# Stability analysis quay structure at the Amazonehaven port of Rotterdam



Master thesis

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"Our perspective is what holds the key whether the solution is ordinary or extraordinary"

-Dewitt Jones-

# Stability analysis quay structure at the Amazonehaven port of Rotterdam

A thesis submitted in fulfillment of the requirements for the degree of

### **Master of Science**

in

### **Civil Engineering**

by Nadevah Mourillon

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# PREFACE

This graduation thesis is conducted as a part of the master specialization track hydraulic structure at the faculty of Civil Engineering and Geoscience of the Delft University of Technology in the Netherlands. This research is carried out at the Public Works Rotterdam that provided the tools to conduct this research.

This master thesis report is about the stability for the quay structure with a relieving platform structure in the port of Rotterdam. The investigations performed poses the problems and provides an extensive literature research of the quay structure at the Amazonehaven, port of Rotterdam. The stability of the quay structure, specifically the effect of the combined wall, has been investigated by performing various calculations with different analytical methods.

Delft, October 14<sup>th</sup> 2015 Nadevah K.N. Mourillon

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## **EXECUTIVE SUMMARY**

The stability of any structure is an important aspect in civil engineering. The subject of this thesis is the stability of the quay structure at the Amazonehaven, port of Rotterdam. The reason for writing this report is to investigate the safety and stability of the quay structure, which was built in 1990, throughout its service lifetime while the toe of the combined wall was deformed. The focal point is the behaviour of the quay structure, specifically the combined wall.

The purpose of this research is to analyse and quantify the influence of the deformed combined wall on the stability of the quay structure. Apart from the deformation of the combined wall the designed penetrated depth was not reached. Measurements were performed on the deformed open tubular piles at the storage yard at Maasvlakte II, port of Rotterdam. It was concluded that the difference between the designed penetration depth and the actual penetrated depth is around 2 meters. Besides the length of the tubular pile also the wall thickness was measured. These findings have been used in the design model of the quay structure for the various design methods.

Literature research was conducted to determine the design aspects of the quay structure at the Amazonehaven. Furthermore the failure mechanisms for the specific quay structure recommended in CUR 211 *handbook Quay Walls* and other design guidelines were researched to obtain better insight into the concept of stability. The analytical method based on the Blum theory, beam on elastic foundation method (D-sheet Piling) and finite element method using Plaxis 3D were applied and compared.

The results of the analytical method, Blum theory, provided a rough estimation of the minimum penetration depth of the combined wall, the bending moments and anchor forces. The method makes use of a simplified model and ground data. It does not include the horizontal soil stress acting on the combined wall. The effects of the relieving platform structure on the bending moment is not considered in this method and had to be emulated.

For the beam on elastic foundation method, D-sheet Piling, two types of quay model are considered. One with the designed trapezium surcharge and a uniformed distributed surcharge model. The relieving platform structure with the M.V.-pile and concrete bearing piles were modelled in SCIA engineering with the loads acting on this superstructure. Four load combinations with different water level difference are applied for the calculation in SCIA engineering. The horizontal reaction force from the relieving platform structure and the vertical reaction force acting on the combined wall will be used in both the quay model in D-sheet. The results from this calculation had significant differences between the minimum penetration depth for the various load combination for both models. Furthermore, the favourable effect of the obliqueness of the combined wall on the bending moment is not taken into account. The second order effects and the plastic soil behaviour are not considered, either. Although these aspects can to be taken into account separately on top of the output of the D-Sheet-calculations, one must conclude that the method of "beam on elastic foundation" is not the ideal tool for the design of a quay structure.

The method based on finite element method, Plaxis 3D, which takes into account the 3-dimensional effects of the quay structure and considers the actual soil behaviour during calculation is a more sophisticated manner for modelling a quay structure.

For this thesis a calibration model (which is the actual designed of the quay structure) and a series of models with different penetration depths of the combined wall are modelled. A comparison is made between the calibration model and model 2B (which was the actual situation of the combined wall in reality). The calibration model which is modelled with or without surcharge turned out to be stable enough to withstand the design loads as did model 2B. After safety analysis (phi, c-reduction) of the soil parameters in both models the safety factor proved to be slightly smaller than 1.5 which indicates that the safety margin used in the previous calculations were correct.

It is advised to design quay structure with relieving platform structures with finite element method, Plaxis 3D. This provides a 3-dimensional view of how the foundation elements and relieving platform structure work and the stability of the quay structure can be modelled in detail. The finite element method, Plaxis 3D, is a better simulation program of the reality however this program is very time-consuming when accurately trying to model the quay structure in Plaxis 3D.

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# **ABBREVIATIONS**

ALS	Accidental Limit State			
СРТ	Cone Penetration Test			
DWT	Dead Weight Tonnage			
E.A.U. GL GWL LLWS LNG LWS MLLWS MSL M.Vpiles N.A.P. NGD	Empfehlungen des Arbeitsaussches Ufereinfassungen Ground level with respect to N.A.P. Ground Water Level Low Low Water Spring with respect to N.A.P. Liquefied Natural Gas Low Water Spring with respect to N.A.P. Mean Lowest Low Water Spring with respect to N.A.P. Mean Sea Level with respect to N.A.P. Müller Verfahren piles The Normal Amsterdam Level (in dutch: Normaal Amsterdams Peil) Nautical Guaranteed Depth			
OWL PoR	Outer Water Level Port of Rotterdam Authority			
SLS	Serviceability Limit State			
ULCV ULS VLOC WL	Ultra Large Container Vessel Ultimate Limit State Very Large Ore Carrier Water Level			

# NOMENCLATURE

φ	Angle of internal friction	[°]
τ	Shear stress of the soil	[kN/m <sup>2</sup> ]
С	Cohesion	[kN/m²]
C′	Effective cohesion	[kN/m <sup>2</sup> ]
$\sigma_n$	Effective soil pressure in normal direction	[kN/m²]
γ	Unsaturated volumetric weight of soil	[kN/m <sup>3</sup> ]
$\gamma_{sat}$	Saturated volumetric weight of soil	[kN/m <sup>3</sup> ]
E <sub>50</sub> <sup>ref</sup>	Secant stiffness in standard drained triaxial test	$[kN/m^2]$
E	Oedemeter stiffness for reference stress	$[kN/m^2]$
Eur	Unloading-reloading stiffness	$[kN/m^2]$
Cu	Undrained shear strength	$[kN/m^2]$
	Dilatation angle	[°]
Ψ θ_	Angle of active sliding plane	[0]
S S	Angle of wall friction	[ ] [9]
b k	Soil permeability in y v-direction	[ ] [m/dav]
R <sub>x,y</sub>	Interface strength ratio ( $R_{\rm c}$ =tan( $\delta$ )/sin( $d'$ )	[]
Vint	Shear strain (at strain shear modulus rodustion of $70\%$ )	[]
Yo,7	Shear modulus for yory small strain	[-]
G <sub>0</sub>	Shear moutius for very small strain	נגוא/ווו <u>ן</u> ר ז
λ *	Modified compression index	[-]
К *	Modified swelling index	[-]
μ	Modified creep index	[-]
α	Obliqueness front retaining wall	[°]
β	Obliqueness ground level	[°]
Ka	Active coefficient of horizontal earth pressure	[-]
K <sub>o</sub>	Neutral coefficient of horizontal earth pressure	[-]
K <sub>p</sub>	Passive coefficient of horizontal earth pressure	[-]
M	Bending moment	[kNm] [m <sup>3</sup> ]
W	Section modulus	[m]
		$\begin{bmatrix} 11 \end{bmatrix}$
El	Flexural stiffness of the beam	[kNm <sup>-</sup> /m]
W		[m]
х	Coordinate along the axis of the beam	[m]
Ν	Normal force in the beam	[kN]
В	Acting width	[m]
f	Total pressure on the beam, including the reaction	
	of the soil springs	[kN/m]
E	Modulus of elasticity	[MPa]
t	thickness of pile	[m]
t <sub>flange</sub>	Thickness of flange intermediate sheet pile	[mm]
t <sub>web</sub>	Thickness of web intermediate sheet pile	[mm]
H <sub>tot</sub>	Total neight intermediate sheet pile	[mm]
B <sub>tot</sub>	hoight	[[[]]]]
li h	width	[[]]]] [mm]
Δ	Area of concrete nile	$[m^2]$
FI	Bending stiffness	[kNm <sup>2</sup> ]
EA	Normal stiffness	[kN]
?	Poisson's ratio	[-]
1	Effective length	[m]
-e <sub>JJ</sub>	Derimeter of the contact surface of soil and grout	[m]
0	rennieter of the contact surface of soll allu grout	[III]

# 1

### INTRODUCTION

This report shows the process and results of the graduation project for the master program Hydraulic structures at Delft University of Technology. The project was executed for the Municipality of Rotterdam and the Port of Rotterdam, with the aim to analyse the stability of the deep-sea terminal of the Frans Swarttouw B.V. at the "Am-azonehaven" in the Port of Rotterdam, which was acquired by "Europees Massagoed Overslagbedrijf" (E.M.O) during construction.

This chapter introduces a general history of the port of Rotterdam, the research description and the project approach that was followed during this research. First, the history of Rotterdam is discussed with the focus on the Amazonehaven, then the research description which will consist of the problem description, objective and research question. And finally, the scope of the thesis is discussed.

### 1.1 History of Rotterdam

Rotterdam is a city in South Holland, the Netherlands, which is geographically located within the Rhine-Meuse-Scheldt river delta at the North Sea. The history of Rotterdam began in the thirteenth century when a dam was constructed along the Rotte river which is a river located in the Rhine-Maas-delta in the Netherlands. Around this location people settled for safety. The city of Rotterdam started out originally as a fish port which in the course of time grew into the largest cargo port in Europe, also known as the port of Rotterdam. [11]

From the mid-nineteenth century until 2002, the port of Rotterdam has been the world's busiest and largest port in Europe. The reason for Rotterdam's success is based on its strategic location on the North Sea, directly at the mouth of the New Meuse channel (Nieuwe Maas)leading into the Rhine-Meuse-Scheldt delta, hereby providing excellent access to the hinterland upstream reaching into Switzerland and France via the rivers Meuse and Rhine. The completion of the New Waterway (Nieuwe Waterweg) in 1872 played a big part in that as well. Thus leading to the nickname "Gateway to Europe" and conversely "Gateway to the World".

Today the port of Rotterdam continues to be the largest port in terms of annual cargo tonnage. This resulted into ever evolving and increased trans-shipment of goods, as well as a shortage of space in the port. Therefore, it was necessary to further develop the port seaward by stretching the port of Rotterdam over a distance of 40 kilometres. This distance consists of the (oldest) city port, the Botlek, the Europoort and the Maasvlakte I & II. In Figure 1.1 1 an overview of the entire port of Rotterdam is given.

The deep-sea terminal for the Frans Swarttouw B.V. at the Amazonehaven, which was built in the early 1990's, is the focus of this master thesis research specifically on the deep sea quay structure. During the construction of the deep-sea terminal, Frans Swarttouw B.V. was acquired by E.M.O., former E.K.O.M. The Amazonehaven is a seaport on the Beerkanaal at the Maasvlakte I. The main companies situated in this area are the container companies 'Europe Container Terminals' (also abbreviated as E.C.T.) on the north side and the 'European Massagoed Overslagbedrijf' (also abbreviated as E.M.O., former E.K.O.M.) on the south side.

In 2010, the Port of Rotterdam Authority facilitated the development of port-related activities in its management area. To make more room for the next generation containerships possible, the Amazonehaven needed to be widened. This widening of the port was mainly effectuated to the south side. The deep sea quay structure is located at the south side of the Amazonehaven, Maasvlakte I. (See Figure 1.1 2) [9]



Figure 1.1 1: Overview port of Rotterdam [28]



Figure 1.1 2: Overview of the 'Europees Massagoed Overslagbedrijf' (E.M.O.)

### 1.2 Research description

### **1.2.1** Problem description

In the port of Rotterdam, quay walls with relieving platforms are used in case of larger retaining height, in combination with large surface loads. It is known that the application of a relieving platform structure will decrease the dimensions of cross-sections that are usually applied. This make the design cost-efficient as it prevents the usage of relative heavy machinery for constructing this quay structure. In 1991 a deep-sea quay structure with a relieving platform structure was constructed at the "Amazonehaven south side" in the port of Rotterdam. This deep-sea quay has been constructed to facilitate the trans-shipment of iron ore and coal. These dry bulk materials are stored in high heaps on the quay thus creating large horizontal loads on the retaining wall. These heavy horizontal loads was reason for the designers to incorporate a relieving platform with the objective to reduce the loads on the retaining wall.

Throughout 2011-2013, the deep-sea quay structure was demolished in order to widen the "Amazonehaven" basin. During demolition works it was revealed that, at the time of construction, the combined wall had undergone a lot of deformation at the embedded end and had not reached its intended depth. This was an unexpected discovery, which led to various questions as to:

- Why had such extensive deformation to the embedded section of the combined wall occurred?
- Why did the tubular piles of the combined wall, over the entire 900m length quay, not reach its pre-calculated depth?
- Would the stability of this deep-sea quay structure be in jeopardy if it was used as originally designed?

The Municipality of Rotterdam and Port of Rotterdam would like to see the unexpected damages to the combined wall and consequences for the stability of this deep-sea quay structure explained. This entails an extensive research with the main focus on the deformed combined wall, throughout its functional service lifetime. Even though it was never fully loaded or used for which it was designed for.

### 1.2.2 Objective

The main focus of this master thesis research is to investigate the structural stability mechanisms of the deep-sea quay structure. These mechanisms are investigated for the deep-sea terminal E.M.O. (former Frans Swarttouw B.V. quay) at the "Amazonehaven" in the port of Rotterdam by means of an analytical and finite element method. The objective of this research is to determine when the deep-sea quay structure will become unstable.

This can be divided into two aspects:

- 1. The influence of the soil layers interaction on the combined wall, in particular the tubular piles.
- 2. The stability of the combined wall, even though the combined wall basically did not reach its designed depth.

These aspects require knowledge of soil behaviour and the possibilities and limitations of the material models and experience with model parameter selection. Furthermore, these aspects mentioned above will be investigated for both the design and the construction phases provided there is enough information available to draw a reasonable conclusion. Optimistically, a better understanding of the stability of the deep-sea quay structure, specifically the combined wall, during its service lifetime can be reached. With these methods of analysis, also the degree of safety or the reliability can be concluded.

### 1.2.3 Research question

From the above mentioned objective, the following research questions are deduced.

The main research questions are:

- What is the structural stability of the deep-sea quay structure throughout its functional service lifetime, specifically the combined wall?
- What is the preliminary evaluation of the deformed combined wall?

The following sub-questions are formulated:

- How (un)safe was the quay-structure if it had been exposed to the full surcharge (design load)?
- How much safety exists against the loss of stability?
- Does the quay structure meet the safety standards with the current calculation rules?
- How reliable are the past design standards in comparison to the current design standards?
- Which is the dominant failure mechanism?
- What has the damage to the combined wall provided to the reduction of the safety?
- Which aspects of the construction contributed to the deformation of the quay structure, specifically of the combined wall?

### **1.3** Outline master thesis

This master thesis research can be divided into four sections.

**Introduction:** The first chapter provides a general overview of the subject and outlines the problem description, objectives and the research questions.

**Literature & design methods**: Chapter two provides an overview of the literature research and field research. A literature research is done to obtain understanding into the different parts of the project and the field research provided a better visualization into the problem. In chapter three the boundary conditions and requirements of this project are described. Chapter four gives a general description on the stability of a quay structure and a brief description into the failure mechanisms. Chapter five describes the design calculation methods which can be used to design a quay structure. The expected results from theses design calculation methods are explained in more detail.

**Analysis:** In chapter six the results from the beam on elastic foundation, D-sheet Piling, are presented and discussed. Chapter seven entails the finite element method calculations with Plaxis 3D which provides a three-dimensional stability research of the quay structure.

**Evaluation & conclusion:** In chapter eight a comparison is made between the data from D-sheet and data from Plaxis 3D. Also the measurements of the deformed open tubular piles are compared to the calculation data to evaluate the length of the combined wall used in the quay models. The results of the varies quay structure models are discussed. Based on the findings of the research, conclusion and recommendations have been formulated in chapter nine.



Figure 1.3 1: Outline master thesis

# 2

## **PROJECT FRANS SWARTTOUW B.V. QUAY STRUCTURE**

This chapter presents the findings of a literature study and measurements of the deformed combined wall.

The literature research provided useful guidelines to solving the above mentioned research questions. The order in which the literature research is performed is by initially searching for all relevant information on the purpose and design of the quay structure. Subsequently, literature on construction methods was consulted.

The measurements in the field are carried out to get a practical insight into the deformations of the combined wall, specifically the tubular piles, and also the construction method which was actually applied on the quay structure can be identified.

### 2.1 Literature overview

In this section the findings of the literature research are discussed. The research is conducted at the Municipality of Rotterdam and consists of searching the (digital) archive that is still available during the relocation of the Municipality of Rotterdam. The rest of the information on this project was achieved by interviewing the personnel of the Municipality of Rotterdam.

In the literature research, background information on the Frans Swarttouw B.V. quay structure (nowadays E.M.O.) is collected. Next, the design methodology is explained starting from general design principles evolving into the more specific design method of the quay structure. Furhermore the construction practice of the quay structure is explained in the same order as the design methodology.

### 2.1.1 Background information

In the 1980's, it was of great importance to the port of Rotterdam, while competing with other major loading ports in countries such as Australia, the United States of America, Canada, South Africa and South America, to be able to facilitate vessels greater than 100 000 dwt, relating to the handling of solid fuel. With this in mind, the engineers designed the lay-out for the new terminal at the Maasvlakte I. The main principles considered when designing this terminal, were:

- 1. Throughput and storage of coal and iron ore
- 2. The depth of the deep-sea quay
- 3. The depth of the operational quay
- 4. Inland barges

With these four principles in mind, the planning for the construction of a deep-sea terminal for the throughput of iron ore and coal was finally realized in 1985. At first, the terminal was supposed to be situated at the 'Kop van de Beer'. The main reason for choosing this location in the beginning was the connection with the 'Dintelhaven' terminal, thus making it a very direct transportation route for barges.

Later on, several plans were developed with or without harbour basin at the Maasvlakte I.



Figure 2.1 1: Amazonehaven during construction



Figure 2.1 2: Overview of the surroundings at the Amazonehaven [7]

Ultimately, another location was chosen for this project, which was named the Amazonehaven. [7]



The project of the Amazonehaven was divided into the following sections:

- A harbour basin where dredging, soil and bank protection must be carried out;
- A quay, with an effective length of about 800 m'. At first the contract depth would be kept at NAP -21,65m with the possibility for future dredging up to NAP -24,00 m;
- An operational and a transitional quay with a total effective length of 650 m'. The contract depth for both quays was set at respectively NAP -12,00m and NAP -8,50 m.

In relation to the positioning of the harbour basin and terrain boundaries, compromises were made between different terrain tenants at the Maasvlakte I, such as E.M.O. (former E.K.O.M.) and E.C.T. For E.M.O., among others future expansion capabilities for dry bulk, north of the current location, needed to be possible and E.C.T. would be able to utilize quay walls on the north side of the harbour basin in the future, with the emphasis on reaching the maximum contract depth.

### 2.1.2 Current design process

When designing a new quay structure there are quite some possibilities which gives the engineers a certain freedom of design. However, due to the local conditions, any project faces limitations that must be taken into account. That is why, during the design process, there are various approaches to be considered, namely:

- The technical approach (the terms of reference are extended into a technical solution)
- Social approach (a problem or vision is extended into a working (technical) solution
- Lifecycle or sustainable approach (from cradle to grave)

These approaches can be elaborated into different process stages and steps which is schematized in figure 2.1 4.

In the technical design stage of the quay structure a more detailed development of the definitive design is given. This design stage consists of different levels which provide a more detailed description of the final design. These levels in the design stage are:

- 1. Preliminary design stage;
- 2. Final design stage;
- 3. Detailed design stage.



Figure 2.1 4: Overview of the design process approach [17]

In all these design stage levels there are choices to be made which are linked to each other. It starts with a rough sketch of the structure for the chosen location which leads to the boundary conditions and requirements. These conditions will lead to the final definitive design structure out of all the alternative design structures. For the design, several design guidelines were developed regarding retaining structures which are used in the Netherlands, in particular:

- CUR 211 Handbook Quay Walls;
- CUR 166 Handbook Sheet Pile Structures;
- EAU (Empfehlungen des Arbeitsausschusses Ufereinfassungen) Guidelines for the Committee for Waterfront Structures, Harbours and Waterways;
- Eurocode 7 (NEN-EN 9997) Geotechnical Design rules

These guidelines use a fundamental semi-probabilistic safety approach which translates into partial safety factors for the load case scenarios.

### 2.1.3 The E.M.O.- quay structure design

For projects like these it is relevant to perform an investigation beforehand to determine the consistency of the soil for the possible effects of drainage of the building pits to the surrounding area (in this case especially for the terrain of the Netherlands Gasunie N.V.). Furthermore, one is also interested in the structure of the different soil layers. This is to determine negative soil friction and soil settlement in these layers. With these data in hand a better image is obtained for the engineers to further realise the quay structure design in detail.

### Soil analysis

Soil investigations need to be performed at the chosen location by a geotechnical engineer who must have the basic and additional knowledge on the soil composition to give a relevant recommendation. This knowledge consists of three components (Figure 2.1 5):

- 1. Soil composition (geology);
- 2. Soil properties (soil behaviour);
- 3. The (calculation) models which are common in geotechnical engineering.

Apart from the basic and additional skills which are essential for a geotechnical engineer to have, they must be able to make schemes, modelling, able to perform hand calculations and analyse computer calculations. Also the geo-hydrological parameters of the location and surrounding area are obtained.

The soil investigation is performed in two parts:

- 1. In-situ research (CPT, sampling and drilling); (Appendix A.1)
- 2. Laboratory research; classification/strength and deformation testing



Figure 2.1 5: The geotechnical triangle [8]

This soil investigation will provide a better geotechnical overview for the design of the quay structure. For this project certain aspects are considered in this investigation, namely:

- The drainage of the building pit during construction of the quay wall and the influence of the drainage on the terrain at the Netherlands Gasunie N.V.;
- To anticipate the cone resistance of the load bearing layers after excavation of approximately 12.0 m of building pit and harbour;
- The bearing capacity of the foundation of the quay structure;
- The installation aspects;
- The dredging of the harbour along the quay structure and also the quality of the dredged sand;
- The calculation of the sheet pile as a part of the quay wall;
- The stability calculation of the entire quay wall structure including the surcharge.

Subsequently, the results from the soil investigations are analysed and evaluated by the geotechnical engineers who can give their recommendations on the soil properties including how the soil will interact with or on the quay structure. Based on these geotechnical recommendations and past experiences with other quay structures at the Maasvlakte, a new quay structure can be chosen out of all the possible design alternatives from the preliminary design phase. Furthermore, the functional and technical requirements can be defined in detail.

#### The design requirements & boundary conditions

Before starting with the calculation of the various elements of the quay structure, it is necessary to define the requirements, in terms of functional and technical, and boundary conditions of the definitive quay structure.

The boundary conditions are environmental factors that cannot be influenced but that are called into being by the presence of the project [1]. Often this involves a geotechnical, environmental and hydraulic description of the current situation.

The functional requirements of the quay structure must indicate what the bottlenecks are and to what extent these must be solved. At this stage, there are no concrete solutions considered, but an indication of the required

performance of the quay structure is given. The deep sea quay structure at the Amazonehaven, with a minimum length of 800 m', should be able to accommodate a bulk cargo carrier with load capacity up to 350 000 dwt. Besides the storage of dry bulk materials, the designer should also take into account the loads from the 3 types of cranes which will be situated on the structure

In the technical requirements document (Programma van eisen) the technical aspects of the quay structure will be defined in detail. With this, the project now becomes less abstract when compared to the functional requirements. The degree of detailing depends on the specific type of structure which is chosen in the final design phase. When working out the details of the aspects, these should always comply with the functional requirements. This is to ensure that the designers are fulfilling the clients' functional demands.

The above mentioned boundary conditions and requirements are explained further in chapter 3. From the previous mentioned requirements, a rough schematization of the quay structure can be derived. The retaining wall consists of two parts:

- 1. A superstructure, which serves as a relieving platform;
- 2. Substructure, which incorporates the foundation elements of the superstructure.

The substructure consists of the following elements:

- A sheet pile wall that has a supporting and retaining function;
- M.V.-piles that together with the sheet pile wall structure will support the resulting horizontal force from the anchor (Appendix A.4);
- Concrete bearing piles which support the relieving platform floor.

For this deep sea quay terminal the German guidelines E.A.U. 1985 were used during the design phase when defining the loading conditions on the structural elements of the quay structure. According to the E.A.U. 1985, a distinction is made between the load combinations of "Lastfall 1" and "Lastfall 3". For "Lastfall 1", the water level difference is about 1 m (common; high safetyfactors) and for "Lastfall 3" is the water level difference about 2 m (extreme; saftey factors about 30% lower).

For the calculation of the structural elements the subsoil is schematized into nine layers which will provide a geotechnical profile of the subsoil. For each layer the corresponding soil parameters and the development of the water pressures are determined. In case of water overpressures due to the loading of coal and iron ore on the superstructure, extra water pressures are modelled into the calculations. To limit the influence of water overpressure in the subsoil, drainage and sand piles are applied underneath and behind the quay structure. Nevertheless, in the clay layer a water pressure difference of about 25 to 40 kN/m<sup>2</sup> is considered.



Figure 2.1 6: Water pressures for 'geval A1'

According to the results of the measurements for the EKOM (now EMO)- quay structure and some additional calculations, it was also decided to apply a large reduction of the moment. The reason for this reduction is due to:

- The occurrence of pressure arching effects between sections of the sheet-piles, which
- will be displaced less ( at the location of the fixed hinge at the bottom and the anchoring at the top), according to the 'Umlagerung' E.A.U.;
- The supporting of the soil pressures by the M.V.- piles and concrete bearing piles.

On top of that, the negative skin friction has a reduction on the granular pressures behind the sheet piles. (Appendix A.2)

After elaborate research and practical experiences, it was concluded that:

- Field moment reduction = 0.75 \* Moment calculated (including water pressures)
- Fixed hinge moment reduction = 0.90 \* Moment calculated (including water pressures)

This moment reduction is allowed if other forces on other elements of the quay structure are adjusted accordingly in the calculations. First, the calculated anchor force should be increased with 15%. According to the E.A.U. when there is a 30% moment reduction on the granular pressure, the calculated anchor force should be increased with 15%. Second, the horizontal equilibrium of the subsoil beneath the concrete relieving floor is taken into consideration. On the left side of the subsoil, which is confined by the sheet pile, a force Ea is exerted which is equal to the active soil pressure and on the right side of the subsoil a force E1 is exerted. The difference between these forces should provide a shear force S in every horizontal cross-section of the subsoil. When determining the shear force S the deformation possibilities in relation to the required continuity of the sheet pile should be taken into account. (See Figure 2.1 7)



Figure 2.1 7: Subsoil equilibrium beneath the relieving platform

From the different recommendations and calculations performed, the dimensions of the elements for the quay structure of the Frans Swarttouw B.V. at the Amazonehaven, Maasvlakte I, is chosen.

### 2.1.4 General construction process

The construction of a quay structure is a very important and complex process. The design aspects play an important role, as design and construction are at least partly intertwined. This relation is relevant for defining the

planning and construction stages of a quay structure. The planning of the construction stages must be done in such a way that it finishes within the planned time period and the structure must meet all the stipulated requirements when inspected.

For this to be possible, the client and engineers must always be able to assess the feasibility of the design. Mostly, this involves an adequate coordination of the various disciplines involved with the construction which are imperative to the design process. There are two construction methods that must be considered during planning:

- 1. Method of construction from the water side;
- 2. Method of construction in a dry building pit.

The use of these methods is determined by the design, construction and the constructional aspects of the project. Besides these methods, there are also other aspects to be taken into account, for instance the various drainage systems, the layout of the construction site, the driveability of the retaining elements, the stability of the supporting soil as a result to the installation of the elements. In the following paragraph the chosen method of construction will be explained for this particular project.

### 2.1.5 The construction method applied for the E.M.O.-quay structure

The implementation aspects for the construction method of the deep sea quay terminal are relevant when deciding on how to proceed. The construction stages can be explained in seven steps to which the quay structure is built (Appendix A5). However, there are certain aspects of the execution of the construction which will be mentioned and explained in more detail [7], namely:

- 1. Excavation and water management;
- 2. Driveability;
- 3. Concrete work;
- 4. E.M.O. quay construction costs

These four aspects are essential to understanding the order in which the construction method is performed.

### 1. Excavation and water management

The construction method used for this quay structure is a dry building pit. The excavation is carried out by using hydraulic excavators and dump trucks. The excavation works includes about 600 000 m<sup>3</sup> of excavation, 300 000 m<sup>3</sup> of replenishment and 100 000 m<sup>3</sup> of soil improvements.

In the case of regulating the water level many aspects are considered before choosing the appropriate type of drainage. These aspects are:

- 1. The outer water level of the Beerkanaal;
- 2. The soil water in the sand above the clay layer at NAP -20.4m;
- 3. The water pressure beneath the clay layer.

The outer water level is turned around by means of auxiliary sheet piles driven into the clay layer and the groundwater level is lowered by means of drains and filters. During land reclamation, in some areas a silt layer is deposited. These layers must be drained because almost only horizontal water transport should occur. In addition, to prevent the forming of too many wells during pile driving through the clay layer, the water stress below the clay layer is lowered by means of deep-well drainage.

### 2. Driveability

For successful pile driving, it is essential that a proper and thorough investigation is acquired on the site conditions. Therefore an accurate assessment can be made of the topographical<sup>1</sup> and geological<sup>2</sup> conditions. [28]

In this project, the pile driving methods used for the various elements of this quay structure are based on the piling recommendations from the geotechnical reports. In these reports the results of the dimensions of the various foundation elements are given in table 2.1-1.

<sup>1</sup>Topography describes the particular environment of the site and details on working restrictions such as noise and vibrations. These restrictions varies according to the proximity and nature of neighbouring buildings, road category, underground services, power supplies, material storage areas etc.

<sup>2</sup>Geological conditions refer to the vertical characteristics of the soil strata.

Elements	Quality	Dimensions	Length [m]	Pieces
Open steel tubular	X-70	ø1420 mm	30 to 32	313
piles <sup>3</sup>	(483 N/mm <sup>2</sup> )	thickness 16-20mm		
Steel sheet piles	Larssen IIIS	t <sub>flange</sub> = 14.1 mm	20 to 24	312
	(triple intermediate	t <sub>web</sub> = 10.0 mm		
	sheet piles)	B <sub>tot</sub> = 1500 mm		
		H <sub>tot</sub> = 400 mm		
M.Vpiles	Peiner Pst 370/153	h = 374 mm	30 to 36	332
	[39]	b = 386 mm		
		t <sub>flange</sub> = 19.2 mm		
		t <sub>web</sub> = 16.0 mm		
Concrete bearing	B55	450 x450 mm	22 to 28	843
piles				

Table 2.1 1: Dimension prope	erties of the various foundation elements
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<sup>3</sup>The open steel tubular piles consist of three segments usually with thicknesses 17-19-17 mm (top-middle-bottom).

From the piling research the required pile block to drive the piles to their desired embedded depth are given. The methods are divided in different manners suited for the particular elements which are driven into the ground (Fig. 2.1-8 and Appendix A.6). [2]

- These piles are driven with a Menck 60 pilling rig on rails in a heavy ridge beam. This gives a very dimensionally stable result. The piles are applied by a 100 tonne auxiliary crane and placed with a vibratory hammer, RBH 160, with 160 strokes. The last ten meters is driven by a D10 diesel hammer.
- The sheet piles are driven into the ground the same way as the tubular piles, however if needed, the pile can be driven with a D46 diesel hammer.
- The M.V.-piles are driven with a Menck 40 provided with an IH S 70 hydraulic hammer with the implementation of grout.
- For the first 15m will these bearing piles be sprayed with the aid of a central spraying pipe. Afterwards, will the bearing piles be driven to their original depth by using a Hitachi KH300 with a D55 diesel hammer.

It can be concluded from the previously mentioned structural elements, with each their own driving method, the open steel tubular piles, steel sheet piles and concrete bearing piles are driven by vibrating and/or spraying till the top of the Pleistocene layer and in the Pleistocene layer the pile will be driven. The M.V.-piles will be fully driven as they derive bearing capacity from the Holocene layer as well to the Pleistocene layer.

### 3. Concrete work

The concrete work for the relieving platform consists of 20 sections of 45 meters in length which is a total of about 50.000 m<sup>3</sup>. Due to this large area that must be laid with concrete, this area will be separated into sections with just about the same workload and a time cycle of a week. The advantages for this way of working are:

- Not too many frame work;
- Large repetition;
- Process is in a factory;
- Concrete deposits are equally distributed in a week.

In Figure 2.1 9 the division of the 20 sections are shown, namely:

- 45 m of work floor of which contains 35 m<sup>3</sup> of reinforced concrete;
- 2 x 22,5m floor II and III of 2x 260 m<sup>3</sup>;
- 2 x 22,5m walls I and II of 2x 270 m<sup>3</sup>;
- 45 m deck of 200 m<sup>3</sup>.

All of this concrete work has to be finished within the planned time frame so the next construction stage of backfilling the soil behind the relieving platform can be done.



Figure 2.1 8: The applied pile driving methods on the foundation elements



Figure 2.1 9: The construction stages of the quay structure at the Amazonehaven, Maasvlakte I

#### 4. E.M.O. quay construction costs

The costs are largely related to the number of  $m^3$  of concrete per  $m^2$  of formwork, namely the contents of the structure. The cost of the concrete work is approximately 40% of the total construction cost. Thus, not only the amount of formwork and therefore the deployment of fewer personnel will also bring back the number of manhours per  $m^2$  formwork.

Simplification of the structure is therefore reflected double in personnel. Foregoing, is clearly to see a comparison of the quay structures that are recently built by the Port of Rotterdam Authority, namely the E.M.O. – and the E.C.T.-quay.

The efforts to simplify the construction can result in a decrease in the number of man-hours, equipment and materials per m3 which will obviously reduce costs. Therefore, changes in construction methods will influence the cost ratio of man-hours, equipment and materials (Figure 2.1 10).



Figure 2.1 10: Comparison of various deep-sea quay structures [7]

In general, a planning is made for the construction stage which is updated every time depending on the progress of the work performed in the field. This is to keep everybody informed on the progress of the quay structure and to see if the deadline can still be met.

### 2.2 Widening of the Amazonehaven, port of Rotterdam

The widening at the south side of the Amazonehaven harbour basin is realized due to the rapid growing dimensions and numbers of the vessels, what are called Ultra Large Container Ships (ULCS), entering the Amazonehaven. These vessels are estimated to be larger than 10 000 TEU, with a length larger than 350 m, width 49 m and a draught of 15.2m, with a capacity between 13 000 and 14 000 TEU. Besides the size of the vessel also the nautical restrictions imposed by the Rotterdam Harbour Master have limited the access to this harbour basin. The largest vessels vessels under the worst wind direction and current conditions are restricted starting at 3 Beaufort and above 5 Beaufort wind speed, the harbour basin is closed for large vessels [40]. This will significantly limit access to commercial operations at the terminal. Therefore, Port of Rotterdam Authority and ECT decided to widen the Amazonehaven harbour basin which the largest vessels will be able to sail and moor in the basin under all wind speed conditions up to 6 Beaufort.

The new harbour basin will have a width of 310 meters over the first kilometre and over the remaining length a width of 275 meters. This is an improvement to the old basin situation where the effective sailing path was about 165 meters.



Figure 2.2 1: Aerial view of the Amazonehaven, widening indicated in yellow [40]

The new harbour basin will have a width of 310 meters over the first kilometre and over the remaining length a width of 275 meters. This is an improvement to the old basin situation where the effective sailing path was about 165 meters.

The widening of the harbour basin is a difficult operation which entails various aspects to be considered beforehand. For ECT, during the widening of the harbour basin their quay should always continue to be operational. Besides, the main challenge is the removal of the old quay structure. This quay structure is a very heavy construction and was not designed for removal. The aspects that should be considered are [40]:

- The demolition of the commercial operations of EMO quay wall on the south side with a length of 950 m;
- The removal of the slopes further on in the harbour basin (1.5 km);
- The dredging of sand between the old and new waterfront;
- And the construction of a new retaining wall.

Basically, the demolition of a deep-sea quay structure is an unique project. The quay structure has a high concrete relieving structure of about 11 m, a massive 18 m wide concrete relieving floor with the thickest part of about 3 meters. Most of the concrete relieving structure is below the water surface with the foundation elements. The removal of the quay structure was provided by Besix Van Oord.



Figure 2.2 2: Drill and blast pontoon positioned (left) and the actual blast (right) [41]

The concrete relieving structure is removed by drilling and blasting of the relieving floor in wet conditions. Throughout the demolition of the concrete structure the impact on the environment should be kept very small, meaning no big fish or marine mammals should be injured during demolition. The M.V.-piles were cut in pieces

and after the removal of sand the sheet piles were removed. The concrete bearing piles are removed with a 300 tons crane with a special clamp and a 105M vibrator. By making some adjustments to the heavy crane by adding 4 clamps the steel tubular piles are removed.

Furthermore, between the old and new quay structure the top layers were excavated and dredged in various stages in cooperation with the demolition stages. The dredged sand from the harbour basin at the Amazonehaven is partly sold on the commercial market and used to reduce the harbour depth of the Amazonehaven basin [40].

### 2.3 Measurements deformed combined wall

In this section the measurements of the deformed steel tubular piles are discussed. The research is conducted with the help of the co-workers from the Municipality of Rotterdam. The open tubular and intermediate sheet piles were stored at the storage yard, at the Maasvlakte in Rotterdam, after demolishing the quay structure at the Amazonehaven in 2011- 2013.

As stated earlier, the deep-sea quay structure was demolished in 2011-2013 because the Amazonehaven harbour basin had to be widened. The reason for this was to keep up with the demands of facilitating vessels larger than 100.000 dwt at the Maasvlakte. During the demolishment of the deep-sea quay structure it was discovered that the combined wall had many deformations to its embedded sections. Also, these piles never reached their intended depth everywhere.

### 2.3.1 Observations during demolition

During the demolition of the old quay structure much damage are discovered at the tip of the open tubular and intermediate sheet piles. This was an unexpected discovery which according to the recorded events that took place during its service lifetime was not known. The events that took place during the lifetime of the quay structure are [40]:

- Overloading of a section of the quay structure which resulted into the vertical deformation of that section for approximately 50 mm. Further no severe damage was observed.
- Temporarily repaired interlock opening of the sheet piles during the night springtide with the aid of divers. Furthermore no damages occurred.

As mentioned before, the most damage occurred at pile tip. The damages to the steel open tubular piles consist of folding of the pile toe, completely closed pile toe, ovalisation at the pile toe or other damages. A rough estimation of the damages to the open tubular piles is to about 15% to 20% of the piles. In the case of the intermediate sheet piles roughly 50% seems to be damaged. All the damages are at the toe of the sheet pile because of the interlock openings that occurred during construction.

### 2.3.2 Measurements at the storage yard at Maasvlakte

To better understand the concept of the deformation of the elements of the combined wall it was imperative to visit the storage yard at the Maasvlakte. The elements of the combined wall are stored here for further investigations to be performed and to be re-used. As previously mentioned, the focus of this research is primarily on the open tubular piles which were measured during the field research.

It was possible to get a better practical insight into the length of the deformation on the open tubular piles. Here, with the help of the co-workers of the Municipality of Rotterdam, measurements were performed on the tubular piles, namely (Appendix A.7):

- The length of the deformation;
- The thickness of the tubes (this is measured per section of the tube);
- The total length of the tube is measured;
- Determine at which side the deformation is located (land or sea side);

The open tubular piles are numbered which makes it very easy to find in which of the 20 sections the pile was located. Over the entire 900 m' quay length, the open tubular piles were numbered from 1 till 313. After evaluating the measurements of the open tubular piles couple of assumptions can be made as to why the elements of the combined wall were deformed. Even though, in some sections there were no visible deformations to the open tubular piles.

First, the intermediate sheet piles are evaluated by looking at the connection between the interlock. It is inevitable to prevent small variations in the positions of the primary elements, which leads to friction between the interlocks resulting in the pile running out of the interlock. Therefore, interlock openings or damages can occur during installation of the open tubular piles[41]. These can rotate as a result of inaccurate positioning or too powerful pile driving. That is why, the intermediate sheet piles must have sufficient deformation capability to follow the deformation of the primary elements. But many times during maintenance of the combined wall, interlock openings are discovered. These openings are repaired by welder divers which can help restore the connection between the interlocks. This section which is maintained is above N.A.P. -25,50 m (construction depth), the rest of the intermediate sheet pile is embedded which is not visible. Also, it cannot be foreseen that the embedded part of the intermediate sheet pile is deformed and have not reached its designed depth. (See Figure 2.3 1)



Figure 2.3 1: Deformed intermediate sheet piles Larssen 3S

Second, the open tubular piles which are the primary elements of the combined wall, are supposed to be both technically and economically more attractive than its predecessors. For this project an open tubular pile is applied with an inclination of 1:5. These piles were driven through many soil layers with high penetration resistance which cannot be overcome very easily. As the pile driving depth increases, the problem of reaching the designed depth increases mostly in the consolidated sand layers. As a result the project reaches a state of stagnation when the pile cannot be driven deeper into the ground.

The reason for this can be that during pile driving, the sand around and inside the tube was too compact which lead to stagnation of the pile driving. Furthermore during driving of the tubes a rather heavy hammer was used. All of this may well have caused the tubular piles to deform at the embedded end (See Figure 2.3 2). The fact that the sand scorched to the inner and outer wall of the tubes (See Figure 2.3 3) confirms stagnation of the penetration of the piles under continued pile driving. The continued pile driving caused the pile end to heat up due to steel-soil friction which in turn caused the sand to scorch to the wall of the tubes.



Figure 2.3 2: Deformed open tubular pile toe in a banana shape (left) deformed open tubular pile toe folded closed (right)



Figure 2.3 3: Scorched sand on the open tubular piles and intermediate sheet piles

### 2.4 Conclusion

In this chapter the deep sea quay structure Frans Swarttouw B.V./ E.M.O. project in the Port of Rotterdam is researched by performing a literature and field research. After examining the available literature and field research a better insight into the whole structure as it was designed and constructed in the 1990's is acquired. Specifically, the concept of the combined walls in relation to the other sub- and superstructure elements. The interaction between these elements are crucial to the understanding of the stability of the quay structure.

Moreover, with this background information in mind the following motivation followed to further investigate the stability of the quay structure with the aid of D-sheet Piling and Plaxis 3D. These computer programs will take the interaction between the soil layers and the steel combined wall during construction into account, i.e. dredging and dewatering of the ground water level. It is possible to simulate the state the steel combined wall was in during its service lifetime. Afterwards, the designed surcharge can be applied to the quay structure model at which the behaviour of the steel combined walls can be further evaluated.
# 3

## **BOUNDARY CONDITIONS AND REQUIREMENTS**

Aforementioned in chapter 2, before being able to conduct any calculations, by hand, D-sheet and Plaxis, a list of the boundary conditions and requirements will be provided. This list is based on an overview of the most important aspects which have to be considered for the design of the quay structure at the Amazonehaven, port of Rotterdam. The geotechnical and hydraulic boundary conditions are described in detail which will show the current characteristic soil situations and the water depth in the harbour. Furthermore, the requirements of the quay structure will be described as well.

### 3.1 Boundary conditions

Boundary conditions are conditions that are imposed by the environment. These conditions are divided into geotechnical and hydraulic boundary conditions.

### 3.1.1 Geotechnical boundary conditions

To determine the geotechnical conditions of the project area a geotechnical investigation was conducted by the Municipality of Rotterdam. The geotechnical investigation was carried out in two stages and took place from May 1987 till July 1988. These investigations consisted of several cone penetration tests (CPT) and drilling. In general, 123 CPT's were carried out in that area, according to the NEN 3680 - "*Grondonderzoeken, Statische sondeermethoden (1982, eerste druk)*", in which the maximum explored depth is about NAP -45m. Also a total of 4 drillings were carried out where continuous undisturbed samples were taken. [2]

According to the cone penetration test investigations, the soil layer consists of mostly loose and stiff sand. Also clay and peat layers can be found in this area. In this thesis the results from all the CPT's which have been performed in this area only one representative soil profile will be used (Figure 3.1 1). In order to determine the representative values of the soil profile in table 16 of the report "*Amazonehaven zuidzijde 900m zeekade E.M.O. 1988 voor kolen en ertsen*" and table 3.1 in CUR 166 (Appendix A.1) are used. The soil profile and its characteristic values are given in Table 3.1 1 & Table 3.1 2.



Figure 3.1 1: Cone penetration test DN 87 port of Rotterdam.

From NAP	To NAP	Layer thickness	Туре
(m)	(m)	(m)	(-)
5.20	-6.00	11.20	Sand (loose)
-6.00	-9.00	3.00	Clay
-9.00	-15.50	6.50	"Leem" (Claylike sand)
-15.50	-19.50	4.00	Sand (Moderate)
-19.50	-22.50	3.00	Clay
-22.50	-25.50	3.00	Sand (Moderate)
-25.50	-29.50	4.00	Sand (Moderate)
-29.50	-36.50	7.00	Sand (Moderate)

Table 3.1 1: Soil profile from CPT DN 87

Layer	Thickness	Soil type	Ysat/ Yd	φ [ <b>o</b> ]	δ	с	Ka	Kp	E
nr.	layer		[kN/m³]		[rad]	[kPa]			[Mpa]
	[m +NAP]								
1	+5,2 till -1,50	Sand(loose)	19/17	30,0	20,0	0	0,28	5,74	15
2	-1,50 till -6,00	Sand	20/18	32,0	21,33	0	0,26	6,83	45
		(moderate)							
3	-6,00 till -10,2	Sand (clay)	21/19	27,0	18,0	0	0,32	4,52	35
4	-10,2 till -11,6	Sand	20/18	32,0	21,33	0	0,26	6,83	45
		(moderate)							
5	-11,6 till -20,4	Sand (clay)	21/19	25,0	16,67	0	0,35	3,91	35
6	-20,4 till -21,5	clay	20/19	20,0	13,33	14	0,43	2,81	4
7	-21,5 till -35,0	Sand	20/18	32,0	21,33	0	0,26	6,83	45
		(moderate)							

### Table 3.1 2 Characteristic values of the soil properties

Where:

δ	Angle of wall friction
φ	Angle of internal friction
С	Cohesion
γ	Unsaturated volumetric weight of soil
$\gamma_{sat}$	Saturated volumetric weight of soil
K <sub>a</sub>	Active coefficient of horizontal earth pressure
K <sub>p</sub>	Passive coefficient of horizontal earth pressure
E	Modulus of elasticity

In this case, Coulomb assumes a straight slip plane failure in the calculation of the earth pressure coefficient. The angle of the wall friction in Table 3.1 2 is assumed to be  $\delta < 2/3 \varphi'$  which corresponds to a straight slip plane as assumed by Coulomb.

### 3.1.2 Hydraulic boundary conditions

### Water levels

In this section, the hydraulic boundary conditions can provide information that can describe the site location of the quay structure. The site location which is being investigated is located near the Beerkanaal and Europahaven. The normative water levels are determined with the data that the Municipality of Rotterdam receives from the operational monitoring system of the Port of Rotterdam . [18] The measured water levels and the exceedance of frequencies for the High and Low Water Levels are given in Table 3.1 3 & Table 3.1 4.



Figure 3.1 2: Location of the area considered in the investigation ("Europees Massagoed Overslagbedrijf" (EMO)

Table 3.1 3: Measured	normative	water	levels
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Туре	
Mean Sea Level [m +NAP]	+0.00
Mean High Water Level [m +NAP]	+1.27
Mean Low Water Level [m +NAP]	-0.66
Tidal difference ( h <sub>tide;mean</sub> ) [m]	1.93
Spring High Water Level [m +NAP]	+1.30
Spring Low Water Level [m +NAP]	-0.60
Tidal difference ( h <sub>tide;spring</sub> ) [m]	1.90
Low Low Water Spring [m +NAP]	-0.90

Table 3.1 4: The exceedance of frequencies for the High and Low Water Levels

Frequencies	1x/year [m +NAP]	1x/ 10 year [m +NAP]	1x/ 50 year [m +NAP]	1x/250 year [m +NAP]
High Water Levels	+2.38	+2.78	+3.23	+3.52
Low Water Levels	-1.10	-1.30	-1.66	-2.00

### **Groundwater levels**

The groundwater level over the length of the harbour area varies between NAP +2.50m and NAP 0.00m The water level head in the Pleistocene layer beneath the clay layer that is approximately between NAP -19.0m and NAP -22.0 m., is between NAP 0.00m and NAP +1.00 m.

### Waves

Wave conditions are divided into wind waves, seiches, swell waves and ship generated waves. The area of the Amazonehaven is dominated by wind and ship-generated waves as the quay is protected from large incoming sea waves. The waves that reach the quay structure are wind generated waves in the harbour basin and ship generated waves. However, it can be assumed that the waves generated by wind are small in a protected area. The reason for this is the small fetch. Since the speed of the passing ships is limited and the fetch is small in the harbour, both ship-generated waves and wind waves will not be taken into account in further calculations.

### **3.2** Functional requirements

The functional requirements contains relevant requirements for the design of the quay structure at the Amazonehaven. The requirements are divided in different categories:

- Retaining;
- Bearing;
- Navigational;
- Protective requirements.

### **Retaining requirements**

As previously mentioned, a quay structure should be able to retain water from the shore safely. Therefore, a certain retaining height for the quay structure is required. The required retaining height is related to the top level of the quay structure which is determined by the expected water levels as shown in Table 3.2 1and the bed level of the harbour basin. The required bed level of the harbour basin follows from the draught of the design vessel (Appendix B.1).

### Table 3.2 1: Retaining levels

Level	Height [+m NAP]
Top of quay structure	+5.00
Contract depth	-24.00
Construction depth	-25.50

### **Bearing requirements**

The bearing requirements are related to the loads imposed by cranes, vehicles and storage of goods and containers. The number, types and loads of cranes placed on the quay structure to transfer cargo between vessels and terminal are relevant due to the loads of the cranes need to be transferred safely to the foundation. Therefore, the exact loads depend on the number and type of cranes used on the deck of the quay structure. The type of crane is also dependent on the type of cargo that has to be transferred, in this case coal & iron ore.

### **Navigational requirements**

The navigational requirements are split up in the identification of a design vessel and berthing requirements. Also as a requirement a quay structure must enable the vessels to berth quickly and safely. The design vessel used in the early designs in the "Amazonehaven" corresponds to a bulk carrier with a dead weight tonnage (DWT) of 250 000. This port is used mostly for bulk material, the largest bulk carrier is chosen as the representative design vessel to take future increase in vessel size. The largest vessel of the VLOC class (Very Large Ore Carrier) at the time was the MS Vale Brasil, this bulk carrier has a DWT 402 347, overall length of 362m and draught of 23 meters (Appendix B.2). This vessel is able to carry iron ore and coal between Brazil and global markets, mostly Europe and Asia (See Figure 3.2 1). Besides the overall length of the design vessel the required length for each berth needs to be determined as well. For a number of vessels that will berth at the quay structure the berth length can be calculated (Appendix B.3). This equation allows for a gap of 15 meters between vessels and an additional 15 meter for the outer berths. An extra margin of 10 % of the vessel length is introduced to avoid additional waiting times due to repositioning of vessels at berth during unloading.[19]



Figure 3.2 1: Classifications of the varies vessels [30]

### **Protective requirements**

Protective requirements are related to berthing / mooring facilities and bottom protections.

### Berthing and mooring facilities

Along a quay structure berthing and mooring facilities are required for vessels (Appendix B.4). These facilities mainly consist of bollards and fenders. Bollards and fenders are placed along the quay wall with a separation distance of 22.5 meters, the resulting bollard force is 1.750 kN (77.78kN/m') and fender force is 2.235 kN (50kN/m') [2]

### **Bottom protection**

The construction depth level of NAP -25.50 m is required to facilitate the dimensions of the vessels propeller wash in combination with the required keel clearance such to determine the need for scour protection. The bottom protection will not be investigated in this thesis research.

### 3.3 Technical requirements

### 3.3.1 Technical lifetime

The technical lifetime of the quay structure is 50 years.

### **3.3.2** External loads on the structure

The loads on the quay wall are (Appendix B.6):

- Terrain load
  - Distance between front of quay and up to 38 m landside : 20kN/m<sup>2</sup>
  - From 38 m behind the front quay wall, triangular load : 90 till max. 450kN/m<sup>2</sup>
  - Heavy loaded part on deck (due to mobile crane) : 500kN/m<sup>2</sup>
  - Heavy loaded part on deck (due to unloading cranes/bridges) : 2 x 2500 kN
- Traffic loads
  - Traffic class 60
  - Mobile crane : 390kN/m<sup>2</sup>
- Bollard load (where c.t.c. is centre to centre) (Appendix B.7)
  - Force : 1750 kN per bollard and normal to the quay wall
  - Max. force : 2500 kN for bollard couple and normal to the quay wall
  - C.t.c. distance between bollards is 4.0 m
  - Bollard couples with c.t.c. distance 22.5 m
- Berthing load at NAP + 2.00 m for small vessels (Appendix B.7)
  - Force : 300 kN
    - C.t.c. distance : 14.9 m
- Fender load at NAP + 4.86 m (Appendix B.7)
  - Force : 50 kN/m
  - Impact force : 1000 kN/m<sup>2</sup>
  - C.t.c. distance between fenders is 22.5 m

### 3.3.3 Crane loads

### Crane details

Several crane loads were considered during the design of the quay structure. A distinction is made between fixed rail cranes and mobile cranes. This information is linked with the number and types of cranes used on the quay structure to transport the goods, specifically dry bulk, to and from the storage zone.

Cranes for seagoing vessels:

The crane that was used in the design of the quay structure, is similar to a Gantry grab crane [33]. The characteristics are given below:

- Distance between front of quay and waterside crane rail :3.25m
  - Distance between front of quay and landside crane rail : 27.25m
    - C.t.c. crane rails : 24.0m
- Average operational wind speed: 28.5 m/s

 Table 3.3 1: Characteristics Grab Crane at the Amazonehaven

Type of crane	Outreach waterside	Rail gauge	Max. ver- tical load	Max.wheel load	Number of wheels	Wheel distance	
	[m]	[m]	[kN]	[kN]	[-]	[m]	
Grab crane	45	24	14356	1795	8	1.38	



Figure 3.3 1: Gantry grab crane

Mobile harbour cranes:

The mobile cranes are used for loading and unloading of inland barges. These cranes are able to move freely along the quay. Two distinction are made for the mobile crane:

- 1. Crane in operation
- 2. Crane in travelling mode.



Figure 3.3 2: Gottwald mobile harbour crane [34]

Mobile harbour crane	
Caterpillar tracks [-]	4
Base [m <sup>2</sup> ]	5 x 1.4
Maximum pressure per track [kN/m <sup>2</sup> ]	390
Maximum load per track [kN]	2713.2
Total crane weight [t]	570
Maximum load [t]	32

Table 3.3 2:	Characteristics	Mobile Harbour	Crane at the	Amazonehaven

### 3.3.4 Retaining requirements

- The retaining height requirements for this quay wall structure are (Appendix B.5):
- Total quay wall length is about : 900 m
- Topside quay wall is about : NAP + 5.00 m
- Temporary contract depth is at : NAP -21.65 m
- Future contract depth is at : NAP -24.00 m
- Construction depth is at : NAP -25.50 m

### 3.3.5 Nautical requirements

The nautical requirements are:

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- Sea vessels (bulk carrier with 350 000 DWT)
  - Overall length : 300.0 m
  - Width : 60.0 m
  - Draught : 23.0 m
    - Berthing angle : 2° (at NAP -23.00m)
    - Berthing velocity : 0.15 m/s
- Inland vessels

•

- Maximum length : 220 m
  - Berthing angle : 15°
- Berthing velocity : 0.25 m/s

## 4

### **STABILITY ANALYSIS**

This chapter presents the limit states which have to be considered during the design process. By analysing the limit states a general description of the failure mechanisms can be deduced. A better understanding into the effects of the failure mechanisms on the researched quay structure can be obtained.

In this chapter the following research question will be answered:

- Which is the dominant failure mechanism?

By researching the failure mechanism for the quay structure the stability of its structure can be analysed.

### 4.1 General stability requirements

In coastal engineering, quay structures with steel combined walls and concrete relieving platforms is a common design. The concrete relieving platform enables the wall to have a larger retaining height resulting in larger depths and thus the berthing of ships of higher tonnage. [13] Besides berthing of ships, a quay structure separates water from shore. Furthermore, the design should include that collapse of the quay structure may not occur during its service lifetime. Therefore three requirements of a structure should be satisfied during the design process, namely:

1. The structure should be sufficiently **strong**. This means that it should not break and/or lose its connections between elements.

2. The structure should be sufficiently **rigid**. This means that it should not deform too much and/or lose its shape.

3. The structure should be **stable**. This means that the structure should not capsize and/ or buckle.

From these requirements can be concluded that the stability and safety of the structure are the most important aspects. The stability of a quay structure is normally checked for horizontal and vertical stability of the foundation elements (Appendix C.3). The safety aspect is usually included in the design rules and guidelines which are drafted from failure mechanisms.

### 4.2 Limit states

Failure mechanisms describe the way a structure fails to temporarily or permanently fulfil its function. A permanent failure of the structure is when a structure collapses. Temporary failure of a structure is when a structure does not yet collapse but does fail to function. That is what is called a limit state. According to the standards a distinction can be made between two limit states, namely:

• Ultimate Limit State (ULS) (during extreme situation)

• Serviceability Limit State (SLS) (during its service time)

By analysing these two limit states for failure of the quay structure a better insight into the failure mechanisms can be reached. This is done by exerting various load combinations on the quay structure. From this a variety of failure mechanisms can be identified for each limit state.

In the Ultimate Limit State (ULS) the following failure mechanisms have to be verified, if relevant to the designed quay wall structure (See Table 4.2 1).

Failure	EQU	STR	GEO	FAT	UPL	HYD	INS
mechanisms	(Equilibrium)	(Structural)	(Geotechnical)	(Fatigue)	(Uplift)	(Hydraulic)	(Installation)
Description	Failure of the	Internal	Failure or	Failure of	Failure of the	Hydraulic	Failure of the
	construction	failure of the	exceptional	the	construction	soil failure	construction or
	itself or part of	construction	deformations	constructi	or subsoil	because of	construction
	it, considered	or	of the subsoil	on or	due to	internal	elements due
	as a rigid body,	exceptional	at which the	constructi	upward	erosion by	to the manner
	where the soil	deformation	strength of the	on	forces by	concentrate	of driving
	strength is	s of the	soil is leading	elements	water	d ground	and/or
	irrelevant	construction	for the	due to	pressure or	water flow	vibration of the
		or structural	resistance to	fatigue	other vertical	(piping) in	elements
		elements	be provided		loads	the subsoil	
						because of	
						hydraulic	
						gradients	

Table 4.2 1 Overview failure mechanisms for the Ultimate Limit State (ULS) [1]

Apart from the mentioned failure mechanisms above also deformations can introduce collapse as well. These mechanisms depend on the type of structure and what is known about the safety of the structure. Regarding the quay structure not every failure mechanism is relevant. The failure mechanisms which are relevant for the quay structure are **structural**, **geotechnical**, **hydraulic** and **installation**.

In the Serviceability Limit State (SLS) deformations of the structure have to be verified. This limit state 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.

If the SLS is exceeded due to too large deformations, the quay wall structure is not able to fulfil its functions. Although, the quay structure will not collapse.

### 4.3 Safety quay structure

Safety of a structure is one of the most important aspect in the design process. Throughout the years, various design approaches were developed which can determine the safety of a quay structure. Originally, the design approach was based on deterministic methods. The deterministic values of the parameters are mostly based on experience, which is a conservative approach. This method of approach makes it impossible to quantify the reliability of the structure.

To make a transition from a design based on experience to a more scientifically based design, a probabilistic approach was developed. The probabilistic approach is based on the principle that the structure should satisfy a specific (im)probability of failure. For this approach all the parameters of the structure are considered to be stochastic. The larger the consequence of failure, the smaller the acceptable probability of failure.

Nowadays, in order to maintain the simplicity of the design method and at the same time avoid complicated specialized probabilistic calculations, the NEN-standards were developed based on a semi-probabilistic approach to safety. In this approach, an element is considered to be sufficiently reliable, if a certain margin is present between the characteristic values of the strength and loads. This margin is created by taking partial safety factors into account.

As mentioned in the previous chapter, during the design of this quay structure at the Amazonehaven in 1985, the German guideline E.A.U. 1985 was used as a basis for design. The German guideline divides the Ultimate Limit State into three sub-limit states and the Serviceability Limit State. These limit states are:

- LS 1A : limit state of loss of support safety
- LS 1B : limit state of failure of structures and components
- LS 1C : limit state of loss of overall stability
- LS 2 : Limit state of serviceability

The German guidelines can be used for several types of structures. Therefore some reductions can be applied to fit the safety factors for the specific type of structure which is being designed. No descriptions are given as to which failure mechanism the safety factors and safety levels are based on.

In the current guidelines, CUR 211 "Handbook Quay Walls" and CUR 166 "Damwandconstructies", the failure mechanisms for the limit states which can occur in a specific structure are covered. The CUR 166 was originally drawn up for the design of a simply supported sheet pile structure. The safety factors for the sheet pile calculations in this guideline are based on the failure mechanism of "Passive earth pressure being insuffient". The safety level for a quay structure in CUR 166 is set at a level III, which is the highest level with a reliability index-value ( $\beta$ -value) of 4,2 (Appendix C.1).

The CUR 211 is based on a quay wall structure with combined wall, superstructure and tension piles, which is much more specific than the structures in the CUR 166. Nevertheless, the CUR 211 uses an almost similar approach as the CUR 166. In this guideline the safety factors are based on the failure mechanism of "*Failure of sheet pile profile*" and not on the "*Passive earth pressure being insuffient*". The reason for this is that in most cases the passive earth pressure will be satisfied due to the deep toe level for the vertical bearing capacity. Therefore, the safety level is set at 2 with a reliability index-value ( $\beta$ -value) of 3,4 (Appendix C.2).

The safety factors are also based on a certain lifetime of the structure (in most cases 50 years). The safety level must be maintained over the lifetime and depends on the limit state of the structure.

In general, for quay structures the 4 main failure mechanisms are:

- 1. Failure of sheet pile wall due to a yielding element or insufficient passive earth pressure
- 2. Failure of the soil due to groundwater flow
- 3. Failure of the soil due to insufficient total stability of the structure in the soil
- 4. Failure of the tensile/anchor element

In the case of a quay structure with a superstructure, additional elements must be verified. For instance:

- Calculation of the foundation elements
- Calculation of the superstructure

### 4.4 Stability research objective

In this research, the fundamental concept of structural stability is the ability to carry its self-weight and the applied load for its service lifetime without collapsing. In the design process limited deflection or rate of deflection is allowed to prevent actual failure that would damage the elements of the structure.

Therefore, as previously mentioned, for the quay structure not every failure mechanism in the ULS are relevant. The relevant failure mechanisms are **structural**, **geotechnical**, **hydraulic** and **installation**. For this thesis research all of the above mentioned failure mechanisms that corresponds with the specific quay structure applied at the Amazonehaven, port of Rotterdam, will be investigated. According to the "*CUR 211 Handbook Quay walls*" the researched quay structure is set at a type 2 structure with the following failure mechanisms. (Figure 4.4 1)



These failure mechanisms in the figure above are briefly discussed [4]:

- 1. Exceedance of the vertical bearing capacity;
- 2. Exceedance of the tensile load capacity;
- 3. Failure of the soil due to horizontal load on the foundation;
- 4. Exceedance of the overall stability;
- 5. Structural failure of the retaining wall;
- 6. Structural failure of the piles, due to pressure, tension, sliding or buckling;
- 7. Failure of the construction due to large displacement of the foundation;
- 8. Failure due to internal erosion, underflow, lateral infiltration or piping.

Besides the above mentioned failures, it is also possible that some other failure mechanisms can occur. These are:

- Failure of the anchor (Appendix C.3) When the force on the anchor is higher than its strength, the Kranz stability will be exceeded. The reason for this failure may be due to the underestimation of the applied load to the anchor, the poor quality or length of the anchor.
- Failure of the relieving floor

Besides the above mentioned failure mechanisms which are the technical requirements of the structure when designing, there are also the functional failure mechanisms. These mechanisms are also crucial in the serviceability state. If the functional failure mechanisms, as indicated in [1], are exceeded the quay structure can no longer as efficiently as possible provide the transferring of goods from ship to land and land to ship. The four functions of a quay structure are:

- Retaining function (the quay structure must be able to safely retain both soil and water)
- Bearing function (the quay structure must be able to safely bear the loads of cranes, vehicles and stored goods)
- Navigation function (ships must be able to berth quickly and safely, and be loaded and unloaded and leave the berth without damaging either the quay structure or themselves)
- Safety function (the ships must be able to berth and leave the quay structure safely)

For the functional failure mechanisms to be satisfied during its service lifetime the deformation behaviour of the quay structure during its use should also be taken into account.

To be able to perform all of these failure checks for a quay structure a computer software is needed. This computer software should be able to numerically solve the problem based on a model in which the soil behaviour and the contained construction elements are integrated.

### 4.5 Conclusion

The focus of this thesis research will be on the failure mechanisms which were previously specified. The various failure mechanisms are linked with the stability of the structure as the most important aspect. Regarding the researched quay structure the stipulated failure mechanisms are researched with the help of the computer software PLAXIS 3D.

The dominant failure mechanism depends on the condition of the combined wall. When performing a numerical calculation with the computer software, PLAXIS 3D, the failure points of the quay structure can be identified. By performing a safety analyses on the model of the quay structure the failure areas will be identified. Not only the failure of the soil layers will be visible, but also the yielding failure of the structural elements.

# 5

## **DESIGN CALCULATION METHODS**

This chapter presents the various design calculation methods that are available when calculating a quay structure. Before numerical integration was possible by computers, sheet piles were calculated by hand calculation according to the classic (analytical) method, Blum. Nowadays, computer program is used for more complex calculation problems than those considered with the classic (analytical) method with Blum, among others beam on elastic foundation method and finite element method are considered. In this chapter only three methods are considered which will be used during this thesis research. These methods are:

- Classic (analytical) method
- Beam on elastic foundation method
- Finite Element Method

In the following paragraphs of this chapter will these methods explained in more detail.

### 5.1 Classic (analytical) calculation method

### 5.1.1 General design approach

According to the theory of Blum, sheet pile is considered as a beam that is loaded by soil and water pressures. The top of the sheet pile structure is connected to the relieving platform by means of an anchorage system, in this case M.V.-piles are considered. The sheet pile structure is supported by the passive soil resistance at the lower part. With a minimum penetration depth, the soil layer providing resistance is just able to ensure the stability of the sheet pile structure. The degree of fixity depends on a number of factors, such as extra pile length to a minimal length, stiffness of the soil resistance and the bending of the sheet pile structures. From the structural analyses point of view two basic concepts are recognised:

- 1. Free earth support sheet pile structures; (See Figure 5.1 1)
- 2. Fixed earth support sheet pile structures. (See Figure 5.1 2)

The deformation behaviour of both concepts is fundamentally different [21]. These methods are based on the failure condition on the passive wedge. With adequate increase of the pile penetration depth (approximately 20 -40%) the sheet pile structure is fully fixed in the soil. Here is when minimum active and maximum passive earth pressure occur.

### 5.1.2 BLUM design approach

For this thesis research an analytical calculation is performed. Blum's method is often used to get a better idea about the minimum required length of the sheet pile structure. This a very simplified method to make a draft design of the sheet pile structure. This analytical calculation is performed with the help of a computer program.



Figure 5.1 1 Free earth support method



Figure 5.1 3 Lateral earth pressure coefficients

K

For the analytical calculation 4 load cases were used with different water levels. This is how the original design calculations were performed. The main reason is to get a general idea on what the minimum required penetration depth will be before failure of the passive earth pressure. From the cone penetration tests (CPT's) results, which are included in Appendix A.1, can the soil profile be determined. As a result the soil parameters are determined by using the Eurocode 7 (NEN-9997-1) and CUR 166. In general, the soil profile consists of 8 layers but to simplify the problem only 3 layers (Table 5.1 1) were used as input into the computer program.

Layer nr.	Thickness layer	Soil type	Y <sub>sat</sub> / Y <sub>dry</sub> [kN/m³]	Ø' [ <sup>0</sup> ]	c' <i>[kPa]</i>	delta [°]	k <sub>a</sub>	К <sub>р</sub>
	[m +NAP]							
1	+5.2020.6	Sand loose	20.5/18.5	30	0	20	0.279	5.737
2	-20.621.7	Clay	17/17	19	5	20	0.415	3.146
3	-21.736.5	Sand Dense	18/20	35	0	20	0.230	7.822

Table 5.1 1: Characteristic values of the soil profile

Table 5.1 2: The 4 Load cases used for the analytical calculations

Case	Lastfall	OWL	GWL
		[m +NAP]	[m +NAP]
A1	1	-1.00	0.00
A2	3	-2.00	0.00
B1	1	0.00	+1.00
B2	3	+0.50	+2.50

The 4 load cases that were used to perform the analytical calculation are given in the table below (See Table 5.1 2) with the different Outer water levels (OWL) and Ground water levels (GWL).

For each load case different surcharges were applied. First, a surcharge of 20 kPa (equal to 20 kN/m2) is applied as originally designed on the relieving platform. Afterwards, a surcharge of 0 till 450 kPa is applied with increments of 50 kPa. As a result an estimation of the minimum penetration depth can be calculated. In the following paragraph the results of the Blum calculations are shown.

### 5.1.3 Results to the analytical calculation

From the results of the analytical calculations no specific answers were found on the stability of the quay structure. The reason for this is that with this method based on the load distribution the penetration depth can be determined. The corresponding moment and shear force can also be derived. However, the actual earth pressure redistribution caused by arching of the soil behind the combined wall is not taken into account in this method. Furthermore the various construction stages are not taken into account in this analytical method.

It can be concluded that with this method a rough sketch of the penetration depth of a combined wall can be found. This method omits a lot of other relevant effects of the soil load distribution on the combined wall and the rest of the construction elements of the quay structure. Therefore, other calculation methods are considered to help in determining the stability of the quay structure.

### 5.2 SCIA engineering

This calculation program performs static, dynamic, stability, nonlinear and other special types of analysis. Scia engineering does not work with finite elements directly. Instead, it employs the displacement- based finite element method on the structural elements for which the finite element mesh is generated "behind the scenes". The results are used for design and checks according to the appropriate technical standards.

Engineers can perform basic checks as well as advanced check for specific situations (e.g. resistance) and structures (e.g. bridges). This can be highly efficient and flexible model for a simple 2D and complex 3D structures.

A analytical calculation with the computer software SCIA engineering is found in Chapter 6.

### 5.3 Beam on elastic foundation method

The calculation method for beam on elastic foundation is more complex than the previous method. With this method it is possible to integrate complex boundary conditions such as multiple anchoring and construction stages. This allows the stress from different construction stages to be transferred to the following stage. Also, with this calculation method a modified modelling of the soil behaviour is necessary. The differential equation for a beam on elastic foundation can be solved by computer software that uses a one-dimensional finite element.



Figure 5.3 1: Spring model schematisation and soil coefficients



Figure 5.3 2: Lateral earth pressure coefficient as a function of the displacements [21]

The computer software D-Sheet Piling is commonly used in the Netherlands when calculating a quay structure and it is based on the theory of elastically supported beam. There are some limitations to this calculation method that are worth mentioning. The soil is schematized as a set of uncoupled elasto-plastic springs (see Figure 5.2 1) without the creep effect. If there is sufficient deformation of the sheet pile wall can the plastic behaviour of the surrounding soil be achieved. This will result into the development of minimum active soil pressure and maximum passive soil pressure. However, this is not the case in reality. Thus, for a more accurate solution, the stress displacement graph could be represented by a multi-linear spring characteristic instead of the used bi-linear spring characteristic. [3]

Analytical description of the beam on elastic foundation method which was calculated with the computer software D-Sheet Piling is found in Chapter 6.

### 5.4 Finite element method

Contrary to the previous methods, the finite element method is based on a model in which the soil behaviour and the contained construction elements are integrated. This method is done according to the stress equilibrium and deformation of soil and bending behaviour of the construction elements are described by a linked system consisting of partial and ordinary differential equations. This method can be numerically solved with the help of computer software which is based on the theory of the finite element system, e.g. PLAXIS. This finite element method

can be used for the solution of two-dimensional and three-dimensional problems. In this software the stresses and deformation in both the soil and the structural elements are divided into small elements. Each node of an element is connected to the node of the adjacent element. All these elements together are the so-called mesh.

The advantage of this method is that it can calculate the displacement of the soil directly around objects, in this case the soil against the combined wall. For that reason, this method should be applied in case that objects nearby are sensitive to the deformations of the soil during the construction of the wall. The effect of the construction on the object can then be calculated. This method is also applied for three-dimensional problems when the distribution of the earth pressures over the primary members and intermediate piles in a combined wall have to be taken into account.

Analytical description of the simulation method and the assumptions and results is found in Chapter 7.

### 5.5 Conclusion and discussion

For the design of a quay structure, specifically the retaining height of a wall, three methods can be used. The method of Blum is the least reliable and is used to give an estimation of the penetration depth. With the beam on elastic foundation method more reliable results for the penetration depth and field moments can be determined. An indication of the penetration depth should be given with the Blum method to be used for the beam on elastic foundation method. The most reliable results can be given with the finite element method. It is a very complex method and it can be used if there is enough knowledge on the soil behaviour.

In this thesis a good indication of the moments, forces, penetration depth and displacement of the quay structure is needed. As a result, the stability of the quay structure can be determined. This can be achieved by applying the third method in three-dimensional; the first method is applied for an indication of the penetration depth which was not relevant to finding a solution to the problem. The second method can be applied to indicate the minimum penetration depth of the combined wall but cannot give a three-dimensional indication of the stability of the quay structure. The computer software, D-sheet, which is used provides a one-dimensional finite element solution to the problem. That is why, the third method is more reliable for this thesis research, but it is also time consuming and requires more knowledge of the soil behaviour.

# 6

## **BEAM ON ELASTIC FOUNDATION METHOD**

This chapter presents the two separate methods that were used to perform various calculations which enable the analysis of the stability of the quay structure. These two methods are both software-based, namely:

- SCIA-engineering, with this software the concrete relieving platform and the foundation elements are analysed;
- D-sheet pilling, with this software the combined wall with the reaction forces from the superstructure and foundation elements are analysed.

In the following paragraphs both calculation methods will be explained in more detail.

### 6.1 General

In chapter 5 four different design calculation methods are described, each with its own applicability. A major difference between the Blum method and the beam on elastic foundation method, is that here it is possible to calculate a sequence of construction stages in which the stress history of the sheet pile wall is used as initial conditions for the next stage. The calculations with the D-sheet piling software , developed by Deltares and based on the theory of elastic supported beam foundation, are restricted to determining sheet pile dimensions. The interface relation between soil and structural elements will be assumed linear in the elastic area and the springs. The behaviour of the soil is represented both horizontally and vertically and mutually uncoupled.

The inclination of the sheet piles and the eccentricity of the saddle in the top are issues that should be taken into account by 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 be calculated and included into the results. In Appendix D Figure D.2-3 the moment distribution of the sheet piles is illustrated.

It should also be noted that the sheet pile walls are considered vertically in the design cases. The favourable effect of inclination of the wall on both active and passive earth pressures as well as the bending moment in the sheet pile wall is not considered. In addition, the unfavourable 2nd order moment , considering the aforementioned, is not taken into account.

Both aspects cancel each other out to a certain degree, which makes the effect of leaving them out insignificant and negligible.

### 6.2 Modelling in SCIA engineering

### 6.2.1 Principle of a relieving platform

A relieving platform is built primarily to reduce the active earth pressure on the sheet pile wall, in this case the combined wall. The redistribution of the stresses is illustrated in the figure 6.2 1.



Figure 6.2 1: Principle of relieving platform [21]

The earth pressure starts on the lower side of the relieving platform instead of the ground level at the top. The line starting from the rear of the relieving platform can be used as the upper bound, at an angle  $\varphi$ , and the lower bound, at an angle  $\vartheta$ , of the transition zone. From the upper bound till the lower bound the influence of the surcharge is valid.

### 6.2.2 Modelling of the relieving platform

In this paragraph the calculation procedure of the relieving platform is described with the help of the calculation software SCIA-engineering. The relieving platform of the quay structure is schematised/modelled as a simple statically indeterminate framework. In this case the relieving platform is long and more rows of bearing piles are necessary, thus making the system a statically indeterminate system. With the framework, the vertical and horizontal reaction forces at the position of the connection between the relieving platform and combined wall are calculated. Figure 6.2 2 below shows the schematization of the relieving platform with tension piles and 2 rows of bearing piles.



Figure 6.2 2: Statistically indeterminate model

In the schematization above the connection between the tension piles, combined wall and relieving platform is simulated as a fixed hinge and the 2 rows of bearing piles as a roller. This simplified model enforces all the horizontal loads to be absorbed by the fixed hinge support. This is a very conservative approach.

The quay structure should be able to resist the various types of loads which are described in the requirements and boundary conditions. Besides the various loads it should be able to carry its self-weight. The self-weight loads imposed on the relieving platform, which result into reaction forces in the foundation elements are shown in the figure below.



Figure 6.2 3: Loads acting on the relieving platform

Description	Load type	Number
Dead load	Permanent	2,4,7
Soil weight	Permanent	3
Effective soil pressure	Permanent	11 of 12
Surcharge load	Variable	1
Water pressure	Variable	5, 13
Crane Load	Variable	6,8
Bollard load	Variable	9
Berthing load	Variable	10

### Table 6.2 1: Loads imposed on the relieving platform

From all of these loads imposed on the relieving platform only the loads that are relevant to the self-weight are taken into account for the calculations in SCIA-engineering. In this case the crane load and berthing load are neglected.

### 6.2.3 Load combinations

### 6.2.3.1 General

In the case of the load combinations, a number of unfavourable load combinations are considered in the limit states. As previously mentioned, the ultimate limit state and the serviceability limit state are considered. These are combinations of permanent and several variable loads. It is important to investigate the governing load combinations for the structural quay elements. By using the load factors  $\gamma$ f and reduction factors  $\Psi$ , the various load combinations are realized. This is determined with the following equations [1]:

$$\begin{split} &\sum_{j \ge 1} \xi_j \cdot \gamma_{G,j} + \gamma_{Q,1} \cdot G_{k,1} + \sum_{j \ge 1} \gamma_{q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \qquad \text{(Eq. 6.2 1)} \\ &\sum_{j \ge 1} G_{k,j} + A_d + \psi_{2,1} \cdot Q_{k,1} + \sum_{j \ge 1} \psi_{2,i} \cdot Q_{k,i} \qquad \qquad \text{(Eq. 6.2 2)} \end{split}$$

Table 6.2 2: Load combinations	in Ultimate Limit State
--------------------------------	-------------------------

	Permanent load G		Variable loads Q		Accidental loads
Fundamental	$\gamma_{f;g^*}G_{rep;max}$	$\gamma_{f;g^*}G_{rep;min}$	γ <sub>f;q</sub> *Q <sub>1;rep</sub>	$\gamma_{f;q^*} \Psi_{0,j^*} Q_{j;rep}$	-
Accidental	$\gamma_{f;g}G_{rep;max}$	$\gamma_{f;g^*}G_{rep;min}$	$\gamma_{f;q^*}\Psi_{1,1^*}G_{1;rep}$	$\gamma_{f;q^*}\Psi_{2,1^*}Q_{1;rep}$	F <sub>a;rep</sub>

### Table 6.2 3: Recommended reduction factors for load combinations

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

### 6.2.3.2 The applied load combinations in the original design

In the original design calculations, a load combination has been made consisting of the following [2]:

- Self-weight of the structure;
- Soil behind the quay wall above the relieving platform floor, depending on the outer and ground water level;
- Horizontal soil pressure excluding the surcharge, also depending on the outer and ground water level;
- Surcharge behind the relieving platform floor;
- Friction on the rear surface, depending on the outer and ground water level;
- Excessive water pressure, depending on the outer and ground water level;
- Friction due to the rear load;
- Surcharge on the relieving floor;
- Crane load (the normative crane load which was in this case the "lostoren");
- Bollards;
- Anchor force from the sheet pile

Afterwards, load combinations were made, using partial factors of about 0.5 and 1.0. The load combinations that were then made, consists of:

- 1. Maximum reaction forces on the combi-wall (case A-1 was calculated);
- 2. Maximum tensile force in the M.V.-piles (all 4 cases were calculated);
- 3. Maximum load on the concrete bearing piles (case A-1 was calculated);

The reason for this is to get an indication of what the capacity of the combined wall, M.V. –piles and the concrete bearing piles will be. This has to be valid for the four load cases.

### 6.2.3.3 Load combinations applied in SCIA-engineering

For the imposed loads mentioned above the following cases are investigated. The following diagram shows the 4 load cases that were modelled in SCIA-engineering.



Figure 6.2 4: Four different load cases of the investigation

In Table 6.2 4 the 4 load cases are further explained on the assumed water levels for the different cases. As shown in Figure 6.2 4 there are two main categories:

A: Low outer water level with the corresponding "logical" groundwater level;

B: High groundwater level with the corresponding low outer water level (as low as possible)

The groundwater and outer water levels are derived from the tides. For the normative loads on the combined wall and concrete bearing piles a lower groundwater level is used.

Cases	Lastfall	Outer water	Ground water	Remarks
	( L.F.)	level	level	
		(OWL)	(GWL)	
		m N.A.P.	m N.A.P.	
A1	1	-1.00	0.00	Occurring low
				tide 35 times per
				year =-1.10
A2	3	-2.00	0.00	The tidal curves
				shows the
				(high)water level
				after extreme
				low water (ca.
				N.A.P1.80 m to
				be about 2.0 to
				2.5 m higher
B1	1	0.00	+1.00	This occurs
				immediately
				after a high
				outer water
				level, where the
				water level is
				lagging
B2	3	+0.50	+2.50	For calculation
				of the
				superstructure
				with gamma =
				1.7
				High G.W.L.
				=+2.00m N.AP.
				Low G.W.L.
				=N.A.P1.50m

Table 6.2 4: The water levels used in the original design [2]

The reaction forces that are calculated with SCIA-engineering are shown in Table 6.2 5. For the 4 cases the governing values are included in the table. These values will lead to a maximum stress in the combined wall which can provide the minimum penetration length and maximum anchor force, for each case. In Appendix E.1 an extensive table with all the reaction forces for each case is given.

Table 6.2 5: The gover	able 6.2 5: The governing reaction forces and eccentric moment in support A				
Cases	Horizontal reactions in support A [kN/m]	Vertical reactions in support A [kN/m]	Total reactions in support A [kN/m]	Eccentricity moment in support A [kNm/m]	
A1	914.81	2651.66	3566.47	2532.19	
A2	944.81	2660.91	3605.72	2560.06	
B1	859.72	2528.02	3387.74	240530	
B2	875.75	2413.35	3289.10	2335.26	









Besides Table 6.2 5 also graphs are plotted, Figures 6.2 5 & 6.2 6, to provide the relation between the reaction forces and moment for each case. In these figures it is shown that case A2 has the highest reaction force and moment of eccentricity values in comparison to the other cases. The reason for this can be in this load case the tidal curves shows (high) water level after extreme low water level (ca. N.A.P. -1.80 m) that the (high) water level to be about 2.0 to 2.5m higher. This extreme change in water level can cause an extreme fluctuation in the water pressure on the passive side which has to be supported by the active side.

For the four load case combinations a partial factor of 1.0 is used. The horizontal reaction force from the superstructure and the vertical reaction force on the combined wall have been used as input for the calculation in D-sheet. The influence of the relieving platform on the combined wall will also be taken into account. In the framework analysis, which is modelled in SCIA-engineering, the surcharge is modelled as a uniform distributed load.

### 6.3 Modelling in D-sheet piling

### 6.3.1 Combined wall modelling

In D-sheet piling software the sheet pile wall, in this case the combined wall, is considered vertical. The inclination effects of the sheet pile wall which are favourable, are not considered. The eccentricity of the axial load reduces the maximum bending moment, meanwhile second order moment effects will be unfavourable to the stability of the sheet pile wall. That is why, the second order effects will not be taken into account. The soil layers that are used in this model are found in the previously mentioned CPT. The structure is modelled in two ways in D-sheet:

- 1. a surcharge of 20 kN/m<sup>2</sup> is applied starting at the rim of the quay reaching 38 meters deep into the terrain; the surcharge increases up to 450 kN/m<sup>2</sup> beyond the aforementioned 38 meters (The original design) (See Figure 6.3 1)
- 2. A uniformly distributed surcharge is applied starting at 18m landside which continues landward until 107 m. Positioning the surcharge closer to the rim would lead to direct failure and is left out for that very reason (See Figure 6.3 2). The uniform surcharge is varied with in crements from 100 kN/m<sup>2</sup> up to 450 kN/m<sup>2</sup>.

Besides the variation of the surcharge the harbour depth is varied also. The calculation is performed with two different harbour depths:

- The contract depth of N.A.P. -24.00m;
- The construction depth of N.A.P. -25.50m

The construction stages have also been used as input into D-sheet. In this software, the soil behaviour effects are not taken into consideration. But the stages are used as input anyway to try to recreate the effects of the construction stages in reality. The construction stages for this quay structure are as follow (Appendix E):

- 1. Excavation till N.A.P. -6.00m;
- 2. Add anchor on the right side of the combined wall;
- 3. Built the relieving platform;
- 4. Excavation on the left side of the combined wall till N.A.P.-25.50m;
- 5. Dewatering
- 6. Add all the surcharges on the right side of the combined wall.



Figure 6.3 1: Combined wall model according to the original design



Figure 6.3 2 Combined wall model with a simplified uniformly distributed surcharge

It can be noted from the figures above that for the load combination of both models, the vertical reaction force to the combined wall and the horizontal force from the superstructure are applied in D-sheet. The vertical reaction force is multiplied by the distance of eccentricity of the saddle in the connection support A (connection between the relieving platform and the combined wall). This will result in an external moment at the top of the combined wall.

The anchor is simulated with the actual properties of the applied M.V.-piles. This a conservative approach. In the table below the properties that were used in the model are given.

	Units	Open tubular piles	Intermediate sheet piles
Thickness (t <sub>t</sub> )	mm	20	14.1
Thickness <i>(t<sub>s</sub>)</i>	mm	-	10
Width <i>(b)</i>	mm	1420	1500
Height <i>(h)</i>	mm	30500	23500
Section area (A)	mm²/m	29530	20133
Weight <i>(G)</i>	kg/m <sup>2</sup>	691	158
Moment of inertia (I)	mm⁴/m	7.2349*10 <sup>9</sup>	3.7053*10 <sup>8</sup>
Section modulus (W)	mm <sup>3</sup> /m	1.0188 *10 <sup>7</sup>	1.5533*10 <sup>6</sup>

Table 6.3 1: Applied characteristics of the open tubular piles and intermediate sheet piles

### Table 6.3 2: Applied characteristics of the combined wall

	Units	Combined wall
System length	тт	2980
Moment of elasticity (EI)	kNm²/m¹	1558195
Moment of inertia (I)	mm <sup>4</sup> /m	7.419*10 <sup>9</sup>
Section modulus (W)	mm³/m	1.045 *10′
Max. bending moment (M <sub>max.</sub> )	kNm	4681.6

### 6.3.2 Results from D-sheet piling

In the following paragraph the results from D-sheet will be analysed for both combined wall models. In the graphs below the maximum moment is given for each load case. In these graphs, the relation between the maximum bending moment and the retaining height is presented with different surcharges with a construction depth of N.A.P. -25.50m

### Quay structure with trapezium distributed surcharges behind the relieving platform

In Table 6.3 3 the relation between the surcharge and the minimum penetration depth needed for the quay structure to retain its stability can be observed. For the load cases the difference between the outer water level and groundwater level is between 1 and 2 meters. As it can be observed with increasing surcharge the minimum penetration depth will increase. For all of the load cases the minimum penetration depth for the combined wall is between N.A.P. -30.7 till -34.5m. In the figures below the maximum bending moment and minimum penetration depth for the various load cases are given (Appendix E).

Surcharges [kN/m <sup>2</sup> ]	Minimum penetration depth load case A1 [-m N.A.P.]	Minimum penetration depth load case A2 [-m N.A.P.]	Minimum penetration depth load case B1 [-m N.A.P.]	Minimum penetration depth load case B2 [-m N.A.P.]
0	30.8	32.1	30.7	32.8
200	32.4	32.7	31.7	33.7
450	33.2	33.6	32.2	34.5

### Table 6.3 3: The relation between the surcharge and the minimum penetration depth for each load case



Figure 6.3 3: Minimum penetration depth for each load case



Figure 6.3 4: Maximum bending moment for each load case

### Quay structure with simplified uniformly distributed surcharges

In Table 6.3 4 the relation between the surcharge and the minimum penetration depth needed for the quay structure to retain its stability can be observed. Here, also the difference between the outer water level and groundwater level is between 1 and 2 meters for the load cases. As it can be observed with increasing surcharge the minimum penetration depth will increase. For all of the load cases the minimum penetration depth for the combined wall is between N.A.P. -32.5 till -34.9 m. In the figures below the maximum bending moment for the various load cases for each calculated penetration depth is given (Appendix E).

Table 6.3 4 The relation between the surcharge and the minimum penetration depth for each load case for the simplified uniformly distributed surcharges

Surcharges [kN/m <sup>2</sup> ]	Minimum penetration depth load case A1 [-m N.A.P.]	Minimum penetration depth load case A2 [-m N.A.P.]	Minimum penetration depth load case B1 [-m N.A.P.]	Minimum penetration depth load case B2 [-m N.A.P.]
0	32.5	32.7	32.5	32.8
200	33.5	33.6	33.5	33.8
450	34.1	34.7	34.6	34.9



Figure 6.3 5: Minimum penetration depth for each load case



Figure 6.3 6: Maximum bending moment for each load case

If we compare the graphs above there is no significant difference between the two models according to the minimum penetrated depth of the combined wall. Furthermore, the graphs above indicate that as the surcharge increases till 450 kN/m2 the penetrated depth for the combined wall increases as well. The penetrated depth of the combined wall will reach its fixitity in the soil with increasing surface loads.

Now, the results for the quay structure with trapezium distributed surcharges behind the relieving platform are given in Figures 6.3 3 and 6.3 4. The results in Figure 6.3 3 indicates that for load cases A1 and B1 the minimum penetration depth are lower than load cases A2 and B2. The difference between these paired load cases is the water level difference. For load cases A2 and B2, the water level difference is about two meters therefore the bending moment exerted on the combined wall will be higher for these load cases. In Figure 6.3 4 the maximum bending moment for load case B2 is higher than load case A2 because the levels for the outer water and the groundwater are situated higher.

Also, the results for the quay structure with a simplified uniformly distributed surcharge are given in Figures 6.3 5 and 6.3 6. The results in these graphs also indicate that the load cases A2 and B2 have a higher bending moment

and minimum penetration depth of the combined wall due to the water level difference.

### 6.4 Conclusion

Beam on elastic foundation method, by means of D-sheet piling, gives the opportunity to analyse the combined wall in a sequence of construction stages. The major disadvantages of this calculation method, as mentioned before, are :

- The superstructure has to be analysed separately, in SCIA engineering, thus the redistribution of the forces should be considered.
- Furthermore, the vertical arching effects of the soil working on the active side, will result in the reduction of bending moments and higher anchor forces.

From the comparison between the two models it can be concluded that by modelling the distribution of the surface loads as a trapezium or uniformly the difference of minimum penetrated depth of the combined wall is considerably small. Furthermore, the field bending moment increases as the surface load increases which leads to a freely imposed bending moment. When the combined wall is shorter than the calculated minimum penetrated depth the quay structure model becomes unstable. With a 2-dimensional software such as D-sheet piling it is apparent that the (in)stability of the quay structure is not investigated for the entire length of a quay section of 45 meters. It is only a cross section per meter that is calculated. Therefore, to implement the 3-dimensional effects, a 3-dimensional method would be very helpful in analysing the actual stability of the entire quay structure.

Consequently, the three dimensional Finite Element Method program Plaxis 3D is expected to be a better tool for analysing the (in)stability of the quay structure along the deep sea terminal at the Amazonehaven.

# 7

### FINITE ELEMENT METHOD

This chapter presents the finite element method analyses for the quay structure at the Amazonehaven, port of Rotterdam. As discussed in the previous chapter it was found that the use of D-sheet Piling software was not suitable for calculating the stability problems of the quay structure for this research. To perform such calculations the switch is made to the finite element program, Plaxis 3D. in this program the quay structure elements, the soil layout and external forces are the input for the various models. With the input for various models the program calculates and evaluates these models and gives the governing safety factors for each model as output. The results can be viewed in various forms.

This finite element software, Plaxis 3D, which have been widely accepted as a powerful tool to solving boundary value problems in geotechnical engineering, can provide an interpretation of the model with the reality based on the 3-dimensional assessment by numerical modelling of the soil behaviour than those results found using the previous calculation methods, Blum and D-sheet piling.

In the following paragraphs the finite element calculation method will be explained in more detail.

### 7.1 Input parameters in Plaxis 3D

Similar to the previous methods an appropriate quay structure is the input model in the finite element method software, PLAXIS. This software allows for more accurate quay structure modelling, specifically oblique installed combined walls, and the analysis and evaluation of plastic soil deformation are also possible. In this paragraph, first the soil data models are defined by analysing the geological soil data from Appendix A. Second, the structure material model are defined by using the information available of the deep sea quay structure at the Amazonehaven (Appendix A). For the first set of calculations only the self-weight of the relieving platform structure is being considered. However, this is not the same as the design of the quay structure but this is how the quay structure was used during its lifetime.

### 7.1.1 Modelling soil and structure material parameters

### 1. Modelling the soil

As mentioned before, the soil layers and properties are based on geological investigations that were carried out in the Amazonehaven. When modelling the foundation subsoil a stratum is created that is large enough so that the boundaries of the model will not affect the calculation results. The length of the quay section is 45 meters as mentioned before. Therefore the width and length of the soil stratum is chosen as 45m x 170m. For the depth of the soil stratum a distance of 50m is modelled. This makes that the dimensions of the soil model are 45m x 170m x 50m (length x width x depth).

The purpose of this research in this chapter is to investigate the stability of the quay structure, as mentioned before. The subsurface consists of various layers of sand with formations of clay and silt in the Holocene layer. The boundary between the Holocene and Pleistocene layer is characterized by a weak clay layer. From field measurements it has been concluded that the relieving platform structure is practically not in contact with the

soil underneath it [14].For the soil model set up in Plaxis 3D the subsurface is divided into four layers, these are:

- 1. The replenished soil layer, consists of dry dense sand, from about N.A.P. 0.0 till +5.0 m;
- 2. The Holocene layers, consists of dense sand, from N.A.P. 0.00 till -22.0 m;
- 3. The clay layer, which is in the Holocene layer, from N.A.P. -20.6 till -21.7 m;
- 4. The Pleistocene layer from N.A.P. -21.7 m and below.



Figure 7.1 1: Schematization soil stratum in Plaxis 3D

By using only four different soil layer of the subsurface, see Figure 7.1 1 and Figure 7.1 2, the complexity of the models is small even though the focus of the research is the effect of various penetration depth of the combined wall on the stability of the quay structure. For the soil materials the physical properties as entered in Plaxis 3D model are based on the clean sand and clean clay material presented in the CUR 166 table 3.1. The major parameters are the self weigth ( $\gamma_{unsat/sat}$ ), the stiffness ( $E_{50}$ ), cohesion ( $c'_{ref}$ ) and the internal angle of friction ( $\varphi'$ ). The used strength and stiffness properties are listed in the tables below. These soil materials the Hardening Soil model is selected which has some advantages over the Mohr-Coulomb model. Since the Hardening Soil model also requires stiffness parameters of the soil which are also listed in the tables below. Also the drained behaviour is used to model the soil. By using this drainage type a long-term soil behaviour without the need to model the precise history of undrained loading and consolidation. For further information on this drainage type the Plaxis 3D reference manual 2013 is referred.

	Sand dense	Clay	Pleistocene
$\gamma_{unsat} [kN/m^3]$	18.5	17.0	18.0
$\gamma_{sat} [kN/m^3]$	20.5	17.0	20.0
C' <sub>ref</sub> [kN/m <sup>2</sup> ]	0.5	5.0	0.1
φ´[°]	34.0	19.0	35.0
ψ [°]	4.0	0.0	5.0

Table 7.1 1: General and strength properties sand, clay and Pleistocene

Table 7 1 2.	Stiffnors pror	ortion for	Hardoning	Soil model
Table 7.1 2:	Summess prop	perties for	Hardening	Solimodel

	Sand dense	Clay	Pleistocene
$E_{50} [kN/m^2]$	65 000	4 000	50 000
$E_{oed}^{ref} [kN/m^2]$	65 000	4 000	50 000
$E_{ur}^{ref} [kN/m^2]$	195 000	12 000	150 000
Power (m)	0.5	0.5	0.5

For more extensive information on the soil properties used in the model and the determination of the soil strength and stiffness properties are presented in Appendix F.

### 2. Phreatic level

The phreatic level for the model is stated at N.A.P. +1.00m. The phreatic level in the soil stratum should be the groundwater level on the right side of the quay structure. An illustration of the soil model and the phreatic level as modelled in Plaxis 3D is added in the Appendix F.



Figure 7.1 2: Schematization quay structure model

### Modelling of the relieving platform structure

The relieving platform structure is modelled (see Figure 7.1 2 ) as a linear elastic non-porous soil material with the behaviour characteristics of concrete. The strength properties of concrete are usually higher compared to that of soil. the non-porous drainage type is applied in order to exclude the pore pressures from the structural elements. This way the thickness of the structure is included in the model and then the interaction between the soil and the concrete structure can be modelled properly. Concrete can become cracked after a certain time which reduces the stiffness of the structure. The concrete relieving platform structure is modelled as un-cracked at which the modulus of elasticity can be considered at its highest. The relieving platform structure height is from +5.0 till -6.0 m N.A.P. and the width of the relieving floor is around 18m and the length of one section is around 45m. The combined wall, M.V.-piles and concrete bearing piles are attached to the concrete relieving floor which has a thickness of around 1,5 m.



Figure 7.1 3: Schematization of the hinge support

The connection between the combined wall and the relieving platform structure consists of an eccentric hinge support. Figure 7.1 3 illustrates the connection modelled in Plaxis 3D. The manner in which it is simulated is by modelling the relieving platform structure and combined wall plates as a hinge. Therefore, the bending moments at the top part of the combined wall are created due to the eccentric connection with the relieving platform structure. As a result the maximum bending moment in the field is lower than without the eccentric connection.

### Modelling of combined wall

The combined wall (see Figure 7.1 2 ) consist of open tubular piles and intermediate sheet pile elements. The

dimension properties of the combined wall are referred to paragraph 2.1.3. For modelling the combined wall it is very important to keep in mind the capacity of the computer used to model this element. In this case, the combined wall dimensions needed to be simplified. Instead of tubular piles and intermediate sheet pile elements, plates were used with the same modulus of elasticity as the original combined wall elements. The moment of inertia for a circle is set equivalent to the moment of inertia for a rectangular plate. From this the length and width of the rectangular plate is determined. (See Figure 7.1 4 and Table 7.1 3)



Figure 7.1 4: Moment of inertia circle equivalent to a rectangular area

Table 7.1 3: Actual tubular piles	and modelled plates properties
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Tubular piles	Plates	
Diameter = 1420 mm	Length = 1420 mm	
Thickness = 20 mm	Width = 716 mm	
Depth in length = 30.5 m	Depth in length = 30.5 m	
Moment of inertia = $0.02156 \text{ m}^4$	Moment of inertia = $0.02156 \text{ m}^4$	
Elastic modulus = 210*10 <sup>6</sup> kN/m <sup>2</sup>	Elastic modulus = 210*10 <sup>6</sup> kN/m <sup>2</sup>	

As previously mentioned, the intermediate sheet piles are modelled as plates as well. However, the material properties of the intermediate sheet piles are the same only the modelled sheet piles are simplified. (See Table 7.1.4)

	Intermediate sheet piles	Plates	
	Diameter = 1500 mm	Length = 1560 mm	
	Thickness = 14.1 mm	Width = 218 mm	
	Depth in length = 23.5 m	Depth in length = 23.5 m	
	Moment of inertia = $5.5580*10^{-4}$ m <sup>4</sup>	Moment of inertia = $5.5580 \times 10^{-4} \text{ m}^4$	
	Elastic modulus = $210*10^6$ kN/m <sup>2</sup>	Elastic modulus = $210*10^6$ kN/m <sup>2</sup>	

7.1 4) Table 7.1 4: Actual intermediate sheet piles and modelled intermediate plates properties

The open tubular piles are modelled with a centre to centre distance of 2.98 m, also known as a system length. The strength and stiffness properties of the combined wall are referred to Appendix F. The combined wall ranges from N.A.P. -6.0 till -36.5m at an angle of 1:5. The intermediate sheet piles ranges from N.A.P. -6.0 till -29.5 m. The tubular piles are modelled as a plate element which does not have the same bearing capacity as an open tubular pile. Therefore, at the toe of the plate element a horizontal plate element is modelled with the same area as the diameter of the open tubular piles.

### M.V.-piles and concrete bearing piles

The M.V.-piles and concrete bearing piles (see Figure 7.1 2) are structural elements which are installed into the subsoil to respectively anchor the quay structure and support the relieving platform structure. These elements are not intended to create a solid earth retaining wall. Therefore, in Plaxis 3D software, there are many types of anchors which can be used to model the M.V.-piles and concrete bearing piles, namely:

- Fixed –end anchors;
- Node-to-node anchors;
- Embedded piles.

For this model the embedded piles approach is used to model both the M.V.-piles and the concrete bearing piles. The reason for choosing this approach is due to the grouted part of the anchor (M.V.-piles) which needed to be created.

The M.V.-pile is modelled at an angle of 45 degrees and with a system length of about three meters which ranges from N.A.P. -4.5 till -28.0m. At the top part of the anchor encased in the relieving platform structure a rigid support is given.

The concrete bearing piles at the back of the quay structure model are aligned into two rows with a centre to centre distance of two meters between the rows and the piles are spaced three meters apart in the field rows. For both rows of concrete bearing piles the top part encased in the relieving platform structure is modelled as a hinged support. By installing these piles under an certain angle the stability of the quay structure can be increased. The first row of piles is installed at an angle of 1 : 3.5 ranging from N.A.P. -4.5 till -27.5m and the second row of piles is at an angle of 1:4 ranging from N.A.P. -4.5 till -27m.

The strength and stiffness properties of these anchors can be referred to Appendix F.

### 7.1.2 Mesh generation of the model

Before Plaxis 3D software can perform calculations a mesh is created for both the soil and the quay structure. The mesh properties are set to medium in order to obtain a mesh fine enough for the finite element model situation. The Plaxis 3D software meshes the model automatically. At the interface between the plate and the soil the mesh is more dense and at larger depths the mesh becomes coarse. The result of the mesh for the quay structure (calibration model) is presented in the figure below.



Figure 7.1 5: Generated mesh model for the quay structure

### 7.1.3 Schematization water level head

The water level head difference is based on water level data from the Port of Rotterdam Authority. In the quay structure model the water level in the harbour basin is set to N.A.P. -1.00m and the groundwater level to N.A.P. +1.00m. So the water level head difference is two meters. Near the relieving platform structure drains are installed that will lower the head difference, because the actual head difference is expected to be smaller. However, the disadvantage with drains are that they can get clogged which can result into a higher water level difference near the relieving platform structure. For further calculations drains are not included in the models.

### 7.1.4 Modelling of the construction stages

To model the deformation behaviour of the deep sea quay structure, the quay structure is implemented in Plaxis 3D with all the construction stages. This is called the staged construction mode. In these construction stages it is indicated which objects, soil layers and forces in the model should be activated or deactivated. The model as used for this research consists of ten calculation stages. The stages that are implemented in Plaxis 3D are (Appendix F):

 Initial phase. The first stage represents the initial phase when no excavations were performed at the Amazonehaven. The ground levels on both sides of the quay structure are equal, N.A.P. +5.00m. In this phase the initial effective stresses and pore pressures are calculated. *In this* stage only the soil layers are activated.

- 2. Between the initial phase and the safety calculation phase, the construction stages of the quay structure are implemented
  - a. Excavation of a trench till N.A.P. -6.50m. Normally, there is a slope of approximately 1:1.5 from surface level till the bottom of the future relieving platform. In the quay structure model the excavation is modelled horizontally till N.A.P. -6.50m. The excavation takes place on both sides of the quay structure. Simultaneously, the water level at both sides is lowered to N.A.P. -7.0m. *In this case, the soil layers from N.A.P.* +5.00m till N.A.P. -6.50m are deactivated.
  - b. After the excavation of the trench the open tubular and intermediate sheet piles are installed till N.A.P. -29.0 and -36.5m. *Apart from the soil layers from previous calculation stage 2a. that are activated the plate elements for the open tubular piles and intermediate sheet piles are activated.*
  - c. Then the M.V. –piles anchors are placed at angle till N.A.P. -28.0m. *Here the M.V.piles are activated and the volume in the relieving platform structure where the em bedded piles (M.V.-piles) are connected are activated. The material property of this volume is for "Gewichtloos Beton". (Appendix F)*
  - d. The first and second rows of concrete bearing piles are placed. Each row at a certain angle and depths between N.A.P. -27.0 and -27.5m. *Just as the M.V.-pile elements are activated both rows of concrete bearing piles elements are activated as well.*
  - e. The concrete relieving platform is casted on top of these foundation members. Afterwards the space behind the relieving platform is filled with sand. *The plate elements for the relieving platform structure are activated and the soil layers on the right side behind the relieving structure element are activated.*
  - f. The last construction phase is the dredging of the harbour basin till the construction depth of N.A.P. -25.5m. *The soil layer on the left side of the quay structure element are deactivated till construction depth.*
  - g. The water level in the harbour basin and the groundwater level will be increased in increments to the levels mentioned in section 7.1.3. *The water levels are activated.*

Now the construction stages of the quay structure model are fully implemented. The loads defined for this model are the self-weight of the relieving platform structure, the water level head difference and the soil on top and behind the relieving platform. These are the loads that are considered to act constantly on the quay structure during its service lifetime.

3. Safety calculation phase. Here, a  $\varphi$  -c reduction takes place until the quay structure model fails.

### 7.2 Deformation behaviour of the quay model

In order to understand the deformation behaviour of the quay structure, specifically the combined wall, a calibration model is set up. This model is based on the original designed quay structure without surface loads. This model will give an insight into the deformations and structural forces of the quay structure which can be compared to the results from the other five models. When comparing the five models with the calibration model the deformation of the foundation elements and relieving platform structure should be closely monitored. The reason for this is the stability of the quay structure model is not only dependent on the deformation of the combined wall but also on the displacement of the foundation elements and relieving platform structure as well.

First the calibration model will be discussed, then the five models will be described. Next the deformation and structural forces will be compared to the calibration model. This is focussed on the effects the various penetration depths of the combined wall can have on the stability of the quay structure.

### 7.2.1 The calibration model

The calibration model is based on the modelling schematization described in section 7.1. This model consists of a section, with a length of about 45m.

The deformed mesh of the quay structure model (See Figure 7.2 1) illustrates the displacement of the relieving platform structure and the combined wall moving towards the waterside due to the soil load and water level head difference. The M.V.-piles retain the quay structure meanwhile the concrete bearing piles supports the relieving platform structure. These foundation elements also slightly move towards the waterside.



Figure 7.2 2: Coordinate system used in Plaxis 3D [25],[26]


Figure 7.2 1: Deformed mesh of the calibration model

The total displacement |u| of the deformed mesh is equal to 0.4585 m. Figure 7.2 3 illustrates the area where maximum displacement is possible of the deformed mesh which is behind the relieving platform structure near the surface level.

In figure 7.2 4, the maximum horizontal displacement  $u_y$  of the deformed mesh which is 0.1050 m. The maximum displacement is directly at the surface level (N.A.P. +5.0m) behind the relieving platform and about 4.5m from the waterside.



Figure 7.2 3: Total displacement |u| deformed mesh calibration model



Figure 7.2 4: Total displacement uy deformed mesh calibration model

The following figure below illustrates the maximum vertical displacement  $u_z$  of the deformed mesh which is about 0.4523 m. The maximum displacement behind the combined wall near the surface level (N.A.P. +5.0m).



Figure 7.2 5: Total displacement uz deformed mesh calibration model

In Appendix F the total displacements |u|, horizontal displacement uy and vertical displacement  $u_z$  are also illustrated.

The structural forces for each foundation element of the calibration model can be evaluated separately. This will show the behaviour of the soil on the elements and if these elements are still capable of retaining the soil pressure before yielding.

#### Maximum structural forces in the combined wall

In Table 7.2 1 the structural forces of the combined wall are given. The forces M, Q and N are given per running meter length combined wall. The maximum bending moment of **4681.6 kNm/m** is not reached this indicates that the steel quality of the combined wall can retain the load of the structural elements and soil. Appendix F illustrates the bending moments of the combined wall and bending moments are created at the top of the combined wall due to the eccentric connection with the relieving platform structure. As a result the maximum field bending moment is lowered. In Appendix F the total displacements, bending moments, shear forces and axial forces in the combined wall are illustrated.

Combined wall	
Maximum total displacement  u  [m]	0.3780
Maximum bending moments ( $M_{22}=M_{xx}$ )	3686
[kNm/m]	
Shear forces (Q <sub>23</sub> =Q <sub>xy</sub> ) [kN/m]	5496
Axial forces $(N_1=N_z)$ [kN/m]	1173

#### Table 7.2 1: Structural forces |u|, M, Q and N in combined wall

#### Maximum structural forces in the M.V.-piles and concrete bearing piles

In Table 7.2 2 and Table 7.2 3 the structural forces of the M.V.-piles and concrete bearing piles are given. Here, the forces M, Q and N are given for the entire length of the foundation elements. Appendix F illustrates the structural forces and displacements working on these bearing piles and anchors.

The bending moments for both the anchors and bearing piles (Figure 7.2 6) illustrate some shielding over the Holocene layers at the intersection of the M.V.-piles and concrete bearing piles. The shielding effect from these piles also depends on the displacement of the combined wall. When the sign changes from negative bending moments to positive bending moments, which happens near and underneath the relieving platform structure, at this location the shear forces reaches its maximum. The horizontal soil stresses acting on the bearing piles and M.V.-pile anchors will be transferred by shear forces to the top and bottom part of the subsoil (Appendix F).

M.V piles				
Maximum total displacement  u  [m]	0.3886			
Maximum bending moments $(M_2=M_x)$	232.8			
[kNm]				
Shear forces ( $Q_{13}=Q_{zy}$ ) [kN]	749.3			
Axial forces (N) [kN]	1318			
Maximum skin friction (T <sub>skin</sub> ) [kN/m]	392.0			

#### Table 7.2 2: Structural forces |u|, M, Q and N in the M.V.-piles

#### Table 7.2 3: Structural forces |u|, M, Q and N in the concrete bearing piles (2 rows)

Concrete bearing piles	
Maximum total displacement  u  [m]	0.4053
Maximum bending moments (M <sub>2</sub> =M <sub>x</sub> )	124.7
[kNm]	
Shear forces (Q <sub>13</sub> =Q <sub>zy</sub> ) [kN]	285.9
Axial forces (N) [kN]	3642
Maximum skin friction (T <sub>skin</sub> ) [kN/m]	447.6



Figure 7.2 6: Bending moments (M2) M.V.-piles and concrete bearing piles [Negative =red area, Positive = blue area]

#### 7.2.2 Quay structure models simulations

To determine the behaviour of the soil, specifically when the combined wall have not reach its intended depth over the entire length of the quay, is by simulating a couple of quay structure models with extreme conditions. The results of these models will be compared to the calibration model based on the structural forces of the foundation elements. For each model the soil stresses and displacements underneath the relieving platform structure and near the combined wall are different from the calibration model.

In this section the models will be analysed and compared for the last construction phase (2f) which is before the safety calculation phase. For the safety check outcomes section 7.3 will be referred to. First the models that are simulated will be described, afterwards a comparison of the structural forces will be made. These results will give an indication on the deformation and the structural forces of the quay structure models.

The manner in which the models are chosen are by simulating the maximum and minimum boundary conditions of the combined wall and then creating models that will vary in penetration depth between these extrema. The calibration model is the maximum boundary condition of the combined wall. The other quay structure models are:

#### o Model 1: Quay structure without the combined wall

This model consists of a quay structure without combined wall. This is not a very realistic simulation, but this results in the minimum boundary condition. As is shown in the figure below the model fails due to the forward

sliding of the soil when dredging the harbour basin. The M.V.-piles and the concrete bearing piles are not capable to hold the soil from sliding forwards.



Figure 7.2 7 Deformed mesh quay structure without combined wall



Figure 7.2 8 Front view of Model 1

#### o Model 2: Quay structure simulated on two supported ends

This model consists of two versions, model 2A and model 2B. Model 2A consists of a combined wall with an opening in the centre starting from N.A.P. -6.0m of about 25 meters. This model simulates a quay structure on two supported ends. Figure 7.2 9 illustrates the soil pushing towards the waterside through the opening. In the last construction phase the quay structure is not capable to retain the soil due to this opening. During dredging of the harbour basin too much soil will slide towards the waterside at which the quay structure fails.



Figure 7.2 9: Model 2A : Deformed mesh quay structure simulated on two supported end



Figure 7.2 10: Front view of Model 2A

Therefore, to be able to perform a safety analysis calculation on this model after all the construction phases have been calculated, a modified version (Model 2B) is created. Model 2B (See Figure 7.2 11) is almost the same as model 2A however the opening in the centre of the combined wall starts at N.A.P. -27.5m. This is just 2 meters below the construction depth of N.A.P. -25.5m.



Figure 7.2 11: Model 2B: Deformed mesh modified quay structure simulated on two supported end



Figure 7.2 12: Front view of Model 2B

#### o Model 3: Quay structure simulated on one support

Here the model also consists of two versions, model 3A and model 3B. Model 3A consists of a combined wall with on either sides of the combined wall an opening starting from N.A.P. -6.0m of about 10 meters wide. This model simulates a quay structure on one support. Figure 7.2 13 illustrates the soil pushing towards the waterside through either sides of the opening. In the last construction phase the quay structure is not capable to retain the soil due to these openings. During dredging of the harbour basin too much soil will slide towards the waterside at which the quay structure fails.



Figure 7.2 13: Model 3A: Deformed mesh quay structure simulated on one support



Figure 7.2 14: Front view of Model 3A

Therefore, to be able to perform a safety analysis calculation on this model, a modified version (Model 3B) is created. Model 3B (See Figure 7.2 15) is almost the same as model 3A however the opening at either sides starts at N.A.P. -27.5m This is also just 2 meters below the construction depth of N.A.P. -25.5m.



Figure 7.2 15: Deformed mesh modified quay structure simulated on one support



Figure 7.2 16: Front view of Model 3B

#### 7.2.3 Comparison of the quay structure models

From the result comparison between the various quay structure models and the calibration model can be concluded that Model 2B and calibration model can be further investigated. Models 1, 2A and 3A were considered based on their extreme conditions which is perfect to analyse with Plaxis. However, these models will not occur in practice but it is always interesting to investigate such conditions.

When comparing the calibration model and model 2B there is a distinct difference in deformation, displacements, bending moments, anchor forces. The total displacements for model 2B is three times larger than the calibration model. This gives an indication that the opening in the combined wall will have an influence on the displacement of the soil. Besides, the moment will increase by almost 1.4 times moment of the calibration model. Because of the increase in moment the anchor force will decrease by 0.80 times the anchor force of the calibration model.

A unity check is performed on these two models which resulted in the calibration model being satisfied. However, the unity check for model 2B is not satisfied. For this model the steel combined wall will yield before failure can occur at the passive side. Normally, steel quality for the combined wall need to be changed but for this thesis

research the steel quality is given.

#### 7.3 Safety analysis

After the first comparison between the various quay structure models and the calibration model in Plaxis 3D a safety analysis is carried out. The safety analysis results in a safety factor which can be obtained by reducing the strength parameters incrementally. Starting from unfactored values that are inputted when creating the soil layers of the models,  $\varphi$ avail and cavail, until equilibrium can no longer be achieved in the calculations. The safety calculation is performed only on the last construction phase when the quay structure is ready for use. The results represents the behaviour of the loads acting on the quay structure, in this case the self-weight, at the defined construction phase. For more detailed description on the method of safety analysis you are referred to the Reference manuals Plaxis and Brinkgreve and Bakker (1991).

The safety analysis is executed for the calibration model and the other five quay structure models which will be compared to each other based on their safety during its service lifetime. The results of the calculation phases reached are presented in the table below. For more information on the construction phases section 7.1.4 is referred.

the second se	
Models	Calculation phases
Calibration model	1 till 3
Model 1	1 till 2f
Model 2A	1 till 2f
Model 2B	1 till 3
Model 3A	1 till 2g
Model 3B	1 till 2g

Table 7.3 1: Calculation phases reached in Plaxis 3D

From the reached calculation phases for each model it is evident that only two models have undergone the entire calculation process. The other four models were not able to proceed through the entire calculation process.

- Model 1: As described in section 7.2.2 this quay structure model is simulated without a combined wall. It is obvious that when simulating the dredging of the harbour basin, this model fails due to anchors and piles cannot retain the soil from moving towards the waterside without a combined wall.
- Model 2A:This quay structure model is simulated with a combined wall with an opening in<br/>the centre starting from N.A.P. -6.0m of about 25 meters, as previously described.<br/>Through the opening soil slides forward to the waterside which creates collapse of<br/>the soil underneath the relieving platform structure. This results in the vertical<br/>effective pressure above the relieving platform structure will become larger than the<br/>vertical effective pressure beneath the relieving platform structure form structure (Appendix F)
- Model 3A: This quay structure model is simulated with of a combined wall with on either sides of the combined wall an opening starting from N.A.P. -6.0m of about 10 meters wide, as previously described. Through the side openings soil also slides forward to the waterside which creates collapse of the soil underneath the relieving platform structure and behind the combined wall. This quay structure model is not a realistic model. This will not occur in practice.
- Model 3B: This quay structure model is simulated the same as model 3A however the opening at either sides starts at N.A.P. -27.5m, as previously described. In this case the calculation phase could not proceed because of a load advancement procedure failure. This occurs when the total load specified in the phase cannot be applied due to the number of load steps is not sufficient. The specified number of load steps have been increased manually. It still needed more number of load steps but at a certain point the calculation is cancelled because it is very time consuming. For more information the Reference manual Plaxis 3D is referred to.

Now the calibration model and model 2B can further be evaluated based on the safety analysis calculation. Also,

these models will be compared to each other based on their (in)stability. The safety factor value (S.F.-value) for the calibration model is equal to **1.621** and for model 2B this is **1.559**. The safety factor value implies the reduction factor that is imposed on the friction angle and cohesion until the quay structure becomes unstable. In the figure below, the development of the safety factor for the calibration model and model 2B is plotted for the phases 1 to 3 as a function of the steps of the quay structure model.



Figure 7.3 1: Safety factors for Calibration model and Model 2B without surcharge

The results shows after a safety analysis calculation the calibration model can withstand more reduction of the strength parameters until failure than model 2B. Underneath the combined wall higher stresses occur and these are very interesting to compare. The figures below shows for both model the potential of numerical methods when investigating failure mechanisms soil-structure interaction should be taken into account. The structural elements, specifically the combined wall, will yield due to the effective horizontal soil pressure from the active side. The effective horizontal soil pressure on the active side underneath the relieving platform structure, which can be recognized by the red points, becomes larger than the effective horizontal soil pressure on the passive side.



Figure 7.3 2: Plastic points calibration model



Figure 7.3 3: Plastic points model 2B

However, the phase displacements of the models a difference can be observed at the passive side. The contour lines are different at the passive side in front of the combined wall. In Figure 7.3 5 at the passive side the phase displacement is larger and at the surface level of the harbour basin. This is because of the opening being two meters below the construction depth. According to the  $\varphi/c$  –reduction calculation for Model 2B and the calibration model the calculated deformations are not realistic, but it can illustrate how the quay structure model will behave. The bending moments in the combined wall will be checked for both models.



Figure 7.3 5: Phase displacement model 2B

#### 7.4 Design model with surcharge

The previous analysis of the quay structure models are based on the quay structure without surcharge. The results of these models are compared to the calibration model based on structural stability. However, the results obtained from these models have not answered the research questions. Therefore, the calibration model and model 2B are created with the designed surcharge.



Figure 7.4 1: Deformed mesh calibration model with designed surcharge



Figure 7.4 2: Deformed mesh calibration model with designed surcharge without soil

The models are compared to each other based on structural stability and unity check. The tables below shows the results of these two models. For further information Appendix F is referred.

	Calibration model with surcharge	Model 2B with surcharge
Combined wall		
Maximum total displacement  u	0.3908	0.412
[m]		
Maximum bending moments	3393	6543
$(M_{22}=M_{xx})$ [kNm/m]		
Shear forces (Q <sub>23</sub> =Q <sub>xy</sub> ) [kN/m]	6687	8658
Axial forces $(N_1=N_z)$ [kN/m]	1258	2672

Tahle '	7 <i>Δ</i> '	1٠	Structural	forces	hul		M	0	and	N	in	combined	wall
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Table 7.4 2: Safety comparison between calibration model and model 2B

	Calibration model with surcharge	Model 2B with surcharge
Unity Check	0.738	1.436
Safety margin	1.355	0.696
Safety factor	1.494	1.473

From Table 7.4 2 the safety factor value (S.F.-value) for both models are plotted in the curve below. The S.F.-value for the calibration model is equal to 1.494 and the S.F.-value for Model 2B is equal to 1.473. In the curve chart also the number of steps it has taken each model to reach the reduction factor which is imposed on the internal angle of friction and cohesion until the quay structure becomes unstable.



Figure 7.4 5: Safety factors for Calibration model and Model 2B with surcharge

#### 7.5 Conclusion

The stability of the deep sea terminal quay structure is investigated with the finite element method, Plaxis 3D. The effect of the combined wall underneath the relieving platform structure is analysed. The M.V.-piles and concrete bearing piles are considered as well. A calibration model with the actual dimensions of the deep sea quay structure at the Amazonehaven are set up to visualize the deformation behaviour in the subsoil and the maximum field bending moments in the combined wall. Finally various design models are considered to evaluate the influence of the self-weight on the combined wall. Apart from the calibration model, model 2B is analysed further with the safety analysis check. The safety factor value determined for the calibration model is 1.621. This S.F.-value is derived from the representative soil parameters that are reduced to a value of failure. This is derived for model 2B as well which is closer to the reality of the situation the combined wall was in. However, the safety margin for both models are 1.263 and 0.864. This indicates that for the calibration model failure will occur for the steel combined wall which reaches its maximum yield stress before the passive soil resistance fails. That is also the case for model 2B, the steel combined wall will fail before the passive soil resistance.

The results shows how the quay structure behaved during its service lifetime. The most important question is, how safe would the structure be with the designed surcharge during its service lifetime? With this in mind, the calibration model and model 2B are created with the designed surcharge. The results illustrated both models will fail due to steel yielding and then failure of the passive soil resistance.

## 8

### DISCUSSION MEASUREMENTS DEFORMED COMBINED WALL AND CALCULATIONS

This chapter presents an evaluation of the measurements on the open tubular piles of the combined wall and the analytical research methods used to analyse the stability of the quay structure. Chapters five and six describe the outcome by using an analytical method based on the Blum theory and the beam on elastic foundation (D-sheet Piling) and chapter seven is based on a finite element method (Plaxis 3D). These methods are based on various assumptions and limitations. The analytical method considers the soil behaviour to be linearly elastic. However, the quay structure is modelled as a finite model where the soil will behave plastically due to the high designed surcharge. The analytical method and beam on elastic foundation method do not entirely simulate the constructional behaviour of a quay structure. The finite element method, which is based on an abundance parameters, does a more accurate job at modelling the deep-sea quay structure.

First of all the results of the research methods are discussed. Secondly the evaluations of the measurements of the deformed combined wall are explained. In addition, a link between the measurements and the quay models will also be concluded.

#### 8.1 Results D-sheet and Plaxis 3D

The results of the analytical method and D-sheet piling are based on the minimum penetration depth of the combined wall for each load case where stability is obtained as well as the bending moments which are present in the combined wall for each load case. However, only the results for D-sheet Piling are considered in this chapter. For the finite element method more data are available to illustrate the stability effect of the quay structure. The results are based on the structural forces and deflections of the combined wall, M.V.-piles and the concrete bearing piles and also on the deformations and stresses of the subsoil.

#### Maximum bending moments in the combined wall

The maximum bending moments in the combined wall for each load case calculated with D-sheet piling software are compared to the maximum bending moments in the quay models calculated with Plaxis 3D. Table 8.1 1 and Table 8.1 2 illustrates the bending moments for the load cases with or without surcharge and a uniform distributed surcharge or trapezium surcharge.

In Table 8.1 3 the bending moment for the quay models calculated with Plaxis 3D are shown.

A comparison can be made between the results in Table 8.1 1 and Table 8.1 3 for the load cases A2 and B2 from D-sheet and the calibration model from Plaxis 3D. The reason for this is that these models have a water level difference of 2 meters. For load case A1 and B1 the water level difference is one meter and the maximum moment for these cases are lower than the calibration model in Plaxis.

In general, the moments calculated with D-sheet piling and Plaxis 3D do not differ too much. The bending moments in the combined wall calculated with D-sheet piling are between 7 to 31% higher than the ones calculated with Plaxis 3D (Figure 8.1 1 and Figure 8.1 2). The obliqueness of the combined wall lowers the field bending moments in the combined wall. This is the case, for the quay model in Plaxis 3D.

	-			
Results D-sheet Piling	A1	A2	B1	B2
No surcharge				
Max. bending moments (M)				
[kNm/m]	3189.2	4107.9	3278.9	4642.5
Trapezium surcharge (450 kN/m <sup>2</sup> /m)				
Max. bending moments (M)				
[kNm/m]	3577.3	4179.3	3416.7	5124.8

Table 8.1 1: Max. bending moment each load case with trapezium surcharge (D-sheet piling)

Table 8.1 2: Max. bending moment each load case with uniform surcharge (D-sheet piling)

Results D-sheet Piling	A1	A2	B1	B2
No surcharge				
Max. bending moments (M)				
[kNm/m]	3409.4	3524.7	3416.7	4852.5
Uniform surcharge (450 kN/m <sup>2</sup> /m)				
Max. bending moments (M)				
[kNm/m]	3583.2	4583.2	3700.1	5136.1

#### Table 8.1 3: Max. bending moment for different quay models (Plaxis 3D)

Results/model Plaxis 3D	Calibration model	Calibration model surcharge	Model 2B	Model 2B surcharge
Max. bending moment				
[kNm/m]	3686	3393	5203	6543



Figure 8.1 1: Comparison max. bending moment between load case A2 (D-sheet) and calibration model (Plaxis 3D)



Figure 8.1 2: Comparison max. bending moment between load case B2 (D-sheet) and calibration model (Plaxis 3D)

The problem of calculating with D-sheet piling software is that the relieving platform structure, the M.V. piles and bearing piles must be calculated separately in a framework in SCIA engineering. The resultant vertical and horizontal forces obtained from the framework serve as input for the D-sheet calculation, where the relieving platform structure is modelled as a spring. However, in Plaxis the relieving platform structure can be integrated in the model.

In the software, D-sheet Piling, it is not possible to model the obliqueness of the combined wall which has favourable effects on the bending moment in the combined wall. Furthermore, the vertical arching effects of the soil working on the active side that will reduce the bending moment are not considered. These are the major disadvantages of this calculation method which resulted in higher bending moment than in Plaxis.



Figure 8.1 3: Comparison max. moment between calibration model and model 2B

Now, the quay models modelled in Plaxis are compared. Figure 8.1 3 illustrates a comparison between the bending moments of both quay models in Plaxis. As it can be observed from the graph the maximum bending moments in Model 2B are higher than in the calibration model. The reason for this is that Model 2B has an opening at the centre of the combined wall which is at N.A.P. -27.5m. Because of this opening the distribution of the moment will be higher at the sides than the centre of the combined wall where the opening is situated.

#### 8.2 Evaluation measurements combined wall

As mentioned in chapter 2 measurements of the deformed combined wall were conducted to obtain a better insight into the deformation of the combined wall, specifically the open tubular piles. However, first the recommended installation method is evaluated. Second, the cause of the damages to the combined wall can be deducted.

#### Installation methods of the combined wall

Before determining the installation methods which should be used on the foundation elements certain geotechnical aspects are investigated beforehand. As previously mentioned, geotechnical investigations (drilling and sounding) are performed to obtain soil mechanical parameters for the design of the quay structure. From the results the bearing capacity of the foundation piles can be calculated. This is important to take into account that after excavation of the building pit and dredging of the harbour basin the cone resistance of the soil will be reduced. In addition, the results of the investigation are used to establish limiting values for point resistance and skin friction for the foundation elements. Subsequently, predictions for drivability for the combined wall, concrete bearing piles and M.V.-piles can be recommended.

In this case, the focus is on the installation methods of the combined wall which consists of open tubular piles and intermediate sheet piles. The following installation methods were adopted for practical reasons, time and effort and to obtain adequate bearing capacity.

The open steel tubular piles were installed by vibration with RBH 160 vibrator and 1600 kN impact force down to the Pleistocene sand layer. The open steel tubular piles were installed under an inclination of 1:5 with the help of a guiding frame with which the position of the piles can be secured. Afterwards, the intermediate sheet piles are installed. To prevent the occurrence of interlock opening during installation the following order of installation was prescribed [42]:

- 1. Vibration;
- 2. Vibration with jetting;
- 3. And driving.

At first instance, for the installation of the open tubular piles a D62 diesel hammer was used. During installation plugging occurred which lead to an increase in blow counts to 400 blows/25cm and long driving times of 8 hours. The hammer was replaced by a D100 diesel hammer with blow counts between 70 and 120 blows/25cm and driving times of about 2 hours. In Figure 8.2 1 the recommended type of hammer according to the number of blows/25cm to penetrated depth is shown. The intermediate sheet piles were installed with a D46 diesel hammer. Nevertheless, interlock openings did occur despite the strict installation methods prescribed.



Figure 8.2 1: Results driveability of open steel tubular piles [42]

#### **Evaluation measurements combined wall**

Measurements in the field were performed to determine the extent of the deformations of the open tubular piles. From the table in Appendix A it is shown that about 22% of the open tubular piles were measured regarding:

- The length of the deformation;
- The wall thickness (this is measured per section of the tube);
- The total length of the tube is measured;
- Determine at which side the deformation is located (land or sea side).

From the documented total length of the tubular piles a minimum length of about 28.3m and a maximum length of about 30.2 m were found. The design length of the open tubular piles was 30.5 meters. This leads to the conclusion that the penetration depths that were actually reached in situ differ, on average, about 2 meters from the designed depth.

Besides the length of the tubular piles a description of the deformation is documented as well. In the table of measurements in Appendix A it can be noted that the majority of the measured tubular piles had no deformation to the pile toe. The tubular piles that were deformed at the toe had the following type of deformations, namely:

- Banana shaped
- Folded closed
- Both sides slightly folded in
- Undamaged



Figure 8.2 2: Banana shaped deformed open tubular pile



Figure 8.2 3: Folded closed deformed open tubular piles



Figure 8.2 4: Both sides slightly folded in deformed open tubular pile



Figure 8.2 5: Undamaged deformed open tubular pile

In general, there are several reasons for the deformation at the pile toe. The most important aspect is the minimum thickness of the wall which is recommended when the tubular pile is subjected to the hard driving force of the hammer. Also the soil type and hammer energy should be considered. According to the American Petroleum Institute (API, 2007) the minimum pile wall thickness can be calculated with the following equation [43],[44]:

t = 6.35 + d/100		Eq. 8.2 1
where: t = the wall thickness d= pile diameter	[mm] [mm]	

According to the equation above the minimum thickness of the wall of the combined wall is about 20mm. From the measurements performed at the storage yard at Maasvlakte I the wall thickness of the open tubular piles are

smaller than the minimum wall thickness. This may be a reason for the deformation of the pile toe during pile driving.

#### 8.3 Conclusion

From the evaluation of the deformed open tubular piles can be concluded that the damage to the pile toe occurred during driving of the pile this include the use of inappropriate hammer, insufficient cushion, tight pile cap, difficult pile conditions, constructions in the ground, concentrated soil resistance and incorrect wall thickness. But not only the deformation and thickness was evaluated the pile length is also taken into consideration. With this in mind, the quay models in Plaxis were modelled. In reality, the minimum length of the tubular pile 28.3 m but for Model 2B a pile length of 21.5 m was chosen. The reason for choosing this length is to analyse the influence of the horizontal stability of the quay structure when the penetration depth is shorter than expected. Also, the bearing capacity might be much higher than the design value from the designed quay structure.

# 9

### **CONCLUSIONS AND RECOMMENDATIONS**

In this chapter the concluding remarks of the master thesis research are presented. For this thesis the deep sea terminal E.M.O - quay at the Amazonehaven in the port of Rotterdam has been researched and used as a model to investigate its structural stability. The reason for choosing this particular quay stems from the fact that during demolition the construction showed severe and unexpected deformations of the combined wall.

The results obtained from the investigation will provide answers to the research questions and objective in Chapter 1. Subsequently, recommendations that should be taken into account during design and construction of the quay structure and recommendations for further research are presented.

#### 9.1 Conclusions

The objective of the master thesis research is to investigate the structural stability of the deep sea quay structure throughout its functional service lifetime, taking into account the aforementioned deformations of the combined wall. According to the literature research the quay structure at the Amazonehaven has never been fully loaded or used as designed. Therefore, field measurements on the tubular piles of the combined wall were performed and afterwards calculations by means of hand, analytical methods and finite element methods were used. The simulation of the quay structure consisted of modelling the quay structure without surface loads as well as with the design surface loads. Based on the results of all these calculations regarding the structural stability of the quay structure, the following main research questions can be answered.

#### 9.1.1 Research questions

The main research questions are:

- What is the structural stability of the deep-sea quay structure throughout its functional service lifetime, specifically the combined wall?
- What is the preliminary evaluation of the deformed combined wall?

To be able to answer these two questions, the following additional questions are answered:

• How (un)safe was the quay structure if it had been exposed to the full surcharge (design load)?

For the calculation of the quay structure various methods were used to provide an insight into the safety of the quay structure.

The hand calculation, Blum's theory, was first used as a first estimation of the internal forces and the penetration depth of the combined wall. This method provides overestimated values so due to its conservative nature and simplicity it can always be used.

The beam on elastic foundation method, D-sheet piling, gives the opportunity to analyse the combined wall in a

sequence of construction phases, but this calculation method does have some disadvantages as well, which are:

- The superstructure, the relieving platform, has to be analysed separately.
- The arching effects of the soil on the active side, resulting in the reduction of the bending moments and higher anchor forces, is not calculated.
- A cross-section of the quay structure per meter is calculated, so 3-dimensional effects are ignored.

The findings of the two aforementioned methods show the structural behaviour of a cross section of the quay. The entire quay structure, a three-dimensional body, is not considered in both methods. Therefore, finite element method, Plaxis 3D, is considered to be a better tool for analysing the stability of the quay structure three-dimensionally, especially as the three-dimensional effects are expected to have played an important role in the stability of the quay in question.

The calibration model (the designed quay structure) with and without surface load resulted in being stable. But for model 2B (which is how the quay structure is in reality) without surface load the soil layer is 1.56 times stronger than the steel of the combined wall. Failure of the combined wall should occur long before the passive soil resistance fails. From these results can be concluded that the quay structure if fully loaded throughout its service lifetime it would have fail due to yielding of the steel combined wall.

#### How much safety exists against the loss of stability?

A safety analyses is performed on the calibration model and model 2B with Plaxis 3D. During the safety analysis calculations the internal friction and cohesion of the soil layers are reduced till failure occurs. In this case, different failure mechanisms can occur with the same parameters. As previously mentioned, these models are calculated with and without surcharge. The safety factor values for both models are:

	Calibration model S.Fvalue	Model 2B S.Fvalue
Without surcharge	1.62	1.56
With surcharge	1.49	1.47







The figure above shows a comparison between the safety factor values of both models. The difference between the model with and without surcharge is significant. Model 2B comes closer to the conditions of the quay structure throughout its functional service lifetime. Evidently, the difference between safety factor values for model 2B is small which demonstrates that even with the full designed loads on the quay structure it will still be stable for a certain period. It would have not immediately collapsed.

#### Does the quay structure meet the safety standards with the current calculation rules?

The deep sea quay structure at the Amazonehaven was designed according to the E.A.U. 1985. The E.A.U. used load cases, "Lastfalls", to determine the design of a quay structure. Lastfall 1 and Lastfall 3 were used as load cases with different outer and groundwater levels. The representative soil parameters were used during calculation of the dimensions of the quay structure. The safety coefficient of steel stress was reduced to 1.5 for Lastfall 1 and 1.2 for Lastfall 3.

For the quay structure modelled in Plaxis the results of the steel stresses are lower than the yield stress in both calibration models, with and without surface loads. The steel stresses in Model 2B without surface load remain below yield stress as the maximum bending moment are reached yet when surface loads come into play, the steel stresses in model 2B become intolerable high.

Even though the steel stress becomes very high in model 2B the safety margin coefficient will be around 1.5. Therefore it can be concluded that the quay structure will meet the safety requirements with the current calculation rules.

## The combined wall did not reach its final penetration depth as indicated in the original design, what is the effect on the retaining capacity of the quay structure?

According to the measurements of the deformed open tubular piles the minimum penetrated depth is about N.A.P. -34.5m. This is not the case for the entire length of the quay structure. Also the harbour basin in front of the quay structure was not dredged at its full construction depth. However, the quay structure did not show excessive deformations throughout its service lifetime. Therefore, it can be concluded that due to the higher passive soil resistance the quay structure was capable to retain soil and water without collapsing.

#### Which is the dominant failure mechanism?

In the computer software, Plaxis 3D, all failure mechanisms linked with the structural stability of the quay structure are investigated. However, regarding the researched quay structure the stipulated failure mechanisms are investigated by performing a safety analysis. From the results in chapter 7 it can be concluded that the dominant failure mechanism for both models with and without surface loads are geotechnical and structural. For geotechnical failure the effective horizontal soil pressure on the active side will become larger than the effective horizontal soil pressure on the passive side. The soil will revolve around the anchors (M.V.-piles) which will be holding the quay structure into place. However, structural failure can occur when the steel of the combined wall reaches its maximum yield stress. Consequently, the steel of the combined wall can yield to a certain point of permanent deformation which can lead the quay structure to collapse due to fracturing of the steel.

#### Which aspects of construction contributed to the deformation of the combined wall?

During the literature research it became clear that various aspects contributed to the deformation of the combined wall. The material steel was not considered during the literature research. Besides literature research, measurements of the deformed tubular piles were conducted regarding the length of deformation of the combined wall. From the information gathered on the construction methods used for the execution of the quay structure certain aspects are evident:

- Throughout driving of the open tubular piles and intermediate sheet piles a rather (maybe too) light hammer was used. The piles were rammed into the ground with an abundance of energy which contributed to the deformation of the embedded end of the piles.
- The piles were driven through many soil layers with high penetration resistance. During this process the surrounding soil is pushed aside which creates higher friction between soil and piles. This caused the piles to heat up, which was confirmed by the fact that sand was scorched to the wall of the actual tubes. Furthermore the heating up of the tubular piles probably caused softening, deformations and even closing up of the embedded end.

#### 9.1.2 Main research questions

The answers to the two main research questions are:

- 1. The actual quay structure, even though not fully loaded, did not become unstable. The finite element method, Plaxis 3D, where the model can be 3-dimensionally analysed was used to model the behaviour of the quay structure throughout its service lifetime. The calibration model (which is the actual designed model) and various models were created and calculated in Plaxis. From these models only model 2B will be compared to the calibration model. Model 2B (which comes closer to reality) with and without the designed surcharge will become unstable when exposed to unfavourable conditions. The deformation behaviour of the combined wall will be too large due to higher loads than anticipated. However, the safety factor value of model 2B comes close to the safety coefficient of 1.5 which was used on the quay structure during the design process. Therefore, it can be concluded that the quay structure was strong enough to retain the loads it was exposed to during its service lifetime. Also the superstructure was strong enough and capable to retain all the excess loads without collapsing.
- 2. The preliminary evaluation of the deformed combined wall, specifically the open tubular piles, can be summarized:
  - The deformation in the combined wall is due to the use of inappropriate hammer, D62 diesel hammer. During installation plugging occurred which lead to an increase in blow counts to 400 blows/25cm. The hammer was then replaced with a heavier hammer,D100 diesel hammer, which did drive the pile to its penetration depth.
  - The wall thickness of the open tubular piles are smaller than the minimum calculated wall thickness needed. The minimum wall thickness is about 20 mm which is sufficient to transfer the energy of the hammer to the subsoil.

#### 9.2 Recommendations

- Based on the results and conclusions described in this thesis, it is recommended to investigate the material steel of the combined wall, specifically the open tubular piles. The influence of the interaction between deep and hard soil layers and open tubular piles during pile driving. These simulations can be carried out with the help from a finite element method software. This can give a better overall view of the actual quay structure throughout its service lifetime.
- The bearing capacity of the deformed open tubular piles can be researched as positioned at the Amazonehaven during its service lifetime. Also, the open tubular piles can be researched in different locations under unfavourable conditions on the bearing capabilities.
- The analytical method, D-sheet piling, is useful for preliminary designs. The results for the minimum penetration depth regarding the effective horizontal stresses in front of the combined wall cannot be compared to the results from the finite element method. The pile wall displacement and pile wall bending moments are not very accurate in D-sheet. Therefore, the use of finite element method to determine displacements and bending moments in the combined wall is preferred. In Plaxis models 3D effects of the soil –structure is taken into account which is not the case in D-sheet. Therefore, results from Plaxis cannot be compared to D-sheet and vice versa.
- The design standards that were used for the original design of the quay structure can be re calculated with the current design standards. The reliability of the current and past design standards can be researched.
- The quay structure models are carried out with specific designed surface loads. However it is recommended to calculate the quay structure in D-sheet and Plaxis 3D with various load combinations. Due to the advanced modelling capabilities of Plaxis and the more reliable results obtained, it is recommended to use the results of D-sheet for the concept design phase. For the final design it is recommended to use Plaxis for more accurate estimation of the forces, moments and stresses in the structural elements.

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