A conceptual design of a quay wall on a steel slag subsoil

MSc thesis report

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A conceptual design of a quay wall on a steel slag subsoil

By

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Vertrouwt op den HEERE tot in der eeuwigheid, want in den HEERE HEERE is een eeuwige rotssteen.

Jesaja 26:4

Preface

This thesis report is part of my master's program Hydraulic and Offshore Structures at the Technical University of Delft. The thesis project is performed in collaboration with Witteveen + Bos and TU Delft. The graduation project aims to develop a design for a quay on a steel slag subsoil.

The subject of this master's thesis design report is about constructing a quay wall on a steel slag subsoil. Quay wall installation in these soil conditions is seldom in worldwide port development projects.

I want to express my gratitude to Dr. Ing. M. Voorendt, Ir. M. Fousert, Dr. M. Korff, Dr. J.G. de Gijt as supervisors and members of my thesis committee. In addition, I want to thank Ir. B. Berkhout at Port of Amsterdam for the feedback and information.

Tom van Koeveringe
04-10-2024
'S Gravenhage

Summary

This report addresses the design of a quay wall on a steel slag subsoil. The reason for this study is the realisation of a wind turbine assembly port. Worldwide hydraulic structures are seldom made in these artificial soils. This steel slag material present in the project area complicates the construction of a quay. Is it possible to install foundation piles in this material? What environmental aspects need to be taken into consideration? Which quay wall type is most suitable for realising the port area?

The study starts by explaining the motivation behind the quay wall structure on this unconventional soil. The main reason for the need for wind assembly ports is to increase the wind turbine installation and maintenance capacity. The problem analysis explores the challenges associated with the steel slag materials, this leads to the problem statement and design objective. The problem can be summarised as follows: Despite the large experience in port developments and quay wall constructions, the ability to efficiently design a quay wall on a varying soil system like steel slag, is still considered complex. This leads to the goal of this thesis, which is to create a conceptual design of a quay wall on a steel slag subsoil at the location.

The design analysis aims to find the most efficient quay wall design, which is possible to construct and even take advantage of the presence of steel slag material to increase structural performance and stability. In the thesis approach, the steps taken to achieve the goals of the study are shown. The report proceeds with the development of a method in which the different characteristics of the steel slag material are examined.

The steel slag materials have some positive and negative effects compared to regular soils. The relatively high friction angle and high density can benefit the structure when applied at the right location.
One of the issues with using steel slag is the risk of environmental implications. When steel slags come in contact with air and water, heavy metals can leak out of the slag causing damage to the ecosystems and humans. The design solution aims to mitigate the environmental risks without exponentially increasing the costs.

A system analysis follows including an area, stakeholder and function analysis. This helped to illustrate the broader environment and requirements for the quay wall construction. The basis of design section outlines the starting points and boundary conditions whereafter the programme of requirements and evaluation criteria are defined. The report includes a functional and structural design.

After analysing potential alternatives for constructing a quay wall, the cofferdam variant was the most promising given the required bearing capacity, height and subsoil. The cofferdam design consists of two combi walls connected with tie rods at two levels. For stability, a grout anchor is connected to the backside of the combi wall. A low permeable environment was created because a clay layer is present between the two walls. The cofferdam dimensions were chosen so that most of the steel slag material would be enclosed between the combined walls. The residual volume of steel slag material is used as a fill material for the piles.

In the structural design, a detailed construction sequence and the design model were provided.
A PLAXIS 2D model based on Finite Element Method (FEM), was made for two cross-sections of the quay wall. Based on the outcome, the elements of the quay wall were verified and optimised. The installation method of the combi wall has a large impact on the cost. Results were analysed and risk-mitigation measures have been advised to provide a controlled construction. Various checks on stability, strength, stiffness and deformation were conducted to ensure this design meets the technical standards.

The validation of the design was then performed to check whether the design objective was adequately formulated and correctly translated into the requirements. As the client was Port of Amsterdam, the design was validated in correspondence with this company.

The report concludes with a discussion of design considerations and the implications of the design choices. Finally, the conclusions and recommendations section summarises the key outcomes of this report.

Based on the outcome of this report, it can be concluded that the construction of a safe and stable quay wall is possible with the right construction measures. It was recommended that further analysis of the environmental impact of the re-usage of steel slag material be conducted.

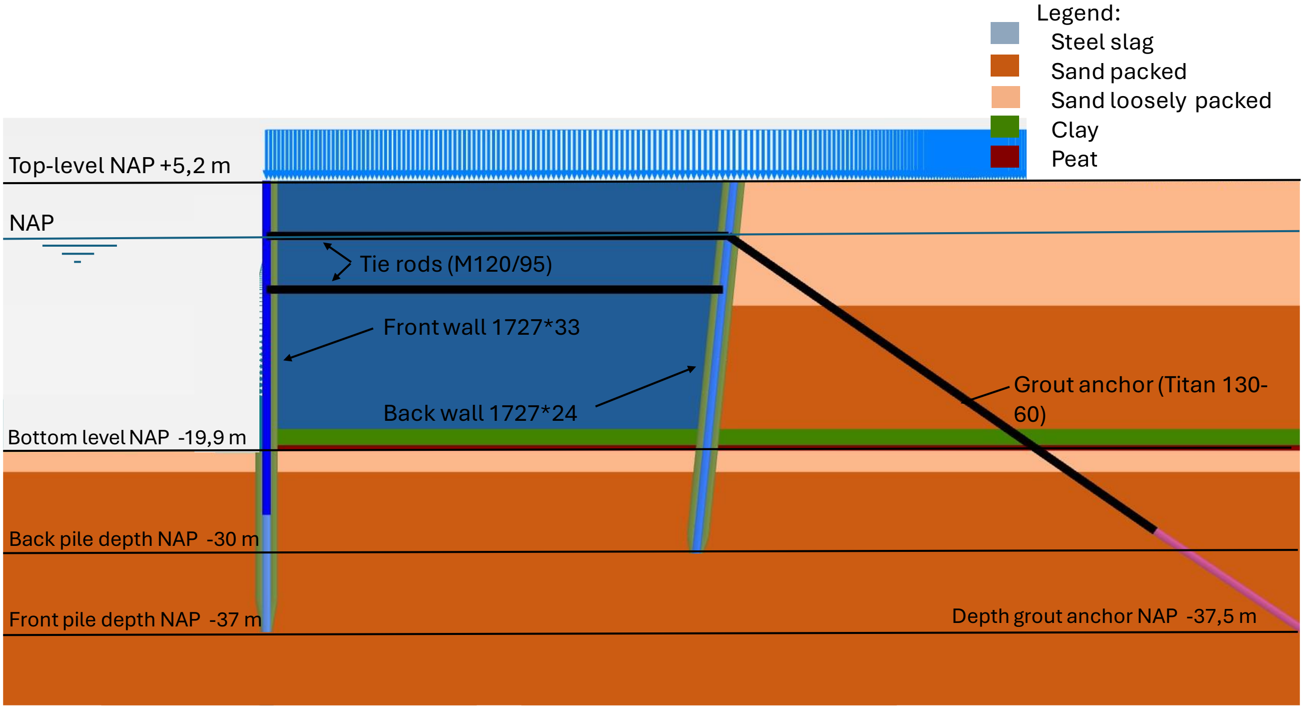

Figure 1 The dimensions of the required elements were calculated.

For further research, it was recommended to perform detailed calculations on the connection between the elements. In addition, a hydrological test could be performed to understand the flows of the rain and groundwater in this design. For the execution of the structure, it was recommended to perform an additional pile driving test with the driving shoes to prevent failure. The test results will show it this setup is suitable for the realisation of the quay wall. Additionally, it can provide extra certainty on the construction time, cost and knowledge.

Contents

1 Introduction

1.1 Motivation: a quay wall on a steel slag subsoil

The ambitions of the Dutch government concerning the energy transition include the installation of wind farms at several locations in the North Sea (Rijksoverheid, 2022). To realise those ambitions, the installation and maintenance capacity of offshore wind turbines will have to be enlarged and so will the vessels used for transportation and installation. To increase installation and maintenance capacity for offshore wind parks, new terminals are necessary which provide dedicated berths for the large jack-up vessels. A new port location was found to facilitate the transport of offshore wind turbines at the former depot for polluted dredged material ‘Averijhaven’. The area shown in Figure 2, close to IJmuiden, is located outside the IJmuiden locks and close to the to-be-installed wind farms. The realisation of the project will contribute to the energy transition of the electricity supply of the Netherlands by increasing the capacity for offshore wind turbine installation.

The new energy port in IJmuiden will be located at the site of the present dredging storage(indicated by the blue box), which must be transformed. The project is named ‘Energiehaven IJmond’ and is initiated by a collaborative partnership (consortium) between the province of Noord-Holland, the municipality of Velsen, Zeehaven IJmuiden and Port of Amsterdam. The Ministry of Infrastructure and Water Management, Rijkswaterstaat, Tata Steel and the Ministry of Economic Affairs are involved in the project (Provincie Noord-Holland, 2022). The port location was selected because it has an open waterway to the North Sea and is relatively close to the proposed wind farm location.

Figure 2 The location of the dredging storage facility (Port of Amsterdam, 2023).

This area was used by Rijkswaterstaat, the executive agency of the Ministry of Infrastructure and Water Management as a dredging sludge storage area, as shown in Figure 3.

The energy port will contribute to the acceleration towards a renewable electricity system in the Netherlands, since, as soon as the operations in the port start, the capacity of the wind turbine installation will increase later. Wind energy will become a significant part of the green energy supply of Dutch society in the upcoming decades (Rijksoverheid, 2022). So, by constructing the quay wall as part of the energy port a small step towards a more sustainable energy source will be made.

Figure 3 Proposed location of the quay wall. A (red colour) indicates the deep sea quay and B (green colour) indicates the coaster quay (Port of Amsterdam, 2023).

This report concerns a study that supports the design of the quay wall as part of the construction of the Energiehaven IJmond.

1.2 Problem analysis

1.2.1 Exploration of the problem

As part of the Dutch government's goal to realise its climate change mitigation goal, the ambitions of the installed wind energy capacity are 21 gigawatts by 2030 and 70 gigawatts by 2050. To realise those ambitions, ports that can facilitate the transportation of wind turbine parts(e.g. blades, foundations) from land to sea are required. In the past decade, the dimensions of the wind turbines and the installation equipment (e.g. vessels) have increased. Therefore a newly designed port could potentially increase the efficiency and capacity of wind turbine installation in the Netherlands. The goal is to install 15 - 20 gigawatts of wind energy using the energy port in IJmuiden (Port of Amsterdam, 2024). According to the proposed planning by the consortium, by 2027 the energy port will be a facility for transporting wind turbines and their foundations (Provincie Noord-Holland, 2022).

Currently, a circular dam, constructed with soil consisting of steel slags is located at the site as shown in Figure 3. The steel slag is a rest product of steel production. The area, enclosed by the steel slag dams, was used as a dredging sludge storage area. Most of the sludge was removed in preparation for the realisation of the Energiehaven IJmond. A deep-sea quay wall must be constructed as part of the port development to facilitate safe ship handling. The proposed location of the quay wall, as shown in Figure 3, has a soil layer profile consisting of steel slags. The total length of the quay wall is 720 m (Port of Amsterdam, 2023). The cross-section of the dike is shown in Figure 4. This dam will have to be changed into a quay wall. The quay wall has to ensure the safe and efficient handling of vessels. The jack-up vessels use spud cans to create an installation platform, these spud cans are placed on the sea datum. The jack-up vessels influence the stability of the quay wall because the spud cans can penetrate the soil. Both the terminal area and the quay wall structure should be able to absorb large loads associated with the transfer of wind turbine elements.

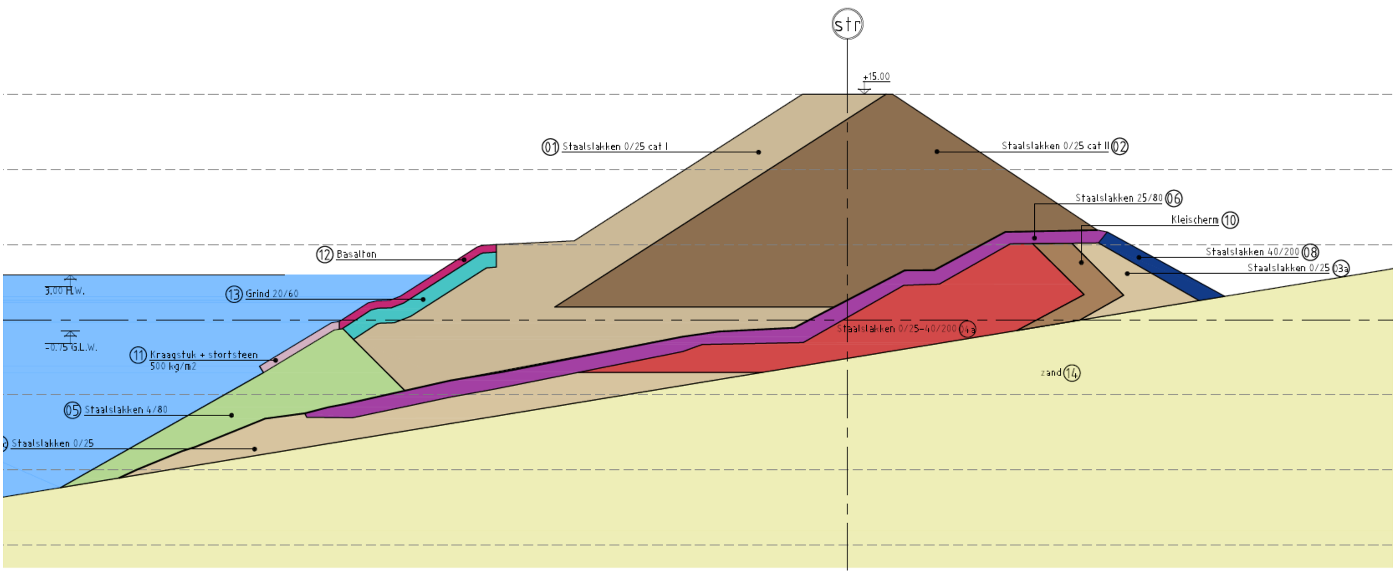

Figure 4 Example of a dike cross-section profile showing the steel stag material layers (Port of Amsterdam, 2023).

1.2.2 Inventory of the current state of technology

The inventory of the research concerning the design of quay walls in a steel slag layer consists of the following. First, the current state of technology in quay wall design and optimisation is shown, followed by an overview of the possibilities of using steel slags and the (health and stability) risks that can occur. The last part shows the implications of using jack-up vessels in combination with a quay wall.

1.2.2.1 Quay wall design and optimisation

Quay wall design and reliability models

The uncertainties of the loads and resistance will have to be known to obtain the most efficient design for a specific quay wall. Although many hydraulic and offshore structures are being modelled, quay wall assessments based on reliability models are scarce. One of the conclusions made by Roubos (2020), based on the results of an assessment is, that the reliability indices seem quite sensitive to changes in the variation of the coefficients such as the soil friction coefficient. It was recommended to install new quay walls with sensors measuring the deformations and anchor forces to obtain more insight and reduce the uncertainty in modelling, doing so new light will be shed on the actual capacity of quay walls (Roubos, 2020).

Quay wall dimensions and optimisation
There are several types of quay walls: Sheet piled quay walls, combi walls, steel cellular or caisson quay walls, gravity blocks quay walls and reinforced counterfort quay walls (NSCC, 2024). The research by NSCC (2024) was performed to optimise the types of quay walls by investigating which input parameter influences the dimensions of quay walls. It was stated by Roubos (2020), that for all types of quay walls the internal friction angle of the soil, wall friction coefficient and terrain loads govern the dimensions of the structures. It was recommended to investigate the dependency of various soil levels on cost. Secondly, the interaction between counterforce structures and deformations in quay walls should be investigated. Lastly, it was advised to use sand with a relatively high internal friction angle to improve the stability and horizontal force transformation. It was recommended to pay attention to the influence of soil parameters on the stability of quay walls (Van Gaalen, 2004), (Roubos, 2020).

A study concluded, that the optimisation of quay walls by modelling the uncertainties, could potentially result in savings of costs by 50 per cent (El-Sayed, 2021). In this study, multiple types of quay walls were analyzed based on a three-dimensional finite element method on a soft clay layer. It was recommended to further research newly constructed quay walls by placing sensor instrumentation to understand the deformations of the quay walls better. With this information, maintenance can be carried out more effectively, with lower costs.

At the Amazonehaven, in the Port of Rotterdam, a deep-sea quay wall was analyzed after the construction. Since the combined wall did not reach its designed penetration depth, the capacity of the quay wall was analyzed. It was concluded that the quay was strong enough to retain the excess loads without collapsing. It was stated that, for further research, the interaction between soil layers and the foundation piles should be investigated (Mourillon, De Gijt, Bakker, Brassinga, & Broos, 2017).

CO_2 emissions of quay walls
The sustainability and CO_2 footprint of quay wall structures is mostly dominated by the concrete and steel as used in the structure. Research has shown that, for a quay wall located in the port of Rotterdam, 46 % of the emissions during construction are related to steel and concrete, 33 % and 13 % respectively. In addition, 36 % of the emissions of greenhouse gases are caused by fossil fuel consumption of construction equipment. This data is based on a 100-meter-long of quay wall and a total CO_2-eq of 1.9 kt. Yet, it was recommended to investigate further alternative designs, such as smaller dimensions of steel piles and prefab concrete quay walls with geo-polymer-based-cements (Taneja, van Rhede-van der Kloot, & Koningsveld, 2021).

1.2.2.2 Steel slags: opportunity or challenge?

Steel slags used in foundations
Steel slag is a waste product or residue of the steel refining industry. Steel slags, as shown in Figure 5, located in the project scope, derived from the steel industry can sometimes be utilised as ground materials by solidification (Kamon, 1993). This could have a positive impact on the bearing capacity of the foundation of the quay wall, however, it strongly depends on the type of steel slag and its soil parameters.


Figure 5 Steel slag from Tata steel (NH nieuws, 2021).

Much research was carried out on the utilisation of steel slag powder as cement or as concrete aggregate. The downside of using steel slag is its significant expansion in reaction with water. However, because of its strength and hardness, it can be used as an aggregate of cement concrete. Investigations were carried out to reuse steel slag, fly ash, and phosphogypsum combined. The solidified steel slag-fly ash-phosphogypsum material properties were better compared to commonly used road-based materials. Therefore the material could be used to increase the reuse of those waste materials on a larger scale (Weiguo Shen, 2009).

Volume instability and heavy metal leaching (mostly related to stainless steel slags) are the two main problems in using steel slags in hydraulic structures (Huang Yi, 2012). Steel slag aggregate mixtures, however, provide a stronger bearing capacity after compaction when using other aggregates. Heavy rainfall does not influence the bearing capacity. And by using carbonic solidification the bearing capacity can be increased. The properties of steel slags are comparable with natural stones like basalt (Motz, 2001).

The chemical components in steel slags depend on the production processes. Worldwide the production of steel slags still increases. In Europe, only 3 % of the produced steel slags are used in civil works, because the use of steel slags comes with environmental implications, as shown in Figure 6. Chemical reactions can occur when steel slags are in contact with water. Toxic elements like heavy metals, alkalis and metalloids can leak into the environment. This results in a change in pH values in the surroundings which destroys aquatic life and affects human bodies if introduced in the food chain. To ensure maximum utilisation of steel slags, innovative solutions are needed to put the material back in the circular economy (Chandel, 2023).

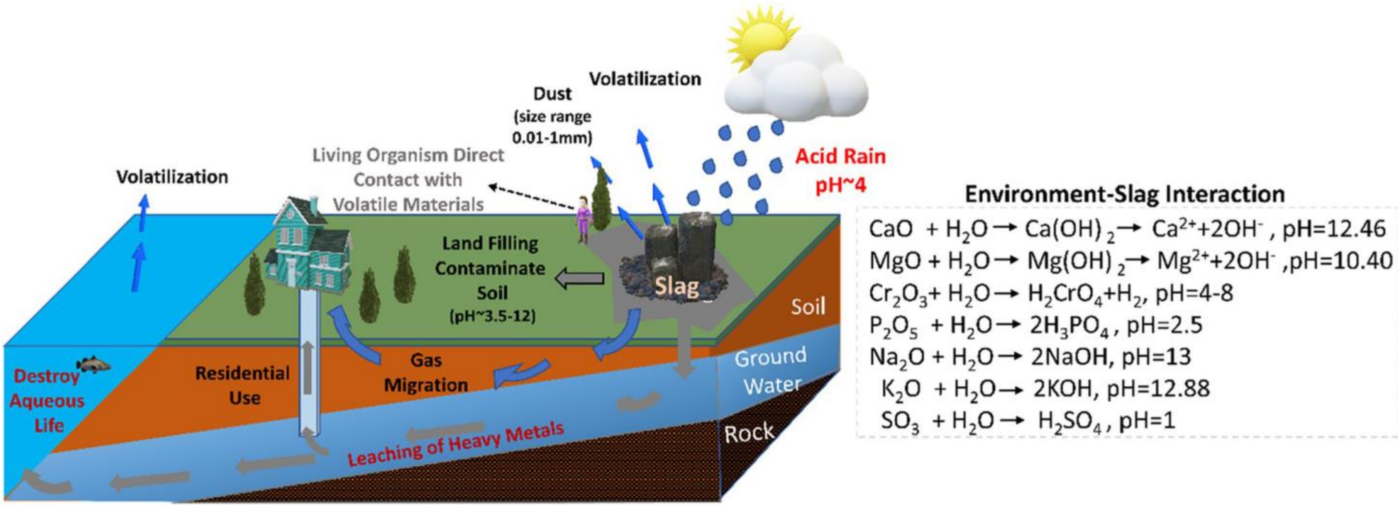

Figure 6 The steel slag's interaction with the environment (Chandel, 2023).

To construct quay walls, foundation piles are driven into the soil to obtain the required strength and stability. At the project location, a test setup was created to check the possibility of using foundation piles in combination with a steel slag soil layer. In the test case, one of the three piles failed, as shown in Figure 7. This underlines the complexity of the system and the importance of performing further research to successfully construct a quay wall at the Energiehaven.

Figure 7 The foundation pile after the test shows the deformation (Witteveen en Bos, 2024).

1.2.2.3 Jack-up vessels and quay walls

Impact of spud can vessels on quay walls
To be able to safely load wind turbine parts on transportation and installation vessels, it is necessary to use jack-up legs in the port. The installation and removal of spud cans impact the stability of the quay wall. As shown Figure 8, it results in extra forces and deformations on the quay wall structure. The spud cans penetrate the passive wedge of the quay wall. When the spud can is removed, the soil is disturbed and loosened. As a result, the passive soil wedges provide less resistance, resulting in higher loads and stresses in both the wall and anchors and subsequently deformations. At the specific location, the impacts of the spud can installation and removal were estimated by an increase of the moment and anchor force of a maximum of 5 per cent. Yet. it was advised to carefully investigate the location to avoid rapid penetration of the soil layers (Kellezi & Kudsk, 2011).

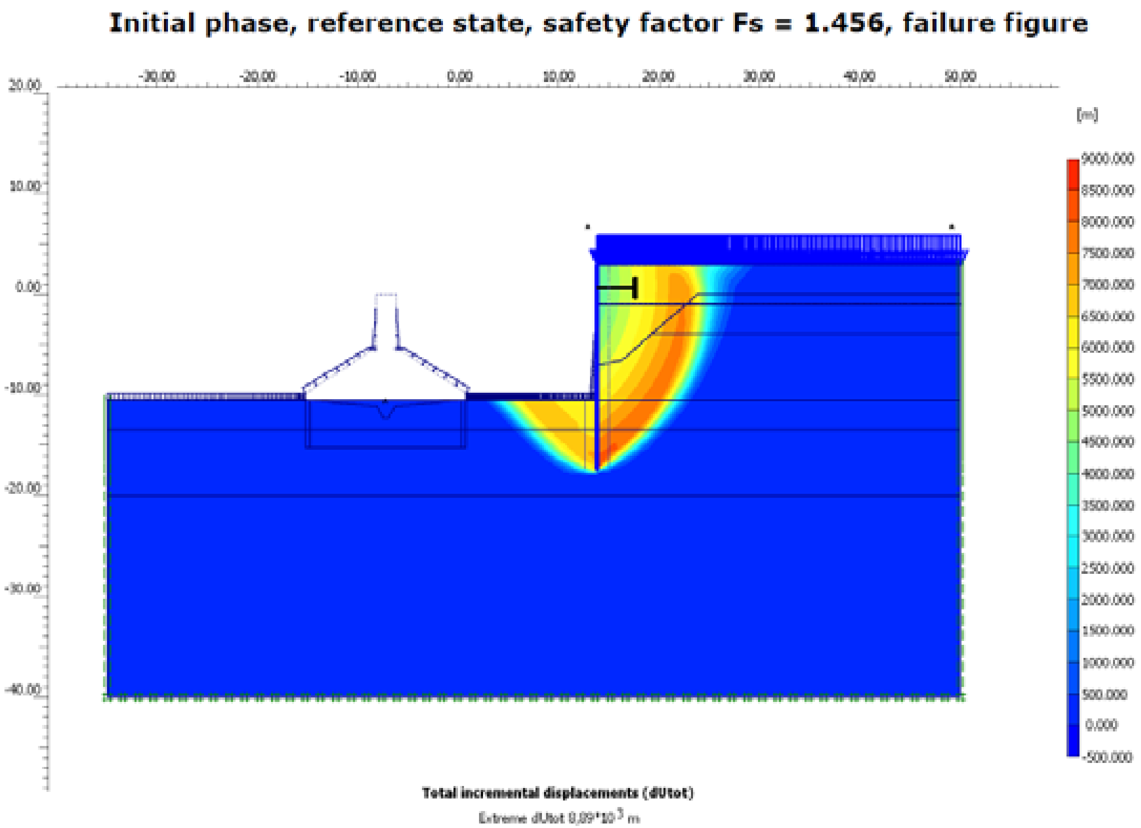

Figure 8 Skirted spud can quay wall interaction, with a safety factor (Kellezi & Kudsk, 2011).

More research was performed to gain knowledge of the spud can penetration, at the port of IJmuiden, and at two quay walls named the Volendamkade and the Monnikendamkade. The vessel named Pacific Orca was selected as a reference since this vessel has a relatively high spud can force.

Table 1 Specifications of the Pacific Orca

Vessel	Diameter spud can [m]	Preload [ton/spud can]	Stress at spud can bottom [kN/m^2]
Pacific Orca	11	7.100	733

As shown in Figure 9, the bearing capacity is obtained for the different soil layers. It can be stated that the spud can penetrate the first two soil layers.

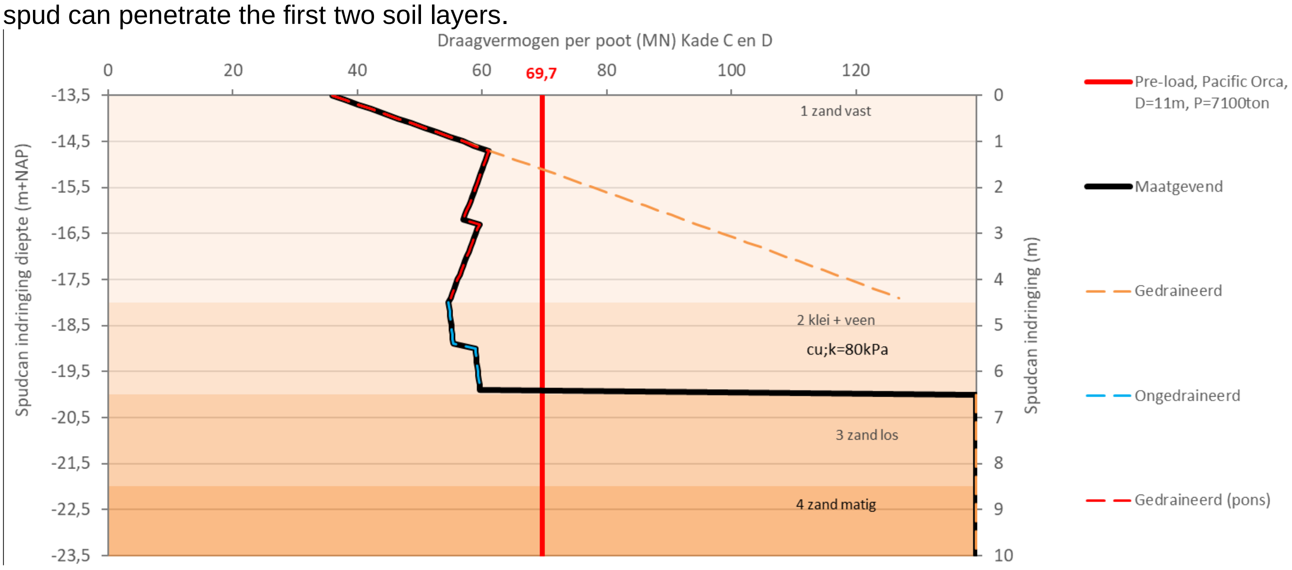

Figure 9 The penetration depth with bearing capacity [MN} for each spud can (Witteveen + Bos, 2021).

Based on these results, an estimation was made about the penetration depth at the location of the new quay wall. In Figure 10 the calculated depth is shown, and compared with the real depth. It can be stated that the measured depth is larger than the calculated depth. In addition, a correlation exists between the depth of the clay layer and the penetration depth. This could mean that the clay layers are squeezed by the load of the spud cans.

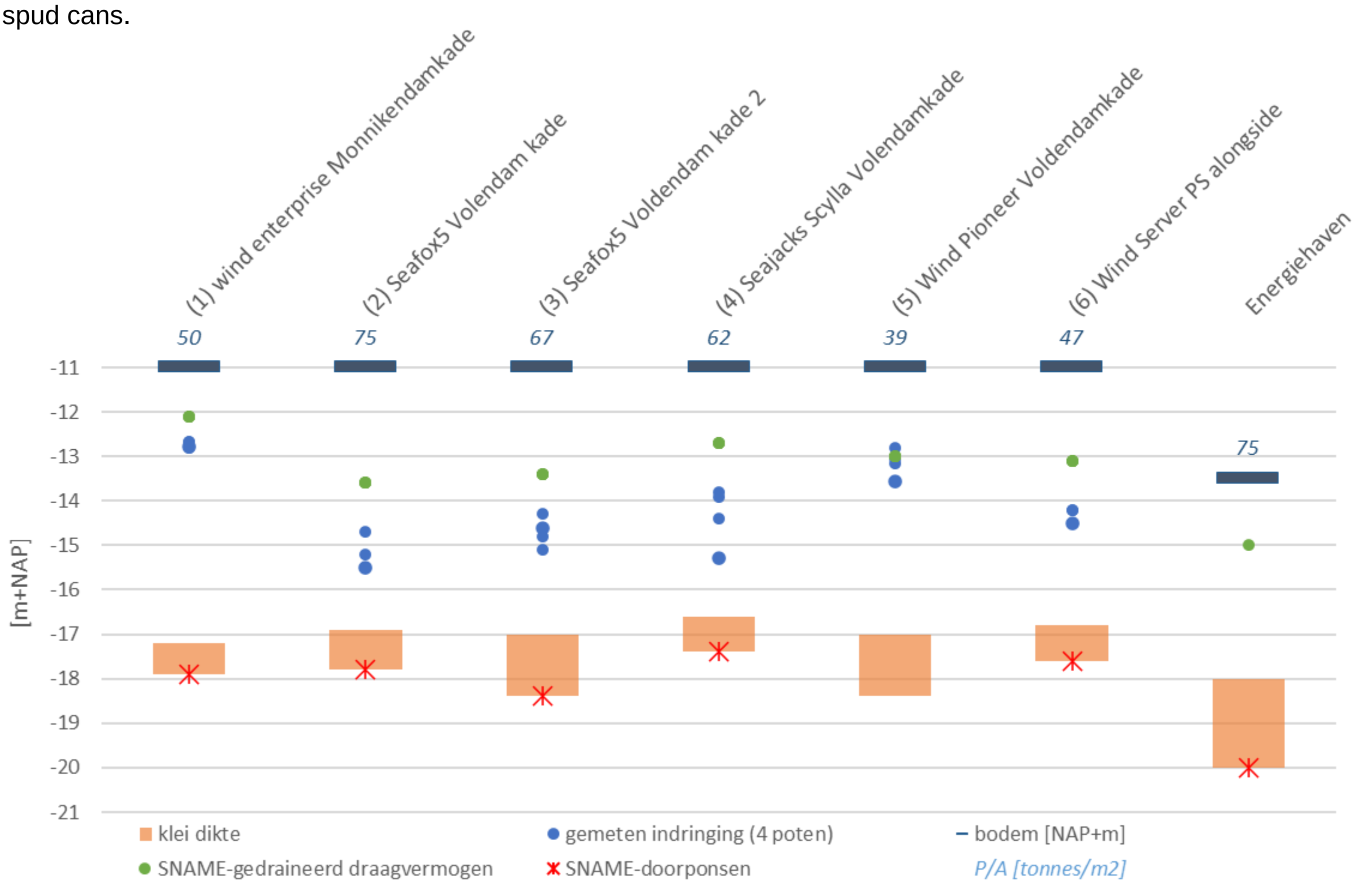

Figure 10 Comparison between the measurements and calculated values (Witteveen + Bos, 2021).

For the calculations at the Energiehaven, it was advised to look at the thickness of the clay layer carefully, however, no total penetration has to be expected (Witteveen + Bos, 2021).

To conclude, the three parts of the overview of the current state of technology show the complexity of the quay wall design. Challenges that might occur during the design and execution of the project Energiehaven IJmond.

1.3 Problem statement

Despite the large experience in port developments and quay wall constructions, the ability to efficiently design a quay wall on a varying soil system like steel slag, is still considered complex. Risk-reducing measures can be taken in both the design and the construction phase. Uncertainties and not yet identified risks could result in over-dimensioning, too high construction costs, and additional CO_2 emissions.

1.4 Design objective and thesis approach

1.4.1 Objective

The objective of this thesis is to create a conceptual design of a quay wall on a steel slag subsoil at the location, as shown in Figure 11.

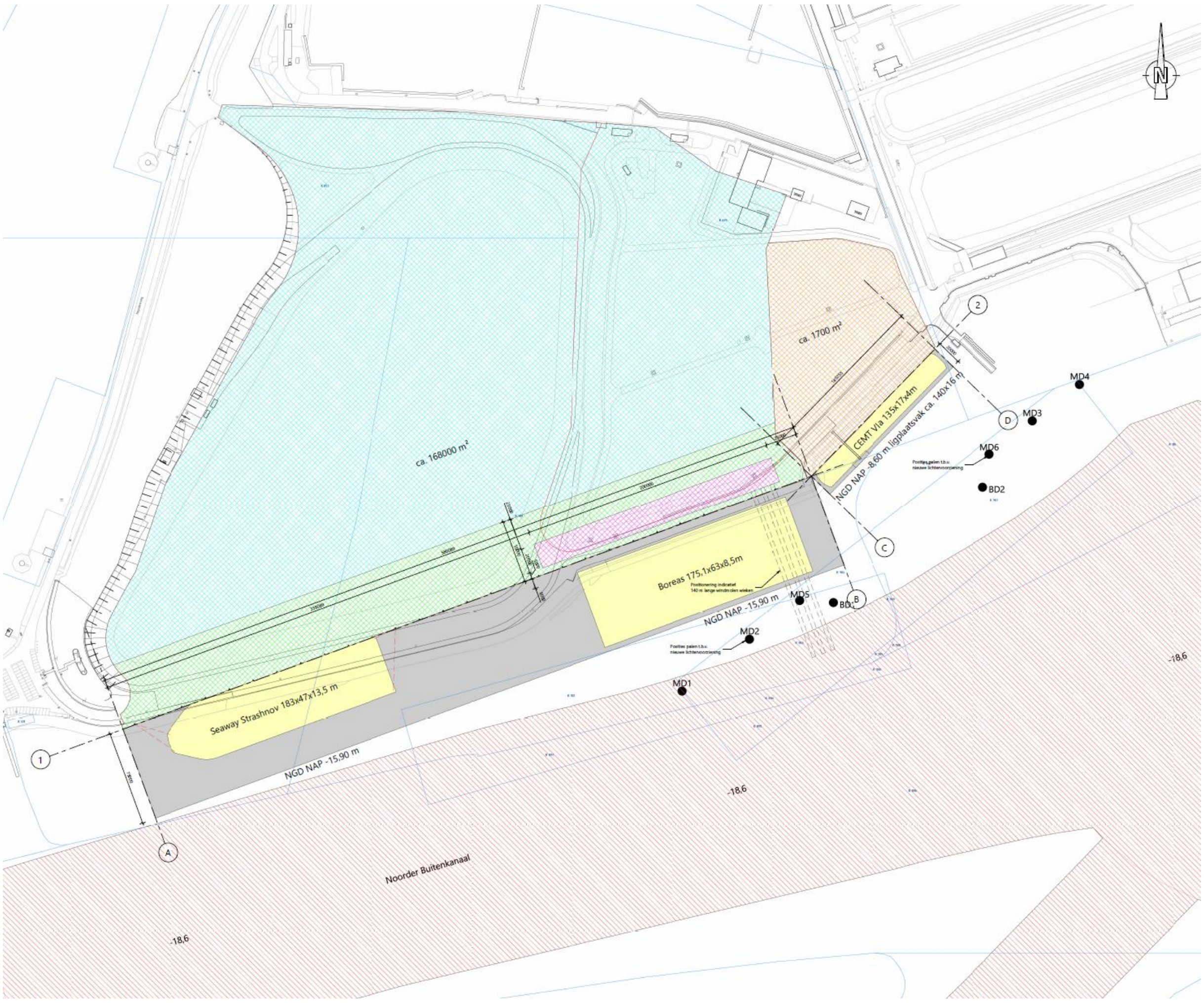

Figure 11 The drawing shows the new port facility with the area and several vessels at the quay wall (Witteveen en Bos, 2024).

1.4.2 Thesis approach

Elementary design cycle

This section presents the design approach that is going to be used for this MSc thesis project. As shown in Figure 12, the elementary design cycle is the basis for the design approach for the design of the quay wall (TU Delft, 2024).

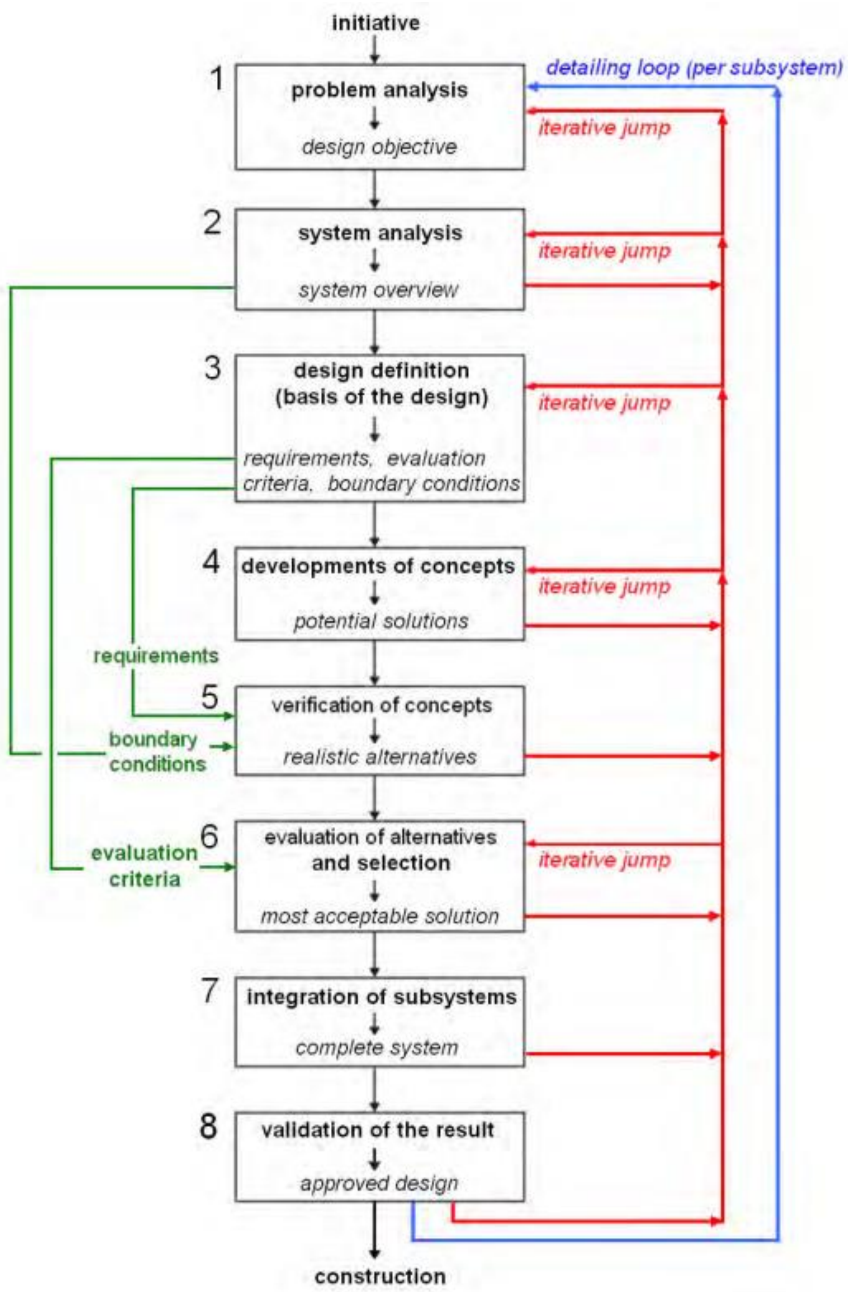

Figure 12 The engineering design cycle (TU Delft, 2024).

Design loops

In an ideal case, spatial-functional and structural design are developed in a combined process. Due to the complexity of the construction of the quay wall, it is decided for this thesis first to create a spatial-functional design, followed by the structural design.
The problem analysis, system analysis, literature study, and basis of the design (step 1,2,3,4) are performed once because this applies to both design loops. The programme of requirements and boundary conditions contain separate sections for the spatial-functional and structural design loops.

The steps for the thesis approach are:

1. **Problem analysis (Chapter 1.2)**
   In the problem analysis, the problem is identified and the objective is defined. An idea of the desired solution occurred.
2. **System analysis (Chapter 3)**
   Performing a system analysis is relevant for designing a quay wall since the quay wall is part of a system and has several functions. Part of the system analysis is the <u>area analysis</u>: the flood defence

system analysis. Another part is the inventory of the stakeholders. Followed by the process and function analysis.

3. **Define the basis of the design (Chapter 4)**
   The third step has the following results:
   a. Starting points: Shows the decisions imposed by the client (Port of Amsterdam).
   b. A programme of requirements: Divided into functional and structural requirements.
   c. A programme of evaluation/selection criteria: The criteria are derived from the stakeholder analysis.
   d. Define the boundary conditions.
4. **Determination of the suitability of steel slags for quay walls (Chapter 2)**
   a. Determine steel slag soil parameters
   b. Describe the feasibility of installing foundation piles in steel slag soil layers.
5. **Perform a spatial-functional design loop (Chapter 5)**
   In the spatial-functional design loop function the aim is to find a technical solution, to fulfil the main functions and determine the necessary components. The steps of this loop are:
   a. Determine the technical systems.
   b. Determine the main dimensions.
   c. Develop, verify, evaluate and select the possible variants.
6. **Perform a structural design loop (Chapter 6)**
   The structural design loop aims to develop ideas to ensure the constructability and structural safety of the quay wall.
   a. Explore the constructability, considering the type of quay walls and the implications of the construction area.
   b. Perform stability checks to determine the stability of the quay wall.
   c. Perform strength checks to ensure the structure can cope with the applied forces.
   d. Perform stiffness/deformation checks.
   e. Execute a final evaluation and selection.
7. **Validate the resulting layout (Chapter 7)**
   The selected conceptual design is validated.

Scope

The scope of the thesis is to design the parts of the quay wall indicated with A, B, and C in Figure 3. Part D will be a different type of quay with other purposes and vessel types and is therefore not accounted for in this report.

In-depth questions

1. How can a quay wall be designed on a steel slag soil layer in the most sustainable way possible?

The answer is provided in the Section: Quay wall type selection.

2. What are the potential risks of constructing a quay wall on a steel slag soil layer?

Answer provided in the Section: Determination of the suitability of steel slags for quay walls.

3. How can steel slags be used most effectively?

Answer provided in the Section: Selection of the quay wall type.
The conclusions made, based on the answers to the in-depth questions can be found in the Section: Answers to in-depth questions.

2 Determination of the suitability of steel slags for quay walls

This chapter develops a method for the verification of the quay wall with a steel slag, which corresponds to step 4 of the thesis approach. The chapter starts with exploring the steel slag parameters, as required for modelling the design of the quay wall. The next subchapter shows the subsoil composition's implications on a quay's constructability. The results of this chapter are used for the structural verification of quay wall elements in Chapter 6.

2.1 Determination of steel slag parameters

The volumes of steel slag soil grading located at the Averijhaven

Based on a 3D model, Rijkswaterstaat calculated the volumes of different gradings of the steel slags. In total 773.620 m^3 of steel is present at the to-be-developed port in IJmond, as shown in Figure 13.

Bepaling Staalslakkenhoeveelheden Averijhavendepot

3D-model								
Laag	**Materialen**		**M^3**	**Aanlegfase**				
01	0/25 cat I (schoon)		149.970	3			149.970	
02	0/25 cat II (IBC)		241.780	3			241.780	
03a	0/25 Binnenzijde Bovenkant	8.170						
03b	0/25 Binnenzijde onderkant	44.030						
03c	0/25 zeezijde	47.390						
03	0/25 totaal		99.590	2		99.590		
04a	0/25 - 40/200 bovenkant	58.350						
04b	0/25 - 40/200 onderkant	58.290						
04	0/25 - 40/200 totaal		116.640	2		116.640		
05	4/80		50.280	3			50.280	
06	25/80		18.380	2		18.380		
07	40/160		1.790	3			1.790	
08	40/200		3.730	2		3.730		
09	10/300		66.460	1	66.460			
	Slakken subtotaal		748.620					
	Opritten (geschat)		25.000	3			25.000	
	Slakken totaal		**773.620**					
10	Kleischerm		12.010	2		12.010		
11	Kraagstuk		2.990					
12	Basalton		2.160					
13	Grind		3.910					
			21.070					
	Totaalhoeveelheid Staalslakken fase			1	66.460			(Aangebracht door Tata Steel)
	Totaalhoeveelheid Staalslakken fase			2		238.340		(Aangebracht door Tata Steel)
	Totaalhoeveelheid Staalslakken fase			3			468.820	(Aangebracht door RWS)
				Totaal		**773.620**		

Figure 13 The volumes of steel slags are separated by the gradings(Rijkswaterstaat, 2011).

The gradings vary from 0/25 mm to 10/300 mm, where more than 50% of the steel slag is within the 0/25 mm boundary.

Nuclear testing

An investigation was made about the steel slags with a grading of 0/25. The goal of the tests was to check whether the nuclear method could be used to determine the degree of compaction.

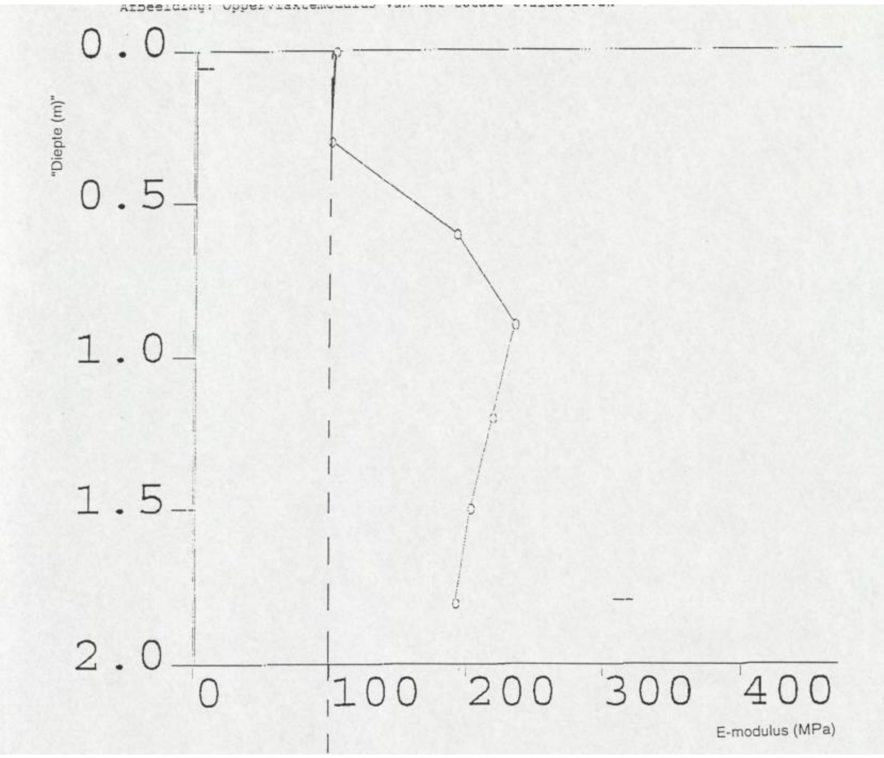

Figure 14 The E-modulus of the steel slag with grading 0/25 mm (Ministerie van Verkeer en Waterstaat, 1998).

It was concluded that this nuclear method is not suitable for this type of material (Ministerie van Verkeer en Waterstaat, 1998). The result of the test is, as shown in Figure 14, that the E-modulus was determined.

Stability of the steel slag dam

In 1997 Fugro performed calculations for the construction of the dike made off steel slags. To do so, the specifications of steel slag used in the calculation are retrieved. As shown in Figure 15, the parameters of steel slags are divided into dry and wet conditions, because steel slag parameters change with the moisture content (FUGRO Ingenieursbureau B.V., 1997).

Grondsoort	γ_d [kN/m^3]	γ_{sat} [kN/m^3]	φ' [^{0}]	c' [kPa]	C_p' [-]	C_s' [-]	c_v [m^2/jaar]
Staalslakken	23,0	26,0	39,4	23,6	1000	-	-
onder water gestort	23,0	26,0	35,0	0,0			
residuele sterkte	23,0	26,0	40,1	7,7			

Figure 15 The parameters used by Fugro to calculate the stability of the steel slag dike (FUGRO Ingenieursbureau B.V., 1997).

For the calculation made in the software Plaxis, the representative soil parameters were used, as shown in Figure 16. The E-modulus is based on the average value, as shown in Figure 14.

Grondsoort	γ_d [kN/m^3]	γ_{sat} [kN/m^3]	Model	E [MPa]	ν [-]
Staalslakken	23,0	26,0	MC	150	0,3
Slib (ongerijpt)	12,5	12,5	MC	1	0,3
Slib (gerijpt)	14,5	14,5	MC	2	0,3
Klei (scherm)	16,5	16,5	MC	3	0,3
Klei	16,5	16,5	MC	3	0,3
Veen	13,0	13,0	MC	3	0,3
Zand, los tot matig gepakt	17,0	19,0	MC	25	0,3
Zand, matig vast tot vast	19,0	21,0	MC	125	0,3

Figure 16 The representative soil parameters used for the finite element calculations (FUGRO Ingenieursbureau B.V., 1997).

It was stated that for the parameters of steel slags, the lower boundaries were taken.

The Library of Material Guidelines of the Netherlands provides parameters for steel slags. It was stated that the parameters differ depending on the grain size (Bodemrichtlijn, 2024).

Table 2 Parameters of steel slags (Bodemrichtlijn, 2024)

Parameter	Value	Unit
Internal friction angle	45-50	Degrees
Cohesion	nihil	kN/m^2
Shrinkage	nihil	%
Expansion(wet to dry)	1 - 4	%
Density	3.100 - 3.400	kg/m^3

Additional soil investigations

In 2013 an investigation at the project area was performed to obtain the different soil layers of the dike ring.

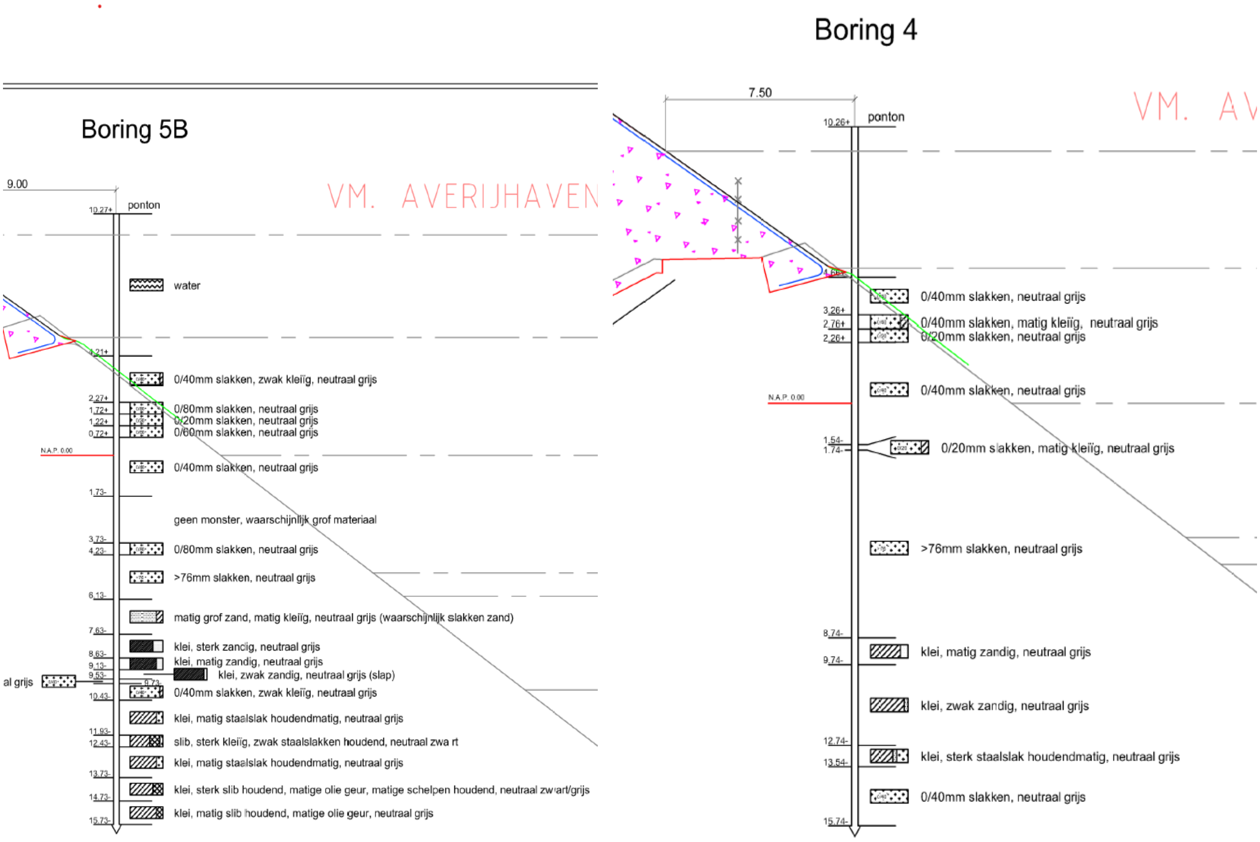

Figure 17 Two examples of the results of the boreholes (Certicon Kwaliteitskeuringen, 2013).

From the results of the boreholes, as shown in Figure 17, it can be concluded that the steel slag grading is not consistent with the depth of the dike. Gradings occur in the range of 0/20-0/80 mm.

Conclusion of the steel slag parameters

As stated in Section 1.2.2, the parameters of the subsoil influence the stability of a quay wall significantly. Especially the internal friction angle [φ'] is important for stability and load distribution. For the construction of the steel slag dams 35 - 40 degrees as friction angle was used for the stability calculation.

Steel slag policies and environmental risks

In the past decade, the usage of steel slags in the Netherlands has increased significantly. The Institute of Public Health and Environment (RIVM) investigated the current state of laws and policies and stated that the risks for the environment and humans are insufficiently covered (Inspectie Leefomgeving en Transport (ILT), 2023). When steel slags are in contact with water negative health effects can occur, such as nose bleeding. Currently, no restrictions are valid for the application of steel slags. Yet, companies using steel slags have to take care of the negative effects on the environment.

2.2 Suitability of foundation pile installation in steel slags subsoil

A test case was performed to check whether or not a pile driving through the steel slag layer is possible. At the test location, three piles were driven into the subsoil. One of the three piles failed, as shown in Figure 18.

Figure 18 The damaged pile, when pulled out of the soil. Indicating the presence of rock or steel slag significantly larger than the pile thickness (Allnamics Geotechnical & Pile Testing Experts, 2024).

The conclusions drawn are the following: Two piles succeeded and this states the possibility of pile driving through the steel slag soil layer. However, the damaged pile, shown in Figure 18, underlines the risks of failure. Most likely this was not an incident and this will occur more often when a larger number of piles will have to be installed. The pile started drifting during the test, which could be caused by the deformations. Another cause could be that the deformation occurred because of the pile-drifting. Therefore during the design and construction of the quay wall, these phenomena should be considered and measures should be taken. Measures could be pre-drilling, applying driving shoes, and avoiding a large number of piles. The test case included a sheet pile wall without pre-drilling. The result showed that the installation of sheet pile elements is possible (Allnamics Geotechnical & Pile Testing Experts, 2024).The results are further analysed in the section named: Determination of the pile and sheet pile installation method.

2.3 Lessons learned from design and construction experience

Many quay walls are constructed worldwide, in various soil conditions and under different load conditions. This experience provides a significant amount of lessons learned for designing a quay wall in a complex situation like in IJmond.

First, soil investigations are of major importance in avoiding unexpected situations. In many projects, soil-related issues come to light during construction, which is too late and results in delays and extra costs. Therefore detailed investigation using the background information is advised. The soil parameters should in some cases be determined for every failure mechanism since the stress and deformation behavior can change. The unknown variation in soil layers can lead to substantial failure of the structure, which states the importance of a sufficient density of soil measurements. For the stability and minimization of settlements of the quay wall, the effects of fill and soil improvement on underlayer soil layers at larger depths cannot be neglected.

The connection between the surrounding structures is sometimes based on old drawings. In time, deformations could have occurred and the drawing could be misleading. The current situation should be determined by measuring the existing structure before the construction starts.

For sheet pile walls combined with piles in between them, interlocking damage can occur. To prevent this significant accuracy is required in positioning the piles. This sheet pile wall is placed afterwards between the piles. The risk of interlocking damage is that it can occur without noticing and reducing the capacity. Sensors can be placed to check if the links between the primary and intermediate piles are damaged. A mitigation measure is to use a pile-driving frame. If obstacles are located in the subsoil pre-drilling may provide an option. The spacing between the piles must be maximized for optimal load transfer and ease of installation. A pile driving analysis could be carried out. In clay layers tensile forces in the pile can occur, which requires larger pre-stressing forces.

For combi walls with a relieving platform, a horizontal sliding plane occurred. It should be checked if sufficient shear stress is present. In the calculations of these walls, often every element is calculated but the total system will react differently than the sum of all the elements.

Another lesson learned from previous projects is to calculate the forces during all the phases of the structures. When sand is reclaimed for port development the sand can be loosely packed. This can result in large displacements during the construction of the quay wall. To mitigate this compaction of the reclaimed soil can be performed.

Foundations in gravel are unpredictable. Sometimes an expensive mitigation measure was used to make sure the piles could be driven into the rocky soil, however, in reality, the piles could be easily placed using a conventional method. This resulted in larger costs (de Gijt & Broeken, 2013).

2.4 Conclusion

The verification of the feasibility of the construction of a quay wall type can be performed by analysing the construction sequence of the quay wall. A quay can be constructed if all steps are possible to execute. As described the foundation of the quay wall is crucial for the stability and bearing capacity. Foundation piles are possible but risk reduction measures can be necessary, resulting in additional costs. When the foundation piles fail due to damage or insufficient bearing capacity, the safety of the quay wall will be reduced. The risks related to the steel slag are also related to the high permeability, as in quay wall design the water behaviour is important for the stability of the soil during all construction sequences. Therefore the determination of the design parameters is important for the verification of the soil stability. Especially when a model is made of the structure to determine the forces on the elements. The learned lessons, as described, must be taken into consideration during the design of a quay wall on a steel slag subsoil.

3 System analysis

This chapter shows the system in which the quay wall design must be constructed, corresponding to step 3 of the design approach. It consists of an area, stakeholder, and function analysis, providing information to draw up a programme of requirements and boundary conditions (Chapter 4).

3.1 Area analysis

In the past, the storage area had a different function as shown in Figure 19. The area consisted of an Averijhaven and a Werkhaven separated by a dam.

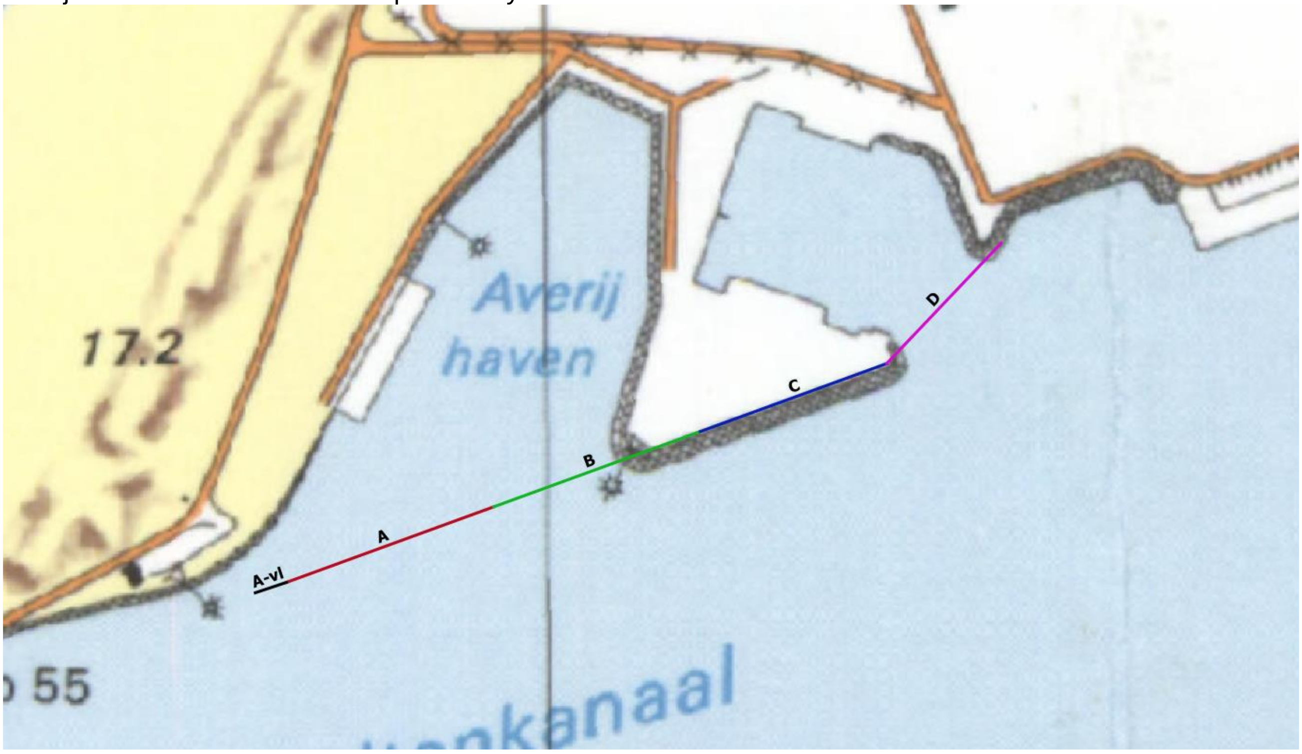

Figure 19 A map showing the layout of the construction area before 1979 (Witteveen + Bos, 2021).

From 1979 until now, the Averijhaven was used as a dredging storage area. To create this storage, a dam consisting of steel slags was made to close off the Averijhaven and the separation dam was removed. Over the years, the dam height was increased to enlarge the storage capacity.

The primary flood defence system is located about 500 m behind the opted location for the to-be-installed quay wall (Nationaal Georegister, 2023). So the quay wall will not be part of the primary flood defence system. The area is located between recreational beaches and the factory of Tata Steel.

On the construction side, several implications should be marked. First, the steel slag subsoil could have an impact on the overall stability port layout. Second, there is a chance that dike protection layers are hidden below the soil layers, which could result in difficulties during the construction of the foundation, as can be seen in Figure 20.

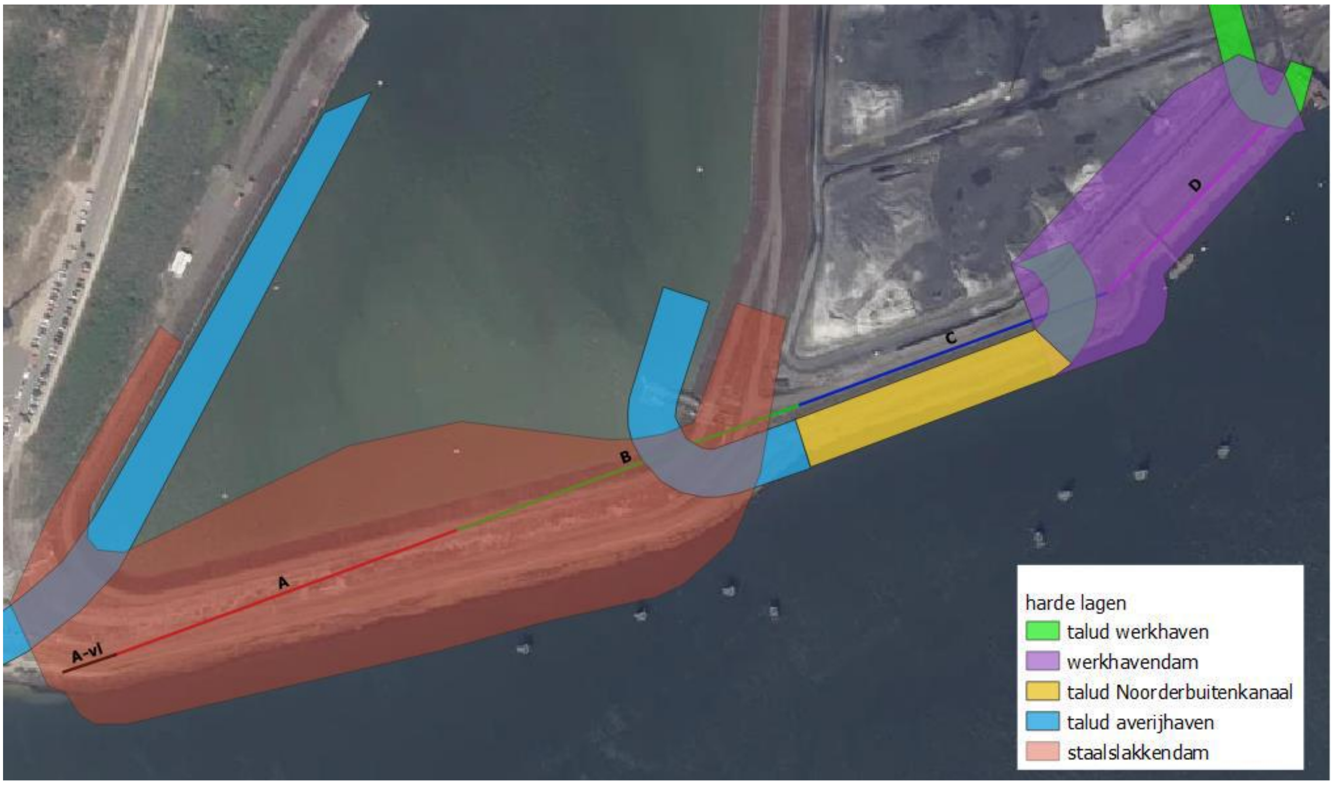

Figure 20: Based on historical maps, An overview of the layers that could hinder the construction works. The line elements indicate the new quay wall (Witteveen + Bos, 2021).

Soil composition

Because of objects located in the subsoil such as dike protection layers, a more detailed survey could be useful to account for constructability. The protection layer consists of blocks of basalt and rock armour (Rijkswaterstaat, 1991). This states the need for further investigation close to the newly constructed quay wall location, especially at the dike ring.

The dike ring consists of several types of steel slags, as shown in Figure 21. Steel slags occur in sizes: 0-6 mm, 4-60 mm, and 40-160 mm (Rijkswaterstaat, 1996).

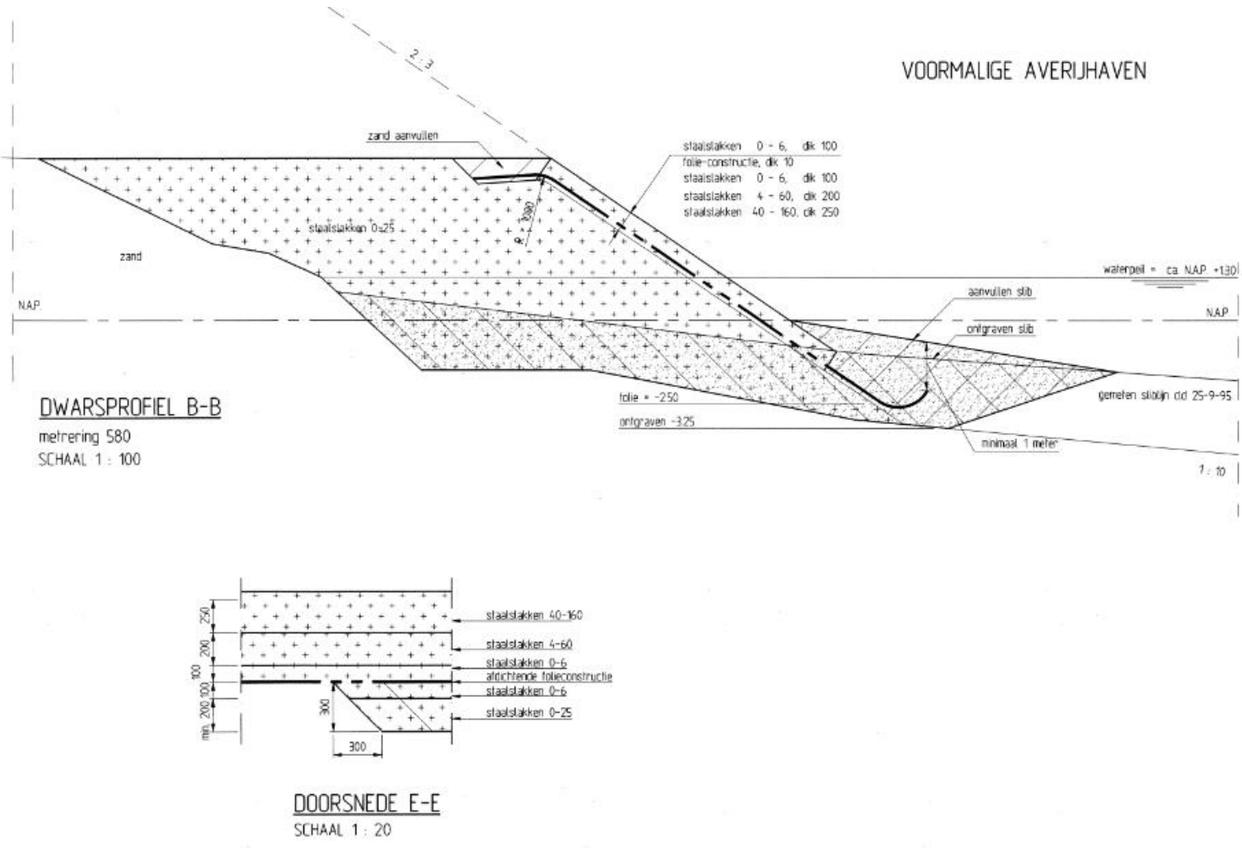

Figure 21 The dike cross-section with the layers consisting of steel slags (Rijkswaterstaat, 1996).

Another important item to consider, especially during the construction is that a cable is indicated in the cross-section D (Rijkswaterstaat, 1996).

A survey executed by Fugro in 2021, is a starting point for the soil layer in the area. In parts, A & B and C & D, the soil layers were surveyed and estimations are shown in Table 3.

Table 3 Soil layers present at the locations(FUGRO, 2019)

Parts A and B		**Parts C and D**	
Soil type	Depth [m + NAP]	Soil type	Depth [m + NAP]
Steel slags	Ground level	Sand and gravel	Ground level
Clay	-18	Sand moderate	-6.5
Peat	-19.5	Sand course	-14
Sand course	-20	clay	-18
Sand moderate to course	-25	Peat	-19.5
		Sand	-20
	-50		-50

In Figure 22 the measured locations are shown.

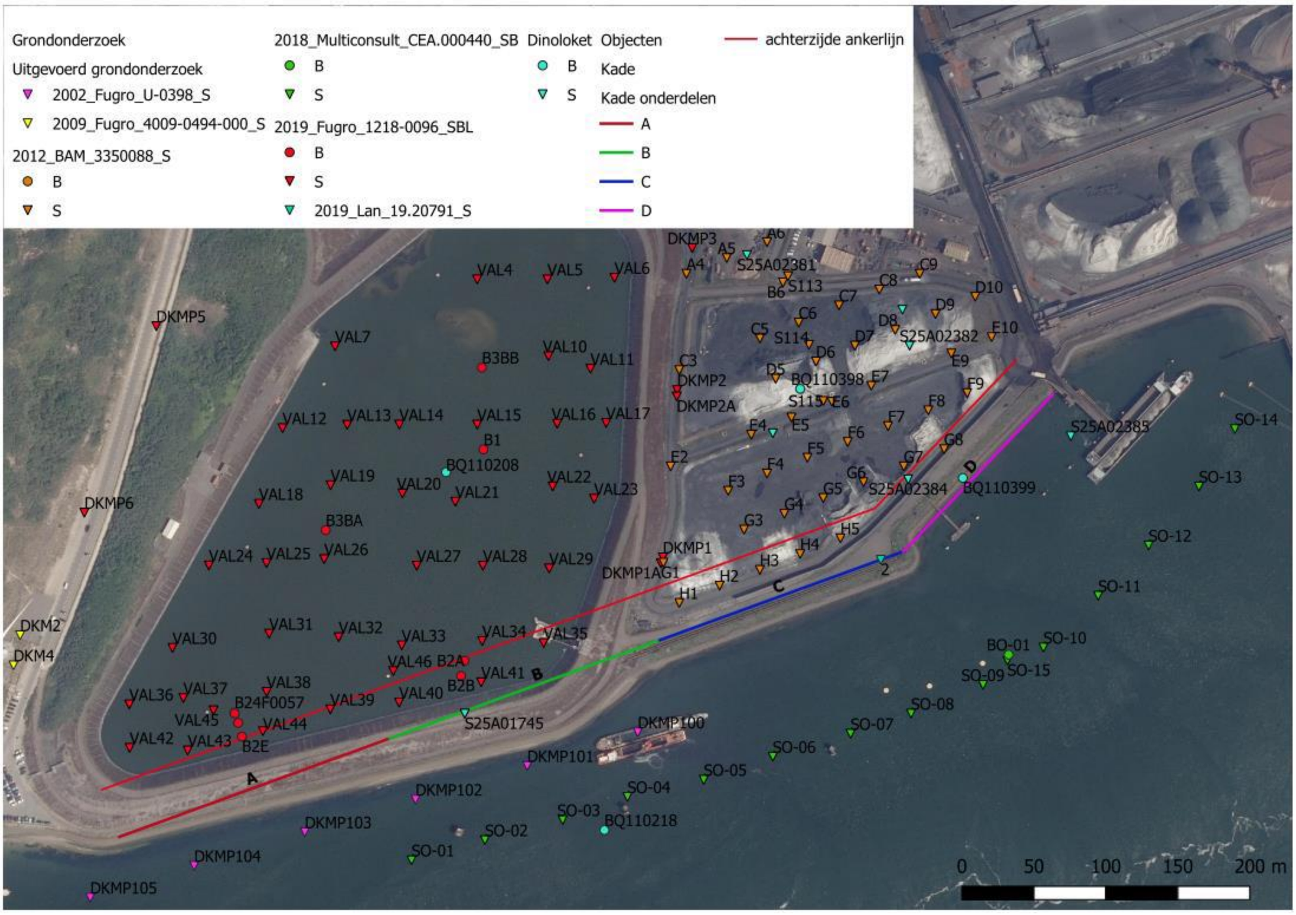

Figure 22 The measured locations at the construction site (Fugro, 2019).

An example of a CTP test result at the project location is shown in Figure 23.

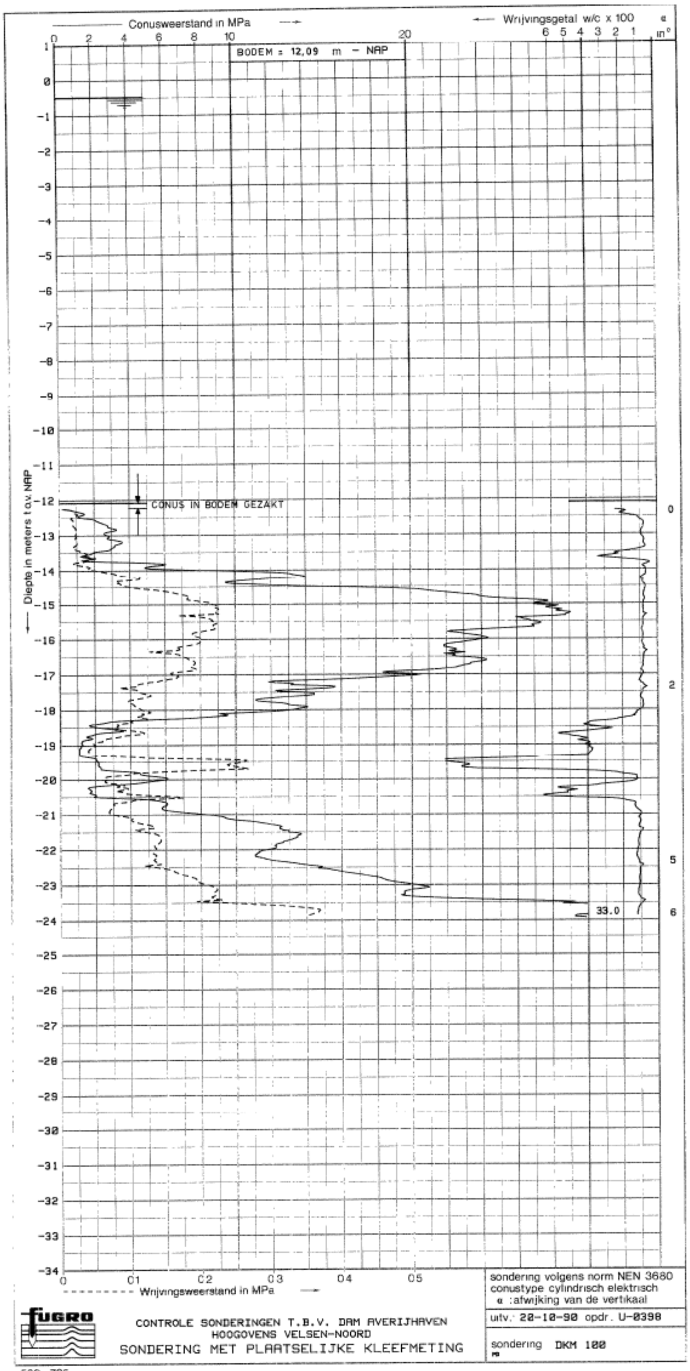

Figure 23 An example of a CTP test result at the Averijhaven at IJmuiden (FUGRO Ingenieursbureau B.V., 1997)

The soil parameters were investigated in the laboratory, the results are shown in Figure 24.

Grondsoort	γ/γ_{sat} [kN/m³]	φ'_{rep} [kPa]	c'_{rep} [°]
Staalslakken	23 / 26	39,4	23,6
Staalslakken (residuele sterkte)	23 / 26	40,1	7,7
Staalslakken (onder water, c' = 5)*	23 / 26	35	7,25
Slib (ongerijpt)	12,5 / 12,5	0	5
Slib (gerijpt), zandig	14,5 / 14,5	24	2
Klei (scherm)	16,5 / 16,5	22,5	5
Klei	17 / 17	22,5	5
Veen	13 / 13	22,5	5
Zand, los tot matig gepakt	17 / 19	30	0
Zand, matig vast tot vast	19 / 21	35	0

Opmerkingen:

γ en γ_{sat} = veldvochtig volumiek gewicht; sat = verzadigd

c'_{rep} / c'_d = representatieve / rekenwaarde effectieve cohesie

φ'_{rep} / φ'_d = representatieve / rekenwaarde effectieve hoek van inwendige wrijving

* is geverifieerd aan de hand van triaxiaalproeven bijlage A

Figure 24 The soil parameters in the construction area (Fugro, 2019).

The parameters for steel slag according to Fugro (2019) are based on an investigation. The soil consists of a clay and steel slag mixture with large variability in ratio. Therefore conservative values were taken.

Water levels

The water level data are provided by Rijkswaterstaat. The average high tide is NAP + 1.01 m, the average low tide is NAP - 0.68 m and the average spring tide is NAP + 1.16 m. The storm height in 1/10000 years is NAP + 5.20 m (Rijkswaterstaat, 2013).

3.2 Stakeholder analysis

The stakeholders and their interests are explained in this analysis. The stakeholders are separated into four categories: Public Service Providers, Private Service Providers, Core Stakeholders, and Periphery Stakeholders. The interests of the clients will form the basis of the programme of requirements and the evaluation criteria.

Public Service Providers

Dutch government:

A consortium was formed by the different governmental organisations to improve the collaboration between the different organisations, however, for this analysis, the organisations were analysed separately.

- Province North Holland: The province of North Holland is involved since the province has the responsibility to stimulate a safe and clean living environment. The provinces assign the locations of wind and solar farms. Due to these responsibilities, the province of North Holland is involved in this project.
- Municipality of Velsen:
    - The municipality of Velsen represents the interest of their inhabitants. The village Wijk aan Zee is located close to the project area, which has about 2200 inhabitants. Since the factory of Tata Steel is responsible for significant disturbance and health issues to households close to the area, the municipality has a high interest in improving environmental issues. A green and clean industry could help to state the ambitions of the local government.
- National government:
    - The national government wants to realise green energy and has an interest in creating this as soon as possible.
- Rijkswaterstaat:
    - The area is owned by Rijkswaterstaat, however, Rijkswaterstaat decided to give the authority of the area to the Port of Rotterdam to take responsibility for this project.
- Ministry of Economic Affairs:

    - This ministry has an interest in stimulating the energy transition by supporting the related project on a national scale. Because of the national interest in the realisation of the renewable energy supply the construction of the port facility will be supported.

European Union:

- Since the EU wants to stimulate the creation of a renewable energy supply, it was decided to support this project financially. Throughout the design phase, this objective is considered through the development of a robust and efficient quay wall. This is to ensure the timely operational readiness of the port facility.

Private Service Providers

Port of Amsterdam

- Port of Amsterdam is responsible for the construction of the Energiehaven. Port of Amsterdam is interested in creating a workable solution for industrial needs most sustainably. By 2030, the port of Amsterdam wants to be in the top of the most sustainable ports of Europe. During the design phase, the client is involved in the choices undertaken. The interests of Port of Amsterdam are translated into requirements and criteria. To achieve these goals it is required to take into account the (environmental) risks of steel slag materials.

Core Stakeholders

Wind turbine-related companies (including vessels)

Several types of vessels are used to install wind turbines; in most cases, spud can vessels are being used to obtain the required stability. In addition, Roll-on-Roll-off vessels will be used in the Energiehaven.

- The port needs to have a safe and efficient port facility, in which vessels can safely enter. The vessels that are currently required to enter the port are:

Table 4 Design vessels with the dimensions.

Name	Length(m)	Draught(m)	Width(m)
Boreas	**175.1**	**8.5**	**63**
Voltaire	**169.3**	**7.5**	**60**
Pacific Orca	**130**	**6.0**	**49**
Seajacks Scylla	**139**	**7.8**	**50**
Aeolus	**139.4**	**5.7**	**38**
Orion	**216.5**	**10.5**	**49**
Blue Tern	**151**	**5.8**	**50**
Seaway Strashnov	**183**	**8.5-13.5**	**47**
Sea Yudin	**183.3**	**5.5-8.9**	**36**
Bokalift 1	**216**	**8.5**	**43**

Periphery Stakeholders

Tata Steel

- The main interest of Tata Steel is to avoid hindering the company. Since the factory of Tata Steel has much negative attention due to environmental issues and local disturbance, the company wants to realise more positive media attention. The realisation of the Energiehaven could contribute to shedding new light on the area. Part of the construction side was owned by Tata Steel. To account for the interest of Tata Steel during the design phase, the constructability, environmental impact, and local disturbance are important.

To account for the stakeholders during the development of the quay wall, the interests of the stakeholders are provided as the basis of the description of the requirements and evaluation criteria.

3.3 Function analysis

This analysis enables finding relationships between functions and components. The quay wall will be part of the port facility and therefore has to be able to function in this system. The main function of the quay wall itself is to transport the offshore wind turbines and guide the vessels. As a side function, the area should be to decommission oil and gas platforms and cold stacking of rigs. In addition, it could be useful to consider other (offshore) maintenance-related works. Since the quay wall is part of the system of the port facility, it must be able to cope with the required vessels, that should be able to moor in the Energiehaven.

Principal functions:

- Enable wind turbine-related parts to be loaded on corresponding vessels.

Preserving functions:

- Prevent the pre-assembly area of the wind turbines from flooding (too often).
- Retain soil for the area behind the quay wall.
- Maintain the hinterlands groundwater system.
- Maintain recreational areas such as dunes and beaches.
- Maintain the nautical function of the Noorderbuitenkanaal as a connection between the North Sea and the Zeesluis IJmuiden.
- Maintain the rainwater discharge function of the area.

Conclusion

The results of the area, stakeholder and function analysis are used to describe the programme of requirements and evaluation criteria in the next chapter. The various analysis shows the importance of design decisions that have to be made.

4 Basis of the design

In this chapter, the basis of the design is explained, which is step 5 of the design approach. To do so, the starting points provided by the client are shown. Next, a programme of requirements is made for the quay wall based on the system analysis. Then the evaluation criteria imposed by the stakeholders are shown. The last part of the chapter provides an overview of the boundary conditions.

4.1 Starting points

The design decisions imposed by the client (Port of Amsterdam) are explained in this chapter.

Location and length of the quay wall
The location is at the Energiehaven at IJmuiden. The area will be used to accommodate wind turbine components deliveries, storage, pre-assembly and loading activities.

The total length of the quay wall is 580 m.

4.2 Programme of Requirements

The Energiehaven must fulfil the following requirements. The programme of requirements is split into functional, structural and environmental requirements.

4.2.1 Functional requirements

The functional requirements specify the design results and are agreed upon by the client.

Principle functional requirements
The requirements relate to the quay wall's principal function and enable wind turbine-related parts to be loaded on corresponding vessels and prevent the port from regular flooding.

- The design lifetime of the quay wall is 50 years.
- The quay wall enables the transfer of wind turbine parts to vessels.
- The quay wall facilitates the governing vessel: Seaway Strashnov.
- The quay wall is suitable for spud can vessels to moor.
- The quay wall design height provides sufficient height, to efficiently transfer equipment to and from the vessels, and prevents the port from (regular) flooding.
- The quay wall has to connect with the other coaster quay wall.
- The quay wall in total has to connect with the parking area and the slopes on both sides of the quay wall.
- The quay is well-accessible and safe to use.
- The quay provides shore-based electricity, drink water and water discharge to moored vessels.

Preserving functional requirements
To maintain the hinterlands groundwater system and the rainwater discharge function of the area.
- The quay wall has a rainwater drainage system.

To maintain the nautical function of the Noorderbuitenkanaal as a connection between the North Sea and the Zeesluis IJmuiden.
- The Noorderbuitenkanaal is operational during the construction and use of the quay wall.

4.2.2 Structural requirements

The structural requirements make sure the design is well-functioning and have structural integrity.
- The quay wall is constructable.
- The quay wall provides sufficient stability.
- The quay wall is sufficiently stiff and strong.
- The quay wall can resist a design load on top of 200 kN/m^2.
- The design is according to the valid norms and guidelines:
    - Eurocode and NEN

    - CUR166
    - CUR211 Handboek kademuren (drainage requirements)
    - EAU
    - Flexible Dolphins

4.2.3 Environmental requirements

- The steel slags that are in contact with water can be used similarly. This means that the steel slags that are located above the water level, must be placed above the water level in the design. The material that is located below the water level can be placed back below the water level.
- The steel slags that are reused have a constructive function in the design.

4.3 Programme of evaluation criteria

4.3.1 Evaluation criteria

The evaluation criteria are based on the wishes of the stakeholders, as described in Section 3.2 Stakeholder analysis.

<u>Local community</u>

- Disturbance to households living in Wijk aan Zee.
- Access to the beach and restaurants during the construction of the quay wall.
- Risks on environmental and health issues related to steel slag usage in the design of the quay wall. (water quality at the beaches)

<u>Port of Amsterdam</u>

- Maintainability of the quay wall.
- Accessibility of the surroundings during construction.
- Accessibility of the construction site.
- Sustainability.
    - Sustainability includes material usage, efficient use of steel slags and construction methods (excavation and transportation of soil).
- Possibility of inspection.
- Steel slag reuse.
    - The steel slags must be reused as a building material or transported to another location.
- Adaptability of the quay wall to future requirements such as higher loads, and sea level rise.
- Risk management.
    - Due to the complex subsoil risks can occur both during the construction phase and its service lifetime. Risks include delay, extra costs and structural failure.

<u>Port exploitation company</u>

- Risks of extra costs, delays, and operational risks.
- Adaptability of the quay wall to future requirements such as higher loads, and sea level rise.

<u>Tata steel</u>

- Accessibility of the construction site (since roads owned by Tata Steel will be used).
- Duration of the construction period.

4.4 Boundary Conditions

4.4.1 Design vessel

The construction of the wind farm located close to IJmuiden was granted to Van Oord (Van Oord, 2024). The vessel van Oord is going to use for the installation of wind turbines is named Boreas.
For the design of the quay wall, the vessels are shown in Table 4. As governing draught, 13.5 m is taken for further calculations.

4.4.2 Water levels

The water levels at the Energiehaven are retrieved from Rijkswaterstaat and shown in Table 5.

Table 5 Water levels retrieved from Rijkswaterstaat (Rijkswaterstaat, 2013).

Frequency	Water levels (NAP)	
Average hightide	+0.97 m	
Average low tide	-0.73 m	
High springtide	+1.15 m	
Low springtide	-0.75 m	
LAT	-0.95 m	
1/10.000 years	+5.15 m	
1/100 years	+3.60 m	-2.50 m
1/10 years	+ 2.95 m	-2.10 m
1 per 2 years	+2.55 m	-1.75 m
2x per year	+2.20 m	-1.55 m
5x per year	+1.95 m	-1.40 m

4.4.3 Currents

A current can occur due to the outflow of the Noordzeekanaal and the Zeesluis Ijmuiden, as shown in Figure 25. This might result in an extra load on the vessels, boulders and quay wall. The pumping system has a maximum capacity of 260 m^3/s (Rijkswaterstaat, 2023). Divided by the area of the channel this would be a current of 0.20 m/s.

Figure 25 Overview of the Noordzeekanaal and the lock at IJmuiden (Rijkswaterstaat, 2021).

The outflow of the Noordzeekanaal is 2.6 billion m^3 of water annually. The lock has a maximum water level difference of 4,94 m, a width of 70 m, and a length of 545 m.

A model made by Arcadis (2021) provides currents for a governing scenario of 500 m^3/s. As shown in Figure 26, this leads to a current of 0.2 - 0.4 m/s at the location of the quay wall. The current on the vessels can be 0.6 - 0.8 m/s, this can lead to a horizontal force on the quay wall.

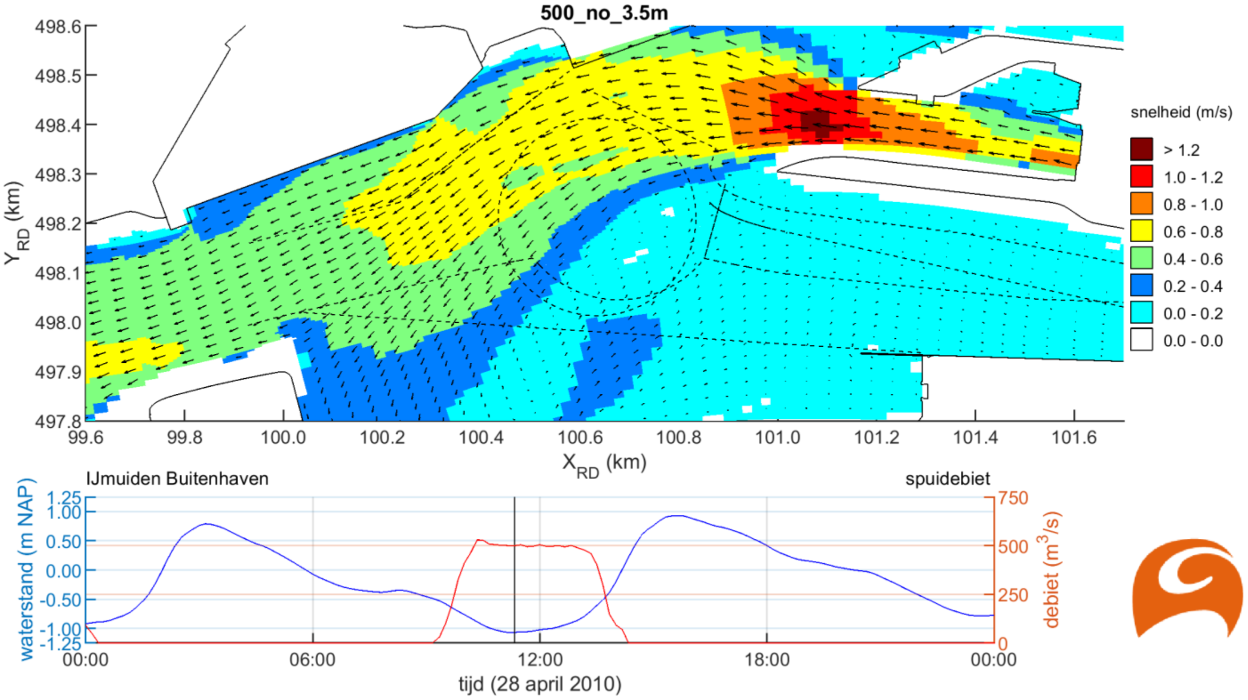

Figure 26 Currents averaged over low water and a discharge of 500 m^3/s (Arcadis, 2021).

4.4.4 Obstacles in the subsoil

As described, the newly constructed quay wall is located at a formerly dike. This dike consists of steel slags to a large extent. In addition, the quay wall foundation will cross two ‘old’ dike protection layers on the east and west sides. This protection layer consists of basalt, rock armour, and concrete blocks.

5 Functional design

In this chapter, the main dimensions of the quay wall are determined, the most suitable quay wall is selected and the complementary technical systems are inventoried (step 5 of the thesis approach).

5.1 Determination of the main dimensions of the quay wall

The two levels that must be determined are the top level of the quay wall and the required bottom level. From these levels, the constructional depth can be obtained.

5.1.1 Top level of the quay wall

The height of the quay wall is based on the connection to surrounding quays, the efficient operation level of the port, and the accepted flooding exceedance probability.

- Since this quay next to the new quay has a height of NAP + 5.2 m and a connection has to be made between the quays it will be useful, for constructional reasons, to construct the new quay at the same level if this is suitable for the new functionality
- This level corresponds to a flood risk smaller than the required risk of once in 10000 years, which is acceptable because the seawater-sensitive materials are placed higher than the quay level in the storage area. The risk of flooding is acceptable for Port of Amsterdam in compliance with their client (Port of Amsterdam, 2023).
- Lastly, this top level is within the workable range height for the design vessels. As they use cranes for lifting. In addition, the design vessel is equipped with a dynamic positioning system (DP3) which enhances the operational range of the vessel (Seaway 7, 2019).

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5.1.2 Bottom level of the quay

The depth is related to the draught of the design vessel. The current maximum required draught of a wind turbine-related installation vessel is 13.5 m, as shown in Table 4. Two main design vessel types govern the quay wall level. First, the required bottom level is calculated for the governing spudcan vessel and then for the design vessel without a jack-up system. This is to compare the two required depth levels.

The governing draught was taken, corresponding to the design vessel Seaway Strashnov (13.5 m). Then the nautical guaranteed depth (NGD) was calculated. Based on the NGD, the constructional depth was calculated. Several margins are considered: first, a dredging and sounding margin (1 m) are used to calculate the nautical guaranteed depth. This margin covers the inaccuracy in the dredging process both in monitoring and execution. For the constructional depth calculation, a maintenance margin (0.5 m) is considered due to the sediment that can occur. The margins are provided by dredging companies based on proven practice. The spud can penetration (3.5 m) is estimated by considering the soil conditions and tests (Witteveen + Bos, 2021).

For jack-up vessels, the calculation provides the NGD of NAP -15.90 m and the constructional depth of NAP -19.90 m, as shown in Figure 27.

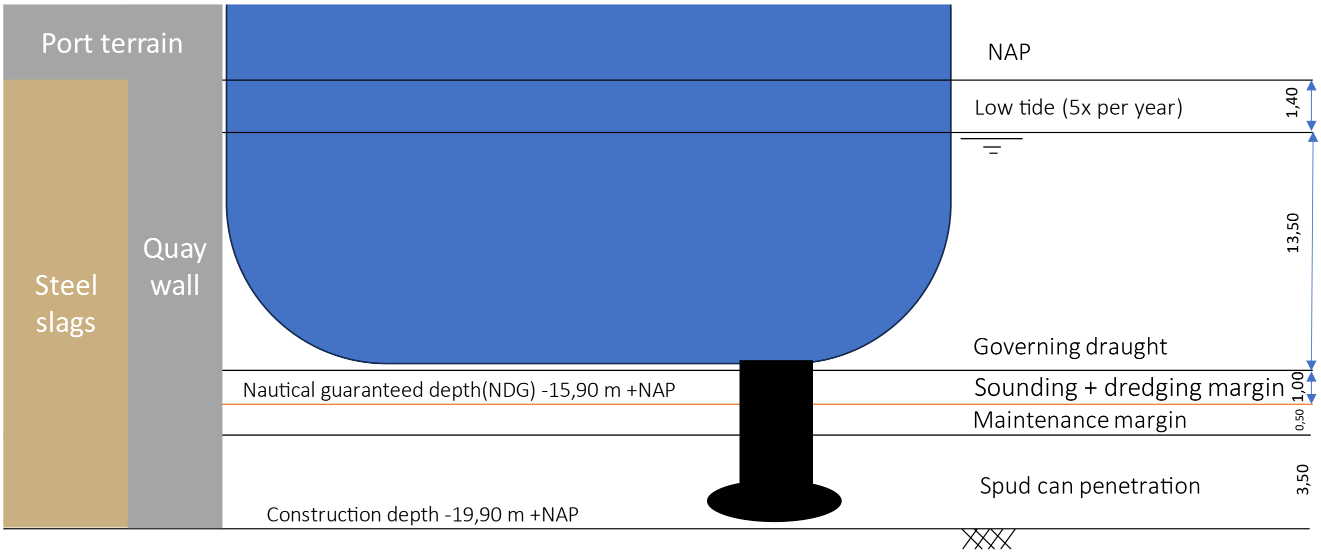

Figure 27 The nautical and construction depth indicated, additionally, the quay wall and spud can vessel draught.

The nautical and construction depths are also calculated for vessels without spud cans. This reduces the construction depth since no soil penetration must be considered. The construction depth is NAP -18.40 m, as shown in Figure 28, for the governing draught without spud cans.

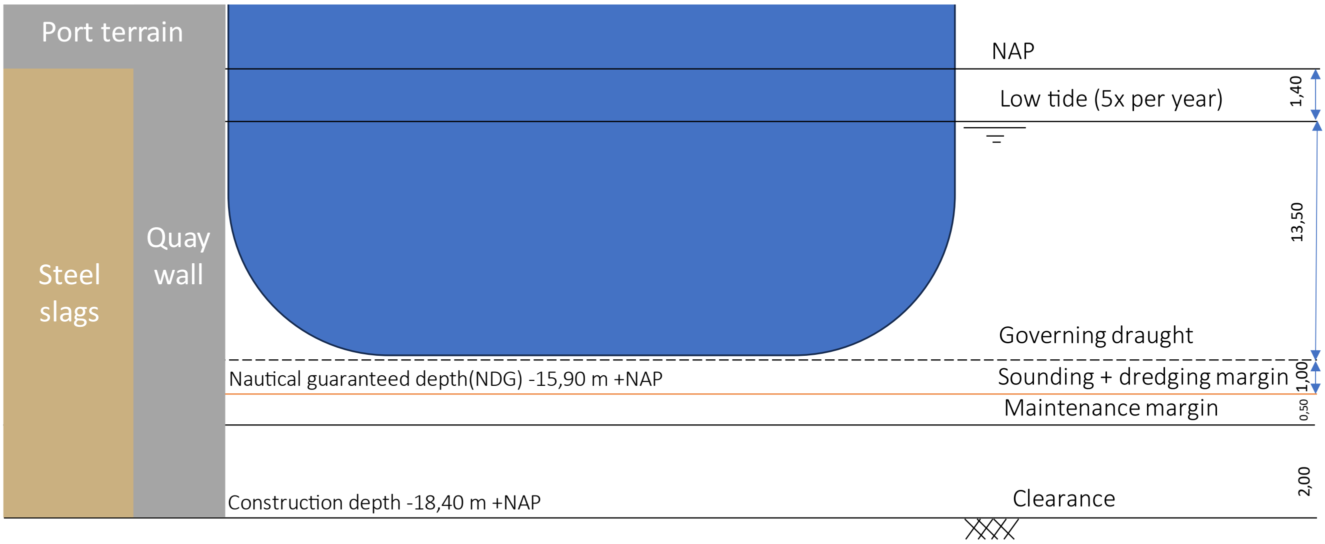

Figure 28 The nautical (NGD) and construction depth with the quay wall and design vessel (without spud cans).

An optimisation could be made since the quay wall will be 580 m long, it can be split up into two parts with different construction depths. This would reduce the construction height of one part and thus material volumes and emissions.

5.2 Quay wall type selection

5.2.1 Inventory of quay wall types

Several types of quay walls exist. They can be divided into four main types (Meijer, 2006):

1. Gravity walls
2.
    a. Embedded walls
    b. Embedded walls with relieving platform
3. Open berth quays

An overview of the soil-retaining structures is shown in Figure 29.

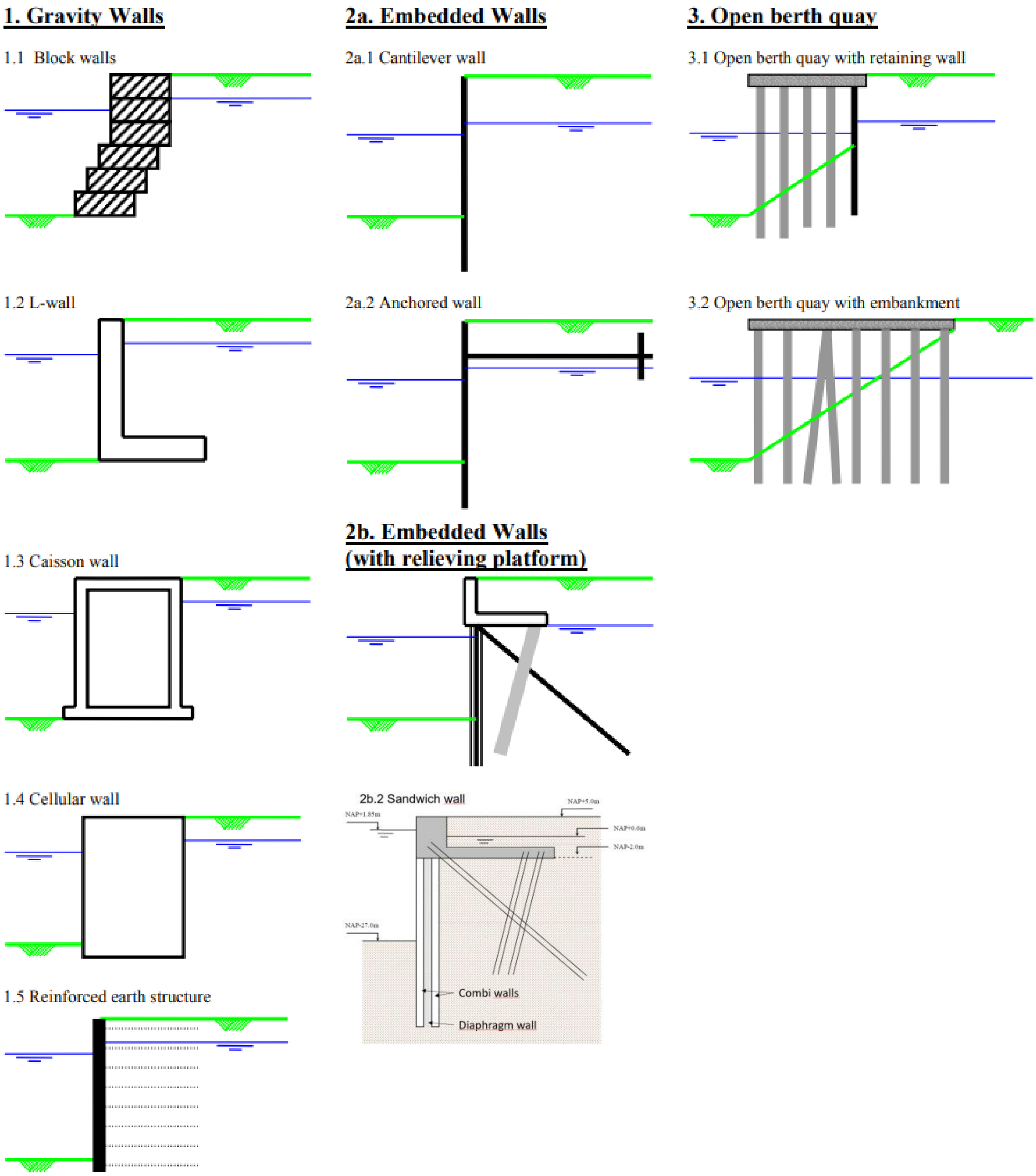

Figure 29 Overview of quay wall types separated in gravity, embedded, and open berth quay walls (adapted from Meijer, 2006).

An explanation of the quay wall types can be found in Appendix C.

5.2.2 Verification of quay wall types

To verify the different types of quay walls, an overview was made in Table 6 with the advantages and disadvantages shown. At this stage of the design process, a selection is made of which types are possible The quay wall type is verified by considering the system analysis and basis of the design. The stakeholders' interests and the programme of requirements are taken into account.

Table 6 An overview of the quay wall type with pros and cons.

Type of quay wall	Advantage	Disadvantage
Sheet pile **Combi wall**	Relatively low costs	Anchors needed. Max. applied force limitation Sensitive to rocks in soil
Combi Wall with relieving platform	Experience in the Netherlands It can be combined with various stabilisation measures	Requires piles and/or anchors(grout)
Diaphragm wall	Strength, high bending moment	Risks for leakage due to the steel slag soil
Caisson	Caissons can be filled with steel slags	Subsoil must be sufficiently strong Caisson production location required Labour-intensive Much soil excavation and transportation. Up to 30% of steel slag reuse
Gravity blocks	Strength	Strong subsoil needed Large concrete volumes and thus emissions Much soil excavation and transportation
L-Wall		Much soil excavation and transportation. Stiff subsoil is required.
Deck on piles	Minimal excavation.	Separate structure for horizontal forces. A large number of piles, so large risk of failure due to the steel slags.
Sandwich wall	Large dimensions and bearing capacity.	No experiences Pile driving and anchors needed Grout body is not advised with steel slag soils Costs
Reinforced soil	Up to 100% of steel slag reuse. Large soil volume of soil excavation and transportation.	Lack of experience in these dimensions. Large deformations and
Cofferdam	Small soil transportation Up to 100% steel slag reuse	Construction of two walls.

Several quay wall types are not feasible for this project due to the dimensions (construction depth 20 m), loads (200 kN/m^2), and soil conditions (steel slag subsoil).

- First, the sheet pile wall is unsuitable, because it will provide insufficient stability and resistance. Due to the steel slags fluid can easily disappear in the subsoils, which causes (constructional) problems for a diaphragm wall and a sandwich wall.
- The gravity-based quay walls (caisson, L-wall, block wall) come with large volumes of soil excavation and transportation. These walls are not constructed on top of the soil, but in the soil, resulting in large volumes of soil transport. The dike must be removed first which results in extra costs and emissions. Creating one of these gravity walls is not impossible, but only if other types are unrealistic and unsustainable.

- The sandwich and diaphragm walls start with an excavation filled with bentonite to support the walls. This bentonite is a fluid material and will disappear in the steel slag subsoil due to its high permeability. In a later stage, the bentonite is replaced by concrete, which might cause leakage problems. Therefore an impermeable layer must be placed first to mitigate the leakage risks. Considering this inefficiency and risks in constructing these quay wall types gives preference to other types.

Considering the earlier-mentioned risks for each variant and the information shown in Appendix C, a selection was made based on critical requirements, as shown in Table 7. These verification steps are based on preliminary calculations and considering the limitations of the quay wall types.

Table 7 Verification of the variants based on three criteria requirements.

Quay wall type	Constructible	Sustainable	Stable
Sheet pile wall	√	√	X
Combi wall	√	√	X
Diaphragm wall	X	√	√
Caisson	√	X	√
Gravity block wall	√	X	√
L-wall	√	X	√
Deck on Piles	√	√	√
Sandwich wall	X	√	√
Reinforced soil	√	√	X
Cofferdam	√	√	√
Cellular wall	X	√	X
Combi wall with relieving platform	√	√	√

If a variant is possible to construct, taking into account the soil conditions the type gets a positive mark (√) and is thus constructible. If the type of quay does not require enormous volumes of soil to be transported and excavated the variant is considered sufficiently sustainable. If the variant can become stable considering the dimensions and loads the type gets a positive mark for stability. Variants with three positive signs which are thus most promising for the design are the combi wall with relieving platform, cofferdam and deck on piles. These are the types that are evaluated.

5.2.3 Evaluation of quay wall types

5.2.3.1 Evaluation criteria

To compare the verified alternatives evaluation criteria are specified. The criteria are derived from the client's wishes and the stakeholder's interests. The wishes and interests are translated into the four most important evaluation criteria.

Ease of construction

As described, the history of the area comes with some complications: the dike protection layers and steel slags. This has consequences for the ease of construction of the quay wall.

Sustainability

In the material usage, steel slags reuse and the soil volumes are considered. Larger volumes of material mean higher CO_2 emissions and thus a larger score on sustainability. The large volumes of materials that must be relocated, cause emissions due to the excavation and transportation of soil. The steel slag volume that can be reused prevents transportation. In addition, the environmental aspect is taken into account.

Maintainability

The score on maintainability is based on the possibility of inspection of the critical elements of the quay. Normally quay walls are inspected once or twice a year. This is related to damage, deformation and defects that can visually be seen.

Adaptability
The dimensions of the wind turbine have increased massively in the last decades. It is possible, that during the design life of the quay wall, it must be upgraded to withstand higher loads than designed for. Adaptability includes the possibility of adapting the quay wall for increased load conditions.

5.2.3.2 Determination of weighting

The weighting of the criteria was determined by comparing the criteria to each other. If one criterion is more important compared to the other, it gets a score of 1, and if it is less important it gets a score of 0. The stakeholder analysis, as shown in the Section 3.2 provides the preferences of the stakeholders. Various stakeholders have various preferences for the weighting, therefore Port of Amsterdam, as the client, dominates this process of determining the weighting values. In Table 8 the ease of construction has the highest weight in the determination of the winning variant.

Table 8 The determination of the weighting factor.

Weighting							
	Ease of construction	Sustainability	Maintainability	Adaptability	Total	Score	Weighting factor
Ease of construction	x	1	1	1	3	6	0.50
Sustainability	0	x	1	1	2	4	0.33
Maintainability	0	0	x	0	0	1	0.08
Adaptability	0	0	0	x	0	1	0.08

5.2.3.3 Evaluating the constructability of the types

The evaluation criteria are based on the interests of the stakeholders as explained in the Section 4.3. Constructability is one of the decision-making criteria, therefore the construction sequences are provided to illustrate the possibilities for construction. In the evaluation phase, the cofferdam, combi wall and deck on piles are considered the most promising options for constructing the quay wall. The construction sequences are shown to illustrate the construction phases of the three best variants. First, the construction sequence of the cofferdam is shown, followed by the combi wall, and last the deck on piles variant.

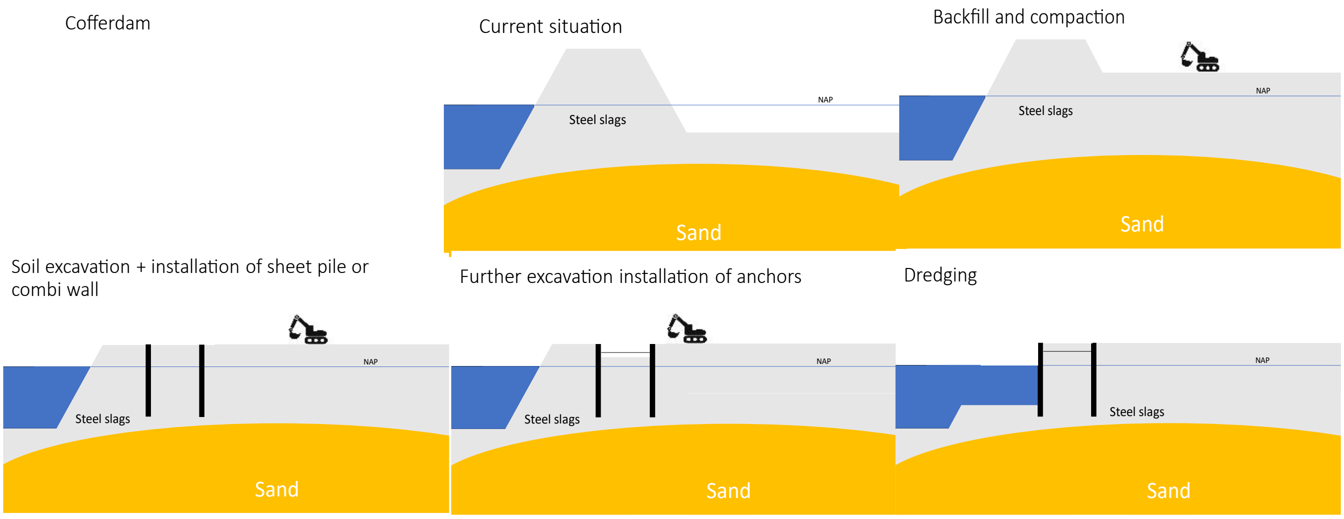

Figure 30 Construction sequence of a cofferdam quay wall.

As shown in Figure 30, the cofferdam construction consists of 4 main steps. The first step is to prepare the port area by backfilling it with sand. Then the steel slag embankment must be excavated until the top level of the quay. Then the combi walls can be installed. After that, the area between the combi walls can be removed and the anchor installed.

The construction sequence of the combi wall with the relieving platform is shown in Figure 31. This method consists of 7 main steps.

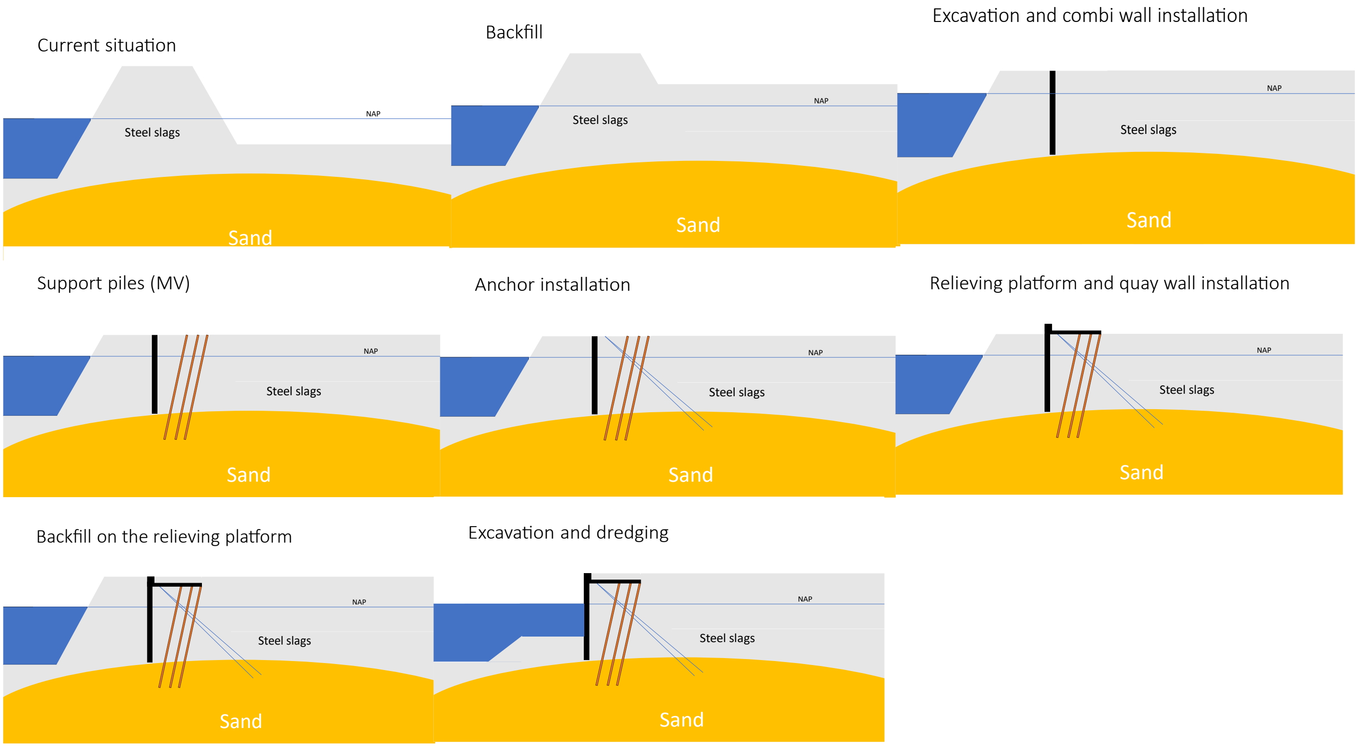

Figure 31 Construction sequence of a combi wall with relieving platform.

The combi wall required high accuracy in the location of the piles, this could be a risk considering the steel slag subsoil.

The next option for constructing a quay wall could be a deck on piles, as shown in Figure 32.

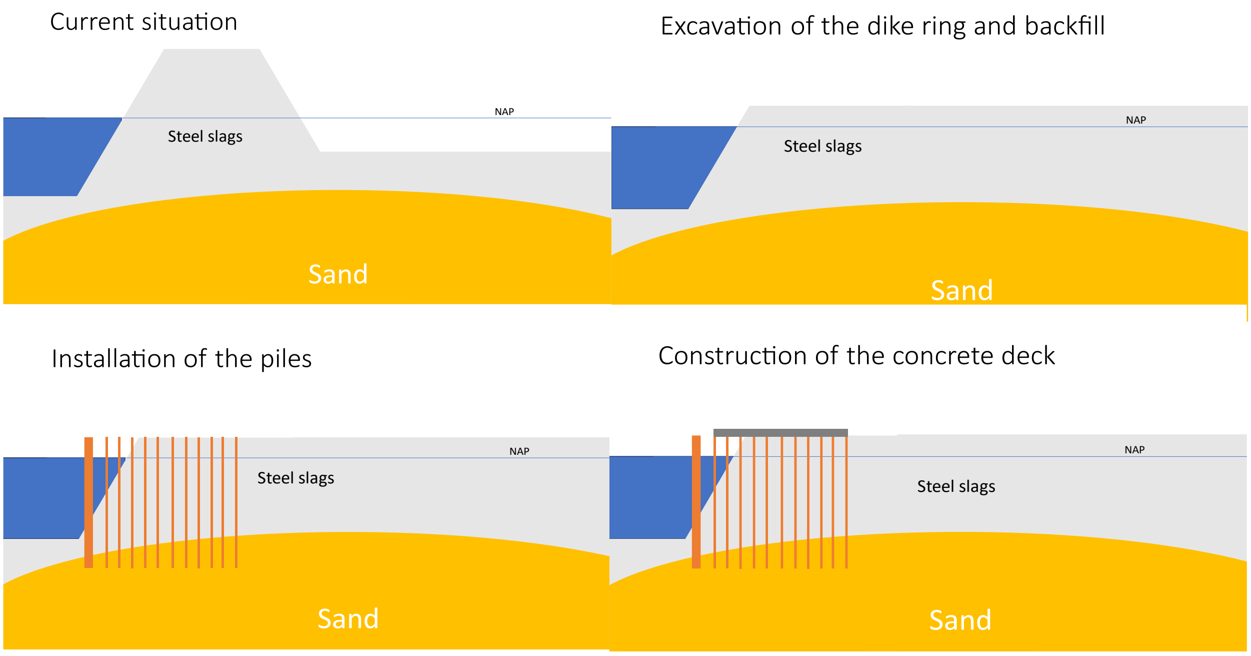

Figure 32 Construction sequence of a deck on piles solution.

Based on the construction sequence, no serious issues or large excavation volumes seem to occur. The main risk is the number of piles and the risks related to the pile drilling. On the other hand, the floor can be modelled and calculated by the uncertainty in the location of the piles. Yet, the number of piles will be higher than the other two variants. A deck on piles cannot be easily upgraded to account for larger loads.

5.2.3.4 Determination of the scores

To compare the alternatives using a range of criteria a Multi Criteria Analysis (MCA) was used. The most important criteria were selected for the MCA analysis.

Table 9 The MCA(multi-criteria analysis) with the three variants.

	Ease of construction	Sustainability	Maintainability	Adaptability	Total score
Criteria Description	Steel slag complications	Material usage, steel slag reuse, soil volumes			
Weighting	0.50	0.33	0.08	0.08	1
Combi Wall	3.25	2.5	3	2	2.88
Deck on piles	2	3	2	1	2.25
Cofferdam	3.5	3.5	2.75	3	3.40

The scores determined by the MCA, as shown in Table 9, are 2.88 for the combi wall followed by the cofferdam with a score of 3.40. The deck on piles variant scored 2.25.

The scores for every variant are explained based on the construction sequences and preliminary calculations.

Ease of construction

The installation of piles in hard soil layers and with obstacles locally comes with risks. The higher the number of piles, the higher the risk of difficulties in construction and the lower the score for the ease of construction. Other complications are described as follows:

- The combi wall with the relieving platform has the lowest number of piles, however, it requires a relatively high installation accuracy for the installation of sheet piles and anchors. In addition, the relieving platform must be constructed in situ and tension piles must be placed. The relieving platform is located above the low water level, so no measures have to be taken to create a dry construction pit. For the construction of the combi wall and the relieving platform, risk can be mitigated by measures, however, this increases the cost.
- The deck on piles quay needs a significantly larger number of piles in the critical areas and gets therefore the lowest score.
- The cofferdam variant has two combi walls, grout anchors and tie rods at two levels. An advantage here is that the second combi wall is placed behind the steel slag dam. Another advantage is that there is some flexibility in the location of the piles since the tie rods can be shortened in situ. Since no relieving platform is constructed, larger piles are needed. An advantage of large pile diameters is a lower risk for damage due to the stronger and stiffer behaviour.

The deck on piles scores the lowest (2), followed by the combi wall (3,25) and then the cofferdam (3,5).

Sustainability

The amount of steel slags that can be reused is considered in the score.

- The combi wall with the relieving platform uses steel and concrete material. The CO_2 emission of these materials was estimated to be 54 megatons.
- The deck on piles variant uses a large number of steel piles and a large volume of concrete. The advantage is that a small volume of soil must be excavated and transported. In addition, a small amount of transportation of steel slag material is required. The CO_2 emissions for the materials of the deck on piles quay were estimated to be 70 Megatons.
- The cofferdam uses steel and concrete for the concrete apron and piles cap. The amount of material is comparable to the combi wall with a relieving platform. The steel slag reuse is an advantage because all the steel slags can be placed in between the combi walls and in the piles. The volume of soil that must be excavated and transported is larger than the other two variants. The estimated CO_2 emissions of the materials is 30 Megatons.

Considering these points the scores are divided as follows: the combi wall scores (2,5), the deck on piles scores (3) and the cofferdam scores (3,5).

Maintainability

For all variants holds that the use of steel slags can result in unforeseen interaction with the construction material.

- The combi wall elements are connected in the concrete relieving platform. Cracks, loose or spalled-off concrete can be inspected at the visual parts. Due to the concrete relieving platform repairs are complex. On the other hand, much experience is available for the construction and design of combi walls with relieving platforms in the Netherlands.
- A quay wall realised by a deck on piles has a concrete floor and piles. The concrete floor can be inspected. The piles above the soil level are visible, however, a large number of piles are covered by the concrete floor. In addition, performing maintenance will be complex due to the concrete deck.
- The cofferdam elements that are critical for stability are the tie rods and anchors. No visual inspection can be carried out on these elements. However, since no concrete floor is constructed repair works can be executed.

The scores for maintainability are combi wall (3), deck on piles (2), and the cofferdam (2,75).

Adaptability

- The option for the combi wall with a relieving platform can be adapted by installing an extra grout anchor.
- Increasing the strength of a deck on piles is hardly possible. Extra piles could be installed through the floor or the floor could be extended or thickened. All these measures will be costly.
- The cofferdam can be adapted to higher loads by installing a relieving platform and/or additional tie rods and grout anchors. Since no concrete floor is placed adapting the quay wall is straightforward.

This results in the best score for the cofferdam(3), second the combi wall (2), and third the deck on piles (1).

5.2.3.5 Ranking of the variants

A cost estimation is made of the three variants, the total construction cost estimation of each variant is shown in Table 10. This includes the material, installation and excavation costs. A more detailed list of the costs is shown in Appendix A. The costs for the technical systems are not considered, because this will be similar for all quay wall types and this is a relatively small value.

Table 10 The three variants with the estimated costs.

Variant	Costs (million euros)
Combi wall with relieving platform	37
Cofferdam	35.9
Deck on piles	65.6

The risks during construction are considered and estimated in costs. A risk for the cofferdam during construction is the need for drainage. A clay layer is present underneath the sand layer according to the drawings (FUGRO Ingenieursbureau B.V., 1997). When this clay layer bursts open the area between the combi wall should be drained for the tie rod installation. A mitigation measure could be a drainage system and/or dividing the cofferdam into parts by sheet piles (additional costs: +- 1-3 million). Therefore a value of 2 million is reserved for this risk, which is included in the total of 36 million euros.

The relieving platform cannot be constructed in wet conditions. To create a dry construction pit a sheet pile wall must be placed. However, considering the soil conditions this will be complex and costly. The traditional installation method for the sheet pile wall cannot be performed due to the rocky layers. So mitigation measures should be considered. In addition, the permeability of the steel slags might cause significant leakage.

However, the saddle, which connects the foundation piles and the concrete floor must be permanently wet during its operational lifetime to prevent corrosion.
It is possible to construct a combi wall without a saddle; a fixed connection is required which causes a larger moment acting on the floor. This results in extra reinforcement steel and thus costs and emissions. In Figure 33, a cross-section of the combi wall with a relieving platform is shown.

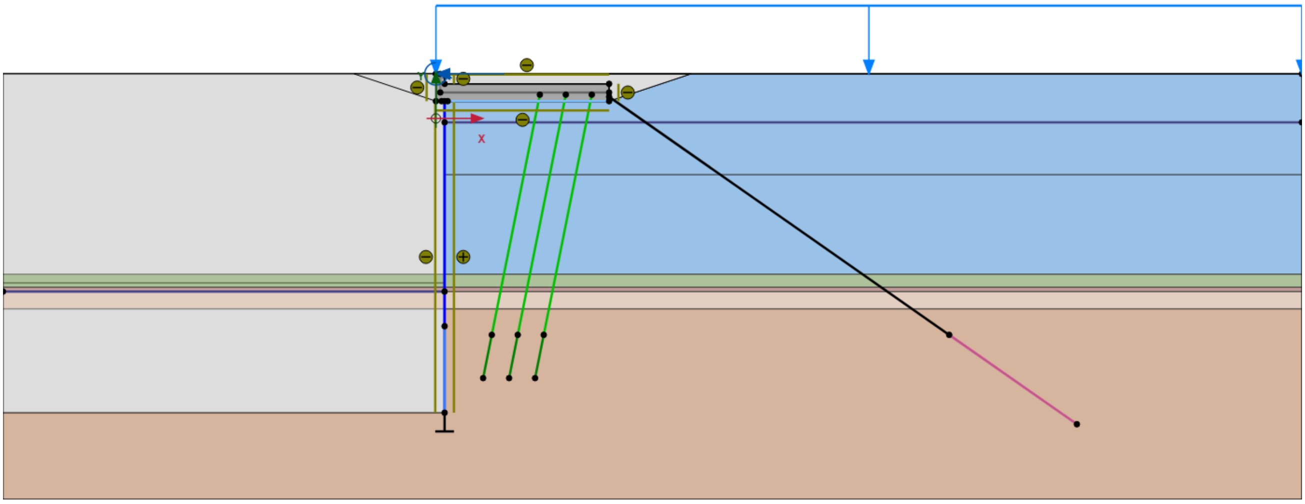

Figure 33 A cross-section of the combi wall with a relieving platform above NAP.

The relieving platform must be located above the water level, this can cause extra costs and emissions. Yet, this option is still possible to construct.

The cofferdam requires at least 2 horizontal tie rods and grout anchors to become a stable quay wall. The back wall of the cofferdam must be placed under an angle of at least 1:10 to provide sufficient resistance, as shown in Figure 34.

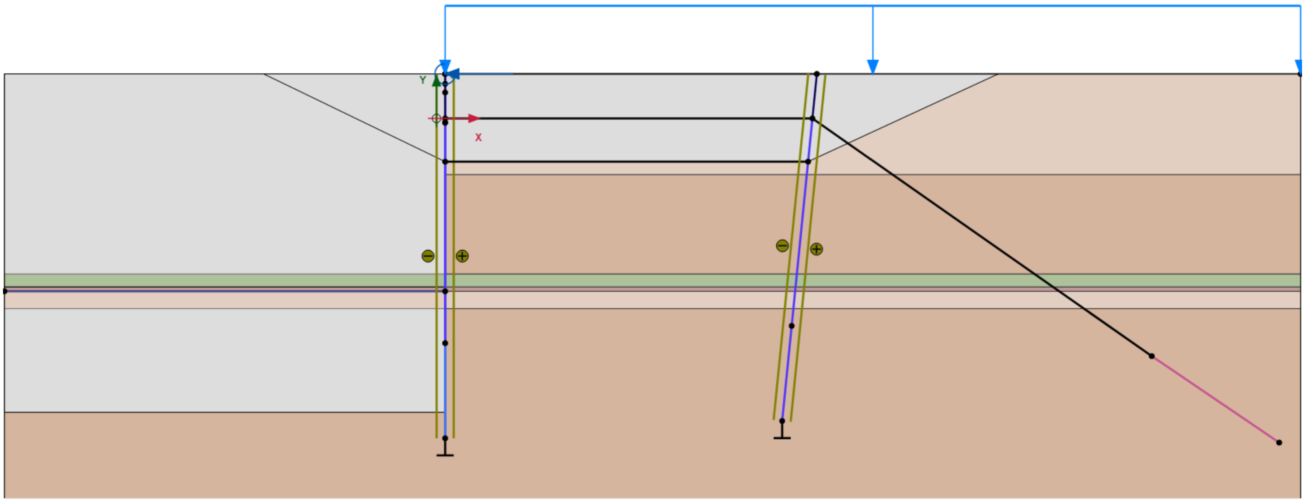

Figure 34 Cross-section of the cofferdam with 2 tie rods and a grout anchor.

To select the most suitable variant the cost and its score are compared. As shown in Figure 35, the deck on piles solution scored the lowest in the multi-criteria analysis and has the highest estimated costs. Therefore this variant will not be further developed. The combi wall with the relieving platform and the cofferdam are located relatively close to each other, both in score and cost.

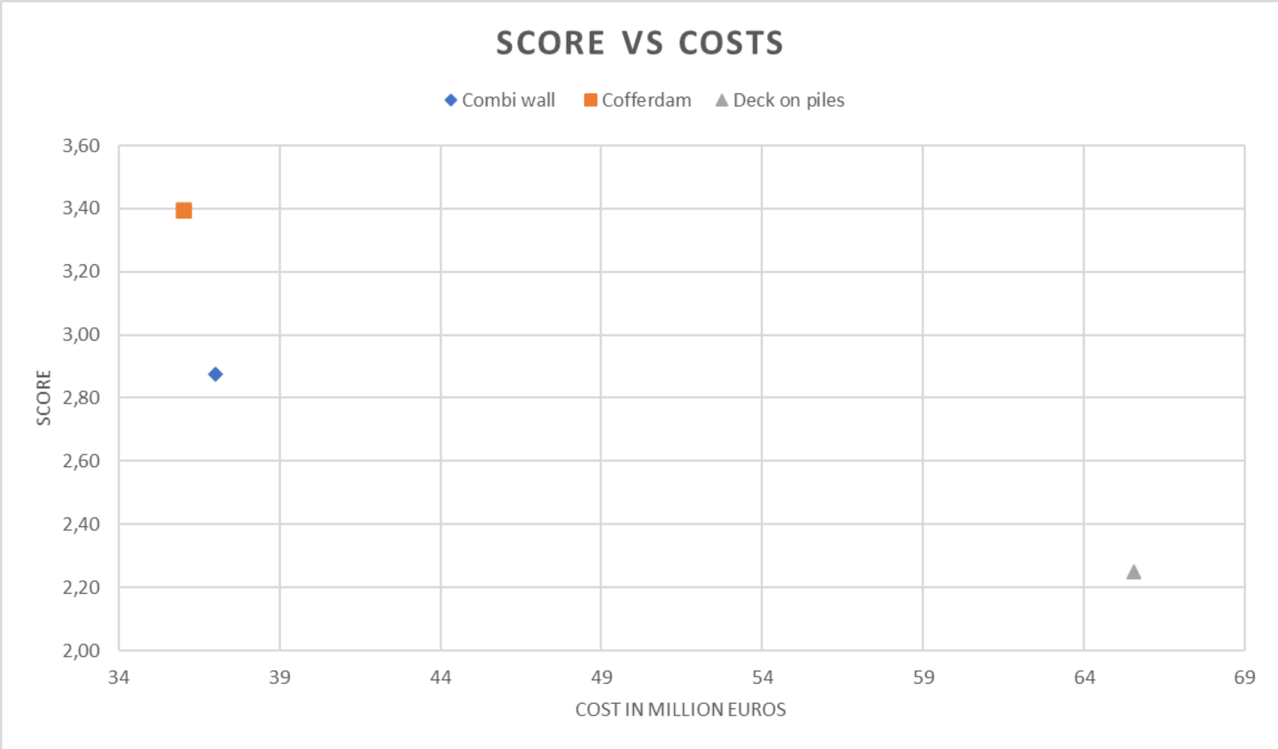

Figure 35 Score vs costs diagram of the three variants.

5.2.3.6 Selection of the quay wall type

Yet the cofferdam has the highest score and the lowest estimated costs. Based on the outcome of the analysis, it can be stated that the cofferdam with anchors, is the most suitable option. Therefore the cofferdam is further developed in the structural design phase.

5.3 Complementary technical systems

The quay wall is part of the port and must function in this system. There should be a working technical system.

The functional design holds the technical functioning of the quay wall as a system. The required technical systems to fulfil the functional requirements are the following:

- To guarantee the safety of the quay wall a rescue ladder system will be installed.
- To guarantee the accessibility of the vessels a fendering system is required.
- To maintain the rainwater discharge function a drainage system will be placed in the quay wall.
- To provide shore-based electricity a power system will be needed to fulfil this function.

5.3.1 Fendering

The fendering system is required for guiding the vessels. The fendering system will be placed in confirmation with the standard of Port of Amsterdam. The mooring arrangements will be constructed on a concrete wall at NAP - 2.0 m and NAP + 5.0 m.

5.3.2 Bollards

The distance between the bollards depends on the design vessel of the quay wall. Port of Amsterdam standardises this, and for the governing vessels, this requires a distance of 20 m between the bollards.

5.3.3 Ladders

The standard ladder system Port of Amsterdam uses has a distance of 30 m in between and is placed from LAT - 1.00 m to the top of the quay wall. Given the water levels, this means from the top of the quay wall until NAP - 2.40 m.

5.3.4 Shore-based electricity

A shore-based electricity system is required to reduce the emissions of the vessels in the port. Since the total length of the quay is 580 m and the design vessel length has a maximum of 300 m at least 3 connection points are required. This results in a cable network behind the quay wall.

5.3.5 Connections to existing dike and quay wall

The quay and port area have to connect to the surrounding structures. The connection on the right side of the quay wall is an existing quay wall, as shown in Figure 36. The top level of the existing quay wall is at NAP + 5.2 m.

Figure 36 The quay wall and terminal area with the connection interfaces indicated (Witteveen + Bos, 2021).

5.3.6 Concrete apron

The concrete apron functions as a protection against corrosion of the combi wall and the fendering and ladder can be connected to the combi wall by the concrete apron.

5.3.7 Pile cap

On top of the piles, a cap is placed to place the bollards and protect the piles. This pile cap can be made of reinforced concrete.

5.4 Overview of the selected alternative: cofferdam

The cross-section of the cofferdam quay wall type with the elements indicated is shown in Figure 37.

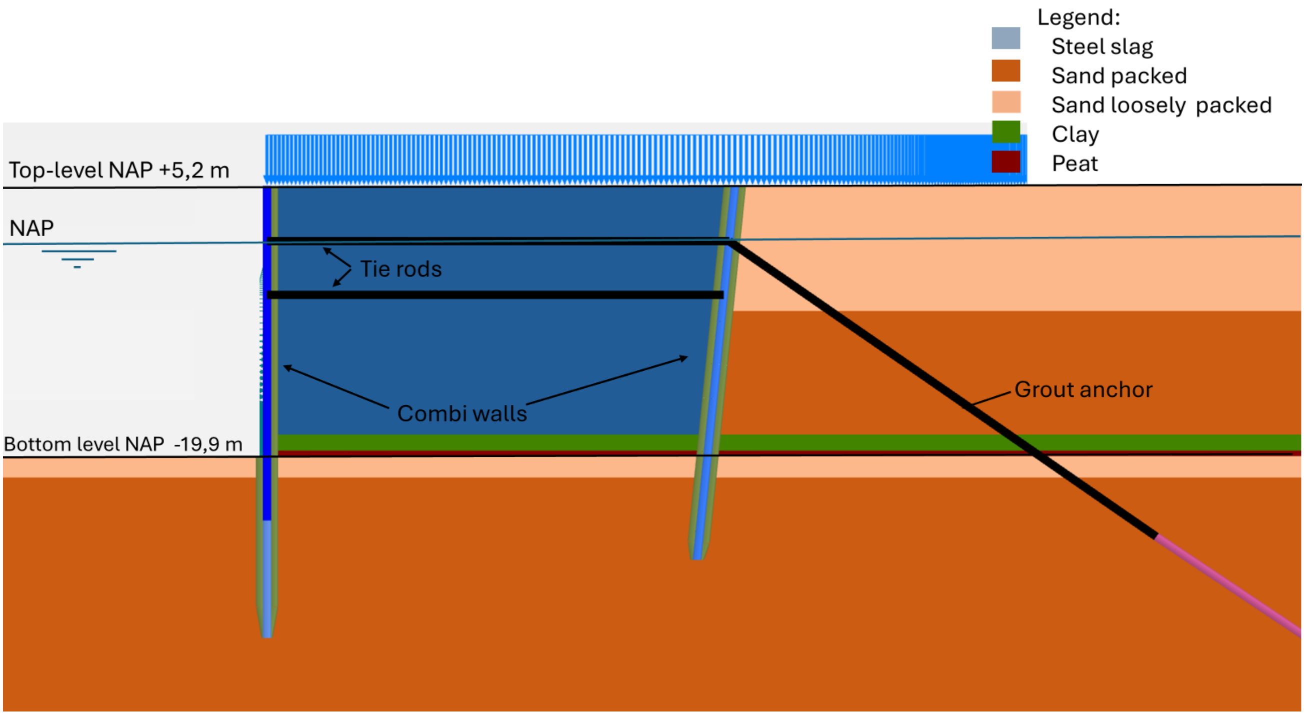

Figure 37 Cross section of the cofferdam with levels and soil layer indicated.

The variant consists of two combi walls with two horizontal tie rods in between. The grout anchor is added to the design for horizontal stability.

6 Structural design

The structural design loop corresponds to 6th step of the design approach. The structural design loop develops ideas to make sure the most suitable variant is constructible, stable, strong and stiff. These are the 4 main parts of the structural design loop. This includes a load inventory and pile driving analyses.

6.1 Detailed construction and use sequence

The construction and use sequence phases illustrate the steps that need to be taken to realise the quay wall. For the calculation of the governing loads, the construction sequences are used to calculate the governing load condition.

The detailed construction steps of the cofferdam are shown in Figure 38 and Figure 39.

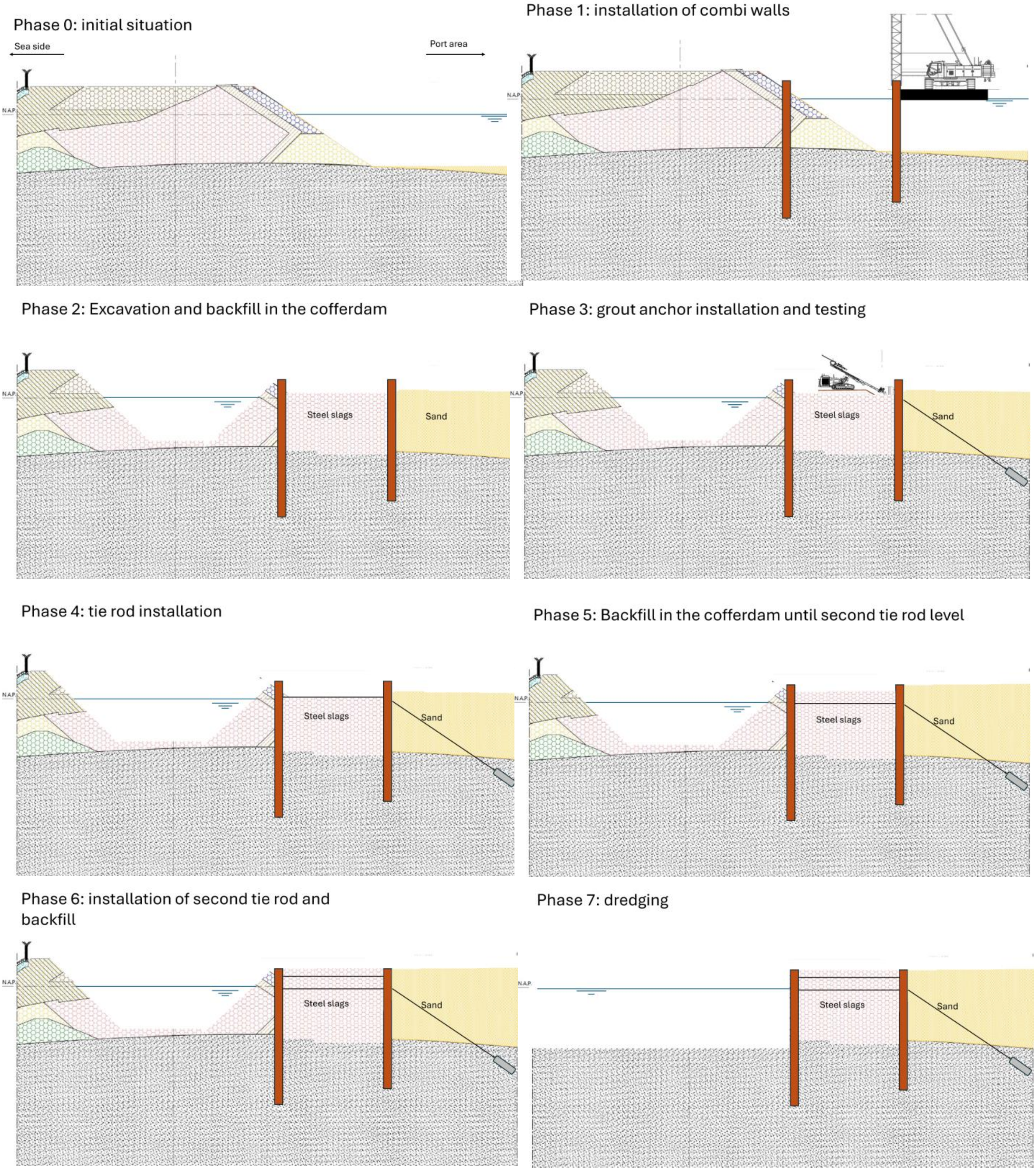

Figure 38 Detailed construction sequence of the cofferdam (part 1).

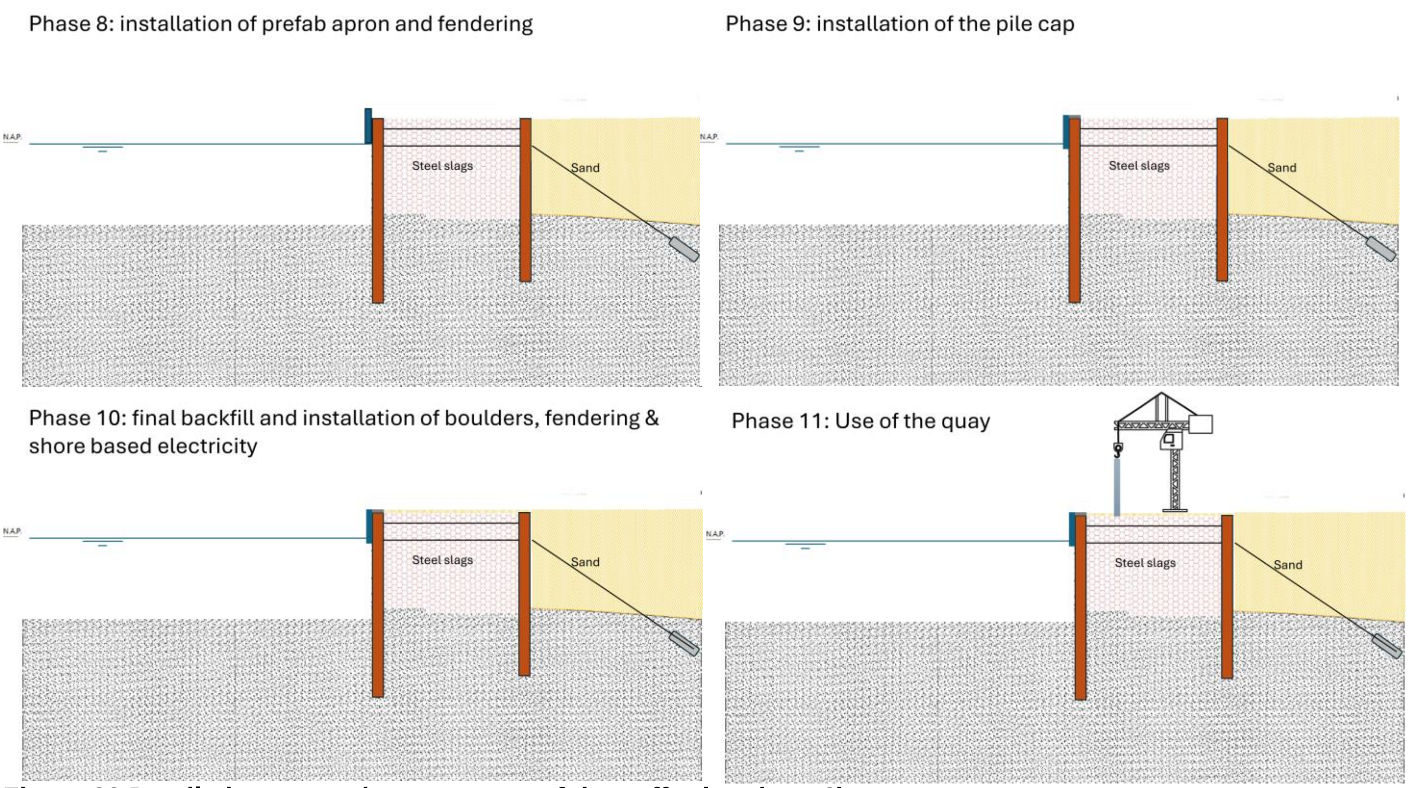

Figure 39 Detailed construction sequence of the cofferdam (part 2)

6.1.1 Determination of failure mechanisms

A structure fails when it does not fulfil its main functions, earth-retaining and load-bearing. The verification is carried out for the ULS (Ultimate Limit State) and SLS (Serviceability Limit State).

The failure mechanisms(ULS) of the quay wall are listed as follows:

- Insufficient stability of combi walls.
    - Sliding
    - Toe resistance too small
    - Structural failure: tilting, collapse.
- Failure of a tension member
    - Tie rod breaks
    - Grout body fails
- Failure of the back wall
    - Supporting earth pressure inadequate
    - Strength inadequate
    - Kranz stability inadequate
- Connection failure
    - Steel fails
    - Structural element fails
- Total stability insufficient
    - Piping
    - Sliding circles

6.1.2 Determination of governing load conditions

The governing load conditions depend on the failure mechanism. In Figure 40 a basic sketch of the loads on a retaining structure is provided.

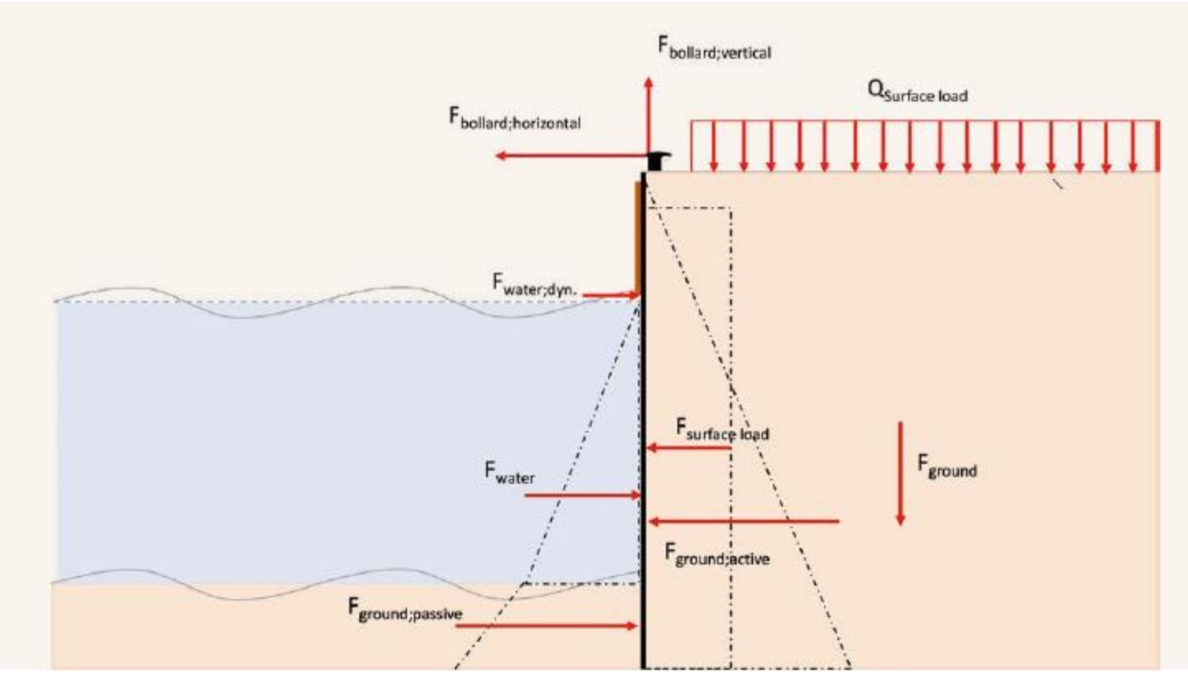

Figure 40 Overview of load on a quay wall (Bonte, 2007).

The possible failure mechanisms of a wall with a tension member are shown in Figure 41.

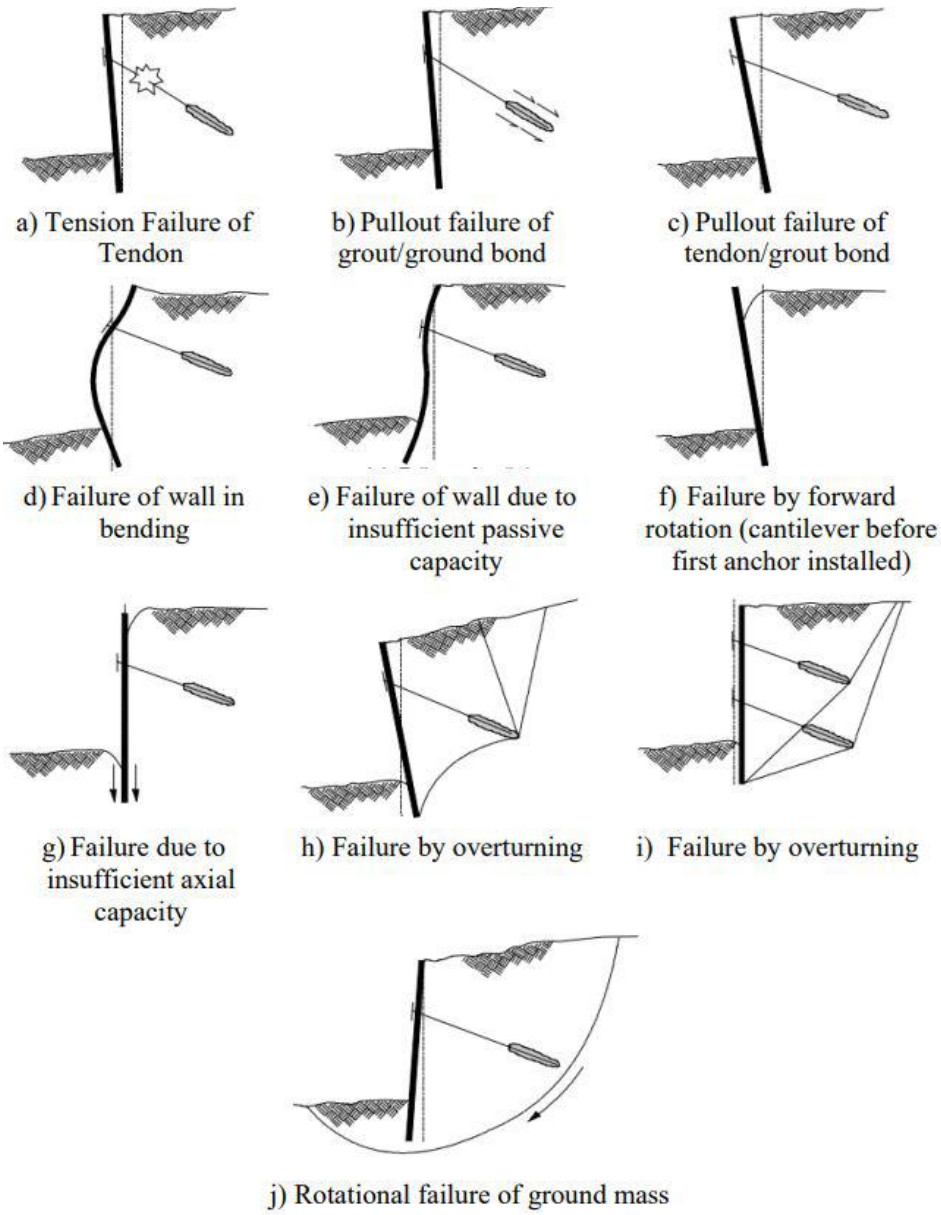

Figure 41 Failure mechanisms of a wall with a tension member (de Gijt & Broeken, 2013).

6.1.3 Surface load conditions

The activities at the port facilities will govern the load on top of the quay wall. Therefore an investigation was made to determine the governing loads on the topside of the quay wall. An example port setup is shown in Figure 42.

Figure 42 Example of a pre-assembly wind turbine port setup (Offshore Magazine, 2023)

For the transportation of the wind turbine parts SPMTs (Self Propelled Modular Transporter) are used. The maximum force per axle of an SPMT is 500 kN/axle which is 150 kN/m^2 (Mammoet, 2024). The wind turbine consists of blades, a tower section, and a nacelle. The parts are stored at specific locations at the port.

The towers and nacelle provided the largest ground pressure. Several wind turbine suppliers show that this requires a ground resistance of up to 600 kN/m^2. The towers are placed vertically at fixed locations. It could be advised to separately design the surface areas which need to carry the towers and nacelle since taking this as a governing situation would result in over-designing the storage area. Another option is to take measures to spread the loads by timber mats or steel plates.

The design surface load is 200 kN/m^2 at the port terrain and on top of the quay wall. The ULS load condition is 1.1 times the SLS load conditions according to the NEN9997.

6.1.4 Bollard load conditions

The bollard load causes a horizontal load on the quay wall caused by the vessel moored. Since wind, currents and waves are acting on the vessel, a tensile force will be transferred to the quay wall by the mooring lines. This force is dependent on water displacement due to the vessel. The weight of the water displacement can be calculated using the following formula:

$$G = L * B * D * C_B * \gamma_w$$

In which:
G: weight of the displaced water [kN];
L: Design vessel length [m];
B: Design vessel width [m];
D: Design vessel draught [m];
C_B: Design vessel block coefficient [-];
γ_w: Water density [kN/m^3];

The Seaway Strashnov has a load capacity of 8.500 tons and a displacement of 77.210 m^3 (Seaway 7, 2019). The bollard force based on the water displacement is shown in Table 11.

Table 11 Bollard force based on water displacement (Port of Amsterdam, 2015)

Water displacement [t]	Bollard force [kN]
76.050 - 95.940	1500

This requires a bollard resistance of 1.500 kN based on the standardized sheet used by the Port of Amsterdam. The complete overview is shown in Appendix B. The bollard force acts on top of the bollard, which has a height of 0.3 m. This will result in a moment acting on the top of the quay wall. The bollards are placed each 15 m so that this load will be spread over this length. And thus the load can be divided by 15 m. This results in the values shown in Table 12.

Table 12 The forces at SLS on the quay wall are initiated by the maximum bollard load.

Moment (kN/m/m)	Point load (70 kN/m)
35	70

For the ULS calculations, these loads are multiplied by a factor of 1.1.

6.1.5 Determination of the retaining height

The retaining height of the quay wall is 24.6 m. In addition, for the ULS calculation, the depth in front of the quay is lower by 0.5 m.

6.1.6 Determination of the governing water level

Governing water level varies for the construction phase. For each phase, the most conservative water level is selected, this holds the maximum load acting on the structure. It is assumed that water is present during construction for the calculation. After construction, at the dredging phase, the water level at the passive side is assumed to be at datum level so at NAP - 19.9 m. On the active side, the groundwater table remains at NAP. For the next phases, the water level is set to LAT which is NAP - 0.95 m.

6.1.7 Determination of the cofferdam width

The distance between the front and back combined wall is determined by the volume of the steel slag. The aim is to store 95% of the available steel slag between the combined walls. This can be achieved when the distance between the wall is 40 m.

6.2 Software selection and model validation

6.2.1 Software selection

The software of Plaxis is used to model the quay wall. This software uses the finite element method which is often used for analysing deformations and stability. The dimensions of the structural elements are obtained by an iterative process. The PLAXIS software supports various models to model soil and structural behaviour. The soil model is set to the hardening soil model. This is a second-order model which can be used to simulate the behaviour of sand and gravel material. The hardening soil model is normally used for the verification of the quay walls.

The soil profile varies over the length of the quay. The results of the soil tests are shown in Figure 43. It is shown, that at the length of about 350 m, the steel slags soil layer changes into sand layers.

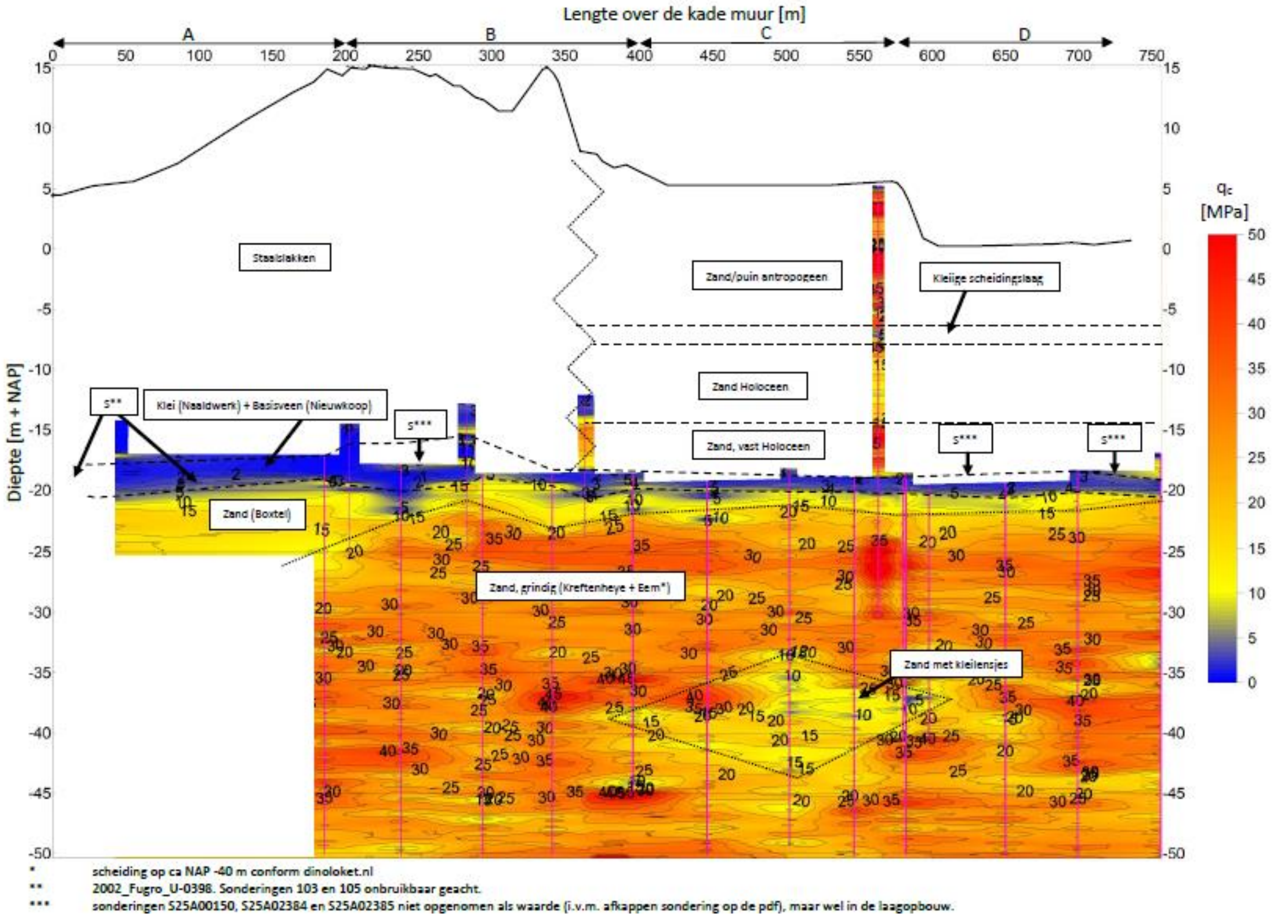

Figure 43 The soil parameters over the length of the quay wall (Fugro, 2021).

Due to the variation in the soil profile, the design quay wall must satisfy the requirements over the length of the quay wall. To ensure the structure is stable in the varying soil the structure will be modelled for two soil profiles.

The soil layer profile is shown in Table 13. The general soil parameter is based on the NEN 9997-1 table 2. b. For all soil layers:

- The void ratio parameters are set to e_{init} 0.5 [-] and n_{init} 0.33 [-].
- The reference stress p^{ref} = 100 kPa.
- The E_{oed}^{ref} is E_{50}^{ref} for sand and for clay and peat E_{50}^{ref} / 2.
- The soil structure interaction parameter R_{inter} is set to 0.8 based on the CUR 211.
- For the steel slag soil a granular material is considered with a lower R_{inter}: 0.10.

Table 13 Soil layer parameters for setup 1 for the PLAXIS 2D model.

Soil type	Depth [NAP m]	Parameters
Steel slags*	+5.15 - -18 m	γ_{sat} 26 kN/m^3 γ_{unsat} 23 kN/m^3 c'_{reft} 7.7 kN/m^2 φ' 40.1 [º] E_{50}^{ref} 5.000 kN/m^2
Clay	-18 - -19.5	γ_{sat} 17 kN/m^3 γ_{unsat} 17 kN/m^3 c'_{ref} 5 kN/m^2 φ' 22.5 [º] E_{50}^{ref} 75.000 kN/m^2
Peat	-19.5 - -20	γ_{sat} 13 kN/m^3 γ_{unsat} 13 kN/m^3 c'_{reft} 5 kN/m^2 φ' 22.5 [º] E_{50}^{ref} 1.000 kN/m^2
Sand, loosely packed	-20 - -22	γ_{sat} 17 kN/m^3 γ_{unsat} 19 kN/m^3 c'_{reft} 0 kN/m^2 φ' 30 [º] E_{50}^{ref} 15.000 kN/m^2
Sand, Moderate to densely packed	-22 - -44	γ_{sat} 19 kN/m^3 γ_{unsat} 21 kN/m^3 c'_{reft} 0 kN/m^2 φ' 35 [º] E_{50}^{ref} 60.000 kN/m^2

*Fugro used for the inner part of the dike c' =7.25 and φ' = 35 kN/m^3 because it was mixed with clay locally.

The combi wall with relieving platform and the cofferdam variant are analysed by using Plaxis. A setup for the cofferdam is shown in Figure 44.

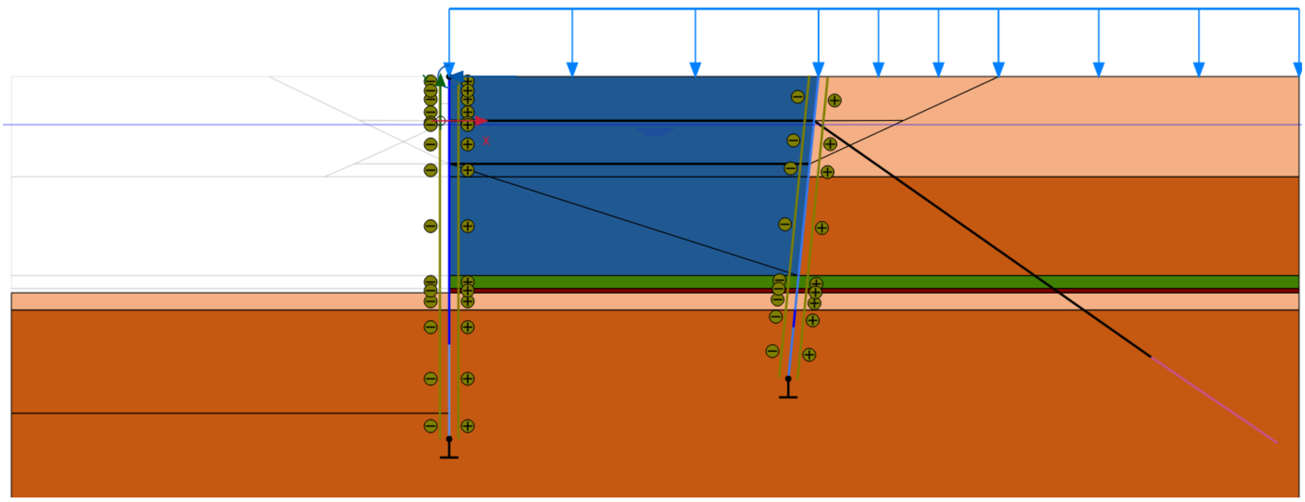

Figure 44 The setup for the cofferdam in PLAXIS 2D.

The second setup for the PLAXIS 2D model is shown in Table 14.

Table 14 The soil layers with parameters for setup 2 for the PLAXIS 2D model.

Soil type	Depth [NAP m]	Parameters
Sand, loosely packed	+5.15 - -8	γ_{sat} 17 kN/m^3 γ_{unsat} 19 kN/m^3 c'_{reft} 0 kN/m^2 φ' 30 [º] E_{50}^{ref} 15.000 kN/m^2
Sand, Moderate to densely packed	-8 - -1	γ_{sat} 19 kN/m^3 γ_{unsat} 21 kN/m^3 c'_{reft} 0 kN/m^2 φ' 35 [º] E_{50}^{ref} 60.000 kN/m^2
Clay	-18 - -19.5	γ_{sat} 17 kN/m^3 γ_{unsat} 17 kN/m^3 c'_{ref} 5 kN/m^2 φ' 22.5 [º] E_{50}^{ref} 5.000 kN/m^2
Peat	-19.5 - -20	γ_{sat} 13 kN/m^3 γ_{unsat} 13 kN/m^3 c'_{reft} 5 kN/m^2 φ' 22.5 [º] E_{50}^{ref} 1.000 kN/m^2
Sand, loosely packed	-20 - -22	γ_{sat} 17 kN/m^3 γ_{unsat} 19 kN/m^3 c'_{reft} 0 kN/m^2 φ' 30 [º] E_{50}^{ref} 15.000 kN/m^2
Sand, Moderate to densely packed	-22 - -44	γ_{sat} 19 kN/m^3 γ_{unsat} 21 kN/m^3 c'_{reft} 0 kN/m^2 φ' 35 [º] E_{50}^{ref} 60.000 kN/m^2

In PLAXIS 2D a mesh is generated. A medium mesh provides sufficiently accurate results and the computation time is acceptable. The mesh is shown in Figure 45.

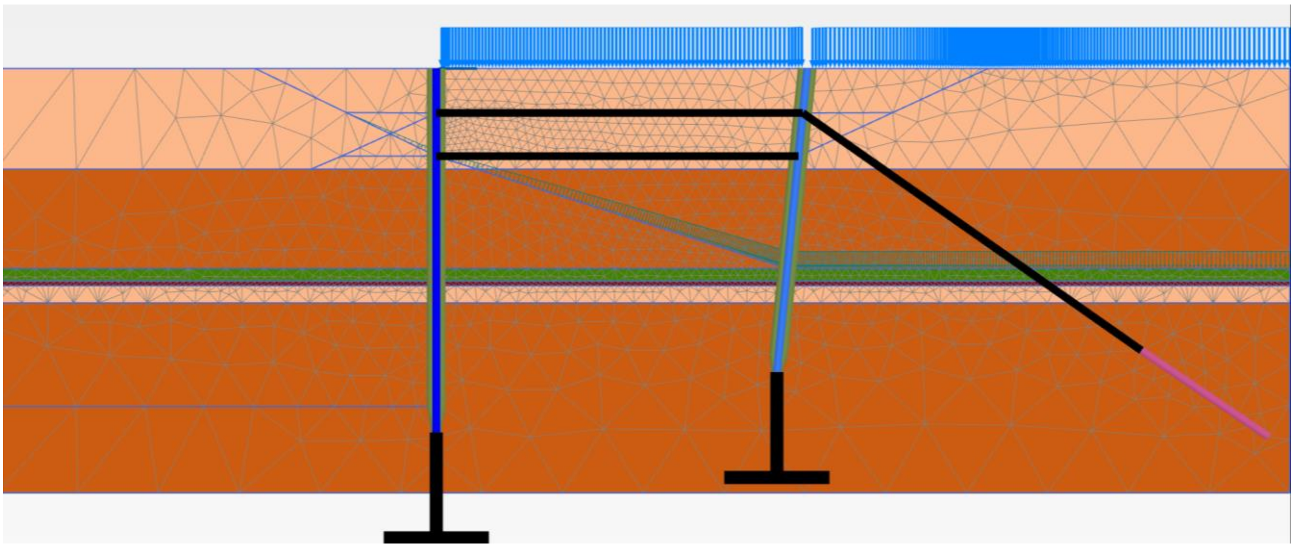

Figure 45 The generated mesh for a cofferdam with grout anchorage.

The construction steps as described in Figure 38 and Figure 39 are modelled in the PLAXIS 2D program. The phases can be seen in Appendix F: Stages in PLAXIS.

6.2.2 Model validation

To ensure the model provides the correct results several checks are performed. The calculations and images are shown in Appendix . First, for several points in the model, the soil stresses are checked by hand calculations. The values that the model provides are similar to the results of the hand calculations. This underlines the model is valid.
Secondly, the model is validated by increasing the bollards' force by 50%. An increase in bollard load should cause a reduction in forces in the tie rods and a reduction of deformation of the front combi wall. The model acts as expected and shows a reduction of deformation of the combi wall of about 10 %. The forces in the tie rods are reduced by 10%.
These checks confirm that the Plaxis model accurately simulates the behaviour of the system. This PLAXIS 2D model provides a reliable basis for further analysis and design.

6.3 Stability checks

6.3.1 Determination of the tie rod levels

The height of the tie rods is an important parameter for the stability of the cofferdam. The tie rod levels have a major impact on the deformation of the combi walls. By iterative calculation, the height was determined. One tie rod is at NAP + 1 m and the other one is at NAP - 5 m. This results in minimal deformation for all construction stages and the lowest forces in the tie rods.

6.3.2 Determination of combi wall length

The length of the combi wall is important for the stability of the cofferdam. For a cofferdam, the stability depends on the stability of each combi wall. From the PLAXIS 2D model the stability is part of the calculation. If the setup is unstable this will result in executive deformations and collapse. By iterative calculation, the pile length was determined so that the results show sufficient stability. The front wall piles have a length of 42 m and the piles of the back wall are 35 m. The sheet pile length for the front wall is 31 m and for the back wall 29 m. This configuration provides sufficient stability for all phases.

6.3.3 Determination of safety against heave

Heave occurs when water level differences are present. As governing situation phases 1,2 & 3 and are considered, because, in these phases, the maximum water level differences will occur. In Figure 46 a basic sketch is provided to illustrate the water flow direction.

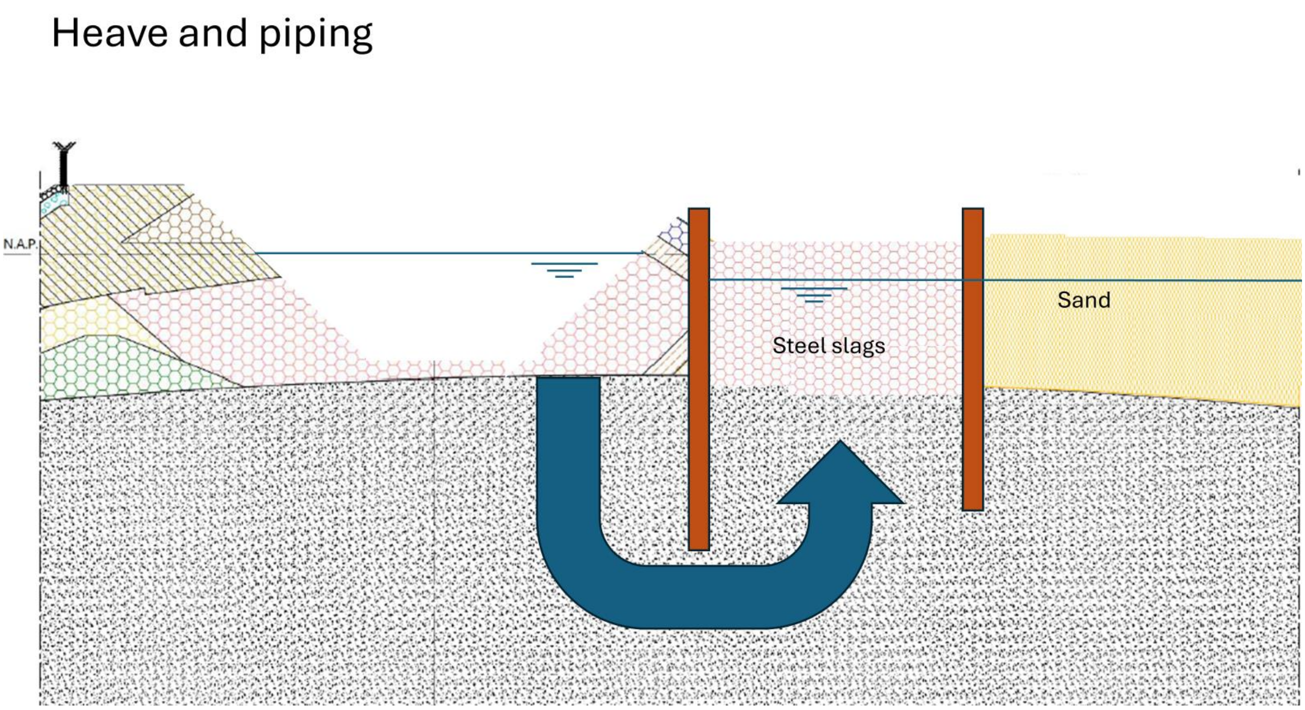

Figure 46 The failure mechanism of heave at the cofferdam.

It is assumed that the maximum water level difference during these phases is 5 m since this depends on the sea and rain conditions. To account for heave the following equilibrium must be satisfied, according to CUR 211 (de Gijt & Broeken, 2013).

$$G_{stb;d} \geq V_{dst;d}$$

In which:
$V_{dst;d} = \gamma_f * V_{dst}$ and $G_{stb;d} = \gamma_G * G_{stb}$

And
$V_{dst;d}$ is the design value of the upward-directed resulting water pressure
$G_{stb;d}$ is the design value of the effective weight of the upper layers
V_{dst} is the characteristic value of the upward-directed resulting water pressure
G_{stb} is the characteristic value of the effective weight of the upper layers
γ_f is the partial factor for construction phase 1.3 and design working life 1.5
γ_G is the partial factor for the self-weight for all phases 0.9

For the calculation, it is assumed that the clay layer of 1.5 m thickness is present over the total length of the quay. In addition, the sand layer provides and steel slag layer that provides weight and resistance. This criterion is satisfied since, during construction $G_{stb;d} = 87$ kN/m^2 and $V_{dst;d} = (10 * 5) * 1.3 = 65$ kN/m^2. The input parameters are shown in Table 15.

Table 15 The parameters of soil layers used for the calculation of the heave resistance.

Layer	Thickness(m)	Density(kN/m^3)
Water level difference	5	50
Clay	1,5	17
Sand	2	19
Steel slag	1	26

During the design lifetime $G_{stb;d} = 580$ kN/m^2 and $V_{dst;d} = 50$ kN/m^2 and thus this criterion is satisfied. Due to the weight of the steel slag fill material the resistance increases significantly. The maximum water level difference that the setup provides sufficient resistance to is approximately 10 m.

6.3.4 Piping

Piping can occur when differences in water levels show up during the construction of the quay wall. The failure mechanism of piping can lead to total failure of the structure and must therefore be taken into account. Often this phenomenon results in significant erosion at the toe of the combi wall. The criteria stated in the CUR 166 Part 1 section 3.3.11 are used to account for piping. (CUR Bouw & Infra, 2018). The parameters used in the formula are shown in Figure 47. The following equation must be satisfied:

$$L_1 + L_2 \geq \gamma_{piping} * C_L * \Delta H$$

In which:
L_1 is the length of the wall from the water level [m]
L_2 is the embedded length of the wall at the seaside [m]
ΔH is the water level difference [m]
γ_{piping} is the partial factor for piping 1 for CUR class I
C_L is the seepage factor of Lane.

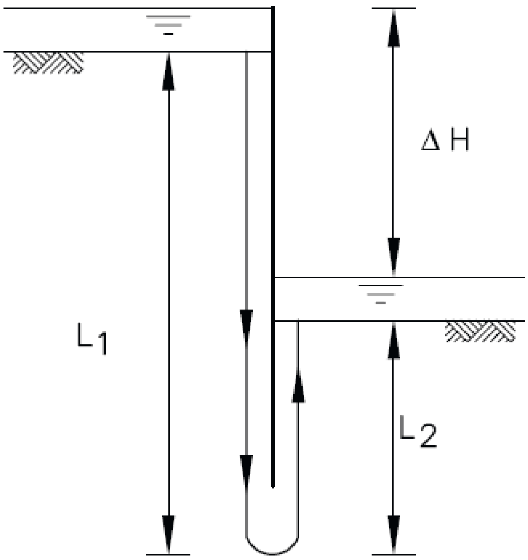

Figure 47 The parameters indicated at the quay wall to determine the safety factor against piping.

Since various soil layers are present at the construction location. The governing situation is the cross-section at which the most permeable layers are present. The factor of Lane for different soil layers is shown in Table 16. For steel slags a C_L of 3.5 is used, which is similar to medium-sized gravel.

Table 16 Lanes' factor for different soil layers

Soil type	$Lane\ factor\ C_L$
Steel slag	3.5
Clay	1.8
Sand	6.0

The water level difference (ΔH) will have a 3 m maximum during the construction. Based on the calculations, a unity check for piping is found: 0.76, which satisfies the requirement.

6.3.5 Verification of soil stability

The failure mechanisms related to soil stability are, as shown in Figure 41, failure due to pullout of grout body (b,c), overturning (h, i), and rotational failure of groundmass (j). The soil-related stability checks are implicitly performed in the Plaxis model. The finite element analysis program PLAXIS 2D results prove that no failure will occur due to these failure mechanisms.

6.4 Strength checks

6.4.1 Verification of tension elements

Failure of the grout anchor or tie rods can result in excessive deformations and loss of overall stability of the quay structure. As a result, the forces on the combi wall will increase and it can fail.

Tie rod 1 (upper)

Based on the PLAXIS 2D model for SLS and ULS the maximum forces are determined to verify the horizontal tie rods. The maximum loads occur when the maximum terrain and bollard load are applied (phase: use of the quay). The tensile resistance of the various anchors is provided in tables by the production company (Schoeder anker, 2024). As a possible anchor ASDO335, M120/95 was selected, which has a diameter of 95 mm. The anchor provides a yield strength of 2516 kN and an ultimate strength of 3615 kN as shown in Table 17. Because the unity checks are smaller than 1, this tie rod fulfils the requirements.

Table 17 The verification of the upper tie rod (M120/95).

Stage	Force (kN)	Resistance(kN)	Unity check
SLS	1200	2516	0.48
ULS	1659	3615	0.66

Tie rod 2 (lower)

The unity checks for the lower tie rod are shown in Table 17. This tie rod satisfies the requirements.

Table 18 The verification of the lower tie rod (M120/95).

Stage	Force (kN)	Resistance(kN)	Unity check
SLS	1513	2516	0.42
ULS	2243	3615	0.62

Grout anchor and grout body

The following check is provided in the CUR 166, for the verification of the grout anchors, according to the NEN 9997-1 (CUR Bouw & Infra, 2018).

$$P_d \leq R_{a;d}$$

Where,

$$R_{a,d} = \frac{R_{a;k}}{\gamma_a}$$

$$R_{a;k} = \frac{R_{a;min}}{ξ} \; when \; n \leq 3$$

$$R_{a;k} = \frac{R_{a;avg}}{ξ} \; when \; n \leq 3$$

In which,

P_d is the anchor calculation value [kN]
$R_{a,d}$ is the resistance force of the grout body [kN]
$R_{a;k}$ is a characteristic resistance force [kN]
$R_{a;min}$ is minimal resistance force of the ground body based on tests [kN]
$R_{a;avg}$ is the average resistance of the grout body based on tests [kN]
$ξ$ is the reduction factor [-]
γ_a the material reduction factor is 1.35 or 1.2 when all anchors are tested.

The reduction factor($ξ$) depends on the number of anchors that will be tested and the number of anchors that are estimated to spread the forces. Since two anchors will be placed in one foundation pile, it is assumed that at least two anchor spread forces. The number of piles that will be tested is assumed to be larger than 10. This results in a reduction factor ($ξ$) of 1,25 [-].

To calculate $R_{a;min}$ the following formula is used:

$$R_{a;min} = f_{k;rep} * L_A$$

In which,

$R_{a;min}$ is the minimal resistance force [kN]
L_A is the length of the grout body
$f_{k;rep}$ is the representative value of the resistance force as a function of q_c [kN/m]
q_c is the average cone resistance in the soil layer of the grout body [kPa]

The grout body starts at NAP- 27.5 m and ends at NAP -35 m and has a length of 15 m. For cone resistance values larger than 20 Mpa, which is governing based on the CPT results, the representative resistance force ($f_{k;rep}$) is 170 kN/m. The grout body needs a hardening time of at least 7 days (Williams form engineering corp., 2024). As anchor type the Titan 130-60 is selected. The dimensions of the grout body and anchor are shown in Figure 48.

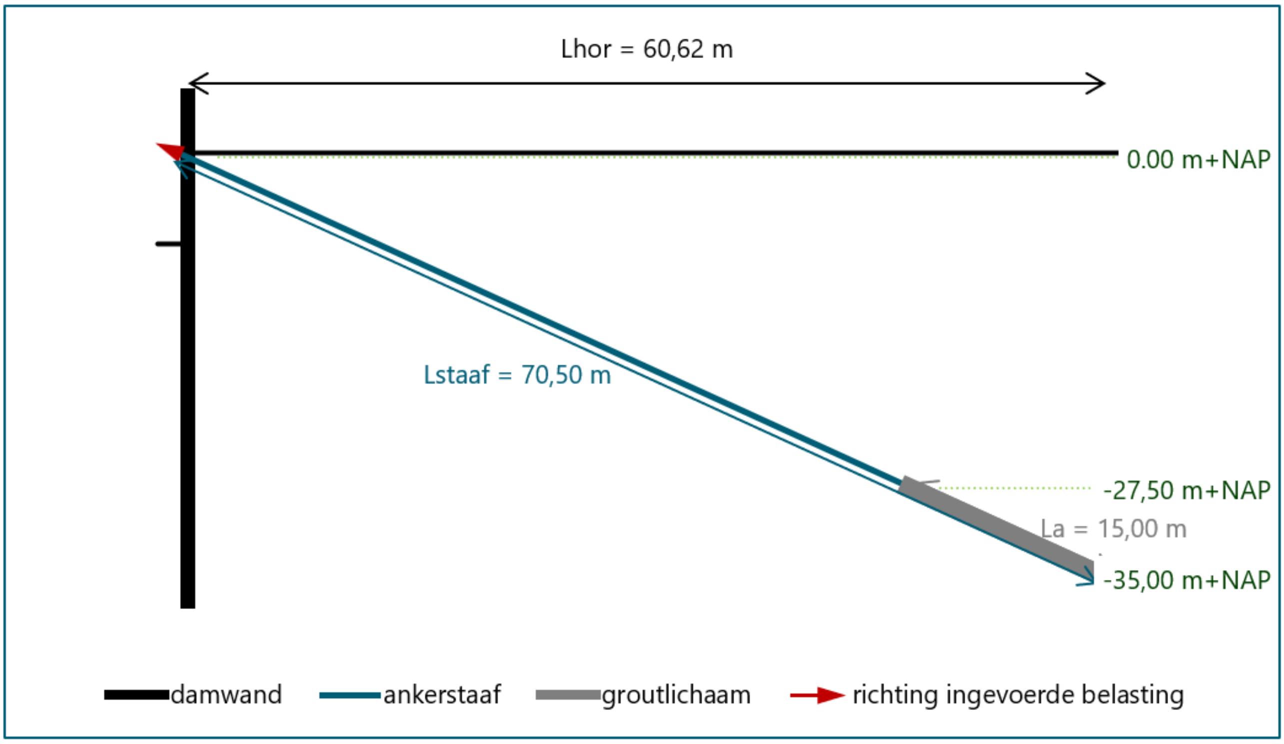

Figure 48 Sketch of the grout anchor and grout body.

This results in a unity check for the anchor and the grout body. It is calculated that one anchor can fail and the anchor next to this anchor can bear this extra load. The unity checks for the governing situations are shown in Table 19. Both unity checks are smaller than the one, meaning the selected materials are calculated to be safe.

Table 19 The unity checks for the grout anchor and grout body.

Verification element	Unity check[-]
Grout body	0.53
Anchor	0.93

6.4.2 Combi wall strength verification

The combi verification is needed since the combi wall is an important element of the design. The failure mechanisms related to the combi wall are a failure due to bending, insufficient passive capacity, forward rotation, and axial capacity. As shown in Figure 41, these are the failure mechanisms d, e, f, g. The maximum forces occur during the use of the quay wall.

The sheet piles are not driven to the same depth as the tubes. For this reason, it is assumed that the secondary elements (sheet piles) do not contribute to the resistance to perform the strength check of the combi wall.
For the verification of the combi wall, the CUR 211 is used. The verification process is based on the out-of-roundness, ovalization, and bending moment. Due to the marine environment, corrosion is taken into account.

6.4.2.1 Corrosion reduction determination

Corrosion leads to a strength and stiffness reduction of the steel elements. Too much corrosion will lead to the failure of the quay wall. To account for this failure the rate of corrosion is calculated. The rate of corrosion depends on several factors, however, it is hard to predict the corrosion rate for the lifetime of the structure. Often cathodic protection is used to protect the steel against corrosion. According to the CUR 211, an outer diameter reduction of 2 mm is used. For the wall thickness, a reduction of 1 mm is used. This reduction is considered in the strength resistance calculation of the combi wall at ULS.

6.4.2.2 Ovalization

The piles of the combi walls can, due to the applied loads, deform from their original shape. This out-of-roundness or ovalisation has an impact on the resistance of the pile. In Figure 49, a sketch is shown with the parameters used in calculations. The out-of-roundness tolerances are based on the fabrication classes.

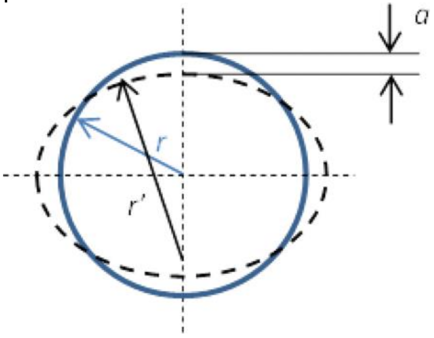

Figure 49 Change of radius with parameters a and r indicated.

6.4.2.3 Determination of bending moment

The maximum bending moments based on the PLAXIS 2D calculation are shown in Table 20. The resistance of the combi wall must be larger than the applied moments due to the loads.

Table 20 Maximum moment of the combi walls.

	Front wall moment [kNm]	Back wall moment [kNm]
ULS	7053	3677
SLS	6153	2867

To calculate the moment resistance of a combi wall the stiffness of the piles and the sheet pile are combined. The formula to calculate this is:

$$I_{combi} = \frac{I_{pile} + I_{n,sheet}}{L}$$

Where:

I_{combi} is the moment of inertia of the combi wall [mm^4/m]
I_{pile} is the moment of inertia of the pile [mm^4]
$I_{n,sheet}$ is the moment of inertia of the sheet pile [mm^4]
L is the distance between the piles

The active soil pressure is calculated, based on the steel slag soil in between the cofferdam. This gives an active stress of 115 kN/m^2. The maximum moment is applied at a depth of NAP -16 m for the front wall. The back wall application point is at NAP - 5 m. The resistance of the piles depends on the material inside the pile. The piles will be filled with steel slag material, providing more resistance against local buckling than sand, because of the higher density, however, the Eurocode provided only code for sand-filled or empty piles. Therefore the piles are modelled as sand-filled, which is a conservative approach.

6.4.2.4 Dimensions of the combi walls

The governing situation for the combi walls is during its user phase (step 11 of the construction sequence). The maximum force at ULS is applied on top of the quay.

Front wall

The unity check with corrosion is 0.93, which means the combi wall can resist the applied force at ULS. The combi wall verification includes some safety margin since the Eurocode only provides code for sand-filled piles. In Appendix D: Forces and deformations the results are shown.
Currently, only empty and sand-filled piles can be calculated by the Eurocode. However, considering the project location's soil conditions, the piles could be filled with steel slags. This material will provide extra stiffness and buckling resistance compared to sand due to the higher density. Therefore calculating the piles as sand-filled is a conservative approach. The combi wall profile is shown in Figure 50.

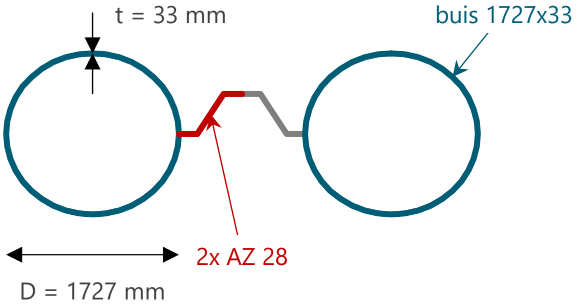

Figure 50 The combi wall front wall (seaside).

The length of the piles in the front wall is 43 m, this is to provide sufficient stability. The sheet pile wall will be 35 m.

Back wall
The unity check for the back wall is 0.75 for the ULS calculation. Due to the smaller forces acting on the back side of the cofferdam a smaller thickness is used, as shown in Figure 51.

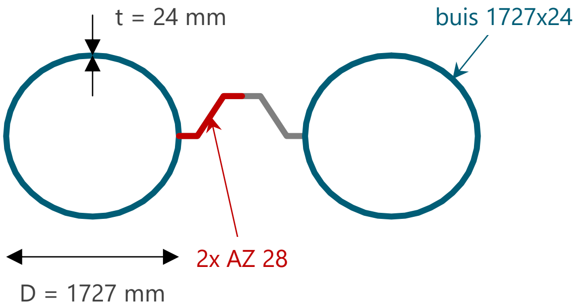

Figure 51 The combi wall profile at the back side of the cofferdam.

The length of the piles of the back wall is 36 m and the sheet pile wall 30 m. For ease of construction, the same diameter of the piles is chosen. The sheet piles profile will be similar for both combi walls. Various diameters could result in various required lengths and thicknesses of the tie rods. The calculations can be seen in Appendix D.

6.5 Stiffness and deformation checks

The accepted deformation is max 1% of the retaining height (CUR Bouw & Infra, 2018). This means a total maximum deformation of 0.25 m at SLS is allowed. After the dredging phase, a maximum of 0.1 m of deformation is allowed. The deformations at the ULS can be governing if they lead to total failure of the structure. For deformations at the SLS stage, the accepted deformation is max 1% of the retaining height (CUR Bouw & Infra, 2018).

The maximum deformation at the ULS stage of the structure is 0.25 m, which does not lead to total failure. The maximum deformation at the SLS stage is 0.16 m which is acceptable. Due to dredging in front of the quay wall, the structure deforms a maximum of 0.06 m.

Based on experience the deformations can vary due to the load variation. To account for this behavior two load cases are chosen. The first one is the terrain load acting on the terrain behind the back wall of the cofferdam. The second load condition is a spread load over the complete structure and terrain. The second load conditions results in the largest deformations and forces.

For the tie rods the anchor ASDO335, M120/95 is chosen. This material provides sufficient resistance and stiffness. The prestressing force is set to 200 kN a larger tension creates too large deformations during the prestressing stage.

In conclusion, it can be stated that the quay wall acts within the boundary provided by the Eurocode.

6.6 Determination of the pile and sheet pile installation method

The quay wall design includes many piles that must be installed into the soil layer. About 480 piles are required and 480 sheet pile will be needed for the realization of this design. This states the importance of carefully investigating the additional measures that will be taken. The installation contractor has to select the appropriate installation method to advance the piled foundation to its required depth successfully. Accurate prediction allows for timely and cost-effective installation work. As the file geometry is calculated previously, correct assumptions must be made regarding the hammer and soil configuration.

At the project location, three piles were drilled to analyse to feasibility of a piled foundation. In Figure 52, an overview is provided with the dimensions of the piles and sheets. For the pile driving analysis (PDA), piles were equipped with strain and acceleration sensors.

datum	buis / plank	locatie	activiteit	voorge-boord?	PDA	VDA	profiel-lengte [m]	profieltype	penetratie start [m]	penetratie eind [m]
29-feb-24	1	P1	voorpoten D-100	N	X		18,0	buis D x t = 1620 x 25 mm	0,00	13,25
01-mrt-24	2	P2	voorpoten D-100	N	X				0,00	5,75
01-mrt-24	3	P3	voorpoten D-100	N	X				0,00	5,50
04-mrt-24	3	P3	afheien S-280	N	X		18,0	buis D x t = 1620 x 25 mm	6,00	15,00
04-mrt-24	2	P2	afheien S-280	N	X				6,00	14,00
04-mrt-24	1	P1	afheien S-280	N	X				13,25	13,75
05-mrt-24		SH4	voorboren						0,00	13,00
05-mrt-24		SH5	voorboren						0,00	13,00
06-mrt-24	A	SH4	intrillen NAAST slot	J		X	21,25	drieling plank PU28	0,00	15,50
06-mrt-24	A	SH5	intrillen IN slot	J		X			0,00	18,75
06-mrt-24	A	SH6	intrillen NAAST slot	N		X			0,00	1,00

Figure 52 Overview of the piles and sheet for the test setup (Allnamics Geotechnical & Pile Testing Experts, 2024).

The second pile that was driven into the soil provided additional PDA results, as shown in Figure 53.

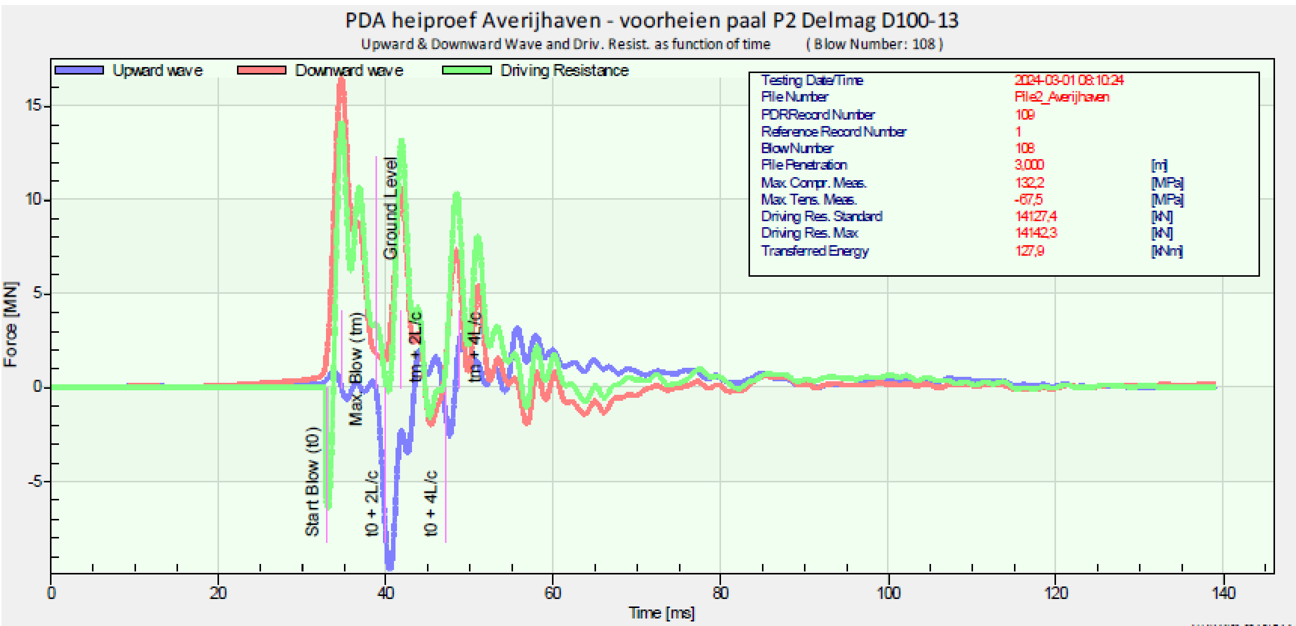

Figure 53 PDA results from predrilling of pile 2 using the Delmag D100-13 hammer (Allnamics Geotechnical & Pile Testing Experts, 2024).

The pattern for the third pile is similar to the second pile. After the predrilling with the Delmag hammer, the larger hammer was installed (IHC S-280) to drill the piles to the required depth. The final penetration depth was 15 m, and it was stated that it was possible to drive the pile further with the same hammer (Allnamics Geotechnical & Pile Testing Experts, 2024). The driving energy was kept between 120 to 140 kNm. The important results of the PDA are shown in Table 21. The steel slag layer does not significantly increase resistance based on the test.

Table 21 Overview of the PDA results for the three piles for two hammers.

Test nr	Depth(m)	max calendar(bls/0,25m)	max transferred energy(kNm)	max compressive stress (MPa)	max tension stress (MPa)	Max piling resistance (MN)
1(P1-D100)	10 13.25	56 160	180 130	186 159	50 20	17,7 21,9
2(P2-D100)	5,75	30	128	141	77	19,9
3(P3-D100)	5,50	32	130	147	75	11,8
1(P1-S280)	13,75	77	270	279	169	38,8
2(P2-S280)	14,00	41	142	149	98	13,7
3(P3-S280)	15,00	36	149	151	66	13,2

Piles 2 and 3 have a maximum piling stress of 160 MPa, smaller than steel's maximum allowed yield strength. Regarding pile 1, it cannot be claimed whether the pile was damaged or displaced due to the damage. However, it is more reasonable that first, the pile was damaged after which the obliquity occurred. The obstacle must be larger than the thickness of the foundation pile which was 25 mm. Based on the outcome of the test case, it can be stated that it is possible to install foundation piles and sheet piles through the steel slag subsoil.

For the sheet piling three tests were performed. For the test one was pre-drilled and one was not. It was concluded that predrilling is needed for the sheet piles. The yield stress at its maximum was 110 MPa, which is acceptable considering the steel quality.

In conclusion, the following lessons are learned.

- The piles of this design are comparable in size to the pile of the test setup, so similar behaviour can be expected.
- No significant increase of stress occurs by driving the pile through the steel slags compared to sand.
- The steel slag soil can be penetrated by the pile and hammer combination, however, the large deformation of pile 1 underlines the need for a stronger pile tip.

Advised risk mitigation measures:

- Predrilling the sheet piles will be needed to penetrate through the steel slag soil layers.
- As a large number of piles will be needed for the construction it is advised to account for a small percentage of damage during installation.
- Predrilling is a costly mitigation measure and therefore not advised for all piles. It can help reduce drilling time, damage, and vibrations. Due to the increase of cost, and uncertainty in the need for the measure, it is not yet advised.
- Driving shoes can be used to strengthen the pile tip. This might help to indicate a large object in the subsoil by preventing pile damage. This also helps in penetrating through hard layers. So the pile possibly can be removed and relocated if necessary. The additional cost depends on the material of the shoes.

6.7 Cross-section with structural elements indicated

The dimensions of the calculated elements are shown in Figure 54.

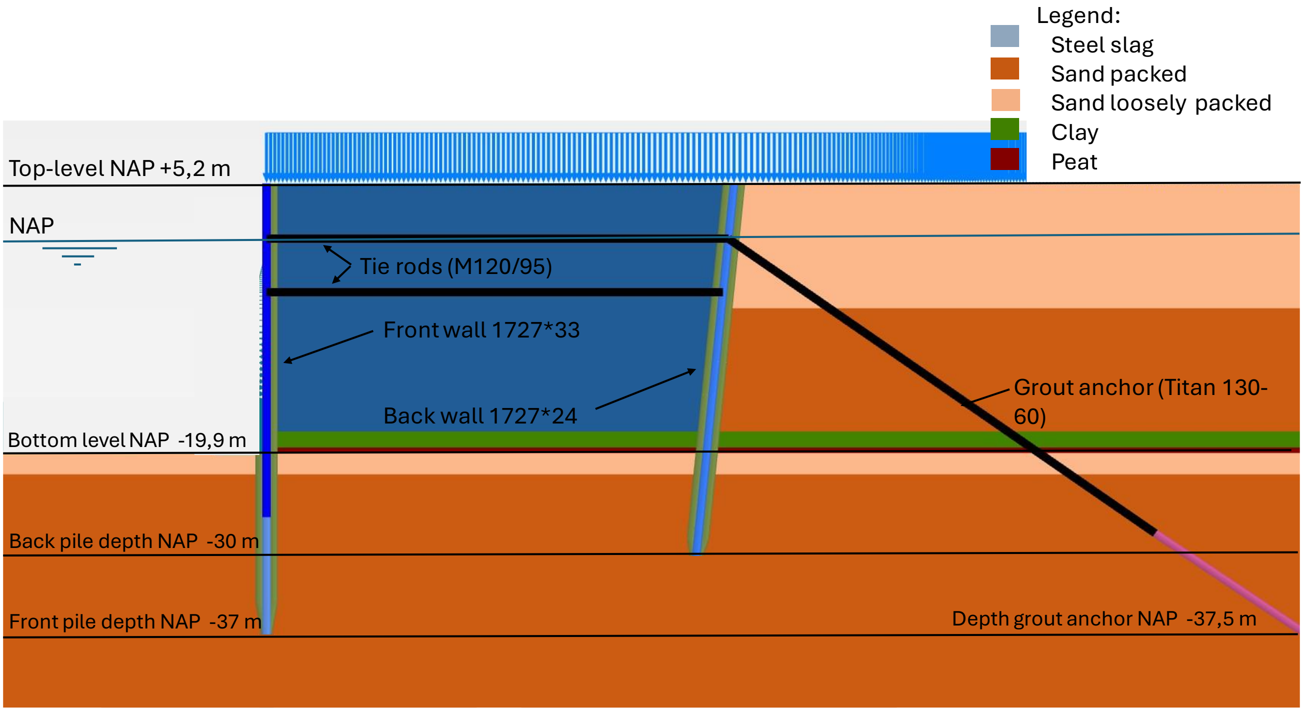

Figure 54 Drawing with the calculated dimensions of the elements indicated.

6.8 Verification of the conceptual design

As stated, the design of the quay wall must satisfy the programme of requirements. The programme of requirements is divided into functional, structural and environmental requirements.

6.8.1 Functional requirements verification

The functional requirements are satisfied by this conceptual design. The lifetime and design vessel type are satisfied by the dimensions of the quay wall. In the list of complementary technical systems, the additional functional requirements are considered.

6.8.2 Structural requirements verification

The structural requirements make sure the design is well-functioning and have structural integrity. The requirements are considered in the calculations and element choices. The calculations made are based on the prescribed norms and codes. In conclusion, this setup should be able to bear the loads and is constructible, sufficiently stable, stiff and strong.

6.8.3 Environmental requirements verification

In this design, the steel slag material has a useful application since the material reduces the load on the combi wall. Additionally, the steel slag material will be placed in between the combined walls and thus reduce contact with the environment.

7 Validation of the resulting conceptual design

Based on the results of the previous chapters the conceptual design was validated. To check whether the design objective is adequately formulated and correctly translated into requirements the client of the project: Port of Amsterdam was asked for the final check.

The design objective was formulated as follows:

This thesis aims to support a conceptual design of a quay wall on a steel slag subsoil.

An engineer by Port of Amsterdam, involved in this project was asked whether the design provides the desired solution. In addition, the design steps taken were discussed and evaluated. The problem statement and the design objective were correctly formulated. The system analysis, which consists of the area, stakeholder, and function analysis, shows a realistic overview of the situation. The system analysis provides a good basis for the next step named: the basis of the design.

The starting points, imposed by the client were shown first. This differs a bit from reality since this design report's starting point differs from the project's starting point. The starting points provided in this report, correspond logically to the starting point in time. The programme of requirements provides a valuable overview of the requirements, however, in reality, this is more detailed. Next, the evaluation criteria and the boundary conditions satisfy its intended function.

The functional design is different from the real project's functional design because the quay wall type was determined in an earlier stage. Additionally, not all quay wall types were considered as a realistic option in an earlier stage. Yet, it is interesting to note that the most suitable variant differs from the chosen variant (combined wall with a relieving platform). The complementary technical systems are similar to the regular quay walls in the Netherlands.

The detailed structural design provides a basis for a further design stage. Most importantly, the dimensions and elements are provided for the cofferdam to create sufficient resistance to the loads.

The quay wall structure will perform its intended function since the quay wall design satisfies the requirements. Several requirements, such as drainage and shore-based electricity systems, describe details that are not yet designed. Although these systems are not included in the design, it is realistic to state that the formerly described quay wall design can satisfy these requirements in a later design stage. The structure of the designed quay wall can fulfil the principle and preserving functions.

In conclusion, it can be stated that the right system was designed to obtain the objective. This conceptual design comes a step closer to the port development, which can be and function as a wind turbine assembly port.

8 Discussion

This conceptual design provides valuable insight into the constructability of a quay in a complex area with a steel slag subsoil. Yet, this design is not without its limitations. In this chapter, the constraints are discussed. The key aspects are the design considerations, challenges and implications are shown. It aims to provide an overview for the stakeholders to understand the outcome of the conceptual design.

8.1 Design considerations

8.1.1 Environmental considerations

Environmental considerations played a significant role in the design process. Risks of steel slag layer increase when the material comes in contact with ground- and rainwater. The larger the volume of the layer the larger the risk for the environment. In this design, the steel slag material is stored in an almost impermeable layer, however, it cannot be guaranteed that there will be no leakage. The health risks for the employees exist when there can be contact between humans and steel slags.

8.1.2 Ecological footprint

To minimize the ecological footprint of the construction and operation a comparison between variants, based on CO_2 emissions was made. Efficient use of materials was aimed for. The steel slags could reduce the lifetime of steel. Interaction between steel slag and construction material can result in additional corrosion. Since steel slag is not often used in the construction of hydraulic structures, not much research has been performed on the interaction between steel slags and steel. The tie rods are placed in the steel slag material in this design. The Eurocode provides corrosion reduction guidelines, however, these are based on regular conditions. It could be possible that the interaction between steel slags and the structure reduces or increases the lifetime of the steel and concrete elements. In this design, no additional material or protection was accounted for, since it is unknown whether the effect is positive or negative.

8.1.3 Soil conditions

The varying soil conditions at the quay wall location, provide complications in the design and the execution phase. The historical context of the site is considered during the process. Yet, the soil composition is partly based on historical data. The parameters of the soil layer are estimated based on the soil investigations at the project site. These parameters could differ along the length of the quay wall.

8.1.4 Quay wall modelling

For the modelling of the quay wall, several cross-sections are considered, however, more cross-sections could be taken into account especially when significant variations in the subsoil are present. In a more detailed design, additional cross-sections or a 3D model should be considered. In addition, various load cases can be considered such as cyclic loading.

8.1.5 Cost and emission estimation

The cost and CO_2 emissions estimations are calculated for the 3 most promising variants. These calculations are based on an estimation of the materials that are required to create the quay wall type.

8.1.6 Execution and risks of the design

Given the location of the quay, it is evident that not all quay wall types could be constructed conventionally. To ensure the selected quay wall type can be constructed without excessive increase in cost and construction time the construction sequences are made upfront of the selection of the most suitable variant. The main risk, for the realisation of this structure, is the installation of the foundation- and sheet piles.

8.2 Implications of design choices

8.2.1 Economic feasibility

The prices of fabrication and installation of the piles can vary from the shown estimation. After making a more precise cost assessment, an improvement in the justification of the most suitable variant could be made. Because the combi wall with the relieving platform and the cofferdam are relatively close to each other both in score and costs, a change in the cost estimation could change the outcome of the selection process. For the costs of the piles for all variants, the same price per unit was used, however, it could be possible that the larger piles require different steel production techniques which might increase the price per unit length of the pile.

8.2.2 Structural integrity

The design ensures the quay wall remains stable and functional under various load conditions. For the 2D model of the quay, the governing load conditions were used. Yet, several unknowns are present during the selection of the governing situation such as the soil parameters at specific locations. This holds that the maximum forces can change slightly in a later design stage, however, in this conceptual design there is room for adjustments and upgrades.

8.2.3 Building a cofferdam

As stated, the execution of the design was of major importance in this conceptual design. Still, the complex soil conditions and the construction of an unusual quay wall type could come with several unforeseen challenges. Based on the design choices made, the design enhances the possibility of construction of a cofferdam at the IJmond port.

9 Conclusions and recommendations

In this chapter, the conclusions for the design of a quay wall will be drawn and the deepening question will be answered. Secondly, the recommendations will be made.

9.1 Main conclusions

This design thesis is conducted to investigate the possibility of the construction of a quay wall for the Energiehaven in IJmuiden. For the thesis, a conceptual design was realised, based on a comprehensive approach to create robust, efficient, and sustainable maritime structures. For the qualification of the design, the Eurocode norms are considered (CUR 166 & CUR 211). A finite method (PLAXIS 2D) was used for the verification of the loads. In short, it can be stated that it is possible to construct a quay wall that can withstand the loads. Based on the cost and score, the most suitable quay wall type is a cofferdam with an additional tie rod. Based on the structural design it can be stated that the cofferdam can be made sufficiently stiff, stable and safe.

9.2 Answers to in-depth questions

9.2.1 How can a quay wall be designed on a steel slag soil layer most sustainably and durably possible?

The steel slag material provides properties that are useful for construction. Compared to sand the higher internal friction angle and higher density can result in more efficient use of construction materials. The active earth pressure is one of the loads on the combi walls. The active earth pressure was calculated for a steel slag-filled cofferdam. This pressure is relatively low compared to a cofferdam filled with sand or clay soils, because of the relatively large friction angle of the steel slags. Based on calculations sand-filled would increase the active earth pressure by 26 %, which results in extra loads on the combi walls. In the Netherlands, steel slags can be used if the material provides an additional value compared to regular soil materials.

9.2.2 What are the potential risks of constructing a quay wall on a steel slag subsoil?

Environmental and ecological issues arise since steel slags can leak heavy metals. In this case, the steel slags have been on the construction site for several decades. After a few years, the leakage of heavy metals reduces significantly. However, at IJmuiden, the steel slags are enclosed by a clay layer and an impermeable layer, so it is hard to predict whether the environmentally harming material is still there or dissipates over time. Still, the risk will increase when this slag material is replaced.

Due to the high permeability water penetration is a risk during construction. In quay wall construction, water level differences can occur. The water can penetrate easily through the steel slag layer. In the cost estimations, 2 million was reserved for installing a pumping system.

9.2.3 How can steel slags be used most effectively?

Steel slags can be used most effectively when the specific parameters which vary from other construction materials like sand and clay are accounted for. In the calculation method for quay walls, the subsoil properties influence the quay wall type selection and the elements' verification. Therefore it is of major importance that the subsoil and its properties are known in an early stage of the design. Moreover, the responsible design team should be aware of the effects of the changes in the parameters. The next step is viewing the odd construction material as a chance for design optimisation rather than a complication.
In the quay wall design, the internal friction angle dominates the loads at the retaining structure. Due to the relatively high internal friction angle of steel slags, the active soil pressure of the quay wall is relatively small. Another use of steel slags is as fill material in the foundation piles. For the calculation of buckling the Eurocode provides values for sand-filled piles. The fill material provides additional resistance against buckling. Steel slag instead of sand fill material will increase the buckling resistance.

9.3 Recommendations

Based on the discussion, the following recommendations are provided for future research and more detailed design:

9.3.1 Further research:

- No calculation method in the Eurocode is provided to calculate the buckling resistance of steel slag-filled foundation piles, however, steel slags are expected to increase the resistance. This solution can potentially increase the functionality of the steel slags in quay wall construction.
- It is advised that the environmental and health risks of using steel slags in a hydraulic structure be further analysed. More specifically for steel slags that were used in the past, and need to be relocated.
- Due to the relatively high permeability of the steel slag soil layer a (geo)hydrological analysis is required to understand the behaviour of the cofferdam for differences in the groundwater table. Due to rain events or extreme seawater levels, the flow inside the cofferdam should be analysed. This ensures the material between the combined walls is stable and no large deformation occurs. This is to guarantee the structure is not sensitive to groundwater-related failure mechanisms like heave, uplift and piping.
- Further research could be performed on the interaction between steel slags and its impact on the lifetime of steel and concrete.

9.3.2 Design recommendations

Further (detailed) design steps could be taken to realise the construction of the cofferdam. As the subtitle of this thesis report states, this is a conceptual design. In a later design stage, additional calculations should be made to develop the details and connections of the elements. After these steps, the technical drawings could be made. For the next design step, several points must be taken into account:

- The cross-sections vary significantly along the quay wall, therefore it could be advised to create a 3D model.
- At the first and last point of the quay, measures might be necessary during construction to create a safe and dry working environment. A solution could be to enclose the cofferdam at the edges with a temporary wall to close off the area inside.
- The mentioned technical systems should be implemented in a later design stage. The complementary systems will be necessary for the functioning of the quay wall. Whereafter a more detailed estimate of the cost and emissions can be made.
- The drainage system's design should minimise the contact between rainwater and steel slag. This is to minimize the risk of dissipation of the heavy metal into the environment.
- The buckling resistance of the foundation piles filled with steel slag material is possibly higher than sand-filled piles, due to the steel slag parameters. This can result in a larger bearing capacity and could be used to reduce the required foundation pile thickness.
- It is advised to perform a second pile-drivability test. The pile for testing should be equipped with a driving shoe, and close to the final location. Another pile could be The test should provide additional information about driving piles through a steel slag layer with mitigation measures.
- A cofferdam with these dimensions is quite a unique design in the Netherlands. This might result in challenges for the contractor. Therefore it is recommended to collaborate with the contractor in the next design and execution stage to avoid delays that could be foreseen.

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Appendix A: Cost estimations

The cost estimation for the combi wall with relieving platform, cofferdam and deck on piles variants.

Costs estimation							
Combiwall with relieving platform							
	Number	Unit	Price	Total			
Combiwall							
Steel piles	5000	ton	1335	6675000			
Down the hole drilling	180		3000	540000			
Installation piles	180		2500	450000			
Sheet piles	1500	ton	1135	1702500			
Installation sheet piles	9000	m2	20	180000			
Lock system	10000	m	55	550000			
Total				10097500		10,1	million
				0			
Concrete floor				0			
Reinforced steel	7000	ton	1750	12250000			
Construction of floor	25000	m3	150	3750000			
Deksloof	3500	m3	200	700000			
Betonshort	6200	m3	150	930000			
Workfloor	2500	m3	150	375000			
Betonproppen	1000	m3	200	200000			
Total				18205000		18,2	million
				0			
Anchorage				0			
Anchors	300		7250	2175000			
Total				2175000		2,2	million
				0			
Pressure piles				0			
Piles	600		8000	4800000			
Total				4800000		4,8	million
				0	**Total**	**35,3**	**million**
Cofferdam							
Combiwall	Number	Unit	Price				
Steel piles	10000	ton	1335	13350000			
Down the hole drilling	360		3000	1080000			
Installation piles	360		2500	900000			
Sheet piles	3000	ton	1135	3405000			
Installation sheet piles	18000	m2	20	360000			
Lock system	20000	m2	55	1100000			
Total				20195000		20,2	million
Anchorage							
Horizontal anchors	600		7250	4350000			
Grout anchors	300		7250	2175000			
			Total	6525000		6,5	million
Ground works							
Steel slags transport	116000	m3	15	1740000			
Others	116000	m3	15	1740000			
			Total	3480000		3,5	million
					Total	**30,2**	**million**
Deck on piles							
Foundation piles	Number	Unit	Price				
Steel piles	30000	ton	1335	40050000			
Down the hole drilling	1400	each	2000	2800000			
Installation of piles	2800	each	1500	4200000			
Sheet piles	2400	each	1135	2724000			
Installation of sheet piles	10500	m2	25	262500			
Predrilling	580	m	20	11600			
			Total	50048100		50,0	
Concrete							
Steel reinforcement	6000	ton	1750	10500000			
Installation of floor	18000	m3	175	3150000			
betonproppen	2100	m3	200	420000			
			Total	14070000		14,1	
Steel							
Mooring posts	1000	ton	1335	1335000			
Installation	20		5000	100000			
			Total	1435000		1,4	
					Total	**65,6**	**million**

Appendix B: Bollard load determination

The table provided by Port of Amsterdam to determine the bollard force.

ndaard bolder	TEU	DWT (scantling)	Displacement	Length Over All	Beam	Design Draft*	Scantling Draft	Berthing velocity	Berthing Angle	Factor of Safety abnormal berthing (ULS)	Slagzij tijdens afmeren	Slagzij tijdens laden/lossen	MBL [ton]	SWL bolder [kN] 2 lijnen	SWL bolder [kN] 3 lijnen	standaard HbR bolder
Sea going tankers																
Coaster		5,000 - 8,000 dwt	5,850 - 9,390 t	100 -116 m	16.00 - 18.00 m	6.00 - 7.10 m	6.40 - 7.50 m	0.15 m/s	7º	1.75	2º	5º	50	600	900	1000
Handysize		10,000 - 25,000 dwt	11,700 - 29,250 t	124 - 170 m	19.00 -25.50 m	7.50 - 8.90 m	8.00 - 9.60 m	0.15 m/s	7º	1.75	2º	5º	50	600	900	1000
Handymax		30,000 - 45,000 dwt	35,100 - 52,650 t	176 - 183 m	28.00 -32.20 m	9.00 - 11.30 m	9.90 - 12.40 m	0.12 m/s	5º	1.5	2º	5º	60	720	1080	1500
Panamax		50,000 - 55,000 dwt	58,500 - 64,350 t	228.6 m	32.20 m	11.00 - 12.30 m	12.30 - 14.10 m	0.12 m/s	5º	1.5	2º	5º	64	768	1152	1500
Aframax		85,000 - 105,000 dwt	99,450 - 122,850 t	244 m	42.00 m	11.00 - 13.40 m	12.10 - 14.70 m	0.10 m/s	3º	1.5	2º	5º	72	864	1296	1500
Suezmax		115,000 - 165,000 dwt	134,550 - 193,050 t	250 - 274 m	44.00 - 50.00 m	13.50 - 15.60 m	15.00 - 17.00 m	0.10 m/s	3º	1.25	2º	5º	83	996	1494	1500
VLCC		260,000 - 319,000 dwt	304,200 - 373,230 t	333 m	58.00 - 60.00 m	17.70 - 21.00 m	19.10 - 22.70 m	0.08 m/s	2º	1.25	2º	5º	103	1236	1854	2000
ULCC		360,000 - 560,000 dwt	421,200 - 655,200 t	341 -460 m	65.00 - 70.00 m	21.40 -22.80 m	23.10 24.70 m	0.08 m/s	2º	1.25	2º	5º	103	1236	1854	2000
Sea going containervessels																
Coaster	400 -1,000 TEU	6,200 - 15,000 dwt	8246 -19,950 t	107 -150 m	17.20 -23.00 m	6.50 -7.60 m	7.70 -9.10 m	0.15 m/s	2º	1.75	2º	5º	50	600	900	1000
Feeder	1,200 - 2,800 TEU	17,700 - 38,500 dwt	23,541 - 51,205 t	160 - 222 m	25.00 - 30.00 m	8.00 - 10.60 m	9.50 - 12.00 m	0.12 m/s	2º	1.5	2º	5º	50	600	900	1000
Panamax	2,800 - 5,100 TEU	38,500 - 66,000 dwt	51,205 - 87,780 t	211 - 294 m	32.20 m	10.70 - 12.00 m	12.00 - 13.50 m	0.10 m/s	1.5º	1.5	2º	5º	64	768	1152	1500
Post-Panamax	5,500 - 10,000 TEU	70,000 - 118,000 dwt	93,100 - 156,940 t	263 - 334 m	40.00 - 45.60 m	12.50 - 13.00 m	14.00 - 14.50 m	0.08 m/s	1º	1.5	2º	5º	83	996	1494	1500
New-Panamax	12,500 - 14,000 TEU	143,000 - 157,000 dwt	190,190 - 208,810 t	366 m	48.40 m	13.50 - 15.00 m	15.00 - 16.50 m	0.08 m/s	1º	1.5	2º	5º	130	1560	2340	2400
ULCV	15,500 - 21,150 TEU	171,000 - 195,000 dwt	227,430 - 259,350 t	397 - 400 m	56.40 - 59.00 m	14.00 - 14.50 m	16.00 m	0.08 m/s	1º	1.5	2º	5º	130	1560	2340	2400
Sea going bulkcarriers																
Ccoaster		5,000 - 8,000 dwt	5,850 -9,360 t	95 - 107m	16.00 -18.20 m	5.70 - 6.80 m	6.10 - 7.30 m	0.15 m/s	7º	1.75	2º	5º	50	600	900	1000
Handysize		10,000 - 30,000 dwt	11,700 - 35,100 t	117 -170 m	19.30 - 27.00 m	7.30 - 9.40 m	7.80 - 10.00 m	0.15 m/s	7º	1.75	2º	5º	50	600	900	1000
Handymax		35,000 - 55,000 dwt	40,950 - 64,350 t	178 - 200 m	28.00 - 32.26 m	9.50 - 11.50 m	10.50 - 12.70 m	0.12 m/s	5º	1.5	2º	5º	60	720	1080	1500
Panamax		65,000 - 82,000 dwt	76,050 - 95,940 t	225 - 229 m	32.26 m	11.20 - 13.40 m	12.60 - 15.10 m	0.12 m/s	5º	1.5	2º	5º	64	768	1152	1500
Capesize		80,000 - 175,000 dwt	93,600 - 204,750 t	225 - 289m	37.00 - 45.00 m	12.10 - 17.00 m	13.10 - 18.40 m	0.10 m/s	3º	1.5	2º	5º	83	996	1494	1500
VLBC		205,000 - 320,000 dwt	239,850 - 374,400 t	300 - 332 m	50.00 - 58.00 m	16.10 - 21.00 m	18.70 - 22.80 m	0.08 m/s	2º	1.25	2º	5º	103	1236	1854	2000
Berge Stahl Vale maz (China max)		365,000 - 400,000 dwt	427,050 - 468,000 t	342 - 362 m	63.50 - 65.00 m	21.20 - 23.00 m	23.00 m	0.08 m/s	2º	1.25	2º	5º	103	1236	1854	2000

Bron: Classificatie Vessels: Propulsion Trends in Tankers, Bulkers and Container vessels Man Diesel:
http://marine.man.eu/docs/librariesprovider6/technical-papers/propulsion-trends-in-tankers.pdf?sfvrsn=20
http://marine.man.eu/docs/librariesprovider6/technical-papers/propulsion-trends-in-bulk-carriers.pdf?sfvrsn=24
http://marine.man.eu/docs/librariesprovider6/technical-papers/propulsion-trends-in-container-vessels.pdf?sfvrsn=20
Bron: Factor of Safety abnormal berthing: Guidelines for the Design of Fenders Systems 2002 Report of WG 33 Table No. 4.2.5
Bron: factor DWT/Dspl. http://www.mandieselturbo.com/files/news/filesof17236/5510_004_02%20low[1].pdf
*) Sea going vessels in salt water
**) Werkelijke metingen laten bij tankers lagere afmeersnelheden zien. Echter de 'second impact' van tankers is groter door een doorgaande afmeerbeweging.

Appendix C: Quay wall type description

The developed design concepts for quay walls are explained in this appendix.

9.3.2.1 Gravity walls

The gravity quay walls are divided into: Block walls, L-walls, Caisson walls, Cellular walls and Reinforced earth structures. For all types of gravity walls, it holds that the soil retaining function of the wall is retrieved from the dead weight of the wall. Often gravity walls are prefabricated and placed on subsoil, which provides sufficient bearing capacity (de Gijt & Broeken, 2013).

Block wall

The most basic type of a gravity wall is the block wall. Stacked prefabricated concrete blocks are mostly used for the construction of quay walls in the presence of stiff subgrades such as rocks. Block walls require a high bearing capacity and need a relatively large amount of concrete. Shapes differ and can be modified (Francois, Lesage, Verbraken, & Schevenels, 2020). Retaining heights over 20 m can be achieved by block walls. An advantage of block walls is the drainage capacity (de Gijt & Broeken, 2013).

Cellular wall

This quay wall type has cylindrical web profiles that are connected as shown in Figure 55. The cells are filled with sand or other fill material. These walls are vulnerable to damage by collision with a rock. This damage leads to a reduction of bearing capacity and instability can occur.

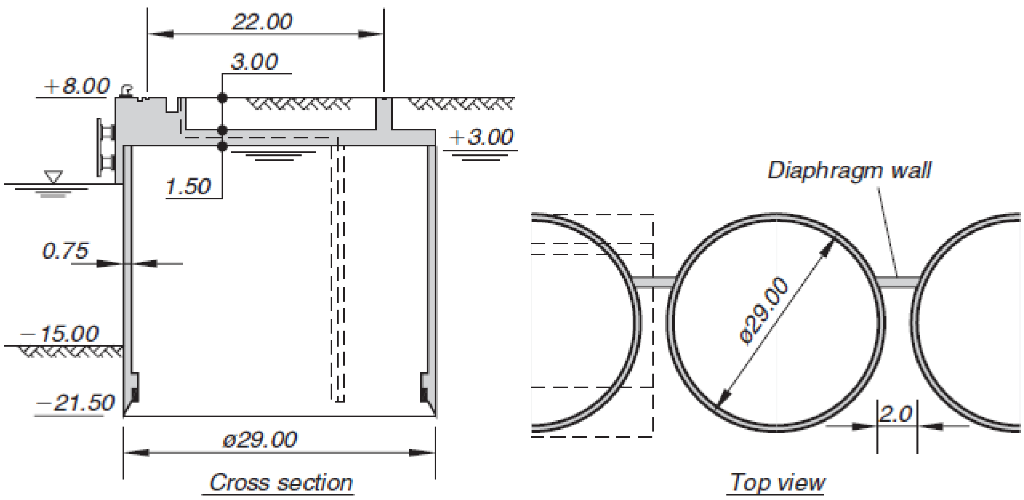

Figure 55 Cross-section of a cellular quay wall (de Gijt & Broeken, 2013).

Caisson wall

Caissons can have multiple functions and applications, whereunder a quay wall. Caissons are mostly designed to avoid large in-situ construction costs since caissons are constructed on land and transported to the location afterwards. In general, caissons can be divided into two types standard and pneumatic caisson. The standard caisson is prefabricated and thereafter transported over water. The pneumatic caisson is constructed at ground level and thereafter subsided in the soil to the desired depth. Considering the bathymetry and soil conditions at the construction site, the pneumatic way will not be a realistic alternative. Since soil excavation will be complex because of steel slags. Moreover, construction of a pneumatic caisson at the location will be hardly possible due to the dike ring. Caissons can be either standing free or partially embedded as can be seen in Figure 56 (Voorendt, Molenaar, & Bezuyen, 2011).

The construction of caissons consists of the following steps:

1. Initiative

2. Planning and design
3. Prefabrication
4. Transport
5. In-situ construction

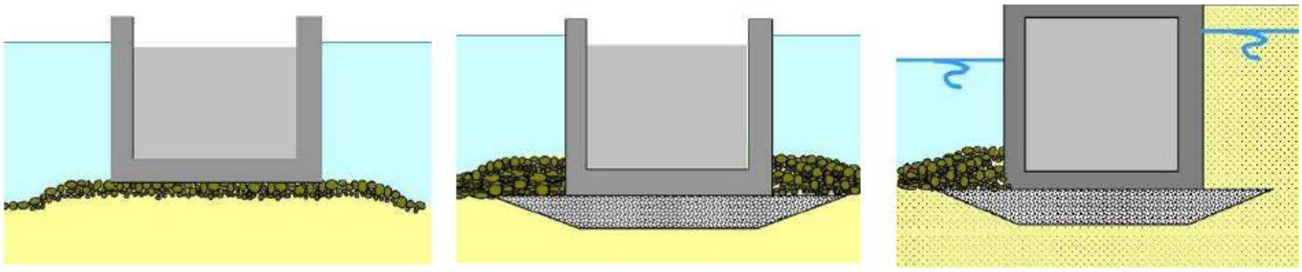

Figure 56 Caisson standing free of partially embedded (Voorendt, Molenaar, & Bezuyen, 2011).

The prefabrication location of the caissons should be as close as possible to the design location. The caisson should be watertight (or controlled acceptable leakage) and tested before transportation. The transport phase of a caisson should be considered since this will apply different forces compared to the final phase. In addition, handling tools such as boulders are needed. The positioning and immersion of standard caissons require a suitable bed layer. A layer of gravel or rubble mount can be fed in by a pipe in a controlled manner and smoothened afterwards by a levelling beam. Because irregularities in the soil will result in peaks in forces to the caisson bottom surface, local pressure should be taken into account. This might result in a relatively thick floor. An option could be to drive piling through hollow intermediate walls in already immersed caisson to obtain higher bearing capacity.

The draught of caissons is determined by the height and width of the caisson. Specific project requirements will determine the height and width of the caisson. The design checks are related to static stability, dynamics stability, rotational stability and shear. The failure mechanisms are tilting, piping and scour, sliding, and collapsing. In this case, the caisson could be filled with steel slag soils, which could be an environmental and efficient way of coping with the steel slags.

L-wall

The L-wall is a retaining structure with a horizontal and vertical slab. This type of gravity wall uses the weight of the soil for stability and requires less bearing capacity of the subsoil compared to a block wall. This wall can be constructed in situ or prefabricated. This structure is often applied when the soil is not sufficiently strong to create a block wall. The contribution of the soil on the horizontal part of the quay wall creates a resisting gravity force. In addition, it could save material compared to a block wall (de Gijt & Broeken, 2013).

9.3.2.2 Embedded walls

Embedded walls exist in several types, such as combi walls, and diaphragm walls.

Sheet pile wall

For limited dimensions and loads, a basic freestanding sheet pile wall can be a feasible solution. The construction material can be wood, steel or concrete. The sheet piles are connected by an interlocking system. If needed anchorage can be applied to gain extra stability for higher retaining heights.

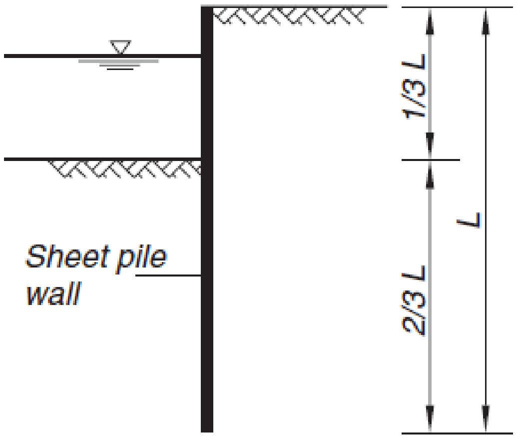

Figure 57 Basic sheet pile wall example (de Gijt & Broeken, 2013).

Combined walls

Combined walls or combi walls are sheet pile walls with different cross-sectional profiles. By doing so, larger loads and moments can be transferred to the soil. In Figure 58, various options are shown for the profile selection. Nowadays the far most walls installed use tubular piles (de Gijt & Broeken, 2013).

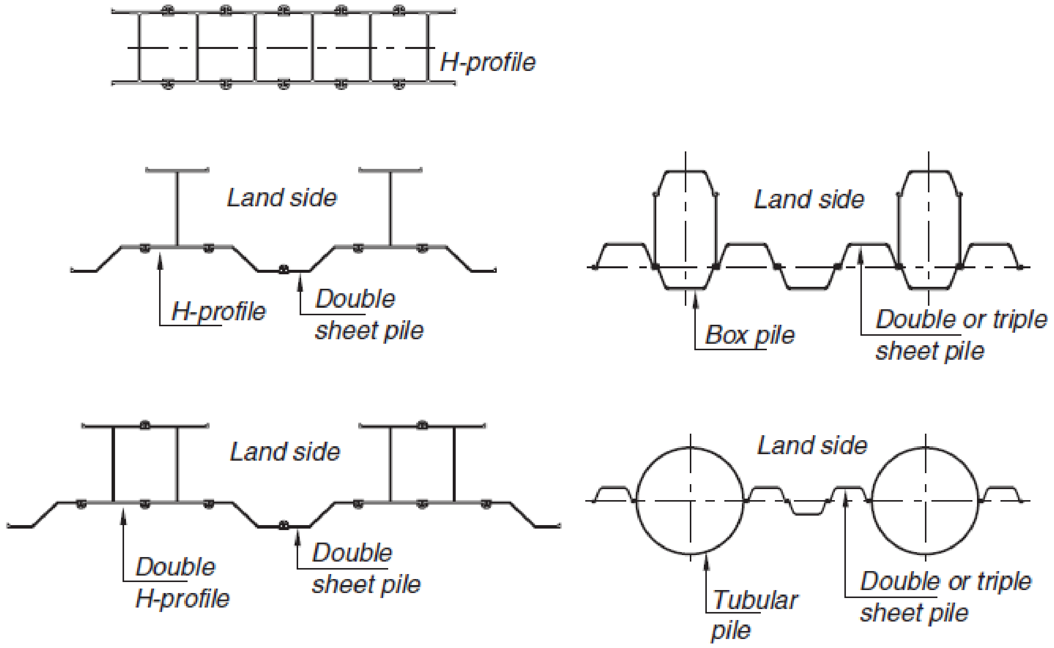

Figure 58 Horizontal cross-sections for the combined wall. Various profiles are shown (de Gijt & Broeken, 2013).

Diaphragm walls

Another option for an embedded wall is a diaphragm wall. This type of wall is in situ created by an excavation of the soil and replacing it with bentonite to avoid instability. When all the soil is removed the bentonite is filled with concrete. Due to the cross-sectional variation of the quay wall, it might be hard to construct this type of structure as it requires an in-situ construction. An example of a diaphragm wall in combination with a deck on piles is shown in Figure 59.

For the construction of a diaphragm wall, an impermeable layer is required, because bentonite and concrete leakage can occur. Considering the soil conditions at the port in IJmond this is a serious risk. In addition, there is a risk that digging the wall is not possible with the conventional excavation way.

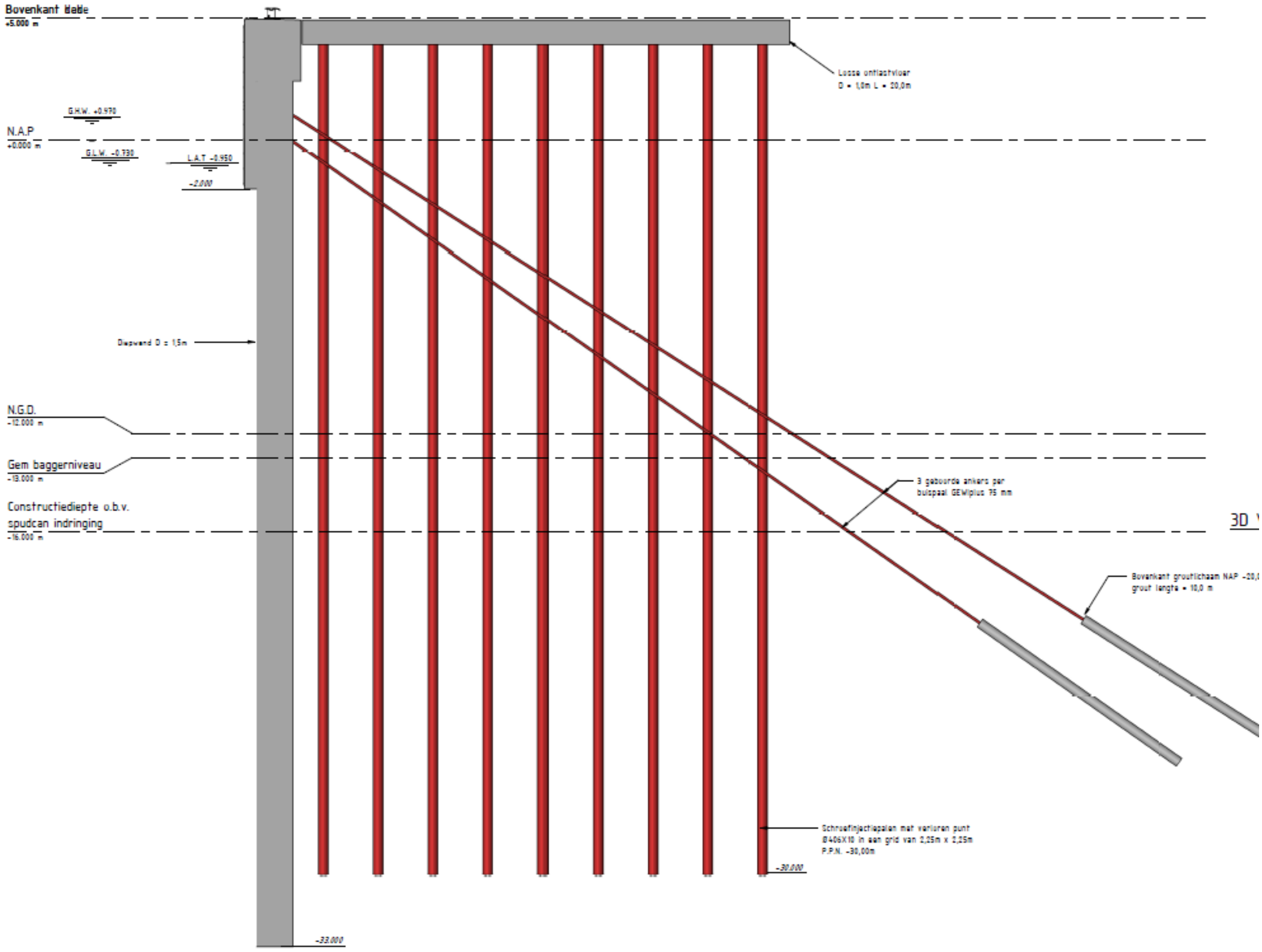

Figure 59 Diaphragm wall combined with a deck on piles example design (Witteveen en Bos, 2021).

Cofferdam

A cofferdam is in principle a combination of two sheet pile walls, connected by anchors as shown in Figure 60.

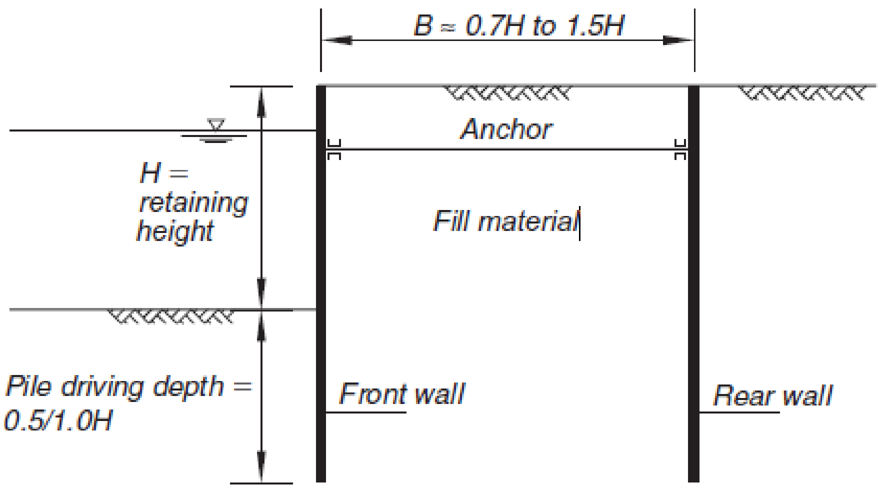

Figure 60 A cross-section of a basic cofferdam as an example (de Gijt & Broeken, 2013).

The soil in between the wall transfers the load to the subsoil. Due to the shear resistance and the weight of the soil, the cofferdam functions as a soil-retaining structure. Mostly the active and passive zones in the soils overlap because of the limited length between the walls (de Gijt & Broeken, 2013).

9.3.2.3 Embedded walls with relieving platform

An option is to create a relieving platform as shown in Figure 61. The relieving platform reduces the horizontal load on the wall structure. It is possible to prefabricate the relieving floor for relatively small dimensions.

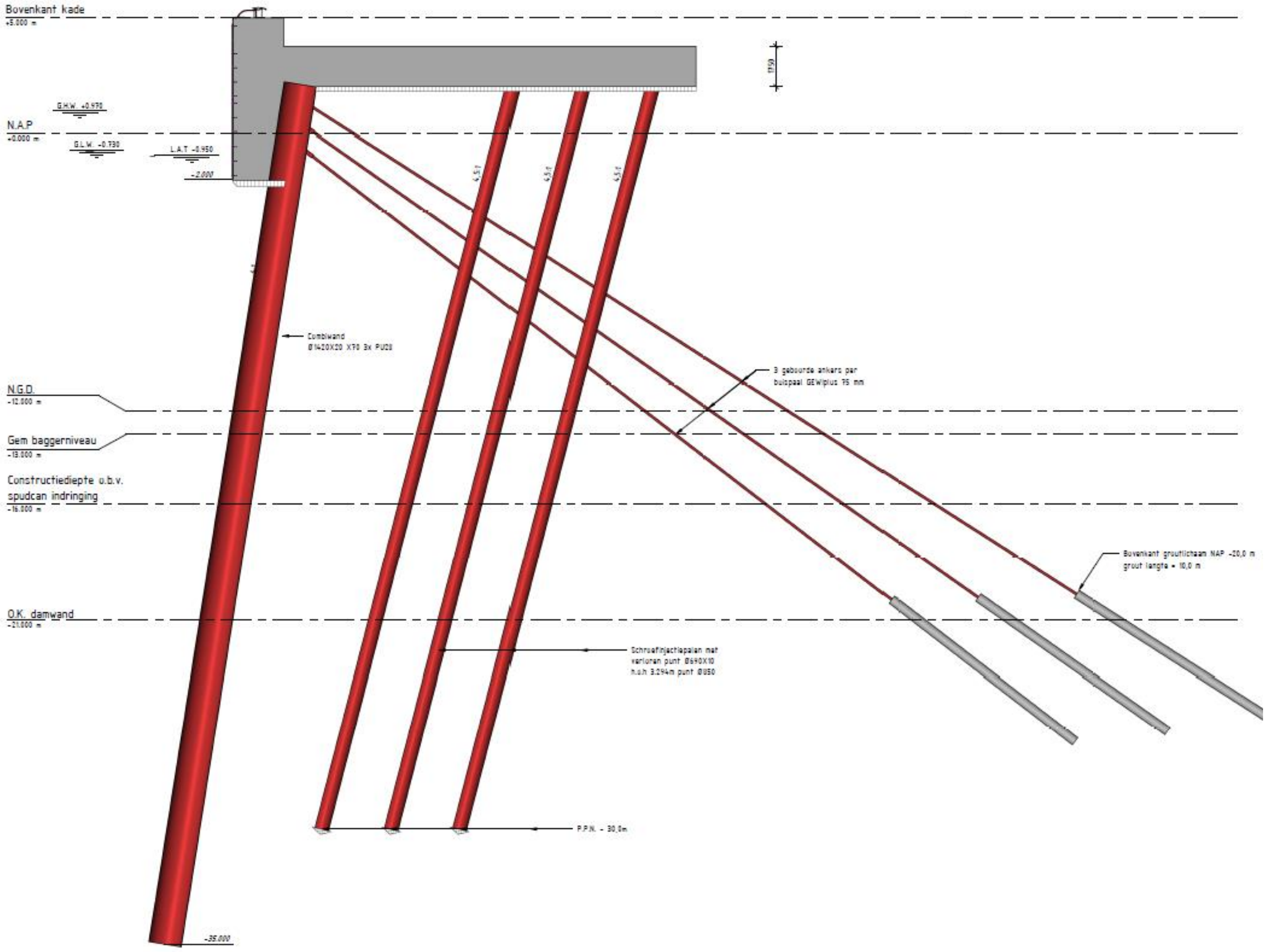

Figure 61 Combi wall with relieving platform (Witteveen en Bos, 2021).

The foundation system consists of tension and bearing piles. The height of the relieving platform can be modified. As shown in Figure 61, it can be combined with anchors to obtain higher stability. Combi walls are often used when high retaining heights, heavy loads, and small deformations are required. In the Netherlands, this construction method is often used.

The relieving platform height

The high-relieving platform usually is constructed above the low water level and can thus be constructed during low tide. It can be connected with a combined wall of diaphragm wall. Lowering the platform height results in a minimization of pile-driving problems. Cast iron saddles are used to create a hinged support between the platform and the sheet pile wall. The wall can be constructed with an angle. The heavy load can require a large amount of pile, then the pile group can work as a screen and cause larger deformations. In addition, a dense pile group can cause higher compression in the subsoil and cause driving problems (de Gijt & Broeken, 2013).

9.3.2.4 Open berth quay walls

A mooring facility can be created by building open berth quay walls. The advantages of this structure are a reduction in soil excavation volumes and can save in costs when large water depths are present. A disadvantage is that during berthing the water is moved by the vessel and can move under the structure. In addition, a large amount of piles is needed and the horizontal force distribution can be limited. An example is shown in Figure 62.

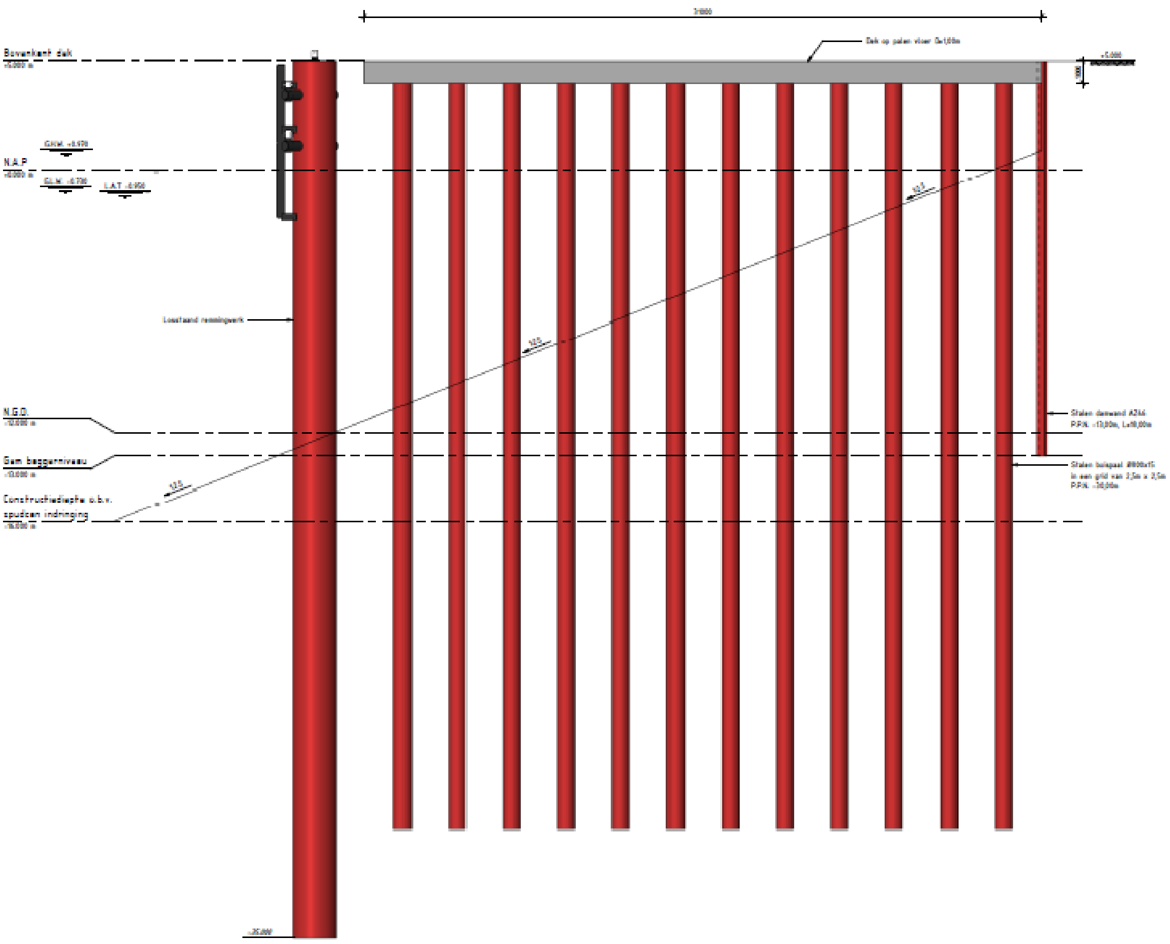

Figure 62 Deck on piles example (Witteveen en Bos, 2021).

A risk that could occur is the instability of the slope of the steel slag dam due to the installation of the piles. The installation of the piles might face implications because of the larger gradings of steel slags, yet, mitigation measures like pre-drilling or down-the-hole-drilling could reduce this risk.

9.3.2.5 Sandwich wall

Because the dimensions of equipment and so the forces on quay walls keep on increasing, a study was performed by Bonto (2007) to create a new quay wall concept. Several futuristic types were considered and the sandwich wall was the most promising. Due to the increasing vessel size and port development requirement larger dimensions of quay wall will be needed. Based on a retaining height of 32 m an example calculation is made. It was estimated that the costs were 1.6 times higher compared to a combi wall quay wall (Bonte, 2007).

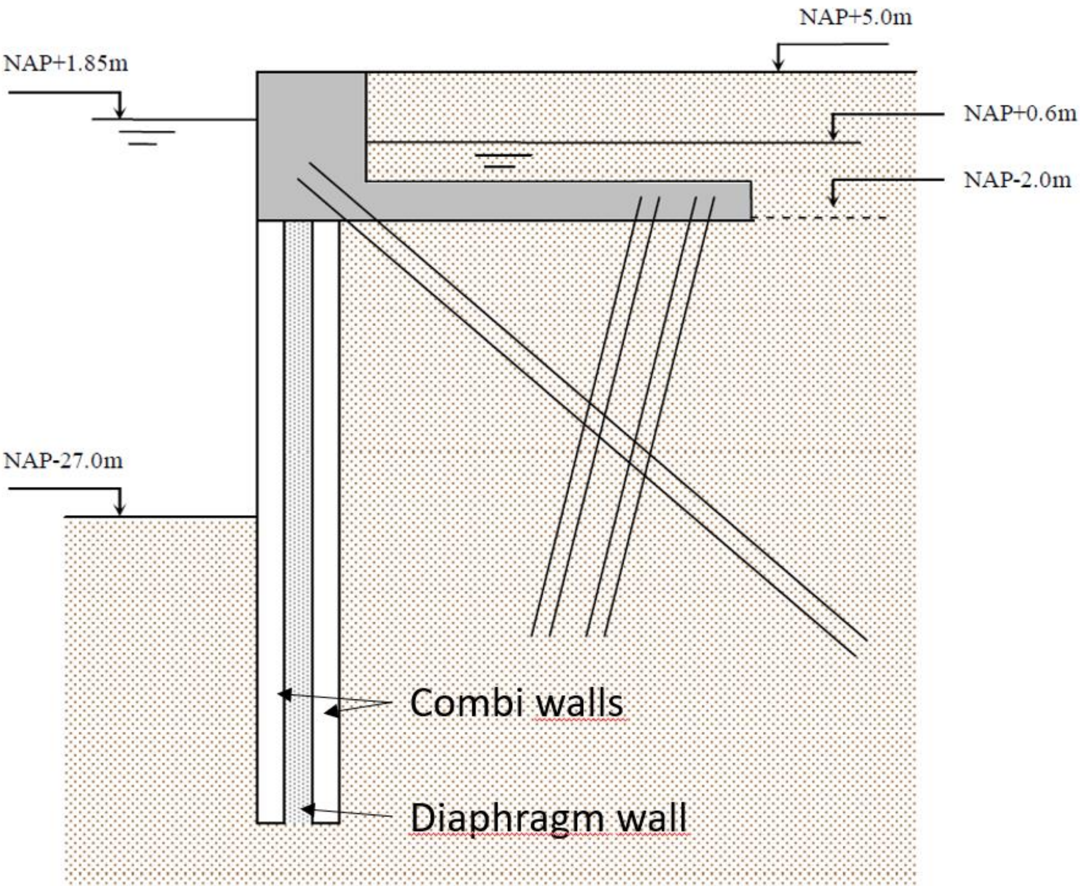

Figure 63 Cross section of a sandwich wall (Bonte, 2007).

9.3.2.6 Reinforced Soil Wall

As stated in Chapter **Error! Reference source not found.** reinforcing soil could have potential advantages for stabilizing the quay wall. It is a relatively new method to anchor sheet pile wall (Wittekoek, et al., 2021). Reinforced soil walls show up under various definitions such as mechanically stabilized earth, geogrid and reinforced earth. It can settle after construction a bit however it is strong and ductile. It was stated by Tensar (2021) that there is no limit to height, depending on the quality of the fill material. As grid high-density polyethylene (HDPE) is used to reinforce the soil. The highest wall built by Tensar is 60 m high (Tensar, 2021).

It can fail due to the exceedance of tensile strength which causes the reinforcement to break. The second reason is failure due to insufficient transfer of the load to the backfill material. The backfill material should ideally have a high internal friction angle and permeability and no cohesion. It is possible to construct this method in wet and dry conditions. In wet conditions guiding systems can be used (Solkema, 1994).

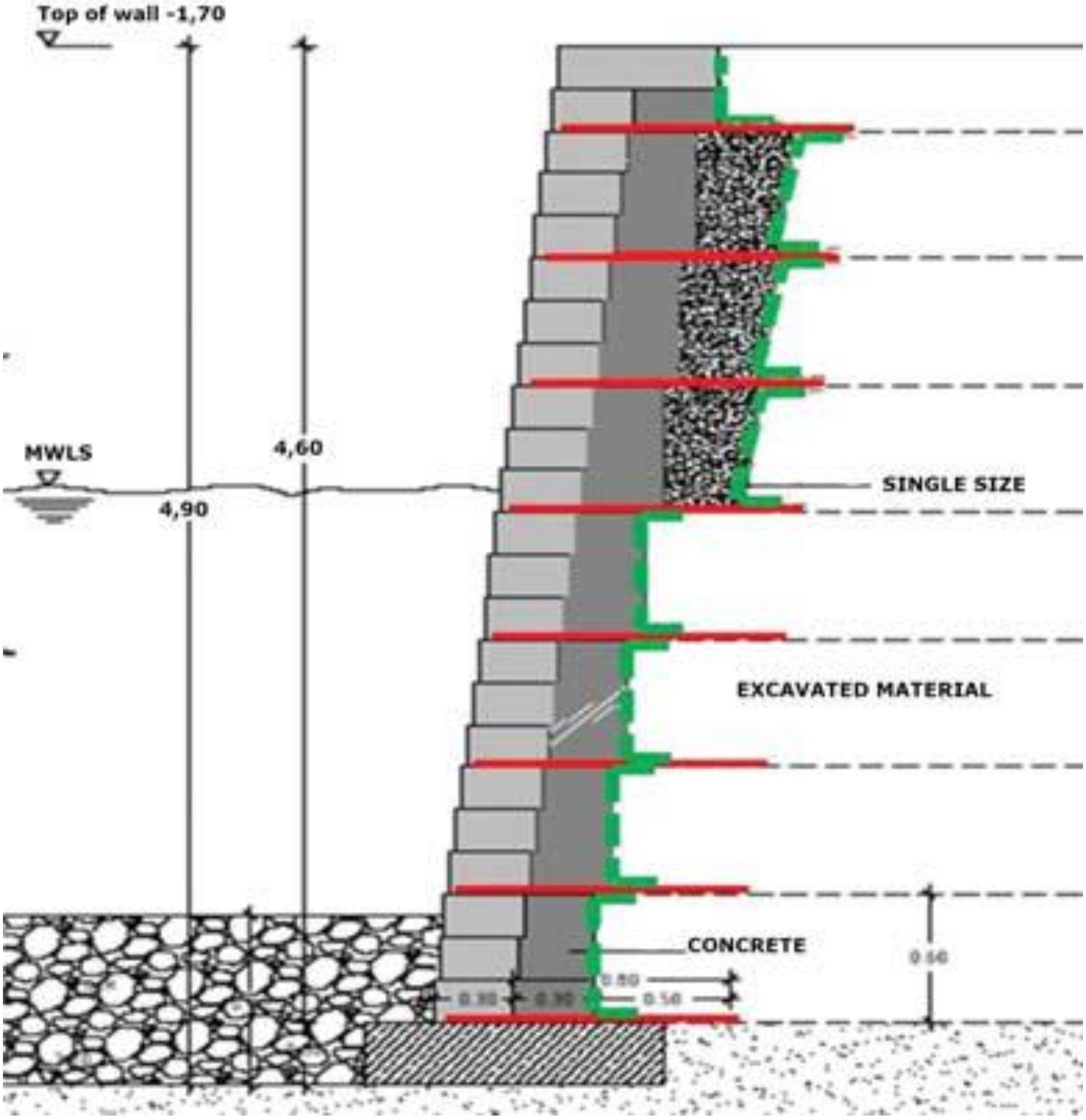

Figure 64 Cross-section of a block wall combined with reinforced soil (Wittekoek, et al., 2021).

Appendix D: Forces and deformations

The forces and deformations, as PLAXIS results, are shown in this appendix. Following by the resistance of the structural elements. The calculation sheets are shown in this appendix.

3.1.1.2.14 Calculation results, Plate, UGT Terreinbelasting + 0.7 x Bolderkracht [Phase_13] (13/142), Bending moments M

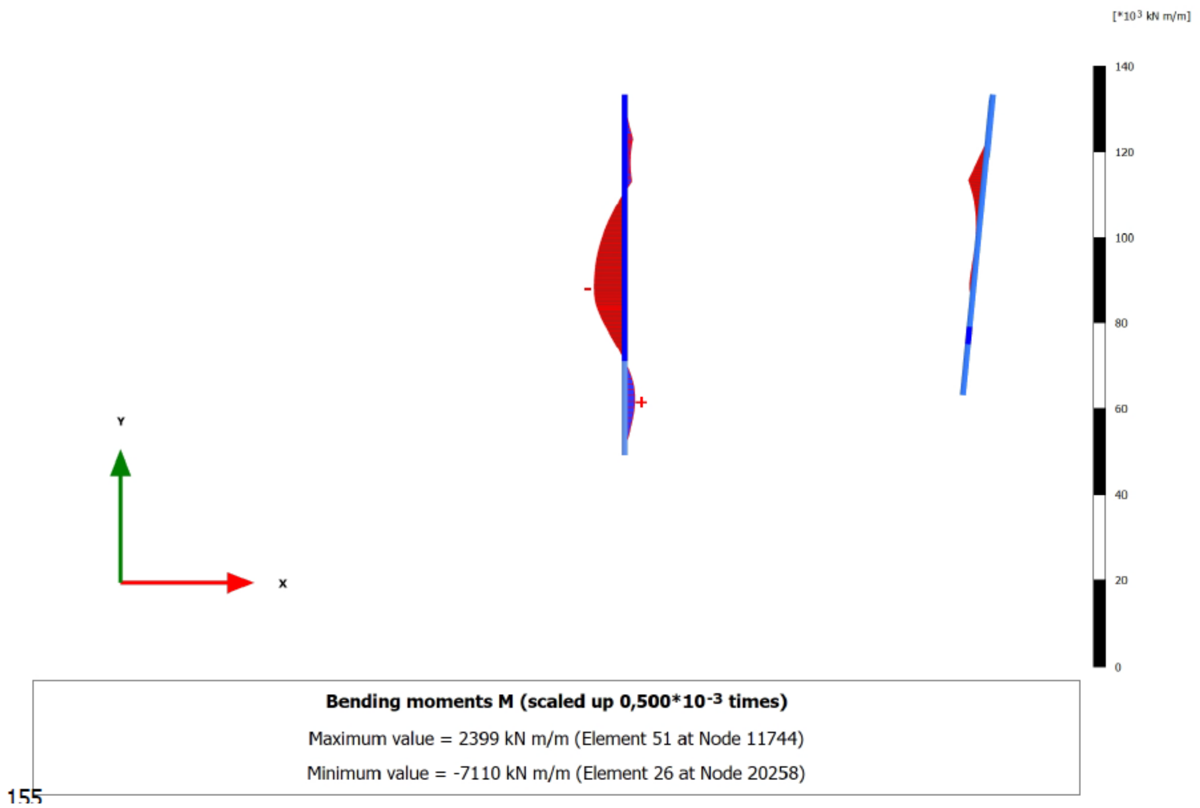

Figure 65 Maximum value of the bending moment in the combi walls.

And the deformations are shown in Figure 66.

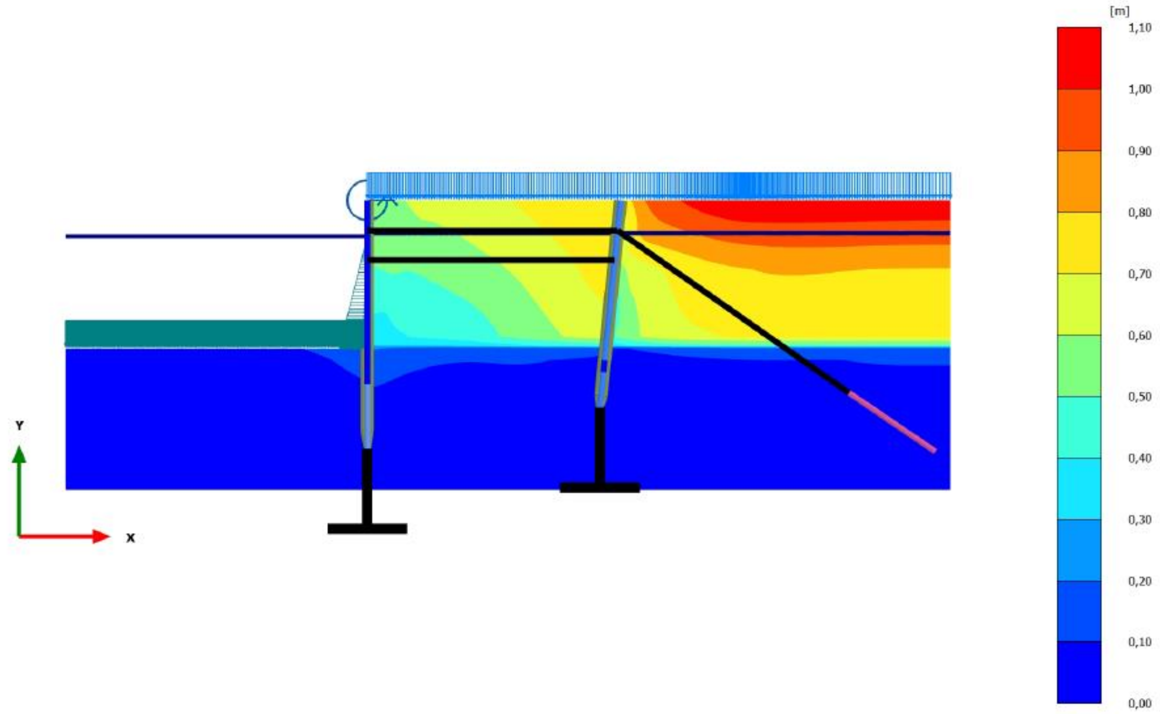

Figure 66 The total deformations during the user phase(which provides the maximum deformations).
The maximum deformation (which is at user phase) of the front and back combi wall are shown in Figure 67.

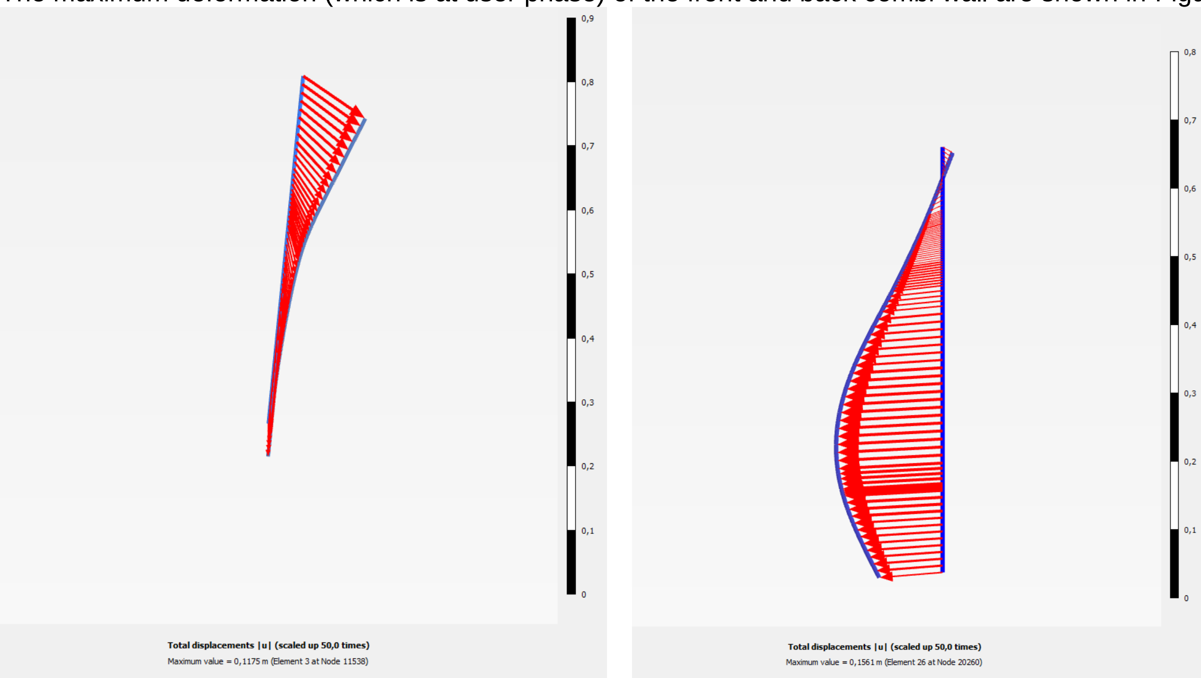

Figure 67 The deformation shape of the back and front wall at the user phase.

	Tie rod	
As	6362	mm2
E	210000	N/mm2
EA	1336020000	N
	1336020	kN
	1336,02	MN
	1,33602	e6 kN
d	100	mm
EA	2	e6
	ULS Load	
Anker 2	2243	kN
Anker 1	1513	kN
	SLS Load	
Anker 2	1659	kN
Anker 1	1200	kN
ULS		
Unity check anker 1	0,42	
Unity check anker 2	0,62	
SLS		
Unity check anker 1	0,48	
Unity check anker 2	0,66	

Figure 68 The calculation for the tie rods verification.

Buispaal				2xAZ28			Combiwall		
		Unit							
Afstand tussen palen	2,8	m							
Diameter inner	1661	mm							
Diameter outer	1727	mm							
Thickness	33	mm	Length	26	m				
Area	1,76E+05	mm2	Area	38900	mm2				
I	6,30E+10	mm4	I	1,07E+09	mm4				
EI cross-section	1,32E+16	Nmm2	EI cross-section	2,25E+14			I combi	2,36E+10	
	1,32E+07	kNm2		2,25E+05	kN/m2/m		W combi	2,65E+07	mm3/m
EI wall	4,73E+06	kN/m2/m					EI	4,95E+15	
EA	3,69E+10			2,25E+14			EI	4,95E+06	
	3,69E+07			2,25E+07	kN/m				
EA1	1,32E+07	kN/m	Mrd	1867250000	N/mm		EA	3,57E+07	kN/m
EA2	6,59E+05	kN/m	Mrd	1867,25	kN/m		EA2	1,79E+06	kN/m
unit weight steel	7,85E+03	kN/m3					M,rd	1,29E+10	N/mm
Volume	1,76E-03	m3					M,rd	1,286E+04	kN/m
Unit weight buispalen	1,38E+01	kN/m	Unit weight damwand	31,1	kN/m		Unit weight	36,0	kN/m

Figure 69 Calculation for the combined walls

The calculation results for empty and sand-filled piles can be seen in Figure 70.

CROSS SECTIONAL VERIFICATION OF TUBULAR PILE

			empty piles			sand-filled piles		
Out of roundness / ovalization			initial	corroded		initial	corroded	
Maximum out of roundness	$U_{r,max}$	=	0,010	0,010	[-]	0,010	0,010	[-]
Ovalization due to initial out of roundness	a_1	=	4,24	4,24	[mm]	4,235	4,235	[mm]
Ovalization due to cross tensile forces	a_2	=	0,00	0,00	[mm]	0,000	0,000	[mm]
Ovalization due to outside soil pressure	a_3	=	7,23	7,92	[mm]	7,225	7,924	[mm]
Meridonial membrane stress	σ_y	=	304,3	313,8	[N/mm^2]	304,316	313,826	[N/mm^2]
Curvature	κ	=	0,0016	0,0017	[1/m]	0,002	0,002	[1/m]
Ovalization as second order effect of tube bending	a_4	=	1,07	1,15	[mm]	1,065	1,151	[mm]
Compression modulus of sand	E_{sand}	=	0	0	[MPa]	**10**	10	[MPa]
Stiffness sand	k_{sand}	=	0	0	[kN/m^3]	11806	11806	[kN/m^3]
Stiffness steel	k_{steel}	=	14663	13370	[kN/m^3]	14663	13370	[kN/m^3]
Total ovalization	a	=	12,53	13,31	[mm]	8,83	9,05	[mm]
Bending moment as a function of the plasticity rate			initial	corroded		initial	corroded	
Radius of curvature	r'	=	0,886	0,889	[m]	0,874	0,875	[m]
Critical strain	ε_{cr}	=	0,0068	0,0065	[-]	0,0100	0,0094	[-]
Relative strain	μ	=	4,930	4,707	[-]	7,221	6,779	[-]
Angle of plasticity rate	θ	=	0,204	0,214	[rad]	0,139	0,148	[rad]
Bending moment as function of plasticity rate	M_R	=	24797	24390	[kNm]	24889	24487	[kNm]
Reduced resistance for effects of ovalisation			initial	corroded		initial	corroded	
Bending moment in point A	M_A	=	-19,00	-19,00	[kNm/m]	-19,00	-19,00	[kNm/m]
Bending moment in point B	M_B	=	24,75	24,75	[kNm/m]	24,75	24,75	[kNm/m]
Tube wall bending moment	$m_{eff;Ed}$	=	21,87	21,87	[kNm/m]	12,12	11,62	[kNm/m]
Tube wall ultimate bending moment	$m_{pl;Rd}$	=	71,78	67,49	[kNm/m]	71,78	67,49	[kNm/m]
Reduction factor for ovalizing bending stresses	g	=	0,953	0,949	[-]	0,975	0,974	[-]
Reduction factor for ovalizing bending deformation	β_g	=	0,990	0,990	[-]	0,993	0,993	[-]
Reduction factor deformation capacity	β_s	=	1,000	1,000	[-]	1,000	1,000	[-]
Reduced bending moment resistance	M_{Rd}	=	23390	22913	[kNm]	24090	23684	[kNm]
Reduced normal force resistance	N_{Rd}	=	44108	43277	[kN]	45127	44407	[kN]
Unity check		=	**0,93**	**0,95**	[-]	**0,90**	**0,92**	[-]

Figure 70 The crosssectional verification of tubular piles

Appendix E: CO_2 emission calculations

The CO_2 emissions calculations were based on the materials used. The three variants that are compared use steel and concrete. For the estimation, it was assumed that the grout body consists of concrete. In Table 22, the used values per material are shown (IEA 50, 2024) & (Princeton, 2024).

Table 22 The CO2 emissions per construction material

Materials	CO_2 emission(ton/ton)
Steel	1,4
Concrete	0,93

The estimated CO_2 emissions for the combi wall with the relieving platform are shown in Table 23

Table 23 The estimated material volumes and CO_2 emissions for the combi wall with the relieving platform.

Combi wall with relieving platform	Total (tons)	Emissions (tons)
Steel	13500	18900
Concrete	38200	35526
	Total (tons)	54426

The estimated CO_2 emissions for the cofferdam are shown in Table 24.

Table 24 The estimated material volumes and CO_2 emissions for the cofferdam.

Cofferdam	Total (tons)	Emissions (tons)
Steel	14000	19600
Concrete	10700	9951
	Total (tons)	29551

The estimated CO_2 emissions for the deck on piles quay are shown in Table 25.

Table 25 The estimated material volumes and CO2 emissions for the deck on piles.

Deck on piles	Total (tons)	Emissions (tons)
Steel	37000	51800
Concrete	20100	18693
	Total (tons)	70493

Appendix F: Stages in PLAXIS

The construction stage is modelled in the PLAXIS 2D finite element program. The construction stages as shown in the Figure 71 to Figure 80.

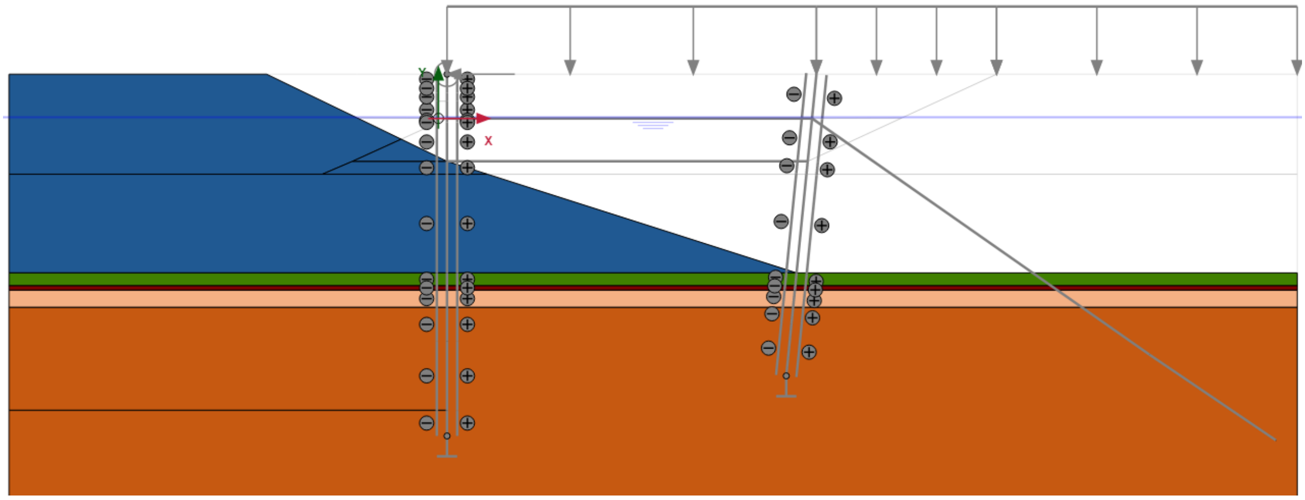

Figure 71 The initial situation at the construction location.

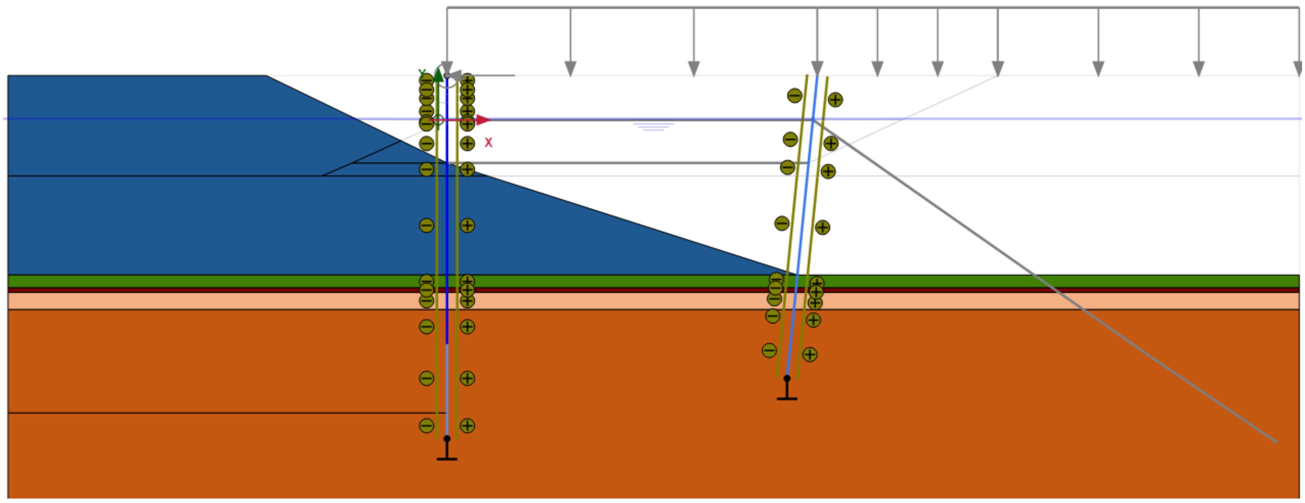

Figure 72 The cofferdam installation.

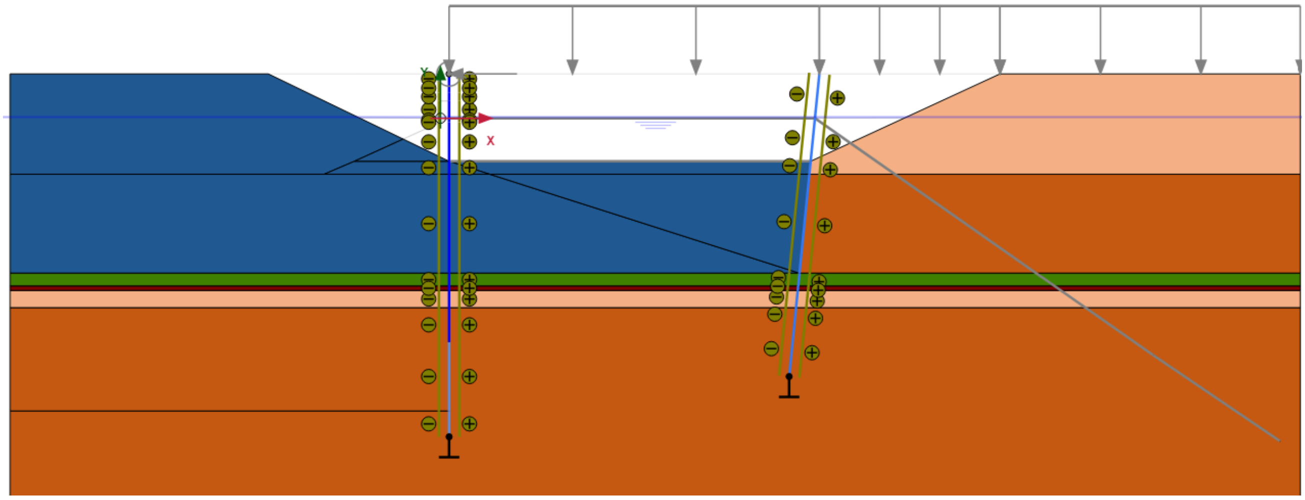

Figure 73 The backfill of the cofferdam and the terrain.

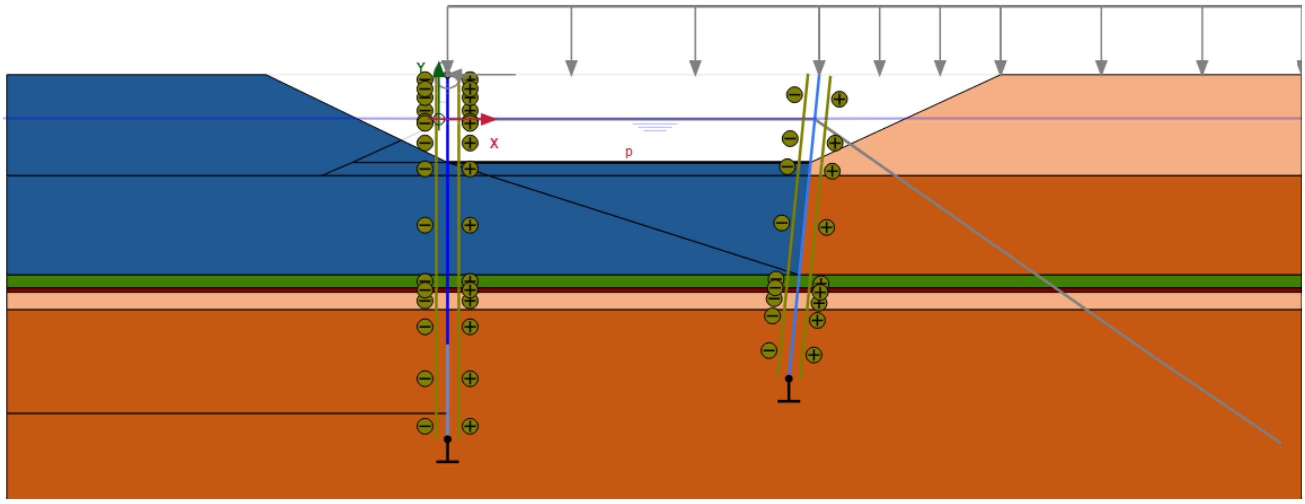

Figure 74 The installation and pre-stressing of the lower tie rods.

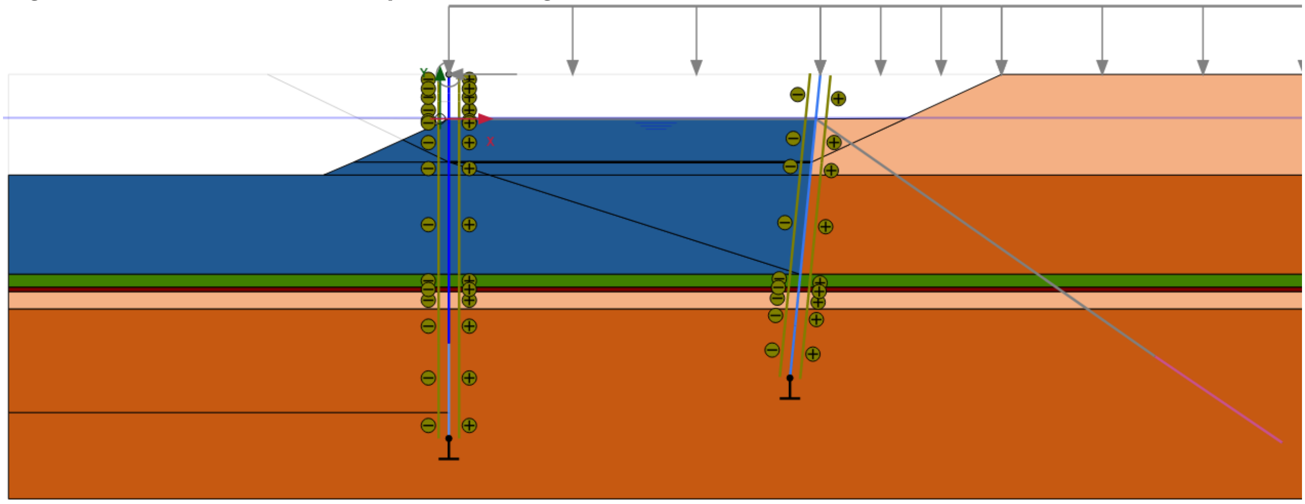

Figure 75 The terrain backfilled to the level of the second tie rods.

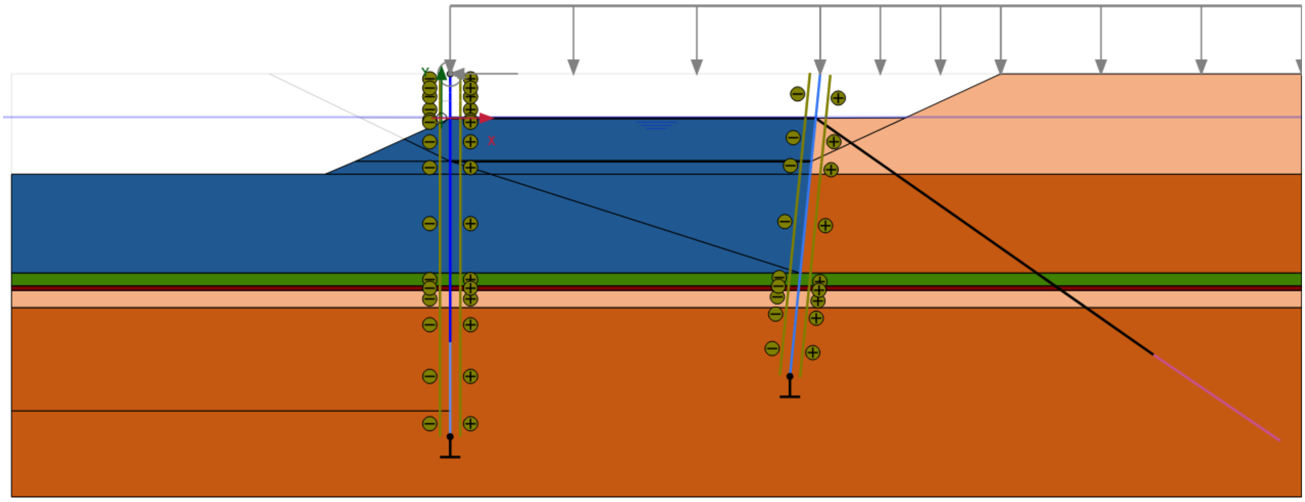

Figure 76 The installation of the upper tie rods and grout anchors.

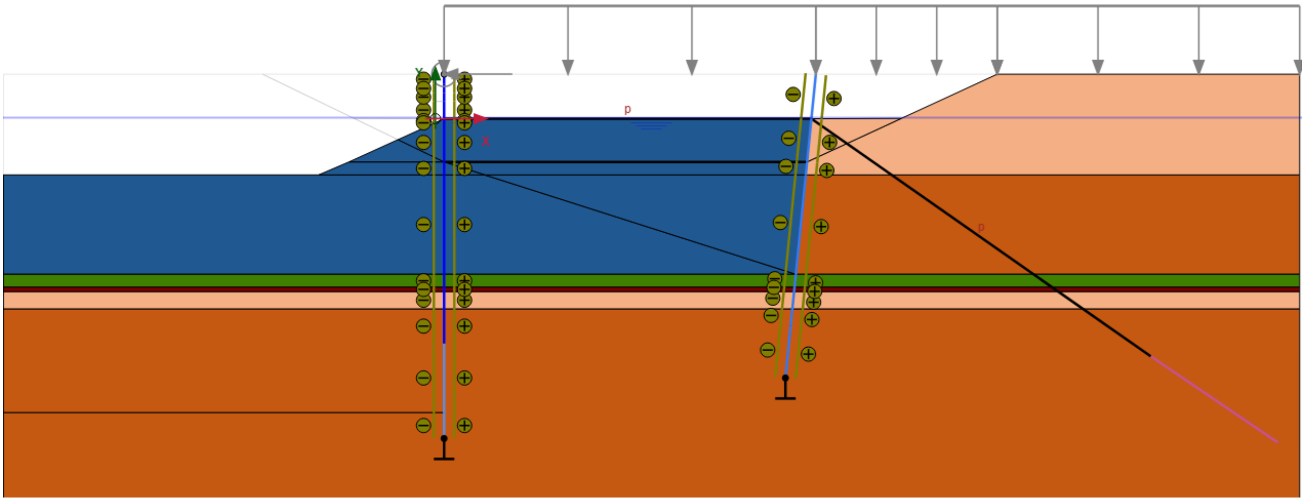

Figure 77 Pre-stressing of the tie rods and the grout anchors.

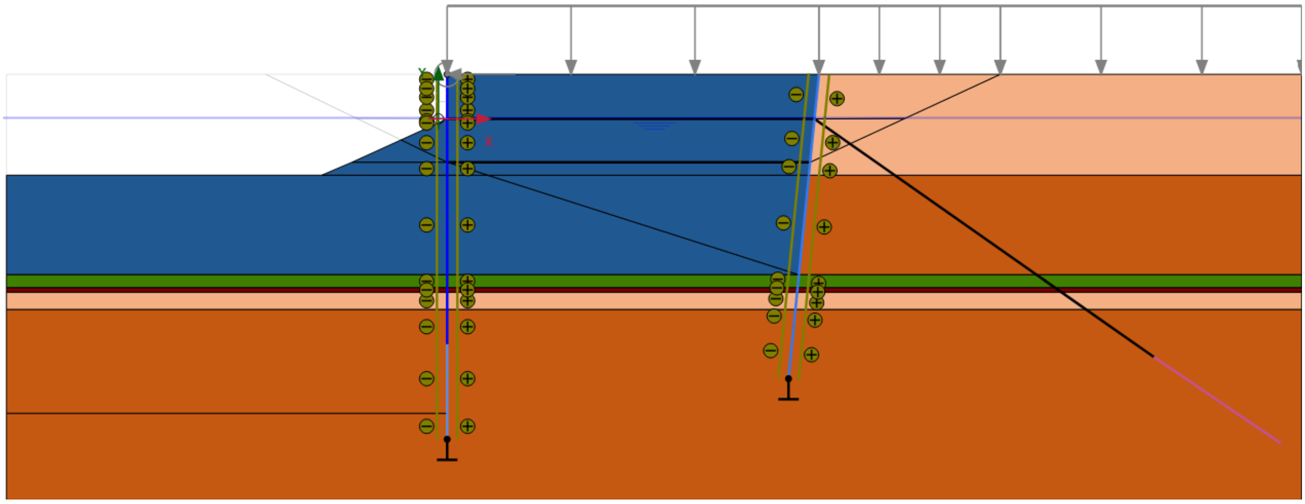

Figure 78 Backfill of the cofferdam and terrain.

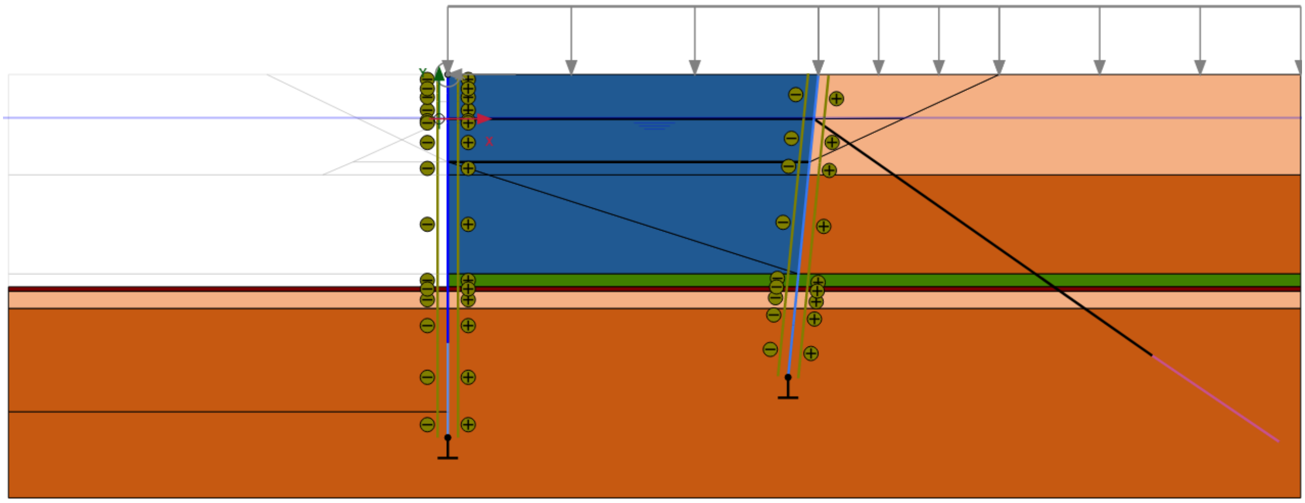

Figure 79 Dredging in front of the quay.

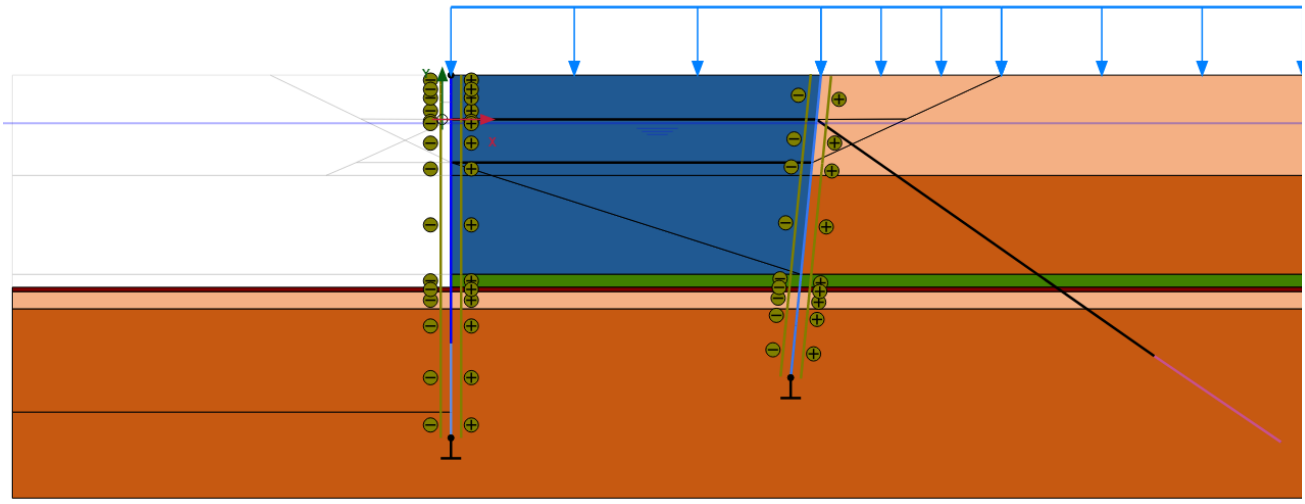

Figure 80 Applying design loads and water levels for SLS and ULS conditions.

Appendix G: Model validation

For the model validation, several screenshots of the model are provided as explained in Section 6.2.2.

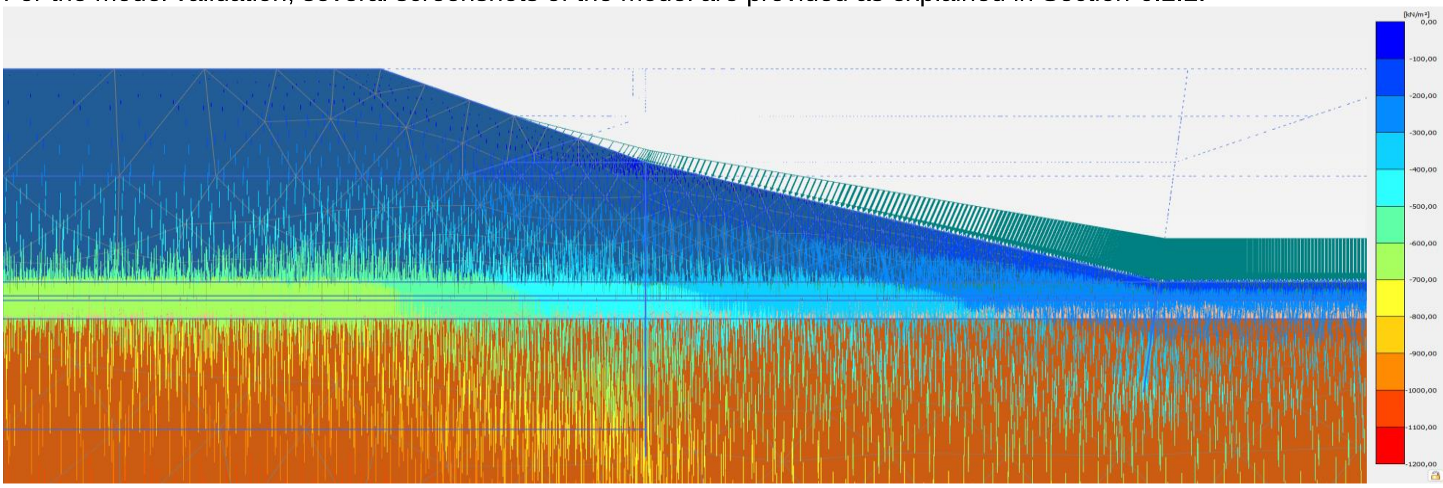

Figure 81 Principal total soil stresses indicated for the initial situation.

The green-coloured layer, as can be seen in Figure 81, indicates a principal total soil stress of 600-650 kN/m^2. The depth of the layer is at -20 m + NAP. The top level is at +5 m NAP.

The calculation by hand is shown in Table 26.

Table 26 Total stress hand calculation results.

Parameter	Value
Soil density (saturated)	26 kN/m^2
Depth from top-level	25 m
Total stress(depth*density) per meter	650 kN/m^2

Extra bollard load

The bollard load is increased by 50%. This has a positive effect on the deformation of the front wall and the tie rod forces.

Parameter	Value
Bollard load	1500 kN
Tie rod 1 (ULS)	1297 kN
Tie rod 2 (ULS	2057 kN
Bollard load	2250 kN
Tie rod 1(ULS)	1174 kN
Tie rod 2(ULS)	1865 kN