads that act on the quay surface directly into the subsoil (Figure 2.3). It consists of a concrete structure (relieve floor), retaining structure (for example sheet pile wall), bearing piles to transmit the vertical loads into the subsoil and tension piles (or anchors) to increase resistance against horizontal pressure. The relieving platform can be installed at various heights.

- ***High relieving platform:*** Here, the horizontal load of the soil is carried by a pile trestle system with tension and bearing piles under de superstructure. The relieving platform usually lies above low water level, this way it can be constructed in a tidal area over a slope in low tide. Depending on the distance between crane tracks, these can be included on the relieving floor, or be provided with a separate foundation;
- ***Deep relieving platform:*** Structures with a relieving platform are mostly used to achieve high retaining heights. The platform is supported by a combined sheet pile wall at waterside and bearing piles at the port side. A tension pile or an anchor is needed to take horizontal loads. The joint between the combined sheet pile wall and relieve floor is made

by a cast iron saddle. This allows the joint to behave as a hinge, giving the possibility to place the combined walls with an angle.

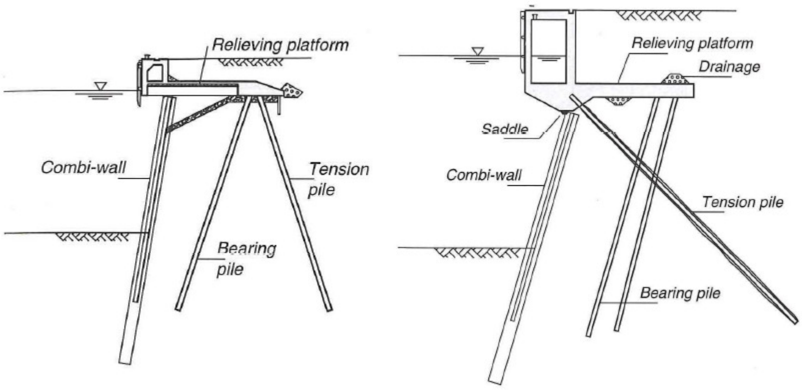

Figure 2.3 Sheet pile wall with high and deep relieving structure [2]

2.1.4 Open berth quay

In this type of quay, the difference in heights is not bridged by a vertical wall, but by a slope (Figure 2.4). It consists of a horizontal deck parallel to the shore and anchored if necessary. The deck is founded on vertical and inclined piles. The slope under the deck must be protected against currents, erosion due to bow thrusts, main propellers and waves. This structure is mainly used when:

- Construction takes place above water;
- There is sufficient space in the water side;
- There is a relative poor subsoil;
- There are existing protected slopes.

These structures are vulnerable for collisions and maintenance under the deck is difficult.

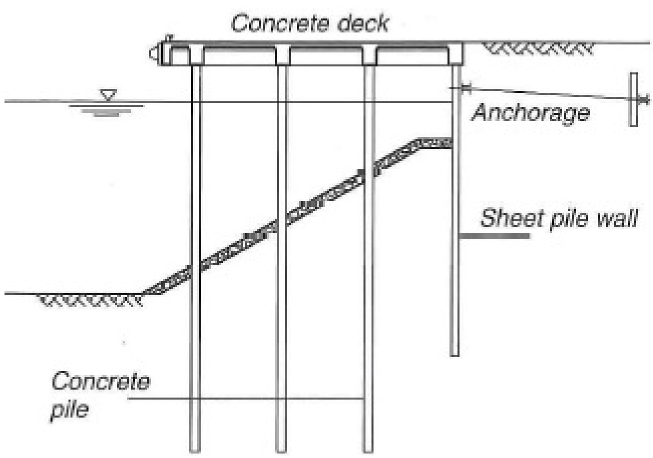

Figure 2.4 Principle of an open berth quay with retaining wall [2]

2.2 Anchors

Anchors are used to increase the resistance of the quay wall against the horizontal pressure. It is known that material used in the wall can be saved by applying an anchor in a sheet pile wall. Several types of anchors were developed in time. Three main categories can be distinguished:

2.2.1 Horizontal anchorage

Conventional horizontal anchors (Figure 2.5) consist of a deadman anchorage that is connected to the sheet piling by a tie rod with an anchor head. The decision of which horizontal anchor will be used depends on the soil conditions. In soft soil sheet piling as anchor head can be used, in rock conditions rock bolts and in soft rocks pile trestles. Also horizontal screw anchors exist, this type of anchor does not have a big anchor wall, the restraint is provided by the threaded rod that is inserted into the soil horizontally. Also two types of tie rod are recognized: bars and cables.

- ***Bar anchor:*** Traditional anchor with a bar and anchor block. The required passive soil pressure is supplied by a vertical anchor wall or by a pile trestle;
- ***Cable anchor:*** The tension member is usually a high quality steel cable. Usually connected to the sheet piling or relieving platform by means of pre-stressed unbonded tendons;
- ***Screw anchor:*** This type of anchor consists of a threaded steel rod welded onto an anchor bar. The anchors are screwed into the desired position with an auger drill.

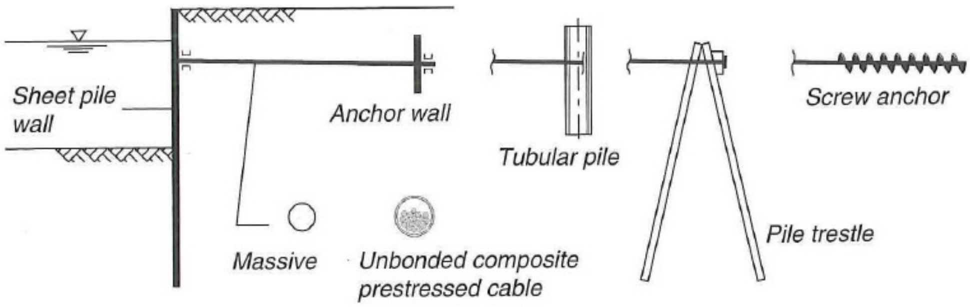

Figure 2.5 Horizontal anchoring [2]

2.2.2 Anchorage with a grout body

The most important characteristic of this type of anchor are the in situ cement grout elements. Handbook quay walls [2] distinguish two types of anchorage with grout body:

- ***Grout anchors:*** These are pre-stressed anchorage consisting of an anchor head and a tendon, part of which is bonded to the ground by grout that has been injected under pressure. The bars are provided with a trapezium screw thread along the whole anchoring length. Grout anchors must be pre-stressed to prevent large deformations. The steel is protected against corrosion and the tensile capacity is derived from the friction between soil and grout body;
- ***Screw injection anchors:*** These anchors are made with a hollow stem auger with a perforated tube that is a few meters long. During drilling, a grout mixture is forced through the perforations in the tube and injected into the soil, thus forming a layer with high strength capabilities.

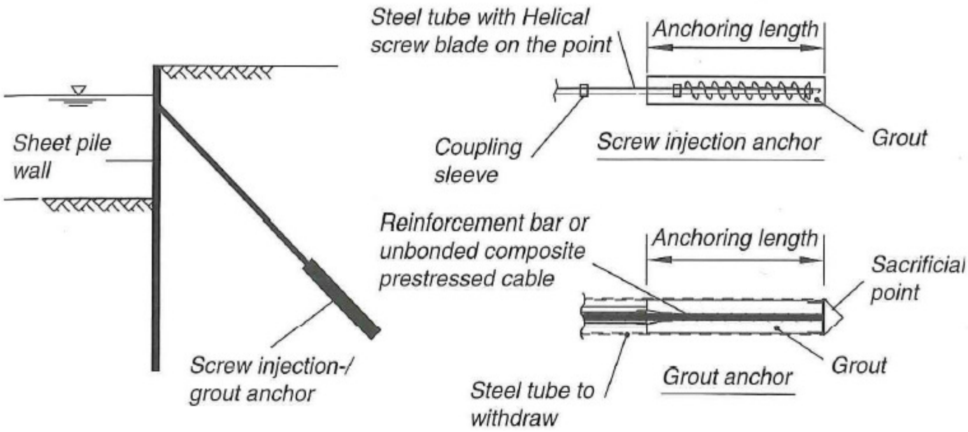

Figure 2.6 Anchors with a grout body [2]

2.2.3 Tension piles

Sheet piling can be anchored with tension piles that may form part of a pile trestle. Some types of tension piles are (Figure 2.7): closed piles, open steel tubular piles, steel H piles and MV piles, in all these types the tensile force is supplied by shaft friction.

- ***Closed or soil displacement pile:*** These are concrete piles and driven, drilled or screwed tubular steel piles with a closed foot;
- ***H*-piles:** The tensile force is supplied by friction between soil and steel pile;
- ***MV-pile:*** This type of tension pile is an H-pile with a grout body around it. The grout assures adhesion of the pile in the soil.

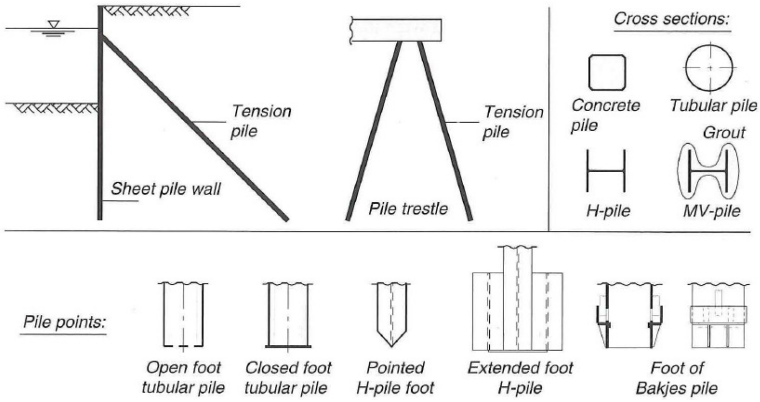

Figure 2.7 Tension piles as anchor [2]

2.2.4 Placement of anchors

Figure 2.8 shows the proper location of an anchor. In plate anchors, the resistance of the anchors derives primarily from the passive soil located in front of them. When the active wedge of the retaining wall and the passive wedge of the anchor cross each other, anchor capacity is lost. The designer should place the anchor at enough distance from the wall, if this is not the case, reduction of the anchor capacity needs to be taken into account. In case of tension anchors (anchors that lean their capacity to the soil friction) the part of the anchor situated in the active wedge of the wall does not contribute to the holding capacity of the anchor.

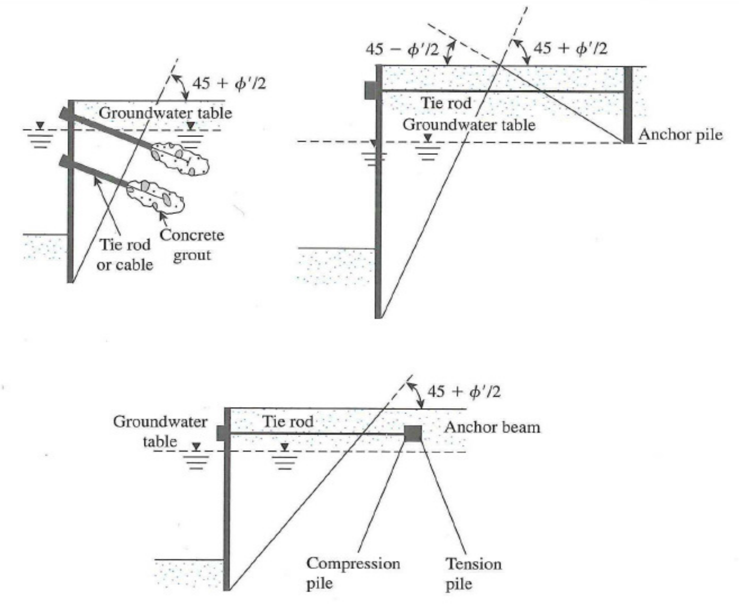

Figure 2.8 Placement of anchors [4]

2.3 Design guidelines

Several design guidelines for retaining structures were developed in the past and each one of them has its advantages and disadvantages. In the Netherlands the most popular guidelines to design quay walls are the CUR166, CUR211, EAU (Empfehlungen des Arbeitsausschusses Ufereinfassungen) and the Eurocode. These guidelines use a fundamental semi-probabilistic safety approach based on partial safety factors for the load case scenarios.

The EAU started as a 'practically proven' design code [5], which means it was based on historical experiences. Later it developed a more fundamental semi-probabilistic safety approach to be able to be compared with the Eurocode.

The CUR166 is the most used guideline in The Netherlands and it is meant specifically for sheet pile structures. Here, representative values of the different parameters can be used based on (for example) the average cone resistance of cone penetration tests. To follow the steps given in this guideline, a safety class for the design has to be chosen (each safety class has its set of partial factors).

The Eurocode also uses a semi-probabilistic approach. Here, the same load combinations are used as in the other mentioned guidelines. In this code, three predefined approaches are given. Each approach gives a set of partial factors which depends on the safety class and type of structure.

The CUR211 is a publication meant for the design of quay walls with relieving structures. In this guideline not only a design approach is given, but also a complete overview of the history, design and execution methods. The last chapters of this guideline include experiences of the past aimed at making the designer aware of the complications and limitations of quay wall design. The last version of this guideline will be published this year (2013) and it will handle the same design approach as given in the CUR 166, also the safety factors and design philosophy are according the NEN-EN 9997 [17]. The NEN-EN 9997 is a compilation of several parts of the Eurocode that applies to geotechnical constructions, namely retaining structures.

2.4 ULS and SLS

When designing a structure difference has to be made in the Ultimate Limit state (ULS) and Serviceability Limit State (SLS).

The ULS is characterized by the use of safety factors. These factors are meant for reducing the strength of the materials, increasing the loads and changing the soil conditions to reach an unfavourable situation (high internal forces). The ULS concentrates in the determination of the strength of the elements.

The SLS does not imply safety factors. The SLS (as the name says) is meant to evaluate the structure at its 'use' period using representative values for the material properties, loads and soil parameters. Here the deformation of the structure is evaluated along with the overall stability of the structure and stability of single elements.

In quay wall design the strength is not always decisive. Depending on the function of the quay wall, strong limits for the deformation are given. In case of anchored sheet piles stability of the anchor can be decisive.

2.5 Verification of stability

The verification of the stability consists of the verification of the possible unfavourable and deep sliding planes (Bishop method), the stability of the anchorage (Kranz method), the verification of heave of the stabilizing soil layer on the passive side of the sheet pile, a phenomenon called piping and local buckling (in case of using steel members in the retaining structure wall).

2.5.1 Bishop method

The method of Bishop is based on a circular sliding plane (Figure 2.9). The condition to have stability is that the destabilizing moment (M_{ad}) has to be smaller or equal than the resisting moment (M_{rd}). Guidelines advise to apply a minimum ratio between the destabilizing and resisting moment (FS>1.3 [2], which is the overall safety factor regarding stability of the quay). For the calculation of these moments, the most unfavourable sliding plane is divided in slices and the summation of moments around the centre of the circular sliding plane caused by the resultant yields to the destabilizing or resisting moment. To find the most critical sliding plane, several circles must be analysed. Nowadays computer software is available to make these types of calculations.

2.5.1 Kranz method

The Kranz method (Figure 2.10) is used to verify the stability of the anchorage, this based on a deep straight sliding plane. If tension piles are used, the sliding plane goes from the shear centre of the sheet piling to the centre of the anchorage area of the tension piles. If an anchorage with anchor wall is used, the slide plane goes from the shear centre of the wall to the toe of the anchor wall. The groundmass in the sliding plane is loaded with the design value of the tensile force of anchorage or pile and must be in equilibrium.

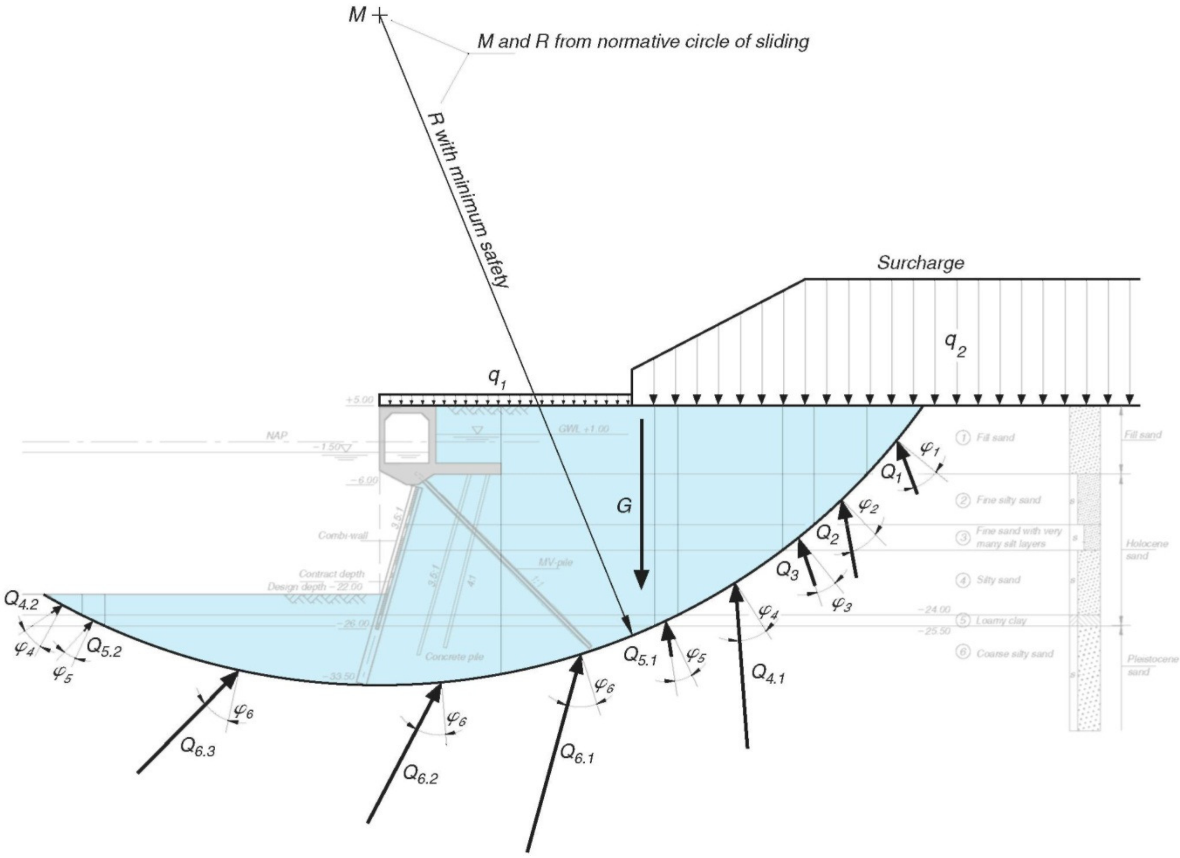

Figure 2.9 Verification of total stability of a quay wall according to Bishop [2]

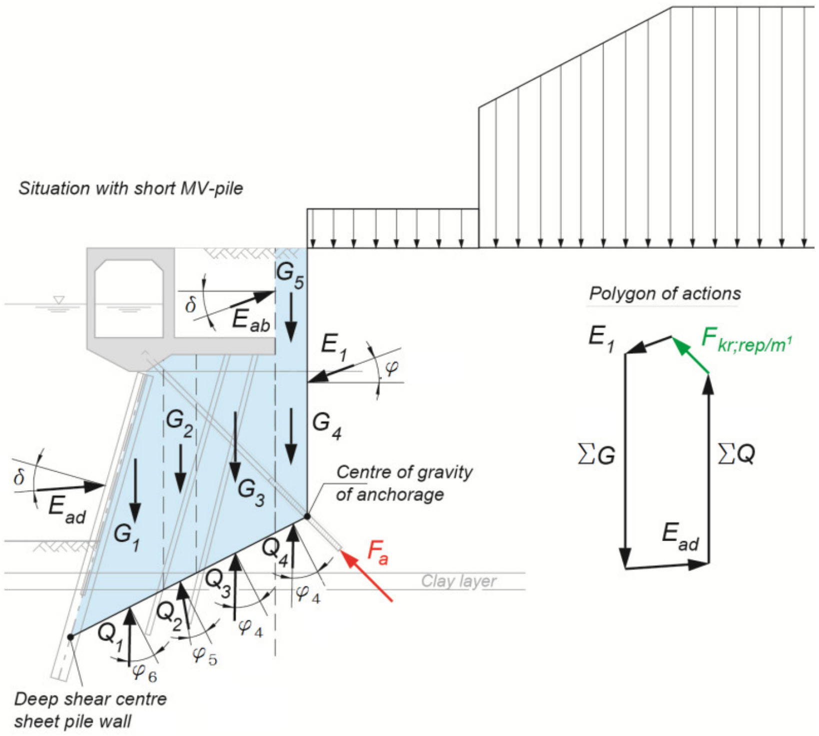

Figure 2.10 Kranz method to determine the anchor stability [2]

2.5.2 Piping and hydraulic failure

Piping (soil boiling) refers to the phenomenon when water flow around the wall exceeds the critical flow velocity. At that point the soil starts to be washed away, internal scour occurs and a "pipe" in the ground appears, this can lead to total failure of the structure. The "pipe" starts on the bed of the harbour and extends via de underside of the sheet piles to create a link with the open water and the high side of the quay. This phenomenon is illustrated in Figure 2.11.

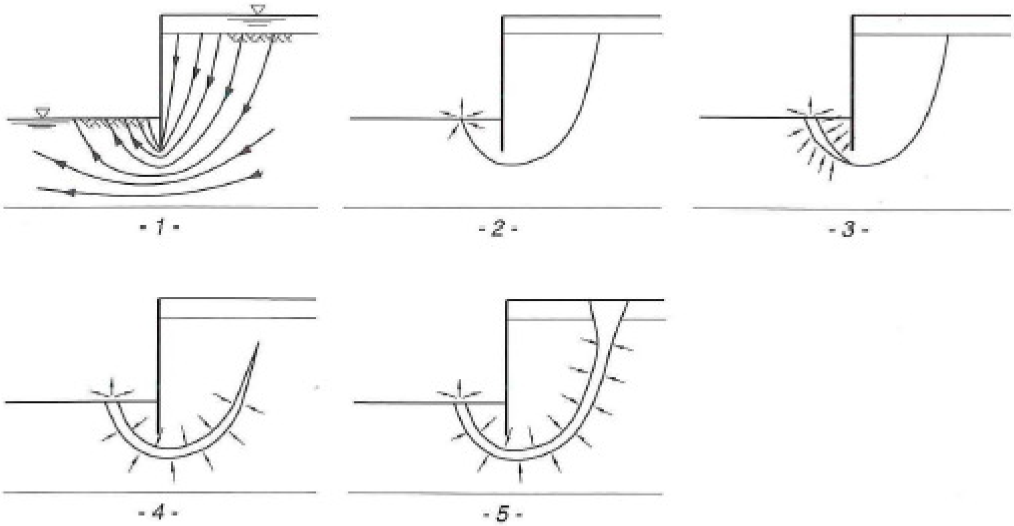

Figure 2.11 Development of piping [2]

Hydraulic failure (heave) occurs when water pressure at the height of the toe exceeds the effective weight of the soil in front of the wall. Also, in tidal areas there is the risk that the semi-permeable ground layers heave due to the water pressure underneath them. Heave greatly reduces the stabilizing capacity of the soil in front of the wall. Verification should be carried out during construction stage and serviceability stage.

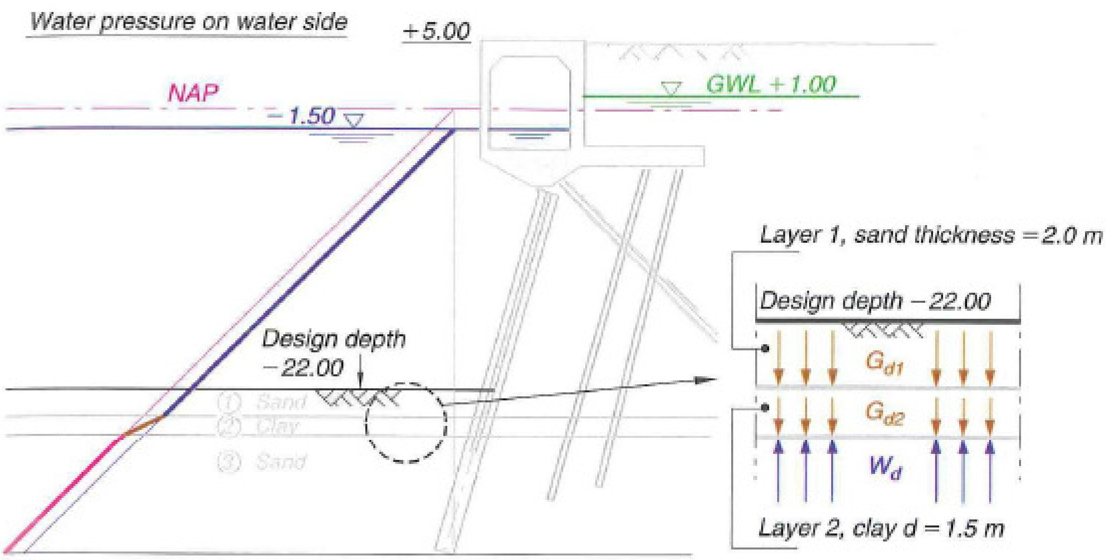

Figure 2.12 Heave phenomenon in quay walls [2]

2.5.3 Local buckling

The tubular primary members of combined wall are often chosen for their flexural strength, stiffness and axial bearing capacity. During the design process of quay walls, because of the increasing retaining heights and loads, these elements tend to increase in diameter while the thicknesses stay small in proportion to the diameter. When the stresses in these elements are too high, local buckling can occur, this can lead to problems during serviceability and even failure of the wall. This phenomenon needs to be taken into account during the design of such elements in combination with ovalisation of the elements.

Ovalisation of the primary elements increase the surface where local buckling can occur, leading to more susceptibility to this phenomenon. The ovalisation of the primary members is a combination of out of roundness during fabrication of the elements, tension forces provided by the secondary members and soil pressure against the primary members. This subject has been studied by Gresnigt and some recommendations to check this phenomenon are given in the Handbook Quay walls [18].

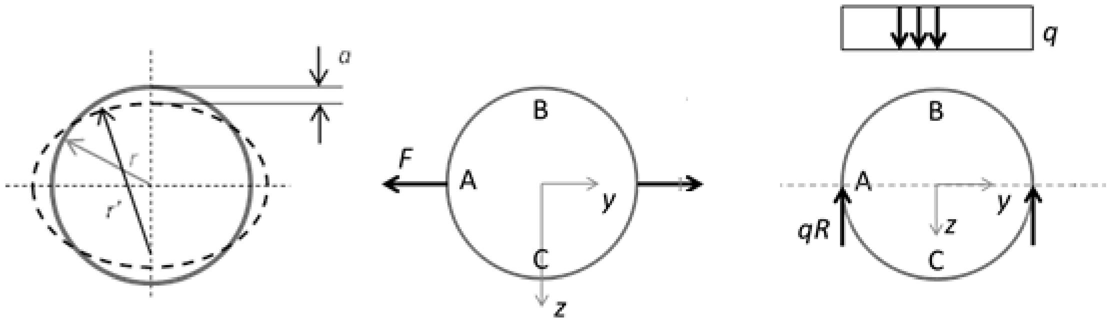

Figure 2.13 Ovalisation due to out of roundness, tension forces from secondary members and soil pressure [18]

2.6 Principle of the relieving structure

A relieving platform reduces the active earth pressure on the retaining wall, which leads to a reduction of bending moments, hence a reduction of the profile and sheet pile lengths is possible.

When the relieving platform is at surface level (high relieving structure), the reduction is limited to the surcharge consisting of crane, storage and traffic loads. When a deep relieving structure is considered, the reduction is much greater because of the soil on top of it.

Figure 2.14 shows the effect in earth pressure reduction on the sheet pile. The area of influence starts where the line at the angle of internal friction of the soil (φ) intersects the axis of the sheet pile (point a). The reduction stops where the line composed by the active sliding planes intersects the axis of the sheet pile (point b) [6].

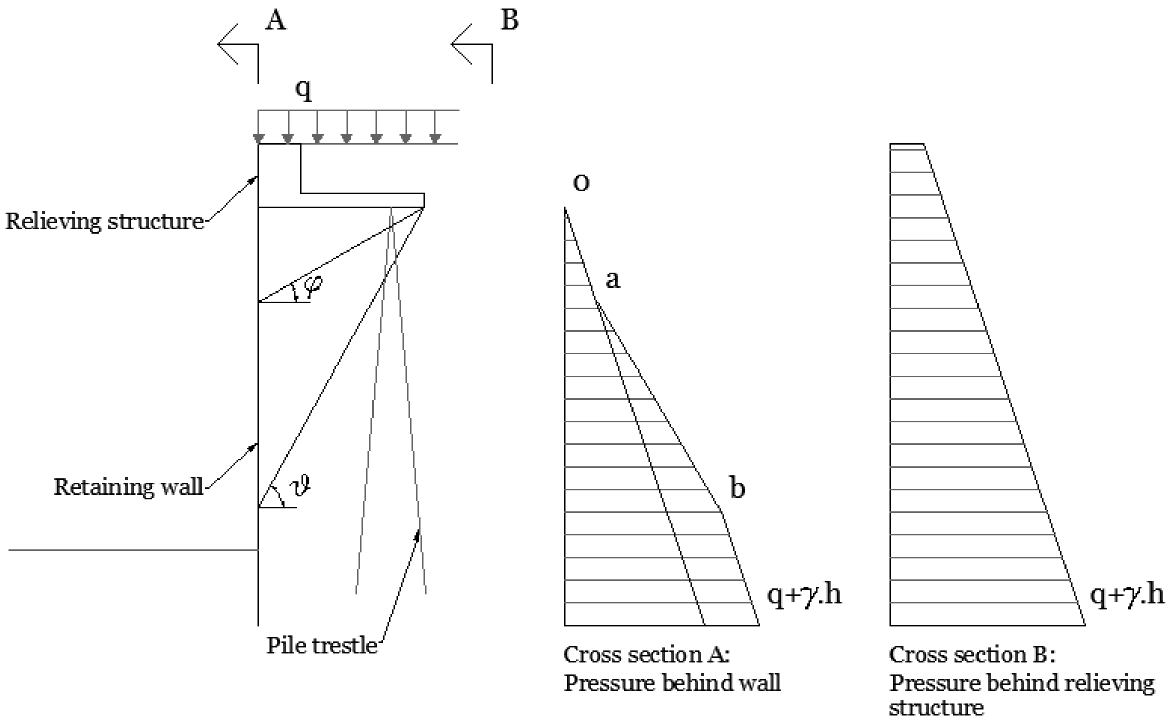

Figure 2.14 Pressure reduction due to the relieving structure

3. Calculation methods

Before numerical integration was facilitated by computers, sheet pile structures where calculated by simple statics, the most known method of hand calculation of sheet piles is the method of BLUM, described in section 3.1. Nowadays, computer software is used for the calculation of a sheet pile wall, among others D-sheet (beam on elastic foundation, section 3.2) and PLAXIS (finite element method, section 3.3).

3.1 Hand calculation methods (analytical calculation method)

Two “classical” methods developed by BLUM can be distinguished:

- Sheet pile walls with a free earth support;
- Sheet pile walls with a fixed earth support.

The principle of both methods is explained in this section, for a detail of the mathematical model see Annex B.

One of the characteristics of the BLUM theory is that the earth pressure coefficients are constant and does not depend on the relative displacement of the soil (Figure 3.3).

3.1.1 Sheet pile with a free earth support

The first method, free earth support, consists of finding the minimum depth of penetration of the sheet pile wall to assure its stability. The sheet pile is assumed to be free supported at the toe, this means that rotation is allowed. This assumption leads to a parabolic distribution of the moment that resembles with the distribution of a simply supported beam.

The disadvantage of this method is that the theoretical depth has to be increased with about 30% to 40%, to ensure stability. Another method used to avoid increasing depth is to apply a safety factor to the passive earth pressure coefficient ($K_{p(design)}=K_p$/safety factor).

The pressure distribution of an anchored sheet-pile wall penetrating a granular soil is schematized in Figure 3.1.

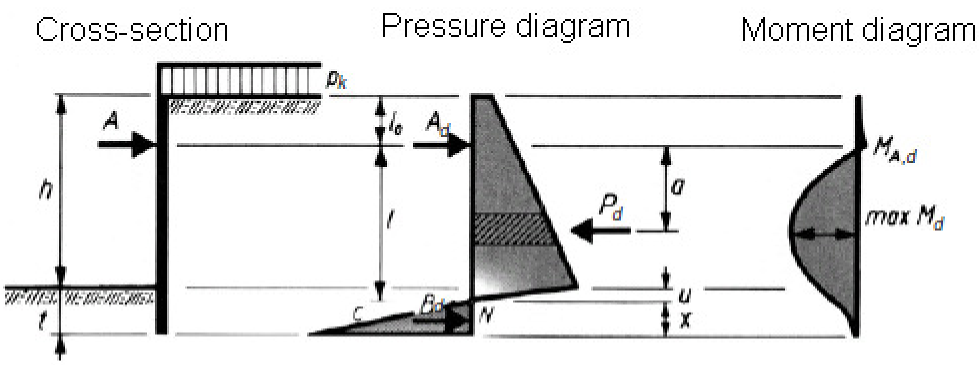

Figure 3.1 Blum method assuming a free earth support [7]

The embedding depth, resultant forces and moments can be calculated with basic knowledge of statics by assuming that the moment at the top and toe is equal to zero. It is important that good soil properties are used and the soil pressures are calculated properly.

3.1.2 Fixed end support

Blum developed a calculation method that is rather conservative, giving an extra sheet pile length as a result. The extra sheet pile length is however compensated with a better redistribution of moments, leading to a lighter sheet pile profile. Blum assumes that the lower part of the sheet pile is fixed in the soil, which means that a fixed end moment will take place and at a certain point the sheet pile does not allows rotation. The schematization according to Blum is shown in Figure 3.2. Blum introduces the concept of an extra concentrated force to account fixity of the sheet pile in the soil. To generate this extra force, the embedment depth needs to be increased with 10 – 20 %.

To calculate the embedding depth, anchor and internal forces, the displacement at anchor level is assumed to be zero. With the expressions of displacement assuming a fixed cantilever beam, the anchor force and internal forces can be calculated.

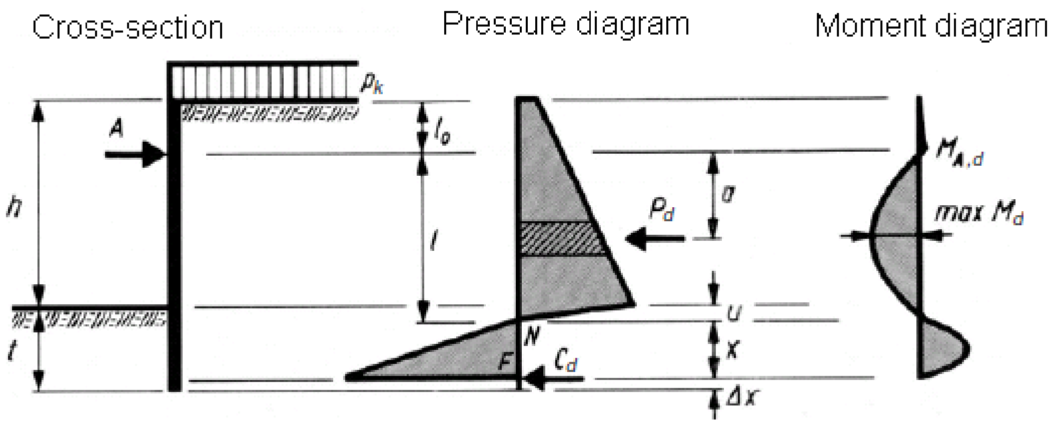

Figure 3.2 Blum method assuming a free sheet pile end [7]

3.2 Beam on an elastic foundation

In this model the retaining wall is modelled as an elastic beam on a foundation of springs. The governing equation to solve this problem is based on the 4th order differential equation of Bernoulli, here the assumption is made that the cross-sections of the beam remain straight and perpendicular to the beam axis. One of the software based on the beam on elastic foundation model is D-sheet developed by Deltares. The advantage of this model is that also the construction stages can be analysed. The calculation includes anchor pre-tensioning, horizontal and normal loads, overall stability of the wall and anchor stability (Kranz-method). The governing equation used in D-sheet is:

$$bEI\frac{d^4w}{dx^4} + N\frac{d^2w}{dx^2} = bf(x,w)$$

Where:

EI	Flexural stiffness of the beam [kNm²/m]
w	Horizontal displacement of the beam [m]
x	Coordinate along the axis of the beam [m]
N	Normal force in the beam [kN]
b	Acting width [m]
f	Total pressure on the beam, including the reaction of the soil springs [kN/m]

With the aid of computer software this equation can be solved numerically. An important aspect is the variation of the earth pressure coefficients due to the displacement of the wall [8] (Figure 3.3).

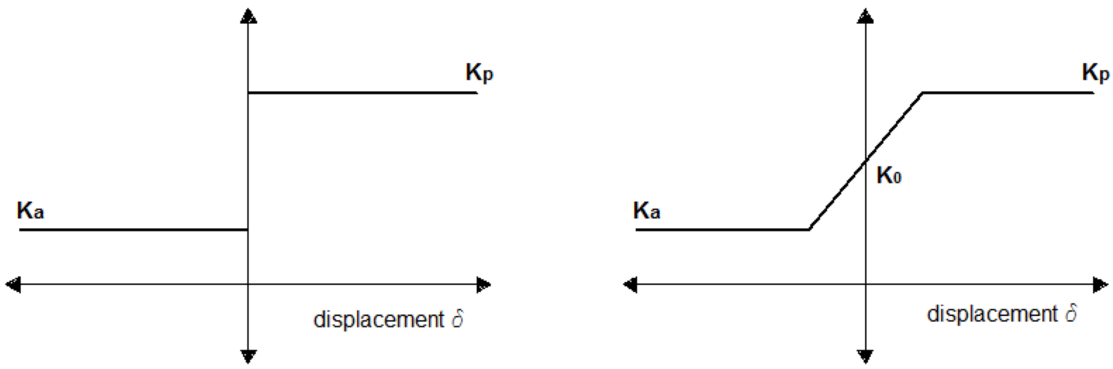

Figure 3.3 Left: Constant earth pressure coefficients. Right: Pressure coefficient in function of the displacement

3.2.1 Application of D-sheet in the design

As mentioned before, D-sheet is software developed by Deltares to design sheet pile walls based on the beam on elastic foundation method. This software has a user friendly interface and gives the possibility to assess designs within a small amount of time. When designing a quay wall with D-sheet several calculation methods can be chosen, namely:

- Standard calculation method using representative values;
- Design of the length of the sheet pile (minimum length);
- Verification of the sheet piling (using pre-defined safety factors):
    - Eurocode 7 general;
    - Eurocode 7 Dutch annex;
    - CUR;
    - Eurocode 7 Belgium annex.
- Allowable anchor force (KRANZ method);
- Overall stability (Bishop Method).

The verification of the sheet pile wall using the EC7 Dutch annex calculates each stage considering 6 steps, varying the modulus of subgrade reaction, water and surface levels at the passive side. The results of each step leads to the highest internal forces in the sheet pile wall and the highest anchor force.

3.3 Finite element method

The finite element method is based on a model in which the behaviour of soil and structure is integrated. Because of the mathematical complexity this method is developed in computer software (for example PLAXIS and DIANA). The advantage of this method is that not only the internal forces in the structure are modelled, but the soil stresses and deformations are also calculated. This software is rather complex and it is advised to use it as verifying tool rather than a design tool (the modelling and calculation are time consuming). The advantages of this method are:

- Dynamic behaviour can be modelled;
- Soil and structural interaction of the wall and anchor are present in the calculation;
- Soil and structure deformations and stresses can be calculated.

The finite element software is available for 2D and 3D calculations, however due to its complicity and computation time, 2D modules are the most popular. 3D modules are used mostly to analyse one certain problem such as: Group effectiveness of foundation piles and load distribution between primary members of a combined wall.

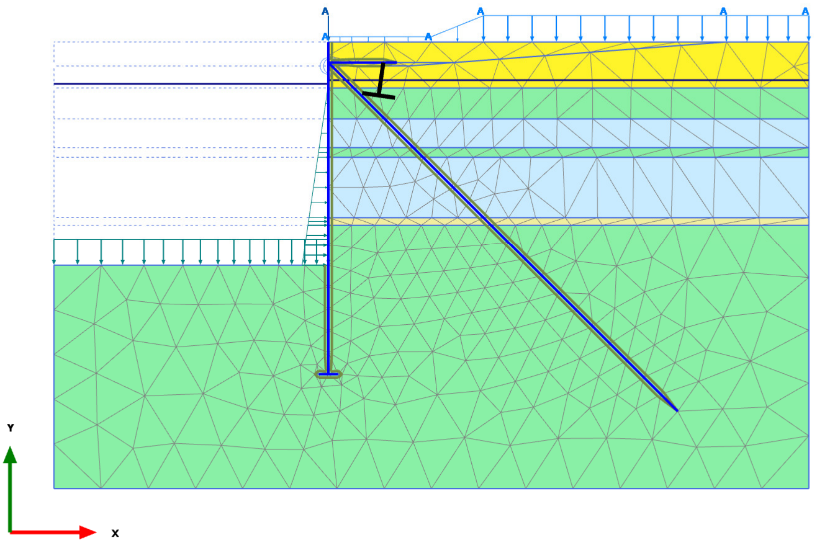

Figure 3.4 Impression of the model used in the FEM calculation (PLAXIS)

3.3.1 Application of PLAXIS in the design

In the CUR166 two methods are described to design sheet pile walls with the FEM [6]:

- **Calculation scheme A** performs a calculation using design values. It is possible to introduce load and material factors to perform calculation in PLAXIS. Normally, two soil stiffness are used to determine the decisive anchor force and internal forces. Experiences showed that deformation may be overestimated due to the use of safety factors in each stage;
- **Calculation scheme B** performs calculations with the characteristic values. The calculation takes into account the characteristic values of soil (c' and φ) and reduces it stepwise until a certain safety is obtained (φ-c' reduction method). The safety factor is defined as the ratio between the tangent of the reduced φ and the tangent of the characteristic φ. The CUR states that the deformations and internal forces are smaller leading to a less conservative design of the structure. Here also two soil stiffness are applied to obtain the decisive forces on the quay.

4. STARTING POINTS

The investigation is realized at the location of the "Amazonehaven" in the port of Rotterdam (Figure 4.1). In this area several cone penetration tests (CPT) were made. One of the CPTs is taken as decisive and the soil profile is derived based on the average cone resistance (see section 4.1.1).

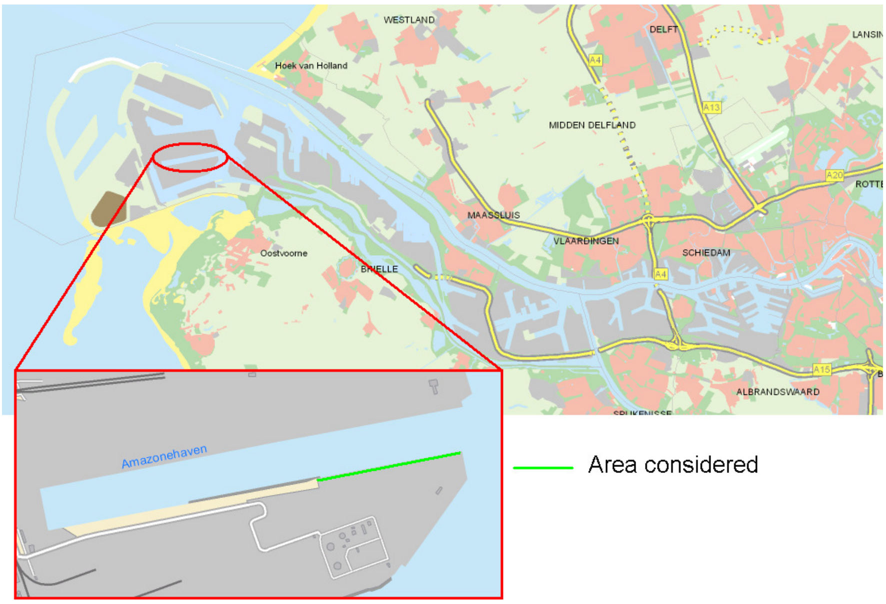

Figure 4.1 Location of the area considered in the investigation

In this section the starting points for the design are given, as well as the material characteristics, loads and load combinations.

4.1 Site conditions

4.1.1 Geotechnical

As mentioned before, the public works Rotterdam made several cone penetration tests (CPTs) and soil investigations in the area. According to these investigations the soil consists mostly of loose and stiff sand, also a clay/peat layer can be found at a depth of around -20 m NAP. For this thesis a CPT is chosen which consists of sand and a clay layer at -20,4 NAP. The CPT is shown in Annex A.

Table 4.1 Soil profile derived from CPT DO77

From NAP	To NAP	Thickness	Type
5,2	-1,5	6,7	Sand (loose)
-1,5	-6	4,5	Sand (moderate)
-6	-10,2	4,2	Sand (clay rests)
-10,2	-11,6	1,4	Sand (moderate)
-11,6	-20,4	8,8	Sand (clay rests)
-20,4	-21,5	1,1	Clay
-21,5			Sand (moderate)

The soil layers in Table 4.1 are determined using the report "Grondonderzoek Bouwput Zeekade Frans Swarttouw Maasvlakte" [9] and table 3.1 of the CUR 166 [6]. With these two documents the representative values of the soil parameters are determined (see Table 4.2, note that an average cone resistance is used to determine these parameters). The coefficients of active and passive earth pressure are determined with the theory of Coulomb [10].

Table 4.2 Soil parameters

From	To	Type	γ [kN/m3]		φ' [o]	δ [rad]	c' [kPa]	K_a	K_p	E [Mpa]
			dry	wet						
5,2	-1,5	Sand (loose)	17	19	30	20,00	0	0,28	5,74	15
-1,5	-6	Sand (moderate)	18	20	32	21,33	0	0,26	6,83	45
-6	-10,2	Sand (clay)	19	21	27	18,00	0	0,32	4,52	35
-10,2	-11,6	Sand (moderate)	18	20	32	21,33	0	0,26	6,83	45
-11,6	-20,4	Sand (clay)	19	21	25	16,67	0	0,35	3,91	35
-20,4	-21,5	Clay	19	20	20	13,33	14	0,43	2,81	4
-21,5		Sand (moderate)	18	20	32	21,33	0	0,26	6,83	45

Where:

δ	Angle of wall friction
φ	Angle of internal friction
c	Cohesion
γ	Weight density
K_a, K_p	Coefficients of active and passive earth pressure

Coulomb assumes straight slip plane failure in the calculation of the coefficient of earth pressure. The angle of wall friction in Table 4.2 is assumed to be $\delta = \pm 2/3 \varphi$ which corresponds to straight slip planes as assumed by Coulomb.

4.1.2 Hydraulic

Water levels

The site location to be analysed is located near the Beerkanaal and Europahaven. The water levels are determined with data of the Public works of Rotterdam [11].

Mean high water level (MHWL)	+1.27 m NAP	
Mean low water level (MLWL):	-0.66 m NAP	
Low low water spring (LLWS):	-0.90 m NAP	
High water with a frequency of (HWL):	1/250 year	+3.52 m NAP
	1/50 years	+3.23 m NAP
Low water with a frequency of (LWL):	1/250 year	-2.00 m NAP
	1/50 years	-1.66 m NAP

Waves

The "Amazonehaven" is situated such that it is protected from sea waves. The waves that reach the quay are the waves from passing ships and wind waves. Because of the limited speed of the ships and small fetch in the harbour both ship waves and wind waves are neglected in the calculation.

4.1.3 Nautical

Design ship

The design ship used in early designs in the "Amazonehaven" corresponds to a bulk carrier with a death weight tonnage (DWT) of 250000. Because this port is used mostly for bulk material, the largest bulk carrier is chosen as the representative design ship to take future increase in ship size. The MS Vale Brazil is the biggest bulk carried known at the time, this bulk carrier has a DWT of 402347, length of 362 m and draught of 23 meters.

4.2Geometry

4.2.1 Design depth

Handbook quay walls [2] show a method to determine the design depth of the quay wall. The requirement is that ships always can access the quay, this means that the depth needs to be calculated taking the lowest water level into account. The retaining height is calculated as follows:

Draught + $0.1 \cdot$ draught (vertical ship movement and net keel clearance is accounted as 10%) + 1.2 m (maintenance margin, sounding accuracy and dredging tolerance)

23 + 2.3 + 1.2 = 26.5 meters. The LLWL is -0.90 m, which means that:

26.5+0.90+5.2 = 32.6 m retaining height from +5.2 m NAP to -27.4 m NAP, this could be reduced if the biggest ships can access the quay only in periods of high water. However it is decided to allow ships to enter the quay at any time.

4.2.2 Cross section geometry

The cross-section geometry of the location is built with the starting points mentioned in section 4.1 and 4.2.1 is shown in Figure 4.2.

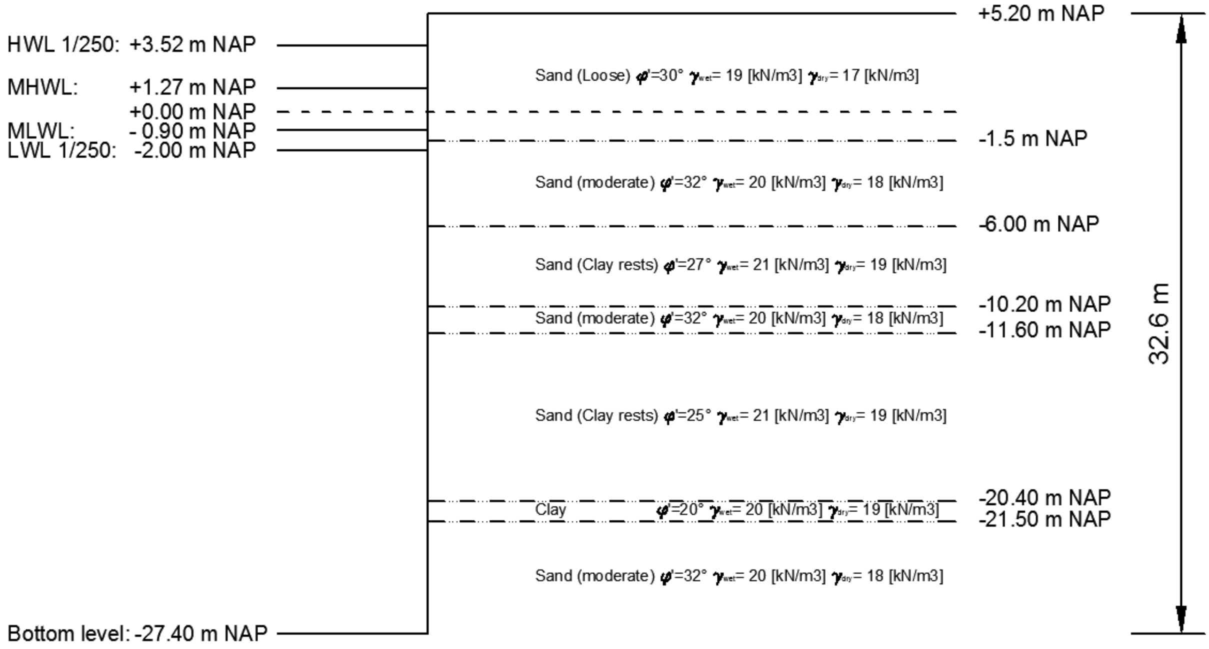

Figure 4.2 Cross section geometry

4.3Materials

The most common materials used for high retaining heights are steel and concrete. Variables that influence the decision of the material are: availability of the materials, construction time, durability and costs. Early studies in the Port of Rotterdam show that a quay made of a diaphragm wall is slightly more expensive than a quay wall made from a combined wall [14]. To limit the investigation it is chosen use steel for the retaining wall and concrete for the relieving structure. In this section the most important data of each material is given.

4.3.1 Steel

The usual steel qualities for tubular piles in combined walls vary between steel with a yield point of circa 345 [N/mm²] and steel with a yield point of 485 [N/mm²] according to the Handbook Quay Wall (2005). It should be noted that the minimum yield stress is designated for a plate thickness of 16 [mm]. For higher thicknesses a stress reduction needs to be taken into account for the design. The stress reduction is applied to take the making of the steel into account. When steel is rolled, it gains more strength by reducing its thickness (up to 16 mm). By high thicknesses (>50 mm) steel does not reach its full yield strength and a reduction of the yield stress needs to be applied (reductions can vary between 10 [N/mm²] to more than 20 [N/mm²]).

Some of the steel elements designed in this thesis have a thickness higher than 50 [mm] when the yield strength reduction is applied. To avoid this problem, high strength steel grade is used in the design, which has minimal yield strength between 500 and 700 [N/mm²] [12]. The EN-1993-1-12 states that no yield strength reduction needs to be applied by thicknesses <50 [mm] and that the standard is applicable without any additional rule. However, because of the limited knowledge in high strength steel behaviour, literature advises to design within the limits of the elasticity theory, without taking into account plasticity of the elements [13].

The parameters for steel used in the design are:

Table 4.3 Nominal values of yield strength f_y and ultimate tensile strength f_u for steel S500 (extract table 1 [12])

EN10025-6 Steel grade and quality	Nominal thickness of the element t mm			
	t<=50 mm		50 mm < t <= 100 mm	
	f_y [N/mm2]	f_u [N/mm2]	f_y [N/mm2]	f_u [N/mm2]
S 500Q/QL/QL1	500	590	480	590

Corrosion of steel

Corrosion is a very important aspect in quay wall made of steel and has to be taken into consideration in the dimensioning of the steel members. If there is no exchange of oxygen-rich water, corrosion is negligible at the soil side (except at the height of the drainage layer). Local environmental conditions and conditions during use (for example contaminants and effects of propeller jets) have great influence on the rate of corrosion. A dominant factor is the vertical positioning of the sheet piles in relation to the high and low water levels. The highest rate of corrosion often occurs in the oxygen-rich area just below low water zone. When designing quays, the corrosion problem can be restricted by choosing a construction level for the steel sheet pile wall with some margin below the low water level.

Measures that can be taken against corrosion are:

- Use of corrosion allowance (extra steel thickness);
- Application of coating;
- Active or passive cathodic protection;
- A combination of measurements.

The life time of the quay wall and the costs have great influence in the decision process regarding measurements taken against corrosion.

4.3.2 Concrete

Concrete is a widely used material that is well known in the hydraulic engineering world due to its durability and resistance against aggressive environments. In hydraulic structures however, special attention has to be given to crack formation. The dimensions of this type of structure are usually large and during the hardening phase special measures to prevent large temperature differentials should be taken. This heat formation can cause crack formation during the hardening process and measurements should be taken to prevent or control this phenomenon.

Several concrete classes are developed in time. The strength depends on the concrete mix, execution and curing of the concrete. In the past, concrete class C30/37 was used to construct the "Amazonehaven", the same concrete class is used in this thesis.

Table 4.4 Material properties concrete C30/37

Class	f_{ck} [N/mm2]	f_{cd} [N/mm2]	f_{ctd} [N/mm2]	f_{ctm} [N/mm2]	E_{cm} [N/mm2]
C30/37	30	20	1.35	2.9	33000

4.4 Special works, drainage

Drainage systems drain off precipitation and restrict excessive water pressure behind the earth retaining structure. In tidal areas this is an important design aspect and should be taken into account when economizing on the dimensions of the water retaining structure. It is advised to place the drain on a place where the access for maintenance is not restricted. A good maintenance of the drain during the lifetime of the quay is very important. The functioning can be threatened by damage to the system caused by settlement, fouling, and silting, blocking or jammed non-return valves.

In this thesis a drain at mean low water level is assumed. This way maintenance during low water is possible.

4.5 Loads

A summary of the loads used in the design of the quay walls is given in this section.

4.5.1 Surface load

Considering the type of goods and services that are handled around the area of the "Amazonehaven" it is assumed that the load consists of iron ore with a weight density of 22 [kN/m³]. The first 15 meters a load of 40 [kN/m²] is present; here loads such as traffic loads are included. This load increases then from 40 [kN/m²] to 200 [kN/m²] in an angle of 40 [°]. Figure 4.3 shows the surface load as used in the calculations.

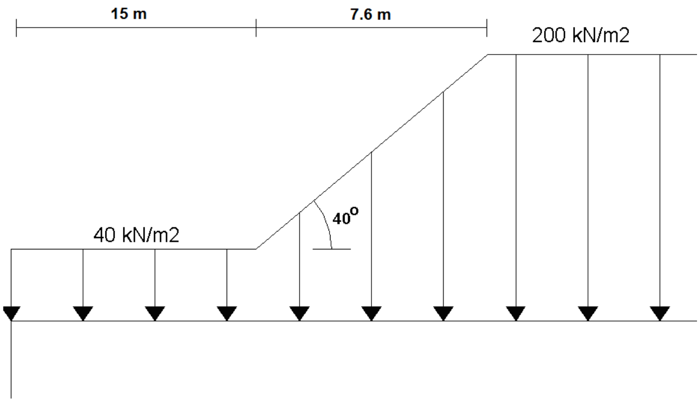

Figure 4.3 Surface load assumed in the calculations

4.5.2 Crane load

The crane used for the design is a grab crane meant for coal/iron ore with the following characteristics:

Table 4.5 Crane properties [2]

Type of crane	Outreach waterside [m]	Rail gauge [m]	Max. Vertical load [kN]	Max. wheel load [kN]	Number of wheels	Wheel distance [m]
Grab crane	45	50-70	12235	1529	8	1,57

The rail gauge at port side is founded on the quay wall while the rail gauge at land side is founded independently (no influence in the design).

4.5.3 Fender and collision load

The load due to berthing is estimated by choosing a fender that can absorb the berthing energy of the ship. It is assumed that the ship berths at a certain angle (5 [°]), the velocities are limited and the contact point with the quay lies 2 [m] from the surface. The berthing energy is calculated with the following equation [15]:

$$E_{kin} = \frac{1}{2} m_s v_s^2 C_H C_E C_S C_C$$

In which:

E_{kin}	Berthing energy [J]
m_s	Mass of the ship [kg]
v_s	Velocity of the ship perpendicular to the structure [m/s]
C_H, C_E, C_S, C_C	Hydrodynamic, Eccentricity, Softness and Configuration coefficient [-]

With this formula and a normal berthing velocity a fender and collision load is calculated:

Berth energy:	1654 [kJ]	Berth load:	1800 [kN]
Collision energy:	4867 [kJ]	Collision load:	3100 [kN]

4.5.4 Bollard load

When the ship is placed next to the quay it is fastened to bollards. This generates a load on the quay wall. Representative values of the horizontal component of this load are given in the EAU 2004 (Table 4.6). The EAU advises to increase this force with 25% for ships with a water displacement bigger than 50000 [ton].

In the case of the MS Vale Brazil, a water displacement bigger than 200000 occurs, which leads to a line pull force of 2500 [kN] per bollard.

Table 4.6 Representative values for the bollard load [16]

Displacement [ton]	Line pull force [kN]
up to 2000	100
up to 10000	300
up to 20000	600
up to 50000	800
up to 100000	1000
up to 200000	1500
>200000	2000

4.5.5 Spreading of the horizontal loads

To be able to model the horizontal load acting on the quay wall, a spreading has to be taken into account. In this thesis it is assumed that a bollard and fender is placed every 15 meters, such that the horizontal force acting on the quay is equal to the total force divided by 15 meters. However, when dealing with a concrete structure a spreading of 45 degrees is most commonly used. This would lead to a higher horizontal force per meter. In this thesis however, all cases where calculated assuming a bollard and fender at a centre to centre distance of 15 meters.

4.6Load combinations and design philosophy

In section 4.6.2 the load combinations described in the EAU are mentioned [16]. In this description, unusual and rare scour are mentioned. The EAU advices to calculate increasing the retaining height to take scour into account. This leads to higher bending moments and anchor forces in the wall. When designing a quay wall one has to decide whether or not to apply a bed protection. Scour holes can lead to overall instability of the quay or failure of the structure element due to much higher loads. An analysis from an economic point of view should be made by the designer whether or not bottom protection is desired. In this thesis work a detail calculation is not made, it is assumed that scour may not occur (this means that bottom protection would need to be applied).

4.6.1 Design philosophy

The design of the quay walls in this thesis is based in the NEN-EN 9997 [17], here a design approach for quay walls in The Netherlands is given.

The dimensions of the elements of a quay wall depend on the requirements in the ULS (ultimate limit state) and SLS (serviceability limit state).

Ultimate limit state

In the ultimate limit state partial material and load factors are applied in the calculation. These factors are given and explained in section 4.6.4.

Three combinations of factors are established in the Eurocode [17]. In the Netherlands, combination 3 is used for the design of quay walls in the ULS (ultimate limit state). The combination of factors is the following: A2, M2 and R3.

In retaining structures, A2 stands for geotechnical actions and is equal to 1.1 (see Annex F) when checking the strength of the quay wall itself (bending moments, shear force, normal force and anchors). M2 stands for material factors (see Table 4.9), the material factors reduces the soil parameters in such a way that higher active pressure and lower passive pressure acts on the quay wall. R3 stands for resistance, in quay walls the Eurocode assumes a value of 1 (no reduction of the resistance).

Next to this combination of factors a combination factor is applied on loads in a load combination, the combination factors are used to simulate the loads in the whole lifetime of the structure. The combination factors are shown in Table 4.11 and explained in section 4.6.4. When these factors are applied design internal forces are calculated which determine the dimensions of the elements.

According to the Eurocode, the combination of the stress due to the design values of the normal force and bending moments in the wall need to be less than the yield strength. It is also advised to do a second check of the internal forces by calculating the stress due to the characteristic values of the normal force and internal moment (SLS). The yield strength of the elements should be higher than 1.2 times the representative value of the calculated stress [6].

The strength needs to be sufficient to prevent structural failure of the elements. The strength criteria are satisfied when:

$$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1 \qquad \text{And} \qquad 1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}} \right) \leq 1$$

This equation is a verification of enough capacity for the bending moments (M) in combination with normal force (N), for more details the reader is referred to the NEN 9997-1 [17] and the CUR166 [2].

Serviceability limit state

In the SLS the deformation must be less than the deformation that allows functioning of the quay wall. Depending on the function of the quay wall, the SLS is decisive for the dimensioning of the elements (due to the required stiffness to prevent large deformations). In the case of a bulk terminal, the deformation in the SLS does not have too strong requirements. Here the ULS will be decisive for the dimensioning of the elements of the wall. The dimension and positioning of the anchor however can be decided by the SLS instead of the ULS.

4.6.2 Load combinations

When designing a quay wall, several load combinations have to be taken into account to find the decisive internal forces that will occur during the life time of the structure. The EAU gives a description of the load combinations that can be used in the calculation. The combinations mentioned here are used as reference for the calculation in this investigation.

Load combination 1 (LC1)

Loads due to active earth pressure and water pressure differences where unfavourable outer and inner water levels occur frequently. Earth pressures resulting from the normal live loads from normal crane loads and pile loads, direct surcharges from dead loads and normal live loads.

Load combination 2 (LC2)

Same as LC1 but with restricted scour caused by currents or ships screw action. Also other forces need to be considered as long these can act together: rare unfavourable water level differences, suction effect of passing ships, pressure due to exceptional local surcharges, loads from frequent waves, compression and impact loads such as line pull on bollards, pressure on fenders, lateral crane loads or from temporary construction conditions.

Load combination 3 (LC3)

As LC2 but with unusual scour, extreme water level conditions, surcharges on larger areas not previously allowed and water pressure with impact of drifting materials.

Accidental load combination (ALC)

Example of exceptional cases are: simultaneous occurrence of extreme water levels and extreme wave loads due to plunging breakers, extreme water level in combination of complete failure of the drainage system or combination of three rare events such as high tide, rare waves and impact of drifting material.

In the design calculation ship collision is taken as exceptional scenario. It is expected that the water overpressure due to the failure of the drainage will not be relevant. Also, as mentioned before, no scour is taken into account.

4.6.3 Water levels for the load combinations

The water levels mentioned in section 4.1.2 are used as a starting point to determine the ground water level in the load combinations. The ground water levels are calculated assuming a reliable drainage system at MLWL. The water levels used in the design are based on the revision chapter 6 of the Handbook Quay Walls (2005) [18].

The assumed water levels used for the calculation in normal conditions are shown in Table 4.7.

Table 4.7 Water levels and ground water levels [18]

Water level fluctuations	Water level (WL)	Ground water level (GW)	Δh_{min}
Minor	MLW	$h_{drainage}$+0.3 m	>0.5 m
Major (rivers)	OLW/OLR	$h_{drainage}$+0.3 m	>0.5 m
Tidal conditions	LLWS	$h_{drainage}$+0.3 m	>0.5 m

Also a difference is made in the accidental load case where the water levels are determined using Table 4.8.

Based in the information in the tables mentioned in this section, the water levels assumed for each load case are the following:

LC1: Water level = -0.9 m Ground water level = -0.36 m Δh = 0.54 m

LC2: Water level = -1.66 m Ground water level = -0.36 m Δh = 1.3 m

LC3: Water level = -2.00 m Ground water level = -0.36 m Δh = 1.64 m

ALC: Water level = -2.00 m Ground water level = -0.36 m Δh = 1.64 m

ALC stand for accidental load case during extreme low water and collision of a ship.

Table 4.8 Accidental water pressure difference with drainage [18]

Accidental actions	Water level fluctuation	Soil conditions	Water level (WL)	Ground water level (GW)	
Extreme low water	-	-	$LW_{1x250\ year}$	$h_{drainage}$+0.3 m	-
Relieving floor	-	-	$LW_{1x250\ year}$	$LW_{1x250\ year}$	-
Failure drainage	Tidal conditions	Impermeable	LW_{1xyear}	MSL	>1.5 m
		Permeable	LW_{1xyear}	MSL	>1.0 m

4.6.4 Safety (Partial) factors

The material and safety factors used for the design of the quay walls in this thesis are according the Eurocode. Here the following safety categories are mentioned:

- Safety category 1 (RC1): Where the risk of danger to life is negligible and the economic damage is low (reference period 15 years);
- Safety category 2 (RC2): Where the risk of danger of life is negligible and the risk of economic damage is high (reference period 50 years);
- Safety category 3 (RC3): Where both the risk of danger of life and the risk of economic damage are high (reference period 50 years).

Safety category 2 (RC2) is normally used in the port of Rotterdam. The reason is that a minimal lifetime of 50 years of the quay wall is expected. Because of the function of the port (handling of freight) the risk of damage of life is slight but the risk of economic damage is very high.

Table 4.9 shows the material factors for the soil used in the calculation for a sheet pile wall with and without relieving structure.

According to the Eurocode, the representative variable loads in a combination of loads are determined by multiplying the characteristic value by a reduction factor. Combinations of loads are based on the occurrence of one extreme value of a variable load, which means that the other loads are reduced. In addition, an accidental load is not combined with the extreme value of other variable or accidental loads. Table 4.9 shows the use of safety and reduction factors for the design in the ultimate limit state. Here a difference is made between fundamental combination and accidental combination.

A fundamental combination is composed of permanent and variable loads that are based on the leading variable load with extreme value. The other simultaneously occurring loads are reduced.

An accidental combination is composed of a combination of permanent loads, one accidental load and possible simultaneously occurring loads. The reduction factors used in the combination are given in Table 4.10.

Table 4.9 Partial factors for soil parameters extracted from the NEN 9997-1:2012 [17]

Soil parameter	Symbol	Simple quay wall	Quay wall with relieving structure
Angle of internal friction [a]	$\gamma_{\varphi'}$	1.175	1.25
Effective cohesion	$\gamma_{c'}$	1.25	1.45
Undrained shear strength	γ_{cu}	1.6	1.75
Density	γ_{γ}	1	1
a This factor is connected with tan φ'			

Table 4.10 Combination of loads in the ultimate limit state [2]

Type of combination	Permanent loads (G_d)	Variable loads (Q_d)		Accidental loads $F_{a;d}$
		Leading variable load	Other simultaneously occurring variable loads	
Fundamental	$\gamma_{f;g} * G_{rep;max}$	$\gamma_{f;q} * Q_{1;d}$	$\gamma_{f;q} * \Psi_{0,j} * Q_{j;rep}$	-
Accidental	$\gamma_{f;g} * G_{rep;max}$	$\gamma_{f;q} * \Psi_{1,1} * Q_{1;rep}$	$\gamma_{f;q} * \Psi_{2,1} * Q_{j;rep}$	$F_{a;rep}$

Table 4.11 Recommended reduction factors for load combinations [18]

Action	Combination factor Ψ_0	Frequent value Ψ_1	Quasi static value Ψ_2
Uniform terrain load (cargo: containers, bulk goods)	0,7	0,5	0,3
Traffic loads (port vehicles)	0,6	0,4	0
Crane loads (crane for cargo handling)	0,6	0,4	0
Mooring loads (bollard pull / hawser load)	0,7	0,3	0
Ship berthing loads (reaction force fendering)	0,7	0,3	0
Earth pressures	1,0	1,0	1,0
(ground)water pressures	1,0	1,0	1,0
Differential settlement	1,0	1,0	1,0
Meteorological loads (wind, waves, temperature, ice)	0,7	0,3	0

4.6.5 Load combinations used in the design

The load combinations in this section are based on the description of the load combination of section 4.6.1, the water levels given in section 4.6.3 and the partial factors in section 4.6.4. These load combinations are used for the design in the ultimate limit state, in other words, the dimensioning of the elements and the check of the strength in the ULS.

		ULS				
		LC1	**LC2.1**	**LC2.2**	**LC3**	**ALS**
1	**Dead load from superstructure**	1	1	1	1	1
2	**Soil pressure**	1	1	1	1	1
3	**Surface load**	1*1.1	1*1.1	1*1.1	0.7*1.1	0.5*1
4	**Crane load**	0.6*1.1	0.6*1.1	0.6*1.1	1*1.1	
5	**Bollard load**		0.7*1.1			
6	**Fender load**			0.7*1.1		
7	**Accidental load (impact)**					1
8	**Water pressure (Average water level)**	x				
9	**Water pressure (Rare water level 1/50 year)**		x	x		
10	**Water pressure (Extreme water levels 1/250 years)**				x	x

The combinations showed in this table are the ones used in the design of all variants, however due to a misunderstanding with the design philosophy of the Eurocode, the used combination for the design is not normative. The influence of this is small and the combinations are left as shown in this section. For more information along with a calculation of the influence in the final results the reader is referred to Annex F.

4.7 Parametric analysis

Two main options are considered in the analysis: The sheet pile wall without relieving structure (Option A) and the sheet pile wall with relieving structure (Option B). Each of these options has sub-options that could significantly influence the design.

The most important aspects that determine the final design of the quay wall are:

- Soil parameters (determined in section 4.1);
- Wall and anchor stiffness;
- Embedding depth (large embedding depths can lead to reduction of construction material);
- Anchor type and position;
- Pressure on the wall.

4.7.1 Wall and anchor stiffness

The wall stiffness is determined by the minimal profile that satisfies the strength criterion mentioned in section 4.6.1. It is known that by longer walls, a better redistribution of moments is obtained. During this thesis several tests have been made by designing the wall larger than minimal required. These tests have not been documented but calculations showed that changing the dimensions of the primary members (steel tubes) does not lead to lower costs (this is valid for this specific case, the designer should make some calculations to see whether or not this applies to his case, for the calculation of costs the reader is referred to Chapter 8). From this test calculation a decision was made to calculate the minimal required length of the elements.

The stiffness of de anchor is determined by means of iteration between the centre to centre distance of the anchors, its length and cross-section.

4.7.2 Anchor position

To have a better understanding of the influence of the anchor position in a quay wall, several positions are evaluated:

- +2.2 m NAP, three meters from surface;
- +0.2 m NAP, five meters from surface;
- - 1.8 m NAP, seven meters from surface;
- -6.0 m NAP, eleven meters and twenty centimetres from surface.

In option B the system line of the slab of the relieving structure is placed at this level. It needs to be noted that level +0.2 m NAP is omitted in option B.

4.7.3 Anchor type

The decision of which anchor to use depends on several factors:

- Costs;
- Risks;
- Surrounding:
    - Enough construction space;
    - Adjacent structures/obstacles;
    - Installation tolerances;
- Construction time;
- Soil investigation.

The objective of the parametric study is to compare the variants in the same conditions. To be able to do this it is assumed that the surrounding does not influence the design (there is enough space for the construction and no obstacles are present in the soil).

Because of the assumption that there are no limitations in space, horizontal plate anchors are chosen as first anchor type. The second anchor type used in the design is the MV-pile. MV-piles are known for their high holding capacity and high stiffness. It is quite common to use MV-pile in case of high anchor loads. For the purpose of this investigation and the limited amount of time, only these two anchor types are evaluated.

4.7.4 Design steps

The first step is to make an estimation of the internal forces in the wall, anchor force and embedding depth, this is calculated analytically by the method described in section 3.1 and annex B. This calculation is not included in the report because its results are used only as first input in the D-sheet calculation. However, to be aware of the exactitude of the analytical method a comparison of a simplified case is made in Chapter 5.

The first dimensions and distribution of forces is obtained by means of an iteration process in D-sheet. The draft design is then modelled and calculated in PLAXIS. The results of PLAXIS are used to check the strength and stability of the wall.

The steps followed are:

1. Determine the start parameters (see Chapter 4);
2. Establish load combinations, safety and reduction factors (see section 4.6);
3. Asses the dimension of the primary elements, embedding length and anchor force with a hand calculation (not included in the report);
4. Use the results of step 3 to model the structure in D-sheet. Start an iteration process between the embedding depth, wall and anchor properties;
5. Use the result in step 4 as input in PLAXIS and perform a calculation;
6. Check the strength of the primary elements with the results obtained in step 5;
7. Check the anchor strength with the results obtained in step 5;
8. Check the deformations;
9. Overall checks (Kranz stability, overall stability, bearing capacity of the primary elements, failure of one anchor and local buckling).

The checks performed in step 9 are the ones made in this thesis. However, the designer has to be aware that other checks need to be considered during design. Some of these checks are:

- Failure of the anchor due to bending: The function of an anchor is to resist normal tension loads. It can occur that the soil under the anchor undergoes settlement and the anchor rod would start carrying the soil above it. This results in bending of the anchor and higher anchor loads;
- Connections: In detailed design connections have to be sufficient strong to transfer the loads between members;
- Piping and heave: As mentioned in section 2.5.2 piping due to ground water flow can lead to overall instability of the quay. It is assumed that the clay layer restrains the flow of water under the quay. If the soil in front of the quay is not heavy enough to prevent heave, extra weigh by (for example) bed protection can be applied

4.7.5 Cases considered in the investigation of Option A

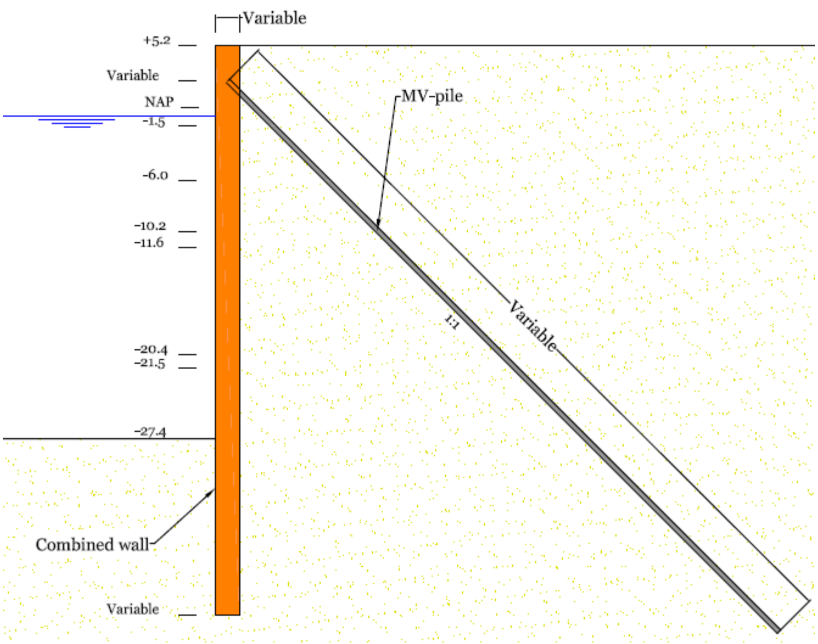

Figure 4.4 Option A1 - Quay wall without relieving structure and MV-pile as anchor

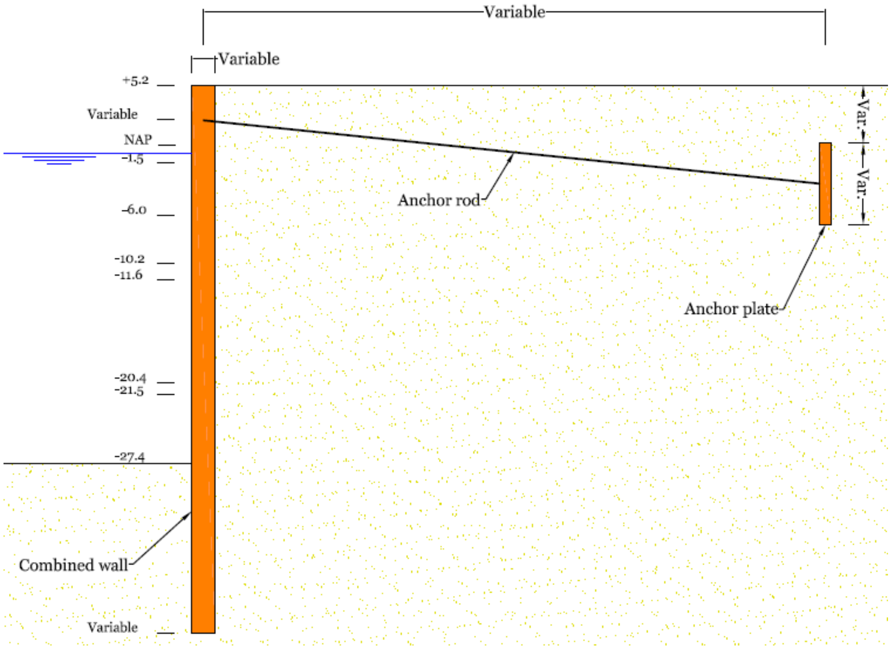

Figure 4.5 Option A2 - Quay wall without relieving structure and anchor plate as anchor

4.7.6 Cases considered in the investigation of Option B

Figure 4.6 and Figure 4.7 show the cases taken into account for the case of Option B. Here the connection between the retaining wall and the relieving structure is considered as a hinge connection as advised in Handbook Quay Walls [2]. In these figures:

H	Distance from surface to system line of the slab
L	Length of the slab of the relieving structure (10 and 20 meters)
d	Thickness of the relieving slab (1 and 1.8 meters, see chapter 7)
s	Slope of the bearing piles (7 for L=10 m and 3 L=20 m, see annex D)
w	Width of the beam of the relieving structure equal to 3 meters based on [9]

Note that when the relieving structure has a length of 20 m 2 rows of bearing piles are applied at a centre to centre distance of 2 meters.

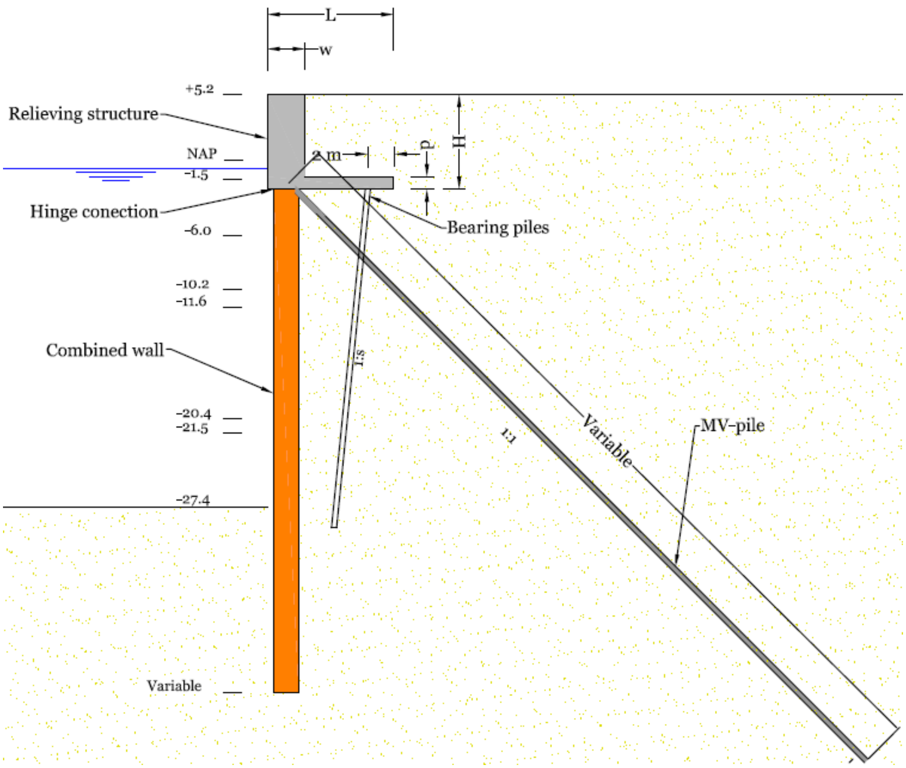

Figure 4.6 Option B1 - Quay wall with relieving structure and MV-pile as anchor

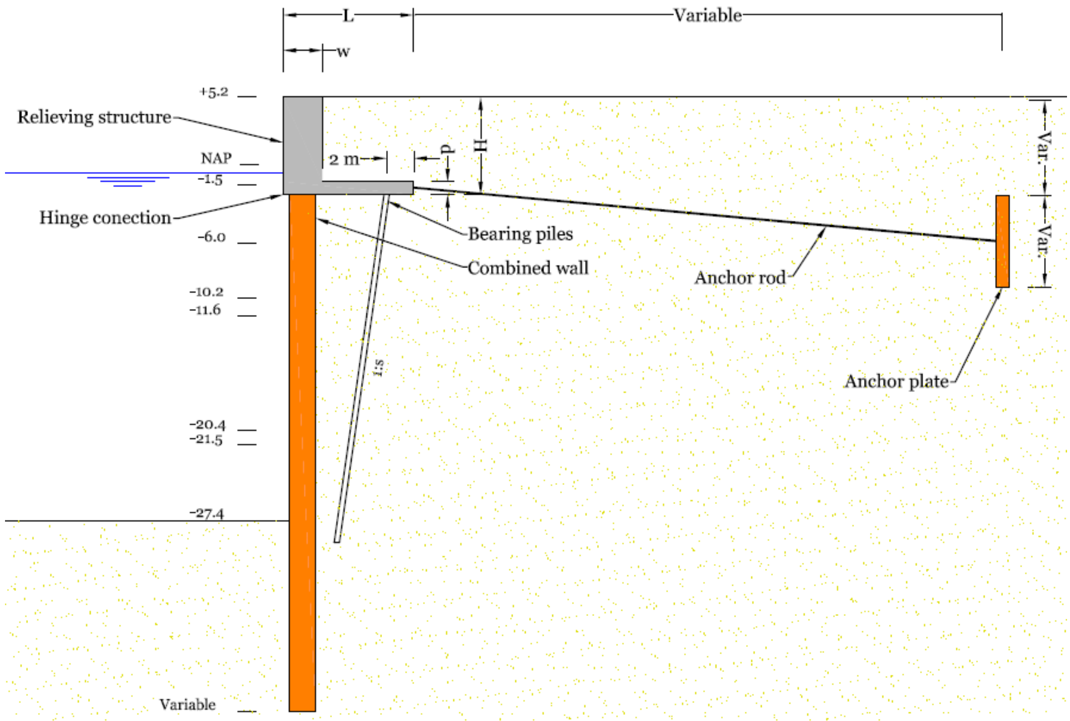

Figure 4.7 Option B2 - Quay wall with relieving structure and Anchor plate as anchor

5. HAND CALCULATION AND VALIDATION

This chapter is included with the purpose of depicting the importance and degree of accuracy of hand calculations. When calculation software is not available, an engineer must be able to give recommendation based on hand calculations and engineering judgement.

The method of BLUM considering a fixed and free end support is evaluated comparing it to the results calculated with D-sheet (beam on elastic foundation method). The validation of the hand calculation is made for a sand profile with the following parameters:

Type	**γ [kN/m3]**		**φ' [o]**	**δ [rad]**	**c' [kPa]**	**K_a**	**K_p**
	dry	**wet**					
Sand	17	19	35	23.3	0	0.22	9.15

In D-sheet a modulus of subgrade reaction of the soil needs to be defined. The CUR166 [6] gives representative values of subgrade reaction modulus for several types of soils. The representative lower boundaries of the modulus of subgrade reaction in function of the relative displacement for stiff sand [6] are:

	[kN/m^3]
k_h50%	40000
k_h80%	20000
k_h100%	10000

The comparison is made for a fixed anchor position and several retaining heights (5, 10, 15, 20 and 25 meters). The anchor level is considered at surface level. A constant water level difference of one meter is considered and kept constant in all calculations.

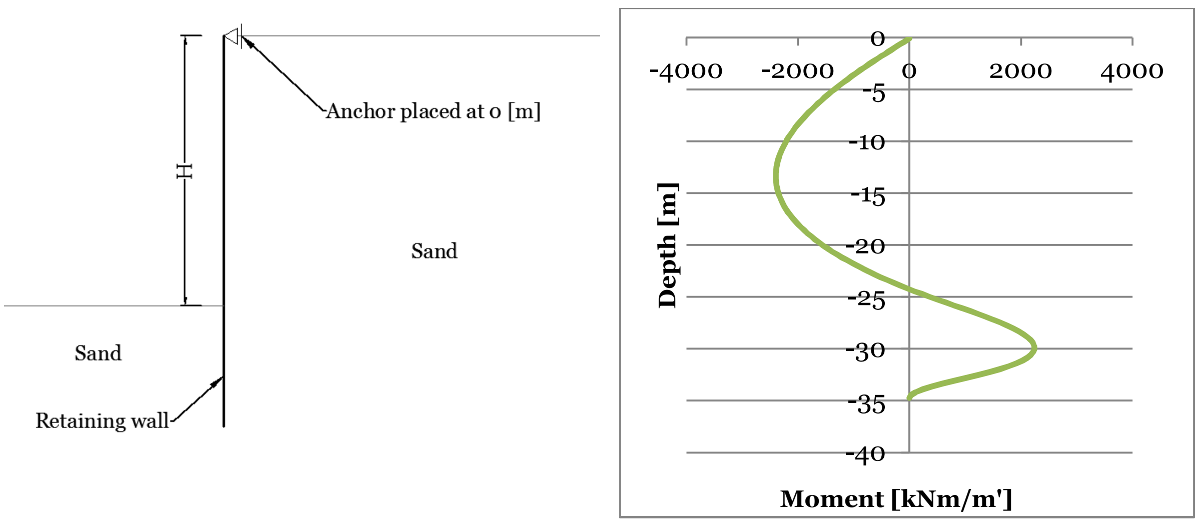

Figure 5.1 Left: sheet pile wall in sand with anchor at surface. Right: Impression of bending moments in the wall

The reader is referred to Annex B for more information about the calculation.

5.1 Results

Figure 5.2 shows the relation between the embedding depth and the retaining height. Here a linear increase of the embedding depth is observed for both methods. This is obtained due to the homogeneity of the soil layer and the parameters that are chosen to be constant in the whole depth. It needs to be noted that this embedding depth needs to be increased according to the literature, see section 3.1.

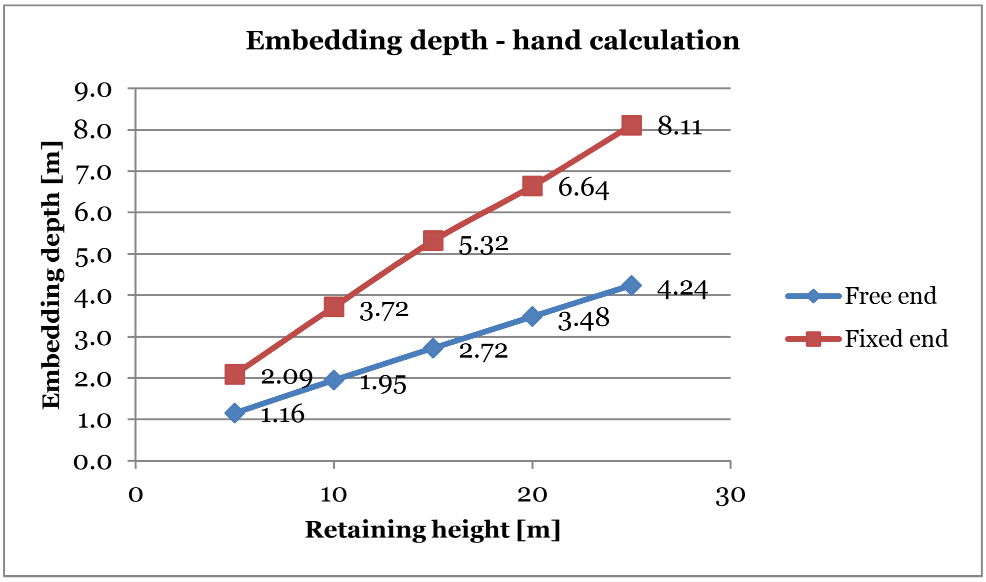

Figure 5.2 Calculated embedding depth in function of the retaining height

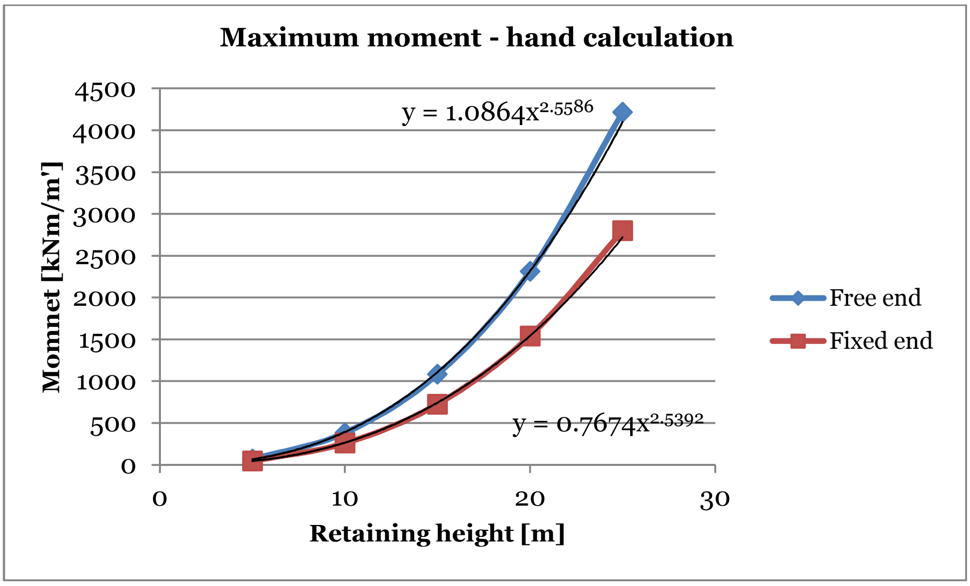

Figure 5.3 Moment in function of the retaining height (hand calculation)

Figure 5.3 shows that the moment calculated with the hand increases with a power of 2.54 in the fixed end method and a power of 2.56 in the free end method. As expected, the maximum moment calculated assuming a free end support is between 40% and 50% higher than the one calculated with the fixed end method. The same with the anchor force, an increase with a power of about 1.62 is obtained for both hand calculation methods. Here a difference of 30% in the anchor force is observed.

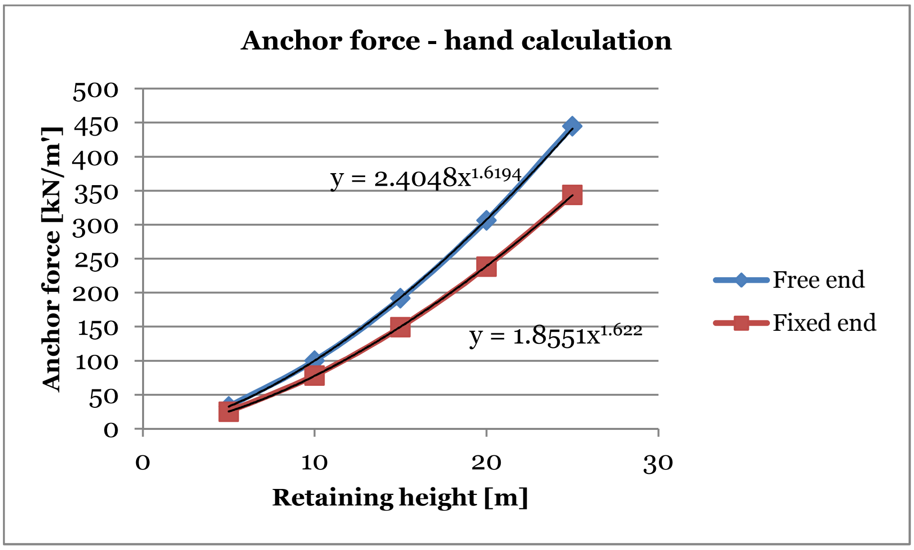

Figure 5.4 Anchor force in function of the retaining height (hand calculation)

The embedding depth shown in Figure 5.2 is the one resulting directly from the calculation. When using this method, the embedding depth has to be increased with 10 to 40%, depending on which method is used (literature states that an increase of 40% for the free end method and 20% for the fixed end method is reasonable [4] and [7]).

This increase in embedding depth is modelled with D-sheet and compared with the results obtained with the hand calculation. D-sheet also takes into account the profile of the sheet pile for the moment distribution and anchor force (stiffness has influence in the soil displacement, hence in the horizontal pressure calculated with D-sheet). For the calculations made a sheet pile with enough moment capacity is chosen for each retaining height. The calculation is made for an increase in embedding depth of 0%, 20% and 40%.

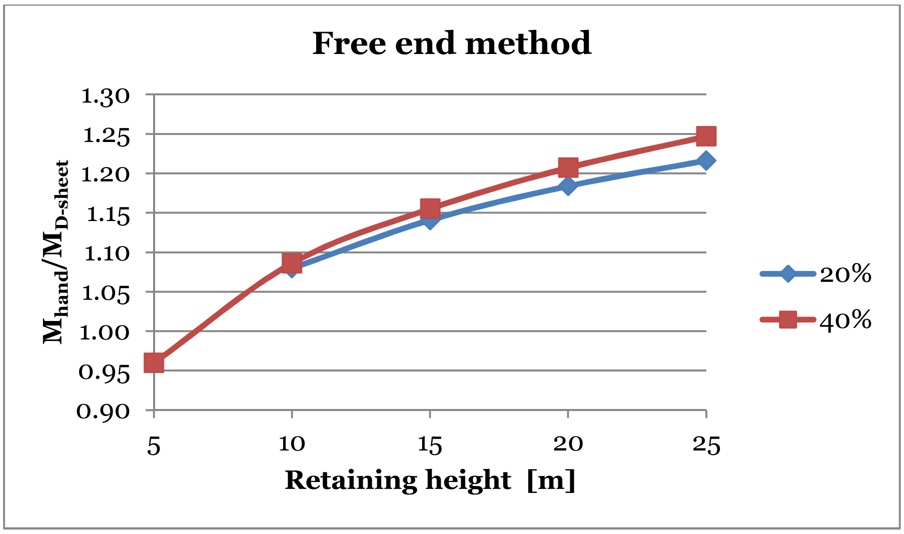

Figure 5.5 Ratio moment hand calculation and moment D-sheet in function of the retaining height (free end method)

First the free end method is discussed. In Figure 5.5 the ratio of the moment calculated with the hand and the moment calculated with D-sheet for an increase in embedding depth of 20% and 40% are shown. The results of this calculation showed that the sheet pile is unstable in all cases when the embedding depth is not increased. In the case of 5 [m] retaining height stability is reached by an increase of at least 40% of the embedding depth (which coincides with the literature). With this method moments for a retaining height of 5 [m] are underestimated (about 5%), while for depths higher or equal to 10 [m] are overestimated (5 to 25%). Literature advises an increment of 40% to be on the safe side, however this calculation shows that for retaining height higher than 10 meters 20% of increase is enough to reach stability of the wall.

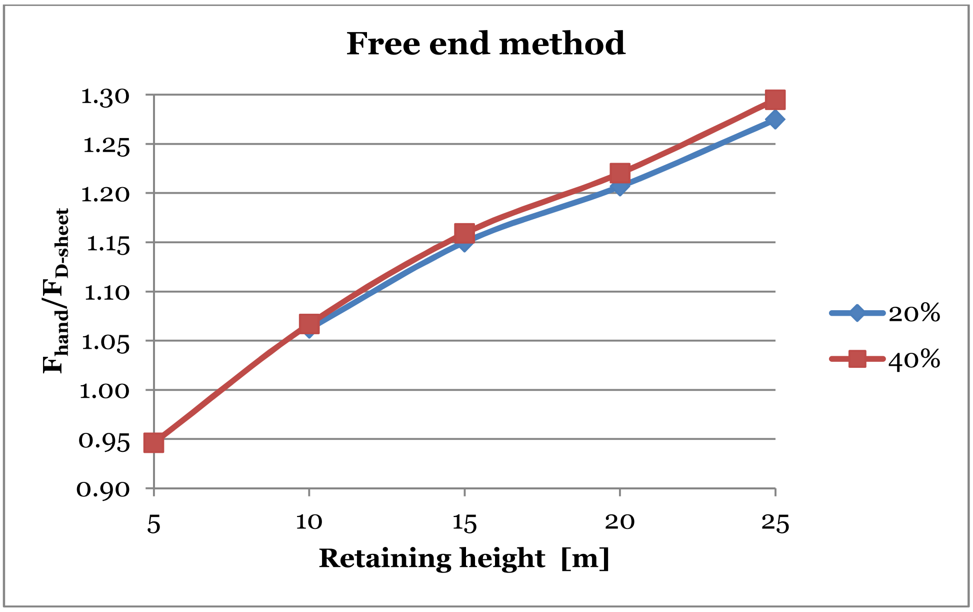

Figure 5.6 Ratio anchor force hand calculation and anchor force D-sheet in function of the retaining height (free end method)

The same can be said for the anchor force (Figure 5.6), for 5 [m] retaining height the anchor force is underestimated and for retaining heights of 10 [m] or more overestimated.

The second method evaluated is the fixed end method. The ratio between the moments calculated with D-sheet and calculated with the hand (Figure 5.7) shows stability of the wall without increasing the embedding depth (the embedding depth calculated with the fixed end method is in average 90% bigger than the one calculated in the free end method).

By retaining heights of 5 [m] the moments and anchor forces (Figure 5.8) are underestimated with an increase of the embedding depth of both 20% and 40%. The reason is that the calculated embedding depth at this height is equal to 2.1 [m], an increase of 20 and 40% (0.42 [m] and 0.84 [m]) lead to a small re-distribution of moments. To clarify this length of the wall was increased until full fixity is reached. The moments stay at a constant value when the wall length is increased to 10 [m] (2 times the retaining height). At this point the ratio $M_{hand}/M_{D\text{-sheet}}$ is equal to 0.8, which means that regardless the increase in embedding depth the moments are underestimated with the hand calculation at this retaining height.

By a retaining height of 10 [m] a good approximation is obtained by an increase of 40% of the embedding depth. For retaining heights higher than 15 [m] an increase of 20% of the embedding depth leads to an over estimation of the moments and anchor forces, thus a more conservative design.

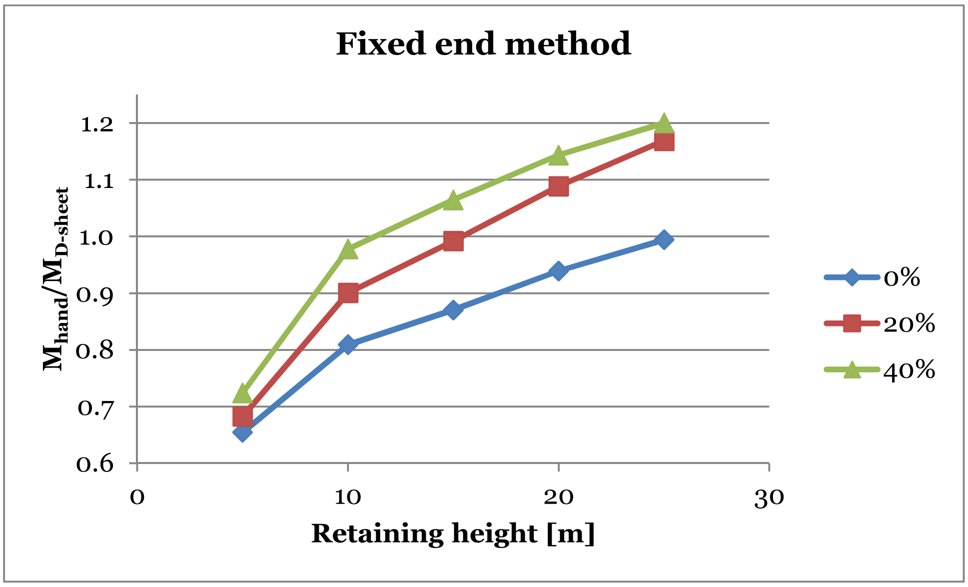

Figure 5.7 Ratio moment hand calculation and moment D-sheet in function of the retaining height (fixed end method)

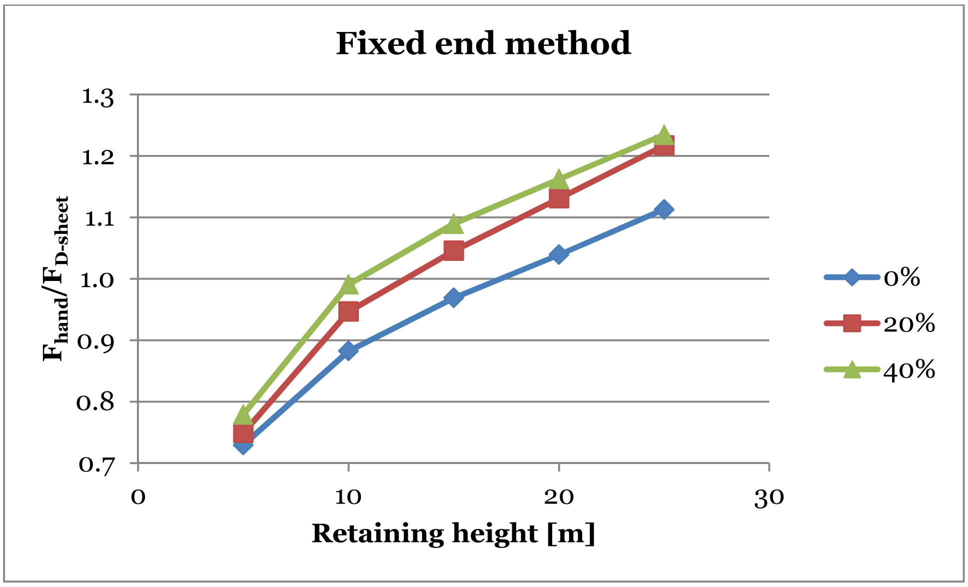

Figure 5.8 Ratio anchor force hand calculation and anchor force D-sheet in function of the retaining height (fixed end method)

5.2 Conclusion and recommendations regarding the validation of the hand calculation

5.2.1 General conclusion

The evaluated hand calculation and calculation software (D-sheet) are powerful tools to estimate bending moments, anchor forces and embedding depths in sheet piles. The designer has to be aware of the limitations of each method to avoid under estimation of the forces. If a designing software is available (D-sheet for example), the result of the hand calculation can function as a good input or as an approximation for a concept design.

5.2.2 Conclusions free end support method

- An underestimation of maximal 6% of the moments and anchor forces is found at retaining heights under 10 [m]. An increase in embedding depth of minimal 40% to ensure stability is required;
- For depths higher or equal to 10 [m] an overestimation of moments and anchor forces is made. The results show that an increase of 20% of the embedding depth leads to an over estimation of 5% at a retaining height of 5 [m] and 25% at a retaining height of 25 [m].

5.2.3 Conclusion fixed end method

- At a retaining height of 5 [m] the hand calculation is less exact than the free end method. An underestimation of 30% of the maximum moment is made at an increase of 20% of the embedding depth. At a certain depth the wall will reach full fixity and the moments will reach a constant value that is still 25% higher than the moment calculated with the hand;
- Retaining heights of 10[m] show good results by an increase of 40% of the embedding depth. When the embedding depth is not increased, the moment is underestimated with 20%, by an increase of 20% of the embedding depth the moment is underestimated with only 10%;
- At retaining heights of 15 [m] or more, an increase of 20% of the embedding depth shows a good estimation of the bending moment and anchor forces (overestimation of 0% to 15%).

5.2.4 Recommendations

The comparison made in this chapter is valid for a sand layer with an anchor at surface level. To gain more insight in the free end and fixed end method it is recommended to extend the investigation in scenarios with different (several) types of soils and varying anchor position.

When dealing with retaining heights of less than 10 [m] it is advised to use appropriate safety factors for the loads acting on the quay wall. Another possibility is to use a safety factor for the coefficients of horizontal pressure. By increasing the active pressure and reducing the passive pressure a more conservative approach is used.

6. OPTION A: QUAY WALL WITHOUT RELIEVING STRUCTURE

In this chapter the results obtained in the calculation of the quay wall without relieving structure (Option A) will be presented and discussed. The detailed calculation is presented in Annex C, here only a summary of the calculation method and results is given.

6.1 Calculation method

The variation in the anchor position is made as presented in section 4.7. The calculation concerns a design with MV-pile (Option A1) and a design with an anchor plate as anchor (Option A2), in both cases several levels are evaluated.

The first estimation of the bending moments and anchor force is made analytically with the evaluated method in chapter 5. With the internal forces and anchor force calculated in this section a first estimate is made for the dimension of the elements which are used as input in D-sheet to calculate the minimum depth and internal forces in all load cases. Here an iteration process is made to define which dimensions of the anchor and primary members are needed. After the final dimensioning of the elements with D-sheet, a PLAXIS calculation is made. The results of the PLAXIS calculation are used to check the strength, deformation and stability of the final design.

6.2 Results

6.2.1 Dimensions

A summary of the dimensions using both anchor types in the design is given in Table 6.1 and Table 6.2. The minimum length needed to assure overall stability of the wall varies slightly with the anchor level and anchor type. When the anchor is placed lower, less embedding depth is needed. Length of the primary members varies from 20 to 70 [cm] between variants when a different anchor type is applied. To reduce costs, the thickness of the primary element is varied in the length. At the level of the maximum moment ±10 [m], a thicker element is applied (the thickness showed in the next tables are at M_{max}).

Table 6.1 Dimensions Option A1: MV-piles

	MV-pile			
	+2.2 m NAP	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Length primary elements [m]	47.2	46.6	46.2	45
Diameter primary members [m]	2.8	2.8	2.8	2.8
Thickness primary elements [mm]	55	50	43	34
MV pile c.t.c. distance [m]	1.8	1.5	1.3	1.3
MV-pile length [m]	75	72	70	67

Table 6.2 Dimensions Option A2: Anchor wall

	Anchor plate			
	+2.2 m NAP	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Length primary elements [m]	47	46.7	46.5	45.7
Diameter primary members [m]	2.9	2.9	2.9	2.9
Thickness primary elements [mm]	50	44	43	33
Anchor rod c.t.c. distance [m]	0.7	0.7	0.7	0.6
Anchor plate length [m]	7	8.5	9	9
Distance from anchor plate to quay wall [m]	55	55	55	55

The variation of embedding depth by using different anchors can be explained by the fact that MV-piles are much stiffer than anchor rods. The embedding depth is determined with D-sheet. In D-sheet, the displacement of the wall influences the distribution of pressures against the wall. Anchors are in fact modelled as a discrete spring (one degree of freedom). The spring stiffness is determined by the cross-section, length and elasticity modulus of the anchor. An MV-pile has a higher spring constant than an anchor rod. A high stiffness leads to a deformation as shown in Figure 6.1 (left), here the passive horizontal pressure acts against the wall from the surface down to the anchor position. This phenomenon leads to high anchor forces, smaller bending moments and a force on the upper side of the wall (with respect of the anchor) that reduces the needed embedding depth.

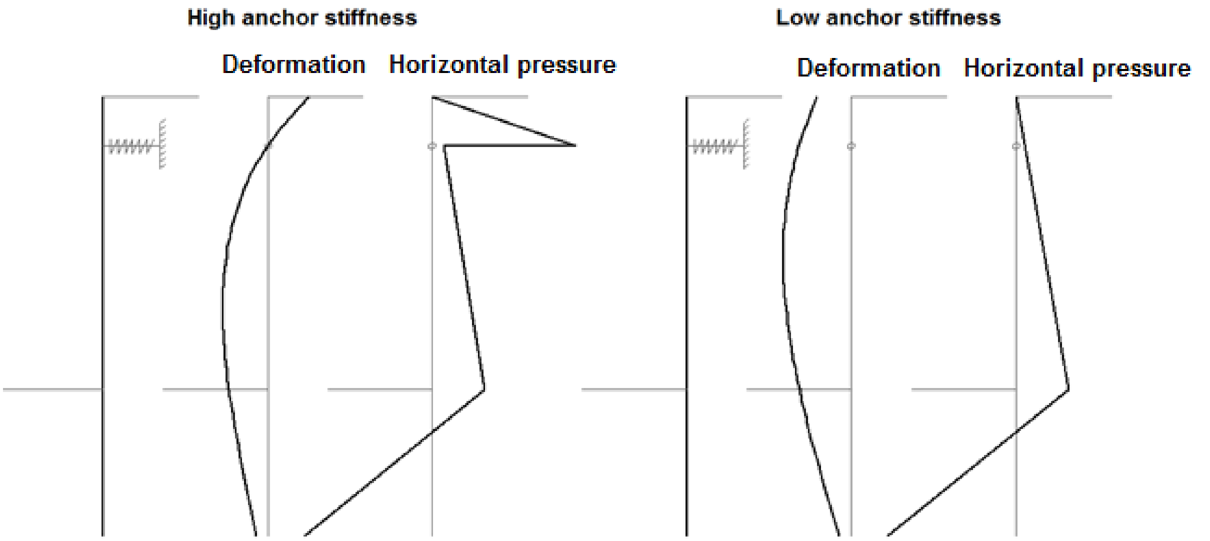

Figure 6.1 Deformation and resultant soil pressure with high and low anchor stiffness

The thickness of the primary elements reduces with the anchor depth as expected. By lowering the anchor the span decreases (distance between anchor and bottom wall). This result in a lower moment (smaller primary elements) and an increase in anchor force.

6.2.2 Anchor force and bending moments

The calculated forces in D-sheet and PLAXIS are shown in Table 6.3 and Table 6.4.

Table 6.3 Maximum moment and anchor force Option A: MV-pile

	MV-pile			
	+2.2 m NAP	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Max. Moment D-sheet [kNm/m]	20382.1	18107.1	15630	10245.1
Max. Anchor force D-sheet [kN/m]	2808.8	3257	3714	3879
Max. Moment PLAXIS [kNm/m]	22890	19780	17190	12222
Max. Anchor force PLAXIS [kN/m]	3351	3401	3443	3654

Table 6.4 Maximum moment and anchor force Option A2: Anchor plate

	Anchor plate			
	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Max. Moment D-sheet [kNm/m]	20657.6	17091	17080	12728
Max. Anchor force D-sheet [kN/m]	1757	1840	1940	2232
Max. Moment PLAXIS [kNm/m]	21190	18750	18130	11880
Max. Anchor force PLAXIS [kN/m]	1757	1840	1940	2232

These tables show that the anchor level has some influence in the distribution of forces. A lower anchor leads to lower bending moments due to the decrease in span. However, higher anchor forces are expected because the distance from surface to anchor increases and the horizontal pressure that is acting above it has to be taken by the anchor (see Figure 6.3). These moments and anchor forces are further discussed in Chapter 7 together with the results of Option B.

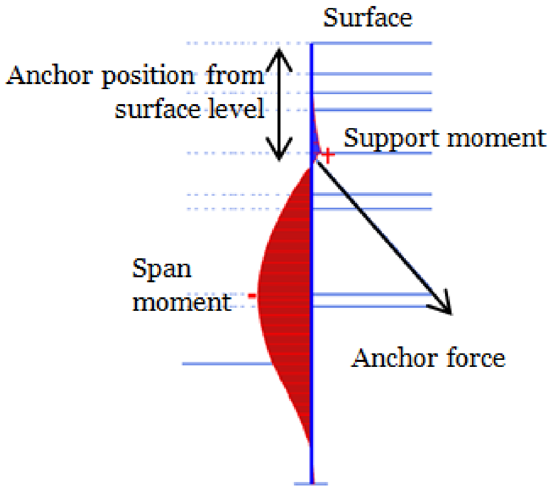

Figure 6.2 Description of terms used in the discussion

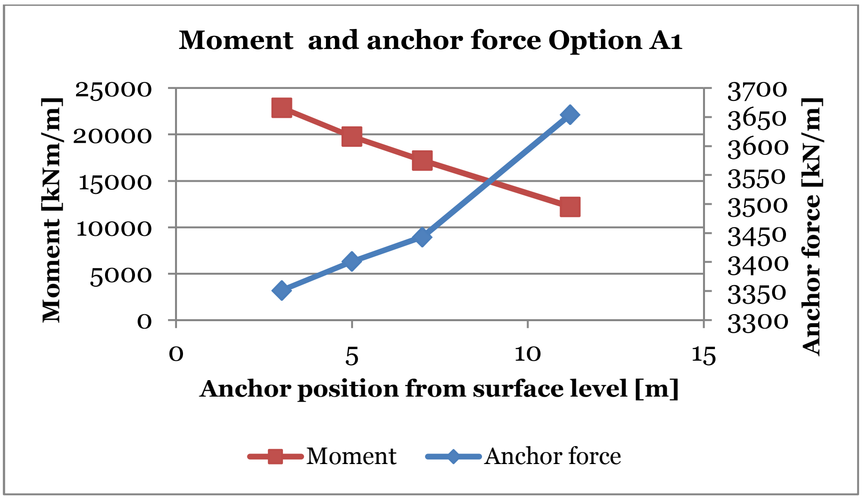

Figure 6.3 Option A1, Moment and anchor force at several anchor levels

6.2.1 Displacements

The displacements calculated with PLAXIS for both anchor types are shown in Table 6.5. In a bulk terminal the limits in displacement are not too strong. Both anchor types lead to acceptable displacements.

Option A1 (MV-pile as anchor) leads to lower displacements than Option A2 (anchor plate as anchor). The reason is that the displacement of Option A2 is not only determined by the elongation of the anchor rod, but also by the horizontal displacement of the anchor plate. Other aspect to take into account is that MV-piles have a higher axial stiffness than anchor rods, leading in general to lower horizontal displacements of the wall. In Chapter 7 the displacements are compared with the ones calculated for Option B.

Table 6.5 Displacements in the SLS calculated with PLAXIS in [m]

Anchor level	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
MV-pile	0.33	0.30	0.29	0.32
Anchor plate	0.42	0.41	0.36	0.46

6.2.2 Results D-sheet and PLAXIS

In Figure 6.4 the ratio between the maximum moment between D-sheet and PLAXIS is given. PLAXIS calculates in most cases higher bending moments. The reason is that D-sheet uses the Boussinesq principle to determine the horizontal pressure due to strip loads that are separated from the wall (annex F), literature showed that the resultant of the horizontal pressure calculated with Boussinesq is lower than the one measured in a field tests described by Bowles [19] (this is explained in more detail in Chapter 7 together with the results of Option B).

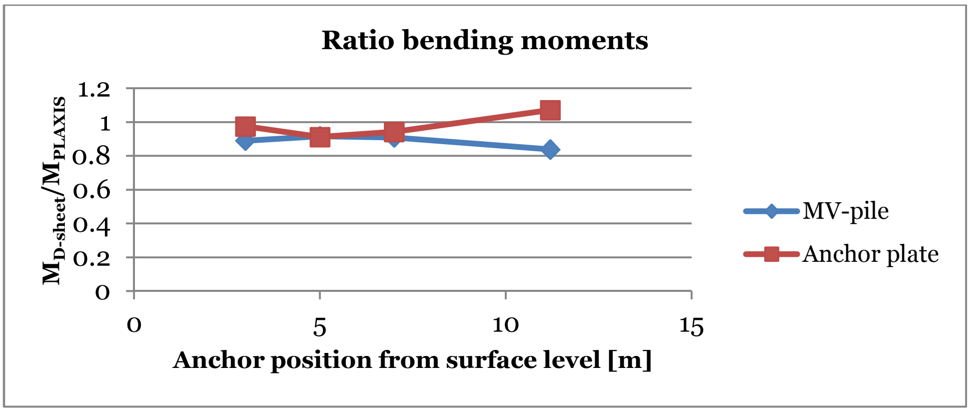

Figure 6.4 Ratio of moments between D-sheet and PLAXIS

In Figure 6.5 the ratio between the anchor force in D-sheet and PLAXIS is given. The anchor forces calculated in D-sheet, in case of the anchor plate, show a good approximation (<10% difference). The reason is that the anchor is placed almost horizontally and no bending of the anchor rod is taken into account in the model, the anchor force is only determined by the elongation of the anchor.

In case of an MV-pile, the relation between the anchor forces calculated in D-sheet and in PLAXIS differs more. Figure 6.5 shows the ratio between the anchor force in D-sheet and in PLAXIS. For high anchor levels the forces in D-sheet are smaller than the forces in PLAXIS. For lower depths the force in D-sheet is higher than in PLAXIS. In PLAXIS active horizontal pressure is present in the whole wall, while in D-sheet passive horizontal pressure is present between the surface and the anchor level (as explained in Figure 6.1, leading to higher anchor forces. This is also reflected in the ratio of moments shown in Figure 6.4, at deeper anchor levels the difference in the moment calculated with D-sheet increases with respect of the ones calculated with PLAXIS. In D-sheet, due to the development of passive pressure above the anchor level higher support moments (at the position of the anchor) are present, this support moment reduces the maximum moment at the span of the wall, increasing the difference in with the moment calculated with PLAXIS.

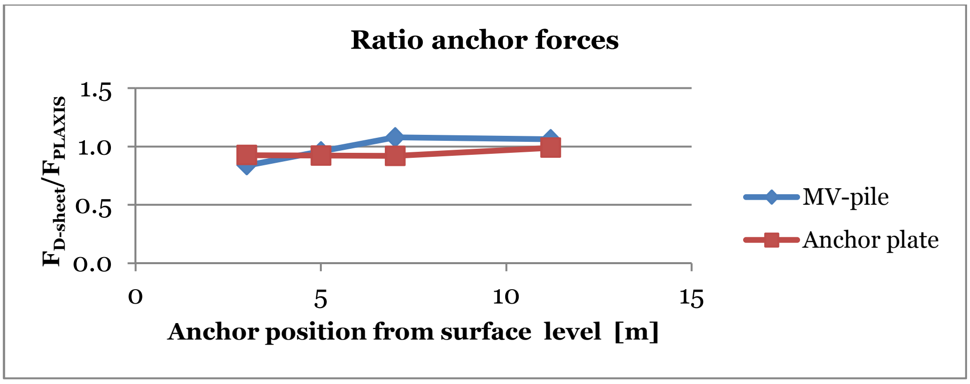

Figure 6.5 Ratio of anchor force between D-sheet and PLAXIS

6.2.3 Anchor design

The length of the MV-piles and anchor plates where first designed to have enough strength. MV-piles of 64.5 meters long at 1.3 to 1.8 meters c.t.c. distance showed enough holding capacity, the same in case of anchor plates of 7 to 9 meters at a distance of 53.5 meters. In a later phase of the design loop, the stability of the anchor showed to be decisive for the length of the MV-piles and position of the anchor plates (the stability is verified with the KRANZ stability criteria [6]).

7. OPTION B: QUAY WALL WITH RELIEVING STRUCTURE

In this chapter the calculation results of Option B are discussed. D-sheet is only able to model vertical walls, without any possibility to introduce other type of structure such as the relieving structure. In PLAXIS on the other hand a much wider field of modelling possibilities is present. With both methods the designer has to be aware of the limitations and differences in the results.

First an explanation about the calculation method used for Option B is given. The results of the calculations of Option B (with PLAXIS and D-sheet) are presented in section 7.2.

7.1 Calculation method

The same design steps as mentioned in section 4.7.4 are followed to calculate the designs. First, the relieving structure is modelled as a framework, at this stage the anchor length and position is not known, based on the design of Option A an anchor force is assumed and the anchors are designed with this force as reference. With the framework model, the reaction forces (Vertical and horizontal) at the position of the connection between the relieving structure and the retaining wall are calculated. Figure 7.1 shows the schematization for the case that the bearing piles are placed in a slope.

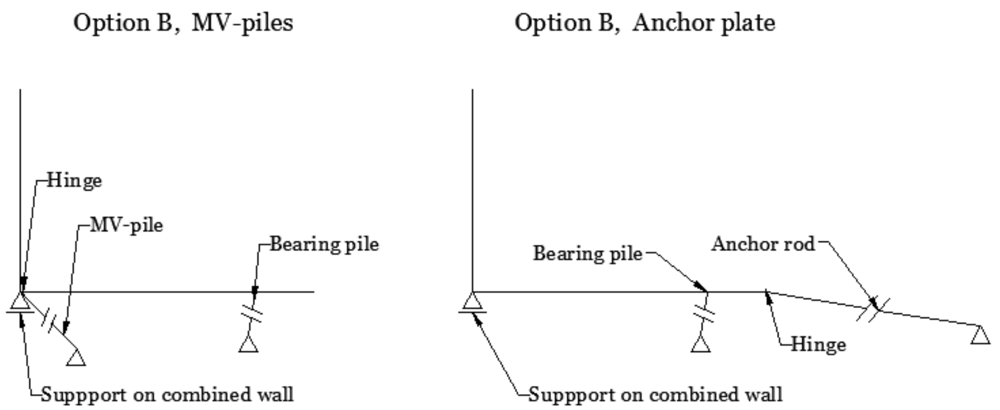

Figure 7.1 Framework schematization of the relieving structure

The vertical and horizontal resultant forces of the framework calculation are used as input in D-sheet, this is done to take into account the influence of the relieving structure on the retaining wall. In the framework analysis the soil is modelled as a distributed load. Figure 7.2 show the loads taken into account in the framework analysis.

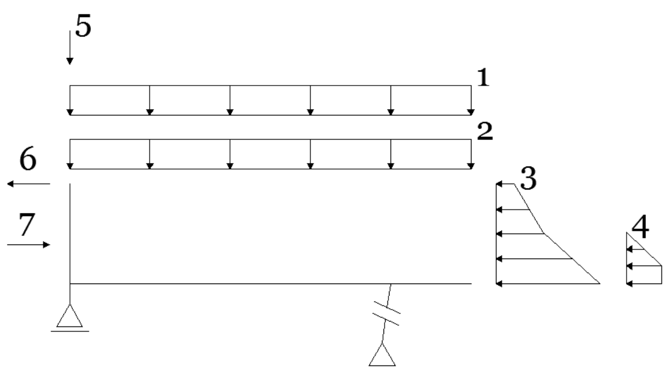

Figure 7.2 Loads considered in the framework analysis

The loads considered in Figure 7.2 are:

1. Surface load;
2. Soil on top of the relieving structure;
3. Horizontal pressure due to soil on the relieving structure, here the assumption is made that the neutral coefficient of horizontal earth pressure is used to calculate the horizontal loads;
4. Water pressure difference;
5. Crane load;
6. Bollard load;
7. Fender load.

The results and more information about the framework calculation are given in Annex D.

The whole system of the superstructure (relieving structure and anchor) is modelled as a horizontal spring in D-sheet. The spring stiffness is determined by the anchor and the bearing piles, which restrain the horizontal displacement. Only axial stiffness is taken into account, which means that stiffness contribution due to bending of the primary elements of the wall, bending of the bearing piles and anchor is neglected. In this stage extra attention has to be given to the model of the surface of the quay in D-sheet. An abrupt change in surface level lead to a high estimation of the bending moment in the retaining wall, in the calculation with D-sheet the surface level is set at the level of the bottom of the relieving structure, the soil behind it is introduced as a vertical surface load. With the forces calculated in D-sheet an estimate of the dimensions and position of the anchors (in case of the anchor plate) can be made.

After estimating the dimensions with the beam on elastic foundation method, the designs are modelled in PLAXIS. The moments in the wall and anchor forces for the serviceability limit state are compared. At this stage some optimization can be made by reducing the diameter of the primary members, this is important for the calculation of the costs. The final design is then checked for overall stability, Kranz stability of the anchor, enough bearing capacity of the primary elements and resistance against local buckling. For details about these calculations along with a calculation of the reinforcement of the relieving structure for one case the reader is referred to Annex D.

7.2 Results Option B

7.2.1 Dimensions

A summary of the dimensions of the design considering both anchor types is given in Table 7.1 and Table 7.2.

Table 7.1 Dimensions Option B1

Length relieving structure [m]	**10**			**20**		
Level relieving structure [m NAP]	**+2.2**	**-1.8**	**-6**	**+2.2**	**-1.8**	**-6**
Length primary elements [m]	45.5	43.6	37	45	40.3	36
Diameter primary members [m]	2.8	2.6	2.5	2.8	2.7	2.6
Thickness primary elements [mm]	48	41	31	38	33	20
MV pile c.t.c. distance [m]	2	2	2	2	2	2
MV-pile length [m]	72	73	73	70	71	71

Table 7.2 Dimensions Option B2

Length relieving structure [m]	10			20		
Level relieving structure [m NAP]	+2.2	-1.8	-6	+2.2	-1.8	-6
Length primary elements [m]	45.5	43.6	37	45	40.3	36
Diameter primary members [m]	2.8	2.7	2.6	2.7	2.5	2.3
Thickness primary elements [mm]	48	42	36	39	30	25
Anchor rod c.t.c. distance [m]	0.85	0.75	0.7	1	1	1
Length anchor rod [m]	50	50	50	40	40	40

A clear reduction of thickness and diameter of the primary members (steel tubes) is observed. However due to the high bending moments present in the wall, the local buckling criteria is decisive, which means that a thicker primary element is needed to limit the stresses in the wall to prevent local buckling. The reduction in the dimensions of the primary elements is important regarding the costs, this is further discussed in Chapter 8.

7.2.2 Anchor force and bending moments

The moments and anchor forces of Option B for both anchor types and a floor length L (L= 10 [m] and L= 20 [m]) calculated with D-sheet and PLAXIS are shown in Table 7.3 and Table 7.2.

Table 7.3 Moments and anchor forces Option B: MV-pile

Length relieving structure	L=10 [m]			L=20 [m]		
Level relieving structure	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Max. Moment D-sheet [kNm/m]	18480	13155.3	9039	15370	9949	5828
Max. Anchor force D-sheet [kN/m]	1837.0	2008.0	2636.0	936.0	1596.0	1271.0
Max. Moment PLAXIS [kNm/m]	18360	12400	9730	13800	8120	5142
Max. Anchor force PLAXIS [kN/m]	2699	3081	3538	1913	1987	2429

Table 7.4 Moments and anchor forces Option B: Anchor plate

Length relieving structure	L=10 [m]			L=20 [m]		
Level relieving structure	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Max. Moment D-sheet [kNm/m]	16201.0	13487.4	8922.0	15470.6	9944.6	5907.0
Max. Anchor force D-sheet [kN/m]	1357.7	1447.7	1860.4	712.0	1137.0	897.0
Max. Moment PLAXIS [kNm/m]	17830	13370	10190	13990	8397	6665
Max. Anchor force PLAXIS [kN/m]	1773	1851	2129.8	1153	1157.5	1557

As in Option A, the moments decrease as the anchor (also the bottom of the relieving structure) is placed at a lower level. Figure 7.3 and Figure 7.4 show the maximum value of the moment in the wall for both anchor types in the serviceability limit state. In this figure a rather linear decrease of the moment in the wall calculated with D-sheet can be observed. In PLAXIS however, a less steep line is found between 7 and 11.2 meters. The reason is that a certain depth the relieving action of the superstructure does not have much effect on the wall. This also depends on the length of the relieving structure, when the length increases the relieving action loses its effect at an earlier height. D-sheet showed some discrepancy at estimating the horizontal loads due to a surcharge separated from the wall, this is further discussed in this chapter.

The moments in the wall between the option with MV-piles and the option with Anchor wall do not differ much as expected. The length on the wall, the horizontal stiffness of the system and the horizontal pressure against the wall determine the moments which in both cases approach.

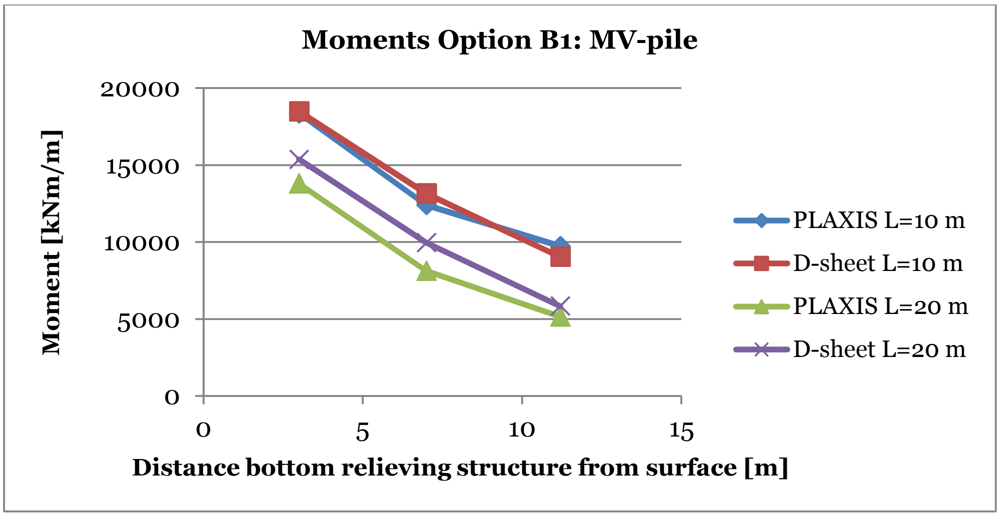

Figure 7.3 Moments in function of the depth. Option B1: MV-piles

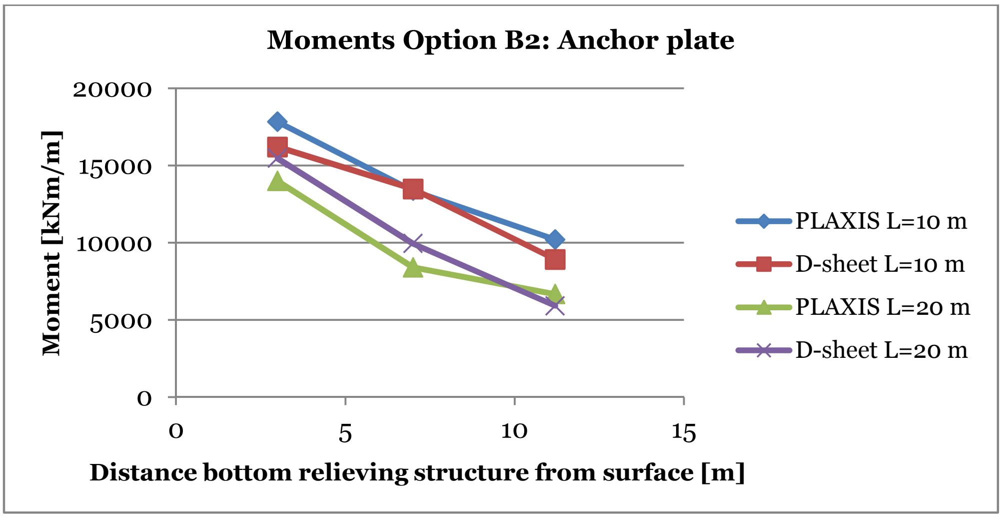

Figure 7.4 Moments in function of the depth Option B2: Anchor plate

The calculated anchor forces for both anchor types are shown in Figure 7.5. Here a discrepancy in the tendency of the anchor forces for the case of a relieving structure of 20 meters was found. Because of this discrepancy extra calculations have been made to ad points in between the heights of the relieving structure.

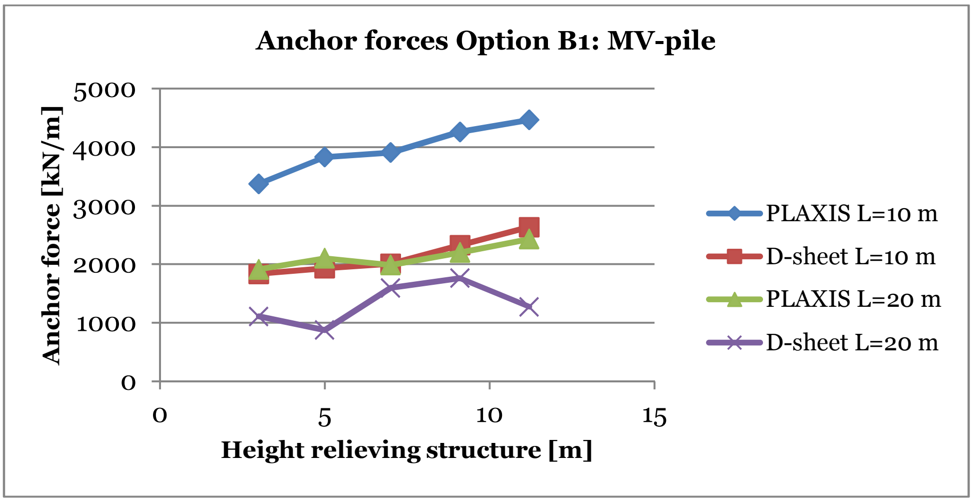

Figure 7.5 Anchor forces in function of the height of the relieving structure Option B: MV-piles

It is clear that in D-sheet and PLAXIS the anchor forces tend to increase at a higher depth, this is also the case for the calculation with PLAXIS for a relieving structure of a length of 20 m. When the relieving structure is placed deeper, more horizontal pressure due to the surface load act's on the relieving structure, which has to be resisted by the anchor and bearing piles.

The results of the calculation with D-sheet for a relieving structure with a length of 20 meters do not show a clear tendency. It is still inconclusive why the anchor forces show such variation (first a decrease in anchor force, then an increase and at the deepest level a decrease). The possible cause lies in the estimation of the horizontal pressure due to surcharges that lie near or outside the straight sliding plane of the wall. The differences obtained with D-sheet and PLAXIS are further discussed in this chapter.

7.2.3 Displacements

The displacements calculated with PLAXIS are shown in Table 7.5. As explained in the case of the moments and anchor forces, the displacements reduce when the length and depth of the relieving structure are increased.

Table 7.5 Displacements Option B

Length of the relieving structure [m]	**10**			**20**		
Anchor level [m NAP]	**+2.2**	**-1.8**	**-6.0**	**+2.2**	**-1.8**	**-6.0**
Option B1: MV-piles	0.29	0.23	0.21	0.23	0.21	0.18
Option B2: Anchor plate	0.30	0.23	0.22	0.21	0.16	0.14

7.3 Discussion engineering aspects Option A and Option B

The displacements calculated with PLAXIS for both cases, with and without relieving structure, showed significant differences. When a relieving structure is applied the displacement of the retaining wall is reduced. As a result of the reduction of the horizontal pressures in combination with a higher horizontal stiffness due to the relieving structure, the resulting displacement of the wall is 30 to 70% less than the case when the relieving structure is absent.

In the same way, the maximal moment acting on the retaining structure is reduced considerably. Lower moments in the wall leads to smaller elements and a more accessible construction (big elements require bigger cranes and more expertise to handle them). The reduction in moments depends on the level of the anchor and the length of the relieving structure. When the anchor is placed at a lower level the moments tend to decrease, by extending the length of the slab of the relieving structure a strong reduction in moments is obtained. The calculations performed showed a reduction of bending moments of about 75% between the highest anchor level in Option A and the biggest relieving structure in Option B (Figure 7.6).

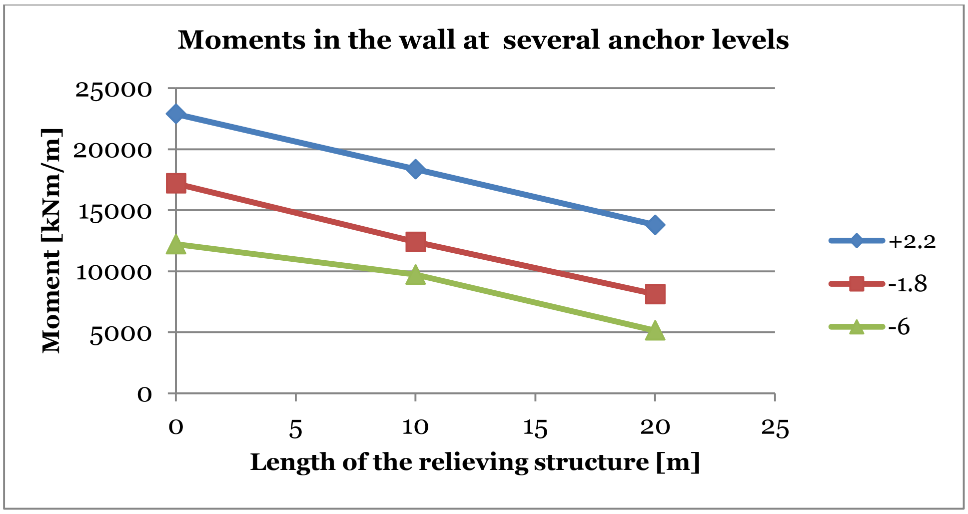

Figure 7.6 Moments in function of the length of the relieving structure MV-pile as anchor

However, the calculations showed that when the relieving structure is present, the reduction of the anchor forces is less strong than the reduction of bending moments. When the relieving structure has length of 10 [m] the reduction varyes between 1% and 19%, by a length of 20 [m] the reduction increases, namely 30% to 40% (comparing it to the case that the relieving structure is absent and considering the same anchor level). Longer relieving structures lead to a higher reduction of pressures on the retaining structure, hence lower anchor forces. Between the highest anchor level in Option A and the biggest relieving structure in Option B the reduction of the anchor force is about 50%. This reduction gives the possibility to use smaller anchor cross-section, shorter anchor lengths or bigger center to center distance between the anchors.

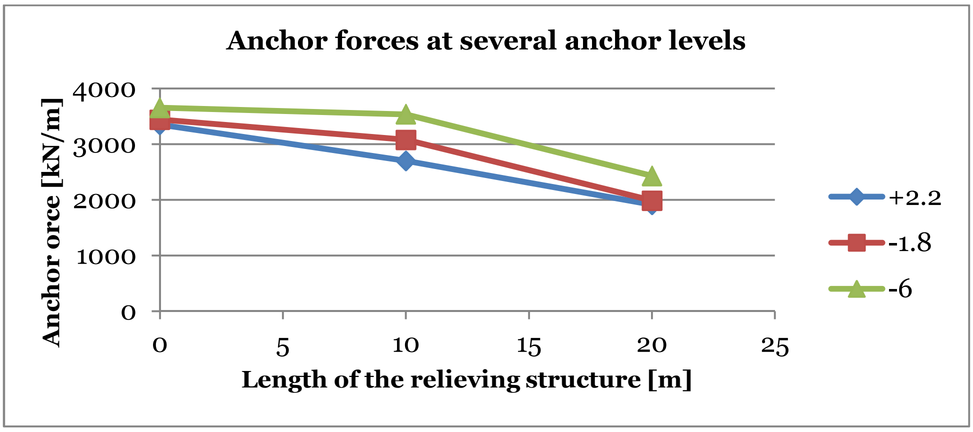

Figure 7.7 Anchor forces in function of the length of the relieving structure MV-pile as anchor

Another important engineering aspect that needs to be considered is that a relieving structure has more capacity of redistribution of forces. In the past, the most probable cause of failure in a quay wall was the anchor. When a relieving structure is applied and an anchor fails, due to the high stiffness of the slab of the relieving structure a great capacity of redistribution of forces is present. The extra loads due to the failure of the anchor would be more conveniently distributed to the adjacent bearing piles and anchors, leading to a higher safety margin and possibility to take actions to prevent failure of the whole structure.

7.4 D-sheet and PLAXIS

The ratio of the moment between D-sheet and PLAXIS for option B is given in Figure 7.8 and Figure 7.9.

In Figure 7.8 can be seen that the moments in the SLS calculated with D-sheet vary slightly from the ones calculated in PLAXIS. For a relieving structure length of 10 meters, the moments calculated with D-sheet have a difference of ±9% and for a length of 20 meters the difference is about +20%.

In Figure 7.9 the same tendency can be observed for the case of an anchor plate and a relieving structure length of 10 meters. When the relieving structure length increases to 20 meters, the moments calculated with D-sheet are higher than the ones calculated with PLAXIS.

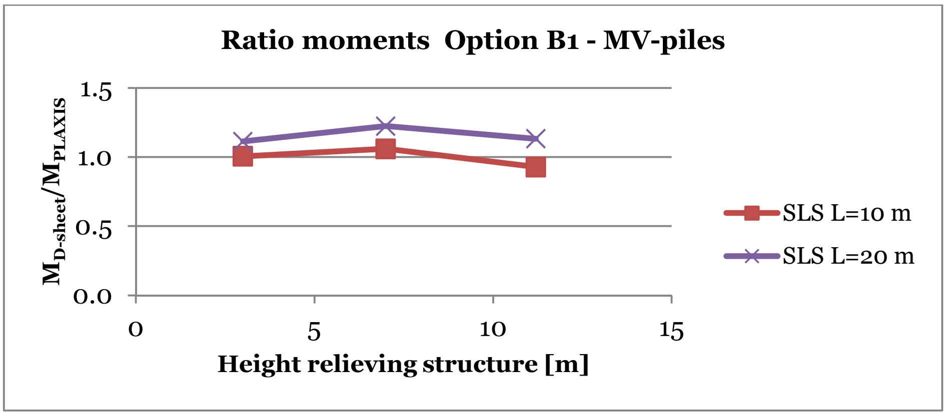

Figure 7.8 Ratio of moments for a relieving structure of length L. Option B - MV-piles

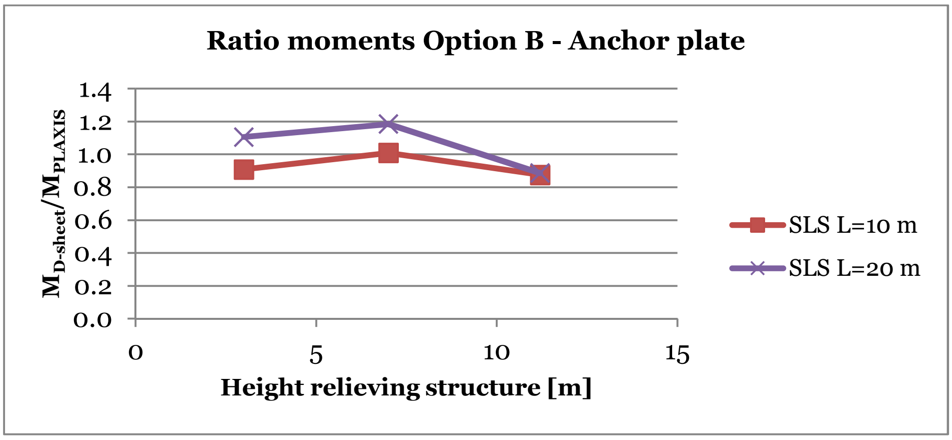

Figure 7.9 Ratio of moments for a relieving structure of length L. Option B - Anchor plate

In general, the moments calculated with D-sheet and PLAXIS do not differ too much. When the relieving structure is absent the moments calculated with D-sheet differ about ±10% (Figure 6.4) than the ones calculated with PLAXIS, conversely, when the relieving structure is present and has a length of 20 [m] the calculated moments with D-sheet are 0% to 20% higher than PLAXIS (Figure 7.8 and Figure 7.9).

Regarding the anchor forces, the calculations showed difference in the anchor forces calculated with D-sheet and PLAXIS for both Option A and Option B (e.g. quay wall with and without relieving structure respectively). In Option A, the calculations with D-sheet showed in most cases a lower anchor force than the one calculated with PLAXIS (Figure 6.5). In the case of an MV-pile the anchor force calculated with D-sheet at the highest anchor level is 20% lower than the one calculated with PLAXIS, as the anchor level is lowered the anchor force increases to a value that is 10% higher than the one calculated with PLAXIS (the reason of this increase is horizontal pressure development from surface to anchor level, this is explained in Chapter 6).

In the case of Option B the difference of the calculated anchor forces between D-sheet and PLAXIS is even higher. One of the problems of calculating Option B with D-sheet is that the relieving structure must be calculated separately with a framework analysis. The resultant vertical and horizontal force obtained from the framework analysis serves as input in the D-sheet calculation, where the relieving structure is modelled as a spring. In PLAXIS however, the relieving structure can be explicitly integrated in the model.

The results calculated with D-sheet in the case of a relieving structure with a length of 10 [m] are 30% to 35% lower than with PLAXIS (Figure 7.5), which means that the force in PLAXIS is a factor 1.4 to 1.55 higher than the one calculated with D-sheet. In the case of a relieving structure of 20 [m] de calculated anchor force with D-sheet does not follow a clear trend with increasing depth of the relieving structure, however the calculated force is 20% and in some cases 60% lower than with PLAXIS (see Figure 7.5, here the calculated anchor force with PLAXIS is a factor 1.25 to 2.5 higher than the one calculated with D-sheet).

The most probable reason of this difference lies in the fact that D-sheet makes an underestimation of the horizontal pressure due to surcharge loads separated from the retaining structure. The horizontal pressure due to a surcharge load is approximated with an equation after Boussinesq [8]. Boussinesq developed this equation based on elastic theory.

In the past, it was proven with field tests that the Boussinesq approach for horizontal pressures due to point loads gave an underestimation of the resultant force against the retaining structure (Bowles, 1997 [20]). The resultant force measured in these field tests were equivalent to the force calculated with Boussinesq near the retaining structure, this difference grew as the load was placed further from the retaining structure. The maximum difference was achieved at the intersection between the straight sliding plane and the surface, at this point the measured forces were a factor 1.55 higher than the ones calculated with Boussineq approach.

It is possible to divide the strip load in several point loads and integrate the Boussinesq equation to obtain an expression for the horizontal pressure of strip loads separated from the retaining wall (this is the case of D-sheet). Because of the uncertainty of the date and circumstances of the field test described by Bowles (1997), more literature review was carried. A scientific paper published by the Islamic Azad University [20] shows similar results as the ones described by Bowles. The paper describes a research carried out with PLAXIS to validate the Boussinesq approach for strip loads. The resultant force calculated with PLAXIS was a factor 1.5 higher than the resultant calculated with Boussinesq.

Taking these two literature reviews and the results obtained in this thesis investigation into account, one can conclude that there are clear discrepancies between the anchor forces calculated with D-sheet and PLAXIS and that the problem lies most probably in the approximation of horizontal pressures due to strip loads developed by Boussinesq. If the designer is not aware of this underestimation, the consequences in the design can be big due to an under dimensioning of the anchors.

7.5 Surcharge modelling in PLAXIS

When dealing with bulk terminals the surcharge on the quay can also be modelled as soil. Figure 7.10 shows the surcharge load modelled as iron ore with an angle of internal friction of 40 degrees and a dry weight density of 22 [kN/m3].

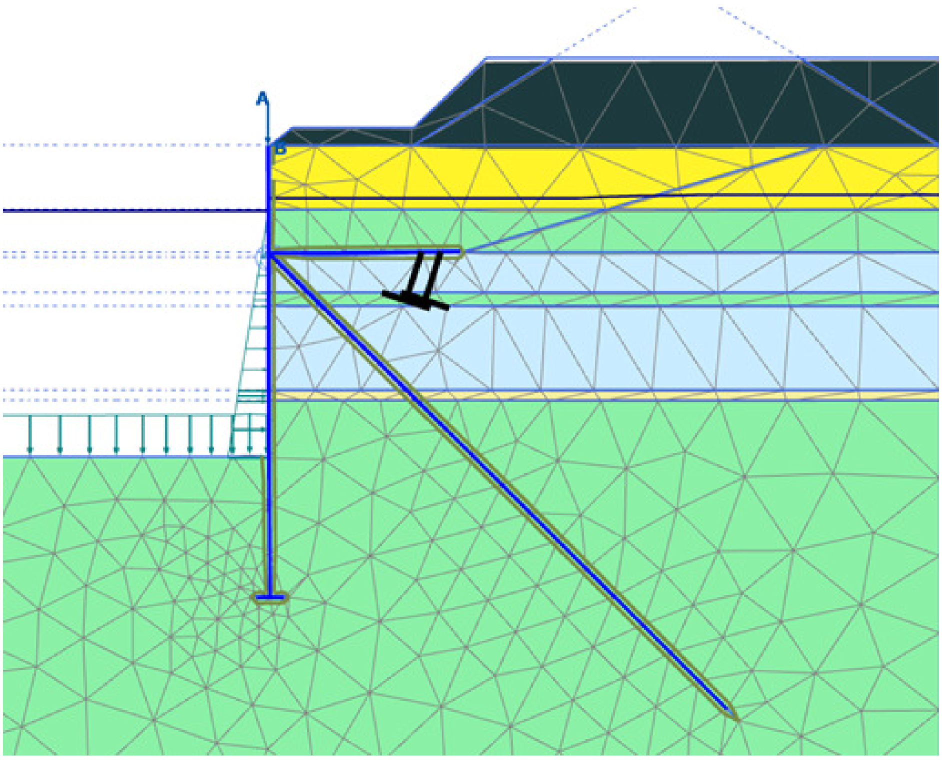

Figure 7.10 Surface load modelled as Iron ore

The anchor force calculated with this model is equal to 2548 [kN/m], which is 6% higher than the load calculated with the iron ore modelled as a uniform load. The reason for this increase is the shear force developed in between the iron ore and the sand layer (see Figure 7.11). When the surcharge is modelled as a load, only vertical forces are introduced, however when the designer deals with bulk material not only vertical forces are present, but also horizontal loads.

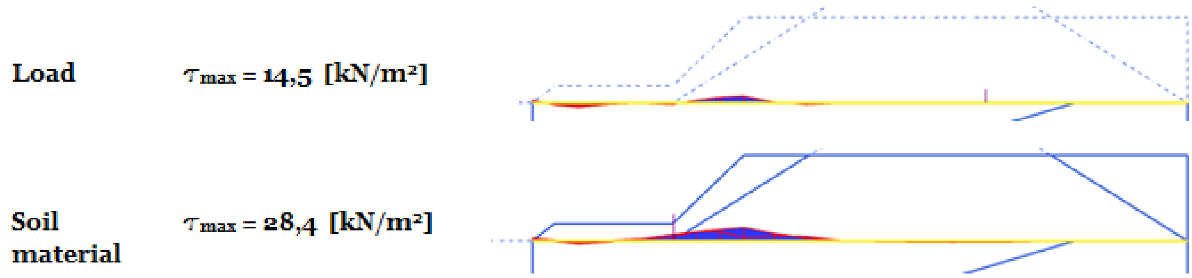

Figure 7.11 Shear stresses for the case of a surcharge modelled as load and as load material

To illustrate this better the surcharge is modelled as an equivalent "triangle" load, here the shear stresses are more clear (see Figure 7.12). However this difference is lower, the difference in anchor forces between both cases is equal to 4% (3150/3020=1.043).

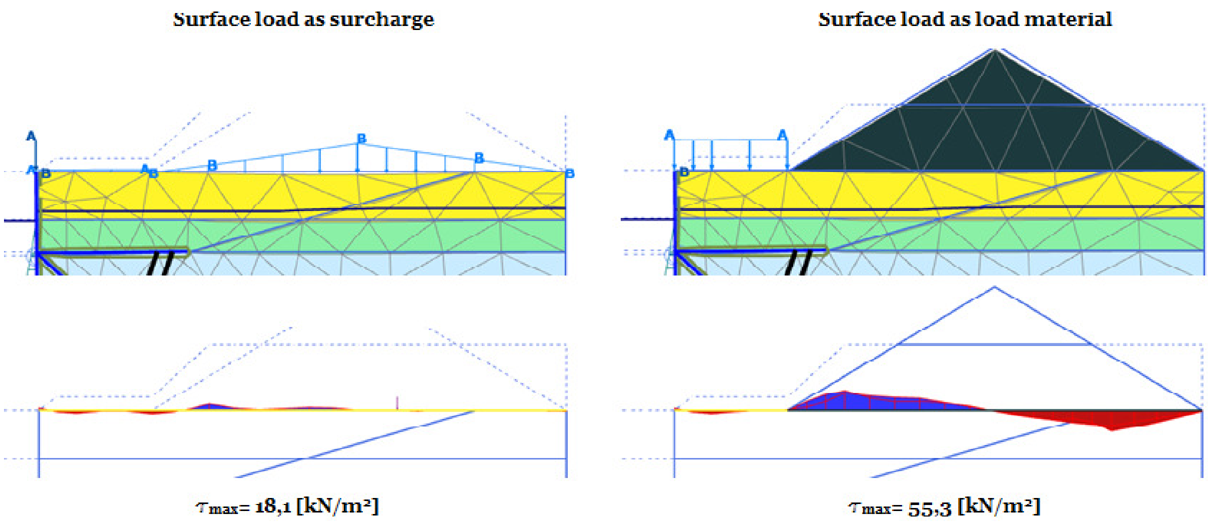

Figure 7.12 Shear stresses due to a triangular surcharge modelled as load and soil material

The calculated moments with the two models showed a difference of less than 1%. The anchor forces calculated with the surcharge modelled as soil are in the range of 6% higher than the ones calculated with a non-uniform strip load.

When the interface of the iron ore layer and the sand layer is investigated a shear stress development is present. Iron ore tends to displace horizontally and the friction with the upper sand layer leads to shear stress and becomes the possible cause of the increase in anchor force.

8. COST ESTIMATION

In this chapter the summary and main assumptions of the cost calculations are given, for more details the reader is referred to annex E.

The purpose of performing a cost calculation of the options considered is to include cost as one of the comparison criteria. Costs are an important part of the overall decision making process as to whether choose for a quay wall design with or without a relieving platform.

The cost of a project is strongly driven by the site conditions, market situation and commercial considerations. The final cost of a structure consists not only by the ones incurred during the construction but also during the service life (inspection and maintenance). The cost of a structure can be divided in direct costs (civil works, earth works, foundation, transportation, installation and man-hours), indirect costs (management and supporting staff, facilities, storage, site preparation, design & site engineering) and additional costs (risk, profit and interests). The costs calculated in this thesis are only for civil works (namely Steel and concrete) and earth works. These costs represent the biggest share of the total costs.

To determine the costs several assumptions have been made being the most important that there is enough space for machinery and facilities. Also that the construction is done in a dry building pit (for other assumptions see annex E).

Another important aspect is that most anchor levels are placed below ground water level, which means that dewatering is needed to allow construction in a dry building pit. The costs of a drainage system are normally given in €/m^3, thus the volume of water extracted during the construction time is needed. Several cost estimates of projects in the archive of the Public works Rotterdam show that the costs of dewatering have a really low impact in the total costs (less than 1%). The costs of dewatering will not be taken into account for the comparison of the costs. To determine the costs of dewatering, the construction period, capacity and type of pump need to be known. The determination of these parameters will lead to more assumptions which for the purpose of this thesis can be neglected due to the low contribution to the total costs.

8.1 Price per element

The price for each element is determined with information of the Archive of the Public Works Rotterdam and PoR (Port of Rotterdam).

8.1.1 Steel

- Delivery steel primary elements*: 1500 €/ton
- Delivery overall steel: 1000 €/ton
- Installation primary elements: 300 €/m (euro per meter pile, diameter 2.8-2.9 m);
- Sheet pile elements**: 25 €/m (reference PU25, 2 or 3 pieces)
- Tubular piles anchor wall**: 100 €/m (piles around 1.4 m)
- Anchor rod**: 56 €/m (per meter rod, 75 mm diameter)
- MV-pile and grouting**: 250 €/m (per meter pile)

* Because of the extra ordinary dimensions (big diameter and thick elements), a higher steel price is used in the cost assessment.

** Installation costs.

8.1.2 Earth works

The prices of earthworks depend on the amount of soil that is moved, the time, the machinery is used, the purity of soil, transport of soil and the circumstances of the construction. When the space is limited (most of the time), temporary structures have to be used to make the excavation pit. Furthermore the machinery has to be able to reach the desired place. The assumption of enough construction space gives the possibility to excavate and apply slopes 1:2 in the excavation sides.

In the archive of the Port of Rotterdam an average value of 2.5 €/m^3 is given for the excavation and transport of soil (assuming clean sand). The same price can be used to calculate costs of refill.

8.1.3 Concrete

The price of a concrete structure is determined by the concrete, reinforcement and formwork. Depending in the number of repetitions costs can be safe in the formwork by applying a steel one (instead of wooden formwork). An integrated fixed price for 1 cubic meter of concrete for universal applications does not exist. The cost components as mentioned above are significantly influenced by project specific (local) conditions and conceptual choices made by engineers.

Nevertheless, overall prices per cubic meter in each country are used to have an indicator to identify trends and support selections in alternatives. The price of concrete including casting of concrete, formwork and reinforcement is set to 350 €/m^3 (Archive Public works Rotterdam). The same way the price for a bearing pile (in the case of option B) is assumed as 2400 €/piece (this includes material, delivery and installation).

8.2Option A – Quay wall without relieving structure

The elements considered in the calculation are shown in Figure 8.1. The costs of dredging at the port side are considered to be the same for all options and therefore are omitted in this cost estimation (only the variable costs are taken into account).

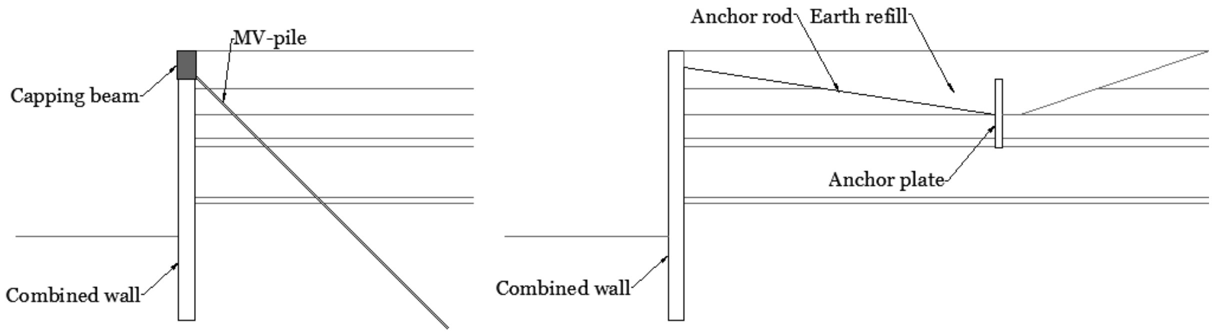

Figure 8.1 Elements considered in the cost estimation, Option A

8.2.1 Results

The results of the cost estimations for Option A are given in Table 8.1. For both options lowering of the anchor leads to lower costs (less steel, less costs). Figure 8.2 shows reduction of the cost by lowering the anchor level (anchor level +2.2 m NAP as reference cost). It is clear from the figure that the reduction in costs can reach a value of 23% by applying an anchor plate with a connection at a level of -6 m NAP. The reduction achieved with the MV-pile is less, namely 17%. In general, a quay wall with an anchor plate as anchor is less expensive than a quay wall with MV-pile (4-12%).

Table 8.1 Costs Option A for both anchor types in thousand euros per meter wall

Anchor level	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Option A1: MV-pile	€ 73.09	€ 68.81	€ 66.00	€ 62.16
Option A2: Anchor plate	€ 61.15	€ 56.30	€ 53.49	€ 48.04

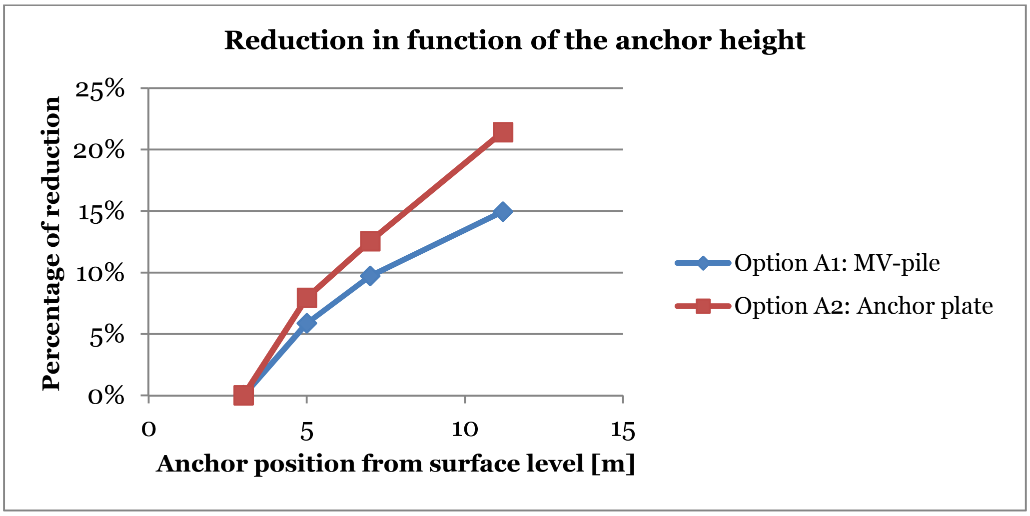

Figure 8.2 Reduction in costs Option A for each anchor type in function of the level of the anchor

These costs are calculated assuming a concrete capping beam that is needed for the connection of the MV-pile and the wall, as shown in Figure 8.3 (where H is equal to the distance from the surface to the anchor + 2 meters). The costs per meter for de designs with both anchor types are shown in Figure 8.4.

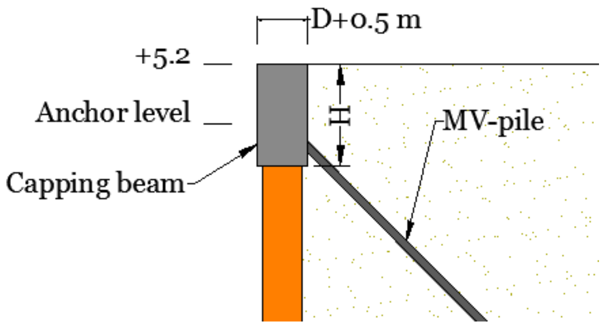

Figure 8.3 Assumptions capping beam for MV-piles

Figure 8.2 and Figure 8.4 show that when the connection of the anchor and the wall lies deeper, a reduction in costs is achieved. When the anchor lies deeper, a reduction in bending moments is achieved, hence thinner primary elements (steel tubes) are applied. The most important contribution of costs lies in the primary members, by reducing its thickness a high reduction in costs is achieved (15% to 21%).

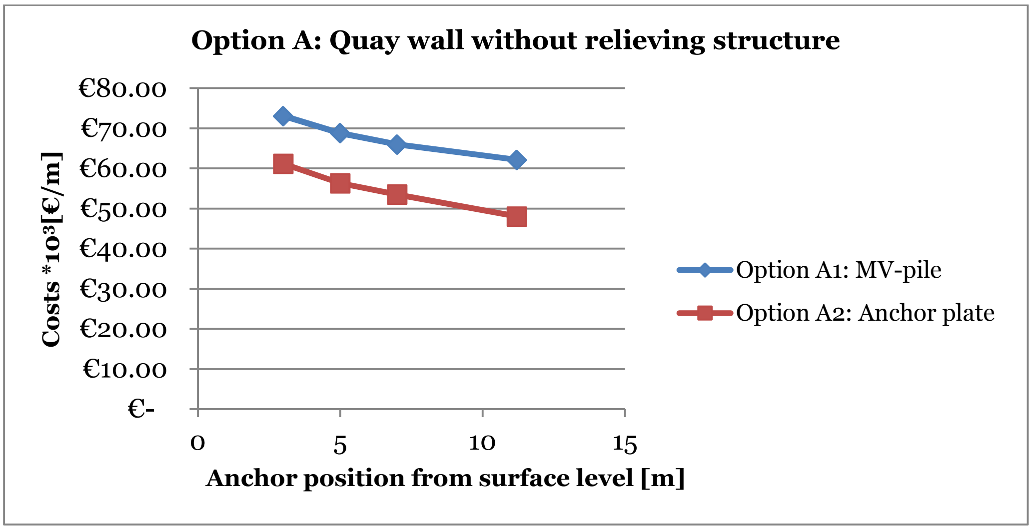

Figure 8.4 Option A - Costs per meter

In Figure 8.4 can be seen that in general a quay wall with an MV-pile as anchor is more expensive than a quay wall with an anchor plate as anchor. However, it has to be noted that a quay wall with an anchor plate is more susceptible for anchor failure than a quay wall with MV-pile. The anchor rods in the case of an anchor plate as anchor have a length of around 55 [m], which means that the high surface load is situated above it. High bending moments may occur due to settlement of the soil under the anchor rods, which can lead to anchor failure due to its low bending stiffness. There are measurements to prevent this failure, for example by applying a hinge connection that gives the possibility to follow soil settlements under the anchor rods. This measurement is quite expensive and would increase the costs of applying an anchor plate. Nevertheless the risk that the anchor rods will not follow the settlements is still there. When an MV-pile is considered this problem is less probable to happen, the MV-piles are places in an angle and the influence of high surface load will be considerably lower.

8.3Option B – Quay wall with relieving structure

The elements considered in the cost estimation of Option B are shown in Figure 8.5.

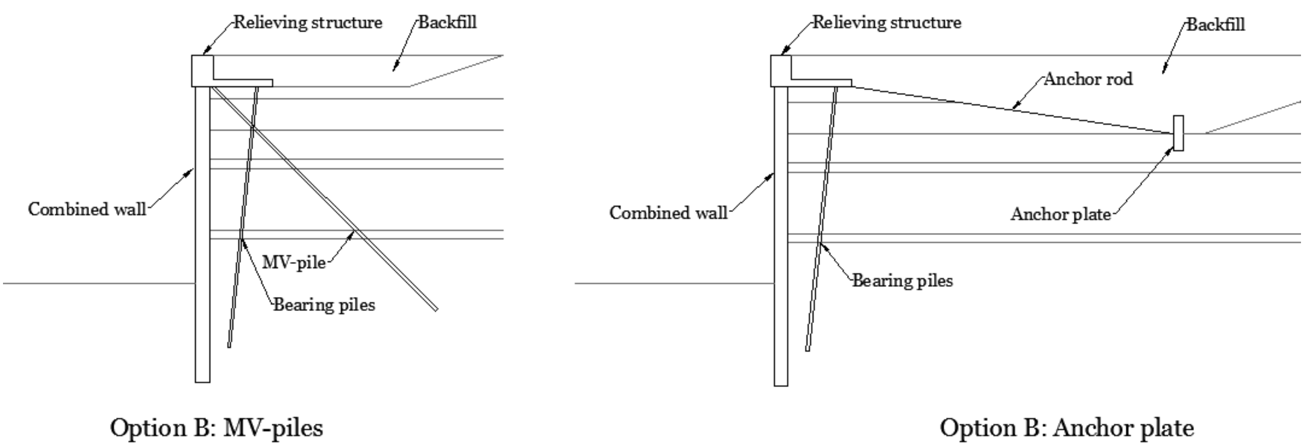

Figure 8.5 Elements considered in the cost estimation, Option B

8.3.1 Results

Table 8.2 Costs Option B for both anchor types and relieving structure length (L) in thousand euros per meter

Length relieving structure	**L=10 m**			**L= 20 m**		
Level relieving structure	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Option B1: MV-pile	€ 77.93	€ 71.54	€ 63.76	€ 78.72	€ 74.43	€ 66.34
Option B2: Anchor plate	€ 72.51	€ 69.16	€ 65.12	€ 72.23	€ 65.44	€ 63.21

The same case is obtained as in Option A. When an anchor plate is used instead of an MV-pile as anchor, the costs are in most cases less than when an MV-pile is applied, however here the same risk of anchor failure is present which would probably increase the costs of a quay wall with anchor plate. In the case of an MV-pile, the relieving structure limits the surface load acting on the anchor, reducing the risk of anchor failure by bending.

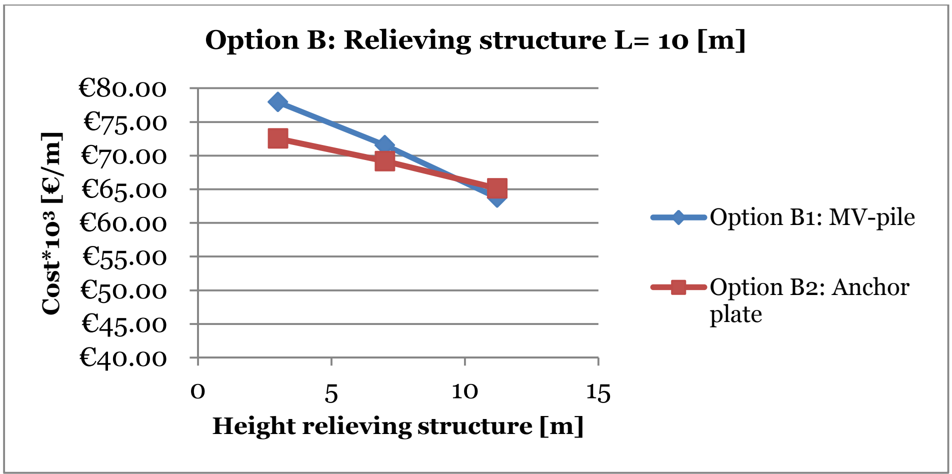

Figure 8.6 Costs Option B with a relieving structure length of L=10 [m]

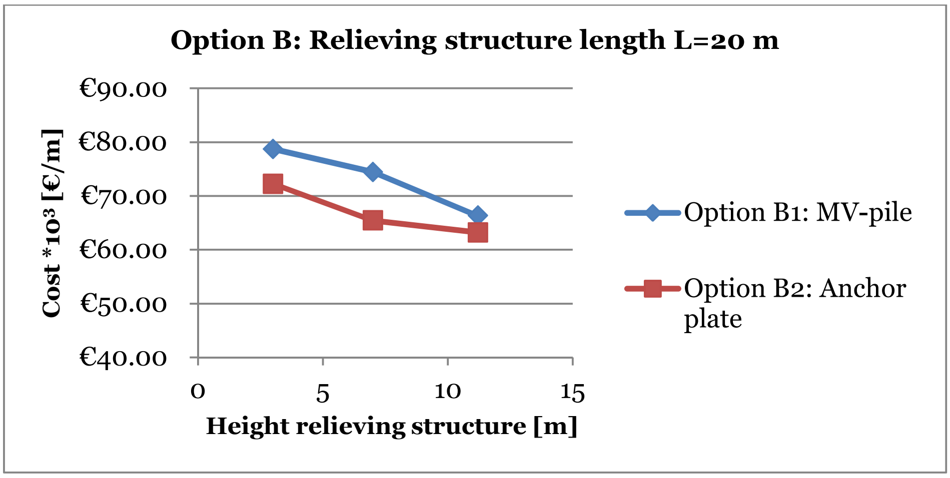

Figure 8.7 Costs Option B with a relieving structure length of L=20 [m]

8.4 Costs in function of the length of the relieving structure

As mentioned before, the calculated costs in this chapter consist of the direct costs of civil works, which means the costs of the steel combine-wall, concrete relieving structure, bearing piles and anchors (delivery of the material and installation of the elements). In Figure 8.8 and Figure 8.9 the costs in function of the length of the relieving structure are given. When the relieving structure is absent, the costs are the lowest. When the relieving structure length increases, the costs tend to increase in the case an MV-pile as anchor and decrease for the case of an anchor plate as anchor.

There are several factors that influence the cost calculation made in this thesis. The fact that the quay wall without relieving structure is less expensive than the quay wall with relieving structure is something that is not expected, but there are certain factors that influence the costs in a significant way.

During the design of the quay wall without relieving structure, cross-sections for the primary elements (steel tubes) are assumed. At the level of the maximum moment the thickness of the primary elements is increased. This way, outside the range of the maximum moment (in this case 20 meters is assumed, however this area is much smaller) savings in steel material are achieved. These savings have a value in the range of €900 - €6800 for the case of a quay wall with MV-pile and €7000-€10000 for the case of an anchor plate as anchor. If this reduction in cross section is not applied, the difference between a quay wall with and without relieving structure will be smaller, in some cases a relieving structure would be less expensive. This reduction in thickness is not applied in the case of a quay wall with relieving structure, local bucking was decisive to determine the thickness of the primary members, reducing this thickness would increase the risk on local buckling of the wall and it was decided to keep the maximum thickness in the whole length of the primary elements.

Other aspect that has to be taken into account is the steel price. The price of the primary elements (steel tubes) is strongly determined by the way of fabrication. The most usual way of making the tubes is with a method called spiral welding, this method showed high savings in the fabrication of steel tubes and it is the reason why the steel price varies around 1000 €/ton. However, this method of fabrication has its limits regarding dimension of the elements. In the Public Works Rotterdam the steel tube dimensions are limited to 1.82 [m] diameter and 25 [mm] thickness, which mean that if these dimensions are exceeded longitudinal welding has to be used to fabricate the steel tubes. This method leads to 1.5 to 2 times higher steel price. Due to the high load that is relative close to the edge of the quay, the primary members calculated in this thesis exceed these dimensions, and is the reason why the steel price is kept constant for every case.

By reducing the centre to centre distance of the primary members in the case of a quay wall with relieving structure, the needed bending stiffness can be achieved with smaller steel tubes. These would not exceed the limits of spiral welding and a lower steel price can be used in the price calculation.

The optimization is not calculated in this research, however when the aforementioned aspects are taken into account (variation in thickness in the length of the steel primary member, lower dimensions by reducing the centre to centre distance of the steel tubes and extra measures to prevent anchor failure), higher costs in the case of Option A and lower costs in the case of Option B can be expected. This way could Option B be more favourable from an economical point of view.

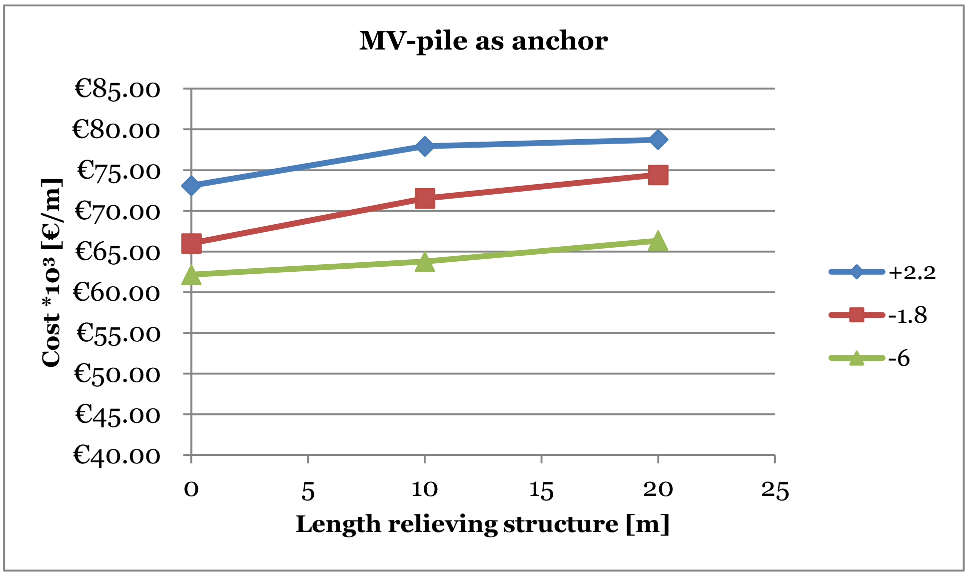

Figure 8.8 Cost in function of the length of the relieving platform for several anchor levels, MV-pile

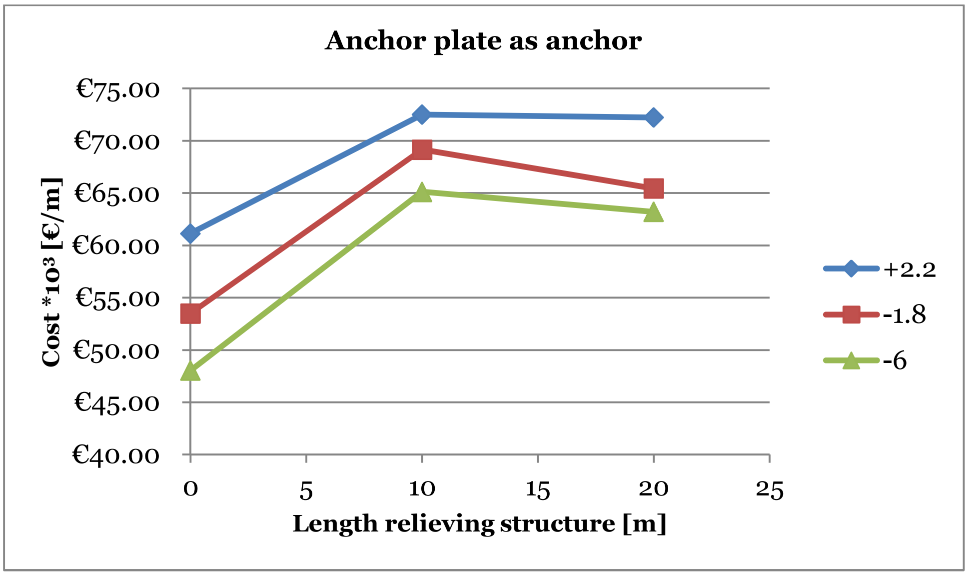

Figure 8.9 Cost in function of the length of the relieving platform for several anchor levels, Anchor plate

9. CONCLUSIONS AND RECOMMENDATIONS

In this chapter the conclusions and recommendations of the present investigation are given. To make this as clear as possible, the research questions established in Chapter 1 are answered based on the results obtained in the investigation. Due to the differences obtained with the calculation methods used, recommendations to take into account during the design of quay walls and recommendations for further investigation are also given.

9.1 Conclusions

"Is a quay wall with a relieving structure more favourable than a traditional quay wall (sheet pile wall) with anchoring, from an engineering and economical point of view?"

Based on the findings of this investigation, following main conclusions regarding this question can be stated:

- From an engineering point of view, a quay wall design with a relieving structure is more favourable than a quay wall without one, due to the smaller displacements, smaller elements used during construction and higher capacity to redistribute loads in case of anchor failure;
- From an economical point of view, a quay wall without relieving structure seems to be more favourable. However, some aspects were not taken into account during the cost estimation (see Chapter 8), This aspects could lead to the conslusion that a quay wall with relieving structure is economically more favourable. The most important aspect is that due to the higher dimensions required for the structure when considering a more stringent load case scenario (combined with a high retaining height as modeled in this thesis) the primary elements (steel tubes) are considered as extraordinary in onshore projects (due to the big dimensions). The price of fabrication of these steel tubes is higher and are more difficult to install. When the load is situated further from the edge of the quay, smaller primary elements of the wall would be needed for the case of a quay wall with relieving structure, this would lead to a reduction in steel price, hence lower costs.

"What is the influence of the anchor type and level in the design of a quay wall?"

To answer this question, two types of anchors were evaluated, horizontal plate anchor and MV-piles.

The anchor type used in a design is strongly driven by the site conditions. Horizontal plate anchors are situated at a long distance from the quay. In this case the calculated distance of the anchor plate needed to ensure stability of the quay were nearly two times the retaining height, , also to install the anchor large areas need to be excavated, which means that no obstacles (structures or foundations) should be present in this area.

The advantage of an MV-pile is that no excavation is needed to install this type of anchor and the angle of installation give the possibility to avoid obstacles near the quay during construction. The main difference between these two anchor types is that the MV-pile leads to less displacement due to its higher stiffness and that using MV-piles lead to higher costs of the quay. The calculated length of the MV-piles lie around 65 and 75 meters, which makes the design more expensive than a design considering horizontal anchors.

The calculation of the anchor forces using both calculation methods showed that in both cases (with and without relieving structure) the level of the anchor showed a reduction of the maximal bending moment in the wall and an increase in anchor force. In all the cases the stability criteria was decisive for the length of the MV-piles and the position of the anchor plate.

"What are the differences in the results using the beam on elastic foundation method and the finite element method? How does this affects in the design of quay walls?"

To be able to answer this question several quay wall designs without relieving structure (Option A) and with relieving structure (Option B) for a specific load situation were made. The calculation of the internal forces in the retaining wall and anchor was performed using two calculation software's, one based on the Beam on elastic foundation method (D-sheet) and the other based on the finite element method (PLAXIS). The results obtained in the calculation with these two methods showed significant differences.

- The calculated bending moments with D-sheet and PLAXIS for both, the quay wall without relieving structure and quay wall with relieving structure have small differences. Both tools are suitable for the dimensioning of the primary elements of the wall.
- In the case of a quay wall without relieving structure, the difference in the calculated anchor forces is low. D-sheet calculates in general lower anchor forces, however, because of the use of safety factors during the design both calculation methods are suitable for the dimensioning of the anchors;
- In the case of a quay wall with relieving structure higher differences between the calculated anchor forces are found. When the relieving structure has a length of 10 [m] PLAXIS estimates anchor forces that are a factor 1.4 to 1.55 higher than D-sheet. When the relieving structure has a length of 20 [m] the anchor forces calculated with PLAXIS are a factor 1.25 to 2.55 higher than D-sheet. The most probable reason of this difference is that D-sheet estimates the horizontal pressure on the wall due to strip loads with an equation after Boussinesq.

"What is the influence of surcharge modelling with the finite element method?"

From the results presented in Chapter 7 it can be concluded that by modelling the load as soil material (iron ore) in PLAXIS, higher bending moments and anchor forces are obtained.

The moments calculated with PLAXIS when the surface load is modelled as a soil are about 1% higher. The calculated anchor forces are about 6% higher. The reason for this increase in anchor forces is the development of shear stress in the contact area between the surface and the iron ore.

This increase in anchor force and bending moments is low and can be neglected during the design phase of the quay walls.

9.2Recommendations

- Based on the conclusions and the investigation results described in this thesis, some recommendations on the use of D-sheet and PLAXIS are given. D-sheet is a powerful tool to calculate sheet pile walls; however when the designer deals with non-uniform loads or loads separated from the quay D-sheet tends to underestimate anchor forces. It is recommended to take adequate safety factors to compensate this difference. From this investigation it is recommended that when a relieving structure is considered a factor 1.5 on the resultant anchor forces should be applied. However, this investigation is carried for a specific load case and this should not be considered as a "Rule of thumb".

  When a relieving structure is present, D-sheet estimates forces that are much smaller than the ones calculated with PLAXIS. Due to the more advanced modelling capabilities of PLAXIS and the corresponding more reliable results obtained, it is recommended to use the results of D-sheet only in the concept design phase (The model and calculation time in D-sheet is small). For final design checks it is advised to use PLAXIS for a more rigorous estimation of the anchor forces and moments;

- During the costs assessment, it is recommended to take into account the fabrication and installation of steel elements. The variation in dimensions of the primary elements of the wall (steel tubes) between the variants considered have influence in the prices of fabrication and installation of these elements, which are of great influence in the costs. By making a more detailed cost assessment, a more justified choice of structure can be made;

- This investigation was carried out for a specific case (load distribution and retaining height). It is recommended to investigate more cases with several retaining heights, soil types, variation in parameters and surcharges. This in order to make a clear assessment of the limits of the Beam on Elastic Foundation method (used by D-sheet) and establish in a more accurate way the differences with regard to the Finite Element method;

- It is also recommended to research the influence in the costs of a quay wall due to high surcharge loads at several distances from the quay wall. This way could be possible to determine the transition point where a quay wall without relieving structure is more favourable than a quay wall with relieving structure.

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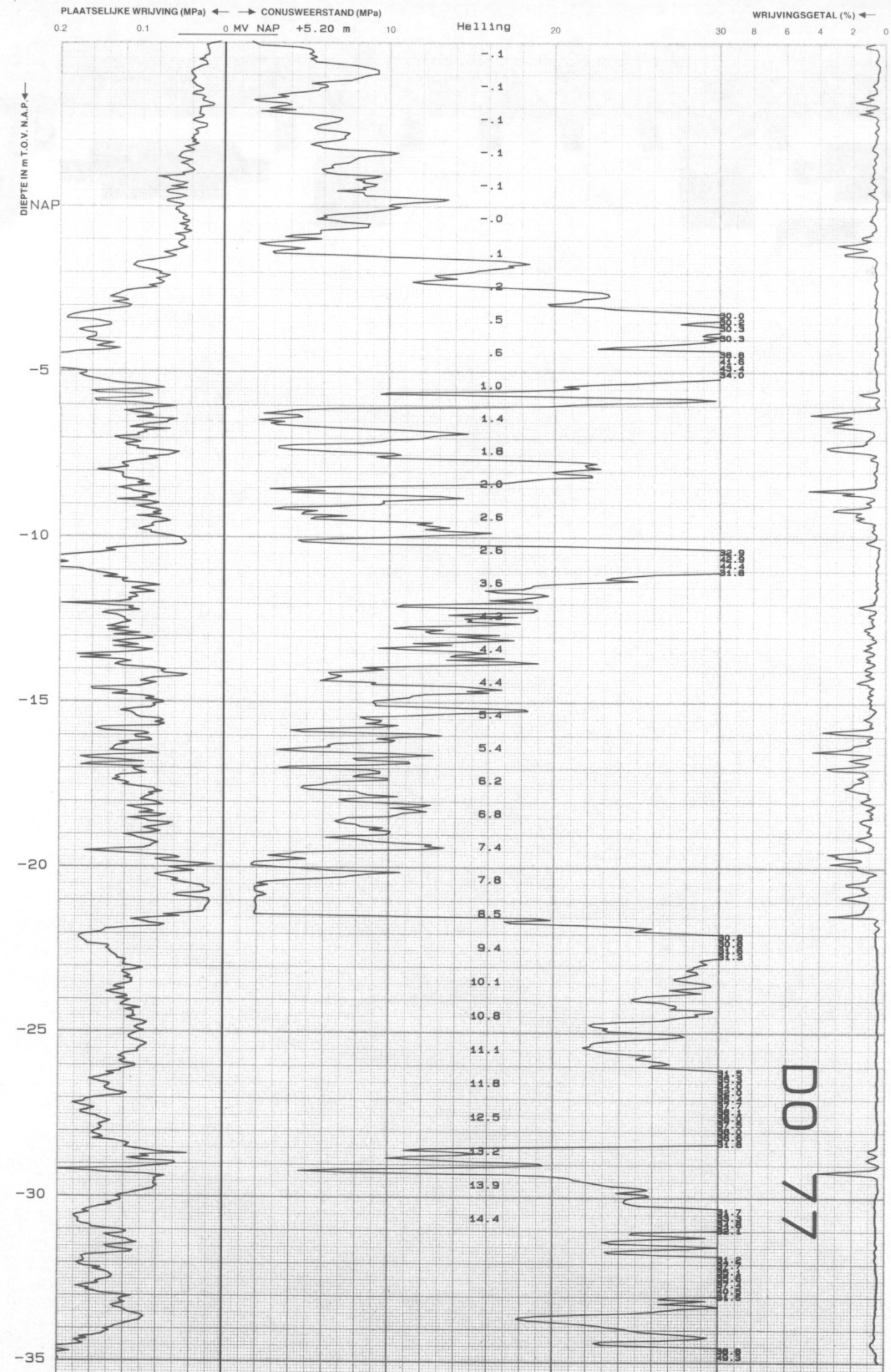
Gemeente Rotterdam

Annex A: Cone penetration test "Amazonehaven"



WERK :	Maasvlakte zeekade Swarttouw
PLAATS:	Rotterdam

GEMEENTEWERKEN ROTTERDAM
INGENIEURSBUREAU GEOTECHNIEK EN MILIEU

MAP :	87/174
DATUM :	88/01/11
SONDERING:	DO 77
PARAAF :	

SONDERING VOLGENS NEN 3680

CONUSTYPE :	Cil.elec kl-mant
CONUS NR. :	GDCFI 5873
MAX. BEREIK:	50 kN

ed 415/4226-'86

Annex B: Hand calculation verification

When calculation software is not available, an engineer must be able to give recommendation based on hand calculations. The method of BLUM is evaluated comparing it to D-sheet (beam on elastic foundation method). The validation of the hand calculation is made for a sand profile with the following parameters:

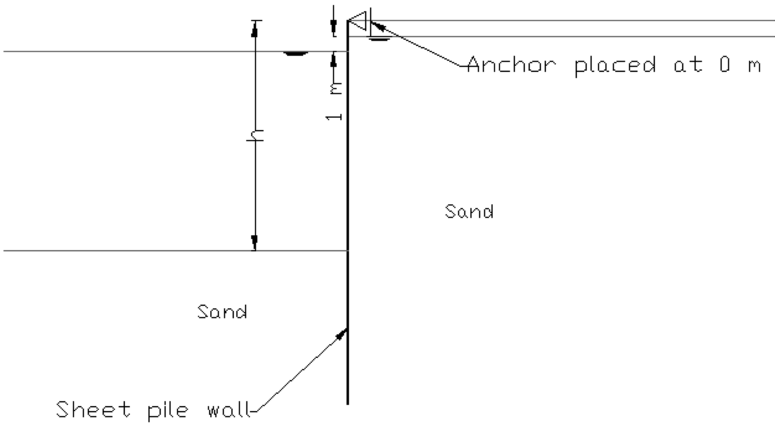
Type	γ **[kN/m3]**		φ' **[o]**	δ **[rad]**	**c' [kPa]**	K_a	K_p
	dry	wet					
Sand (loose)	17	19	35	23.3	0	0.22	9.15

In D-sheet a modulus of subgrade reaction of the soil needs to be defined. The CUR166 gives representative values of subgrade reaction modulus for several types of soils. The representative values of the modulus van subgrade reaction in function of the relative displacement are:

	[kN/m3]
k_h50%	40000
k_h80%	20000
k_h100%	10000

Variation in the retaining height and anchor level is made. Five retaining heights are considered: 5, 10, 15, 20 and 25 meters. Also two anchor levels are considered: anchor at surface level and 1/5 of the retaining height below surface level. The next figure shows the layout of the model.

The water levels are kept at two meters from the surface at water side and one meter at land side.



Layout hand calculation verification

B.1. Hand calculation method

The hand calculation is performed with the free end support and fixed end support beam method of BLUM. The free end support beam method calculates the maximal bending moment and embedding depth based on horizontal equilibrium and moment at the toe equal to zero. The fixed end method assumes a fixed support at the toe, displacement at the anchor point and toe and moment at the toe equal to zero.

The two methods are shown in Figure 1.1 and Figure 1.2 with their respective equations:

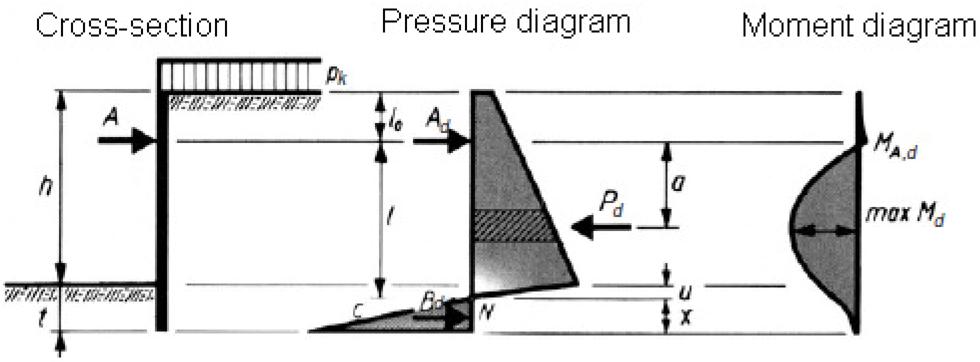


Figure 1.1 Schematization sheet pile with a free end support

$$A = \sum_{-l_0}^{+l} P - \frac{c}{2} \cdot x^2$$

$$\xi^2 (2\xi + 3) = \frac{6}{c \cdot l^3} \sum_{-l_0}^{+l} P \cdot a = m$$

Where: $A =$ Anchor force in [kN]

$\sum P =$ Resultant force due to earth pressure [kN]

$a =$ Distance of force P to anchor in [m]

$u =$ Distance from bottom quay to point zero pressure in [m]

$l =$ Distance from anchor to point of zero pressure in [m]

$x =$ Distance from point of zero pressure to toe of sheet pile in [m]

$x = \xi \cdot l$ x in function of l

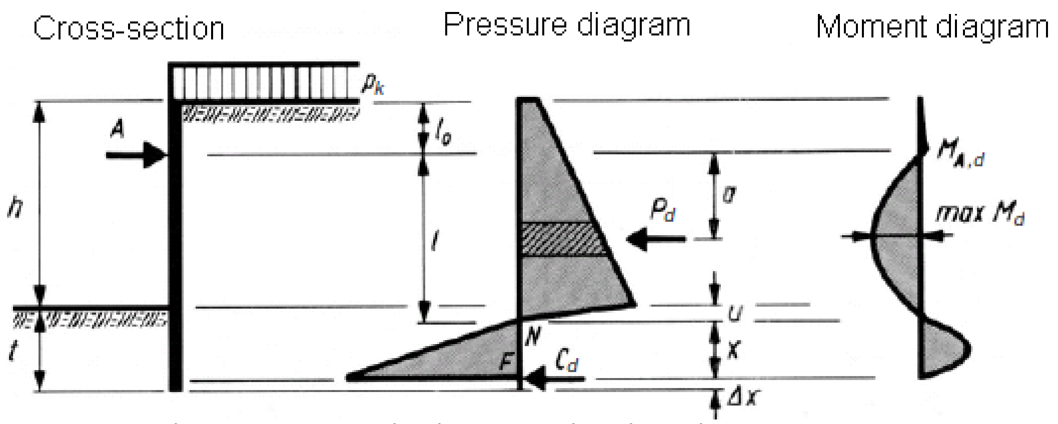


Figure 1.2 Schematization sheet pile with a fixed end support

$$A = \frac{1}{l+x}\left\{\sum_{0}^{-l_0} P'_n\left[(l+x)+a'_n\right] + \sum_{0}^{+l} p_n\left[(l+n)-a_n\right] - \frac{c}{6}\cdot x^3\right\}$$

$$\xi^3\left(0,8\cdot\xi^2+2,5\xi+2,0\right) = (1+\xi)^2\cdot m - n$$

$$m = \frac{6}{c\cdot l^3}\sum_{-l_0}^{+l} P\cdot a \qquad n = \frac{6}{c\cdot l^5}\sum_{0}^{+l} P\cdot a^3$$

The factor ξ is solved by iteration. The advantage of the fixed end method is that a better distribution of moments is acquired (smaller moments for the dimensioning of the sheet pile), the price of this distribution is a much longer sheet pile wall.

After the hand calculation is finished, the length of the calculated embedding depth needs to be increased with 40% in the free end support method (to assure stability of the sheet pile wall) and 20% in the fixed end support method (to develop the fixity force).

B.2. D-sheet

Several calculations of D-sheet are made to validate the hand calculation. The starting points are the results of the hand calculation, then the embedding depth is increased with 0%, 20%, 40% and 60% (for both, the free end and fixed end method).

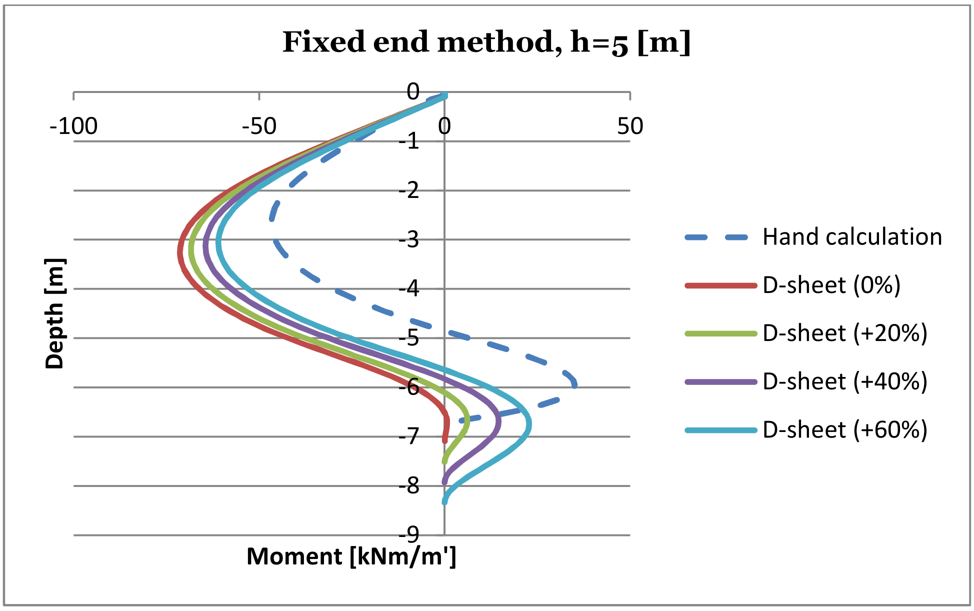
B.3. Results

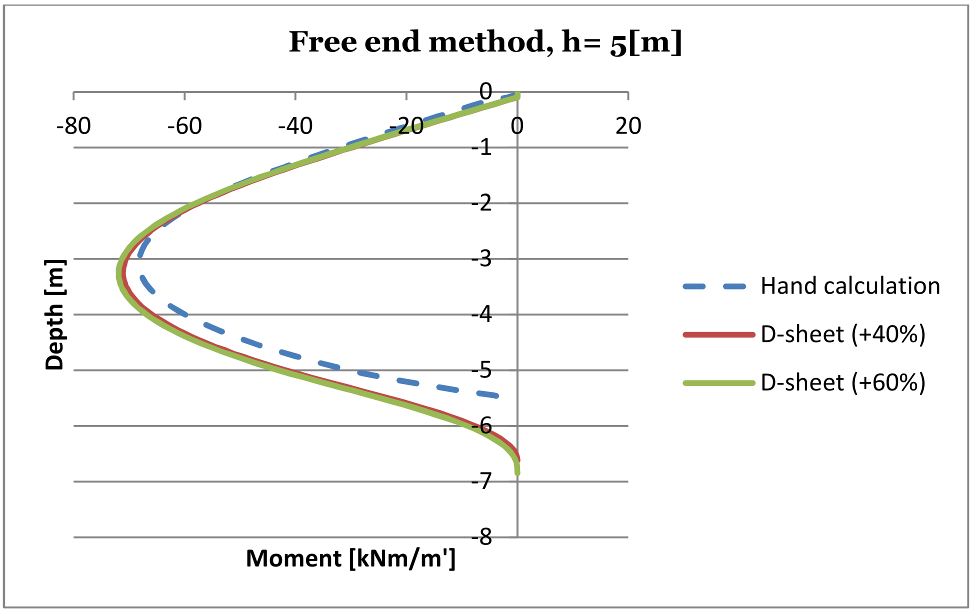
The result for all cases is summarized in the next table:

		Hand calculation		
Retaining height	**Method**	**Embedding depth [m]**	**"Span" moment [kNm/m']**	**Anchor force [kN/m]**
5	Free end	1.16	68.24	32.64
10		1.95	383.18	100.27
15		2.72	1083.74	191.70
20		3.48	2313.61	306.50
25		4.24	4216.50	444.62
5	Fixed end	2.09	46.70	25.17
10		3.72	259.16	78.28
15		5.32	725.91	149.50
20		6.64	1541.13	238.40
25		8.11	2798.86	344.00

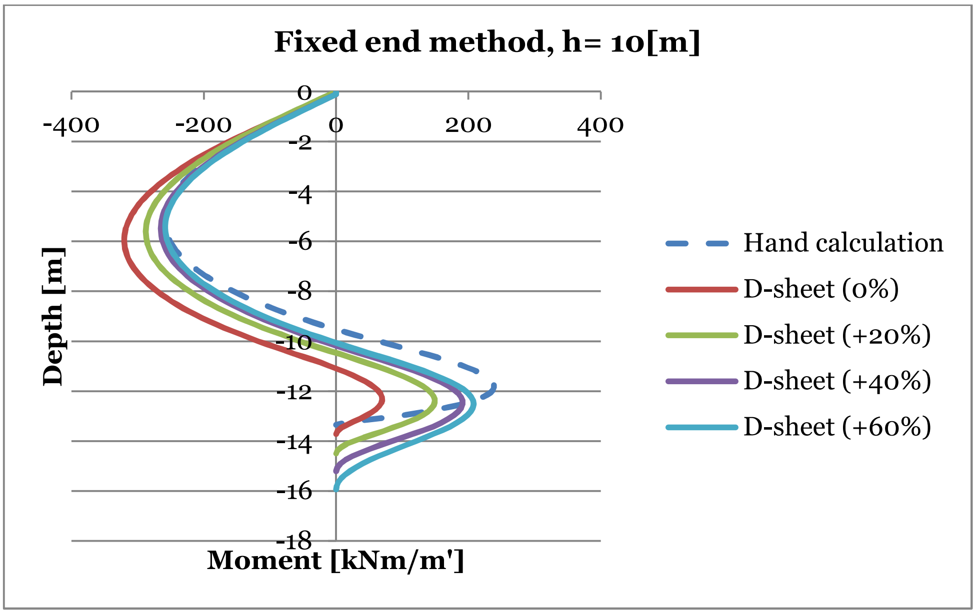
		D-sheet							
		Moment with increased embedding depth [kNm/m']				Anchor force with increased embedding depth [kN/m']			
Retaining height	**Method**	**0%**	**20%**	**40%**	**60%**	**0%**	**20%**	**40%**	**60%**
5	Free end	Unstable	Unstable	71.11	71.9	NA	NA	34.5	34.7
10		Unstable	354.92	352.84	344.08	NA	94.4	94	92.6
15		Unstable	950.03	938.27	907.72	NA	166.7	165.4	162.1
20		Unstable	1954.4	1917	1834.09	NA	254	251.2	244.7
25		Unstable	3467.1	3382.2	Fixed end	NA	348.8	343.4	NA
5	Fixed end	71.35	68.4	64.48	61.01	34.5	33.6	32.3	31.2
10		320.32	287.77	265.07	258.39	88.7	82.7	79	77.8
15		834.04	731.89	681.97	668.71	154.3	143	137.2	118
20		1641.1	1415.5	1347.7	1340.48	229.4	210.8	205.1	204.3
25		2815.7	2394.7	2332.5	2329.7	309.1	282.7	278.7	135.7

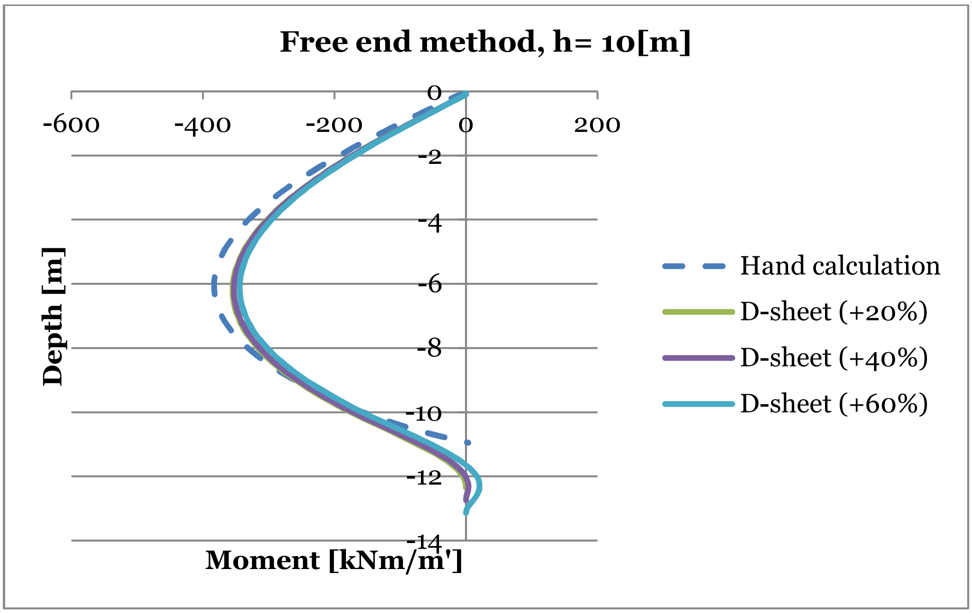
a) 5 meters retaining height



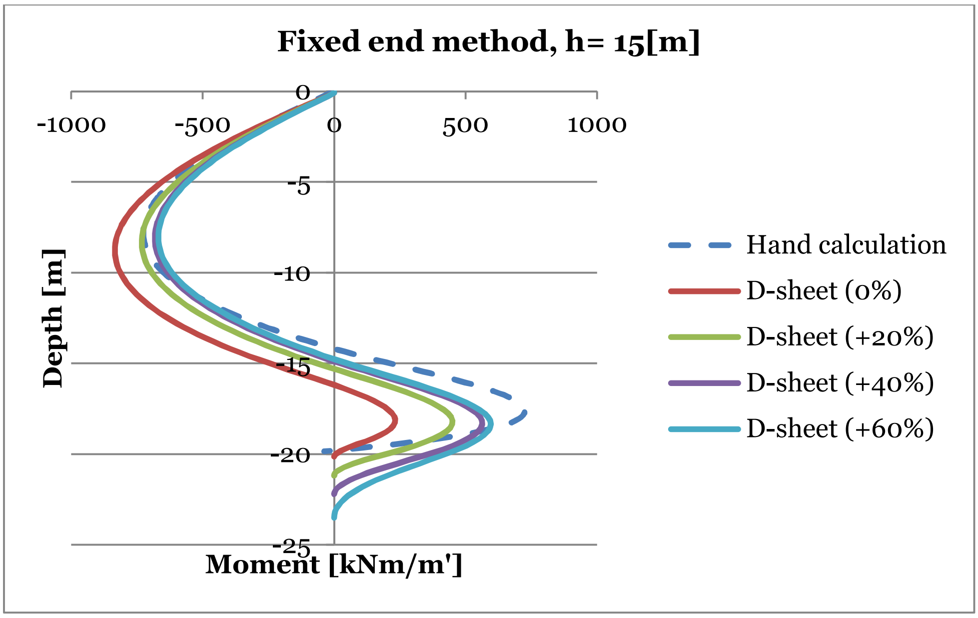


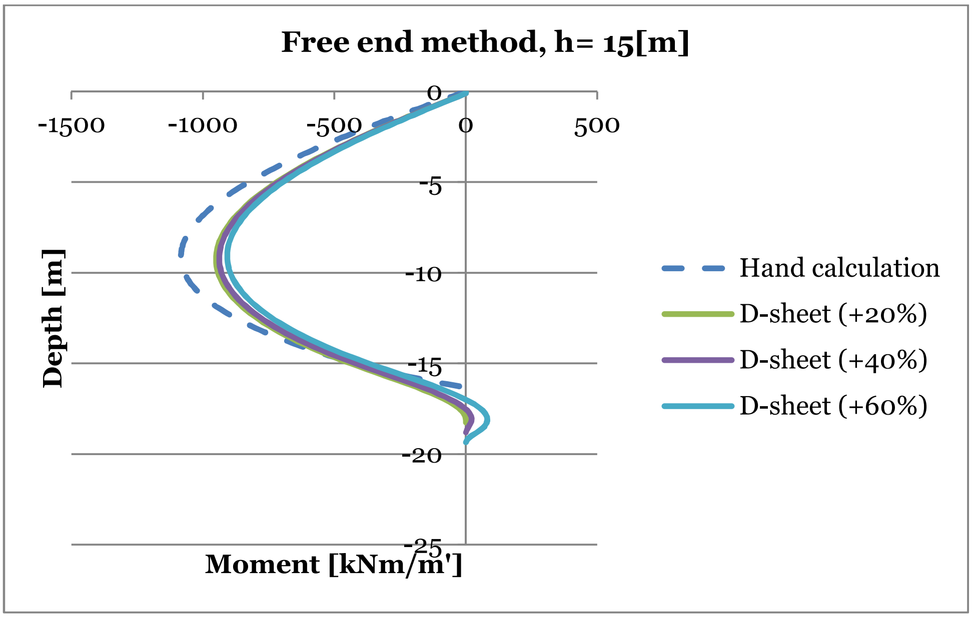
b) 10 meters retaining height



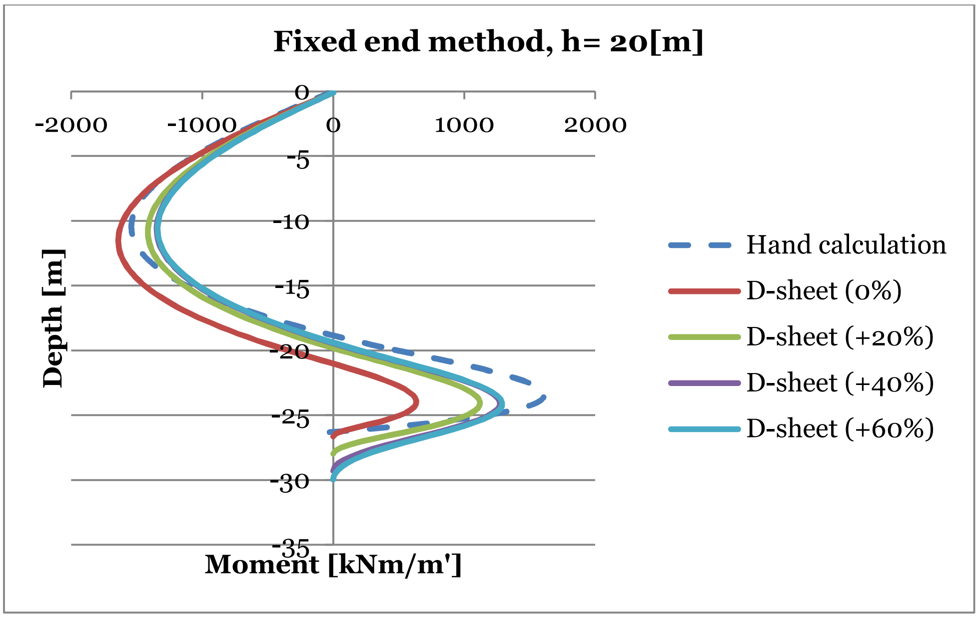


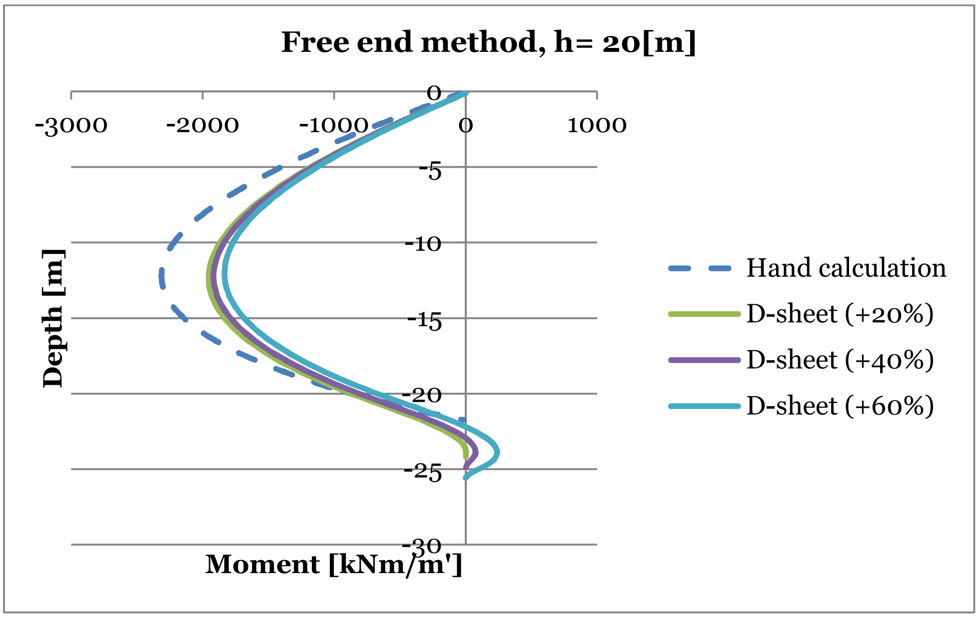
c) 15 meters retaining height



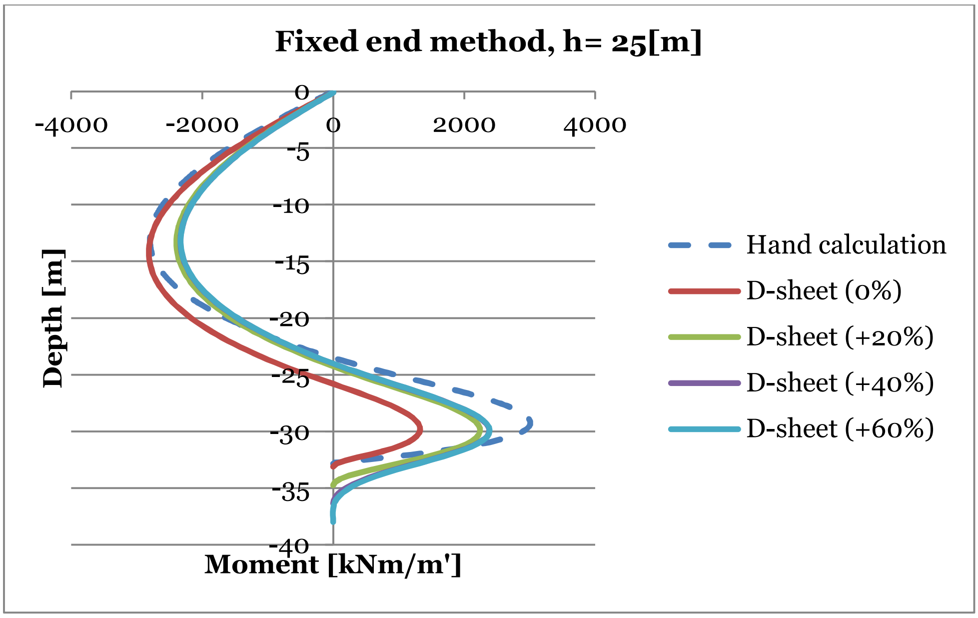


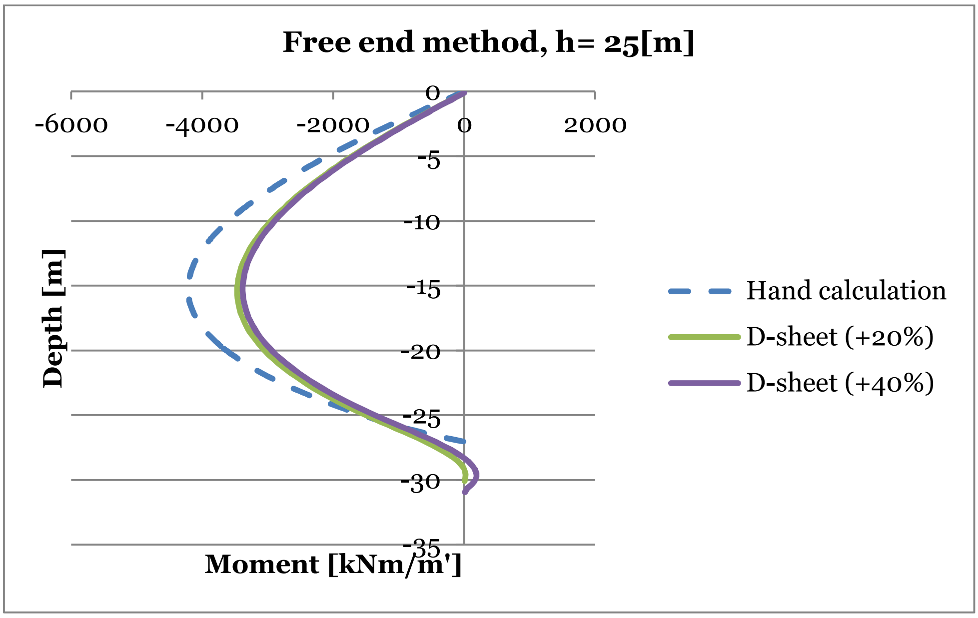
d) 20 meters retaining height





e) 25 meters retaining height





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Annex C: Calculation Option A: combined sheet pile wall without relieving structure

In this attachment the calculation of the combined sheet pile wall without relieving structure is explained in more detail. As mentioned in chapter 4, the most important aspects that determine the internal forces of the wall are:

- Soil parameters;
- Wall and anchor stiffness;
- Anchor position;
- Embedding depth.

The soil parameters depend of the location and the other parameters are determined by the designer.

Wall and anchor stiffness

The wall stiffness is determined by the minimal profile that satisfies the strength criterion mentioned in chapter 4, section 4.6.1.

Embedding depth

The minimal embedding depth to ensure stability of the quay wall is determined in section 2.1 of this annex.

Anchor position

To have a better understanding of the influence of the anchor position in a quay wall, several positions are evaluated:

- +2.2 m NAP, three meters from surface;
- +0.2 m NAP, five meters from surface;
- - 1.8 m NAP, seven meters from surface;
- -6.0 m NAP, 11.2 m from surface.

Anchor type

Two types of anchors will be analysed. A horizontal anchor (anchor plate) and tension piles (MV-pile).

C.1 Calculation method

The first estimation of the bending moments and anchor force is made with the hand calculation method evaluated in chapter 5. This first assumption is used as input in D-sheet to calculate the minimum depth and internal forces in all load cases. D-sheet is used because it gives the possibility to model the soil as a set of springs. Because of its user friendly interface D-sheet it is an excellent tool to make draft designs.

After the final dimensions of the elements are determined, a PLAXIS calculation is made. The results of the PLAXIS calculation are used to check the strength, deformation and stability of the final design in all load case scenarios.

The steps to obtain the final design are the following:

1. Determine the start parameters (water levels, soil parameters, design depth, etc.);

2. Establish load combinations, safety and reduction factors;
3. Asses the dimension of the primary elements, embedding length and anchor force with a hand calculation;
4. Use the results of step 4 to model the structure in D-sheet. Start an iteration process between the embedding depth, wall and anchor properties;
5. Use the result in step 4 as input in PLAXIS and perform a calculation;
6. Check the strength of the primary elements with the results obtained in step 5;
7. Check the anchor strength with the results obtained in step 5;
8. Check the deformations;
9. Overall checks (Kranz stability, overall stability, bearing capacity of the primary elements and failure of one anchor).

Step 1 and 2 are determined in chapter 4. Step 3 is omitted in this report (these are only used as input in the first iteration in D-sheet). The rest of the steps are explained in section 2 of this annex.

1.1 Calculation method of anchors

In section 3.3 of this annex, anchor forces are determined and vary between 2000 – 6000 [kN/m] in the ULS (ultimate limit state). From the several anchor types described in chapter 2 only the anchor wall and MV-pile will be considered.

1.1.1 Anchor wall

The anchor wall is basically a sheet pile wall placed at a certain distance parallel to the quay wall. It is connected to the quay wall by means of anchor rods. When the quay wall starts to deform, a tension force is transmitted to the anchor wall and it will tend to move in the same direction. The soil in front of the anchor wall (in between the quay wall and anchor wall) will generate passive resistance and (if the anchor wall has sufficient length) will make equilibrium with the anchor force (and active pressure behind the anchor wall). The principle is shown in Figure A, the surface load is considered active only in the active side of the anchor wall to calculate with an unfavourable situation.

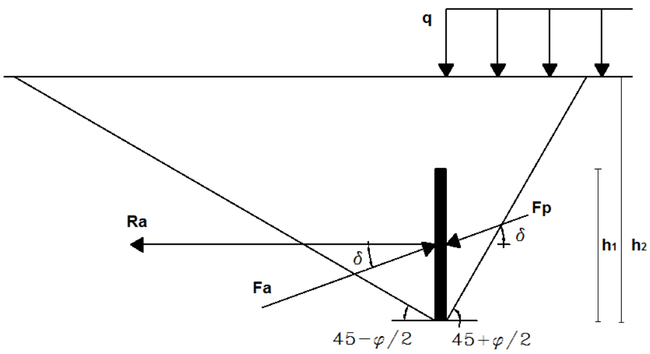


Figure A Principle of forces of an anchor plate

Approach formulas have been developed to estimate the maximum allowable force of an anchor wall. In the Netherlands the CUR166 give some formulas to calculate the holding capacity of plate anchors. These formulas are for plate anchors with determined width and length. The formula given for a continuous wall only takes the passive resistance of the soil into account and neglects the active pressure caused by the surface load and the active soil pressure:

$$R_a = K_p 0.5 \gamma h_2^2$$

Where:

R_a Anchor holding capacity [kN]

K_p Passive earth pressure coefficient [-]

γ Weight density of the soil [kN/m3]

h_2 Height from surface to anchor toe [m]

A better approach is made by taking the active pressure into account and by neglecting the pressure contribution from the surface until the head of the anchor:

$$R_a = (K_p - K_a)0.5\gamma(h_2^2 - h_1^2) - K_a q$$

It should be noted that this estimates the holding capacity assuming a horizontal displacement without rotation and full earth mobilisation.

1.1.2 MV-pile

MV-piles are made of steel tension piles and a grout body that assures adhesion to the soil. The holding capacity of this anchor is provided by means of friction between the soil and grout body (tension pile interaction). As mentioned in chapter 2 MV-piles can be made of hollow sections or H profiles. The advantage of an H profile is that long contact surfaces can be obtained with relative small sections. Figure B shows the principle of an MV-pile. The effective length that contributes to the holding capacity is situated outside the active wedge of the combined sheet pile wall.

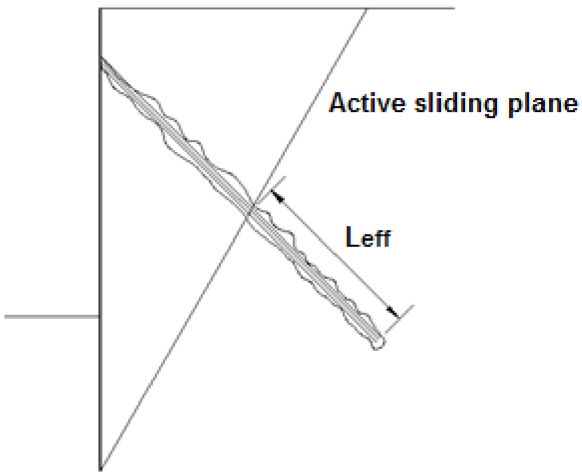


Figure B Principle of the effective length in a MV-pile

The basic formula used to calculate the hold capacity of an MV-pile is:

$$R_a = L_{eff} O \tau$$

Where:

L_{eff} Effective length [m]

O Perimeter of the contact surface of soil and grout [m]

τ Shear stress of the soil [kN/m^2]

The shear stress is determined as a percentage of the average cone resistance of the layer. In the Netherlands a value of 1.2% of the cone resistance is advised. In practice this has shown too conservative results and a value of 1.4% of the cone resistance give better results, however the maximum shear stress should be kept at 250 [kN/m^2] (CUR166, 6th edition, 2012) which is still conservative. This is strongly dependent on the country, for example in Belgium people calculate without a limit shear stress, the shear stress reaches then values of 280 – 300 [kN/m^2]. In this thesis the Dutch approach will be used where 1.4% of the average cone resistance is equal to the shear stress and a safety factor 1.4 is applied to the holding capacity of the anchor (CUR166, 6th edition, 2012).

The perimeter of the contact surface is difficult to determine because it is not known how the grout will distribute around the pile (see Figure C.b). Because of this uncertainty the perimeter will be assumed as b*h [m^2] as shown in Figure C.a. This perimeter is the so called "short way" perimeter, in reality the contact surface will be larger, resulting in a larger holding capacity.

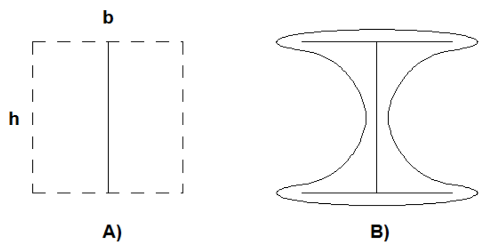
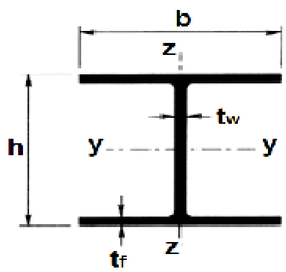


Figure C Perimeters for the calculation of an MV-pile A) "Short" way B) "Diabolo" shape

A decision of the cross-section has to be made for further calculation. HP profiles are made especially for foundation matters with an equal web and flange thickness. The advantages of the HP profile as MV-pile are (www.arcelormittal.com, June 2013):

- A stiff web that gives enough stability during pile driving;
- High bearing capacity (>5000 [kN] each pile);
- Unlimited length due to the weld characteristics.

During the iteration process in section C.2, a HP 400x194 is chosen as cross section for the MV-pile:



HP 400x194 Profile	
Mass [kg/m]	194
h [mm]	364
b [mm]	398
tw [mm]	22
tf [mm]	22
A [cm2]	247.5

Figure D Cross-section MV pile, HP400x194 (www.arcelormittal.com, June 2013)

1.2 Moment capacity combined wall

The secondary elements in a combined wall have a small contribution in the moment capacity and stiffness of the wall. It is decided to assume that the primary members resist all the loads and the secondary elements just transfer the load to the primary elements. It is assumed that the secondary elements consist of 3 PU profiles with a width of 0.6 [m] each. The moment of inertia and the moment capacity of the wall are then:

$$I_{system} = \frac{I_{tube}}{L_{system}} = \frac{\pi}{64 L_{system}} \left(D_e^4 - D_i^4 \right) \qquad M_{capacity} = \frac{I_{system} \cdot f_y}{z}$$

Where:

I_{system}	Moment of inertia of the system in [m4]
I_{tube}	Moment of inertia of the primary element [m4]
L_{system}	Centre to centre distance primary members [m]
D_e, D_i	Outer and inner diameter of the primary member [m]
$M_{capacity}$	Moment capacity of the wall [kNm/m’]
z	Distance from the neutral axis to the ultimate fibre (end fibre $z = 0.5 D_e$) [m]

C.2 Calculations

2.1 D-sheet calculation (step 4)

The dimensions obtained in the hand calculation are used as a starting point in the D-sheet calculation. In this step, iteration between the length of the wall, dimension of the primary elements and anchor properties is made. The last iteration step (final dimensions) is described in this section. The holding capacity and dimensions of the anchors are determined as explained in section c.1 of this annex.

2.1.1 Combined sheet pile wall with MV-pile

Model description

In this model, an MV-pile is situated at an angle of 45 degrees. In D-sheet, the soil interaction along the anchor is not modelled, the anchor parameters is used to calculate the anchor stiffness, the angle to calculate the contribution to the normal force and the length to check the Kranz stability of the anchor.

Figure E shows the model used for the MV-pile at +0.2 m NAP. The hatch area represents the crane load that is introduced as a normal force in the wall.

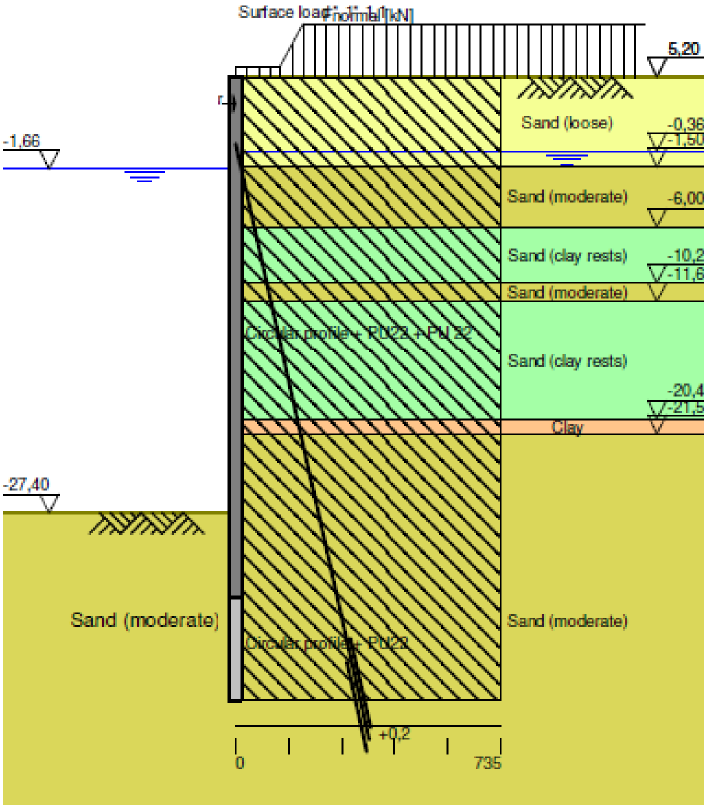


Figure E Impression combined sheet pile wall with MV-piles in D-sheet

Results

The calculation in each load case shows that the maximum moment is obtained in LC2.2 and the maximum anchor force in LC2.1.

a) MV-pile at +2.2 m NAP

Minimal length primary elements: 47.2 [m] (toe at -42 m NAP)

The secondary elements are situated about 3 meters under the zero pressure point at a height of -33.8 m NAP.

Cross section primary members

Figure F shows the moment distribution in the wall for the combination with the maximum moment and maximum anchor force. The maximal moment is only present at a certain point and it would be too conservative to dimension the whole wall with one thickness (thickness has a great influence in the costs). The section thickness will be reduced at certain areas. To verify the influence in the internal force distribution, two calculations are performed for a hollow section with the same diameter but different thickness.

Primary members of a diameter of 2.8 [m] and a thickness of 55 [mm] are needed satisfy the maximal moment requirement. The maximal moment occur around -19.2 m NAP (Figure F). At -19.2 ±10 m NAP the moment is equal to 21000 [kNm/m] which is equal to a reduction of 15 mm of thickness in

27.2 [m] wall. In the same figure the shear force distribution is shown, here the anchor force has a value of 3290.4 [kN/m].

The moment and shear force distribution of the wall with a thickness of 55 [mm] is shown in Figure F and for a thickness of 40 [mm] in Figure G. The variation of the maximal moment and anchor force is low (magnitude <1%) and a calculation of a combined profile is not necessary.

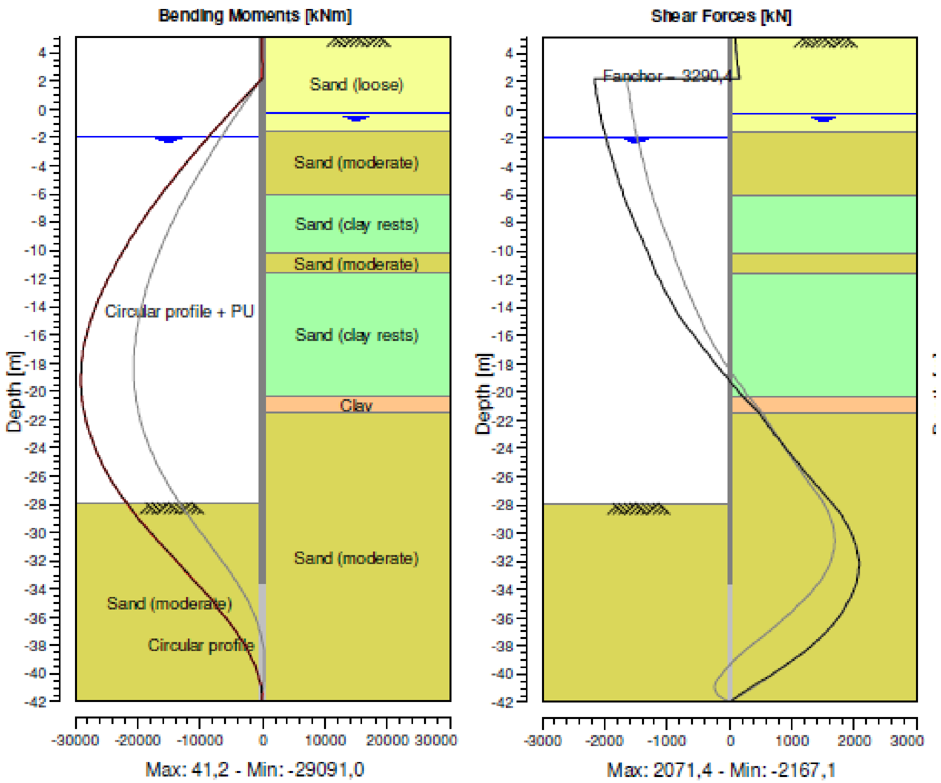


Figure F Moment and shear force distribution Option A, MV-pile at +2.2 m, Max. thickness

MV-pile

The maximal anchor force is obtained in LC2.1 and has a value of 3709 [kN/m]. The method described in section 1.1.2 of this annex is used for the calculation of the holding capacity of the MV-pile.

When piles are too close of each other the influence area of the piles overlap each other, this means that a reduction in the holding capacity of each pile needs to be applied. Several studies have been made in the past to determine the efficiency of a group of tension piles in non-cohesive soil, the reduction depends on the pile configuration, soil and loading direction. For the row MV-piles, the same efficiency will be assumed as for a group piles loaded by a normal tension force. The assumed efficiency factors for a tension pile in a group of piles are (Pile design and construction Rules of thumb, Ruwan Rajapakse, 2008):

c.t.c. distance	Efficiency
3D	0.67
5D	0.80
6D	0.87
>8D	1

With the given efficiency factors and the assumption that $\tau = 0.014 q_{c;av} \leq 250[kN/m^2]$ the results are:

q_c [Mpa]	τ [kPa]	$\tau \cdot O$ [kN/m]	L_{eff} [m]	R_{rep} [kN]
10.7	149.8	228.3	9.04	2063.8
20	250	381	30.98	11804.6
			Total	13868.4

c.t.c. distance [m]:	1.8
R_d [kN]:	9906
R_d [kN/m]:	5503.319
% reduction:	25.3
R_d*reduction [kN/m]:	4113.1
Total length [m]	64.5

Where:

q_c Average cone resistance at layer [Mpa]
L_{eff} Effective length in layer [m]
τ Shear stress at layer [kPa]
R_{rep} Representative anchor force
O Contact perimeter [m]
R_d Design anchor force = R_{rep}/1.4

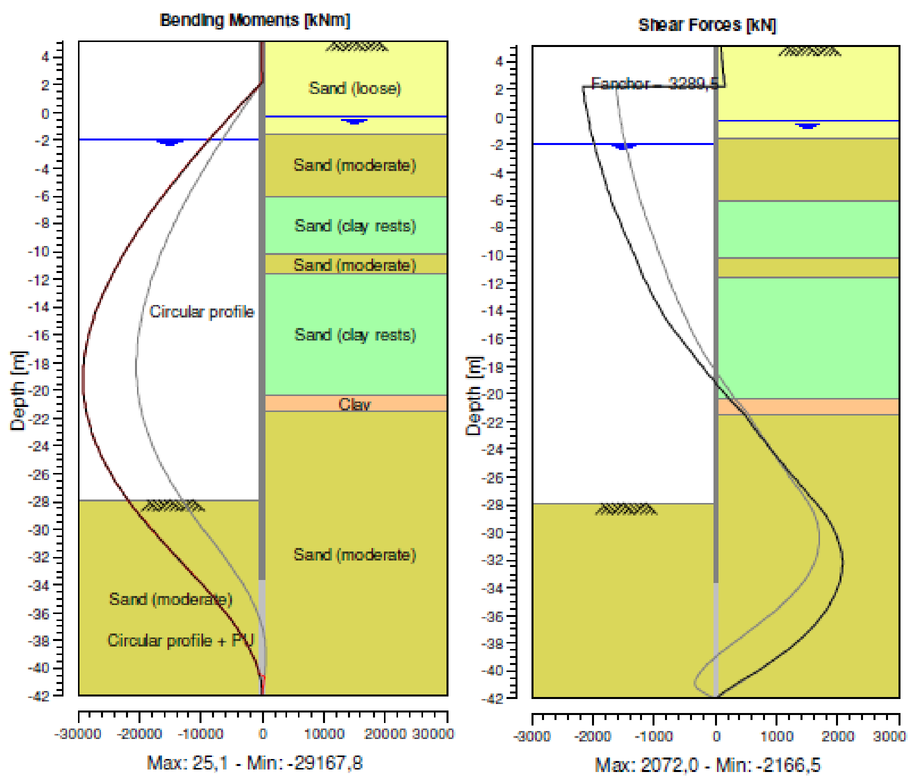


Figure G Moment and shear force distribution Option A, MV-pile at +2.2 m, Min. thickness

The cross section of the situation with MV-piles at +2.2 m NAP is illustrated in Figure H.

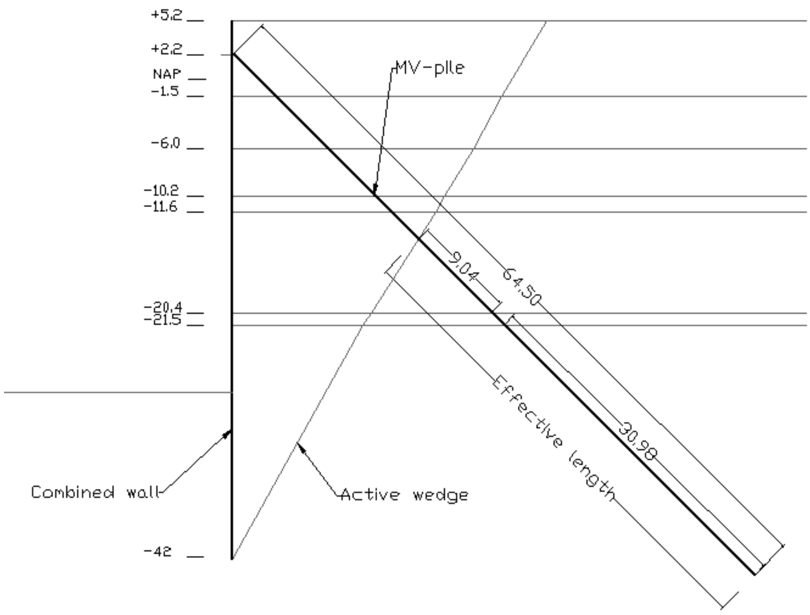


Figure H Length and affective length MV-pile at +2.2.

b) MV-pile at +0.2 m NAP

Minimal length primary elements: 46.6 [m] (-41.4 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found around -20.4 m NAP and has a value of 26071 [kNm/m], a tubular section of 2.8 [m] diameter and a thickness of 50 [mm] satisfies the needed capacity. At -20.4 ±10 m NAP the moment is equal to 18000 [kNm/m] which is equal to a reduction of 17 mm of thickness in 26.6 [m] wall. The moment and shear force distribution of the wall with a thickness of 50 [mm] is shown in Figure J and for a thickness of 33 [mm] in Figure K. As mentioned before, the variation of the maximal moment and anchor force is small and a calculation of a combined profile is not necessary.

MV-pile

The anchor force has a value of 3744 [kN/m], the same profile is used (HP 400x194). The results of the calculation are (Figure L):

q_c [Mpa]	τ [kPa]	$\tau \cdot O$ [kN/m]	L_{eff} [m]	R_{rep} [kN]
10.70	149.80	228.30	7.60	1735.04
20.00	250.00	381.00	33.81	12882.21
			Total	14617.25

c.t.c distance [m]	1.50
R_d [kN]	10440.89
R_d [kN/m]	6960.60
% reduction	29.8
R_d*reduction [kN/m]	4886
Total length [m]	64.50

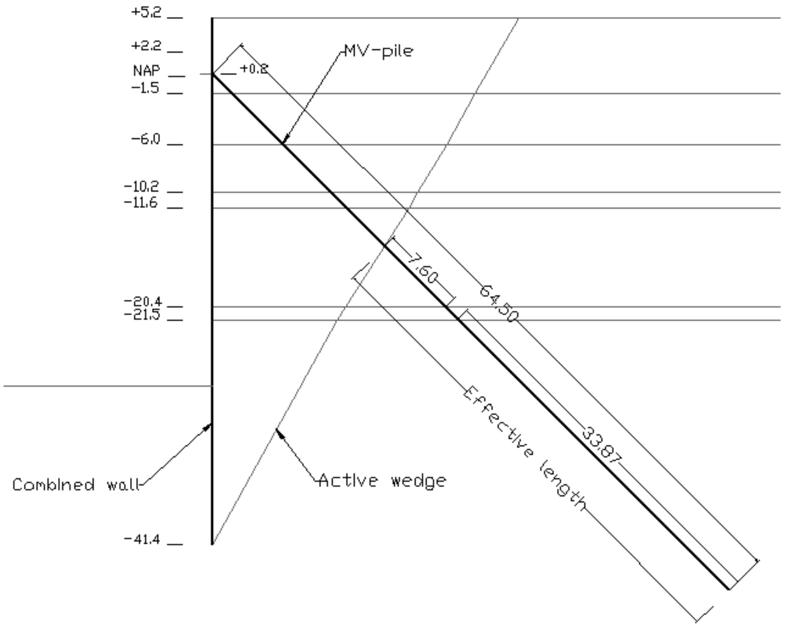


Figure I Length and affective length MV-pile at +0.2 m NAP

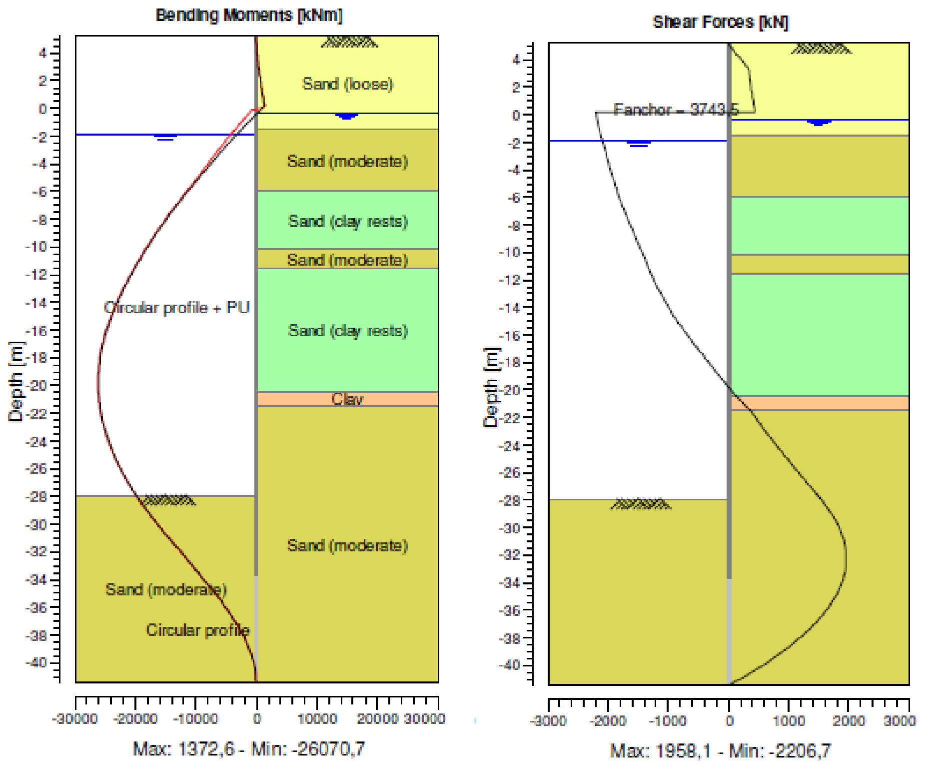


Figure J Moment and shear force distribution Option A, MV-pile at +0.2 m, Max. Thickness

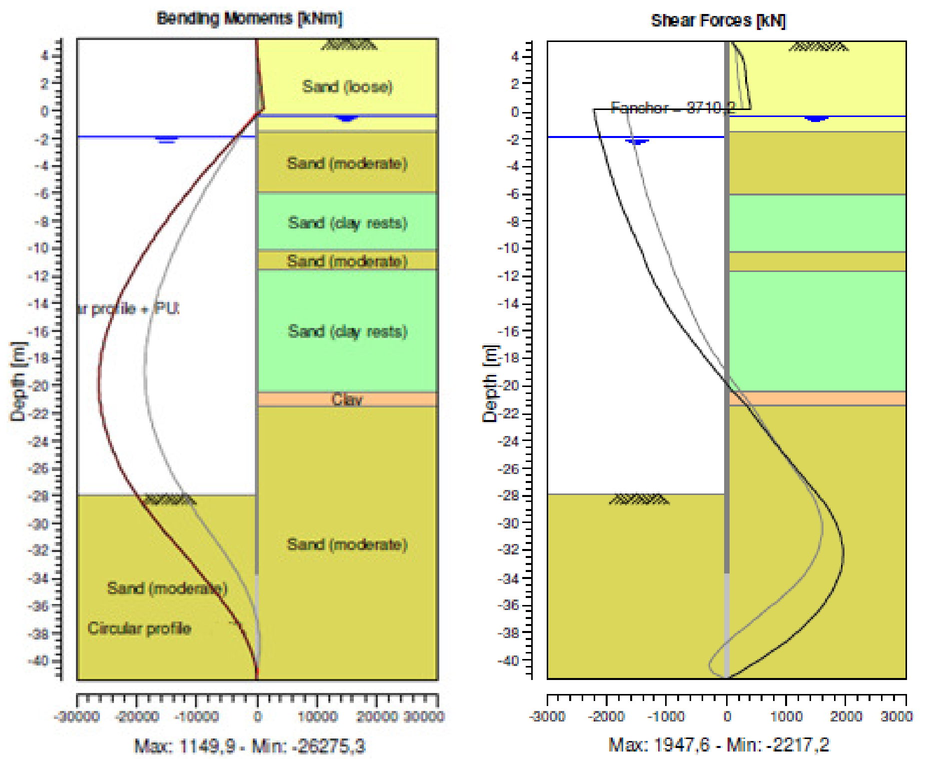


Figure K Moment and shear force distribution Option A, MV-pile at +0.2 m, Min. Thickness

c) MV pile at -1.8 m NAP

Minimal length primary elements: 46.2 [m] (-41 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found around -21.7 m NAP and has a value of 22086 [kNm/m], a tubular section of 2.8 [m] diameter and a thickness of 43 [mm] satisfies the needed capacity. At -21.7 ±10 m NAP the moment is equal to 14500 [kNm/m] which is equal to a reduction of 13 mm of thickness in 26.2 [m] wall. The moment and shear force distribution of the wall with a thickness of 43 [mm] is shown in Figure M and for a thickness of 30 [mm] in Figure N.

MV-pile

The length of the MV-piles is illustrated in Figure L. The anchor force is equal to 4550 [kN/m]. The results of the calculation are:

q_c [Mpa]	τ [kPa]	$\tau \cdot O$ [kN/m]	L_{eff} [m]	R_{rep} [kN]
10.70	149.80	228.30	6.06	1383.47
20.00	250.00	381.00	36.64	13959.84
			Total	15343.31

c.t.c. distance [m]	1.30
R_d [kN]	10959.50
R_d [kN/m']	8430.4
% reduction	32.83
R_d*reduction [kN/m]	5662.93
Total length [m]	64.5

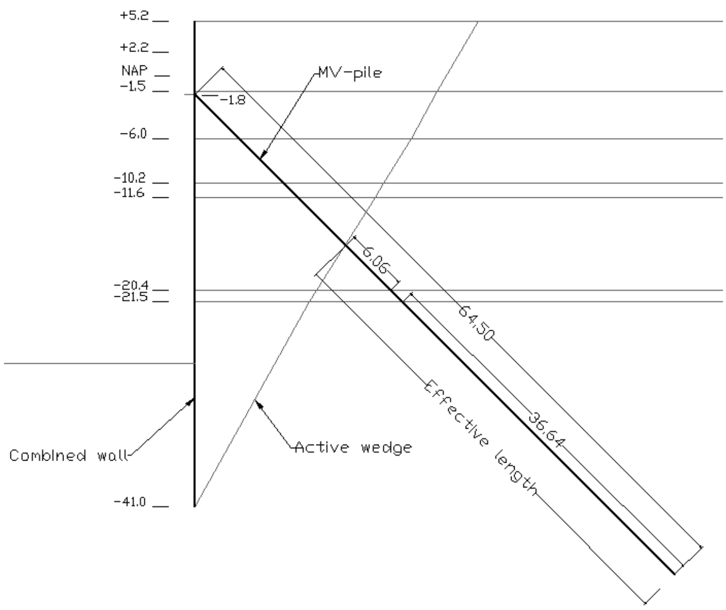


Figure L Length and affective length MV-pile at -1.8 m NAP

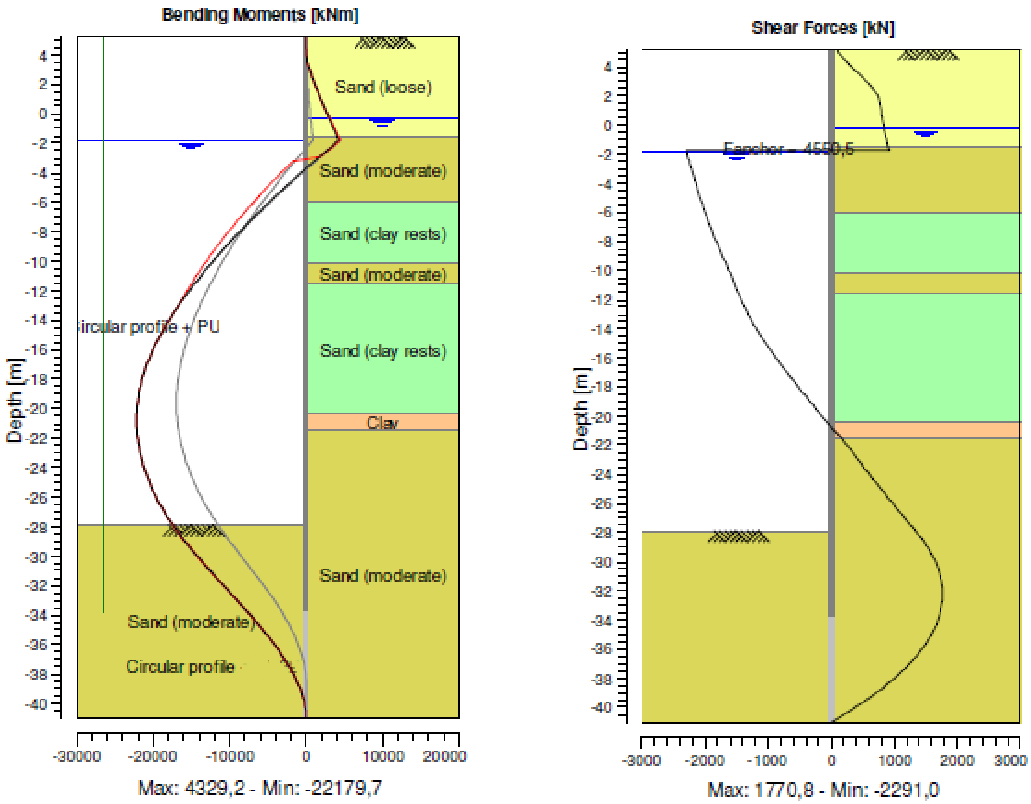


Figure M Moment and shear force distribution Option A, MV-pile at -1.8 m, Max. thickness

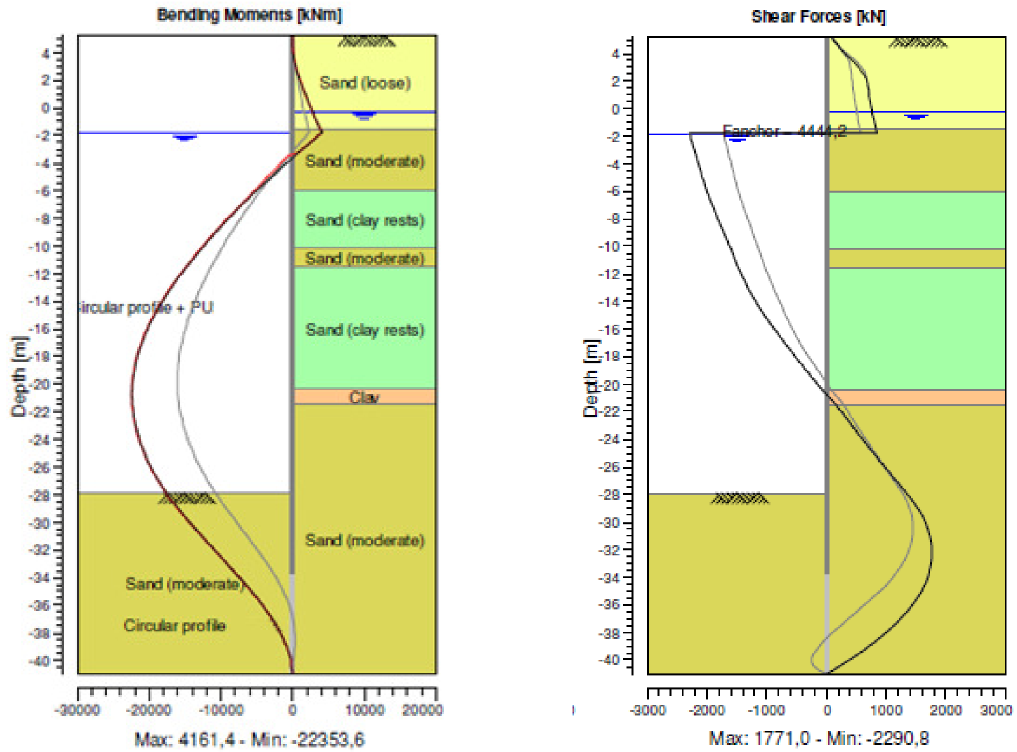


Figure N Moment and shear force distribution Option A, MV-pile at -1.8 m, Min. thickness

d) MV pile at -6.0 m NAP

Minimal length primary elements: 45 [m] (-39.8 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found around -23 m NAP and has a value of 13732 [kNm/m], a tubular section of 2.8 [m] diameter and a thickness of 33 [mm] satisfies the needed capacity. In this case, a thickness reduction of 3 [mm] is applied. The moment at anchor level is almost equal to the maximal (absolute) moment. The moment and shear force distribution of the wall with a thickness of 30 [mm] is shown in Figure P.

MV-pile

The cross-section of this situation is illustrated in Figure O. The anchor force is equal to 5569 [kN/m], the results of the calculation are:

q_c [Mpa]	τ [kPa]	$\tau \cdot O$ [kN/m]	L_{eff} [m]	R_{rep} [kN]
10.70	149.80	228.30	3	684.89
20.00	250.00	381.00	44.08	16794.4
			Total	17479.25

c.t.c. distance [m]	1.30
R_d [kN]	12485.2
R_d [kN/m']	9603.89
% reduction	32.83
R_d*reduction [kN/m]	6451,3
Total length [m]	66

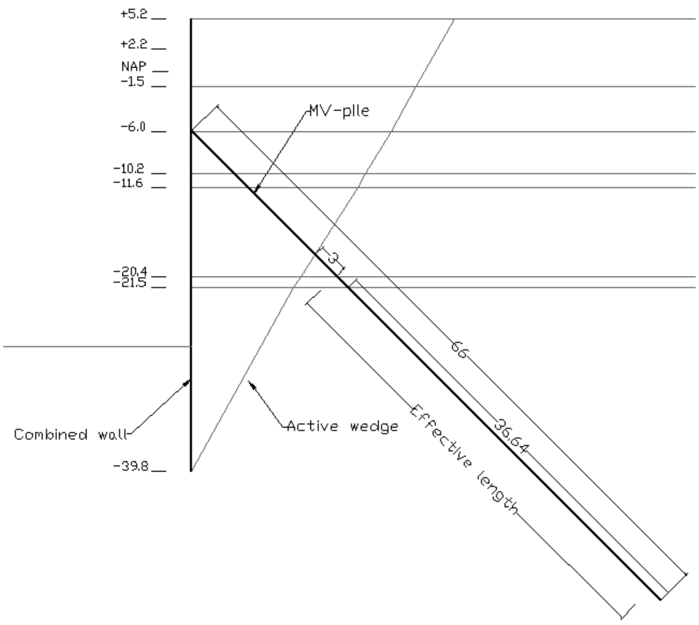


Figure O Length and affective length MV-pile at -6.0 m NAP

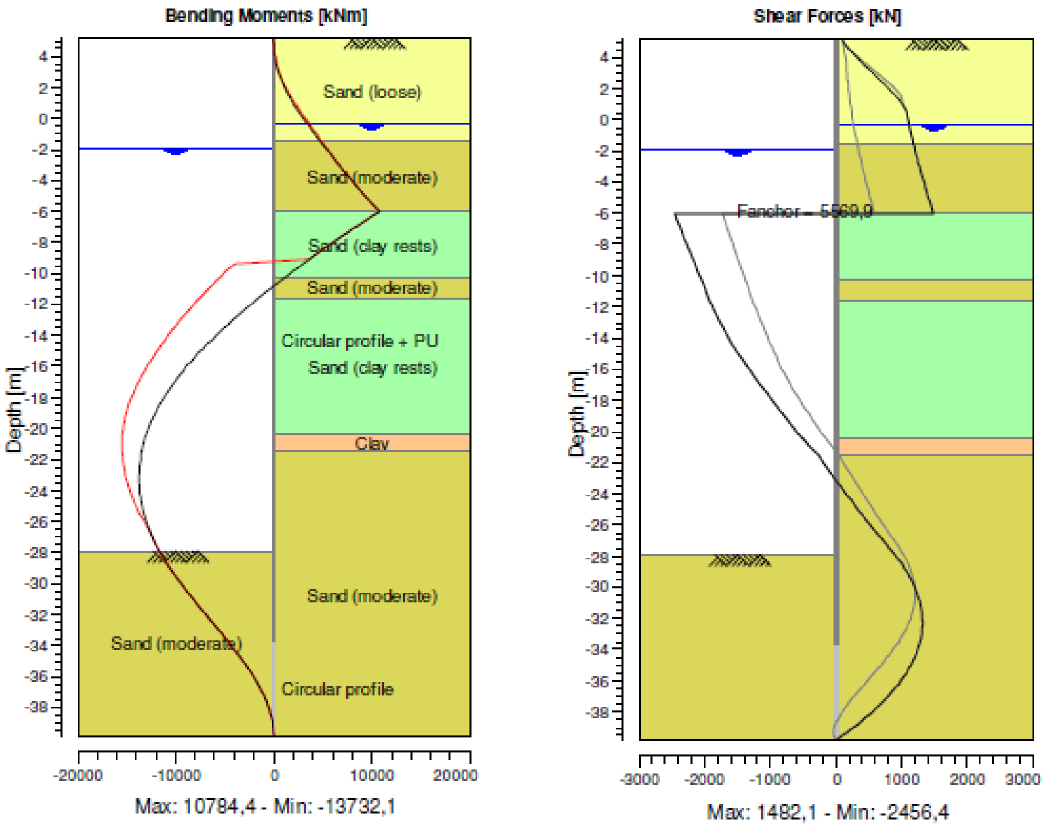


Figure P Moment and shear force distribution Option A, MV-pile at -6.0 m

2.1.2 Combined sheet pile wall with anchor wall

Model description

The same approach is used as the approach for the design of the combined sheet pile wall with MV-piles as anchors.

In this model, an anchor plate is placed at a distance of 53.5 m from the retaining wall. The initial distance is determined by placing the anchor in such a way that the active wedge of the retaining wall and the passive wedge of the anchor don't cross each other. In a further step the anchor position is changed, the final model is described in this section (see figures R, S and X for the final position).

In D-sheet, the soil displacement caused by the anchor is not taken into account for the calculation. The input of the anchor parameters are used to calculate the anchor stiffness. The anchor has an angle that varies depending of the anchor position; the angle is used to calculate de normal force contribution on the wall. The stability of the anchor is determined in a later phase with the method of Kranz for anchor stability. Figure Q shows an example of the model used. In this figure the crane load is applied as a normal force (hatched area) on the wall.

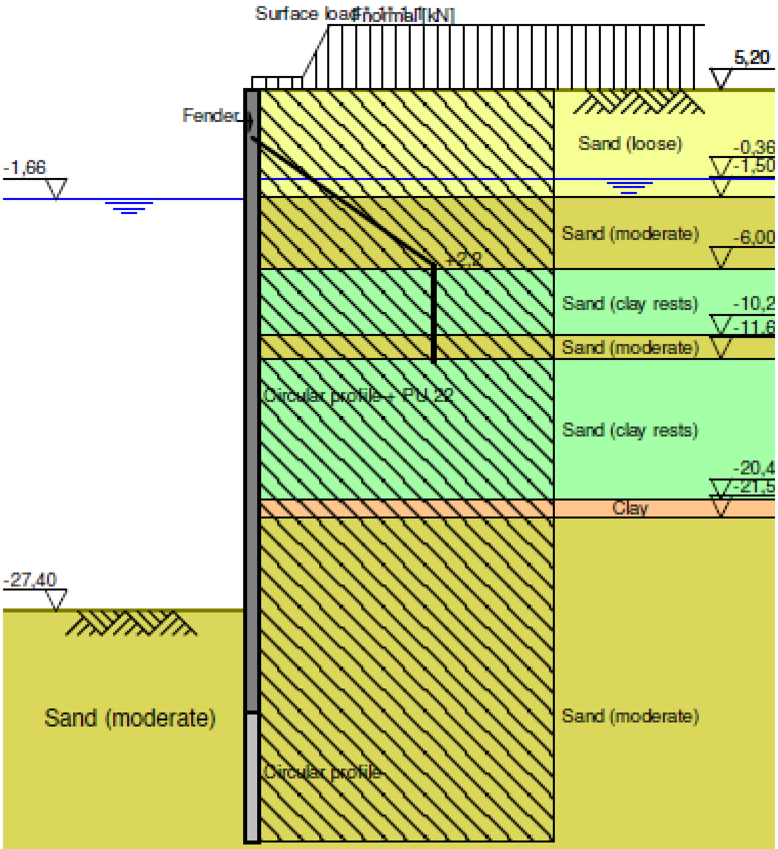


Figure Q Impression combined sheet pile wall with anchor wall and without relieving structure in D-sheet

Results

a) Anchor rod connection at +2.2 m NAP

Minimal length primary elements: 47 [m] (toe at -41.8 m NAP)

The secondary elements are kept the same as for the option with MV-pile (situated 3 meters under the zero pressure point at a height of -33.8 m NAP).

Cross section primary members

The maximal moment calculated in D-sheet has a value of 29124.1 [kNm/m]. A profile with a diameter of 2.9 [m] and a thickness of 50 [mm] has a capacity of 33358.2 [kNm/m] (extra capacity needed to satisfy the combination of bending moment and normal force). The maximal moment takes place at -19.2 m NAP. At -19.2 ±10 m NAP the maximal moment is about 20600 [kNm/m]. A thickness reduction of 15 [mm] can be applied (to reduce costs). A calculation is made with two different wall stiffness to show the influence of the stiffness in the internal force distribution. The two distributions are showed in Figure R and Figure S. The difference in moments is low and the calculation is continued as a uniform profile.

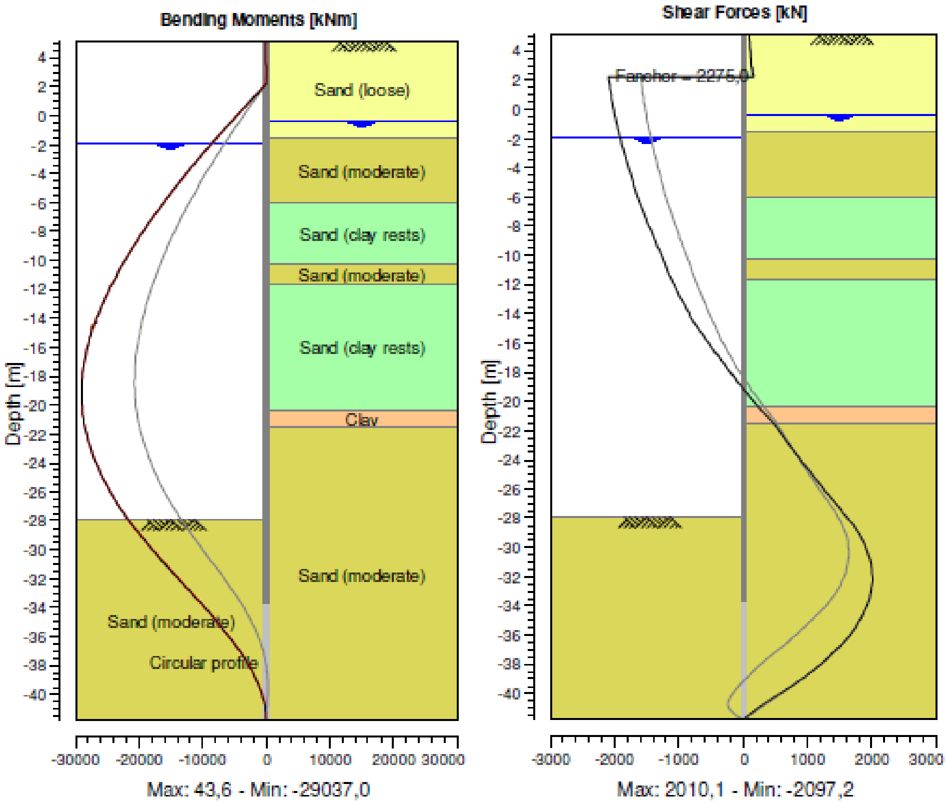


Figure R Moment and shear force distribution Option A, anchor wall at +2.2 m, Max. Thickness

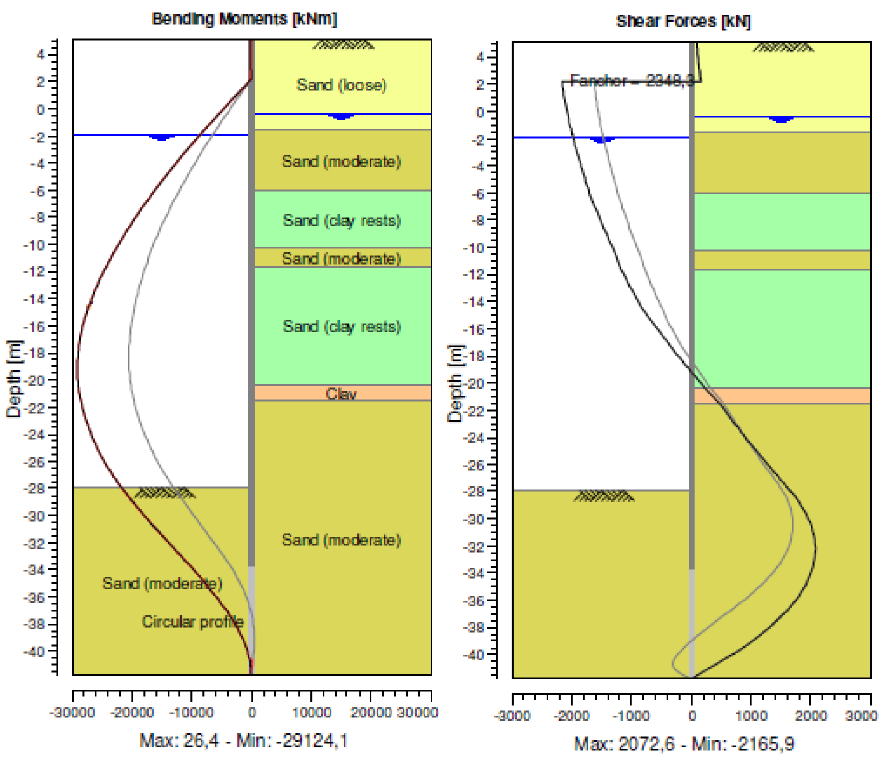


Figure S Moment and shear force distribution Option A, anchor wall at +2.2 m, Min. thickness

Anchor wall

The dimensioning of an anchor wall depends strongly in the strength of the anchor rods. Standard anchor rods of 75 [mm] diameter and steel quality 670/800 can take forces of max. 2.5 [MN/m] (when placed at 1 [m] c.t.c. distance). The minimal c.t.c. distance of the anchor rods determined as the anchor force divided by the anchor rod capacity. In a later step this distance is modified due to the check failure of one anchor.

As mentioned in section 1.1.1 of this annex, the anchor plate capacity is calculated by means of horizontal equilibrium between the anchor force, the passive and active earth pressure. The anchor is placed such that the passive wedge of the anchor and the active wedge of the wall don't cross each other. Figure T shows the layout of the anchor wall.

The minimal anchor plate height is:

Height from surface to top anchor wall [m]	5.00
Height anchor wall [m]	7.00
Contribution passive side [kN/m]	5132.79
Contribution active side [kN/m]	-222.45
contribution surface load [kN/m]	-382.08
Ra [kN/m]	4528.27
Rd [kN/m]	3234.48

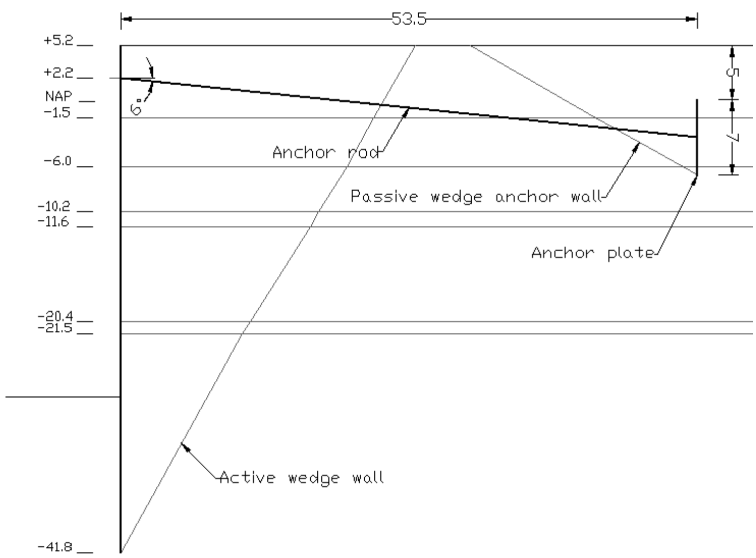


Figure T Anchor plate at +2.2 m NAP

b) Anchor rod connection at +0.2 m NAP

Minimal length primary elements: 46.7 [m] (-41.5 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found around -19.8 m NAP and has a value of 26537 [kNm/m], a tubular section of 2.9 [m] diameter and a thickness of 44 [mm] satisfies the needed capacity. At -19.8±10 m NAP the moment is equal to 18000 [kNm/m] which is equal to a reduction of 14 mm of thickness in 26.6 [m] wall. The moment and shear force distribution of the wall with a thickness of 44 [mm] and for a thickness of 30 [mm] are shown in Figure V and Figure W respectively.

Anchor wall

As mentioned before, in this thesis it is assumed that the anchor rod is strong enough and is places at 1 [m] c.t.c. distance.

The anchor plate capacity is calculated by means of horizontal equilibrium between the anchor force, the passive and active earth pressure. The calculation of the minimal anchor wall length is show in next table, the result of the iteration is shown in Figure U.

Height from surface to top anchor wall [m]	5.00
Height anchor wall [m]	8.50
Contribution passive side [kN/m]	6217.25
Contribution active side [kN/m]	-299.22
contribution surface load [kN/m]	-478.08
Ra [kN/m]	5439.95
Rd [kN/m]	3885.68

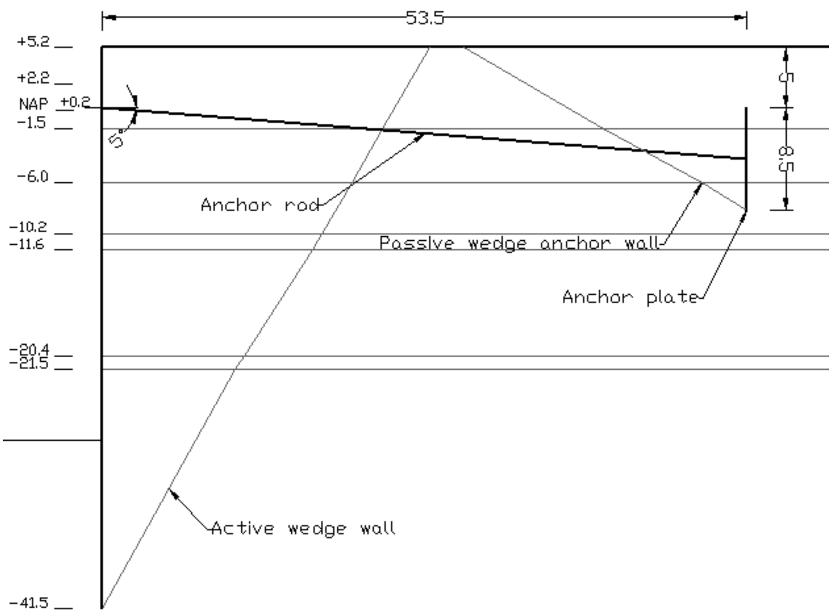


Figure U Anchor wall connected at +0.2 m NAP

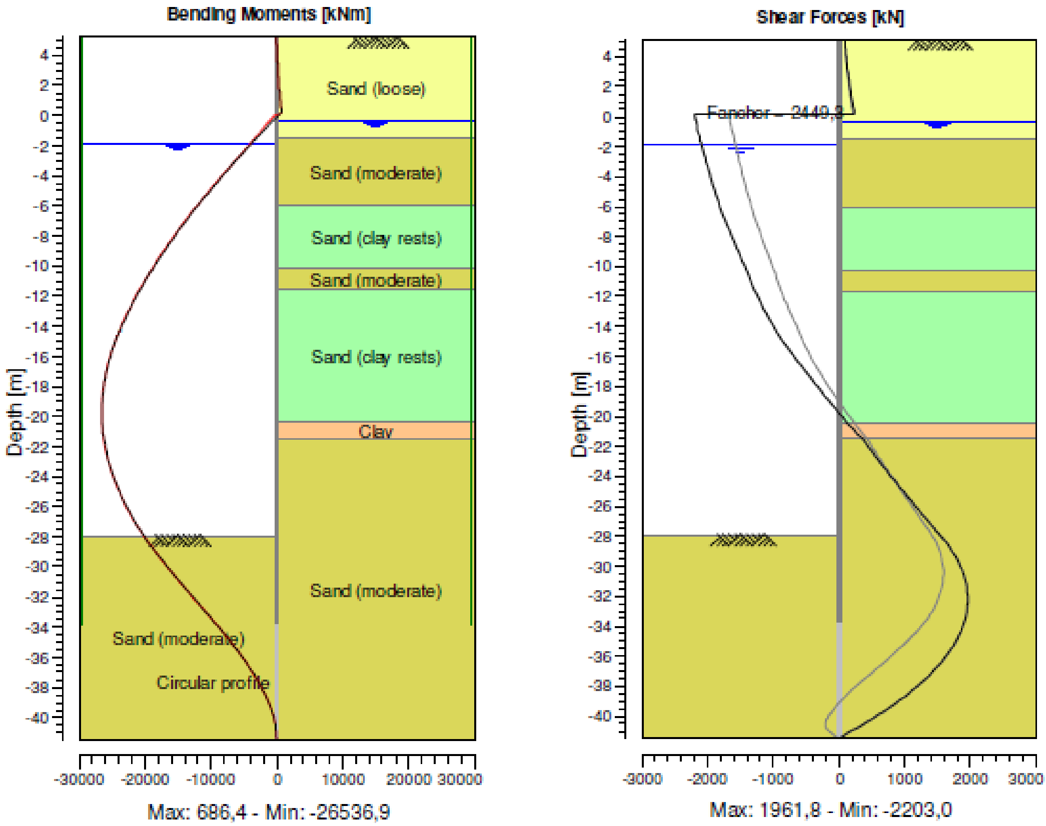


Figure V Moment and shear force distribution Option A, anchor wall at +0.2 m, Max. thickness

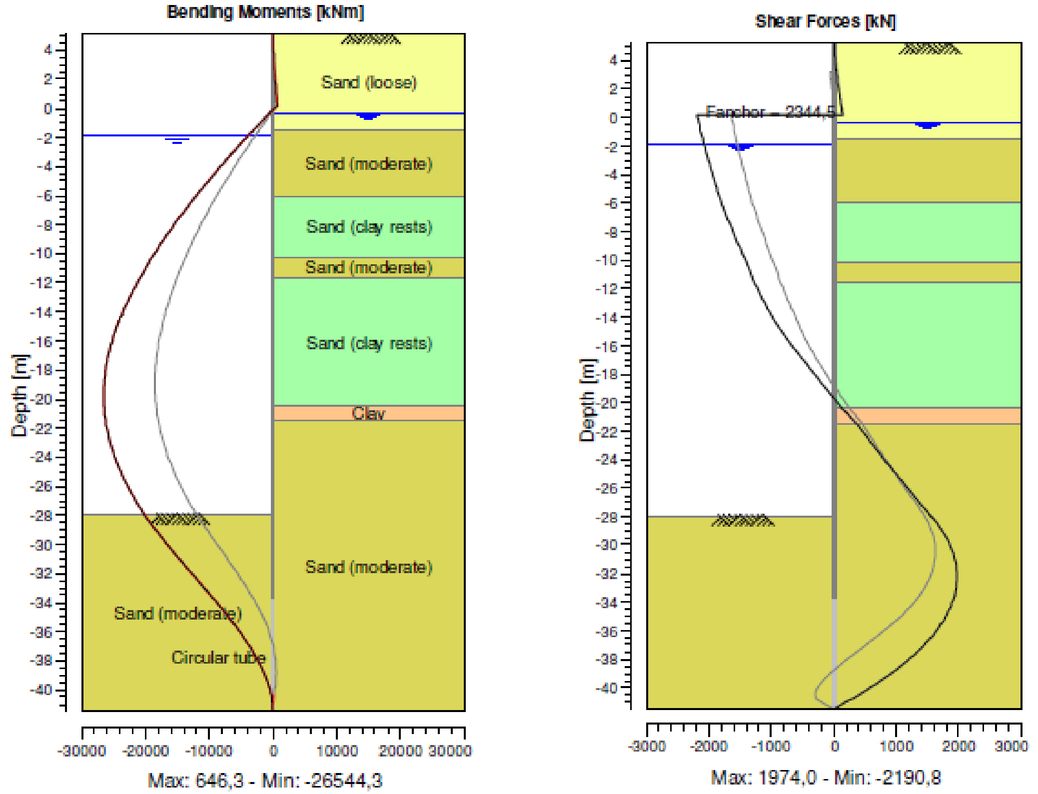


Figure W Moment and shear force distribution Option A, anchor wall at +0.2 m, Min. thickness

c) Anchor rod connection at -1.8 m NAP

Minimal length primary elements: 46.5 [m] (-41.3 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found at -20.4 m NAP and has a value of 24184 [kNm/m]. A primary member of 2.9 [m] diameter and 37 [mm] thickness satisfies the criteria for the moment capacity At -20.4 ± 10 m NAP the moment is equal to 15900 [kNm/m]. Here a reduction of 12 [mm] can be applied. The moment and shear force distribution of the wall with both thicknesses is shown in Figure X and Figure Y.

Anchor wall

The anchor plate capacity is calculated by means of horizontal equilibrium between the anchor force, the passive and active earth pressure. The calculation of the minimal anchor wall length is show in next table, the result of the iteration is shown in Figure Z.

Height from surface to top anchor wall [m]	5.00
Height anchor wall [m]	9.00
Contribution passive side [kN/m]	6603.60
Contribution active side [kN/m]	-326.57
contribution surface load [kN/m]	-510.08
Ra [kN/m]	5766.95
Rd [kN/m]	4119.25

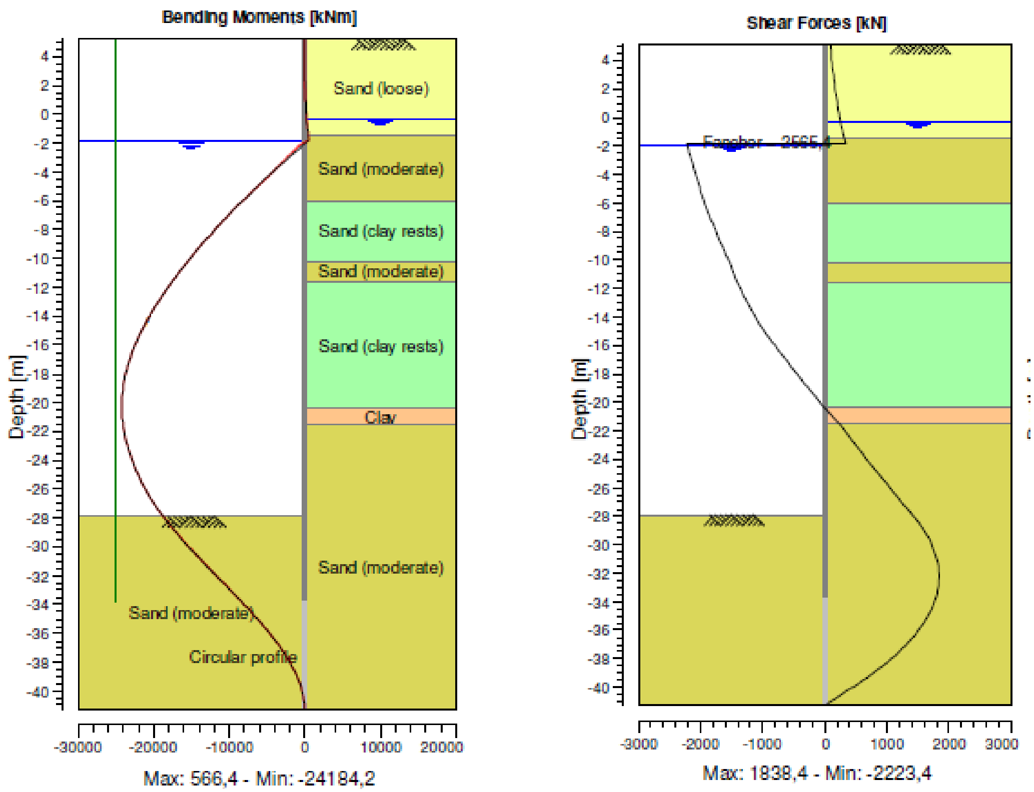


Figure X Moment and shear force distribution Option A, anchor wall at -1.8 m, Max. thickness

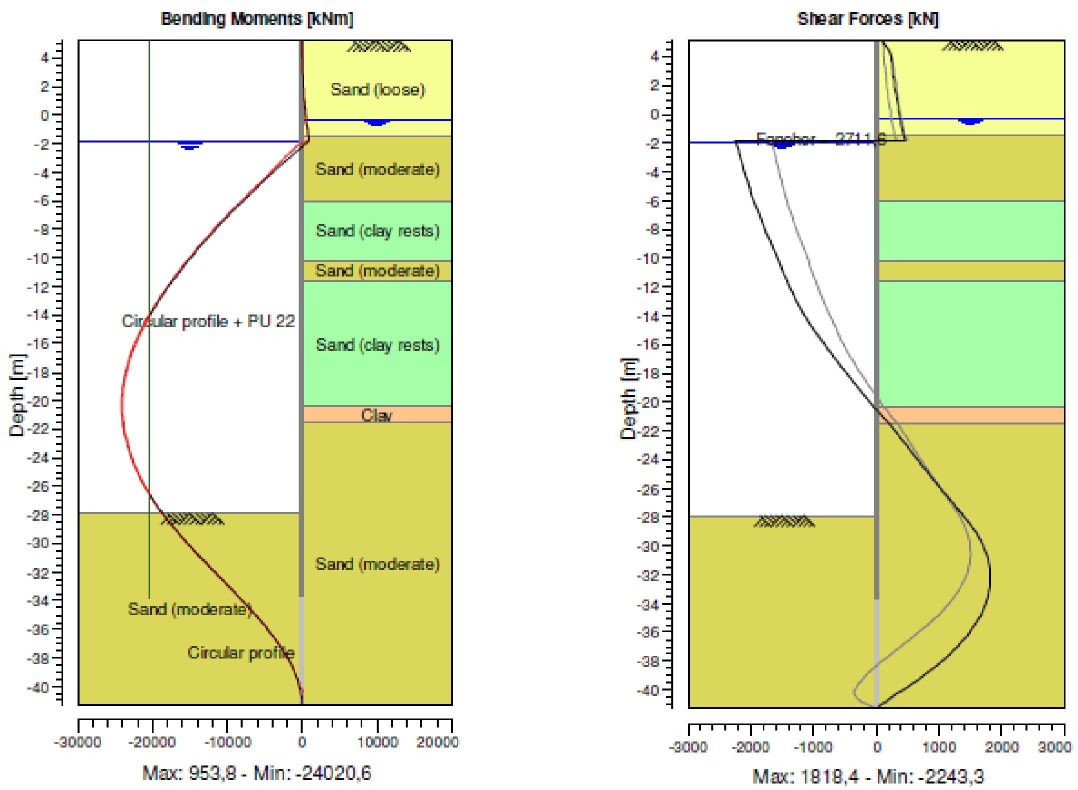


Figure Y Moment and shear force distribution Option A, anchor wall at -1.8 m, Min. Thickness

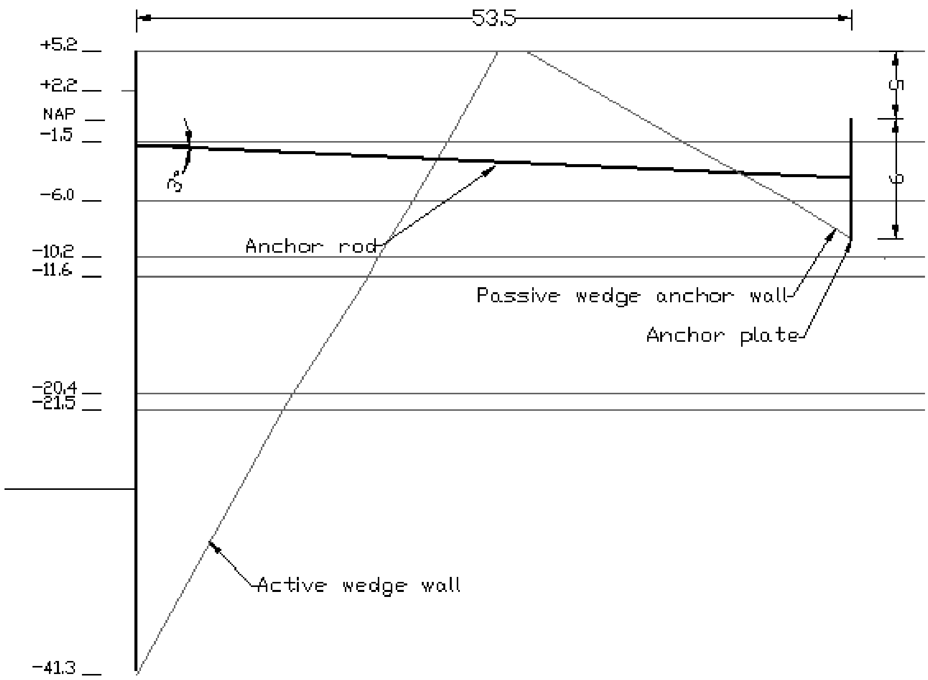


Figure Z Anchor wall connected at -1.8 m NAP

d) Anchor rod connection at -6.0 m NAP

Minimal length primary elements: 45.7 [m] (-40.5 m NAP)

Length secondary elements: 39 [m] (-33.8 m NAP)

Cross section primary members

The maximal moment is found at -21.8 m NAP and has a value of 18354 [kNm/m]. A primary member of 2.9 [m] diameter and thickness of 33 [mm] satisfies the criteria for the moment capacity. At -21.8 ± 10 m NAP the moment has a value of 10000 [kNm/m], the thickness can be reduced to 20 [mm] outside this area. The moment and shear force distribution of the wall with a thickness of 33 [mm] and 20 [mm] is shown in Figure AA and Figure BB.

Anchor wall

The anchor plate capacity is calculated by means of horizontal equilibrium between the anchor force, the passive and active earth pressure. The calculation of the minimal anchor wall length is show in next table, the result of the iteration is shown in Figure CC.

Height from surface to top anchor wall [m]	7.00
Height anchor wall [m]	9.00
Contribution passive side [kN/m]	7267.37
Contribution active side [kN/m]	-380.07
contribution surface load [kN/m]	-520.10
Ra [kN/m]	6367.21
Rd [kN/m]	4548.00

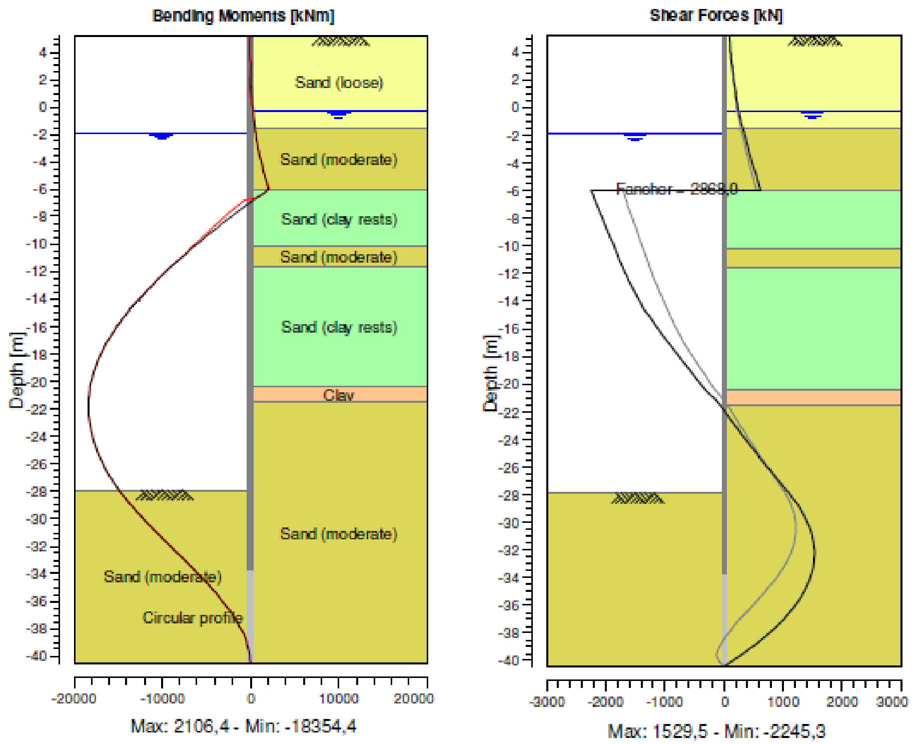


Figure AA Moment and shear force distribution Option A, anchor wall at -6.0 m. Max thickness

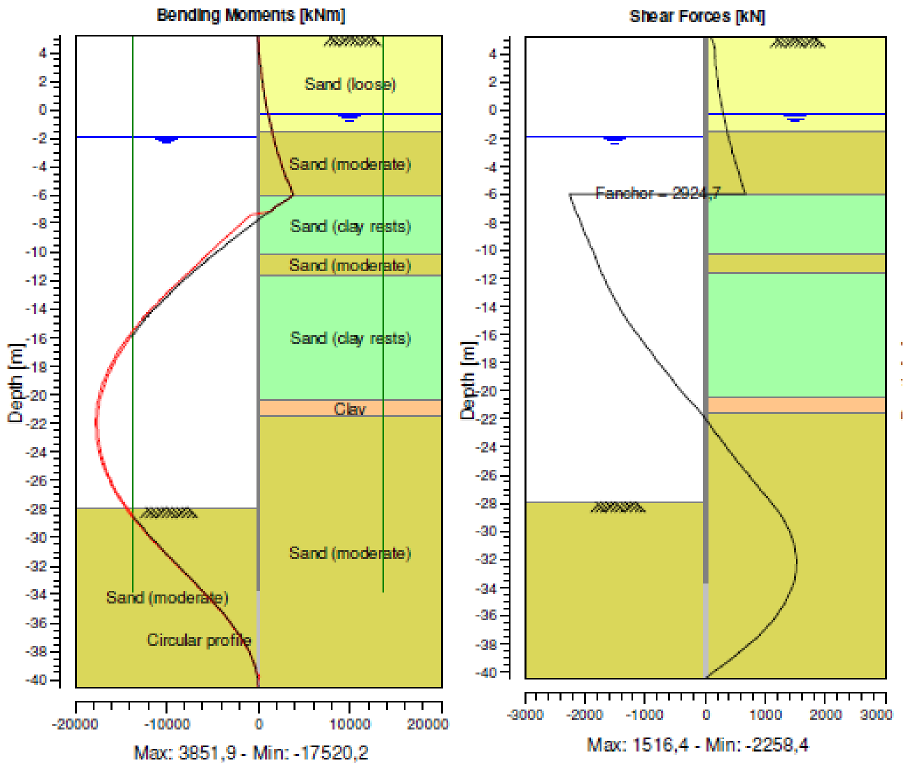


Figure BB Moment and shear force distribution Option A, anchor wall at -6.0 m. Min thickness

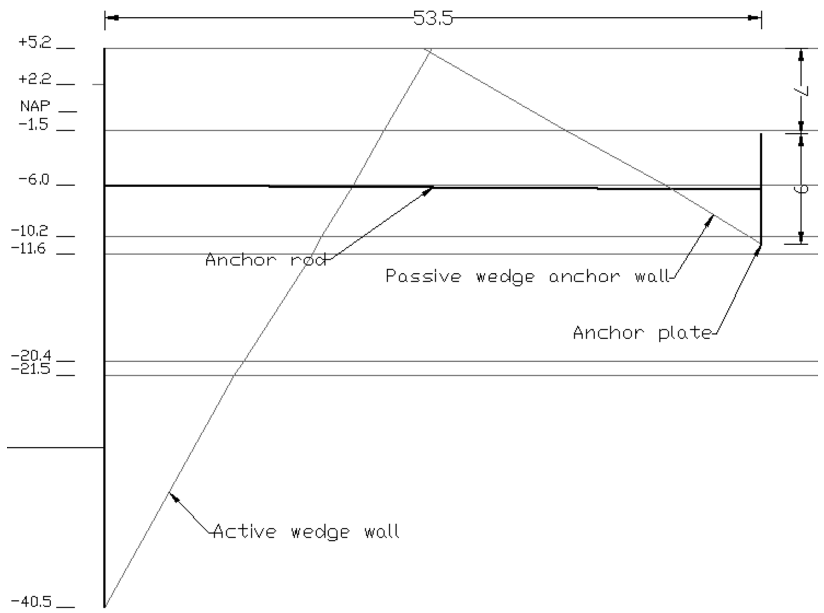


Figure CC Anchor wall connected at -6.0 m NAP

2.2 PLAXIS calculation (Step 5)

The results obtained in section 2.1 (step 4) are modelled in PLAXIS.

2.2.1 Model and results MV-pile

The MV-piles are modelled as a plate with parameters equal to the parameters of one MV-pile reduced by the centre to centre distance between the piles.

An impression of the model for the situation with the anchor at +2.2 m NAP is shown in Figure DD.

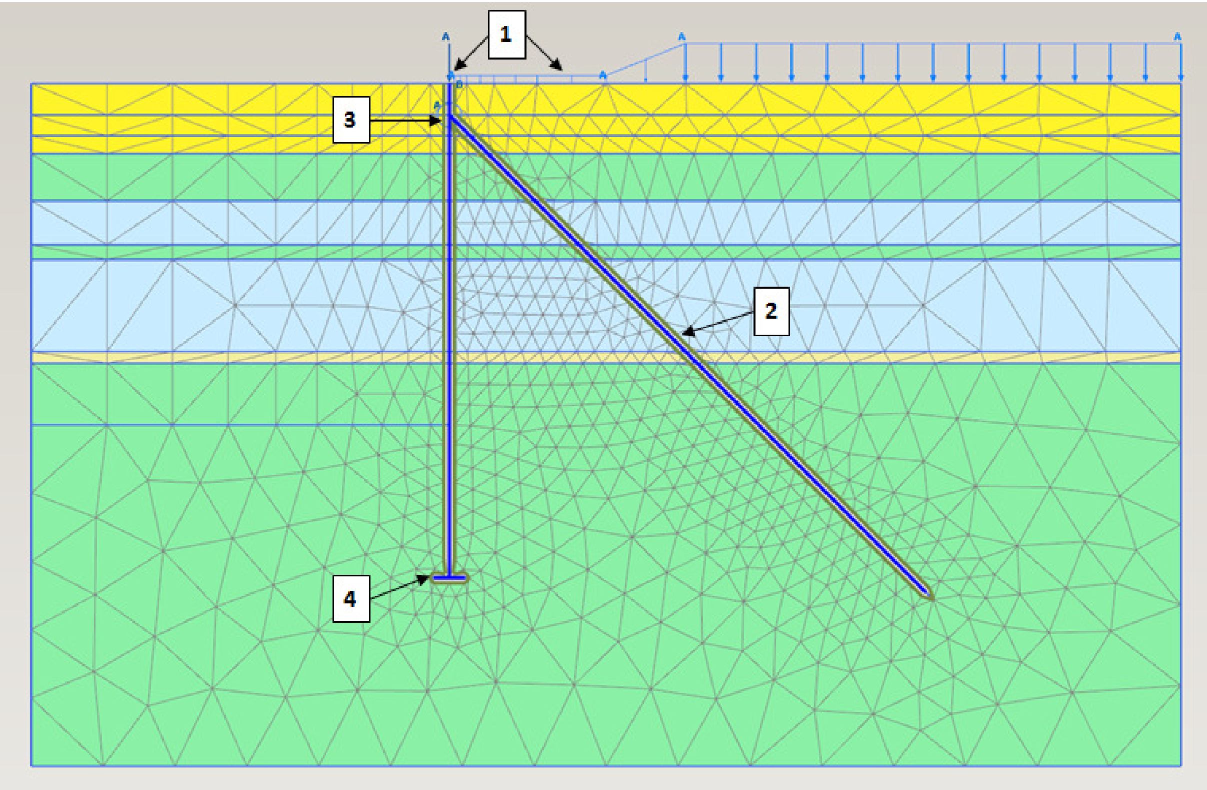


Figure DD Example PLAXIS model of a combined sheet pile wall with a MV-pile row at +2.2 m NAP

In the model:

1. Variable loads (Surface load crane load, fender load and bollard load);
2. MV-pile modelled as a plate with EI and EA = $(EI;EA)_{MV\text{-}pile}/L_{c.t.c.\ piles}$;
3. The connection between wall and MV-pile is modelled as a hinge;
4. Plate elements in PLAXIS that are used to model piles do not have end bearing capacity. The bearing capacity of the plate element is modelled as a really stiff plate as long as the diameter of the primary members. This way the deformation in the vertical direction is not completely restraint.

The results of the PLAXIS calculation are given in Table 2.1 of this annex. The maximal moment and the maximal anchor force in the ULS are present in LC2.1. In SLS, the maximal displacement and moment are found in LC3, the maximal anchor force in LC2.1.

Table 2.1 Result PLAXIS calculation Option A - MV-piles

Anchor level [m NAP]	+2.2	+0.2	-1.8	-6.0
SLS				
Displacement [m]	0.32	0.3	0.29	0.32
Moment [kNm/m]	22890	19780	17190	12222
Normal force [kN/m]	5268	5240	5330	5453
Anchor force [kN/m]	3351	3401	3443	3654
ULS				
Moment [kNm/m]	28470	24730	21170	15910
Normal force [kN/m]	5286	5244	5372	5457
Anchor force [kN/m]	3914	3913	3953	4499

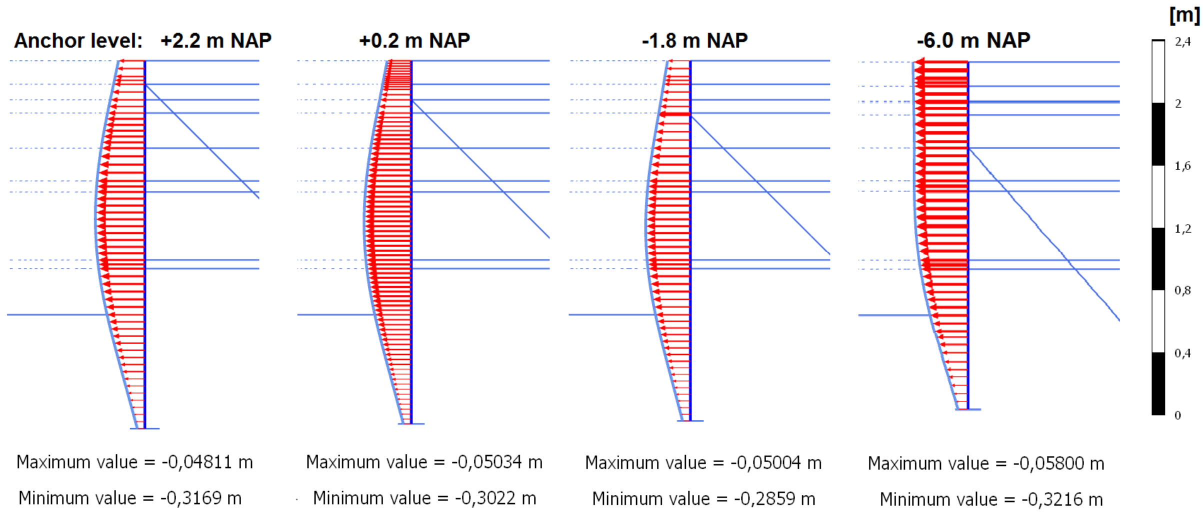


Figure EE Maximum displacement SLS Option A - MV-piles

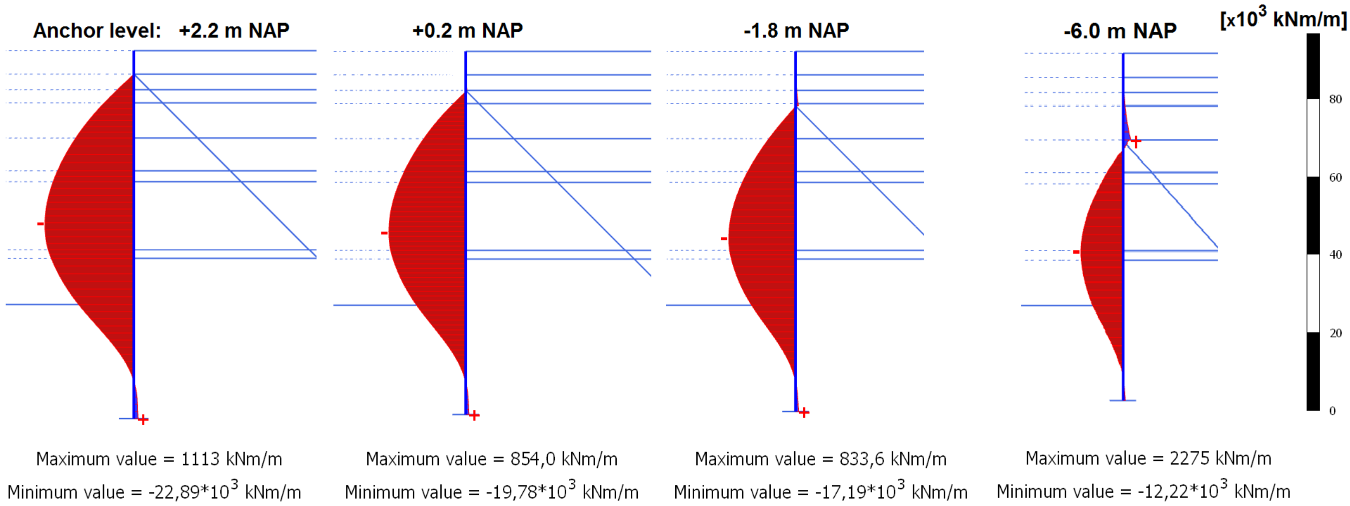


Figure FF Maximum moments SLS Option A - MV-piles

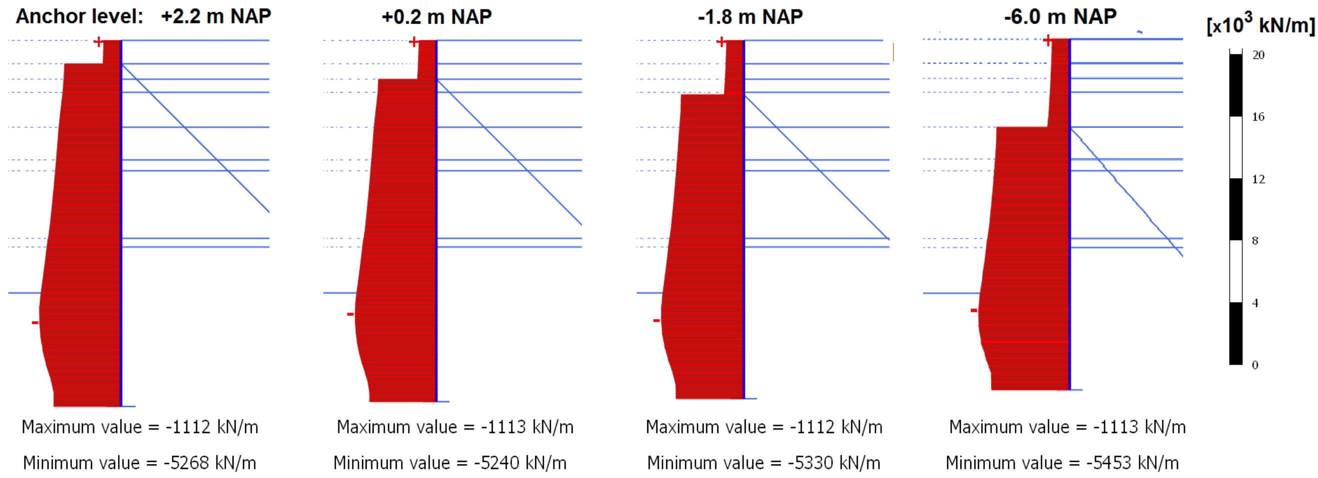


Figure GG Maximum normal force SLS Option A - MV-piles

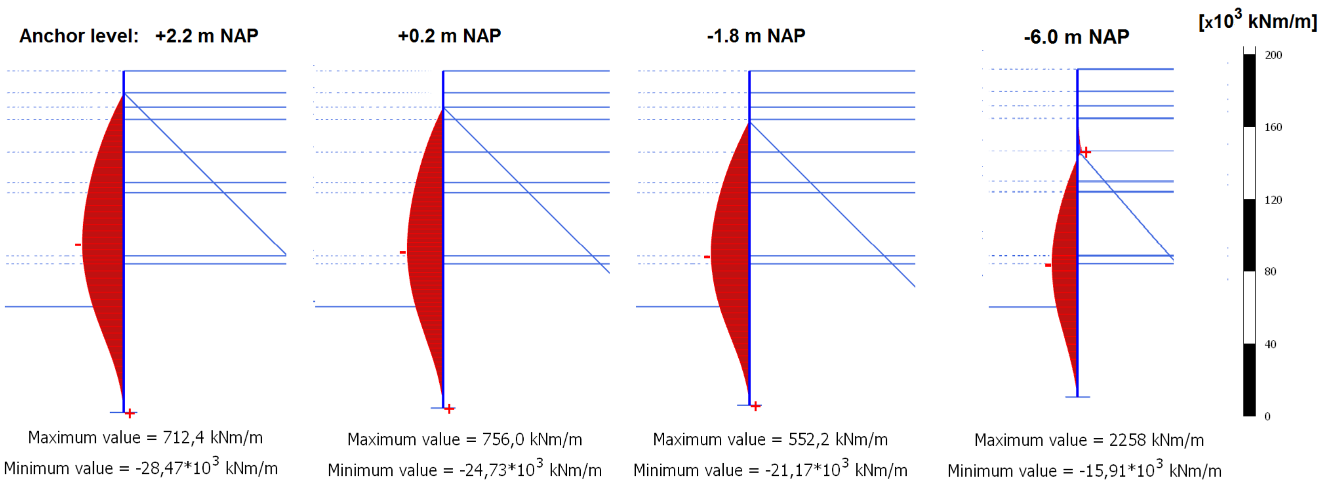


Figure HH Maximum moments ULS Option A - MV-piles

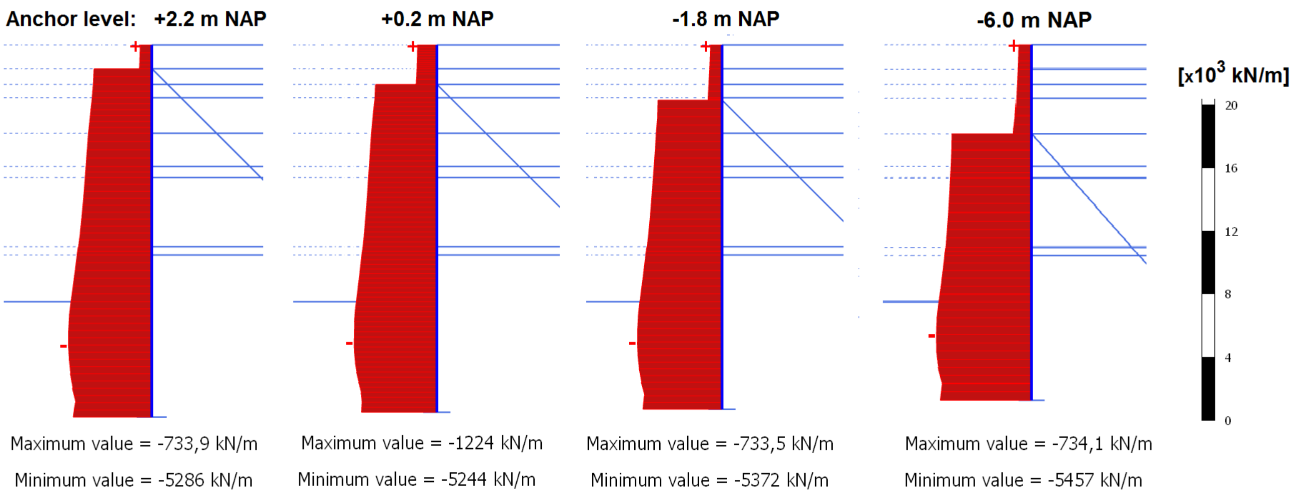


Figure II Maximum normal force ULS Option A - MV-piles

2.2.1 Model and results anchor wall

The results obtained in section 2.1 (step 4) are modelled in PLAXIS. The anchor wall is modelled as a plate. The parameters of the plate are determined by means of iteration in PLAXIS. A stiff plate is desired to prevent bending of the anchor plate and to restrain more displacement of the wall. The cross section of the plate is determined based on the moment distribution obtained in PLAXIS.

An impression of the model for the situation with the anchor at +2.2 m NAP is shown in Figure JJ.

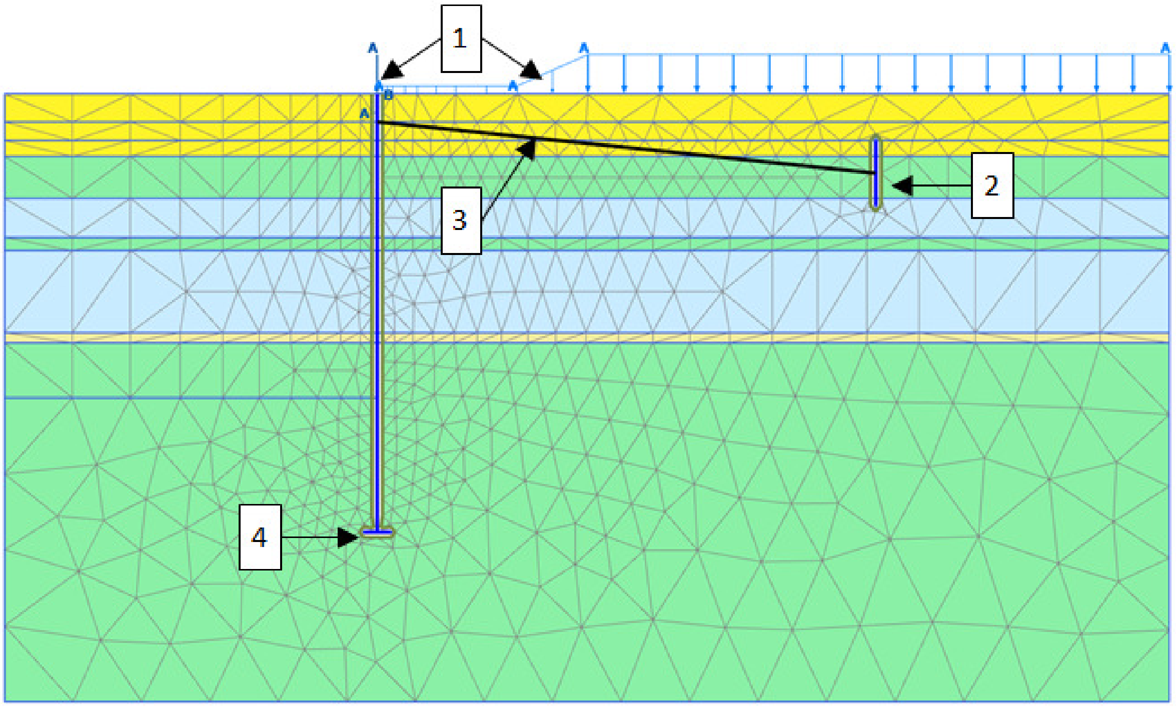


Figure JJ Example PLAXIS model of a combined sheet pile wall with a MV-pile row at +2.2 m NAP

Where:

1. Variable loads (Surface load crane load, fender load and bollard load);
2. Anchor plate modelled as a stiff plate;
3. Anchor rod modelled as a steel bar of 75 [mm] diameter;
4. As mentioned before. Plate elements in PLAXIS that are used to model piles do not have end bearing capacity. The bearing capacity of the plate element is modelled as a really stiff plate as long as the diameter of the primary members. This way the deformation in the vertical direction is not completely restraint.

The results of the PLAXIS calculation are given in Table 2.2 of this annex. The maximal moment and the maximal anchor force in the ULS are present in LC2.1. In the SLS, the maximal displacement and moment are found in LC3, the maximal anchor force in LC2.1.

Table 2.2 Max. Absolute values PLAXIS calculation option A - Anchor wall

Anchor level [m NAP]	+2.2	+0.2	-1.8	-6.0
SLS				
Displacement [m]	0.415	0.405	0.364	0.461
Moment [kNm/m]	21190	18750	18130	11880
Normal force [kN/m]	2841	2775	2835	2586
Anchor force [kN/m]	1898	1996	2110	2257
ULS				
Moment [kNm/m]	27650	24170	23010	16600
Normal force [kN/m]	2881	2765	2949	2723
Anchor force [kN/m]	2319	2396	2473	2722

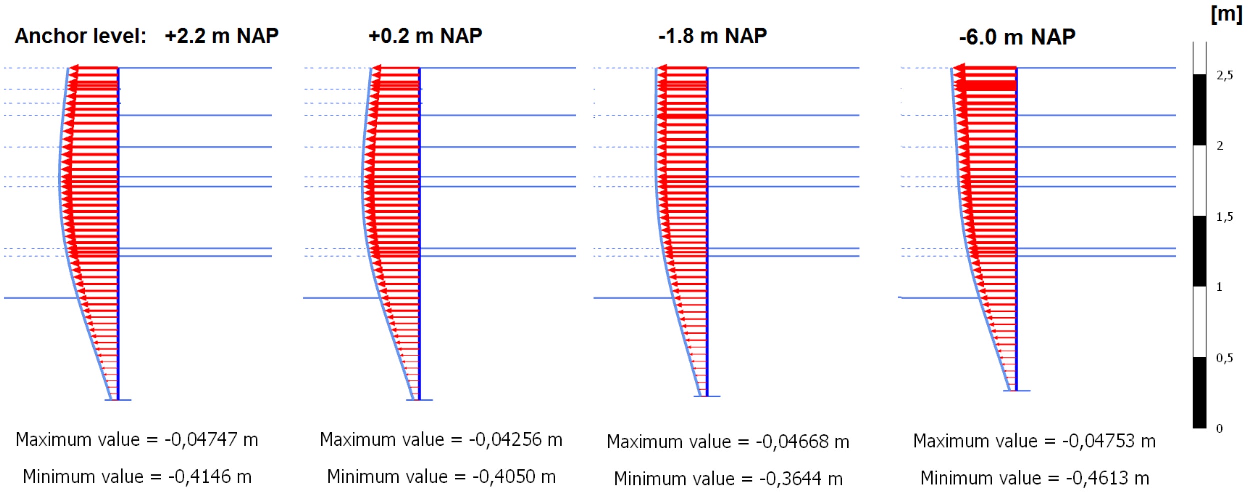


Figure KK Maximum displacement SLS Option A - Anchor plate

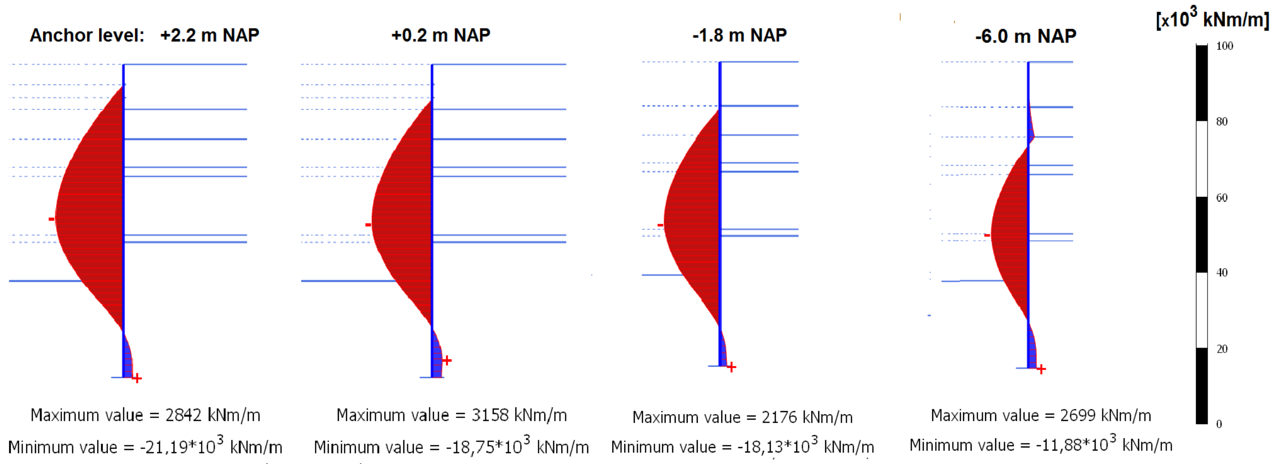


Figure LL Maximum moment SLS Option A - Anchor plate

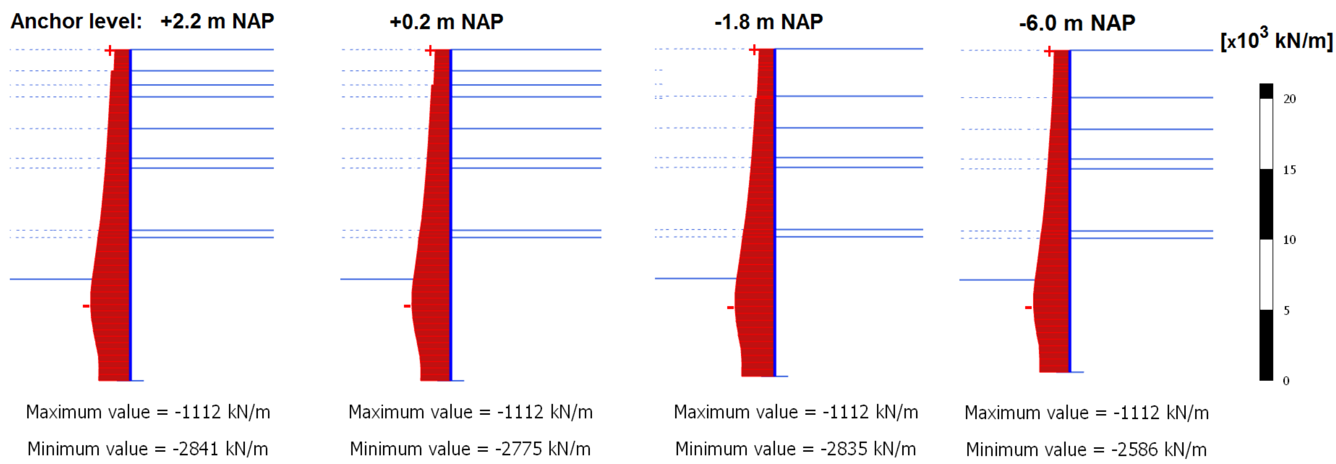


Figure MM Maximum normal force SLS Option A - Anchor plate

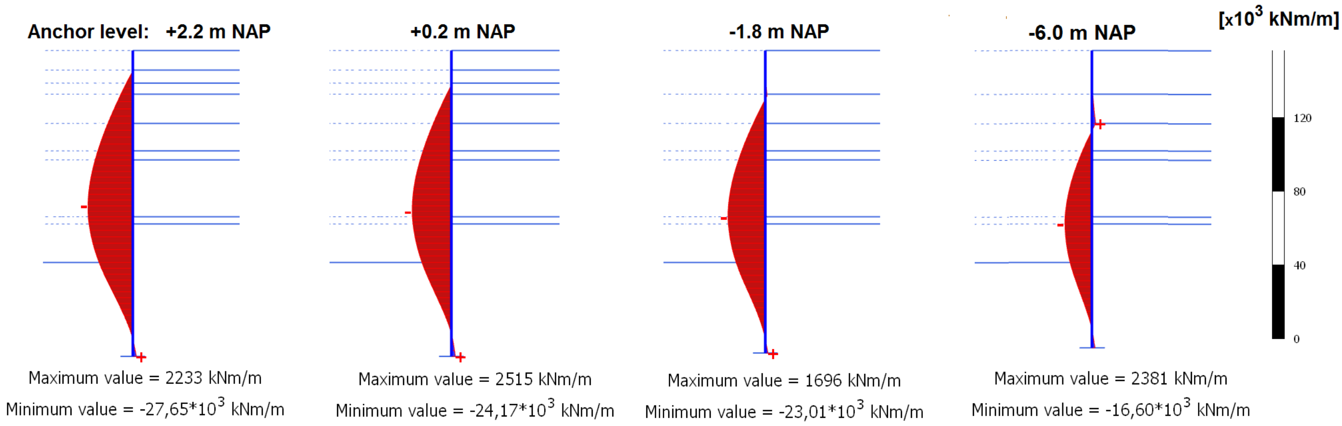


Figure NN Maximum moment ULS Option A - Anchor plate

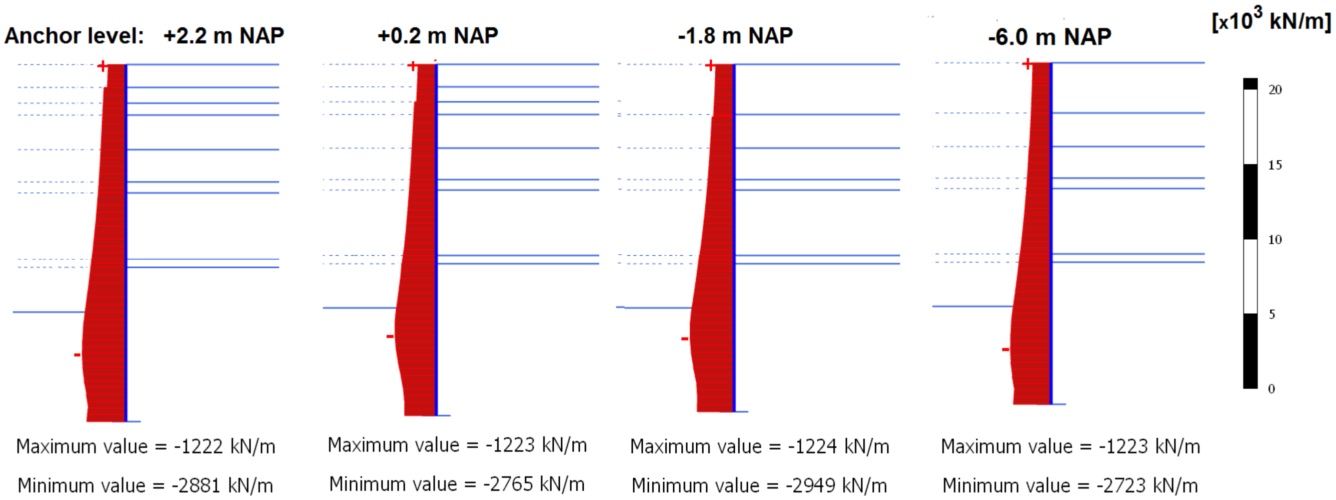


Figure OO Maximum normal force ULS Option A - Anchor plate

2.3Strength check primary elements (step 6)

The combination of moments and normal force is evaluated by means of a unity check. Two limits have to be controlled:

$$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1 \qquad \text{And} \qquad 1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}} \right) \leq 1$$

In the ULS, safety factors, reduction actors and soil parameter factors are used. The results are dependent on the way they are used (subject to interpretation). By applying a factor 1.2 to the representative values of the internal forces a 20% safety factor is considered acceptable (CUR166, 6th edition, 2012).

The unity checks for the three cases is show below, it has to be noted that the maximum bending moment and normal force are not present in the same place. The unity checks are made at the height of the maximal moment, where the check is decisive.

2.3.1 Summary of the properties of the wall

a) Combined wall with MV-pile

Properties wall				
Anchor position [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.0**
Diameter primary member [m]	2.8	2.8	2.8	2.8
Thickness [mm]	55	50	43	33*
Moment of inertia [m4]	0.447	0.4085	0.3540	0.2746
Yield stress [MPa]	480	500	500	500
System length [m]	4.6	4.6	4.6	4.6
Moment capacity [kNm/m]	33310.4	31714.34	27480.74	21318.04
Normal force capacity [kN/m]	49492.4	46953	45146.93	31180.65

b) Combined wall with anchor plate

Properties wall				
Anchor position [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.0**
Diameter primary member [m]	2.9	2.9	2.9	2.9
Thickness [mm]	50	44	37**	33
Moment of inertia [m4]	0.4547	0.4026	0.3410	0.3054
Yield stress [MPa]	500	500	500	500
System length [m]	4.7	4.7	4.7	4.7
Moment capacity [kNm/m]	33358.17	29538.90	25020.87	22408.80
Normal force capacity [kN/m]	54063.24	47873.40	40551.06	36317.71

* In a tater phase changed to 34 [mm] to satisfy the local buckling criteria

** In a later phase changed to 43 [mm] to satisfy the local buckling criteria

2.3.2 Unity check

a) Combined wall with MV-pile

ULS	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Absolute value moment [kNm/m]	28470	24730	21170	15910
Acting normal force [kN/m]	4539	5244	5378	4807
SLS				
Absolute value moment [kNm/m]	22890	19780	17190	12222
Acting normal force [kN/m]	4469	5240	5330	4908
Unity check				
$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1$	0.95	0.89	0.89	0.90
$1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}} \right) \leq 1$	0.93	0.88	0.89	0.88

b) Combined wall with anchor plate

ULS	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Absolute value moment [kNm/m]	27650	24170	23010	16600
Acting normal force [kN/m]	1761	1732	1702	1623
SLS				
Absolute value moment [kNm/m]	21190	18750	18130	11880
Acting normal force [kN/m]	2070	2060	2031	1965
Unity check				
$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1$	0.86	0.85	0.96	0.79
$1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}} \right) \leq 1$	0.81	0.81	0.93	0.70

All the designs satisfy the strength criteria, it needs to be noted that here the local buckling criteria was decisive.

2.1 Check anchor holding capacity (step 7)

The anchor capacity is also controlled in two ways. First the holding capacity must be bigger than 1.1 times the anchor force obtained in the ULS calculation. The other control is that the holding capacity has to be bigger than 1.32 times the anchor force in the SLS calculation (1.1*1.2*anchor force in SLS).

$$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1 \quad \text{and} \quad 1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$$

a) Combined wall with MV-piles

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor holding capacity [kN/m]	4113	4886	5663	6451

Anchor force in the ULS [kN/m]	3914	3913	3953	4499
Anchor force in the SLS [kN/m]	3351	3401	3443	3654

Check anchor holding capacity				
$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1$	1.05	0.88	0.77	0.77
$1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$	1.08	0.92	0.80	0.75

This table shows that the capacity of the anchor at +2.2 m NAP is not enough. An extra length of 3 meters is needed to meet this requirement. The anchors at +0.2 m NAP, -1.8 m NAP and -6.0 m NAP meet the requirement of the minimal holding capacity (an extra capacity of 10%-25% is achieved).

It needs to be noted that the stability is decisive for the length of the MV-piles and is determined further in this annex.

b) Combined wall with anchor plate

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor holding capacity [kN/m]	3235	3886	4909	6180

Anchor force in the ULS [kN/m]	2319	2396	2473	2722
Anchor force in the SLS [kN/m]	1898	1996	2110	2257

Check anchor holding capacity				
$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1$	0.79	0.68	0.54	0.49
$1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$	0.77	0.68	0.68	0.66

Another aspect that needs to be checked in this case is the anchor rod capacity, the same way a unity check is made where the anchor rod capacity has to be equal to 1.25 times the anchor force.

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor holding capacity [kN]	2524	2524	2524	2524

Anchor force per rod in the ULS [kN]	2319	2396	2473	2722

Check anchor holding capacity				
$1.25\frac{F_{anchor;ULS}}{R_{d;rod}} \leq 1$	1.15	1.19	1.22	1.35

When the anchor rods do not satisfy when they are placed at a centre to centre distance of 1 [m]. By reducing this distance the safety factor on the anchors can be met:

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor rod c.t.c. distance [m]	0.8	0.8	0.8	0.7
Anchor holding capacity [kN]	2524	2524	2524	2524

Anchor force per rod [kN]	1855.2	1916.8	1912	1936.2

Check anchor holding capacity				
$1.25\frac{F_{anchor;ULS}}{R_{d;rod}} \leq 1$	0.92	0.95	0.98	0.94

2.2 Displacements (step 8)

It is known that the functioning of a bulk terminal is not hindered by large deformations. However the deformations that take place in all variants is less than 0.5 [m], which is considered reasonable for bulk terminals.

2.3 Overall checks (step 9)

In this chapter the overall stability of the quay wall, the bearing capacity of the bearing piles, the kranz stability of the anchor and failure of one anchor is evaluated.

2.3.1 Overall stability

The overall stability is checked with the Bishop's method (Manual quay walls, CU211, 2005). D-sheet is used to find the decisive circle. The results for each case are:

a) Combined wall with Mv-piles

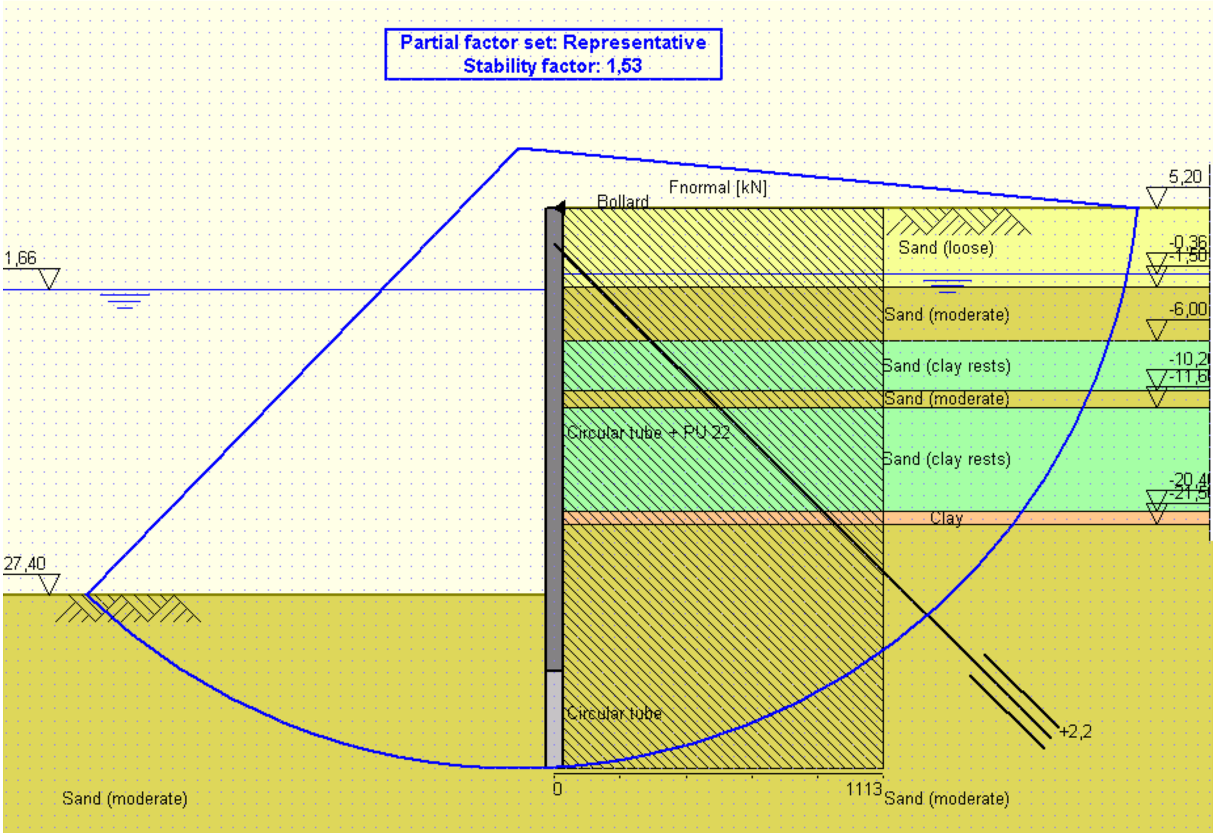


Figure PP Impression Bishop Stability criteria D-sheet Anchor position +2.2 m NAP, LC2.1.

	Stability factors			
Anchor position	**LC1**	**LC2.1**	**LC2.2**	**LC3**
+2.2 m NAP	1.56	1.53	1.53	1.52
+0.2 m NAP	1.53	1.51	1.51	1.49
-1.8 m NAP	1.51	1.49	1.49	1.48
-6.0 m NAP	1.46	1.44	1.44	1.42

Here factors higher or equal to 1.42 are achieved, satisfying the overall stability criterion (F_s>1.3).

b) Combined wall with anchor plate

The same calculation is performed for the combined wall with anchor plate. Here, all cases lead to a safety factor higher than 1.40, which is higher than 1.3 (Manual quay wall, CUR211, 2005).

	Stability factors			
Anchor position	**LC1**	**LC2.1**	**LC2.2**	**LC3**
+2.2 m NAP	1.55	1.52	1.52	1.51
+0.2 m NAP	1.53	1.50	1.51	1.49
-1.8 m NAP	1.51	1.48	1.48	1.47
-6.0 m NAP	1.44	1.44	1.42	1.40

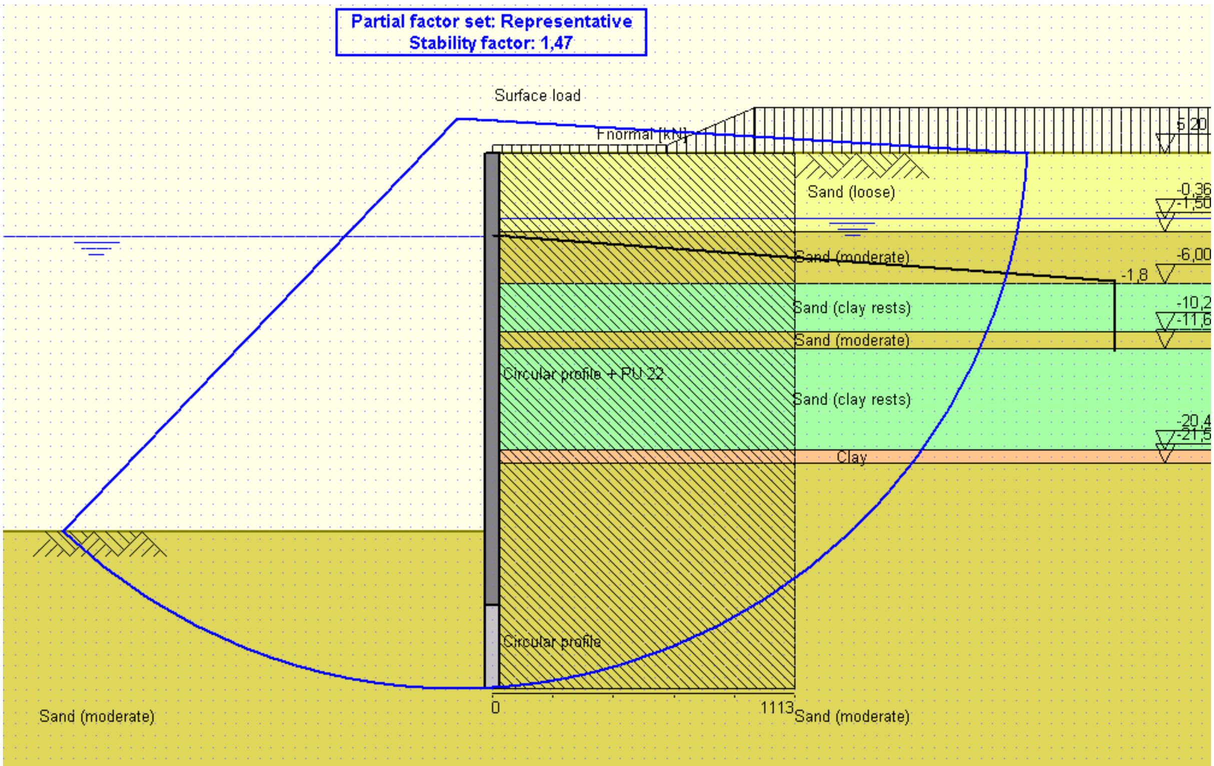


Figure QQ Impression Bishop Stability criteria D-sheet Anchor position -1.8 m NAP, LC3.

2.3.2 Bearing capacity primary members

The calculation is performed following the approach given in the Manual Quay walls (CU211, 2005). Here several assumptions are made, for a detail calculation more insight is needed in the parameters.

The bearing capacity is determined as the sum of the end bearing capacity and the shaft friction (inner and outer sides) of the pile. An important phenomenon is the plug formation. Plug formation can be developed in sandy soils due to the increase of stresses during driving of the piles. This heavy compressed plug may form in the pile as a result of which arching action may cause the development of a big upward directed force. This resistance cannot exceed the maximum resistance of the soil above the plug, also the plug formation influences the soil outside the pile increasing shaft friction in the outside wall.

Experience shows that the plugging phenomenon occurs at a depth of $8D_o$ – $10D_o$, at smaller penetration depth this phenomenon must be controlled. This principle is schematized in Figure RR. This is not the focus of this study and only a rough estimation of the bearing capacity is made.

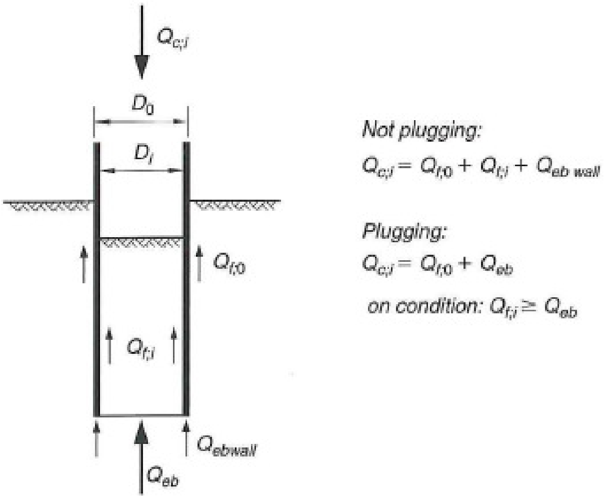


Figure RR Bearing capacity of a tubular pile (Manual quay walls, CUR211, 2005)

The end bearing capacity is defined as:

The tubular pile shaft friction on the outside of the tubular pile is defined as:

$$Q_{f;0} = \pi D_0 \int_0^{\Delta L} f_{c;o;z} dz$$

Where:

$f_{c;o;z}$ Maximum external friction with compression at depth z [kPa];

D_0 Outside diameter of the pile [m];

ΔL Pile length over which the friction force acts [m].

The ultimate end bearing capacity of the plugged pile is defined as:

$Q_{eb} = q_{eb} \pi D_0^2 / 4$ Where q_{eb} is the ultimate resistance in kPa and can be determined with:

$$\frac{q_{eb}}{p_a} = 8.5 \left(\frac{q_{c;av}}{p_a} \right)^{0.5} (DR)^{0.25}$$

Where:

$q_{c;av}$ Average of the measured cone resistance $q_{c;z}$ over the area $\pm 1.5 D_0$;

p_a Reference pressure 100 kPa;

DR displacement ratio = $1-(D_i / D_0)^2$ and D_i, D_0 Internal and external diameter respectively.

In case the penetration depth is less than $10D_0$, it is necessary to verify that the ultimate end bearing capacity of the plug is not greater than the sum of the total internal friction in the pile and the end bearing capacity of the wall of the tubular pile. The expression for this verification is:

$$Q_{eb} = \frac{q_{eb}\pi D_0^2}{4} < \left\{ \pi D_i \int_0^{PileTip} f_{ci}(z)dz + Q_{ebwall} \right\}$$

Where:

f_{ci} is the maximum friction for compression within the open tubular pile in kPa, this is assumed as $f_{ci} = 0.8\% \cdot q_{av} \leq 0.12[Mpa]$ (Tomlinson, 2008)

Q_{ebwall} the ultimate end bearing capacity of the wall of the tubular pile in kN/m,

$Q_{ebwall} = A_{tube} \cdot q_{av}[MN]$.

Reduction of the cone resistance

A factor that needs to be considered is the reduction of the cone resistance that is the effect of changes in overburden pressure. Changes in overburden pressure can result from excavations, scour of a river or seabed, or the loading of the ground surface by placing fill material. When piles are installed before excavation the reduction can be calculated as (Manual Quay walls,2005):

$$q_{c;z;excavation} = q_{c;z}\sqrt{\left(\frac{\sigma'_{v;z;excavation}}{\sigma'_{v;z;0}}\right)}$$

In which:

$q_{c;z;excavation}$	Cone resistance after excavation [Mpa]
$q_{c;z}$	Cone resistance before excavation [Mpa]
$\sigma'_{v;z;excavation}$	Vertical effective stress after excavation [Mpa]
$\sigma'_{v;z;0}$	Initial vertical stress before excavation [Mpa]

The reduction in cone resistance is calculated for each anchor depth and the design depth.

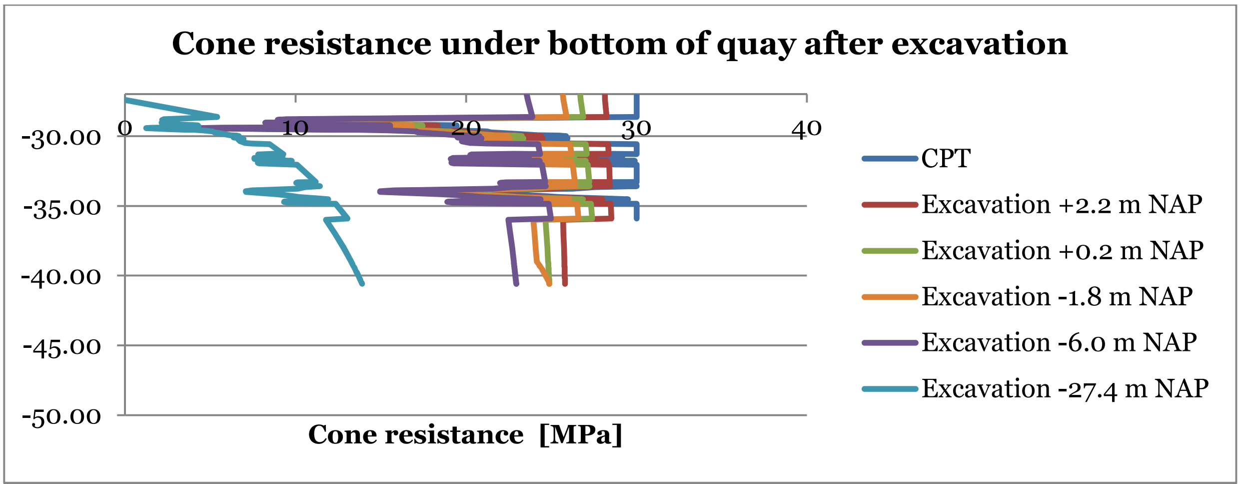


Figure SS Cone resistance reduction under bottom of quay due to excavation

The results for all variants following the calculation mentioned are:

a) Combined wall with MV-piles

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Embedded depth [m]	14.6	14	13.6	12.4
Diameter [m]	2.8	2.8	2.8	2.8
Thickness [mm]	34	33	30	30
Cone resistance port side in [Mpa]				
Average initial	24.28	24.28	24.28	24.28
Average reduced from -27.4 m NAP to -31 m NAP	4.75	4.75	4.75	4.75
Averaged reduced from -31 m NAP to -35 m NAP	9.62	9.62	9.62	9.62
Averaged reduced from -35 m NAP to bottom primary members	13.17	13.14	12.99	12.91
Cone resistance quay side in [Mpa]	22.61	21.59	20.89	19.42
Contribution to force [MN]				
Outside shaft friction port side	9.83	9.54	9.27	8.68
Outside shaft friction quay side	15.41	14.78	14.36	13.09
Inner shaft friction	6.74	6.54	6.53	5.94
End bearing capacity (plugged)	23.47	23.47	23.47	23.47
End bearing capacity (unplugged)	2.71	2.63	2.40	2.40
Total bearing capacity (plugged) [MN]	48.72	47.79	47.10	45.24
Total bearing capacity (Unplugged) [MN]	34.70	33.49	32.56	30.10
Bearing capacity needed [MN]	26.75	26.53	27.18	27.61
Unity check [MN]	1.30	1.26	1.20	1.09

The unity check is carried with a safety factor of 1.1. The calculation shows enough bearing capacity of the piles. This was expected considering the big dimensions of the primary elements.

b) Combined wall with anchor plate

	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Embedded depth [m]	14.4	14	13.5	11.9
Diameter [m]	2.9	2.9	2.9	2.9
Thickness [mm]	35	30	30	20
Cone resistance port side in [Mpa]				
Average initial	24.28	24.28	24.28	24.28
Average reduced from -27.4 m NAP to -31 m NAP	4.75	4.75	4.75	4.75
Averaged reduced from -31 m NAP to -35 m NAP	9.62	9.62	9.62	9.62
Averaged reduced from -35 m NAP to bottom primary members	13.66	13.57	13.46	13.17
Cone resistance quay side in [Mpa]	23.32	22.25	21.51	19.50
Contribution to force [MN]				
Outside shaft friction port side	10.39	10.14	9.82	8.89
Outside shaft friction quay side	15.74	15.31	14.76	13.01
Inner shaft friction	6.81	7.01	6.73	6.57
End bearing capacity (plugged)	25.36	25.36	25.36	25.36
End bearing capacity (unplugged)	2.94	2.53	2.53	1.69
Total bearing capacity (plugged) [MN]	51.50	50.81	49.94	47.26
Total bearing capacity (Unplugged) [MN]	35.88	34.98	33.84	30.16
Bearing capacity needed [MN]	14.89	14.30	14.93	13.78
Safety factor [MN]	2.41	2.45	2.27	2.19

The bearing capacity of the wall is by far met in the case of a combined wall with anchor plate. The reason is that the contribution to the vertical force made by the anchor is really small. In the case of the MV-pile, the contribution of the anchor to the vertical force is really high, leading to high needed bearing capacity.

2.3.3 Kranz stability

The Kranz stability is calculated as explained in chapter 2, where stability is reached when the ratio between the anchor force according to Kranz and the needed anchor force is higher than 1.5 (CUR166, 6th edition, 2012). The anchor force according to Kranz is calculated by means of equilibrium of forces in the failure plane of the anchor. Some assumptions of this failure plane are made (Figure TT):

- In the case of a sheet pile wall with an MV-pile, the failure plane is equal to the lines connecting the lowest zero shear point of the sheet pile wall, the shear centre of the MV pile (in this cased assumed to be at ½ of the effective length) and a vertical line to the surface;
- In the case of a sheet pile wall with an anchor plate the same failure plane is taken except that instead of the shear centre of the MV-pile the plane goes through the lowest point of the anchor plate.

Taken this failure planes the Kranz calculation is performed. All of the variants show instability of the anchor, the needed position and length is changed in order to satisfy this criterion. The anchor dimensions are presented in Table 2.3.

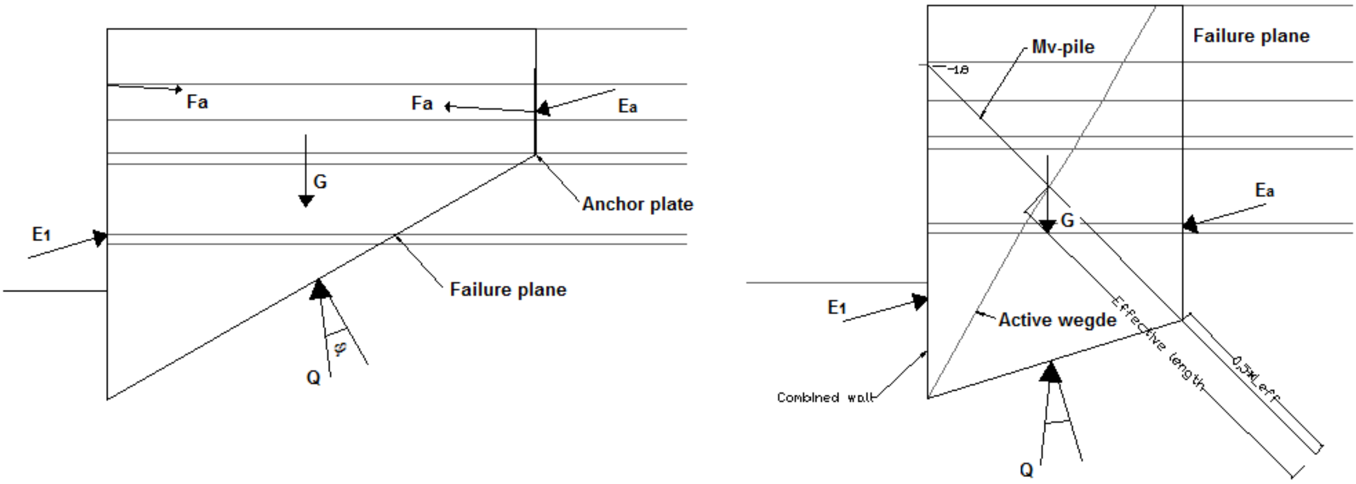


Figure TT KRANZ stability of anchors

Final result

Table 2.3 Results KRANZ stability

	MV-pile			
Position anchor [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.00**
Length MV-pile [m]	75	72	70	67
Safety factor	1.61	1.56	1.59	1.67

	Anchor plate			
Position anchor [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.00**
Toe anchor plate [m NAP]	-12.3	-13	-14	-14.5
Distance from quay wall [m]	55	55	55	55
Safety factor	1.50	1.51	1.53	1.52

Kranz calculation details

	MV-pile			
Position anchor [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.00**
Anchor force from calculation (SLS) [kN/m]	3363	3401	3443	3654
Horizontal distance from quay wall to shear level MV-pile [m NAP]	34.62	33	31.9	29.51
Zero shear level Wall [m NAP]	-42	-41.4	-41	-39.8
Surface load [kN/m²]	200	200	200	200

Horizontal components

Contribution of force on sheet pile wall [kN/m]	3723.95	3642.86	3589.33	3431.21
Contribution reaction of soil weights [kN/m]	4869.29	4951.63	5264.24	6161.55
Contribution of soil behind the failure plain [kN/m]	-4767.10	-4848.91	-4981.15	-5271.10
Horizontal component Franz [kN/m]	3826.15	3745.59	3872.42	4321.65
Angle anchor [degrees]	45	45	45	45
Fkranz in anchor direction [kN/m]	5410.99	5297.06	5476.43	6111.74
Safety factor (F_{kranz}/F_a)	1.61	1.56	1.59	1.67

	Anchor plate			
Position anchor [m NAP]	**+2.2**	**+0.2**	**-1.8**	**-6.00**
Anchor force from calculation (SLS) [kN/m]	1898	1996	2110	2257
Distance to shear level MV-pile/anchor wall [m NAP]	55	55	55	55
Zero shear level Wall [m NAP]	-41.8	-41.5	-41.3	-40.5
Surface load [kN/m²]	200	200	200	200

Horizontal components

Contribution of force on sheet pile wall [kN/m]	3696.82	3656.31	3629.44	3522.99
Contribution reaction of soil weights [kN/m]	724.25	1041.58	1455.96	1851.86
Contribution of soil behind the failure plain [kN/m]	-1612.72	-1716.00	-1866.81	-1943.7
Horizontal component Franz [kN/m]	2808.34	2981.89	3218.60	3431.19
Angle anchor [degrees]	9	7	3	2.5
F_{kranz} in anchor direction [kN/m]	2843.35	3004.28	3223.01	3434.46
Safety factor (F_{kranz}/F_a)	1.50	1.51	1.53	1.52

2.3.4 Failure of one anchor

To check this failure mechanism, the SLS load combination is taken in which the anchor force is decisive. When an anchor fails, adjacent anchors have to be able to take 1.5 times the anchor force. This increased anchor force is compared to the design value of the holding capacity of the anchor:

a) MV-pile

Anchor level	**+2.2 m NAP**	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor holding capacity [kN/m]	4113.1	4886	5663	6451.3
Max. Anchor force [kN/m]	3351	3401	3443	3654
Centre to centre distance anchors [m]	1.80	1.50	1.30	1.30
Force per anchor [MN]	6.03	5.10	4.48	4.75
1.5*Force per anchor [MN]	9.05	7.65	6.71	7.13
Anchor capacity [MN]	7.40	7.33	7.36	8.39
Safety anchor	0.82	0.96	1.10	1.18
Extra capacity needed [MN]	1.64	0.32	0.00	0.00
Extra length	4.32	0.85	0.00	0.00

The design holding capacity of the MV-piles cannot take ½ of the force of one nearby anchor. To be able to do this, all anchors need to be provided with an extra length that varies between 0 and 4.5 meters. This calculation is made in case of the length needed to satisfy the strength criteria, however, to satisfy the stability criteria the anchors need to be much longer.

b) Anchor wall

The anchor rods are decisive for the holding capacity of the anchor plate. When one anchor rod fails adjacent anchors have to be able to resist re-distributed force. It is assumed that the force is increased in 50%. To satisfy this criterion, the centre to centre distance of the anchor rods is changed.

Anchor level	**+2.2 m NAP**	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor rod capacity [kN]	2524	2524	2524	2524
Max. Anchor force [kN/m]	1898	1996	2110	2257
Centre to centre distance anchors [m]	1	1	1	1
Force per anchor SLS [MN]	1.898	1.996	2.11	2.257
1.5*Force per anchor [MN]	2.847	2.994	3.165	3.3855
Anchor capacity [MN]	2.524	2.524	2.524	2.524
Needed c.t.c. distance [m]	0.89	0.84	0.80	0.75

2.3.5 Local buckling

The control for local buckling is carried out as in the Handbook Quay Walls [18].

Local buckling check		Option A MV-pile +2.2 m NAP	Option A MV-pile +0.2 m NAP	Option A MV-pile -1.8 m NAP	Option A MV-pile -6.0 m NAP
Diameter	[m]	2.8	2.8	2.8	2.8
Thickness	[mm]	55.0	50.0	43.0	34.0
Soil pressure q	[kN/m²]	150.0	150.0	150.0	150.0
Stifness EI	[kNm²]	8.94E+07	8.17E+07	7.08E+07	5.65E+07
Yield stress	[N/mm²]	480.0	500.0	500.0	500.0
Young modulus	[N/mm²]	200000.0	200000.0	200000.0	200000.0
Moment capacity	[kNm]	153227.6	145886.0	126411.4	100926.0
Normal force capacity	[kN]	227664.9	215984.5	186219.5	147724.0
Acting moment per tube	[kNm]	130962.0	113758.0	97382.0	73186.0
Acting normal force per tube	[kN]	21477.4	24104.0	24518.0	22576.8
Acting moment per meter	[kNm/m]	28470.0	24730.0	21170.0	15910.0
Acting normal force per meter	[kN/m]	4669.0	5240.0	5330.0	4908.0
Output					
Reduction factor M_{Rd}	[-]	0.99	0.93	0.87	0.78
Moment resistance agains local buckling	[kNm]	1.52E+05	1.36E+05	1.10E+05	7.92E+04
Normal force recistance against local buckling	[kN]	2.26E+05	2.14E+05	1.84E+05	1.45E+05
Unity check	[-]	0.88	0.86	0.92	0.97
Conclusion unity check		OK	OK	OK	OK
Calculation steps					
Ratio D/t	[-]	50.91	56.00	65.12	82.35
Radius	[m]	1.40	1.40	1.40	1.40
Out of roundness tolerance*	[-]	0.01	0.01	0.01	0.01
Initial out of roundness	[mm]	7.00	7.00	7.00	7.00
Ovalisation and moments due to tensile forces secondary members					
Tensile force	[kN]	210.00	210.00	210.00	210.00
Ovalisation (decrease in radius)	[mm]	4.40E-04	4.82E-04	5.56E-04	6.96E-04
Ma	[kNm]	-93.58	-93.58	-93.58	-93.58
Mb	[kNm]	53.42	53.42	53.42	53.42
Ovalisation and moments due to soil pressure from one side					
Ovalisation	[mm]	2.69E-04	2.94E-04	3.39E-04	4.25E-04
Ma	[kNm]	-36.75	-36.75	-36.75	-36.75
Mb	[kNm]	47.86	47.86	47.86	47.86
Total ovalisation	[mm]	7.00	7.00	7.00	7.00
Actual radius (r')	[m]	1.42	1.42	1.42	1.42
Critical strain (epsilocr)	[-]	0.01	0.01	0.01	0.00
Reducing factors for the moment resistance					
g	[-]	0.99	0.99	0.99	0.98
meff;sd	[kNm]	14.53	14.53	14.53	14.53
mpl;Rd	[kNm]	330.00	284.09	210.11	131.36
Safety factor (gammao)	[-]	1.10	1.10	1.10	1.10
c1	[-]	1.96	1.96	1.94	1.90
Beta g	[-]	1.00	1.00	1.00	1.00
Beta s (empty)	[-]	1.00	0.94	0.88	0.80
Slenderness (mu)	[-]	2.99	2.52	2.03	1.39
* Depends the fabrication class and the diameter. Ur=0.01 for Class B D>=1.25 m					

Local buckling check		Option A			
		Anchor plate			
		+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Diameter	[m]	2.9	2.9	2.9	2.9
Thickness	[mm]	50.0	44.0	43.0	33.0
Soil pressure q	[kN/m²]	150.0	150.0	150.0	150.0
Stifness EI	[kNm²]	9.09E+07	8.05E+07	7.88E+07	6.11E+07
Yield stress	[N/mm²]	500.0	500.0	500.0	500.0
Young modulus	[N/mm²]	200000.0	200000.0	200000.0	200000.0
Moment capacity	[kNm]	156783.4	138832.9	135818.7	105321.4
Normal force capacity	[kN]	223838.5	197392.5	192973.9	148614.6
Acting moment per tube	[kNm]	129955.0	113599.0	108147.0	78020.0
Acting normal force per tube	[kN]	9729.0	9682.0	9545.7	9235.5
Acting moment per meter	[kNm/m]	27650.0	24170.0	23010.0	16600.0
Acting normal force per meter	[kN/m]	2070.0	2060.0	2031.0	1965.0
Output					
Reduction factor M_{Rd}	[-]	0.92	0.86	0.85	0.76
Moment resistance agains local buckling	[kNm]	1.44E+05	1.20E+05	1.16E+05	8.05E+04
Normal force recistance against local buckling	[kN]	2.22E+05	1.95E+05	1.91E+05	1.46E+05
Unity check	[-]	0.91	0.95	0.94	0.98
Conclusion unity check		OK	OK	OK	OK
Calculation steps					
Ratio D/t	[-]	58.00	65.91	67.44	87.88
Radius	[m]	1.45	1.45	1.45	1.45
Out of roundness tolerance*	[-]	0.01	0.01	0.01	0.01
Initial out of roundness	[mm]	7.25	7.25	7.25	7.25
Ovalisation and moments due to tensile forces secondary members					
Tensile force	[kN]	217.50	217.50	217.50	217.50
Ovalisation (decrease in radius)	[mm]	4.98E-04	5.63E-04	5.75E-04	7.41E-04
Ma	[kNm]	-100.39	-100.39	-100.39	-100.39
Mb	[kNm]	57.30	57.30	57.30	57.30
Ovalisation and moments due to soil pressure from one side					
Ovalisation	[mm]	3.04E-04	3.43E-04	3.51E-04	4.52E-04
Ma	[kNm]	-39.42	-39.42	-39.42	-39.42
Mb	[kNm]	51.34	51.34	51.34	51.34
Total ovalisation	[mm]	7.25	7.25	7.25	7.25
Actual radius (r')	[m]	1.47	1.47	1.47	1.47
Critical strain (epsilocr)	[-]	0.01	0.00	0.00	0.00
Reducing factors for the moment resistance					
g	[-]	0.99	0.99	0.99	0.98
meff;sd	[kNm]	15.58	15.58	15.58	15.58
mpl;Rd	[kNm]	284.09	220.00	210.11	123.75
Safety factor (gammao)	[-]	1.10	1.10	1.10	1.10
c1	[-]	1.95	1.94	1.93	1.89
Beta g	[-]	1.00	1.00	1.00	1.00
Beta s (empty)	[-]	0.92	0.87	0.87	0.78
Slenderness (mu)	[-]	2.40	1.99	1.92	1.24
* Depends the fabrication class and the diameter. Ur=0.01 for Class B D>=1.25 m					

2.4 Cross section anchor wall

The cross section of the anchor wall is determined based on the bending moments due to the anchor force. The bending moments are calculated with PLAXIS. Table 2.4 show the bending moment acting on the plate for each anchor level.

Table 2.4 Moment acting on anchor plate

Anchor position [m NAP]	+2.2	+0.2	-1.8	-6
Moment [kNm/m]	2202	2896	2966	3468

The moments are too high for a sheet pile wall. A combined wall of 1.1 [m] diameter and 20 [mm] thickness and 2 PU profiles is chosen as an anchor plate (bending moment capacity = 3755 [kNm/m]).

2.5 Displacements D-sheet

The displacements calculated in D-sheet for each anchor position are:

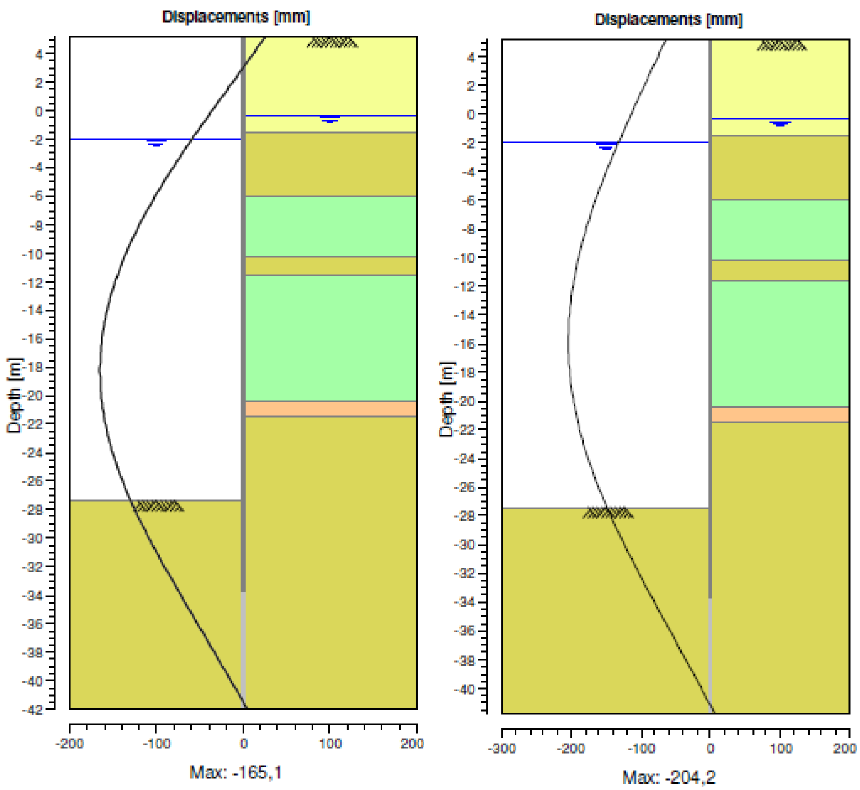


Figure UU Displacement D-sheet SLS anchor level +2.2 m NAP, left MV pile, right anchor plate

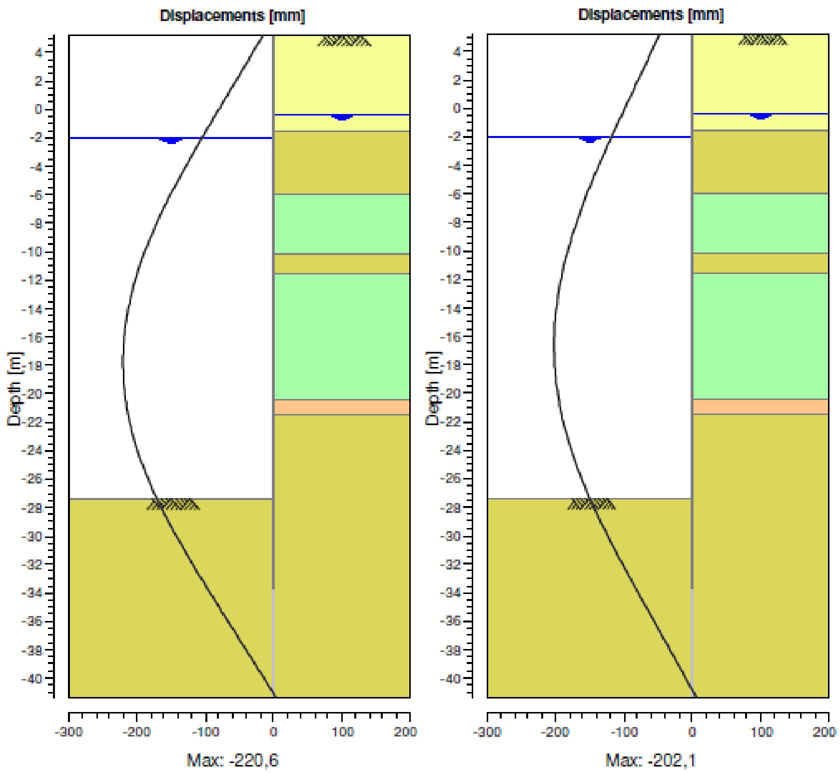


Figure VV Displacement D-sheet SLS anchor level +0.2 m NAP, left MV pile, right anchor plate

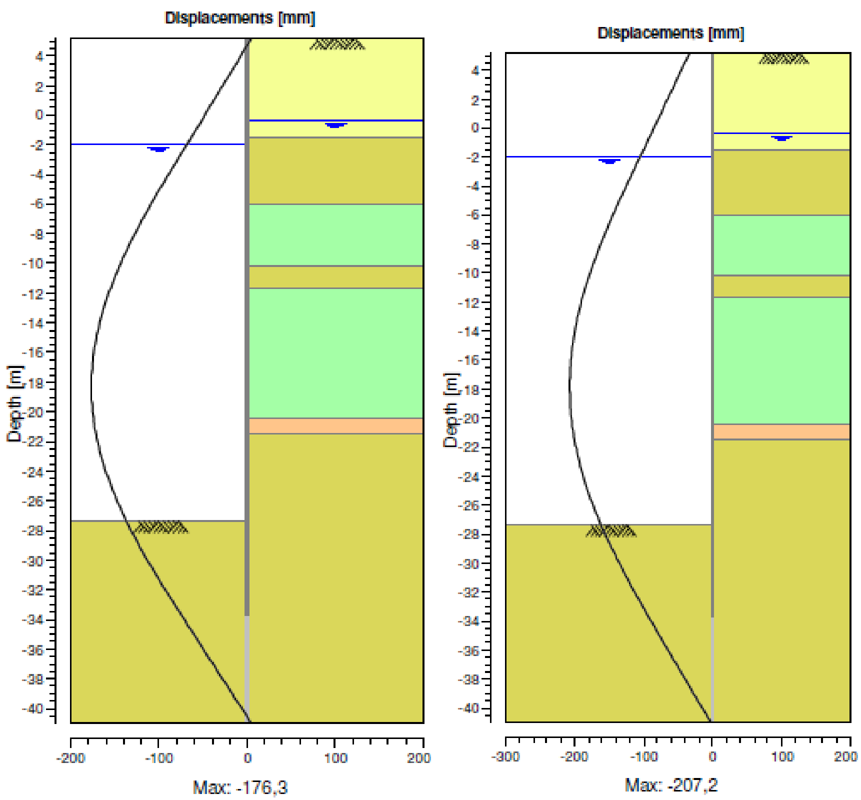


Figure WW Displacement D-sheet SLS anchor level -1.8 m NAP, left MV pile, right anchor plate

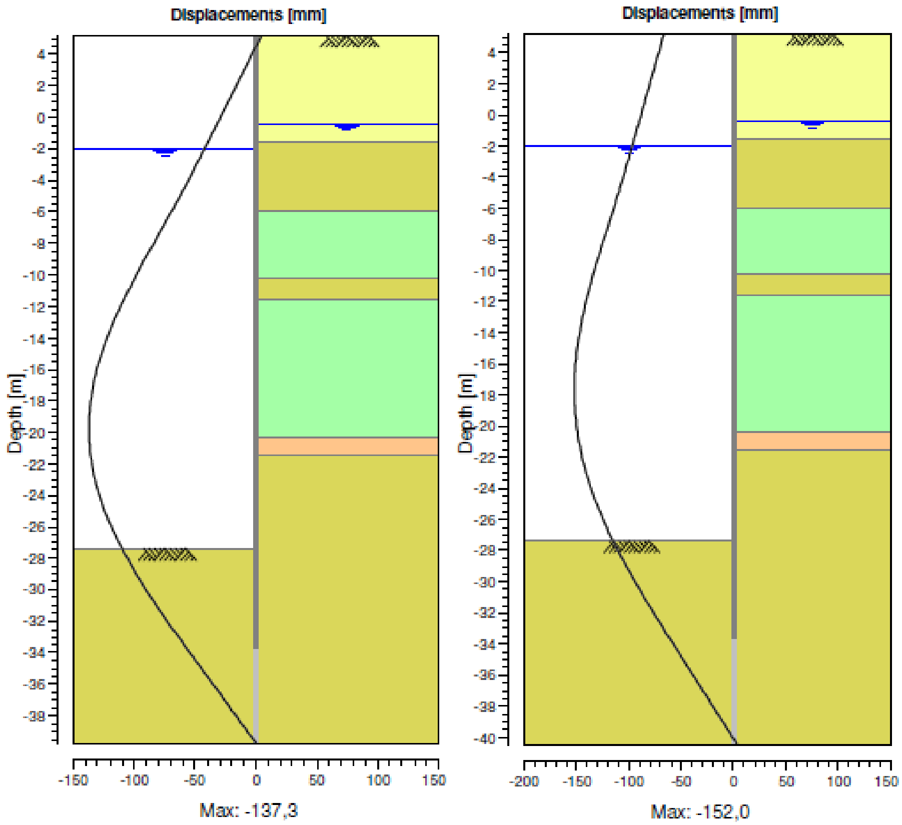


Figure XX Displacement D-sheet SLS anchor level -6.0 m NAP, left MV pile, right anchor plate

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Annex D: Calculation Option B: Combined sheet pile wall with relieving structure

Annex D: Calculation Option B: Combined sheet pile wall with relieving structure

In this attachment the calculations of the combined sheet pile wall with relieving structure is explained in more detail. As mentioned in chapter 4, the same approach as the one used in the design of Option A is used.

In this case the relieving structure is positioned at three heights (where the system line of the relieving structure will be placed). Namely:

- +2.2 m NAP, three meters from surface;
- - 1.8 m NAP, seven meters from surface;
- -6.0 m NAP, 11.2 m from surface.

The anchor is assumed to be connected at the same level as the system line of the relieving structure. In this case also two anchor types are evaluated (MV-pile and anchor plate).

D.1. Layout Option B

The layout of Option B for both anchor types is showed in chapter 4. The dimensions of the relieving structure are determined based on rules of thumb and designs in the past at the "Amazonehaven". The basic layout (without anchor) is showed in Figure 1.a. In this figure L is equal to 10 and 20 meters and H is equal to 3, 7 and 11.2 meters plus half thickness of the slab of the relieving structure.

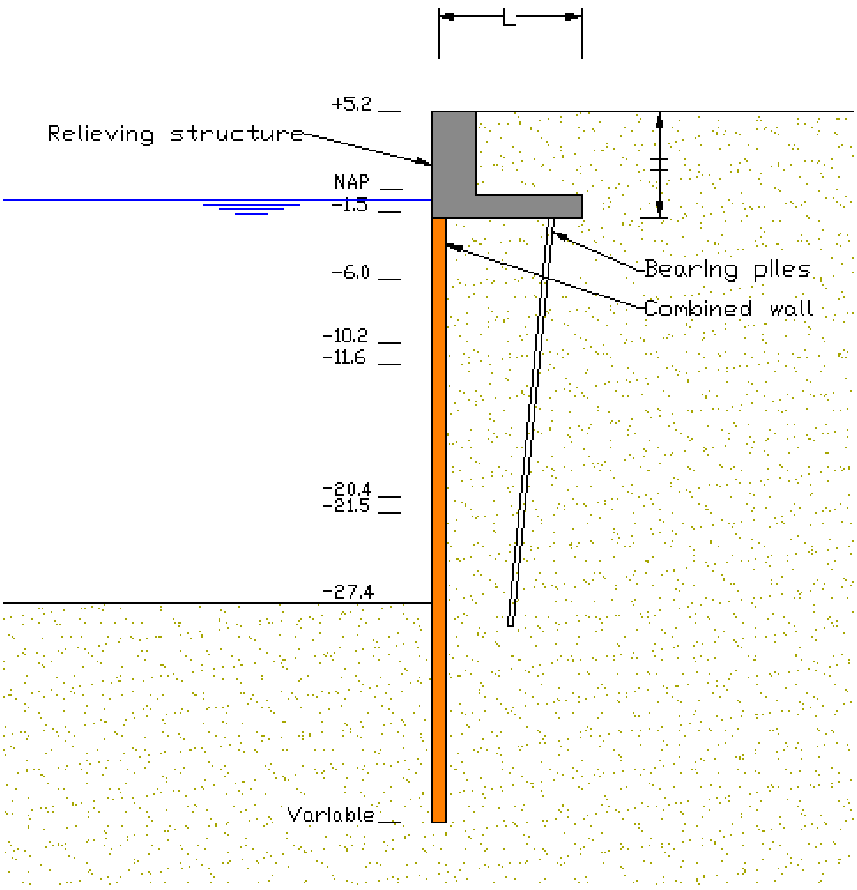

Figure 1.a Layout Option B (without anchor)

D.2. Calculation method

First, the relieving structure needs to be dimensioned. It is assumed that the relieving structure is infinite stiff, such that neutral horizontal pressures act on the relieving structure. The distribution of forces in the relieving structure is dependent on the bending stiffness of the elements. The designer has to be aware of the fact that concrete has a reduced stiffness when it is cracked. To take this into account the elasticity modulus of concrete is reduced with 60% when it is loaded only in bending and 80% when it is loaded by axial tension (Reference Dr. ir. C.R. Braam, Lecturer Concrete structures, TU Delft).

The same approach as in option A is used. First the relieving structure is modelled as a framework. Handbook Quay Walls (2005) advises to make a statically determined structure. However in this thesis is decided to include the anchors and bearing piles in the framework, obtaining a statically undetermined structure, this way the influence of the anchor in the horizontal spring stiffness is taken into account. In this model the soil is modelled as vertical and horizontal loads.

In D-sheet, the whole relieving structure is modelled as a horizontal spring. To determine the spring stiffness a horizontal load at the level of the slab of the relieving structure is applied, the spring stiffness is equal to the force divided by the displacement. The resultant vertical and horizontal forces from this framework calculation are used in D-sheet as input to determine the bending moments and anchor forces. With these results the wall can be dimensioned. After this, the wall is modelled in PLAXIS and the results are compared. The results showed that the design is too conservative and the design was optimized in PLAXIS. The results of this optimization are used to check the stability and strength of the wall and its elements.

2.1 Application of D-sheet and PLAXIS

The design process of option B is more complicated than option A. The main problem here is that D-sheet can be used to calculate retaining walls without including any other type of structure. In this case the relieving structure should be modelled as a spring. The calculation of the spring stiffness is explained in section 3.1.2 of this annex. When this is done, the forces on the retaining wall can be calculated with D-sheet and the relieving structure as a framework separately. The advantage of PLAXIS is that the calculation of the retaining wall and relieving structure is made together taking into account the deformation of all plate elements, deformation of soil, displacement of the supports and interaction between these aspects that influence the force distribution in the structure.

2.2 Calculation steps

The calculation steps followed are:

1. Determine the start parameters (water levels, soil parameters, design depth, etc.);
2. Establish load combinations, safety and reduction factors;
3. Asses the dimensions of the relieving structure, check the sliding of the quay due to the clay layer and calculate the internal forces;
4. Asses the dimension of the primary elements of the wal, embedding length and anchor force with D-sheet (model the anchor and relieving structure as a spring support). Start an iteration process between the embedding depth, wall and anchor stiffness;
5. Use the result in step 4 as input in PLAXIS and perform a calculation;
6. Check the strength of the primary elements with the results obtained in step 5;
7. Check the anchor strength with the results obtained in step 5;
8. Check the deformations;

9. Overall checks (Kranz stability, overall stability, bearing capacity of the primary elements, failure of one anchor and loacal buckling).

Finally, the reinforcement of the relieving structure needs to be calculated and cracking needs to be controlled.

D.3. Calculations

3.1 Relieving structure (step 3)

The relieving structure is made of concrete and has two main parts: The slab that it has the function to transfer the loads to the foundation members and the beam that transfers the horizontal loads and some of the vertical loads to the slab that transfers these to the bearing members and anchor. The length of the relieving structure depends on the relieving effect that needs to be achieved in the retaining wall. Due to the limited time in this thesis it is chosen to take two floor lengths into consideration, namely 10 and 20 meters. This way the influence of the length and height of the relieving structure in the design can be investigated.

3.1.1 Sliding due to clay layer

When a clay layer is present, risk of sliding of the quay wall is present. Enough resistance to sliding is achieved when there is enough soil under the relieving structure. The resistance against sliding due to the clay layer can be controlled by means of simple equilibrium. Table 3.a shows the result of the calculation, where: H1 is the horizontal force of the soil below the relieving structure (this is the reaction, thus opposite sign), w1 is the friction of the wall and soil. H2 is the horizontal pressure of the soil behind the relieving structure, here the influence of surface load is taken into account. G is the soil mass under the relieving structure. Rv is the resultant of the vertical forces and Rh the horizontal component of the resultant force that has an angle φ with the clay layer. When Rh+H1-H2<0 an extra anchor force has to be taken into account to compensate this. If Rh+H1-H2>0 no extra anchor force is needed.

The calculation is given in Table 3.a, in some cases an extra anchor force is necessary.

Table 3.a Extra anchor force due to clay layer

	Length floor = 10 m			Length floor = 20 m		
Level relieving structure [m NAP]	**+2.2**	**-1.8**	**-6.00**	**+2.2**	**-1.8**	**-6.00**
H1 [kN/m]	978.23	608.93	384.90	978.23	608.93	384.90
W1 [kN/m]	312.24	189.79	117.79	312.24	189.79	117.79
H2 [kN/m]	2114.76	2159.25	1899.57	2114.76	2159.25	1899.57
W2 [kN/m]	1074.42	1087.44	925.17	1074.42	1087.44	925.17
G [kN/m]	2557.80	1990.00	1570.00	5115.60	3980.00	3140.00
ΣV = -Rv [kN/m]	3319.98	2887.65	2377.38	5877.78	4877.65	3947.38
Rh [kN/m]	1208.37	1051.02	865.30	2139.34	1775.32	1436.73
H1+Rh-H2 [kN/m]	71.84	-499.30	-649.37	1002.81	225.00	-77.94
Extra anchor force [kN/m]	0.00	499.30	649.37	0.00	0.00	77.94

Note: In this calculation no horizontal force due to bearing piles is taken into account, this contribution could lead to enough horizontal resistance.

3.1.2 Horizontal stiffness

The relieving structure is schematized as framework. Handbook quay walls (2005) advices to model the relieving structure as a statically determined structure. In this thesis several models are considered but only small differences in the horizontal stiffness is obtained, the reason is that the main contribution to the horizontal stiffness comes from the anchor and bearing piles.

The elements that contribute to the horizontal stiffness of the relieving structure are mainly the combined wall, the bearing piles, the anchor and the slab of the relieving structure. Because it is assumed that the slab is connected by a hinge it does not has much resistance to horizontal loads. The contribution to the horizontal stiffness of the combined wall is by means of bending, of the bearing piles and anchor by means of bending and/or axial stiffness (depending on the slope).

The contribution of the combined wall is neglected and modelled as a translation support that can only resist vertical forces. The bearing piles and anchors are modelled as beams, but as mentioned before the axial stiffness contributes the most to the horizontal stiffness of the system.

Figure 3.a shows the framework model of the determination of the spring stiffness. Here, the MV-piles and anchor rod are taken into account. It needs to be noted that the slope of the bearing piles has a positive influence in the horizontal stiffness of the relieving structure. In this thesis two slopes are assumed, 1:7 for a length of 10 meters and 1:3 for a length of 20 meters. The slope of the bearing piles was determined in such way that they do not come in conflict with the combined wall and the points are situated at the lowest sand layer (bearing layer).

The model is shown in Figure 3.a and the spring stiffness determined for each case are:

Table 3.b Spring stiffness for a slab length (L) used in the D-sheet calculation in [kN/m]

Anchor position [m NAP]	MV-pile		Anchor plate	
	L=10 [m]	L=20 [m]	L=10 [m]	L=20 [m]
+2.2	16949	18868	15625	23810
-1.8	16949	18868	14085	20408
-6.0	16949	18868	15152	25000

3.1.3 Main dimensions and schematization of the relieving structure

The connection with the primary member is modelled as a hinge. The dimensioning of the anchors will take place together with the dimensioning of the wall. The anchors and bearing piles are modelled as bars with a corrected spring stiffness equal to one MV-pile (or bearing pile) divided by the centre to centre distance of the piles. The models for each anchor type are shown in Figure 3.a.

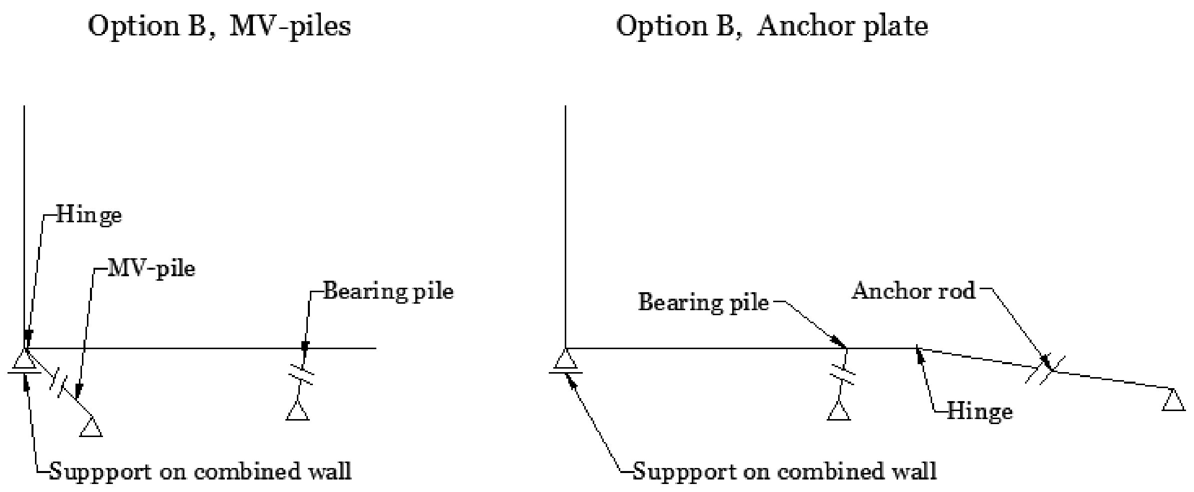

Figure 3.a Schematization relieving structure

3.1.1 Main dimensions superstructure

The thickness of the floor and wall in the relieving structure is determined based on rules of thumb. The "infinite" length of the quay gives the possibility to calculate the internal forces as a beam with a width of 1 meter. The rule of thumb used for the height of a concrete beam is h=1/10*L (Constructieleer Gewapend Beton, C.R. Braam, 2008). With this rule of thumb the thickness of the floor of 1 meter in case of a span length of 10 m and 2 m in case of a span of 20 meter. During the calculation the thickness of the floor of 20 m length is changed to 1.8 m to reduce the costs of concrete.

3.1.2 Loads on superstructure

The assumption of an infinite stiff structure leads to the use of the neutral coefficient of horizontal pressure (K_0=1-sinφ). Vertical forces acting on the superstructure consist of the surface load, crane load, water pressure and dead weight of the concrete. The horizontal loads are: Bollard load, fender load, collision load, soil and water pressure behind the wall (note that the horizontal force of the crane also needs to be taken into account, this is not done in the design of the quay wall without relieving structure, to be consistent it will be omitted here as well).

The representative values of the forces taken into account on the relieving structure are shown in the next tables.

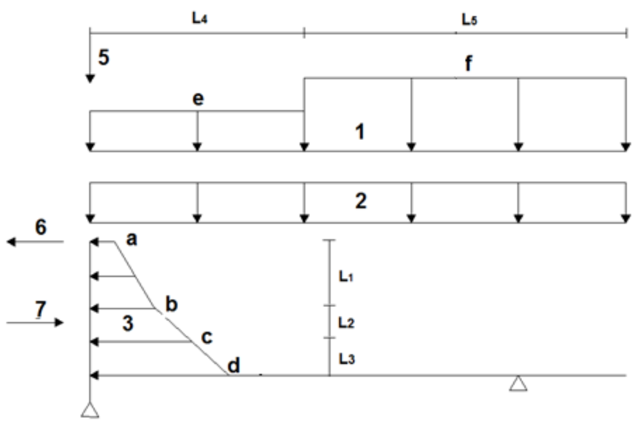

Figure 3.b Schematization of forces

Table 3.c Forces on the relieving structure for a length of 10 meters

			Slab length L=10 m		
			+2.2 m NAP	**-1.8 m NAP**	**-6 m NAP**
1	Surface load on platform [kN/m²]	e	40.0	40.0	40.0
		f	0.0	0.0	0.0
2	Vertical soil on platform [kN/m²]		51.0	107.3	149.3
3	Horizontal load due to surface load and soil [kN/m²]	a	20.0	20.0	20.0
		b	45.5	66.8	66.8
		c	NA	72.2	72.2
		d	NA	73.6	93.3
5	Crane load [kN/m]		1113.0	1113.0	1113.0
6	Bollard load [kN/m]		167.0	167.0	167.0
7	Fender load [kN/m]		120.0	120.0	120.0
	Distance in [m]	L1	3	5.5	5.5
		L2	0	1.2	1.2
		L3	0	0.3	4.5
		L4	10	10	10
		L5	0	0	0

Table 3.d Forces on the relieving structure for a length of 20 meters

			Slab length L=20 m		
			+2.2 m NAP	**-1.8 m NAP**	**-6.0 m NAP**
1	Surface load on platform [kN/m²]	e	40.0	40.0	40.0
		f	90.0	90.0	90.0
2	Vertical soil on platform [kN/m²]		51.0	107.3	149.3
3	Horizontal load due to surface load and soil [kN/m²]	a	20.0	20.0	20.0
		b	45.5	66.8	66.8
		c	NA	72.2	72.2
		d	NA	73.6	93.3
5	Crane load [kN/m]		1113.0	1113.0	1113.0
6	Bollard load [kN/m]		167.0	167.0	167.0
7	Fender load [kN/m]		120.0	120.0	120.0
	Distance [m]	L1	5.5	5.5	5.5
		L2	0.3	1.2	1.2
		L3	0	0.3	4.5
		L4	15	15	15
		L5	5	5	5

The horizontal loads mentioned in Table 3.c and Table 3.d are only taking into account the influence of a uniform surface load of 40 [kN/m²], after 15 meters the load increases linearly to a value of 200 [kN/m²]. The extra horizontal load due to this increase is shown in Table 3.e.

Table 3.e Extra horizontal pressure due to increase of surface load

L=10		L=20	
Distance from surface in [m]	**Horizontal pressure in [kN/m²]**	**Distance from surface in [m]**	**Horizontal pressure in [kN/m²]**
0.0	0.0	0.0	40
3.0	0.0	1.5	40
6.7	24.8	3.0	80
7.3	26.7	6.7	75.2
9.1	41.3	11.2	75.2
11.2	48.1		

To remain consistent, the ground water level is taken at -0.36 m NAP, each load case has a different water pressure difference. It needs to be noted that submerge subject to an uplift force equal to the volume times the water density in [kN/m³].

3.1.3 Summary of parameters used in for the framework calculation

Materials	E [N/mm2]	Weight density [kN/m3]
Concrete of relieving structure (C30/37)*	6600	25
Concrete bearing piles (C45/55)**	36000	0
Steel MV-piles**	210000	0
* The elasticity modulus is reduces with 80% to take a cracked stiffness into account. The weight density of the concrete is reduced with 10 [kN/m3] in case the element is submerged		
** The weight of the concrete piles and MV-piles is not taken into account		

Element	A [m²/m]	I [m⁴/m]
Vertical part relieving structure	3.00E+00	2.25E+00
Slab 10 m long	1.00E+00	8.33E-02
Slab 20 m long	1.80E+00	4.86E-01
Bearing pile	1.25E-01	2.60E-03
MV-pile	1.24E-02	2.89E-04
Anchor rod	4.00E-03	1.55E-06

Where:

E Elasticity modulus
A Cross-sectional area
I Moment of inertia

3.1.4 Result calculation of internal forces (SCIA Engineering)

a) Reaction forces

Table 3.f Forces acting on top of the retaining wall. MV-pile, relieving structure L=10 m in [kN/m]

	Anchor position [m NAP]	MV-pile						Anchor plate					
		2.2		-1.8		-6		2.2		-1.8		-6	
Vertical	LC1	1817	1469	2638	2360	3912	3776	2063	1735	3578	3375	4552	4509
	LC2.1	2056	1623	2977	2609	4382.1	4146	2272	1895	3838	3574	4894	4770
	LC2.2	1680	1363	2437	2205	3601	3525	1934	1635	3417	3249	4332	4347
	LC3	1817	1921	2643	2363	3978	4287	2063	2204	3583	3377	4568	4990
Horizontal	LC1	-2.5	10.11	277.4	341	864.7	1017.1	-379	-363	469	589	634	865
	LC2.1	173	133.2	469	479	1090	1198	-202	-226	651	728	649	1029
	LC2.2	-125	-84	150.6	244	702	883	-501	-457	347	494	488	758
	LC3	-2.5	21.7	281	344.4	916	1074	-379	-282	473	591	647	955

Table 3.g Forces acting on top of the retaining wall. MV-pile, relieving structure L=20 m in [kN/m]

	Anchor position [m NAP]	MV-pile						Anchor plate					
		2.2		-1.8		-6		2.2		-1.8		-6	
Vertical	LC1	1801	1442	2350	2005.5	3022.2	2730	2453	2108	3099	2772	3909	3628
	LC2.1	1859	1486	2489	2112	3260	2914	2479.2	2128	3172	2828	4030	3723
	LC2.2	1791	1433	2280	1951	2879	2617	2449.3	2105.1	3061	2743	3835	3674
	LC3	1802	1882	2350	2005.3	3035	3182	2453	2493	3099	2772	3910	4018
Horizontal	LC1	-15.4	-2.1	270	331.6	853.1	1003	-346	-331	454	570	596	814
	LC2.1	156	130.1	458	465	1075	1181	-184	-207	531	706	780	969
	LC2.2	-135	-94.1	146	237	692	871	-458	-417	336	486	459	714
	LC3	-15.4	9.25	274	334.74	903	1060	-346	-256	458	573	609	899

3.2 D-sheet calculation (step 4)

The results obtained in the framework analysis are used as input in the D-sheet calculation. In this step, iteration between the length of the wall, dimension of the primary elements and anchor properties is made. The last iteration step (final dimensions) is showed in this section.

Model description

In this model, the anchor and relieving structure is modelled as a discrete spring. The spring stiffness used is calculated earlier in this annex. The resultant vertical and horizontal forces calculated in the framework analysis are introduced as a normal force and external horizontal force in D-sheet.

Because D-sheet is not able to introduce the relieving structure explicit in the calculation, two models are evaluated. The models are shown in Figure 3.c. Left the soil around the relieving structure is extracted and in the right a horizontal surface level is assumed, here the soil behind the relieving structure is introduced as a surface load. The model from the right showed a better approximation of the horizontal pressure against the wall and is used further in the calculation.

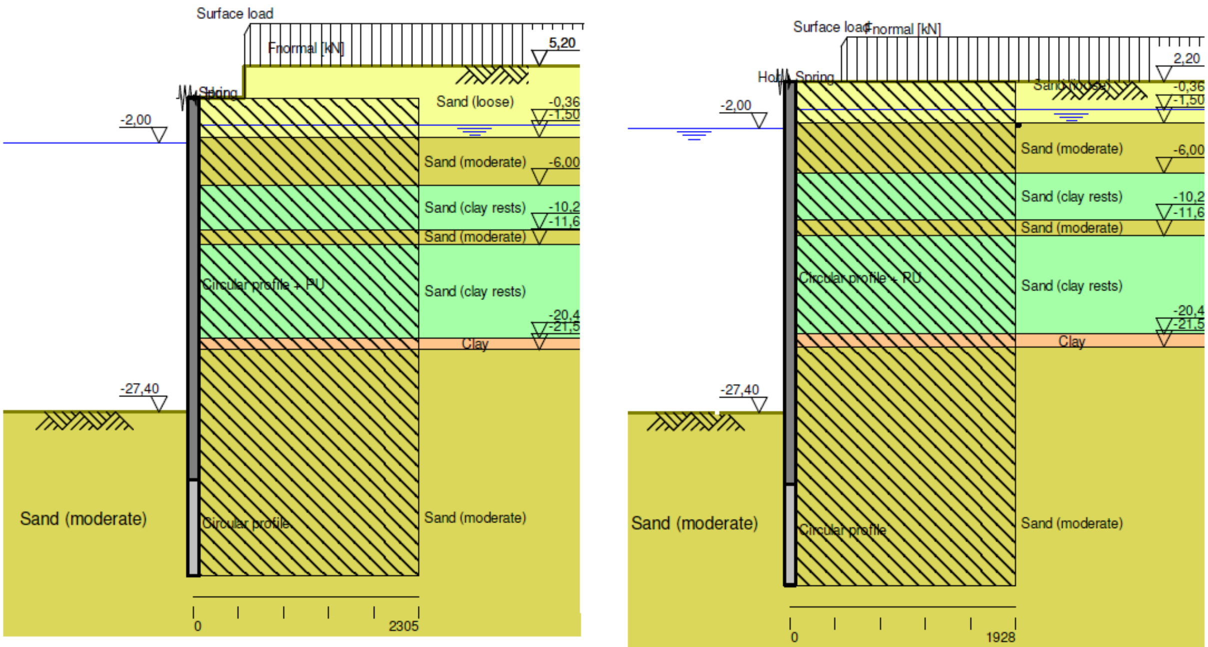

Figure 3.c Model 1 (left) and Model 2 (right)

3.2.1 Combined sheet pile wall with MV-pile

The summary of the resulting dimensions, moments and anchor forces of the wall calculated with D-sheet is shown the next tables. The diameter of the elements is kept constant at a value of 2.8 m.

Table 3.h Summary of internal forces. Option B - MV piles

Length relieving structure [m]	Length floor L=10 m			Length floor L=20 m		
Anchor level	+2.2 m NAP	-1.8 m NAP	-6 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Length primary elements [m]	45.5	42.0	37.0	45.0	40.3	36.0
Thickness [mm]	46.0	39.0	33.0	42.0	30.0	20.0
Max. moment ULS [kNm/m]	29246.0	22604.9	17275.0	25728.2	18642.0	12419.0
Max. Anchor force ULS [kN/m]	2875.4	3011.0	3680.0	1999.0	2886.0	2659.6
Max. Moment SLS [kNm/m]	18480.0	13155.3	9039.0	15378.2	9949.0	5828.0
Max. Anchor force SLS [kN/m]	1837.0	2008.0	2636.0	1109.0	1596.0	1271.0

3.2.2 Combined sheet pile wall with anchor wall

Table 3.i Summary of internal forces. Option B - Anchor plate

Length relieving structure [m]	Length floor L=10 m			Length floor L=20 m		
Anchor level	+2.2 m NAP	-1.8 m NAP	-6 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Length primary elements [m]	45.0	42.0	37.0	45.0	40.3	35.8
Thickness [mm]	46.0	39.0	33.0	42.0	30.0	20.0
Max. moment ULS [kNm/m]	29099.0	22433.0	17098.0	25933.0	18703.4	12406.1
Max. Anchor force ULS [kN/m]	2070.2	2139.0	2606.3	1438.0	2041.0	1997.0
Max. Moment SLS [kNm/m]	16201.0	13487.4	8922.0	15460.9	9890.3	5864.0
Max. Anchor force SLS [kN/m]	1357.7	1447.7	1860.4	802.8	1368.0	1230.0

3.3 PLAXIS calculation (Step 5)

3.3.1 Model and results MV-pile

The MV-piles are modelled as a plate with parameters equal to the parameters of one MV-pile reduced by the centre to centre distance between the piles. The bearing pile is modelled as a fixed-end anchor with the stiffness of one concrete bearing pile of 450x450 [mm], the length of de bearing piles is determined based on the straight sliding plane (the point bearing capacity has to be situated outside this area). The relieving structure is modelled as plate elements of several thicknesses, a reduced elasticity modulus is used to take cracking into account (stiffness reduction).

An impression of the model for the situation with the anchor at +2.2 m NAP is shown in Figure 3.d.

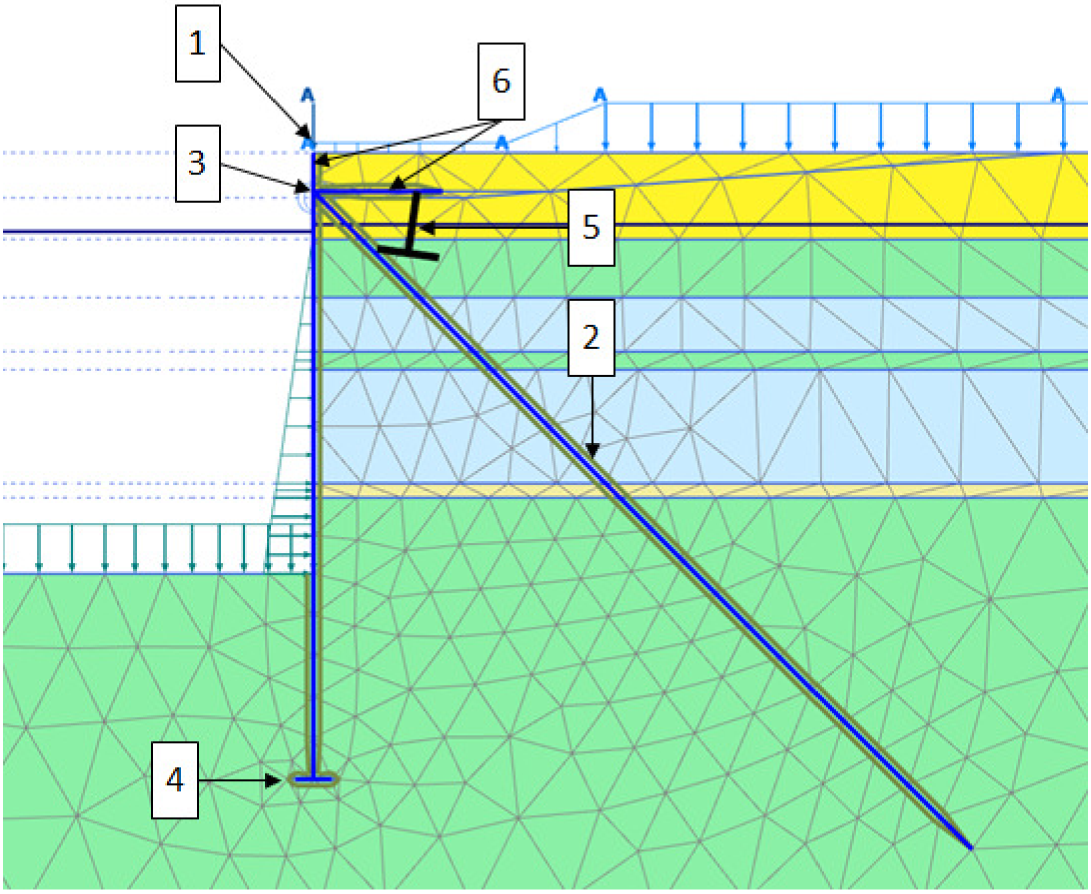

Figure 3.d Example PLAXIS model of Option B with a MV-pile row at +2.2 m NAP

In the model:

1. Variable loads (Surface load crane load, fender load and bollard load);
2. MV-pile modelled as a plate with EI and EA = $(EI;EA)_{MV\text{-}pile}/L_{c.t.c.\ piles}$;
3. The connection between wall – relieving structure and MV-pile – relieving structure with the relieving structure is modelled as a hinge;
4. Plate elements in PLAXIS that are used to model piles do not have end bearing capacity. The bearing capacity of the plate element is modelled as a really stiff plate as long as the diameter of the primary members. This way the deformation in the vertical direction is not completely restraint;
5. Bearing piles;
6. Relieving structure.

The results of the PLAXIS calculation are:

Table 3.j Result PLAXIS calculation Option B - MV-piles, without optimization

Length relieving structure	L=10 m			L= 20 m		
Anchor level [m NAP]	+2.2	-1.8	-6.0	+2.2	-1.8	-6.0
	SLS					
Max. displacement wall [m]	0.2894	0.2290	0.2134	0.2289	0.2053	0.1799
Max. displacement top relieving structure [m]	0.2002	0.2368	0.4373	0.1324	0.1481	0.2434
Moment [kNm/m]	18360	14160	9730	13800	8959	5142
Anchor force [kN/m]	2699	3053	3538	1913	2356	2429
	ULS					
Moment [kNm/m]	26650	20650	13390	19280	12920	7421
Anchor force [kN/m]	3373	3967	4467	2727	3305	3471

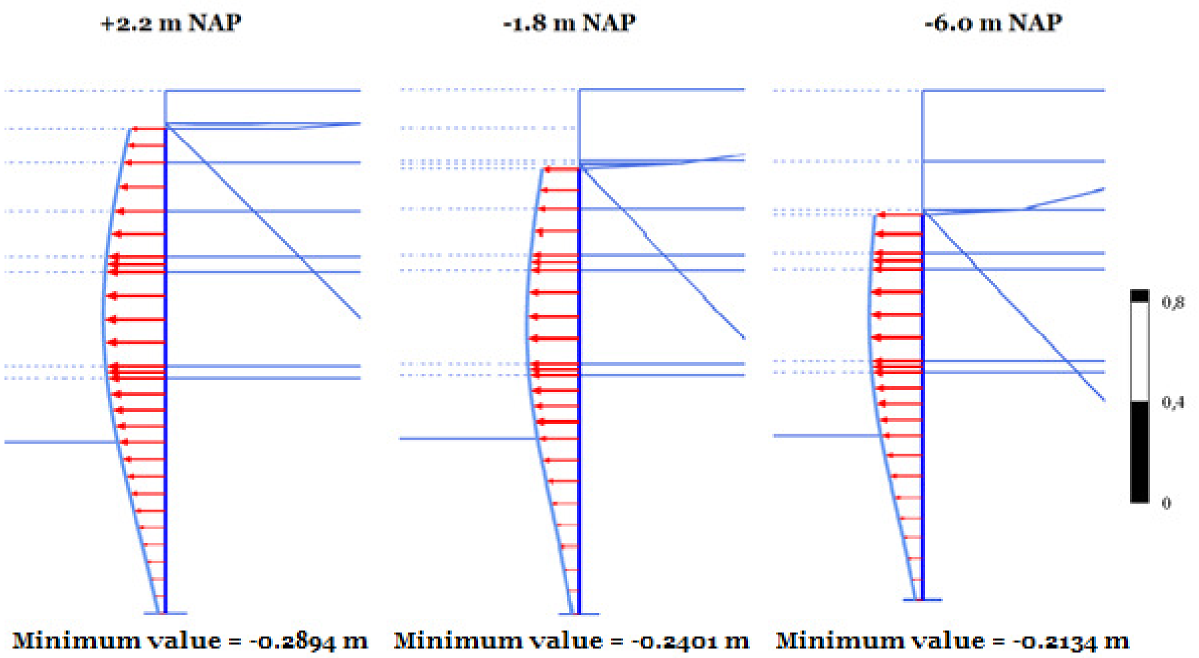

Figure 3.e Displacements Option B - MV-piles L=10 m

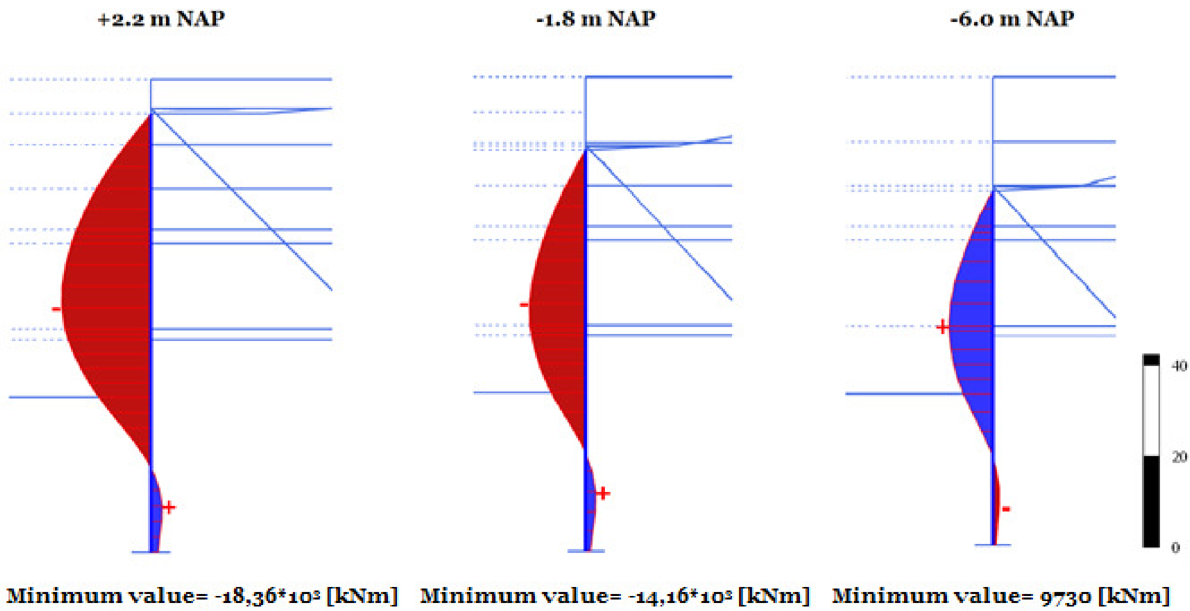

Figure 3.f Moments SLS Option B - MV-piles L=10 m

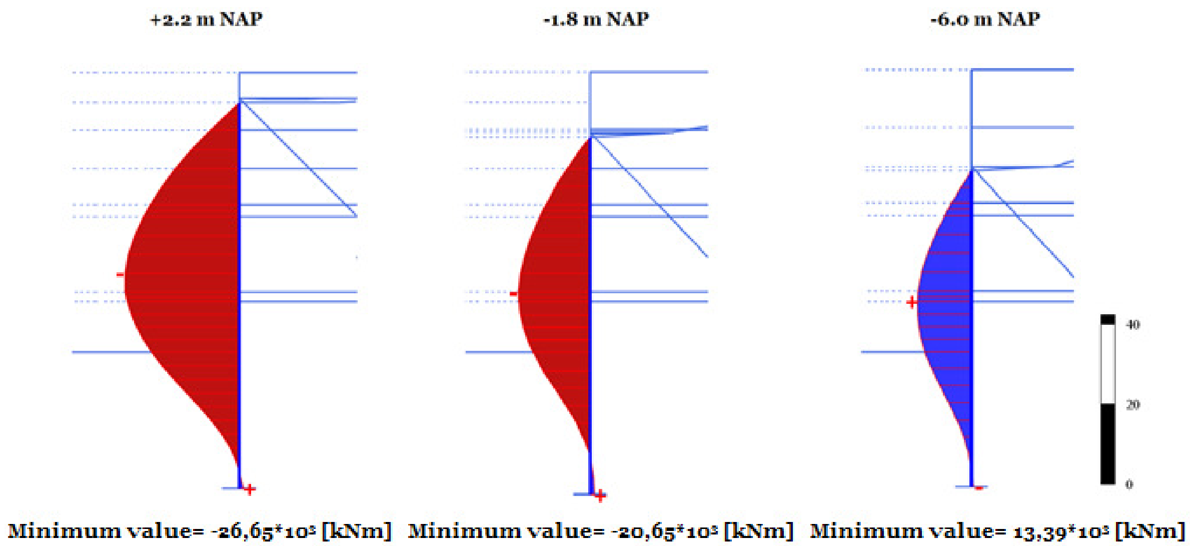

Figure 3.g Moments ULS Option B - MV-piles L=10 m

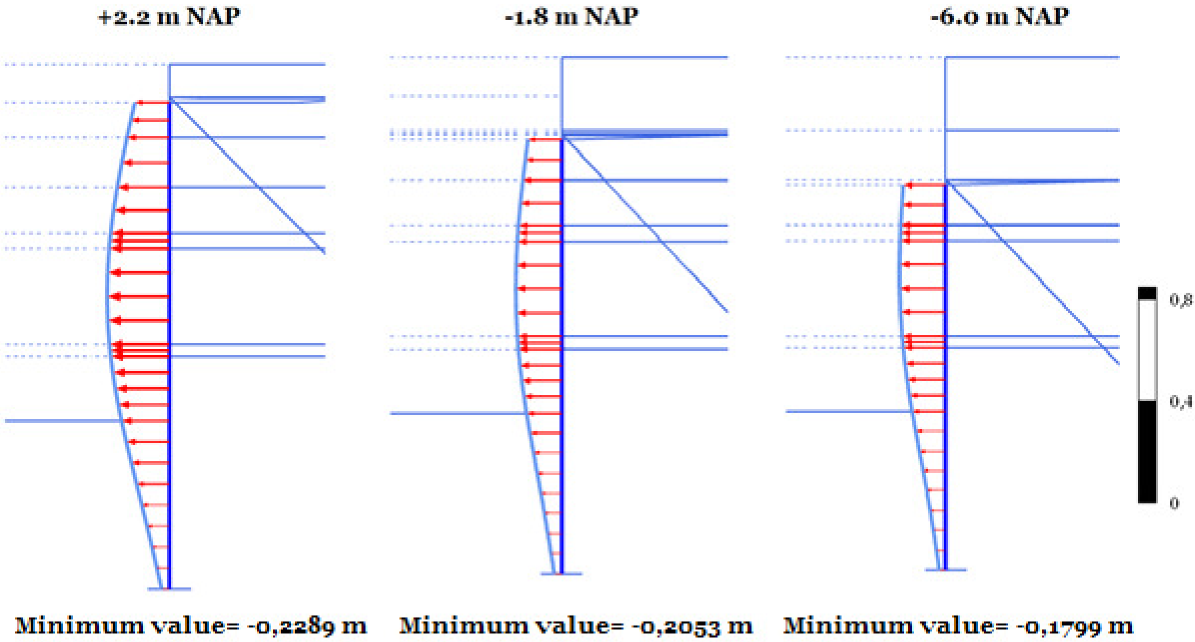

Figure 3.h Displacements Option B – MV-pile L=20 m

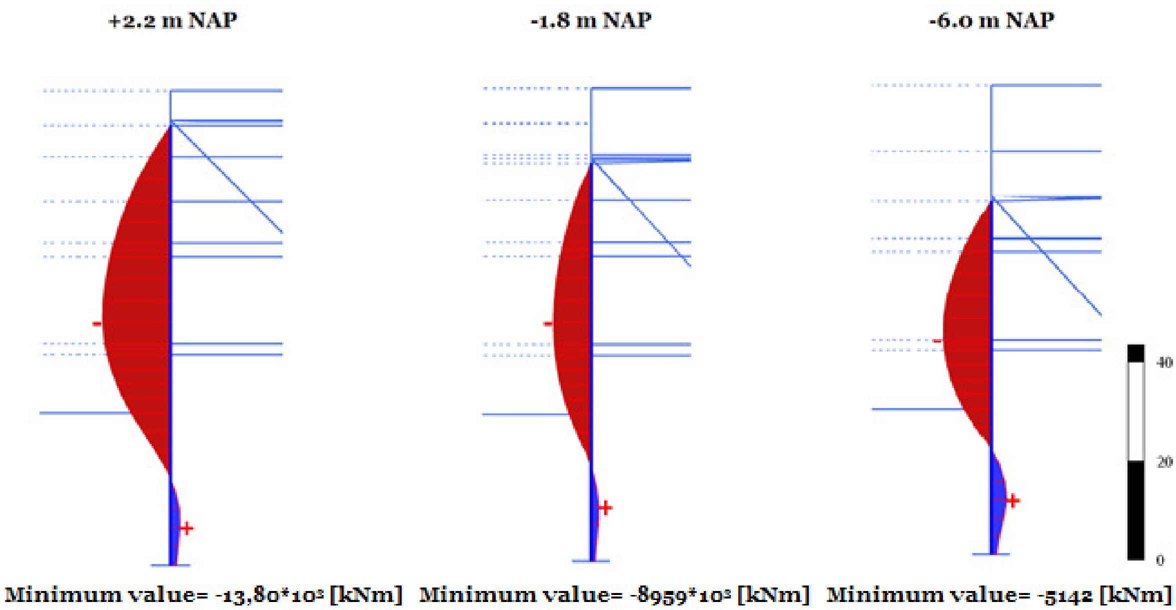

Figure 3.i Moments SLS Option B - MV-pile L=20 m

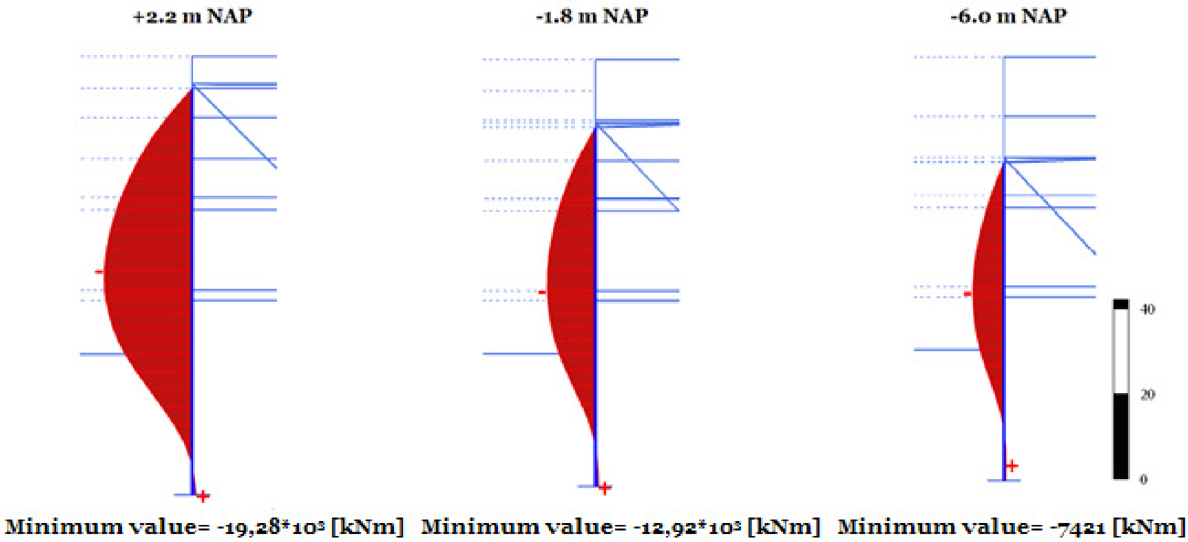

Figure 3.j Moments ULS Option B - MV-piles L=20 m

Table 3.j shows the result of the PLAXIS calculation without optimization. Here, the moments in the ULS are smaller than the moments in the ULS calculated in D-sheet. The first dimensioning of the elements was according to D-sheet, which means that optimisation can take place. This is important to have good economical comparison. The results of the optimisation are:

Table 3.k Result PLAXIS calculation Option B - MV-piles, with optimization

Properties wall	**L= 10 m**			**L=20 m**		
Anchor position [m NAP]	**+2.2**	**-1.8**	**-6**	**+2.2**	**-1.8**	**-6**
Diameter primary member [m]	2.8	2.6	2.5	2.8	2.7	2.6
Thickness [mm]	48	41*	31	38	29**	20
	SLS					
Moment [kNm/m]	18630	13270	8902	13320	11840	4850
Normal force [kN/m]	5023	5306	5280	4637	4256	5120
Anchor force [kN/m]	2694	3063	3507	1912	2161	2409
	ULS					
Moment [kNm/m]	26870	20200	12810	18910	14800	7121
Normal force [kN/m]	5164	5522	5396	4392	4556	5368
Anchor force [kN/m]	3657	3943	4481	2736	2824	3480

* In a later phase changed to 42 [mm] due to the local buckling criteria

** In a later phase changed to 33 [mm] due to the local buckling criteria

3.3.1 Model and results anchor wall

The same parameters for the relieving structure are used. Here the difference lies in the anchor, which is modelled as shown in Figure 3.k.

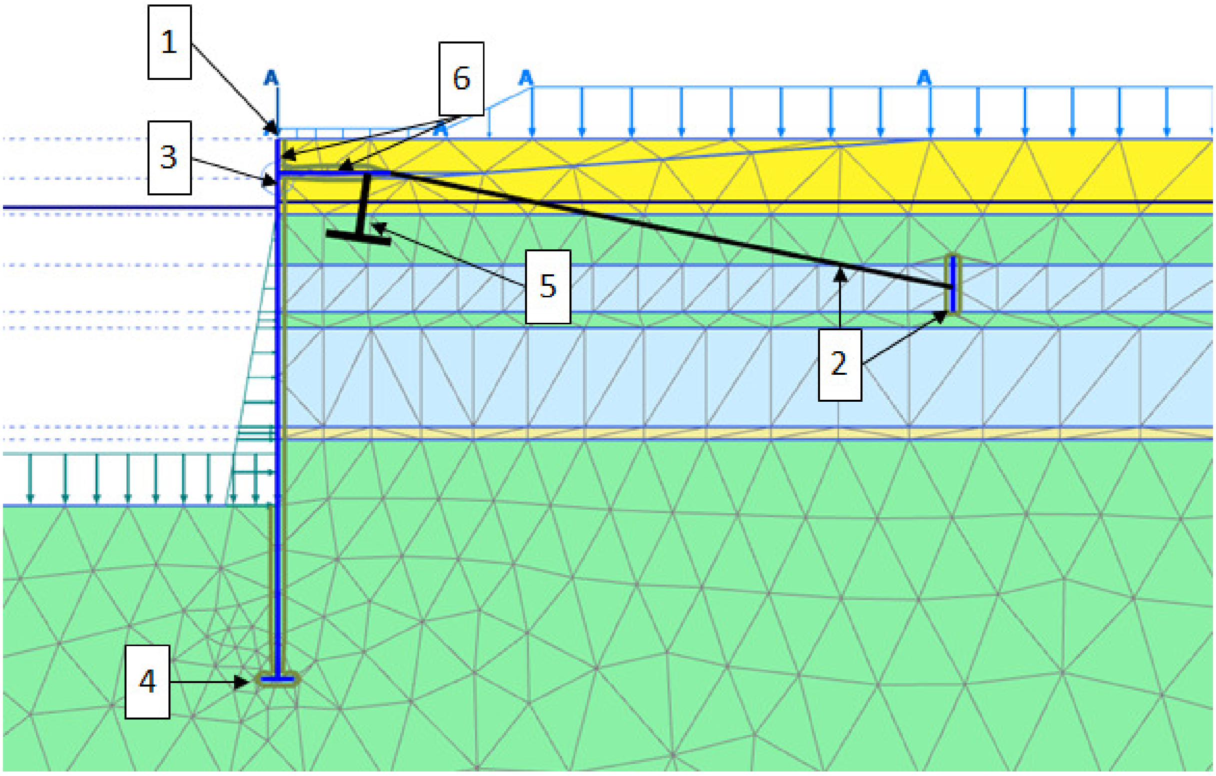

Figure 3.k Example PLAXIS model of Option B with a Anchor rod connection at +2.2 m NAP

1. Variable loads (Surface load crane load, fender load and bollard load);
2. Anchor rod modelled as a node to node anchor with an EA equal to a steel bar of 75 [mm]. The anchor plate is modelled as a plate with enough moment resistance;
3. The connection between wall – relieving structure is modelled as a hinge;
4. Plate elements in PLAXIS that are used to model piles do not have end bearing capacity. The bearing capacity of the plate element is modelled as a really stiff plate as long as the diameter of the primary members. This way the deformation in the vertical direction is not completely restraint;
5. Bearing piles;
6. Relieving structure.

The results of the calculation are:

Length relieving structure	L=10 m			L= 20 m		
Anchor level [m NAP]	**+2.2**	**-1.8**	**-6.0**	**2.2**	**-1.8**	**-6.0**
	SLS					
Max. displacement wall [m]	0.2998	0.2300	0.2158	0.2088	0.1592	0.1357
Max. displacement top relieving structure [m]	0.2291	0.2831	0.4184	0.086	0.1365	0.1455
Moment [kNm/m]	17830	13370	10190	13990	8397	6665
Anchor force [kN/m]	1773	1851	2129.8	1153	1157.5	1557
	ULS					
Moment [kNm/m]	26140	19400	14550	19900	12140	8860
Anchor force [kN/m]	2357	2442	2580.3	1588	1625	1980

Here also an optimisation is made to calculate the costs:

Properties wall	L= 10 m			L=20 m		
Anchor position [m NAP]	**2.2**	**-1.8**	**-6**	**2.2**	**-1.8**	**-6**
Diameter primary member [m]	2.8	2.7	2.6	2.7	2.5	2.3
Thickness [mm]	48	41*	33**	35***	30	25
	SLS					
Moment [kNm/m]	17830	14160	12800	13330	8397	6665
Normal force [kN/m]	2979	3444	3771	3271	3408	3846
Anchor force [kN/m]	1773	1925	2219	1085	1158	1557
	ULS					
Moment [kNm/m]	26140	21330	16630	19410	12140	8860
Normal force [kN/m]	2959	3305	3650	3246	3489	3943
Anchor force [kN/m]	2357	2550	2773	1605	1625	1981

* In a later phase changed to 42 [mm] to satisfy the local buckling criteria

** In a later phase changed to 36 [mm] to satisfy the local buckling criteria

*** In a later phase changed to 39 [mm] to satisfy the local buckling criteria

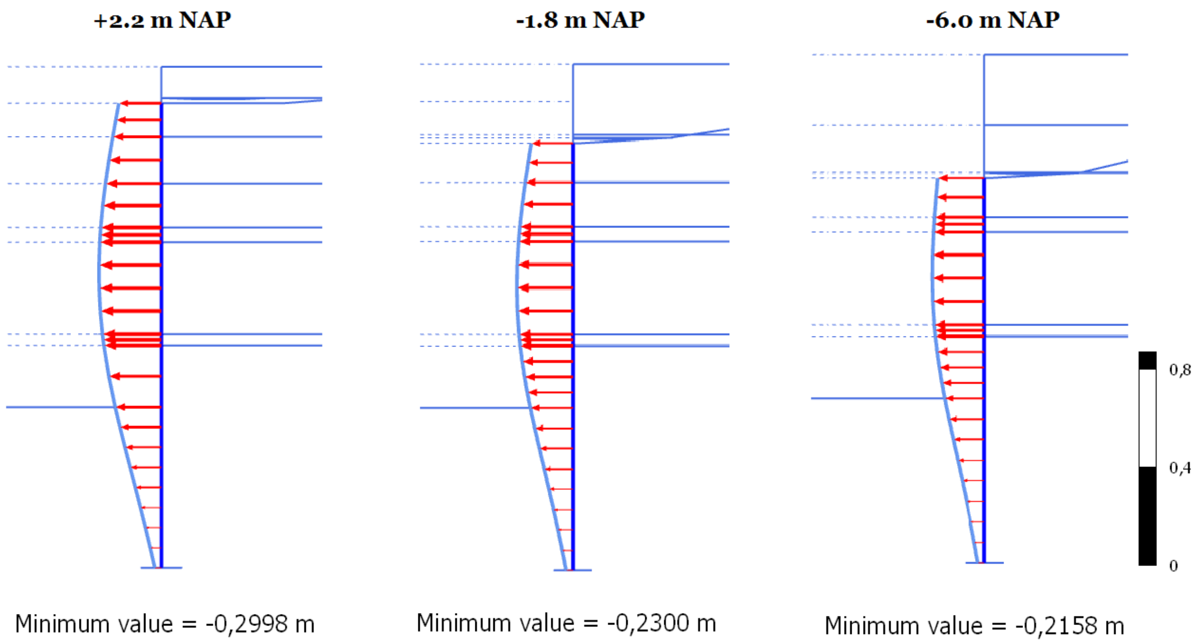

Figure 3.l Displacements Option B – Anchor plate L=10 m

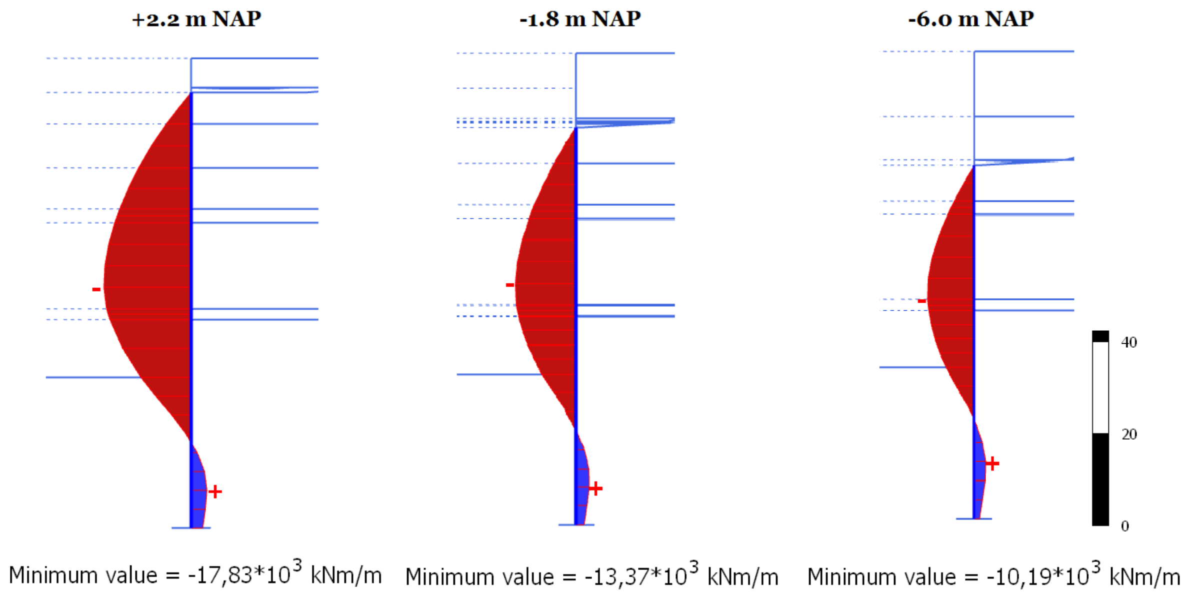

Figure 3.m Moments SLS Option B - Anchor plate L=10 m

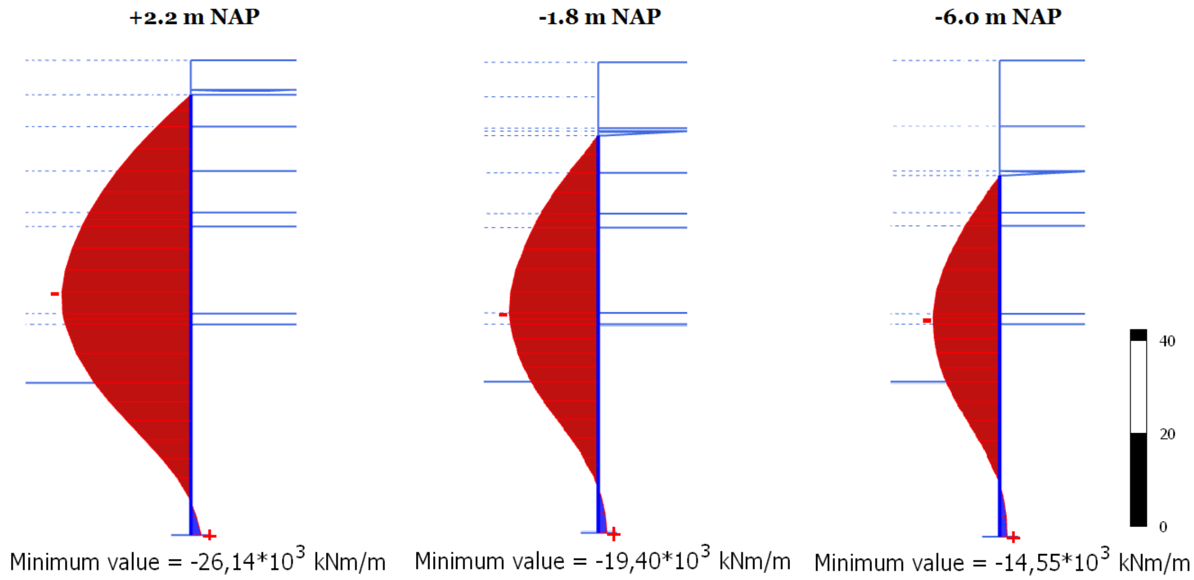

Figure 3.n Moments ULS Option B - Anchor plate L=10 m

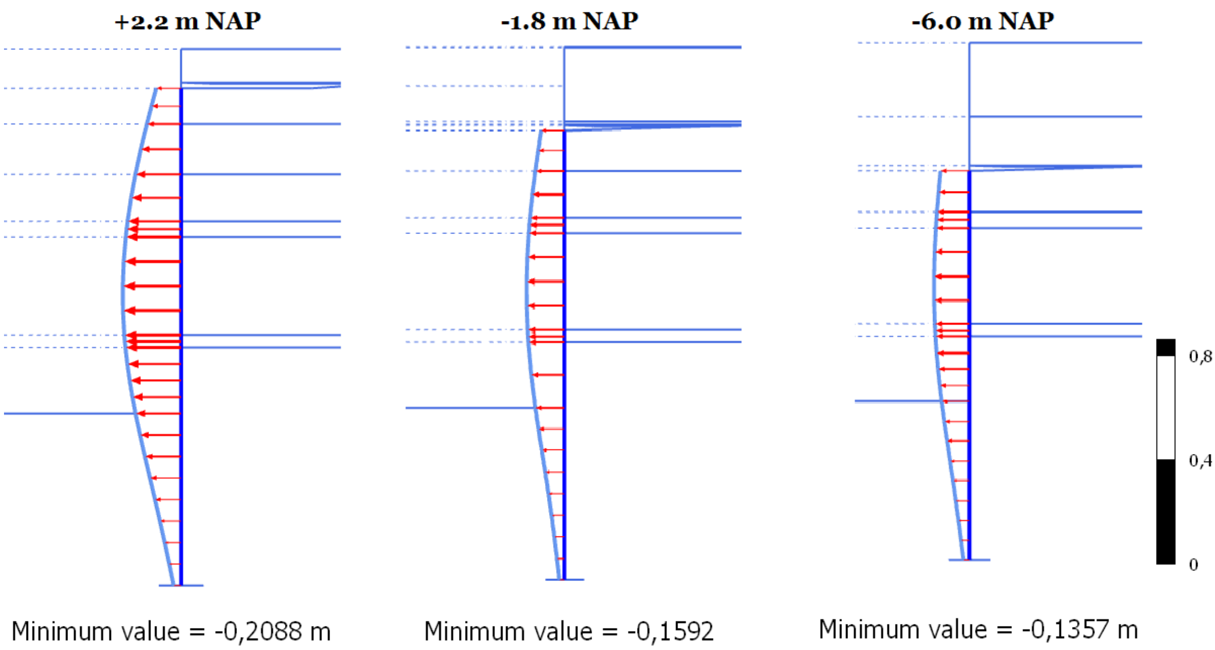

Figure 3.o Displacements Option B – Anchor plate L=20 m

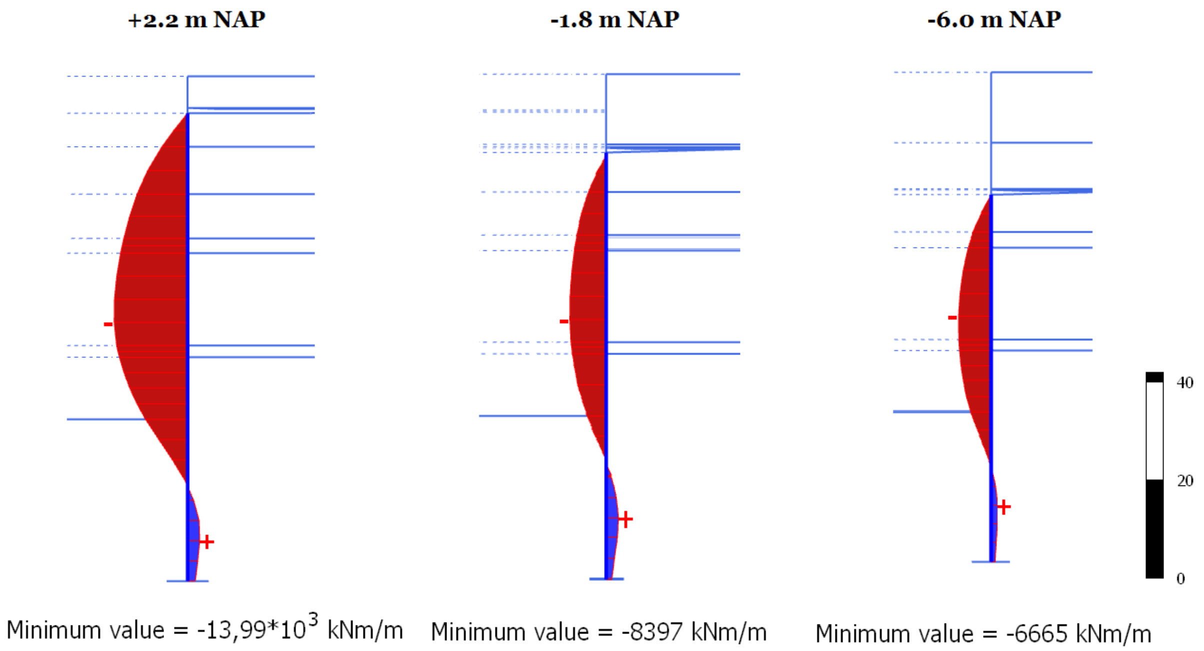

Figure 3.p Moments SLS Option B - Anchor plate L=20 m

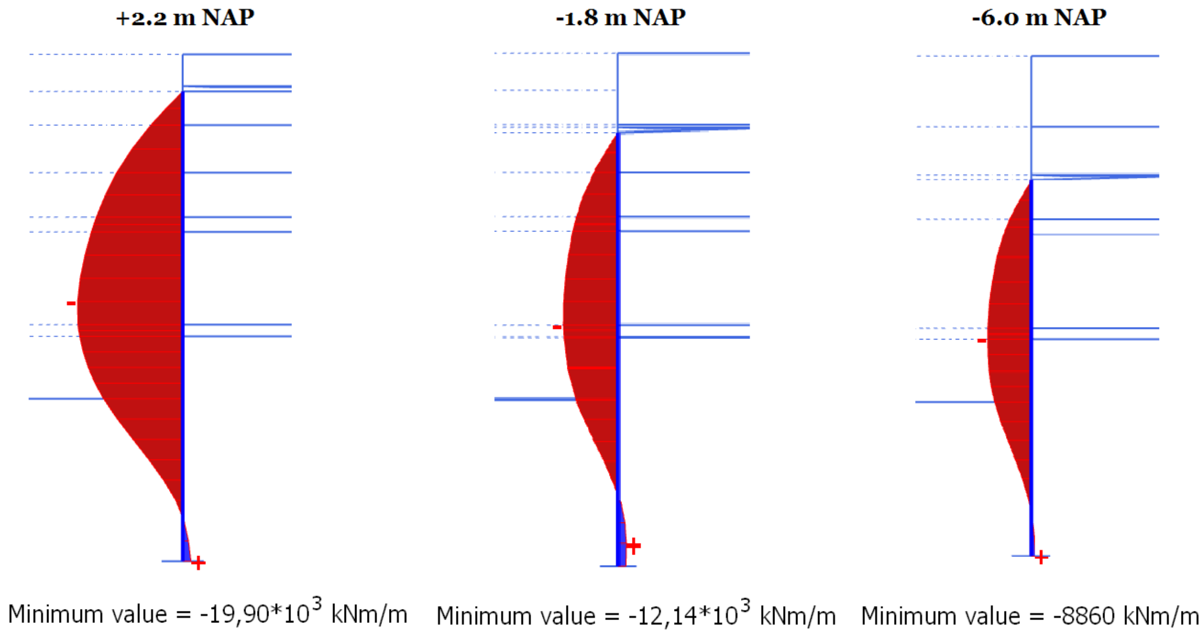

Figure 3.q Moments ULS Option B - Anchor plate L=20 m

3.4 Strength check primary elements (step 6)

3.4.1 Summary of the properties of the wall

a) Combined wall with MV-pile

Properties wall	L= 10 m			L=20 m		
Anchor position [m NAP]	+2.2	-1.8	-6	+2.2	-1.8	-6
Thickness [mm]	48	41*	31	38	29*	20
Diameter primary member [m]	2.8	2.6	2.5	2.8	2.7	2.6
Moment of inertia [m^4]	0.393	0.270	0.183	0.314	0.217	0.135
Yield stress [MPa]	500	500	500	500	500	500
System length [m]	4.6	4.4	4.3	4.6	4.4	4.4
Moment capacity [kNm/m]	30511	23591	17047	24416	17863	11791
Normal force capacity [kN/m]	45108	37456	27960	35840	27038	18421

b) Combined wall with anchor plate

Properties wall	L= 10 m			L=20 m		
Anchor position [m NAP]	2.2	-1.8	-6	2.2	-1.8	-6
Thickness [mm]	48	39*	33*	35*	30	25
Diameter primary member [m]	2.8	2.7	2.6	2.7	2.5	2.3
Moment of inertia [m^4]	0.39	0.26	0.23	0.27	0.18	0.14
Yield stress [MPa]	500	500	500	500	500	500
System length [m]	4.6	4.5	4.4	4.5	4.3	4.1
Moment capacity [kNm/m]	30511	23756	19164	21415	16517	12260
Normal force capacity [kN/m]	45108	36226	30242	32559	27069	21790

* In a later phase changed due to local buckling

c) **Combined wall with anchor plate**

3.4.2 Unity check

a) Combined wall with MV-pile

Length relieving structure	L= 10 m			L=20 m		
Anchor position	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
ULS						
Absolute value moment [kNm/m]	26870	20200	12810	18910	14800	7121
Acting normal force [kN/m]	4410	4938	4930	3931	4236.3	4671
SLS						
Absolute value moment [kNm/m]	18630	13270	8902	13320	11840	4850
Acting normal force [kN/m]	4104	4409	4458	3635	3824	5088
$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1$	0.98	0.99	0.93	0.88	0.99	0.86
$1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}}\right) \leq 1$	0.84	0.82	0.82	0.78	0.97	0.83

b) Combined wall with anchor plate

Length relieving structure	L= 10 m			L=20 m		
Anchor position	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
ULS						
Absolute value moment [kNm/m]	26140	21330	16630	19410	12140	8860
Acting normal force [kN/m]	2357	2550	2773	2308	2679	3362
SLS						
Absolute value moment [kNm/m]	17830	14160	12800	13330	8397	6665
Acting normal force [kN/m]	2193	2651	2924	2543	2842	3450
$\frac{M_{ULS}}{M_{capacity}} + \frac{N_{ULS}}{N_{capacity}} \leq 1$	0.91	0.97	0.96	0.98	0.83	0.88
$1{,}2 \cdot \left(\frac{M_{SLS}}{M_{capacity}} + \frac{N_{SLS}}{N_{capacity}}\right) \leq 1$	0.76	0.80	0.92	0.84	0.74	0.84

3.1 Check anchor holding capacity (step 7)

3.1.1 Anchor holding capacity

The minimal length of the piles to satisfy the strength criteria is given in this section.

a) MV-piles

Length relieving strucure	**10 [m]**			**20 [m]**		
Anchor position	**+2.2 m NAP**	**-1.8 m NAP**	**-6 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6 m NAP**
Representative holding capacity [kN/m]	5954	6682	7234	4514	4608	5454
Design holding capacity [kN/m]	4253	4773	5167	3224	3292	3896
Length pile [m]	68	69	69	58	55	57

b) Anchor plate

Length relieving strucure	**10 [m]**			**20 [m]**		
Anchor position	**+2.2 m NAP**	**-1.8 m NAP**	**-6 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6 m NAP**
Representative holding capacity [kN/m]	3863.7	4219.6	4449.7	3543.7	3543.7	4349.2
Design holding capacity [kN/m]	2759.8	3014	3178.3	2531.2	2531.2	3106.6
Heigth anchor plate [m]	5.5	5	5.5	5	5	5

As in Option A an anchor rod of 75 [mm] diameter with a design holding capacity of 2524 [kN] is assumed.

3.1.2 Unity check

The anchor capacity is also controlled in two ways. First the holding capacity must be bigger than 1.1 times the anchor force obtained in the ULS calculation. The other control is that the holding capacity has to be bigger than 1.32 times the anchor force in the SLS calculation (1.1*1.2*anchor force in SLS).

$$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1 \qquad \text{and} \qquad 1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$$

a) Option B with MV-piles

Length relieving structure	**10 [m]**			**20 [m]**		
Position Anchor	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor holding capacity [kN/m]	4253	4773	5167	3224	3292	3896
Anchor force in the ULS [kN/m]	3657	3943	4481	2736	2824	3480
Anchor force in the SLS [kN/m]	2694	3063	3507	1912	2161	2409
Check anchor holding capacity						
$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1$	0.946	0.909	0.954	0.933	0.944	0.983
$1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$	0.836	0.847	0.896	0.783	0.867	0.816

b) Combined wall with anchor plate

Length relieving structure	**10 [m]**			**20**		
Anchor level	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor holding capacity [kN/m]	2759.8	3014	3178.3	2531.2	2531.2	3106.6
Anchor force in the ULS [kN/m]	2357	2550	2773	1605	1625	1981
Anchor force in the SLS [kN/m]	1773	1925	2219	1085	1158	1557
Check anchor holding capacity						
$1.1\frac{F_{anchor;ULS}}{R_d} \leq 1$	0.94	0.93	0.96	0.70	0.71	0.70
$1.1*1.2\frac{F_{anchor;SLS}}{R_d} \leq 1$	0.85	0.84	0.92	0.57	0.60	0.66

Another aspect that needs to be checked in this case is the anchor rod capacity, the same way a unity check is made where the anchor rod capacity has to be equal to 1.25 times the anchor force.

Length relieving structure	10 [m]			20 [m]		
Anchor level	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor holding capacity [kN]	2524	2524	2524	2524	2524	2524
Anchor force per rod in the ULS [kN]	2357	2550	2773	1605	1625	1981
Check anchor holding capacity						
$1.25\frac{F_{anchor;ULS}}{R_{d;rod}} \leq 1$	1.17	1.26	1.37	0.79	0.80	0.98

When the anchor rods do not satisfies the strength criteria, the centre to centre distance is reduced.

Length relieving structure	10 [m]			20 [m]		
Anchor level	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Anchor rod c.t.c. distance [m]	0.85	0.75	0.7	1	1	1
Anchor holding capacity [kN]	2524	2524	2524	2524	2524	2524
Anchor force per rod [kN]	2003.5	1912.5	1941.1	1605	1625	1981
Check anchor holding capacity						
$1.25\frac{F_{anchor;ULS}}{R_{d;rod}} \leq 1$	0.99	0.95	0.96	0.79	0.80	0.98

3.2 Overall checks (step 9)

In this chapter the overall stability of the quay wall, the bearing capacity of the bearing piles, the Kranz stability of the anchor and failure of one anchor is evaluated.

3.2.1 Overall stability

In quay walls with relieving structure this phenomenon is less probable to happen and is left out of the calculation.

3.2.2 Bearing capacity primary members

The bearing capacity of the primary elements is calculated as explained in annex C. Here the total bearing capacity is the lowest value of the bearing capacity with and without plug formation (it is known that with plug formation a much higher bearing capacity is reached). The results are showed in the next tables. In this calculation, reduction of the cone resistance due to excavation is taken into account.

In both cases (MV-pile and anchor plate) enough bearing capacity is achieved.

a) Combined wall with MV-piles

Length relieving structure	L=10 m			L=10 m		
Level bottom relieving structure	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Embedded depth [m]	15.90	18.00	15.60	15.40	14.70	14.60
Diameter [m]	2.8	2.6	2.5	2.8	2.6	2.6
Thickness [mm]	48	38	31	38	29	20

Cone resistance port side in [Mpa]

Average initial	24.28	24.28	24.28	24.28	24.28	24.28
Average reduced from -27.4 m NAP to -31 m NAP	4.75	4.75	4.75	4.75	4.75	4.75
Averaged reduced from -31 m NAP to -35 m NAP	9.62	9.62	9.62	9.62	9.62	9.62
Averaged reduced from -35 m NAP to bottom primary members	13.97	14.35	13.17	13.87	13.73	13.71

Cone resistance quay side in [Mpa]	23.61	22.53	20.53	23.52	21.86	20.31

Contribution to force [MN]

Outside shaft friction port side	10.95	11.38	9.20	10.64	9.49	9.43
Outside shaft friction quay side	16.78	17.64	14.70	16.26	14.41	14.31
Inner shaft friction	5.87	7.58	6.69	6.61	6.75	7.39
End bearing capacity (plugged)	25.92	22.35	20.66	25.92	22.35	22.35
End bearing capacity (unplugged)	3.92	2.89	2.27	3.11	2.21	1.53

Total bearing capacity (plugged) [MN]	53.65	51.37	44.56	52.82	46.24	46.09
Total bearing capacity (Unplugged) [MN]	37.53	39.49	32.86	36.63	32.86	32.66

Bearing capacity needed [MN]	25.42	25.68	24.97	23.46	22.05	24.78
Safety factor [MN]	1.48	1.54	1.32	1.56	1.49	1.32

b) Combined wall with anchor plate

Length relieving structure	L=10 m			L=10 m		
Level bottom relieving structure	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Embedded depth [m]	15.90	18.00	15.60	15.40	14.70	14.60
Diameter [m]	2.8	2.7	2.6	2.7	2.5	2.3
Thickness [mm]	48	41	33	35	30	25
Cone resistance port side in [Mpa]						
Average initial	24.28	24.28	24.28	24.28	24.28	24.28
Average reduced from -27.4 m NAP to -31 m NAP	4.75	4.75	4.75	4.75	4.75	4.75
Averaged reduced from -31 m NAP to -35 m NAP	9.62	9.62	9.62	9.62	9.62	9.62
Averaged reduced from -35 m NAP to bottom primary members	13.97	14.35	13.17	13.87	13.73	13.71
Cone resistance quay side in [Mpa]	23.61	22.53	20.53	23.52	21.86	20.31
Contribution to force [MN]						
Outside shaft friction port side	10.95	11.82	9.56	10.26	9.12	8.34
Outside shaft friction quay side	16.78	18.32	15.29	15.68	13.85	12.66
Inner shaft friction	5.87	7.41	6.70	6.76	6.51	6.40
End bearing capacity (plugged)	25.92	24.10	22.35	24.10	20.66	17.49
End bearing capacity (unplugged)	3.92	3.24	2.51	2.77	2.20	1.69
Total bearing capacity (plugged) [MN]	53.65	54.24	47.20	50.04	43.64	38.49
Total bearing capacity (Unplugged) [MN]	37.53	40.79	34.07	35.47	31.68	29.08
Bearing capacity needed [MN]	15.07	17.05	18.25	16.19	16.50	17.78
Safety factor [MN]	2.49	2.39	1.87	2.19	1.92	1.64

The bearing capacity of the wall is by far met in the case of a combined wall with anchor plate. The reason is that the contribution to the vertical force made by the anchor is really small. In the case of the MV-pile, the contribution of the anchor to the vertical force is really high, leading to high needed bearing capacity.

3.2.3 Kranz stability

The Kranz stability is calculated as explained in chapter 2, where stability is reached when the ratio between the anchor force according to Kranz and the needed anchor force is higher than 1.5 (CUR166, 6th edition, 2012). The anchor force according to Kranz is calculated by means of equilibrium of forces in the failure plane of the anchor. Some assumptions of this failure plane are made (Figure 3.r):

- In the case of a quay wall with an MV-pile, the failure plane is equal to the lines connecting the lowest zero shear point of the sheet pile wall, the shear centre of the MV pile (in this cased assumed to be at ½ of the effective length) and a vertical line to the surface;
- In the case of a quay wall with an anchor plate the same failure plane is taken except that instead of the shear centre of the MV-pile the plane goes through the lowest point of the anchor plate.

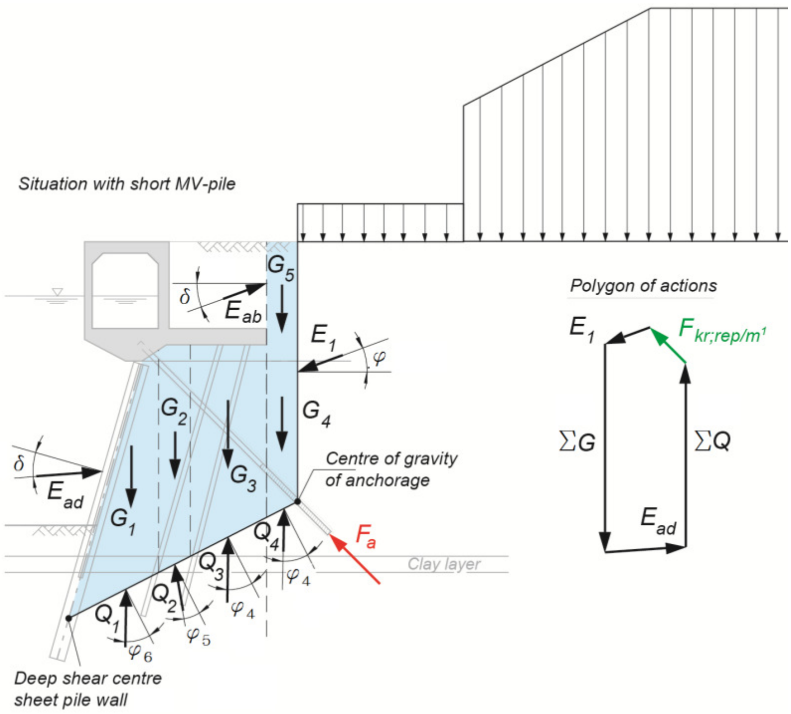

Figure 3.r KRANZ stability of anchors

Table 3.l Results KRANZ stability, Option B, MV-piles

Relieving structure length	**10 [m]**			**20 [m]**		
Position anchor [m NAP]	**+2.2**	**-1.8**	**-6.00**	**+2.2**	**-1.8**	**-6.00**
Length MV-pile [m]	72	73	73	70	71	71
Safety factor	1.50	1.58	1.62	1.62	1.48	1.59

Table 3.m Results KRANZ stability, Option B, Anchor plate

Relieving structure length	**10 [m]**			**20 [m]**		
Position anchor [m NAP]	**+2.2**	**-1.8**	**-6.00**	**+2.2**	**-1.8**	**-6.00**
Toe anchor plate [m NAP]	-8	-13.2	-16.5	-8	-8	-15
Distance from quay wall [m]	60	60	60.4	60	65	60
Safety factor	1.71	1.50	1.52	2.52	2.12	1.68

Kranz calculation details Length relieving structure L=10 [m]

	MV-pile			Anchor plate		
Position anchor [m NAP]	**+2.2**	**-1.8**	**-6.00**	**+2.2**	**-1.8**	**-6.00**
Anchor force from calculation (SLS) [kN/m]	2694	3063	3507	1773	1925	2219
Distance to shear level MV-pile/anchor wall [m NAP]	33.4	33.2	31.98	60	60	60.4
Zero shear level Wall [m NAP]	-40	-40	-40	-40	-39.3	-40
Surface load [kN/m2]	200	200	200	200	200	200
Horizontal components						
Contribution of force on sheet pile wall [kN/m]	2831.18	2253.81	1984.20	2831.18	2166.57	1984.2
Contribution reaction of soil weights [kN/m]	4596.68	6352.91	7758.19	1157.13	2389.05	3612.32
Contribution of soil behind the failure plain [kN/m]	-4573.42	-5182.63	-5716.91	-1019.71	-1698.78	-2260.7
Horizontal component Franz [kN/m]	2854.44	3424.09	4025.49	2968.60	2856.85	3335.85
Angle anchor [degrees]	45	45	45	12	9	8
Fkranz in anchor direction [kN/m]	4036.78	4842.4	5692.9	3034.92	2892.46	3368.64
Safety factor (Fkranz/Fa)	1.50	1.58	1.62	1.71	1.50	1.52

Kranz calculation details Length relieving structure L=20 [m]

	MV-pile			Anchor plate		
Position anchor [m NAP]	**+2.2**	**-1.8**	**-6.00**	**+2.2**	**-1.8**	**-6.00**
Anchor force from calculation (SLS) [kN/m]	1912	2161	2410	1085	1158	1557
Distance to shear level MV-pile/anchor wall [m NAP]	32.5	32.1	31.3	60	60	60
Zero shear level Wall [m NAP]	-40	-40	-40	-40	-40	-40
Surface load [kN/m2]	200	200	200	200	200	200
Horizontal components						
Contribution of force on sheet pile wall [kN/m]	2831.18	2253.81	1984.20	2831.18	2253.81	1984.2
Contribution reaction of soil weights [kN/m]	3785.65	5004.96	6321.67	837.56	513.06	2630.23
Contribution of soil behind the failure plain [kN/m]	-4432.19	-4997.80	-5596.83	-1019.71	-1025.38	-2021.5
Horizontal component Franz [kN/m]	2184.64	2260.98	2709.04	2649.03	1741.49	2592.97
Angle anchor [degrees]	45	45	45	14	5	8
Fkranz in anchor direction [kN/m]	3089.55	3197.51	3831.16	2730.13	1748.14	2618.45
Safety factor (Fkranz/Fa)	1.62	1.48	1.59	2.52	1.51	1.68

3.2.4 Failure of one anchor

To check this failure mechanism, the SLS load combination is taken in which the anchor force has its higher value. When an anchor fails, adjacent anchors have to be able to take 1.5 times the anchor force. This increased anchor force is compared to the design value of the holding capacity of the anchor:

a) MV-pile

Length relieving structure	**L= 10 m**			**L= 20 m**		
Level anchor	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor holding capacity [kN/m]	4253	4773	5167	3224	3292	3896
Max. Anchor force [kN/m]	2694	3063	3507	1912	2161	2409
Center to center distance anchors [m]	2.00	2.00	2.00	2.00	2.00	2.00
Force per achor [MN]	5.39	6.13	7.01	3.82	4.32	4.82
1.5*Force per anchor [MN]	8.08	9.19	10.52	5.74	6.48	7.23
Anchor capacity [MN]	8.51	9.55	10.33	6.45	6.58	7.79
Safety anchor	1.05	1.04	0.98	1.12	1.02	1.08
Extra capacity needed [MN]	0.0	0.0	0.2	0.0	0.0	0.0
Extra length	0.0	0.0	0.5	0.0	0.0	0.0

b) Anchor wall

The anchor rods are decisive for the holding capacity of the anchor plate. When one anchor rod fails adjacent anchors have to be able to resist re-distributed force. It is assumed that the force is increased in 50%. To satisfy this criterion, the centre to centre distance of the anchor rods is changed.

Length relieving structure	**L= 10 m**			**L= 20 m**		
Level anchor	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Anchor holding capacity [kN]	2524	2524	2524	2524	2524	2524
Max. Anchor force [kN/m]	1773	1925	2219	1085	1158	1557
Center to center distance anchors [m]	0.85	0.75	0.7	1	1	1
Force per achor [MN]	1.51	1.44	1.55	1.09	1.16	1.56
1.5*Force per anchor [MN]	2.26	2.16	2.33	1.63	1.74	2.34
Anchor capacity [MN]	2.52	2.52	2.52	2.52	2.52	2.52
Safety anchor	1.12	1.17	1.08	1.55	1.45	1.08

3.3 Local buckling check

Local buckling check		Option B					
		MV-pile					
		L=10 m			L=20 m		
		+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Diameter	[m]	2.8	2.6	2.6	2.8	2.7	2.6
Thickness	[mm]	48.0	42.0	31.0	38.0	33.0	20.0
Soil pressure q	[kN/m²]	130.0	113.0	103.4	123.0	70.0	60.0
Stifness EI	[kNm²]	7.86E+07	5.52E+07	4.13E+07	6.29E+07	4.92E+07	2.70E+07
Yield stress	[N/mm²]	500.0	500.0	500.0	500.0	500.0	500.0
Young modulus	[N/mm²]	200000.0	200000.0	200000.0	200000.0	200000.0	200000.0
Moment capacity	[kNm]	140352.6	106207.3	79396.9	112315.2	91063.8	51880.2
Normal force capacity	[kN]	207495.9	168760.1	125096.6	164864.5	138247.4	81053.1
Acting moment per tube	[kNm]	123602.0	88880.0	56364.0	86986.0	66600.0	31332.4
Acting normal force per tube	[kN]	20286.0	21727.2	21692.0	18082.6	19063.4	20552.4
Acting moment per meter	[kNm/m]	26870.0	20200.0	12810.0	18910.0	14800.0	7121.0
Acting normal force per meter	[kN/m]	4410.0	4938.0	4930.0	3931.0	4236.3	4671.0
Output							
Reduction factor M_{Rd}	[-]	0.91	0.89	0.78	0.82	0.79	0.74
Moment resistance agains local buckling	[kNm]	1.28E+05	9.45E+04	6.22E+04	9.25E+04	7.23E+04	3.82E+04
Normal force recistance against local buckling	[kN]	2.06E+05	1.68E+05	1.24E+05	1.63E+05	1.37E+05	7.97E+04
Unity check	[-]	0.98	0.97	0.96	0.96	0.96	0.92
Conclusion unity check		OK	OK	OK	OK	OK	OK
Calculation steps							
Ratio D/t	[-]	58.33	61.90	83.87	73.68	81.82	130.00
Radius	[m]	1.40	1.30	1.30	1.40	1.35	1.30
Out of roundness tolerance*	[-]	0.01	0.01	0.01	0.01	0.01	0.01
Initial out of roundness	[mm]	7.00	6.50	6.50	7.00	6.75	6.50
Ovalisation and moments due to tensile forces secondary members							
Tensile force	[kN]	182.00	146.90	134.42	172.20	94.50	78.00
Ovalisation (decrease in radius)	[mm]	4.34E-04	3.99E-04	4.89E-04	5.13E-04	3.23E-04	4.34E-04
Ma	[kNm]	-81.11	-60.79	-55.62	-76.74	-40.61	-32.28
Mb	[kNm]	46.29	34.70	31.75	43.80	23.18	18.42
Ovalisation and moments due to soil pressure from one side							
Ovalisation	[mm]	2.65E-04	2.43E-04	2.98E-04	3.13E-04	1.97E-04	2.65E-04
Ma	[kNm]	-31.85	-23.87	-21.84	-30.14	-15.95	-12.68
Mb	[kNm]	41.48	31.09	28.45	39.25	20.77	16.51
Total ovalisation	[mm]	7.00	6.50	6.50	7.00	6.75	6.50
Actual radius (r')	[m]	1.42	1.32	1.32	1.42	1.37	1.32
Critical strain (epsilocr)	[-]	0.01	0.01	0.00	0.00	0.00	0.00
Reducing factors for the moment resistance							
g	[-]	0.99	0.99	0.99	0.99	0.99	0.98
meff;sd	[kNm]	12.59	9.44	8.63	11.91	6.30	5.01
mpl;Rd	[kNm]	261.82	200.45	109.20	164.09	123.75	45.45
Safety factor (gamma0)	[-]	1.10	1.10	1.10	1.10	1.10	1.10
c1	[-]	1.96	1.96	1.93	1.94	1.96	1.90
Beta g	[-]	1.00	1.00	1.00	1.00	1.00	1.00
Beta s (empty)	[-]	0.92	0.90	0.79	0.83	0.80	0.75
Slenderness (mu)	[-]	2.38	2.18	1.35	1.67	1.41	0.61

* Depends the fabrication class and the diameter. Ur=0.01 for Class B D>=1.25 m

Local buckling check		Option B					
		Anchor plate					
		L=10 m			L=20 m		
		+2.2 m NAP	-1.8 m NAP	-6.0 m NAP	+2.2 m NAP	-1.8 m NAP	-6.0 m NAP
Diameter	[m]	2.8	2.7	2.6	2.7	2.5	2.3
Thickness	[mm]	48.0	41.0	36.0	39.0	30.0	25.0
Soil pressure q	[kN/m²]	130.0	113.0	103.4	123.0	70.0	60.0
Stifness EI	[kNm²]	7.86E+07	6.06E+07	4.77E+07	5.77E+07	3.55E+07	2.31E+07
Yield stress	[N/mm²]	500.0	500.0	500.0	500.0	500.0	500.0
Young modulus	[N/mm²]	200000.0	200000.0	200000.0	200000.0	200000.0	200000.0
Moment capacity	[kNm]	140352.6	112134.2	91670.3	106902.7	71022.5	50265.3
Normal force capacity	[kN]	207495.9	171246.6	144990.8	163015.7	116396.0	89339.0
Acting moment per tube	[kNm]	120244.0	95985.0	73172.0	87345.0	52202.0	36326.0
Acting normal force per tube	[kN]	10842.2	11475.0	12201.2	10386.0	11519.7	13784.2
Acting moment per meter	[kNm/m]	26140.0	21330.0	16630.0	19410.0	12140.0	8860.0
Acting normal force per meter	[kN/m]	2357.0	2550.0	2773.0	2308.0	2679.0	3362.0
Output							
Reduction factor M_{Rd}	[-]	0.91	0.87	0.83	0.85	0.79	0.76
Moment resistance agains local buckling	[kNm]	1.28E+05	9.71E+04	7.63E+04	9.05E+04	5.60E+04	3.82E+04
Normal force recistance against local buckling	[kN]	2.06E+05	1.70E+05	1.44E+05	1.61E+05	1.15E+05	8.86E+04
Unity check	[-]	0.94	1.00	0.97	0.97	0.95	0.99
Conclusion unity check		OK	OK	OK	OK	OK	OK
Calculation steps							
Ratio D/t	[-]	58.33	65.85	72.22	69.23	83.33	92.00
Radius	[m]	1.40	1.35	1.30	1.35	1.25	1.15
Out of roundness tolerance*	[-]	0.01	0.01	0.01	0.01	0.01	0.01
Initial out of roundness	[mm]	7.00	6.75	6.50	6.75	6.25	5.75
Ovalisation and moments due to tensile forces secondary members							
Tensile force	[kN]	182.00	152.55	134.42	166.05	87.50	69.00
Ovalisation (decrease in radius)	[mm]	4.34E-04	4.23E-04	4.23E-04	4.83E-04	3.29E-04	3.10E-04
Ma	[kNm]	-81.11	-65.55	-55.62	-71.35	-34.82	-25.26
Mb	[kNm]	46.29	37.42	31.75	40.73	19.87	14.42
Ovalisation and moments due to soil pressure from one side							
Ovalisation	[mm]	2.65E-04	2.58E-04	2.58E-04	2.95E-04	2.01E-04	1.89E-04
Ma	[kNm]	-31.85	-25.74	-21.84	-28.02	-13.67	-9.92
Mb	[kNm]	41.48	33.53	28.45	36.49	17.81	12.92
Total ovalisation	[mm]	7.00	6.75	6.50	6.75	6.25	5.75
Actual radius (r')	[m]	1.42	1.37	1.32	1.37	1.27	1.17
Critical strain (epsilocr)	[-]	5.94E-03	4.98E-03	4.32E-03	4.61E-03	3.41E-03	2.85E-03
Reducing factors for the moment resistance							
g	[-]	0.99	0.99	0.99	0.99	0.99	0.99
meff;sd	[kNm]	12.59	10.18	8.63	11.08	5.40	3.92
mpl;Rd	[kNm]	261.82	191.02	147.27	172.84	102.27	71.02
Safety factor (gamma0)	[-]	1.10	1.10	1.10	1.10	1.10	1.10
c1	[-]	1.96	1.95	1.95	1.94	1.95	1.95
Beta g	[-]	1.00	1.00	1.00	1.00	1.00	1.00
Beta s (empty)	[-]	0.92	0.87	0.84	0.86	0.80	0.77
Slenderness (mu)	[-]	2.38	1.99	1.73	1.85	1.36	1.14
* Depends the fabrication class and the diameter. Ur=0.01 for Class B D>=1.25 m							

3.4 Cross section anchor wall

The cross section of the anchor wall is determined based on the bending moments due to the anchor force. The bending moments are calculated with PLAXIS. Table 3.n show the bending moment acting on the plate for each anchor level.

Table 3.n Moment acting on anchor plate

Length relieving structure	**10 m**			**20 m**		
Anchor position [m NAP]	**+2.2**	**-1.8**	**-6.0**	**+2.2**	**-1.8**	**-6.0**
Moment [kNm/m]	1749	1891	2409	1178	1225	1776

The moments are too high for a sheet pile wall. A combined wall of 1.1 [m] diameter and 20 [mm] thickness and 2 PU profiles is chosen as an anchor plate (bending moment capacity = 3755 [kNm/m]).

3.5 Reinforcement relieving structure

The reinforcement is calculated for the case where the bottom of the relieving structure is at -6,0 m NAP, has a slab of 20 meters and an MV-pile as anchor.

3.5.1 Option B – MV piles

Starting points:

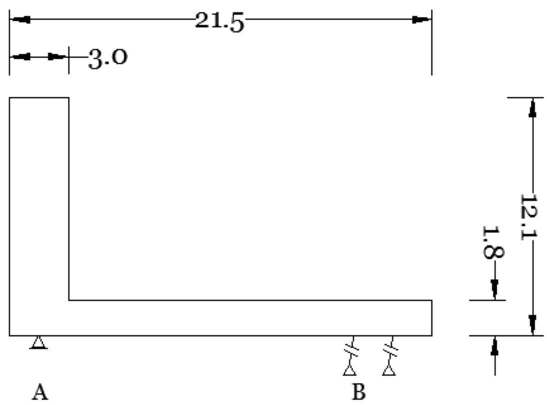

Figure 3.s Dimensions relieving structure in meters

Concrete class:	C30/37
Compression strength:	f_{cd}= 20 [N/mm²]
Tensile strength:	f_{ctm}= 2.9 [N/mm²]
Environmental class:	XC4, XS3, XF4
Concrete cover:	c_{nom}= 55 [mm], $c_{applied}$= 70 [mm]
Requirement crack width:	w_{slab}= 0.3 [mm]; w_{wall}= 0.15 [mm]
Reinforcement steel:	B500
Design yield strength:	f_{yd}= 435 [N/mm²]

a) Slab

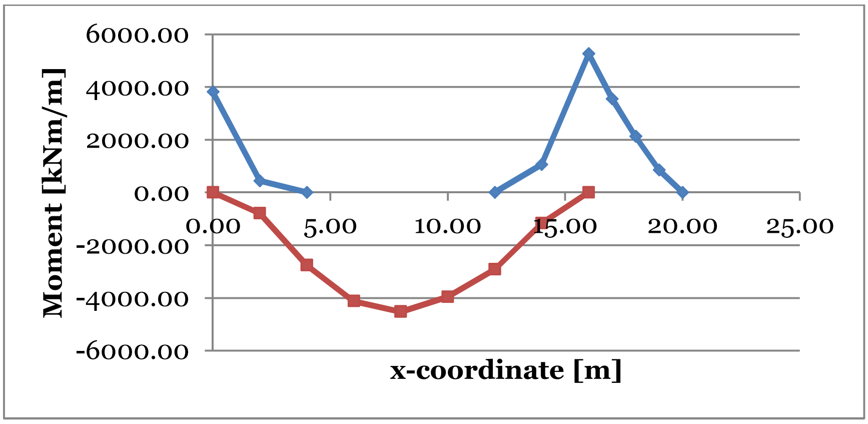

Figure 3.t Envelop moment diagram of slab relieving structure

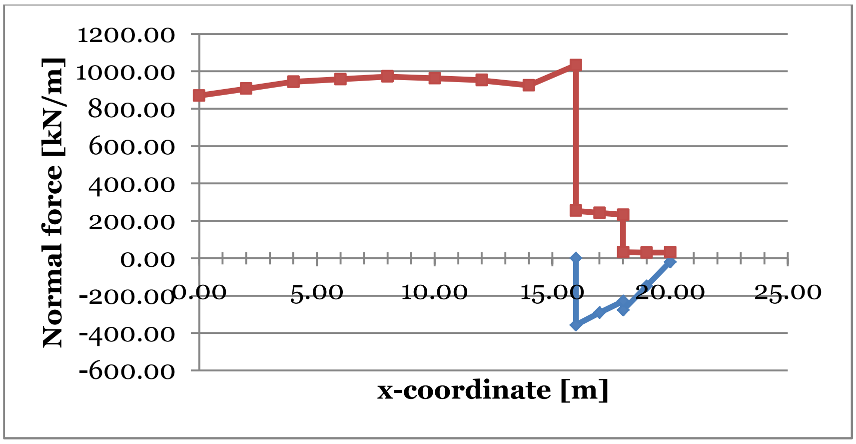

Figure 3.u Envelop normal forces diagram slab relieving structure

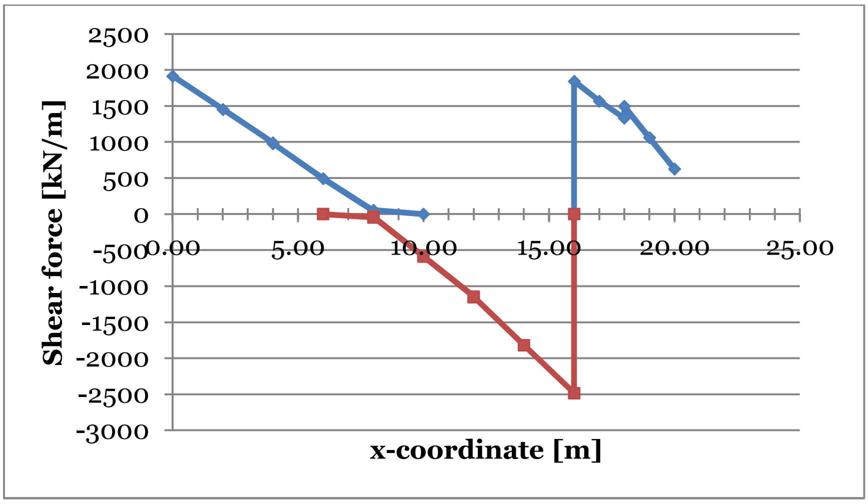

Figure 3.v Envelop shear force diagram slab

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Bottom net

Assumed stirrups: $\phi 16$

Main reinforcement: $\phi 32\_100$

Moment in the ULS: M_{Ed}= 4524 [kNm/m]

Normal force in the ULS: N_{Ed}= 1032 [kN/m]

$$d = h - c - \phi_{stirrup} - \phi_{mr}/2 = 1800 - 70 - 16 - 16 = 1698[mm]$$

Estimation needed reinforcement:

$$A_{s;requ} = \frac{M_{Ed} - N_{Ed} \cdot e}{f_{yd} \cdot 0.9d} + \frac{N_{Ed}}{f_{yd}} = \frac{4524 \cdot 10^6 - 1032 \cdot 10^3 \cdot (1698 - 0.5 \cdot 1800)}{435 \cdot 0.9 \cdot 1698} + \frac{1032000}{435} = 7939[mm^2/m]$$

Applied reinforcement:

$$A_s = 0.25 \cdot \pi \cdot \phi_{mr}^2 \cdot n = 0.25 \cdot \pi \cdot 32^2 \cdot 10 = 8045.5[mm^2/m]$$

$$A_{SN} = N / f_{yd} = 1032000/435 = 2373[mm^2/m]$$

$$A_{SM} = A_S - A_{SN} = 8045 - 2373 = 5672[mm^2/m]$$

$$N_S = A_{SM} f_{yd} = 2466.5[kN/m];$$

$$N_S = N_c = \frac{3}{4} x_u f_{cb} b = \frac{3}{4} x_u 20 \cdot 1000 = 15 x_u \rightarrow x_u = 164.4[mm]$$

$$z = d - 0.39 x_u = 1633.9[mm]$$

$$M_{Rd} = A_S f_{yd} z = 4030[kNm/m] > M_{Ed} - N \cdot e = 3700.5[kNm/m]$$

Crack width control mid span

$$h_{eff} = \min(2.5(h-d); (h-x)/3; h/2) = 255[mm]; \rho_{eff} = A_S/(h_{eff} b) = 0.032;$$

Moment in the SLS: $M_{Eq} = 4335[kNm/m]$

$$\alpha_e = E_S(1 + \varphi(\infty; t_0))/E_c = 2 \cdot 10^5 (1 + 1.7)/33000 = 16.36$$

$$S_{r;\max} = k_3 c + k_1 k_2 k_4 \frac{\phi}{\rho_{eff}} = 3.4 \cdot 70 + 0.8 \cdot 1 \cdot 0.425 \cdot 32/0.032 = 578[mm]$$

$$\sigma_S = \frac{M_{Eq} - N_{Eq} e}{A_S z f_{yd}} + \frac{N_{Eq}}{A_S f_{yd}} = 409.7[N/mm^2]$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_S - k_t \dfrac{f_{ct}}{\rho_{eff}}(1 + \alpha_e \rho_{eff})}{E_s} \geq 0.6\frac{\sigma_S}{E_S} \rightarrow \varepsilon_{sm} - \varepsilon_{cm} = 0.00186 > 0.0012$$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 1.06[mm]$ > 0.3*70/55 = 0.38 [mm]

To be able to control cracking the steel stress needs to be reduced. Extra reinforcement is needed to satisfy crack width, because the procedure is the same, only the results of the calculation are presented:

Reinforcement: $\phi 40\_75$; $A_S = 16755.2[mm^2/m]$; $d = 1694[mm]$

$h_{eff} = 265[mm]$; $\rho_{eff} = 0.063$

$\alpha_e = 16.36$; $S_{r;\max} = 410.7[mm]$; $\sigma_S = 197.05[N/mm^2]$

$\varepsilon_{sm} - \varepsilon_{cm} = 8.44 \cdot 10^{-4} > 5.93 \cdot 10^{-4}$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 0.35[mm]$ < 0.3*70/55 = 0.38 [mm] OK

Top net

The same way the reinforcement of the top net is calculated, because hi moments are expected at the supports, a standard net is applied and extra bars at the supports (A and B) are applied.

Main reinforcement: $\phi 25\_200$

Support A (retaining wall)

M_{Ed}= 3809 [kNm/m]; $d = 1701.5[mm]$

$A_{S;requ} = M_{Ed} / (f_{yd} 0.9d) = 5718[mm^2/m]$

Extra reinforcement: $\phi 32\_200$

$A_{S;applied} = 6476[mm^2/m]$; $\phi_{eq} = 28.93[mm]$; $d = 1699.54[mm]$

$N_S = 2817.06[kN/m]$; $N_S = N_c = 15x_u \rightarrow x_u = 187.8[mm]$

$z = 1626.3[mm]$ $M_{Rd} = 4581.4[kNm/m]$ OK

Crack width control support A

$h_{eff} = 251.15[mm]$; $\rho_{eff} = 0.0258$; $M_{Eq} = 3713[kNm/m]$

$\alpha_e = 16.36$; $S_{r;\max} = 619.24[mm]$; $\sigma_S = 361.7[N/mm^2]$

$\varepsilon_{sm} - \varepsilon_{cm} = 0.00149 > 0.00108$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 1.12[mm]$ > 0.3*70/55 = 0.38 [mm] NOT OK

New reinforcement to reduce steel stress: $\phi 25\_200 + \phi 40\_150$

$A_S = 10831.95[mm^2/m]$; $\phi_{eq} = 35.2[mm]$; $d = 1696.4[mm]$; $h_{eff} = 259[mm]$;

$\rho_{eff} = 0.0418$; $\alpha_e = 16.36$; $S_{r;\max} = 310.12[mm]$; $\sigma_S = 223.42[N/mm^2]$

$\varepsilon_{sm} - \varepsilon_{cm} = 8.83 \cdot 10^{-4} > 6.7 \cdot 10^{-4}$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 0.27[mm]$ < 0.3*70/55 = 0.38 [mm] OK

Support B (above bearing piles)

M_{Ed}= 5255.05 [kNm/m]; $d = 1701.5[mm]$

$A_{S;requ} = M_{Ed} / (f_{yd}\, 0.9d) = 7888{,}8[mm^2/m]$

Extra reinforcement: $\phi 40\_150$

$A_{S;applied} = 10831.95[mm^2/m]$; $\phi_{eq} = 35.2[mm]$; $d = 1696.4[mm]$

$N_S = 4711.9[kN/m]$; $N_S = N_c = 15x_u \rightarrow x_u = 314.13[mm]$

$z = 1573.89[mm]$ $M_{Rd} = 7416.01[kNm/m]$ OK

Crack width control support B

$A_S = 10831.95[mm^2/m]; \phi_{eq} = 35.2[mm]; d = 1696.4[mm]; h_{eff} = 259[mm]$;
$M_{Eq} = 4009[kNm/m]$

$\rho_{eff} = 0.0418$; $\alpha_e = 16.36$; $S_{r;\max} = 310.12[mm]$; $\sigma_S = 235.15[N/mm^2]$

$\varepsilon_{sm} - \varepsilon_{cm} = 9.4213 \cdot 10^{-4} > 7.045 \cdot 10^{-4}$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 0.29[mm]$ < 0.3*70/55 = 0.38 [mm] OK

Shear reinforcement

$d = 1694[mm];$ $k = 1 + \sqrt{\frac{200}{d}} = 1.344;\ \upsilon_{min} = 0.035 \cdot k^{3/2}\sqrt{f_{ck}} = 0.299[N/mm^2]$

Support A: $\upsilon_{Ed} = V_{Ed}/(bd) = 1.131[N/mm^2]$

Support B: $\upsilon_{Ed} = V_{Ed}/(bd) = 1.47[N/mm^2]$ Shear reinforcement needed.

$\theta = 21.8[°]$ $\nu_1 = 0.54$

$V_{Rd;max} = b \cdot z \cdot \nu_1 \cdot f_{cd}/(\cot\theta + \tan\theta) = 6308.4[kN/m]$ OK

Stirrups: $\phi 16$; One stirrup (2 legs): $A_{SW;prov} = 402.12[mm^2]$

$A_{SW;requ} = \frac{\upsilon_{Ed} bs}{0.9 f_{yd}\cot\theta} \rightarrow$ support A: $\phi 16\_300 \rightarrow A_{SW;requ} = 347[mm^2/m]$ OK

Support B: $\phi 16\_250 \rightarrow A_{SW;requ} = 375.5[mm^2/m]$ OK

b) Wall

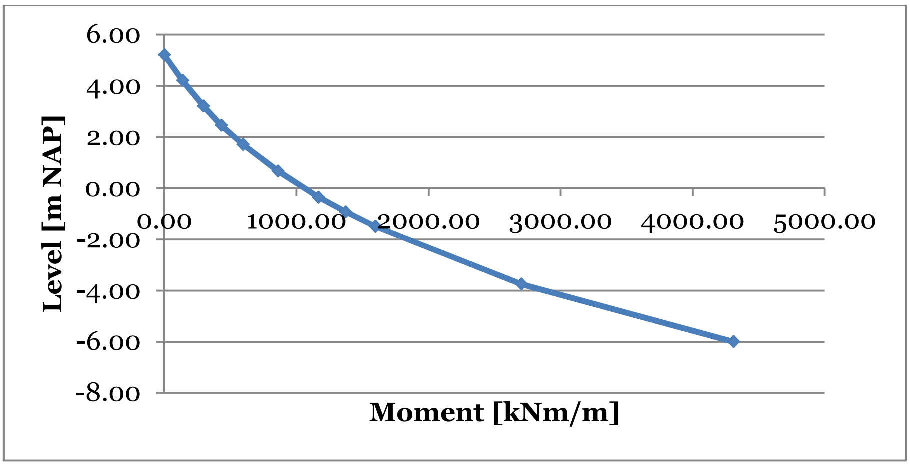

Figure 3.w Envelop moment diagram of wall relieving structure

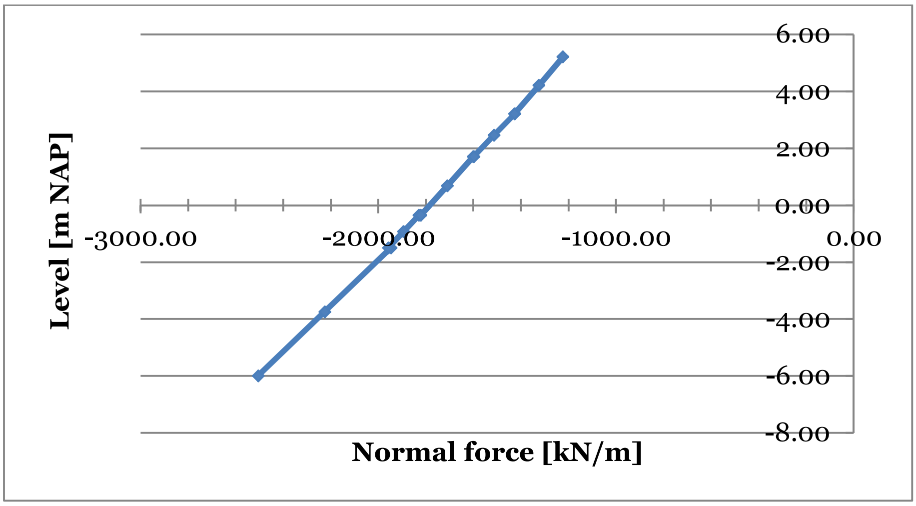

Figure 3.x Envelop normal forces diagram wall relieving structure

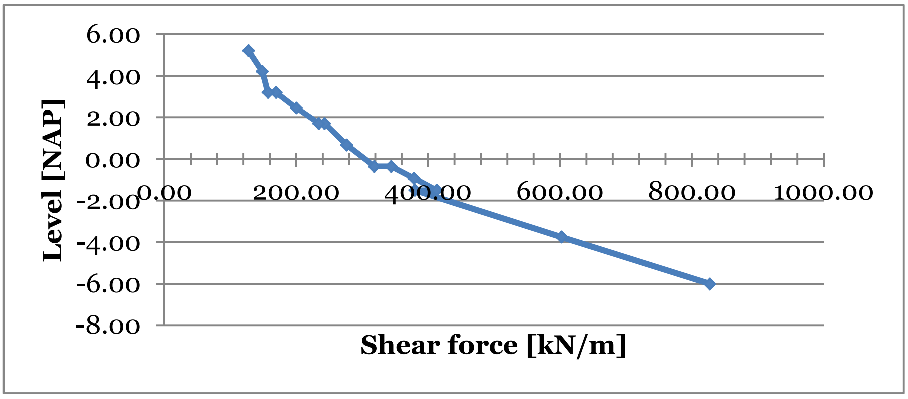

Figure 3.y Envelop shear force diagram wall

Main reinforcement wall

Normal force is favourable, therefore neglected. $\phi 32\_100$; cover = 60 [mm]

M_{Ed}= 4308.5 [kNm/m]; $d = 2924[mm]$

$A_{S;applied} = 8042[mm^2/m]$; $N_S = 3498.27[kN/m]$; $N_S = N_c = 15x_u \rightarrow x_u = 233.22[mm]$

$z = 2833.04[mm]$ $M_{Rd} = 9910.75[kNm/m]$ OK

Crack width control wall

$h_{eff} = 190[mm]$; $\rho_{eff} = 0.0423$; $M_{Eq} = 4083[kNm/m]$

$\alpha_e = 16.36$; $S_{r;\max} = 149[mm]$; $\sigma_S = 179.2[N/mm^2]$

$\varepsilon_{sm} - \varepsilon_{cm} = 6.64 \cdot 10^{-4} > 5.38 \cdot 10^{-4}$

Crack width: $w_k = S_{r;\max}(\varepsilon_{sm} - \varepsilon_{cm}) = 0.1[mm]$ < 0.15*60/55 = 0.16 [mm] OK

Shear force reinforcement wall

$d = 2924[mm]$ $\quad V_{Ed} = 827.3[kN/m] \rightarrow \upsilon_{Ed} = 0.283[N/mm^2]$

$k = 1 + \sqrt{\dfrac{200}{d}} = 1.262$; $\upsilon_{\min} = 0.035 \cdot k^{3/2}\sqrt{f_{ck}} = 0.272[N/mm^2]$; shear reinforcement necessary

$\theta = 21.8[°]$ $\quad \nu_1 = 0.54$

$V_{Rd;\max} = b \cdot z \cdot \nu_1 \cdot f_{cd} /(\cot\theta + \tan\theta) = 10888[kN/m]$ OK

Stirrups: $\phi 16$; One stirrup (2 legs): $A_{SW;prov} = 402.12[mm^2]$

$A_{SW;requ} = \dfrac{\upsilon_{Ed} bs}{0.9 f_{yd} \cot\theta} \rightarrow$ support A: $\phi 16\_300 \rightarrow A_{SW;requ} = 86.74[mm^2/m]$ OK

Summary

Slab

Bottom net**:** $\phi 40\_75$

Top net: $\phi 25\_200$ Extra at support A and B: $\phi 40\_150$

Shear reinforcement: $\phi 16\_300 \rightarrow$ Support A
$\phi 16\_250 \rightarrow$ Support B

Wall

Main reinforcement: $\phi 32\_100$

Shear reinforcement: $\phi 16\_300$

NOTE:

An important aspect that has to be taken into account is the following. In the calculation of the shear reinforcement the c.t.c. distance in calculated the length direction of the relieving structure (10 and 20 meters). The calculation is made for a meter width of the relieving structure. Two legs of the stirrup are taken in to account, which means that in the depth of the relieving structure the legs of the stirrups are placed at a distance of 0.5 m. It is possible to change these legs depending on the distance of the main reinforcement.

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Annex E: Cost estimation

In this attachment the cost of all the calculated quay walls is estimated. The cost of a project is strongly driven by the geographical area, market situation and commercial considerations. The final cost of a structure consists not only by the ones made during the construction but also during the service life (inspection and maintenance). The costs can be divided in direct costs and indirect costs.

Direct costs:

- Civil works: concrete, reinforcement/pre-stressing, formwork, steel profiles;
- Earth works: excavations, dredging, reclamations and fills, dewatering, preloading/(accelerated) consolidation;
- Foundation: Piles, sheet piles, anchorages including soil and grout body anchors;
- Transportation (both horizontal and vertical): Cranes, pontoons and lifts;
- Installation: Driving rigs, self-elevation platforms and pontoons;
- Man-hours related to items above.

Indirect costs:

- Site office including man-hours for management and supporting staff;
- Site facilities for storage, repair and maintenance;
- Site preparation: levelling, roads and fences;
- Auxiliaries: Oil, gas, water and electrical supply;
- Small tools: Positioning/survey tools (total stations), compressor, compactors, etc;
- Design and site engineering.

Additional costs:

- Risk;
- Profit;
- Head office;
- Interests.

In several cases the environment determines which option is the most suitable for a determined case. A structural engineer has to think in advance about the construction methods to save costs during the design of the structure.

1.1 Assumptions

The calculation of costs in this thesis is made to be able to compare all the variants. These costs are a rough estimation and several assumptions are made.

- The construction is made from land in a dry building pit, there is enough space for machinery and facilities;
- Sheet pile driving has no influence to the surroundings (Noise, soil and water pollution);
- Sheet piles/MV-piles are easy to be driven in the soil (no obstruction due to stiff layers or obstructions in the soil);
- Dredging on port side and costs due to bottom protection are the same for all variants;
- The soil is clean (no extra taxes due to polluted soil);
- The ground water consists of salt water that can be released in a close harbour;
- Additional costs such as risk, profit, interest and uncertainties are the same for each variant.

1.2 Cost per item

The cost estimation made is focused in the comparison of each variant (not in the overall costs). Costs that are assumed to be the same do not influence the relative costs (differences between variants). These costs are:

- Risk, profit, interest and uncertainties;
- Dredging port side and bottom protection;
- Quay drainages and corrosion protection.

The cost that are taken into account are described in this section, the value of each one is obtained from the archive of the Port of Rotterdam, where cost estimation of projects in the past are given.

1.2.1 Steel

The costs of steel can be divided in cost of material and costs of installation. The cost of delivery of steel vary between 900 and 1100 €/ton (archive Public works Rotterdam). This price includes the fabrication of the steel elements of a "normal size" (tubes until 1.82 meters diameter and 25 mm thickness). When the size of the tubular members is higher than 1.82 [m] or thicker than 25 [mm] higher fabrication costs need to be applied. Considering this, a value of 1000 €/ton is assumed for normal sizes (diameter <1.82 [m] and thickness < 25 [mm]) and 1500 €/ton for tubes with a diameter or thickness bigger than 1.82 [m] and 25 [mm] respectively (Reference Port of Rotterdam). The price of 1000 €/ton is maintained for the delivery of MV-piles, tubular piles, sheet piles and anchor rods.

The components of each variant vary in length and cross section, so different prices for the installation are assumed for each one of them. The price of installation depends in the surrounding, accessibility, dimensions of the elements (size of the hammer) and number of elements. In the archive of the Public works of Rotterdam, several prices of the installation of tubular piles and sheet piles are given. It is assumed that these values give a good estimation of the costs of installation of the steel elements:

- Delivery steel primary elements: 1500 €/ton
- Delivery overall steel: 1000 €/ton
- Installation primary elements: 300 €/m (euro per meter pile, diameter 2.8-2.9 m);
- Sheet pile elements: 25 €/m (reference PU25, 2 or 3 pieces)
- Tubular piles anchor wall: 100 €/m (piles around 1.4 m)
- Anchor rod: 56 €/m (per meter rod, 75 mm diameter)
- MV-pile and grouting: 250 €/m (per meter pile)

1.2.2 Concrete

The price of a concrete structure is determined by the concrete, reinforcement and formwork. Depending in the number of repetitions costs can be safe by applying a steel formwork (instead of wooden formwork). An integrated fixed price for 1 cubic meter of concrete for universal applications does not exist. The cost components as mentioned above are significantly influenced by project specific (local) conditions and conceptual choices made by engineers.

Nevertheless, overall prices per cubic meter in each country are used to have an indicator to identify trends and support selections in alternatives. The price of concrete including casting of concrete, formwork and reinforcement is set to 350 €/m^3 and the price for a bearing pile (In the case of option B) is assumed as 2400 €/piece (Archive Public works Rotterdam).

1.2.3 Earth works

The prices of earthworks are dependent on the amount of soil that is moved, the time, the machinery is used, the purity and transport of soil and the circumstances of the construction. When the space is limited (most of the times), temporary structures have to be used to make the excavation pit. Furthermore the machinery has to be able to reach the desired place. The assumption of enough construction space gives the possibility to excavate and apply slopes 1:2 in the excavation sides.

In the archive of the Port of Rotterdam an average value of 2-2.5 €/m^3 is given for the excavation and transport of soil (assuming clean sand). The same price can be used to calculate costs of refill.

1.2.4 Dewatering

Various dewatering systems can be used to bring the phreatic level of the building pit to the desired level. The difference between each type lays on the permeability of the soil and the required reduction of the phreatic level.

- Open drainage (pumping from open sumps): primary suitable for clay or peat soils and a phreatic level reduction of 2 m. If the soil is sand, a reduction of 0.5 m can be achieved;
- Horizontal drainage: suitable for a permanent reduction of the phreatic level of temporarily reduction of large areas. This type of drainage are normally placed at a maximum depth of 5 m from the surface;
- Well point drainages:
    - Closed vacuum drainage: used in high permeable soils. The tube works as filter and a phreatic level reduction of 4 m can be achieved;
    - Open vacuum drainage: suitable for non-permeable soils along the entire height. Reduction of about 6 m is attained;
    - Deep well drainage: Pump suspended at the bottom of the filter. Capacity much greater than the other types of drainage, the drawdown is limited by the capacity of the pump used.

The costs of a drainage system are normally given in €/m^3, thus the volume of water extracted during the construction time is needed. Several cost estimates of projects in the archive of the Public works Rotterdam show that the costs of dewatering have a really low impact in the total costs (less than 1%).

The costs of dewatering will not be taken into account for the comparison of the costs. To determine the costs of dewatering, the construction period, capacity and type of pump need to be known. The determination of these parameters will lead to more assumptions that can be neglected due to the low contribution to the total costs.

1.3 Dimensions

1.3.1 Option A: Quay wall without relieving structure

The dimensions of the variants are:

a) MV-pile as anchor

Diameter tubular piles:	2.8 m
Length tubular piles:	47.2, 46.6, 46.2 and 45 [m]
Thickness:	40, 33, 30 and 30 [mm] (anchor position +2.2,+0.2,-1.8 and -6.0 m NAP)
Increase in thickness:	15, 17, 13 and 4 [mm] (increase at level max moment +- 10 [m])

Secondary elements: 3xPU32, width 600 [mm] and length 39 [m]

MV-piles HP400x194: Length 75, 72, 70 and 67 [m] (anchor position +2.2,+0.2,-1.8 and -6.0 m NAP)
C.t.c. distance: 1.8, 1.5, 1.3 and 1.3 [m]

b) Anchor plate as anchor

Diameter tubular piles: 2.9 m
Length tubular piles: 47, 46.7, 46.5 and 45.7 [m]
Thickness: 35, 30, 25 and 20 [mm] (anchor position +2.2,+0.2,-1.8 and -6.0 m NAP)
Increase in thickness: 15, 14, 18 and 13 [mm] (increase at level max moment +- 10 [m])

Secondary elements: 3xPU32, width 600 [mm] and length 39 [m]

Distance of anchor plate from quay wall: 55 [m]
C.t.c. distance anchor rods: 0.8, 0.8, 0.8 and 0.6 [m]
Length anchor plate: 7.0, 8.5, 9.0, 9.0 [m]
Profile anchor plate: Combined profile, tubes 1.1 m_20 mm and 2xPU

1.3.2 Option B Quay wall with relieving structure

a) Length relieving structure L=10 m

Concrete structure

Vertical wall: Width 3 meters. Height: 3.5, 7.5 and 11.7

Slab: Length 10 meters. Thickness: 1 meter

Bearing piles: Prefab concrete piles 450x450 mm, One row c.t.c 2 [m]

MV-pile as anchor

Diameter tubular piles: 2.8 m, 2.6 m and 2.6 [m]
Length tubular piles: 45.5, 42 and 37 [m]
Thickness: 48, 42 and 31 [mm] (anchor position +2.2,-1.8 and -6.0 m NAP)

Secondary elements: 3xPU32, width 600 [mm] and length of 36, 32 and 28 [m]

MV-piles HP400x194: Length 72, 73 and 73 [m] (anchor position +2.2, -1.8 and -6.0 m NAP)
C.t.c. distance anchors: 2 [m]

Anchor plate as anchor

Diameter tubular piles: 2.8, 2.7 and 2.6 [m]
Length tubular piles: 45, 42 and 37 [m]
Thickness: 48, 41 and 36 [mm] (anchor position +2.2,-1.8 and -6.0 m NAP)

Secondary elements: 3xPU32, width 600 [mm] and length 39 [m]

Distance of anchor plate from quay wall: 60 [m]
C.t.c. distance anchor rods: 0.85, 0.75 and 0.7 [m]

Length anchor plate: 5 [m]
Profile anchor plate: Combined profile, tubes 1.1 m_20 mm and 2xPU

b) Length relieving structure L=20 meters

Concrete structure

Vertical wall: Width 3 meters. Height: 3.5, 7.5 and 11.7

Slab: Length 20 meters. Thickness: 1.8 meter

Bearing piles: prefab concrete piles 450x450 mm, two row s c.t.c 2 [m]

MV-pile as anchor

Diameter tubular piles: 2.8, 2.7 and 2.6 [m]
Length tubular piles: 45, 40.3 and 36 [m]
Thickness: 38, 33 and 20 [mm] (anchor position +2.2,-1.8 and -6.0 m NAP)

Secondary elements: 3xPU32, width 600 [mm] and length 36, 32 and 27 [m]

MV-piles HP400x194: Length 70, 71 and 71 [m] (anchor position +2.2, -1.8 and -6.0 m NAP)
C.t.c. distance anchors: 2 [m]

Anchor plate as anchor

Diameter tubular piles: 2.7, 2.5 and 2.3 [m]
Length tubular piles: 45, 40.3 and 35.8 [m]
Thickness: 39, 30 and 25 [mm] (anchor position +2.2,-1.8 and -6.0 m NAP)

Secondary elements: 3xPU32, width 600 [mm] and length 36, 32 and 27 [m]

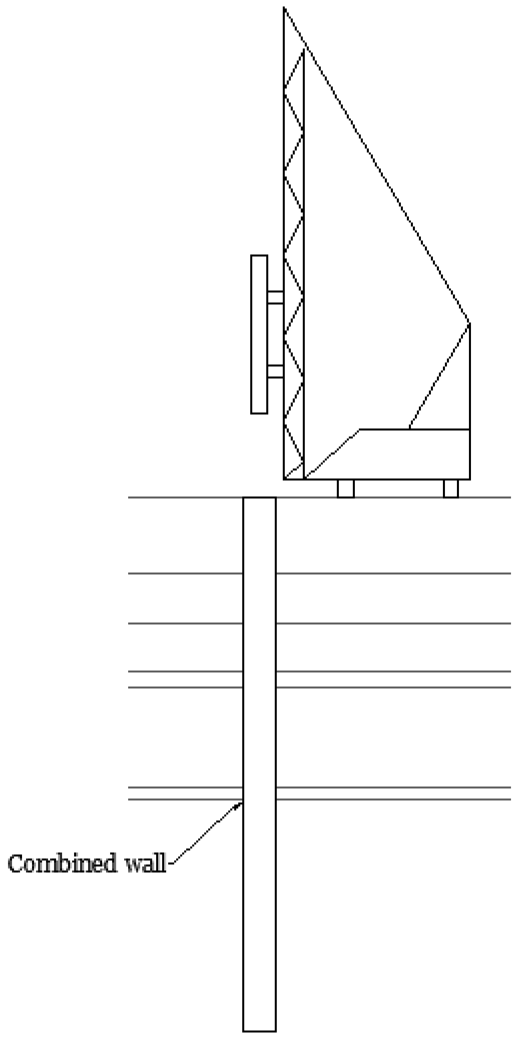
Distance of anchor plate from quay wall: 60 [m]
C.t.c. distance anchor rods: 1 [m]
Length anchor plate: 5 [m]
Profile anchor plate: Combined profile, tubes 1.1 m_20 mm and 2xPU

1.4 Construction phases

The costs are based in the following construction phases.

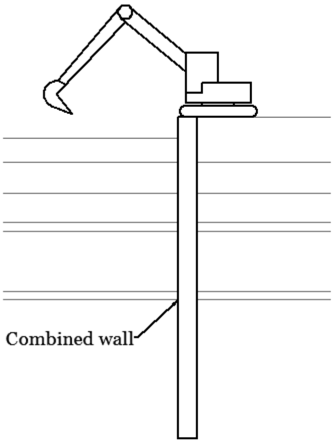
1.4.1 Option A: Quay wall without relieving structure

Phase 1: Sheet pile driving.

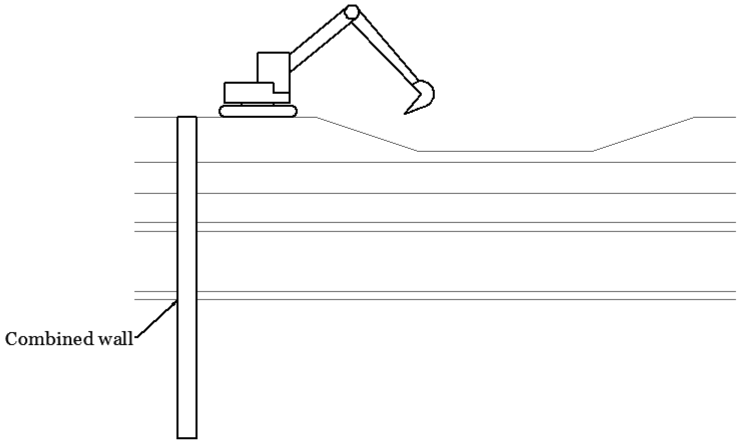


Phase 2: Excavation until anchor level

The slope of the excavation pit is assumed to be 1:2 to allow access of the machinery and equipment.

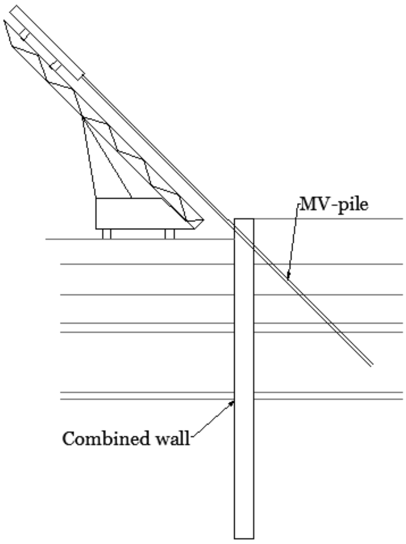


Option A: MV-piles

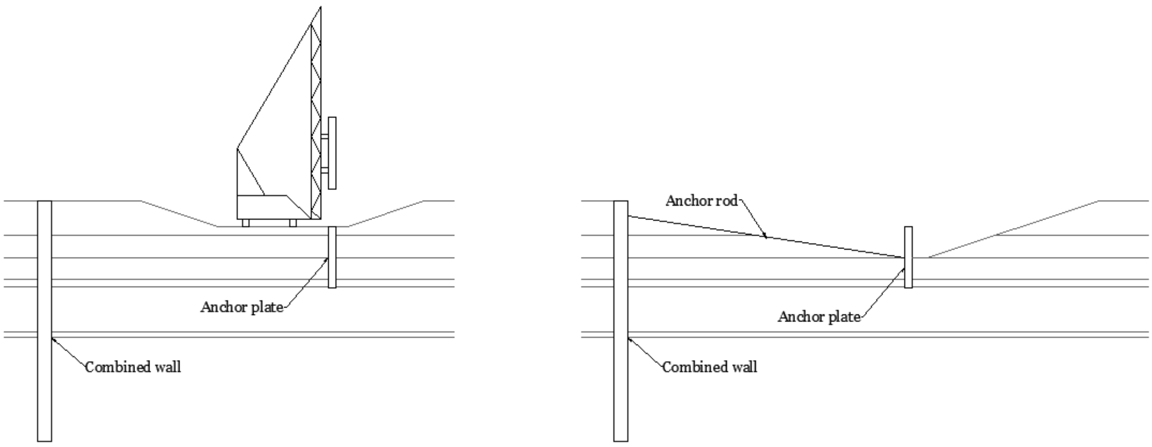


Option A: Anchor plate

Phase 3: Placement of the anchor

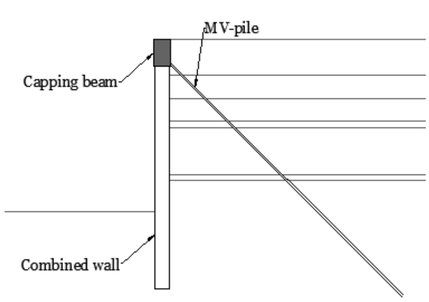


Option A: MV-piles

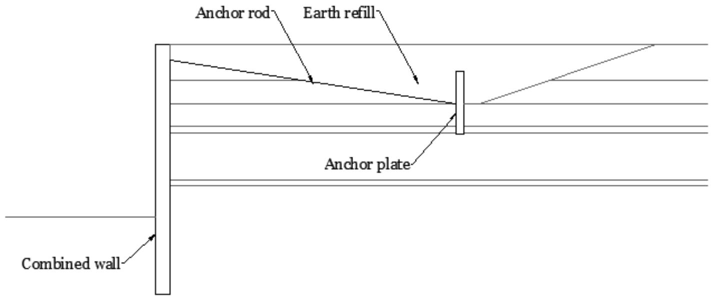


Option A: Anchor plate

Phase 4: Backfill and dredging portside



Option A: MV-piles

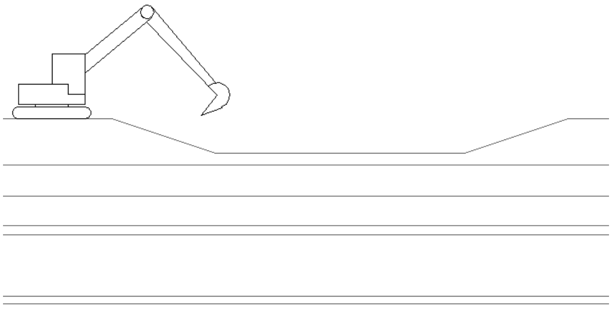


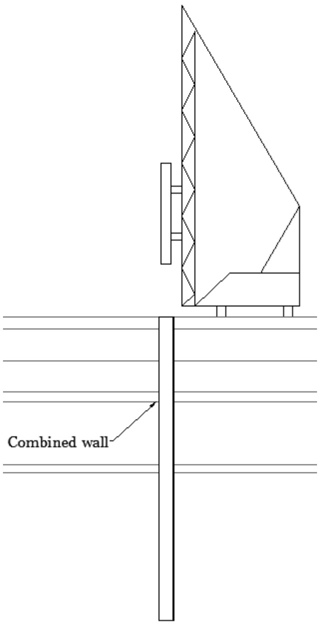
Option A: Anchor plate

The costs of excavation/dredging in the port side have no impact in the difference in costs of each variant.

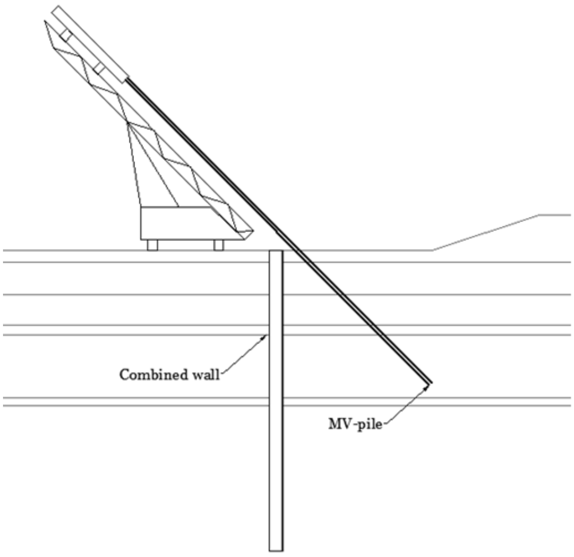
1.4.2 Option B: Quay wall without relieving structure

Phase 1: Excavation until bottom relieving structure and placement combi-wall

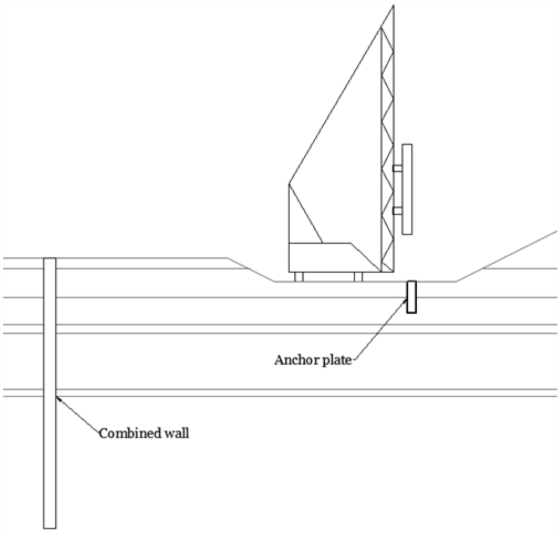




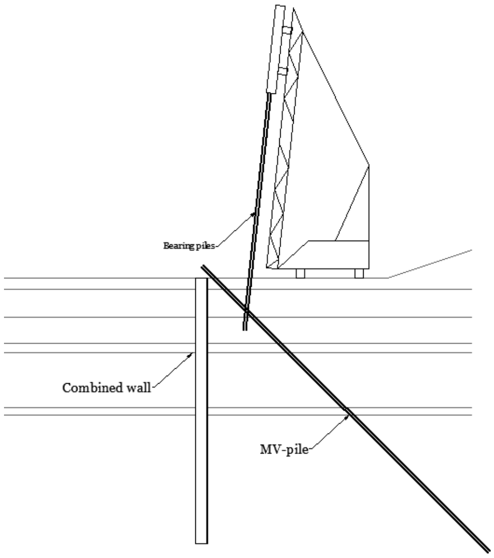
Phase 2: Placement of the anchor and bearing piles



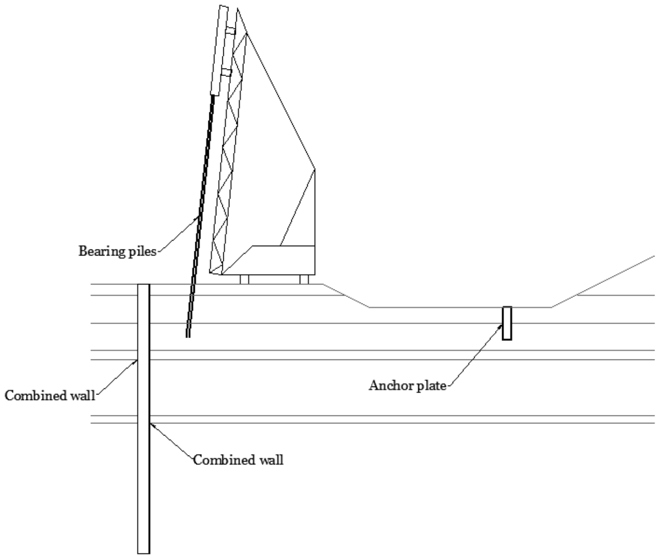
Option B: MV-piles



Option B: Anchor plate

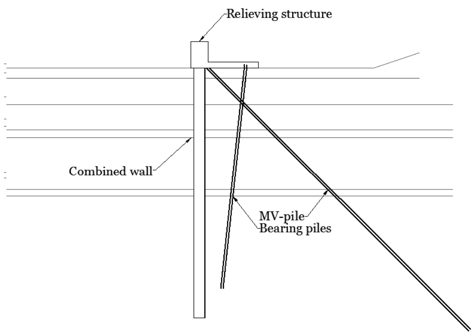


Option B: MV-piles

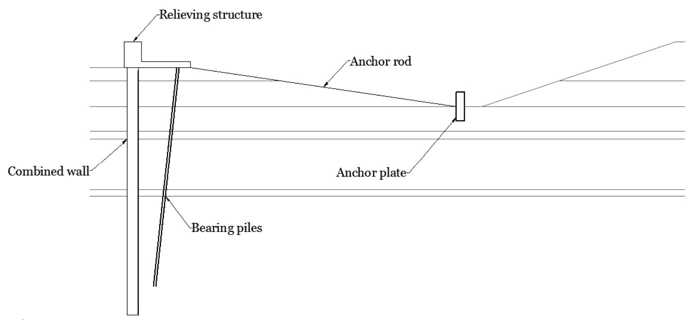


Option B: Anchor plate

Phase 3: Cast of the relieving structure

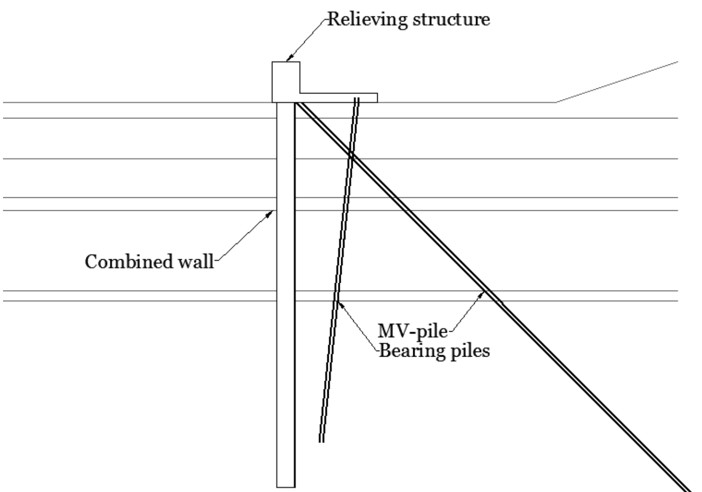


Option B: MV-piles

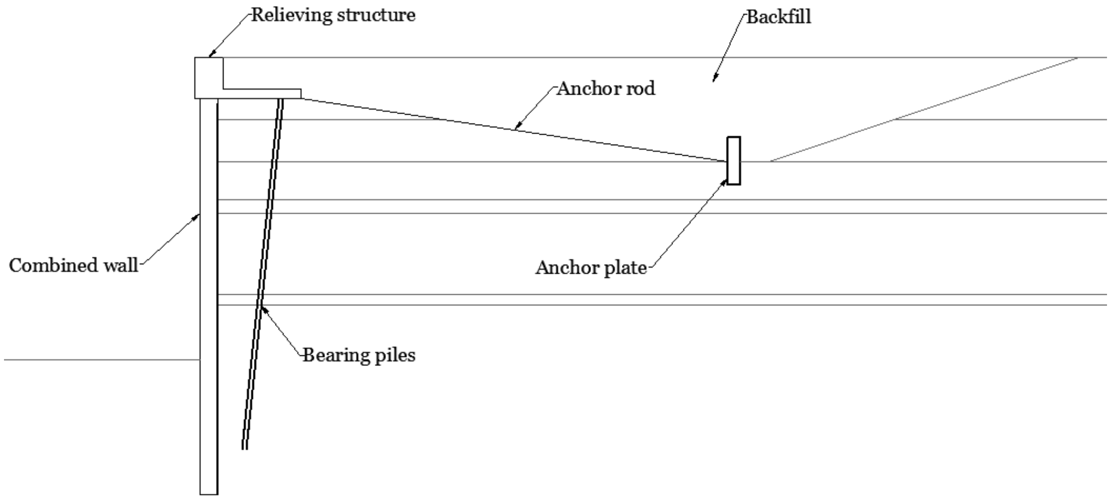


Option B: Anchor plate

Phase 4: Backfill and dredging portside



Option B: MV-piles



Option B: Anchor plate

1.5 Material quantities

1.5.1 Option A: Quay wall without relieving structure

A summary of the material quantities is given in tables D.1 and D.2. For the detailed calculation of the material quantities see tables D.5 and D.6.

Table D.1 Material quantity Option A: MV-pile

Earth works	**Anchor position**	**+2.2 m NAP**	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Area of excavation (refill) in 1000 m	[m²]	0	0	0	0

Primary members

Area	[m²]	3.47E-01	2.87E-01	2.61E-01	2.61E-01
Steel volume primary members	[m³/piece]	14.64	11.36	9.71	8.30
Steel weight	[ton/piece]	110.97	86.13	73.63	62.95
Pieces in 1000 m	[-]	218.39	218.39	218.39	218.39
Steel in 1000 m	[ton]	24235.42	18809.93	16081.00	13746.66

Extra thickness primary elements	[mm]	15.00	17.00	13.00	4.00
Extra area steel	[m²]	1.27E-01	1.45E-01	1.11E-01	3.44E-02
Extra volume steel	[m³/piece]	2.55	2.90	2.23	0.69
Total extra volume steel per 1000 m	[m³]	556.77	633.80	486.46	150.17
Total steel + extra thickness	[ton]	2.86E+04	2.38E+04	1.99E+04	1.49E+04

Secondary members

Cross-section secondary elements 3xPU32	[m²]	4.36E-02	4.36E-02	4.36E-02	4.36E-02
Length	[m]	39.00	39.00	39.00	39.00
Volume steel secondary elements	[m³/piece]	1.70	1.70	1.70	1.70
Number of pieces per 1000 m	[-]	217.39	217.39	217.39	217.39
Steel in 1000 m secondary members	[ton]	2899.85	2899.85	2899.85	2899.85

Anchors

MV-pile c.t.c. distance	[m]	1.80	1.50	1.30	1.30
MV-pile length	[m]	75.00	72.00	70.00	67.00
MV-pile area cross section	[m²]	2.48E-02	2.48E-02	2.48E-02	2.48E-02
Steel per piece	[ton/piece]	14.58	13.99	13.60	13.02
Number of pieces per 1000 m	[-]	556.56	667.67	770.23	770.23
Total steel mv piles	[ton]	8.11E+03	9.34E+03	1.05E+04	1.00E+04

Concrete

volume concrete	[m³/m]	16.5	23.1	29.7	43.56
Volume concrete in 1000 [m]	[m³]	16500	23100	29700	43560

Table D.2 Material quantity Option A: Anchor plate

Earth works	Anchor position	+2.2 m NAP	+0.2 m NAP	-1.8 m NAP	-6.0 m NAP
Area of excavation (refill) in 1000 m	[m²]	1.15E+06	1.30E+06	1.46E+06	2.00E+06

Primary members

Area	[m²]	3.15E-01	2.70E-01	2.26E-01	1.81E-01
Steel volume primary members	[m³/piece]	14.81	12.63	10.50	8.27
Steel weight	[ton/piece]	112.26	95.78	79.61	62.70
Pieces in 1000 m	[-]	213.77	213.77	213.77	213.77
Steel in 1000 m	[ton]	2.40E+04	2.05E+04	1.70E+04	1.34E+04

Extra thickness primary elements	[mm]	15.00	14.00	18.00	13.00
Extra area steel	[m²]	0.13	0.12	0.16	0.12
Extra volume steel	[m³/piece]	2.65	2.49	3.20	2.33
Total extra volume steel per 1000 m	[m³]	567.14	531.40	684.67	497.11
Total steel + extra thickness	[ton]	2.85E+04	2.46E+04	2.24E+04	1.73E+04

Secondary members

Cross-section secondary elements 3xPU32	[m²]	4.36E-02	4.36E-02	4.36E-02	4.36E-02
Length	[m]	39.00	39.00	39.00	39.00
Volume steel secondary elements	[m³/piece]	1.70	1.70	1.70	1.70
Number of pieces per 1000 m	[-]	212.77	212.77	212.77	212.77
Steel in 1000 m secondary members	[ton]	2838.15	2838.15	2838.15	2838.15

Anchors

Anchor rod c.t.c. distance	[m]	0.80	0.80	0.80	0.70
Anchor rod length	[m]	55.00	55.00	55.00	55.00
Anchor rod cross section	[m²]	4.42E-03	4.42E-03	4.42E-03	4.42E-03
Steel per piece anchor rod	[ton/piece]	1.91	1.91	1.91	1.91
Number of pieces per 1000 m	[-]	1251.00	1251.00	1251.00	1429.57
Total steel anchor rods	[ton]	2386.78	2386.78	2386.78	2727.48
Cross-section primary members Anchor plate	[m²]	6.79E-02	6.79E-02	6.79E-02	6.79E-02
Volume steel anchor plate in 1000 m	[m³]	294.91	358.10	379.17	379.17
Total steel anchor plate in 1000 m	[ton]	2315.62	2811.82	2977.22	2977.22

Table D.3: Material quantity Option B: MV-pile

Earth works	Anchor position	L=10 m +2.2 m NAP	L=10 m -1.8 m NAP	L=10 m -6.0 m NAP	L=20 m +2.2 m NAP	L=20 m -1.8 m NAP	L=20 m -6.0 m NAP
Area of excavation (refill)	[m²]	101750	282818	551930	170670	444010	804989
Primary members							
Area	[m²]	4.15E-01	3.30E-01	2.40E-01	3.30E-01	2.66E-01	1.62E-01
Steel volume primary members	[m³/piece]	19	14	9	15	11	6
Steel weight	[ton/piece]	143	105	67	113	81	44
Pieces in 1000 m	[-]	218	228	234	218	228	228
Steel in 1000 m	[ton]	31266	23960	15755	24569	18562	10100
Secondary members							
Cross-section secondary elements 3xPU32	[m²]	4.36E-02	4.36E-02	4.36E-02	4.36E-02	4.36E-02	4.36E-02
Length	[m]	36	32	28	36	32	27
Volume steel secondary elements	[m³/piece]	1.57E+00	1.39E+00	1.22E+00	1.57E+00	1.39E+00	1.18E+00
Number of pieces per 1000 m	[-]	217	227	233	217	227	227
Steel in 1000 m secondary members	[ton]	2677	2488	2227	2677	2488	2099
Anchors							
Mv pile c.t.c. distance	[m]	2	2	2	2	2	2
Mv-pile length	[m]	72	73	73	70	71	71
MV-pile area cross section	[m²]	2.48E-02	2.48E-02	2.48E-02	2.48E-02	2.48E-02	2.48E-02
Steel per piece	[ton/piece]	14	14	14	14	14	14
Number of pieces per 1000 m	[-]	501	501	501	501	501	501
Total steel mv piles	[ton]	7010	7107	7107	6815	6913	6913
Concrete							
Volume vertical wall	[m³/m]	1.05E+01	2.25E+01	3.51E+01	1.17E+01	2.37E+01	3.63E+01
Volume slab	[m³/m]	1.15E+01	1.15E+01	1.15E+01	3.87E+01	3.87E+01	3.87E+01
Total volume concrete per meter	[m³/m]	2.20E+01	3.40E+01	4.66E+01	5.04E+01	6.24E+01	7.50E+01
Volume concrete in 1000 [m]	[m³]	2.20E+04	3.40E+04	4.66E+04	5.04E+04	6.24E+04	7.50E+04
Bearing piles in 1000 m	[pieces]	500	500	500	1000	1000	1000

Table D.4 Material quantity Option B: Anchor plate

Earth works	Anchor position	L=10 +2.2 m NAP	L=10 -1.8 m NAP	L=10 -6.0 m NAP	L=20 +2.2 m NAP	L=20 -1.8 m NAP	L=20 -6.0 m NAP
Area of excavation (refill)	[m²]	1.3E+6	2.0E+6	2.5E+6	1.2E+6	1.9E+6	2.3E+6
Primary members							
Area	[m²]	0.41	0.35	0.29	0.33	0.23	0.18
Steel volume primary members	[m³/piece]	19	15	11	15	9	6
Steel weight	[ton/piece]	143	112	81	111	71	48
Pieces in 1000 m	[-]	218	223	228	223	234	245
Steel in 1000 m	[ton]	31266	24930	18570	24831	16613	11878
Secondary members							
Cross-section secondary elements 3xPU32	[m²]	0.044	0.044	0.044	0.044	0.044	0.044
Length	[m]	36	32	28	36	32	27
Volume steel secondary elements	[m³/piece]	1.57	1.39	1.22	1.57	1.39	1.18
Number of pieces per 1000 m	[-]	217	222	227	222	233	244
Steel in 1000 m secondary members	[ton]	2677	2432	2177	2736	2545	2252
Anchors							
Anchor rod c.t.c. distance	[m]	0.85	0.75	0.7	1	1	1
Anchor rod length	[m]	50	50	50	40	40	40
Anchor rod cross section	[m²]	0.0044	0.0044	0.0044	0.0044	0.0044	0.0044
Steel per piece anchor rod	[ton/piece]	1.73	1.73	1.73	1.39	1.39	1.39
Number of pieces per 1000 m	[-]	1177	1334	1430	1001	1001	1001
Total steel anchor rods	[ton]	2042	2314	2480	1389	1389	1389
Cross-section primary members Anchor plate	[m²]	0.0679	0.0679	0.0679	0.0679	0.0679	0.0679
Volume steel anchor plate in 1000 m	[m³]	211	211	211	211	211	211
Total steel anchor plate in 1000 m	[ton]	1654	1654	1654	1654	1654	1654
Concrete							
Volume vertical wall	[m³/m]	10.50	22.50	35.10	11.70	23.70	36.30
Volume slab	[m³/m]	11.50	11.50	11.50	38.70	38.70	38.70
Total volume concrete per meter	[m³/m]	22.00	34.00	46.60	50.40	62.40	75.00
Volume concrete in 1000 [m]	[m³]	22000	34000	46600	50400	62400	75000
Bearing piles in 1000 m	[pieces]	500	500	500	1000	1000	1000

1.6 Costs

1.6.1 Option A: Quay wall without relieving structure

Table D.3 Costs per 1000 meter Option A: MV-pile

Anchor level	**+2.2 m NAP**	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ -	€ -	€ -	€ -
Steel primary members	€ 42,910,732.93	€ 35,679,824.32	€ 29,850,975.38	€ 22,388,722.81
Placing primary members	€ 2,764,833.91	€ 2,594,488.70	€ 2,437,246.96	€ 2,083,453.04
Steel secondary elements	€ 2,899,846.02	€ 2,899,846.02	€ 2,899,846.02	€ 2,899,846.02
Placing secondary elements	€ 211,956.52	€ 211,956.52	€ 211,956.52	€ 211,956.52
Steel MV-pile	€ 8,111,950.28	€ 9,342,168.26	€ 10,477,903.59	€ 10,028,850.58
Placing MV-pile	€ 10,416,666.67	€ 10,000,000.00	€ 9,722,222.22	€ 9,305,555.56
Concrete	€ 5,775,000.00	€ 8,085,000.00	€ 10,395,000.00	€ 15,246,000.00
TOTAL	€ 73,090,986.32	€ 68,813,283.82	€ 65,995,150.69	€ 62,164,384.53
Costs per meter wall	€ 73,090.99	€ 68,813.28	€ 65,995.15	€ 62,164.38

Table D.4 Costs per 1000 meters Option A: Anchor plate

Anchor level	**+2.2 m NAP**	**+0.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ 2,873,437.50	€ 3,245,300.00	€ 3,660,800.00	€ 5,009,200.00
Steel primary members	€ 42,675,719.54	€ 36,969,046.41	€ 33,590,772.55	€ 25,959,827.82
Placing primary members	€ 3,014,100.00	€ 2,994,861.06	€ 2,982,035.11	€ 2,930,731.28
Steel secondary elements	€ 2,838,147.17	€ 2,838,147.17	€ 2,838,147.17	€ 2,838,147.17
Placing secondary elements	€ 207,446.81	€ 207,446.81	€ 207,446.81	€ 207,446.81
Steel Anchor plate + anchor rod	€ 4,702,400.71	€ 5,198,604.78	€ 5,364,006.14	€ 5,704,702.40
Placing anchor rod and anchor plate	€ 4,842,210.43	€ 4,842,210.43	€ 4,842,210.43	€ 5,392,210.43
Concrete	€ -	€ -	€ -	€ -
TOTAL	€ 61,153,462.16	€ 56,295,616.66	€ 53,485,418.21	€ 48,042,265.90
Costs per meter wall	€ 61,153.46	€ 56,295.62	€ 53,485.42	€ 48,042.27

Table D.4 Costs per 1000 meter Option B: MV-pile

Length relieving structure	**L=10**		
Anchor level	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ 254,375.00	€ 707,046.00	€ 1,379,825.00
Steel primary members	€ 46,898,755.40	€ 35,940,355.82	€ 23,632,214.87
Placing primary members	€ 2,981,041.30	€ 2,876,236.36	€ 2,592,495.35
Steel secondary elements	€ 2,676,780.94	€ 2,487,513.60	€ 2,227,192.41
Placing secondary elements	€ 195,652.17	€ 181,818.18	€ 162,790.70
Steel MV-pile	€ 7,010,124.26	€ 7,107,487.10	€ 7,107,487.10
Placing MV-pile	€ 9,018,000.00	€ 9,143,250.00	€ 9,143,250.00
Concrete	€ 7,700,000.00	€ 11,900,000.00	€ 16,310,000.00
Bearing piles	€ 1,200,000.00	€ 1,200,000.00	€ 1,200,000.00
TOTAL	€ 77,934,729.08	€ 71,543,707.06	€ 63,755,255.43
Costs per meter wall	€ 77,934.73	€ 71,543.71	€ 63,755.26

Length relieving structure	**L=20**		
Anchor level	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ 426,675.00	€ 1,110,025.00	€ 2,012,471.75
Steel primary members	€ 36,853,610.25	€ 27,843,497.93	€ 15,150,645.68
Placing primary members	€ 2,948,282.61	€ 2,759,817.27	€ 2,465,345.45
Steel secondary elements	€ 2,676,780.94	€ 2,487,513.60	€ 2,098,839.60
Placing secondary elements	€ 195,652.17	€ 181,818.18	€ 153,409.09
Steel MV-pile	€ 6,815,398.59	€ 6,912,761.43	€ 6,912,761.43
Placing MV-pile	€ 8,767,500.00	€ 8,892,750.00	€ 8,892,750.00
Concrete	€ 17,640,000.00	€ 21,840,000.00	€ 26,250,000.00
Bearing piles	€ 2,400,000.00	€ 2,400,000.00	€ 2,400,000.00
TOTAL	€ 78,723,899.56	€ 74,428,183.42	€ 66,336,223.01
Costs per meter wall	€ 78,723.90	€ 74,428.18	€ 66,336.22

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Table D.5 Costs Option B: Anchor plate

Length relieving structure	**L=10**		
Anchor level	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ 3,237,888.50	€ 4,914,825.00	€ 6,125,000.00
Steel primary members	€ 46,898,755.40	€ 37,395,203.12	€ 27,854,873.15
Placing primary members	€ 2,981,041.30	€ 2,812,600.00	€ 2,533,827.27
Steel secondary elements	€ 2,676,780.94	€ 2,432,235.52	€ 2,176,574.40
Placing secondary elements	€ 195,652.17	€ 177,777.78	€ 159,090.91
Steel MV-pile	€ 3,696,281.76	€ 3,968,352.93	€ 4,133,538.99
Placing MV-pile	€ 3,921,917.65	€ 4,361,133.33	€ 4,627,800.00
Concrete	€ 7,700,000.00	€ 11,900,000.00	€ 16,310,000.00
Bearing piles	€ 1,200,000.00	€ 1,200,000.00	€ 1,200,000.00
TOTAL	€ 72,508,317.73	€ 69,162,127.68	€ 65,120,704.72
Costs per meter wall	€ 72,508.32	€ 69,162.13	€ 65,120.70

Length relieving structure	**L=20**		
Anchor level	**+2.2 m NAP**	**-1.8 m NAP**	**-6.0 m NAP**
Earthworks	€ 3,080,000.00	€ 4,811,700.00	€ 5,787,500.00
Steel primary members	€ 37,246,402.70	€ 24,919,720.85	€ 17,816,502.72
Placing primary members	€ 3,013,500.00	€ 2,823,717.91	€ 2,630,252.20
Steel secondary elements	€ 2,736,264.96	€ 2,545,362.75	€ 2,252,413.23
Placing secondary elements	€ 200,000.00	€ 186,046.51	€ 164,634.15
Steel MV-pile	€ 3,042,964.08	€ 3,042,964.08	€ 3,042,964.08
Placing MV-pile	€ 2,867,240.00	€ 2,867,240.00	€ 2,867,240.00
Concrete	€ 17,640,000.00	€ 21,840,000.00	€ 26,250,000.00
Bearing piles	€ 2,400,000.00	€ 2,400,000.00	€ 2,400,000.00
TOTAL	€ 72,226,371.74	€ 65,436,752.10	€ 63,211,506.37
Costs per meter wall	€ 72,226.37	€ 65,436.75	€ 63,211.51

Annex F: Supplementary work and theory

This attachment is made to clarify some aspects of this investigation regarding the design philosophy and the Boussinesq approximation for strip loads.

F.1 Design philosophy

During the process of this research some misunderstanding regarding the Eurocode were found. These misunderstandings and the consequences are explained in this section.

1.1 Load combinations

The table shown in Chapter 4 section 4.6.5 of the main report show the load combinations used in this investigation for the dimensioning for the quay wall. In this table the surface load is taken as extreme and it is not reduced in the load combinations, however, the designer should consider all combinations of forces in each load case scenario to assess which one is decisive. The next table show the combinations that the designer should take into account. The combinations used in this thesis are: LC1.1, LC2.1.1, LC2.2.1 and LC3.1.

		ULS									
		LC1.1	**LC1.2**	**LC2.1.1**	**LC2.1.2**	**LC2.1.3**	**LC2.2.1**	**LC2.2.2**	**LC2.2.3**	**LC3.1**	**LC3.2**
1	**Dead load from superstructure**	1	1	1	1	1	1	1	1	1	1
2	**Soil pressure**	1	1	1	1	1	1	1	1	1	1
3	**Surface load**	1*1.1	0.6*1.1	1*1.1	0.7*1.1	0.7*1.1	1*1.1	0.7*1.1	0.7*1.1	0.7*1.1	1*1.1
4	**Crane load**	0.6*1.1	1*1.1	0.6*1.1	1*1.1	0.6*1.1	0.6*1.1	1*1.1	0.6*1.1	1*1.1	0.6*1.1
5	**Bollard load**			0.7*1.1	0.7*1.1	1*1.1					
6	**Fender load**						0.7*1.1	0.7*1.1	1*1.1		
7	**Water pressure (Average water level)**	x	x								
8	**Water pressure (Rare water level 1/50 year)**			x	x	x	x	x	x		
9	**Water pressure (Extreme water levels 1/250 years)**									x	x

To clarify the differences between the approaches used in this investigation two calculations are done, one for Option A: MV-pile at -6.0 m NAP and the other one for Option B: MV-pile at +2.2 m NAP. The next tables show the results of the two calculations. In both, the values used for the design are not normative, however the difference is negligible.

Option A MV-6.0 m NAP

	LC1.1	**LC1.2**	**LC2.1.1**	**LC2.1.2**	**LC2.1.3**	**LC2.2.1**	**LC2.2.2**	**LC2.2.3**	**LC3.1**	**LC3.2**
Moment [kNm/m]	14500	13530	15890	13710	13940	15250	13980	14330	12030	16390
Anchor force [kN/m]	3225	3623	4467	3899	4053	3261	3436	3671	3461	4498

Option B AP+2.2 m NAP

	LC1.1	**LC1.2**	**LC2.1.1**	**LC2.1.2**	**LC2.1.3**	**LC2.2.1**	**LC2.2.2**	**LC2.2.3**	**LC3.1**	**LC3.2**
Moment [kNm/m]	25110	20780	26010	22000	21680	26140	21330	21780	25590	26760
Anchor force [kN/m]	2154	1839	2357	2057	2085	2176	1801	1861	2211	2302

Aforementioned table shows that the normative combination is LC3.2 (yellow) and the used values for dimensioning are not normative (green). However, in both options the difference is small (moments 3% and anchor forces less than 1%), from this comparison it can be concluded that this would not affect the designs or the conclusions of this thesis.

1.2 Load factors

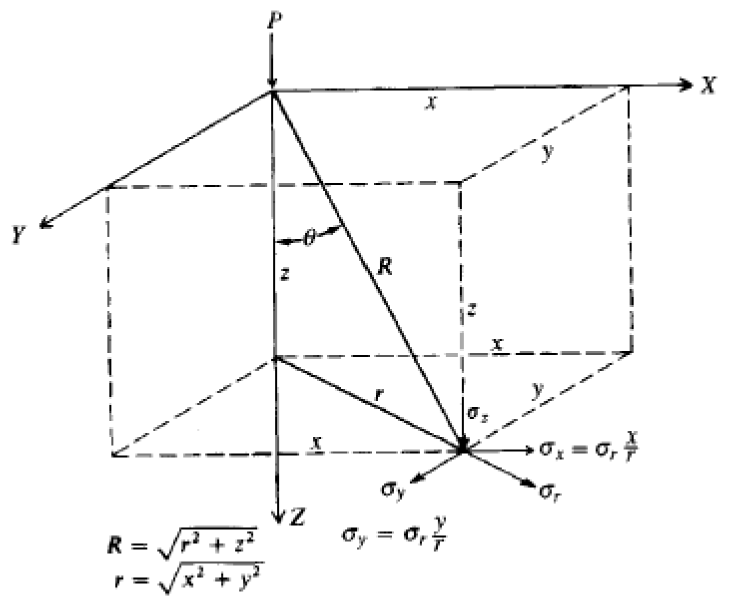
In the Eurocode it is stated that the safety factor for variable loads may be reduced when partial factors for soil parameters are applied. However, this depends on the safety class of the quay. In this case the safety factor is reduced from 1.3 to 1.1, which is not correct. The safety factor may only be reduced to 1.1 when dealing with a structure of safety class RC3. In this thesis a factor 1.1 is used in the design of the structures, which leads to less safety. The consequence of this is lower moments (14%) and lower anchor forces (10%) in the ULS (see table below for the results of a calculation with factor 1.3). However, during the research the stability of the anchor was decisive during the design and higher anchor forces can be taken due to the extra length provided to meet the stability criteria. Regarding the combined elements less safety is provided which is not desired.

Option B AP+2.2 m NAP factor 1.3					
	LC1.1	LC2.1.1	LC2.2.1	LC3.1	
Moment [kNm/m]	27970	29010	29020	25590	29800
Anchor force [kN/m]	2368	2597	2378	2211	2523

The comparison in forces made in Chapter 6 and 7 are done in the SLS and the mentioned aspects do not influence the conclusions taken.

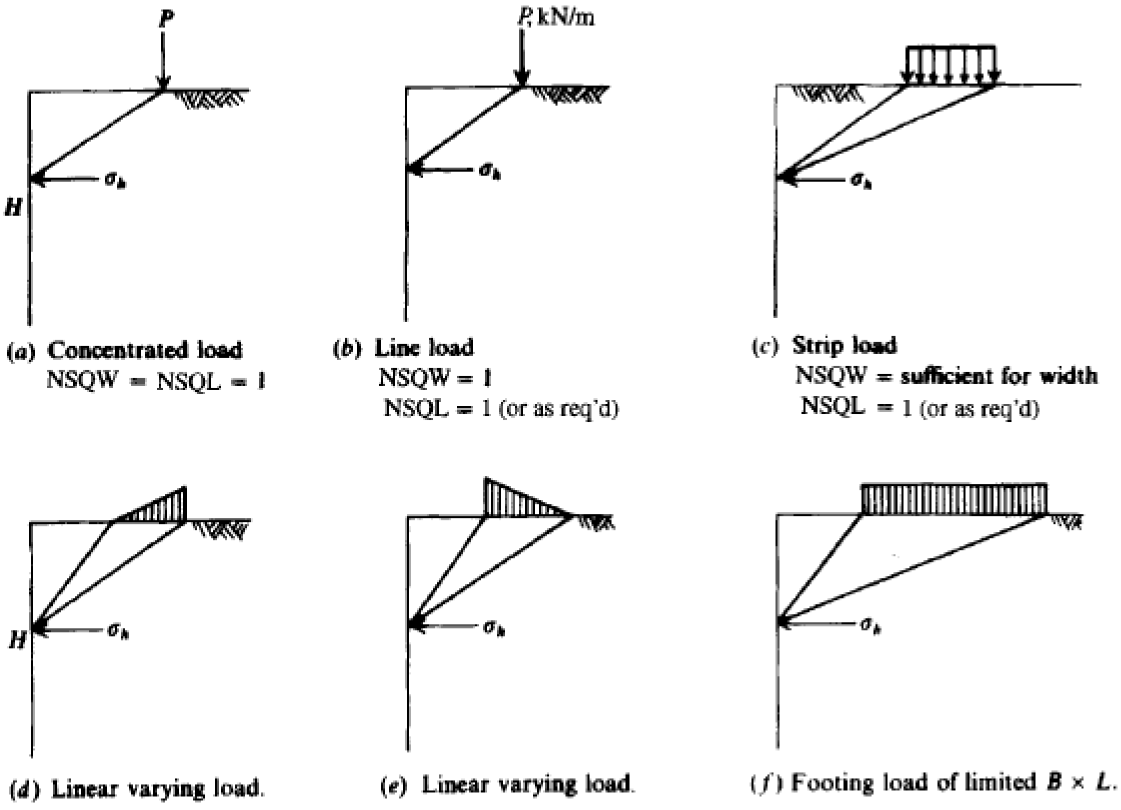
F.2 Boussinesq theory for strip loads

The Boussinesq approach for point loads is showed in the next figure. This equation is bases on the theory of elasticity and by varying the depth, the pressure on the wall due to a point load on the surface can be estimated.



$$\sigma_r = \frac{P}{2\pi}\left[\frac{3r^2 z}{R^5} - \frac{1-2\mu}{R(R+z)}\right]$$

When dealing with strip loads, this equation can be integrated to obtain the pressure profile due to (non-) uniform strip loads.



(a) Concentrated load NSQW = NSQL = 1

(b) Line load NSQW = 1 NSQL = 1 (or as req'd)

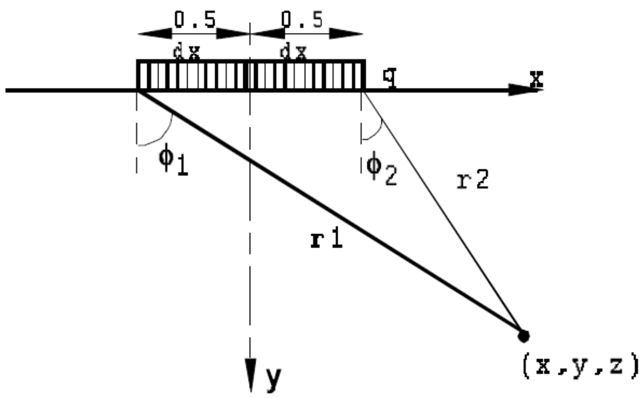
(c) Strip load NSQW = sufficient for width NSQL = 1 (or as req'd)

(d) Linear varying load.

(e) Linear varying load.

(f) Footing load of limited B × L.

This is the approach that D-sheet uses to estimate the horizontal pressure due to surcharges on the retaining structure, the equation is as follow:



$$\sigma_H = \frac{K\ P}{\pi}\left[(\phi_1 - \phi_2) + \sin\phi_1 \cos\phi_1 - \sin\phi_2 \cos\phi_2\right]$$