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"Parametric Analysis of Quay Walls with Relieving Platform, by means of Elastic Supported Beam and Finite Element Method"

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"Engineering is the art of modelling materials we do not wholely understand, into shapes we cannot precisely analyse so as to withstand forces we cannot entirely assess, in such a way that the public has no reason to suspect the extend of our ignorance"

A.R.Dykes 1976

Preface

This report is the final piece of the Master of Science thesis titled “Parametric Analysis of Quay Walls with a relieving platform, by means of Elastic Supported Beam and Finite Element Method”.

This thesis is part of the Hydraulic Engineering– Hydraulic Structures MSc program at the faculty of Civil Engineering and Geosciences of Delft University of Technology. This study has been performed in cooperation and under the supervision of Gemeentewerken Rotterdam. This thesis work has been assessed and supported by the graduation committee, which consist of the following members:

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I would like to thank all the graduation committee and all the colleagues of Gemeentewerken Rotterdam who gave me their support during my thesis work. Special thanks to my family for their moral and material support all this period and to my god-father who is the “instigator” of my Odyssey in the Netherlands…

Rotterdam, May 2008

Anastasia Karamperidou

Summary

In cases where the sheet pilling is allowed, when great retaining heights have to be achieved a combined quay wall structure is normally used. This structure consists of a sheet pile wall, a relieving platform, an anchor and a pile foundation system.

The content of the present MSc thesis is divided into two major parts. The first part (Part A) contains mostly hand calculations for the determination of the loads and for the analysis of the sheet pile wall, while on the second part (Part B) advanced computer programs will be used for the same purpose. In Part A the structure is divided and analysed separately. For the relieving platform’s analysis and the Blum’s Analysis of the Sheet Pile Wall the loads were determined manually and the structures were analysed with respect to classic mechanics.

A parametric analysis of seven different quay walls is investigated, for various loading combinations of given loads. In details, three different depths and two different widths of the relieving platform were investigated. In addition a quay wall combined only from sheet piles, without a relieving platform is also analysed.

The sheet pile wall, in Part A, is analysed by three different approaching theories of Blum. The fixity of the end point of the sheet piling is considered once as completely free and in the other two as fully fixed. The Free Earth Support method is used in the first case, while in the second, two different methodologies, both considering a fully fixity at the toe level are used (Fixed Earth Support method). Graphs concerning the effect of the various dimensions of the relieving platform, the embedded depth and the anchor force are produced.

In the Elastic Supported Beam theory (Msheet) analysis the ground is simulated according to a more advanced model. According to that theory, the Sheet Pile Wall is schematised as a beam on an elastic foundation, where the ground is simulated by a set of uncoupled elasto-plastic springs. On that stage the axial forces, estimated from the static analysis of the relieving platform are imposed on the top of the sheet pile wall as external loads.

In Part B, the same process is repeated but this time with a Finite Elements Method. PLAXIS created from Plaxis will be used for that purpose. This type of analysis is based on a model in which the behaviour of soil and structure are integrated.

Through a parametric analysis and a financial assessment of a quay wall, the economic effects of the relieving platform separately on the sheet pile wall and on the whole structure will be estimated.

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CHAPTER 1

PROBLEM DESCRIPTION

1.1 Objective

In cases where the sheet pilling is allowed, when great retaining heights have to be achieved a combined quay wall structure is normally used. This structure consists of a sheet pile wall, a relieving platform, an anchor and a pile foundation system.

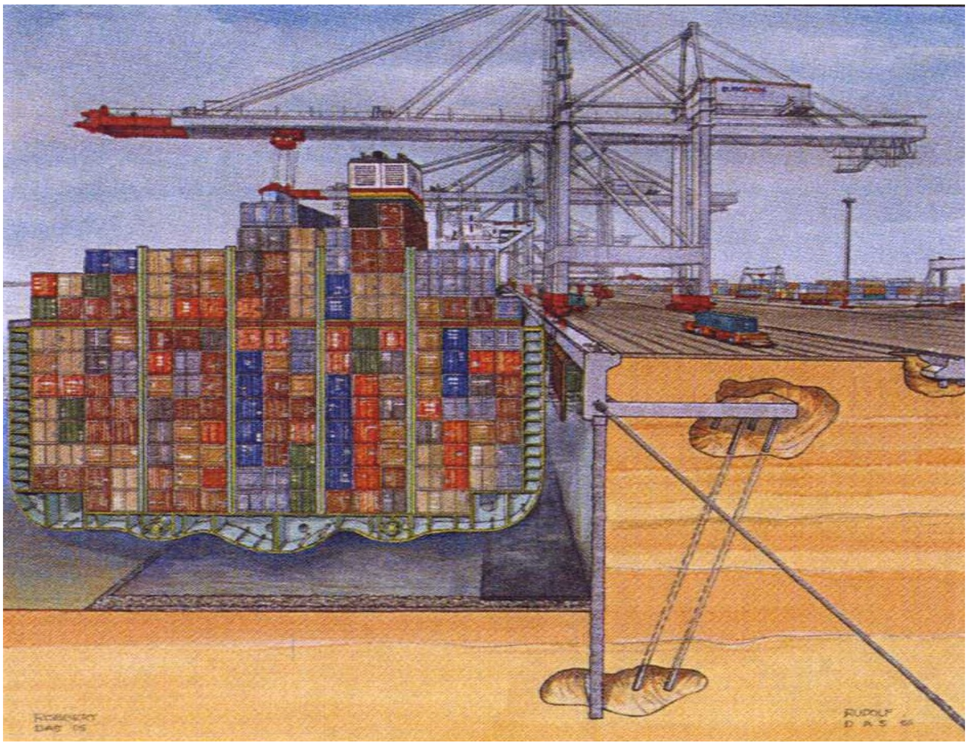


Figure 1.1: Schematisation of a container terminal

In this type of quay wall the horizontal load is significantly reduced by the presence of the relieving platform. Such structures with a relieving platform have been already used in the port of Rotterdam and elsewhere. In any specific situation, various motives influence the choice of the construction depth and width of the platform, including:

- Savings on the costs of the sheet pile wall by reducing moments and pile depths;
- Shortening the length of sheet piles to limit installation risks;
- Restricting the length of the foundation members such as tension and bearing piles in relation to availability and feasibility;
- Saving on the number of tension members in the pile trestle system by increasing the vertical load component of the soil;

During the preliminary design of a quay wall, experience gained from previous projects with similar boundary conditions is often used. Dimensions of the relieving platform applied in previous cases are used as a first estimation.

The aim of the present MSc Thesis is to investigate the influence of the dimensions of the relieving platform in the sheet pile wall design, and how this affect the final cost of the structure. Through a parametric analysis and a financial assessment of a quay wall, the economic effects of the relieving platform separately on the sheet pile wall and on the whole structure will be estimated.

1.2 Layout of report

The content of the present MSc thesis is divided into two major parts. The first part contains mostly hand calculations for the determination of the loads and for the analysis of the sheet pile wall, while on the second part advanced computer programs will be used for the same purpose. The outline of the analysis is illustrated in the following figure:

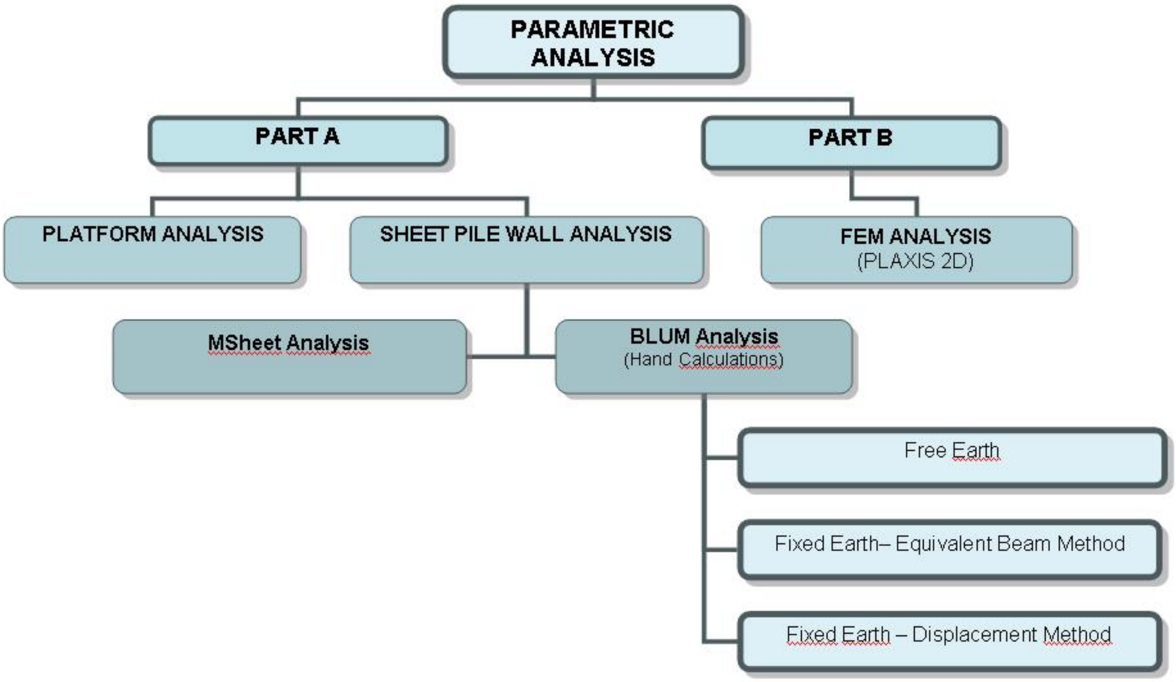


Figure 1.2: Outline of the analysis methods

In chapter 2, introductive information is given concerning the development of quay walls, the functions and the various types of these specific structures. Moreover, analytical descriptions of the acting loads is presented and further the design philosophy and the available calculation methods are described in details in chapters 3 and 4.

In Part A the structure is divided and analysed separately. For the relieving platform's analysis and the Blum's Analysis of the Sheet Pile Wall the loads were determined manually and the structures were analysed with respect to classic mechanics.

A parametric analysis of seven different quay walls is investigated, for various loading combinations of given loads. In details, three different depths and two different widths of the relieving platform were investigated. In addition a quay wall combined only from sheet piles, without a relieving platform is also analysed.

Each model of the superstructure is analysed by means of a two dimensional frame solver software program (*Frame Solver 2D*, *ESADS*) for four different load combinations taking into account differences in the water levels. On that way, the axial forces acting on the sheet pile wall are determined. Moreover, a first impression of the forces acting on the pile foundation system, lying on the land side, is obtained.

The sheet pile wall, in Part A, is analysed by three different approaching theories of Blum. The fixity of the end point of the sheet piling is considered once as completely free and in the other two as fully fixed. The Free Earth Support method is used in the first case, while in the second, two different methodologies, both considering a fully fixity at the toe level are used (Fixed Earth Support method).
As it is shown in chapter 7 the results between these two methodologies don't vary. Graphs concerning the effect of the various dimensions of the relieving platform, the embedded depth and the anchor force are produced.

In the Elastic Supported Beam theory (Msheet) analysis the ground is simulated according to a more advanced model. According to that theory, the Sheet Pile Wall is schematised as a beam on an elastic foundation, where the ground is simulated by a set of uncoupled elasto-plastic springs. On that stage the axial forces, estimated from the static analysis of the relieving platform are imposed on the top of the sheet pile wall as external loads.

In Part B, the same process is repeated but this time with a Finite Elements Method. PLAXIS created from Plaxis will be used for that purpose. This type of analysis is based on a model in which the behaviour of soil and structure are integrated.

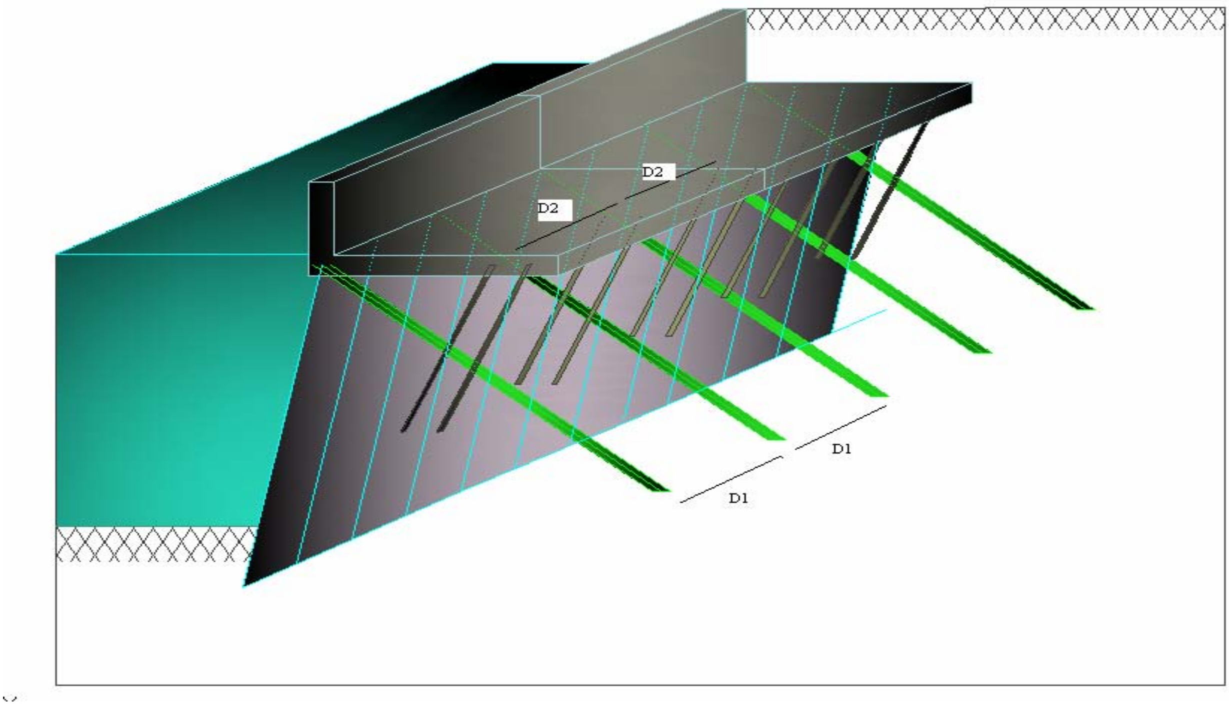


Figure 1.3: Three Dimensional (3D) representation of quay wall

1.3 References

[1.1] Ir. J.G.de Gijt; *"Developments in the Port of Rotterdam in Relation to the history of quay wall construction in the world"*, Delft University of Technology/Gemeentewerken Rotterdam, 2008

[1.2] CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*, 2005

CHAPTER 2

INTRODUCTION

2.1 General - Development of quay walls

The continuously increasing dimensions of the ships play a significant role in the design of ports and lengths of quay walls. Over the history the loading capacity, thus the dimensions of the ship have dramatically increased. This fact affected the needed retaining height in front of these structures, the length of the quays and the width if the harbour basin.

In the following figure this increment of the retaining height for the port of Rotterdam is illustrated. It can be noted that the bottom depth in the area of Maasvlakte 1 reaches the -23.00 m to -25.00 m, resulting to a total retaining height of 30.00 m.

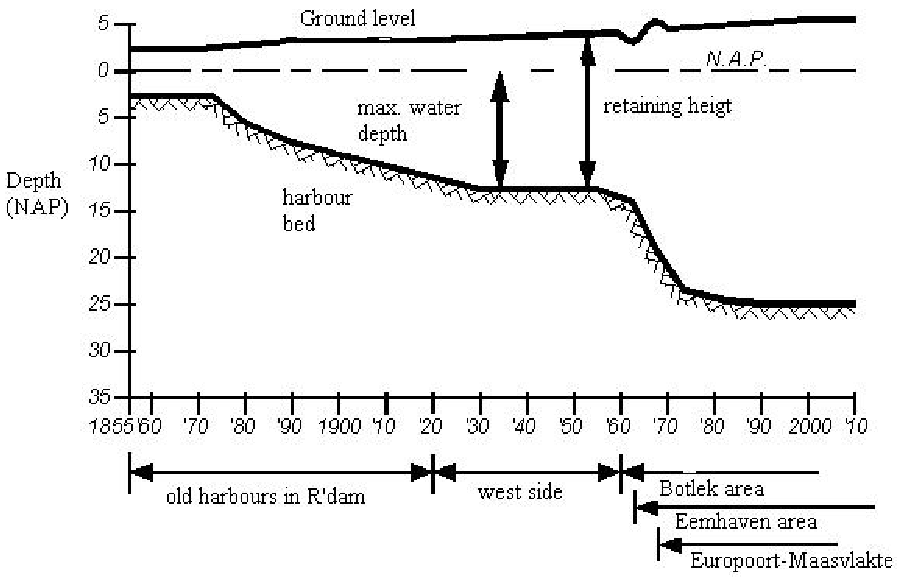


Figure 2.1: Development of the water depth (in front of quay walls) in Port of Rotterdam

The design of terminals is specified according to the type of the handling cargo. With respect to the form in which cargo is transported the following division is made:

- Dry Bulk;
- Liquid Bulk;
- Roll on/Roll off;
- Containers;
- Others;

The last category is almost identical with conventional general cargo, which includes breakbulk cargo, mass-breakbulk cargo or neobulk and bagged commodities.

Over the past 30 years container shipping has spread across the globe, taking over a major share of the general cargo trade. The first generation of container ships were general cargo vessels converted to carry containers. Since then several classes of container ships have been built with increasing dimensions and capacities.

For a long time the size of the container ships was influenced by the width of Panama Canal. The further development was restricted by this parameter due to the great importance of this canal to the seaways between the east and the west coasts of the United States. The transport of cargos has become increasingly important since in the end of nineties the demand for transport to Europe from the east American coast has increased. This fact permitted the further development of the container ships, which are characterized as Post Panamax ships. The scenarios predict that in the near future the load capacity of container ships will reach the 15000-18000 TEU.

The type of terminal that will be investigated in the present study is a future container terminal. In the following table the development of the loading capacity and the dimensions of the container vessels are illustrated.

Table 2.1: Development of average size of container ships

Class	**Period** [years]	**DWT** [average]	**TEU**	**Length** [m]	**Beam** [m]	**Draught** [m]
1st generation	End '60	14000	300-1100	200	27	9
2nd generation	'70	30000	800-1700	240	30	10.50
3rd generation	Begin '80	45000	1700-3000	300	32	11.50
4th generation	Mid '80	57000	4000-4500	310	32.30	12.50
Post Panamax	After '90	67000	4300-8000	340	39.4-45	13.50
6th generation	End '90	104000	8000	347	42.80	14.52
7th generation	After 2003	123000	12500-18000	400	63.80	14.70-21

2.2 Functions of quay walls

Quay walls are earth-retaining structures that are used for the mooring of ships. They should be designed and constructed to resist safely the vertical loads caused by useful loads, trucks, cranes etc., as well as the horizontal loads from ship impacts, wind, soil pressure, etc. The aforementioned loads vary according to the type and the magnitude of the terminal.

One can say that quay walls are subjected to heavy loads, thus the design and construction of such structures is quite demanding and complicated. Moreover, quay wall's design and construction become more complex when the users and managers specify their demands.

In order to ensure that the handling of the freight is executed as quickly as possible, the designers took into account except of the present situation the possible future developments. These are anticipated developments in transhipment and freight storage, navigation and dimensions of vessels and demands arising from the local conditions and the future user of the terminal.

The requirements and functions that a quay wall must satisfy, vary from different points of view:

- For ships, the most crucial requirement is the retaining height. There must be sufficient draught so that the big ships can berth;
- For the handling of the freight, it is also essential that the quay wall provides sufficient area and bearing capacity for present and future transhipment, storage and transport;
- Moreover, the quay wall should be designed and constructed on such a way that the total cost (including construction and maintenance costs) and the quality have been optimised;

Figure 2.2: Big Container terminal - Maasvlakte 2 extension

In addition to providing berthing facilities, the quay wall must retain the soil for the area behind the quay, provide bearing capacity to carry loads coming from the freight and the cranes, function as a water retaining structure for the areas lying behind it.

2.3 Main types of quay walls

To fulfil the aforementioned functions of quay walls, many different construction methods have arisen in various countries.

The berth fronts of quay walls are mainly constructed according to one of the following two principles:

- *Solid Berth Structures*: the fill is extended right out to the berth front where the vertical front wall is constructed;
- *Open Berth Structures*: From the top of a dredged or filled slope and out to the berth front a load bearing slab is constructed on piles;

2.3.1 Solid Berth Structures

Generally solid structures are more resistant to impacts than open structures, and they can be divided into two mail types, depending on the principle on which the front wall is constructed in order to obtain sufficient stability:

- *Gravity quay walls*: the front wall of the structure with its own dead weight and bottom friction is able to confront the horizontal and vertical loads. Sometimes the dead weight is strengthened by the soil lying above it. Typical examples of this type of structures are the block wall, L-wall, cellular wall etc;
- *Sheet Pile quay wall*: the front wall is adequate to resist any horizontal loads acting on the structure and must therefore be anchored behind the quay. Examples include anchored sheet piles, combined walls, diaphragm walls and cofferdams.

2.3.1.1 Gravity quay walls

As it is already mentioned, the soil retaining function of the gravity quay walls derives from the dead weight of the structure, which is sufficiently heavy to resist in shearing, tilt or sliding. These types of structures are suitable for areas where the bearing capacity is large enough, and where the subsoil is not suitable for driving piles. (Rocky subsoil or very firm sand).

These structures often consist of prefabricated elements, solution that can be attractive from an economical point of view especially for long quays. The upper-part of the structure is equipped with facilities for berthing, such as bollards and fenders. It is also necessary to protect the bed of the harbour, in the adjacent area of the retaining structure, against erosion caused by the propellers of ships. Drainage is also necessary in order to prevent excess pore pressures behind the quay wall.

The final choice of the type of quay wall depends on the local conditions and the total cost of the structure. For example, block quay walls are suitable for hard subsoils. A more economical attractive solution is the L-wall and the caisson quay walls. The cellular wall and the reinforced earth structure is suitable for cases with dry building pit and low retaining height respectively.

__Block wall__

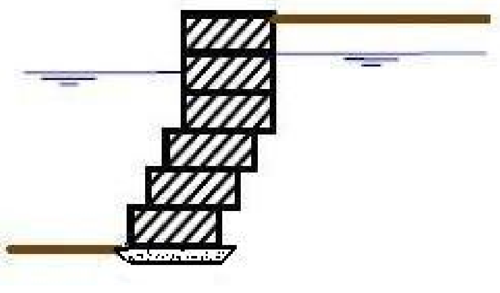


Figure 2.3: Block quay wall

This type of structure consists of blocks of concrete or natural stone, piled on the top of each other. The blocks are placed from the water side on a natural foundation consisting of a layer of gravel or stones. The great weight of the blocks and the total weight of the whole structure make it suitable only for stiff subsoils with a high bearing capacity. By using this method retaining heights up to 20 m can be achieved with a relatively low cost of labour but a big amount of building material.

The joints between the blocks provide a good drainage function of the quay wall, but in order to prevent the loss of soil it is necessary to apply a filter structure behind the wall. Naturally, these filters consist of a sufficiently thick filling of rock material.

L-wall

L-walls constitute the most representative example of structures which owe their overall stability to the weight of the soil that rests on them. The summation of the soil weight and the dead weight of the structure contribute to the shear stresses built up. On that way the desired friction is activated and the stability of the structure against the horizontal soil pressures is assured.

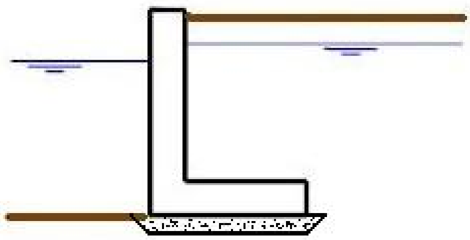


Figure 2.4: L-wall quay wall

Large prefabricated L-quay walls can be built in any subsoil, but it has to be placed on a gravel or stone bed layer, and a filter from rock filling material on the rear side should always be considered.

Caisson wall

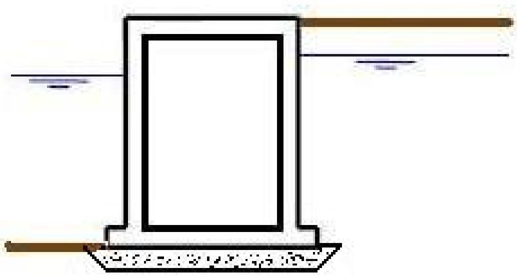


Figure 2.5: Caisson quay wall

Caisson quay walls are big hollow cellular concrete structure as it is shown is the figure. They are built in a dry construction dock and after they are floated in the specific location where they are then sunk. The total weight of this structure, as of the L-wall, consists of the summation of the dead weight and the weight of the filling material. This can be soil, crushed stones or gravel.

The alternative of a caisson quay wall is economical in material use, but it is labour intensive. To construct the caissons and to transport them in the project location requires a construction dock, a pontoon and a waterway over which the transportation will take place. These parameters make this solution difficult and rarely used.

Cellular wall

Cellular walls are constructed by driving straight instead of corrugated web profiles to form cylindrical or partially cylindrical cells that are linked to each other. Relatively little material and labour is demanded. However due to the small thickness of the sheet profiles this type of quay walls is vulnerable to damage when collision occurs and to corrosion especially in aggressive environments.

Reinforced Earth wall

In this case tensile elements like steel strips, rods and polymer reinforcements such as geogrid and geotextiles are inserted into the ground. The primary mechanism of stress transfer between the reinforcement and soil is the friction between the contact surfaces of tension elements and soil. When geogrids are used extra resistance is created by soil particles enclosed in the openings in the grid. With steel strips the resistance can be increased by adding rolled transverse ribs.

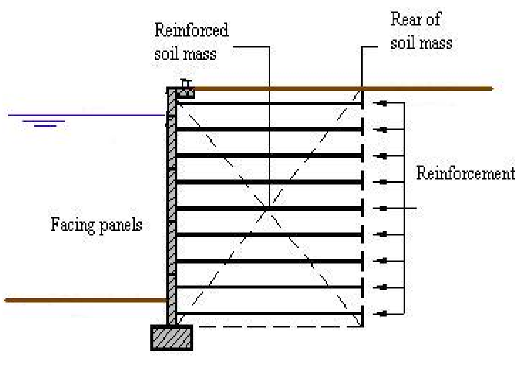


Figure 2.6: Reinforced earth quay wall

2.3.1.2 Sheet pile walls

These wall structures are used in easily penetrable soils with low bearing capacity. The sheet pile wall consists of vertical elements (mostly corrugated profiles are used) that are driven up into the subsoil. Moreover, this type of wall can be anchored, and the choice between the various anchoring systems depends on the loading and environmental conditions. Drainage system is necessary in order to avoid the excess pore pressure behind the structure.

<u>*Anchored sheet pile walls*</u>

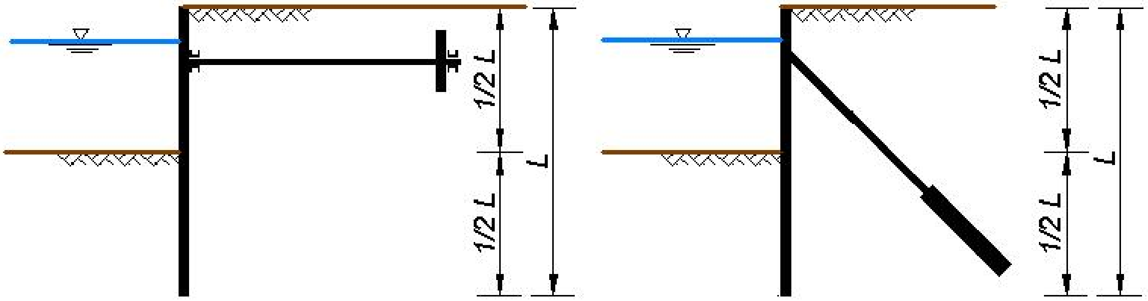
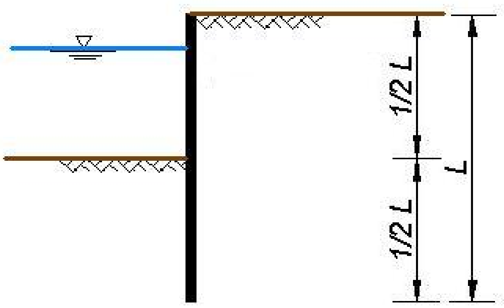


Figure 2.7: Sheet piling with anchored wall and with grout anchor

In case where high retaining heights are demanded, an anchored sheet pile system is always used. In principle, the static system of that wall can be simulated as a beam simple supported on two supports. The support that lies on the underside part of the wall can be totally free or entirely or partly fixed, depending on the used embedded length. Different approaches have been developed for the fixity of the toe. Analytical description is found on chapter 7: "Sheet Pile Analysis – Blum approaching methods".

The anchorage functions as a supporting point for the sheet pile wall. Usually it consists of a tie rod with an anchorage at the end part. Other anchorage options are also available, as the horizontal anchoring (bar, cable and screw anchor), anchors with a grout body (grout and screw injection anchors) and tension piles (closed or soil displacement pile, H-piles, open tubular steel and MV piles).

<u>*Freestanding sheet pile walls*</u>



In the case that the sheet pile is not anchored, the wall is acting as a cantilever beam that is elastically fixed in the toe. A higher embedded depth is needed in that case. On the base the supporting pressure that is necessary for the equilibrium is mobilised by the passive earth pressure. This type of quay walls is used only for small retaining heights. For higher situations, some kind of anchoring systems is always necessary.

Figure 2.8: Freestanding sheet piling

Sheet pile systems

The main types are single sheet piling, diaphragm walls and fixed cofferdams. For quay walls with high retaining heights, suspending into heavy loads at the same time, heavy structures that may consist of combined walls are mostly used. A combined wall consists of heavy primary elements that are deeply embedded into the subsoil, and secondary elements that are welded to each other between the primary elements. These secondary elements are normally shorter compared to the primary ones, since the soil pressure is transferred to the primary elements by arch action.

A diaphragm wall is a reinforced concrete wall that is constructed in situ. The thickness of the wall varies between 0.50 m to 2.00 m and the width of the panels from 2.50 m to 7.20 m. The concrete wall has high bearing capacity and is relatively stiff, so that the deformations are minimal.

A cofferdam wall consists of two sheet pile walls with a soil filling space between the two walls, which transfer the horizontal and vertical loads to the subsoil. The front and the rear wall are often connected by one or more anchors. The retaining function of the cofferdam derives from the shear resistance and the total weight of the soil between the walls. The special with this case is that the walls are close enough that the active zone of the front wall and the passive zone of the rear wall overlap. For that reason, the rear wall should not be considered as a normal anchoring wall.

Sheet piles with a relieving platform

With this special structure the designers managed to relieve the sheet pile wall from the soil pressures reaching very high retaining heights. The presence of the superstructure considerably reduces the horizontal loads on the front wall, thus the bending moments on the wall. The structure consists of a bearing and earth retaining sheet piling on the water side and a foundation system of tension and bearing piles on the earth side.

The platform creates a horizontal link between the sheet pile wall and the foundation piles. Also an anchorage is used, usually at the connecting point of the sheet piling and the relieving platform.

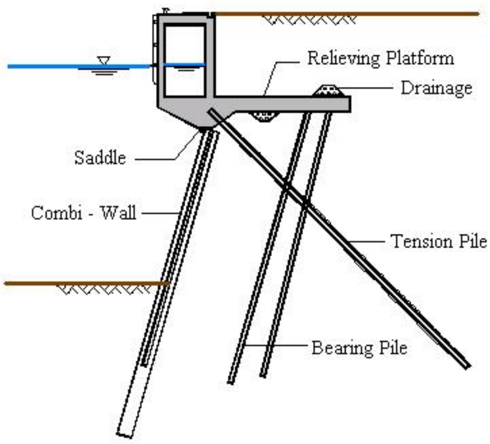


Figure 2.9: Structure with relieving platform

When the retaining height of the quay front is high, a simple anchored sheet pile wall structure will normally not be the most economical solution. The relatively high cost of soil improvements prompted the designers to research for alternative options.

Relieving platforms constitute probably an interesting solution from an economic point of view in cases were the loads and the retaining heights are relatively great. Moreover, in cases where the retaining heights are small but the terrain loads are high, this type of structure can be also economically justified. These types of structures have also been used in cases where the utilisation of sheet piles with small moments of resistance has been pursued and the allowable deformations are very limited.

This type of quay wall has been broadly designed in the port of Rotterdam and terminals with retaining heights up to 20-25 m are suitable to accommodate the biggest vessels of nowadays. This type of structure constitutes the subject of this study; the role and function of the relieving platform are investigated in the following chapters.

2.3.2 Open Berth Structures

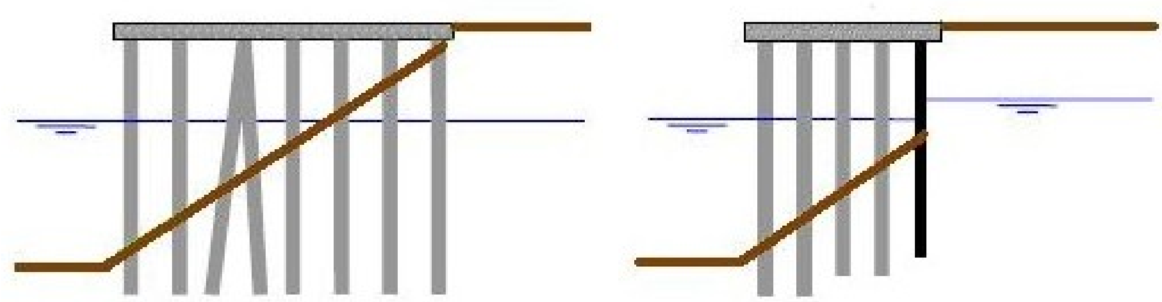


Figure 2.10: Open berth quay over a slope and with a retaining wall

In this case the height difference is bridged by a slope instead of a vertical wall that it is used in the previous cases. The structure consists of a horizontal deck that is sometimes anchored. The deck is founded on vertical and inclined piles, while underneath the deck a slope revetment is lying in order to withstand the wave attack and the currents caused by the propellers and the bow thrusters of the ships. These jetty-like structures are mainly used in cases where the construction is necessary to take place above the water and in subsoils with a low bearing capacity. These type of structure demands sufficient space on the sea side and they are vulnerable in damage when collision occurs.

2.4 References

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CHAPTER 3

LOADS ACTING ON QUAY WALLS

3.1 General

In order to make a safe design an accurate prediction of the loads is needed. When these structures are verified for safety the effects of unfavourable loads and combination of them are considered. Information for the design values of loads were collected from various sources mainly from Handbook of Quay Walls.

The verification of safety in the various limit states is based on a categorization of the loads. This is executed for the determination of the correct combination of loads, and the used partial factors as well. In Handbook of Quay Walls a distinction is made between the following categories of loads:

- Permanent loads;
- Variable loads;
- Accidental loads;

Permanent loads
The variation of these loads is remarkable slight in magnitude during the reference period of the structure. These loads are:

- The dead weight of the structure and the additional self weight of the equipments such as: covering layers, fendering systems, bollards, and the weight of the soil acting on the relieving platform;
- Water and earth pressures caused by the weight of the soil under different water level conditions;
- Friction forces between the soil and soil on relieving platform;
- Friction forces between soil and relieving platform;

Variable loads
This type of loads vary during the reference period of they are not always present. These loads are:

- Loads caused by earth pressures as a result of site loads (terrain loads);
- Loads caused by water pressures;
- Hydrodynamic water pressures cause by the groundwater flows and waves;
- Ship's operation;
- Bollard forces;
- Fender forces;
- Terrain loads caused by the storage of the freight;
- Crane loads;
- Traffic loads;
- Environmental loads (wave loads, ice loads, loads cause by temperature variations, seismic loads);

Based on differences in the various transfer mechanism, a distinction should be made between direct variable loads on the sheet piles and loads that act directly or indirectly on the relieving platform and are transferred to the subsoil via the foundation system. The former loads are horizontal earth pressures that result from the variable site loads and taking into account the relieving effect of the superstructure, act on the sheet pile wall.

Accidental loads
Loads with a very small probability of occurrence are termed as accidental loads. For quay wall structures these are:

- Loads caused by extreme water levels and variations of the water level (flooding of the area combined with a non-functional drainage system);
- Extreme site load by bulk goods (coal and ores in an emergency situation);
- Impacts from falling freight;
- Collision loads;
- Seismic loads;
- Loads caused from an incidental extreme deepening of the harbour bottom;

It has to be noted that in very frequent use of the quay wall, crane loads and traffic loads, and especially the vertical component of these loads can lead to fatigue of the structural members. This problem affects only the directly loaded parts of the structures such as the crane tracks and the quay decks.

Representative and design values
Calculations for the dimensioning and verification of structures are based on the design values of the design parameters. These values are obtained by multiplying or dividing the representative values with the correct partial factors. Most of the times, it is assumed that the representative value is equal to the characteristic value. Depending on the variable, the characteristic value has a probability of 5% that the variable is smaller or exceeds that value. For a more scientific approach the determination of the characteristics values should be executed by a statistical analysis. This is possible only when many data are available, otherwise it is allowed by the NEN codes to use the nominal value of the parameters. All the loads are assumed that act in a representative cross section, where distributed loads are expressed per meter.

3.2 Specific Loads on a Quay Wall structure

3.2.1 Vertical loads

Earth pressures
In addition to horizontal earth pressures, possible vertical friction forces will develop. With entirely neutral earth pressures no friction occurs. In the case of passive and active pressures are activated, the maximum vertical friction force is related to the resultant of the maximum active earth pressure E_a and it is directed downwards. For the situation of “earth on earth”, the maximum friction is a set at $E_a \cdot \tan\varphi$. The friction between “earth and structure” is: $E_a \cdot \tan\delta$. Finally, it has to be noted that the friction is considered as a permanent load.

Terrain loads
Terrain loads are working on the quay due to storage. General design values are recommended by Handbook of Quay Walls and EAU 2004. The difference between these codes is that in Handbook of Quay Walls the container is assumed to be 17% unloaded. That leads to a smaller load compared to EAU which takes into account fully loaded containers. In order to make a safe approach to the problem the recommendations of EAU will be used in that case.

Table 3.1: Average container loads according to EAU 2004

EAU 2004 – average container loads		
Weight 20ft container	200	[kN]
Weight 40ft container	300	[kN]
Light traffic (cars)	5	[kN/m^2]
General traffic (HGV's)	10	[kN/m^2]
General Cargo	20	[kN/m^2]
Container empty, stacked 4 high	15	[kN/m^2]
Container full, stacked 2 high	35	[kN/m^2]
Container full, stacked 4 high	55	[kN/m^2]

Crane loads

It is likely to occur that the future container cranes will have the ability to lift more than one container each time. The imposed loads are very high; therefore the foundation system of such elements is of a major importance. Recommendations regarding the design values of the crane loads can be found in Handbook of Quay Walls. These loads include the weight of the crane and the hoisting load, wind load and dynamic loads from crane movements and tilting while moving.

Figure 3.1: First of the 16 cranes arriving on the Euromax terminal

Table 3.2: Crane loads

Rail distance [m]	Bearing capacity waterside [kN]	Bearing capacity landside [kN]	Dead weight [kN]	Max wheel load waterside [kN]	Max wheel load landside [kN]	Wheel distance [m]
15.24	410	410	5150	293	174	1.75
15.24	500	500	8100	474	433	1.20
20.00	500	500	9770	568	542	1.00
30.48	500	500	8970	408	609	1.24
35.00	670	670	12122	691	691	1.05
48.00	450	450	7350	420	383	1.50

Traffic load

The traffic load can be due to the landward transportation of cargos through vehicles and other transport systems. In the present case, it is assumed that the traffic load is included in the design value of terrain load.

3.2.2 Horizontal loads

Earth pressures

Earth pressures are generated by the self-weight of the ground and by surcharges. Different approaches are taken into account for the calculation of the earth pressures on different structural members of quay wall. In the followings a distinction is made for the earth pressures on the sheet pile wall and on the superstructure.

- *Earth pressures acting on the sheet pile wall*

The determination of the earth pressures depends on the deformation and the stiffness of the structure. In principle two calculation methods based on different approaches to the determination of the earth pressures are available. A further description of these methods is given in “Chapter 4: Design philosophy and calculation methods”. On that point a brief reference of them will be done.

According to Blum theory, the calculation starts from the failure situation of the ground where minimal active and maximum passive earth pressures occur. This method is very useful for initial calculations when the designers want to obtain a first impression of the embedded depth and the dimensions of the sheet pile wall.

According to the calculation model with an elastic-supported beam, the ground is represented by a set of elasto-plastic springs. Because of the practical applicability of this approach it constitutes the most frequently used method.

For detailed investigations, especially for the prediction of deformations a calculation method that is based on an advanced Finite Element Method (FEM) is usually used. This method is based on a model in which the behaviour of the soil and structure are integrated.

- *Earth pressures acting on the relieving platform*

After completion of the relieving platform, the excavation area is filled and well compacted. This causes horizontal stresses and depending on the stiffness of the superstructure, increases the earth pressures. During the operation of a quay wall, a situation develops in which the superstructure is loaded and stressed against the fill material. In extreme loading situation, in addition to small deformations, active earth pressures will arise. For safety reasons, it is recommended by the codes that for calculation of fundamental loading combinations, neutral earth pressures should be used. For extreme combinations of loads, active earth pressure should be used instead. Consequently, in the Serviceability Limit State (SLS), neutral earth pressure should be used.

Waves

The quay walls in Maasvlakte 2 are mostly protected against waves coming from the sea. The only waves that can reach the quay are developed by wind in the harbour basin itself and stern waves from passing vessels. The wind waves are very low and short since the fetches in the inner side of the harbour are small.

The waves caused from the passing ships will also be low since the speed of the ships which approach the quay in order to berth is already very limited. Moreover such forces are assumed to be taken care of by the fact that the structure is also designed for impact and mooring forces. For breakwaters and similar structures heavily exposed to waves the wave actions must of course be studied very closely in each case.

For these reasons and for simplicity reasons as well, waves will not be taken into account and are neglected, consequently the water pressure will be assumed totally hydrostatic.

Currents

Currents due to passing ships in that case will be neglected. The quay wall is able to resist since the speed of the vessels is low thus the caused currents have a small magnitude. In a detailed calculation it would probably be necessary to apply a proper bottom protection in order to withstand the currents from the ship's propellers.

Ice forces

The study of forces due to formation of ice in the harbour basin has not been given high priority until now. However, maritime structures like dolphins, bridge pillars etc. are surrounded by an ice slab during the winter season or they are exposed to drift ice, thus one must take into consideration that both horizontal and vertical ice forces can be of importance.

The magnitude of the ice forces depends on type of the structure; the properties of the ice and to some extend on the conditions under which the ice was formed. Research has shown that ice formed in fresh river water has higher strength and modulus of elasticity than ice formed in salt sea water.

The structure which will be analysed in the followings is subjected to heaviest horizontal loads. Moreover, the vertical force can cause problems to light structures; for instance light piers on timber piles. For the aforementioned reasons and for simplicity reasons ice forces will not be considered in the present investigation.

Crane loads

The horizontal loads coming from the horizontal movements of crane usually are caused by swaying of the lifted containers. The design value of these loads is calculated as a percentage of the vertical crane load, these being 10% to 15%. These loads include the weight of the crane and the hoisting load, wind load and dynamic loads from crane movements and tilting while moving.

Bollard loads

A ship coming alongside is usually stopped partly by its own engines and partly by the use of spring hawsers. On that way, mooring forces are transmitted to bollards and dolphins which are situated on the quay wall. The force on the bollards depends on the water displacements due to berthing, wind and currents. These ship motions lead to horizontally concentrated tensile forces in the mooring lines. The water displacement can be calculated with the following formula:

$$G = L \cdot B \cdot D \cdot C_B \cdot \rho_w$$

in which:

G: water displacement [tons];
L: length of the design ship [m];
B: width of the design ship [m];
D: draught of the design ship [m];
C_B: Block coefficient of the design ship;
p_w: density of the water [ton/m^3];

Table 3.3: Bollard forces according to water displacements

Displacements [tons]	Bollard force [kN]	Percentage [‰]
<2000	100	5.00
<10000	300	3.00
<20000	600	3.00
<50000	800	1.60
<100000	1000	1.00
<200000	1500	0.75
>200000	2000	1.00

Fender loads

The estimation of fender loads is based on the theory of kinetic energy absorption. The fender system which is used depends on the adopted amount of energy by the fender. The energy that has to be absorbed is described by the following formula:

$$E_d = \frac{1}{2} \cdot G \cdot v_S^2 \cdot C_E \cdot C_M \cdot C_S \cdot C_C$$

in which:

G: mass of the ship;
V_s: berthing velocity;
C_M: virtual mass coefficient;
C_E: eccentricity coefficient;
C_S: softness coefficient;
C_C: configuration coefficient;

Seismic loads

Seismic or earthquake loads act at the centre of gravity of the structure as a horizontal force equal to the design coefficient times the weight of the structure. In the calculated weight of the structure, it should be added one half of the live load. The actual seismic load due to earthquake will depend on the magnitude of the disturbance, the type of structure and the soil conditions in the adjacent area.

In NEN codes the seismic activity of the Netherlands and the neighbouring countries is specified. More specifically, NEN 6702 shows the zones that are susceptible to seismic activity, and that may be taken as a guideline by the designers. Two aspects must be taken into account concerning the effects of an earthquake, these are:

- The occurrence of acceleration forces on the structure and soil masses of the active and passive sliding wedge;
- The reduction of shear resistances in non cohesive soils with loose packing, as a result of liquefaction, may possibly occur;

During earthquakes the ground experiences strong cyclic accelerations. During earthquakes, for instance, saturated sand is sometimes densified in a short time, which causes large pore pressures to develop, so that the sand particles may start to float in the water. This phenomenon is called liquefaction.

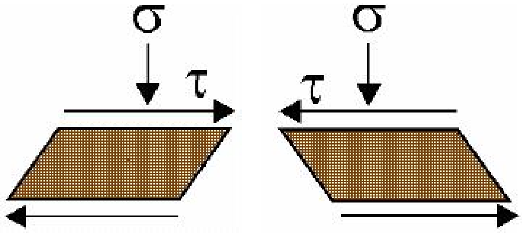


Figure 3.2: Shear stresses in soil particles

Structures can survive an earthquake due to the fact that most of the times they are supported on piles penetrated into stiff soil. Otherwise soil improving measurements are considered as compaction, gravel columns, drains, cementation etc.

Generally, unless the quay wall is a massive or gravity type structure, the seismic effect on the design is relatively small. For the aforementioned reason, and moreover taking into consideration the seismographic behaviour of the area, it can be assumed that the seismic loads can be neglected.

Loads caused by variations in temperature

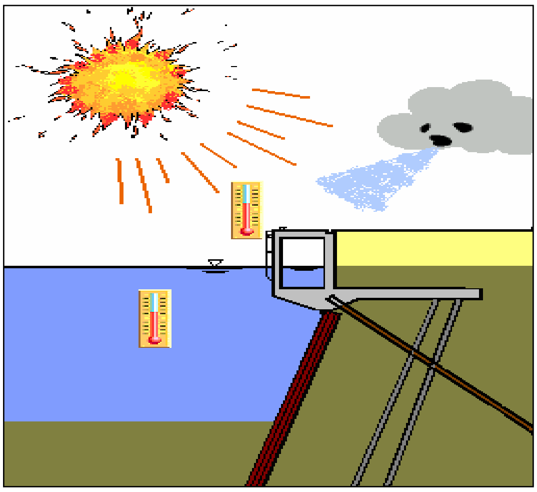


Figure 3.3: Thermal effect in quay walls

It is necessary to take into account the loads resulting from variations in temperature. The variations in temperature between the members of the structure are caused by general climate and seasonal influences. These effects are only taken into account in Serviceability Limit State (SLS).

This phenomenon is very important for the verification of crack formation. For other parts of the quay wall structure this loads can be neglected. In the present study, for simplicity reasons, this loading parameter will be neglected even for the relieving platform.

3.3 References

[3.1] CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*

[3.2] Carl A. Thoresen; *"Port Design – Guidelines and Recommendations"*, Tapir Publishers

[3.3] Gregory P.Tsinker; *"Port Engineering, Planning, Construction, Maintenance and Security"*

[3.4] Prof. Arnold Verruijt; *"Soil Mechanics"*, Delft University of Technology

CHAPTER 4

DESIGN PHILOSOPHY
AND CALCULATION METHODS

4.1 Design philosophy

4.1.1 General

For the design of a quay wall structure a deterministic approach to safety is still sometimes used for simplicity reasons. This method determines a margin between the characteristic values of loads and strengths, which has to be respected in order to assure the safety of the structure.

The probabilistic approach is based on the principle that the structure should satisfy a specific probability of failure. For this analysis all the parameters of the structure are considered as stochastic. Nowadays, in order to maintain the simplicity of a design method and to avoid at the same time complicated specialised probabilistic calculations; Dutch norms are based on semi-probabilistic methods.

The design method described in Handbook of Quay Walls will be followed as design code for this study. This method has been broadly used in designing quay walls for the port of Rotterdam, and one can say that is well applicable for special structures like quay walls.

According to the Dutch standards three limit states are specified. The two of them are unltimate limit states (ULS1A, ULS1B) and one serviceability limit state (SLS). ULS1A treats the strength and the stability of the structure and happens when a failure mechanism occurs due to:

- Failure of a sheet pile;
- Loss of overall stability;
- Insufficient bearing capacity of the foundation;
- Insufficient passive soil resistance;
- Failure of the piling system or the anchorage;
- Internal erosion of the soil/scouring/piping;

ULS1B occurs when deformations of the quay wall lead to severe structural damage to parts of the structure or to nearby structures or installations. SLS concerns deformations under serviceability loads and occurs when:

- Deformations affect the appearance or the efficient use of the structure or structures or installations located in the adjacent area;
- Deformations exceed values which are acceptable for serviceability limit state or which do not meet specific deformation requirements;

4.1.2 Determination of representative values, design values and normative load combinations

- *Design approach to the sheet pile wall*

According to Handbook of Quay Walls, for the design of the sheet pile wall, a set of partial factors that are specifically developed for calculation of the sheet piles in a quay wall is used.

Table 4.1: Partial safety factors for unfavourable representative soil parameters

Parameter	Partial Safety Factor
Volumetric weight of soil "γ"	1.0
Angle of internal friction "φ"	1.0
Cohesion "c"	1.0
Friction angle between soil and wall "δ"	1.0
Soil coefficient "K_h"	1.0
Young's modulus "E"	1.0
Poison's ratio "v"	1.0

In contrast with the NEN codes, in Handbook of Quay Walls the value 1.00 is taken for all the partial factors for soil properties, which means that the calculations are executed for representative soil properties. Depending on the magnitude of the deformations they can be passive, neutral of active stresses. The earth pressures on the sheet pile caused by surcharges are determined from the design values of the surcharges.

Different safety factors are applied for the normal forces, the transverse forces and the bending moments and for the loads as it is shown in the following table:

Table 4.2: Partial safety factors for geometrical parameters

Parameter	Partial Safety Factor
Bottom level	1.20
Groundwater level	2.00
Free water level	0.60

Table 4.3: Partial safety factors for the results of the sheet pile calculation

Parameter	Partial Safety Factor
Normal forces	1.3
Transverse forces	1.3
Bending moments	1.3
Anchor force resulting from sheet pile calculation	1.2
Mobilised earth pressures	1.3

The safety factors that are listed above are applicable for all limit states. In case of serviceability limit state al the values should be replaced with 1.0. This is not the case for the partial safety factors of the soil parameters which stay the same in each limit state.

The advantage of this method is that the distribution of the internal forces corresponds in a more accurate way the physical behaviour of the structure. The method in Handbook of Quay Walls is based on safety class 2. According to that, a failure of a quay wall leads to large economical damage but small personal risk.

- *Design approach to the relieving platform and the foundation*

For the relieving platform and the foundation of the quay wall, the general line of approach given in the series of NEN-standards is followed. In contrast with the NEN codes, in Handbook of Quay Walls the value 1.00 is taken for all the partial factors for soil properties for the fill behind the superstructure. Depending on the magnitude of the deformations they can be passive, neutral of active stresses.
The design values of the loads on the relieving platform and foundation in the ultimate limit state are determined by using the partial factors shown in the following table:

Table 4.4: Partial load factors for the ultimate limit state

Safety Category	Combination of loads	Permanent loads $\gamma_{f;g}$ Unfavourable $\gamma_{f;gmax}$	Permanent loads $\gamma_{f;g}$ Favourable $\gamma_{f;gmin}$	Variable loads $\gamma_{f;q}$	Accidental loads $\gamma_{f;a}$
1	Fundamental 1	1.2 (1.15)	0.9 (0.95)	1.2	-
2	Fundamental 1	1.2 (1.15)	0.9 (0.95)	1.3	-
3	Fundamental 1	1.2 (1.15)	0.9 (0.95)	1.5	-
1-2-3	Fundamental 2	1.35 (1.3)	0.9 (0.95)	-	-
1-2-3	Accidental 3	1.0	1.0	1.0	1.0

Often it is necessary to verify the serviceability limit state. In unusual circumstances the deformation requirements may be so strict that verification is necessary. Verification calculations are based on a combination of loads with partial safety factors as shown in table 4.2.

Table 4.5: Partial safety factors for the serviceability limit state

Safety Category	Combination of loads	Permanent loads $\gamma_{f;g}$ Unfavourable $\gamma_{f;gmax}$	Permanent loads $\gamma_{f;g}$ Favourable $\gamma_{f;gmin}$	Variable loads $\gamma_{f;q}$	Accidental loads $\gamma_{f;a}$
1-2-3	Infrequent 4	1.0	1.0	1.0	-
1-2-3	Frequent 5	1.0	1.0	1.0	-

- *Load Combinations*

In the limit states a number of unfavourable combinations of loads that are composed from permanent and several variable loads are considered. For that reason combinations of loads that are normative for the various constructive members have to be investigated.

In the load combinations, it has to be taken into account that the probability of a simultaneous combination of loads must be smaller than the probability that one of the loads occurs. In addition to the permanent loads, there is a possibility that a leading variable load combined with other variable loads occurs. Depending on the nature of the loads, the variable loads in a load combination are reduced by means of reduction factors ψ_i.

Two loading combinations are considered:

- Fundamental combination, fundamental situations during serviceability;
- Special load combination, in extreme or special situation;

For these two load combinations, three types of loads can be determined:

- Permanent loads;
- Variable loads;
- Special loads;

Table 4.6: Load combination philosophy

Design values of loads in load combinations in the ULS					
Combination type	**Permanent loads G_d**		**Variable loads Q_d**		**Special loads $F_{a,d}$**
	Unfavourable	**Favourable**	**Dominant Variable**	**Simultaneously occurring Var. loads**	
Fundamental	$\gamma_{f:g\ max} * G_{rep\ max}$	$\gamma_{f:g\ min} * G_{rep\ min}$	$\gamma_{f:q} * Q_{1:rep}$	$\gamma_{f:q} * Q_{0.j:rep}$	-
Special	$\gamma_{f:g\ max} * G_{rep\ max}$	$\gamma_{f:g\ min} * G_{rep\ min}$	$\gamma_{f:q} * \psi_{1.1} Q_{1:rep}$	$\gamma_{f:q} * \psi_{2.1} Q_{1:rep}$	$\gamma_{f:g\ max} * G_{rep\ max}$

Depending on the limit state, load combination and type of load, the representative values can be distinguished in three categories:

- Combination loads;
- Momentaneous loads, a variable load that will probably occur in the load combination;
- Quasy-permanent loads, a variable load present over a longer period;

Table 4.7: Load factors for load combinations

Type of load	**Combination factor Ψ_0**	**Momentaneous factor Ψ_1**	**Quasy-permanent factor Ψ_2**
Soil pressure	1.00	1.00	1.00
Water pressure	1.00	1.00	1.00
Variable favourable loads	0.70	0.60	0.50
Meteorological loads	0.70	0.30	0

*Meteorological loads are loads due to waves, currents, air and water temperature, snow, ice and earthquakes

4.2 Calculation methods

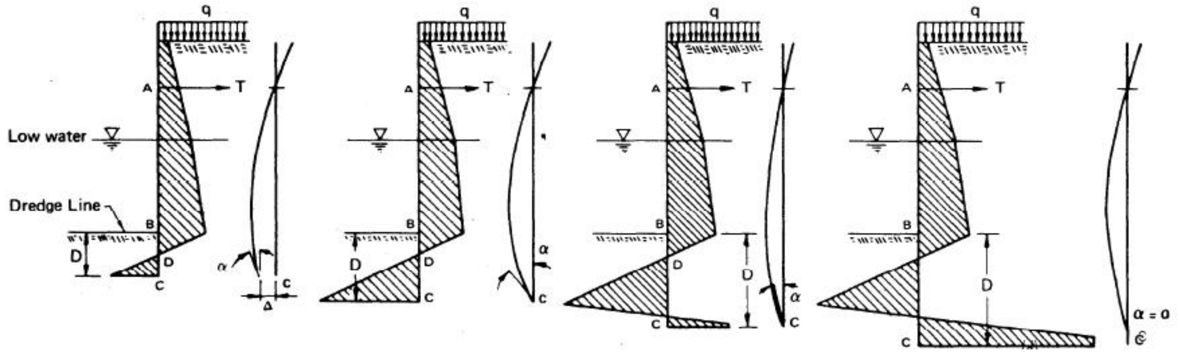
The design of sheet pile retaining walls requires several successive operations:

- Evaluation of the forces and lateral pressures that act on the wall;
- Determination of the required depth of piling penetration;
- Computation of the maximum bending moments in the piling;
- Computation of the stresses in the wall and selection of the appropriate piling section,
- Design of the waling and anchorage system;

For the designing of the sheet pile wall in the quay structure, two calculation methods are mostly used because of their applicability and simplicity. These are the standard calculation method of Blum and the method of a beam placed on elastic foundation. Recent developments in numerical modelling provided a third method which is based on the finite elements mode, where the properties of both soil and structure are introduced.

4.2.1 Blum model

Anchored walls derive their support by two means: passive pressure on the front of the embedded portion of the wall and anchor tie rods near the top of the piling. In principal for higher walls the use of high-strength steel piling, reinforced sheet piling, relieving platform or additional anchors may be necessary. The following figure shows the general relationship between embedded depth, lateral pressure distribution and elastic line or deflection shape.



(a) (b) (c) (d)

Figure 4.1: Effect of embedded depth on pressure distribution and deflected shape

The first case (a) is commonly called the free earth support method. The passive stresses in front of the wall are insufficient to prevent lateral deflection and rotations in point C. The next cases show the effect of increasing the embedded depth. In cases (b) and (c) the passive stresses pressure has increased enough to prevent lateral deflection at C; however, rotation still occurs. In case (d) passive pressures have sufficiently developed on both sides of the wall to prevent both lateral deflection and rotation at C. This case is commonly called the fixed earth support method because point C is essentially fixed.

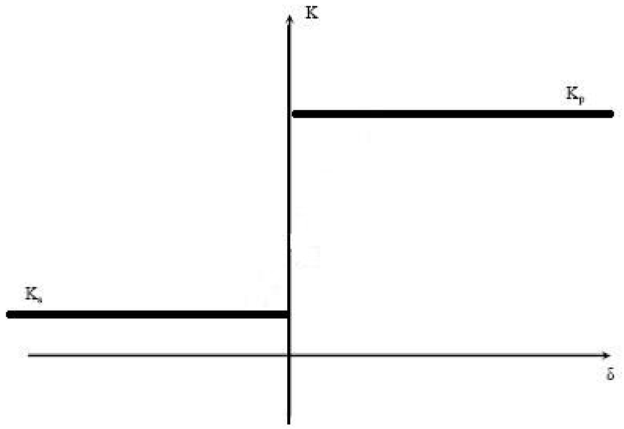


Figure 4.2: Lateral earth stresses coefficients for Blum theory

Some different methods in current use for the design of anchored sheet pile walls are grouped and discussed in the followings.

- Free Earth Support Method
- Fixed Earth Support Method (Equivalent Beam)
- Fixed Earth Support Method (Displacement Method)

Analytical description of these three different approaches is found in the relative calculation chapter (Chapter 7: “Sheet Pile Analysis – Blum approaching methods”).

4.2.2 Beam on elastic foundation

The formation of equations for the subgrade reaction method resulted in a fourth order differential equation. Solving this was for a long time a major problem that hindered application of the method for retaining wall design. The development of the computer programs facilitated numerical integration of the equations and radically changed the nature of the problem.

In this model the ground is schematised as a set of elasto-plastic springs. With important deformations of the sheet pile wall the plastic branch of the ground spring is reached and minimum active and maximum passive earth pressure develop. Because the earth pressures depend on the deformation of the sheet pile wall the calculation is an iterative process. After each calculation step a verification of whether the calculated earth pressures correspond with the displacements is made.

The iterative process comes to an end when the results have converged. The available computer program is based on uncoupled springs, thus the arching effect of the ground is not taken into account.

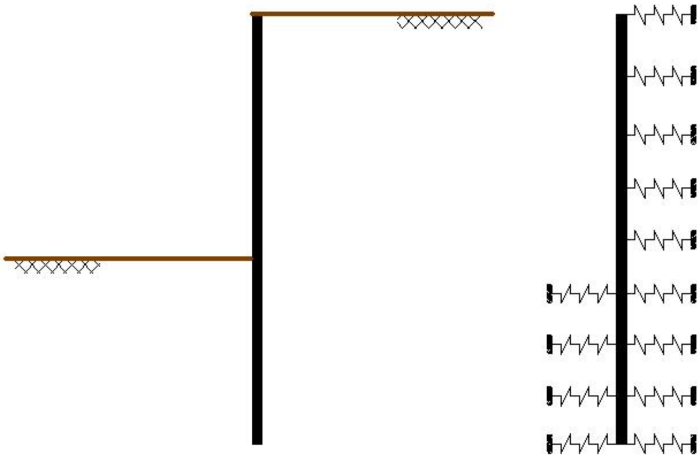


Figure 4.3: Spring model schematisation and soil coefficients

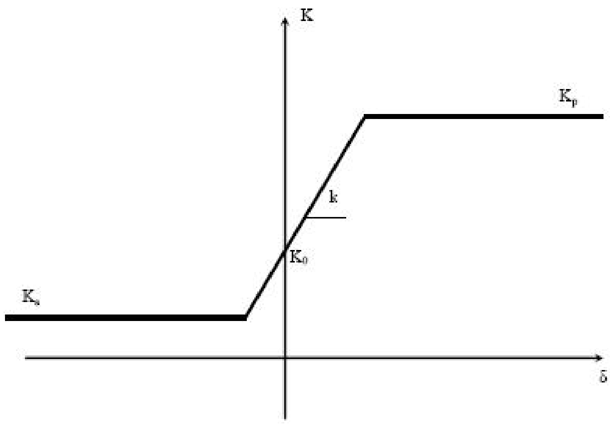


Figure 4.4: Lateral earth stresses coefficients for spring model

With this calculation method it is possible to calculate a sequence of phases in which the "stress history" of the sheet pile can be used as initial conditions to the following phase. Moreover, computer programs do not take into account inclined sheet pile walls and axial loads either. The second order effect on the redistribution of forces has to be investigated separately.

4.2.3 Finite Element analysis

Finite element method is based on a model in which the behaviour of soil and structure are intergraded. The properties of soil are introduced by means of stresses deformation relations. This method can be used for the verification of the overall stability and the deformations of the structure. Finite elements method is used to analyse successfully problems like:

- Horizontal deformations of the foundation members;
- Bending moments in piles and extra horizontal loads;
- Deformation of the relieving platform in various phases;
- Arching affect of the soil on the active part of the sheet pile wall;
- Relieving effect of the relieving platform in a weak cohesive soil;

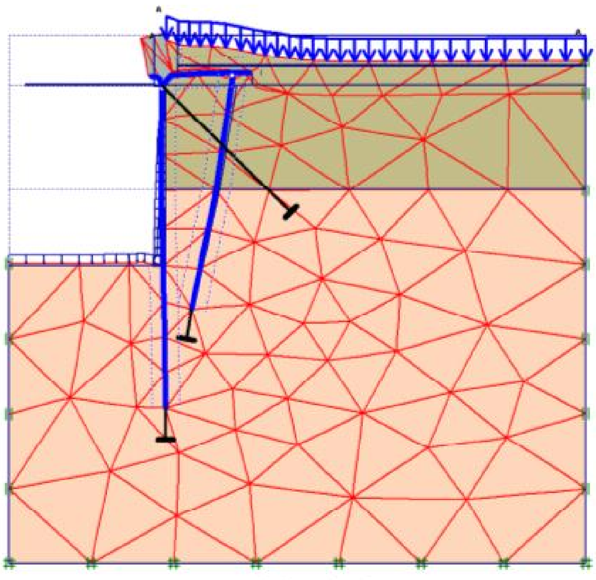


Figure 4.5: Deformed mesh from FEM - PLAXIS

The finite element method can also be used for three-dimensional problems, for example to investigate the distributions of earth pressures over the primary members and intermediate piles in combined wall.

Analytical description of the simulation method and the assumption is found in Chapter 9: "Part B – Finite Element Analysis".

4.3 References

[4.1] CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*

[4.2] EAU 2004; *"Recommendations of the Committee for Waterfront Structures Harbours and Waterways"*

[4.3] *"Steel Sheet Piling – Design Manual"*, U.S.Department of Transportation/FHWA

[4.4] Luc de Lattre; *"A century of design methods for retaining walls – the French point of view"*, Laboratoire Central de Ponts et Chaussees

[4.5] PLAXIS manual; *"PLAXIS, Finite Element Code for Soil and Rock Analysis, version 8"*

[4.6] MSheet manual; *"Manual – Reference Section"*

CHAPTER 5

PART-A
HAND CALCULATIONS-GENERAL

5.1 General

A parametric study of the structure, with respect to different dimensions of the relieving platform, is carried out in order to investigate the effects of the superstructure to the sheet pile wall. In order to check the differences between the outcomes, the retaining height is considered stable.

Two different soil profiles will be investigated; the first (soil profile 1) is representative of the Maasvlakte area and the second (soil profile 2) represents the area near the city of Rotterdam, the Netherlands. CPTs from these areas are available in the Appendices.

5.2 Boundary conditions

5.2.1 Determination of the retaining height

The construction depth of a quay wall depends on many parameters such as tolerances due to dredging and stone fill, and is based by definition on the low low water spring (LLWS). In addition, the required retaining height depends on the draught of the design vessel and the keel clearance as it is illustrated in the following figure:

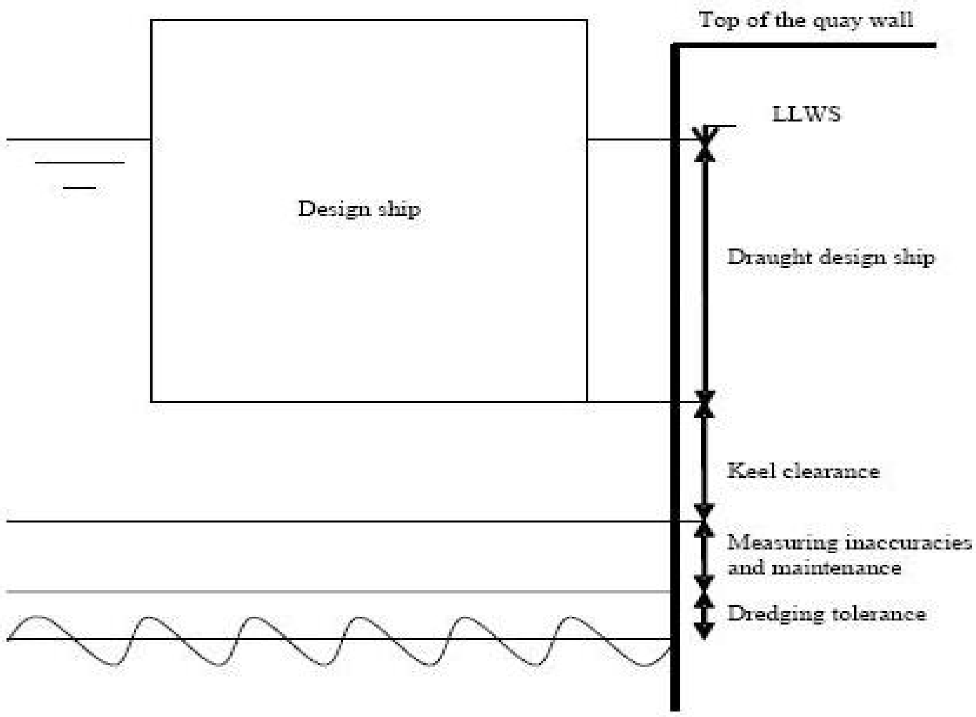


Figure 5.1: Determination of the retaining height

The required contract depth is equal to the level of LLWS – 1.1▪the maximum draught; consequently the total retaining height is equal to:

LLWS – 1.1▪the maximum draught – inaccuracies - keel clearance – tolerances - maintenance fluctuations

The type of quay wall that is investigated in that case constitutes an appropriate structure for the accommodation of 7th generation container vessels. A representative design container vessel should be able to transport 12500 TEU, and in this case the **Southampton ++** container vessel is chosen. The characteristics of the ship are summarised in the following table:

Table 5.1: Design ship characteristics

Characteristic	Value	
Length	382	[m]
Beam	57	[m]
Draught	17	[m]
Block Coefficient	0.686	[-]

The necessary retaining height of the quay wall can now be determined. The following table illustrates the different components which together form the final retaining height.

Table 5.2: Determination of total retaining height

1.1▪Draught of design ship	1.1▪17=18.7	[m]
Keel clearance (10-15% of draught)	2.55	[m]
Measuring inaccuracies	0.5	[m]
Fluctuations due to maintenance	0.5	[m]
Dredging tolerances	0.8	[m]
Ground level, NAP	+5.0	[m]
L.L.Water Spring Rotterdam, NAP	-1.77	[m]
Retaining height	≈29.8 [m]→**30 [m]**	

5.2.2 Water levels

For the elaboration of the study, hydraulic boundary conditions similar to those of Maasvlakte at the Port of Rotterdam are used. That was chosen due to the fact that the conditions in the area determine the applicability and feasibility of a quay wall to a large extend.

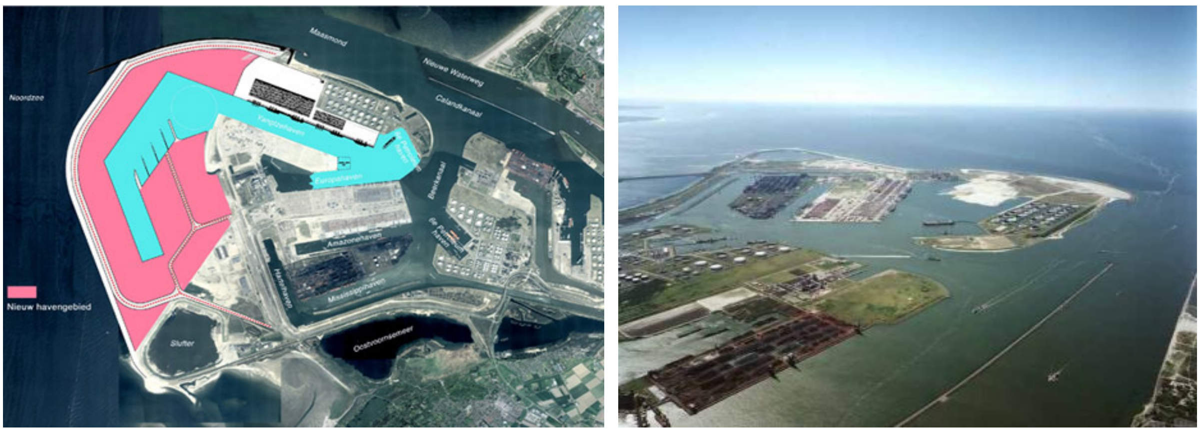


Figure 5.2: Maasvlakte 2 area, Port of Rotterdam

Rotterdam is the Europe's market leader for container transhipment, thus an economically attractive quay wall for the port of Rotterdam may constitute a possible construction to different locations.

In Handbook of Quay Walls a special probabilistic water level is described. This analysis is based on the probabilistic distribution function of high and low waters. From this analysis mean values and standard deviations can be determined.

The water levels according to Handbook of Quay Walls are summarised in the followings:

Table 5.3: Design values for water levels

Parameter	**Mean Value**	**Standard Deviation**	**Design Value**
High Water Level	+1.26	0.33	+1.46
Low Water Level	-1.22	0.27	-1.38
Low Low Water Spring	-1.61	0.27	-1.77
Groundwater Level	+0.80	0.25	+1.30

The design free water levels will be:

$$h_{LW} = \mu_{50,LW} - \gamma_{SF} \cdot \sigma_{LW} = -1.22 - 0.6 \cdot 0.27 = -1.38 \text{mNAP}$$
$$h_{LLW} = \mu_{LLW} - \gamma_{SF} \cdot \sigma_{LLW} = -1.61 - 0.6 \cdot 0.27 = -1.77 \text{mNAP}$$
$$h_{HW} = \mu_{HW} + \gamma_{SF} \cdot \sigma_{HW} = +1.26 + 0.6 \cdot 0.33 = +1.46 \text{mNAP}$$

The high groundwater level can be derived from the free water levels:

$$h_{g,HW} = \mu_{50,gHW} + \gamma_{SF} \cdot \sigma_{g,HW} = +0.80 + 2 \cdot 0.25 = +1.30 \text{mNAP}$$

It can be noted, that the groundwater level follows the tide near the free water level. But the fluctuation in the groundwater is less than a tide, since the underground flow needs time in order to adapt to the new pressure difference. In every location, the classification of the soil substrata should be taken into account, since the presence of an impermeable clay layer can make the underground flow impossible. In this case study, a rough assumption of a uniform sand layer is made, thus the groundwater can easily follow the variations of the free water.

It is usually assumed that the tide has a time shift of 2 hours with the ground water. For simplicity reasons, no drainage systems will be applied and the low groundwater level is assumed at the same magnitude of the Low Water Level. This assumption constitutes a safe approach for the further design.

5.2.3 Ground levels

The future ground level in Maasvlakte2 will be +5.0 m NAP. The chosen retaining height is 30 m, consequently the bottom level is -25.0 m NAP.

5.2.4 Soil profiles

In the followings the mechanical parameters of the soil profiles which were used in several stages are illustrated. It can be noted that different parameters are involved in different calculation means such as MSheet and PLAXIS. The Finite Element Method is very sensitive to the strength parameters of the soil, thus the input data were chosen according to NEN codes.

- **Soil Profile 1 – Uniform Sand soil Profile**

In the following table the basic soil parameters are summarised. Extra mechanical parameters needed for the simulation of the soil profile in Elastic Supported Beam (Msheet) and Finite Element Method (PLAXIS) are describing in the related chapters.

Table 5.4: Design values for soil mechanical parameters of soil profile 1 (sand q_c:15Mpa)

Soil parameter	Design Value	
Sand (+5.00 m NAP → -35.00 m NAP)		
Volumetric weight of dry soil “γ_d”	18	[kN/m^3]
Volumetric weight of saturated soil “γ_{sat}”	20	[kN/m^3]
Friction angle between soil and wall “δ”	20	[^{o}]
Angle of internal friction “φ”	30	[^{o}]
Dilatancy angle	2	[^{o}]

- **Soil Profile 2 – Clay and Sand soil Profile**

Table 5.5: Design values for soil mechanical parameters of soil profile 2

Soil parameter	Design Value	
Clay (+5.00 m NAP → -15.00 m NAP)		
Volumetric weight of dry soil “γ_d”	15	[kN/m^3]
Volumetric weight of saturated soil “γ_{sat}”	16	[kN/m^3]
Friction angle between soil and wall “δ”	11	[^{o}]
Angle of internal friction “φ”	17	[^{o}]
Dilatancy angle	0	[^{o}]
Cohesion	5	[^{o}]

*The same mechanical parameters of table 5.4 stand for the sand part in soil profile 2

5.2.5 Loads on the structure

<u>*Bollard loads*</u>
The design container vessel displaces more than 200.000 tons of water, thus the force on the berthing vessel that corresponds to this tonnage is approximately 2000 kN.

The effect of a concentrated point load acting directly on a concrete deck slab, or the increased loading area on which a concentrated point load is acting, is estimated as follows. This load is linearly redistributed through the superstructure over an angle of 45^o, as it is illustrated in the following figure.

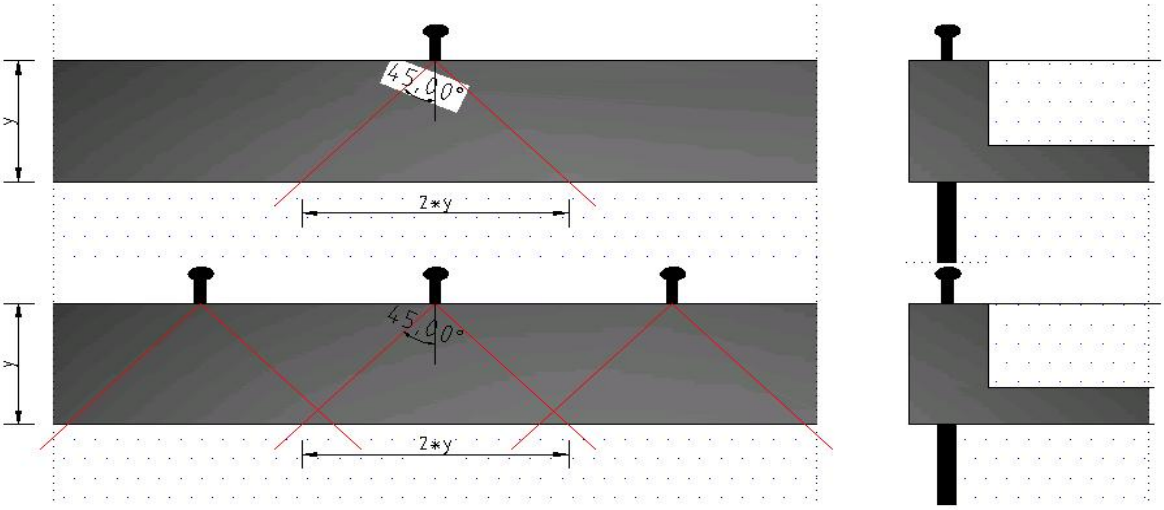


Figure 5.3: Redistribution of bollard loads over the quay

The redistributed line load as it is obvious depends on the depth of the platform. For the extreme case that the platform is laying in the surface the bollard load will be taken into account equal to **667 kN**, which come from a distribution over 1.5 m. For the intermediate case with h=6.5 m, the total load acts over a distance of 2*(6.5-1.5/2)=11.5 m, which results in approximately **175 kN/m**. Finally for h=11.5 m the redistributed load is approximately equal to 87 kN/m over a distance of 23 m.

Bollards should be provided at intervals of about 5-30 m depending on the size of the ship along the berthing face and the bollard load capacity. In the case of a design ship as Southampton ++, bollards should be placed every 30 m.

Moreover, for deep cases there is the possibility that loads from two individual bollards to overlap. The redistributed load would be two times higher, approximately equal to 175 kN/m. In order to make a safer approach to the problem, for the latter case of h=11.5 m also the design load of **175 kN/m** will be used.

The vertical component of the bollard load will be neglected since it has a relieving action to the structure.

Fender loads

The same stands for the fender loads. As a design value in that case will be used the reaction force of 2539 kN, for SCN 1800 – E1.3 which consist the EUROMAX fender system.
This load is redistributed as previous over an angle of 45^{o} for the two cases of h=6.5 m and h=11.5 m. This results in a design value of approximately **250 kN/m**. For the extreme case that the height of the relieving platform is equal to zero the fender load is acting directly on sheet pile wall.

Figure 5.4: Fender system on a quay wall

Crane loads

The design values, same with those of EUROMAX quay, will be assumed. The vertical load is equal to 1600 kN/m and the horizontal load is +/-60 kN/m. The mutual distance of the crane legs is 30.48 m.

The vertical and horizontal crane load on the landward side will not be included in the following calculations, since these loads are normally supported by separate spread foundations. Crane foundations support the tracks of rail-mounted cranes. The representative value of the vertical load and dynamic load of harbor cranes varies between 300 and 800 kN/m. In addition to vertical loads, as it has already mentioned the horizontal loads equal to 10% to 15% of the vertical ones. The simplest spread foundation consists of a structure that is built up from the following parts: foundation of stabilized sand, ballast bed of quarry stone, hardwood concrete or steel sleepers and the rail structure.

For heavier cranes, as in the present study case, continuous reinforced concrete beams are usually used. The crane track is designed as an elastic beam on elastic foundation in which both the stiffness of concrete structure and bed influence the actions in the cross section. The bearing resistance and settlement should be verified. Crane girders can be equipped with joints and shear connectors, but can also be constructed as a continuous reinforced concrete beam. In the latter case extra longitudinal reinforcement must be used to control cracking that may result if deformation is prevented. The level of the rail structures must be sufficiently adjustable.

In the following figures, a typical cross section of a crane girder is presented as well as a picture from the procedure of the construction:

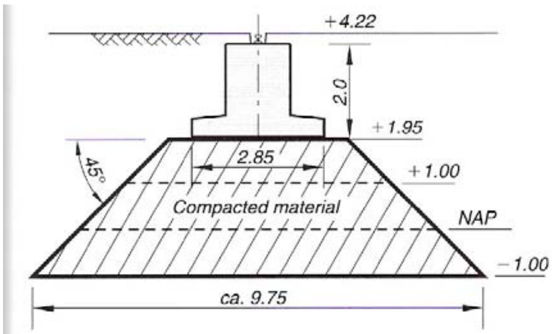


Figure 5.5: Schematisation of landward crane girder

Figure 5.6: Landward crane girder – under construction

Terrain loads
In addition, it is assumed that between the two crane legs full containers of 2 layers will be used, and that can happen in case of an emergency, which leads to 35 kN/m^2. For the area behind of the landward crane leg 4 layers are assumed, which leads to 55 kN/m^2. Taking into consideration the future development, one can say that it is a reliable assumption to use **40 kN/m^2** and **60 kN/m^2** respectively. These are the used design values for EUROMAX container terminal. Moreover, on that way the traffic load is also taken into account.

In the followings the 7 different designs are presented. The length of the MV-pile and the bearing piles is not illustrated on scale. The dimensions of these elements are noted in the followings.

5.3 Parametric analysis - Seven different cases

The analysis of the quay wall is executed with respect to the Handbook of Quay Walls CUR 211 recommendations. The aim is to conclude into the optimisation of quay wall structures, through a parametric analysis, in cohesive and not cohesive soils where a relieving platform is present. In this two dimensional analysis 7 different designs of quay walls will be analysed. Extreme values of the ***width (b)*** and the ***depth (h)*** of the platform will be used and intermediate situations will be interpolated.

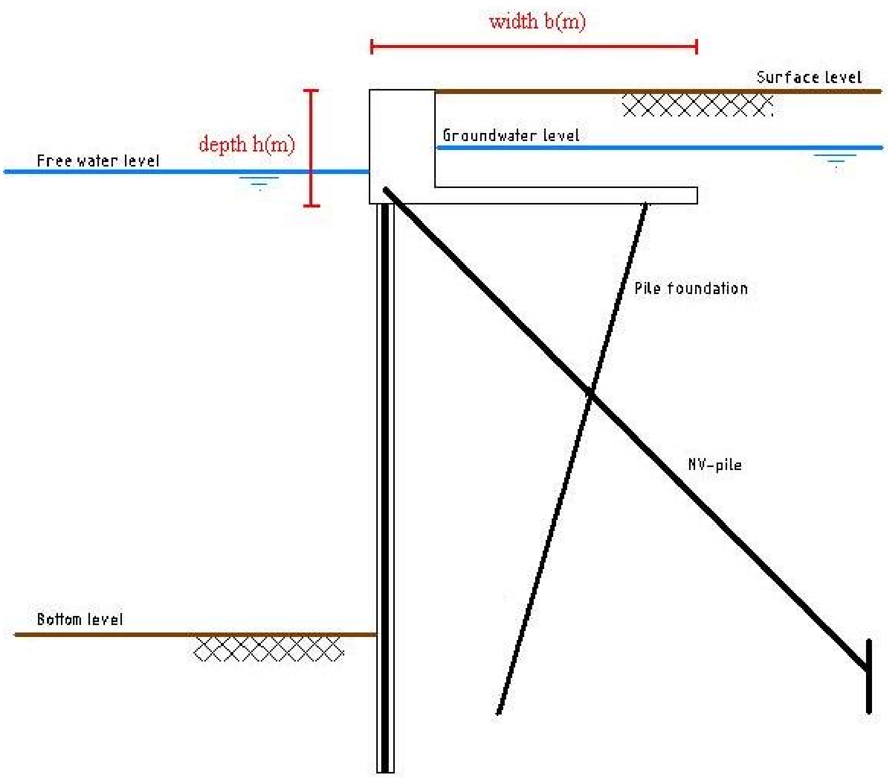


Figure 5.7: Schematisation of a Quay Wall with a relieving platform

The following diagram shows the cases that are investigated in that phase. In order to give a clear representation of these cases a schematisation of these 7 different designs are presented below:

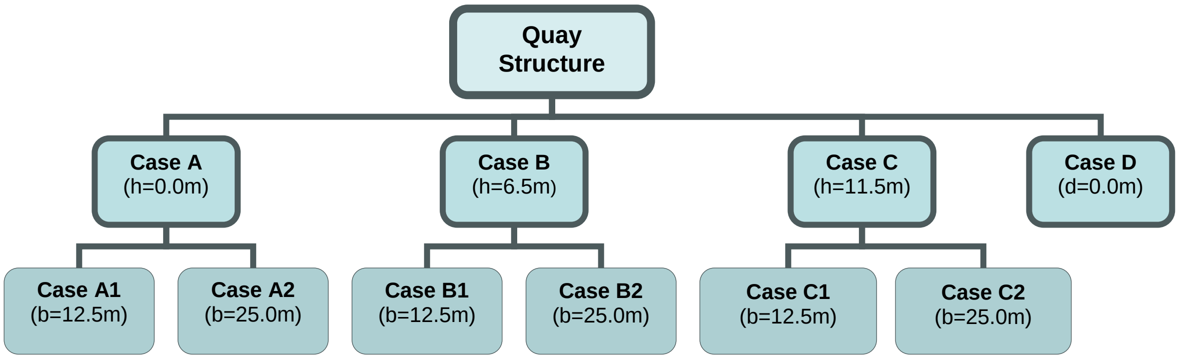
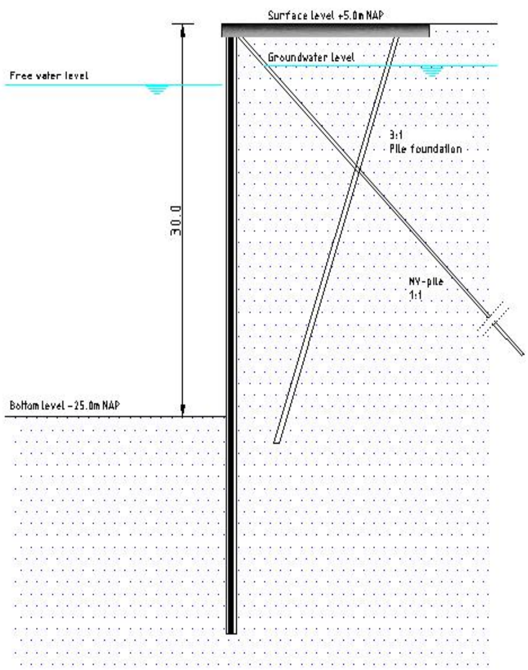


Figure 5.8: Seven different cases of investigation

<u>Case A: Extreme situation with the relieving platform on the surface level</u>

Case A1

Case A2



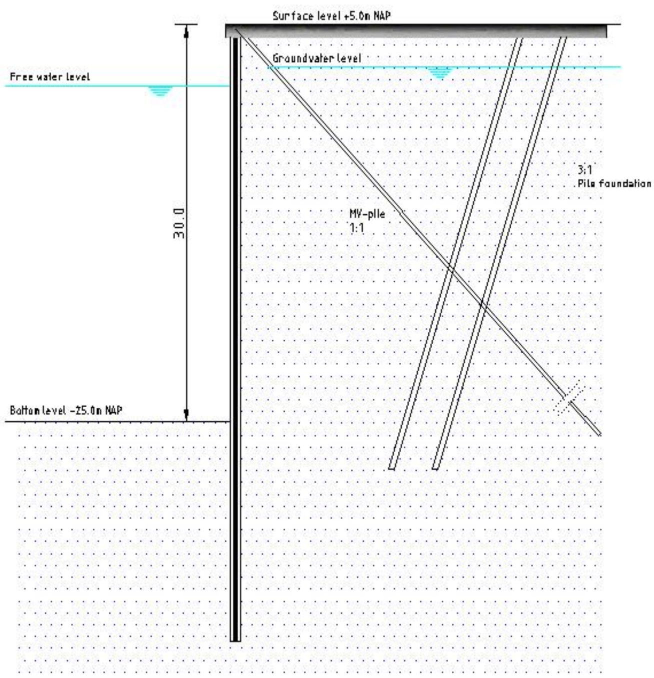


Figure 5.9: Design cases A1 and A2

Case B: Intermediate situation with the relieving platform at depth h=6.5 m

Case B1 **Case B2**

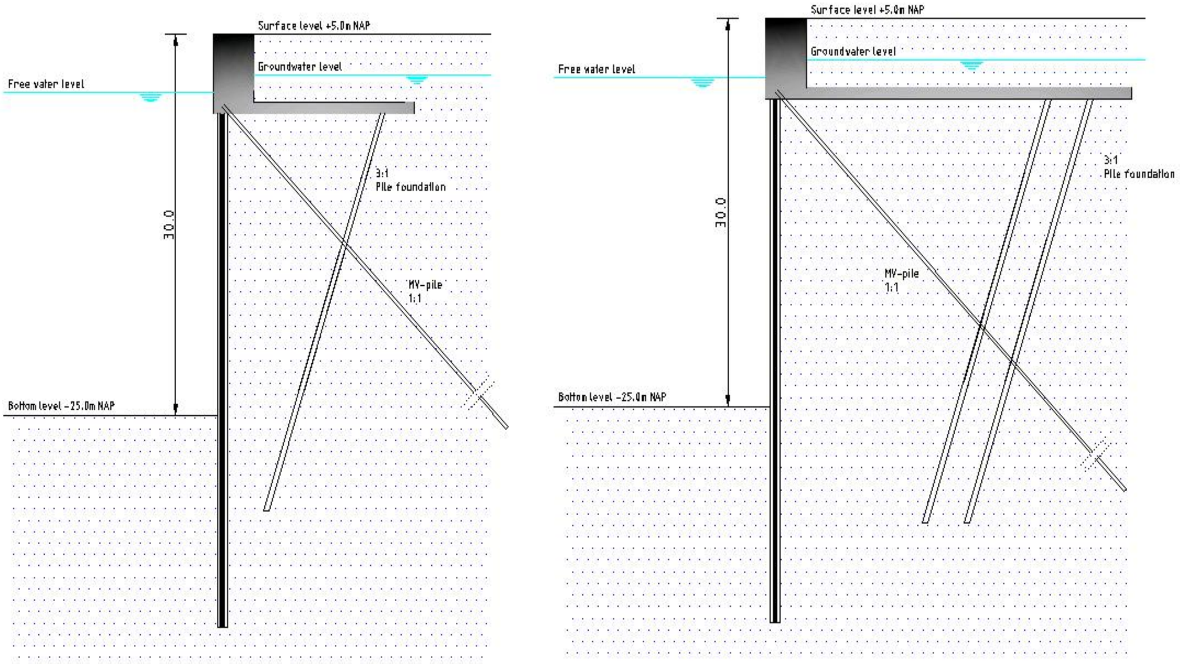


Figure 5.10: Design cases B1 and B2

Case C: Extreme situation with the relieving platform at depth h=11.5 m

Case C1 **Case C2**

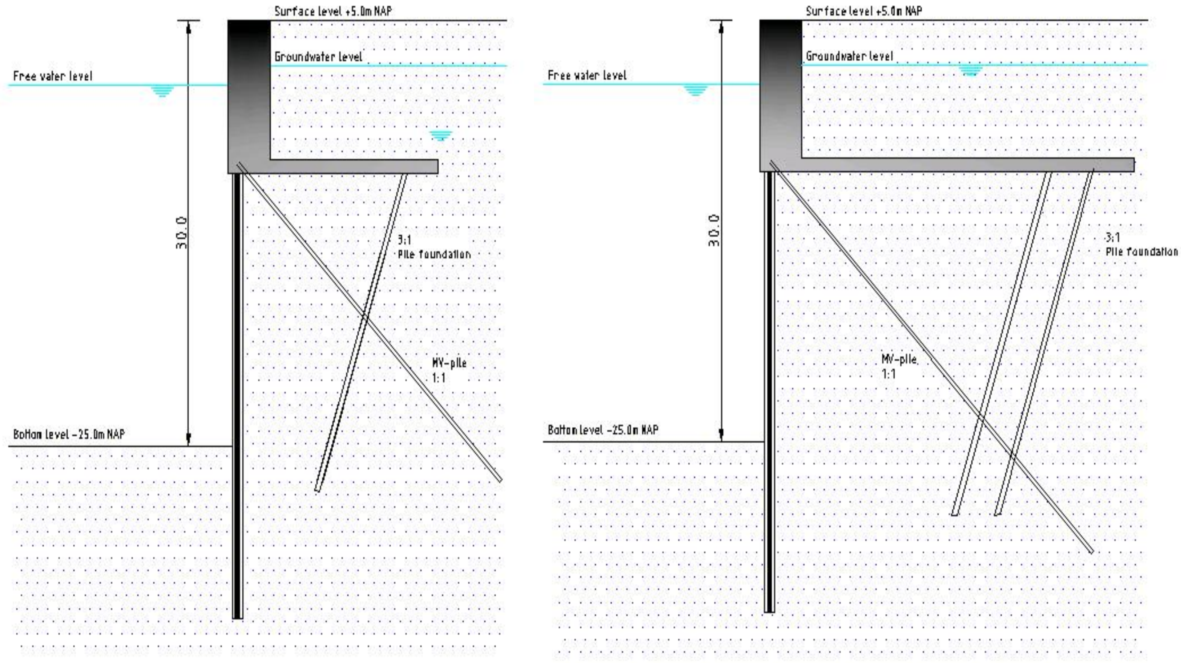


Figure 5.11: Design cases C1 and C2

Case D: Extreme situation without a relieving platform

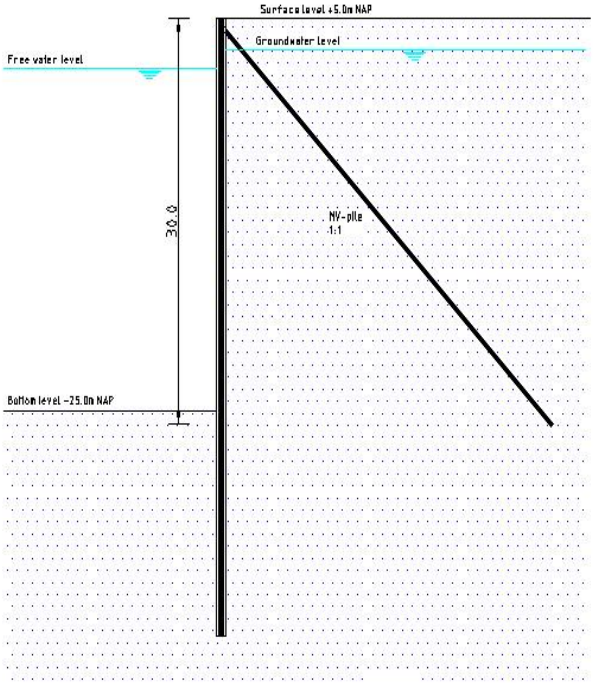


Figure 5.12: Design case D

An additional remark has to be made with respect to the construction phases of the quay wall. It is usual that the construction phases will lead to additional load cases. As an example, when the soil in front of the sheet pile wall is partly excavated before applying the anchorage.

The sheet pile wall can be temporarily considered as a cantilever; while after the installation of the anchorage it can be schematised as a simple supported beam. However, the construction phases will not lead to additional load cases for the sheet pile wall. The construction phases are schematised in figure:

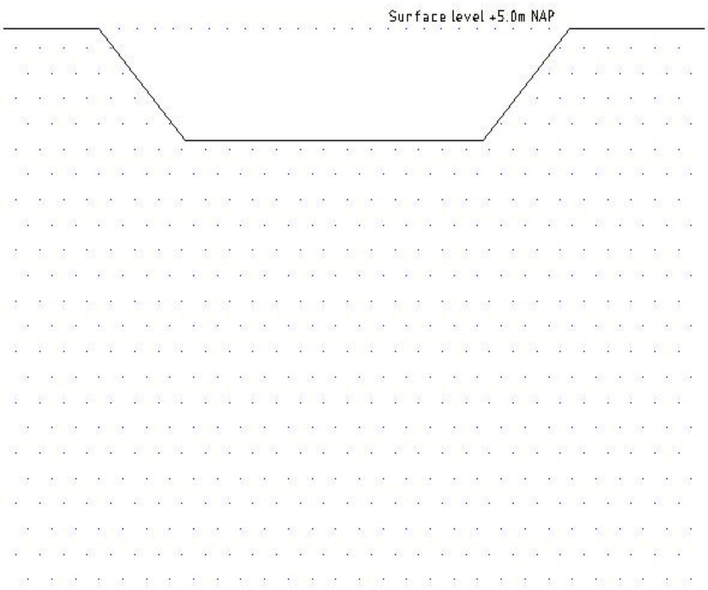


Figure 5.13: Phase 1 - Small excavation

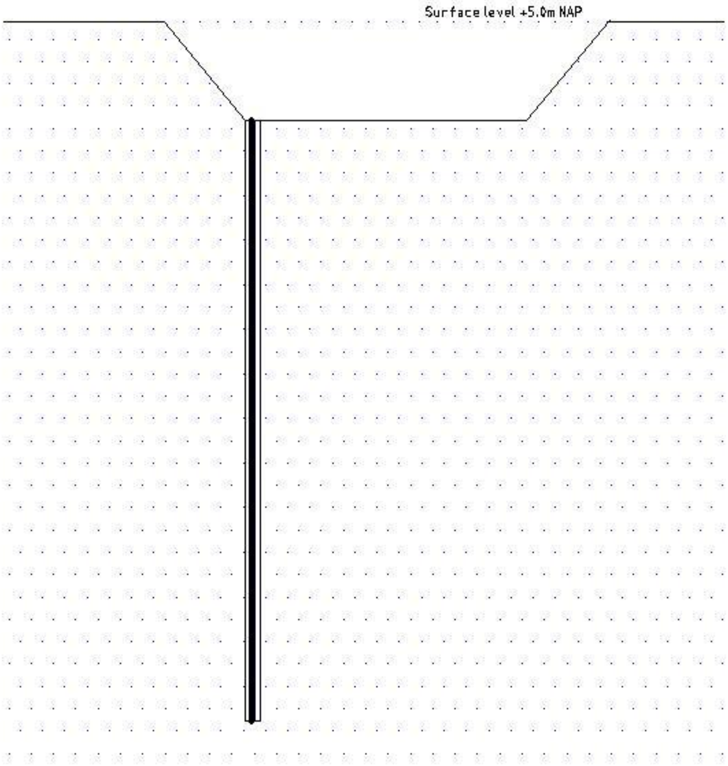


Figure 5.14: Phase 2 - Driving of Sheet pile wall

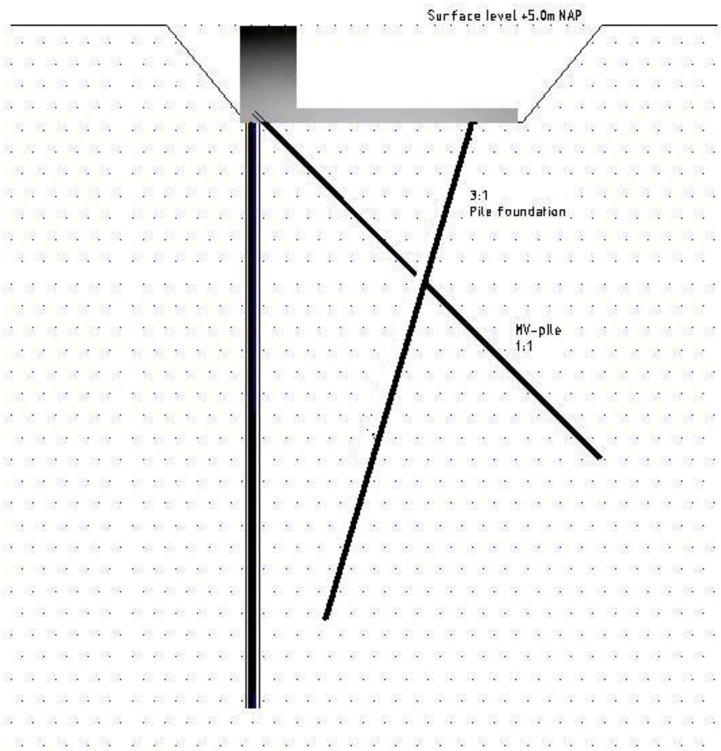


Figure 5.15: Phase 3 - Construction of Quay Wall

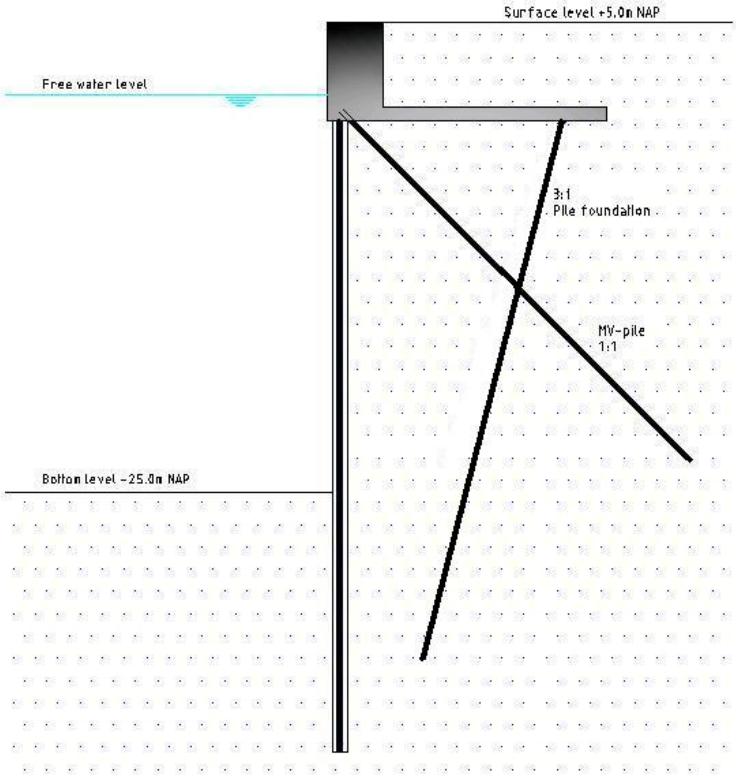


Figure 5.16: Phase - 4: Front excavation and fill of the back side

In construction phase 3 the relieving platform will be constructed. The relieving platform structure is connected to the wall structure in such a way that it provides the anchorage. Due to this configuration the sheet pile wall is anchored before the excavation at the front of the wall start. On that way, it is assumed that the construction phases do not lead to additional load cases.

5.4 References

[5.1] CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*

[5.2] EAU 2004; *"Recommendations of the Committee for Waterfront Structures Harbours and Waterways"*

[5.3] Emiel Meijer; "*Comparative analysis of Design Recommendation for Quay Walls*", Delft University of Technology/Gemeentewerken Rotterdam, 2006

[5.4] Priscilla Bonte; "*Sandwich Wall as the wall of the future*", Delft University of Technology/Delta Marine Consultants B.V., 2007

[5.5] Archive/Library of Gemeentewerken Rotterdam, Rotterdam

CHAPTER 6

PART-A
HAND CALCULATIONS-PLATFORM ANALYSIS

6.1 General

The use of a relieving platform primarily reduces the active earth pressures on the upper most part of the sheet pile wall. The redistribution of the stresses is illustrated in the following figure.

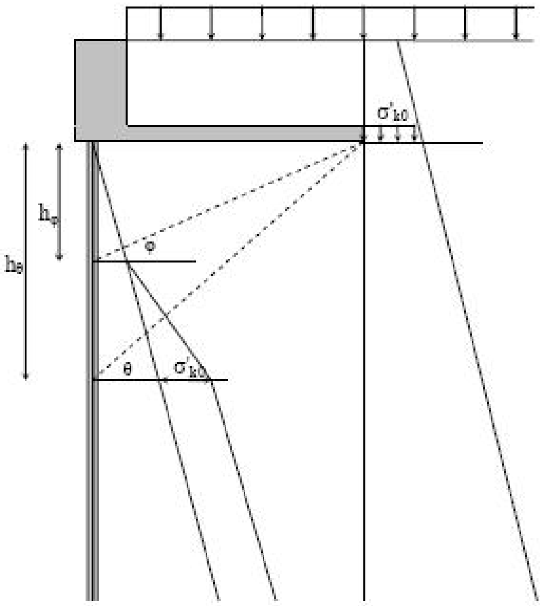


Figure 6.1: Principle of relieving platform

It has to be noted that the following redistribution is based on the Rankine's theory distribution of stresses, the principle of which is illustrated in the above figure.

In this chapter the calculation procedure of the relieving platform is described step-by-step. For the sake of clarity, the superstructure is assumed that is possessing great horizontal strength and stiffness, so that no passive or active stresses are activated.

The relieving platform of the quay wall is schematised to a simple statically determinate frame. In the case of a long relieving platform the presence of more bearing piles is necessary, thus the system is a statically indeterminate one. For that case in the following calculations two bearing piles were used and the acting axial forces on those elements can be spread, in a more detailed design, to several rows.

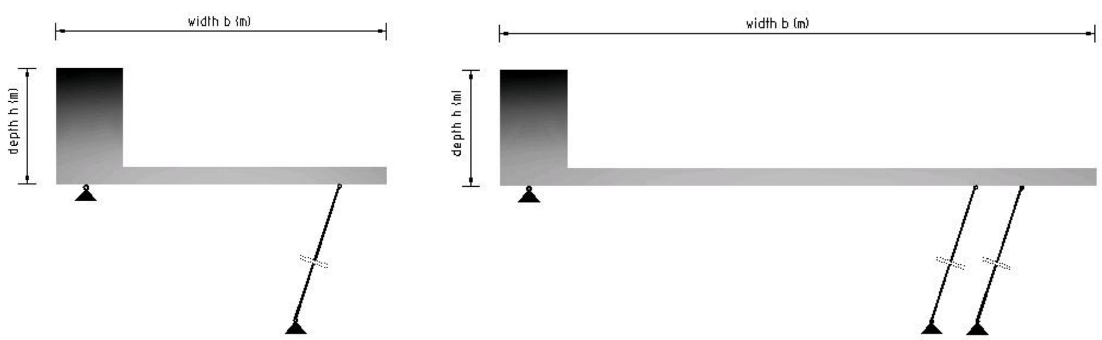


Figure 6.2: Schematisation of the relieving platform with different widths

This concept of the schematisations is based on the followings:

- Tension and bearing piles are considered as hinged bars that take up only normal forces;
- Several rows of piles were needed are grouped into one or two piles, positioned in the centre of gravity of the rows;
- In the position of a pile trestle the support is schematised as a hinge;
- If there is an anchor, it can be schematised as a flexible bar;
- In the position of the inclined pile row the support is schematised to an inclined hinged bar that only takes up axial loads;

6.2 Loads Acting on the relieving platform

The loads acting directly on the relieving platform are given on Chapter 3: "Loads acting on Quay Walls". Water pressures caused on differences between the water level and the phreatic level, as well horizontal earth pressures, the magnitude of which depends on the deformations of the relieving platform, are important determinants. With fundamental combinations of loads, neutral earth pressures are considered. For accidental combinations of loads the extra horizontal deformation is assumed so that the development of active earth pressures is possible.

Moreover, horizontal deformations of the earth mass under the relieving platform appear as imposed deformations of the foundation piles. With a dense pile group, these forces can be of such a magnitude that the reducing action of the relieving platform is negatively influenced. This effect can be taken into account by adding an extra horizontal anchor force. The magnitude of that force can be computed by simulating the foundation piles as fixed supported beams with actual bending stiffness exposed to the specific deformation. This may result in a reduction of the earth pressures on the sheet piles. In the present study, for simplicity reasons, that was not taken into consideration.

6.3 Load Combinations

In the verification of various limit states (ULS, SLS) several load combinations including permanent and variable loads are considered. The designers have to investigate a number of these load combinations in order to define the normative design values of bending moments, internal forces, displacements, stresses etc, for all the elements of the structure, such as the sheet pile, the bearing piles, the anchor and the relieving platform.

In the ultimate limit state the following types of combinations of loads should be investigated:

- Fundamental combinations of loads
- Accidental combinations of loads

In the presence calculations only the fundamental combination of loads will be studied, for different water levels.

Fundamental combinations of loads that are analysed for quay structures include:

- Combination of loads consisting permanent loads and a leading variable load caused by unfavourable water pressure at low tide, combined with one meteorological load and two simultaneously occurring variable loads or partial loads, caused by surcharges, crane loads, traffic loads and bollard loads;

- Combination of loads consisting of permanent loads and leading variable load caused by unfavourable water pressure with high phreatic level and a maximum gradient, combined with one meteorological load and two simultaneously occurring variable or partial loads, for example surcharges, crane loads, traffic loads and bollard loads;
- Combination of loads consisting of permanent loads and a leading variable load caused by unfavourable resulting water pressure at low free surface water level with simultaneous ground water level, combined with one meteorological load and two simultaneously occurring variable or partial loads caused for example by surcharges, crane loads, traffic loads and bollard loads;

The objective of that approach is to find the crucial combination of loads that lead to normative loads on the various foundation members such as sheet piles, relieving platform and bearing and tension piles. For the composition of loads in a combination specific associated reduction factors are used.

The calculation of the response of the groundwater level to the variation of the surface water level can be modelled. For this it is necessary to take into account a certain delay in the response of the groundwater level. A low surface water level, with high ground water is normative for the stresses occurring in the sheet pile and anchorage. The aforementioned load combinations are presented in the following tables.

Load Combination A: (LCA)

Table 6.1: Load combinations LCA1, LCA2, LCA3

High Ground Water Level [+1.30 m] + Low Free Water Level [-1.38 m]			
	LCA1	**LCA2**	**LCA3**
Earth Pressures	1.0	1.0	1.0
Water Pressures	1.0	1.0	1.0
Terrain Loads	0.7	0.7	0.7
Crane Loads	0.7	-	-
Bollard Loads	-	0.7	-
Fender Load	-	-	0.7

Load Combination B: (LCB)

Table 6.2: Load combination LCB

Low Ground Water Level [-1.38 m] + Low Free Water Level [-1.38 m]	
	LCB
Earth Pressures	1.0
Water Pressures	1.0
Terrain Loads	0.7
Crane Loads	0.7
Bollard Loads	-
Fender Load	-

6.4 Analysis of the relieving platform

The relieving platform will be investigated by two different static models. In the first static model the connection point was simulated by a combination of **a bar with an angle of 45^o and a roller support**. On that way, the anchor is simulated with its own cross-section and material characteristics whilst the sheet pile wall is entered in the model as an infinite stiff support. This simulation will lead to higher axial forces on the sheet pile wall since its stiffness is assumed infinite. (That is happening in cases A2, B2 and C2, where the static system is undetermined and the stiffness of the members plays role in the distribution of forces).

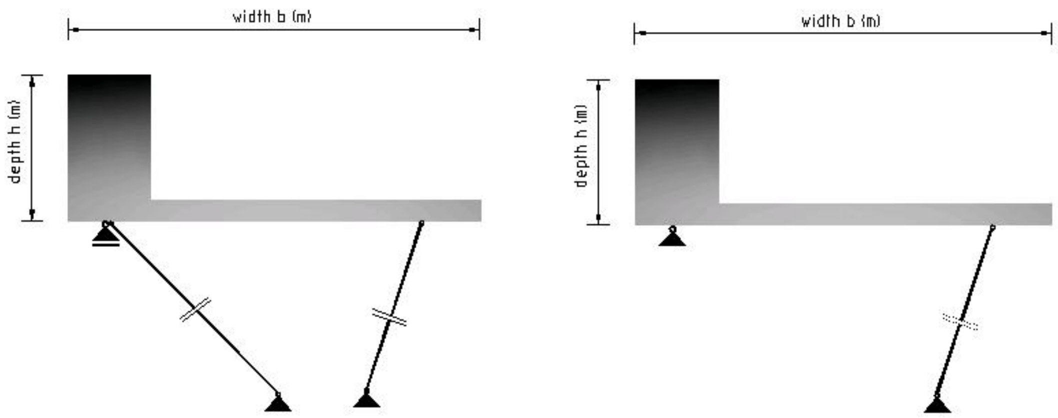


Figure 6.34: Two different static models of the relieving platform

In the second static model, which is also proposed by the Handbook of Quay Walls, the connecting point is simulated as a **fixed hinge**. On that way, the distribution of forces will be on the same extent to the horizontal and vertical direction. It has to be noted that the horizontal reaction in the connecting point was analysed in two directions (the anchor plane and the vertical direction) respecting the direction of the vectors, in order to get the total axial force.

It has to be noted also that, the modulus of elasticity of each embedded element was decreased by the factor 2.5, since the mutual distance of them was assumed equal to 2.5 m. The characteristics of each element are summarised in the following tables.

Table 6.3: MV-pile characteristics

MV pile	
Profile	HP 360 x 152
Cross-section (A)	193.8 [cm^2]
Moment of inertia (I)	$4395 \cdot 10^5$ [mm^4]
Modulus of Elasticity (E)	205 [GPa]
Angle	45^o
Length	35-50 [m]
Centre to centre distance	2.5 [m]

Table 6.4: Bearing piles characteristics

Bearing piles	
Diameter (D)	0.67 [m]
Modulus of Elasticity (E)	36 [GPa] (B55)
Inclination	3:1
Length	25-35 [m]
Centre to centre distance	2.5 [m]

Table 6.5: Concrete platform characteristics

Concrete Relieving Platform	
Horizontal Part	
Cross-section (A)	15000 [cm^2]
Moment of inertia (I)	$28.125 \cdot 10^{10}$ [mm^4]
Modulus of Elasticity (E)	15 [GPa] (B35)
Vertical Part	
Cross-section (A)	30000 [cm^2]
Moment of inertia (I)	$225 \cdot 10^{10}$ [mm^4
Modulus of Elasticity (E)	15 [GPa] (B35)

The loads, the two different static systems and the coming results of each case are illustrated in the followings pages.

- CASE A1

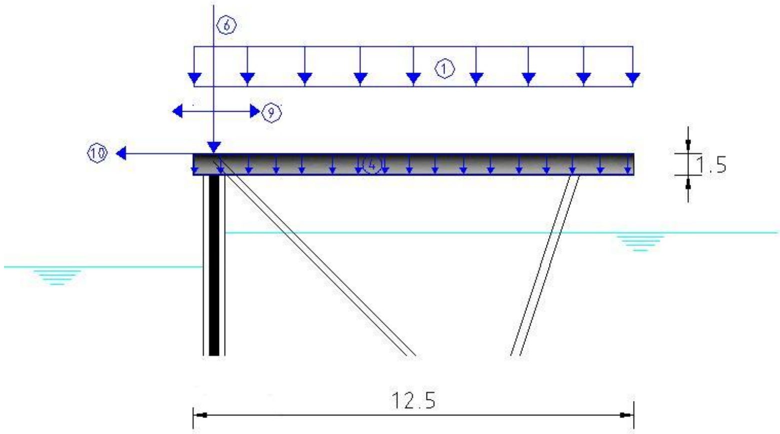


Figure 6.4: Loads acting on the relieving platform A1

- CASE A2

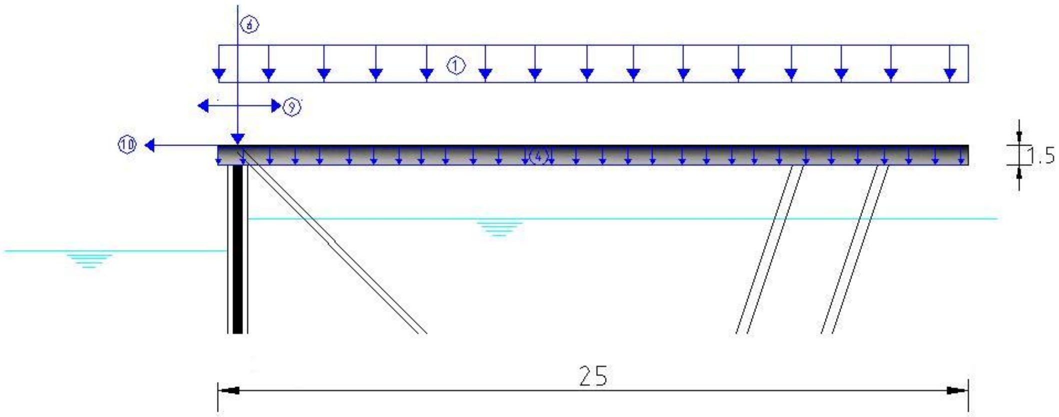


Figure 6.5: Loads acting on the relieving platform A2

Table 6.6: Loads on the relieving platform

Vertical Loads			
1	Terrain load	40	[kN/m^2]
4	Dead load	37.5	[kN/m^2]
6	Crane load	1600	[kN/m]
Horizontal Loads			
9	Crane load	60	[kN/m]
10	Bollard load	667	[kN/m]

- CASE B1

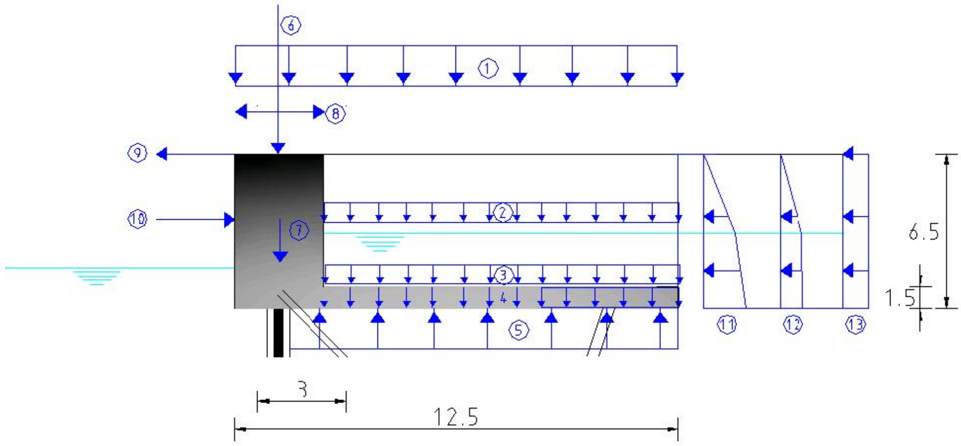


Figure 6.6: Loads acting on the relieving platform B1

- CASE B2

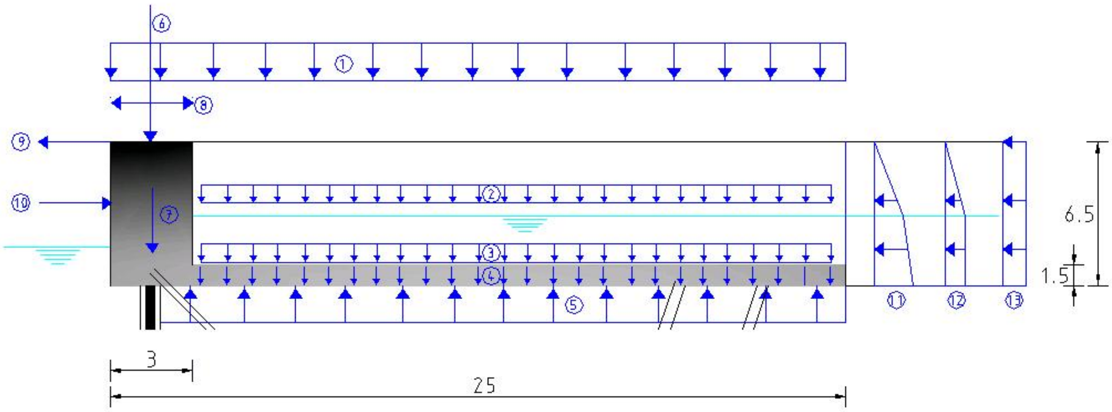


Figure 6.7: Loads acting on the relieving platform B2

Table 6.7: Loads on the relieving platform

Vertical Loads			
1	Terrain load	40	[kN/m^2]
2	Soil weight (dry)	66.6	[kN/m^2]
3	Soil weight (saturated)	26	[kN/m^2]
4	Dead load 1	37.5	[kN/m^2]
5	Uplift	28	[kN/m^2]
6	Crane load	1600	[kN/m]
7	Dead load 2	375	[kN/m]
Horizontal Loads			
8	Crane load	60	[kN/m]
9	Bollard load	175	[kN/m]
10	Fender load	250	[kN/m]
11	Soil pressures	61.25	[kN/m^2]
12	Hydrostatic pressures	26.8	[kN/m^2]
13	Terrain load	20	[kN/m^2]

- CASE C1

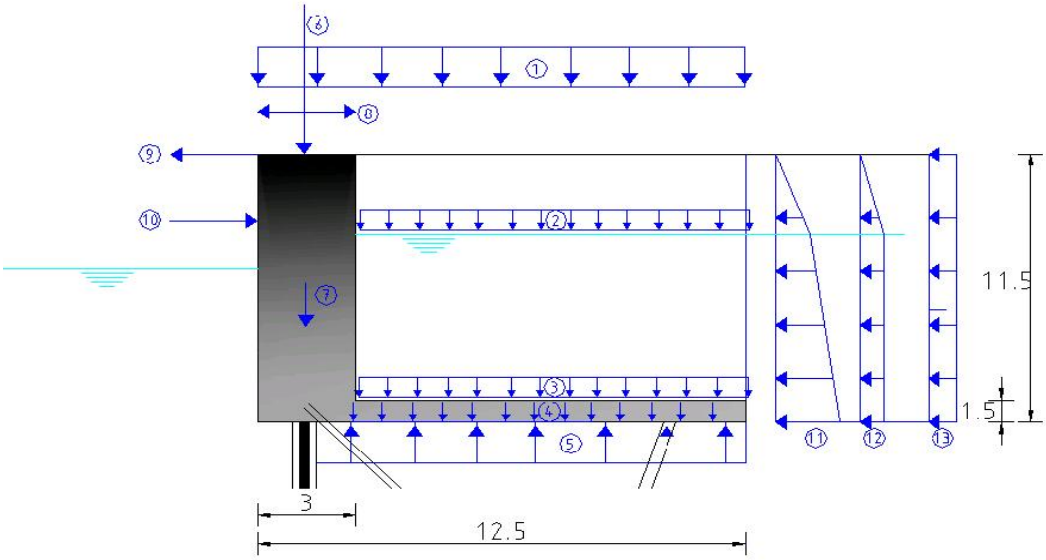


Figure 6.8: Loads acting on the relieving platform C1

- CASE C2

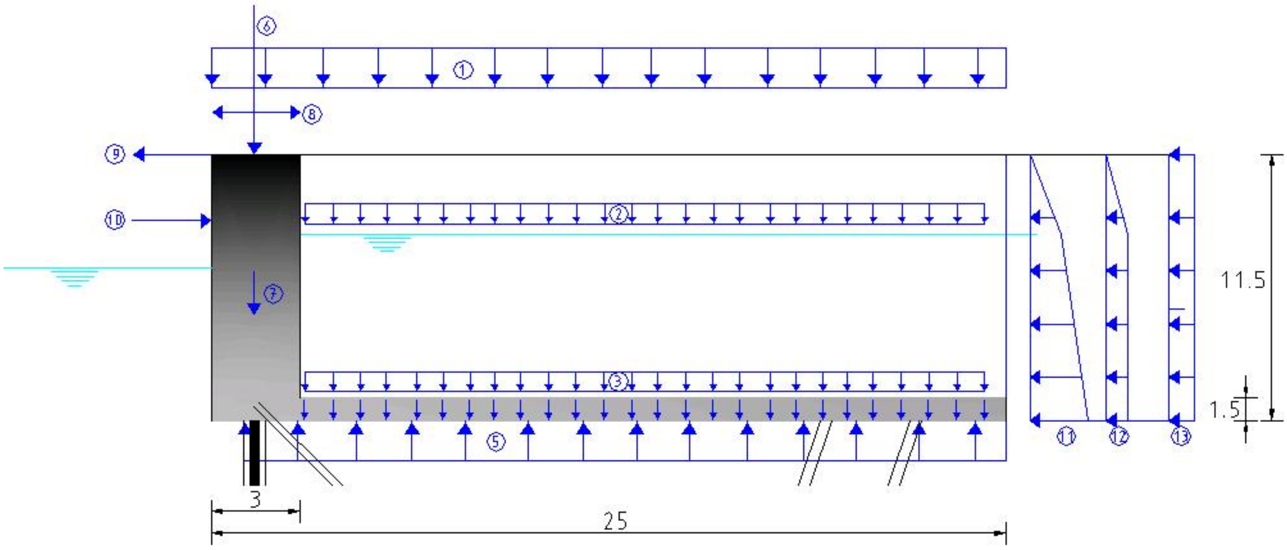


Figure 6.9: Loads acting on the relieving platform C2

Table 6.8: Loads on the relieving platform

Vertical Loads			
1	Terrain load	40	[kN/m^2]
2	Soil weight (dry)	66.6	[kN/m^2]
3	Soil weight (saturated)	156	[kN/m^2]
4	Dead load 1	37.5	[kN/m^2]
5	Uplift	78	[kN/m^2]
6	Crane load	1600	[kN/m]
7	Dead load 2	750	[kN/m]

Horizontal Loads			
8	Crane load	60	[kN/m]
9	Bollard load	175	[kN/m]
10	Fender load	250	[kN/m]
11	Soil pressures	33.25, 111.25	[kN/m^2]
12	Hydrostatic pressures	26.8	[kN/m^2]
13	Terrain load	20	[kN/m^2]

6.5 Results from the Static Analysis

The different results from the two different static systems are presented below. Only the extreme values, coming from the normative load combination each time is illustrated in the following tables. All the results are presented analytically in the appendices.

Table 6.9: Extreme values coming from the first static system

CASE	Reaction in connecting point [kN/m]	Anchor Force [kN/m]	Bearing Pile 1 [kN/m]	Bearing Pile 2 [kN/m]
A1	1330.948	430.557	513.713	-
A2	1294.777	125.730	918.952	336.486
B1	2346.000	267.263	1021.713	-
B2	2176.772	-409.226	1767.662	658.305
C1	3776.104	811.206	1462.847	-
C2	3502.177	-244.360	2695.778	1155.057

Table 6.10: Extreme values coming from the second static system

CASE	Reaction in connecting point [kN/m]	Anchor Force [kN/m]	Bearing Pile 1 [kN/m]	Bearing Pile 2 [kN/m]
A1	1330.900	430.556	513.713	-
A2	1283.628	126.780	906.567	292.922
B1	2346.547	267.264	1021.713	-
B2	2163.232	-412.858	1859.208	623.792
C1	3776.104	811.207	1462.847	-
C2	3454.000	-242.634	2760.547	1138.645

As it can be seen from the above, the results between the two models present a small deviation. The extreme values coming from these two models are used in further calculations. The summation of the axial force in the connection point and the vertical component of the anchor force is applied as an external load in the sheet pile wall.

6.6 References

[6.1] CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*, 2005

[6.2] Prof. Arnold Verruijt; *"Soil Mechanics"*, Delft University of Technology, 2001-2006

[6.3] Karl Terzaghi, Ralph Peck, Gholamreza Mesri; *"Soil Mechanics in Engineering Practice"*

[6.4] Emiel Meijer; *"Comparative analysis of Design Recommendation for Quay Walls"*, Delft University of Technology/Gemeentewerken Rotterdam, 2006

[6.5] Priscilla Bonte; "*Sandwich Wall as the wall of the future*", Delft University of Technology/Delta Marine Consultants B.V., 2007

CHAPTER 7

PART-A
SHEET PILLE WALL ANALYSIS – BLUM METHOD

7.1 General

According to Blum’s theory, the sheet pile is considered as a beam that is loaded by soil and water pressures. On the top the sheet pile wall is connected to the relieving platform by means of an anchorage system, i.e. in the present case the MV-piles are considered. At the lower part, the sheet pile wall is supported by the passive soil resistance, resulted by the soil under the bed of the labour. With a minimal embedded depth, the soil layer providing resistance is just able to ensure the stability of the sheet piles. The degree of fixing depends on a number of factors, such as extra sheet pile length in relation to the minimal length, stiffness of the resistance soil and the bending stiffness of the sheet piles.

The various calculation methods lead to different results. From the point of view of structural analyses two basic concepts are recognised:

- Free earth support sheet piles;
- Fixed earth support sheet piles;

As it has already mentioned the sheet pile wall calculation will be executed with three different approaches of Blum’s theory. This theory is based on the failure condition of the soil where the minimum active and maximum passive earth pressures occur. The first approach that leads to small embedded depths and high anchor forces takes into consideration a partially fixed end at the toe of the wall. Fully fixity is assumed in the second and third approach where higher embedded depths and smaller anchor forces are reached. Between these extremes, a variety of intermediate cases like partially fixed sheet pile systems are also possible.

7.2 Assumptions

Firstly, it has to be noted that in order to provide a margin of safety, the final coming results of the aforementioned methods should be examined and moderated. That can be done by two methods, either by an addition of 20-40% of the final embedded depth or the use of a reduced value Kp’, by dividing it with a safety factor of 1.5.

In the present study the second way will be used, since according to that way the soil properties are reduced and that is closer to the philosophy of the recent more advanced calculation means.

Moreover, the several methods have been developed in order to estimate the lateral earth pressure coefficients (Rankine theory, Coulomb theory, NEN, etc). For the calculation of sheet pile walls that also have a bearing function, the favourable working of the angle friction δ is taken into account in the determination of horizontal soil pressures. Consequently, the Rankine’s theory can be avoided in that case.

For the calculations of Part A, the NEN coefficients have been used instead of the Coulomb theory coefficients, since they are accurate enough and broadly used in projects relative to Port of Rotterdam.

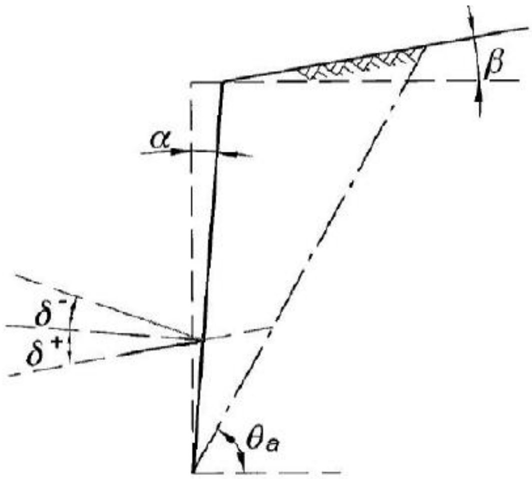


Figure 7.1: Illustration of the parameters involved in the earth pressure coefficient formulas

The expressions for the coefficients of the earth pressures according to the Dutch code NEN are:

$$K_a = \frac{\cos^2(\varphi+\alpha)}{\cos^2\alpha\left[1+\sqrt{\dfrac{\sin(\varphi+\delta)\cdot\sin(\varphi-\beta)}{\cos(\alpha-\delta)\cdot\sin(\alpha+\beta)}}\right]^2}$$

$$K_p = \frac{\cos^2(\varphi-\alpha)}{\cos^2\alpha\left[1-\sqrt{\dfrac{\sin(\varphi-\delta)\cdot\sin(\varphi+\beta)}{\cos(\alpha-\delta)\cdot\sin(\alpha+\beta)}}\right]^2}$$

For the present case where $\varphi=30^o$ $\delta=20^o$ $\beta=0^o$ $\alpha=0^o$ we get:

K_a=5.74 and **K_p=0.28.**

7.3 Blum model

Blum's method is often used as an introduction to sheet pile walls in many textbooks. The basis of the method is the assumption of limit earth pressures acting on the sheet pile wall. This means that this method uses calculations with minimum active and maximum passive earth pressures.

The earth pressure distribution in Blum's method is based on *Rankine's* earth pressure theory and the calculations can be carried out as a supported beam calculation. Because of the simplicity, the Blum design method is still frequently used in the development of draft designs. This can be a first estimation of the minimum length of sheet piles, the pile length at which fixed end is achieved and the estimation of bending moments and anchor forces.

A disadvantage of the Blum's method is that the actual earth pressures on site differ from the limit earth pressures, due to the fact that it does not take into account the earth pressure redistribution caused by arching in the soil behind the sheet pile wall. As a result the fixing moments are too large, the moments in the span are too small and the anchor forces are too low.

According to clause R77 of EAU, earth pressure redistribution may be taken into account by correcting the calculation results according to Blum as follows: the part of the bending moment resulting from the active effective earth pressure may be reduced by 33%, but the anchor force must then be increased by 15%.

This is possible due to arching occurring on sheet piles which are directly driven in the soil. In case that the back side is filled on stages, then that is not the case and the maximum bending moments should be taken into account. For safety reasons this reduction will not be taken into consideration in the present calculations.

7.3.1 Free earth Support Method

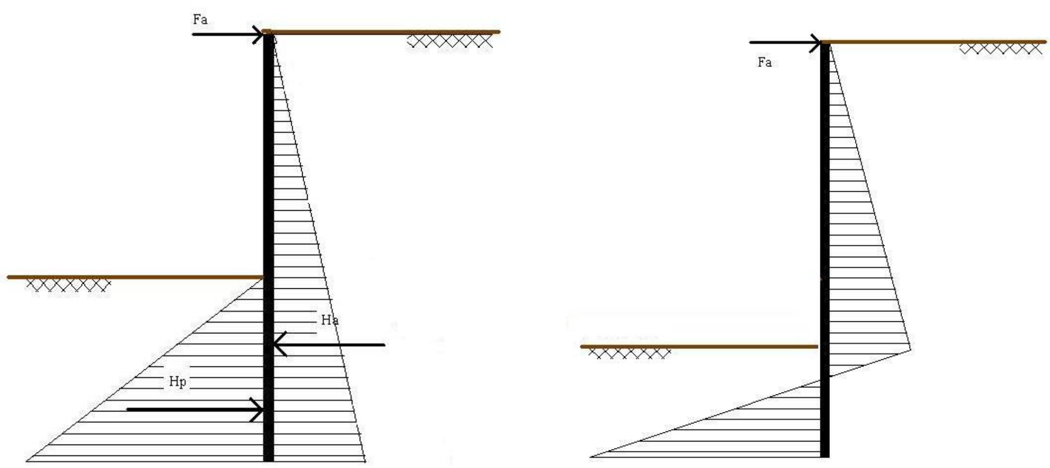


Figure 7.2: Free earth support method

The anchored wall is only just in equilibrium when the moment of the horizontal loads on the wall relative to the anchored point is zero. Of course, in this situation the anchor force must be able to be absorbed fully.

The limit state is described by the following boundary conditions:

$\Sigma M_{TOP} = 0$

$\Sigma F_{HOR} = 0$

The forces acting on the sheet pile wall are listed in the followings:

- Active and passive soil pressures;
- Hydrostatic water pressures;
- Surcharge of the soil pressures (depending on the platform dimensions);
- Anchor force;

The needed embedded depths and the anchor forces are summarised in the table below for each case. The calculations were executed by an Excel sheet that is found in the appendices.

Table 7.1: Results of Free Earth Support method

CASE	Anchor Force [kN] (horizontal component)	Embedded depth [m]
D	1513.99	12.78
A1	1276.38	12.44
A2	1178.90	12.65
B1	1076.30	11.50
B2	902.33	11.10
C1	797.37	10.41
C2	590.89	8.40

7.3.2 Fixed Earth Support Method (Equivalent Beam Method)

This method is based on the assumption that the elastic line of the wall will take the shape indicated in the following figure.

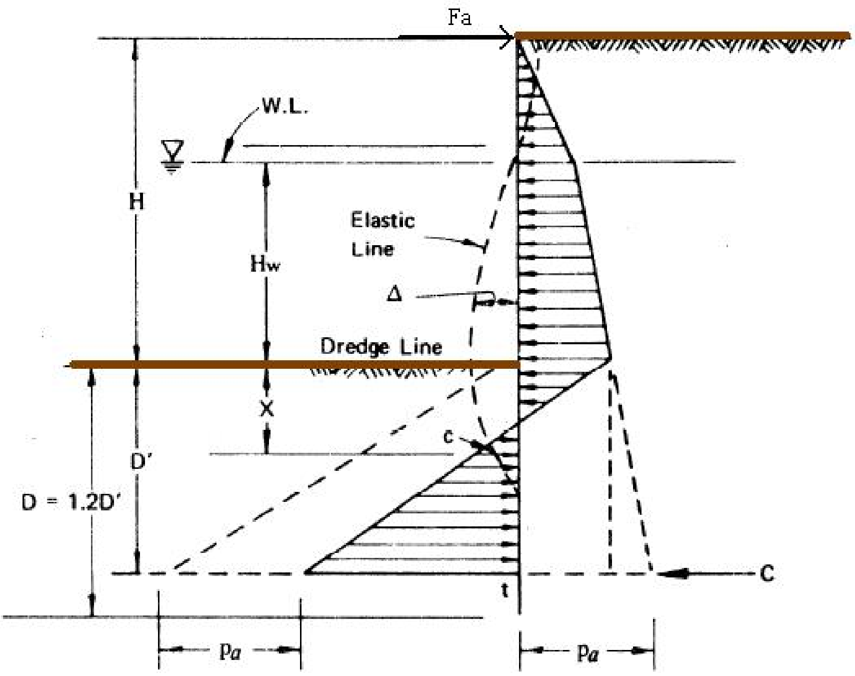


Figure 7.3: Fixed earth support method-Equivalent Beam Method

The deflected shape reverses its curvature at the *point of contraflexure* c and becomes vertical at point t. Consequently the beam acts like a partially built in beam subjected to bending moments.

To produce the deflected shape, the wall must be driven deep enough so that the soil provides the required restraint on the bulkhead deformations. The equivalent beam method assumes a hinge at the *point of contraflexure*, since the bending moments there are zero.

The part above the hinge can then be treated as a separate freely supported beam with an overhanging end as shown in the following figure.

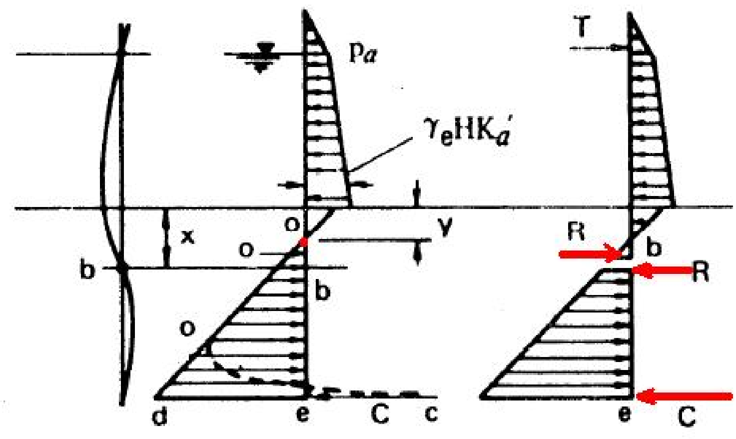


Figure 7.4: Point of Contraflexure and Reaction on the Equivalent Beam

The reactions R and T and the bending moments can be determined from statics and simple beam theory. The lower portion, below the *point of contraflexure*, can also be analysed as a separate freely supported beam on two supports R and C.

Also in this case the forces acting on the sheet pile wall are:

- Active and passive soil pressures;
- Hydrostatic water pressures;
- Surcharge of the soil pressures (depending on the platform dimensions);
- Anchor force;

The needed embedded depths and the anchor forces are summarised in the table below for each case. The calculations were executed, also in this case, by an Excel sheet that is found in the appendices.

Table 7.2: Results of Fixed Earth Support method (equivalent beam method)

CASE	Anchor Force [kN] (horizontal component)	Embedded depth [m]
D	1263.86	20.62
A1	1054.76	19.98
A2	923.30	19.03
B1	878.32	18.36
B2	709.57	16.33
C1	647.16	16.28
C2	496.18	13.36

7.3.3 Fixed Earth Support Method (Displacement Method)

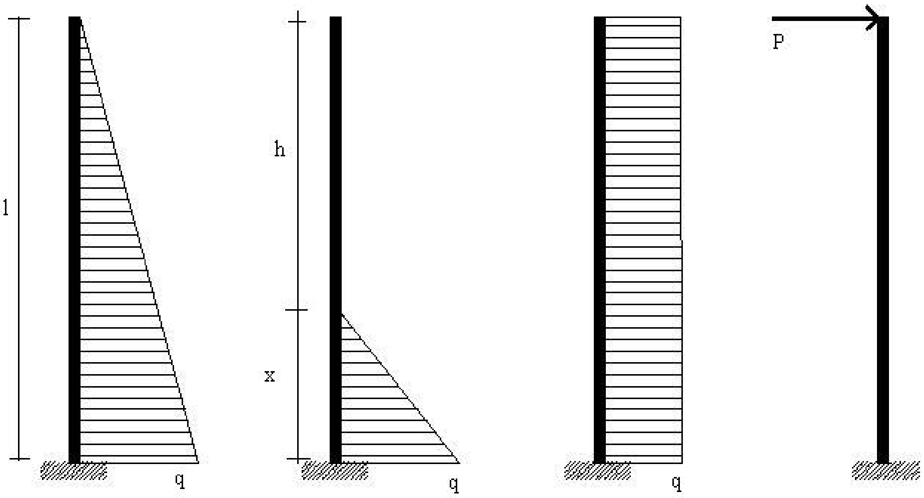
According to that approach, the sheet pile wall is considered as fully clamped at its toe, with the additional condition that the bending moment at the toe is zero. The shear force will be unequal to zero. This shear force is supposed to be the resultant force of the stresses in the vicinity of the toe, including some length beneath.

The clamping of the edge is supposed to be so strong that the displacement and the rotation (first derivative of the displacement) are zero, and even the second derivative is zero, so that the bending moment is zero.

The length of the sheet pile wall will be determined by the conditions of the equilibrium, with active and passive soil stresses, and the condition that the horizontal displacements is zero at the level of the anchor.

The forces acting on the sheet pile wall are the same with previous cases:

- Active and passive soil pressures;
- Hydrostatic water pressures;
- Surcharge of the soil pressures (depending on the platform dimensions);
- Anchor force;



(a) (b) (c) (d)

Figure 7.5: Fixed earth support method-Displacement Method

The displacements on the top are expressed by simple statics and beam theory. Analytically, for the aforementioned loading situations the expressions are:

(a): $w_i = \dfrac{q \cdot l^4}{30 \cdot EI}$

(b): $w_i = \dfrac{q \cdot x^4}{30 \cdot EI} + \dfrac{q \cdot x^3 \cdot h}{24 \cdot EI}$

(c): $w_i = \dfrac{q \cdot l^4}{8 \cdot EI}$

(d): $w_i = \dfrac{P \cdot l^3}{3 \cdot EI}$

The calculations were executed in Maple 10 and the needed embedded depths and the anchor forces are summarised in the table below for each case:

Table 7.3: Results of Fixed Earth Support method (displacement method)

CASE	Anchor Force [kN] (horizontal component)	Embedded depth [m]
D	1257.98	20.16
A1	1043.35	20.01
A2	910.66	20.52
B1	885.10	18.26
B2	698.54	17.98
C1	659.53	16.04
C2	494.04	13.42

7.4 Comparison of the methods

The different results according to the aforementioned methods are summarised in the following graphs. For different dimensions of the relieving platform the equations coming from the trend lines can be used, in order to estimate the embedded depth and the anchor force. An example is also illustrated in the end of the present paragraph.

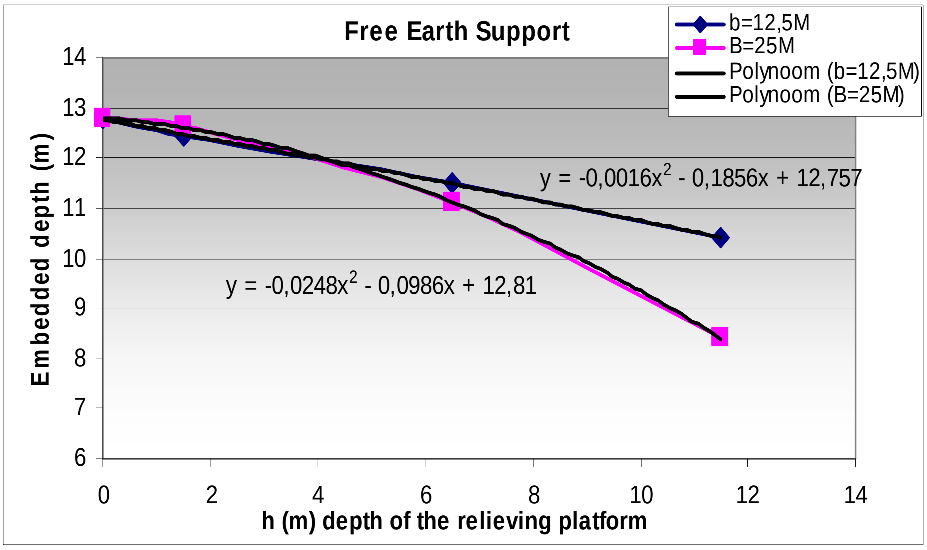


Figure 7.6: Embedded depth for different relieving platforms (Free Earth Support method)

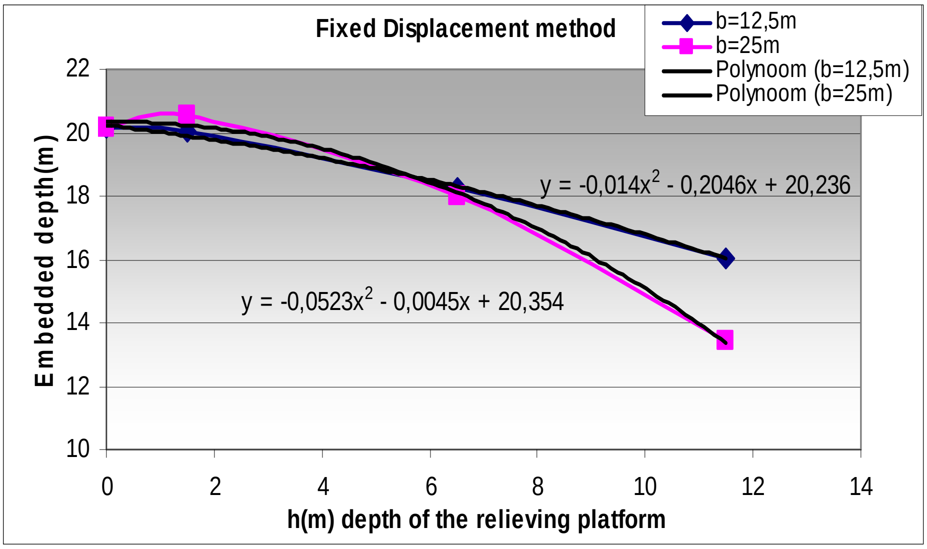


Figure 7.7: Embedded depth for different relieving platforms (Fixed- Displacement method)

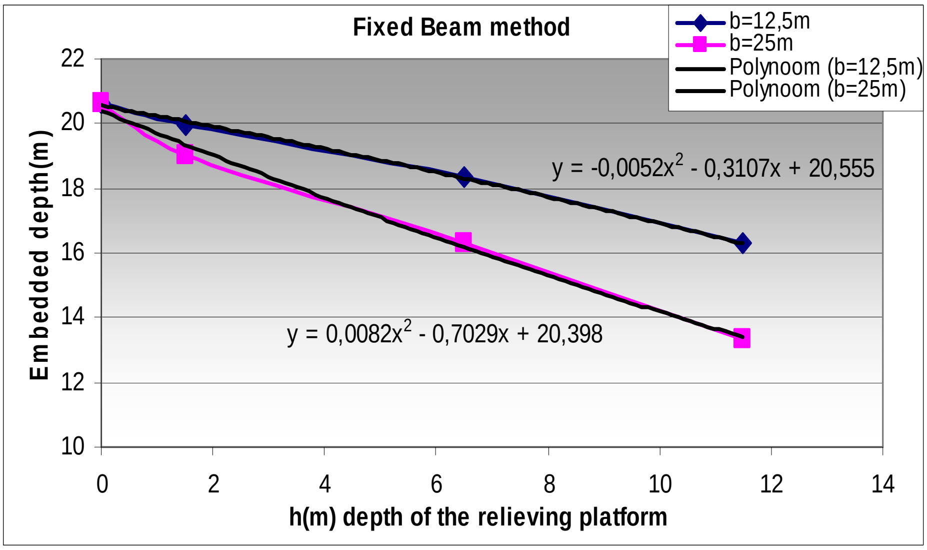


Figure 7.8: Embedded depth for different relieving platforms (Fixed- Beam method)

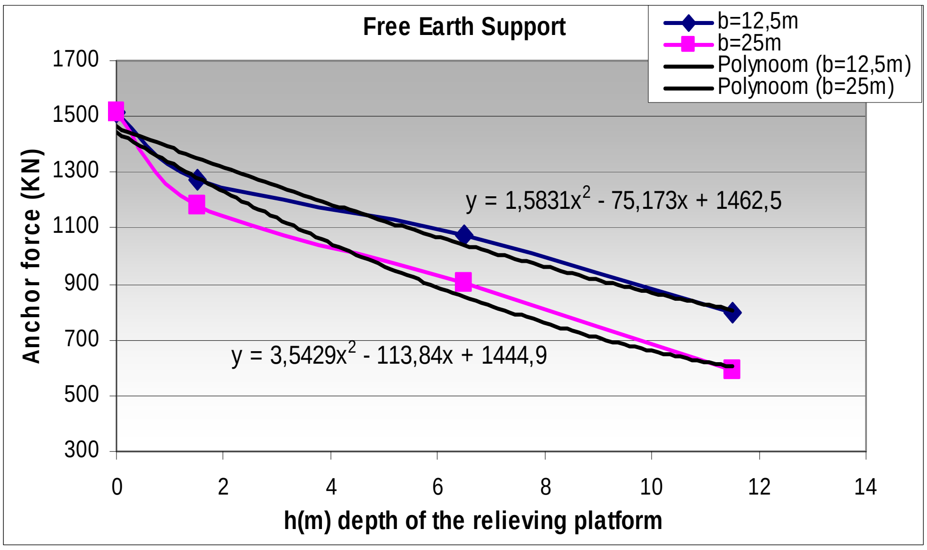


Figure 7.9: Anchor force for different relieving platforms (Free Earth Support method)

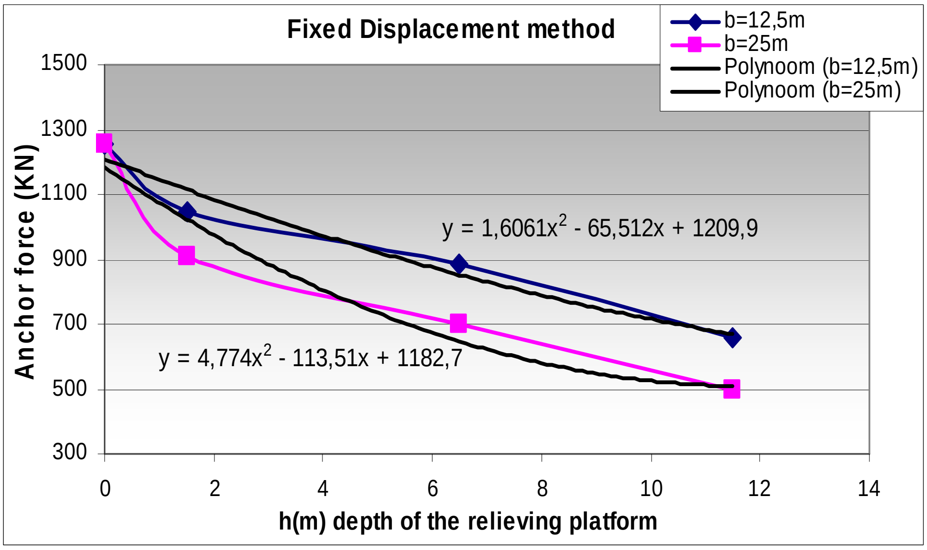


Figure 7.10: Anchor force for different relieving platforms (Fixed- Displacement method)

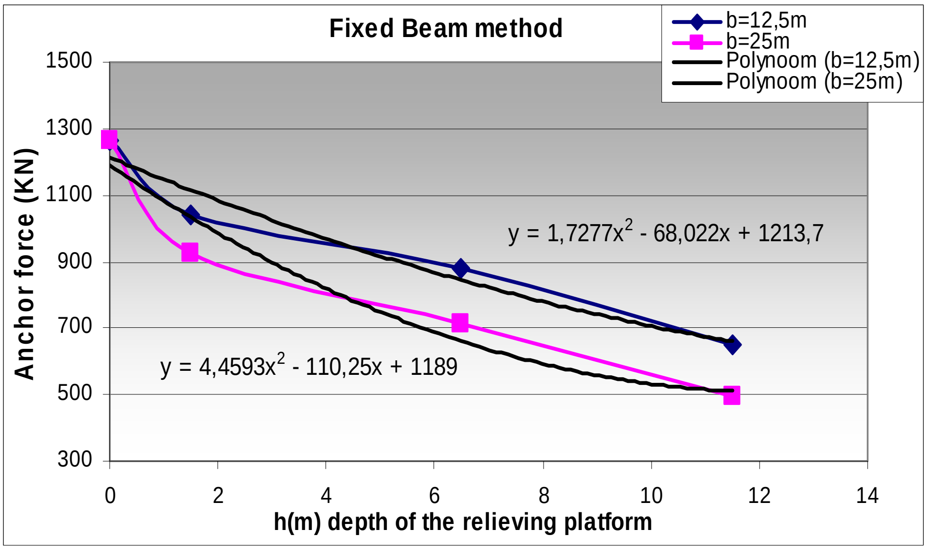


Figure 7.11: Anchor force for different relieving platforms (Fixed- Beam method)

According to the Free Earth Support method the anchor forces are overestimated compared with the values provided for the Fixed Earth Support method, and at the same time the embedded depths are smaller.

As it can be seen from the previous results, the differences between the two variants of the Fixed Earth Support methods are insignificant. In general, a quay wall structure may be based on fixed sheet pile walls for the following reasons:

- The risk of loss of stability due to insufficient passive earth pressures is minimised;
- The bearing function of the sheet pile may be a normative parameter for the determination of the total length of the sheet piles; consequently a longer embedded depth is favourable;
- The ultimate bending moment is reduced thus a lighter profile is required;
- The anchor forces are reduced thus a smaller anchor is required;

In the case that hard layers are present in the substrata high driving risks may occur, thus the choice of a free earth supported sheet pile wall might be preferable.

7.5 Example

In order to make clear the usefulness of the previous graphs an example is illustrated in the followings. Here, a quay wall structure with a retaining height of 30 m exposed in the same loading conditions is assumed and the relieving platform differs from the already investigated cases.

The relieving platform that was chosen for this specific structure has a depth of **h= 8.0 m** and a width equal to **d=19.0 m**

Firstly, the needed embedded depth will be determined as follows. It has to be noted that in the equations of the trend lines that appear in the graphs the variable x is the depth (h) of the relieving platform and the y is in the one case the embedded depth and in the second the anchor force.

- **Embedded depth determination**

<u>Free Earth Support Method</u>

From graph 7.1 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=10.43 m
- For width b=12.5 m and for h=8.0 m → y=11.17 m

With a linear interpolation between these values we get the final required embedded depth for the case of width b=19.0 m.

Consequently we get: **y=10.79 m**

<u>Fixed Earth Support Method (Displacement method)</u>

From graph 7.2 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=16.97 m
- For width b=12.5 m and for h=8.0 m → y=17.70 m

With a linear interpolation between these values we get the final required embedded depth for the case of width b=19.0 m.

Consequently we get: **y=17.32 m**

<u>Fixed Earth Support Method (Equivalent Beam method)</u>

From graph 7.2 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=15.30 m
- For width b=12.5 m and for h=8.0 m → y=17.74 m

With a linear interpolation between these values we get the final required embedded depth for the case of width b=19.0 m.

Consequently we get: **y=16.47 m**

- **Anchor force determination**

Free Earth Support Method

From graph 7.4 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=760.93 kN
- For width b=12.5 m and for h=8.0 m → y=962.43 kN

With a linear interpolation between these values we get the final anchor force for the case of width b=19.0m.
Consequently we get: **y=857.65 kN**

Fixed Earth Support Method (Displacement method)

From graph 7.5 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=580.16 kN
- For width b=12.5 m and for h=8.0 m → y=788.59 kN

With a linear interpolation between these values we get the final anchor force for the case of width b=19.0 m.

Consequently we get: **y=680.21 kN**

Fixed Earth Support Method (Equivalent Beam method)

From graph 7.6 and using the equation of the trend lines for the different widths we get:

- For width b=25.0 m and for h=8.0 m → y=592.40 kN
- For width b=12.5 m and for h=8.0 m → y=780.10 kN

With a linear interpolation between these values we get the final anchor force for the case of width b=19.0 m.

Consequently we get: **y=682.49 kN**

7.6 Preliminary Design of the Sheet Pile Wall

The objective of the calculations on that point is to make a first determination of the optimum dimensions and the quality of the steel for the sheet pile wall on the basis of the calculated distribution of forces.

As it has already mentioned before, from the Fixed Earth method, specifically the Equivalent Beam method, the maximum bending moment and the reactions on the two supports can be determined. This is executed by using the simple beam theory from simple mechanics. By using the frame solver software program (Frame Solver 2D - ESADS) we get the following values, as illustrated in the following table;

Table 7.4: Maximum Bending Moments from Fixed Earth method-Equivalent Beam method

CASE	max. Bending Moment [kNm/m]	Distance [m] (from anchor point)
D	13922.07	18.28
A1	12020.47	17.80
A2	10394.24	17.88
B1	8259.18	15.43
B2	6054.51	14.91
C1	4927.71	12.70
C2	3214.25	11.53

With the above maximum bending moments and taking into consideration the total axial force, (summation of the vertical reaction and the vertical component of the anchor force) the required profiles for the sheet pile wall are determined.

In general for a quay wall like the case considered, due to the high retaining height and the heavy loads, a combined sheet pile system is considered. As it has already mentioned, the intermediate piles provide the soil tightness for the sheet pile wall and transfer the loads from soil and water pressures to the primary elements. In principle triple intermediate piles are used, but in cases where big stiffness is required double intermediate piles can be utilised.

The design values of moments, normal forces and transverse forces are determined by multiplying the results of the sheet pile calculation by a partial load factor 1.30, see also Chapter 4: “Design philosophy and calculation methods”. Each normative cross-section must satisfy the following condition:

$$M_{s;d} \leq M_{r;d} = \frac{M_{r;rep}}{\gamma_m}$$

in which:

$M_{s;d}$: design value of the distribution of forces caused by loads;
$M_{r;d}$: design value of the resistance of the structure;
$M_{r;rep}$: representative value of the reistance of the structure;

γ_m : partial factor for material properties;

Moreover, in order to protect the structure against corrosion, a measure that can be used is an extra allowance on the thickness of the materials. Other usual protection measurements are the coating and the cathodic protection.

The ultimate limit state of a combined wall is verified by using the following expression:

$$\sigma_{yield} \geq \frac{M_{max;d} \cdot e}{I} + \frac{N_d'}{A}$$

in which:

σ_{yield} : design value of yield stress
$M_{max;d}$: design value of the moment to be verified
N_d' : design value of the axial compression force acting on the sheet pile wall
I : moment of inertia of the considered cross-section;
e : eccentricity of the extreme fibre of he considered cross-section
A : area of the considered cross section

7.7 Sheet pile profiles

In general U-shaped and Z-shaped profiles are utilised in a sheet pile wall construction. The major disadvantage of the U-shaped profiles is that bending moments are possible to occur in the longitudinal direction, the so called oblique bending.

On the other side that is not the case for the Z-shaped profile. In contrast they have the disadvantage that the highest bending stresses are consecrated in the adjacent area of the interlocks, pushing them to open.

A combined wall consists of two profiles, in the present case a tubular profile (primary element) and an infill profile (secondary elements). The resistance against the bending moment will be produced mostly by the primary elements of the combined wall, and the infill elements will resist against the hydraulic failure.

The tubular profiles are in most cases longer that the infill profiles, due to the fact that the determination of the infill profiles is based on the hydraulic resistance. The stiffness of the combined wall should be taken into account for the calculations of the sheet pile wall in the upper part, while in the bottom part the stiffness of the tubular profiles only should be considered.

The difference in stiffness between the intermediate elements and the primary ones is so large that probably arching will occur, so that the primary elements receive almost all the internal forces. The intermediate elements in this case are considered as double PU sheet piles as it is illustrated below.

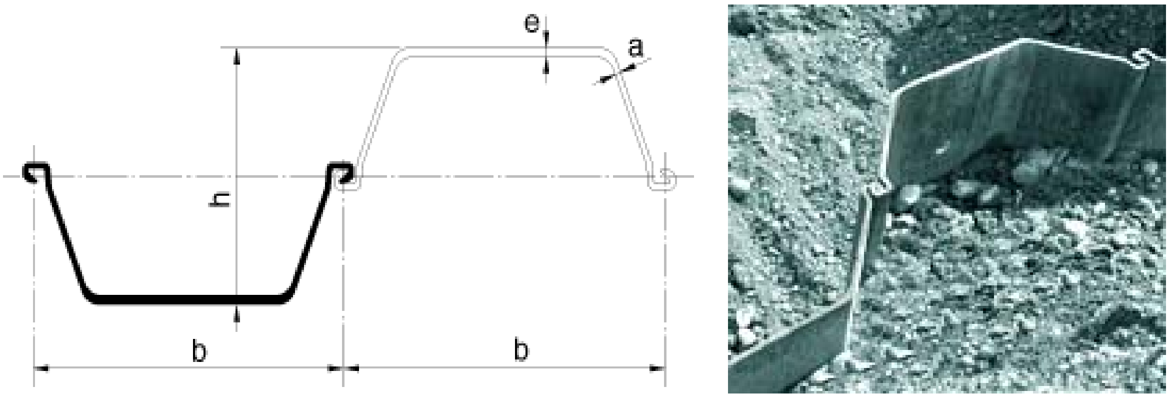


Figure 7.12: Illustration of PU Sheet Piles

The characteristics of the intermediate piles used in the design of the sheet pile wall are summarised in the following table. For simplicity reasons, in each case the same intermediate sheet piles are used, i.e PU 22.

Table 7.5: Characteristics of PU 22 intermediate sheet piles

Intermediate Elements (PU 22)		
Thickness (t_e)	12.1	[mm]
Thickness (t_a)	9.5	[mm]
Width (B)	1200	[mm]
Height (H)	450	[mm]
Total Area (A)	21950	[mm^2]
Section modulus (W)	2.640E+06	[mm^3]
Moment of Inertia (I)	5.936E+08	[mm^4]
Mass (G)	172.3	[kg/m^1]

In this stage, a preliminary determination of the required profiles of the sheet piles will be executed by taking into consideration the acting loads and the characteristics of the cross sections.

The stress of the combined wall will be specified by the aforementioned formula:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A},$$

where (the axial force is the summation of the reaction coming from the analysis of the relieving structure and the vertical component of the anchor). The moment of inertia of the system (I_{syst}) is determined as follows:

$$I_{syst} = \frac{I_{tub} + I_{sh.pile}}{L_{syst}}$$

where the moment of inertia of the sheet pile is defined from tables and the moment of inertia of the tubes can be easily determined by:

$$I_{tub} = \frac{\pi}{64} \cdot \left(D_{ext}^4 - D_{int}^4\right)$$

The section modulus of the combined wall is determined from the following equation:

$$W_{syst} = \frac{I_{syst}}{0.5 \cdot D}$$

The maximum acting bending moment was determined by the Fixed Support method (Equilibrium method) by treating the sheet pile as a simple supported beam, The total axial force (summation of the vertical reaction and vertical component of the anchor force) is estimated below and is noted as ΣF.

As it is shown in the following calculations, the yielding stress is not yet reached, but this extra thickness of the wall can be used for protection against corrosion.

7.8 Dimensioning according to Blum results

Table 7.6: Characteristics of Combi-walls and stress control (Blum analysis)

CASE	Bending Moment [kNm/m]	Total Axial Force [kN/m]	Primary Elements		Steel Quality	Yielding Stress [N/mm^2]	Max. stress [N/mm^2]
			Diameter [mm]	Thickness [mm]			
D	13922.0	2382.2	2500	45	X65	443	395.27
A1	12020.0	2667.9	2420	35	X65	443	419.44
A2	10394.2	2525.7	2120	35	X65	443	429.88
B1	8259.1	3626.9	2320	30	X65	443	386.38
B2	6054.5	2666.4	1920	30	X65	443	358.67
C1	4927.7	5239.6	1920	30	X65	443	358.90
C2	3214.2	3844.0	1420	30	X65	443	345.77

Note: The graphs of the static analysis with “Beam Solver 2D” are presented in the Appendices

7.9 References

[7.1] [CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*, 2005

[7.2] Prof. Arnold Verruijt; *"Soil Mechanics"*, Delft University of Technology, 2001-2006

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[7.5] Arcelor Group; *"Piling Handbook"*, Arcelor RPS

[7.6] ARBED Societe Anonyme; *"Practical Design of Sheet Pile Bulkheads"*, Luxembourg

[7.7] Phd.MSc.Dick Arjen Kort; *"Steel Sheet Pile Walls in Soft Soils"*, Delft University of Technology, 2002

[7.8] *"Steel Sheet Piling – Design Manual"*, U.S.Department of Transportation/FHWA

[7.9] Luc de Lattre; *"A century of design methods for retaining walls – the French point of view"*, Laboratoire Central de Ponts et Chaussees

[7.10] *"Beton Kalender 1984, volume 2"* Giourdas Editions

[7.8] Emiel Meijer; *"Comparative analysis of Design Recommendation for Quay Walls"*, Delft University of Technology/Gemeentewerken Rotterdam, 2006

[7.9] Priscilla Bonte; "*Sandwich Wall as the wall of the future*", Delft University of Technology/Delta Marine Consultants B.V., 2007

CHAPTER 8

PART-A
SHEET PILLE WALL ANALYSIS –
ELASTIC SUPPORTED BEAM METHOD (MSHEET)

8.1 General

In principle, due to its practical applicability the calculation of the sheet pile wall is carried out by the method of the beam supported on the uncoupled elasto-plastic springs. This means that the effect of arch working of the ground, which causes an important reduction of the moment in the span in non-cohesive soil, is not taken into account.

Only with sufficient deformations of the sheet pile wall can plastic strains of the soil and active earth pressures or passive soil resistances occur. If there is no displacement the earth pressure is neutral. That leads to an iterative process; after each calculation step a verification of whether the estimated pressures correspond to the appropriate deformations is made.

According to Handbook of Quay Walls, due to the high distribution capacity of the relieving platform, the average values are used as design values for followings parameters:

- Coefficients of subgrade reaction K_h of the soil;
- Mechanical parameters of the sheet pile wall;
- Spring stiffness of the anchor;

A major difference between the beam on elastic foundation method and the aforementioned Blum method, is that here it is possible to calculate a sequence of phases in which the stress history is of the sheet pile wall is used as initial conditions for the next phase. However, as it happens with all the engineering problems, the reliability of the results should be treated with scepticism, since the schematisation of the soil and the interface between soil and structure is far from being perfect.

The inclination of the sheet piles and the eccentricity of the saddle in the top of it are issues that should be considered from the designers. The effect of inclination on the earth pressures can be taken into account with adjusted earth pressure coefficients. In addition the second order moments that are induced from the axial load might also calculated and included into the results. In the following figure the moment distribution of the sheet piles is illustrated:

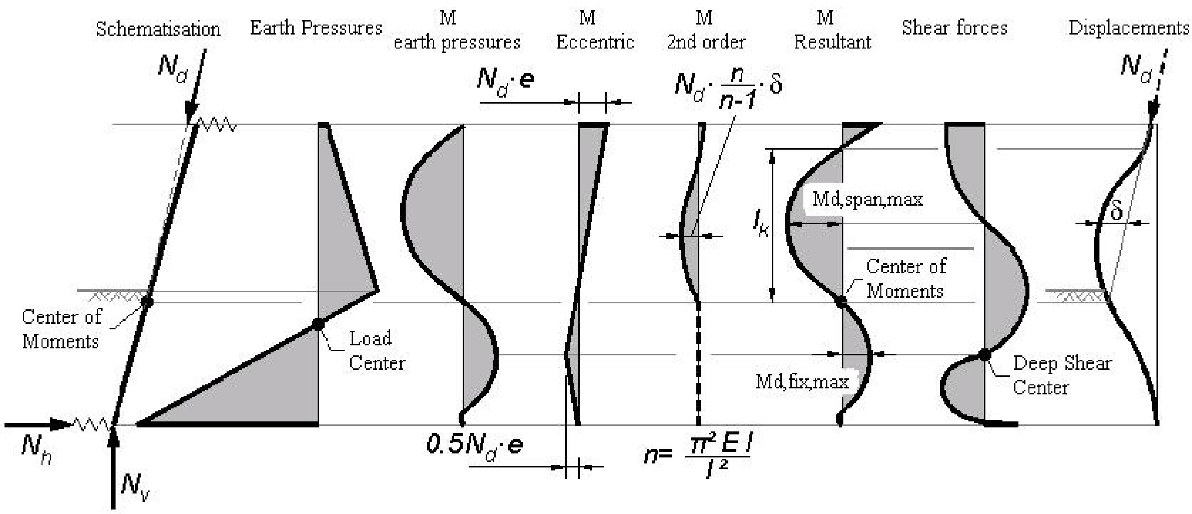

Figure 8.1: Principle of the moment distribution of the sheet pile wall

In the present study the sheet pile wall is considered vertical, so the favourable effect of the sheet pile inclination is not considered. Moreover, it is clear that the eccentricity of the axial load reduces the maximum bending, while second order moments will act unfavourably for the sheet pile wall stability. For simplicity reason, second order effects are taking into account.

The mechanical parameters of the soil are summarised in the following tables:

Table 8.1: Design values for soil mechanical parameters of soil profile 1 (sand q_c:15MPa)

Soil parameter	Design Value	
Sand (moderate)		
Volumetric weight of dry soil “γ_d”	18	[kN/m^3]
Volumetric weight of saturated soil “γ_{sat}”	20	[kN/m^3]
Friction angle between soil and wall “δ”	20	[^{o}]
Angle of internal friction “φ”	30	[^{o}]
Cohesion	0	[kPa]

Table 8.2: Design values for soil mechanical parameters of soil profile 2 (sand q_c:15MPa –clay c_u:50KPa)

Soil parameter	Design Value	
Clay (moderate)		
Volumetric weight of dry soil “γ_d”	17	[kN/m^3]
Volumetric weight of saturated soil “γ_{sat}”	17	[kN/m^3]
Friction angle between soil and wall “δ”	11	[^{o}]
Angle of internal friction “φ”	17.5	[^{o}]
Cohesion	5	[kPa]

*The same mechanical parameters of table 8.1 stands for the sand part in soil profile 2

The average coefficients of subgrade reactions which are proposed in CUR211 are used in this stage.

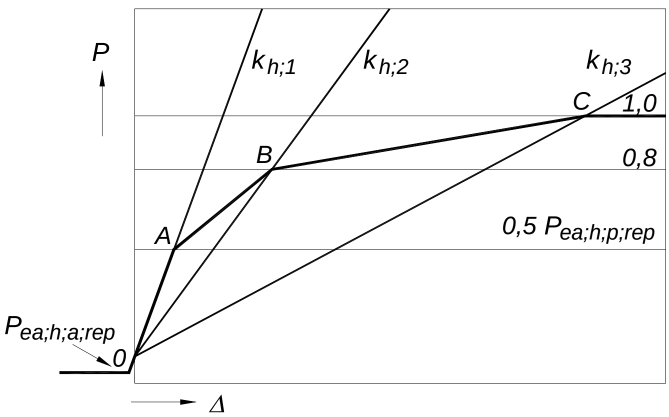

Figure 8.2: Construction of the deformation-dependent coefficient of subgrade reaction

Table 8.3: Average values of the horizontal coefficients of subgrade reaction with increased stress

Soil Type	K_{h1} [kN/m^3] $p_0<p_h<0.5\ p_{ea;h;p;rep}$	K_{h2} [kN/m^3] $p_0<p_h<0.8\ p_{ea;h;p;rep}$	K_{h3} [kN/m^3] $p_0<p_h<1.0\ p_{ea;h;p;rep}$
Sand (q_c:15MPa)	20000	10000	5000
Clay (f_{und}: 50 KPa)	4000	2000	800

When this approach is used, in contrast with method of Blum, even for piles exceeding the minimum length no fully fixed sheet pile wall is found. In the present study, and according to the philosophy of Handbook of Quay Walls, the toe of the sheet piling is assumed fully fixed. For that purpose and in addition for the satisfaction of the vertical capacity of the sheet piling, the maximum embedded depth that is estimated in the previous chapter is used.

8.2 Elastic Supported Beam Method (MSheet design)

The design values of the earth pressures and soil resistances in the ultimate limit state (ULS), which are caused by effective pressure and pore water pressure are determined by using a sheet pile calculation that is based on the design values of the soil properties. As it has already mentioned, according to Handbook of Quay Walls, the design is carried out with the representative values since the partial factors for the mechanical parameters of soil is 1,0.

The influence of surcharge on the soil pressures is calculated by using the design value of the surcharge. This value is considered equal to the representative value of the surcharge that is equal to the extreme site load given in terms of reference. The estimated values of the terrain load as it is shown in Chapter 5: "Hand Calculations – General" is not representative of an extreme site load. A uniform terrain load of 60 kN/m^2 is assumed as a surcharge in order to consider in that way a partial factor.

As it has already mentioned, the connection point of the relieving platform and the sheet piles according to Handbook of Quay Walls is a hinge. Despite this philosophy, the connection point can be accomplished in various ways; a fixed moment connection is also possible. In this case, the detailing in the connection is very demanded and difficult to get realised. This is because the internal distribution of forces is strongly depended on the deformation of the quay system. Thus the hinge connection is more realistic and it can be achieved with the aid of a cast steel saddle.

The maximum design value of the vertical reaction coming from the static analysis of the relieving platform, increased by the vertical component of the design value of the anchor force, is imposed as external loads in this phase.

The yielding stress of the combined wall will be specified by the aforementioned formula (for explanations see previous Chapter):

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A}$$

The results of the MSheet analyses and the cross-section check are presented in the followings.

8.3 Results of MSheet analysis

8.3.1 Sheet pile design for soil profile 1 (uniform sand)

Table 8.4: Characteristics of Combi-walls and stress control for soil profile 1 (MSheet analysis)

CASE	Bending Moment [kNm/m]	Total Axial Force [kN/m]	Primary Elements		Steel Quality	Yielding Stress [N/mm^2]	Max. stress [N/mm^2]
			Diameter [mm]	Thickness [mm]			
D	13214.1	3443.5	2320	45	X65	443	390.48
A1	11584.8	3372.8	2220	40	X65	443	419.71
A2	9115.6	2812.1	2120	35	X65	443	391.80
B1	8624.8	4138.2	2220	35	X65	443	377.63
B2	5585.0	2938.7	1820	30	X65	443	367.04
C1	4812.6	5666.5	1720	35	X65	443	358.85
C2	1769.8	3853.0	1220	25	X65	443	331.02

8.3.2 Sheet pile design for soil profile (clay – sand)

Table 8.5: Characteristics of Combi-walls and stress control for soil profile 2 (MSheet analysis)

CASE	Bending Moment [kNm/m]	Total Axial Force [kN/m]	Primary Elements		Steel Quality	Yielding Stress [N/mm^2]	Max. stress [N/mm^2]
			Diameter [mm]	Thickness [mm]			
D	12489.5	3555.9	2220	45	X65	443	395.19
A1	11669.3	3451.1	2220	40	X65	443	414.44
A2	8618.7	2740.4	2020	35	X65	443	395.59
B1	8988.8	4325.8	2220	35	X65	443	393.81
B2	6113.6	2944.8	1820	30	X65	443	395.34
C1	5478.6	6157.1	1920	35	X65	443	356.32
C2	3451.3	4065.5	1420	30	X65	443	371.47

8.4 References

[8.1] [CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*, 2005

[8.2] Msheet manual; *"Manual – Reference Section"*

[8.3] Prof. Arnold Verruijt; *"Soil Mechanics"*, Delft University of Technology, 2001-2006

[8.4] Emiel Meijer; "*Comparative analysis of Design Recommendation for Quay Walls"*, Delft University of Technology/Gemeentewerken Rotterdam

[8.5] Priscilla Bonte; "*Sandwich Wall as the wall of the future*", Delft University of Technology/Delta Marine Consultants B.V.

[8.6] *"Archive of Gemeentewerken Rotterdam*", Rotterdam

CHAPTER 9

PART-B
SHEET PILLE WALL ANALYSIS –PLAXIS
(FINITE ELEMENT CODE FOR SOIL AND ROCK ANALYSIS)

9.1 General

Finite elements analysis is based on a model in which the behaviour of soil and structure is integrated. The mechanical parameters of soil are introduced by means of stress deformation relations; thus the method is very sensitive in the correct choice of the modulus of elasticity of the soil. With this method, fundamental calculations of stresses and deformations of soil and structure are carried out; and according to the stiffness of each element the internal forces are determined.

PLAXIS 2D is a two dimensional finite element computer program used to perform deformation and stability analyses for various types of geotechnical applications.

In addition to Mohr – Coulomb model, which is a simple and well known non-linear model, the Hardening Soil Model is used for the present analysis. This is an advanced elasto-plastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. This second order model can be used to simulate the behaviour of sand and gravel as well as softer types of soil such as clays and silts.

For further information about the scientific background of the model, the reader is referred to the PLAXIS scientific manual.

9.2 Mechanical parameters and model description

The used mechanical parameters of the soil are summarised in the following table:

Table 9.1: Design values for soil mechanical parameters of soil profile 1 (sand))

Soil parameter	**Design Value**	
Sand (moderate)		
Volumetric weight of dry soil “γ_d”	18	[kN/m^3]
Volumetric weight of saturated soil “γ_{sat}”	20	[kN/m^3]
Friction angle between soil and wall “δ”	20	[^{o}]
Angle of internal friction “φ”	30	[^{o}]
Dilatancy angle	2	[^{o}]
Plastic straining due to primary loading	15	[MPa]
Plastic straining due to primary compression	15	[MPa]
Elastic unloading/reloading	45	[MPa]

Table 9.2: Design values for soil mechanical parameters of soil profile 2 (clay – sand)

Soil parameter	Design Value	
Clay (moderate)		
Volumetric weight of dry soil "γ_d"	17	[kN/m^3]
Volumetric weight of saturated soil "γ_{sat}"	17	[kN/m^3]
Friction angle between soil and wall "δ"	11	[^{o}]
Angle of internal friction "φ"	17.5	[^{o}]
Cohesion	5	[kPa]
Plastic straining due to primary loading	2	[MPa]
Plastic straining due to primary compression	2 [	[MPa]
Elastic unloading/reloading	6	[MPa]

*The same mechanical parameters of table 9.1 stands for the sand part in soil profile 2

It has to be noted that the moduli of elasticity of the soils which are proposed from NEN 6740:2006 is used. It is known that the modulus of elasticity of a sand layer is difficult to predict and the values given from different codes have a significant variation. From experience and for safety reasons, the modulus of elasticity of sand which is proposed for loose sand is used, instead of the one of the moderate sand.

Each model is based on the same basic assumptions. As it has already mentioned the connection point of the sheet piles and the relieving platform, according to Handbook of Quay Walls is a hinge. In order to simulate that hinge in PLAXIS an imaginary beam was entered on the base of the relieving platform. The stiffness of the beam is very small so that it follows completely the deformations of the relieving platform. Interfaces haven't been added between the beam and the superstructure so that they cooperate. The position of the imaginary beam is illustrated in the following figure:

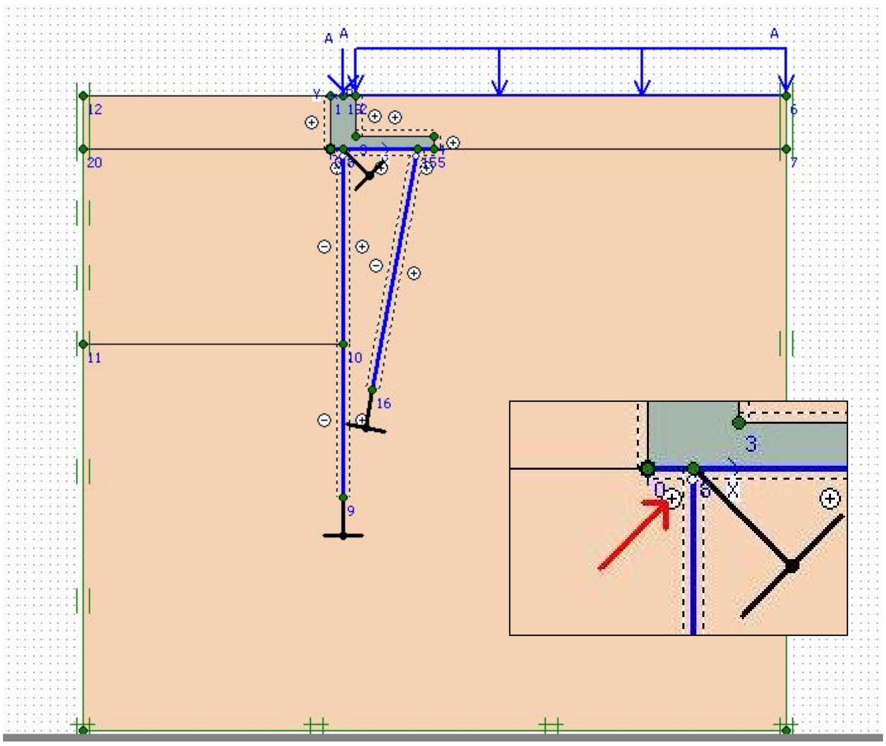

Figure 9.1: Model simulation (case B1)

The toe level in the sheet pile wall is the same with previous cases, consequently a fixed end is assumed. The toe is simulated in the program by a very stiff anchor, i.e. EA=3*10^5 kN. For the simulation of the bearing piles a smaller stiffness is applied, i.e. EA=1*10^5 kN.

From the static analysis of the relieving structure the two normative load combinations were determined. These are the loading cases LCA1 and LCA2; one for the axial load and the other for the anchor force. There two load combinations are investigated in PLAXIS and the extreme values of internal forces are used for the cross-section check.

The ultimate limit state of a combined wall, as before, is verified by using the following expression:

$$\sigma_{yield} \geq \frac{M_{max;d} \cdot e}{I} + \frac{N_d'}{A}$$

The extra thickness of the wall can be used for protection against corrosion. The same intermediate sheet piles are used also here; thus double PU 22.

The results of the PLAXIS analyses and the cross-section check for both soil profiles are presented in the followings. The deformed mesh as well as the shear forces and the horizontal displacements for each case are presented in the appendices.

9.3 Results from PLAXIS analysis

9.3.1 Sheet pile design for soil profile 1 (uniform sand)

Table 9.3: Characteristics of Combi-walls and stress control for soil profile 1 (PLAXIS analysis)

CASE	Bending Moment [kNm/m]	Total Axial Force [kN/m]	Primary Elements		Steel Quality	Yielding Stress [N/mm^2]	Max. stress [N/mm^2]
			Diameter [mm]	Thickness [mm]			
D	11300.0	3180.0	2220	40	X65	443	398.09
A1	8400.0	2480.0	1820	35	X65	443	407.85
A2	7920.0	1910.0	1820	35	X65	443	403.85
B1	7560.0	2930.0	1820	35	X65	443	408.01
B2	5820.0	3610.0	1620	35	X65	443	395.02
C1	4860.0	3870.0	1520	35	X65	443	364.60
C2	3980.0	4210.0	1420	35	X65	443	360.09

9.3.2 Sheet pile design for soil profile (clay – sand)

Table 9.4: Characteristics of Combi-walls and stress control for soil profile 1 (PLAXIS analysis)

CASE	Bending Moment [kNm/m]	Total Axial Force [kN/m]	Primary Elements		Steel Quality	Yielding Stress [N/mm^2]	Max. stress [N/mm^2]
			Diameter [mm]	Thickness [mm]			
D	14940.0	2980.0	2320	45	X65	443	428.34
A1	9670.0	2950.0	2020	40	X65	443	389.18
A2	9360.0	3430.0	1920	40	X65	443	412.45
B1	8610.0	3590.0	1920	40	X65	443	386.99
B2	6800.0	3850.0	1820	35	X65	443	391.70
C1	4720.0	4570.0	1620	35	X65	443	355.54
C2	4090.0	4690.0	1620	30	X65	443	340.09

9.4 Supplementary Works

Drainage systems are used in order to reduce the excess pore pressure and lower the phreatic level in the landward area. In addition, drainage is also applied to consolidate compressible earth layers, in combination with load increments by site loads.

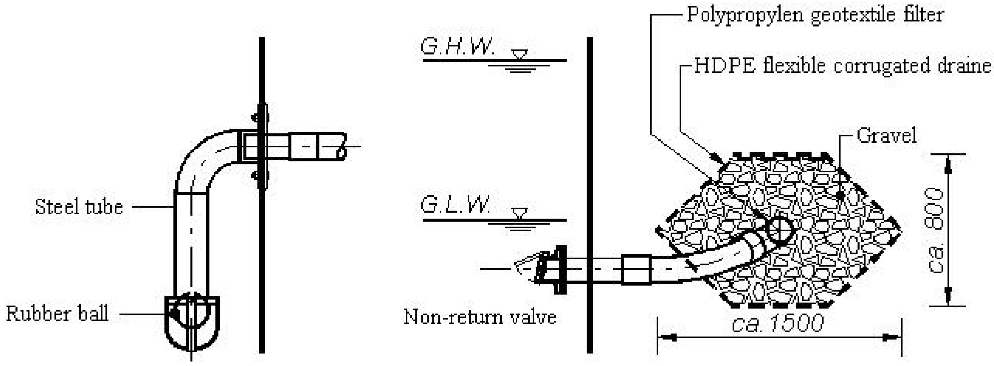

Figure 9.2: Drainage systems with non return valves

In principle, drainage systems drain off precipitation and restrict excessive water pressure behind the quay wall. Especially in tidal areas, this concept should be well considered as well the maintenance of the system during the lifetime of the structure. The functioning of the drainage system can be threatened due to settlements, silting or blocking of the non-return valves.

Moreover, if the quay is backfilled on weak compressible soil, like clay, initially high excess pore pressures will develop, followed by big settlements. These pore pressures lead to very high pressures on the sheet pile wall, which will be dissipated only after a long period of time. The installation of vertical drains can reduce the pore pressures and at the same time the settlement development will be accelerated to such a degree that the area can be used in a short time. Vertical drains can be made as sand or synthetical drains, and they are normally installed in a triangular grid with mutual distances of about 2.5 m

Figure 9.3: Vertical drainage systems

Case B2 for the second soil profile, where clay is present was also investigated in PLAXIS considering and undrained behaviour of clay. The terrain load is equal to 60 kN/m^2 and as it can be seen form the results the maximum value of the excess pore pressures in the area just above the relieving platform is equal to -80 kN/m^2 and in the adjacent backward area of the quay wall is bout -60 kN/m^2, in the same magnitude of the surcharge.

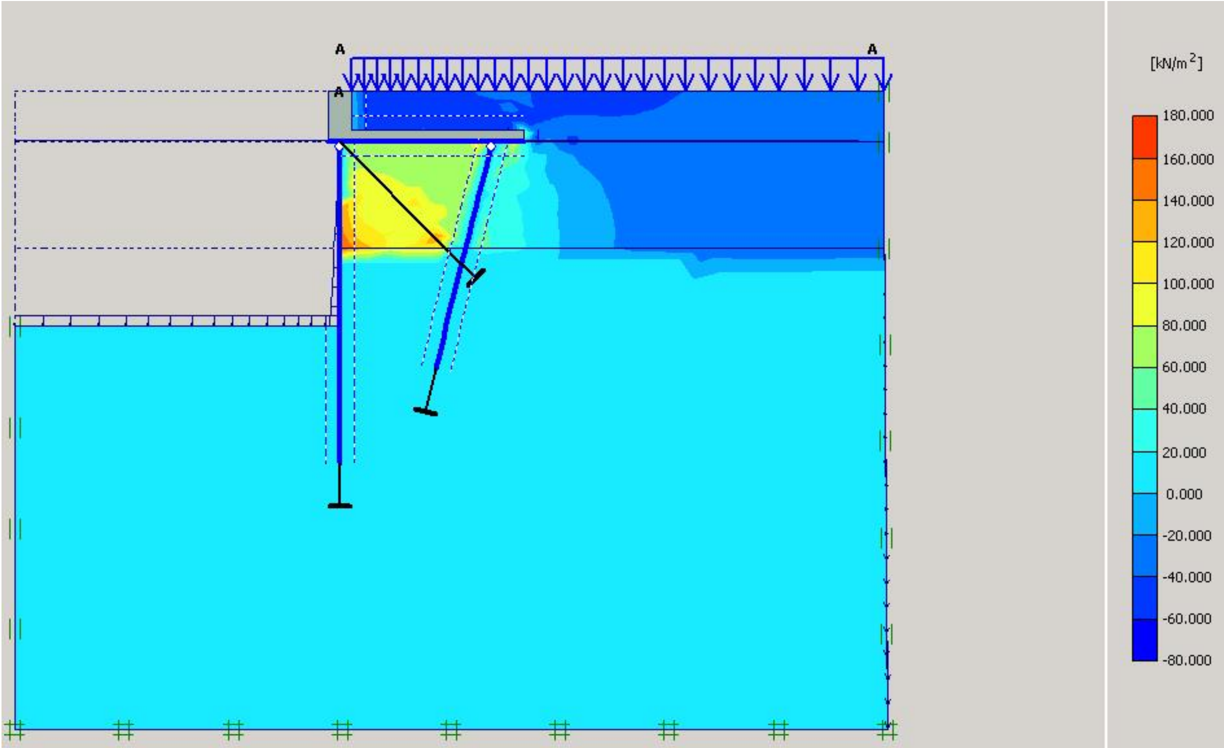

Figure 9.4: Excess pore pressure PLAXIS analysis

9.5 References

[9.1] [CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*

[9.2] PLAXIS manual; *"PLAXIS, Finite Element Code for Soil and Rock Analysis, version 8"*

[9.3] Prof. Arnold Verruijt; *"Soil Mechanics"*, Delft University of Technology

[9.4] Karl Terzaghi, Ralph Peck, Gholamreza Mesri; *"Soil Mechanics in Engineering Practice"*

[9.5] Dr. Garth N. Wells; *"The Finite Element Method: An Introduction"*, Delft University of Technology

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[9.5] "*Archive of Gemeentewerken Rotterdam*", Rotterdam

CHAPTER 10

COST ESTIMATION

10.1 General

In principle, a financial assessment of the structure should be carried out in order to give to the owner the opportunity to choose one of the alternative designs. The financial assessment should provide insight into the total investment costs, operational costs, and maintenance and demolition costs.

Various methods have been developed in order to determine the total costs of the projects. CROW has developed a cost estimate system for the civil engineering projects, and at the same time many organisations have collected dimensionless parameters that are based on data of the costs of previous projects. Both the methods are valuable and useful, and which method is chosen depends on the phase of the project and the associated uncertainty and required specifications.

In the present study a rough estimation of the total cost is executed in order to get an insight of the magnitude and how that varies relative to the different relieving platform's dimensions. For this purpose, dimensionless parameters based on already executed projects from Public Works of Rotterdam are used.

According to the literature some indices are available based on which an estimation of the construction costs can be made. During the planning phase and the development of alternatives, the construction cost can be determined, permitting a rough comparison between the different designs. In the Netherlands the cost per retaining height [€/m] are as follows:

Table 10.1: Costs in relation to retaining height

Retaining Height [m]	Cost per Retaining Height [€/m]
5-10	350-650
10-20	650-1000
20-30	1000-1300

The previous table does not include the costs of engineering, bottom protection, fendering and dredging in front of the quay wall. Consequently, for a quay wall of 30 m retaining height, in accordance to the previous table an estimation of the total cost is estimated to 30.000 € – 39.000 € per longitudinal meter.

10.2 Total cost components

In the present cost estimation bottom protection, fendering, dredging and supplementary costs are not included. As it is presented in the following graph the total cost is a summation of the material cost, the excavation and refill cost and the dewatering cost. Analytical description of the cost analysis for each component is found in the Appendices, where the used excel files are presented.

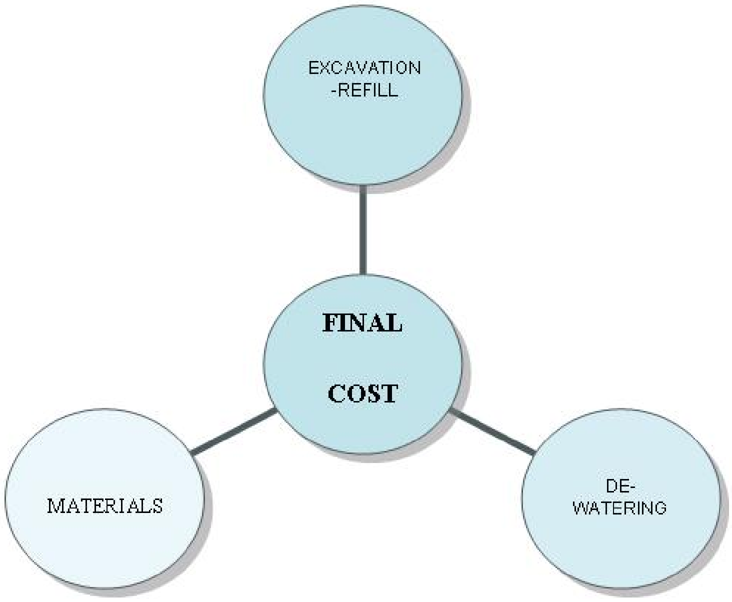


Figure 10.1: Components of total cost estimation

10.2.1 Material Costs

In the used values costs for the use of equipment, production and formwork are included. Analytically the used values are presented in the following table:

Table 10.2: Material costs

Material		
Sheet pile Wall	1031.0	[€/ton]
Relieving platform	345.0	[€/m^3]
Anchors	200.0	[€/m]
Bearing piles	240.0	[€/m^3]

- **Sheet Pile Wall**

As it has already mentioned, for quay walls with high retaining height that must bear heavy loads, big structures that consist of combined walls are needed. In the present case, tubular piles with intermediate double sheet piles are used. This construction system is economically attractive, since the open tubular pile can easily vibrated or driven through sand layers.

Differences in the dimension of the relieving platform affect significantly the total length and the cross-section profile of the sheet pile wall; thus the final cost of the element. The cost of the sheet pile wall relative to different dimensions of the superstructure, as it is carried out from the different calculation methods and different soil profiles, is presented below.

Blum method

As it can be seen from the following graph, the effect of the width of the relieving platform on the cost of the sheet pile is more intent in higher depths.

It is remarkable that in the case that the relieving platform is placed just in the surface the cost differences are not big enough. That is because in that case, the reduction is more or less limited to the surcharge consisting of crane, storage and traffic loads.

Table 10.3: Costs and percentage of reduction – Blum method – Soil profile 1

CASE	Thousands [€ /m]	Percentage of reduction [%]
D	20.8	0.0
A1	16.1	14.2
A2	15.7	16.5
B1	12.9	34.7
B2	11.7	37.8
C1	8.4	55.2
C2	7.1	62.3

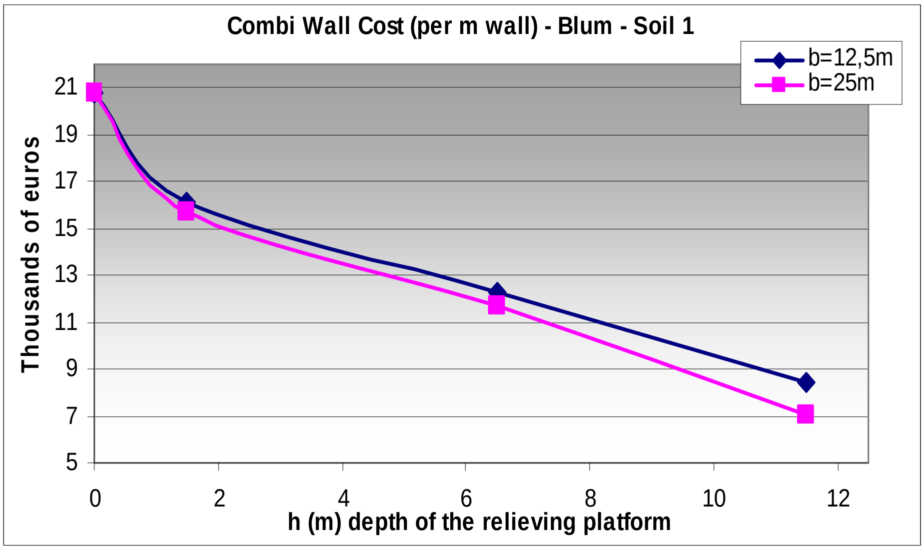


Figure 10.2: Combi wall cost – Blum method

In the following graph, the previous reduction is translated into percentages. The reference point for that comparative analysis is the most expensive case, i.e. CASE D, where the relieving platform is absent.

The maximum reduction that can be achieved by placing a deep and long relieving platform is more or less 63%, comparing to the case of a single sheet pile wall.

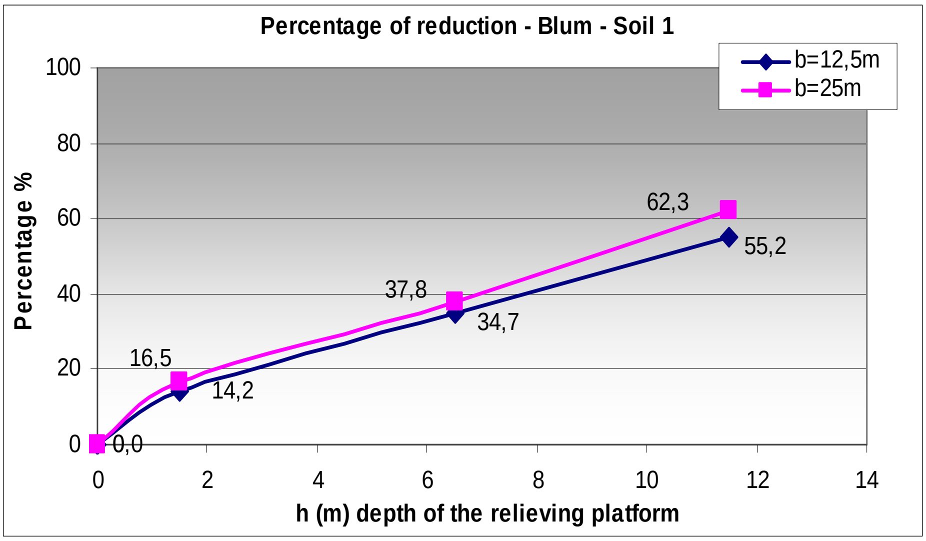


Figure 10.3: Combi wall cost reduction– Blum method

Elastic Supported Beam method (MSheet)
With this method two different soil profiles are investigated. It can be conclude as it comes out from the graphs and tables below, that the presence of a big clay layer leads to more expensive solutions.

In addition, one can say that the percentage of reduction relative to the different dimensions of the relieving platform is approximately at the same levels in both soil profiles.

Table 10.4: Costs and percentage of reduction – MSheet method – Soil profile 1and 2

CASE	Soil Profile 1		Soil Profile 2	
	Thousands [€ /m]	Percentage of reduction [%]	Thousands [€ /m]	Percentage of reduction [%]
D	20.5	0.0	20.2	0.0
A1	17.2	15.8	17.7	12.4
A2	15.7	23.4	15.5	23.4
B1	13.8	32.4	13.8	31.7
B2	11.6	43.5	11.6	42.9
C1	9.3	54.6	9.5	52.9
C2	6.0	70.6	7.1	65.0

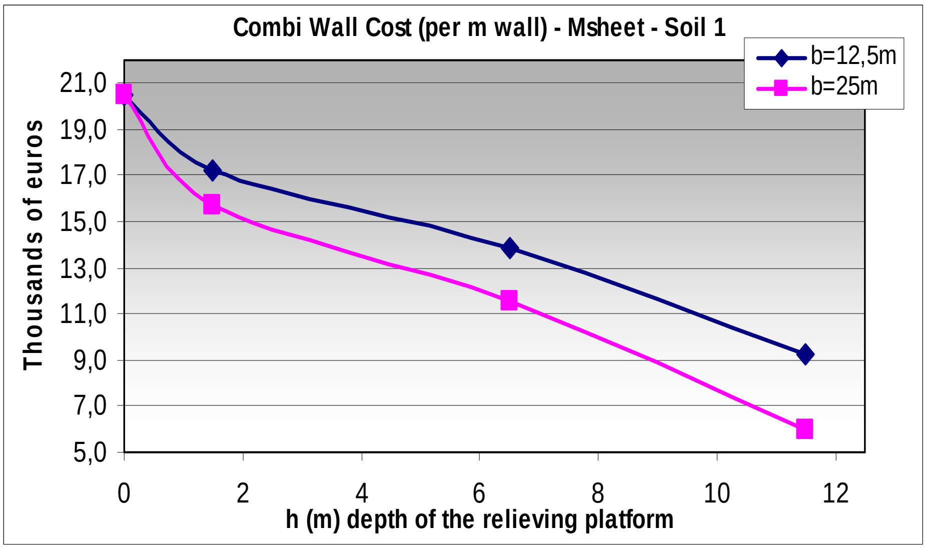


Figure 10.4: Combi wall cost – MSheet method – Soil profile 1

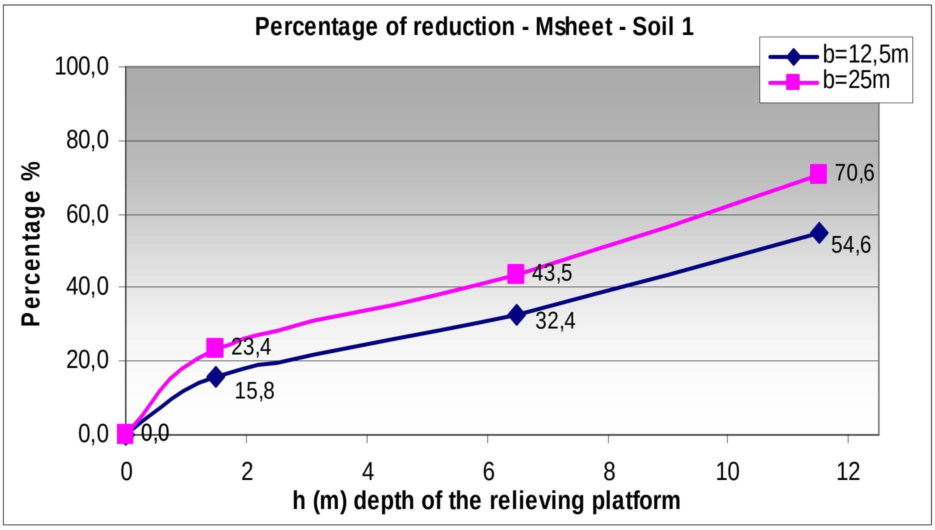


Figure 10.5: Combi wall cost reduction– MSheet method – Soil profile 1

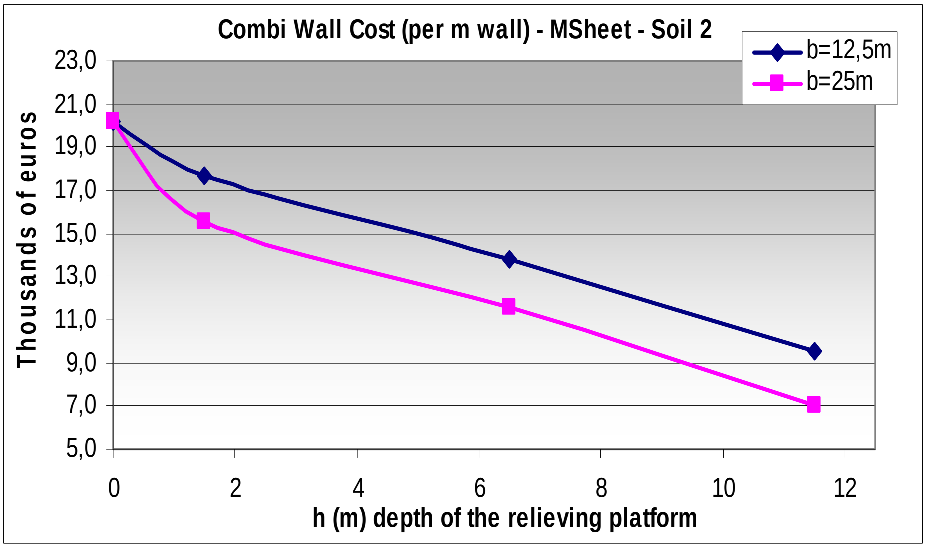


Figure 10.6: Combi wall cost – MSheet method – Soil profile 2

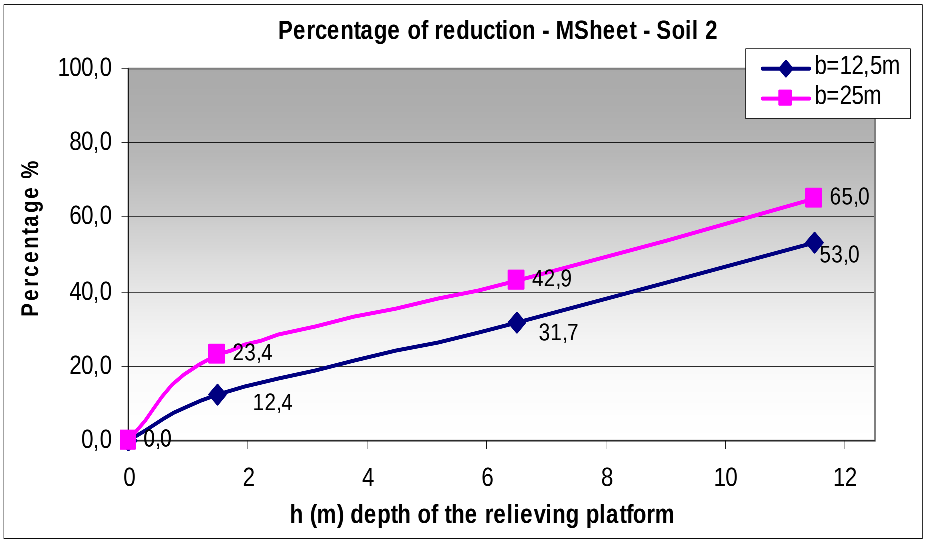


Figure 10.7: Combi wall cost reduction– MSheet method – Soil profile 2

<u>Finite Element Method (PLAXIS)</u>

With this method the two different soil profiles (Soil Profile 1 and 2) are also investigated and the results are presented below:

Table 10.5: Costs and percentage of reduction – PLAXIS method – Soil profile 1and 2

CASE	Soil Profile 1		Soil Profile 2	
	Thousands [€ /m]	Percentage of reduction [%]	Thousands [€ /m]	Percentage of reduction [%]
D	18.3	0.0	20.5	0.0
A1	15.3	16.2	17.3	15.3
A2	15.1	17.2	17.1	16.3
B1	13.2	27.7	15.0	26.9
B2	12.7	30.5	13.1	36.1
C1	9.0	50.8	9.1	55.3
C2	7.9	56.5	7.3	64.4

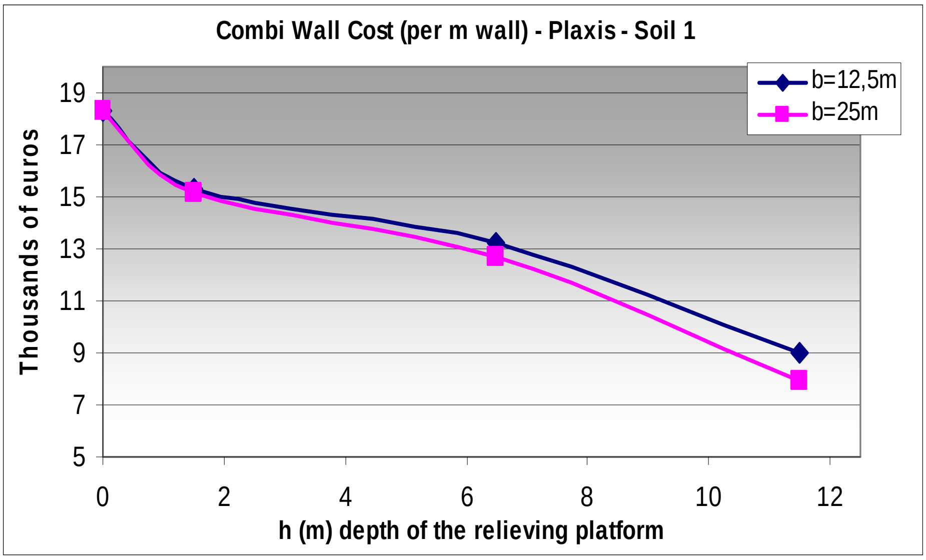


Figure 10.8: Combi wall cost – PLAXIS method – Soil profile 1

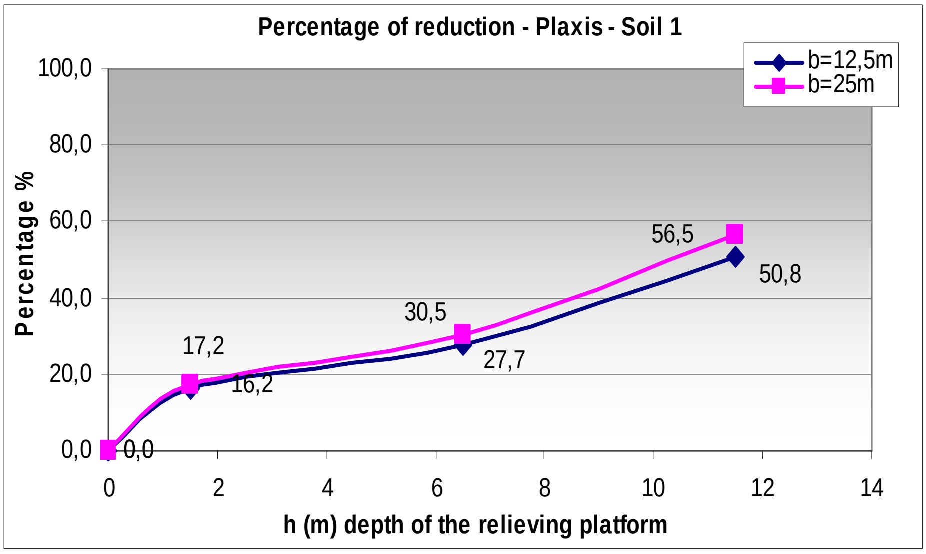


Figure 10.9: Combi wall cost reduction– PLAXIS method – Soil profile 1

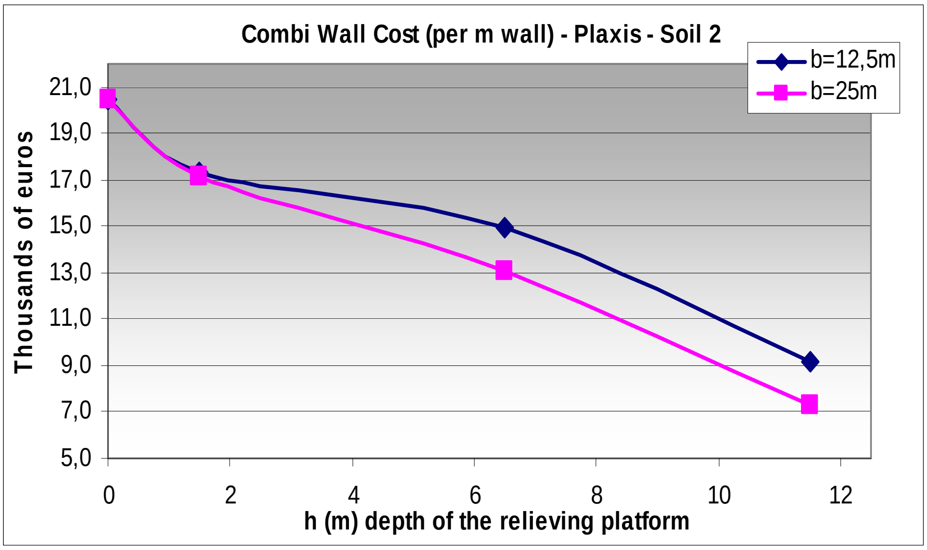


Figure 10.10: Combi wall cost – PLAXIS method – Soil profile 2

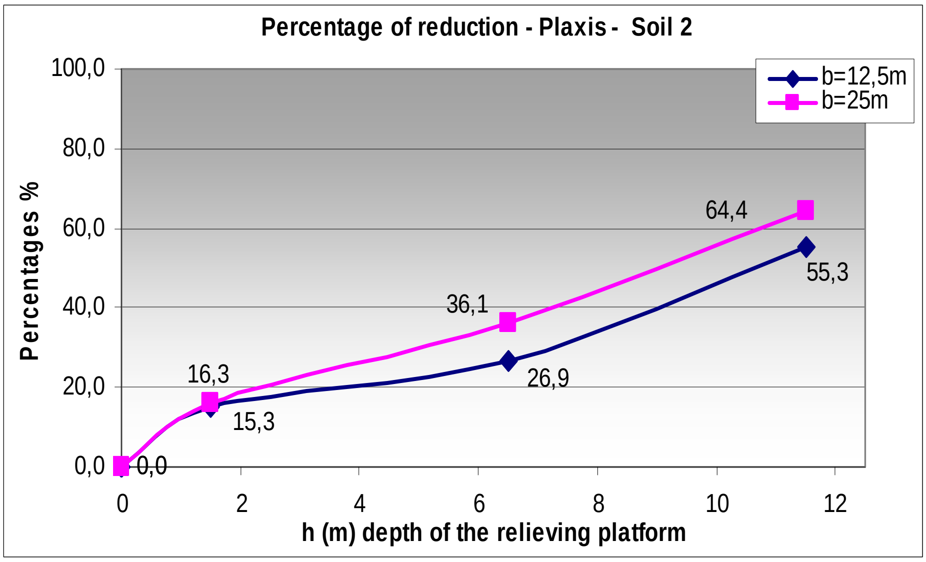


Figure 10.11: Combi wall cost reduction– PLAXIS method – Soil profile 2

- **Relieving platform**

The cost of the relieving platform depends on the volume of the superstructure and it is clear that it increases relating to the width and the depth. In the following figure a rough approximation of the superstructure cost is presented.

Further details for the cost estimation can be found in the relative excel file which is found in the appendices.

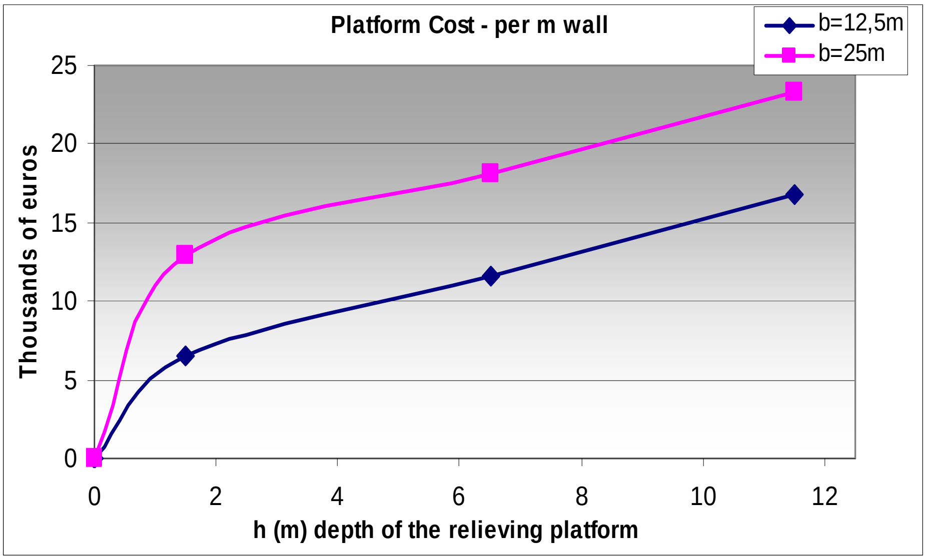


Figure 10.12: Relieving platform cost

- **Anchors**

Since the same cross-section profile of MV – pile is applied in each case; the anchor cost is influenced by the mutual distances and the length of the element. In the present study, a smaller anchor length is applied for the cases C1 and C2, where a deep excavation is executed. It can be noted that in a more detailed design smaller lengths and bigger mutual distances could be also applied, but the differences in the total cost of the quay wall is not significant. Details for the cost estimation process can be found in the excel file in Appendices.

- **Bearing Piles**

In the cases where a long relieving platform is present more than one row of bearing pile are used. In order to take this fact into account, for the cases where the length of the platform is 25m, the volume of concrete was increased by a factor of 2, which is a realistic hypothesis. For further details elative to the cost estimation of the foundation piles, the reader is referred to the Appendices.

10.2.2 Excavation – Refill Costs

The excavation and refill process differ for the two different soil profiles. In the first case of uniform sand subsoil, the same amount of soil will be used in order to fill the area behind the relieving platform. In the second soil substrata, the excavation area will be filled with sand which will not be provided in the field. Consequently a higher cost value is used in the refill phase of the second soil profile, including at the same time labour and purchasing costs.

The total excavation area is formed in such a way that stable slopes are constructed, and is illustrated in the following figure marked in brown.

For the refill phase the area (2) was excluded since it constitutes a part of the future dredged area. And the areas above the relieving platform is included in the calculations are marked in red.

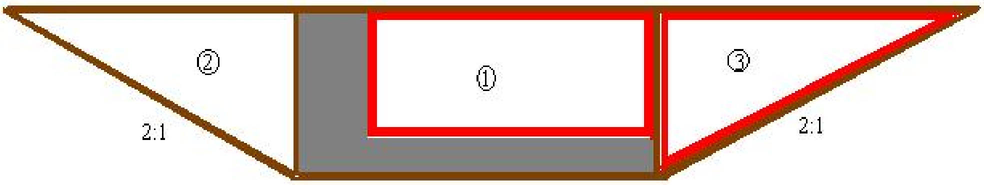


Figure 10.13: Excavation and refill areas

Table 10.6: Excavation and refill cost

Excavation phase		
Sand	2.5	[€/m^3]
Clay	3.5	[€/m^3]
Refill phase		
Sand (soil profile 1)	2.5	[€/m^3]
Sand (soil profile 2)	10.0	[€/m^3]

Further details for the calculation process can be found in the Appendices.

10.2.3 Dewatering Costs

The dewatering cost is relative to the volume of the soil that will be excavated under the groundwater level. Consequently, the volume of soil which is determined between the High Groundwater Level and the excavation depth is used for the cost estimation.

Table 10.7: Dewatering cost

Dewatering		
Both soil profiles	1.63	[€/m^3]

10.3 Total Costs

In that part, the total costs of each case is presented, as they are coming out from the Elastic Supported Beam method (MSheet) and Finite Element Method (PLAXIS), for the two different soil profiles. The following values constitute a rough approximation of the final cost of each structure.

Table 10.8: Total Costs – Msheet

CASE	Soil Profile 1		Soil Profile 2	
	Thousands [€ /m]	Percentage of increment [%]	Thousands [€ /m]	Percentage of increment [%]
D	24.5	0.0	24.2	0.0
A1	28.9	18.3	29.5	21.6
A2	35.1	43.4	35.0	44.3
B1	31.0	26.9	31.9	31.5
B2	36.5	49.2	37.9	56.3
C1	31.7	29.6	34.0	40.4
C2	36.6	49.5	40.8	68.4

Table 10.9: Total Costs – PLAXIS

CASE	Soil Profile 1		Soil Profile 2	
	Thousands [€ /m]	Percentage of increment [%]	Thousands [€ /m]	Percentage of increment [%]
D	22.3	0.0	24.5	0.0
A1	27.0	21.3	29.1	18.9
A2	34.5	34.5	36.6	49.6
B1	30.4	36.6	33.0	35.0
B2	37.6	37.6	39.4	61.1
C1	31.4	40.9	33.7	37.7
C2	38.5	38.5	41.0	67.8

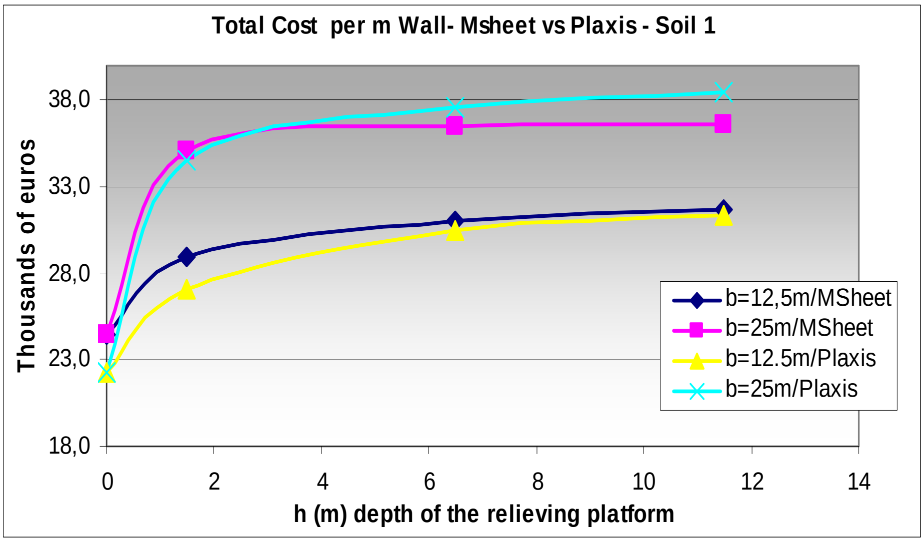


Figure 10.14: Total Costs –MSheet vs PLAXIS – Soil 1

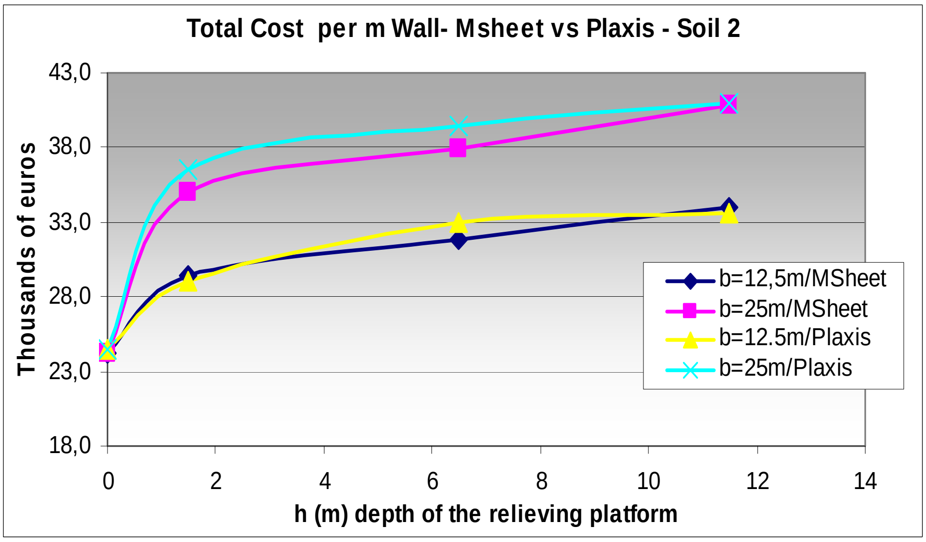


Figure 10.15: Total Costs – MSheet vs PLAXIS – Soil 2

As it is shown from the followings, the case where the relieving platform is absent is the cheapest one. This is discussed in Chapter 11: "Conclusions". Separate graphs for each method and soil profile are found in the appendices.

10.4 References

[10.1] [CUR,Port of Rotterdam, Gemeentewerken Rotterdam; *"Handbook of Quay Walls"*

[10.2] Archive of Gemeentewerken Rotterdam

CHAPTER 11
CONCLUSIONS

11.1 General

In this section, the gathered conclusions of the present study are considered. The several results of the mathematical models and calculation methods that are used in the design of quay wall structures are compared. The structural background of the mathematical models is not explained in the present study, since that is beyond the scope of the MSc thesis.

Moreover, issues resulting from the financial assessment of the structure are discussed. The magnitude of influence of each structural element in the different designs is presented and further parameters that can affect the final cost are mentioned.

11.2 Comparison between the different analysis methods

First of all, it can be conclude while a Finite Element program is used, an accurate prediction of the modulus of elasticity of the soil is of a great importance. A mistaken or overestimated value can lead to significantly different results of the bending moments and the anchor forces.

As it is shown in the previous chapters, different maximum bending moments were estimated for each analysis method. These differences are due to the dissimilar considered models which are assumed in each method.

Blum theory
Blum's method provides overestimated values but due to its simplicity it can always be used as a first estimation of the internal forces and the embedded depth. Taking into account that the philosophy of recent codes, like Handbook of Quay Walls, proposes the fully fixity in the sheet pile wall toe, Fixed Earth Support approach by means of the Equivalent Beam method is a simple and useful way to approximate the subject.

Elastic Supported Beam
Elastic Supported Beam theory, by means of MSheet, gives the opportunity to analyse the sheet pile wall in a sequence of construction phases. The major disadvantages of this calculation method are summarised in the followings.

The structure has to be analysed separately, thus special attention in the redistribution of forces has to be noticed.

Furthermore, the vertical arch working of the soil on the active area, resulting in reduced bending moments and higher anchor forces is not taking into account automatically. This is not the case for a Finite Element program like PLAXIS, where the whole structure is studied in several construction stages.

It has to be noted that in the Elastic Supported Beam method (MSheet) the effect of the width of the relieving platform is more obvious. This might be explained from the different applied ways for the stress distribution. In Blum theory, the manual estimation of the earth pressures due to the relieving platform effect was based on Rankine's theory. In MSheet, additional stresses due to surcharge is based on Boussinesq's stress distribution theory.

In addition, in manual estimation of acting loads in the sheet pile wall, the lateral earth coefficients were assumed constant over the depth, when in MSheet the c, phi, delta method is used.

Finite Element Method

In the present study, the Finite Element Analysis is executed by means of PLAXIS. This method is very sensitive of the modulus of elasticity of the used materials.

At the beginning, the representative modulus of elasticity for a moderate sand (q_c=15MPa, E=45 MPa) according to NEN 6740; 2006 code was used. The coming results of the bending moments on the sheet piling differ significantly, i.e. with a factor of 2-2,5 with those coming from the Elastic Supported Beam theory (MSheet).

Several trials have been executed with different modulus of elasticity, in order to investigate in which value the results from both methods are approximately equal. As it has been already mentioned, that is the case for the representative modulus of elasticity of a loose sand (q_c=5MPa, E=15MPa).

In the following table the different values of the maximum bending moments in the sheet pile wall are presented.

Table 11.1: Max. Bending Moments from different analysis methods

CASE	Soil Profile 1			Soil Profile 2	
	Blum [kNm/m]	**MSheet [kNm/m]**	**PLAXIS [kNm/m]**	**MSheet [kNm/m]**	**PLAXIS [kNm/m]**
D	13922.0	13214.1	11300.0	12489.5	14940.0
A1	12020.4	11584.8	8400.0	11669.3	9670.0
A2	10394.2	9115.6	7920.0	8618.7	9360.0
B1	8259.1	8624.8	7560.0	8988.8	8610.0
B2	6054.5	5585.0	5820.0	6113.6	6800.0
C1	4927.7	4812.6	4860.0	5478.6	4720.0
C2	3214.2	1769.8	3980.0	3451.3	4090.0

As it can be seen from the above table, in case of the soil profile 1, the maximum bending moments coming from the Finite Element Method calculations (PLAXIS) are smaller than those coming from the Elastic Supported Beam method (MSheet). This is explained by the vertical arching that is not taking into consideration on the Elastic Supported Beam calculations.

It is remarkable that in the second soil profile, where a large clay layer is present, the values between the two methods are very close. This is possible, because arching effect is less intent in cohesive soils like clay comparing to non-cohesive soils like sand. In the following figures, this is shown from the directions of the effective stresses.

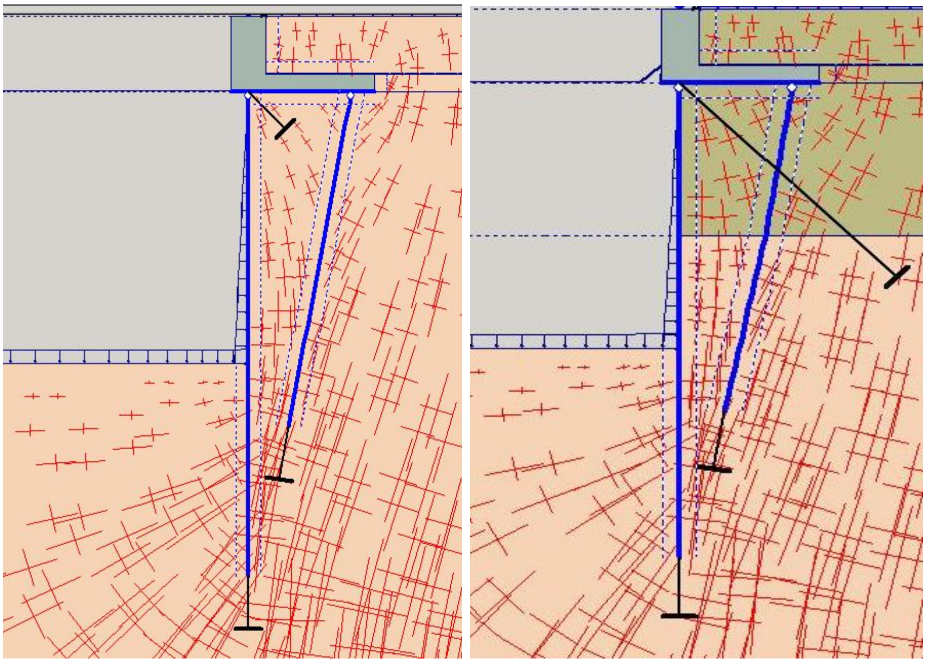


Figure 11.1: Direction of the effective stresses in both soil profiles - arching effect

Moreover the influence of the mutual distances of the foundation piles for the cases of a short relieving platform is investigated. That was introduced in the Finite Element Method (PLAXIS) by two different concepts. Firstly, different bending stiffness of the foundation pile was inserted and secondly the pile is simulated by an anchor, as it is shown in the following figures.

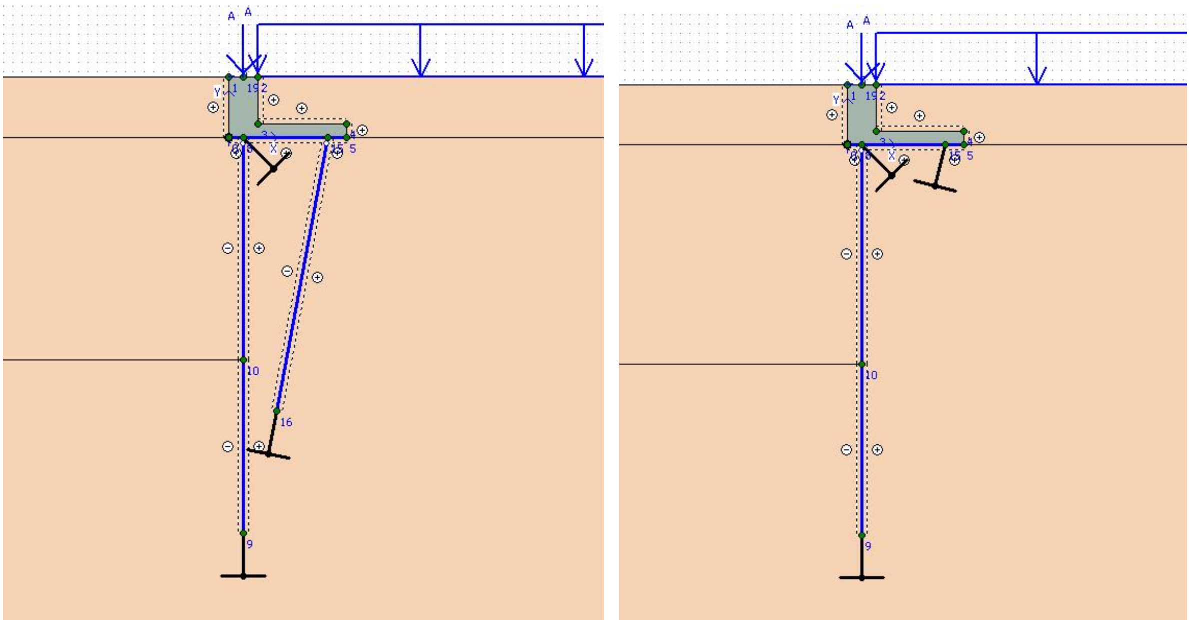


Figure 11.2: Different simulation of the bearing piles

The differences between the cases where a pile is present are minor, even if the mutual distances differ significantly. When the bearing pile is schematised as an anchor with the same equivalent length and stiffness parameters, the maximum bending moment in the sheet pile wall is bigger; approximately 10%.

Table 11.2: Max. Bending Moments for different bearing pile schematisation

CASE B1	max. Bending Moment [kNm/m]
c.t.c 1.25m	7450
c.t.c 2.5m	7560
c.t.c 5.0m	7600
Anchor schematisation	8500

The Finite Element Method (PLAXIS) can also be used for three- dimensional problems, like the screen effect behind the sheet piling, which can cause a reduction of the bending moments in the sheet piling. Consequently, a three dimensional problem is better to be analysed by a three dimensional Finite Element Method program, since from the aforementioned simulations on a two dimensional software program only the extreme boundaries can be provided.

11.3 Cost assessment

First of all, it can be concluded that the total cost coming from the two different methods; i.e. Elastic Supported Beam method (MSheet) and the Finite Element Method (PLAXIS), is approximately equal. This is shown in the following graphs where the total cost of each structure, estimated by the two aforementioned methods are presented for the two different investigated soil profiles.

The difference of the total cost is remarkable, especially in cases where the relieving platform is present, between the two soil profiles. This can be explained considering that in soil profile 2 the sheet pile wall was more expensive and in addition the excavation and refill cost differ significantly.

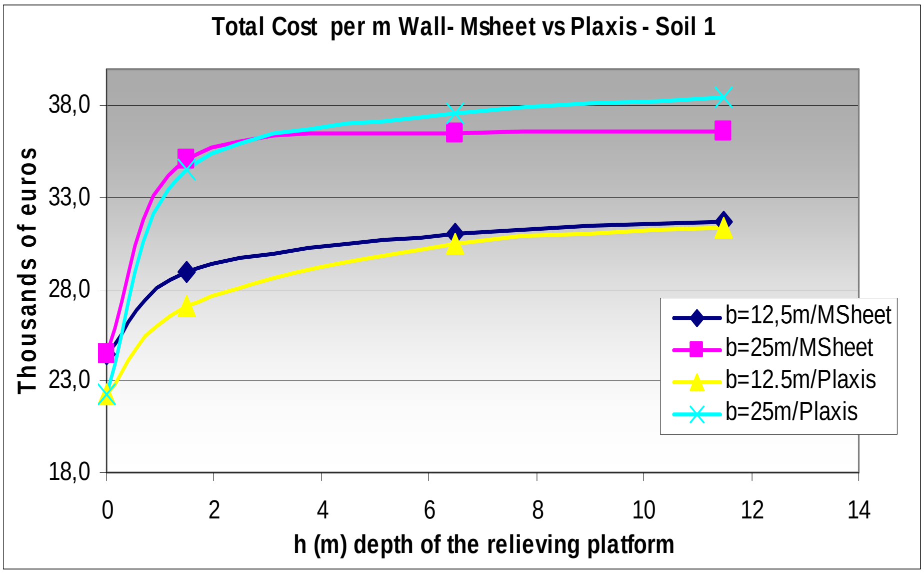


Figure 11.3: Total costs for soil profile 1

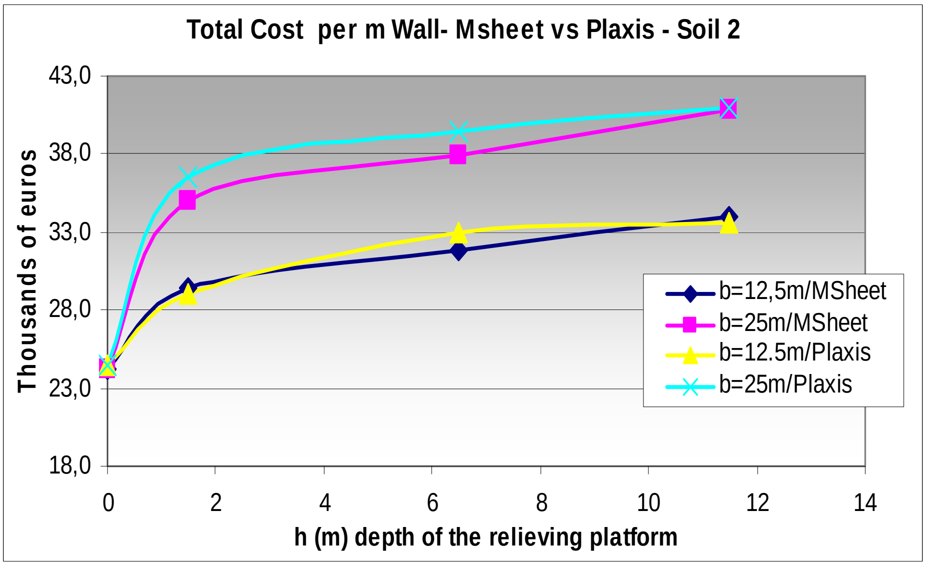


Figure 11.4: Total costs for soil profile 2

As it has already mentioned, the centre to centre distances between the tension piles as well as the maximum length, could be investigated in detail so that the influence of these elements would be clearer.

In spite of that fact, the influence of these elements in the total cost is not dominant. That can be concluded, considering the following two graphs, where it is evident that the normative components are the sheet pile wall and the relieving platform.

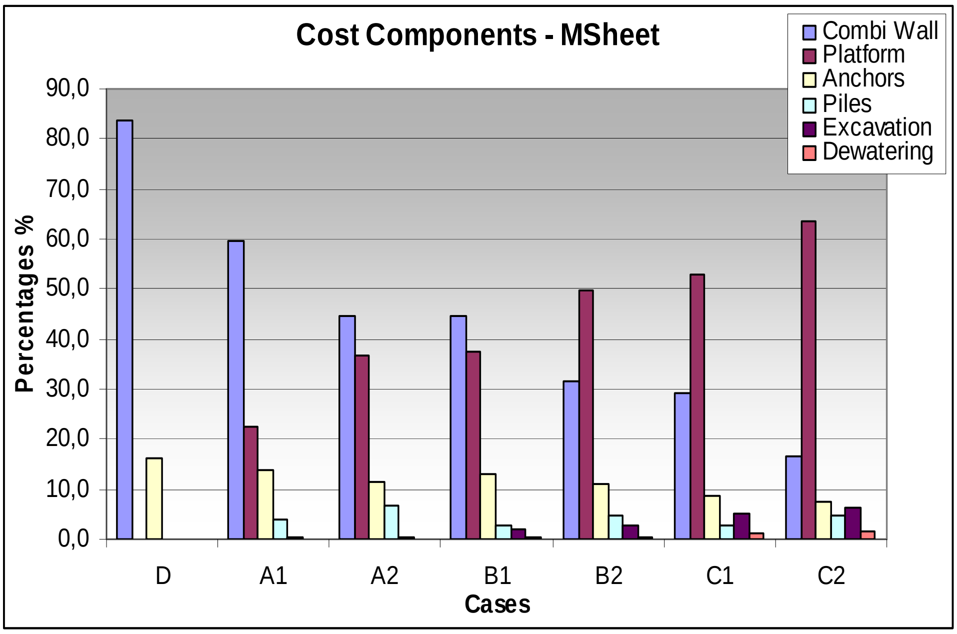


Figure 11.5: Cost components - MSheet

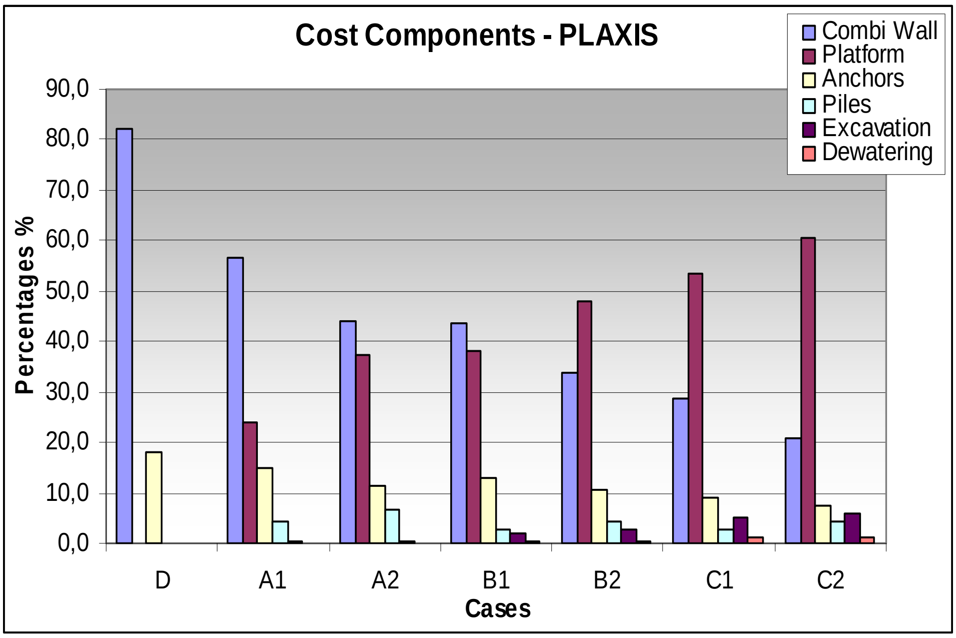


Figure 11.6: Cost components - PLAXIS

From figures 11.3 and 11.4, it is evident that the cheapest solution according to the present rough estimation is Case D, where the quay wall is constructed exclusively from a Combi – Wall supported by anchors.

In the port of Rotterdam quay walls with a relieving platform have been chosen many times when high retaining heights and heavy loads are involved. Without the relieving platform, it would not be possible to construct the sheet pile wall with available equipments. In order to achieve such deep driving depths, offshore equipments should be used increasing dramatically the installation cost of the sheet pile wall.

Moreover, the maximum length of the intermediate elements is about 24 m. Special orders of 31 m increase also the final cost. Longer sheet piles are constructed by welding separate elements and that constitutes also another factor that increases the cost.

In addition to the previous, installation risks, the vulnerability of the structure and high demands in relation to allowable deformations should be taken into account for the final choice.

APPENDIX A

Soil Mechanical Characteristics
CPT's

Rotterdam is located near Northsea in the Rhine-Meuse Delta. Due to the geological history and location of the area, the soil conditions can vary significantly over a short distance.
That variation is the result of meandering rivers and rises and drops in the sea level in the past. At present situation, as it is presented in the following figure, the extensive area can be divided in three areas with their own typical soil profiles.

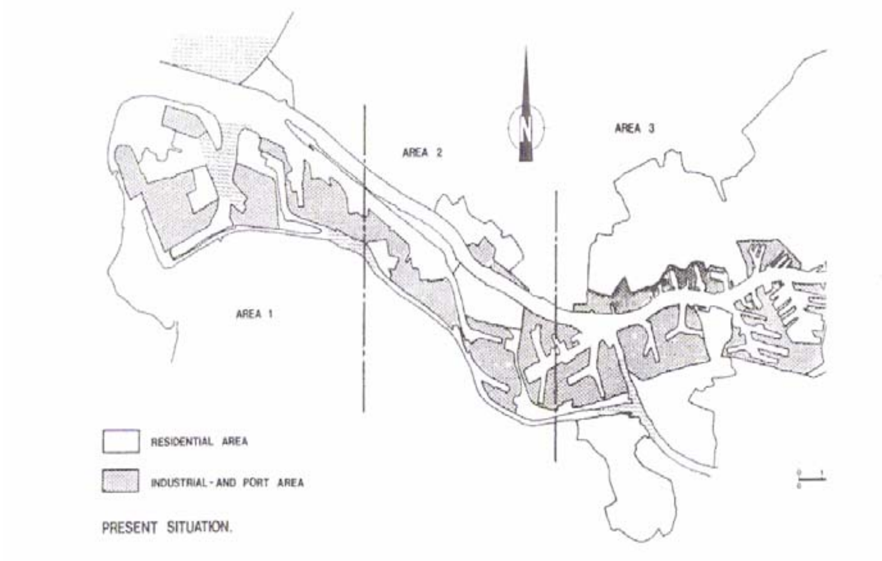


Figure A.1: Soil mechanical characteristics Port of Rotterdam

The three areas are:

- The city area up to the river Oude Maas
- The area between the river Oude Maas and the Maasvlakte
- The Maasvlakte area.

The following CPTs are representative for the Maasvlakte 1 area, which was reclaimed by the sea, and for the area near the city of Rotterdam.

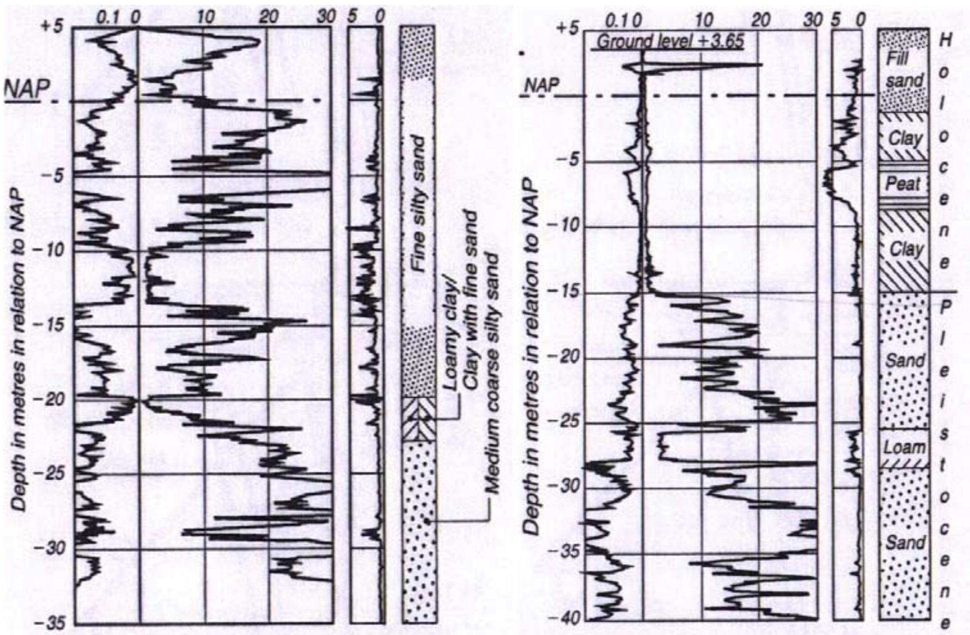


Figure A.2: CPT-result in the area of Maasvlakte 1 and near the city of Rotterdam

APPENDIX B

Static Analysis of the Relieving Platform

Case A1

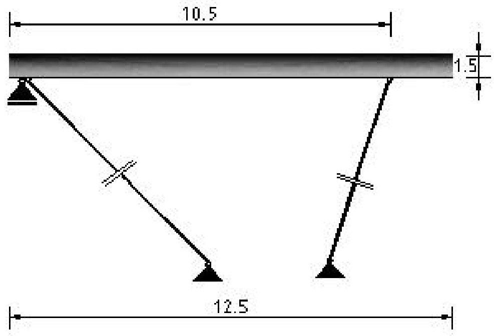


Figure B.1: Schematization of the relieving platform – case A1- (static model 1)

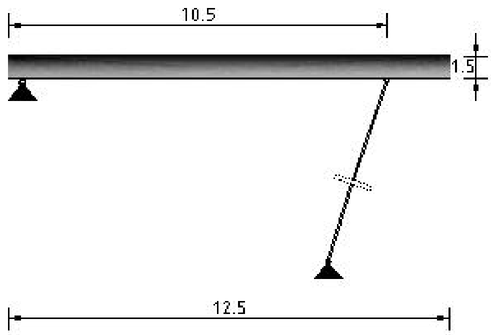


Figure B.2: Schematization of the relieving platform – case A1- (static model 2)

Table B.1: Results of Static Analysis – case A1

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	1130,948	635,848	168,948	1330,948
Anchor Force	[KN]	170,343	-430,557	229,74	170,343
Bearing Pile 1	[KN]	513,713	513,713	513,713	513,713
Bearing Pile 2	[KN]	-	-	-	-
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	1330,900	635,847	168,948	1330,900
Anchor Force	[KN]	170,342	-430,556	229,739	170,342
Bearing Pile 1	[KN]	513,713	513,713	513,713	513,713
Bearing Pile 2	[KN]	-	-	-	-

Case A2

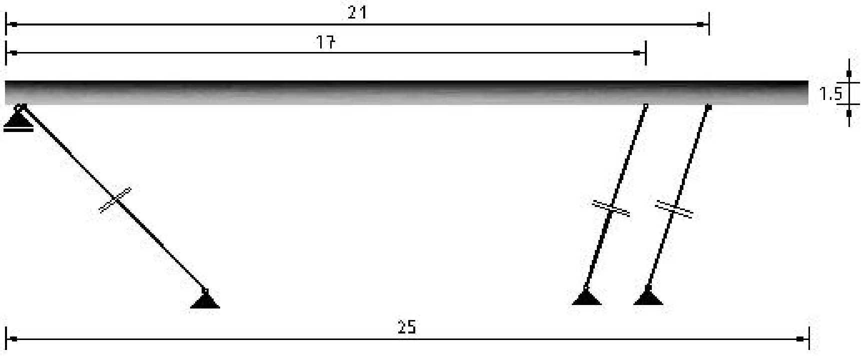


Figure B.3: Schematization of the relieving platform – case A2- (static model 1)

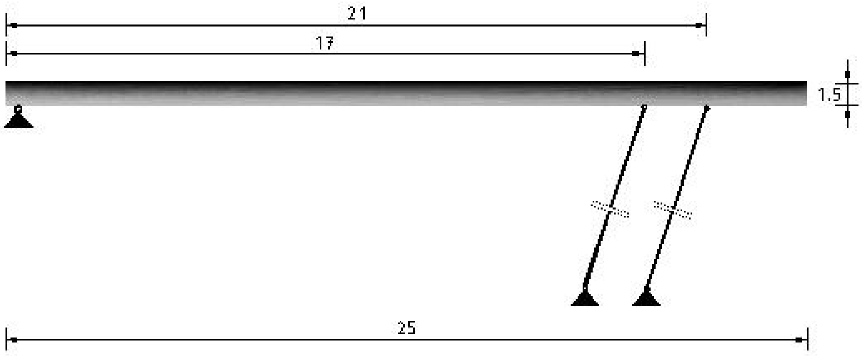


Figure B.4: Schematization of the relieving platform – case A2- (static model 2)

Table B.2: Results of Static Analysis – case A2

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	1294,777	585,547	135,635	1294,777
Anchor Force	[KN]	470,196	-125,73	527,564	470,196
Bearing Pile 1	[KN]	860,249	918,952	852,379	860,249
Bearing Pile 2	[KN]	330,416	282,896	336,486	330,416
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	1283,628	588,528	122,511	1283,628
Anchor Force	[KN]	474,120	-126,780	532,316	474,120
Bearing Pile 1	[KN]	906,567	906,567	906,379	906,567
Bearing Pile 2	[KN]	292,922	292,922	292,811	292,922

Case B1

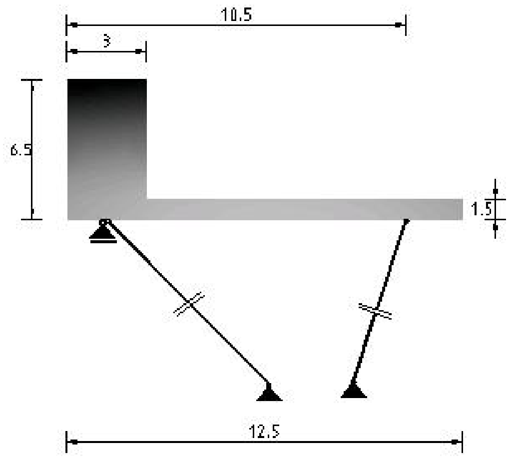


Figure B.5: Schematization of the relieving platform – case B1- (static model 1)

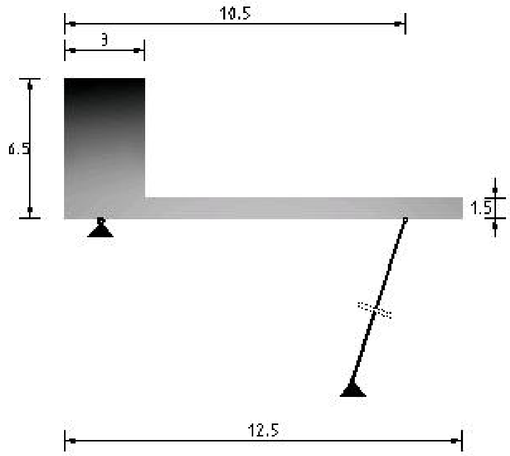


Figure B.6: Schematization of the relieving platform –case B1- (static model 2)

Table B.3: Results of Static Analysis – case B1

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	2346	1367,78	879,5089	2319,491
Anchor Force	[KN]	-138,004	-267,263	209,9457	-102,868
Bearing Pile 1	[KN]	907,6214	856,7226	1021,713	909,9216
Bearing Pile 2	[KN]	-	-	-	-
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	2346,547	1367,780	879,509	2319,491
Anchor Force	[KN]	-138,004	-267,264	209,946	-102,868
Bearing Pile 1	[KN]	907,621	856,723	1012,713	909,922
Bearing Pile 2	[KN]	-	-	-	-

Case B2

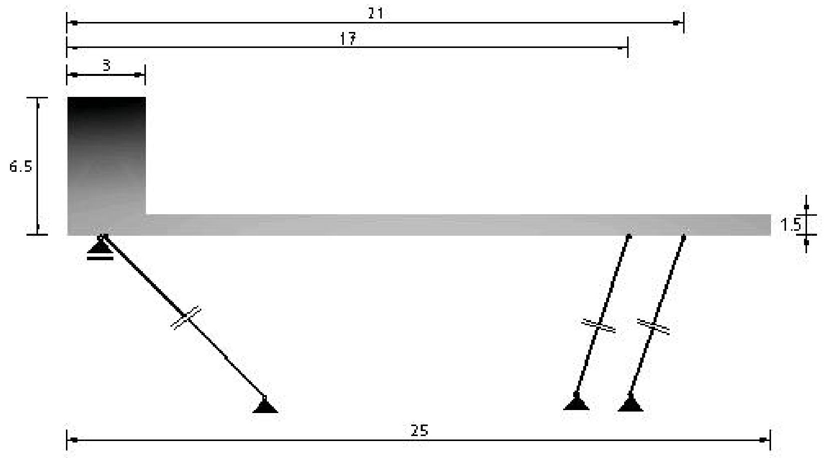


Figure B.7: Schematization of the relieving platform – case B2 - (static model 1)

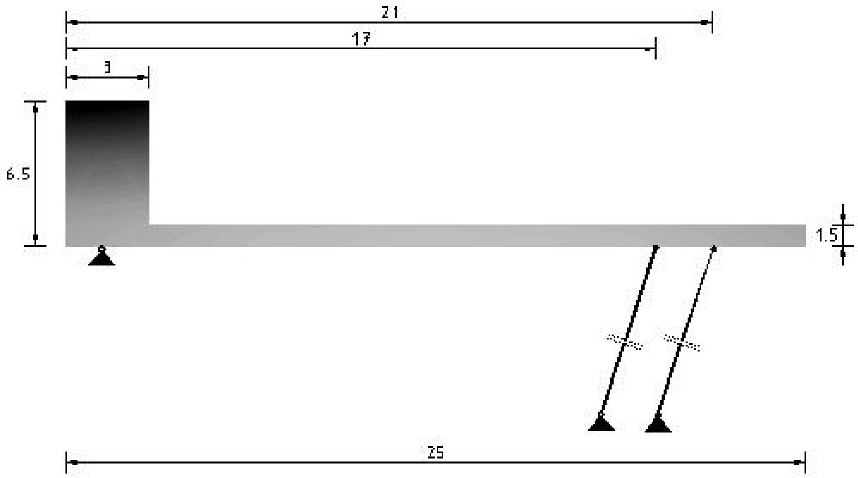


Figure B.8: Schematization of the relieving platform case B2 - (static model 2)

Table B.4: Results of Static Analysis – case B2

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	2176,772	1183,436	748,1026	1776,241
Anchor Force	[KN]	539,3918	409,2261	878,686	574,0958
Bearing Pile 1	[KN]	1666,661	1612,946	1767,662	1666,03
Bearing Pile 2	[KN]	641,0853	658,3049	612,5513	642,7613
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	2163,232	1173,164	726,047	1761,831
Anchor Force	[KN]	544,179	412,858	886,485	579,191
Bearing Pile 1	[KN]	1722,856	1655,579	1859,208	1725,840
Bearing Pile 2	[KN]	595,595	623,792	538,446	594,344

Case C1

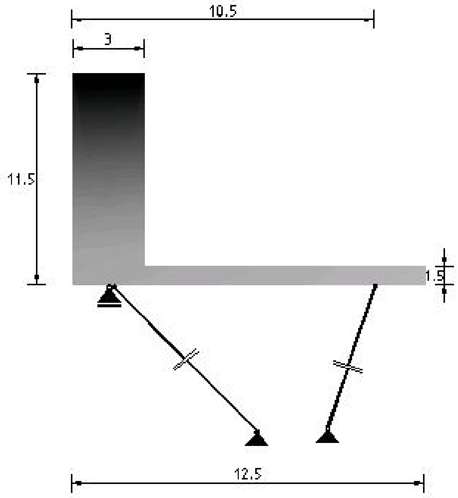


Figure B.9: Schematization of the relieving platform –case C1 - (static model 1)

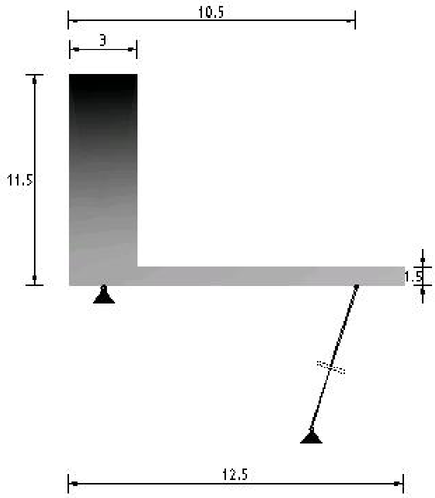


Figure B.10: Schematization of the relieving platform – case C1 - (static model 2)

Table B.5: Results of Static Analysis – case C1

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	3776,104	2853,852	2164,962	3620,289
Anchor Force	[KN]	-656,235	-811,206	-250,627	-485,686
Bearing Pile 1	[KN]	1246,929	1154,090	1462,847	-1284,678
Bearing Pile 2	[KN]	-	-	-	-
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	3776,104	2853,852	2164,962	3620,289
Anchor Force	[KN]	-656,236	-811,207	-250,627	-485,687
Bearing Pile 1	[KN]	1246,929	1154,090	1462,847	1284,678
Bearing Pile 2	[KN]	-	-	-	-

Case C2

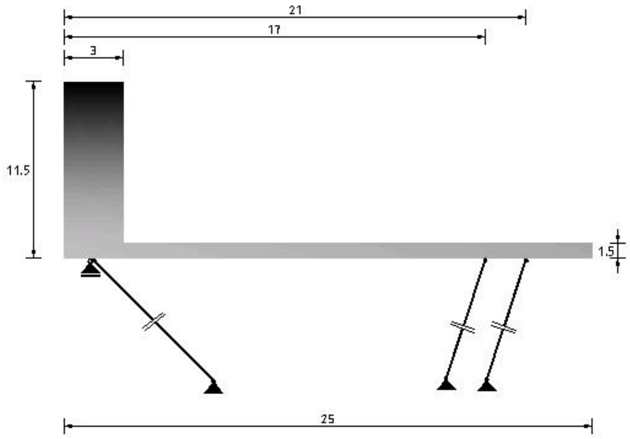


Figure B.11: Schematization of the relieving platform – case C2 - (static model 1)

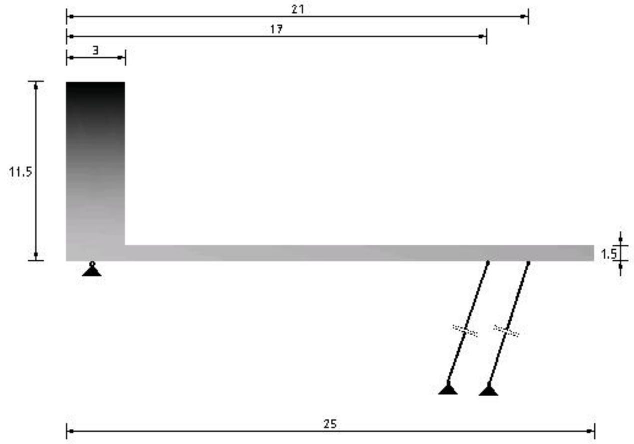


Figure B.12: Schematization of the relieving platform - case C2 - (static model 2)

Table B.6: Results of Static Analysis – case C2

First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	3502,177	2540,665	2007,733	3388,361
Anchor Force	[KN]	366,424	224,360	716,785	509,079
Bearing Pile 1	[KN]	2451,331	2340,128	2695,778	2480,734
Bearing Pile 2	[KN]	1113,460	1155,057	1023,365	1106,169
First Case (Roller Support)		**LCA1**	**LCA2**	**LCA3**	**LCB**
Reaction in the connecting point	[KN]	3454,000	2511,445	1914,381	3322,060
Anchor Force	[KN]	396,269	242,634	775,167	550,543
Bearing Pile 1	[KN]	2484,441	2360,402	2760,547	2526,735
Bearing Pile 2	[KN]	1086,657	1138,645	970,933	1068,930

APPENDIX C

Blum Method - Equivalent Beam Method –
Frame Solver 2D Static Analysis

Case D – Soil Profile 1

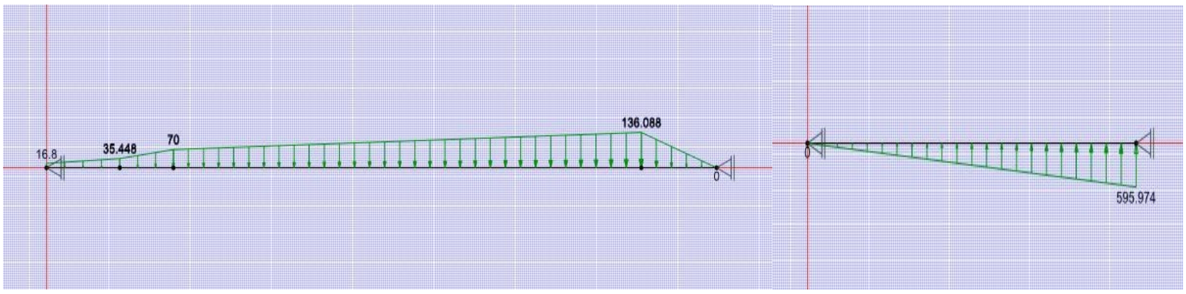


Figure C.1: Acting Loads on Sheet Pile wall – Case D

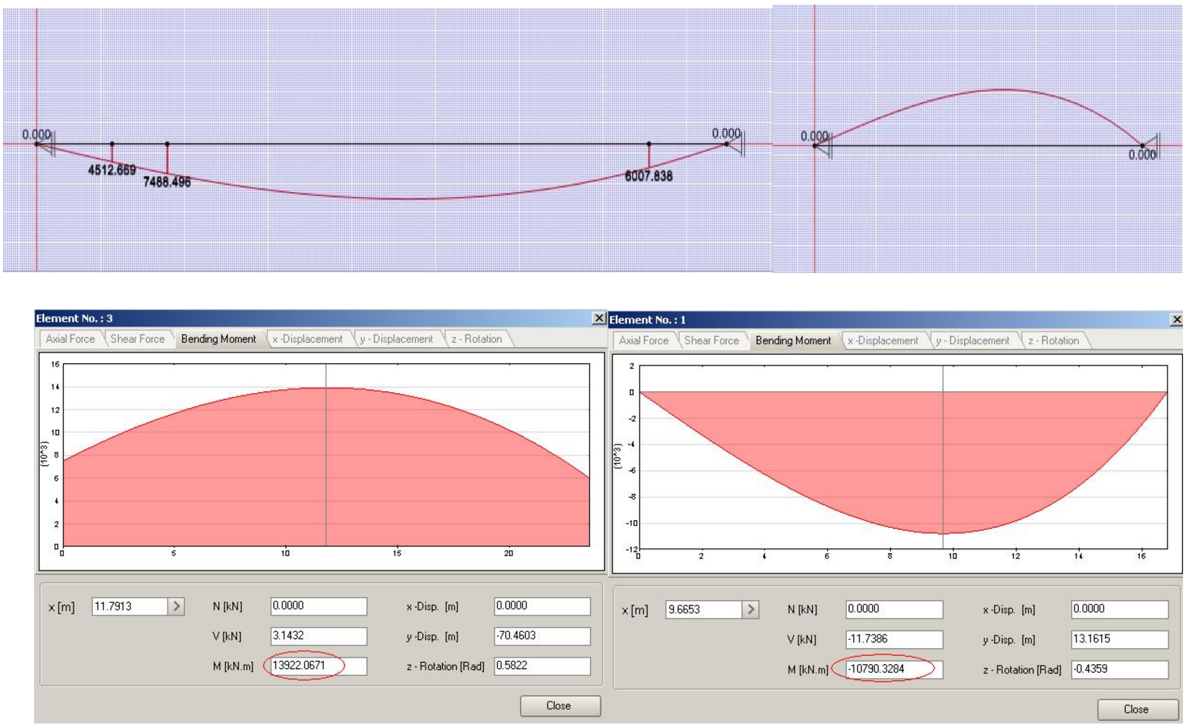


Figure C.2: Bending Moments on the Sheet Pile wall – Case D

Table C.1: Preliminary design of case D

Primary Elements			System		
External Diameter (D)	**2500**	[mm]	Length	3,750	[m]
Thickness (t)	**45**	[mm]	Moment of Inertia	6,991E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	1,468E+07	[kNm^2/m^1]
Moment of Inertia (I)	2,616E+11	[mm^4]	Section Modulus (W)	0,050024	[m^3/m]

Table C.2: Loads acting on Case D

External loads		
Vertical crane load (F_y)	1120,0	[kN]
Vertical component of the bollard load (F_x)	466,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1262,2	[kN]
Total Axial Force (Nmax)	2382,2	[kN]
Maximum Bending Moment (M)	13922,1	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d{}'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 395{,}27 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A1 – Soil Profile 1

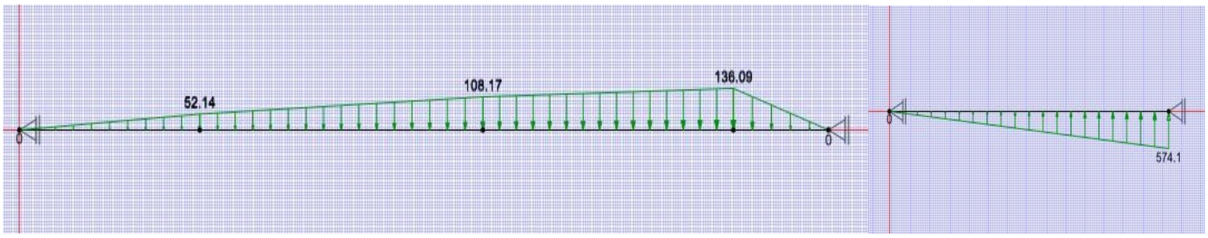


Figure C.3: Acting Loads on Sheet Pile wall – Case A1

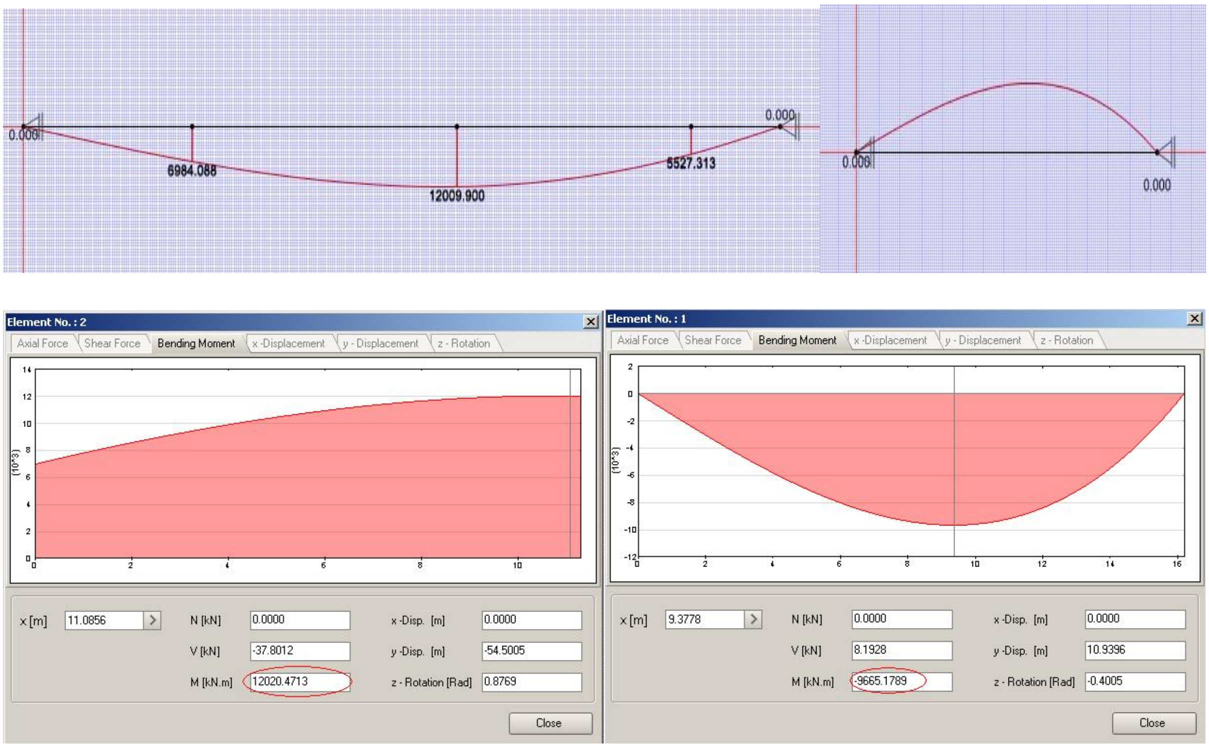


Figure C.4: Bending Moments on the Sheet Pile wall – Case A1

Table C.3: Preliminary design of case A1

Primary Elements			**System**		
External Diameter (D)	**2420**	[mm]	Length	3,670	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	5,098E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	1,071E+07	[kNm^2/m^1]
Moment of Inertia (I)	1,865E+11	[mm^4]	Section Modulus (W)	0,042132	[m^3/m]

Table C.4: Loads acting on Case A1

Reactions from platform analysis		
Maximum vertical reaction (Fy)	1330,9	[kN]
Vertical component of horizontal reaction (Fx)	304,4	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1337,0	[kN]
Total Axial Force (Nmax)	2667,9	[kN]
Maximum Bending Moment (M)	12020,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 419{,}44\ \mathrm{N/mm^2} < \sigma_y = 443\ \mathrm{N/mm^2}$$

Case A2 – Soil Profile 1

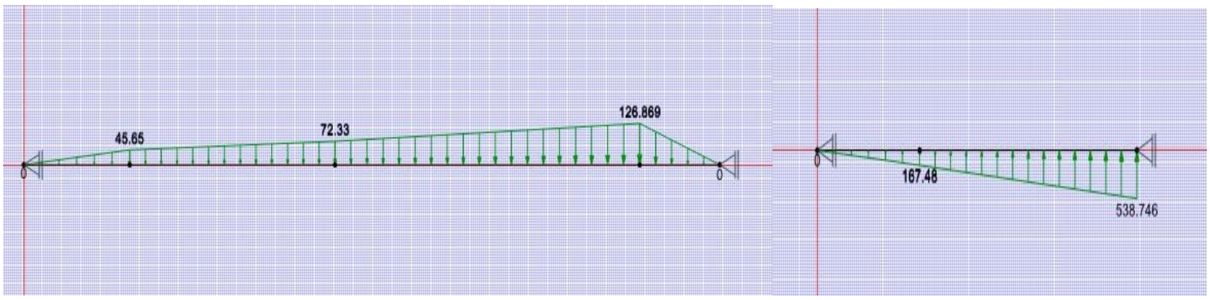


Figure C.5: Acting Loads on Sheet Pile wall – Case A2

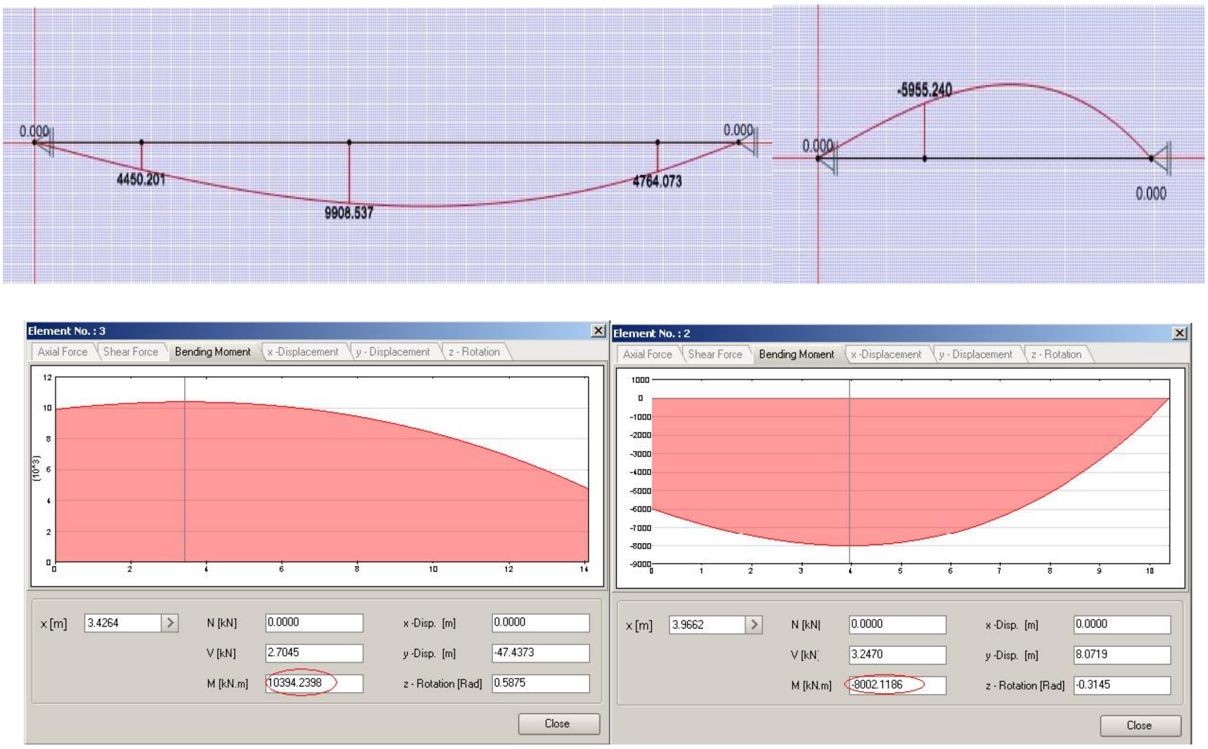


Figure C.6: Bending Moments on the Sheet Pile wall – Case A2

Table C.5: Preliminary design of case A2

Primary Elements			System		
External Diameter (D)	**2120**	[mm]	Length	3,370	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	3,715E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	7,802E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,246E+11	[mm^4]	Section Modulus (W)	0,035047	[m^3/m]

Table C.6: Loads acting on Case A2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	1283,6	[kN]
Vertical component of horizontal reaction (F_x)	89,6	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	945,4	[kN]
Total Axial Force (Nmax)	2525,7	[kN]
Maximum Bending Moment (M)	10394,2	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 429{,}88\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case B1 – Soil Profile 1

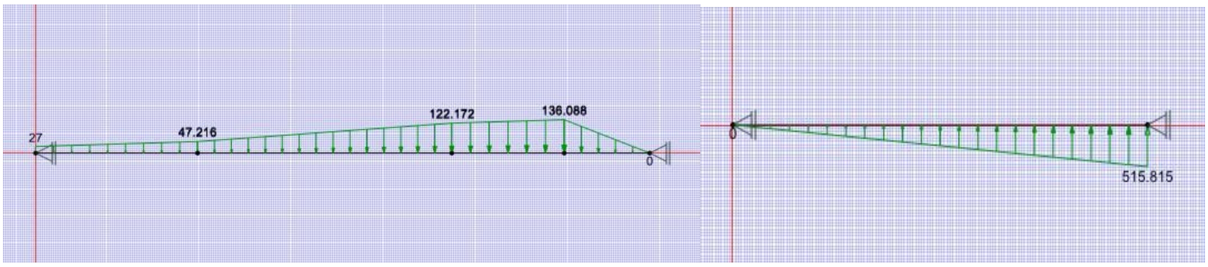
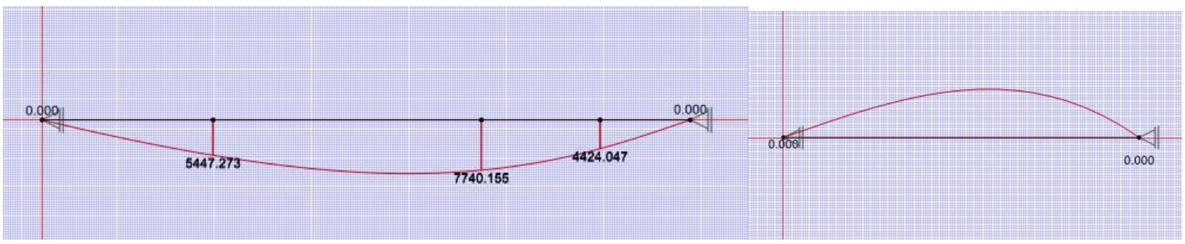


Figure C.7: Acting Loads on Sheet Pile wall – Case B1



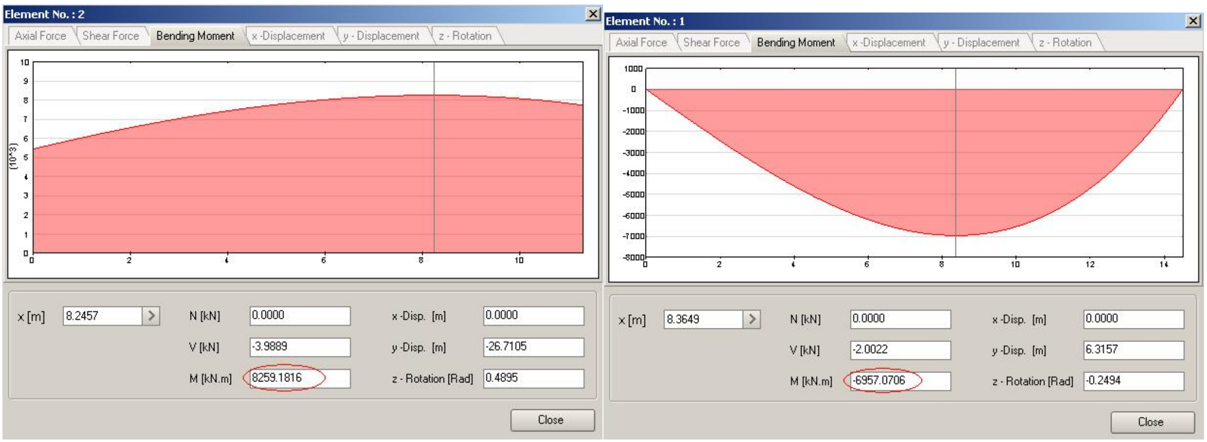


Figure C.8: Bending Moments on the Sheet Pile wall – Case B1

Table C.7: Preliminary design of case B1

Primary Elements			System		
External Diameter (D)	**2320**	[mm]	Length	3,570	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	3,980E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	8,359E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,415E+11	[mm^4]	Section Modulus (W)	0,03431	[m^3/m]

Table C.8: Loads acting on Case B1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2346,5	[kN]
Vertical component of horizontal reaction (F_x)	188,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	878,0	[kN]
Total Axial Force (Nmax)	3413,5	**[kN]**
Maximum Bending Moment (M)	8259,1	**[kNm]**

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 386{,}38 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case B2 – Soil Profile 1

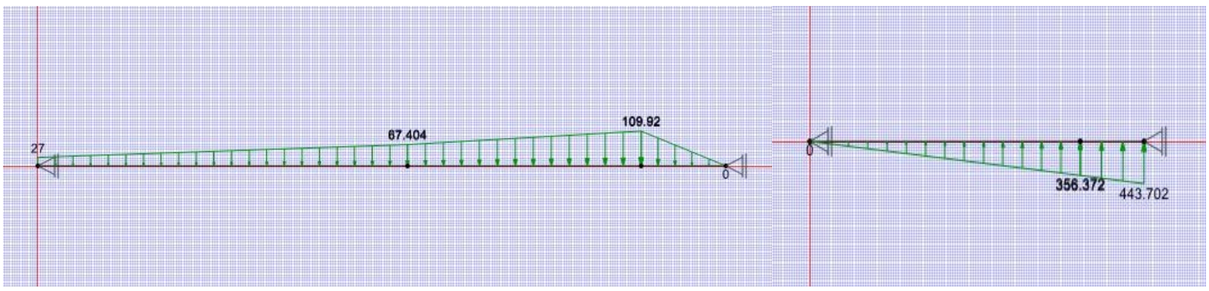


Figure C.9: Acting Loads on Sheet Pile wall – Case B2

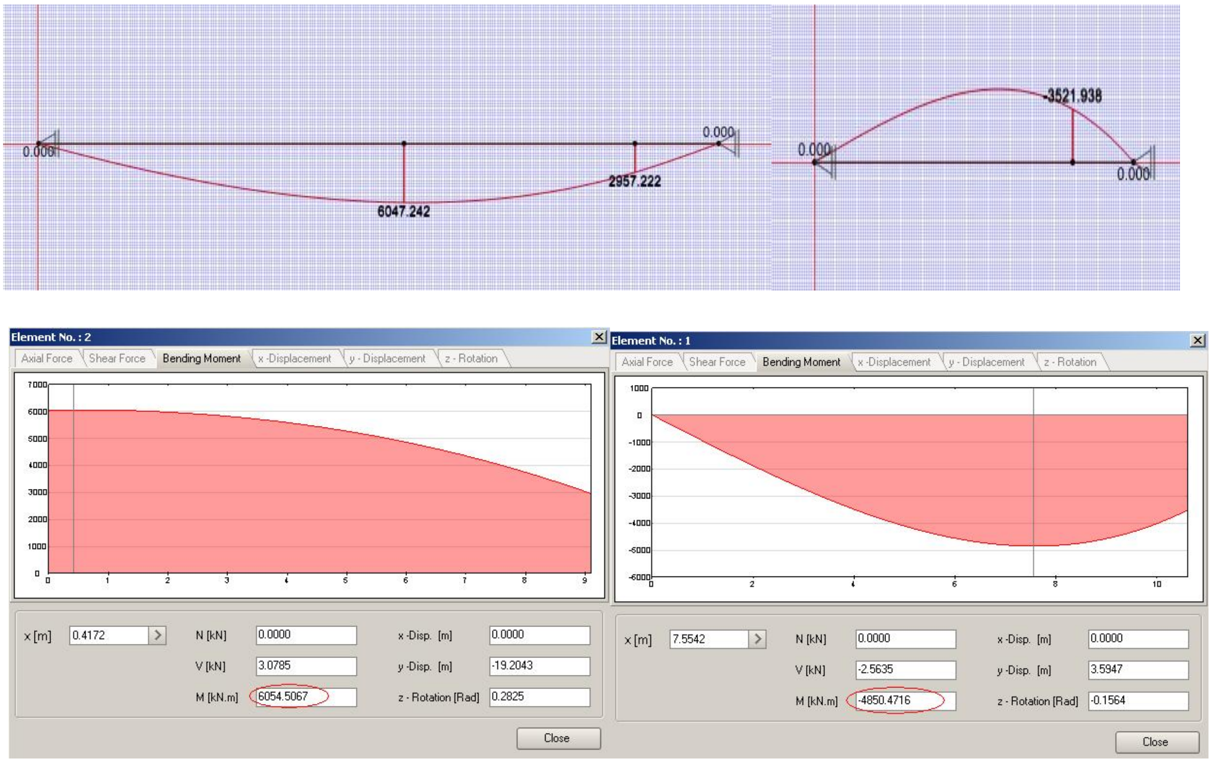


Figure C.10: Bending Moments on the Sheet Pile wall – Case B2

Table C.9: Preliminary design of case B2

Primary Elements			System		
External Diameter (D)	**1920**	[mm]	Length	3,170	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	2,528E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,310E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,956E+10	[mm^4]	Section Modulus (W)	0,026333	[m^3/m]

Table C.10: Loads acting on Case B2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2163,2	[kN]
Vertical component of horizontal reaction (F_x)	-291,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	711,3	[kN]
Total Axial Force (Nmax)	2582,6	[kN]
Maximum Bending Moment (M)	6054,5	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 358{,}67\ \text{N/mm}^2 < \sigma_y = 443\ \text{N/mm}^2$$

Case C1 – Soil Profile 1

Figure C.11: Acting Loads on Sheet Pile wall – Case C1

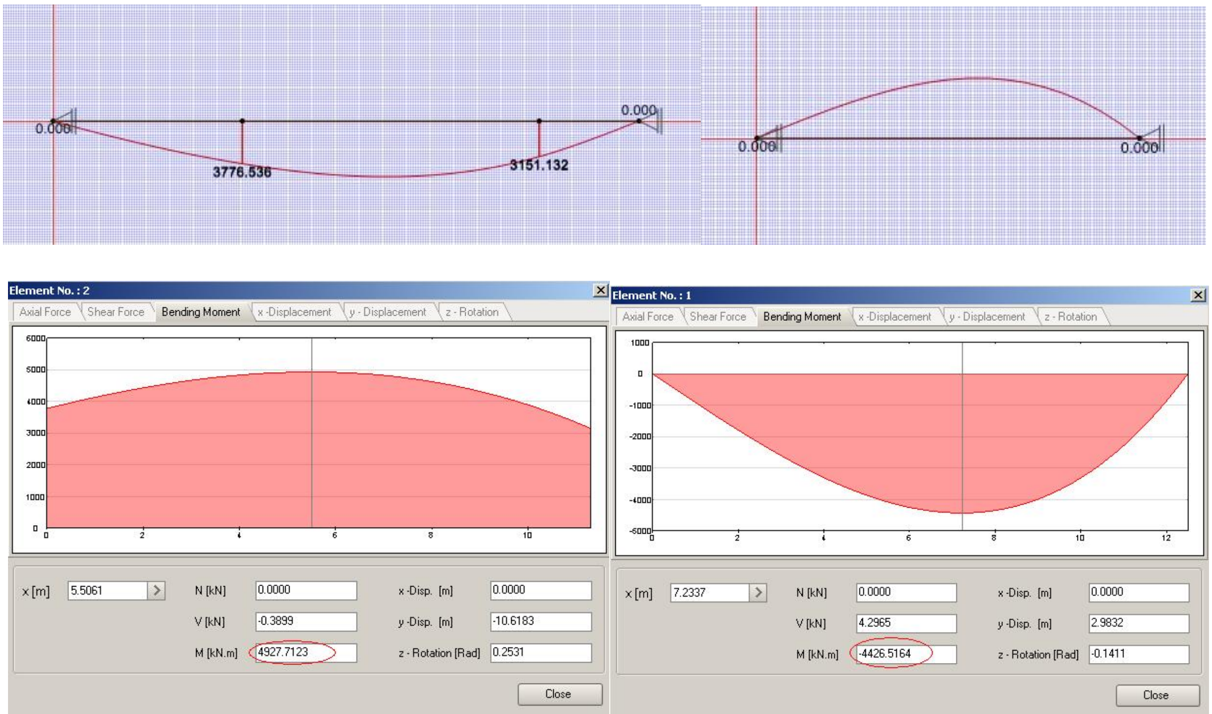


Figure C.12: Bending Moments on the Sheet Pile wall – Case C1

Table C.11: Preliminary design of case C1

Primary Elements			**System**		
External Diameter (D)	**1920**	[mm]	Length	3,170	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	2,528E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,310E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,956E+10	[mm^4]	Section Modulus (W)	0,026333	[m^3/m]

Table C.12: Loads acting on Case C1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3776,1	[kN]
Vertical component of horizontal reaction (F_x)	573,6	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	645,9	[kN]
Total Axial Force (Nmax)	4995,6	[kN]
Maximum Bending Moment (M)	4927,7	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 358{,}90 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C2 – Soil Profile 1

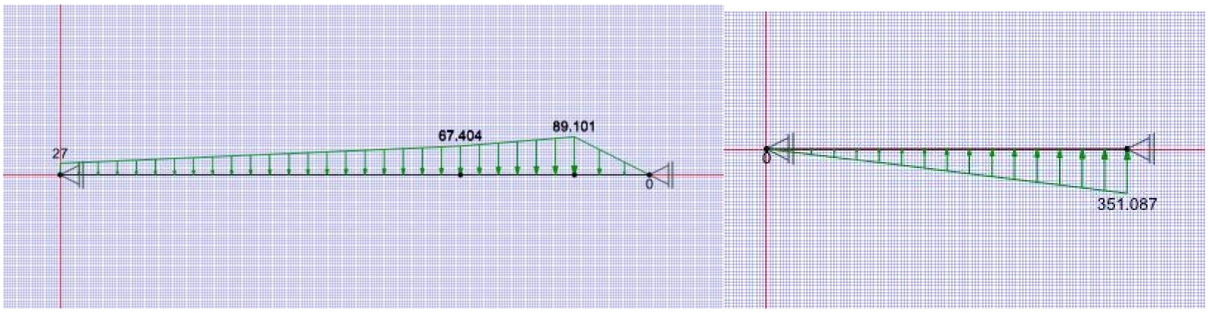


Figure C.13: Acting Loads on Sheet Pile wall – Case C2

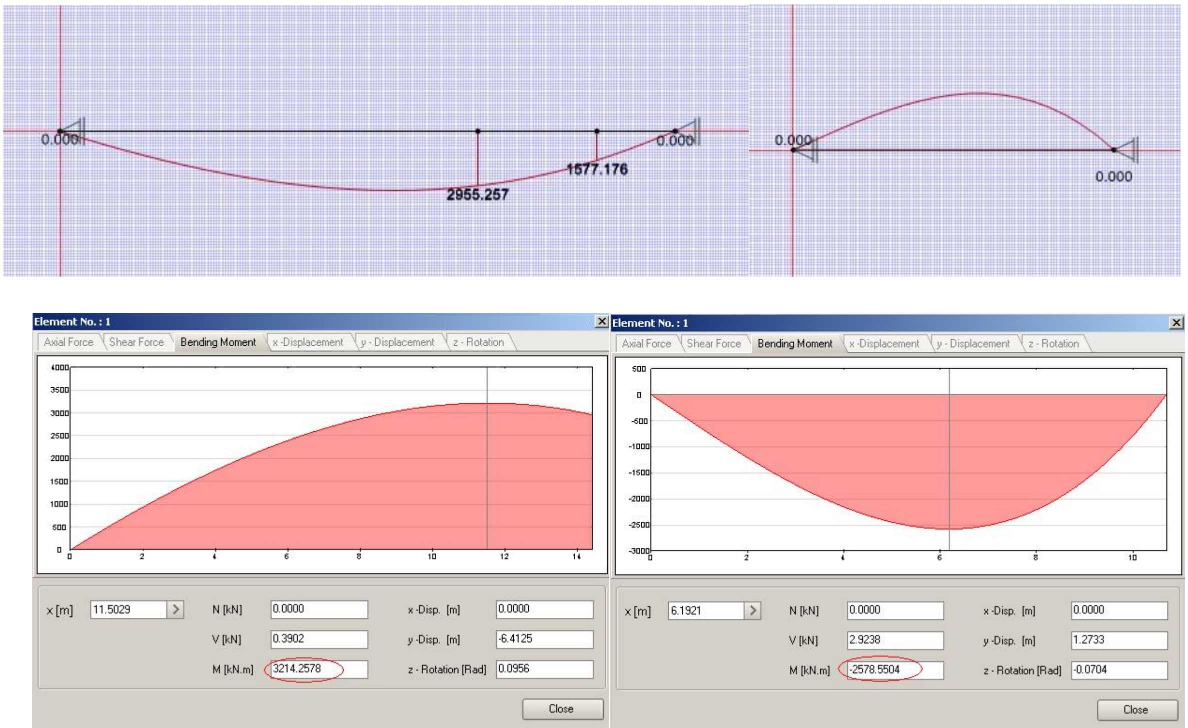


Figure C.14: Bending Moments on the Sheet Pile wall – Case C2

Table C.13: Preliminary design of case C2

Primary Elements			System		
External Diameter (D)	**1420**	[mm]	Length	2,670	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	1,208E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	2,536E+06	[kNm^2/m^1]
Moment of Inertia (I)	3,165E+10	[mm^4]	Section Modulus (W)	0,017014	[m^3/m]

Table C.14: Loads acting on Case C2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3454,0	[kN]
Vertical component of horizontal reaction (F_x)	-171,5	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	496,6	[kN]
Total Axial Force (Nmax)	3779,0	[kN]
Maximum Bending Moment (M)	3214,2	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 345{,}77 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

APPENDIX D

Sheet Pile Analysis – Elastic Supported Beam (Msheet)

Case D – Soil Profile 1

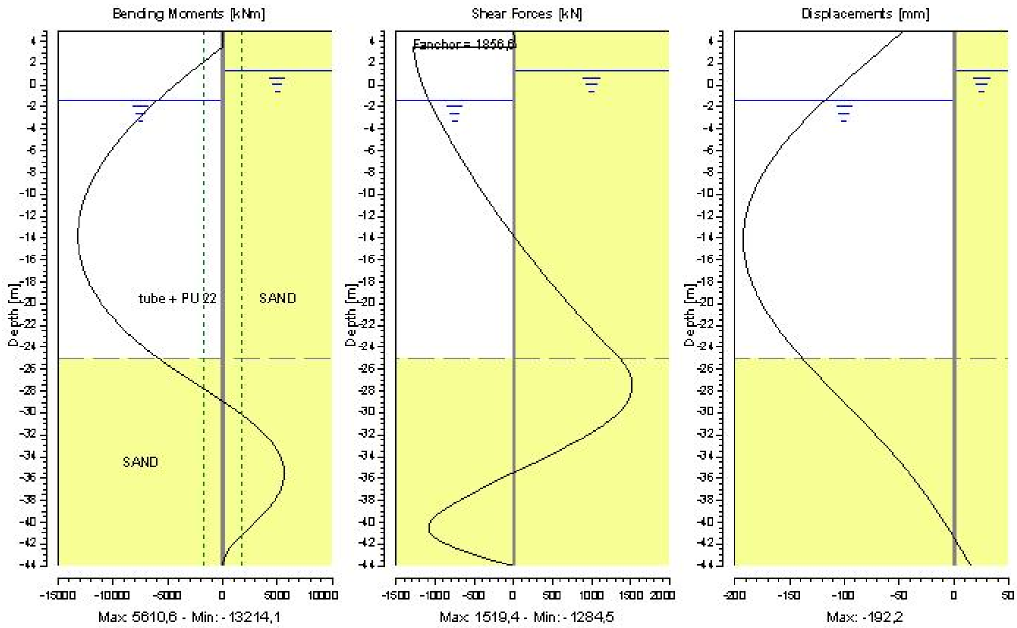


Figure D.1: Representative values of internal forces – case D

Table D.1: Preliminary design of case D

Primary Elements			**System**		
External Diameter (D)	**2320**	[mm]	Length	3,570	[m]
Thickness (t)	**45**	[mm]	Moment of Inertia	5,847E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	1,228E+07	[kNm^2/m^1]
Moment of Inertia (I)	2,082E+11	[mm^4]	Section Modulus (W)	0,050405	[m^3/m]

Table D.2: Loads acting on Case D

External loads		
Vertical crane load (F_y)	1120,0	[kN]
Vertical component of the bollard load (F_x)	466,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1856,6	[kN]
Total Axial Force (Nmax)	3443,5	[kN]
Maximum Bending Moment (M)	13214,1	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 390{,}48 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A1 – Soil Profile 1

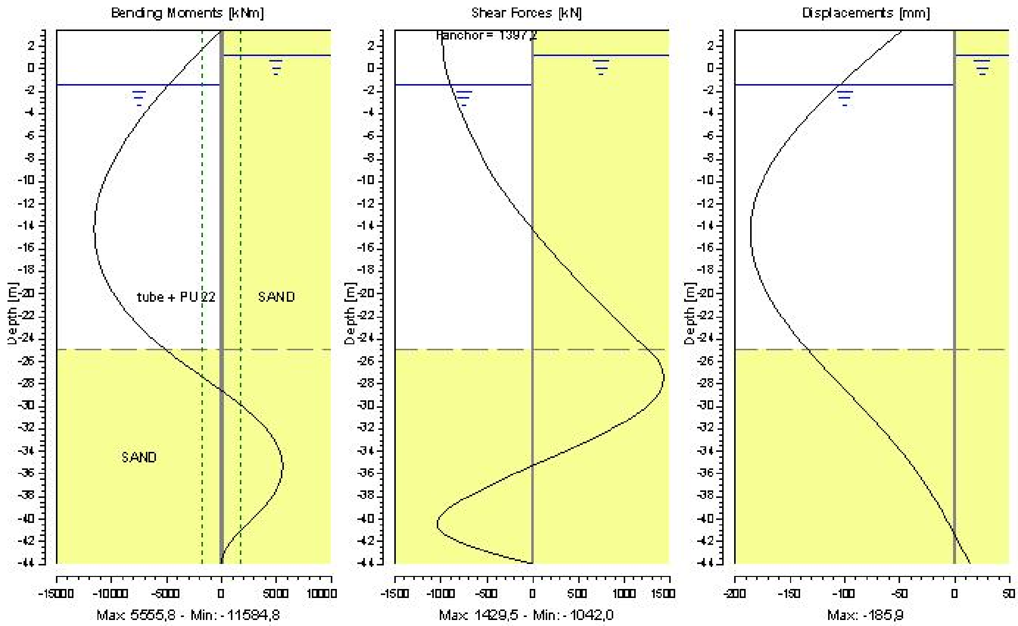


Figure D.2: Representative values of internal forces – case A1

Table D.3: Preliminary design of case A1

Primary Elements			System		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	4,709E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	9,888E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,628E+11	[mm^4]	Section Modulus (W)	0,041540	[m^3/m]

Table D.4: Loads acting on Case A1

Reactions from platform analysis		
Maximum vertical reaction (Fy)	1330,9	[kN]
Vertical component of horizontal reaction (Fx)	304,5	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1397,2	[kN]
Total Axial Force (Nmax)	3372,8	[kN]
Maximum Bending Moment (M)	11584,8	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 419{,}71 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A2 – Soil Profile 1

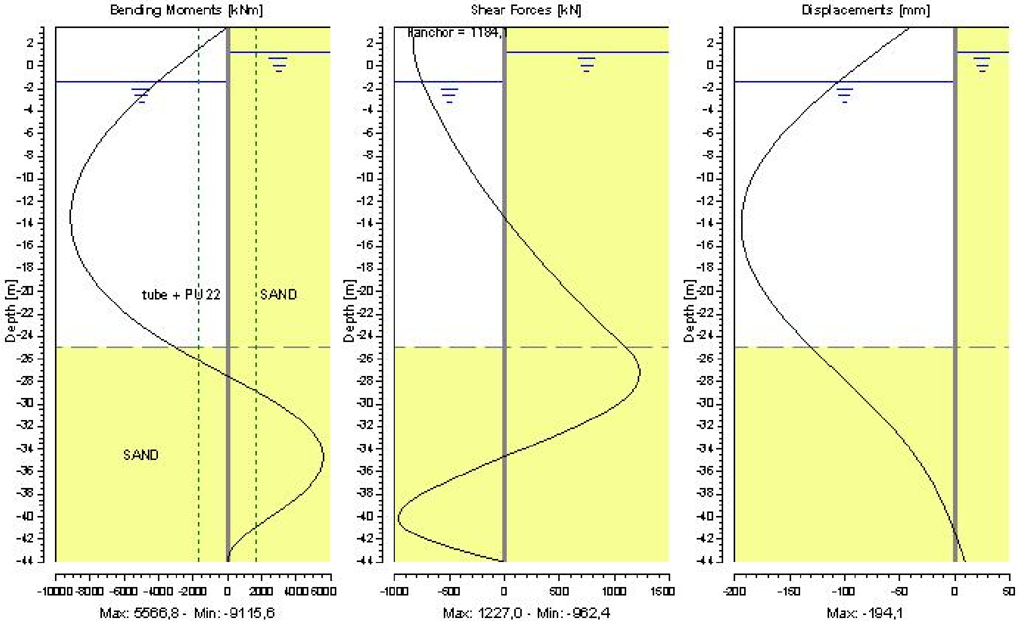


Figure D.3: Representative values of internal forces – case A2

Table D.5: Preliminary design of case A2

Primary Elements			**System**		
External Diameter (D)	**2120**	[mm]	Length	3,370	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	3,715E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	7,802E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,246E+11	[mm^4]	Section Modulus (W)	0,035047	[m^3/m]

Table D.6: Loads acting on Case A2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	1283,628	[kN]
Vertical component of horizontal reaction (F_x)	89,647	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1184,1	[kN]
Total Axial Force (Nmax)	2812,1	[kN]
Maximum Bending Moment (M)	9115,6	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 391{,}8\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case B1 – Soil Profile 1

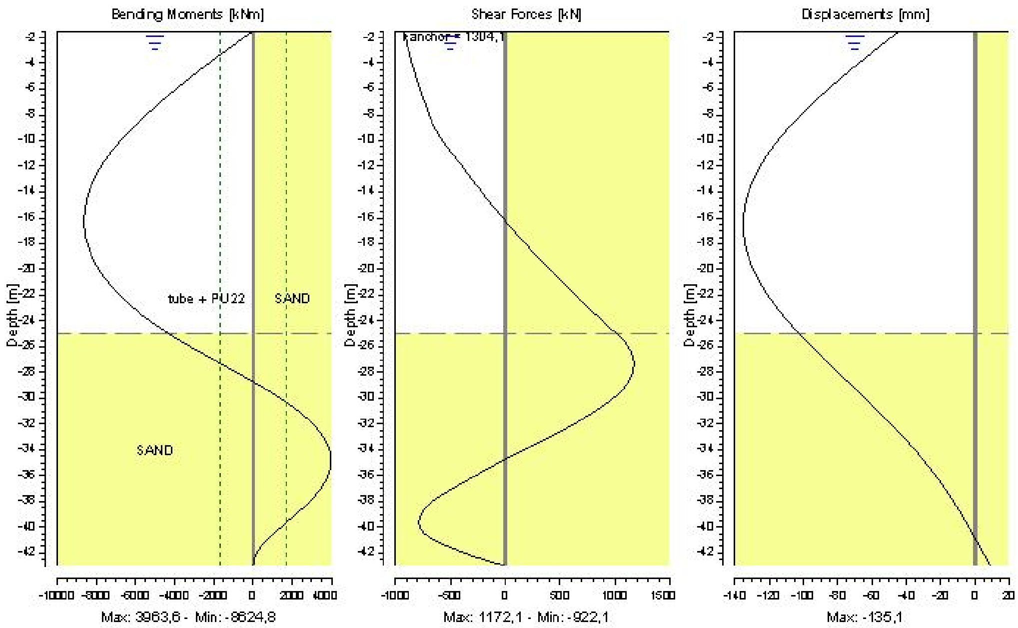


Figure D.4: Representative values of internal forces – case B1

Table D.7: Preliminary design of case B1

Primary Elements			**System**		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	4,150E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	8,715E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,434E+11	[mm^4]	Section Modulus (W)	0,037387	[m^3/m]

Table D.8: Loads acting on Case B1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2346,5	[kN]
Vertical component of horizontal reaction (F_x)	188,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1304,1	[kN]
Total Axial Force (Nmax)	4138,2	**[kN]**
Maximum Bending Moment (M)	8624,8	**[kNm]**

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 377{,}63 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case B2 – Soil Profile 1

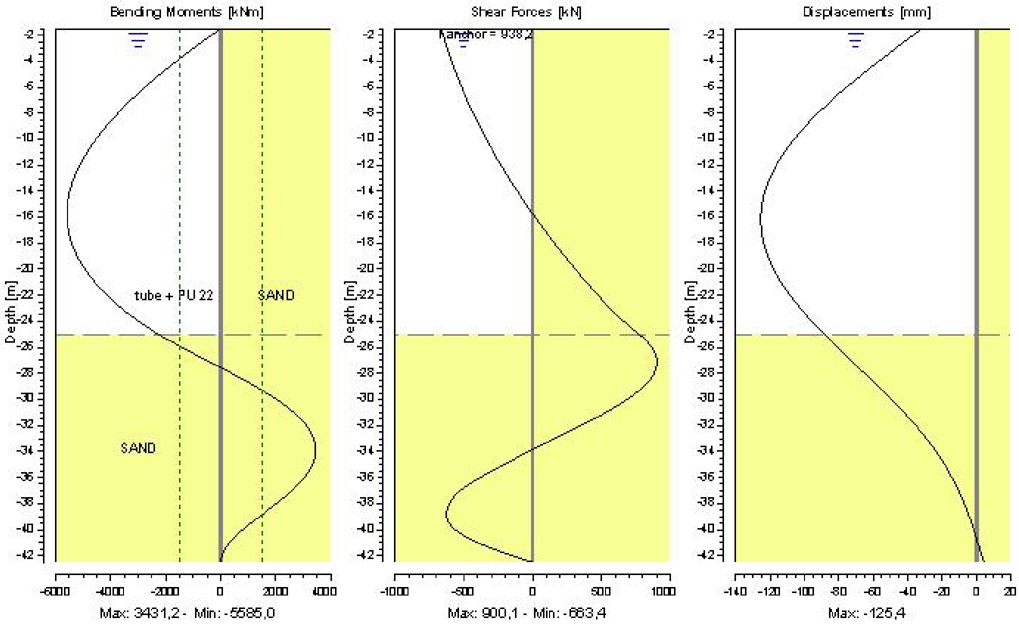


Figure D.5: Representative values of internal forces – case B2

Table D.9: Preliminary design of case B2

Primary Elements			System		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	2,221E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	4,664E+06	[kNm^2/m^1]
Moment of Inertia (I)	6,759E+10	[mm^4]	Section Modulus (W)	0,024406	[m^3/m]

Table D.10: Loads acting on Case B2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2163,2	[kN]
Vertical component of horizontal reaction (F_x)	-291,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	938,2	[kN]
Total Axial Force (Nmax)	2938,7	[kN]
Maximum Bending Moment (M)	5585,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 367{,}04 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C1 – Soil Profile 1

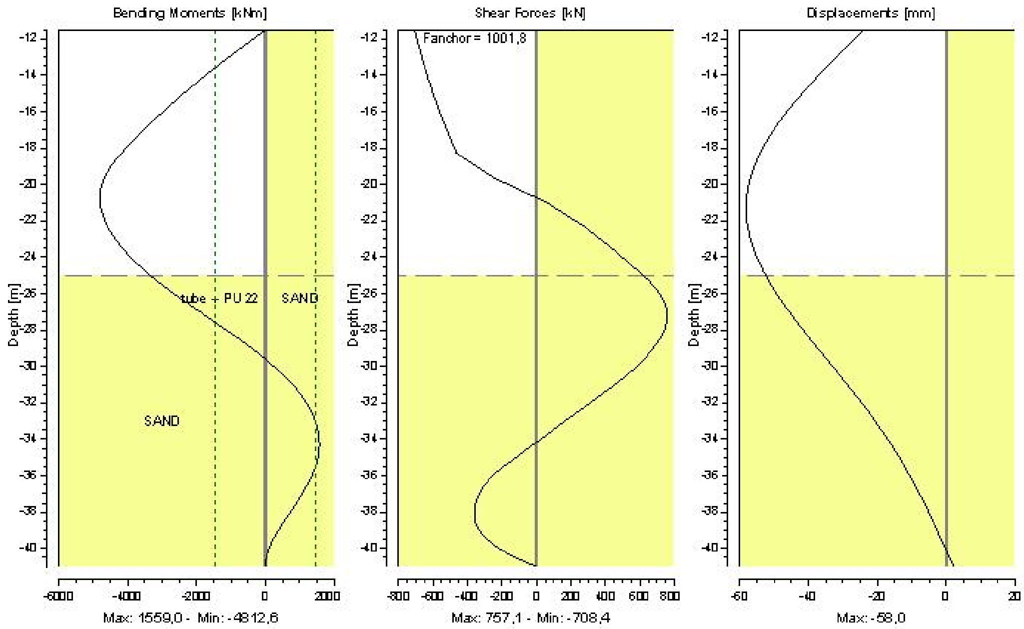


Figure D.6: Representative values of internal forces – case C1

Table D.11: Preliminary design of case C1

Primary Elements			System		
External Diameter (D)	**1720**	[mm]	Length	2,970	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,235E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	4,693E+06	[kNm^2/m^1]
Moment of Inertia (I)	6,578E+10	[mm^4]	Section Modulus (W)	0,025988	[m^3/m]

Table D.12: Loads acting on Case C1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3776,1	[kN]
Vertical component of horizontal reaction (F_x)	573,6	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1001,8	[kN]
Total Axial Force (Nmax)	5666,5	[kN]
Maximum Bending Moment (M)	4812,6	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 358{,}85 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C2 – Soil Profile 1

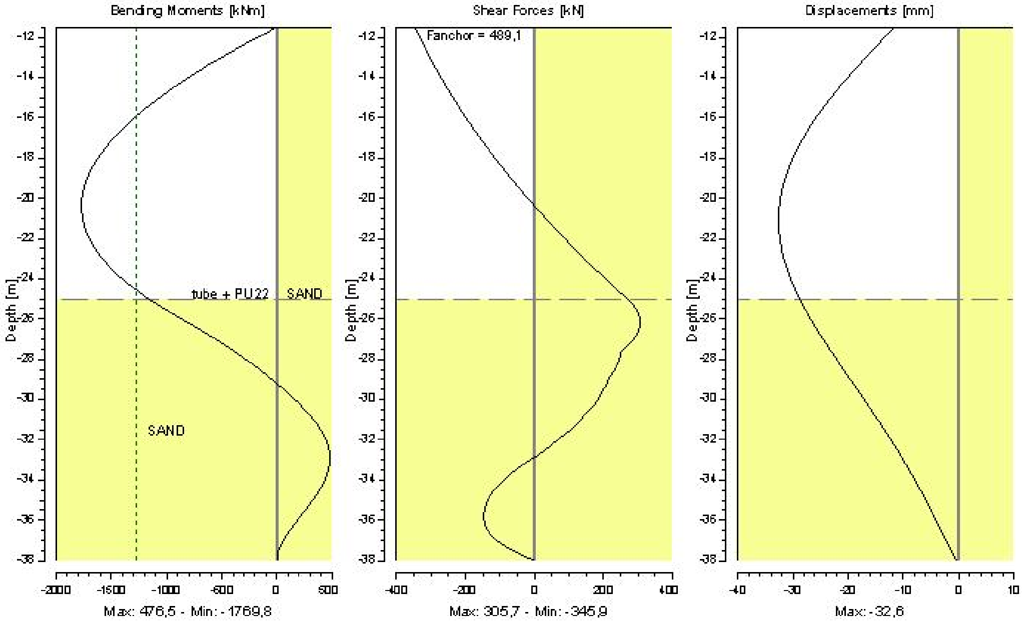


Figure D.7: Representative values of internal forces – case C2

Table D.13: Preliminary design of case C2

Primary Elements			**System**		
External Diameter (D)	**1220**	[mm]	Length	2,470	[m]
Thickness (t)	**25**	[mm]	Moment of Inertia	7,026E-03	$[m^4/m^1]$
Steel Quality	**X 70**	[-]	Stiffness	1,475E+06	$[kNm^2/m^1]$
Moment of Inertia (I)	1,676E+10	$[mm^4]$	Section Modulus (W)	0,011518	$[m^3/m]$

Table D.14: Loads acting on Case C2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3454,0	[kN]
Vertical component of horizontal reaction (F_x)	-171,5	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	489,1	[kN]
Total Axial Force (Nmax)	3853,0	[kN]
Maximum Bending Moment (M)	1769,8	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 331{,}02 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case D – Soil Profile 2

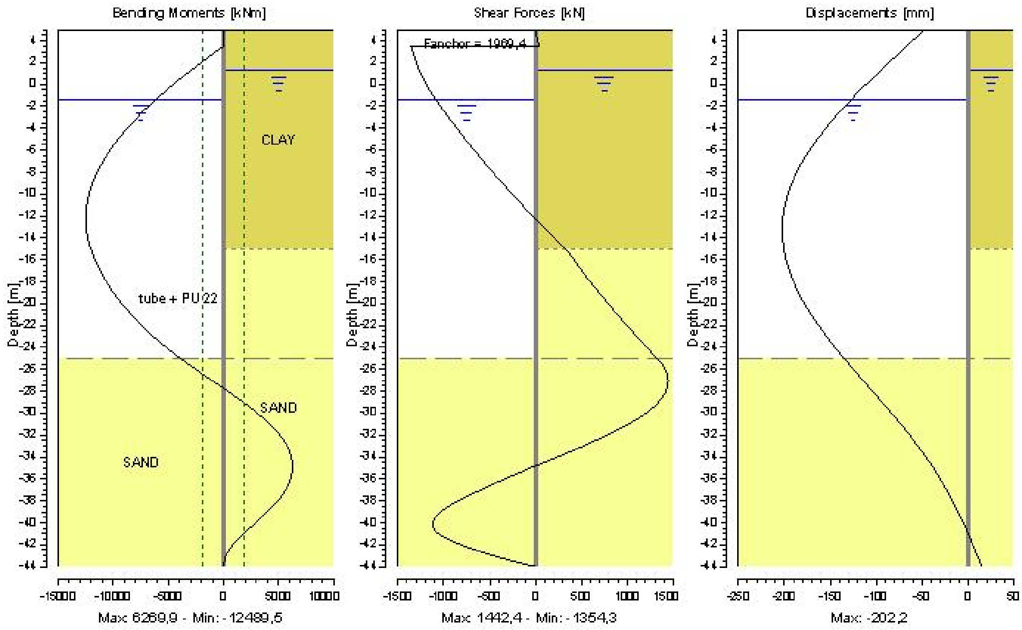


Figure D.8: Representative values of internal forces – case D

Table D.15: Preliminary design of case D

Primary Elements			System		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**45**	[mm]	Moment of Inertia	5,259E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	1,104E+07	[kNm^2/m^1]
Moment of Inertia (I)	1,819E+11	[mm^4]	Section Modulus (W)	0,047378	[m^3/m]

Table D.16: Loads acting on Case D

External loads		
Vertical crane load (F_y)	1120,0	[kN]
Vertical component of the bollard load (F_x)	466,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1969,4	[kN]
Total Axial Force (Nmax)	3555,9	[kN]
Maximum Bending Moment (M)	12489,5	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 390{,}48\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case A1 – Soil Profile 2

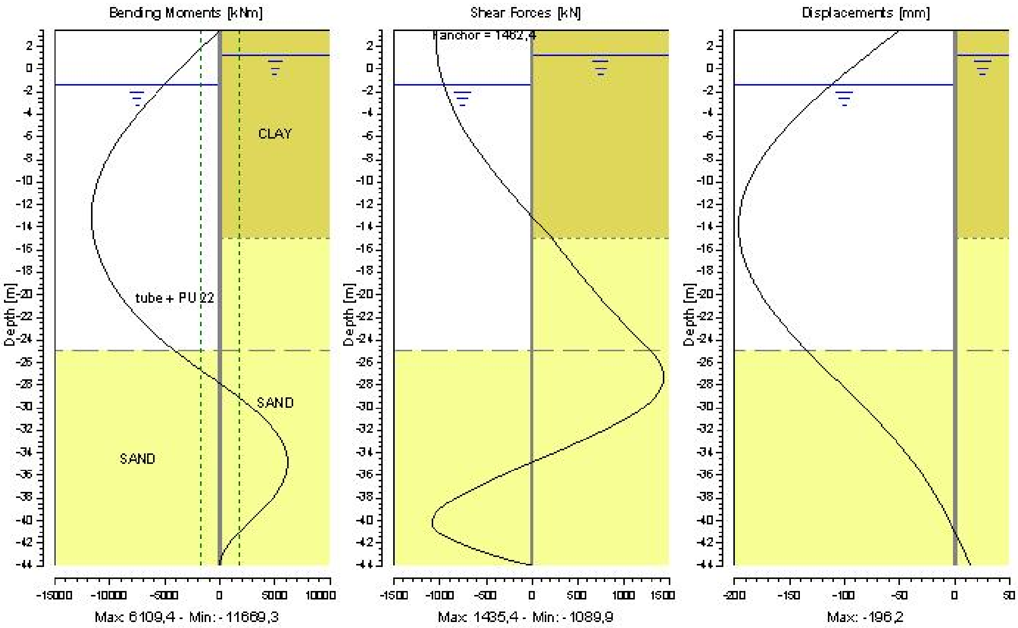


Figure D.9: Representative values of internal forces – case A1

Table D.17: Preliminary design of case A1

Primary Elements			System		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	4,709E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	9,888E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,628E+11	[mm^4]	Section Modulus (W)	0,042423	[m^3/m]

Table D.18: Loads acting on Case A1

Reactions from platform analysis		
Maximum vertical reaction (Fy)	1330,9	[kN]
Vertical component of horizontal reaction (Fx)	304,4	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1462,4	[kN]
Total Axial Force (Nmax)	3451,1	[kN]
Maximum Bending Moment (M)	11669,3	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 414{,}44 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A2 – Soil Profile 2

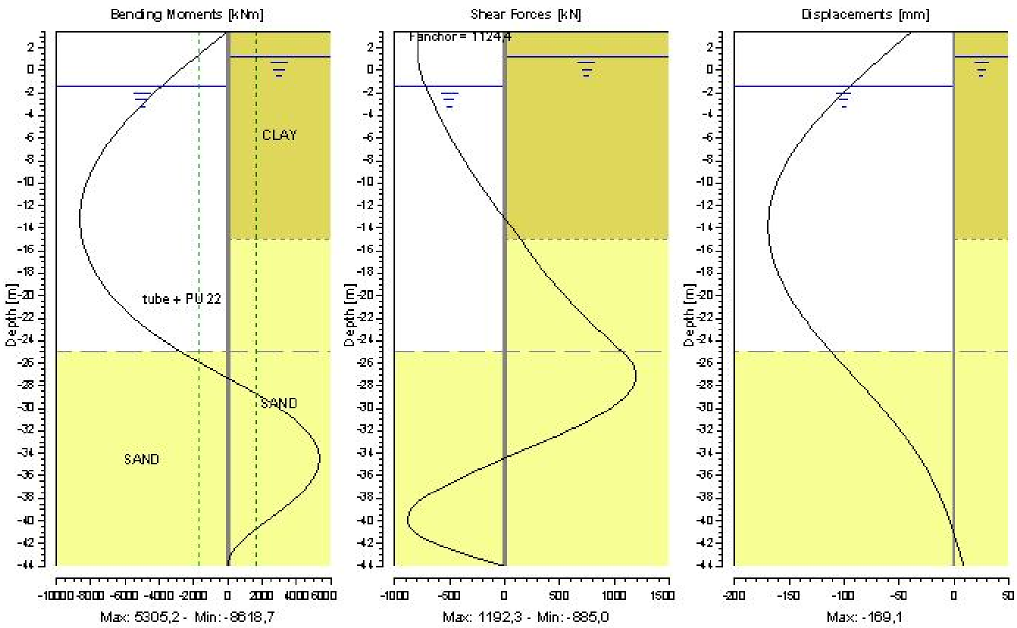


Figure D.10: Representative values of internal forces – case A2

Table D.19: Preliminary design of case A2

Primary Elements			System		
External Diameter (D)	**2020**	[mm]	Length	3,270	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	3,307E-02	$[m^4/m^1]$
Steel Quality	**X 70**	[-]	Stiffness	6,944E+06	$[kNm^2/m^1]$
Moment of Inertia (I)	1,075E+11	$[mm^4]$	Section Modulus (W)	0,032742	$[m^3/m]$

Table D.20: Loads acting on Case A2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	1283,6	[kN]
Vertical component of horizontal reaction (F_x)	89,6	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1124,4	[kN]
Total Axial Force (Nmax)	2740,4	[kN]
Maximum Bending Moment (M)	8618,7	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 395{,}59\ \text{N/mm}^2 < \sigma_y = 443\ \text{N/mm}^2$$

Case B1 – Soil Profile 2

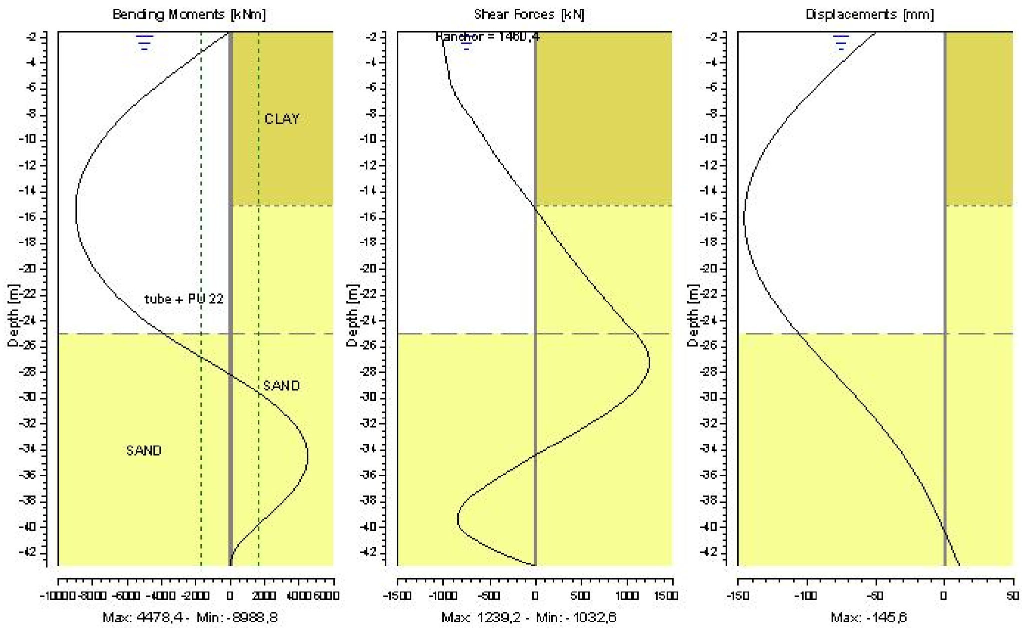


Figure D.11: Representative values of internal forces – case B1

Table D.21: Preliminary design of case B1

Primary Elements			System		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	4,150E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	8,715E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,434E+11	[mm^4]	Section Modulus (W)	0,03738	[m^3/m]

Table D.22: Loads acting on Case B1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2346,5	[kN]
Vertical component of horizontal reaction (F_x)	188,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1460,4	[kN]
Total Axial Force (Nmax)	4325,8	**[kN]**
Maximum Bending Moment (M)	8988,8	**[kNm]**

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 393{,}81 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case B2 – Soil Profile 2

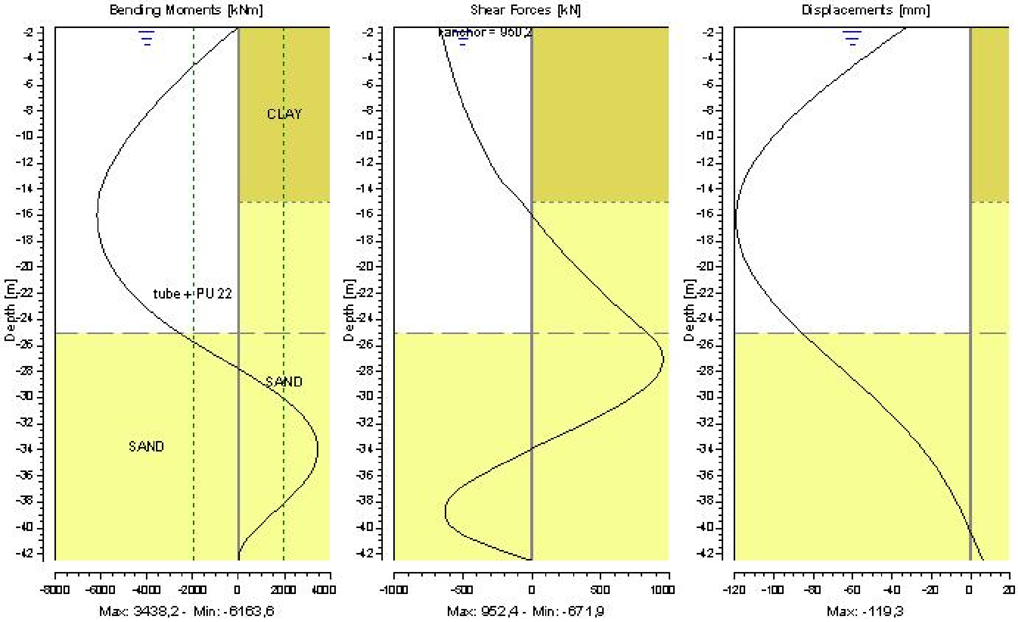


Figure D.12: Representative values of internal forces – case B2

Table D.23: Preliminary design of case B2

Primary Elements			System		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	2,221E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	4,664E+06	[kNm^2/m^1]
Moment of Inertia (I)	6,759E+10	[mm^4]	Section Modulus (W)	0,024406	[m^3/m]

Table D.24: Loads acting on Case B2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	2163,2	[kN]
Vertical component of horizontal reaction (F_x)	-291,9	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	943,3	[kN]
Total Axial Force (Nmax)	2944,8	[kN]
Maximum Bending Moment (M)	6113,6	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 395{,}34 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C1 – Soil Profile 2

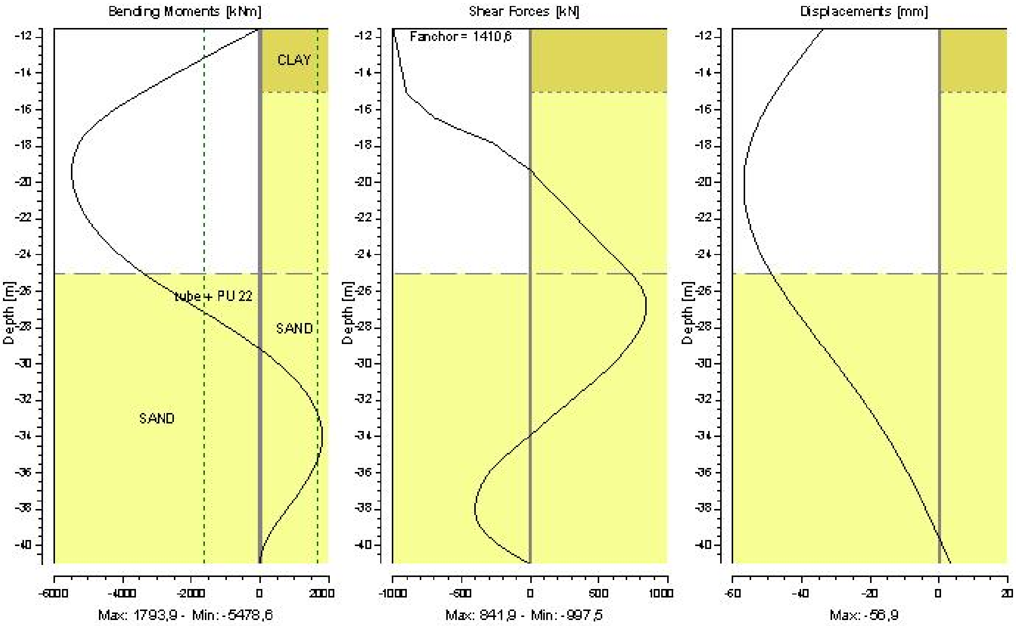


Figure D.13: Representative values of internal forces – case C1

Table D.25: Preliminary design of case C1

Primary Elements			**System**		
External Diameter (D)	**1920**	[mm]	Length	3,170	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,924E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	6,140E+06	[kNm^2/m^1]
Moment of Inertia (I)	9,209E+10	[mm^4]	Section Modulus (W)	0,030458	[m^3/m]

Table D.26: Loads acting on Case C1

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3776,1	[kN]
Vertical component of horizontal reaction (F_x)	573,6	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	1410,6	[kN]
Total Axial Force (Nmax)	6157,1	[kN]
Maximum Bending Moment (M)	5478,6	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 356{,}32 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C2 – Soil Profile 2

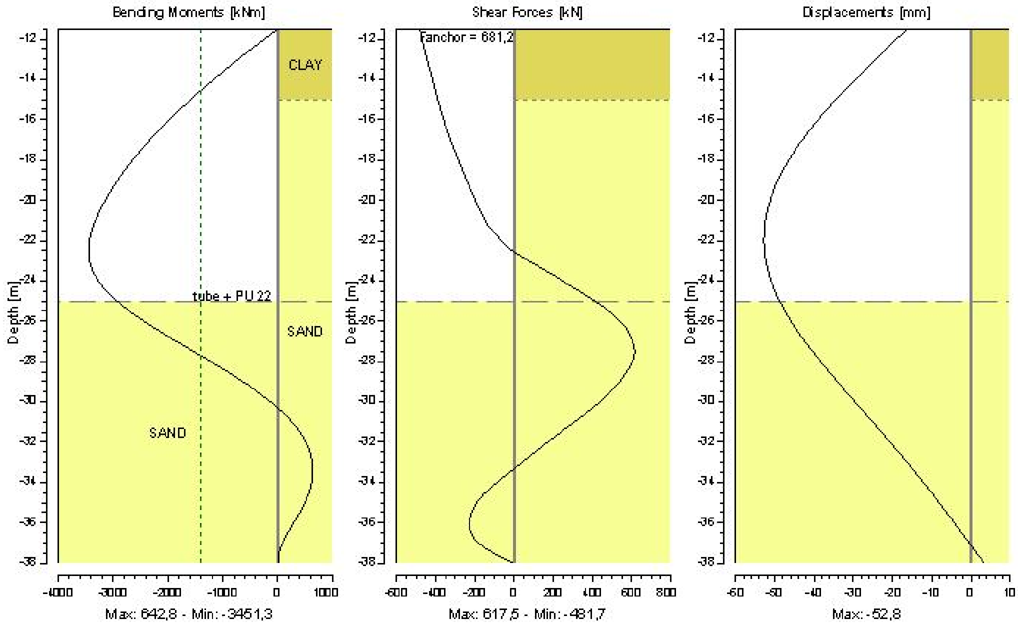


Figure D.14: Representative values of internal forces – case C2

Table D.27: Preliminary design of case C2

Primary Elements			System		
External Diameter (D)	**1420**	[mm]	Length	2,670	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	1,208E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	2,536E+06	[kNm^2/m^1]
Moment of Inertia (I)	3,165E+10	[mm^4]	Section Modulus (W)	0,017014	[m^3/m]

Table D.28: Loads acting on Case C2

Reactions from platform analysis		
Maximum vertical reaction (F_y)	3454	[kN]
Vertical component of horizontal reaction (F_x)	-171,568	[kN]
Reaction from Sheet pile analysis		
Vertical component of Anchor force T	681,2	[kN]
Total Axial Force (Nmax)	4065,5	[kN]
Maximum Bending Moment (M)	3451,3	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 371{,}47 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

APPENDIX E

Sheet Pile Analysis – Finite Element Method (PLAXIS)

Case D – Soil Profile 1

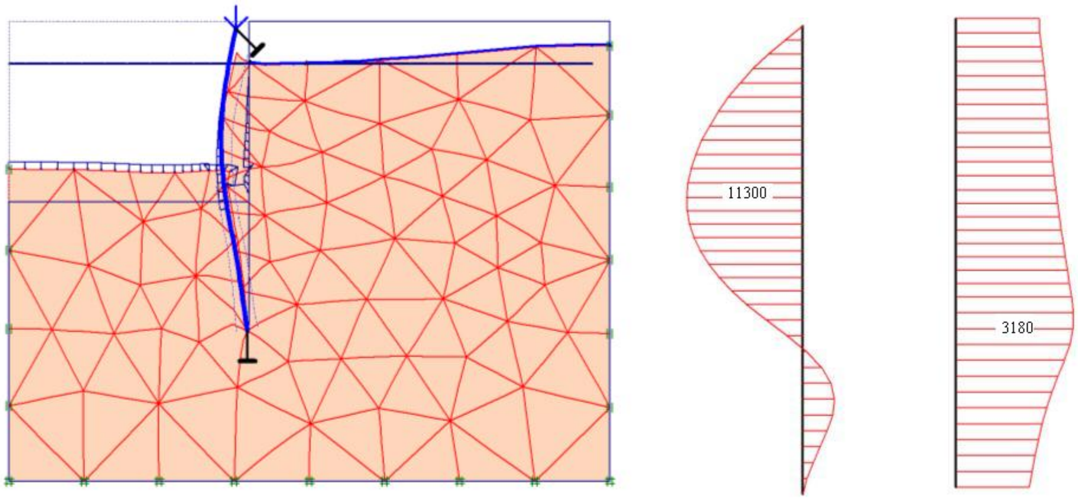


Figure E.1: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case D

Table E.1: Preliminary design of case D

Primary Elements			**System**		
External Diameter (D)	**2220**	[mm]	Length	3,470	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	4,709E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	9,888E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,628E+11	[mm^4]	Section Modulus (W)	0,042423	[m^3/m]

Table E.2: Loads acting on Case D

Loads		
Total Axial Force (Nmax)	3180,0	[kN]
Maximum Bending Moment (M)	11300,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 398{,}66 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A1 – Soil Profile 1

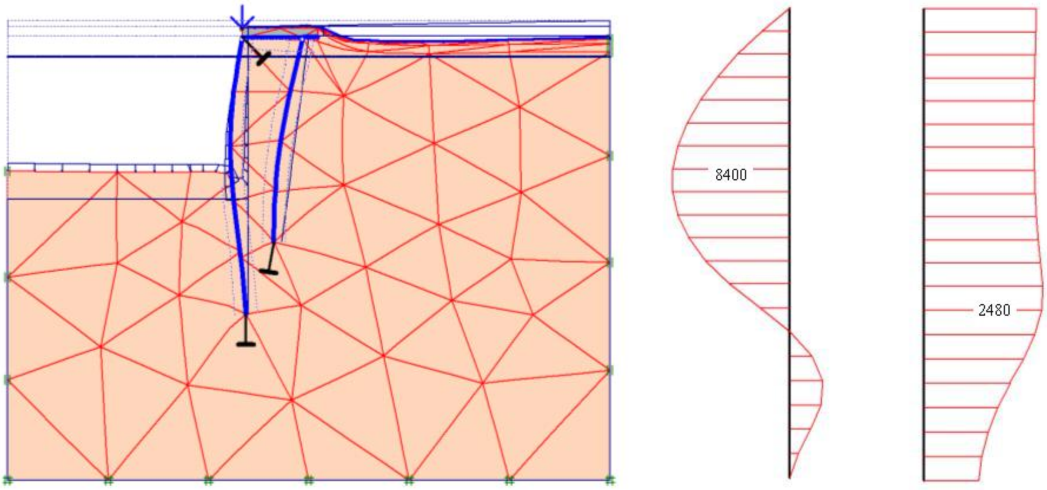


Figure E.2: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case A1

Table E.3: Preliminary design of case A1

Primary Elements			System		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,567E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,390E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,820E+10	[mm^4]	Section Modulus (W)	0,030458	[m^3/m]

Table E.4: Loads acting on Case A1

Loads		
Total Axial Force (Nmax)	2480,0	[kN]
Maximum Bending Moment (M)	8400,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 407{,}85 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A2 – Soil Profile 1

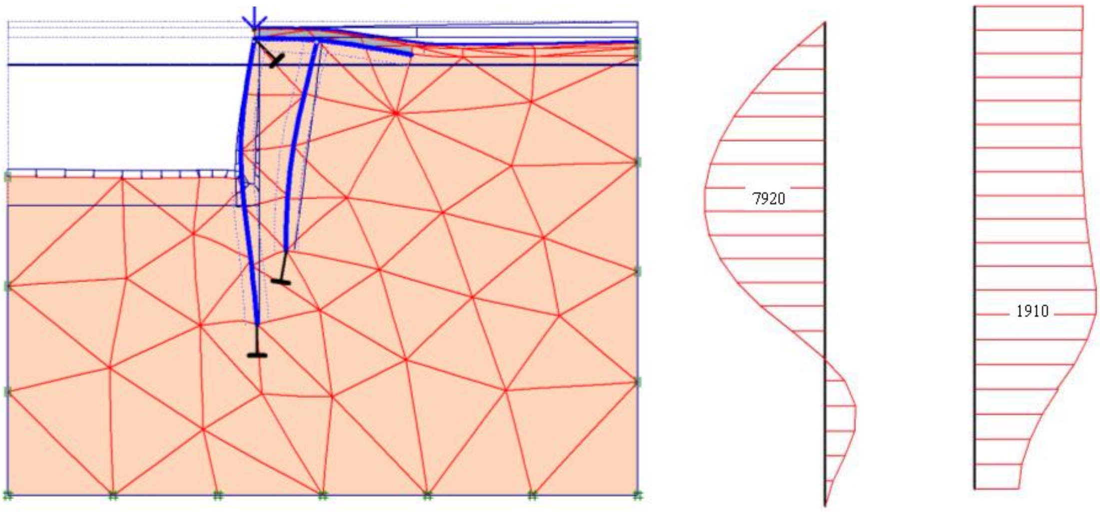


Figure E.3: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case A2

Table E.5: Preliminary design of case A2

Primary Elements			System		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,567E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,390E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,820E+10	[mm^4]	Section Modulus (W)	0,030458	[m^3/m]

Table E.6: Loads acting on Case A2

Loads		
Total Axial Force (Nmax)	1910,0	[kN]
Maximum Bending Moment (M)	7920,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 403{,}85\ \text{N/mm}^2 < \sigma_y = 443\ \text{N/mm}^2$$

Case B1 – Soil Profile 1

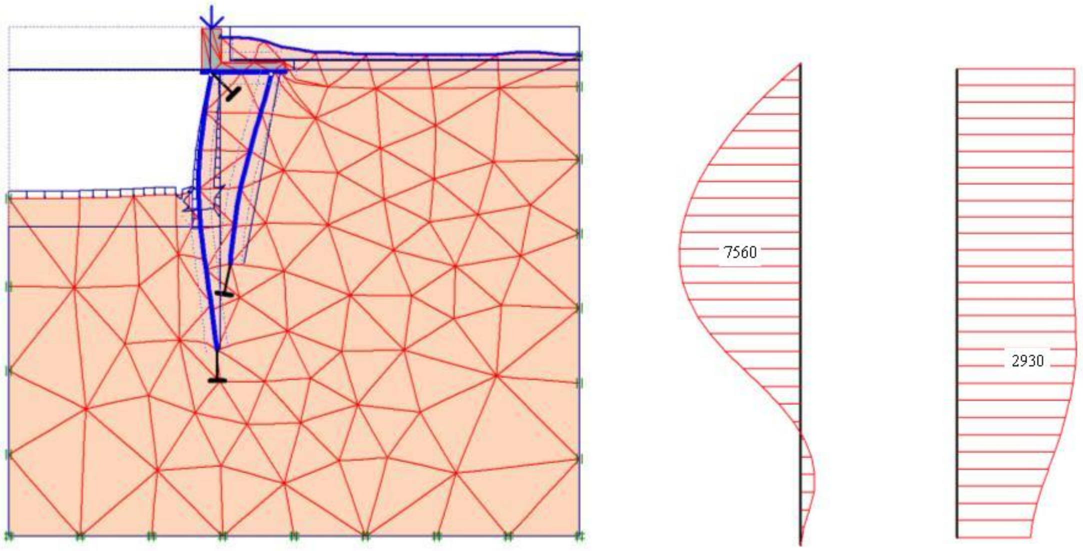


Figure E.4: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case B1

Table E.7: Preliminary design of case B1

Primary Elements			System		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,567E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,390E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,820E+10	[mm^4]	Section Modulus (W)	0,030458	[m^3/m]

Table E.8: Loads acting on Case B1

Loads		
Total Axial Force (Nmax)	2930,0	**[kN]**
Maximum Bending Moment (M)	7560,0	**[kNm]**

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 408{,}01 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case B2 – Soil Profile 1

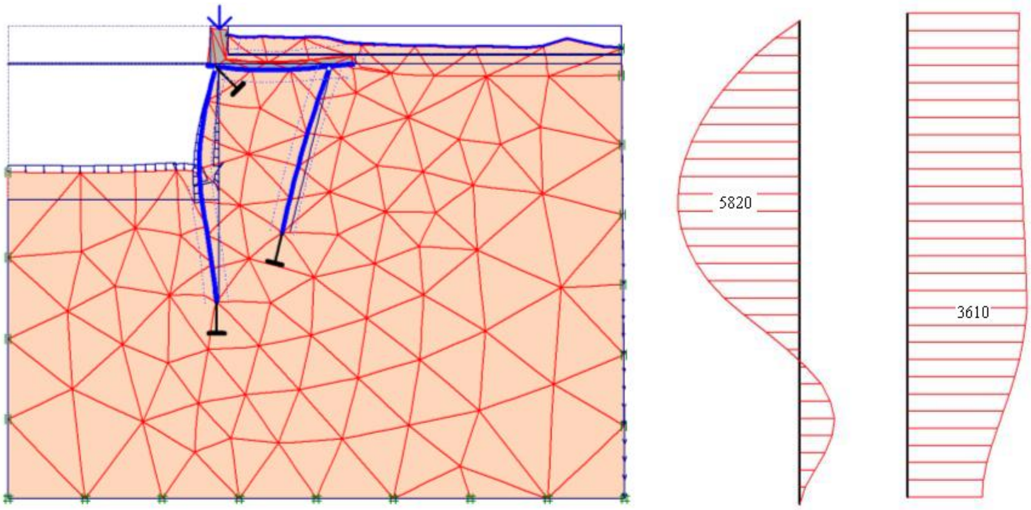


Figure E.5: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case B2

Table E.9: Preliminary design of case B2

Primary Elements			System		
External Diameter (D)	**1620**	[mm]	Length	2,870	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	1,929E-02	$[m^4/m^1]$
Steel Quality	**X 70**	[-]	Stiffness	4,050E+06	$[kNm^2/m^1]$
Moment of Inertia (I)	5,476E+10	$[mm^4]$	Section Modulus (W)	0,023814	$[m^3/m]$

Table E.10: Loads acting on Case B2

Loads		
Total Axial Force (Nmax)	3610,0	[kN]
Maximum Bending Moment (M)	5820,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 395{,}02\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case C1 – Soil Profile 1

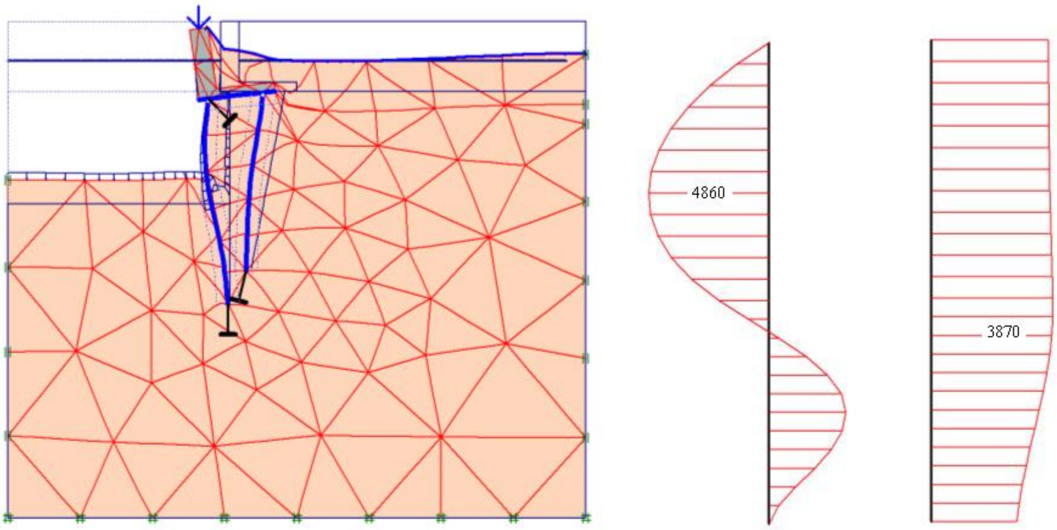


Figure E.6: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)– case C1

Table E.11: Preliminary design of case C1

Primary Elements			System		
External Diameter (D)	**1520**	[mm]	Length	2,770	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	1,647E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	3,459E+06	[kNm^2/m^1]
Moment of Inertia (I)	4,503E+10	[mm^4]	Section Modulus (W)	0,013689	[m^3/m]

Table E.12: Loads acting on Case C1

Loads		
Total Axial Force (Nmax)	3870,0	[kN]
Maximum Bending Moment (M)	4860,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 364{,}60\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case C2 – Soil Profile 1

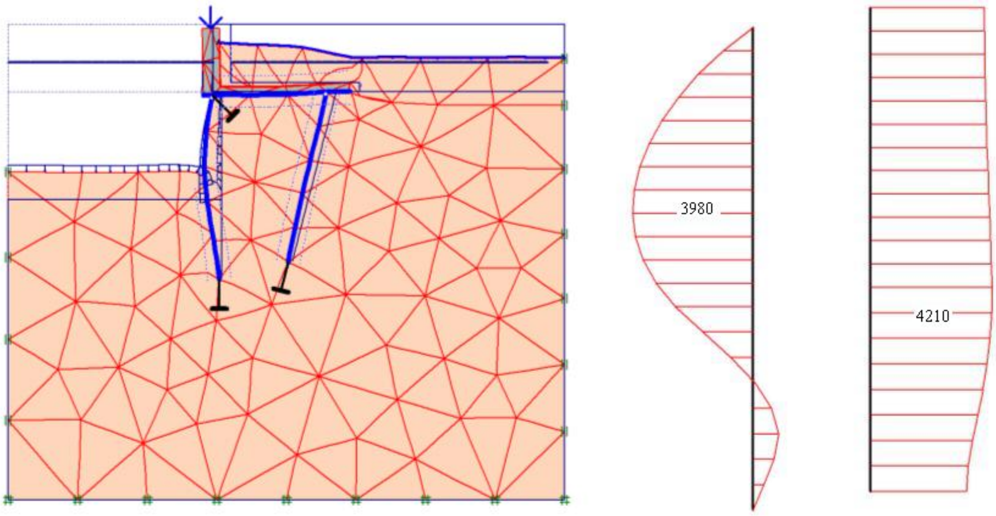


Figure E.7: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)– case C2

Table E.13: Preliminary design of case C2

Primary Elements			**System**		
External Diameter (D)	**1420**	[mm]	Length	2,670	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	1,391E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	2,921E+06	[kNm^2/m^1]
Moment of Inertia (I)	3,654E+10	[mm^4]	Section Modulus (W)	0,011872	[m^3/m]

Table E.14: Loads acting on Case C2

Loads		
Total Axial Force (Nmax)	4210,0	[kN]
Maximum Bending Moment (M)	3980,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 360{,}09 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case D – Soil Profile 2

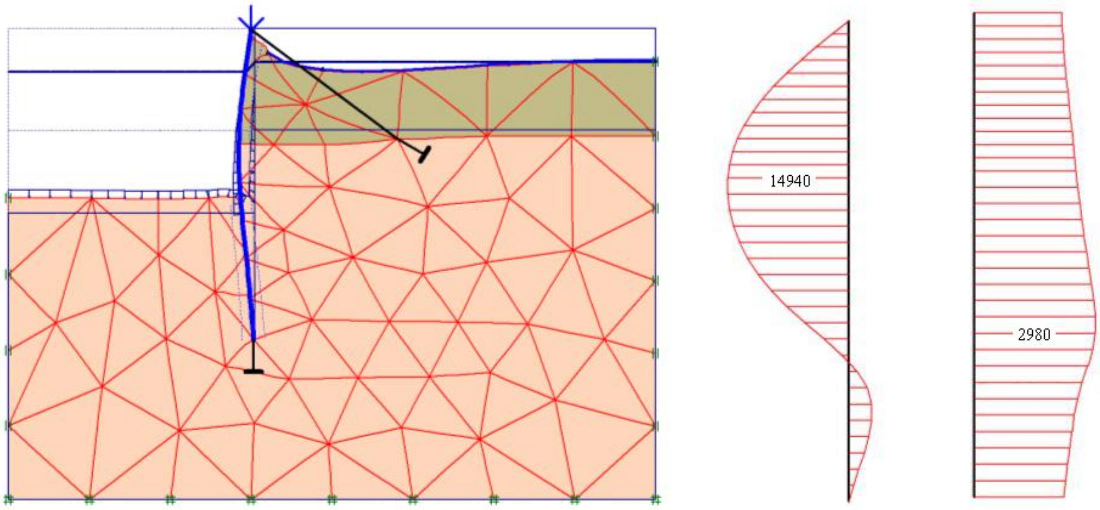


Figure E.8: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case D

Table E.15: Preliminary design of case D

Primary Elements			System		
External Diameter (D)	**2320**	[mm]	Length	3,570	[m]
Thickness (t)	**45**	[mm]	Moment of Inertia	5,847E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	1,228E+07	[kNm^2/m^1]
Moment of Inertia (I)	2,082E+11	[mm^4]	Section Modulus (W)	0,050405	[m^3/m]

Table E.16: Loads acting on Case D

Loads		
Total Axial Force (Nmax)	2980,0	[kN]
Maximum Bending Moment (M)	14940,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 428{,}34\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case A1 – Soil Profile 2

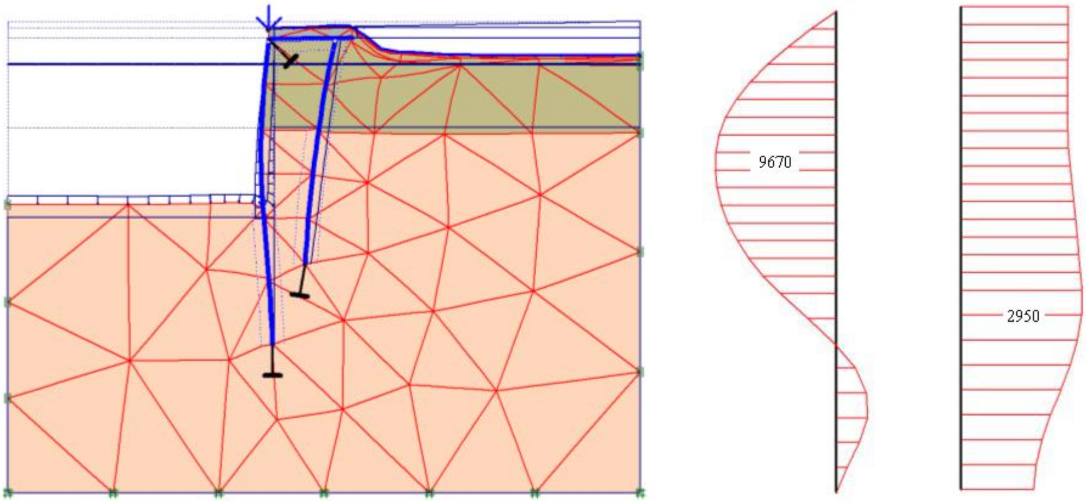


Figure E.9: Deformed mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m) – case A1

Table E.17: Preliminary design of case A1

Primary Elements			System		
External Diameter (D)	**2020**	[mm]	Length	3,270	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	3,748E-02	$[m^4/m^1]$
Steel Quality	**X 70**	[-]	Stiffness	7,872E+06	$[kNm^2/m^1]$
Moment of Inertia (I)	1,220E+11	$[mm^4]$	Section Modulus (W)	0,037108	$[m^3/m]$

Table E.18: Loads acting on Case A1

Loads		
Total Axial Force (Nmax)	2950,0	[kN]
Maximum Bending Moment (M)	9670,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 389{,}18 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case A2 – Soil Profile 2

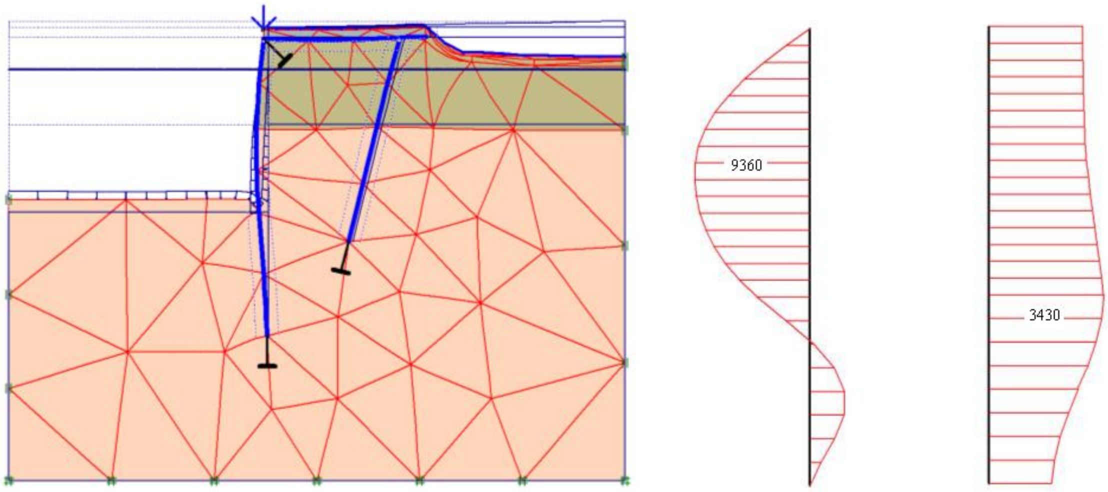


Figure E.10: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)– caseA2

Table E.19: Preliminary design of case A2

Primary Elements			**System**		
External Diameter (D)	**1920**	[mm]	Length	3,170	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	3,313E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	6,957E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,044E+11	[mm^4]	Section Modulus (W)	0,034510	[m^3/m]

Table E.20: Loads acting on Case A2

Loads		
Total Axial Force (Nmax)	3430,0	[kN]
Maximum Bending Moment (M)	9360,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 412{,}45\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case B1 – Soil Profile 2

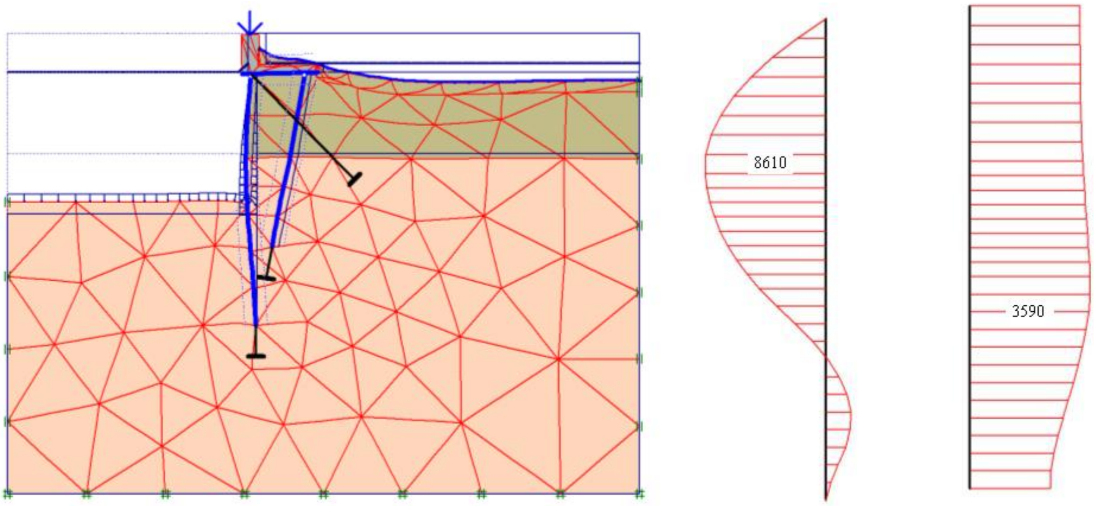


Figure E11: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)– case B1

Table E.21: Preliminary design of case B1

Primary Elements			**System**		
External Diameter (D)	**1920**	[mm]	Length	3,170	[m]
Thickness (t)	**40**	[mm]	Moment of Inertia	3,313E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	6,957E+06	[kNm^2/m^1]
Moment of Inertia (I)	1,044E+11	[mm^4]	Section Modulus (W)	0,034510	[m^3/m]

Table E.22: Loads acting on Case B1

Loads		
Total Axial Force (Nmax)	3590,0	**[kN]**
Maximum Bending Moment (M)	8610,0	**[kNm]**

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 389{,}99\ N/mm^2 < \sigma_y = 443\ N/mm^2$$

Case B2 – Soil Profile 2

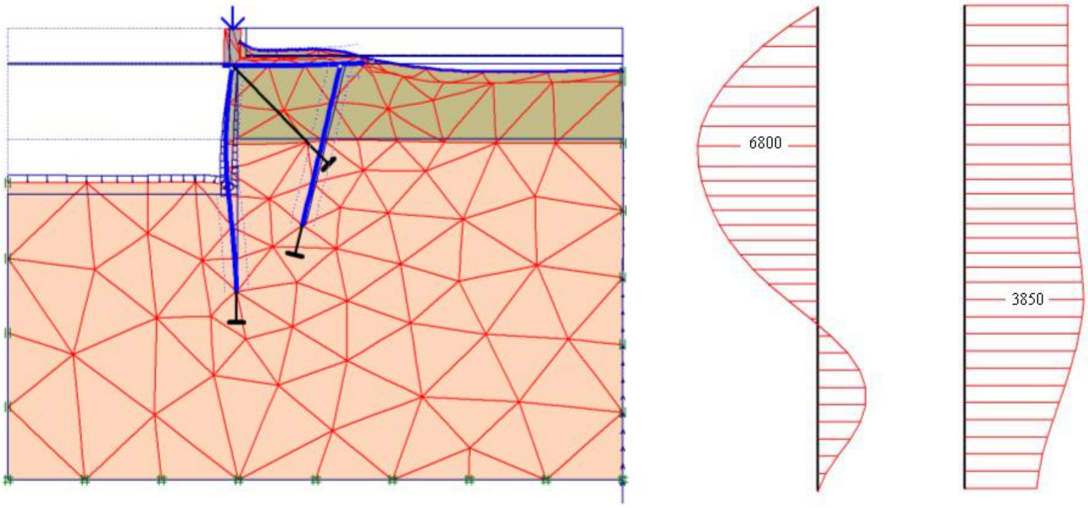


Figure E12: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)– case B2

Table E.23: Preliminary design of case B2

Primary Elements			**System**		
External Diameter (D)	**1820**	[mm]	Length	3,070	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	2,567E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	5,390E+06	[kNm^2/m^1]
Moment of Inertia (I)	7,820E+10	[mm^4]	Section Modulus (W)	0,028208	[m^3/m]

Table E.24: Loads acting on Case B2

Loads		
Total Axial Force (Nmax)	3850,0	[kN]
Maximum Bending Moment (M)	6800,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 391{,}70\ \text{N/mm}^2 < \sigma_y = 443\ \text{N/mm}^2$$

Case C1 – Soil Profile 2

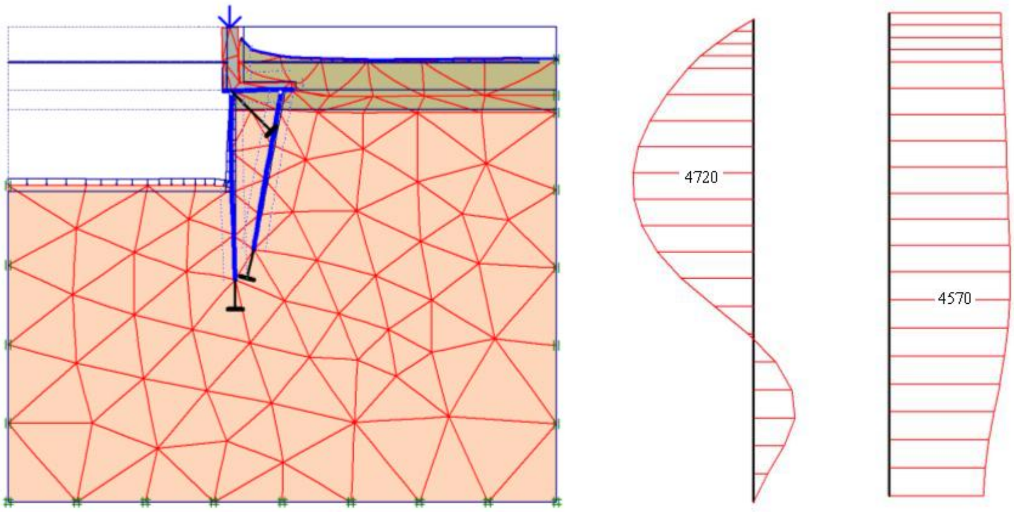


Figure E13: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)–case C1

Table E.25: Preliminary design of case C1

Primary Elements			**System**		
External Diameter (D)	**1620**	[mm]	Length	2,870	[m]
Thickness (t)	**35**	[mm]	Moment of Inertia	1,929E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	4,050E+06	[kNm^2/m^1]
Moment of Inertia (I)	5,476E+10	[mm^4]	Section Modulus (W)	0,023814	[m^3/m]

Table E.26: Loads acting on Case C1

Loads		
Total Axial Force (Nmax)	4570,0	[kN]
Maximum Bending Moment (M)	4720,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 355{,}54 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

Case C2 – Soil Profile 2

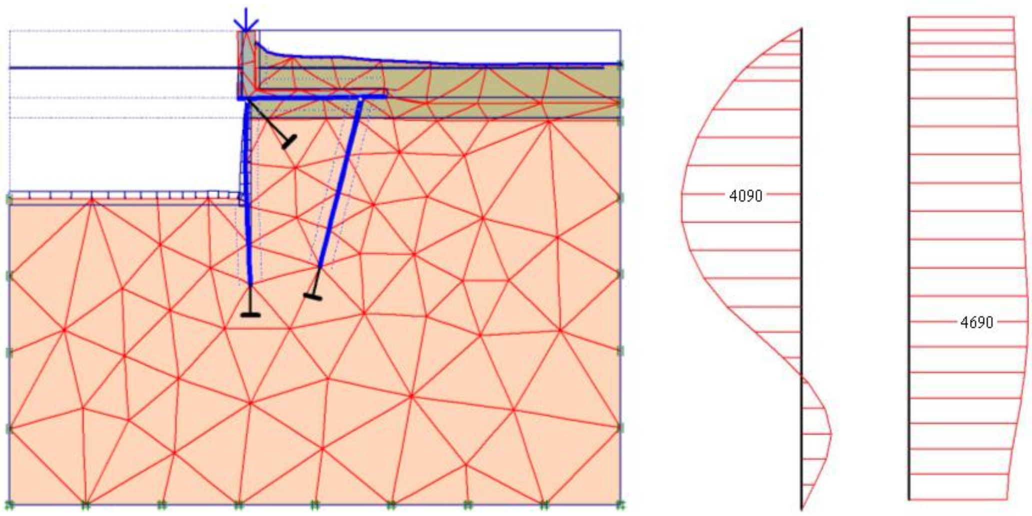


Figure E.14: Deformed Mesh/Diagrams of bending moment (KNm/m) and axial force (KN/m)–caseC2

Table E.27: Preliminary design of case C2

Primary Elements			**System**		
External Diameter (D)	**1620**	[mm]	Length	2,870	[m]
Thickness (t)	**30**	[mm]	Moment of Inertia	1,671E-02	[m^4/m^1]
Steel Quality	**X 70**	[-]	Stiffness	3,510E+06	[kNm^2/m^1]
Moment of Inertia (I)	4,737E+10	[mm^4]	Section Modulus (W)	0,023814	[m^3/m]

Table E.28: Loads acting on Case C2

Loads		
Total Axial Force (Nmax)	4690,0	[kN]
Maximum Bending Moment (M)	4090,0	[kNm]

Check of the yielding stress:

$$\sigma = \frac{M_{max;design}}{W_{system}} + \frac{N_d'}{A} = \frac{1.3 \cdot M_{max,}}{W_{system}} + \frac{1.3 \cdot N'}{A} = 340{,}09 \text{ N/mm}^2 < \sigma_y = 443 \text{ N/mm}^2$$

APPENDIX F

Blum Calculations
Free Earth– Fixed Earth Support Method

FREE EARTH SUPPORT - CASE D

Table F.1: Excel file – Free Earth Support – case D

x	T	final x	Final pile wall
12,7796	-1980,886	12,8	42,8
	Forces	Lever (top)	Moments (top)
1	-3124,77	38,5197	-120365,1786
2A	-6617,38	30,6497	-202820,8245
2B	7636,076	29,7497	227171,2152
3A	4611,837	28,5197	131528,3707
3B	-1710,48	29,7497	-50886,35816
4	718,6973	21,3898	15372,79108
T	-1980,886	0	0
Sum	1513,986		0,015646117

FREE EARTH SUPPORT - CASE A1

Table F.2: Excel file – Free Earth Support – case A1

x	T	final x	Final pile wall
12,436	-1276,381	12,4	40,9
	Forces	Lever (top)	Moments (top)
1	-2958,996819	36,7907	-108863,4656
2A	-6492,96648	28,9207	-187780,9192
2B	7502,38848	28,0207	210221,9268
3A	4222,905362	27,2907	115245,9026
3B	-1680,53522	28,0207	-47089,71721
4	1224,262957	29,7007	36361,42601
5	-540,6775614	33,4667	-18094,67572
T	-1276,381	0	0
Sum	1276,38072		0,477606849

FREE EARTH SUPPORT - CASE A2

Table F.3: Excel file – Free Earth Support – case A2

x	T	final x	Final pile wall
12,6552	-1178,898	12,7	41,2
	Forces	Lever (top)	Moments (top)
1	-3064,228147	36,9368	-113182,7822
2A	-6572,197635	29,0668	-191032,7542
2B	7587,538035	28,1668	213716,6663
3A	4268,251227	27,4368	117107,1553
3B	-1699,60872	28,1668	-47872,53889
4	668,1656356	32,2468	21546,20362
5	-9,022616716	31,23013333	-281,777523
T	-1178,898	0	0
Sum	1178,897779		0,172353473

FREE EARTH SUPPORT - CASE B1

Table F.4: Excel file – Free Earth Support – case B1

x	T	final x	Final pile wall
11,541	-1076,316424	11,5	35,0
	Forces	Lever (top)	Moments (top)
1	-2548,45823	31,194	-79496,60602
2	946,107	17,5205	16576,26769
3	1719,020353	23,364	40163,19154
4	1481,605344	25,764	38172,08009
5	-521,9580441	29,534	-15415,50887
T	-1076,316424	0	0
Sum	0		-0,575579157

FREE EARTH SUPPORT - CASE B2

Table F.5: Excel file – Free Earth Support – case B2

x	T	final x	Final pile wall
11,1355	-902,3297233	11,1	34,6
	Forces	Lever (top)	Moments (top)
1	-2372,521093	30,9236	-73367,05143
2	935,1585	17,3177	16194,84111
3	1679,465004	23,0736	38751,41569
4	660,2273117	27,8996	18420,12192
5	0	0	0
T	-902,3297233	0	0
Summation	0		-0,67270969

FREE EARTH SUPPORT - CASE C1

Table F.6: Excel file – Free Earth Support – case C1

x	T	final x	Final pile wall
10,4076	-797,3701955	10,4	28,9
	Forces	Lever (top)	Moments (top)
1	-2072,487036	25,4384	-52720,75421
2	780,5052	14,4538	11281,26606
3	1170,059073	19,2684	22545,16624
4	1194,31271	21,6784	25890,78866
5	-275,0197518	25,4384	-6996,062453
T	-797,3701955	0	0
Sum	0		0,404290348

FREE EARTH SUPPORT - CASE C2

Table F.7: Excel file – Free Earth Support – case C2

x	T	final x	Final pile wall
8,3755	-590,885	8,4	26,9
	Forces	Lever (top)	Moments (top)
1	-1342,160822	24,08366667	-32324,15384
2	725,6385	13,43775	9750,948753
3	1011,3595	17,91366667	18117,15697
4	196,0476598	22,72666667	4455,509815
T	-590,8848384	0	0
Sum	0		-0,538306646

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE D

Table F.7: Excel file – Fixed Earth Support – case D

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	62,16	1,850	31,980	114,996	1987,8768
2	34,498	2,467	31,363	85,106566	1081,960774
3	95,709	5,050	28,780	483,33045	2754,50502
4	46,645	5,500	28,330	256,5475	1321,45285
5	1652	18,200	15,630	30066,4	25820,76
6	779,838	22,130	11,700	17257,81494	9124,1046
7	260,609	31,276	2,554	8150,807084	665,595386
8	T	0	33,830	0	0
9	R	33,830	0	0	0
T	R		Sum	56415,00254	42756,25543
1730,76	1667,60				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE A1

Table F.8: Excel file – Fixed Earth Support – case A1

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	12,188	1,466	30,874	17,86761	376,2923
2	29,916	3,550	28,790	106,2018	861,2816
3	53,204	4,000	28,340	212,816	1507,801
4	117,368	6,060	26,280	711,2501	3084,431
5	1,914	6,446	25,894	12,33764	49,56112
6	589,703	12,875	19,465	7592,426	11478,57
7	316,849	14,760	17,580	4676,691	5570,205
8	1078,455	23,515	8,825	25359,87	9517,365
9	139,181	25,176	7,164	3504,021	997,0927
10	260,612	29,776	2,564	7759,983	668,2092
11	T	0	32,340	0	0
12	R	32,340	0	0	0
T	R		Sum	49953,46	34110,81
1054,76	1544,63				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE A2

Table F.9: Excel file – Fixed Earth Support – case A2

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	12,197	1,100	31,089	13,4167	379,192533
2	29,938	3,550	28,639	106,2799	857,394382
3	46,656	4,000	28,189	186,624	1315,185984
4	435,205	9,665	22,524	4206,256325	9802,55742
5	127,139	11,253	20,936	1430,695167	2661,782104
6	1017,683	21,465	10,724	21844,5656	10913,63249
7	383,68	23,810	8,379	9135,4208	3214,85472
8	234,009	29,729	2,460	6956,853561	575,66214
9	T	0	32,189	0	0
10	R	32,189	0	0	0
T	**R**		Sum	43880,11205	29720,26178
923,30	1363,20				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE B1

Table F.10: Excel file – Fixed Earth Support – case B1

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	194,94	3,610	23,720	703,7334	4623,977
2	72,979	4,810	22,520	351,02899	1643,487
3	534,013	12,875	14,455	6875,41738	7719,158
4	423,876	14,760	12,570	6256,40976	5328,121
5	607,195	21,015	6,315	12760,2029	3834,436
6	34,581	21,843	5,487	755,352783	189,7459
7	260,608	24,776	2,554	6456,82381	665,5928
8	T	0	27,330	0	00
9	R	27,330	0	0	0
T	**R**		Sum	34158,969	24004,52
878,32	1249,87				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE B2

Table F.11: Excel file – Fixed Earth Support – case B2

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	389,61	7,215	19,555	2811,03615	7618,82355
2	291,515	9,620	17,150	2804,3743	4999,48225
3	611,354	18,965	7,805	11594,32861	4771,61797
4	192,81	20,476	6,294	3947,97756	1213,54614
5	179,719	24,590	2,180	4419,29021	391,78742
6	T	0	26,770	0	0
7	R	26,77	0	0	0
T	**R**		Sum	25577,00683	18995,25733
709,57	955,44				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE C1

Table F.12: Excel file – Fixed Earth Support – case C1

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	194,94	3,610	18,730	703,7334	3651,2262
2	72,979	4,813	17,527	351,247927	1279,102933
3	532,596	12,860	9,480	6849,18456	5049,01008
4	501,238	14,740	7,600	7388,24812	3809,4088
5	261,288	19,780	2,560	5168,27664	668,89728
6	T	0	22,340	0	0
7	R	22,340	0	0	0
T	**R**		Sum	20460,69065	14457,64529
647,16	915,88				

FIXED EARTH SUPPORT - EQUIVALENT BEAM METHOD- CASE C2

Table F.13 Excel file – Fixed Earth Support – case C2

	FORCES	Arm (A)	Arm (B)	Moment (A)	Moment (B)
1	389,61	7,215	13,985	2811,03615	5448,69585
2	291,515	9,620	11,58	2804,3743	3375,7437
3	274,334	16,465	4,735	4516,90931	1298,97149
4	44,153	17,143	4,057	756,914879	179,128721
5	120,286	19,400	1,800	2333,5484	216,5148
6	T	0	21,200	0	0
7	R	21,200	0	0	
T	**R**		Sum	13222,78304	10519,05456
496,18	623,72				

APPENDIX G

Total Cost per meter Quay Wall
Different Analysis Methods / Soil Profiles

Table G.1 Excel file – Combi Wall Cost – Soil Profile 1 - MSheet

	density	area	depth	system length	n. of cross sections	weight (tons)	total weight in 100m	COST (100 m)	COST/m (euros)	THOUSANDS (euros)
A1	7843,00	0,1601	47,50	3,57	28,01	59,65	1670,81	1722601,14	17226,011	**17,23**
A2	7843,00	0,1375	47,5	3,37	29,67	51,22	1519,82	1566933,01	15669,330	**15,67**
B1	7843,00	0,143	41,5	3,47	28,82	46,54	1341,12	1382692,74	13826,927	**13,83**
B2	7843,00	0,107	41	3,07	32,57	34,40	1120,40	1155133,34	11551,333	**11,55**
C1	7843,00	0,1155	29,5	2,97	33,67	26,72	899,78	927675,21	9276,752	**9,28**
C2	7843,00	0,0693	26,5	2,47	40,49	14,41	583,50	601587,34	6015,873	**6,02**
D	7843,00	0,1843	49	3,57	28,01	70,82	1983,63	2045119,39	20451,194	**20,45**

Table G.2 Excel file – Combi Wall Cost – Soil Profile 2 - MSheet

	density	area	depth	system length	n. of cross sections	weight (tons)	total weight in 100m	COST (100 m)	COST/m (euros)	THOUSANDS (euros)
A1	7843,00	0,1601	47,50	3,47	28,82	59,65	1718,96	1772243,83	17722,44	**17,72**
A2	7843,00	0,132	47,5	3,27	30,58	49,17	1503,69	1550307,67	15503,08	**15,50**
B1	7843,00	0,143	41,5	3,47	28,82	46,54	1341,12	1382692,74	13826,93	**13,83**
B2	7843,00	0,107	41	3,07	32,57	34,40	1120,40	1155133,34	11551,33	**11,55**
C1	7843,00	0,1265	29,5	3,17	31,55	29,27	923,23	951846,01	9518,46	**9,52**
C2	7843,00	0,0881	26,5	2,67	37,45	18,32	685,99	707260,74	7072,61	**7,07**
D	7843,00	0,1772	49	3,47	28,82	68,10	1962,55	2023385,14	20233,85	**20,23**

Table G.3 Excel file – Combi Wall Cost – Soil Profile 1 - PLAXIS

	density	area	depth	system length	n. of cross sections	weight (tons)	total weight in 100m	COST (100 m)	COST/m (euros)	THOUSANDS (euros)
A1	7843,00	0,1265	47,50	3,17	31,55	47,12	1486,55	1532633,40	15326,33	**15,33**
A2	7843,00	0,121	47,5	3,07	32,57	45,08	1468,29	1513807,72	15138,08	**15,14**
B1	7843,00	0,121	41,5	3,07	32,57	39,38	1282,82	1322589,91	13225,90	**13,23**
B2	7843,00	0,11	41	2,87	34,84	35,37	1232,55	1270758,90	12707,59	**12,71**
C1	7843,00	0,1045	29,5	2,77	36,10	24,18	872,95	900014,04	9000,14	**9,00**
C2	7843,00	0,099	26,5	2,67	37,45	20,58	770,77	794667,14	7946,67	**7,95**
D	7843,00	0,1601	49	3,47	28,82	61,53	1773,24	1828209,42	18282,09	**18,28**

Table G.4 Excel file – Combi Wall Cost – Soil Profile 2 - PLAXIS

	density	area	depth	system length	n. of cross sections	weight (tons)	total weight in 100m	COST (100 m)	COST/m (euros)	THOUSANDS (euros)
A1	7843,00	0,1476	47,50	3,27	30,58	54,97	1681,00	1733109,29	17331,09	**17,33**
A2	7843,00	0,1413	47,5	3,17	31,55	52,63	1660,22	1711690,23	17116,90	**17,12**
B1	7843,00	0,1413	41,5	3,17	31,55	45,98	1450,51	1495476,73	14954,77	**14,95**
B2	7843,00	0,121	41	3,07	32,57	38,91	1267,37	1306655,09	13066,55	**13,07**
C1	7843,00	0,11	29,5	2,87	34,84	25,45	886,83	914326,53	9143,27	**9,14**
C2	7843,00	0,0975	26,5	2,87	34,84	20,27	706,40	728299,26	7282,99	**7,28**
D	7843,00	0,1843	49	3,57	28,01	70,82	1983,63	2045119,39	20451,19	**20,45**

Table G.5 Excel file – Combi Wall Cost – Soil Profile 1 - Blum

	density	area	depth	system length	n. of cross sections	weight (tons)	total weight in 100m	COST (100 m)	COST/m (euros)	THOUSANDS (euros)
A1	7843,00	0,1540	47,50	3,67	27,25	57,36	1562,92	1611372,97	16113,73	**16,11**
A2	7843,00	0,1375	47,5	3,37	29,67	51,22	1519,82	1566933,01	15669,33	**15,67**
B1	7843,00	0,1305	41,5	3,57	28,01	42,48	1189,93	1226820,96	12268,21	**12,27**
B2	7843,00	0,1117	41	3,17	31,55	35,91	1132,84	1167952,89	11679,53	**11,68**
C1	7843,00	0,1117	29,5	3,17	31,55	25,84	815,09	840356,35	8403,56	**8,40**
C2	7843,00	0,0881	26,5	2,67	37,45	18,32	685,99	707260,74	7072,61	**7,07**
D	7843,00	0,1969	49	3,75	26,67	75,67	2017,87	2080421,86	20804,22	**20,80**

Table G.5 Excel file – Platform Cost

	h(m) depth	b (m) width	3rd dimension	Volume (m^3)	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	100	1875	6468,75	6,46875
A2	1,5	25	100	3750	12937,5	12,9375
B1	6,5	9,5	100	3375	11643,75	11,64375
B2	6,5	22	100	5250	18112,5	18,1125
C1	11,5	9,5	100	4875	16818,75	16,81875
C2	11,5	22	100	6750	23287,5	23,2875
D	0	0	100	0	0	0

Table G.6 Excel file – Anchor Cost

	length (m)	euros/anchor	num of anchors in 100m	COST/m	THOUSANDS (euros)
A1	50	10000	40	4000	**4**
A2	50	10000	40	4000	**4**
B1	50	10000	40	4000	**4**
B2	50	10000	40	4000	**4**
C1	35	7000	40	2800	**2,8**
C2	35	7000	40	2800	**2,8**
D	50	10000	40	4000	**4**

Table G.7 Excel file – Bearing Pile Cost

	Length (m)	Cross Section	Volume (m3)	Number of Piles in 100m	COST/m (euros)	THOUSANDS (euros)
A1	35	0,35	12,25	40	1176	1,176
A2	70	0,35	24,5	40	2352	2,352
B1	25	0,35	8,75	40	840	0,84
B2	50	0,35	17,5	40	1680	1,68
C1	25	0,35	8,75	40	840	0,84
C2	50	0,35	17,5	40	1680	1,68
D	-	-	-	-	-	-

Table G.8 Excel file – Excavation phase – Soil Profile 1

Excavation phase	depth	length	area	extra area	total	third dimension	Volume m3	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	18,75	2,25	23,25	100	2325	58,125	0,058125
A2	1,5	25	37,5	2,25	42	100	4200	105	0,105
B1	6,5	12,5	81,25	42,25	165,75	100	16575	414,375	0,414375
B2	6,5	25	162,5	42,25	247	100	24700	617,5	0,6175
C1	11,5	12,5	143,75	132,25	408,25	100	40825	1020,625	1,020625
C2	11,5	25	287,5	132,25	552	100	55200	1380	1,38

Table G.9 Excel file – Refill phase – Soil Profile 1

Refill phase	depth	length	front thickness	slab thickness	concrete area	refil area	extra refill area	third dimension	Total area	Volume m3	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	0	0	18,75	0	2,25	100	2,25	225	5,625	0,005625
A2	1,5	12,5	0	0	18,75	0	2,25	100	2,25	225	5,625	0,005625
B1	6,5	12,5	3	1,5	33,75	47,5	42,25	100	89,75	8975	224,375	0,224375
B2	6,5	25	3	1,5	52,5	110	42,25	100	152,25	15225	380,625	0,380625
C1	11,5	12,5	3	1,5	48,75	95	132,25	100	227,25	22725	568,125	0,568125
C2	11,5	25	3	1,5	67,5	220	132,25	100	352,25	35225	880,625	0,880625

Table G.10 Excel file – Total Excavation&Refill Cost – Soil Profile 1

	Total Cost (THOUSANDS euros)
A1	0,06
A2	0,11
B1	0,64
B2	1,00
C1	1,59
C2	2,26

Table G.11 Excel file – Excavation phase – Soil Profile 2

Excavation phase	depth	length	area	extra area	total	third dimension	Volume m3	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	18,75	2,25	23,25	100	2325	81,375	**0,08**
A2	1,5	25	37,5	2,25	42	100	4200	147	**0,15**
B1	6,5	12,5	81,25	42,25	165,75	100	16575	580,125	**0,58**
B2	6,5	25	162,5	42,25	247	100	24700	864,5	**0,86**
C1	11,5	12,5	143,75	132,25	408,25	100	40825	1428,875	**1,43**
C2	11,5	25	287,5	132,25	552	100	55200	1932	**1,93**

Table G.12 Excel file – Refill phase – Soil Profile 2

Refill phase	depth	length	front thickness	slab thickness	concrete area	refil area	extra refill area	third dimension	Total area	Volume m3	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	0	0	18,75	0	2,25	100	2,25	225	22,5	**0,02**
A2	1,5	12,5	0	0	18,75	0	2,25	100	2,25	225	22,5	**0,02**
B1	6,5	12,5	3	1,5	33,75	47,5	42,25	100	89,75	8975	897,5	**0,90**
B2	6,5	25	3	1,5	52,5	110	42,25	100	152,25	15225	1522,5	**1,52**
C1	11,5	12,5	3	1,5	48,75	95	132,25	100	227,25	22725	2272,5	**2,27**
C2	11,5	25	3	1,5	67,5	220	132,25	100	352,25	35225	3522,5	**3,52**

Table G.13 Excel file – Total Excavation&Refill Cost – Soil Profile 2

	Total Cost (THOUSANDS euros)
A1	**0,10**
A2	**0,17**
B1	**1,48**
B2	**2,39**
C1	**3,70**
C2	**5,45**

Table G.14 Excel file – Dewatering Cost

	depth	length	water level	area	extra area	total	volume m3	COST/m (euros)	THOUSANDS (euros)
A1	1,5	12,5	0	0	0	0	0	0	**0,00**
A2	1,5	25	0	0	0	0	0	0	**0,00**
B1	6,5	12,5	3,7	35	7,84	50,68	5068	82,6084	**0,08**
B2	6,5	25	3,7	70	7,84	85,68	8568	139,6584	**0,14**
C1	11,5	12,5	3,7	97,5	60,84	219,18	21918	357,2634	**0,36**
C2	11,5	25	3,7	195	60,84	316,68	31668	516,1884	**0,52**

Table G.15 Excel file – Total Cost – Soil Profile 1 - MSheet

	Combi wall	Platform	Anchors	Bearing Piles	Excavation-Refill	Dewatering	Total Cost (THOUSANDS euros)
A1	17,23	6,46875	4	1,176	0,06375	0	28,93
A2	15,67	12,9375	4	2,352	0,110625	0	35,07
B1	13,83	11,64375	4	0,84	0,63875	0,0826084	31,03
B2	11,55	18,1125	4	1,68	0,998125	0,1396584	36,48
C1	9,28	16,81875	2,8	0,84	1,58875	0,3572634	31,68
C2	6,02	23,2875	2,8	1,68	2,260625	0,5161884	36,56
D	20,45	0	4	0	0	0	24,45

Table G.16 Excel file – Total Cost – Soil Profile 2 - MSheet

	Combi wall	Platform	Anchors	Bearing Piles	Excavation-Refill	Dewatering	Total Cost (THOUSANDS euros)
A1	17,72	6,46875	4	1,176	0,103875	0	29,47
A2	15,50	12,9375	4	2,352	0,1695	0	34,96
B1	13,83	11,64375	4	0,84	1,477625	0,0826084	31,87
B2	11,55	18,1125	4	1,68	2,387	0,1396584	37,87
C1	9,52	16,81875	2,8	0,84	3,701375	0,3572634	34,04
C2	7,07	23,2875	2,8	1,68	5,4545	0,5161884	40,81
D	20,23	0	4	0	0	0	24,23

Table G.17 Excel file – Total Cost – Soil Profile 1 - PLAXIS

	Combi wall	Platform	Anchors	Bearing Piles	Excavation-Refill	Dewatering	Total Cost (THOUSANDS euros)
A1	15,33	6,46875	4	1,176	0,06375	0	27,03
A2	15,14	12,9375	4	2,352	0,110625	0	34,54
B1	13,23	11,64375	4	0,84	0,63875	0,0826084	30,43
B2	12,71	18,1125	4	1,68	0,998125	0,1396584	37,64
C1	9,00	16,81875	2,8	0,84	1,58875	0,3572634	31,40
C2	7,95	23,2875	2,8	1,68	2,260625	0,5161884	38,49
D	18,28	0	4	0	0	0	22,28

Table G.18 Excel file – Total Cost – Soil Profile 2 - PLAXIS

	Combi wall	Platform	Anchors	Bearing Piles	Excavation-Refill	Dewatering	Total Cost (THOUSANDS euros)
A1	17,33	6,46875	4	1,176	0,103875	0	29,08
A2	17,12	12,9375	4	2,352	0,1695	0	36,58
B1	14,95	11,64375	4	0,84	1,477625	0,0826084	33,00
B2	13,07	18,1125	4	1,68	2,387	0,1396584	39,39
C1	9,14	16,81875	2,8	0,84	3,701375	0,3572634	33,66
C2	7,28	23,2875	2,8	1,68	5,4545	0,5161884	41,02
D	20,45	0	4	0	0	0	24,45

Table G.19 Excel file – Total Cost – Soil Profile 1 - Blum

	Combi wall	Platform	Anchors	Bearing Piles	Excavation-Refill	Dewatering	Total Cost (THOUSANDS euros)
A1	16,11	6,46875	4	1,176	0,06375	0	27,82
A2	15,67	12,9375	4	2,352	0,110625	0	35,07
B1	12,27	11,64375	4	0,84	0,63875	0,0826084	29,47
B2	11,68	18,1125	4	1,68	0,998125	0,1396584	36,61
C1	8,40	16,81875	2,8	0,84	1,58875	0,3572634	30,81
C2	7,07	23,2875	2,8	1,68	2,260625	0,5161884	37,62
D	20,80	0	4	0	0	0	24,80

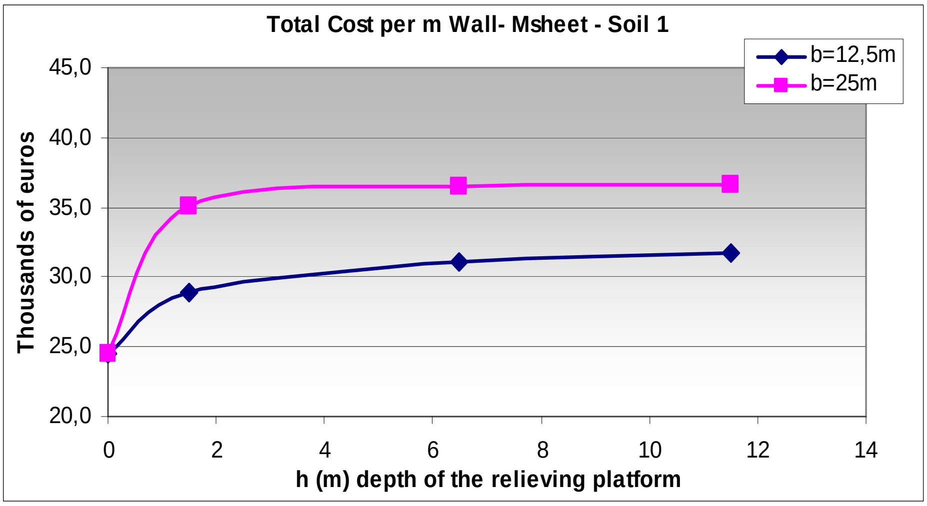


Figure G.1: Total Cost – Msheet – Soil Profile 1

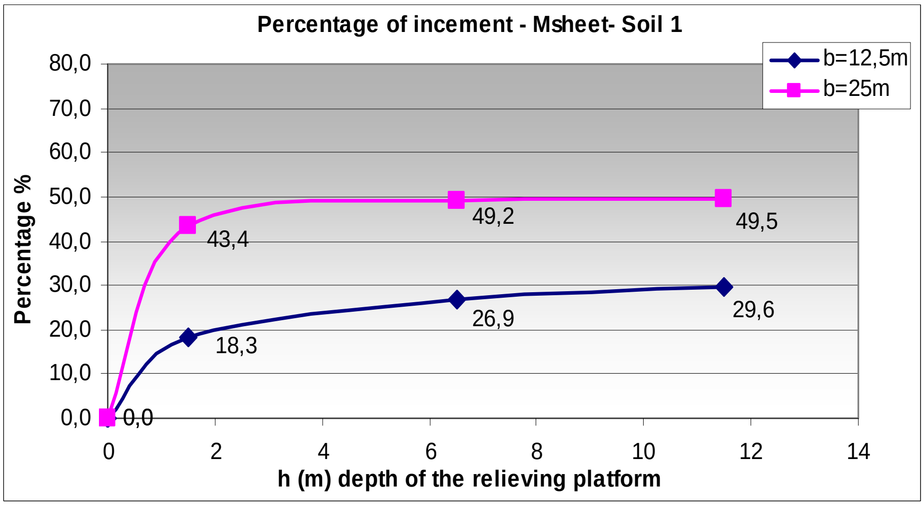


Figure G.2: Percentage of increment – Msheet – Soil Profile 1

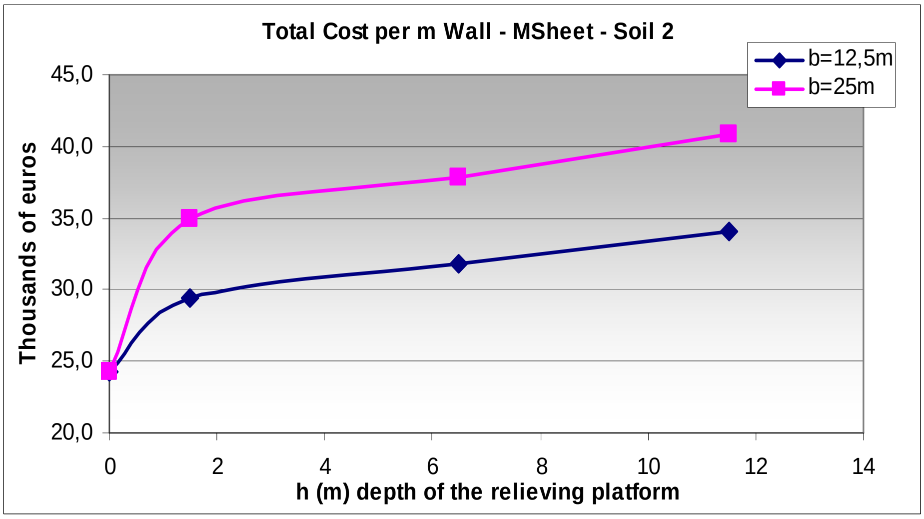


Figure G.3: Total Cost – Msheet – Soil Profile 2

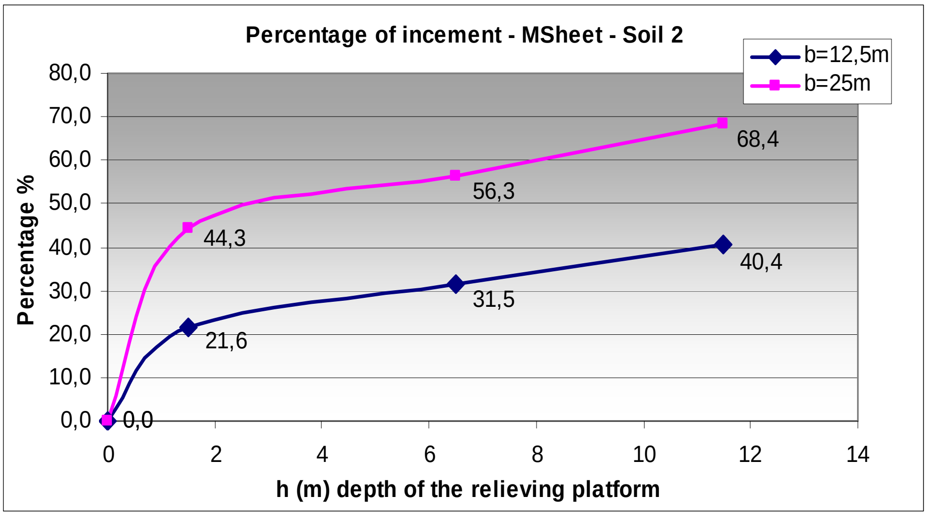


Figure G.4: Percentage of increment – Msheet – Soil Profile 2

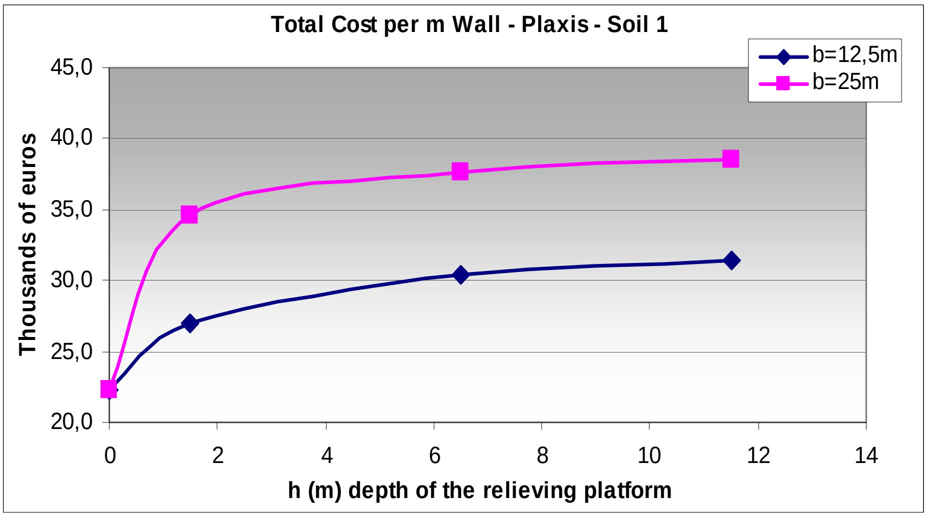


Figure G.5: Total Cost – PLAXIS – Soil Profile 1

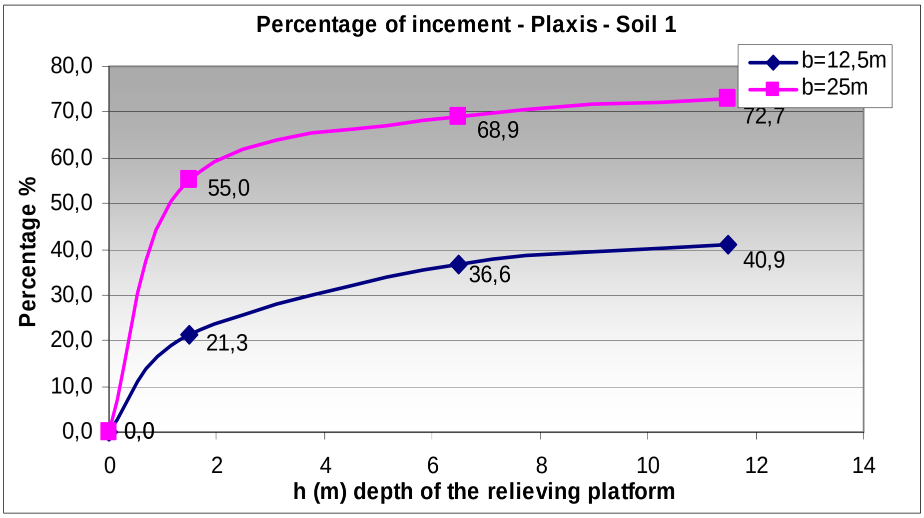


Figure G.6: Percentage of increment – PLAXIS– Soil Profile 1

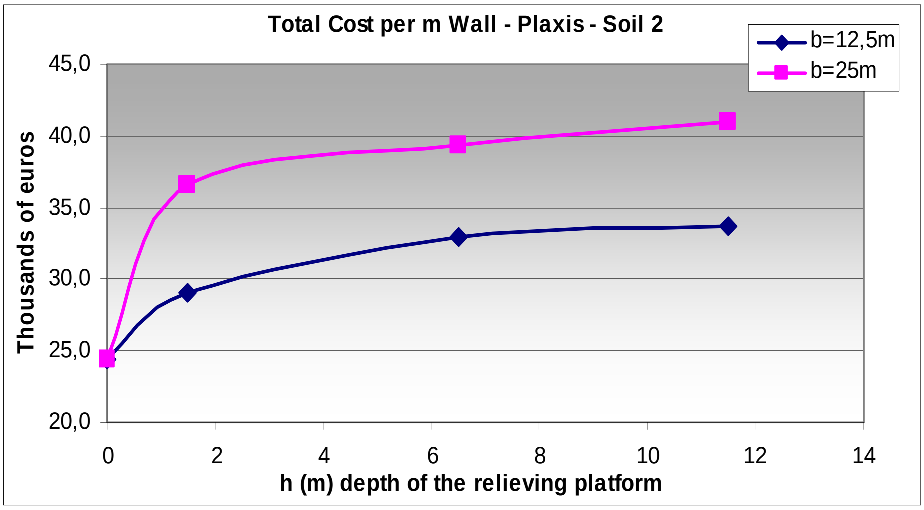


Figure G.7: Total Cost – PLAXIS – Soil Profile 2

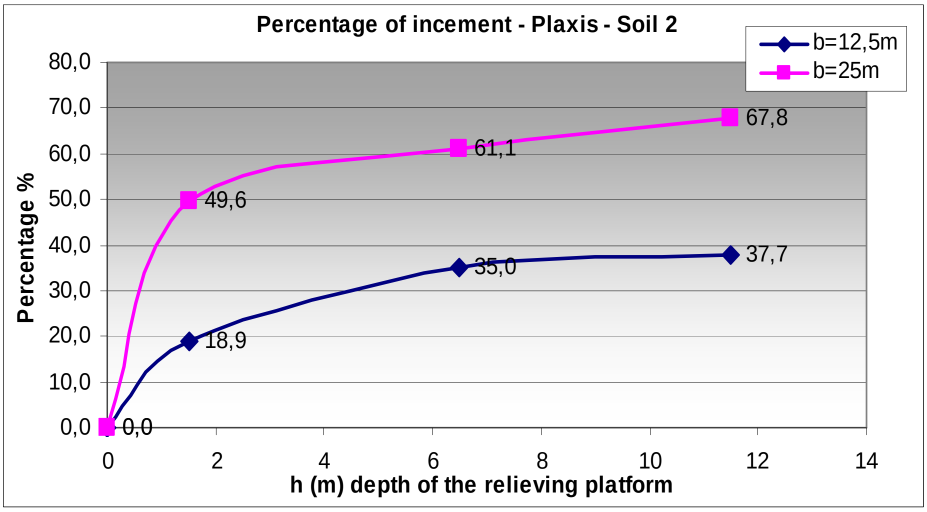


Figure G.8: Percentage of increment – PLAXIS – Soil Profile 2

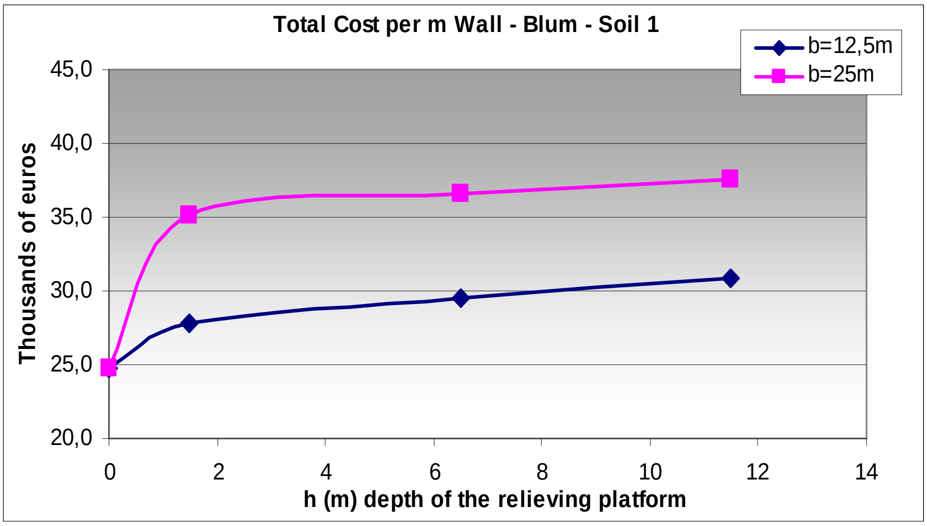
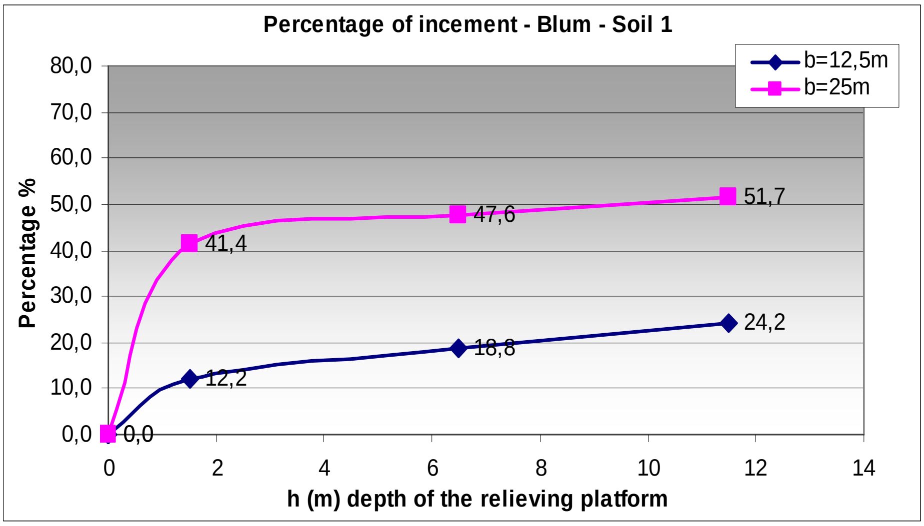


Figure G.9: Total Cost – Blum – Soil Profile 1



G.10: Percentage of increment – Blum– Soil Profile 1