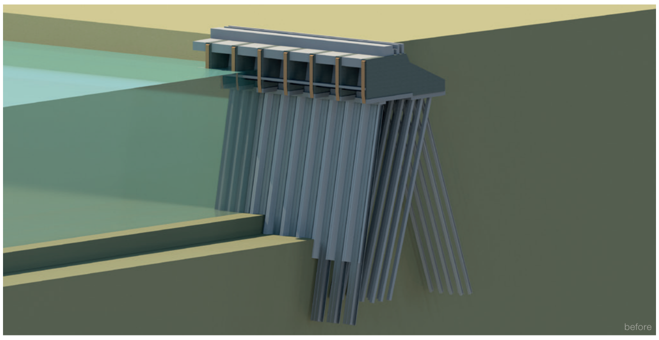
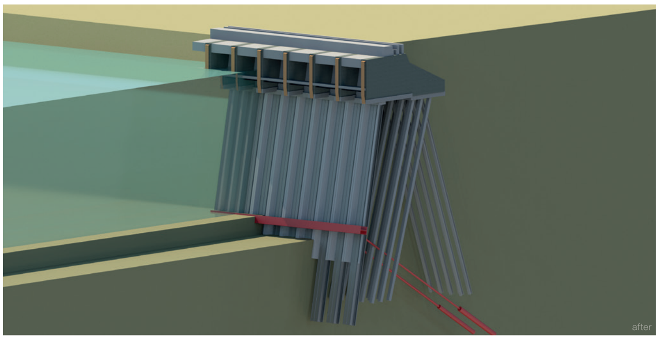
Port of Rotterdam





DEEPENING OF AN EXISTING COMBI WALL

SEPERATELY PROVIDED REPORTS

ROTTERDAM UNIVERSITY
UNIVERSITY OF APPLIED SCIENCES

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Deepening of an Existing Combi Wall

Separately Provided Reports

12 June 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
Supervisor school:	**H.J. Dommershuijzen MSc**

Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

Table of contents

Introduction

In addition to the main final thesis report this report is arranged. This report provides the interim reports and documents for the Rotterdam University of Rotterdam which are separately provided. This document consists of the following documents in the same order.

Interim reports:

1. *Project Plan;*
2. *Executed deepening projects;*
3. *Reference structures Botlek area;*
4. *Structural engineering reference structure;*
5. *Inventory and preselection;*
6. *Trade-off selection;*
7. *Documents for the Rotterdam University of Applied Sciences:*
    - *List of achieved Competences;*
    - *Approval form for the database of the Rotterdam University of Applied Sciences;*

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1 Project Plan

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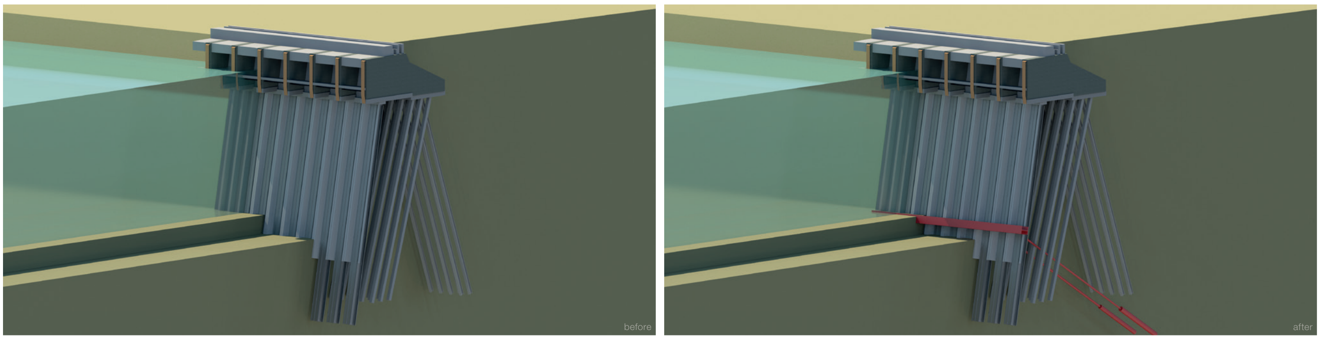
Port of Rotterdam



DEEPENING OF AN EXISTING COMBI WALL

PROJECT PLAN

ROTTERDAM UNIVERSITY
UNIVERSITY OF APPLIED SCIENCES

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Deepening of an Existing Combi Wall

Project plan

12 juni 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
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Summary

This project plan describes the approach of the graduation thesis project ”Project plan”. This project will be executed by Jordy Schutte, student of the University of applied sciences Rotterdam. This thesis is for graduating the bachelor Civil engineering at University of Applied Sciences Rotterdam.

The Port of Rotterdam Authority will be the supervisory company. Witteveen+Bos will advise the student during the graduation thesis. The project will be performed from February 1st 2017 until June 9st 2017.

The project will investigate growing solution for the reference quay wall of the Botlek area. Deepening the port bed is the reason a solution for the current quay structure needs to be investigated.

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1 General information

This document is intended to explain the approach of the graduation thesis of the study Civil Engineering at the University of applied science Rotterdam. The supervisory company will be the Port of Rotterdam Authority. The project will be supported by the engineering and consultation company Witteveen+Bos (W+B).

1.1 Port of Rotterdam Authority

The Port of Rotterdam Authority (PoR) manages, operates and develops the port and industrial area of Rotterdam and is responsible for maintaining a safe and smooth handling of all shipping. The organisation structure of the Port of Rotterdam Authority is consistent with this. The Port of Rotterdam Authority has a turnover of approximately €600 million and employs 1,100 people in a wide range of positions in commercial, nautical and infrastructural areas. The Port of Rotterdam Authority is an unlisted public limited company. The shares in the Port of Rotterdam Authority are held by the Municipality of Rotterdam (approx. 70%) and the Dutch government (approx. 30%).

1.2 Location of graduation thesis

The graduation thesis takes place at the main office of the Port of Rotterdam Authority. The main office of the Port of Rotterdam Authority is located in the World Port Centre at the WIlhelminapier 909 in Rotterdam.

The department Port Development of the Port of Rotterdam Authority will be the supervisory department for the graduation thesis.

1.3 Problem description

The global standards for the size and draft of the seagoing vessels are increasing, although the water depths of the port remain the same. The global increasing of the size and draft of the seagoing vessels are the reason the growing supply of mooring facilities with more draft.

The Port of Rotterdam Authority is aware of the increasing supply of mooring facilities with more draft. The Port of Rotterdam Authority will execute different deepening project. This graduation thesis will focus on deepening project of the Botlek area.

The Botlek area is one of the biggest Petrochemical harbour complexes of Europe. For that reason, the Botlek area is a valuable area for the Port of Rotterdam Authority. The current position in the petrochemical industry is one of the best of Europe.

The Port of Rotterdam Authority wants to keep in a high position in Europe and the world, so the Port of Rotterdam needs to be attainable for the bigger seagoing vessels. The Port of Rotterdam Authority wants to receive as many vessels as possible. If the water level is below the draft of the vessel, the vessels are not able to enter the Port of Rotterdam, so the port needs to be deepened.The location of the Botlek area is presented in figure 1. The Botlek area is framed in the blue rectangle.

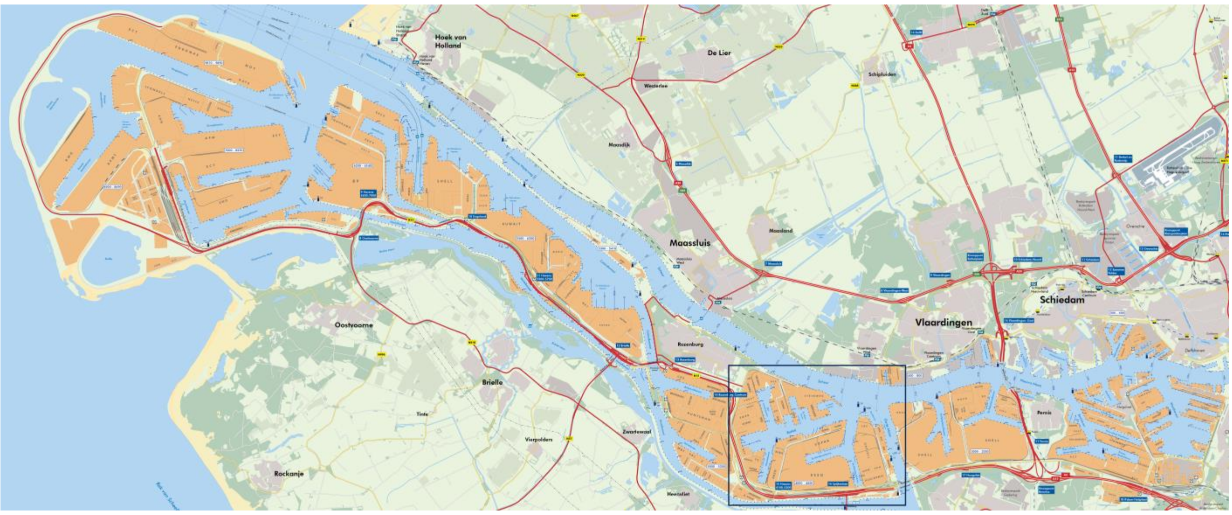

Figure 1 Port of Rotterdam overview with the Botlek area

The Port of Rotterdam Authority will invest in deepening the harbour and upgrading of their assets in the Botlek area the next years. These investments will also revive the Botlek area. The biggest investment considers the dredging of the Botlek port bed 2 meters from NAP–15.0 meters to NAP -17.0 meters.

After the Botlek area is dredged, larger vessels with more draft can enter and moor in the Botlek area. The deepening of the harbour can be a risk for the stability of the existing quay wall. It is nowadays unknown if the actual quay structure are still useful in the new situation after the deepening the harbour. This graduation thesis consists of 1 representative combi-wall structure.

Figure 2 displays an overview of the project location the Botlek area.

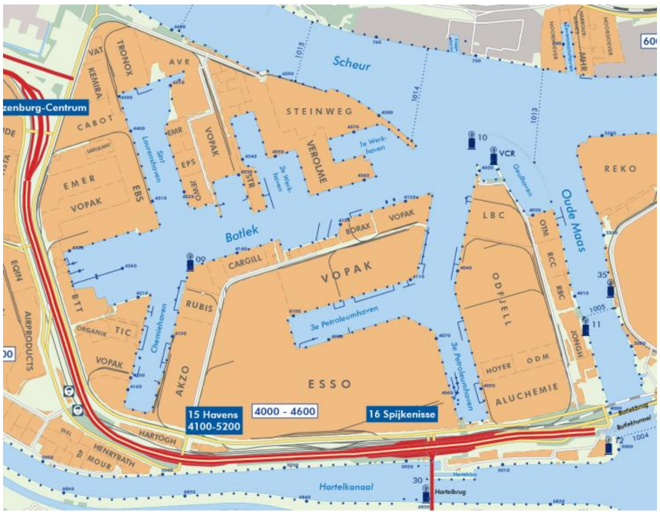

Figure 2 Overview of the problem location

1.4 Research purpose

The purpose of the research defines a suitable solution for the reference combi-wall structure. This solution should be suitable for more type of quay wall structures. The effects of the solution will be compared to the computer model of the reference quay structure made by the student.

1.5 Primary research question

The most important question for this research is:

What is the most optimal solution for deepening the construction depth of an existing combi wall structure, with at least 2 meters, without compromising reliability?

1.6 Secondary research questions

The secondary research questions which will be investigated are:

1. What adjustments have been made in the past, to deepened combi wall structures in the Port of Rotterdam and other ports?
2. What is a representative combi wall of the Botlek area?
3. What are the failure mechanism and critical structural members of the reference combi wall structure?
4. What are the preferred solutions for deepening a combi-wall structure?

1.7 Success of the project

The project will succeed if the following goals are accomplished:

- The Port of Rotterdam Authority can apply the thesis results in future projects;
- The student will graduate for the bachelor Civil Engineering;
- The University of Applied Sciences is pleased with the final result.

2 Project activities

The project will consist of nine internal phases, during these internal phases different research methods and software will be used.

2.1 Project approach

The project will be executed as the flowchart below.

The left side of the figure presents the internal phases and the right side presents the internal results.

Phases	•Main phase activities
Initiation	•Method of approach small version •Method of approach extended version
Preliminary investigation	•Inventory executed deepening projects quay structures in the past •Inventory and selection reference quay structure of the Botlek area and design conditions
Structural engineering reference model	•Modelling of soil-structure interaction reference combi-wall structure •Minimum requirements of the solutions
Inventory solutions	•Interview and brainstorm session •Inventory of deepening solutions
Preselection	•Preselection criteria •Preselected solutions
Final selection by trade-off matrix	•Trade-off matrix to final design •Effect of the deepening solution on the soil-structure interaction
Most preferred solution	•Substantiation of the most feasible solution

Figure 3 phases and internal results during graduation thesis

2.2 Activities per internal phases

Every internal phase can be divided into sub-activities as the following list:

1. Preparation
   1.1. Plan of approach
   1.2. Time schedule
2. Research reference design
   2.1. Literature research
   2.2. Research similar projects
   2.3. Research references of the design Botlek area Quay structure
   2.4. Research design conditions
   2.5. Finite Element Method reference Quay structure
3. Longlist alternative
   3.1. Interview experts
   3.2. Process the interview results
   3.3. Developing feasible alternatives into longlist
4. Shortlist alternatives by trade-off matrix
   4.1. Trade-off matrix of the longlist to shortlist
5. Structural engineering shortlist
   5.1. Finite Element Method calculation shortlist
6. Final solution by trade-off matrix
   6.1. Trade-off matrix of the shortlist to final design
7. Detailed engineering final design
   7.1. Finite Element Method extended calculation Final design
8. Final report
   8.1. Write main final report
   8.2. Write conclusions
   8.3. Write recommendations for the research
   8.4. Combine all the documents of phase 1 until 7
9. Thesis defence
   9.1. Prepare graduation thesis presentation
   9.2. Defence final report

2.3 Research methods

The methods which will be used for the graduation thesis are:

- Literature review;
- Field research;
- Interview sessions with experts;
- Brainstorm sessions with experts;
- Computer engineering calculations.

2.4 Research software

The software which will be used for the graduation thesis are:

- Portmaps, Port of Rotterdam Authority/ ESRI;
- Word, Microsoft;
- Excel, Microsoft;
- Plaxis Finite Element Method, Plaxis b.v.

3 Project results

The project results will be a final thesis with appendixes and a final presentation. To accomplish the final thesis and the final presentation, internal phase results have to be made.

3.1 Internal phase results

During this project several internal phase result will be made, the following results/products will be made:

1. Project plan;
2. Research executed deepening projects quay structure in the past;
3. Report references quay structures and design conditions of the Botlek area including Finite Element Method (FEM);
4. Interviews and brainstorm session reports;
5. Longlist of alternatives including trade-off matrix;
6. Shortlist of alternatives including computer calculations and trade-off matrix;
7. Calculation and final process final alternative;
8. Final thesis of the research with conclusions, recommendations and appendixes;
9. Final presentation thesis.

3.1.1 Research report executed deepening projects quay structures in the past

This report is about deepening project of quay structure in the past. Several projects with alternatives for the existing quay structure will be examined. The research consists of the Port of Rotterdam and other projects around the world. The result will be an overview of executed deepening project around the world with the adjustments of the existing quay structure.

3.1.2 Report references quay structures and design conditions of the Botlek area including Finite Element Method (FEM)

The reference quay structure and design conditions are the most relevant quay structure and the safety value, ground parameters and structural parameters of the reference structure. To investigate the weak/failure points of the structure, the structure will be modulated with the Finite Element Method software Plaxis. This modulated model will be the base model for the calculation of the alternatives. The research is intended to be modulated with Plaxis but in case of time D-sheet will be used instead of Plaxis. D-sheet will be the back-up plan for the modelling of the structure.

3.1.3 Longlist of alternatives including trade-off matrix

The longlist of alternatives will be made according to executed projects in the past, brainstorm and interviews sessions with experts and literature research. These alternatives will be filtered to a shortlist by a trade-off matrix. The trade-off matrix will contain the following criteria:

- 2,8 meter deepening;
- Multidisciplinary;
- Feasibility;
- New or upgrade.

3.1.4 Shortlist of alternatives including computer calculations and trade-off matrix

The alternatives of the shortlist will be modulated in Plaxis. The results of Plaxis will be added to the trade-off matrix. The feasibility of the alternative structures will also add to the final trade-off matrix. The final trade-off matrix will contain the following criteria:

- Global execution costs;
- Lifetime;
- Maintainable;
- Execution time
- Expendable;
- Results Plaxis.

3.1.5 Calculation and final underpinning best alternative

At the end of the research, the final best alternative is chosen by the trade-off matrix. This best alternative will be structural calculated in detail in Plaxis. The cost will be calculated in more detail and the technical drawing will be more detailed.

3.1.6 Final thesis of the research with conclusions, recommendations and appendixes

The final result of the graduation thesis will be the thesis report with conclusions, recommendations and appendixes. The next documents will be part of the final thesis:

- Project plan;
- Research executed deepening projects quay structures in the past;
- Report references quay structures and design conditions of the Botlek area including Finite Element Method (FEM);
- Interviews and brainstorm session reports;
- Longlist of alternatives including trade-off matrix
- Shortlist of alternatives including computer calculations and trade-off matrix;
- Calculation and final underpinning best alternative.

4 Project time schedule and organisation

The project will be executed according to the time schedule and with the presented project organisation.

4.1 Time schedule

A time schedule is made based on the activities mentioned in chapter 2 Project activities. In this time schedule all the activities are scheduled within the given timespan for this thesis. The project will be executed from February 1st 2017 until June 31st 2017 (22 weeks). Inside of that period the following deadlines has to be attained:

- Hand in interim report (1 week before the interim presentation)
- Interim presentation (around half April, to be scheduled)
- Hand in draft final report (at least 3 weeks before final report deadline)
- Go/No Go – Meeting with teacher (at least 2 weeks before final report deadline)
- Hand in 3 Thesis reports + 1 USB stick + company evaluation + Authorization form (June 13th)
- Thesis defense (23 June - 4 July)

The period of research and writing of the graduation thesis is from February 1st 2017 until June 9th 2017 (18 Weeks). The detailed time schedule is attached in **Fout! Verwijzingsbron niet gevonden.**. The national holidays and the study trip to Madagascar from April 2nd 2017 until April 15th 2017 are included in the time schedule. The time schedule is made in consultation with the supervisors and can be adjusted when necessary.

The global time schedule for the research is presented in the bullet points below:

- Preparation 1 week;
- Research Reference design 4 weeks;
- Longlist alternatives 2 weeks;
- Shortlist alternatives by trade-off matrix 1 weeks;
- Structural engineering shortlist 3 weeks;
- Final alternative by trade-off matrix 1 week;
- Detailed engineering final design 2 weeks;
- Final Report 3 weeks;
- Thesis Defence 2 weeks.

4.2 Project organisation

The organisation for the graduation thesis project consist on a student, supervisors and an advisor.

4.2.1 Student

Name:	J.J. Schutte (Jordy)
Function:	Graduation student University of applied sciences Rotterdam
Address:	Klarinetsingel 136, 3335DE, Zwijndrecht, the Netherlands
Telephone number:	+31 6 28 98 30 11
email address 1:	jj.schutte@portofrotterdam.com
email address 2:	jordyschutte@hotmail.com
Availability:	Monday, Tuesday, Wednesday, Thursday, Friday

4.2.2 Supervisors

Name:	ir. A.A. Roubos (Alfred)
Function:	Supervisor PoR Project Engineer PoR
Address:	Wilhelminakade 909, 3072 AP Rotterdam, the Netherlands
Telephone number:	+31 6 52 84 52 96
email address 1:	aa.roubos@portofrotterdam.com
Availability:	Tuesday, Thursday, Friday

Name:	ir. H.J. Dommershuijzen (Harry)
Function:	Supervisor University of applied sciences Rotterdam (UoASR) Lecture UoASR
Address:	G.J. de Jonghweg 4-6, 3015 GG Rotterdam, the Netherlands
Telephone number:	+31 6 13 49 08 23
email address 1:	h.j.dommershuijzen@hr.nl
Availability:	Monday, Tuesday, Wednesday, Thursday, Friday

4.2.3 Advisor

Name:	ir. D.J. Jaspers Focks (Dirk-Jan)
Function:	Advisor Witteveen+Bos (W+B) Team leader Geotechnical and Hydraulic Engineering W+B
Address:	Leeuwenbrug 27, 7400 AE Deventer, the Netherlands
Telephone number:	+31 6 20 94 50 82
email address 1:	D.jaspersfocks@witteveenbos.nl
Availability:	Monday, Tuesday, Wednesday, Thursday, Friday

4.2.4 Relations

The relations between the student, supervisors and the advisor are presented on Table 1 Relations project organisation.

Person(s)	Relation	Person(s)
Student	Deliver graduation thesis to	Supervisor UoASR Supervisor Por Advisor W+B
Supervisor PoR	Deliver company evaluation to	Student Supervisor UoASR
Supervisor PoR	Deliver graduation thesis evaluation to	Student Supervisor UoASR
Supervisor UoASR Supervisor PoR Advisor W+B	Deliver support and expertise to	Student
Supervisor UoASR Supervisor PoR	Evaluate graduation thesis defense with	Student

Table 1 Relations project organisation

4.3 Meetings

During the execution of the project, 2 visits at the graduation location of the supervisor UoASR will be planned. The supervisor PoR will also be present at these meetings. The student is responsible for scheduling these meetings.

Meetings with the supervisor PoR will take place at least every week. If the supervisor PoR is not available, a meeting with one of his colleague will be scheduled.

Meeting with the supervisor UoASR will take place at least every 3 weeks.

If an internal phase result is completed, a meeting will be scheduled to review the documents. This meeting will be with all the members of the project organisation.

The meetings with the supervisor UoASR can be by phone, but the preference is an actual meeting.

Meetings with the advisor W+B will take place when the student need advice.

5 Quality

The quality of the products will be maintained by following up the points below.

5.1 Maintaining quality

To maintain the quality of the products each product will be planned, made, reviewed and revised. This cycles can be done as the plan, do, check and act method. The completed product will be checked and approved by the supervisor PoR and the supervisor UoASR. The products are final if the feedback of the supervisors is processed in the products.

5.2 Layout

The documents for this project will be created in according to the Huisstijl handbook of Port of Rotterdam Authority. Every document will have the same header and footer as this document.

5.3 References

References which are used in the documents will be listed at the end of the documents by the APA-method.

5.4 Archiving

The completed products will be saved to a Dropbox folder and on a USB-stick. Twice save the documents will occur that documents can be lost.

6 Competencies

The thesis had to be checked by the competencies set by the University of Applied Sciences Rotterdam. The competencies are made to maintain a high-quality level. The student needs to measure the competencies during the thesis. The target competencies of the thesis which has to be achieved can be pictured as the following Table 2 and Table 3.

Level	Factors	Description
1	Task	Simple, structured, application of known methods
	Context	Known, simple, monodisciplinary
	Independency	Directive counseling (directed by teacher)
2	Task	Complex, structured, application of known methods in variable situations
	Context	Known, complex, monodisciplinary, practical project under supervision
	Independency	Counselling if necessary
3	Task	Complex, unstructured, application of methods adapted to the situation
	Context	Unknown, complex, multidisciplinary, in practice
	Independency	Independent

Table 2 level of competencies

Number	Competences	Number	Core tasks	Product
1	Initiate	1.1	Detect and analyse (the need of) a civil engineering project in the built environment	Thesis report and thesis presentation
		1.2	Develop a list of requirements for a civil engineering project	
2	Design	2.1	Design solution variants. For example schemes, drawings and/or calculations for civil engineering problems	Thesis report and thesis presentation
		2.2	Rate solution variants and choose the most suitable	
		2.3	Calendaring and collection of information	
3	Specify	3.1	Schematization of the real situation in a (simplified) calculation model	Thesis report and thesis presentation
		3.2	Detailing and/or calculating and drawing of a (part of a) civil	
		3.3	Setting up of contract-, costing- license documents and organize and accompany the forming of contracts	
4	Realize	4.1	Preparing for implementation of a civil engineering project	Internship, 3th year
		4.2	Building site managing: Leading, guarding, evaluating and optimizing of the building process based on a building plan	
5	Maintain	5.1	Setting up a plan for maintaining and managing civil engineering projects	Course managing and maintaining, 3th year
6	Monitor, test and evaluate	6.1	Using the plan-do-check-act cycle	Thesis report and thesis presentation and operating in the company
		6.2	Environment-conscious and socially responsible handling	
7	Analysing and investigating	7.1	Executing research	
8	Communicating and cooperating	8.1	Describing and visualizing of information	
		8.2	Operate in teams	
9	Managing and undertaking	9.1	Control the personal learning process	
		9.2	Working structured and lead the processes	

Table 3 compencies to be achieved

7 Bibliography

Port of Rotterdam Authority. (n.d.). *Baggeren voor een betere doorvaart*. Retrieved February 1, 2017, from Havenkrant: https://www.portofrotterdam.com/nl/havenkrant/havenkrant-editie-30/baggeren-voor-een-betere-doorvaart

Port of Rotterdam Authority. (n.d.). *Organisation*. Retrieved February 2, 2017, from https://www.portofrotterdam.com/en/port-authority/organisation

Witteveen+Bos. (n.d.). *Key facts and figures*. Retrieved February 3, 2017, from Witteveen+Bos: http://www.witteveenbos.com/en/key-facts-and-figures

Witteveen+Bos. (n.d.). *Organisation*. Retrieved February 3, 2017, from Witteveen+Bos: http://www.witteveenbos.com/en/organisation

2 Executed deepening projects

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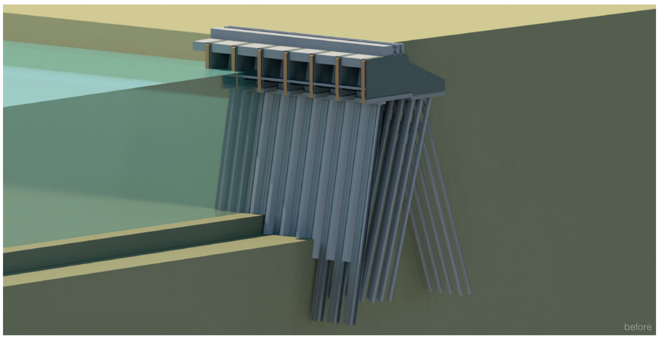

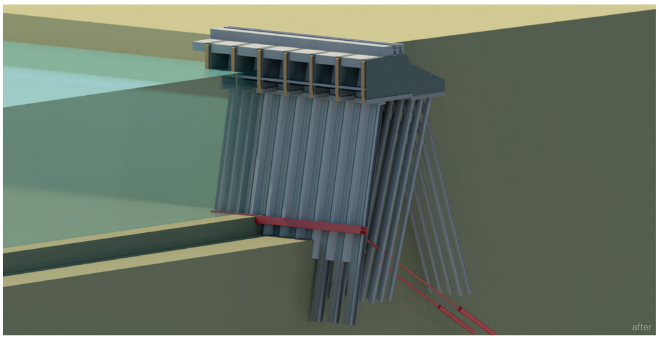

DEEPENING OF AN EXISTING COMBI WALL

EXECUTED DEEPENING PROJECTS

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Deepening of an Existing Combi Wall

Executed deepening projects

12 juni 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
Supervisor school:	**H.J. Dommershuijzen MSc**

Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

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Summary

The global standards for the size and draft of the seagoing vessels are increasing, although the water depths of the port remain the same. The global increasing of the size and draft of the seagoing vessels are the reason of the growing demand for mooring facilities with more draft.

The demand for mooring facilities with more draft forces the Port Authorities around the world to deepen the ports. Several deepening projects are executed in front of quay wall structure. The deepening of the port bed in front of quay wall structures can be a risk for the stability of the existing quay wall structure. The quay wall structure needs to be adjusted to remain the same safety value after the deepening works. Two projects in Rotterdam and four projects around the world are analysed.

The deepening projects which are investigated are:

- Sint Laurenshaven, Botlek, Port of Rotterdam, The Netherlands;
- Pier 6, Waalhaven, Port of Rotterdam, The Netherlands;
- Port of Felixstowe, United Kingdom;
- Port of Ravenna, Italy;
- Port of Kaohsiung, Taiwan;
- Port of Yokohama, Japan.

The deepening of the port beds are done next to several types of quay structures. The types of quay wall structures are a caisson, a combi-wall with relieving floor and an anchored sheet pile wall. The solutions for the quay wall structures are different.

The executed solution for the investigated deepening projects are:

- Add asphalt matrasses in front of the existing structure;
- Inject the ground in front of the existing structure with grout;
- Add an extra wall in front of the existing structure;
- Add a low underwater anchor near to the port bed;
- Add an extra wall with an addition anchor in front of the existing structure;
- Inject the ground behind the existing structure with grout.

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Table of contents

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1 Introduction

The global standards for the size and draft of the seagoing vessels are increasing, although the water depths of the port remain the same. The global increasing of the size and draft of the seagoing vessels are the reason of the growing demand for mooring facilities with more draft.

The global grown of the size and draft forces the Port Authorities around the world to deepen the port beds. Several deepening projects are executed in front of quay wall structure. The deepening of the port bed in front of quay wall structures can be a risk for the stability of the existing quay wall structure.

The quay wall structure needs to be adjusted to remain the same safety value after the deepening works. Two projects in Rotterdam and four projects around the world are analysed. The analysed deepening project in front of quay wall structure are described in the following chapters.

2 Deepening projects Port of Rotterdam

The Port of Rotterdam Authority is aware of the increasing demand of mooring facilities with more draft. The Port of Rotterdam Authority executed different deepening project. Two deepening project with the solutions for the quay wall structure will be descripted in this analysis.

2.1 Sint Laurenshaven

The quay wall structure is in the Port of Rotterdam, the Netherlands. The deepening in front of the quay wall is executed in the Sint Laurenshaven in the Botlek area. Figure 1 and

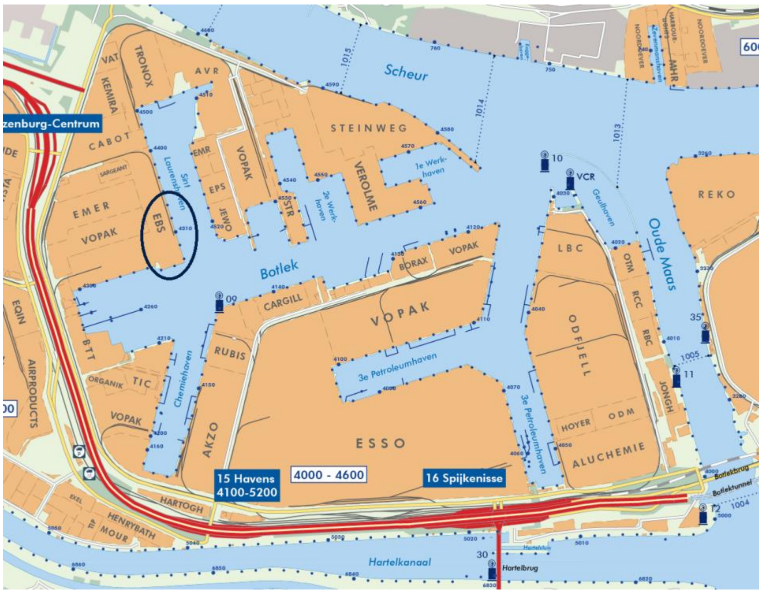

Figure 2 presents the location of the deepening project relative to Europe and the Botlek area.

Figure 1 Location deepening project Sint Laurenshaven relative to Europe

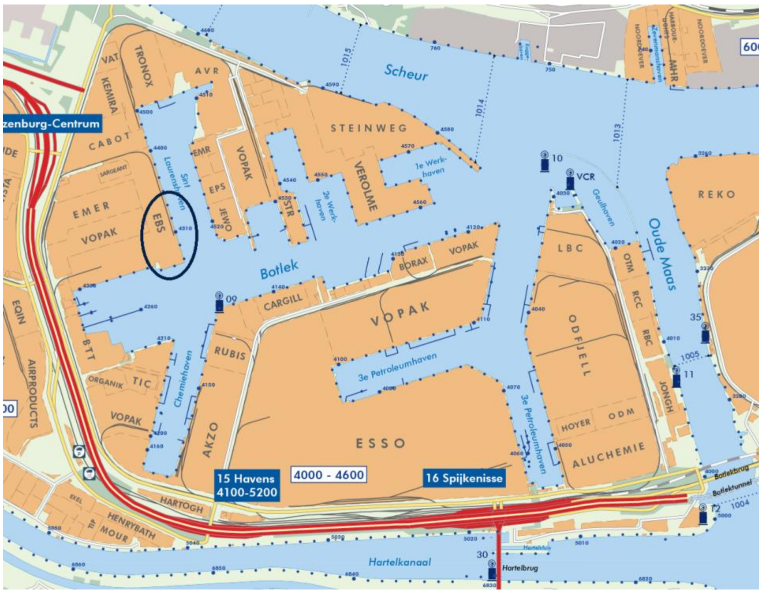

Figure 2 Location deepening project Sint Laurenshaven relative to the Botlek

2.1.1 Type of structure

The structure of the Sint Laurenshaven terminal is a combi-wall with a low relieving structure. The combined wall is made of double king piles and shorter intermediate sheet piles. The original port bed level is -14.00 meters. The principle cross section of the structure before the executed deepening and adjustments are pictured on

Figure 3 Cross section Sint Laurenshaven before deepening.

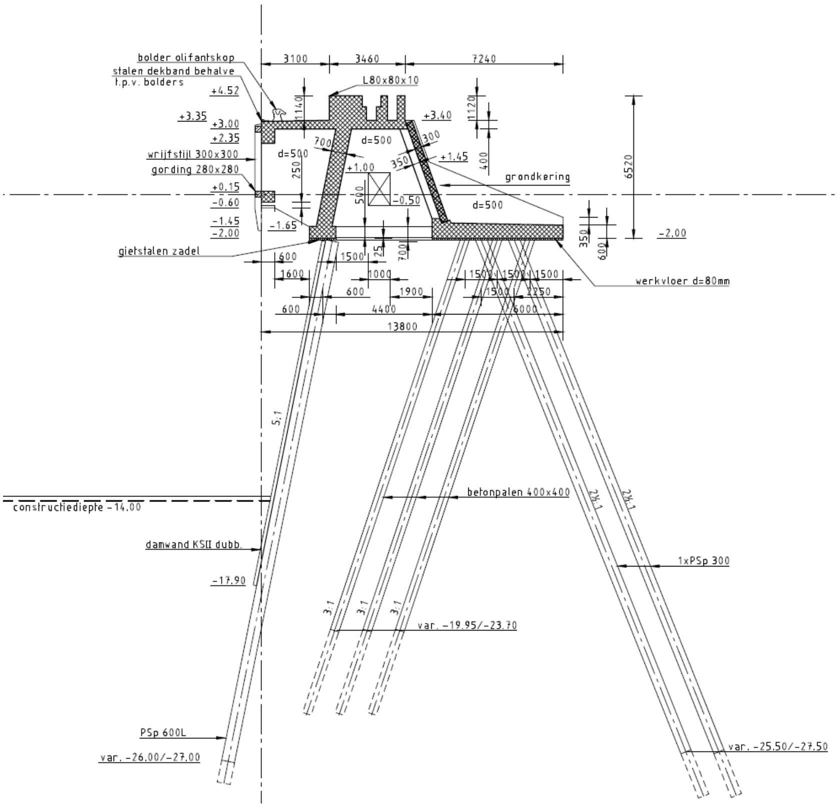

Figure 3 Cross section Sint Laurenshaven before deepening

2.1.2 Solution

The deepening could be done after the quay wall structure has been adjusted. The solution for the quay wall structure is the addition of asphalt matrasses with crushed stones in front of the quay wall. The solution provides an additional load in front of the quay wall. That additional load increases the passive ground pressures. The additional pressure of the passive ground by the asphalt matrasses provides the same equilibrium, so it is possible to deepen the port bed one meter up to -15.00 meters. See Figure 4 for the principle cross section of the deepened quay structure with the asphalt matrasses and the crushed stones.

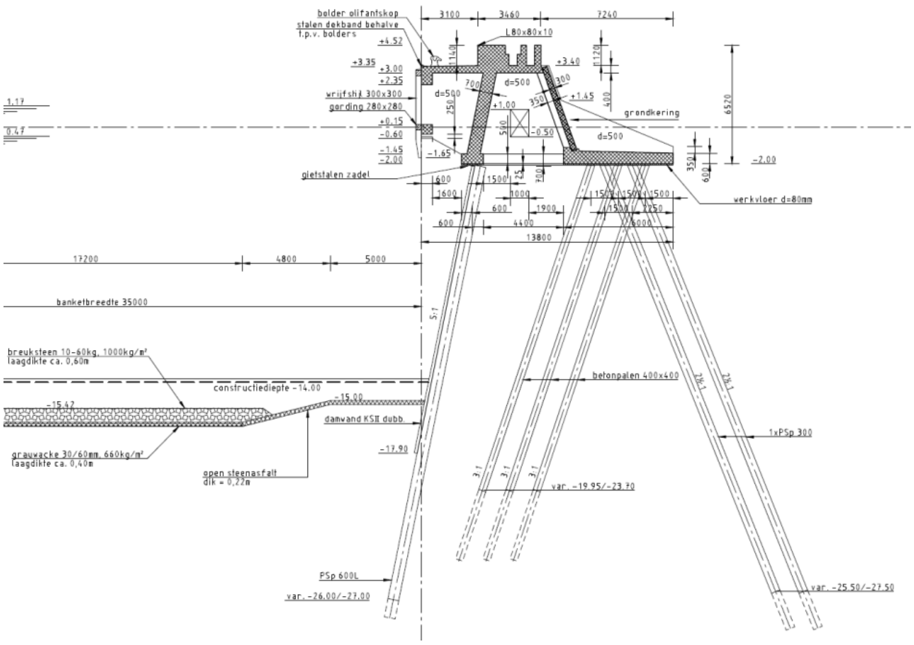

Figure 4 Cross section Sint Laurenshaven with asphalt matrasses after deepening

2.2 Waalhaven, Rotterdam

The achieved deepening of the project is 1.15 meters and 1.5 meters.

2.2.1 Location

The quay wall structures are in the Port of Rotterdam, the Netherlands. The deepening in front of the quay wall is executed near to Pier 5 and Pier 6 in the Waalhaven area. Figure 5 and Figure 6 presents the location of the deepening project relative to Europe and the Waalhaven area.

Figure 5 Location deepening project Pier 6 relative to Europe

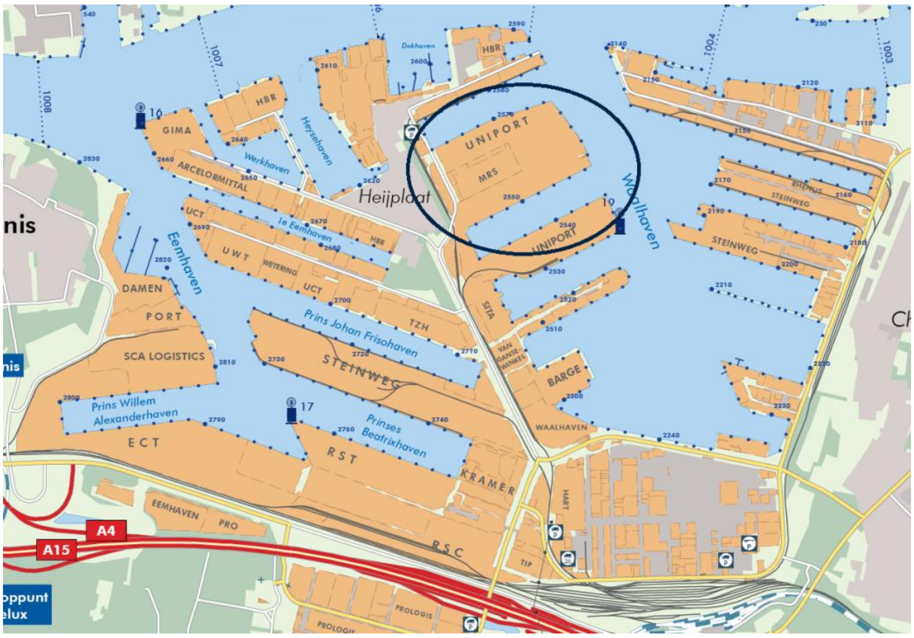

Figure 6 Location deepening project Pier 5 and Pier 6 relative to the Waalhaven area

2.2.2 Type of structure

The quay wall structure of Pier 5 is a caisson structure. The caisson structure is part of the gravity wall category of quay wall structures. That structure was damaged during the second world war. For that reason the structure must be repaired, as addition the structure is also deepened. and The original port bed level was -10.50 meter. The principle cross section of Pier 5 is de base of the drawing of the solution in Figure 8.

The quay wall structure of Pier 6 is a caisson structure. The caisson structure is part of the gravity wall category of quay wall structures. The original port bed level is -12.00 meter. The original cross section before the adjustment and the deepening is shown in Figure 7.

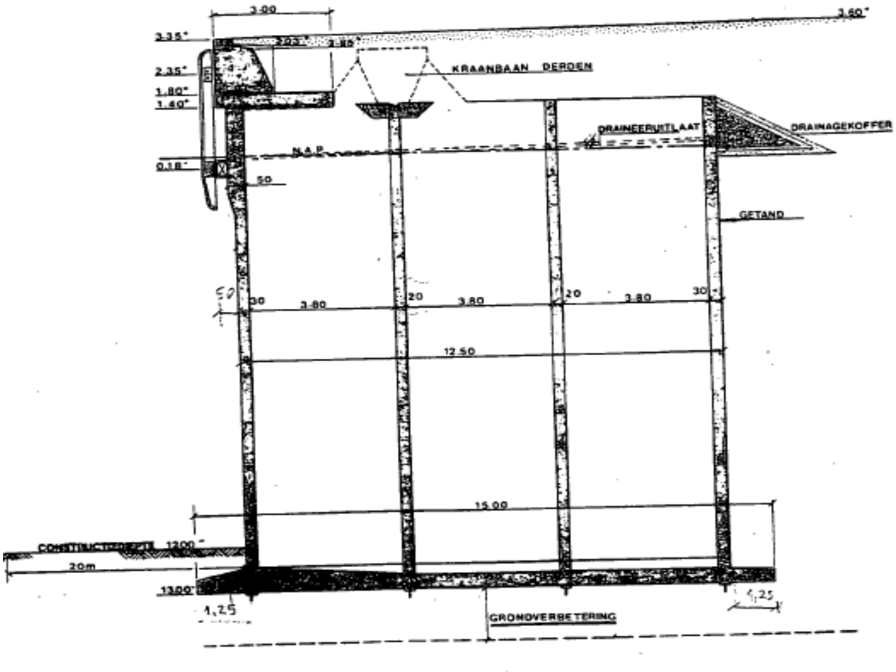

Figure 7 original caisson structure Pier 6 before deepening

2.2.3 Solutions

Pier 5 is adjusted as new structure, which includes the partly removal of the old caisson and the construction of a front wall with a relieving structure with bearing and tension piles. These elements are shown in Figure 9. The achieved deepening is 1.15 meters.

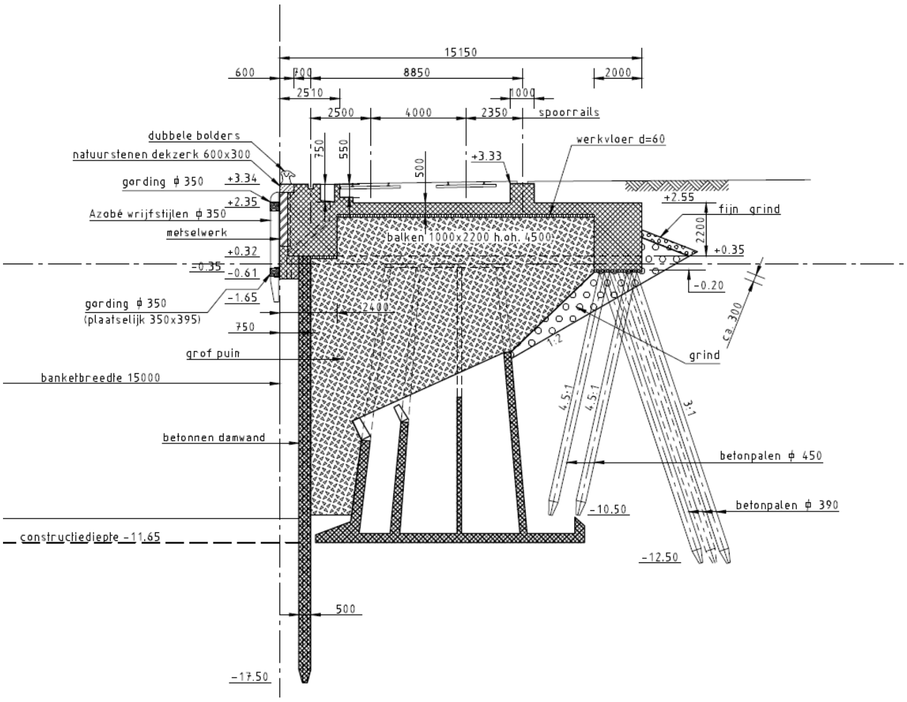

Figure 8 principle cross section of the deepening of Pier 5

The solution of Pier 6 is to inject the ground in front of the caisson with grout. The addition of the grout injection prevents the geotechnical failure of the overall stability. The resistance and the mass of the back-force moment are increased so the caisson is overall stable. The illustration of the grout injected ground and the caisson are pictured in Figure 9.

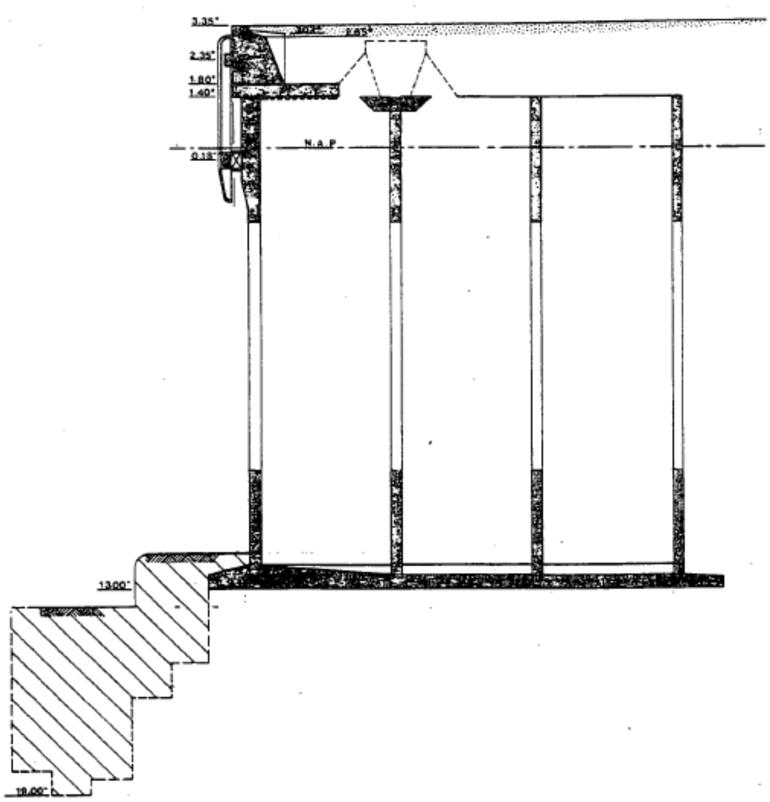

Figure 9 cross section Pier 6 with grout injection in front of the caisson

3 Deepening projects around the world

The global increasing of the vessel drafts forces the port authorities all over the world to deepen the port. Four deepening projects around the world will be discussed in the subchapter below.

3.1 Felixstowe, United Kingdom

The Port of Felixstowe is Britain’s biggest and busiest container port of the United Kingsdom, and one of the largest in Europe. The port of Felixstowe is dealing with 42% of Britain's containerised trade.

The port handles more than 4million TEUs (Twenty-foot Equivalent Units) and welcomes approximately 3,000 ships each year, including the largest container vessel.

The deepening is executed in 1983. The achieved deepening of the project is 2.04 m.

3.1.1 Location

The quay wall structure is in the Port of Felixstowe, the United Kingdom. The deepening in front of the quay wall is executed in front of the Languard terminal. Figure 10 and Figure 11 presents the location of the deepening project relative to Europe and the Port of Felixstowe.

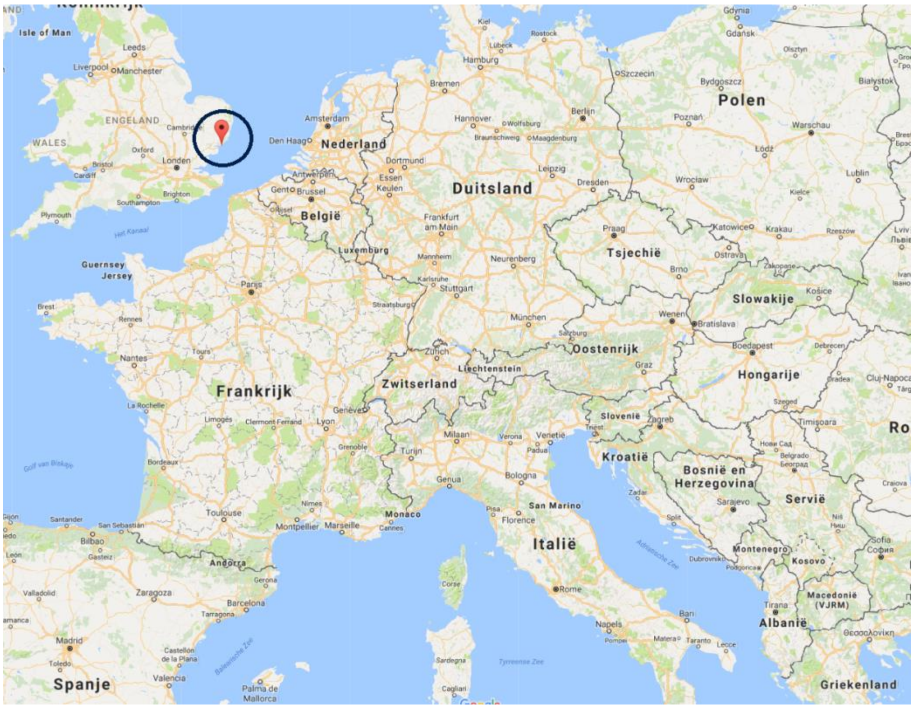

Figure 10 Location deepening project Languard terminal relative to Europe

Figure 11 Location deepening project Languard terminal relative to the port of Felixstowe

3.1.2 Type of structure

The original type of quay wall structure is a combi-wall with anchorage. The wall consist of king pile with intermediate Larssen piles. The original port bed level is -11.88 meter. The structure is anchored by an anchor wall with tension strings. The original cross section is shown in Figure 12.

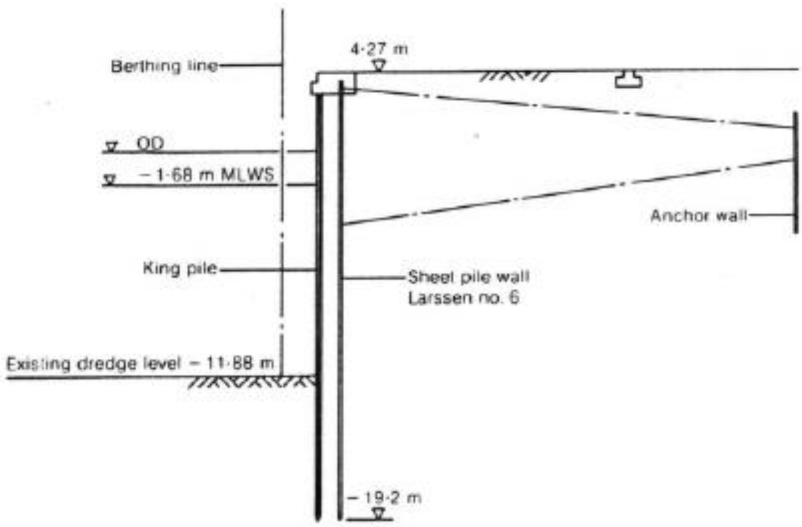

Figure 12 Cross section Languard terminal before deepening

3.1.3 Solution

The executed solution is an extra wall in front of the existing wall structure. The additional wall contains of 1.067 meter diameter tubular piles. The additional wall is connected to the existing wall by a concrete connection. The additional wall extends the wall length so the passive ground pressure surface and the active pressure will be equilibrium. See Figure 13 for the illustration of the additional piles with the concrete connection.

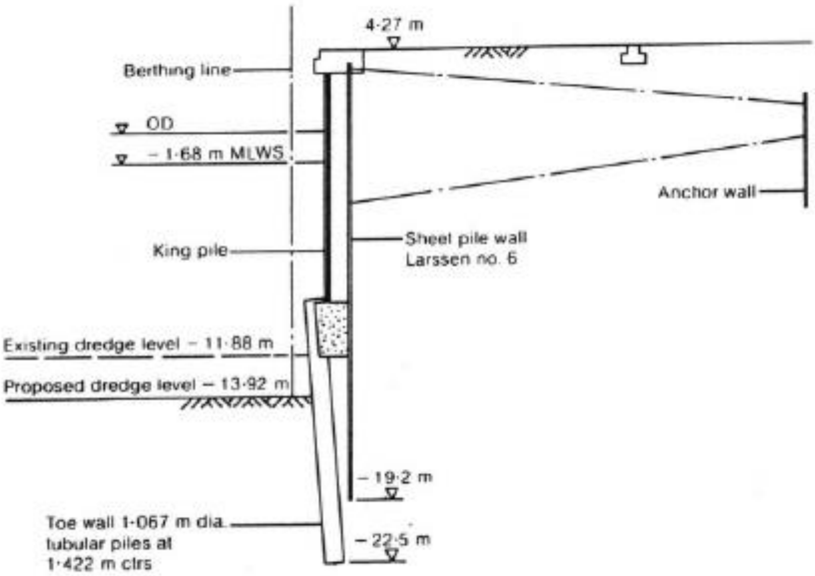

Figure 13 Cross section Languard terminal with the additional wall

3.2 Port of Ravenna, Italy

The Port of Ravenna represents the only port in the Emilia-Romagna Region of Italy.

By virtue of its strategic geographic position, the Port of Ravenna is a leading port in Italy for its trade with the markets of the Eastern Mediterranean and Black Sea (almost 40% of the national total and excluding coal and oil products) and plays an important role as regards trade with the markets of the Middle and Far East.

3.2.1 Location

The quay wall structure is in the Port of Ravenna, Italy. The deepening is executed in the inner port of Ravenna. Figure 14 and Figure 15 presents the location of the deepening project relative to Europe and the Port of Ravenna.

Figure 14 Location deepening Ravenna port relative to Europe

Figure 15 Location deepening project relative to the port of Ravenna

3.2.2 Type of structure

The structure of the port of Ravenna is a diagraph wall with relieving floor. The cross section before deepening is not available so the principle of a wall with high relieving platform is added for the illustration of the quay wall structure. The original port bed level is -10.00 meter. The relieving floor is supported by two bearing and one tension pile. See Figure 16 for the principle cross section of a wall with relieving platform.

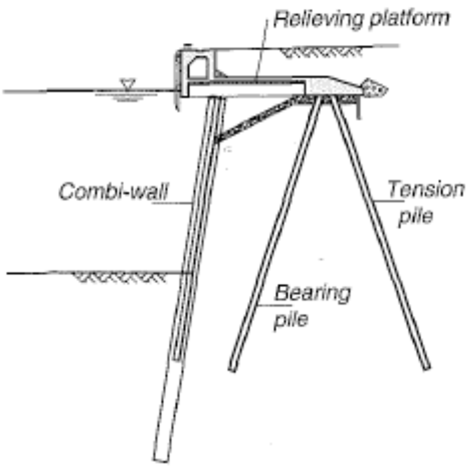

Figure 16 Principle of a high relieving platform

3.2.3 Solution

The deepening of the quay wall structure is achieved by the admission of an underwater anchor at -8.00 meter. The admission of the anchorage is theoretical underpinned and experimental tested. The results of the theoretical underpinning and experimental field results are close to each other. The positive (+) and negative (-) results of the tests are:

+ increasing of the factor of safety;
+ reducing of the bending moments;
- increasing of the horizontal displacements;
- increasing of the original anchor forces.

The structural failure of the front wall is prevented by reducing the bending moments. the achieved deepening is 2.00 meter. The illustration of the solution of the structure is pictured in Figure 17.

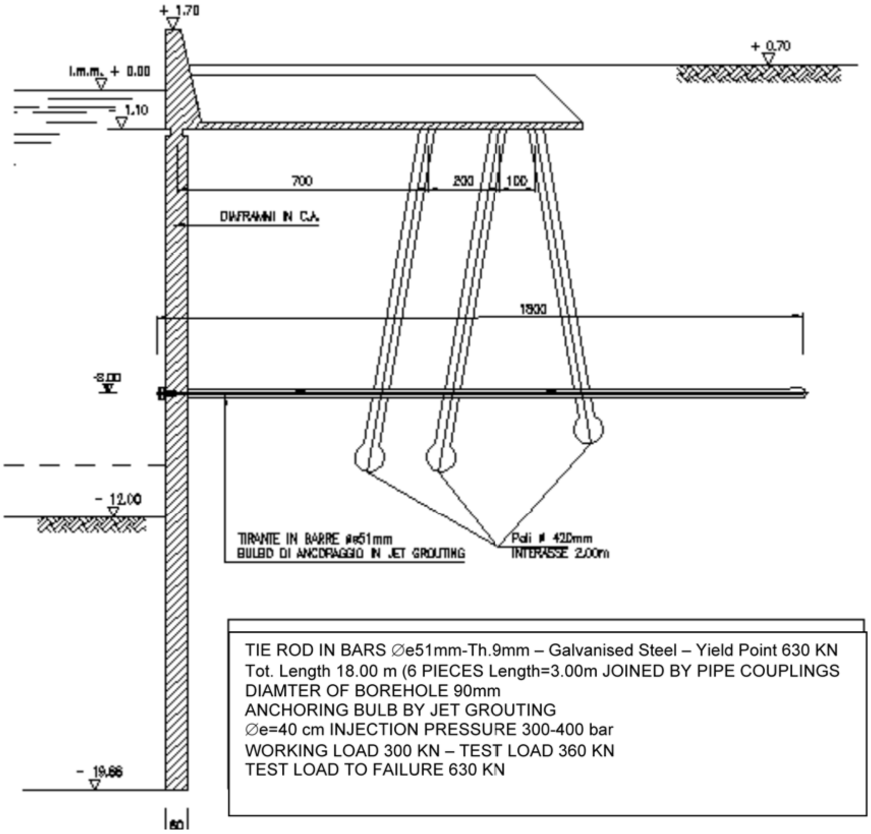

Figure 17 Cross section Ravenna with an additional underwater anchor

3.3 Port of Kaohsiung, Taiwan

The Port of Kaohsiung is the largest harbor in Taiwan, handling approximately 14 million twenty-foot equivalent units (TEU) worth of cargo in 2016. The port is located in southern Taiwan, adjacent to Kaohsiung City. It is operated by Taiwan International Ports Corporation, the Taiwan's only state-owned harbor management company.

3.3.1 Location

The quay wall structure is located in the Port of Kaohsiung, Taiwan. The deepening in front of the quay wall is executed in front of the container terminal . Figure 18 and Figure 19 figures the location of the deepening project relative to Europe, Asia and the Port of Kaohsiung.

Figure 18 Location of the deepening project relative to Europe and Asia

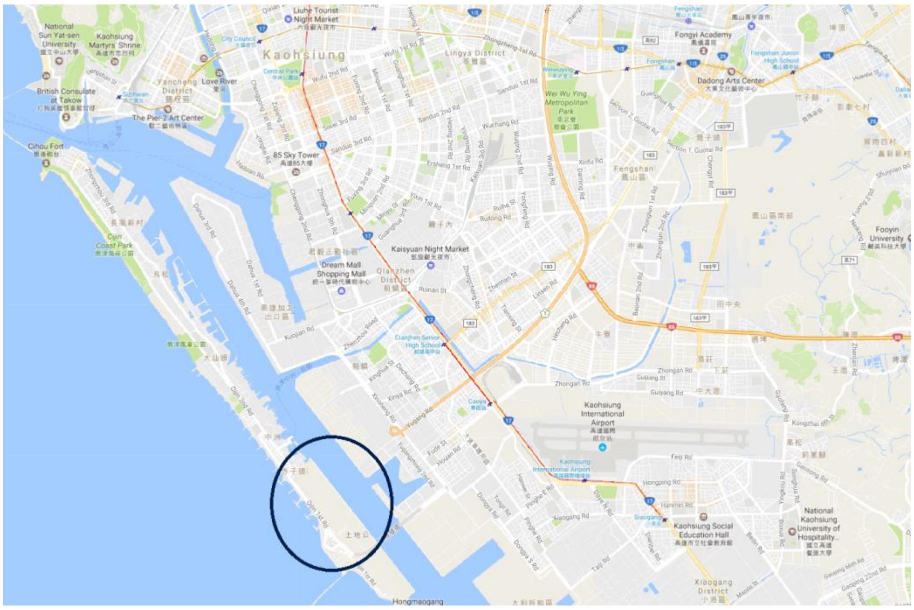

Figure 19 Location of the deepening project relative to the port of Kaohsiung

3.3.2 Type of structure

The current quay wall structure is an anchored sheet pile wall. The anchorage is done by a pile trestle. This pile trestle is also the foundation for the crane rail. The original port bed level is -12.00 meter. The illustration before the deepening project is not available, so the principle cross section of an anchored sheet pile is figured in Figure 20

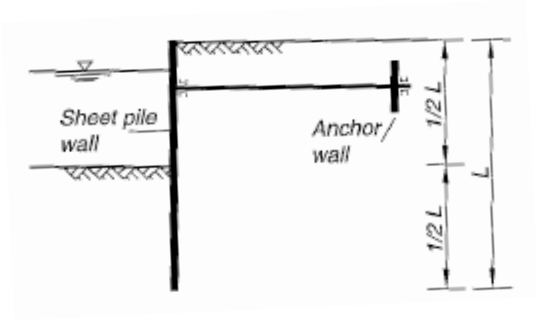

Figure 20 Principle of a anchored sheet pile wall with anchor wall

3.3.3 Solution

The solution for the deepening project of the Port of Kaohsiung is an additional sheet pile wall and an additional underwater anchor. The new sheet pile wall is connected to the original wall by a concrete fill so the two sheet pile walls can work as one. The failure of the structural strength and the failure of the passive ground pressure are resisted with this solution. The illustration of the solution is shown in Figure 21.

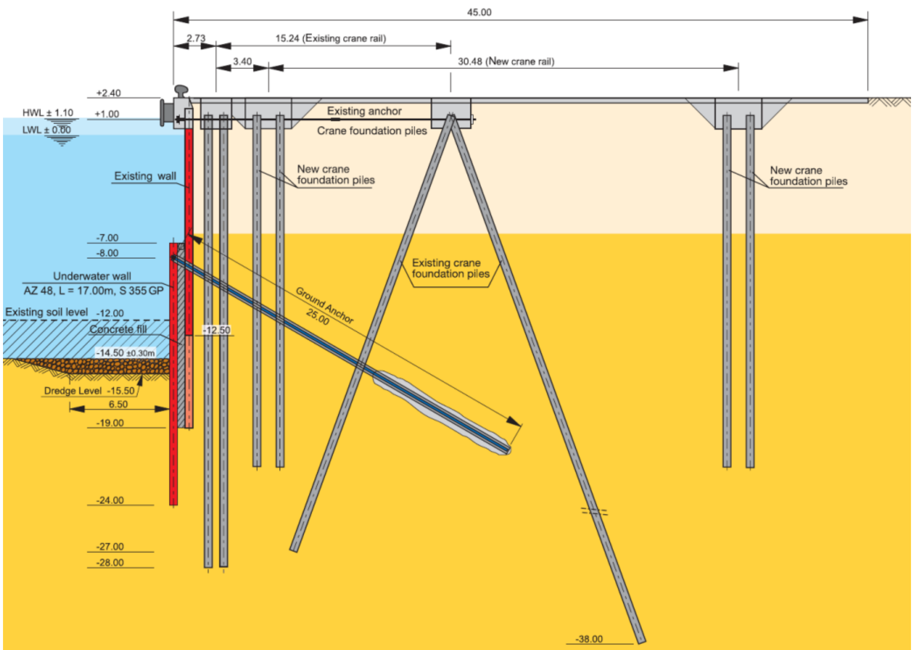

Figure 21 Extra wall with additional anchor, Kaohsiung, Taiwan

3.4 Port of Yokohama, Japan

The Port of Yokohama is located on the north-western edge of Tokyo Bay. It is a naturally blessed port with a spacious water area on the eastern side and undulated hills on the northern, western and southern sides. In addition to its natural assets, the port has been equipped with various facilities, such as inner and outer breakwaters, that protects the port from the effects of winds and tides. It also has an ample water depth.

3.4.1 Location

The quay wall structure is in the Port of Yokohama, Japan. The deepening in front of the quay wall is executed in front of a container terminal. Figure 22 and Figure 23 presents the location of the deepening project relative to Europe, Asia and the Port of Yokohama.

Figure 22 Location of the deepening project Yokohama relative to Europe and Asia

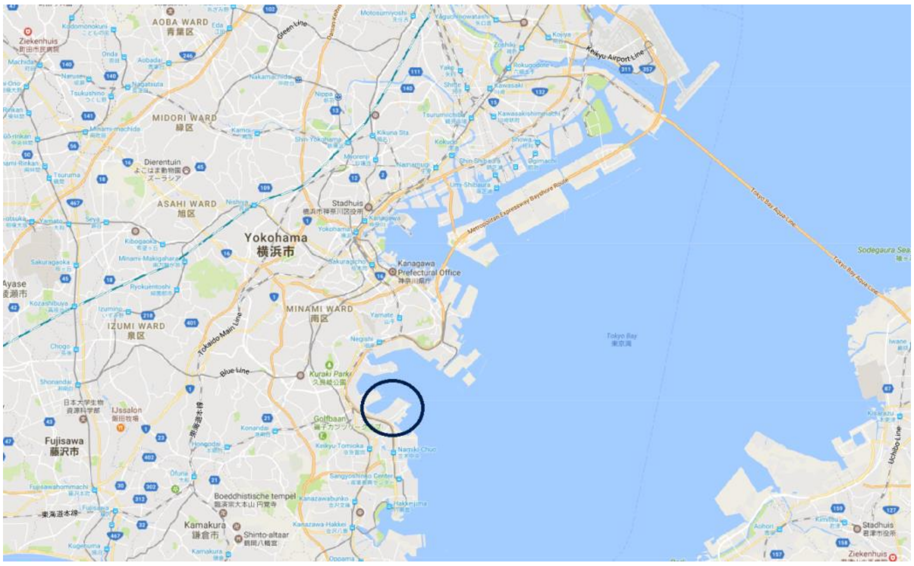

Figure 23 Location of the deepening project relative to the port of Yokohama

3.4.2 Type of structure

The current quay wall structure is an anchored sheet pile wall. The anchorage is done by an anchor plate. The original port bed level is -9.00 meter. The illustration before the deepening project is not available, so the principle cross section of an anchored sheet pile is figured in Figure 24.

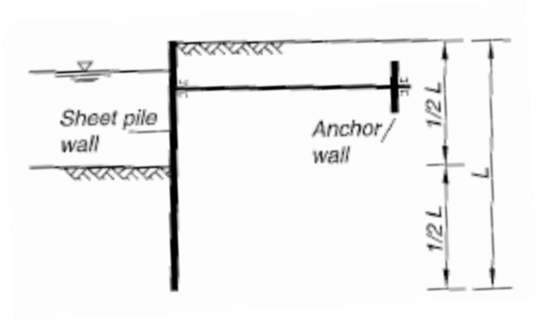

Figure 24 Principle of a anchored sheet pile wall with anchor wall

3.4.3 Solution

The solution for the deepening of this quay wall structure is injecting the ground behind the wall with grout. The soil behind the wall will behave as concrete. The effect of the grout injection of the ground behind the wall in unknown. The expected effects are:

- Reduction of the active soil pressure so the failure of the insufficient passive resistance of front wall will be solved;
- Increasing of the overall stability by the Bishop method;
- Reduction of the bending moments of the front wall;
- Reduction of the anchor forces.

See Figure 25 below for the illustration of the grout injection of the ground behind the wall.

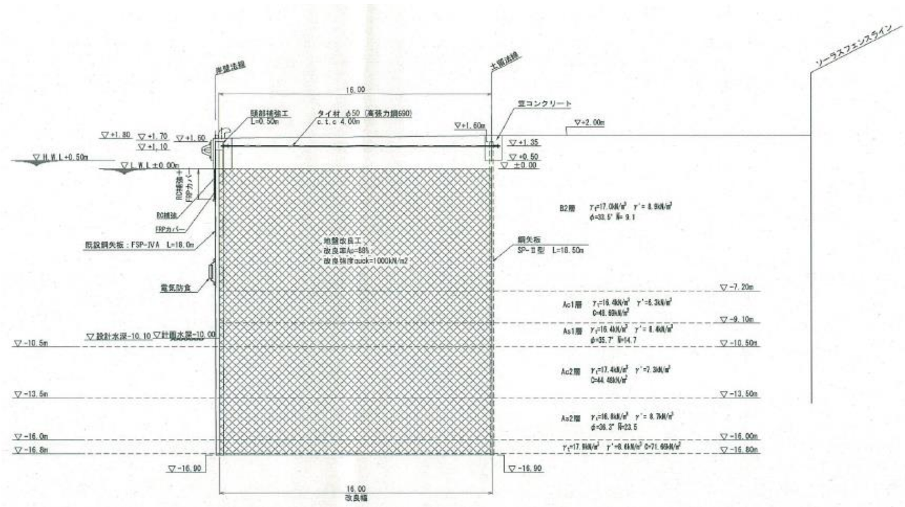

Figure 25 Cross section Yokohama with solution after deepening

4 Conclusion

The conclusion of the report are the investigated deepening projects and the executed solutions of the deepening. This report answers the first secondary research question: “• What adjustments have been made to combi-wall structures in respect to deepening projects in the Port of Rotterdam and global in the past?”.

The deepening projects which are investigated are:

- Sint Laurenshaven, Botlek, Port of Rotterdam, The Netherlands;
- Pier 6, Waalhaven, Port of Rotterdam, The Netherlands;
- Port of Felixstowe, United Kingdom;
- Port of Ravenna, Italy;
- Port of Kaohsiung, Taiwan;
- Port of Yokohama, Japan.

The deepening of the port beds is done next to several types of quay structures. The types of quay wall structures are a caisson, a combi-wall with relieving floor and an anchored sheet pile wall. The solutions for the quay wall structures are different.

The executed solution for the investigated deepening projects are:

- Add asphalt matrasses in front of the existing structure;
- Inject the ground in front of the existing structure with grout;
- Add an extra wall in front of the existing structure;
- Add a low underwater anchor near to the port bed;
- Add an extra wall with an addition anchor in front of the existing structure;
- Inject the ground behind the existing structure with grout.

5 Bibliography

Google. (2017). *Maps*. Retrieved February 14, 2017, from Google Maps: https://www.google.nl/maps/

Hutchison Ports, Port of Felixstowe. (2017). *About us*. Retrieved February 15, 2017, from Port of Felixstowe: https://www.portoffelixstowe.co.uk/

L'Autorità Portuale di Ravenna. (2017). *The Port of Ravenna*. Retrieved February 15, 2017, from L'Autorità Portuale di Ravenna: http://www.port.ravenna.it/porto-di-ravenna-en/

SBRCURnet. (2014). *Publicaion 221E "Quay walls, Second Edition".* Rotterdam, The Netherlands: CRC Press/Balkema.

Tijssen, G., & Veldhuijsen, R. (1988). *Renovatie kademuren nog steeds in ontwikkeling.* Delft: TU Delft.

Uniport Multipurpose Terminals BV. (2017). *About us*. Retrieved February 21, 2017, from Uniport Multipurpose Terminals BV: http://uniport.nl/about-uniport/

3 Reference structures Botlek area

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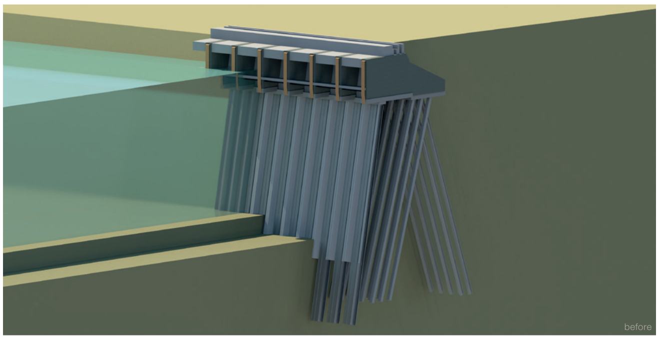

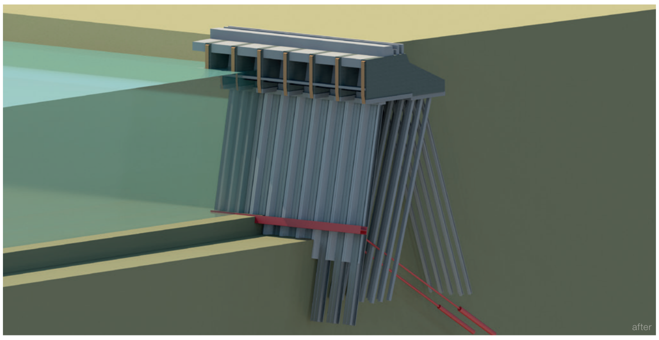

DEEPENING OF AN EXISTING COMBI WALL

REFERENCE STRUCTURES BOTLEK AREA

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Deepening of an Existing Combi Wall

Reference quay structure Botlek

12 juni 2017, Rotterdam, The Netherlands

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Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

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1 Summary

Quay wall structures are developed for several functions, the primary functions of quay wall structures are:

- Retaining: the structure must retain soil and water for the area behind the quay;
- Bearing: the structure must bear loads of transshipment of freights, carry loads, crane loads and storage loads;
- Mooring: the structure must have sufficient draft for the bigger vessels, enough mooring facilities to moor the vessels safe and efficient;
- Protection: the structure can serve as retaining wall during high waters.

Quay wall structures can be divided into different types. The chart below pictures the various types of quay wall structures.

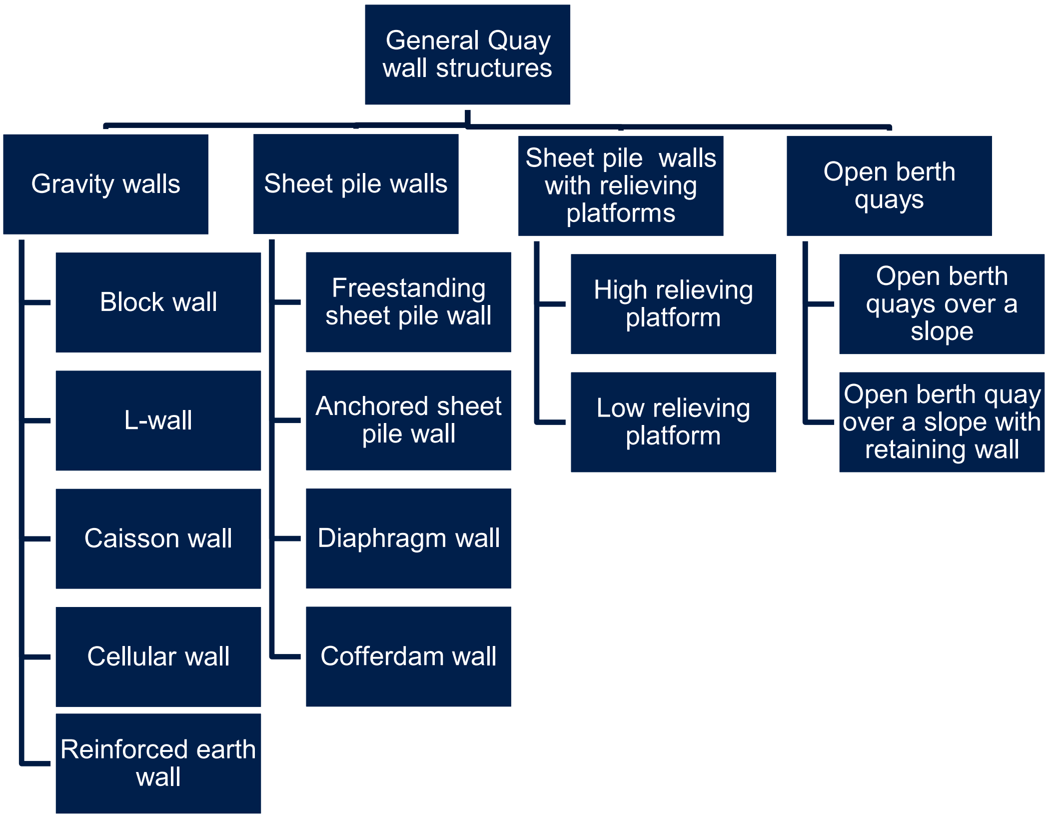

Figure 1 main types of quay wall structures

The quay wall structures of the Botlek area are various. Single sheet pile walls with anchors are constructed of shallow water. The deep water harbours are constructed as combi-wall with relieving structure. This thesis will focus on combi-walls with relieving structures. In consultation with the supervisors, the quay structure of Sint Laurenshaven, port number 4313 is the reference structure for this thesis.

The principle cross section of the quay wall structure of Sint Laurenshaven, port number 4313 is pictured as Figure 2 Principle cross section reference structure thesis

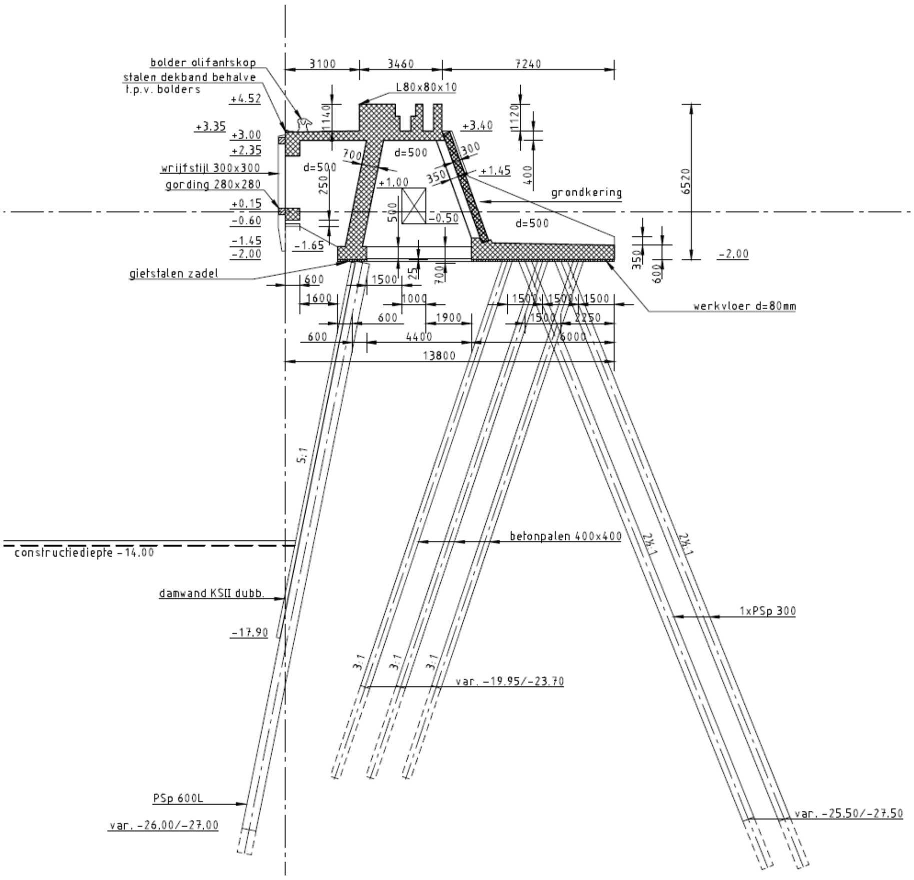

Figure 2 Principle cross section reference structure thesis

Table of contents

2 Main types of quay walls

This report is made according to the second secondary research question: What are the main types of combi-wall structures of the Botlek area?

The results of this report is an overview of the type of quay wall structures in the Botlek area and the underpinning of the selection of the reference quay wall structure. The reference quay wall structure will be used to test the solution for the deepening of the port.

Quay wall structures can be divided into different type of structures. The type of structures will be explained below.

2.1 Function quay wall

The primary function of a quay wall is that ships can berth alongside. The development of quay walls is space saving compared to older used slope structures. The main functions of a quay wall are:

- Retaining: the structure must retain soil and water for the area behind the quay;
- Bearing: the structure must bear loads of transshipment of freights, carry loads, crane loads and storage loads;
- Mooring: the structure must have sufficient draft for the bigger vessels, enough mooring facilities to moor the vessels safe and efficient;
- Protection: the structure can serve as retaining wall during high waters.

2.2 Main types of quay wall structures

To fulfil the functions of the quay wall, several quay wall structures type are engineered. The main type of quay wall structures nowadays are presented in Figure 3 main types of quay wall structures.

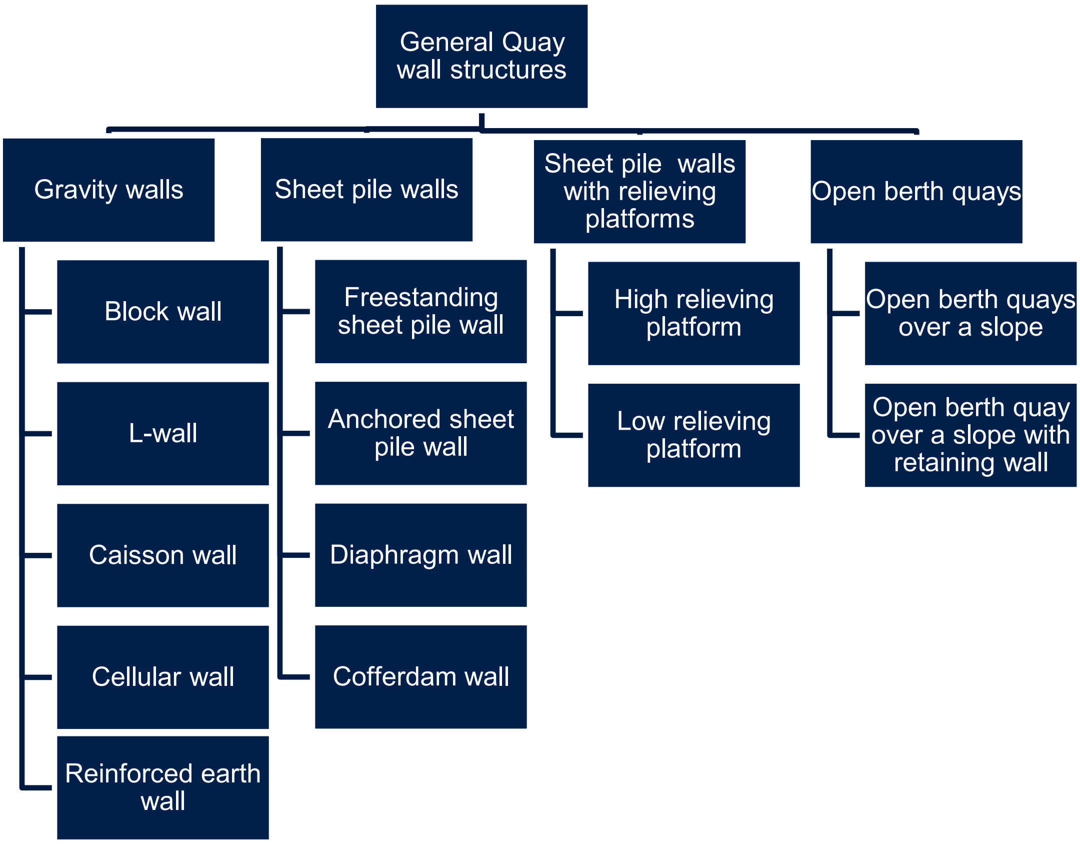

Figure 3 main types of quay wall structures

The subchapters below explains the different types of quay wall structures.

2.3 Gravity walls

The retaining function of gravity wall is derived from the self-weight of the structures. The wall is that heavy so the structures cannot tilt of slide, because of the generated resistance to shearing.

The choose of the type of gravity wall structure depends on the local soil conditions and the relation between costs of materials and labour.

2.3.1 Block wall

The block wall is the simplest type of gravity walls. The block wall consists of blocks of concrete or natural stones. These stones are piled on top of each other. The block wall is placed on a foundation of gravel of crushed stones. Vertical and horizontal joints provide a good drainage, so water overpressure is limited behind the wall. The drainage is sufficient if the sand grains cannot wash out. The prevent the washing out a filter structure behind the wall is needed. The principle of a block wall is presented in Figure 4 Principle of a block wall.

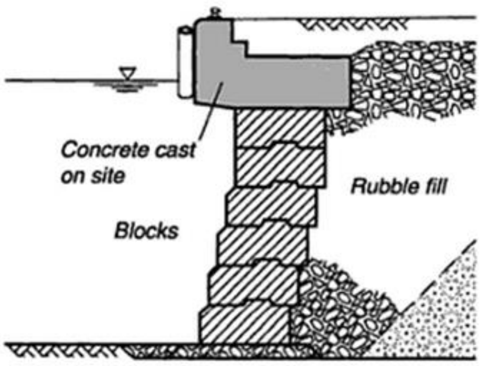

Figure 4 Principle of a block wall

2.3.2 L-wall

The stability of an L-wall consists of the weight of the concrete structure and the weight of the soil that rest on them. L-wall structures can be used if the soil conditions are not sufficient for the construction of a block wall. The L-wall is placed on a foundation of gravel or crushed stones. The principle of a block wall is presented in Figure 5 Principle of an L-wall.

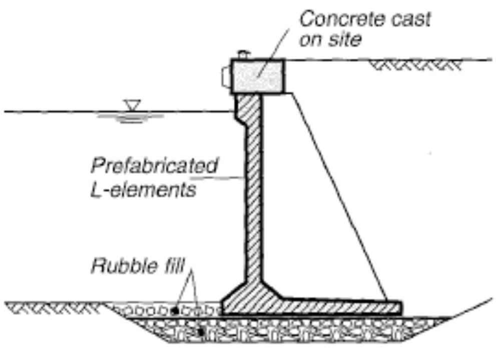

Figure 5 Principle of an L-wall

2.3.3 Caisson wall

Caisson wall structures are made of large hollow cellular concrete elements. The caissons are usually built in a dry dock. From the building dock the caissons are floated to the construction site. At the construction site the caisson pieces are sunk onto the firm prepared subsoil. The caisson are filled with soil to withstand the horizontal soil pressure. The principle of a caisson is presented in Figure 6 Principle of a caisson.

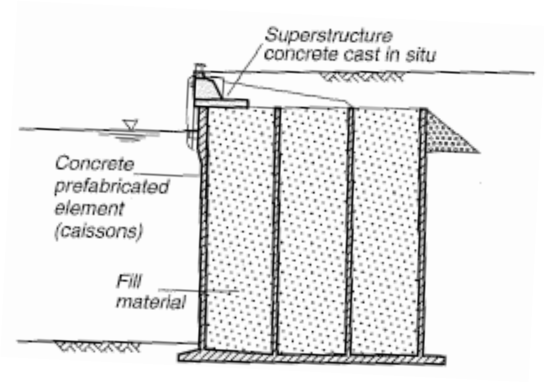

Figure 6 Principle of a caisson

2.3.4 Cellular wall

Cellular walls are constructed by driving straight web profiles to form cylindrical or partially cylindrical cells. The web profiled are connected to each other by interlocking profiles. The cellular wall rest on the bottom of the harbour. The cellular wall is filled with sand or other materials. The structure is relative thin, so it is vulnerable to collision and corrosion. The principle of a cellular wall is presented in

Figure 7 Principle of a cellular wall and

Figure 8 Principe execution method cellular wall.

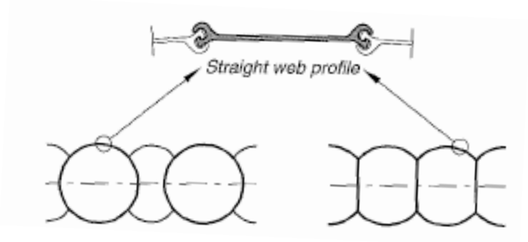

Figure 7 Principle of a cellular wall

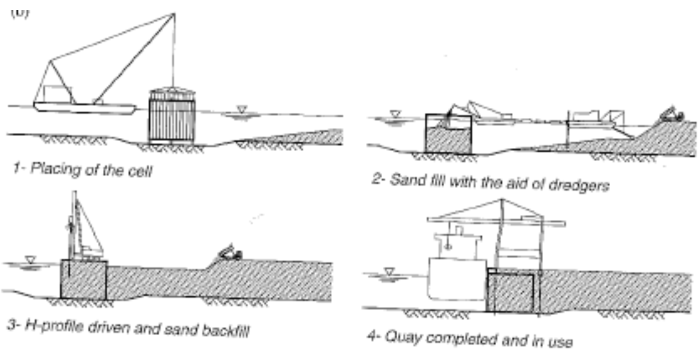

Figure 8 Principe execution method cellular wall

2.3.5 Reinforced earth wall

The reinforced earth wall consist of tension elements as steel strips, steel rods or polymer reinforcement such as geogrid of geotextiles. The tension elements are inserted into the soil and increase the friction between the soil and the wall. The horizontal tension elements are connected to vertical bars on the wall. The stresses are transferred from the soil to the horizontal tension reinforcements to the vertical bar, to the wall. The principle of a reinforced earth wall is presented in Figure 9 Principle of a reinforced earth wall.

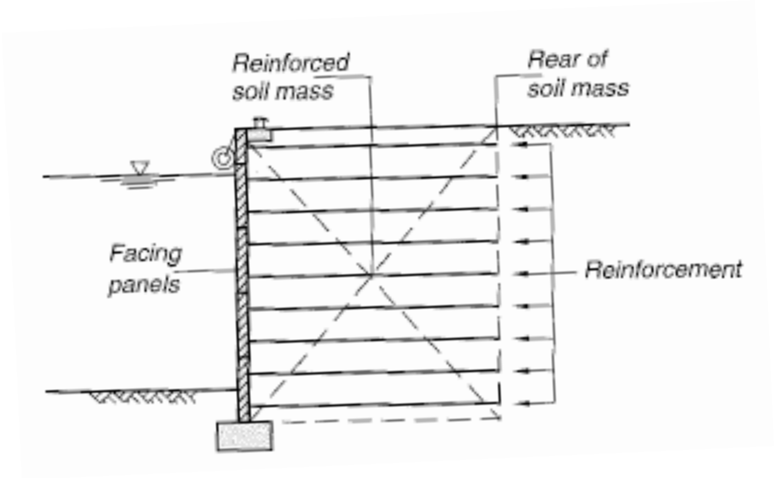

Figure 9 Principle of a reinforced earth wall

2.4 Sheet pile walls

Sheet pile wall structures derive their soil retaining function and stability from the fixation capacity of the soil. Sheet pile wall can be used in combination with anchors. The sheet pile structures are used on locations where the subsoil is easy to penetrate and has reduced bearing capacity. The interlocking system connects the sheet piles. A drainage is necessary to prevent the reducing of the pore pressure behind the wall by overstrained water. The structure types are freestanding sheet pile wall and anchored sheet pile wall. The sheet piles consist of several types sheet pile systems and anchor types.

2.4.1 Freestanding sheet pile wall

A freestanding sheet pile wall is not anchored. The freestanding sheet pile wall acts as a cantilever beam to transfer horizontal pore pressures to the subsoil. The supporting pressure that is necessary to gain equilibrium is the passive earth pressure at the toe of the construction. The principle of a freestanding sheet pile wall is presented in Figure 10 Principle of a freestanding sheet pile wall.

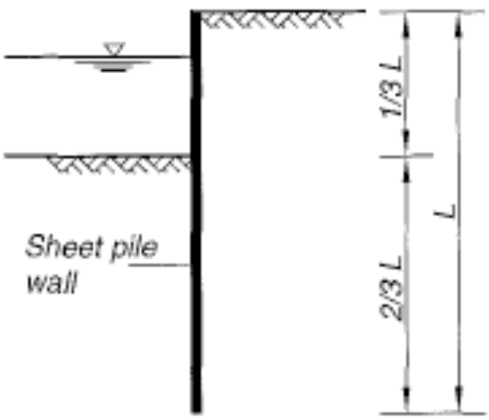

Figure 10 Principle of a freestanding sheet pile wall

2.4.2 Anchored sheet pile wall

Higher retaining height is the reason anchorages are used in combination with sheet pile walls. The anchored wall will behave as a girder in two supports. The anchor as one support and the soil as one support. The different types of anchoring are explained in section 2.4.4 Anchorages. The figures Figure 11 Principle of a anchored sheet pile wall with anchor wall and Figure 12 Principle of a anchored sheet pile wall with grout anchor presents the principles of an anchored sheet pile wall.

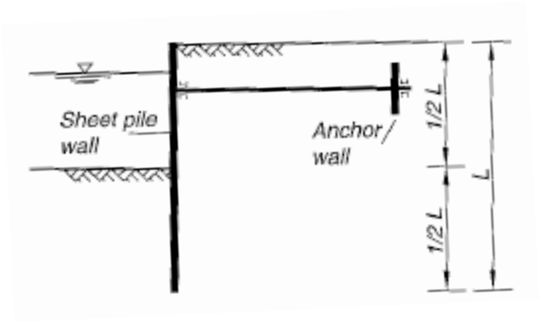

Figure 11 Principle of a anchored sheet pile wall with anchor wall

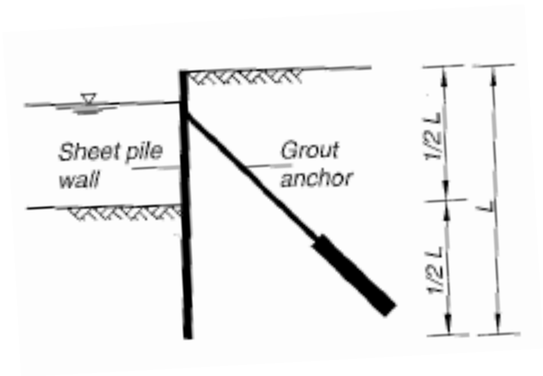

Figure 12 Principle of a anchored sheet pile wall with grout anchor

2.4.3 Sheet pile systems

The main types of sheet pile systems are single sheet piling, combined sheet piling, diaphragm wall and fixed cofferdam.

2.4.3.1 Single sheet pile

The single sheet pile walls can be made of wood, concrete and steel. Nowadays steel profile are the most uses single sheet pile. The different types of single sheet piles systems is presented in

Figure 3 main types of quay wall structures.

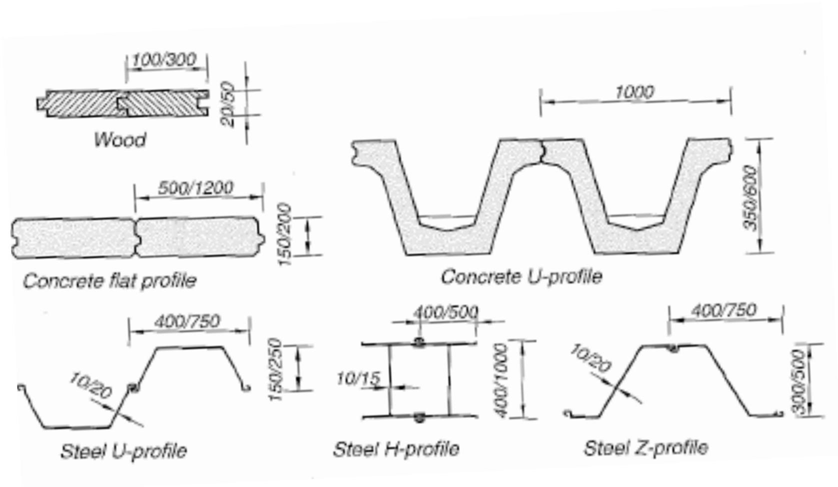

Figure 13 Types of single sheet pile walls

2.4.3.2 Combined wall

Quay walls with higher retaining height combined walls are needed. The combined walls can bear more loads and heavier structures. A combined wall consist of heavy primary elements which are deeply embedded in the subsoil. The standard steel sheet piles are the intermediate piles. These intermediate can be short the primary piles because the horizontal pressure is transferred to the primary piles. The combined walls are often used in Rotterdam. The open piles are relatively east to vibrated through the firm sand layers.

See
Figure 14 Types of combined walls for the visualisation of the main types of combined wall systems.

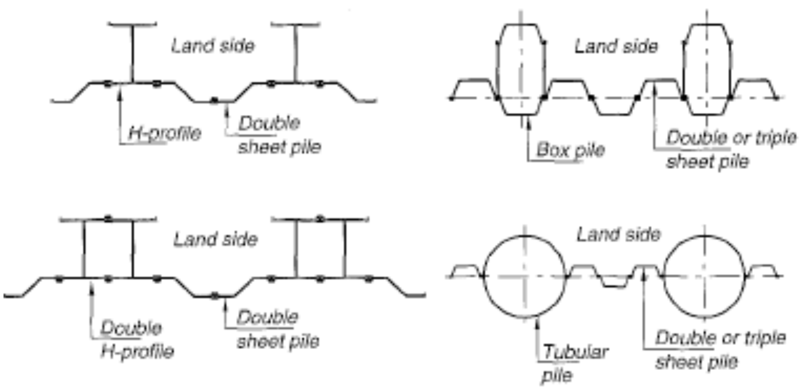

Figure 14 Types of combined walls

2.4.3.3 Diaphragm wall

A diaphragm wall is a reinforced concrete wall. These wall are made in situ. A deep narrow trench is excavated on the location where the quay wall is to be constructed. During the excavation the trench is filled with bentonite slurry to prevent the collapse of the wall. These walls have a high bearing capacity and are very stiff so the deformations are minimal. The principle of a diaphragm wall is presented in Figure 15 Principle of a diaphragm wall.

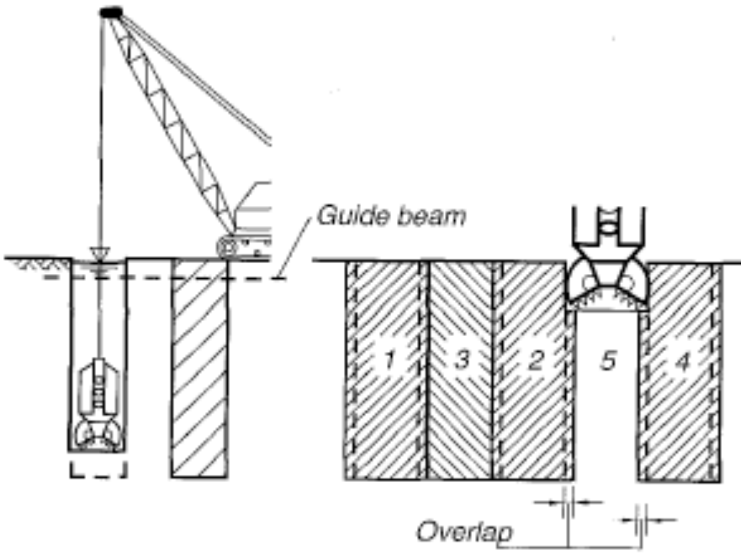

Figure 15 Principle of a diaphragm wall

2.4.3.4 Cofferdam wall

A cofferdam consist of two sheet pile walls. The two sheet pile walls are connected to each other by one or more anchorages. The space between the two sheet piles is filled with soil, which transfers the horizontal and vertical pressures to the subsoil. The two walls are that close to each other that the active and passive ground zones overlaps. It is assumed that the two walls with the soil in between works as one system. The principle of a cofferdam wall is presented in Figure 16 Principle of a cofferdam wall.

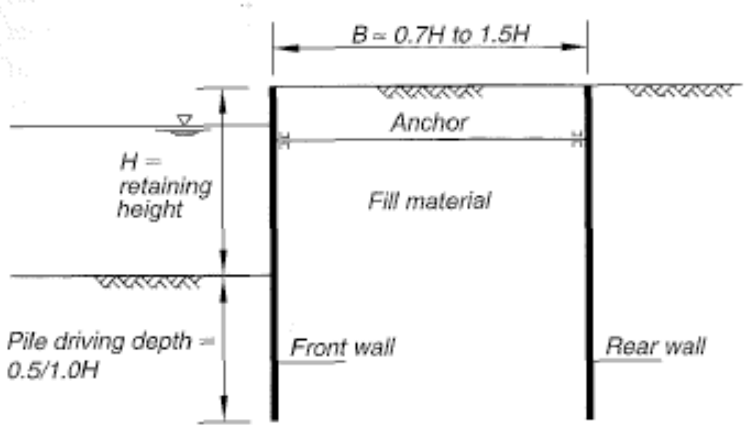

Figure 16 Principle of a cofferdam wall

2.4.4 Anchorages

The anchorage types functions as an upper support point for a sheet pile. The forces of the wall are transferred to trough the anchor to the soil behind the wall. In general the anchorages types can be divided in three types; horizontal anchorage, anchorages with grout body and tension piles.

2.4.4.1 Horizontal anchorage

The most conventional type of anchorage is the horizontal anchorage. The anchorages can be connected to the wall by bars, cables and screws. The types of horizontal anchorage are presented in Figure 17 Types of horizontal anchorage.

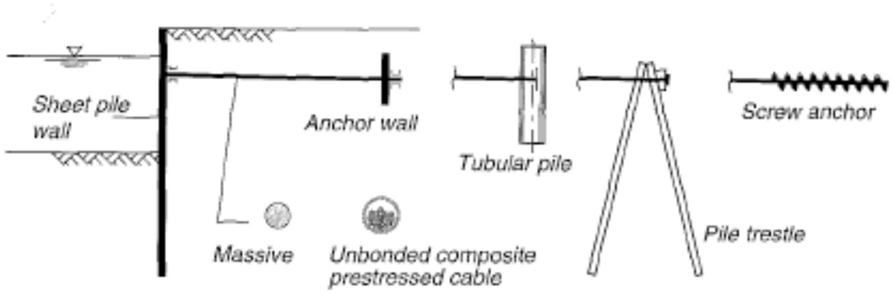

Figure 17 Types of horizontal anchorage

2.4.4.2 Anchorage with grout body

The anchorage with grout body consists of cement grout elements made in situ. The different type of grout body anchorages are grout anchor and screw injection anchor. The grout body anchorage must be pre-tensioned because the sheet pile will have a large deformation. The types of anchorage with grout body are presented in Figure 18 Types of anchorages with grout body.

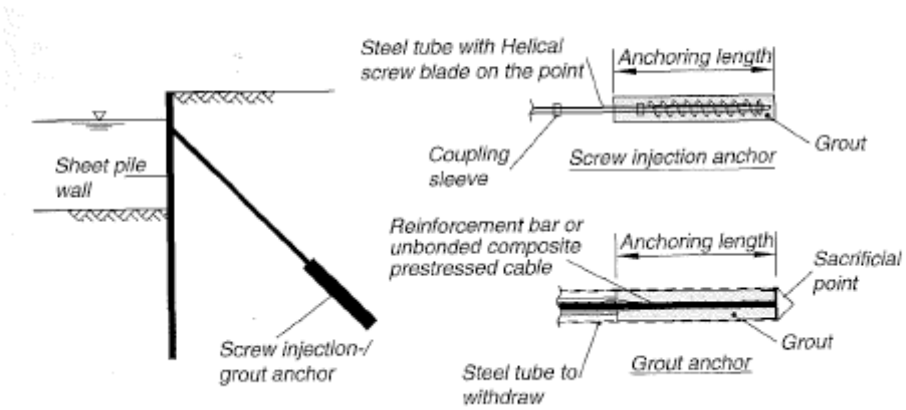

Figure 18 Types of anchorages with grout body

2.4.4.3 Tension piles

Sheet piles can also be anchored by tension piles. The tensile force is supplied by the shaft friction of the pile and the soil. Closed piles, open steel piles, steel H-piles and MV-piles can be used as tension pile anchorage. The different types of tension piles are presented in Figure 19 Types of tension piles.

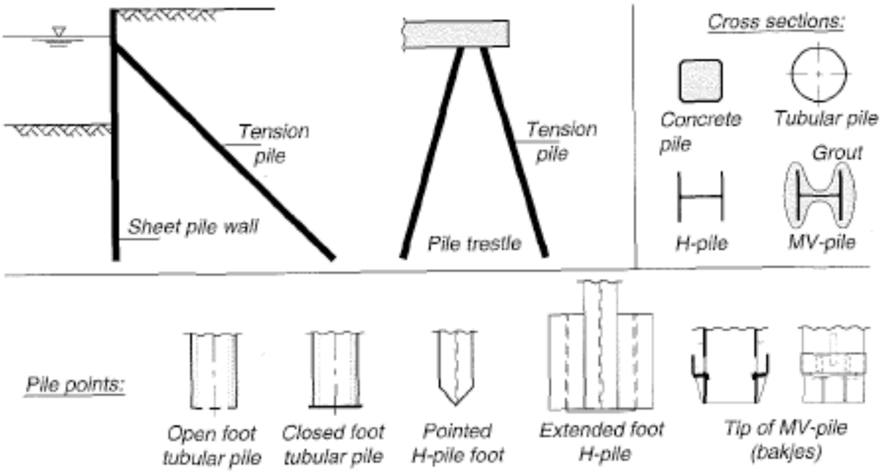

Figure 19 Types of tension piles

2.5 Sheet pile wall with relieving platform

The relieving platform reduces the horizontal load on the front of the wall. The loads on the quay surface will directly be transferred to the subsoil by the concrete platform. The total structure consists of retaining sheet pile wall, a concrete relieving floor and bearing piles to transmit the vertical loads into the subsoil and tension piles or anchors to increase resistance against horizontal pressure. Sheet pile walls with relieving platforms can be applied for high retaining heights, heavy loads and high demand for deformations. Relieving platform structure can be divided into two type; high and low relieving platform structures.

2.5.1 High relieving platform

The method of construction is based on the transfer of the horizontal loads of the soil by a pile trestle system with tension and bearing piles. The high relieving platform is usually constructed above the low water level. The elements of the relieving platform are often prefabricated. The principle of a high relieving platform is pictured in Figure 20 Principle of a high relieving platform

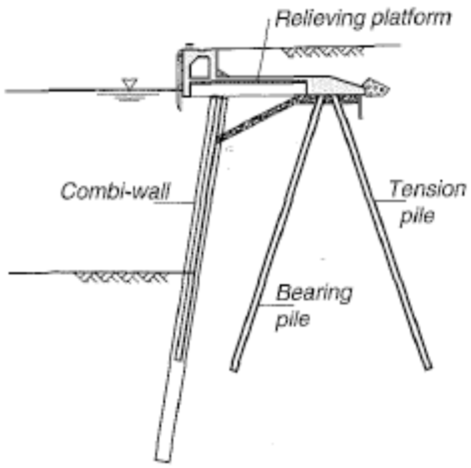

Figure 20 Principle of a high relieving platform

2.5.2 Low relieving platform

Structures with relieving platforms have been developed for high retaining heights. The platform is supported by foundation elements: one on the water side on the sheet pile wall and on the land on the bearing piles and one row of tension piles. Cast iron saddles between the relieving platform and the sheet pile wall create a hinge, so the vertical force will not be transferred to the wall. The principle of a low relieving platform is pictured in Figure 21 Principle of a low relieving platform.

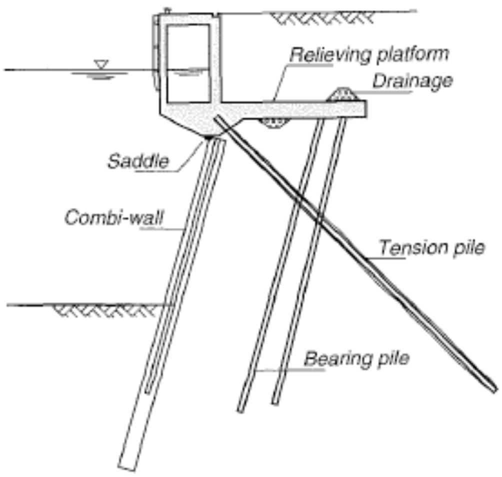

Figure 21 Principle of a low relieving platform

2.6 Open berth quays

The open berth quay structures are different from all the other quay structures. The retaining height is not bridged by a vertical wall but by a slope.

2.6.1 Open berth quays over a slope

The open berth quay over a slope is a jetty-like structure. The vertical forces are transferred to the subsoil by the vertical pile. The horizontal forced are transferred to the subsoil by the bearing and tension piles. The element of an open berth quay over a slope are usual prefabricated and constructed on site. The open berth quay structure is illustrated in Figure 22 principle of an open berth quay on a slope.

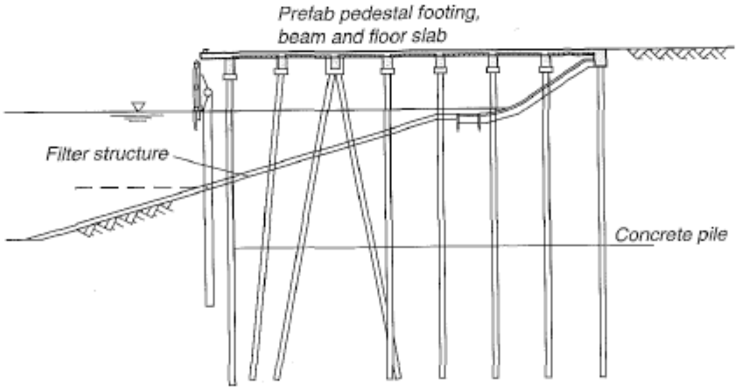

Figure 22 principle of an open berth quay on a slope

2.6.2 Open berth quays over a slope with retaining wall

To reduce the width of the structure a vertical sheet pile can be constructed to retain a part of the forces. The principle of an open berth quay with a retaining wall is pictured in Figure 23 Principle of an open berth quay with a retaining wall.

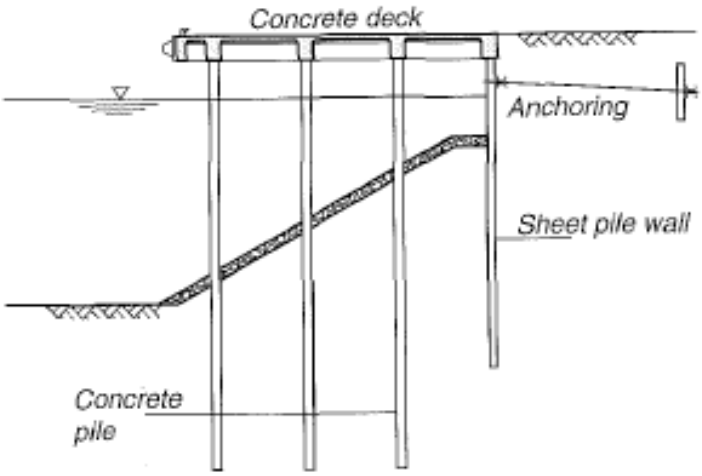

Figure 23 Principle of an open berth quay with a retaining wall

3 Types quay constructions Botlek

This thesis will focus on the quay structures at the Botlek. The main quay wall structure of the Botlek will be explained in the sub-chapters below.

3.1 Sint Laurenshaven, port number 4313

The quay construction is located at the Sint Laurenshaven and port number 4313-4316. See Figure 24 Location of the Sint Laurenshaven quay for the location of the quay wall of Sint Laurenshaven in the Botlek area.

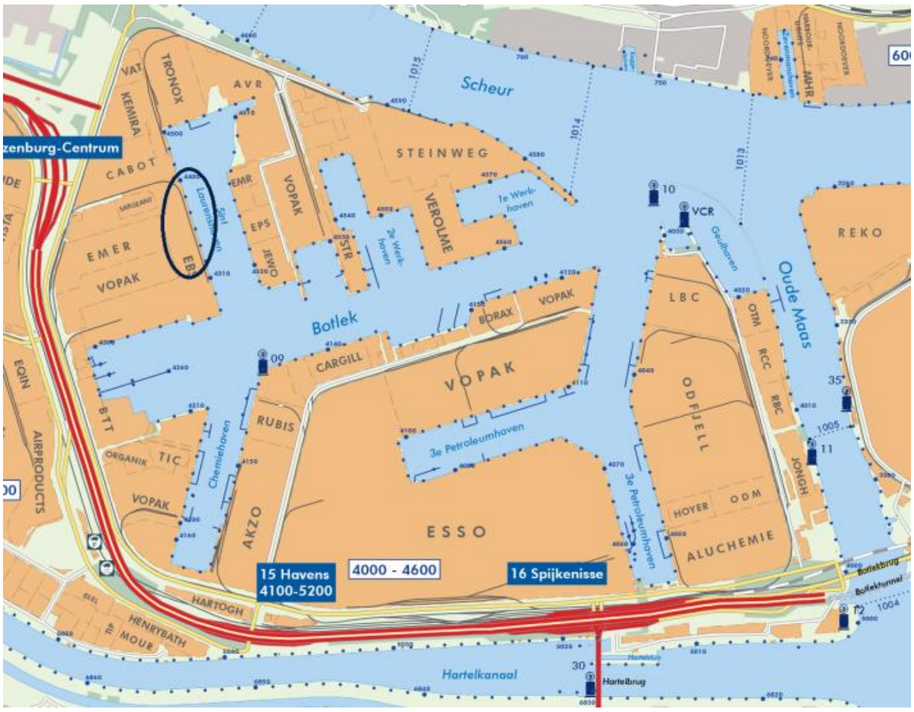

Figure 24 Location of the Sint Laurenshaven quay structure in the Botlek

3.1.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 1963
- Type construction: Sheet pile wall with high relieving platform
- Type sheet pile: combined wall

3.1.2 Principe cross section

The principle cross section of the Sint Laurenshaven quay wall is illustrated in Figure 25 Cross section quay wall structure Sint Laurenshaven.

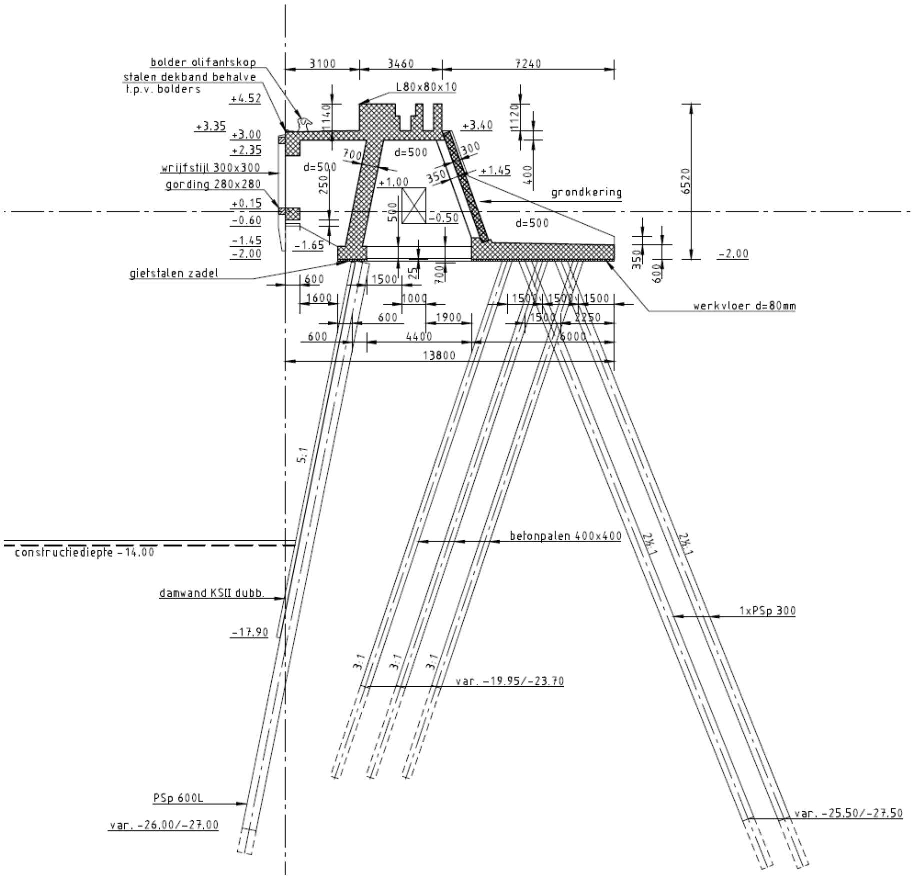

Figure 25 Cross section quay wall structure Sint Laurenshaven

3.2 Torontohaven, port number 4540

The quay construction is located at the Torontohaven and port number 4540-4543. See

Figure 26 Location of the Torontohaven quay structure in the Botlek for the location of the quay wall of Torontohaven in the Botlek area.

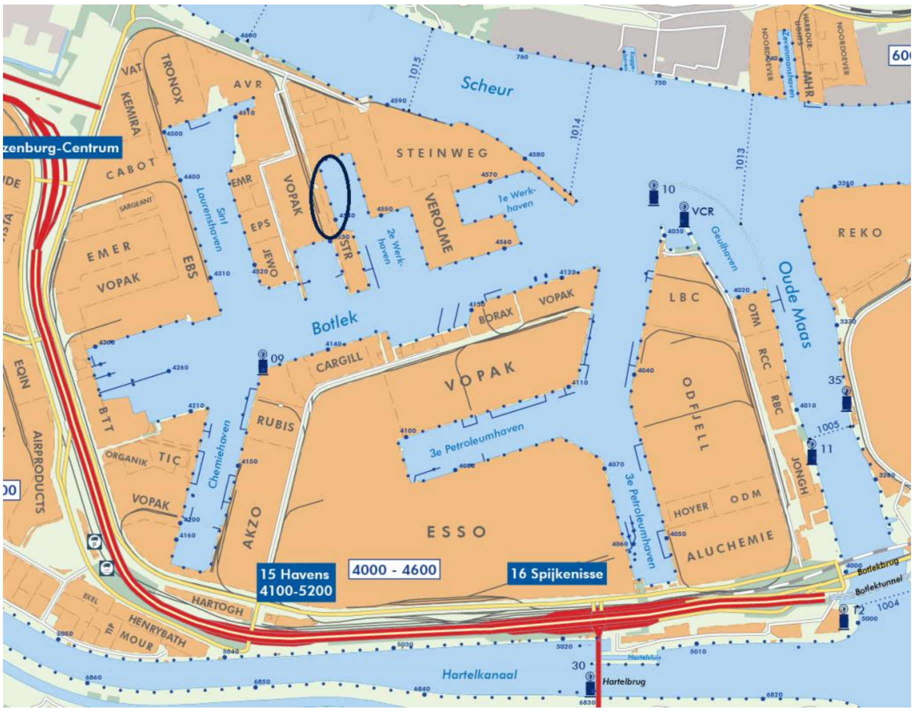

Figure 26 Location of the Torontohaven quay structure in the Botlek

3.2.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 1968
- Type construction: Sheet pile wall with high relieving platform
- Type sheet pile: single sheet pile

3.2.2 Principe cross section

The principle cross section of the Torontohaven quay wall is pictured in Figure 27 Cross section quay wall structure Torontohaven.

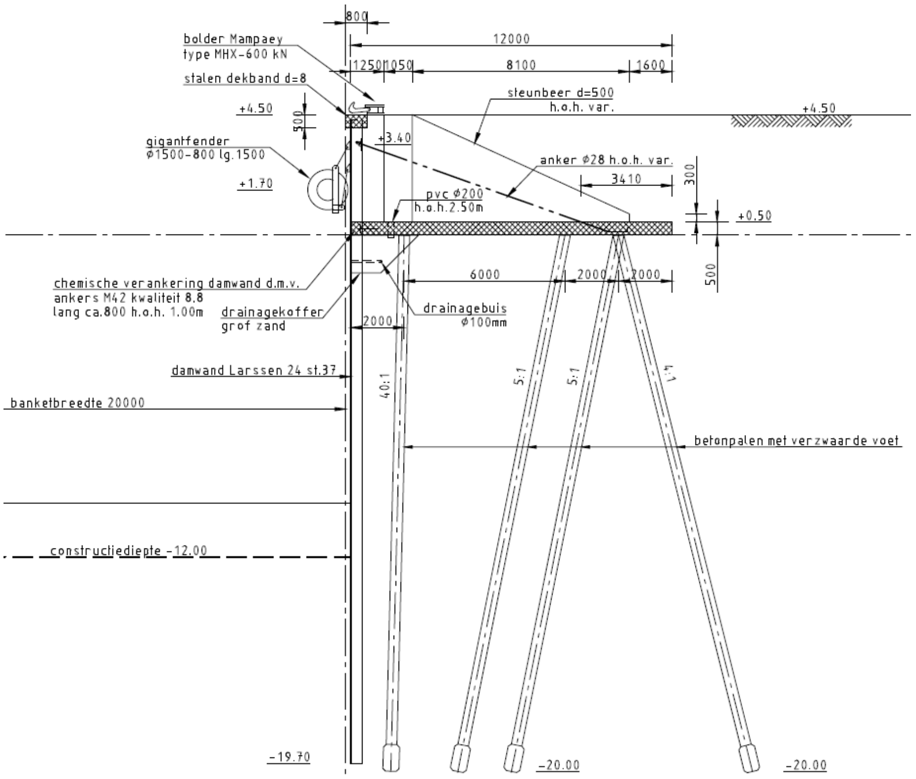

Figure 27 Cross section quay wall structure Torontohaven

3.3 2e Petroleumhaven, port number 4080

The quay construction is located at the 2e Petroleumhaven and port number 4080-4082. See Figure 28 Location of the 2e Petroleumhaven quay structure in the Botlek for the location of the quay wall of 2e petroleumhaven in the Botlek area.

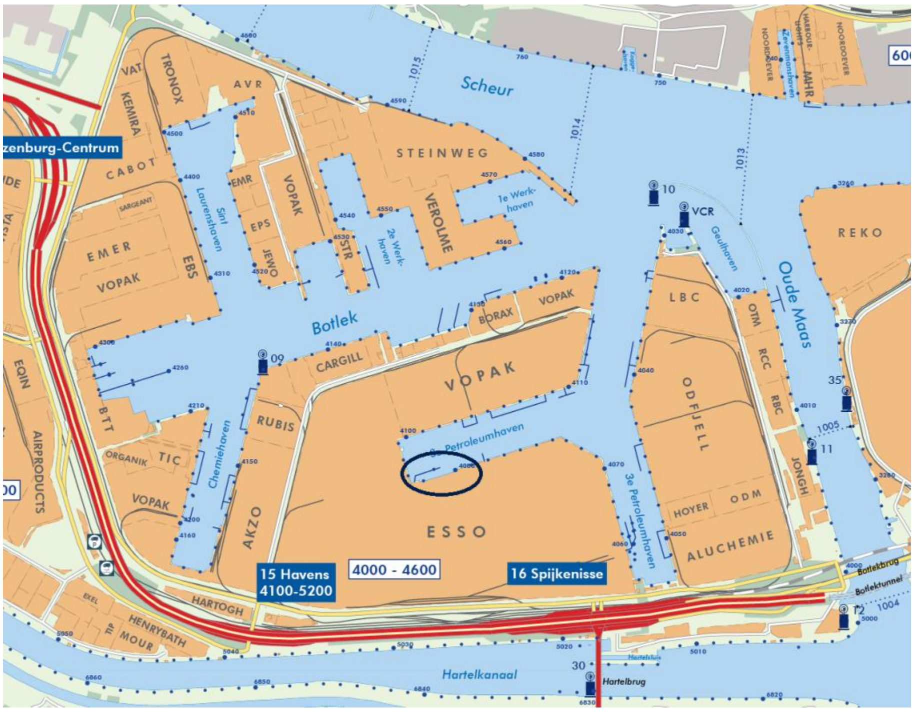

Figure 28 Location of the 2e Petroleumhaven quay structure in the Botlek

3.3.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 1987
- Type construction: anchored sheet pile wall
- Type sheet pile: combi wall

3.3.2 Principe cross section

The principle cross section of the 2e Petroleumhaven quay wall is illustrated in Figure 29 Cross section quay wall structure .

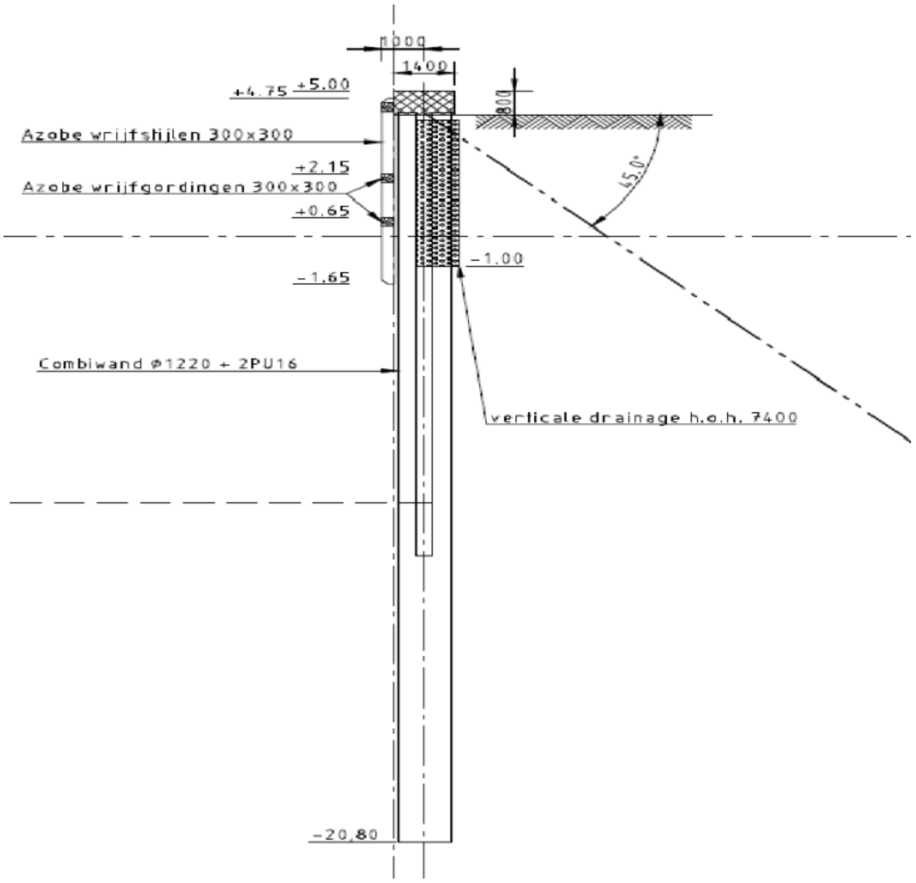

Figure 29 Cross section quay wall structure 2e Petroleumhaven

3.4 3e Petroleumhaven, port number 4045

The quay construction is located at the 3e Petroleumhaven and port number 4045-4049. Figure 30 Location of the 3e Petroleumhaven quay structure in the Botlek for the location of the quay wall of 3e Petroleumhaven in the Botlek area.

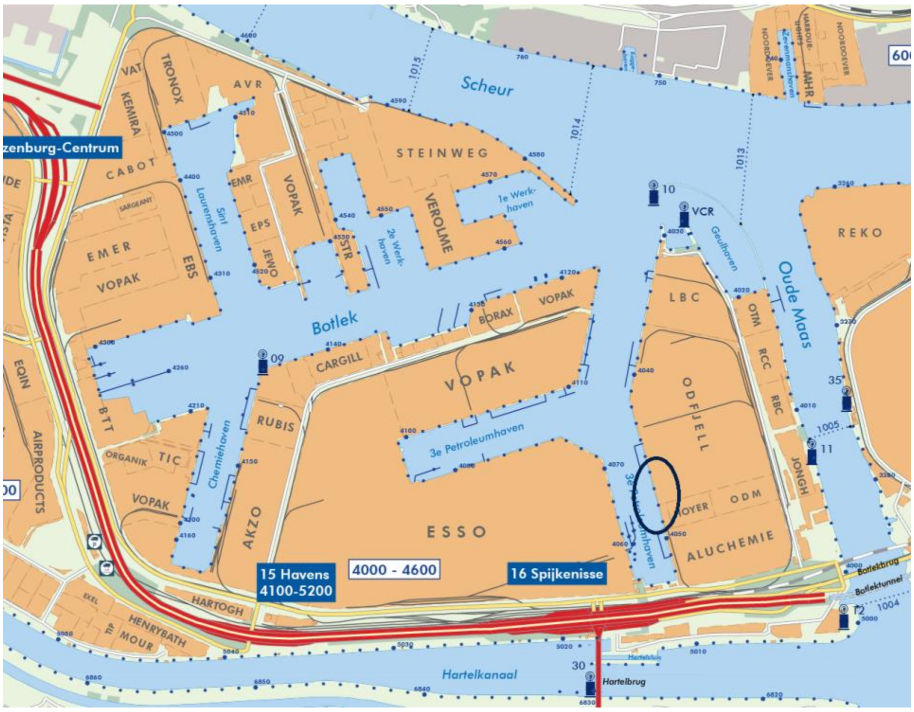

Figure 30 Location of the 3e Petroleumhaven quay structure in the Botlek

3.4.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 2008
- Type construction: Anchored sheet pile wall
- Type sheet pile: combined wall

3.4.2 Principe cross section

The principle cross section of the 3e Petroleumhaven quay wall is pictured in Figure 31 Cross section quay wall structure 3e Petroleumhaven.

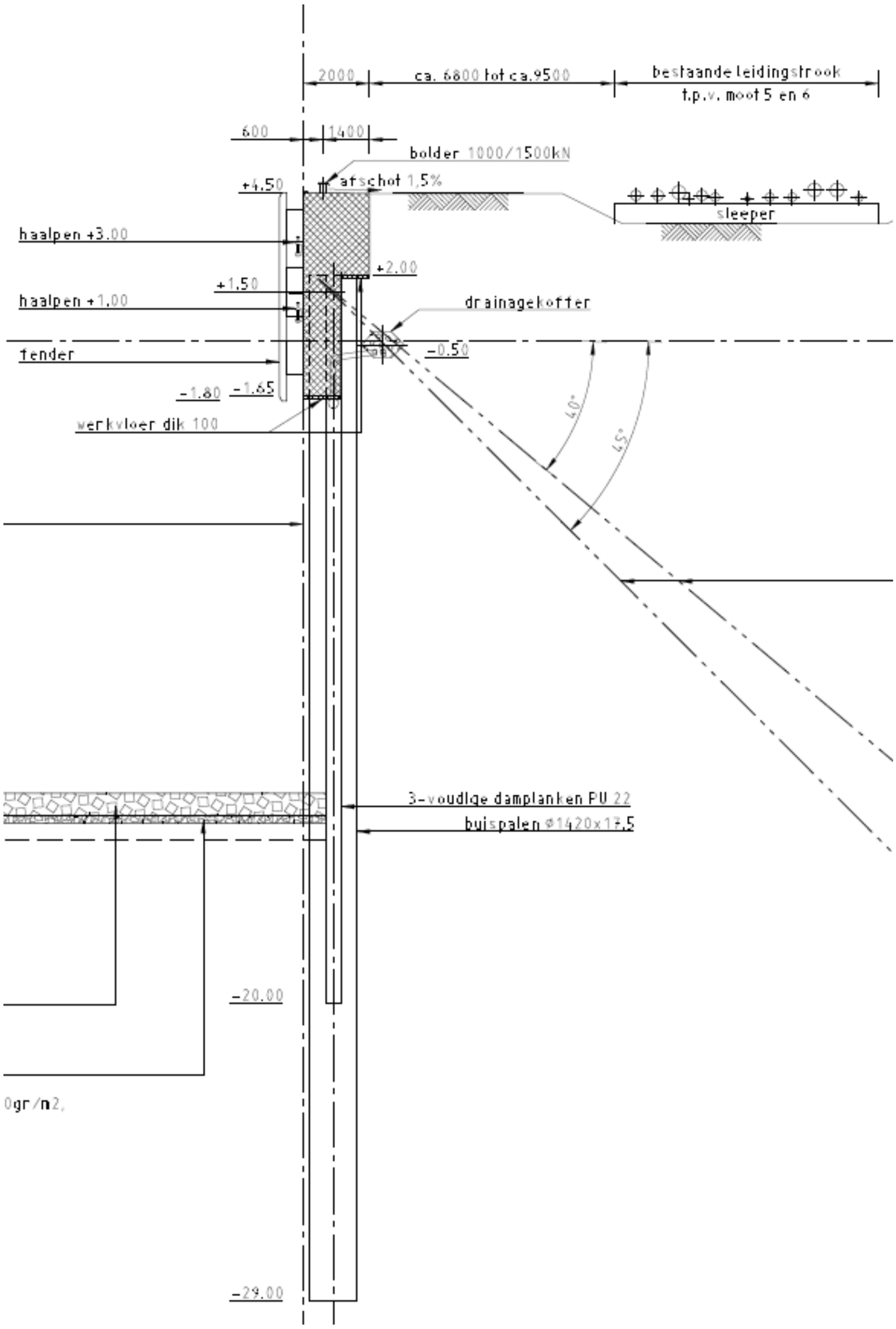

Figure 31 Cross section quay wall structure 3e Petroleumhaven

3.5 Sint Laurenshaven, port number 4515

The quay construction is located at the Sint Laurenshaven and port number 4515-4518. See Figure 32 Location of the Sint laurenshaven quay structure in the Botlek for the location of the quay wall of Sint laurenshaven in the Botlek.

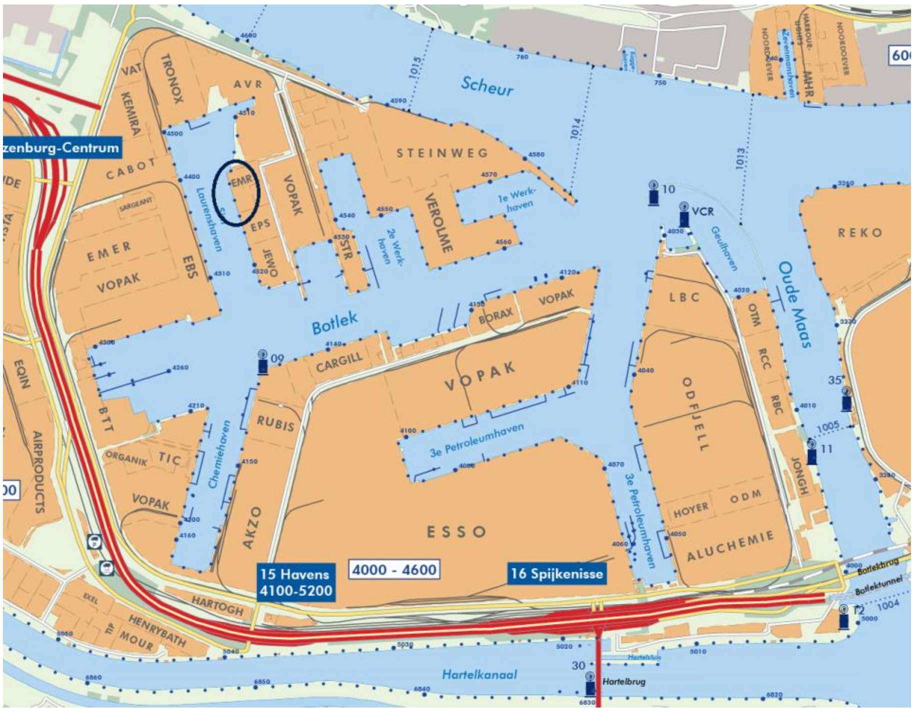

Figure 32 Location of the Sint laurenshaven quay structure in the Botlek

3.5.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 1957
- Type construction: Sheet pile wall with low relieving platform
- Type sheet pile: combined wall
- Adjusted and strengted with additional combi-wall with anchorage.

3.5.2 Principe cross section

The adjustment of the quay wall is the additional relieving floor on a combi-wall with anchorage. The combi-wall with anchorage are added to the structure because of the change of the surface load of the quay wall from ore to scrap. The adjustment does not provides deepening of the construction depth. The principle cross section of the Sint Laurenshaven quay wall with the adjustment is pictured in Figure 33 Cross section quay wall structure Sint Laurenshaven.

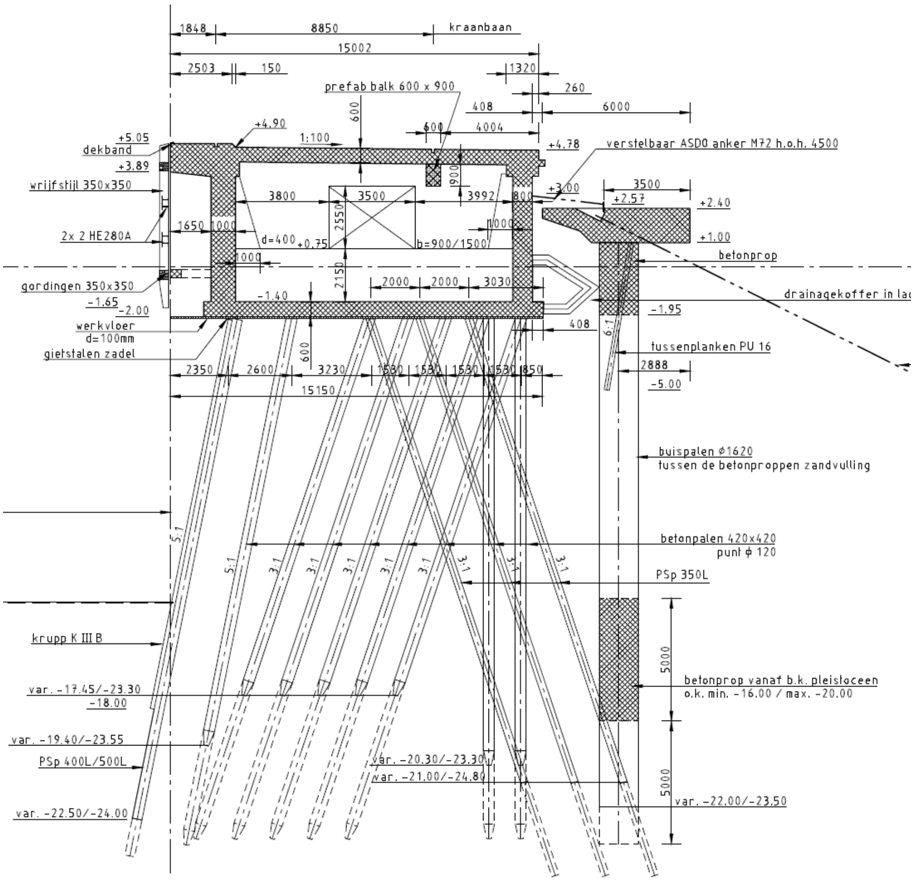

Figure 33 Cross section quay wall structure Sint Laurenshaven

3.6 2e Werkhaven, port number 4552

The quay construction is located at the 2e Werkhaven and port number 4552-4557. See Figure 34 Location of the 2e Werkhaven quay structure in the Botlek for the location of the quay wall of the 2e Werkhaven in the Botlek area.

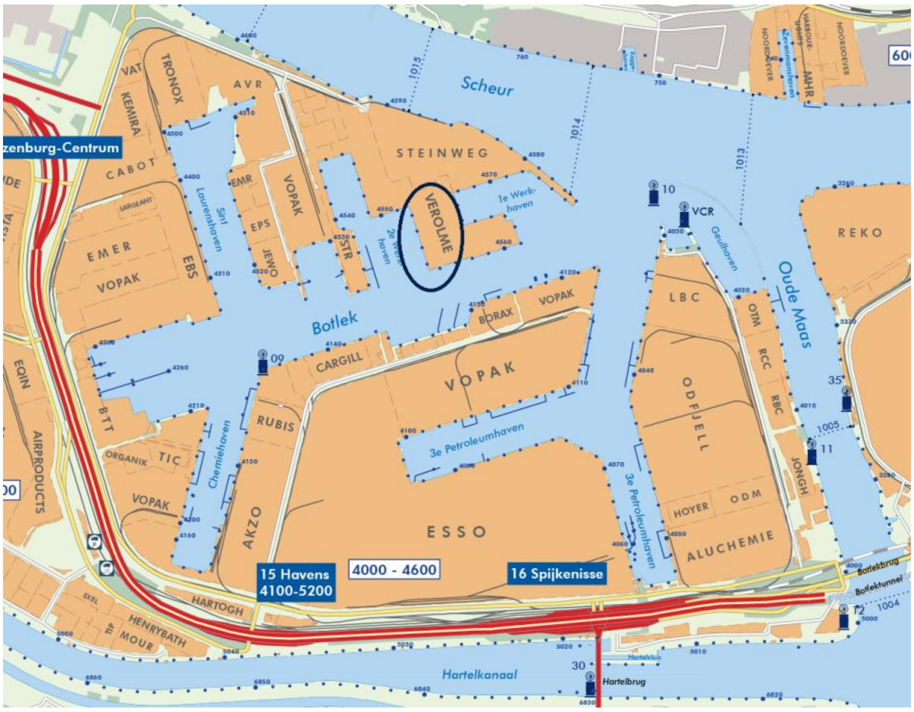

Figure 34 Location of the 2e Werkhaven quay structure in the Botlek

3.6.1 Type of quay wall

The characteristics of the quay wall structure are:

- Construction year: 1969
- Type construction: Sheet pile wall with low relieving platform
- Type sheet pile: single sheet pile

3.6.2 Principe cross section

The principle cross section of the 2e Werkhaven quay wall is pictured in Figure 35 Cross section quay wall structure 2e Werkhaven.

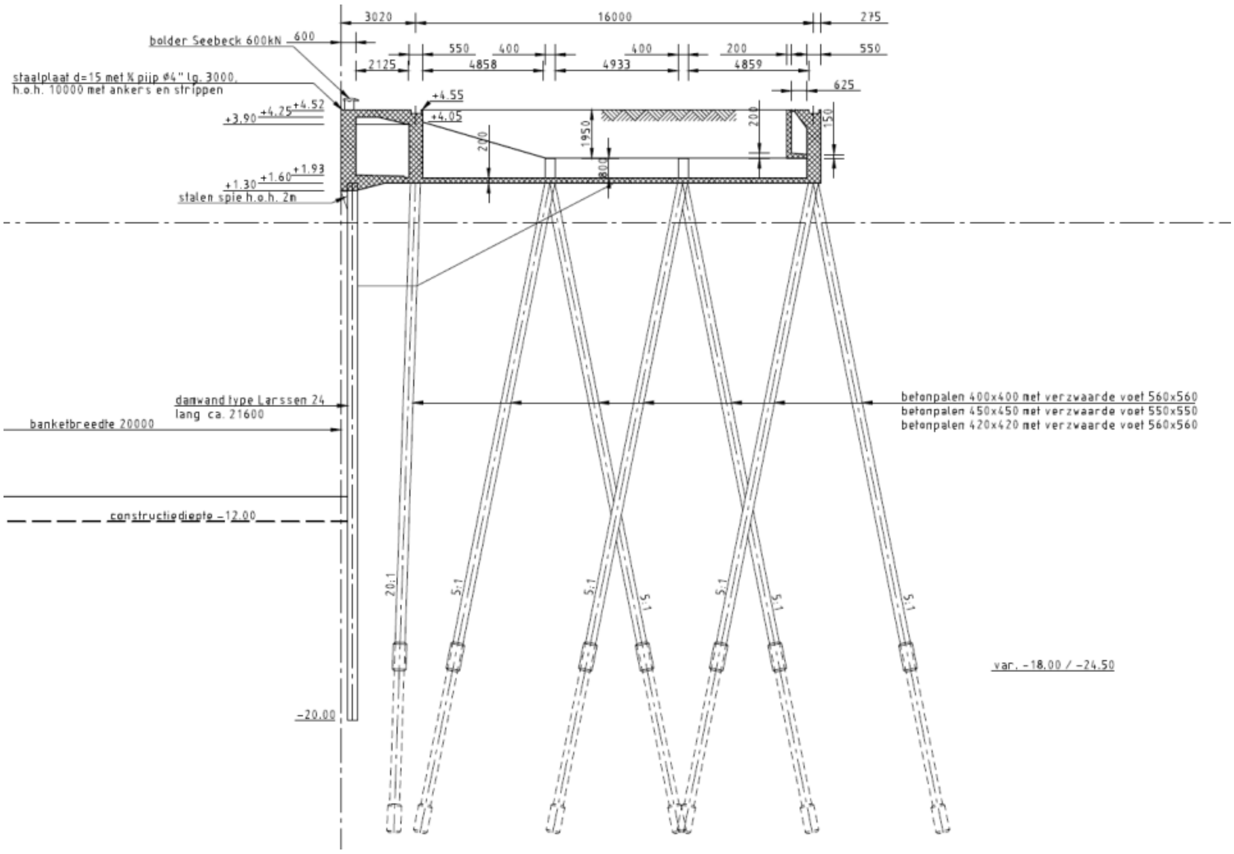

Figure 35 Cross section quay wall structure 2e Werkhaven

4 Selection reference quay wall structure

The quay wall structures of the Botlek area are various. Single sheet pile walls with anchors are constructed of shallow water. The deeper water depths are constructed as combi-wall with relieving structure. This thesis will focus on combi-walls with relieving structures.

The Sint Laurenshaven, port number 4313 quay wall structure is chosen as reference structure because of:

- The executed deepening before;
- The complex structure of the bearing and tension pile;
- The unusual combi-wall: Peiner piles:
- The challenging angle of the front wall.

The principle cross section of the quay wall structure of Sint Laurenshaven, port number 4313 is pictured in Figure 36 Principle cross section reference structure thesis.

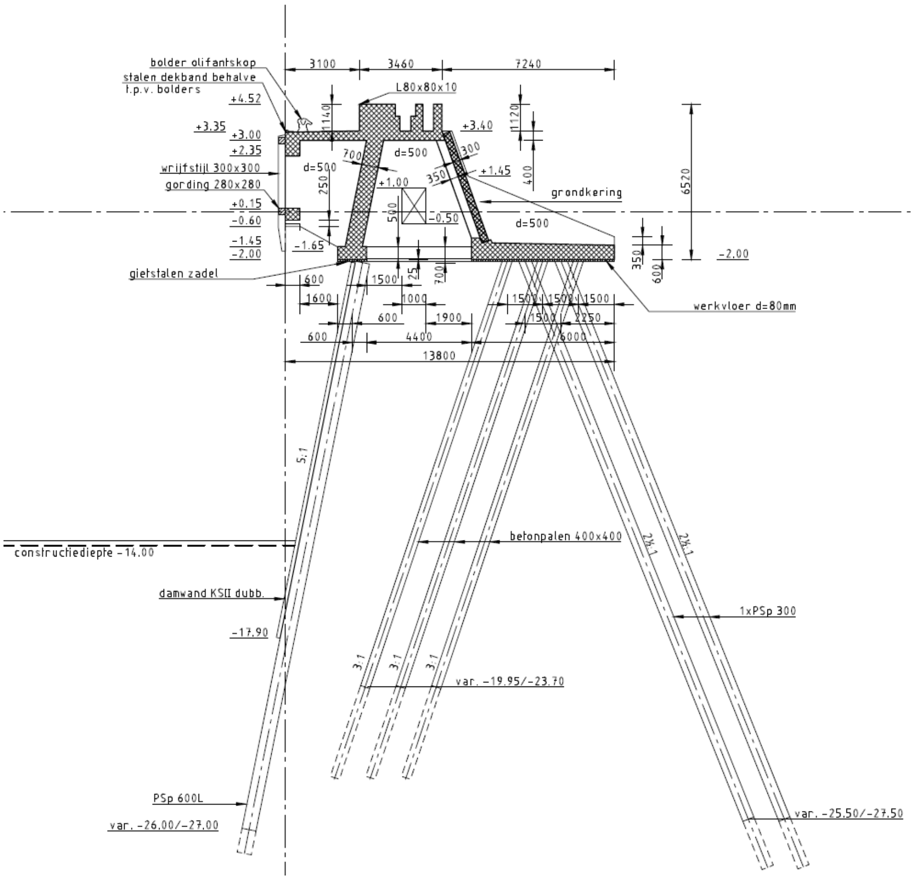

Figure 36 Principle cross section reference structure thesis

5 Bibliography

Leatemia, I. (2007). *Herbeschouwing kademuren.* Rotterdam: Gemeentewerken Rotterdam.

Port of Rotterdam Autority. (n.d.). *Portmaps kademuren en glooiingen*. Retrieved 3 12, 2017, from https://portmaps.ad.portofrotterdam.com/

SBRCURnet. (2014). *Publicaion 221E "Quay walls, Second Edition".* Rotterdam, The Netherlands: CRC Press/Balkema.

4 Structural engineering reference structure

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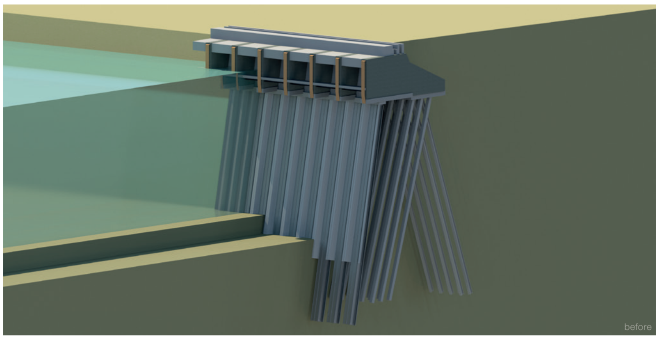

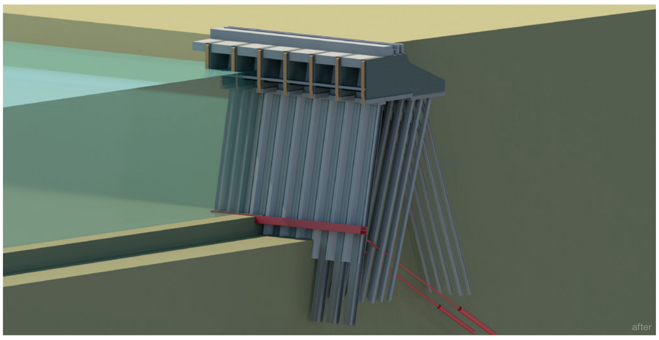

DEEPENING OF AN EXISTING COMBI WALL

STRUCTURAL ENGINEERING REFERENCE STRUCTURE

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Deepening of an Existing Combi Wall

Structural engineering reference structure

12 juni 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
Supervisor school:	**H.J. Dommershuijzen MSc**
Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

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Summary

The Eurocode divides the limit state into ULS = Ultimate limit state and SLS = Serviceability limit state. The ULS is limit state with the design approach for the extreme values. The SLS is the check for the structure while in use. The failure mechanisms which will be checked by the Plaxis model are:

- SLS = Serviceability limit state
- ULS = Ultimate limit state
    - STR = Structural limit state
        - Failure of front wall
        - Failure of anchor rod
        - Failure of bearing piles
        - Failure of tension piles
    - GEO = Geotechnical limit state
        - Failure of pile bearing
        - Insufficient passive resistance of front wall
        - Failure of anchor tension resistance
        - Overall stability Bishop
        - Overall stability Kranz
    - HYD = Hydraulic soil failure limit state
        - Heave
        - Piping (erosion)

The SLS limit state is used in the original calculation, so the quay wall structure will be calculated as SLS as base. The purpose of the research is not to prove the solution for the specific quay wall structure, but for a type of structure. The failure mechanism of the ULS will be checked by the SLS partial factors. The effect of the deepening and the solutions will be checked and not the exact results for this quay wall structure.

The HS model includes the stiffness and the deformations of the soil. The Hardening soil model will be used for this research. The HS model is the most suitable for relieving structure because of the influence of the structure to the soil.

The figures below shows the light grey parts of the concrete superstructure and the dark grey parts counterforts.

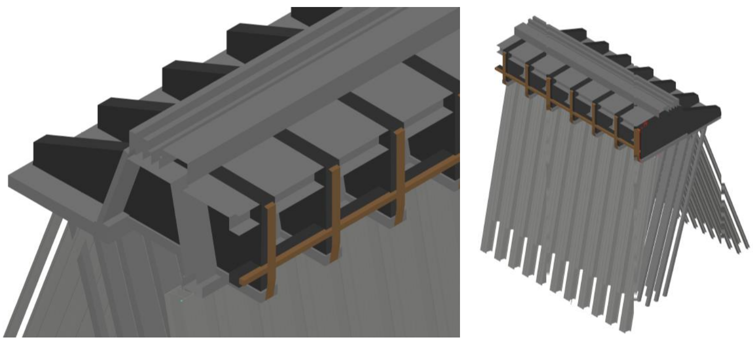


Figure 1 overview of the structure

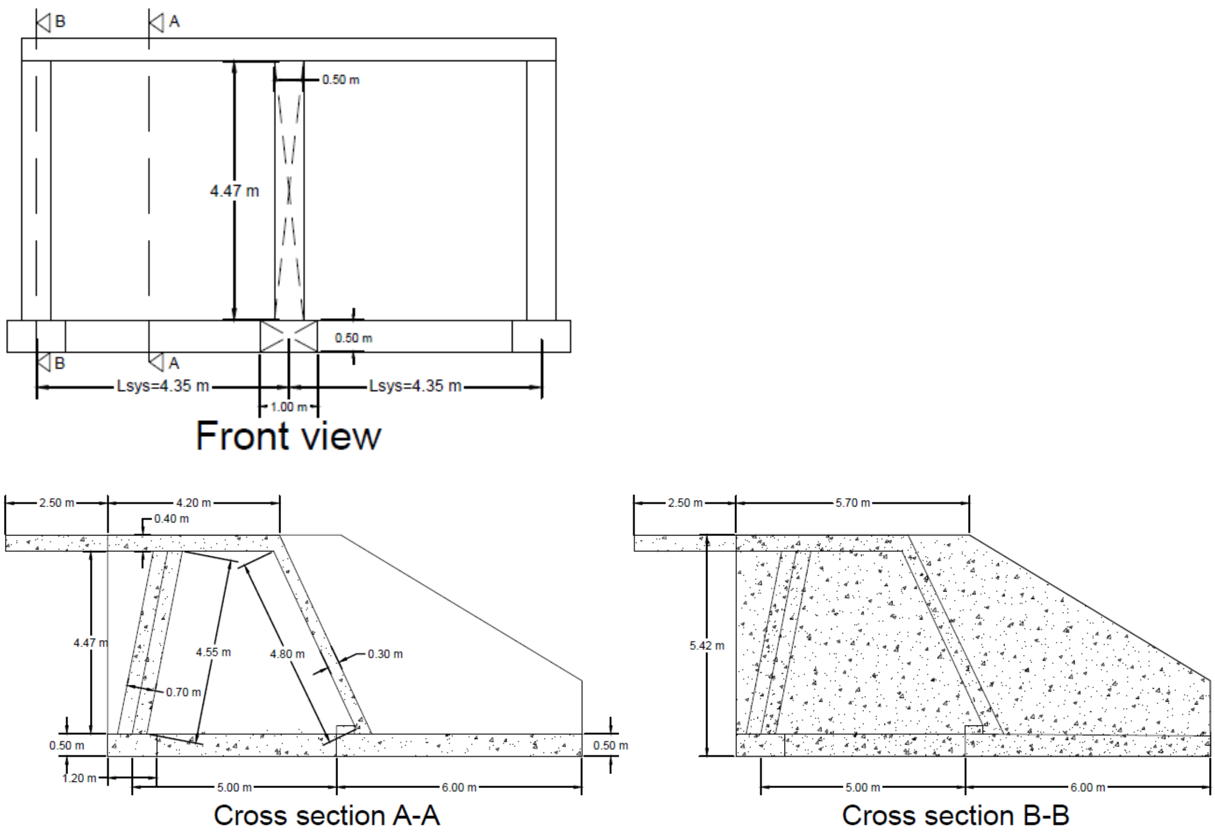


Figure 2 overall overview of the superstructure front view and cross section A-A and B-B

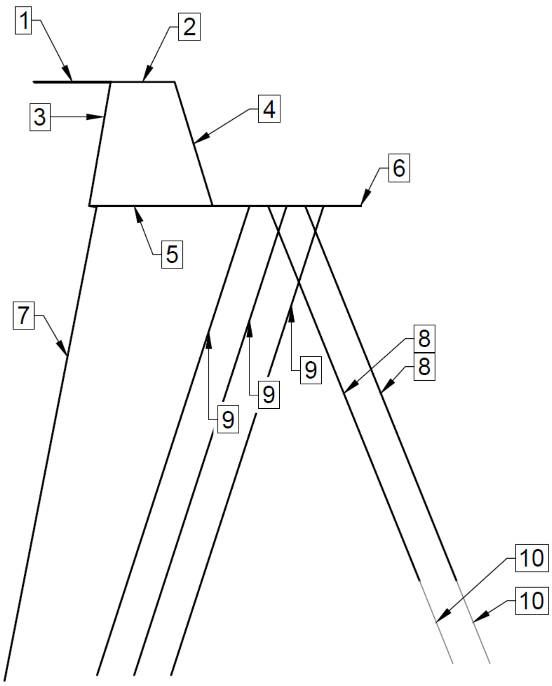


Figure 3 number of the structural elements according to the next chapters of this report

The summary of the parameters in the plaxis model are shown in the tables below.

Table 1 summary plate parameters

Plate parameters					
nr	Name	EA (kN/m)	EI (kNm/m)	v (-)	w (kN/m/m)
1	horizontal 1	10,712,500	106,294	0.2	8.40
2	horizontal 2	11,900,000	158,666	0.2	9.60
3	Vertical 1	20,825,000	850,354	0.2	16.80
4	Vertical 2	8,925,000	66,937	0.2	7.20
5	Counterfort full	18,704,885	42,670,000	0.2	15.10
6	Counterfort reduced	26,860,488	10,550,000	0.2	9.66
7	Psp60L	5,200,000	358,000	0	2.02

Table 2 summary node-to-node parameters

Node-to-node parameters			
nr	Name	EA (kN/m)	Lspacing (m)
8	Tension pile	3,255,000	1.5

Table 3 summary embedded beam row parameters

Embedded beam row parameters						
nr	Name	E (kN/m2)	A (m2)	Ispacing (m)	Fmax (kN)	Tmax (kN/m2)
9	Bearing pile	30,000,000	0.16	1.5	1,600	192
10	Tension pile end piece	210,000,000	0.0155	1.5	0	192

The loads of the structure are modulated as figure X.

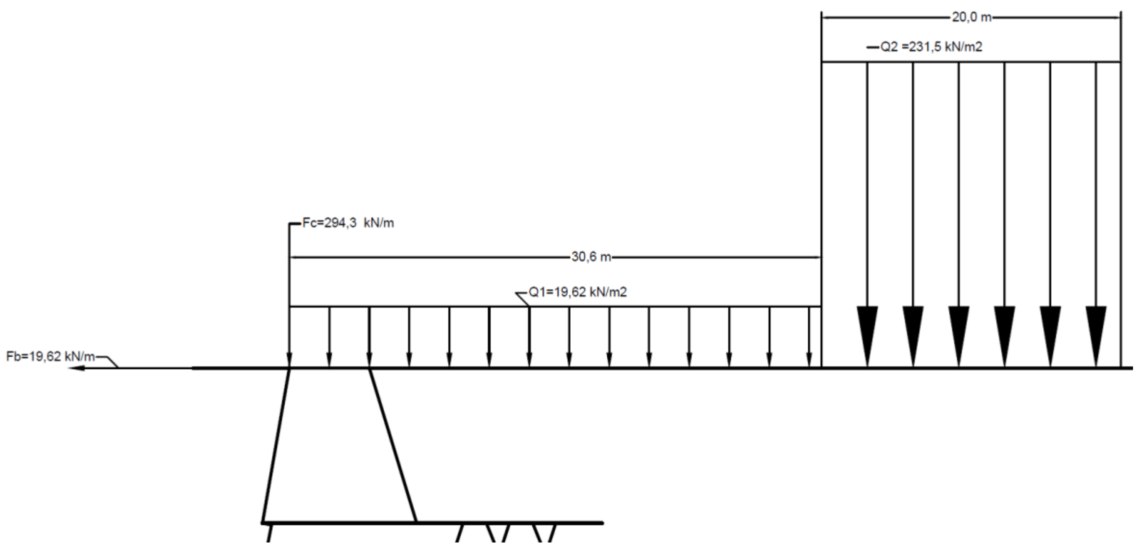


Figure 4 modulation of the loads

The ground parameters are determined by the original calculation and with the experienced formulas by the NEN-6740. The parameters are determined according to the cone penetration test pictured on Figure 5. The parameters of the plaxis model are shown in Table 4.

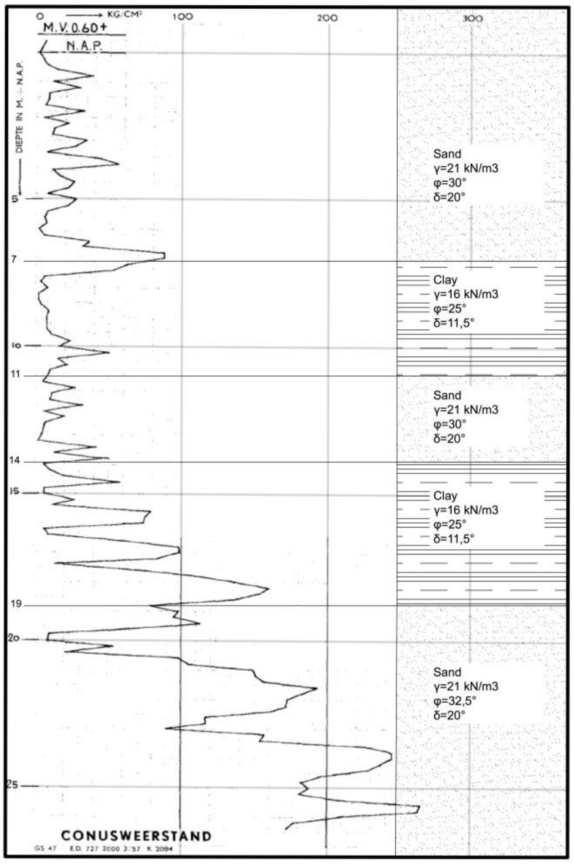


Figure 5 cone penetration test results

Table 4 ground parameter plaxis model

Soil parameter			**Sand**	**Clay**	**Sand**	**Clay**	**Sand**
Symbol	Description	Unit					
γsat	Saturared weight density of the soil	kN/m3	21	16	21	16	21
γunsat	Unsaturared weight density of the soil	kN/m3	18				
E50 ref	Secant stiffness modulus at a 50% deviatoric stress	kPa	28600	6000	18400	6000	31700
Eoed ref	Oedometric stiffness modulus	kPa	28600	3000	18400	3000	31700
Eur ref	Unloading reloading stiffness modulus	kPa	85800	15000	55200	15000	95100
ψ	Dilatancy angle	°	30	0	30	0	32.5
φ	Internal angle of friction	°	30	25	30	25	32.5
Rinter	Interface	-	0.8	0.66	0.8	0.66	0.8
C'ref	Effective cohesion in drained conditions	kPa	0	10	0	10	0
m	amount of stress dependency (power)	-	0.5	1	0.5	1	0.5

The plaxis model is chosen for plaxis instead of D-sheet in this graduation thesis because of:

- The geometry of the structure, difficult to module the angle of the front wall into D-sheet;
- Easy modulation of the relieving platform;
- The difficulty of the modulation of the possible solutions into D-sheet;
- The advanced geotechnical calculation of Plaxis;
- Advanced calculation method of the clay layers.

The following points are seen as uncertainty of the plaxis model:

- Assumed represented high parameters instead of real parameter in the plaxis model;
- Theoretical calculations are conservative compared to the real situation;
- Extremely high surface load;
- The inaccuracy margin of ± 30% of Plaxis deformations (cur 221);
- Large influence of the clay layers, more than D-sheet and blum calculations;
- Schematisation of the structure into plaxis.

The summary of the deviation and the effect of the deepening to the structure are shown in the table below. The deviation is calculated after deepening to before deepening. The results of the comparison are shown in Table 5.

Table 5 summary of the values and the effect of the deepening to the structure elements

Phase	Annotation	Load combination 4 before deepening	Load combination 4 after deepening	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.121	-10.03%
Structural				
Bending moment front wall	kNm/m	953	1115	16.99%
Shear force front wall	kN/m	229	232	1.49%
Normal force front wall	kN/m	799	847	5.98%
Normal force bearing pile 3	kN	595	599	0.67%
Bending moment bearing pile 3	kNm/m	92	119	29.35%
Normal force tension pile 1	kN	95	119	25.26%
ULS				
Deformations x top quay wall	m	0.13	0.18	40.25%

According to the results of the plaxis calculation the following structure elements are the biggest increase of the value so these elements are critical:

- Front wall : Bending moment;
- Tension pile 1: Normal force;
- Deformations;
- Reduction of the passive pressure.

The following failure mechanisms are critical according to deepening of the port bed 2.8 meters:

- Structural:
    - Failure of front wall;
    - Failure of tension piles;
- Geotechnical:
    - Insufficient passive resistance of front wall
    - Failure of anchor/pile tension resistance

The most feasible solution must meet the following requirements to make the upgrade feasible:

- Safety factor ≥ 1.25;
- Bending moment front wall ≤ 935 kNm;
- Normal force tension pile 1 ≤ 95 kN;
- Deformations x top quay wall ≤ 0.13 m.

Table of contents

1 Introduction

This document is made according to the third secondary research question: What are the failure mechanism and the failure structure member of the reference combi-wall structure?

The result of this report is the conclusion of the failure mechanism and failure structural elements. The modulated model is a reference model for quay wall structures with a low relieving floor. The values of the modulated will be compared with the results after deepening and the solution. This comparison will present the effect of the deepening and the solution according to the failure mechanism and the failure structural elements. The safety factor of the model is not the quay safety factor.

The safety factor is not the exact safety factor because of:

- Calculation with assumed ground parameters;
- Calculated with the design approach SLS;
- Corrosion effect not modulated.

1.1 Principle cross section

The principle cross section of the reference quay structure of EBS is shown on Figure X.

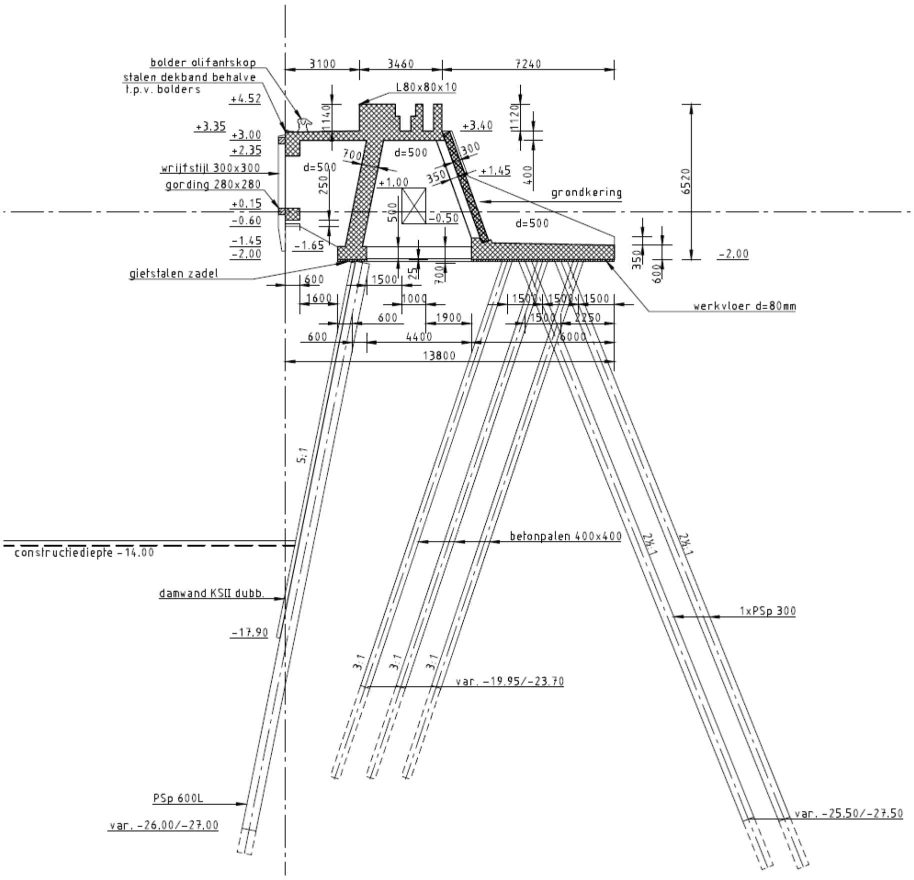


Figure 6 Principle cross section reference structure thesis

2 Failure mechanism

Quay wall structures can fail by several failure mechanisms. The calculation of the structure will be done by the probabilistic safety verification. This calculations uses design values for soil properties, loads, geometry, strength and stiffness.

2.1 Limit states

The Eurocode divides the limit state into ULS = Ultimate limit state and SLS = Serviceability limit state. The ULS is limit state with the design approach for the extreme values. The SLS is the check for the structure while in use. The failure mechanisms which will be checked by the Plaxis model are:

- SLS = Serviceability limit state
- ULS = Ultimate limit state
    - STR = Structural limit state
        - Failure of front wall
        - Failure of anchor rod
        - Failure of bearing piles
        - Failure of tension piles
    - GEO = Geotechnical limit state
        - Failure of pile bearing
        - Insufficient passive resistance of front wall
        - Failure of anchor tension resistance
        - Overall stability Bishop
        - Overall stability Kranz
    - HYD = Hydraulic soil failure limit state
        - Heave
        - Piping (erosion)

The failure mechanisms which not will be calculated are:

- ULS = Ultimate limit state
    - EQU= Equilibrium
    - STR = Structural limit state
        - Failure of superstructure
        - Failure of joints between elements
    - FAT= Fatique
    - UPL= Uplift

The SLS limit state is used in the original calculation, so the quay wall structure will be calculated as SLS as base. The purpose of the research is not to prove the solution for the specific quay wall structure, but for a type of structure. The failure mechanism of the ULS will be checked by the SLS partial factors. The effect of the deepening and the solutions will be checked and not the exact results for this quay wall structure.

3 Basics of design

This part is about the transfer of the original design calculation to the research. The original calculation will be used as the basics of the design and the calculations. The structure consists of a superstructure and a foundation. The modulation and the schematisation is checked and approved by Dirk-Jan Jasper Fock, Witteveen+Bos.

3.1 General assumptions

The general assumptions of the design are:

- The calculation will be for a cross section of 1.0 metre width, so every load and parameter will be calculated to the structure of 1.0 metre width.
- The parameters are characteristics;
- $g = 9.81\ m/s^2$
- $\gamma_{gw} = 10\ kN/m^3$
- $\gamma_w = 10\ kN/m^3$
- Design approach SLS partial factor 1.0.

3.1.1 Vessel

The representative vessel of the current quay structure is the Panamax vessels. The aim is the make the quay wall structure assessable for the new panama vessels. See the figure and the table below for the illustration and the vessel characteristics.

Table 6 characteristics of the Panamax and new Panamax vessels

Vessel type Panamax vs new Panamax		
Type	Panamax	New Panamax
Length	294.13	366 m
Width	21.31	49 m
Draft	12.04	15.2 m
DWT	52,500	120,000
TUE	5,000	13,000

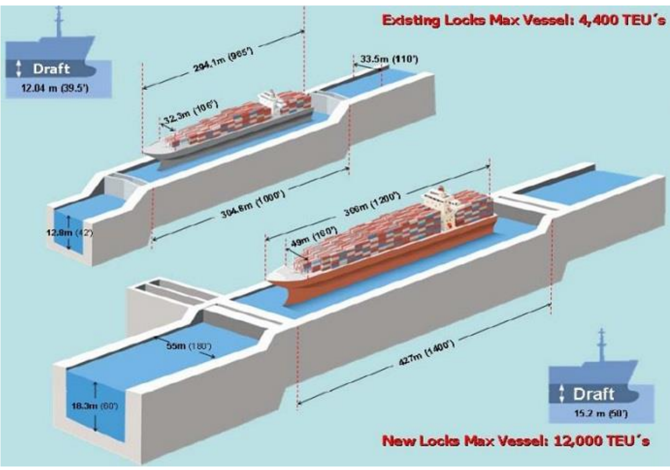


Figure 7 illustration of the Panamax and the new Panamax vessels

3.1.2 Illustration of the structure

The structure is an open structure besides of the counterforts. This counterfort transfers the horizontal forces of the front wall to the tension piles and the horizontal forces to the bearing piles. The figures below shows the light grey parts of the concrete superstructure and the dark grey parts counterforts.

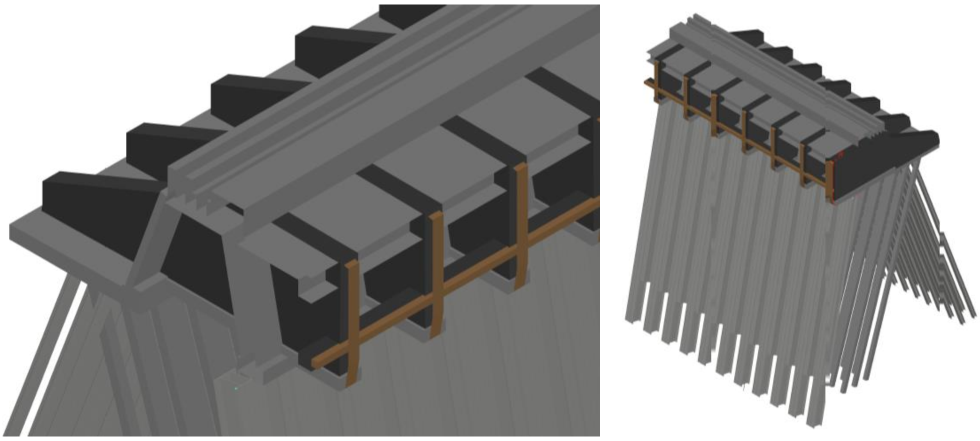


Figure 8 overview of the structure

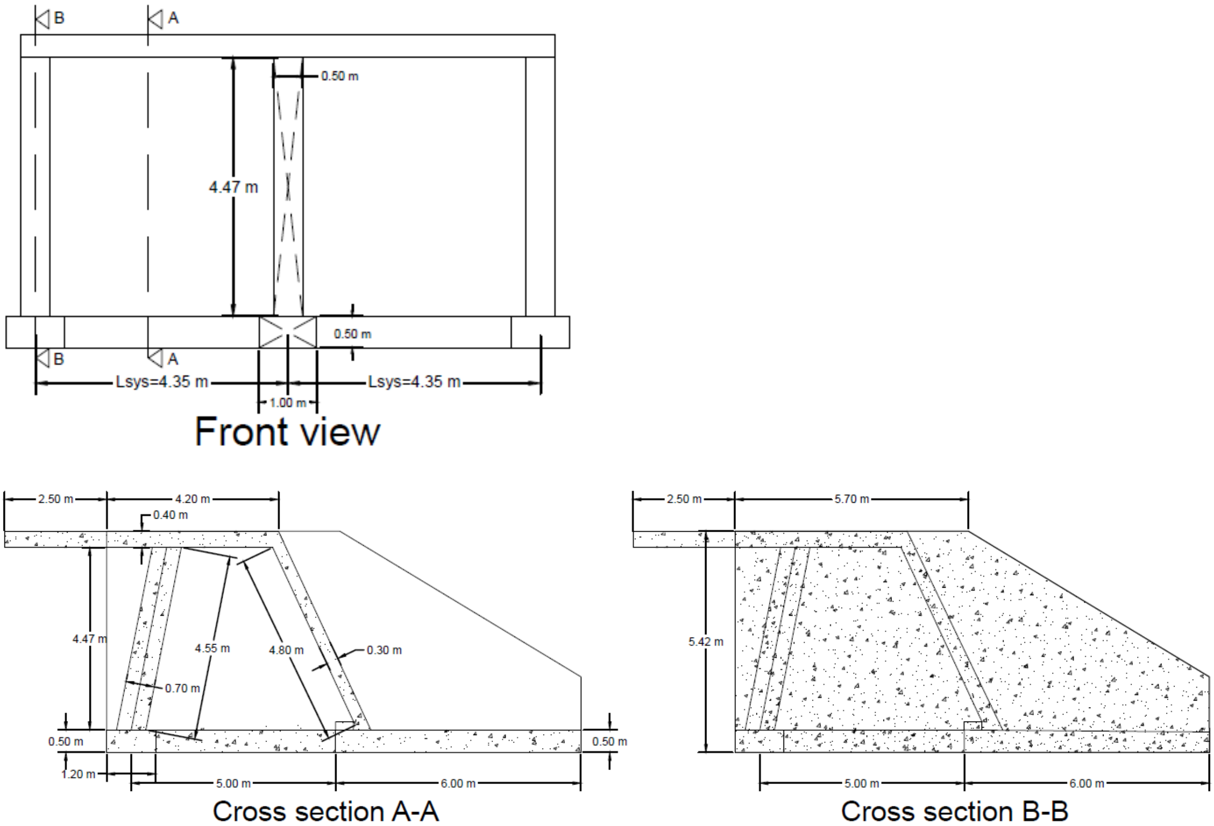


Figure 9 overall overview of the superstructure front view and cross section A-A and B-B

The structure is divided into different structural member. The structural members are exploded to underpin the location of the structural elements. The structure element will be explained in the paragraphs below.

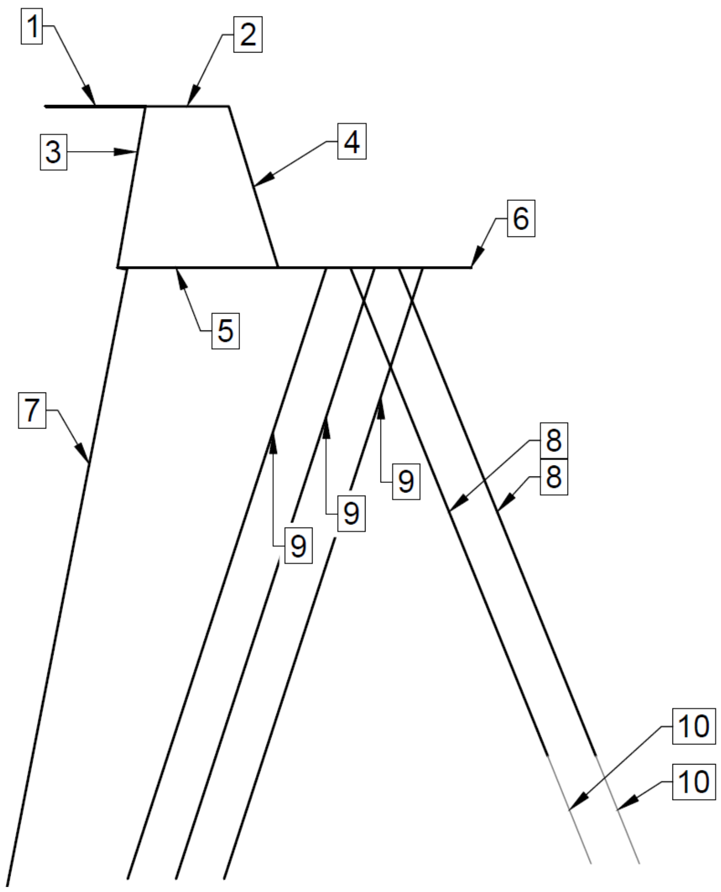


Figure 10 number of the structural elements according to the next chapters of this report

3.2 Superstructure (1, 2, 3, 4, 5 and 6)

The superstructure of the quay wall consists of counterfort, relieving floor and a solid concrete structure. The structure elements are be descripted in the paragraphs below.

3.2.1 General parameters

The parameters of the concrete which are assumed for the superstructure are according to the original calculation.

$$f_{ck} = 300\frac{kg}{cm2} = 2{,}943\frac{N}{cm2} = 29{,}430\frac{kN}{m2} = 29.43\frac{N}{mm2}$$

$$f_{ck} = 30\ \frac{N}{mm2}$$

The concrete class which will be used is C25/30.

NEN-EN 1992-1-1 is used for the determination of the stiffness parameters of concrete.

$$E = 22{,}250 + 250 * f_{ck}$$

$$E = 22{,}250 + 250 * 30$$

$$E = 29{,}750\ N/mm^2 = 29.75\ kN/mm^2 = 29{,}750{,}000\ kN/m^2$$

Assumed is the $Rinter = 0.7$, so the interface of concrete to soil is 0.7, also assumed is $\nu = 0.2$.

3.2.1 Plate horizontal 1(1)

This element is to retain the vertical surface forces, the crane forces and the bollard forces. . The figure below shows the dimensions of the plate.

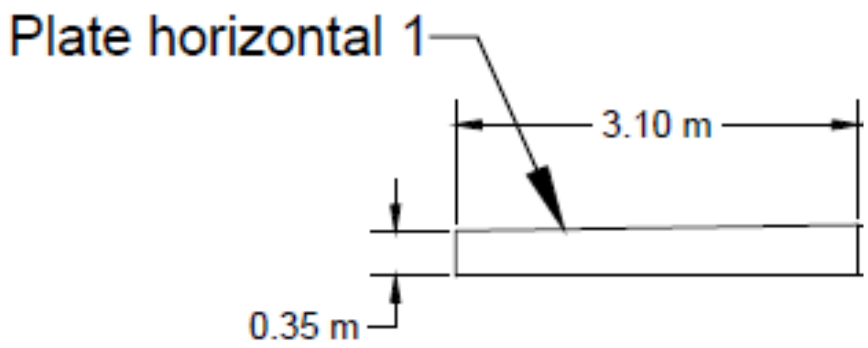


Figure 11 overview of the plater horizontal 1

The parameters of the element are:

$\mathrm{A} = 0.35 * 1 = 0.35\, m^2$

$E = 29{,}750{,}000\ kN/m^2$

$I = \frac{1}{12} * 1 * 0.35^3 = 0.00357\, m^4/m$

$w = A * \gamma = 0.35 * 24 = 8.4\ kN/m$

$EA = 10{,}412{,}500\ kN/m$

$EI = 106{,}294\ kNm^2/m$

3.2.2 Plate horizontal 2 (2)

This element is to retain the vertical surface forces, the crane forces and the bollard forces. The figure below shows the dimensions of the plate.

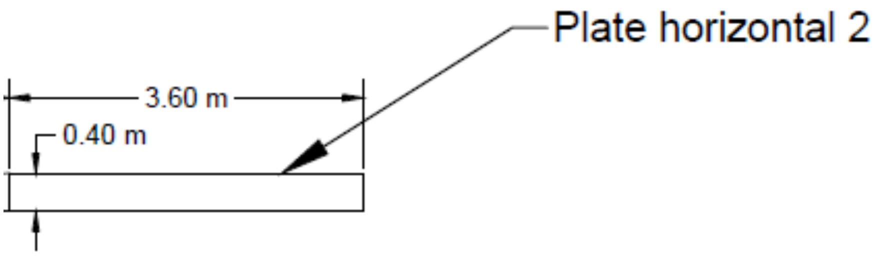


Figure 12 overview of the plater horizontal 1

The parameters of the element are:

$$A = 0.4 * 1 = 0.4\ m^2$$

$$E = 29{,}750{,}000\ kN/m^2$$

$$I = \frac{1}{12} * 1 * 0.4^3 = 0.0053\ m^4/m$$

$$w = A * \gamma = 0.4 * 24 = 9.6\ kN/m$$

$$EA = 11{,}900{,}000\ kN/m$$

$$EI = 158{,}666\ kNm^2/m$$

3.2.3 Plate vertical 1(3)

This element is transfers the horizontal forces of the crane and the surface to the front wall. The figure below shows the dimensions of the plate.

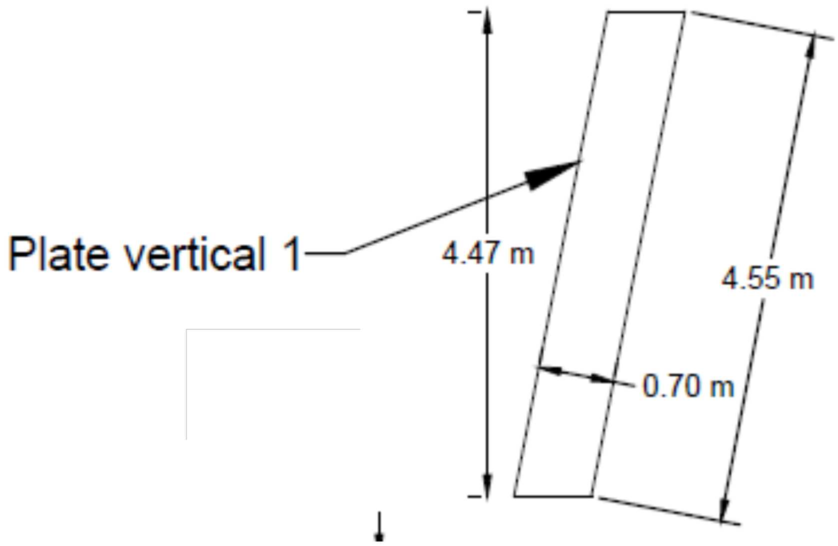


Figure 13 overview of the plater vertical 1

The parameters of the element are:

$A = 0.7 * 1 = 0.7\ m^2$

$E = 29{,}750{,}000\ kN/m^2$

$I = \frac{1}{12} * 1 * 0.7^3 = 0.029\ m^4/m$

$w = A * \gamma = 0.7 * 24 = 16.8\ kN/m$

$EA = 20{,}825{,}000\ kN/m$

$EI = 850{,}354\ kNm^2/m$

3.2.4 Plate vertical 2 (4)

This element is to retain the horizontal ground stresses. The figure below shows the dimensions of the plate.

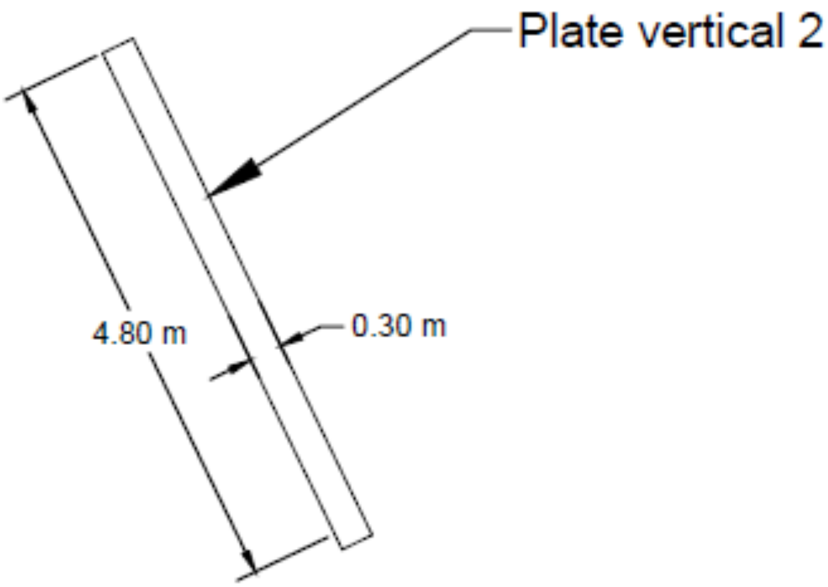


Figure 14 overview of the plater vertical 2

The parameters of the element are:

$$A = 0.3 * 1 = 0.3\ m^2$$

$$E = 29{,}750{,}000\ kN/m^2$$

$$I = \frac{1}{12} * 1 * 0.3^3 = 0.00225\ m^4/m$$

$$w = A * \gamma = 0.3 * 24 = 7.2\ kN/m$$

$$EA = 8{,}925{,}000\ kN/m$$

$$EI = 66{,}937\ kNm^2/m$$

3.2.5 Floor relieving structure (6)

The floor of the relieving structure will be modelled as build. The measurements of the floor are the same as the original calculation. See the figure below for the drawing of the floor.

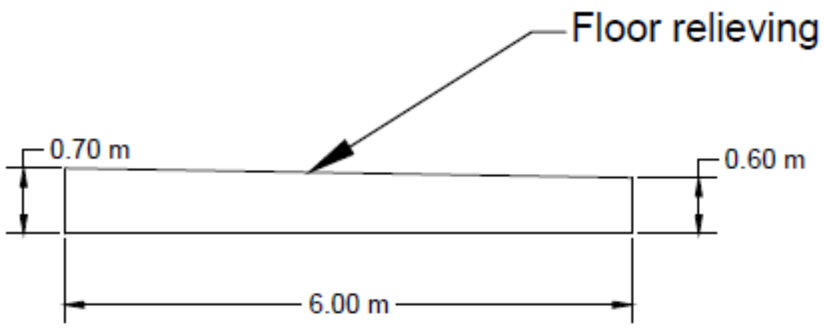


Figure 15 overview of the concrete element of the relieving floor

The parameters of the concrete are the same as the whole superstructure:

$Average\ height = 0.65\ m$

$\gamma = 24\ kN/m^2$

$E = 29{,}750{,}000\ kN/m^2$

$Rinter = 0.7$

If the floor will be modulated as plate the parameters will be:

$\gamma = 24\ kN/m^2$

$E = 29{,}750{,}000\ kN/m^2$

$A = 1 * 0.65 = 0.65\ m^2$

$I = \frac{1}{12} * 1 * 0.65^3 = 0.0229\ m^4/m$

$w = A * \gamma = 0.65 * 24 = 15.6\ kN/m$

$EA = 19{,}337{,}500\ kN/m$

$EI = 680{,}641\ kNm^2/m$

$Rinter = 0.7$

3.2.6 Counterforts (5+6)

The forces and stress of the wall and the soil are transferred by the counterforts. The cross section of counterfort is divided into 2 elements.

3.2.6.1 Parameters counterfort

The counter wall will be modulated as 2 structural elements, because the counterfort wall height is constructed as a slope. The slope of the counterforts can be seen on figure 3 and figure 4. The cross sections of the counterfort will also be divided in to two cross sections because of sloping shape of the counterforts. The full and the reduced cross section are shown in figure X.

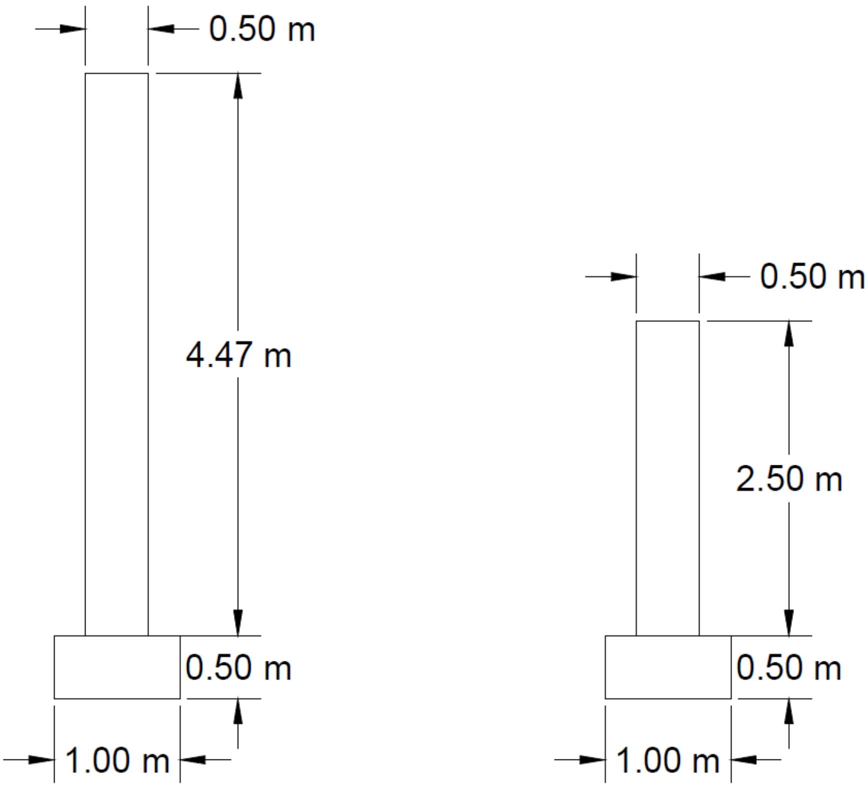


Figure 16 cross sections of the counterfort full and the reduced counterfort

The stiffness and strength of the counter fort will be modelled as plates in Plaxis. The system distance between two counterforts reduces the parameters of the plaxis calculation. The parameters of the concrete are assumed and calculated as below:

C25/30

$f_{ck} = 30\ N/mm2$

$E = 29{,}750\ N/mm^2 = 29.75\ kN/mm^2 = 29{,}750{,}000\ kN/m^2$

FULL COUNTERFORT (5)

The schematisation and explanation of the symbols of the calculation are shown in Figure 17.

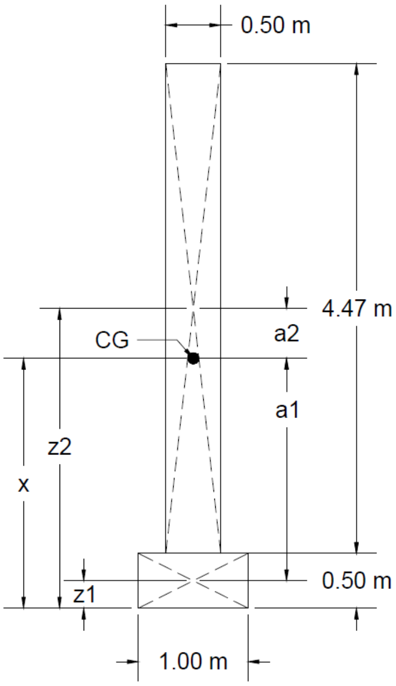


Figure 17 schematisation full counterfort

$$Centre\ of\ gravity\ full\ (x) = \frac{\sum A * z^2}{\sum A} = \frac{\left(0.5 * 1 * \frac{0.5}{2}\right) + 0.5 * 4.47 * \left(0.5 + \frac{4.47}{2}\right)}{0.5 * 1 + 0.5 * 4.47} = 2.281 \text{ m}$$

$$I_{full} = \sum \frac{1}{12} * b * h^3 + \sum A * a^2$$

$$I_{full} = (I_1 + A_1 * {a_1}^2) + [I_2 + A_2 * {a_2}^2]$$

$$I_{full} = \left(\frac{1}{12} * 1.0 * 0.5^3\right) + (1.0 * 0.5 * (2.281 - 0.25)^2) + \left[\frac{1}{12} * 0.5 * 4.47^3\right] + \left[0.5 * 4.47 * \left(\frac{4.47}{2} + 0.5 - 2.281\right)^2\right]$$

$$I_{full} = 6.24\ m^4$$

$$A = 0.5 * 1.0 + 4.47 * 0.5 = 2.735\ m^2$$

$$EA_{full} = 29{,}750{,}000 * 2.735 = 81{,}370{,}000\ kN/m$$

$$EI_{full} = 29{,}750{,}000 * 6.24 = 185{,}500{,}000\ kNm^2/m$$

$$w_{full} = \frac{A * \gamma}{l_s} = \frac{((0.5 * 4.47) + (1.0 * 0.5)) * 24}{4.35} = 15.1\ kN/m$$

REDUCED COUNTERFORTS (6)

The schematisation and explanation of the symbols of the calculation are shown in Figure 18.

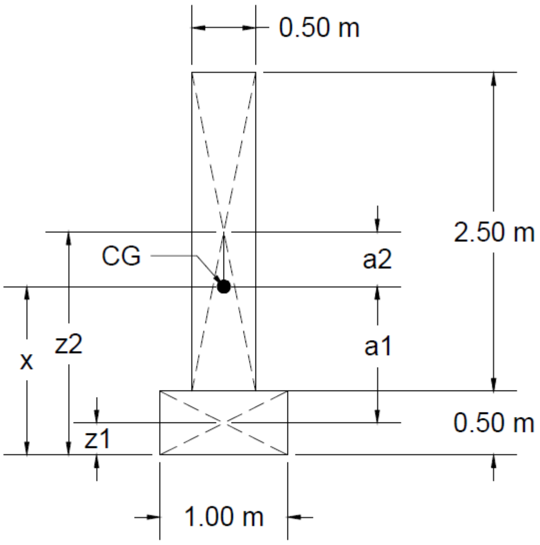


Figure 18 schematisation reduced counterfort

$$Centre\ of\ gravity\ reduced = \frac{\sum A * z^2}{\sum A} = \frac{\left(0.5 * 1 * \frac{0.5}{2}\right) + 0.5 * 2.5 * \left(0.5 + \frac{2.5}{2}\right)}{0.5 * 1 + 0.5 * 2.5} = 1.321 \text{ m}$$

$$I_{reduced} = \sum \frac{1}{12} * b * h^3 + \sum A * z^2$$

$$I_{reduced} = I_1 + A_1 * {a_1}^2 + I_2 + A_2 * {a_2}^2$$

$$I_{reduced} = \left(\frac{1}{12} * 1.0 * 0.5^3\right) + (1.0 * 0.5 * (1.321 - 0.25)^2) + \left[\frac{1}{12} * 0.5 * 2.5^3\right] + \left[0.5 * 2.5 * \left(\frac{2.5}{2} + 0.5 - 1.321\right)^2\right]$$

$$I_{reduced} = 1.46\ m^4$$

$$A = 0.5 * 1.0 + 2.5 * 0.5 = 1.75\ m^2$$

$$EA_{reduced} = 29{,}750{,}000 * 1.75 = 52{,}000{,}000\ kN/m$$

$$EI_{reduced} = 29{,}750{,}000 * 1.46 = 43{,}600{,}000\ kNm^2/m$$

$$w_{reduced} = \frac{A * \gamma}{l_s} = \frac{((0.5 * 2.5) + (1.0 * 0.5)) * 24}{4.35} = 9.66\ kN/m$$

The counterforts will be modulated as plates. The full counterfort parameters will be of the front to the first bearing pile. The reduced counter fort will be the remaining part of the relieving floor. The parameters of the plaxis model will be per metre. The parameters of the counterfort and the relieving floor will be combined to one plate with the represented parameters for the schematization of the structure. Per system length of 4.45 metre, the counterfort represents 1 metre of the floor and the relieving floor represents 3.35 metre. Based on these values the characteristics for the plaxis calculation will be determined.

Representative parameters plate full counterfort:

$$EA_{r,full} = \frac{EA_{full}}{l_s} = \frac{81{,}366{,}250}{4.35} = 18{,}700{,}000\ kN/m$$

$$EI_{r,full} = \frac{EI_{full}}{l_s} = \frac{185{,}500{,}000}{4.35} = 42{,}670{,}000\ kNm^2/m$$

$$w_{r,full} = 15.1\ kN/m$$

Representative parameters plate full counterfort:

$$EA_{r,reduced} = \frac{EA_{reduced} + 3.35 * EA_{relievingfloor}}{l_s}$$

$$EA_{r,reduced} = \frac{52{,}062{,}500 + 3.35 * 19{,}337{,}500}{4.35} = 26{,}860{,}000\ kN/m$$

$$EI_{r,reduced} = \frac{EI_{reduced} + 3.35 * EI_{relievingfloor}}{l_s}$$

$$EI_{r,reduced} = \frac{43{,}600{,}000 + 3.35 * 680{,}641}{4.35} = 10{,}550{,}000\ kNm^2/m$$

$$w_{r,reduced} = 9.66\ kN/m$$

3.2.6.2 Summary parameters counterfort

The parameters for the plaxis calculation are:

Full counterfort (5):

$\gamma = 24\ kN/m^2$

$E = 29{,}750{,}000\ kN/m^2$

$A = 2.735\ \ m^2$

$EA_{r,full} = 18{,}700{,}000\ kN/m$

$EI_{r,full} = 42{,}670{,}000\ kNm^2/m$

$w_{r,full} = 15.1\ kN/m$

Reduced counterfort (6):

$\gamma = 24\ kN/m^2$

$E = 29{,}750{,}000\ kN/m^2$

$A = 1.75\ m^2$

$EA_{r,reduced} = 26{,}860{,}000\ kN/m$

$EI_{r,reduced} = 10{,}550{,}000\ kNm^2/m$

$w_{r,reduced} = 9.66\ kN/m$

3.3 Foundation (7, 8, 9 , 10)

The foundation of the quay wall structure is constructed by a combi-wall with a relieving platform. That platform transfers the forces to the subsoil by bearing and tension piles. See the figure X below for the illustration of the foundation of the original calculation.

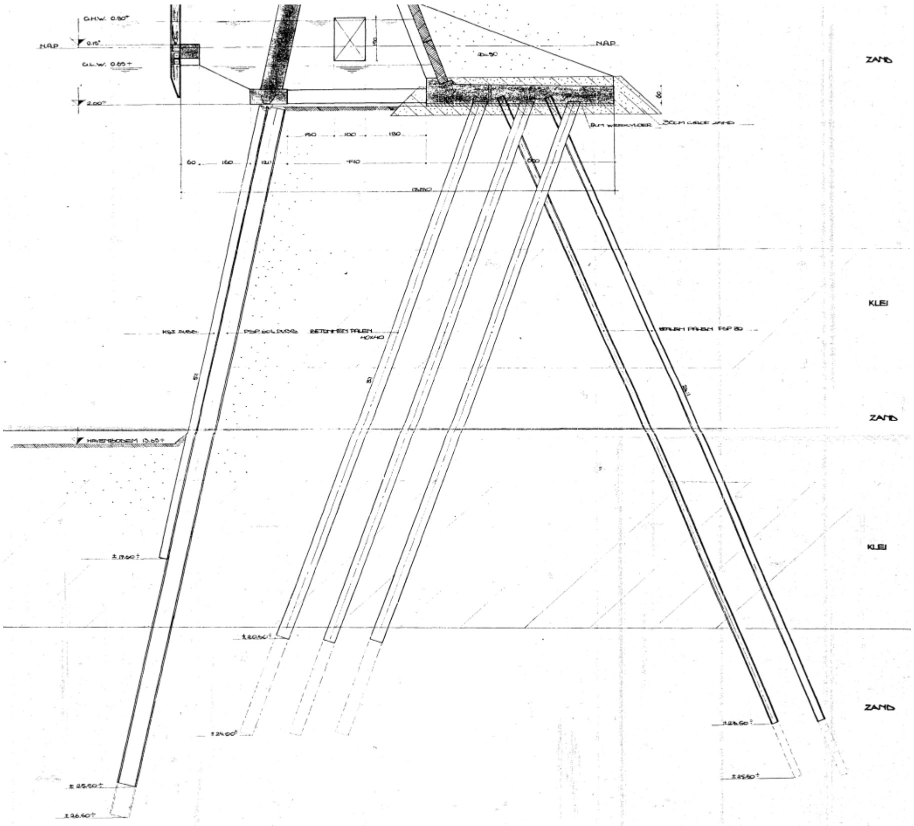


Figure 19 illustration of the foundation

3.3.1 Combi-wall (7)

The original calculation assumed a type of front wall. The chosen type of combi-wall is 2 PSP60L with KSII intermediate piles. PSP60L piles are connected by locks and act like one. The parameters of the KSII intermediate piles are unknown. See the figure below for the parameters of the PSP60L piles.

Profil-bezeich-nung	2 Bohlen mit 2 Schlössern							
PSp	F cm²	G kg/m	J_x cm⁴	i_x cm	W_x cm³	J_y cm⁴	i_y cm	W_y cm³
30 L	244	192	46 060	13,7	2 530	74 200	17,4	2270
30 S	290	228	56 170	13,9	2 990	87 060	17,3	2650
35 L	330	259	84 980	16,1	4 030	141 900	20,7	3630
35 S	368	289	96 030	16,1	4 510	162 000	21,0	4130
40 L	356	280	116 800	18,1	4 950	150 400	20,5	3890
40 S	404	317	132 400	18,1	5 560	174 700	20,8	4500
50 L	401	315	198 300	22,2	6 940	169 700	20,6	4380
50 S	443	348	220 800	22,3	7 660	190 600	20,7	4900
60 L	436	342	297 700	26,1	8 860	184 400	20,6	4740
60 S	467	366	326 500	26,4	9 660	200 100	20,7	5150
80 S	656	515	758 700	34,0	17 300	284 800	20,8	7310

Figure 20 characteristics of the double Psp piles

The characteristics of the wall are:

$$Piling\ depth\ PsP60L = -26.00\ metres$$

$$Pilling\ depth\ intermediate\ KSII\ piles = -17.60\ metres$$

$$f_y = 235\ N/\mathrm{mm}^2 = 0.235\ \mathrm{kN/mm}^2 = 235{,}000\ \mathrm{kN/m}^2$$

$$\mathrm{A\ psp60L\ double\ pile} = \frac{436\ \mathrm{cm}^2}{l_s} = \frac{436\ \mathrm{cm}^2}{1.66} = 0.026\ m^2$$

$$I_y\ \mathrm{psp60L\ double\ pile} = 297{,}700\ \mathrm{cm}^4$$

$$I_y\ \mathrm{psp60L\ double\ pile\ per\ metre} = \frac{297{,}700}{l_s} = \frac{297{,}700}{1.66} = 179{,}337\ \mathrm{cm}^4/\mathrm{m} = 0.00179\ \mathrm{m}^4/\mathrm{m}$$

$$W_y = 8{,}600\ \mathrm{cm}^3 = \frac{8{,}600}{1.66} = 5{,}180\ cm^3 = 0.0052\ m3$$

$$\mathrm{w}\ PSP60\ L = 342\frac{kg}{\mathrm{m}} = 342 * 9.81 = 3{,}355\ \mathrm{kN}$$

$$\mathrm{w}\ PSP60\ L = \frac{3{,}355}{l_s} = \frac{3{,}355}{1.66} = 2.02\ \mathrm{kN/m}$$

$$E = 210{,}000\ N/mm^2 = 210\ kN/mm^2 = 210{,}000{,}000\ kN/m^2$$

The calculation of the strength and stiffness paremeters of the combi-wall is based on the PSP60L pile, so the intermediate pile are ignored as structural parameter. The parameters per metre will be the parameters of the PSp60L double sheet piles. All the forces and stress will be adopted by the double PSP 60 L piles.

$$EA = 200{,}000{,}000 * 0.026 \; = 5{,}200{,}000 \; kN/m$$

$$EI \; = 200{,}000{,}000 * 0.00179 = 358{,}000 \; kN\mathrm{m}^2/m$$

3.3.1.1 Reliability factor front wall

According to the original calculation the reliability factor is determined. The factor is determined by the check of the stresses in the front wall. The parameters are the original parameters used for the original determination.

The reliability factor is determined according to the following parameters:

$$M_{max} = \; 155.21 \; tm/1.5 \; m^1$$

$$N_{min} = 52.5 \; t/1.5 \; m^1$$

$$N_{max} = \; 110.5 \; t/1.5 \; m^1$$

$$N_{o.w.} = 2{,}960 \; kg$$

$$M_{re} = \; 52.5 * 0.3 = 15.75 \; tm/1.5 \; m1$$

$$W_y = 8{,}860 \; \mathrm{cm}^3$$

$$\mathrm{A}_{2 \; \mathrm{PSp20L}} = \; 0.0436 \; m^2 = 436 \; cm^2,$$

The reliability factor will be calculated by the appearanced and performenced stresses.

$$\sigma_{s,a} < \sigma_{s,p}$$

$$\sigma_{s,a} = \frac{M_{max}}{W_y} + \frac{N_{max}}{\mathrm{A}_{2 \; \mathrm{PSp20L}}} + \frac{N_{o.w.}}{\mathrm{A}_{2 \; \mathrm{PSp20L}}} - \frac{M_{re}}{W_y}$$

$$\sigma_{s,a} = \frac{151.21 * 10^5}{8.860} + \frac{110.5 * 10^3}{436} + \frac{2960}{436} - \frac{15.75 * 10^5}{8.860}$$

$$\sigma_{s,a} = 1.706 + 253 + 7 - 178 = 1.788 \; kg/cm^2$$

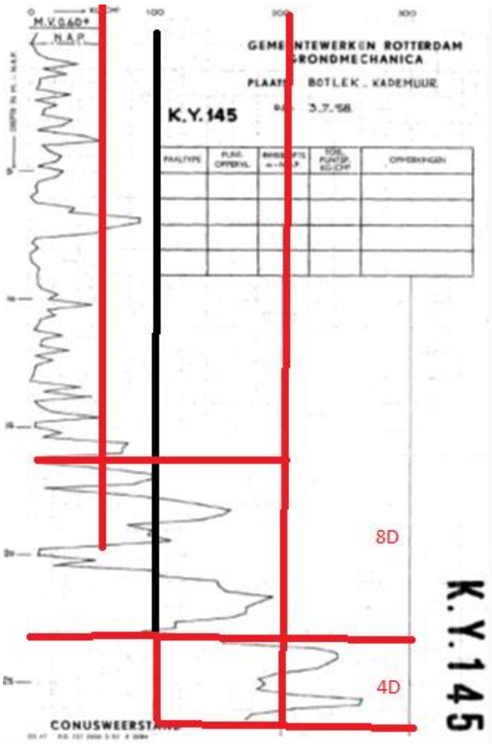
$$\sigma_{s,a} = 1.788 \; kg/cm^2 = 175 \; N/mm2$$

$$\sigma_{s,p} = 235 \; N/mm2$$

$$\text{Reliability factor} = \frac{\sigma_{s,p}}{\sigma_{s,a}} = \frac{235}{175} = \; 1.34$$

3.3.1.2 The bearing capacity of the front

The bearing capacity of the subsoil will be calculated by the method of Koppejan. The calculation will be as follows:



$$f_{b,max} = \frac{1}{2}\left[\frac{1}{2} * (p_1 + p_2) + p_3\right]$$

$$p_1 = 10\ MPa$$

$$p_2 = 20\ MPa$$

$$p_3 = 5\ MPa$$

$$f_{b,max} = \frac{1}{2}\left[\frac{1}{2} * (10 + 20) + 5\right] = 10\ MPa = 10.000\ kN/m^2$$

The cross section of the bearing capacity will be the boxed area between the two peiner piles. See the figure below for the boxed area between the two peiner piles.

$$A = 0.6 * 0.4 = 0.24\ m^2$$

$$F_{b,max} = f_{b,max} * A = 10.000 * 0.24 = 2.400\ kN$$

$$F_{p,max} = 1100\ kN$$

$1100\ kN < 2.400\ kN$, so is suitable.

$$\text{Reliabilty factor} = \frac{2.400}{1.100} = 2.18$$

3.3.1.3 Summary parameters combi-wall

$f_y = 235\ N/\mathrm{mm}^2$

$E = 210.000.000\ kN/m^2$

I_y psp60L double pile per metre $= 0.0179\ \mathrm{m}^4/\mathrm{m}$

$W_y = 0.0052\ \mathrm{m}^3$

A psp60L double pile $= 0.026\ m^2$

w $PSP60\ L = 2.02\ \mathrm{kN/m}$

$EA = 200.000.000 * 0.026 = 5.200.000\ kN/m$

$EI = 200.000.000 * 0.00179 = 358.000\ kN\mathrm{m}^2/m$

$Moment\ arm = 0.3\ m$

Reliability factor $= 1.34$

$F_{b,max} = 2.400\ kN$

3.3.2 Bearing piles (9)

The current calculation does not provide the f_{ck} of the concrete. The f_{ck} will be assumed by determination the current calculation value. The obtain the force the bearing piles are 3 in one row. See the picture below for the schematisation of the bearing and tension piles of the current quay structure.

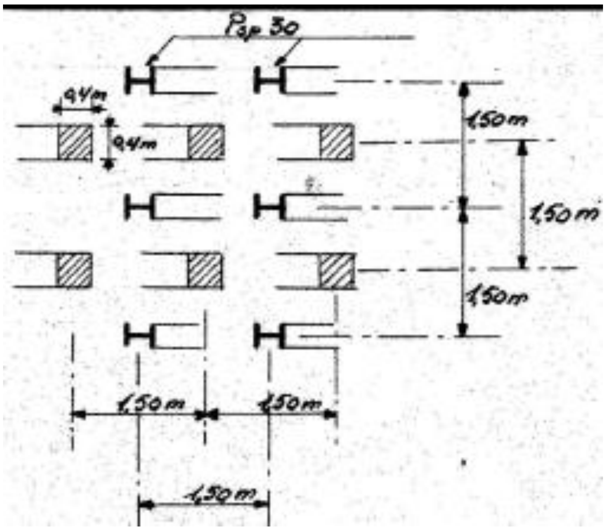


Figure 21 schematisation of the bearing and tension piles

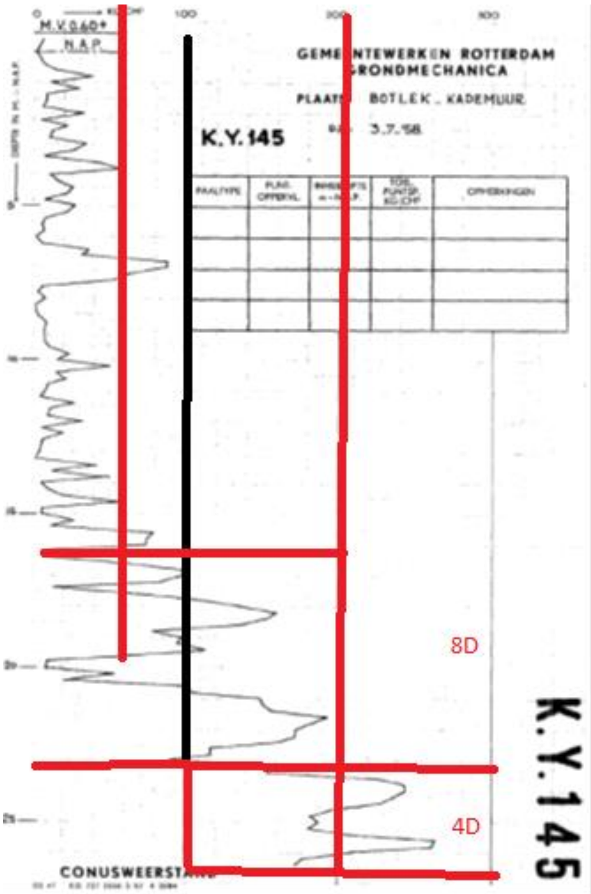
The distance ($Lspacing$) between 2 rows of bearing pile is determined in the original calculation, $Lspacing = 1.5\ m$. The distance between each pile in one row is 1.5 m. The calculated forces on the bearing piles has been checked and approved.

Parameters bearing piles

D= 0.4 m

A= 0.16 m

$$W_y = \frac{1}{6} * b * h^2 = \frac{1}{6} * 0.4 * 0.4^2 = 0.011\ m^3$$



The bearing capacity of the subsoil will be calculated by the method of Koppejan. The calculation will be as follows:

$$f_{b,max} = \frac{1}{2}\left[\frac{1}{2} * (p_1 + p_2) + p_3\right]$$

$$p_1 = 10\ MPa$$

$$p_2 = 20\ MPa$$

$$p_3 = 5\ MPa$$

$$f_{b,max} = \frac{1}{2}\left[\frac{1}{2} * (10 + 20) + 5\right] = 10\ MPa = 10.000\ kN/m^2$$

$$F_{b,max} = f_{b,max} * A = 10.000 * 0.16 = 1.600\ kN$$

$$F_{p,max} = 550\ kN$$

$550\ kN < 1.600\ kN$, so is suitable.

$$\text{Reliabilty factor} = \frac{1600}{550} = 2.9$$

C20/25 is assumed for the concrete of the bearing piles. The characteristics of the concrete according to the NEN-EN 1992-1-1 are presented in the table below.

Table 7 Material properties concrete C20/25

Class	f_{ck}	$f_{ck, cube}$	f_{cd}	f_{ctm}	f_{ctd}	E_{cm}
C20/25	20	25	13.3	2.21	1.03	30.000

The γ of concrete is assumed of 24 kN/m^2.

$f_{ck} = 20\ N/mm^2 = 20.000\ kN/m^2$

$E = 30.000\ N/mm^2 = 30.000\ .000\ \ kN/m^2$

The distance ($Lspacing$) between 2 rows of bearing pile is determined in the original calculation, $Lspacing = 1.5\ m$.

3.3.2.1 Summary parameters bearing piles

$\gamma = 24\ kN/m^2$

$E = 30.000.000\ \ kN/m^2$

$A = 0.4 * 0.4 = \ 0.16\ m^2$

$Lspacing = 1.5\ m$

$\text{Reliabilty factor} = 2.9$

3.3.3 Tension piles (8+10)

PSp300 is unknown what the parameters are, the PSp370 is the smallest PSp pile. These values will be used for the tension piles for the calculations. The obtain the force the bearing piles are 3 in one row. See the picture below for the schematisation of the bearing and tension piles of the current quay structure.

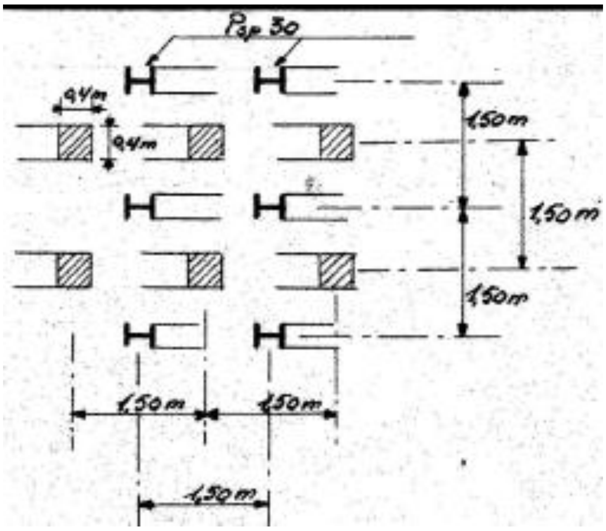


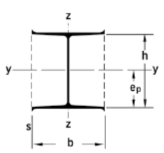
Figure 22 schematisation of the bearing and tension piles

EA is needed for the computer calculations. This is the stiffness parameter of the wall.

According to the original calculation the EA is determined.

$$E = 210.000\ N/mm^2 \ = 210\ \ kN/mm^2 = \ 210.000.000\ kN/m^2$$

The determination of the A is done by the sheet pile parameters of the site of sheet pile supplier Peiner Träger GmbH. The sheet pile parameters are pictured in figure X.



Sheet piles	Section PSp[9]	Section modulus	Section modulus	Weight	Width	Height	Perimeter developed	outline	Coating area one side incl. interlocks	Cross-sectional area steel	Cross-sectional area outline	Moment of inertia	Moment of inertia	Radius of gyration	Radius of gyration	Edge distance
		W_y	W_z		b	h						I_y	I_z	i_y	i_z	e_p
		cm³	cm³	kg/m	mm	mm	cm	cm	m²/m	cm²	cm²	cm⁴	cm⁴	cm	cm	cm
	370	2285	800	122	380	370	225	158	0.39	155	1422	42274	15192	16.5	9.9	18.5
	400	2523	801	127	380	400	231	164	0.39	162	1536	50469	15210	17.6	9.7	20.0
	500	3278	801	136	380	500	251	184	0.39	173	1916	81947	15211	21.8	9.4	25.0
	600	5274	1169	188	460	600	301	220	0.47	239	2774	158226	26886	25.7	10.6	30.0
	700	6353	1169	199	460	700	321	240	0.47	253	3234	222343	26889	29.6	10.3	35.0
$W_{ye_p} = \frac{I_y}{e_p}$	800	7980	1216	221	460	800	339	260	0.47	281	3694	319198	27973	33.7	10.0	40.0
	900	9221	1216	232	460	900	359	280	0.47	295	4154	414958	27975	37.5	9.7	45.0

Figure 23 Parameters PSP sheet piles

The EA stiffness parameter can be calculated with the sheet pile parameters. The EA is as the following calculation.

$E = \ 210.000.000 \ kN/m^2$

$A = 155 \ cm^2 = 0.0155 \ m^2$

$W_y = 2.285 \ cm^3$

$I_y = W_y * e_p = 2285 * 18.15 = 41.473 \ cm^4 = 0.00041473 \ \ m^4$

$EA = 210.000.000 * 0.0155 \ = \ 3.255.000 \ kN/pile$

The distance ($Lspacing$) between 2 rows of tension pile is determined in the original calculation.

$Lspacing = 1.5 \ m$

3.3.3.1 Check current calculation

The calculated forces on the tension piles has been checked and approved. The reliability factor will be calculated by the tension forces appearanced and performed.

According to the Cur 166, the $t_{max} = 0.012 * q_c$. According to the original calculation the $A = 155 \ cm^2 = 1.55 \ m^2$.

$t_{max} = 0.012 * q_c = 0.012 * 16 = 0.192 \ MPa = 192 \ KPa = 192 \ kN/m^2$

The tension will only be adopted by the bottom sand layer. The tension piles are 4 metre piled in the bottom sand layer. The total tension resistance is

$F_{a,max} = t_{max} * A * L = 192 * 1.55 * 4 = 1190 \ kN$

The tension forces which are calculated are 200 kN, so the safety factor is:

$$\text{Reliabilty factor} = \frac{1190}{200} = 5.95$$

The tension piles will be modulated as node-to-node anchor with embedded beam row end piece in Plaxis. The Skin resistances of these beams will be 192 kN/m as maximum at the bottom and 0 kN/m as maximum at the top.

3.3.3.2 Summary parameters tension piles and the anchorage

$EA = \ 3.255.000\ kN/pile$

$Lspacing = 1.5\ m$

$\text{fy} = 235\ N/mm^2$

$T_{top,max} = 0\ kN/m$

$T_{bottom,max} = 192\ kN/m$

$\text{Reliabilty factor} = 5.95$

3.4 Ground

Assumption according to the original calculation the following ground parameters can be divided. The presented parameters are characteristics.

Table 8 Characteristics ground parameters original calculation

Top layer NAP	Bottom layer NAP	Type ground	φ	δ	γunsat	γsat
4.5	-7	Sand	30	20	18	21
-7	-11	Clay	25	11.5		16
-11	-14	Sand	30	20		21
-14	-19	Clay	25	0		16
-19	-35	Sand	32.5	20		21

The stiffness parameters are assumed according to the cone penetration test near to the location of the quay wall structure. The results of the cone penetration test are as shown in the figure X below.

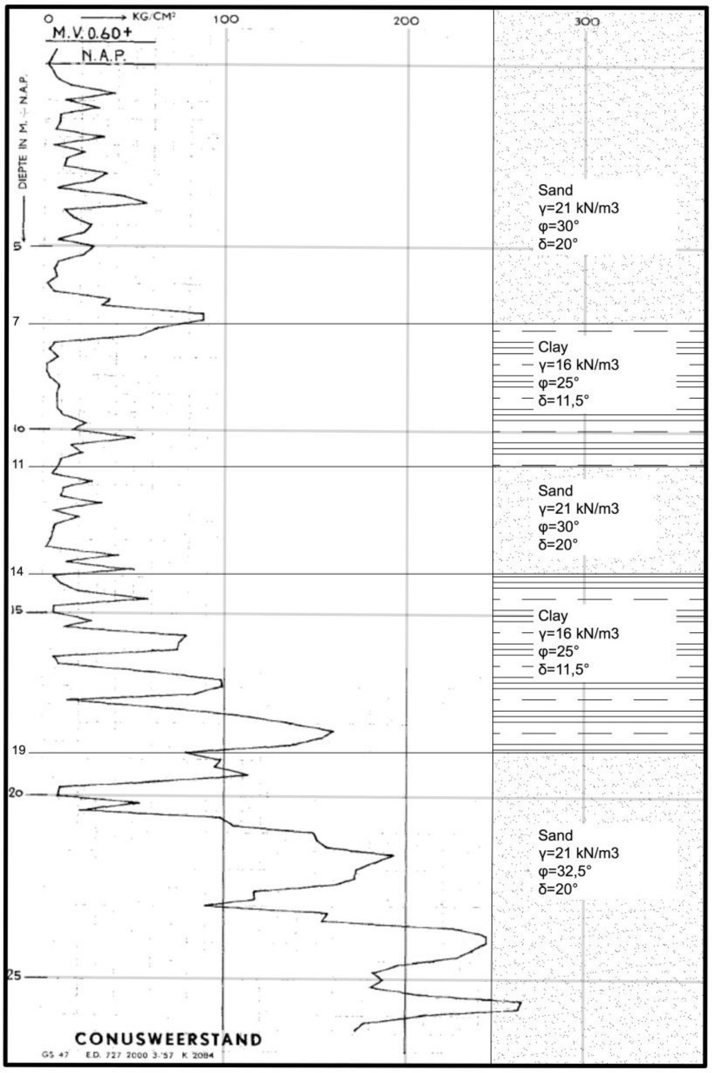


Figure 24 cone penetration test results

The soil stiffness can be determined by experienced formulas by the NEN-6740. The formulas of the parameters of the soil stiffness are:

$$E_{50,ref} = 3 * q_c * \sqrt{\frac{P_{ref}}{\sigma'_v}}$$

$$Sand: E_{50,ref} = E_{oed,ref}$$
$$E_{ur,ref} = 3 * E_{50,ref}$$

$$Clay: E_{50,ref} = 2 * E_{oed,ref}$$
$$E_{ur,ref} = 5 * E_{50,ref}$$

The determination of the σ'_v of the layers is in the middle of the layers. The value of the calculation of the parameters are as the table below.

Layer	layer thickness	γsat	σ'v (kpa) middle layer	qc (Mpa)	qc (kpa)
Sand	5	21	27.5	5	5000
Clay	4	16	67	3	3000
Sand	3	21	95.5	6	6000
Clay	5	16	127	6	6000
Sand	16	21	230	16	16000

The other unknown parameters are assumed by the NEN-6740 and the experience of the supervisor and experts:

$$m\ (power)\ sand\ =\ 0.5$$

$$m\ (power)\ \ clay =\ 0.9$$

$$R_{inter}\ sand = 0.8$$

$$R_{inter}\ clay = 0.66$$

$$C_{Ref}\ sand = 0\ kN/m^2$$

$$C_{Ref}\ \ clay = 5\ \ kN/m^2$$

$$\psi\ sand \approx \varphi\ - 30$$

$$\psi\ clay \approx 0°$$

$$\upsilon\ sand = 0.2$$

$$\upsilon\ clay = 0.2$$

3.4.1 Summary ground parameters

The table below shows the summary of the characteristics ground parameters which will be used for the calculations. The characteristics ground parameters are determined by the NEN-6740. The parameters are checked and approved by the supervisor and experts.

Table 9 characteristics ground parameters for Plaxis model

Soil parameter			Sand	Clay	Sand	Clay	Sand
Symbol	Description	Unit					
γsat	Saturared weight density of the soil	kN/m3	21	16	21	16	21
γunsat	Unsaturared weight density of the soil	kN/m3	18				
E50 ref	Secant stiffness modulus at a 50% deviatoric stress	kPa	28600	6000	18400	6000	31700
Eoed ref	Oedometric stiffness modulus	kPa	28600	3000	18400	3000	31700
Eur ref	Unloading reloading stiffness modulus	kPa	85800	15000	55200	15000	95100
ψ	Dilatancy angle	°	30	0	30	0	32.5
φ	Internal angle of friction	°	30	25	30	25	32.5
Rinter	Interface	-	0.8	0.66	0.8	0.66	0.8
C'ref	Effective cohesion in drained conditions	kPa	0	10	0	10	0
m	amount of stress dependency (power)	-	0.5	1	0.5	1	0.5

3.5 Water levels

The water levels of Botlek area are determined with data of the Public works of Rotterdam. The Geulhaven is the nearest measure point to the Botlek. The water levels for the calculations are:

Mean Water Level (MWL): +0.00 metre NAP

Mean High Water (MHW): +2.68 metre NAP

Mean Low water (MLW): -1.57 metre NAP

The calculation of the quay wall structure assumed the Mean Water Level of +0.00 metre NAP and the ground water level is determined on -0.00 metre NAP according to the original calculation.

3.6 Geometry overview

The starting points of the calculation of the water level and ground conditions are shown in figure X.

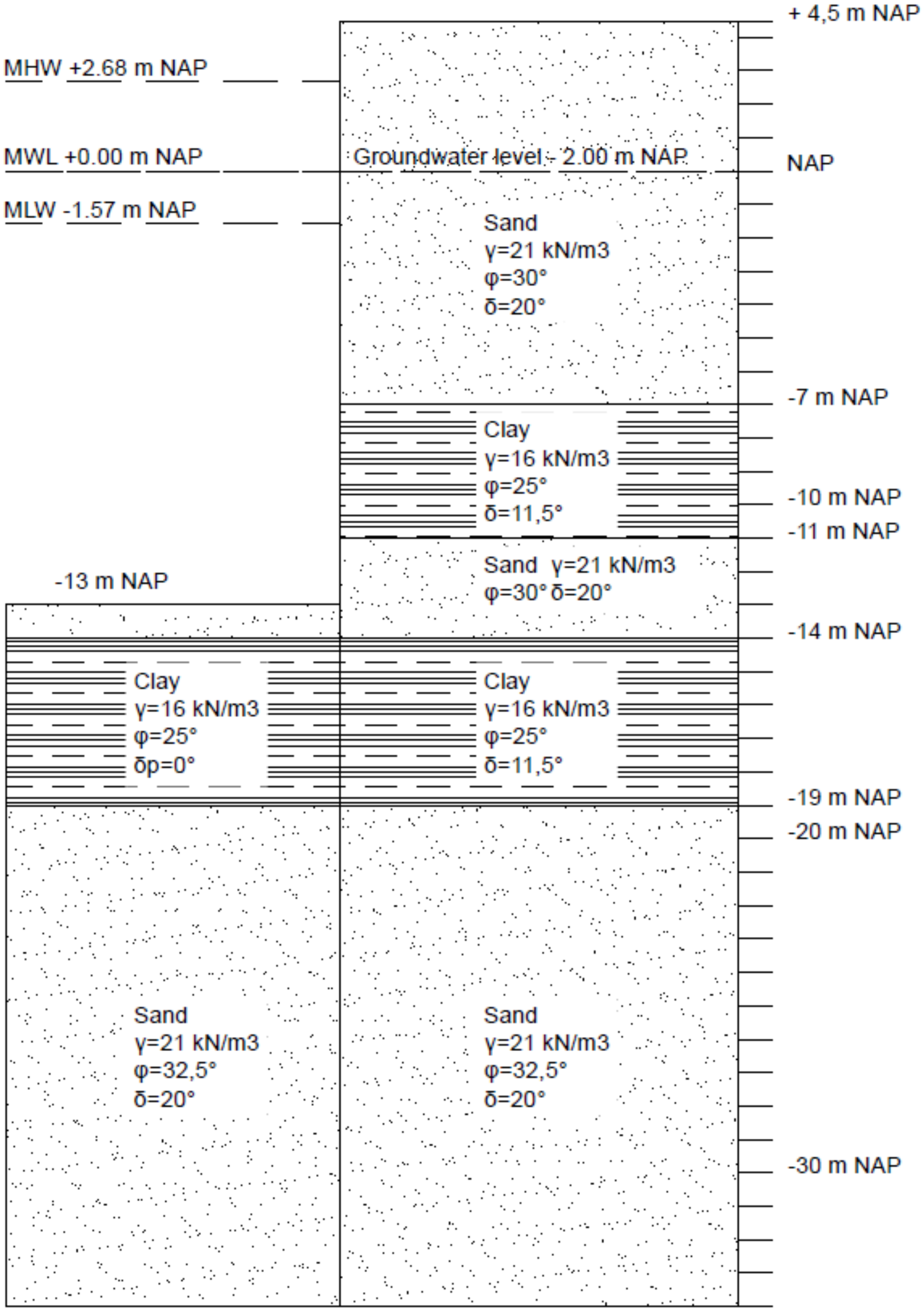


Figure 25 water levels and main ground parameters

3.7 Loads

Several loads are present on the quay wall structure of EBS. The different loads are described below.

3.7.1 Waves

The forces of the waves and the current are not part of the research.

3.7.2 Bollard forces

The bollard force of the original calculation is used for the research. The bollard forces are 80 tons per 40 metres. The bollard force per metre is 2 tons=19.62 kN.

The force for the bollard in the calculation will be $F_b = 19.62\ \mathrm{kN/metre}$.

3.7.3 Surface load

The surface load will be the same as the original calculations. This will be done because of the current reduction of the loads. The loads which are used in the original calculation are more than the current loads. The following surface loads will be used for the calculation.

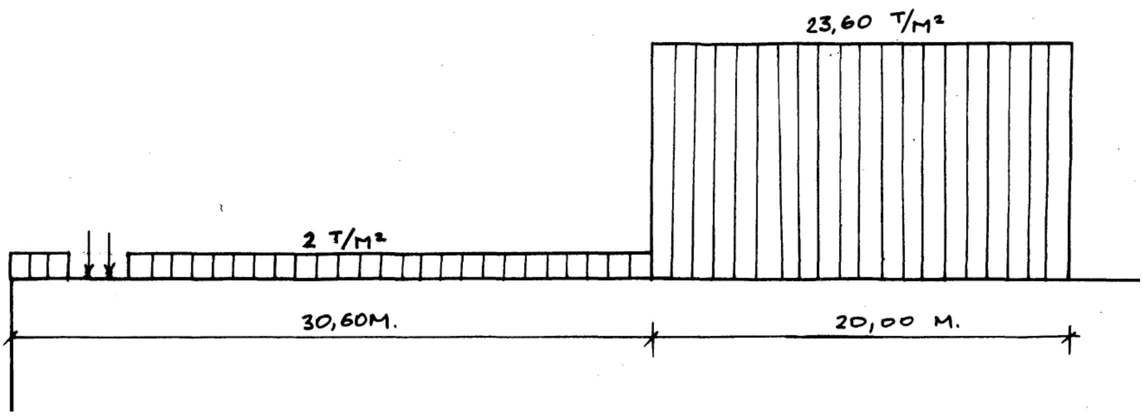


Figure 26 surface loads

The surface load of $\mathrm{q1} = 2\ \mathrm{tons/metre} = 19.62\ \mathrm{kN}/m^2$ and the surface load of $\mathrm{q2} = 23.6\ tons/\mathrm{metre} = 231.5\ kN/m^2$

3.7.4 Crane loads

The total load of the crane on the front of the crane is according to the original calculation 500 tons. The Figure 27 and Figure 28 explains the location and the layout of the front wheels. The load of the front wheels is spread out by 4*4= wheels.

So the wheel load of the crane is 500/16=30 tons/ m, 30 tons=294.3 kN/m.

The used crane force for the calculation is $F_c = 294.3\ \mathrm{kN/m}$.

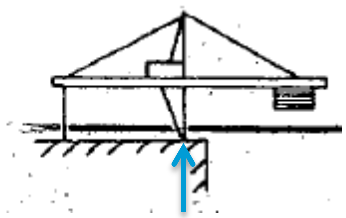


Figure 27 Side view crane front quay with annotation of the front wheels

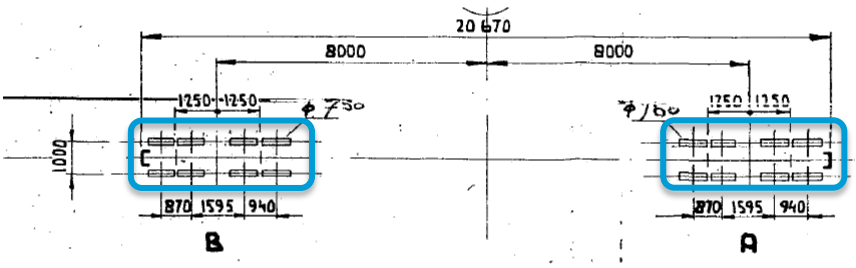


Figure 28 view from above of the front wheels

4 Finite elements method

The computer program Plaxis is used for this master thesis. Plaxis is chosen in consultation with the supervisor and advisor. Finite elements method (Plaxis) is a more complex calculation method compared to spring supported beam model (Dsheet). The calculation model types are various and the possibilities are more. The main advantage of Plaxis compared to Dsheet is the interaction between all the structure elements and the soil. Dheet only convers the interaction between the front wall and the soil. The main disadvantage of Plaxis is the higher required level of knowledge of soil behaviour and the big amount of parameter which are needed.

Plaxis is chosen instead of D-sheet because of:

- The geometry of the structure, difficult to module the angle of the front wall into D-sheet;
- Easy modulation of the relieving platform;
- The difficulty of the modulation of the possible solutions into D-sheet;
- The advanced geotechnical calculation of Plaxis;

4.1 Soil model

The behaviour of the soil can be modelled in several ways. The relevant models for this study are:

• Linear elastic model. This model represents Hooke's law of isotropic linear elasticity. The model involves two elastic parameters; Young's modulus (E) and Poisson's ratio (v). This model is primarily used for stiff structures in the soil

• Mohr-Coulomb (M-C) model. This is used as a first approximation of soil behaviour in general. The model involves the parameters: Young's modulus (E), Poisson's ratio (v), cohesion (C),internal friction angle (φ) and dilatancy angle (ψ).

• Hardening Soil (HS) model. This is an elasto-plastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. It also involves compression hardening to simulate irreversible compaction of the soil under primary compression. This model can be used to simulate behaviour of sands and gravel as well as softer soils. In addition to the M-C strength parameters, the HS model invloves three reference stiffness parameters for a given reference stress, i.e. the 50% secant stiffness (E50), the oedometric stiffness (Eoed) and the unloading-reloading elasticity modulus (Eur).

The HS model includes the stiffness and the deformations of the soil. The Hardening soil model will be used for this research. The HS model is the most suitable for relieving structure because of the interface and the influence of the structure to the soil.

4.2 Calculation method

In the calculation program the user can define the types of calculations to be performed and the types of loadings or construction stages to be activated during the calculations. The calculation program considers only deformation analysis and distinguishes between Plastic calculations, φ -c reduction, Consolidation analysis, and Dynamic calculations. The first two options are relevant for this study and explained more:

- A Plastic analysis is used to make elastic-plastic deformations analyses. The analysis does not take include the effect of decay of excess pore pressures in time. Small deformation theory is used and the stiffness matrix is based on the original undeformed geometry. This type of calculations is suitable for most practical geotechnical applications.
- φ -C reduction is a safety analysis. Basically, the soil strength parameters (φ and C) of the soil are reduced stepwise. The method should be used when it is desired to calculate a global safety factor and soil mechanical failure is the only relevant mechanism. When using φ -C reduction in combination with advanced soil models (for example HS model) these models will actually behave as a Mohr-Coulomb model, because stress-dependent stiffness behaviour.

The calculation method φ -C reduction is used to determine the safety factor of this model and the effect of the safety factor of the deepening and the solutions.

4.3 Model parameters for Plaxis

The parameters which are needed for the Finite elements method calculation are summed in the table below. All the parameters of the model are characteristic. The highlighted parameters are dominant for this research, the dominant parameters are chosen in cooperation with the supervisors.

Table 10 Parameters Plaxis

Parameters Plaxis		
Symbol	Description	Unit
Soil parameter		
γsat	Saturared weight density of the soil	kN/m3
γunsat	Unsaturared weight density of the soil	kN/m3
E50 ref	Secant stiffness modulus at a 50% deviatoric stress	kPa
Eoed ref	Oedometric stiffness modulus	kPa
Eur ref	Unloading reloading stiffness modulus	kPa
ψ	Dilatancy angle	°
φ	Internal angle of friction	°
Rinter	Interface	-
C'ref	Effective cohesion in drained conditions	kPa
m	amount of stress dependency (power)	-
Plate parameter		
EA	Axial stiffness	kN/m
EI	Flexural rigidity	kNm2/m
w	specific weight	kN/m
W	Resisting moment	m3/m
fy	Yiels stress steel	kPa
Anchor parameter		
EA	Axial stiffness	kN/m
fy	Yield stress steel	kPa
Lspacing	speсing between anchor	m
Embedded pile row parameter		
E	Modules of elasticity of soil	kN/m2
γ	Weight density	kN/m3
A	Surface area	m2
I	Moment of inertia of the cross-sectional area	m4
Lspacing	speсing between anchor	m
Ttopmax	skin resistance	kN/m
Tbottom max	skin resistance	kN/m
Fmax	Base resistance	kN
Loads parameter		
q	Surcharge load	kN/m2
Fb	Bollard force	kN/m
Fh	Hawser force	kN/m

4.4 Reliability class

This quay walls structure will be calculated as reliability class RC2/CC2. The structure is for barges and vessel with a retaining height > 5.0 m, but not in flood defence/LNG-plants or nuclear plants. For those reasons the reliability class is RC2/CC2.

4.5 Design approach

The original calculation uses the SLS as design approach. The serviceability limit state approach every force as the value so every force will be calculated and multiplied with 1.0.

Design value force = $F_d = y_f * F_{rep}$

Design value ground = $X_d = X_{rep}/y_m$

Design value of geometrical data= $A_d = A_{norm} \pm \Delta A$

Division of the loads for the design approach will be:

- Permanent:
  - Dead weight structural elements
  - Soil pressure
  - (Ground) water pressure
- Variable:
  - Pressure differences due to water level differences
  - Hawser forces
  - Fender forces
  - Terrain/traffic load
  - Crane loads
- Exceptional:
  - Ship collision

4.6 Design steps

This plaxis research will be done by the design steps of the CUR 166.

1. Determine the characteristics parameters (see Chapter 4);
2. Establish load combinations, safety and reduction factors (see section 5.5);
3. Asses the dimension of the primary elements, embedding length and anchor force with a hand calculation (already done in the original calculation, not done for this report)
4. Use the result in step 3 as input in PLAXIS and perform a calculation;
5. Check the moment of the primary elements with the results obtained in step 4
6. Check the shear and normal forces the primary elements with the results obtained in step 4
7. Check the anchor strength with the results of step 4;
8. Check the deformations;
9. Overall checks (Kranz stability, overall stability, bearing capacity of the primary elements, failure of one anchor, piping and local buckling).

4.7 Load combination

The loads of the structure are modulated as figure X.

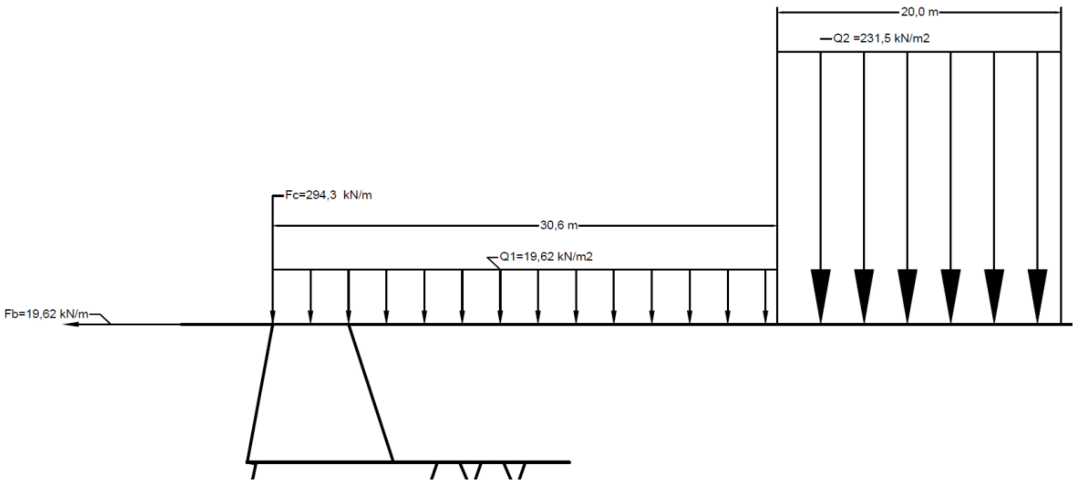


Figure 29 modulation of the loads

This research consists of 7 load combinations. The dominant load combination is determined after the modulation of the model. See the table X below for the activated load of the load combinations.

Table 11 load combinations

load combination	Groundwater level	Water level	Q1	Q2	Fb	Fc
0	0.00 metre NAP	0.00 metre NAP				
1	0.00 metre NAP	0.00 metre NAP	x		x	x
2	0.00 metre NAP	0.00 metre NAP	x	x	x	x
3	0.00 metre NAP	-1.57 metre NAP	x		x	x
4	0.00 metre NAP	-1.57 metre NAP	x	x	x	x
5	0.00 metre NAP	2.68 metre NAP	x		x	x
6	0.00 metre NAP	2.68 metre NAP	x	x	x	x

The load combinations are connected to models of the plaxis calculation. The load combinations are calculated before the deepening and after de the deepening. The table below describes the load combinations translation to the plaxis calculation.

Table 12 plaxis models connected to load combinations

Plaxis model	load combination	Before deepening	After deepening
Load combination 0 before deepening	0	x	
Load combination 0 after deepening	0		x
Load combination 1 before deepening	1	x	
Load combination 1 after deepening	1		x
Load combination 2 before deepening	2	x	
Load combination 2 after deepening	2		x
Load combination 3 before deepening	3	x	
Load combination 3 after deepening	3		x
Load combination 4 before deepening	4	x	
Load combination 4 after deepening	4		x
Load combination 5 before deepening	5	x	
Load combination 5 after deepening	5		x
Load combination 6 before deepening	6	x	
Load combination 6 after deepening	6		x

4.8 Sensitivity analyses

The parameters of the basics of design are all characteristic. The design value can be determined by the design approached and the formula for the design value.

The characteristic parameter can be assumed as the reference value for the design values and the upper and lower limits of the model parameters. The lower and higher limit parameters show the uncertainty of the plaxis model. The influence of these lower and higher limits of the model will be checked and compared. The current design philosophy in CUR 211, CUR 166 and Eurocode is characterized by the use of characteristic values of the parameters. The characteristic value of a parameter implies there is a 5% probability that the value is higher (solicitation) or lower (resistance).

The lower and upper values are influenced by:

- Importance (α_i);
- Uncertainty (V_x);
- The probability of exceedance is accounted for by applying a target reliability index (β).

The time independence solicitation and resistance values were derived by the following equation:

$$x_{max} = x_s = y_s * x_{ref}$$

$$x_{min} = x_r = y_r * x_{ref}$$

$$y_s = \frac{S_k}{S^*} = \frac{1 + \alpha_i V_x \beta}{1 + k_s V_x}$$

$$y_r = \frac{R_k}{R^*} = \frac{1 + k_s V_x}{1 + \alpha_i V_x \beta}$$

$k_s = 1.64$ for a 5% reliability

$V_x = dependance\ of\ the\ parameter\ type$

$\beta = 3.8\ for\ RC2$

The distribution, uncertainty and importance of model parameters are shown in the table X below.

Table 13 parameters of the sensitivity analyses

Notation	type	xref	ks	Vx	beta	ai	yr	ys	xmax	xmin	Annotation
q1	solicitation	20	1,64	0,2	3,8	0,7	**0,867**	**1,154**	**17**	**23**	kN/m2
q2	solicitation	230	1,64	0,2	3,8	0,7	**0,867**	**1,154**	**199**	**265**	kN/m2
φ sand	resistance	30	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**27**	**34**	°
φ clay	resistance	25	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**22**	**28**	°
φ sand lower	resistance	32,5	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**29**	**36**	°

4.9 Implementation in Plaxis

The calculated and assumed parameters need to be implemented into the plaxis model. The paragraphs of this chapter describe the implementation of the determined parameters into the plaxis model.

4.9.1 Modulation choices

The large surface load q2 is modulated on top of a plate. This plate is add to the model because of the prevention of the local failure of the soil under the surface load.

The superstructure is connected as a hinge to the combi-wall by a cast iron saddle. This saddle supports on the end of the PSP 60 L piles, so it is eccentric supported. The eccentric support reduces the bending moments of the sheet pile wall. The eccentricity of the superstructure to the wall is 0.5*height of the PSP 60 L. The extra moment of the superstructure will be generated by the moment arm of 0.5*600=300 mm. The modulation of the saddle is shown in figure X.

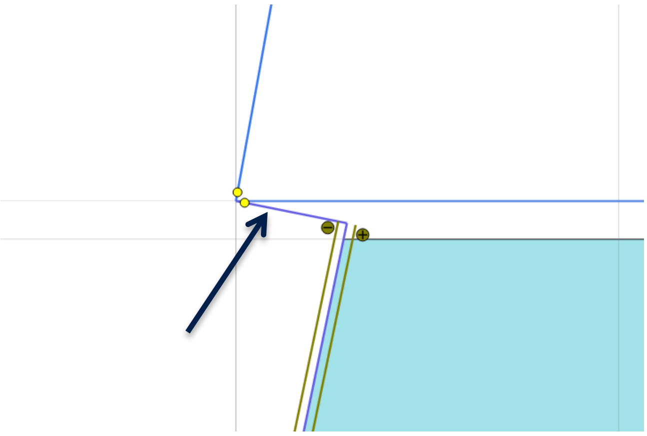


Figure 30 saddle modulation plaxis

The modulation of the subsoil is done with a fissure between the subsoil and the superstructure. The interface of the soil to the superstructure is eliminated. The side effects of bearing of the subsoil, the friction of the superstructure to the subsoil and the sticking of the subsoil to the superstructure are eliminated by the modulation of the fissure. The illustration of the fissure of the subsoil to the superstructure is shown in figure X.

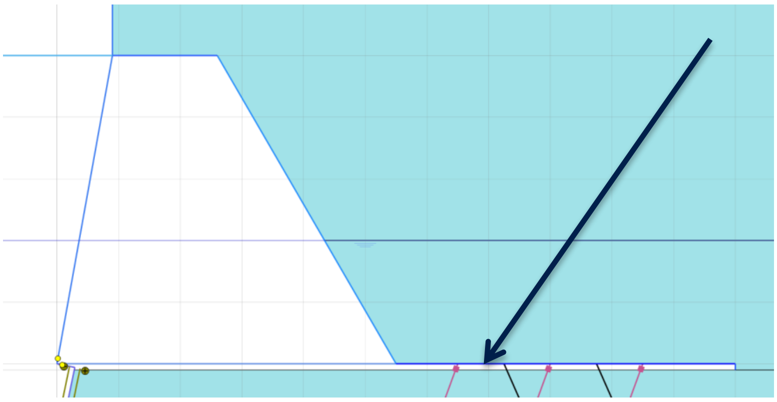
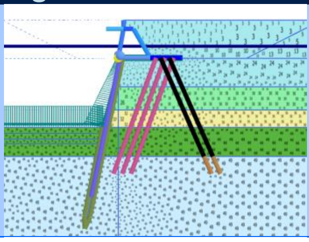
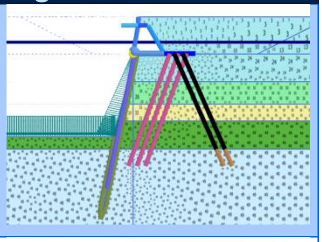
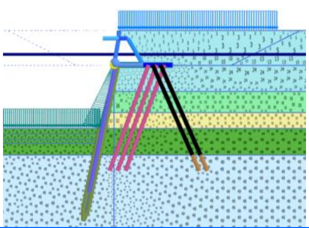
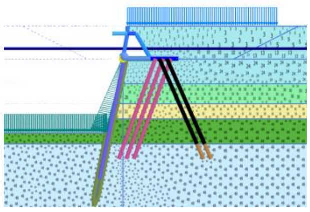
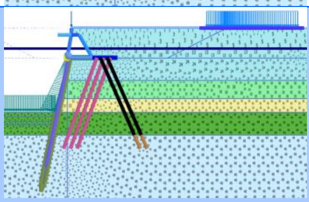
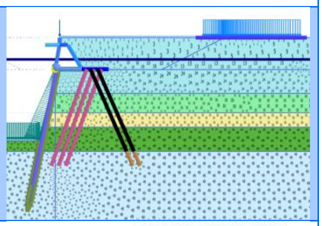
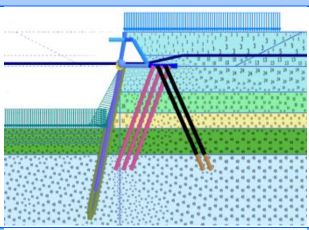
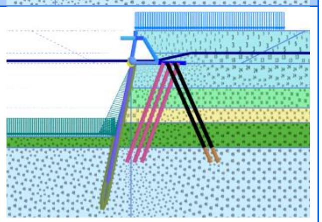
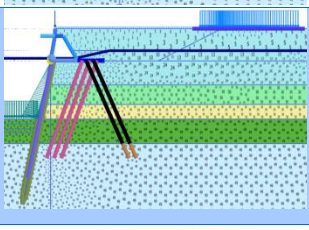
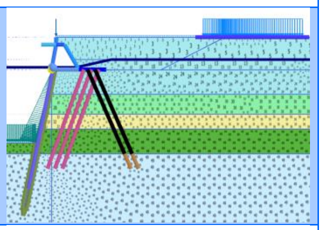
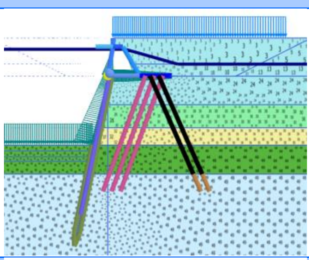
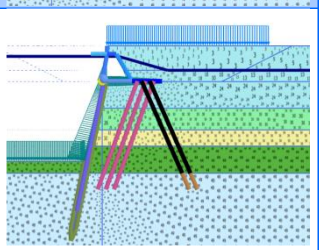
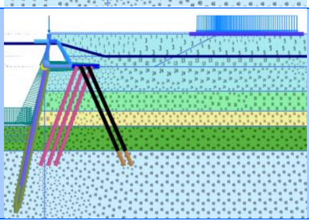
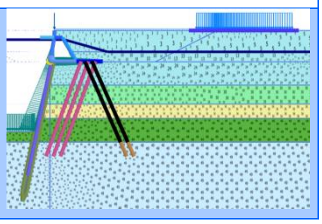


Figure 31 fissure modulation plaxis

4.9.2 Plaxis models

The implementation into plaxis is done in different models of plaxis. The load combinations before and after deepening are modulated into plaxis. The figures of the plaxis models which are used are presented in Table X.

Table 14 plaxis models figures

Plaxis model	Figure	Plaxis model	Figure
Load combination 0 before deepening		Load combination 0 after deepening	
Load combination 1 before deepening		Load combination 1 after deepening	
Load combination 2 before deepening		Load combination 2 after deepening	
Load combination 3 before deepening		Load combination 3 after deepening	
Load combination 4 before deepening		Load combination 4 after deepening	
Load combination 5 before deepening		Load combination 5 after deepening	
Load combination 6 before deepening		Load combination 6 after deepening	

4.9.3 Modulation materials

The construction elements of the quay wall will be modulated as the figures below. The values are determined in chapter 4.

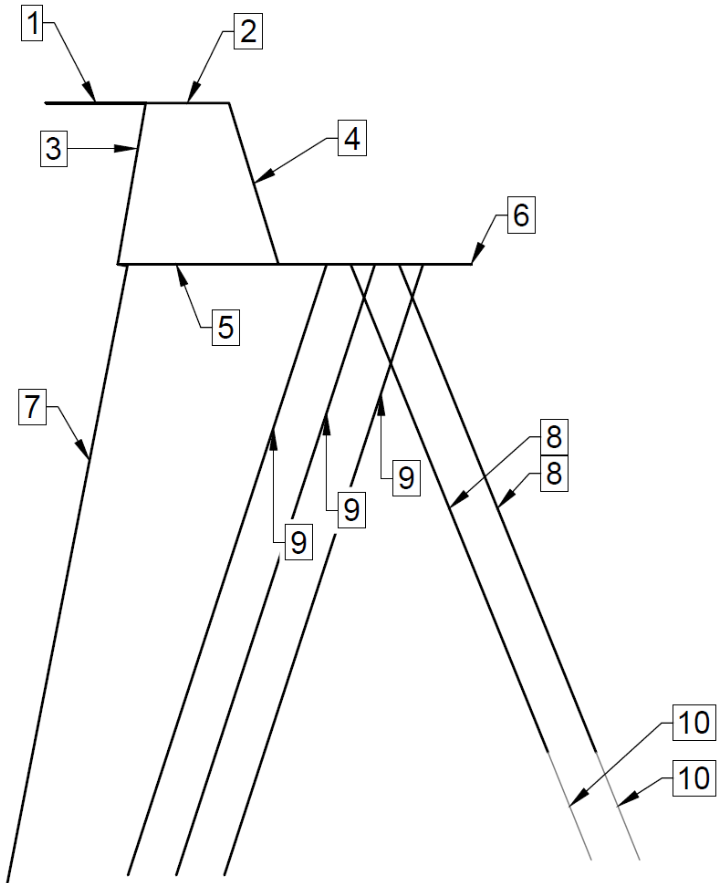


Figure 32 numbers of the structure elements

4.9.4 Input parameters

The input parameters of the FEM model are determined and explained in chapter 4. The determined parameters of chapter 4 are used for the FEM model. The print screens of the input screens of plaxis are added to this master thesis in appendix X.

The summary of the parameters in the plaxis model are shown in the tables below.

Table 15 summary plate parameters

Plate parameters					
nr	Name	EA (kN/m)	EI (kNm/m)	v (-)	w (kN/m/m)
1	horizontal 1	10,712,500	106,294	0.2	8.40
2	horizontal 2	11,900,000	158,666	0.2	9.60
3	Vertical 1	20,825,000	850,354	0.2	16.80
4	Vertical 2	8,925,000	66,937	0.2	7.20
5	Counterfort full	18,704,885	42,670,000	0.2	15.10
6	Counterfort reduced	26,860,488	10,550,000	0.2	9.66
7	Psp60L	5,200,000	358,000	0	2.02

Table 16 summary node-to-node parameters

Node-to-node parameters			
nr	Name	EA (kN/m)	Lspacing (m)
8	Tension pile	3,255,000	1.5

Table 17 summary embedded beam row parameters

Embedded beam row parameters						
nr	Name	E (kN/m2)	A (m2)	Ispacing (m)	Fmax (kN)	Tmax (kN/m2)
9	Bearing pile	30,000,000	0.16	1.5	1,600	192
10	Tension pile end piece	210,000,000	0.0155	1.5	0	192

5 Structural assessment existing quay wall

This chapter will show the most important results of the plaxis calculation. The effect of the deepening to the structure element and the safety factor are explained and presented.

The results and graphs of the plaxis calculation are added to the report in appendix B. The most important tables, graphs and results are shown in the sub-chapters below.

5.1 Uncertainty of the model

The following points are seen as uncertainty of the plaxis model:

- Assumed representive high parameters instead of real parameter in the plaxis model;
- Theoretical calculations are conservative compared to the real situation;
- Extremely high surface load;
- The inaccuracy margin of ± 30% of Plaxis deformations (cur 221);
- Large influence of the clay layers, more than D-sheet and blum calculations.

5.2 Results before deepening

The different load combinations before deepening are presented in the figures and tables below.

5.2.1 Front wall

The comparison of the bending moment, shear force and normal force of the front wall in the different load combinations are shown in figure X, Figure x and figure X.

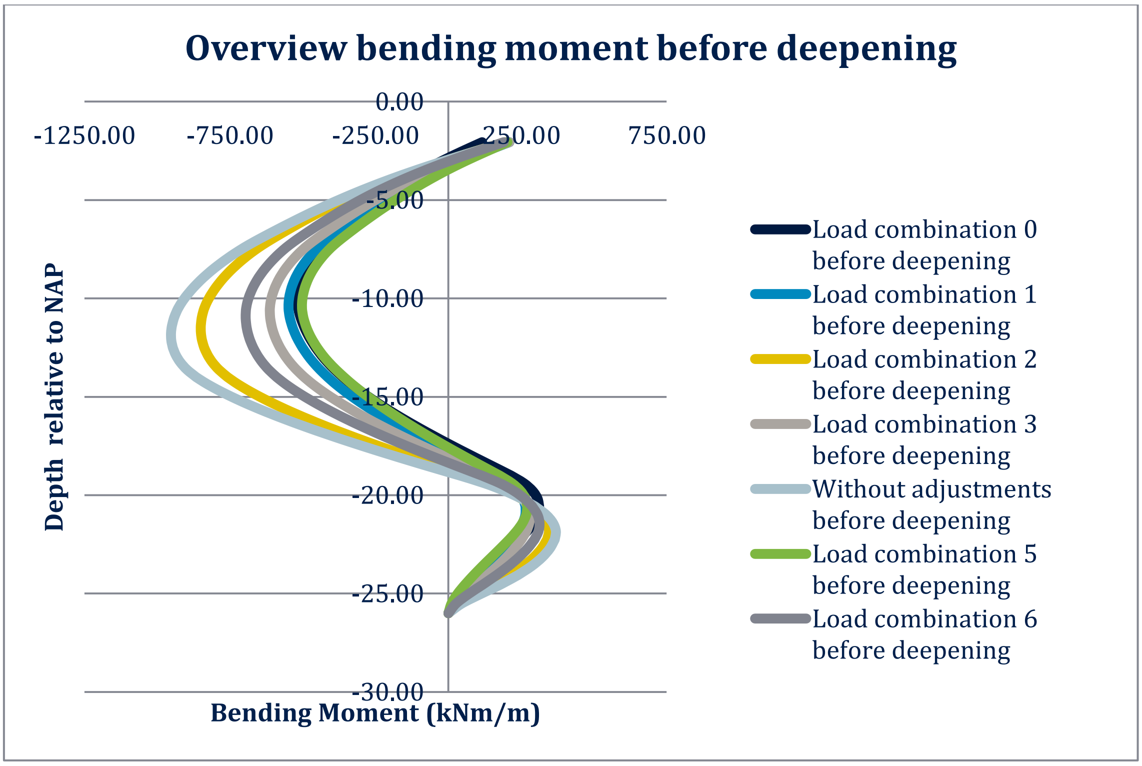


Figure 33 Bending moment of the front wall in different load combinations

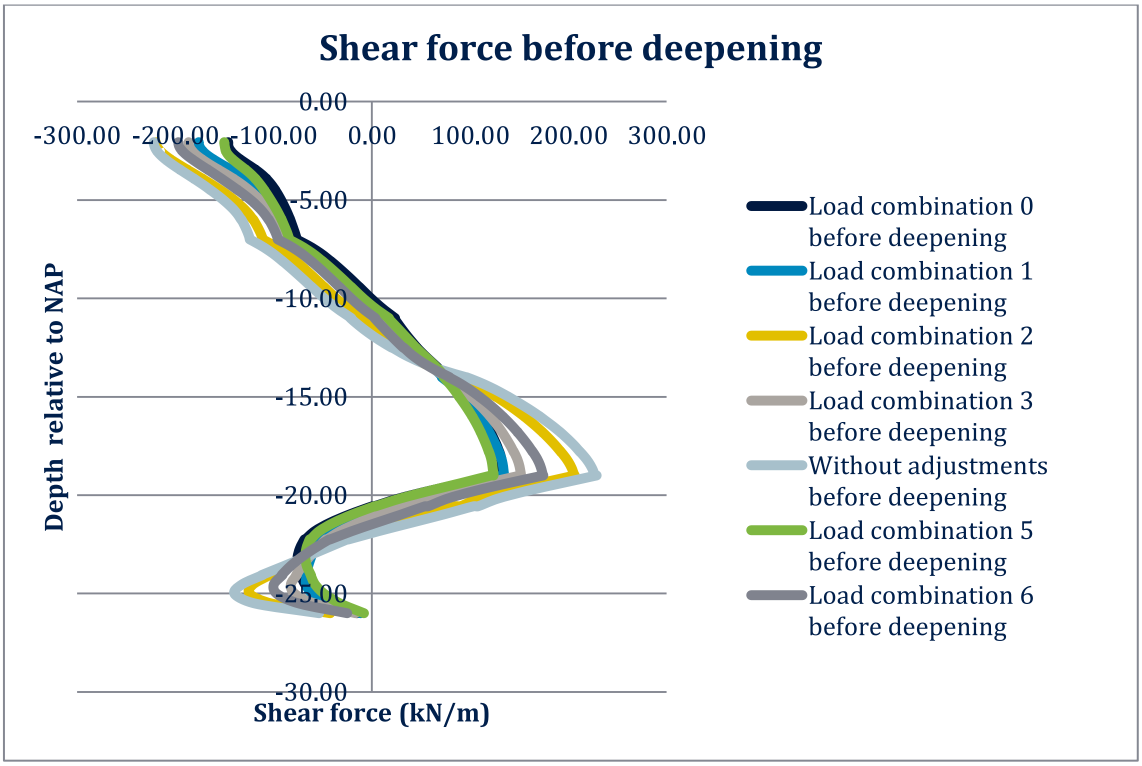


Figure 34 shear forces of the front wall in different load combinations

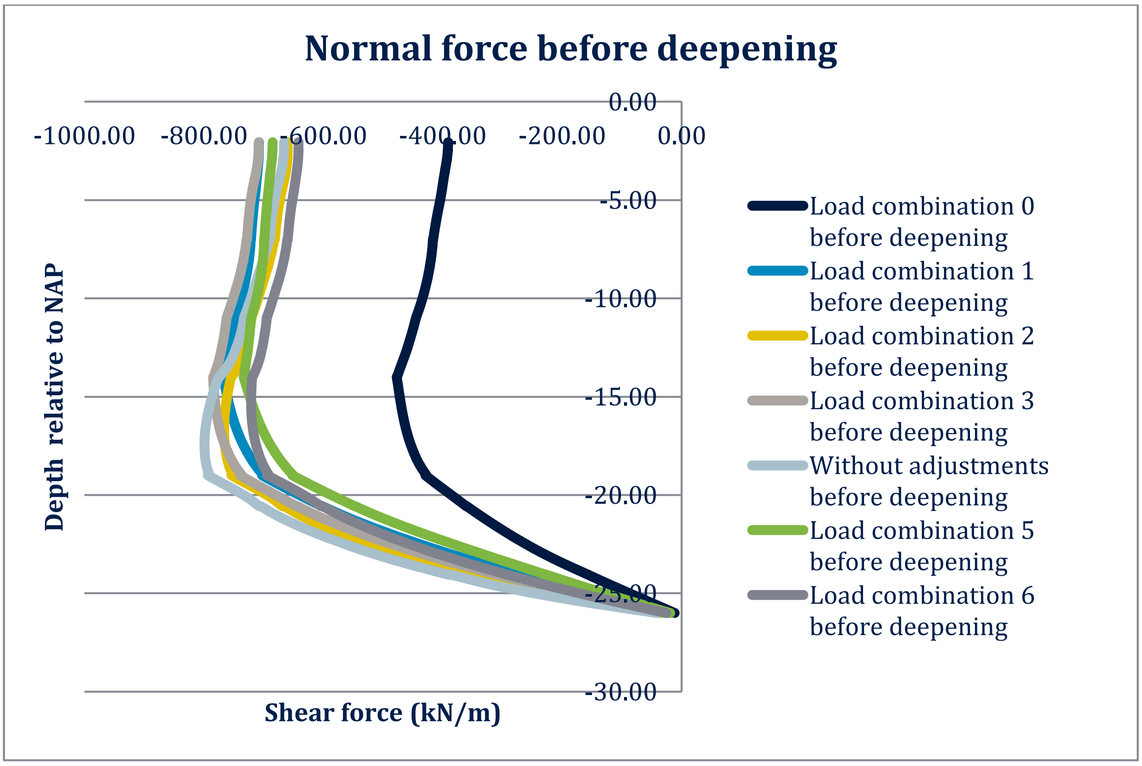


Figure 35 Normal forces of the front wall in different load combinations

5.2.2 Bearing and tension piles

The comparison of the bearing and tension piles of different load combinations before deepening are shown in figure X.

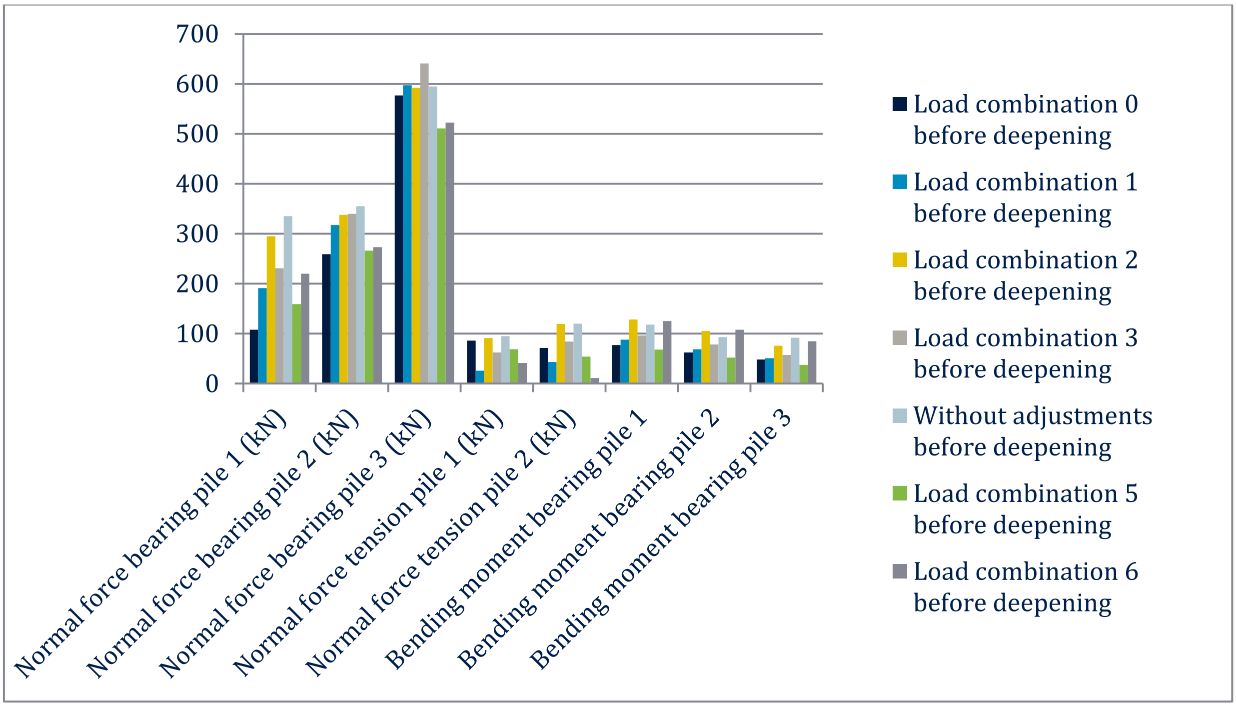


Figure 36 comparison of the bearing and tension pile in different load combinations

5.2.3 Hydraulic

The CUR166 describes a formula of the check for piping. The piping and heave failure mechanism can be check by the following formula by lane.

$$L_1 + L_2 \geq \Delta H * C_l * y_{piping}$$

In the formula are:

$L_1 = length\ of\ the\ groundwater\ level\ to\ depth\ of\ the\ intermiete\ piles$

$L_2 = \ \ length\ of\ the\ portbed\ level\ to\ depth\ of the\ intermiete\ piles$

$\Delta H = difference\ between\ LLW\ and\ grounwater\ level$

$C_l = seepage\ path\ length\ factor\ see\ table\ X$

$y_{piping} = partial\ safety\ factor\ of\ piping\ see\ table\ X$

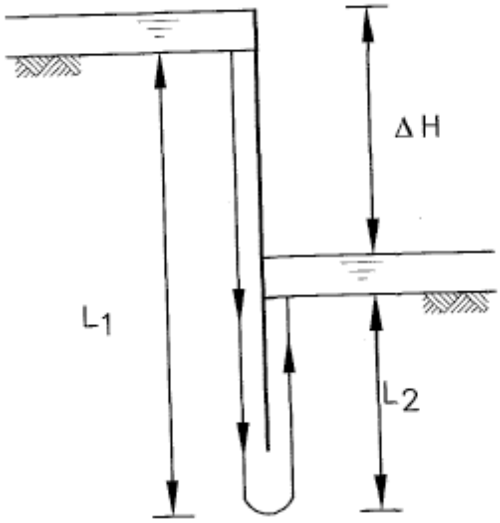


Figure 37 schematization of the formula of Lane

Table 18 CL factor (CUR166)

Grondsoort	Aanbevolen waarden voor C_L
zeer fijn zand of silt	8,5
fijn zand	7,0
middel grof zand	6,0
grof zand	5,0
fijn grind	4,0
middel grof grind	3,5
grof grind	3,0
stenen	2,5

Table 19 y_{piping} (CUR166)

veiligheidsklasse	β	γ_{piping}
I	2,5	1,0
II	3,4	1,5
III	4,2	2,0 *)

$L_1 + L_2 \geq \Delta H * C_l * y_{piping}$

$15.9 + 3.1 \geq 1.57 * 6 * 1.5$

$19.0 \geq 14.13, so\ no\ piping\ for\ this\ situation$

The ground geometry of this specific quay wall structure prevents the seepage. The clay layers are hardly pervious, so a continue seepage path length cannot occur.

5.2.4 Geotechnical

The geotechnical stability of the phases are checked and all the phase are before deepening above 1.25, so stable. The geotechnical charts are add to this report in appendix C. The geotechnical chart of load combination 0 before deepening is shown in figure X.

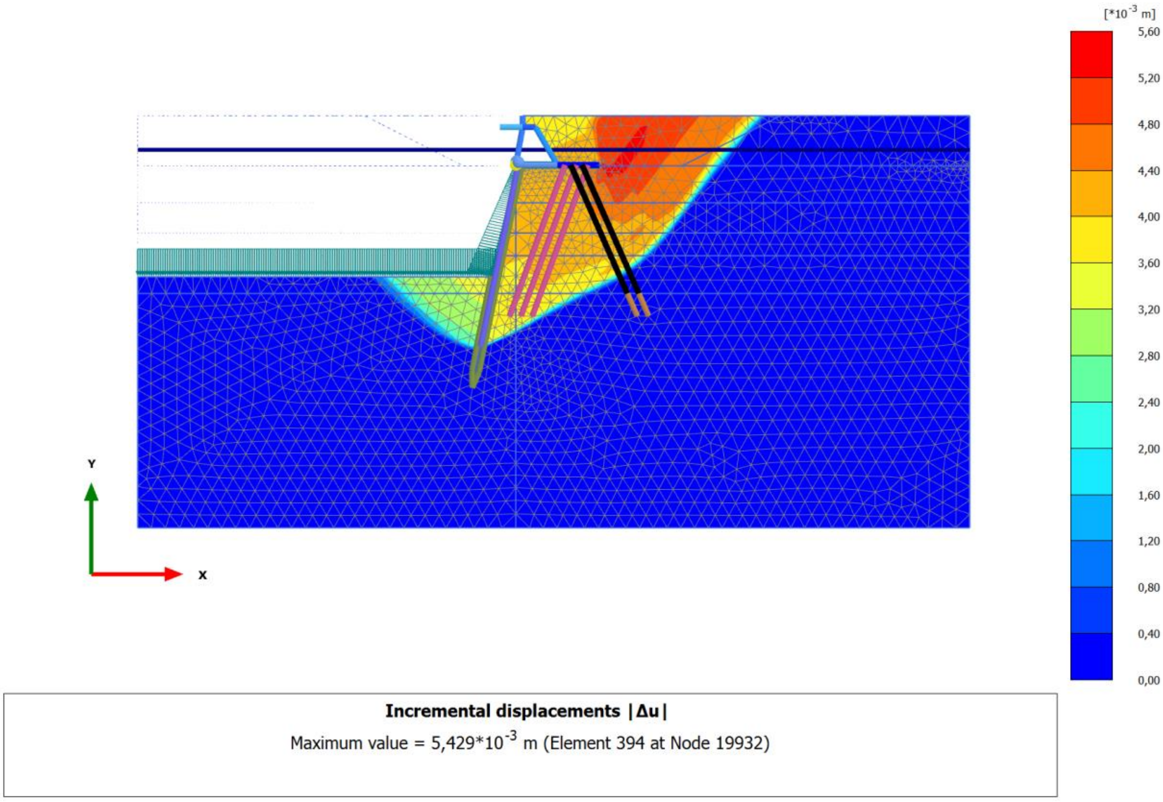


Figure 38 load combination 0 before deepening incremental displacements

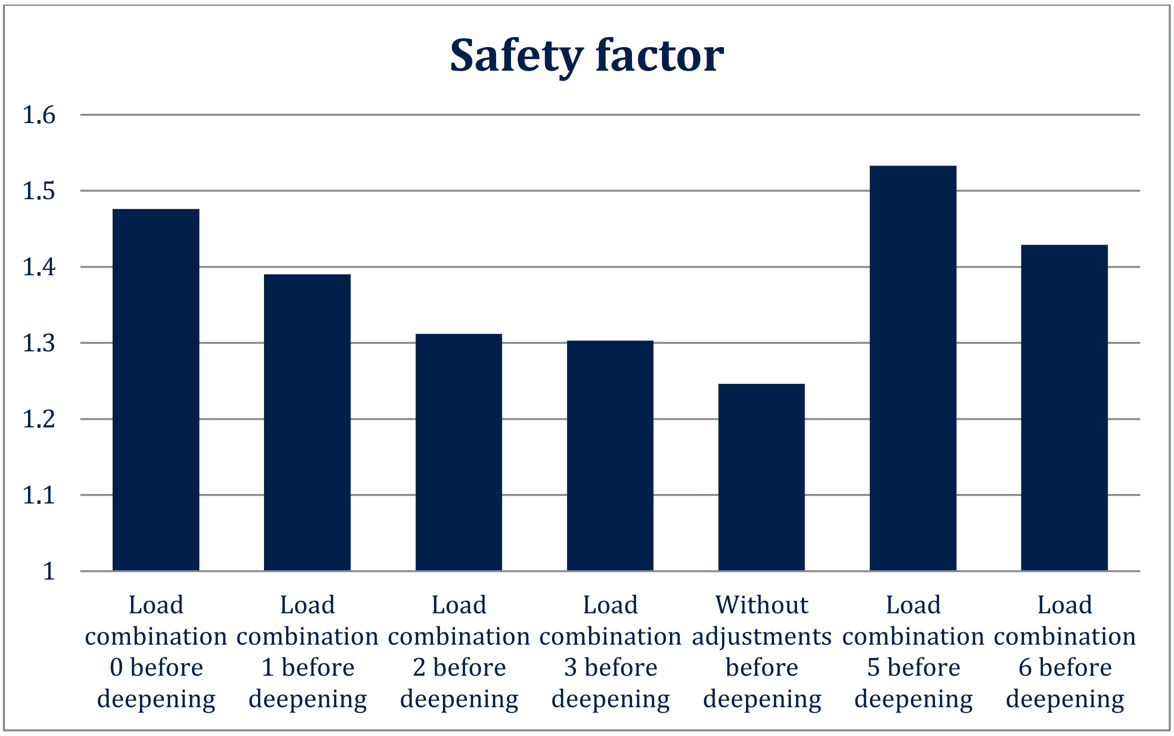
5.2.4.1 Local failure of ground between primary piles

The pilling depth of the intermediate piles does have influence on the possibility of piping. Besides of the pining issue the pilling depth is also important for the stability of the ground between the primary piles. That ground will disappear when the pilling depth is too short. The local failure can occur after de deepening.

5.2.5 Safety factor

The safety factor of the different load combination is the plaxis model is determined by the φ -C reduction. The results of the φ -C reduction safety factors are shown in Table X.

Table 20 safety factor before deepening



5.2.6 Conclusion

Load combination 4 is the dominant load combination. All the forces and moments are the biggest and the safety factor the lowest in load combination 4. For these reasons load combination 4 will used in the next phases. Load combination 4 will be used to determine the effect of the deepening and to determine the effects of the solutions according to this reference model. The reference value of the plaxis calculation of combination 0 and the dominant load combination 4 are shown in Table X.

Table 21 summary of the reference values

Phase	Load combination 0 before deepening	Load combination 4 before deepening
Safety factor	1.476	1.246
Bending moment front wall (kNm/m)	518	953
Shear force front wall (kN/m)	146	229
Normal force front wall (kN/m)	477	799
Normal force bearing pile 3 (kN)	577	595
Bending moment bearing pile 3 (kNm/m)	48	92
Normal force tension pile 1 (kN)	86	95
Deformations x top quay wall (m)	0	0.13337

5.3 After deepening results

The next phases of the master research consist of load combination 4. The fourth load combination consists of a different water level and the entire loads are present. The results and effects of the deepening are the less favourable, so load combination four will be used to show the effect of the solutions. The results of load combination 4 are shown in the paragraphs below.

5.3.1 Front wall

The comparison of the bending moment, shear force and normal force of the front wall are shown in figure X, Figure x and figure X.

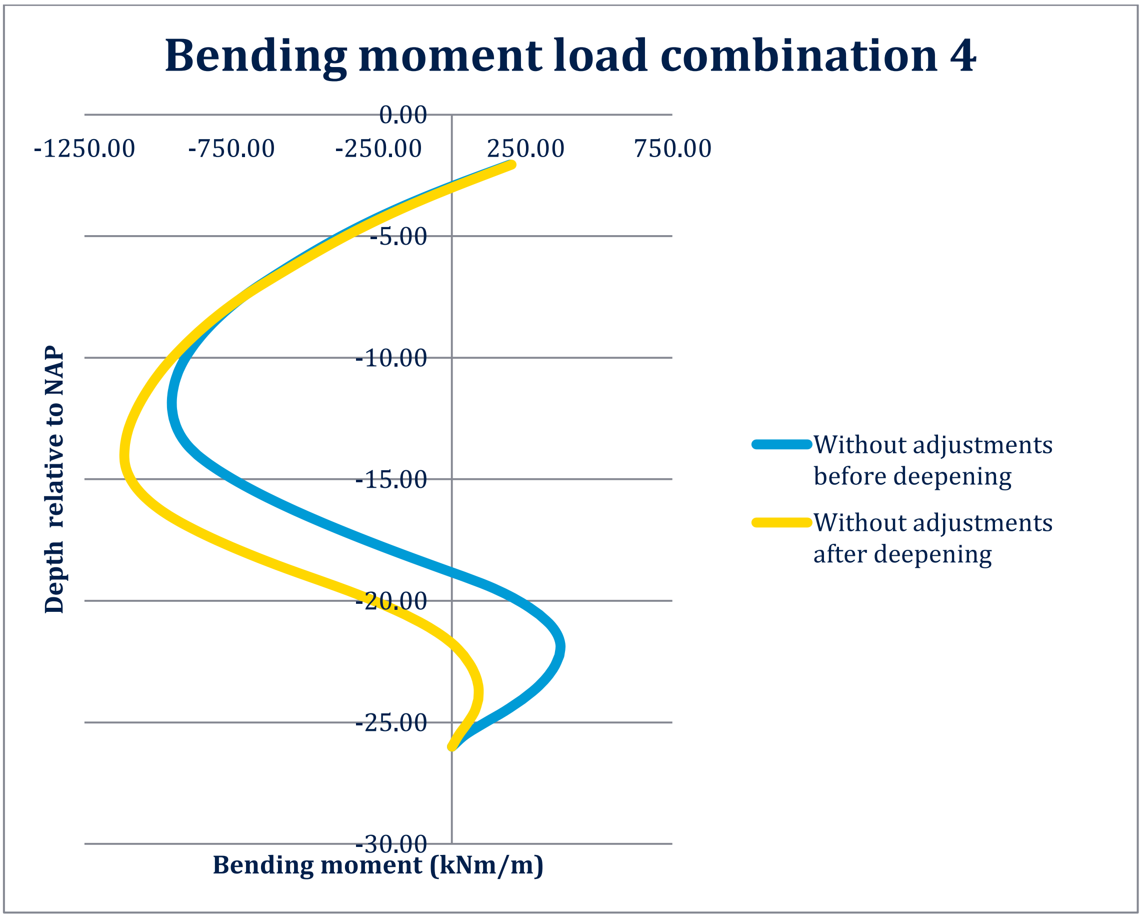


Figure 39 bending moment of the front wall before and after deepening compared to the zero situation

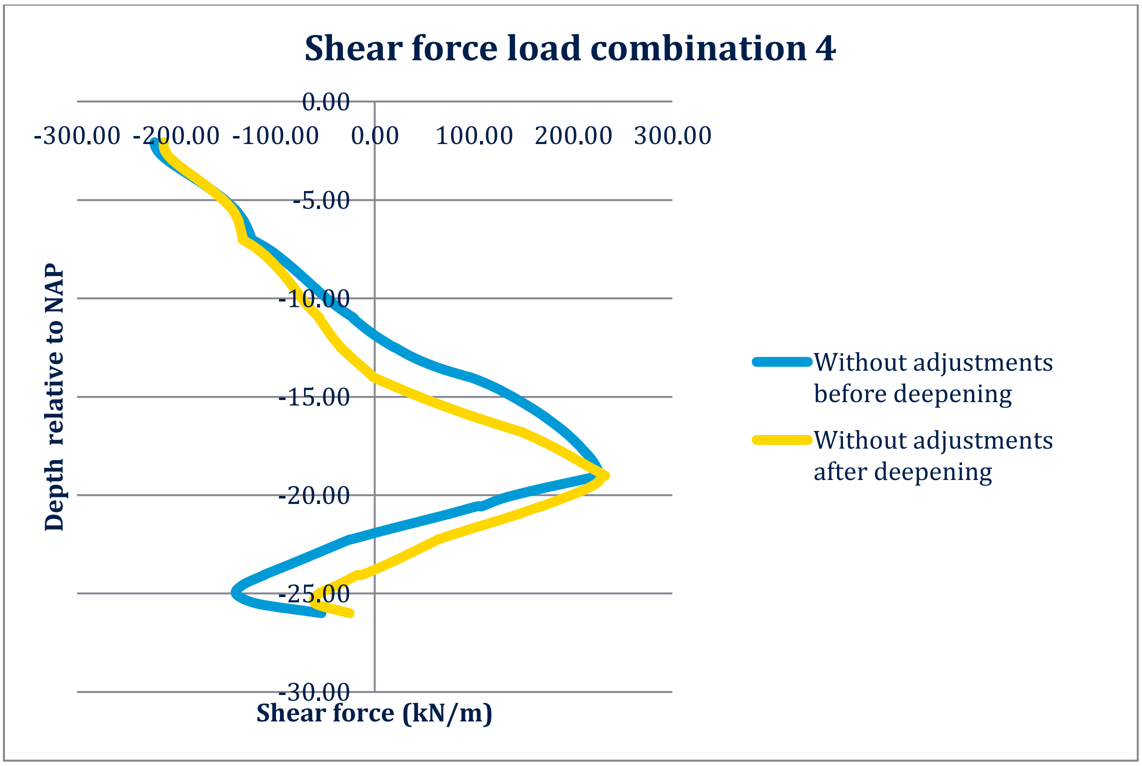


Figure 40 Shear force of the front wall before and after deepening compared to the zero situation

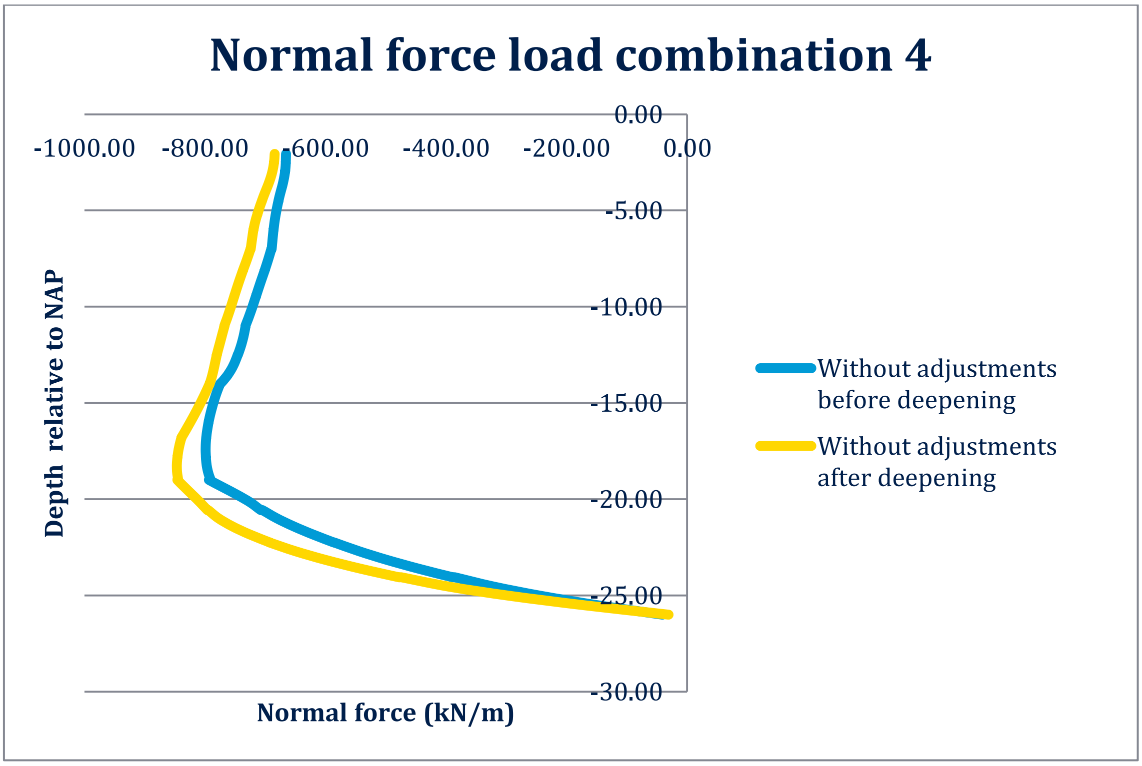


Figure 41 Normal force of the front wall before and after deepening compared to the zero situation

5.3.2 Bearing and tension piles

The comparison of the bearing and tension piles of load combination 4 before and after deepening is shown in figure X.

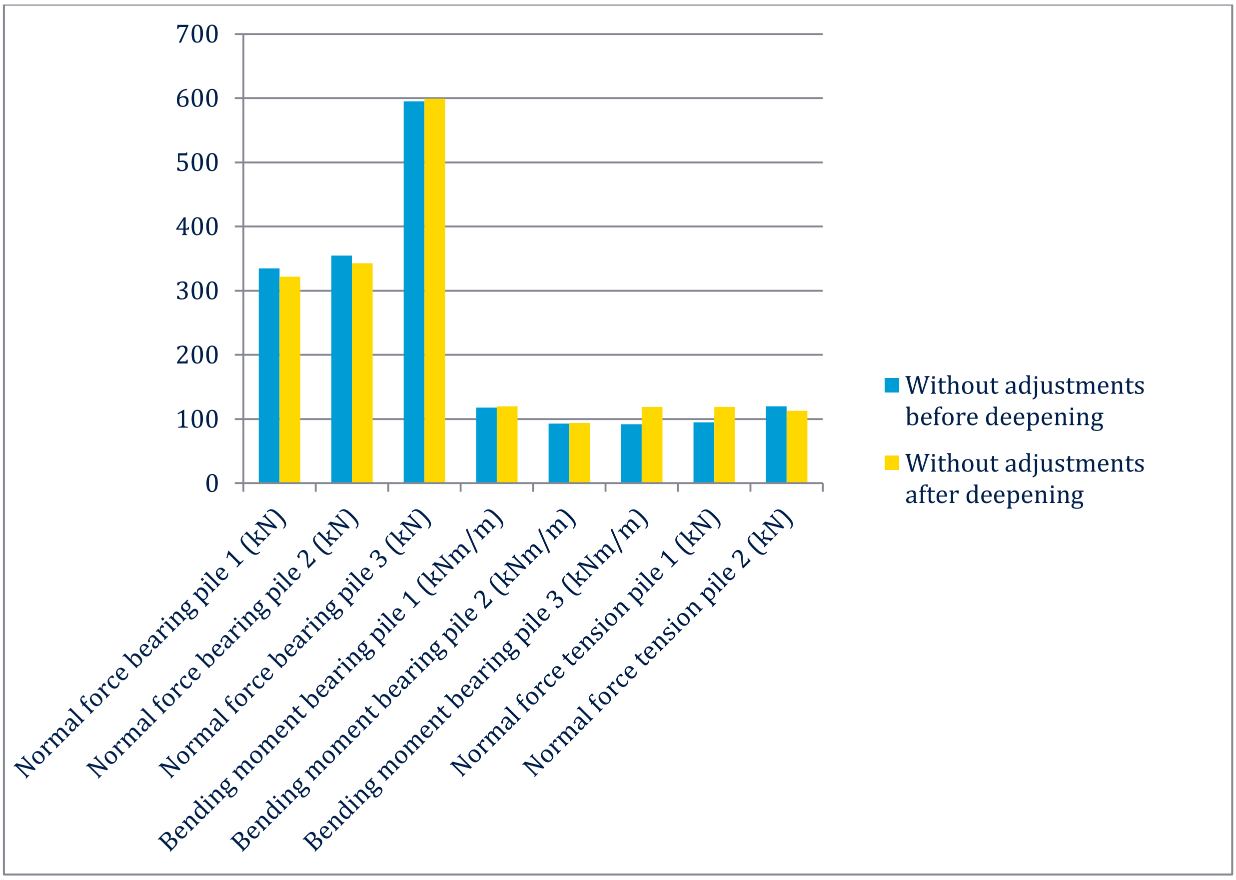


Figure 42 remaining values of the structure elements before and after deepening

5.3.3 Hydraulic

The hydraulic check is made according to the formula of lane mentioned in chapter 7.1.3.

$$L_1 + L_2 \geq \Delta H * C_l * y_{piping}$$

$$15.9 + 1.1 \geq 1.57 * 6 * 1.5$$

$$17.00 \geq 14.13\ , so\ no\ piping\ for\ this\ situation$$

The ground geometry of this specific quay wall structure also prevents the seepage. The clay layers are hardly pervious, so a continue seepage path length cannot occur.

The formula of the lane and the clay layers of the soil approve the piping prevention of the structure. Besides of the approval of the piping by the formula of Lane the piping and the local failure of the soil between the primary piles are critical. These failure mechanism are critical because of the influence of the propeller of the vessel and the unknown specific pilling depth of the intermediate piles. For those reasons the portbed level can become below the depth of the intermediate piles, so the soil behind the structure can erode.

5.3.4 Geotechnical

The geotechnical stability is checked by the φ -C reduction. The safety factor of the dominant load combination 4 after deepening is below 1.25, so the structure after deepening is not stable. The sliding surface of the total structure can be seen on the plaxis output. The geotechnical charts of the plaxis calculation of load calculation 4 before and after deepening are shown in figure X and Figure X.

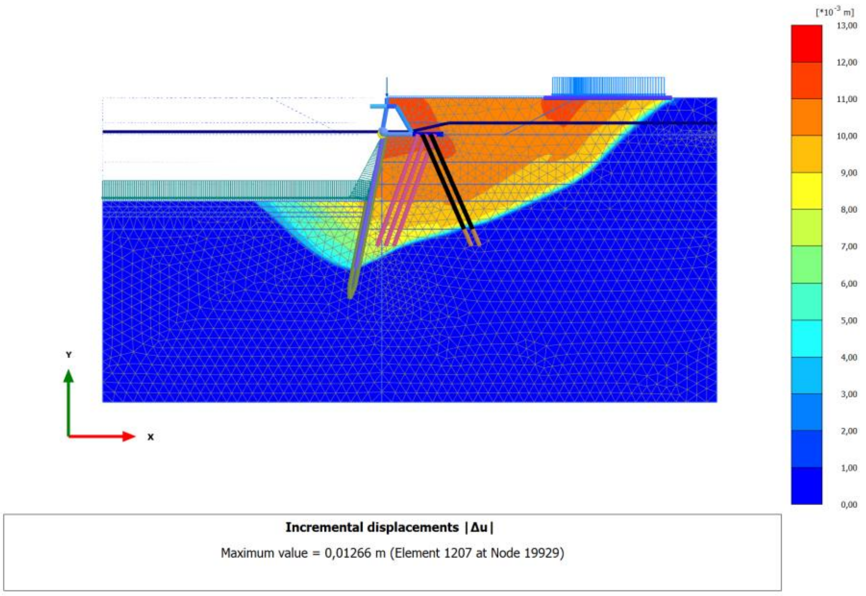


Figure 43 incremental displacements load combination 4 before deepening

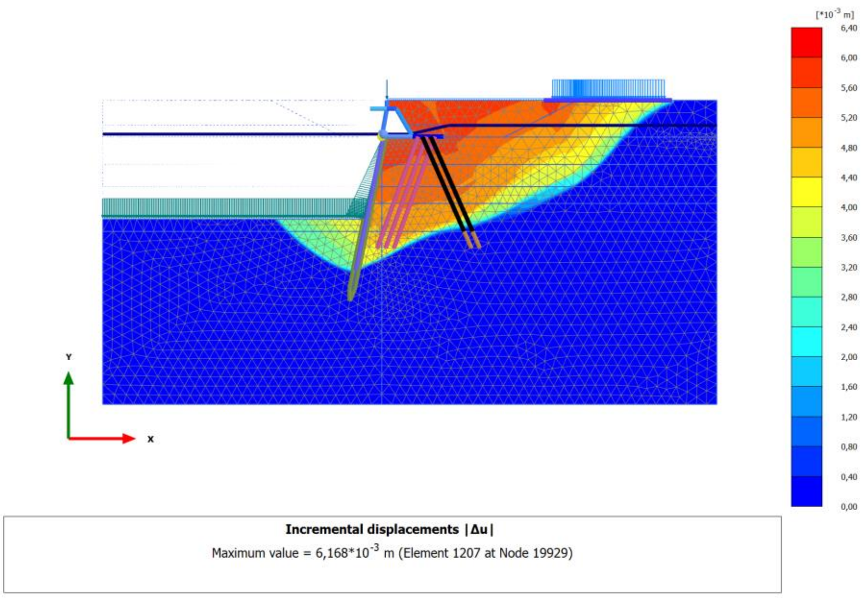


Figure 44 incremental displacements load combination 4 after deepening

5.3.5 Safety factor

The safety factor of the different load combination is the plaxis model is determined by the φ -C reduction. The results of the φ -C reduction safety factors are shown in Table X.

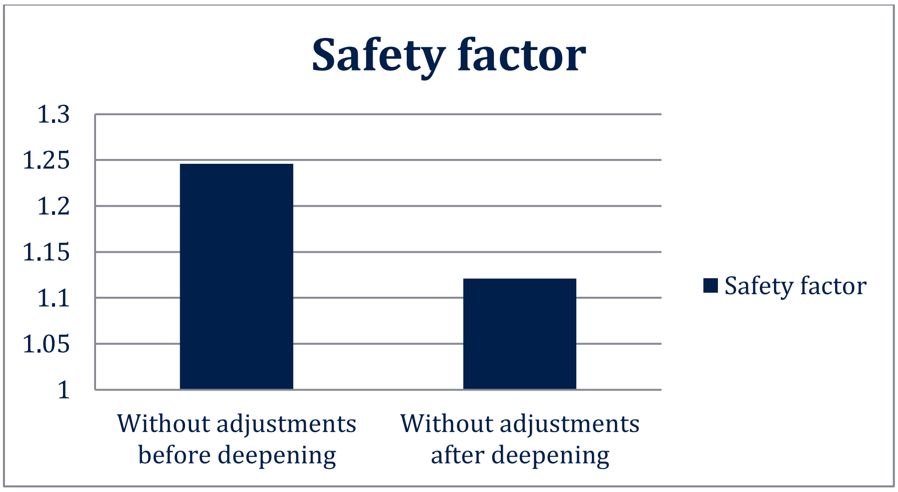


Figure 45 safety factor of the model before and after deepening

5.3.6 Summary

The summary of the deviation and the effect of the deepening are shown in the table below. The deviation is calculated of load combination 4 before and load combination 4 after deepening.

Table 22 summary of the values and the effect of the deepening to the structure elements

Check	Annotation	Load combination 4 before deepening	Load combination 4 after deepening	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.121	-10.03%
Structural				
Bending moment front wall	kNm/m	953	1,115	16.99%
Shear force front wall	kN/m	229	232	1.49%
Normal force front wall	kN/m	799	847	5.98%
Normal force bearing pile 3	kN	595	599	0.67%
Bending moment bearing pile 3	kNm/m	92	119	29.35%
Normal force tension pile 1	kN	95	119	25.26%
ULS				
Deformations x top quay wall	m	0.13	0.19	40.25%

The structure elements which deviate large are the critical structure elements. According to the results of the plaxis calculation the following structure elements are critical:

- Front wall : Bending moment;
- Tension pile 1: Normal force;
- Deformations;
- Pilling depth of the intermediate piles;
- Reduction of the passive pressure.

The following failure mechanisms according to the deepening are critical according to the critical structural members:

- Structural:
  - Failure of front wall;
  - Failure of tension piles;
- Geotechnical;
  - Insufficient passive resistance of front wall;
  - Local failure of geotechnical stability between the primary piles;
  - Failure of anchor/pile tension resistance.

5.4 Requirements deepening solutions

The deepening solutions upgrade the current quay wall structures. The upgrade solutions are modulated into plaxis to determine the effect of the upgrade. The results of the plaxis calculation need at least to be the same values as before the deepening to be feasible. The result of the plaxis calculation are ± 5% because of the unconsertancy of the model and the conservative approach of the model. The minimum requirements to be the most feasible solutions are:

- Safety factor ≥ 1.25 -5%;
- Bending moment front wall ≤ 935 +5% kNm;
- Normal force tension pile 1 ≤ 95 +5% kN;
- Prevention of eroding of the soil between the primary piles;
- Deformations x top quay wall ≤ 0.13 m.

5.5 Validation model

The plaxis model is checked and approved by Dirk-Jan Jasper Focks, Witteveen+Bos. The model is also compared to the deformation measurement and maximum parameters of the current quay wall structure. The deformations are measured for the first time in 1964. The current measurements are compared to the zero situation of 1964. The maximum deformation is 22 millimetres subsidence in the z direction. The deformations perpendicular (x-y) to the quay wall is 76 millimetres. The construction is 76 millimetres deformed to the front and 22 millimetres sagged.

The plaxis model deforms 130 millimetres to the front and 0 millimetre sagged. The order of magnitude of the real measurement and the plaxis model are equal. The differences of the measurement and the plaxis model are because of:

- Assumed representive high parameters instead of real parameter in the plaxis model;
- Theoretical calculations are conservative compared to the real situation;
- Extremely high surface load;
- The inaccuracy margin of ± 30% of Plaxis deformations (cur 221);
- Large influence of the clay layers, more than D-sheet and blum calculations.

The results of the plaxis calculation will also be checked by the maximum appearance moments, forces and bearing capacity. The maximum represented moments, forces and bearing capacity are:

- Structural
  - Combi-wall:
    - $M_{r,max} = f_y * W_y = 235{,}000 * 0.0052 = 1{,}222\ kNm$
    - $N_{r,max} = f_y * A = 235{,}000 * 0.026 = 6{,}110\ \mathrm{kN}$
    - $Q_{r,max} = f_y * A = 235{,}000 * 0.026 = 6{,}110\ \mathrm{kN}$
  - Tension pile:
    - $N_{r,max} = f_y * A = 235{,}000 * 0.0155 = 3{,}643\ \mathrm{kN}$
  - Bearing pile:
    - $M_{r,max} = f_{ck} * W_y = 20{,}000 * 0.0106 = 213\ kNm$
    - $N_{r,max} = f_{ck} * A = 20{,}000 * 0.16 = 3{,}200\ \mathrm{kN}$
- Geotechnical:
  - Bearing capacity front wall
    - $F_{b,max} = f_{b,max} * A = 10{,}000 * 0.24 = 2{,}400\ kN$
  - Bearing pile
    - $F_{b,max} = f_{b,max} * A = 10{,}000 * 0.16 = 1{,}600\ kN$
  - Tension pile
    - $F_{a,max} = t_{max} * A * L = 192 * 1.55 * 4 = 1{,}190\ kN$

Table X shows the results of the plaxis calculation of the deepening of load calculation 4 compared to the maximum impregnable values of the moments, forces and bearing capacity.

Table 23 compare of the plaxis results and the impregnable values

Phase	Load combination 4 after deepening	Original calculation	maximum value structural	maximum value Geotechnical
Safety factor	1.134		-	-
Bending moment front wall (kNm/m)	1115	933	1222	-
Shear force front wall (kN/m)	232		6110	-
Normal force front wall (kN/m)	847		6110	2400
Normal force bearing pile 1 (kN)	554	550	3200	1600
Normal force bearing pile 2 (kN)	343	550	3200	1600
Normal force bearing pile 3 (kN)	599	550	3200	1600
Normal force tension pile 1 (kN)	119	200	3643	1190
Normal force tension pile 2 (kN)	113	200	3643	1190
Deformations x top quay wall (m)	0.185	-	-	-

The orders of magnitude of the plaxis results are acceptable. The plaxis model is approved as reverence model by Dirk-Jan Jasper Focks, so this model will used to determine the effect of the solutions.

The reduction of the force after deepening occurs because of the deformation and rotation of the relieving platform. The relieving platform rotates on bearing pile 3. See figure X below for the deformation and rotation of the relieving platform.

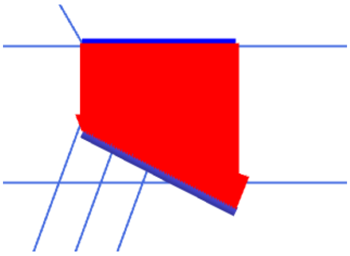


Figure 46 deformation and rotation of the relieving platform

5.6 Sensitivity analyses

The sensitivity analysys of the parameters is executed for the results of the load calculation 4. The results of the different parameters will be compared to each other.

The imput parameters are shown in table X.

Notation	type	xref	ks	Vx	beta	ai	yr	ys	xmax	xmin	Annotation
q1	solicitation	20	1,64	0,2	3,8	0,7	**0,867**	**1,154**	**17**	**23**	kN/m2
q2	solicitation	230	1,64	0,2	3,8	0,7	**0,867**	**1,154**	**199**	**265**	kN/m2
φ sand	resistance	30	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**27**	**34**	°
φ clay	resistance	25	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**22**	**28**	°
φ sand lower	resistance	32,5	1,64	0,1	3,8	0,8	**0,893**	**1,120**	**29**	**36**	°

5.6.1 Sensitivity φ

The results of the sensitivity analyses are shown in table x.

Table 24 sensitivity analyses

Phase	phase 39 load combination 4	LC4 min phi	deviation	LC4 max phi	deviation
Safety factor	1.246	1.093	-12.28%	1.42	13.96%
Bending moment front wall (kNm/m)	953	1,239	29.96%	768	-19.40%
Shear force front wall (kN/m)	229	239	4.71%	225	-1.33%
Normal force front wall (kN/m)	799	800	0.12%	761	-4.74%
Normal force bearing pile 1 (kN)	335	352	5.07%	273	-18.51%
Normal force bearing pile 2 (kN)	355	369	3.94%	346	-2.54%
Normal force bearing pile 3 (kN)	595	574	-3.53%	654	9.92%
Bending moment bearing pile 1	120	117	-2.50%	135	12.50%
Bending moment bearing pile 2	118	120	1.69%	116	-1.69%
Bending moment bearing pile 3	93	133	43.01%	89	-4.30%
Normal force tension pile 1 (kN)	95	95	0.00%	98	3.16%
Normal force tension pile 2 (kN)	120	121	0.83%	122	1.67%
Deformations x top quay wall (m)	0.13	0.28	109.94%	0.077	-42.27%

The minimum parameters of the φ effects the critical structure elements. The critical elements moments and force increase. The minimum parameters are negative, but the same critical elements increase, so the conclusion remains the same.

The maximum parameters are positive according to the critical structural element. The conclusion also remains the same, but the structure will be safer as the model with the reference parameters.

The sensitivity of the surface load is determined in the load combination. The influence of the large surface load is large. The conclusion still remains the same, but the normal force of the bearing piles is also critical if the large surface load is disabled.

5.6.2 Sensitivity stiffness superstructure

The stiffness of the structure is calculated as simple structure element. The moment of inertia is calculated by the parameters of the cross section. The calculation does not includes the addition of the effect of the plate work and the tubular work of the structure. These reasons are the base of the sensitivity analyses. The counterfort parameters are assumed as 20 m4 and the result are compared to the original calculation with the moment of inertia as 6 m4. The results of the sensitivity analyses are shown in table X.

Phase	Annotation	Load combination 4 before deepening	Load combination 4 after deepening high I	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.243	-0.24%
Structural				
Bending moment front wall	kNm/m	953	952	-0.11%
Shear force front wall	kN/m	229	228	-0.22%
Normal force front wall	kN/m	799	799	-0.01%
Normal force bearing pile 3	kN	595	594	-0.17%
Bending moment bearing pile 3	kNm/m	92	90	-2.22%
Normal force tension pile 1	kN	95	96	1.04%
ULS				
Deformations x top quay wall	m	0.13	0.14	7.14%

The effect of the high stiffness does not have influence on the results, so the high value of the stiffness of the counterforts is used in the calculation, because of the plate and the tubular work of the structure.

5.6.3 Sensitivity soil difference

The plaxis model is also modulated as whole sand. The differences of the results of load combination 4 are shown in the figures and tables below.

Check	Load combination 4 before deepening	LC4 sand	deviation
Safety factor	1.246	1.41	13.00%
Bending moment front wall (kNm/m)	953	529.00	-44.49%
Shear force front wall (kN/m)	229	182.06	-20.33%
Normal force front wall (kN/m)	799	826.80	3.47%
Normal force bearing pile 1 (kN)	335	182.00	-45.67%
Normal force bearing pile 2 (kN)	355	324.00	-8.73%
Normal force bearing pile 3 (kN)	595	694.00	16.64%
Bending moment bearing pile 1	120	99.00	-17.50%
Bending moment bearing pile 2	118	90.00	-23.73%
Bending moment bearing pile 3	93	78.00	-16.13%
Normal force tension pile 1 (kN)	95	175.00	84.21%
Normal force tension pile 2 (kN)	120	178.00	48.33%
Deformations x top quay wall (m)	0.13337	0.05	-66.26%

The influence of sand instead of the clay layers is huge. The structure is much safer and the values of the structure elements are positive changed. The conclusion of the primary investigation will change if the soil is total sand, because of the less influence of the deepening to the solution. The failure mechanism and critical structure element still remain the same.

6 Conclusion

The summary of the deviation and the effect of the deepening to the structure are shown in the table below. The deviation is calculated after deepening to before deepening.

Table 25 summary of the values and the effect of the deepening to the structure elements

Check	Annotation	Load combination 4 before deepening	Load combination 4 after deepening	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.121	-10.03%
Structural				
Bending moment front wall	kNm/m	953	1,115	16.99%
Shear force front wall	kN/m	229	232	1.49%
Normal force front wall	kN/m	799	847	5.98%
Normal force bearing pile 3	kN	595	599	0.67%
Bending moment bearing pile 3	kNm/m	92	119	29.35%
Normal force tension pile 1	kN	95	119	25.26%
ULS				
Deformations x top quay wall	m	0.13	0.18	40.25%

According to the results of the plaxis calculation the following structure elements are the biggest increase of the value so these elements are critical:

- Front wall : Bending moment;
- Tension pile 1: Normal force;
- Deformations;
- Reduction of the passive pressure.

The following failure mechanisms are critical according to deepening of the port bed 2.8 meters:

- Structural:
  - Failure of front wall;
  - Failure of tension piles;
- Geotechnical:
  - Insufficient passive resistance of front wall
  - Failure of anchor/pile tension resistance

The structural assessment concludes which failure mechanism and which structure elements are critical. These critical structure elements do have a value before deepening. The most feasible deepening solution must meet the value of the structure element before deepening, to be feasible without reducing the reliability. The values are assumed as acceptable within a range around the value before deepening, because of the uncertainty of the model, the conservative modulation of the solutions and the inventory purpose of this research. The values of the critical structure elements are considered to be acceptable if the value after deepening is in between the lower limit and upper limit, which are shown in Table 26. These value are in 95% of all the cases, so the range of 5% is assumed as variation.

Table 26 lower limit and upper limit of the requirements

Criteria	Requirement	Lower limit	Upper limit
Range	**0**	**-2.5%**	**2.5%**
Safety factor	1.25	1.22	1.28
Maximum stress front wall	212	207	217
Shear force front wall	229	223	235
Normal force tension pile 1	95	93	97

The deepening solutions must meet the following requirements to be considered as feasible:

- Safety factor, $1.22 < \mathrm{SF} \leq 1.28$;
- Front wall, $207\ N/mm^2 \leq\ \sigma_{max}\ \leq 217\ N/mm^2$;
- Shear force, $223\ kN \leq\ F_s\ \leq 235\ kN$;
- Prevention of eroding/piping of the soil between the primary piles.

7 Notation

Symbol	Description	Unity
General		
g	Free fall acceleration	m/s^2
γ_{gw}	Weight density of ground water	kN/m^3
γ_w	Weight density of water	kN/m^3
CG	Centre of gravity	-
Concrete		
f_{ck}	Characteristic	N/mm^2
E	Young´s modulus	kN/m^2
R_{inter}	Interface	$-$
γ	Weight density	kN/m^3
I	Moment of inertia of the cross-sectional area	m^4
w	Self-weight of the element	kN/m^1
Steel		
f_y	Yield stress of steel	N/mm^2
I_y	Moment of inertia of the cross-sectional area	m^4
W_y	Moment of resistance of the cross-sectional area	m^3
E	Young´s modulus	kN/m^2
$\sigma_{s,a}$	Appearanced stress	N/mm^2
$\sigma_{s,p}$	Performenced stress	N/mm^2
A	Cross section area	m^2
w	Self-weight of the element	kN/m^1
Loads and forces		
M_{max}	Maximum moment	kNm
M_{re}	Reduction moment of eccentricity	kNm
N_{min}	Maximum normal force	kN
N_{max}	Minimum normal force	kN
F_c	Crane load	kN
F_b	Bollard load	kN
q	Surcharge load	kN/m
Soil		
γsat	Saturared weight density of the soil	kN/m^3
$\gamma unsat$	Unsaturared weight density of the soil	kN/m^3
$E_{50,ref}$	Secant stiffness modulus at a 50% deviatoric stress	kPa
$E_{oed,ref}$	Oedometric stiffness modulus	kPa
$E_{ur,ref}$	Unloading reloading stiffness modulus	kPa
ψ	Dilatancy angle	°
φ	Internal angle of friction	°
δ	Angel of wall friction	°
R_{inter}	Interface	$-$
C_{Ref}	Effective cohesion in drained conditions	kPa
m	amount of stress dependency (power)	$-$
υ	Poisson-factor	$-$
σ'_v	Horizontal pore pressure	N/mm^2

8 Bibliography

Gumucio, I. J. (2013). *Design of Quay Walls using the Finite.* Civil Engineering. Delft: Tudelft.

Maritime-Connector.com. (2017). Retrieved February 22, 2017, from Maritime-Connector: http://maritime-connector.com/wiki/panamax/

Peiner Träger GmbH. (2017). *Sheet piling sections*. Retrieved February 17, 2017, from Peiner Träger GmbH: http://www.peiner-traeger.de/en/products/product-range/sheet-piling-sections.html

SBRCURnet. (2014). *Publicaion 221E "Quay walls, Second Edition".* Rotterdam, The Netherlands: CRC Press/Balkema.

Stichting Cur. (2003). *Cur-rapport '2003-7 'Bepaling geotechnische parameters'.* Gouda: Stichting Cur.

Stichting CURNET. (2008). *Cur-publicatie 166 (vijfde druk) 'Damwandconstructies'.* Gouda: CURNET.

Tijssen, G., & Veldhuijsen, R. (1988). *Renovatie kademuren nog steeds in ontwikkeling.* Delft: TU Delft.

Tu delft. (1999). *Grondmechanica.* Delft: Delft University Press.

Appendix A: Input print screens Plaxis

The result of the Plaxis calculation are available on request, which can be done by E-mailing to:

Jordy Schutte

jordyschutte@hotmail.com

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Appendix B: Structural results plaxis

The result of the Plaxis calculation are available on request, which can be done by E-mailing to:

Jordy Schutte

jordyschutte@hotmail.com

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Appendix C: Geotechnical results plaxis

The result of the Plaxis calculation are available on request, which can be done by E-mailing to:

Jordy Schutte

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5 Inventory and preselection

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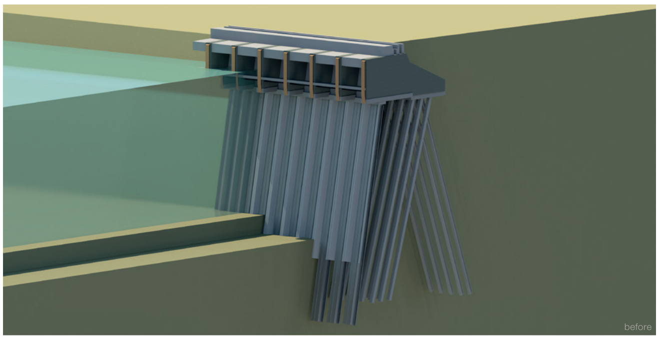

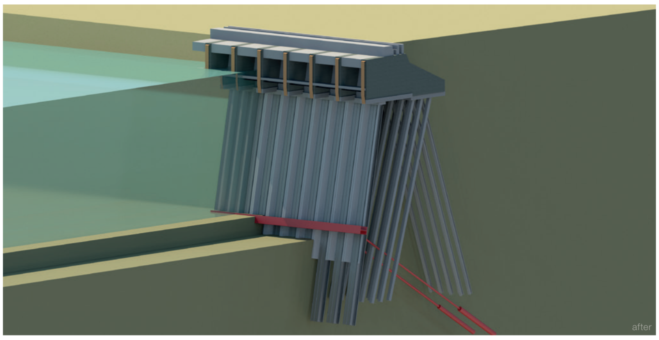

DEEPENING OF AN EXISTING COMBI WALL

INVENTORY AND PRESENTATION

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Deepening of an Existing Combi Wall

Inventory and preselection

12 juni 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
Supervisor school:	**H.J. Dommershuijzen MSc**

Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

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Summary

The preselection of alternatives is made by evaluation of the executed projects in the past, brainstorm and interviews sessions with experts and literature research. These alternatives are filtered to a shortlist by a per-selection. The selection is made by minimum criteria. The solutions which comply with all or the most of the selection criteria are considered to continue to the short list. The ranking and the assessment of the solutions is done in cooperation with the supervisors. The solutions were arranged through the following criteria:

- At least 2 meter deepening;
- Multidisciplinary solution;
- Technical feasibility;
- New structure or upgrade.

According to the ranking of the solutions the best solutions of the pre-selection provides to the short list. The short list solutions will be modulated in Plaxis and will be rated to the final solution. The following solutions will proceed to the shortlist.

- Excavation below the relieving floor;
- Grout injection behind the retaining wall;
- Grout injection at the toe of the retaining wall;
- Additional wall with concrete connection in the toe of the current wall;
- Additional underwater anchorage;
- Additional high relieving platform.

See Figure 1 for the illustration of the solutions which proceed to the shortlist.

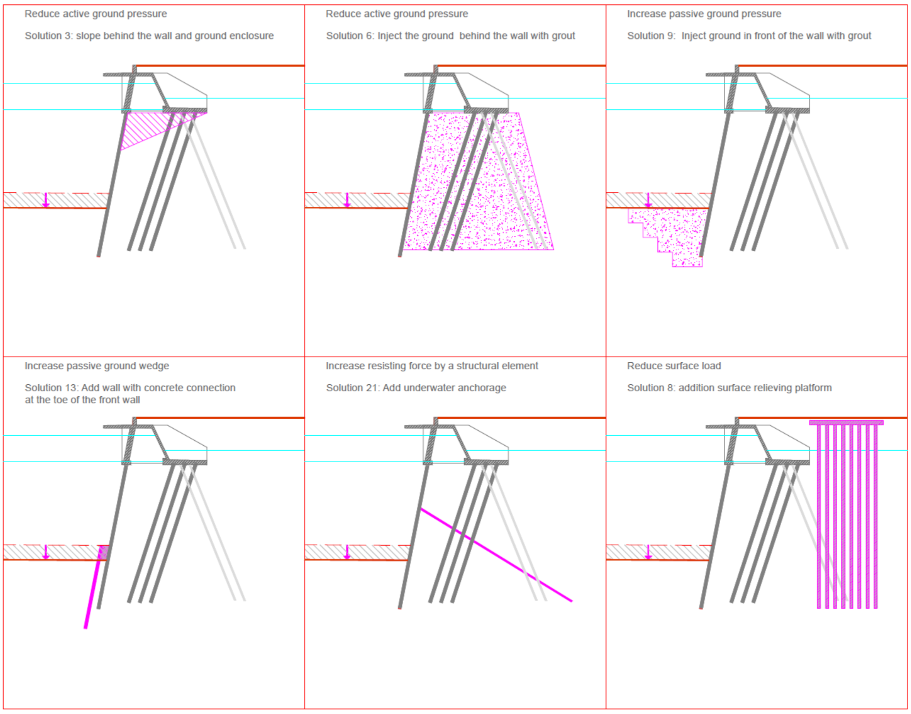

Figure 1 Selected solutions of the pre-selection

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Table of contents

1 Introduction

The previous phases of the bachelor thesis includes the preliminary investigation and the structural engineering of the reference quay wall structure, Sint Laurenshaven. The failure structural members and the failure mechanism are the results of the structural engineering. These results are the failure of structural members and the failure mechanisms. These failure mechanisms with the corresponding failure of structural member is shown in the following bullet points:

- Structural;
  - Failure of front wall;
    - Failure of structural member is the bending moment of the front wall;
  - Failure of tension piles;
    - Failure of structural member is the normal force of the tension pile;
- Geotechnical;
  - Insufficient passive resistance of front wall;
    - Failure of structural member is the reduction of the passive wedge;
  - Failure of anchor/pile tension resistance;
    - Failure of structural member is the normal force of the tension pile;
  - The deformations of the total structure.

The effect of the deepening need to be reduced, so solution for the deepening needs to be established. The solutions are determined by own inventions, desk research, a brainstorm session on the main office of Royal Haskoning DHV and interview with several experts.

The expert which are interviewed:

- Davy Bijleveld, Gebr. De Koning;
- Dirk-Jan Jaspers Focks, Witteveen+Bos;
- Harm Kortman, Port of Rotterdam;
- Hein van Laar, Hakkers;
- Henk Brassinga, Port of Rotterdam;
- Maarten Meijler, Port of Rotterdam;
- Marco van der Berg, De Klerk;
- Marinus de Heus, Jetmix:
- Maurice Krul, W. Smit duik- & bergingsbedrijf;
- Rob Vinks, Dimco;
- Willem-Jan Nederlof, Dimco.

The result of this research are pre-selected solutions which proceed to the trade-off selection.. This trade-off selection is the next phase of this thesis. The solutions are filtered through minimum criteria, which are explained in chapter 5.

In order to visualise the structure which will be deepened the principle cross section before deepening is shown in figure X. The principle cross section after the deepening work is represented in Figure X.

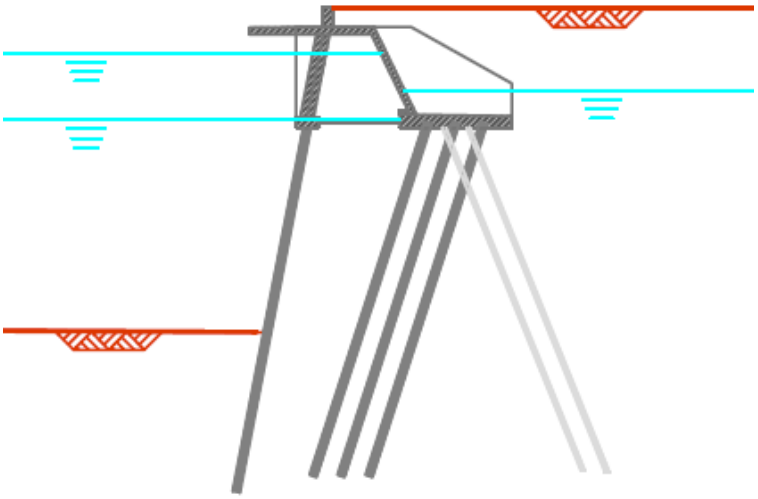


Figure 2 principle cross section before deepening

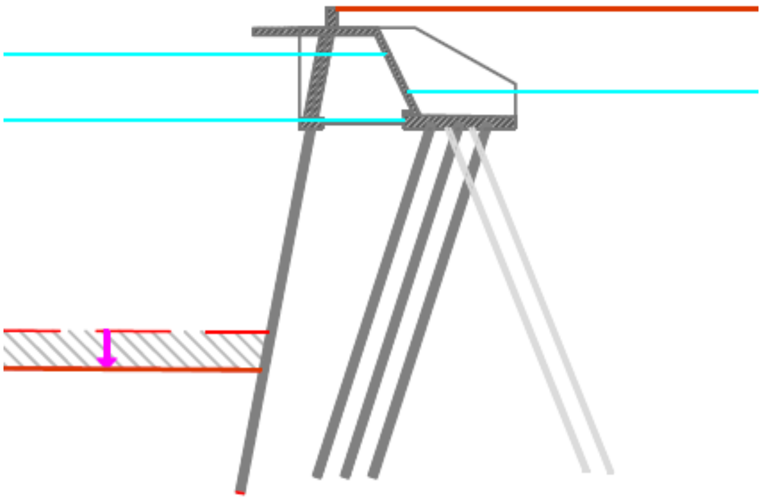
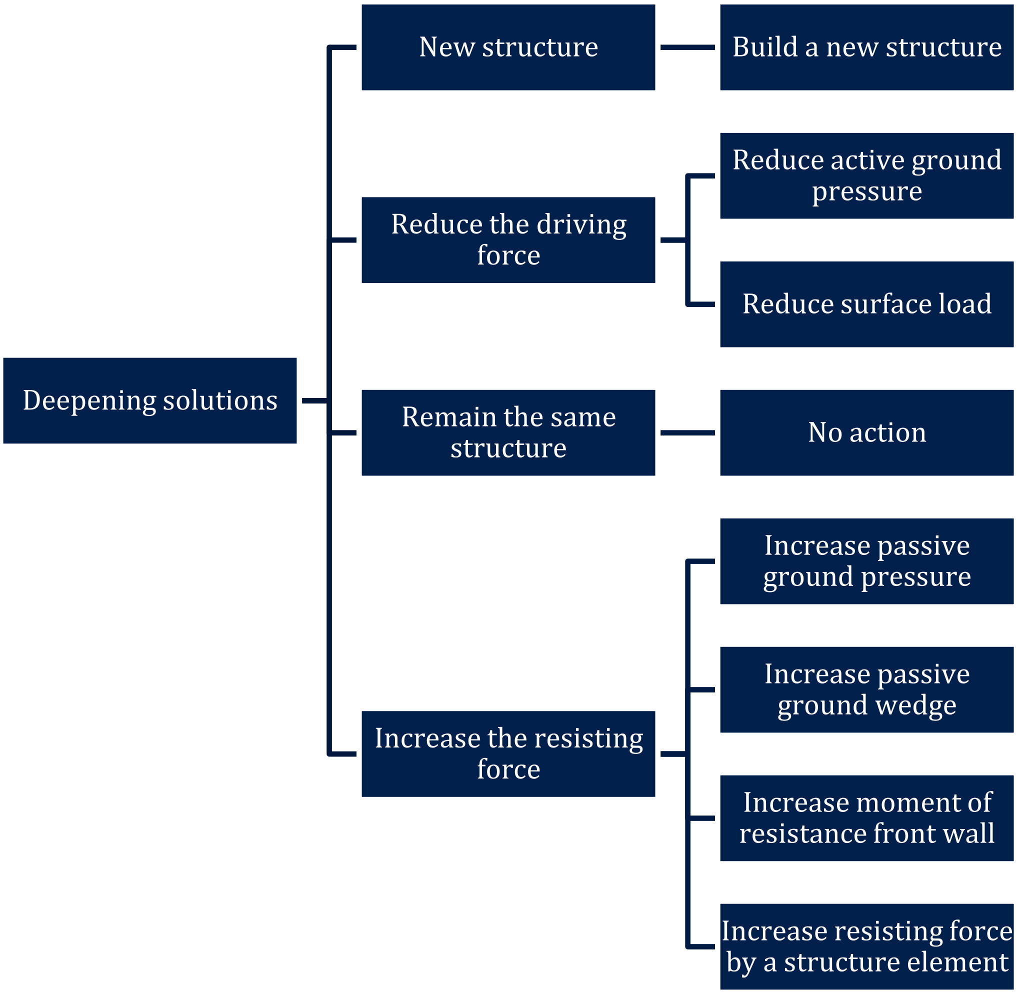


Figure 3 principle cross section after deepening

2 Type of upgrading solutions

The solution for the main failure mechanism can be divided as the chart below.



The solutions will be presented as the solution types as the chart above.

The failure mechanism after deepening are:

- Structural:
  - Failure of front wall;
  - Failure of tension piles;
- Geotechnical:
  - Insufficient passive resistance of front wall
  - Failure of anchor/pile tension resistance

The type of solution of no action and building a new structure is not part of this research

The longlist alternatives are described in chapter 5 of this report. The ranking and the selection of the solutions can be found in chapter 6.

3 Inventory

The solutions for the deepening will be divided into the type of upgrading techniques mentioned in chapter 4.

3.1 Reduce active pressure

3.1.1 Solution 1: refill ground behind the wall with light-weight material

3.1.1.1 Illustration of the solution

See Figure 4 for the illustration of solution 1.

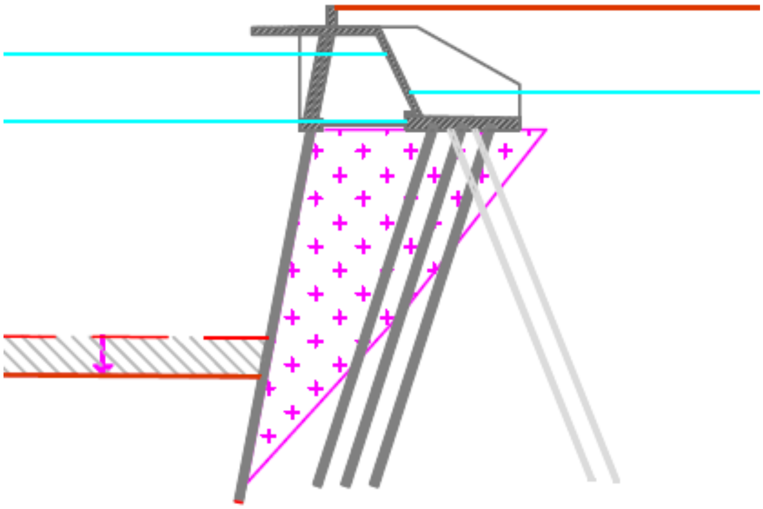


Figure 4 solution 1

3.1.1.2 Description

The refill of the ground behind the structure with light-weight material is a complex solution. The removal and the placement of the ground behind that wall are difficult. The ground behind the wall is enclosed by the superstructure, the front wall and the further ground, so hard to remove and refill.

3.1.1.3 Advantages

The advantages of this solution are:

- No adjustment to structure elements;
- Reduction of the active pressure.

3.1.1.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Hard to execute;
- Hard to maintain the slope.

3.1.2 Solution 2: refill the ground above the structure with light-weight material

3.1.2.1 Illustration of the solution

See Figure 5 for the illustration of solution 2.

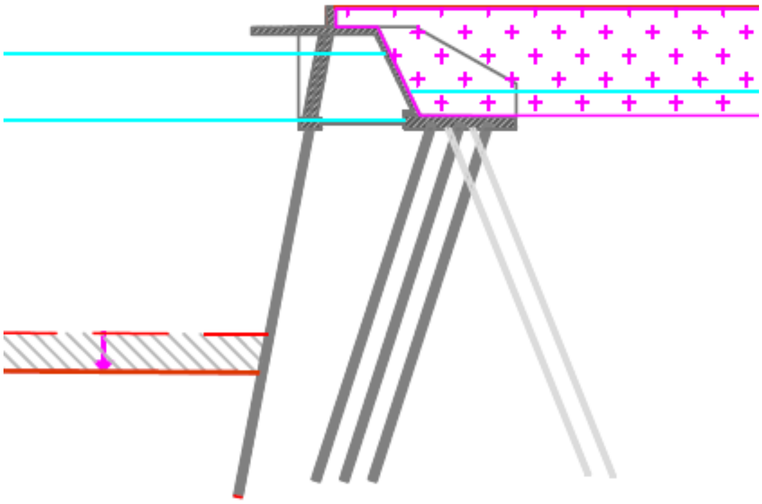


Figure 5 solution 2

3.1.2.2 Description

The subsoil above the structure will be removed and replace by light-weight materials like EPS. The execution of the refill is easier as solution 1, but the effect of the solution will be less.

3.1.2.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure;
- Reduction of the normal force of the bearing pile;
- Easy to execute.

3.1.2.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Many uses of the quay surface during the execution, so large downtime;

3.1.3 Solution 3: slope behind the wall

3.1.3.1 Illustration of the solution
See Figure 6 for the illustration of solution 3.

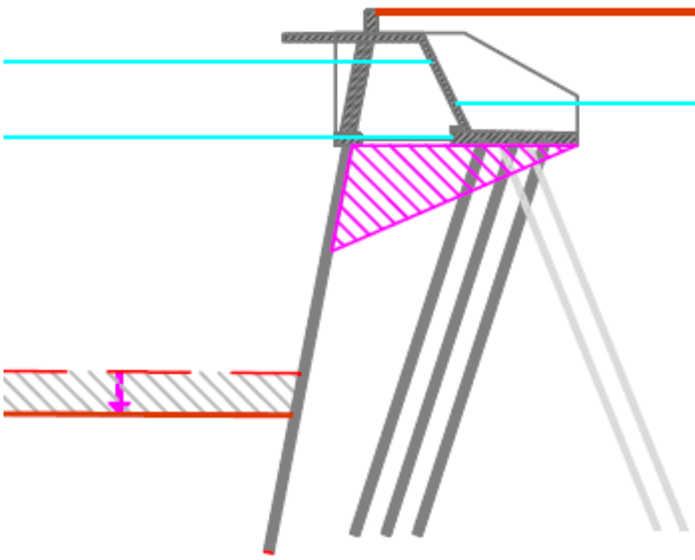


Figure 6 solution 3

3.1.3.2 Description
This solution is an alternative of solution 1. The remove ground will not be refilled with light-weight material, but covert with a underwater concrete floor. The concrete floor is to retrain the slope under the structure.

3.1.3.3 Advantages
The advantages of this solution are:

- No adjustment to the current structure;
- Simple reduction of the active pressure.

3.1.3.4 Disadvantages
The disadvantages of this solution are:

- Difficult to remove the soil under the structure;
- Local weakening of the superstructure of foundation during the execution.

3.1.4 Solution 4: soil mix wall behind the structure

3.1.4.1 Illustration of the solution

See Figure 7 for the illustration of solution 4.

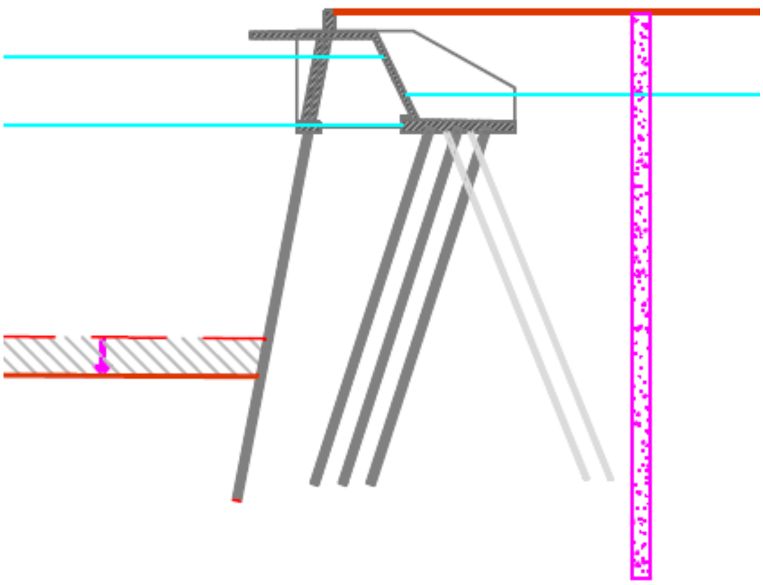


Figure 7 solution 4

3.1.4.2 Description

The soil mix wall will be constructed by a specialised equipment. The specialised equipment will dig a small rectangle trench and will mix the soil with a concrete mixture. The soil mix wall can be strengthened by the application of steel profiles. The soil mix wall can be seen as a new structure.

3.1.4.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;
- Less influence of the surface load on the current situation.

3.1.4.4 Disadvantages

The disadvantages of this solution are:

- The solution is a new structure;
- Unknown behaviour of the soil mix wall.
- Does not solve all the failure mechanism;

3.1.5 Solution 5: additional sheet pile behind the wall

3.1.5.1 Illustration of the solution

See Figure 8 for the illustration of solution 5.

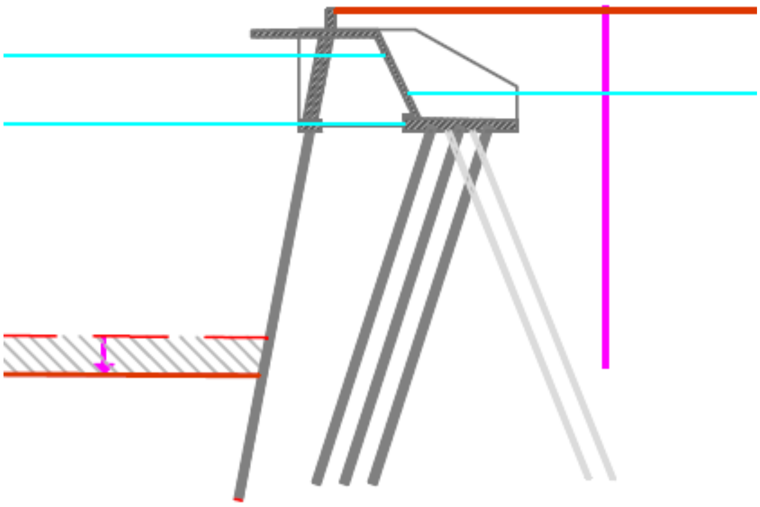


Figure 8 solution 5

3.1.5.2 Description

Solution 5 is a variation of solution 4. Instead of the soil mix wall and sheet pile wall will be used. The sheet pile will be driven from the surface, so work space is needed.

3.1.5.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;
- Less influence of the surface load on the current situation.

3.1.5.4 Disadvantages

The disadvantages of this solution are:

- The solution is a new structure;
- The surface above the structure needs to be obstacle free, so downtime;
- Does not solve all the failure mechanism;

3.1.6 Solution 6: grout injection behind the wall

3.1.6.1 Illustration of the solution

See Figure 9 for the illustration of solution 6.

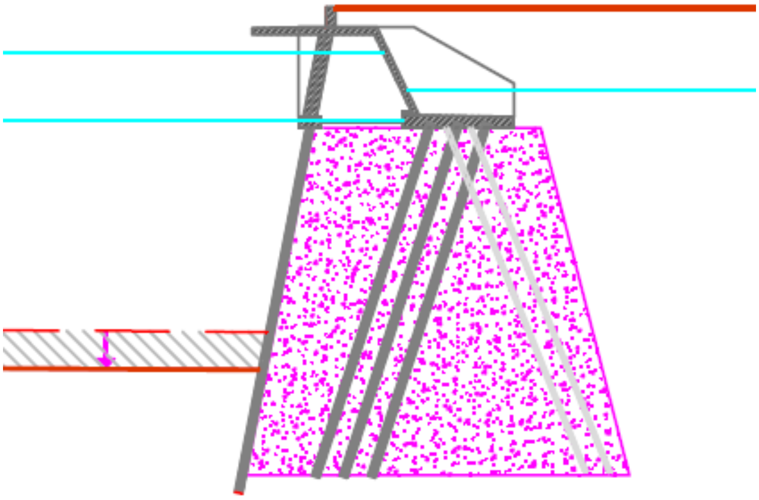


Figure 9 solution 6

3.1.6.2 Description

This solution is based on the executed deepening project in Yokohama, Japan. The ground behind the structures is injected with grout because of the earthquake resistance of the overall structure with the grout injection. The grout mixture will injected with high pressure and replaces the soil.

3.1.6.3 Advantages

The advantages of this solution are:

- Does solve all the failure mechanism;
- Will behave like a gravity quay structure;
- More deepening possible;
- Long life time;
- Earthquake proof.

3.1.6.4 Disadvantages

The disadvantages of this solution are:

- Difficult to execute because of the current pile configuration;
- Expensive execution method.

3.2 Reduce surface load

3.2.1 Solution 7: additional low relieving platform

3.2.1.1 Illustration of the solution

See Figure 10 for the illustration of solution 7.

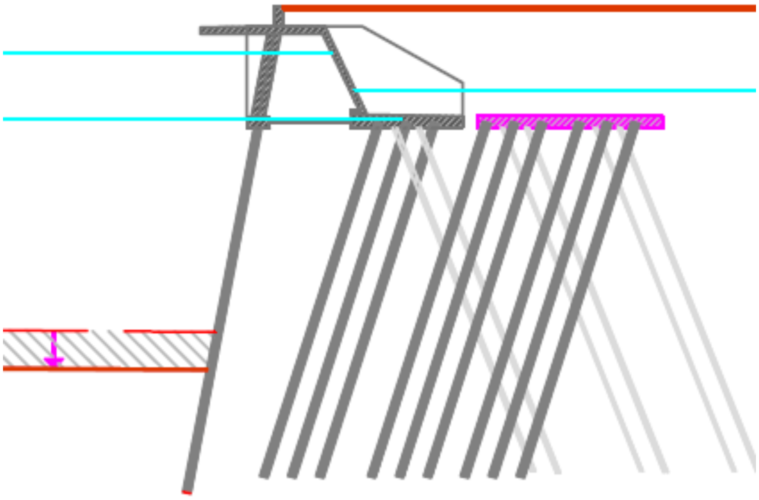


Figure 10 solution 7

3.2.1.2 Description

This solution extends the relieving floor by a separated structure. The platform will relieve the horizontal stresses on the front wall. The foundation of the relieving platform could be bearing pile in combination with tension piles. Another possibility for foundation are bearing and tension piles as one pile. The additional relieving floor will be constructed dry, so the ground need to be removed and the ground water needs to be lowered. The execution method is a disadvantage

3.2.1.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;
- No influence of the surface load to the current construction.

3.2.1.4 Disadvantages

The disadvantages of this solution are:

- Difficult to execute with the current tension piles;
- Does not solve all the failure mechanism;
- The surface above the structure needs to be obstacle free, so downtime.

3.2.2 Solution 8: additional high relieving platform

3.2.2.1 Illustration of the solution

See Figure 11 for the illustration of solution 8.

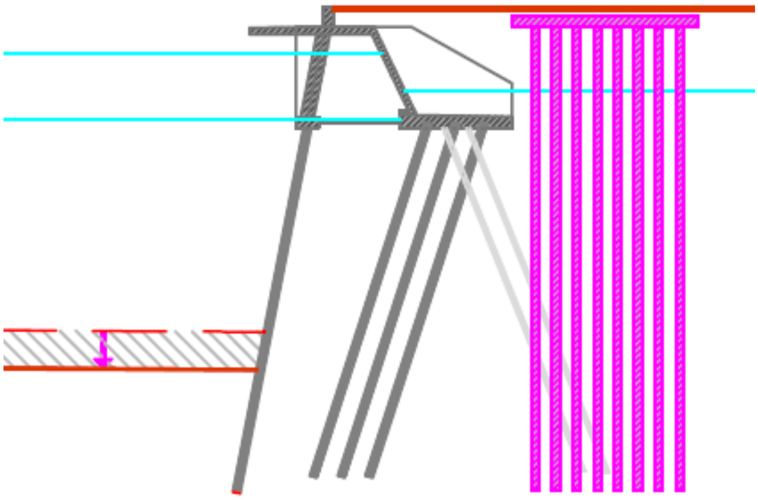


Figure 11 solution 8

3.2.2.2 Description

Solution 8 is an alternative of solution 7. Instead of a low relieving platform an extra high relieving platform will be constructed. This relieving platform directly transfers the surface load to the subsoil.

3.2.2.3 Advantages

The advantages of this solution is:

- No adjustment to the current structure and geometry;
- No influence of the surface load to the current construction.

3.2.2.4 Disadvantages

The disadvantages of this solution are:

- Difficult to execute with the current tension piles;
- Does not solve all the failure mechanism;
- The surface above the structure needs to be obstacle free, so downtime.

3.3 Increase passive pressure

3.3.1 Solution 9: grout inject in front of the wall

3.3.1.1 Illustration of the solution

See Figure 12 for the illustration of solution 9.

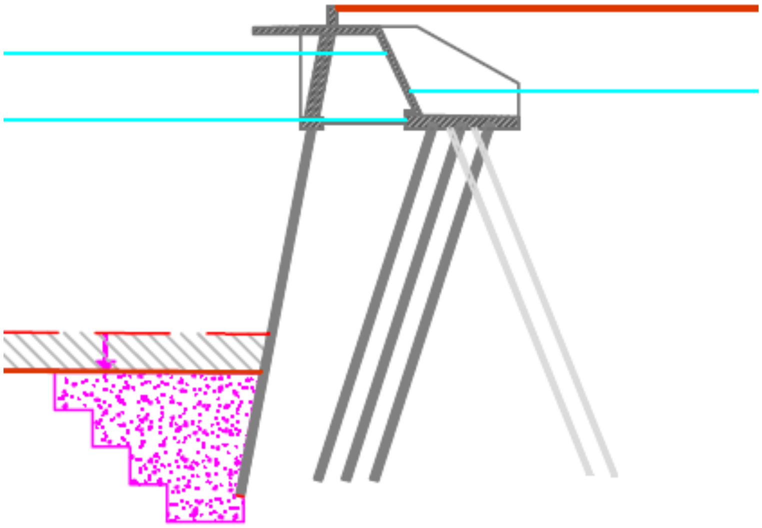


Figure 12 solution 9

3.3.1.2 Description

This type of solution is based on the executed deepening of the Waalhaven, Rotterdam. The ground in front of the structure will be replaced by grout injection. The grout injection will be done before the deepening works and in segments. The grout will replace the soil and will have a solid structure.

3.3.1.3 Advantages

The advantages of this solution are:

- Prevention of piping;
- No adjustment to the current structure and geometry;
- Solves all the failure mechanism;
- Less downtime;
- No scour protection needed.

3.3.1.4 Disadvantages

The disadvantages of this solution are:

- Unknown behaviour according to the front wall;
- Difficult to check the quality of the mixture and the connection to the wall;
- Grout injection hard in the clay layers.

3.3.2 Solution 10: Grout injection in front and behind the wall

3.3.2.1 Illustration of the solution

See Figure 13 for the illustration of solution 10.

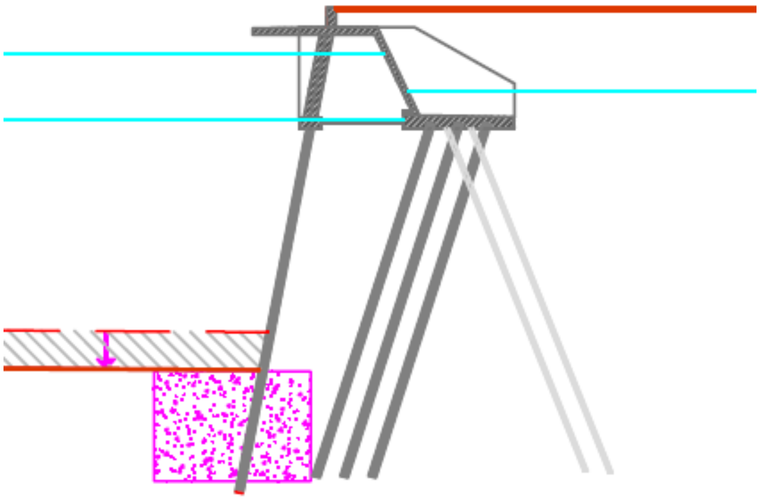


Figure 13 solution 10

3.3.2.2 Description

The solution with the fixation is a variation of solution 9. The grout will also be injected behind the wall. The injection in front and behind the wall will fixate the front wall.

3.3.2.3 Advantages

The advantages of this solution are:

- Prevention of piping;
- Solves all the failure mechanism;
- More bearing capacity;
- No scour protection needed.

3.3.2.4 Disadvantages

The disadvantages of this solution are:

- Unknown behaviour according to the front wall;
- Difficult to check the quality of the mixture and the connection to the wall;
- Grout injection hard in the clay layers.

3.3.3 Solution 11: addition of heavy material in front of the wall

3.3.3.1 Illustration of the solution

See Figure 14 for the illustration of solution 11.

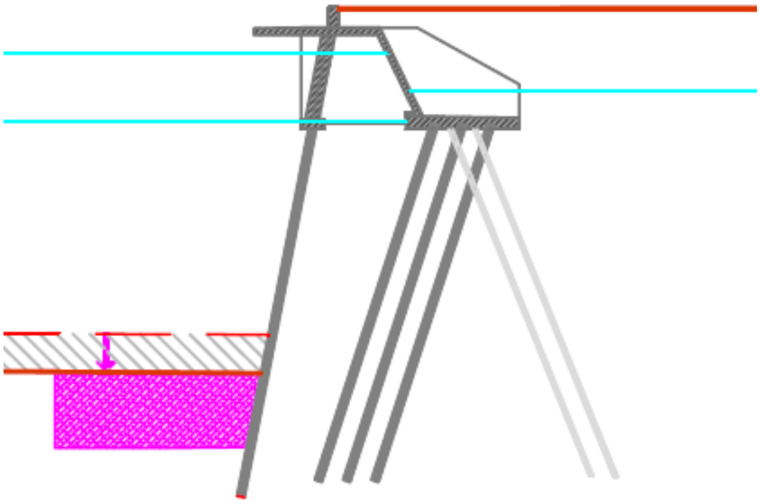


Figure 14 solution 11

3.3.3.2 Description

The solution is based on the executed deepening project of the Botlek, Rotterdam. The replacement of heavy material is an often used method in the Port of Rotterdam. The comment used material are asphalt matrasses. The ground in front of the wall will be removed and afterwards the heavy material will be placed.

3.3.3.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;
- Quick solution;
- Easy to execute;
- Solves all the failure mechanism.

3.3.3.4 Disadvantages

The disadvantages of this solution are:

- The ground in front of the wall needs to be removed at first, so the structure could fail during the execution;
- Maximum deepening possible; more or less 1 meter to 1,5 meter.

3.4 Increase passive wedge

3.4.1 Solution 12: Additional jet grout wall behind the wall

3.4.1.1 Illustration of the solution

See Figure 15 for the illustration of solution 12.

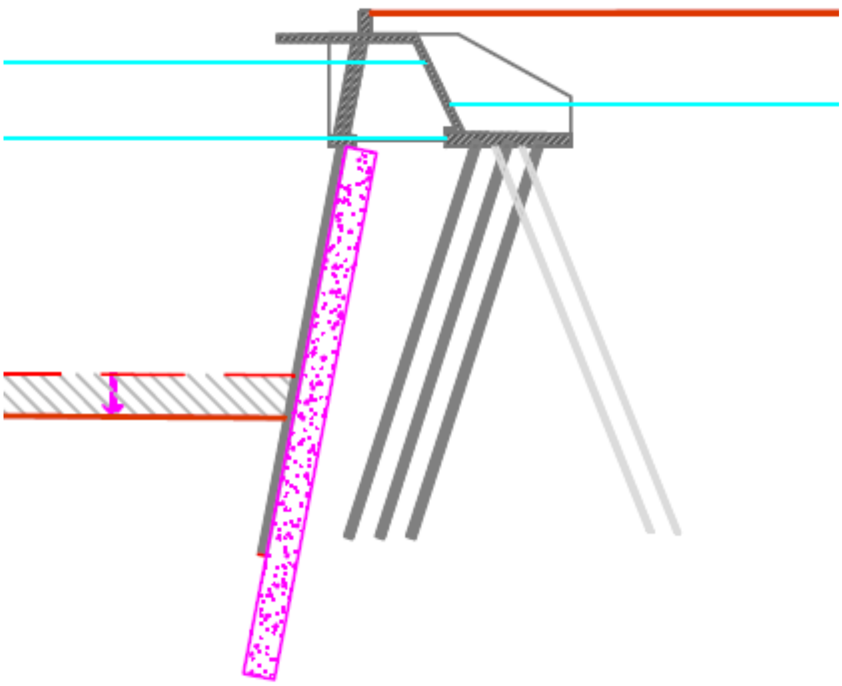


Figure 15 solution 12

3.4.1.2 Description

An additional wall will be constructed behind the front wall. This additional wall can be made of grout injection or cutter soil mix. The soil mix wall can be strengthened by the application of steel profiles. This additional wall will be seen as upgrade of the current structure.

3.4.1.3 Advantages

The advantages of this solution are:

- Solves all the failure mechanism;
- More bearing capacity of the front wall.

3.4.1.4 Disadvantages

The disadvantages of this solution are:

- Unknown behaviour of the soil mix wall according to the front wall;
- Local weakening of the superstructure by the work injection holes.

3.4.2 Solution 13: additional sheet pile wall with concrete connection

3.4.2.1 Illustration of the solution

See Figure 16 for the illustration of solution 13.

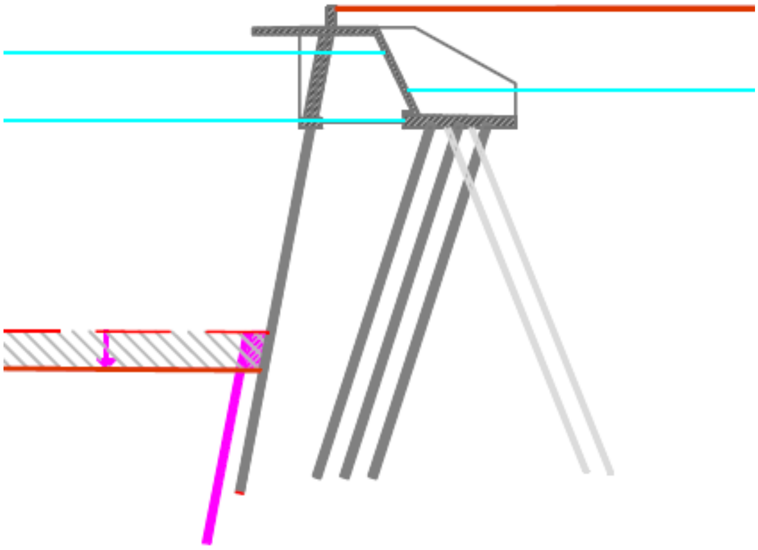


Figure 16 solution 13

3.4.2.2 Description

This solution is based on an extra sheet pile wall in front of the current quay wall structure. The sheet pile will be piled into the same angle as the current wall. The connection between the current wall and the new wall will be done by concrete so the connection is ground tight and moment fixed.

3.4.2.3 Advantages

The advantages of this solution are:

- Solves all the failure mechanism;
- No adjustment to the current structure and geometry.

3.4.2.4 Disadvantages

The disadvantages of this solution are:

- Unknown behaviour of the total structure;
- Difficult execution because of the current structure.

3.4.3 Solution 14: extension of the current front wall

3.4.3.1 Illustration of the solution

See Figure 17 for the illustration of solution 14.

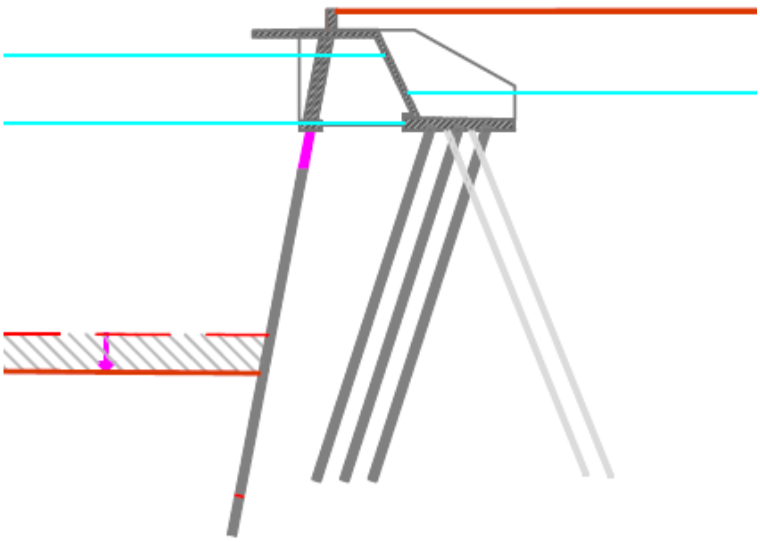


Figure 17 solution 14

3.4.3.2 Description

The solution is based on the extension of the current front wall. The current combi-wall will be piled to more depth and will be extended. The material properties will be the same. The extended combi-wall can adopt more water depth.

3.4.3.3 Advantages

The advantages of this solution are:

- Predictable effects of the solution;
- Extendable.

3.4.3.4 Disadvantages

The disadvantages of this solution are:

- Hard to execute because of the current superstructure;
- No piping prevention.

3.4.4 Solution 15: Additional wall with corbelling

3.4.4.1 Illustration of the solution

See Figure 18 for the illustration of solution 15.

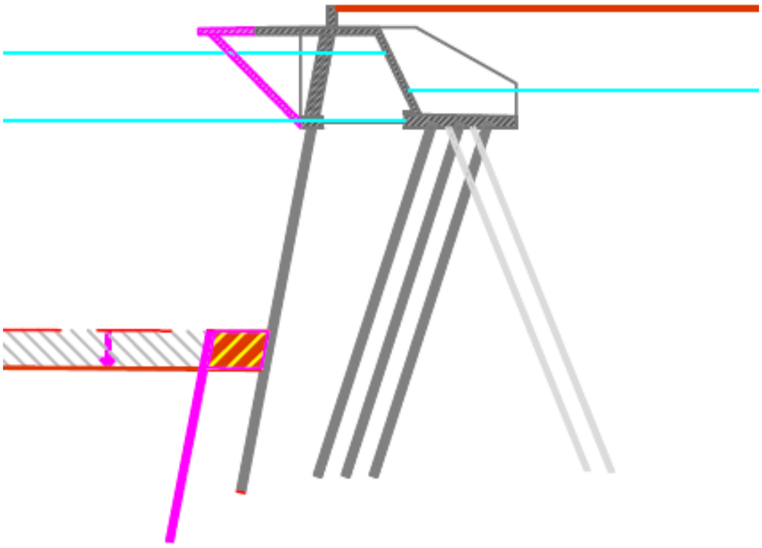


Figure 18 solution 15

3.4.4.2 Description

This solution is an alternative of solution 14. Instead of a concrete connection the connection is made by the ground. The new sheet pile wall will be constructed several meters out of the current structure.

3.4.4.3 Advantages

The advantages of this solution are:

- Solves all the failure mechanism;
- Corbelling of the fender;
- Piping prevention.

3.4.4.4 Disadvantages

The disadvantages of this solution are:

- Unknown behaviour of the total structure;
- Difficult execution because of the current superstructure.

3.4.5 Solution 16: additional wall with full grout connection

3.4.5.1 Illustration of the solution

See Figure 19 for the illustration of solution 16.

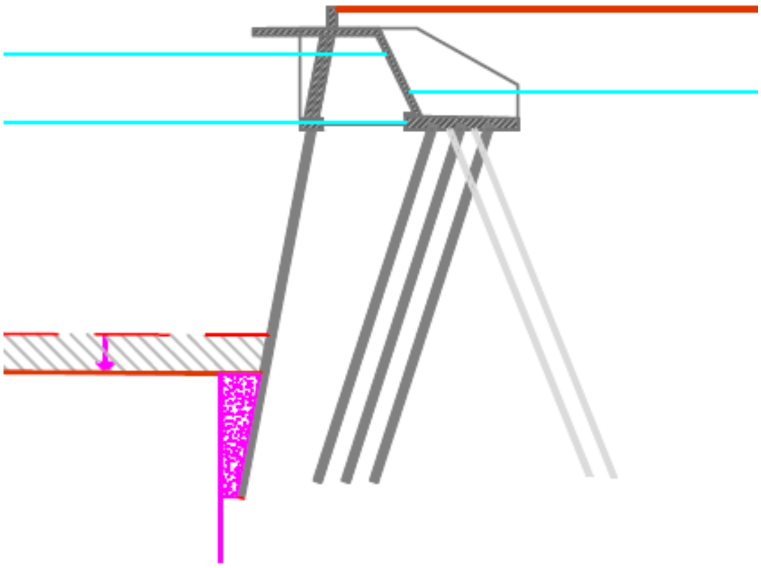


Figure 19 solution 16

3.4.5.2 Description

This solution is an optimization of solution 14. The sheet pile will be piled vertical so the current structure will be avoided. The new sheet pile wall will be connected to the current wall by a full concrete connection.

3.4.5.3 Advantages

The advantages of this solution are:

- Solves all the failure mechanism;
- Sheet pile can easy be constructed;
- Piping prevention.

3.4.5.4 Disadvantages

The disadvantages of this solution are:

- Difficult to execute the grout injection because of the shorter intermediate piles;
- Unknown behaviour of the total structure.

3.4.6 Solution 17: Cutter soil mix wall in front of the structure

3.4.6.1 Illustration of the solution

See Figure 20 for the illustration of solution 17.

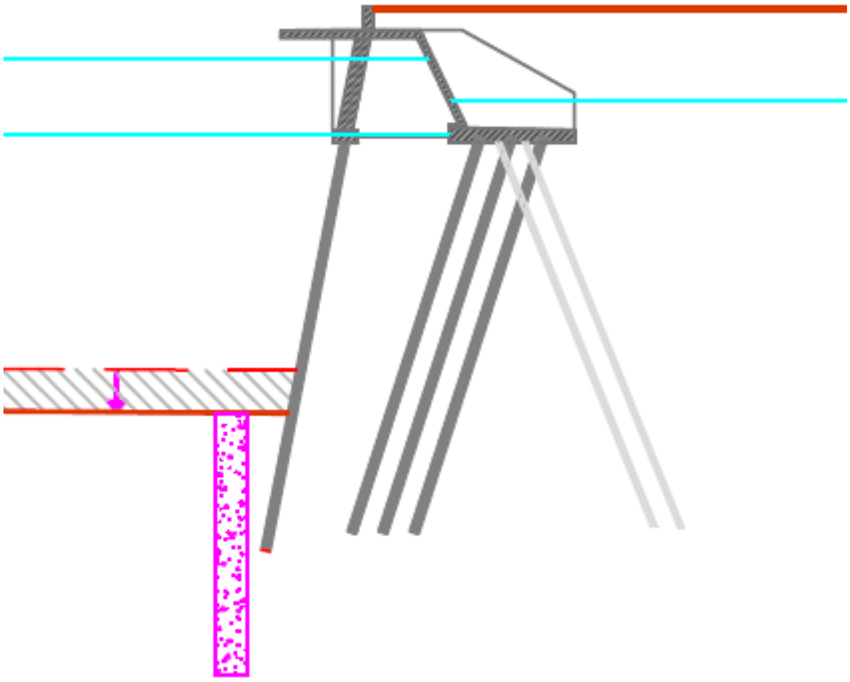


Figure 20 solution 17

3.4.6.2 Description

This solution is an new soil mix wall in front of the current structure. These soil mix wall are nowadays only constructed on land, so the execution method underwater is unknown.

3.4.6.3 Advantages

The advantages of this solution are:

- Long life time;
- No adjustment to the current structure.

3.4.6.4 Disadvantages

The disadvantages of this solution are:

- Hard to execute under water;
- No connection between the new wall and the current.

3.5 Increase moment of resistance front wall

3.5.1 Solution 18: additional steel on the front wall

3.5.1.1 Illustration of the solution

See Figure 21 for the illustration of solution 18.

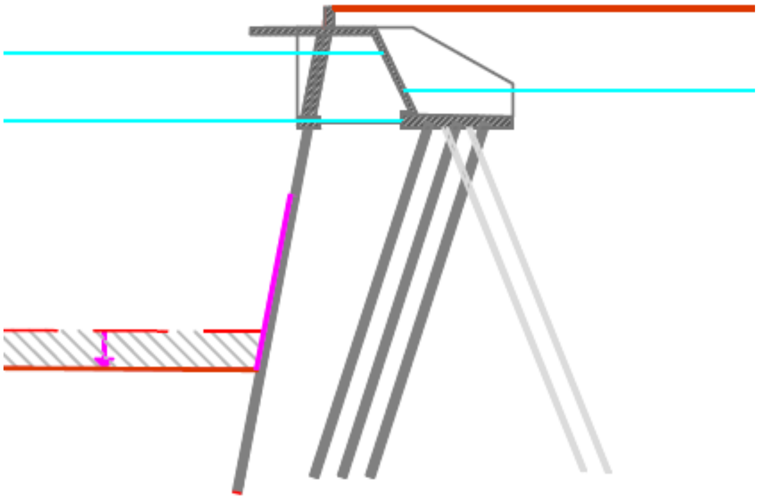


Figure 21 solution 18

3.5.1.2 Description

This solution is an often executed method in the Port of Rotterdam. This is an temporary solution to strengthen the current front wall. The execution is easy and quick.

3.5.1.3 Advantages

The advantages of this solution are:

- Easy execution;
- Proven solution;
- Can be combined with other solutions.

3.5.1.4 Disadvantages

The disadvantages of this solution are:

- No long life time expectation;
- Only solve one failure mechanism: exceeded bending moment front wall.

3.6 Increase resisting force by a structure element

3.6.1 Solution 19: multiple anchorage

3.6.1.1 Illustration of the solution

See Figure 22 for the illustration of solution 19.

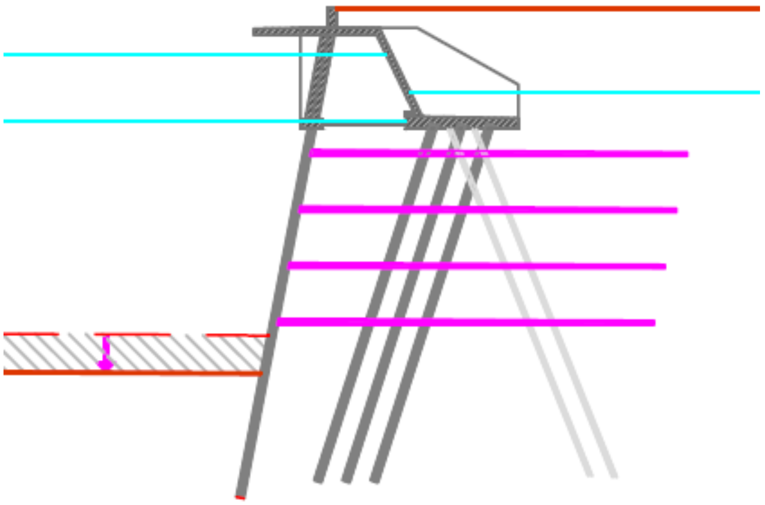


Figure 22 solution 19

3.6.1.2 Description

This solution provides several additional anchorage. The addition of the multiple anchorage provides also multiple horizontal resisting forces. The behaviour of the total structure is unknown.

3.6.1.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;.
- Several small anchorage instead of 1 large anchor.
- Spread out of the force of the front wall.

3.6.1.4 Disadvantages

The disadvantages of this solution are:

- The anchorage cannot be executed in the clay layers;
- Local weakening of the front wall by the anchorage connections;
- Unknown effect of the anchorage;
- Difficult execution because of the pile configuration.

3.6.2 Solution 20: additional low underwater anchor

3.6.2.1 Illustration of the solution

See Figure 23 for the illustration of solution 20.

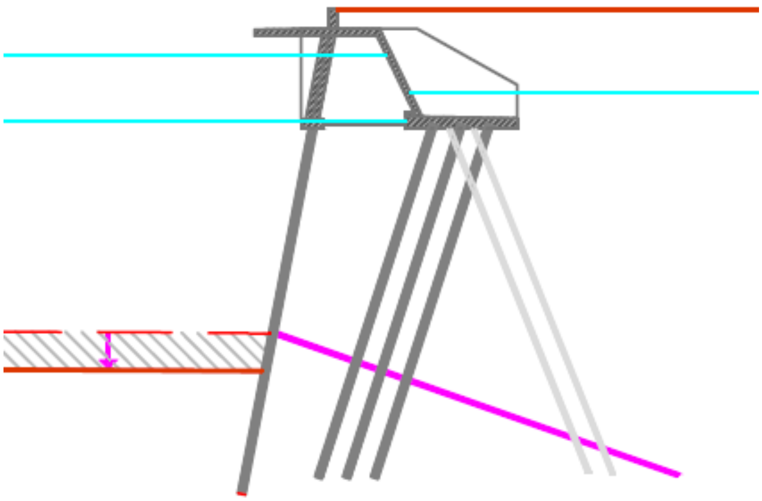


Figure 23 solution 20

3.6.2.2 Description

This solution is based on the executed deepening project of Ravenna, Italy. The solution consist of an additional underwater anchorage. This solution add the anchorage at the current port bed level.

3.6.2.3 Advantages

The advantages of this solution are:

- Short execution time;
- Long life time.

3.6.2.4 Disadvantages

The disadvantages of this solution are:

- Local weakening of the front wall by the anchorage connections;
- Unknown effect of the anchorage.

3.6.3 Solution 21: additional middle underwater anchor

3.6.3.1 Illustration of the solution

See Figure 24 for the illustration of solution 21.

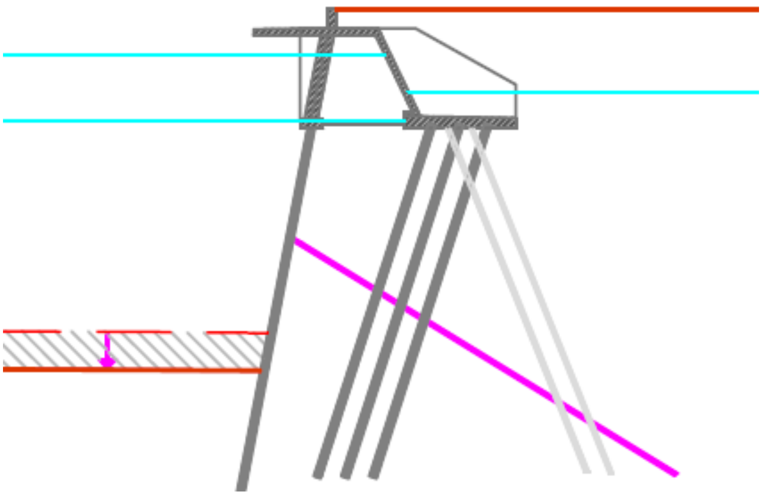


Figure 24 solution 21

3.6.3.2 Description

This solution is an alternative of solution 20. The variation is the addition of an anchor at the middle of the front wall instead of the current port bed level.

3.6.3.3 Advantages

The advantages of this solution are:

- Short execution time;
- Long life time.

3.6.3.4 Disadvantages

The disadvantages of this solution are:

- Local weakening of the front wall by the anchorage connection;
- Unknown effect of the anchorage.

3.6.4 Solution 22: additional high underwater anchor

3.6.4.1 Illustration of the solution

See Figure 25 for the illustration of solution 22.

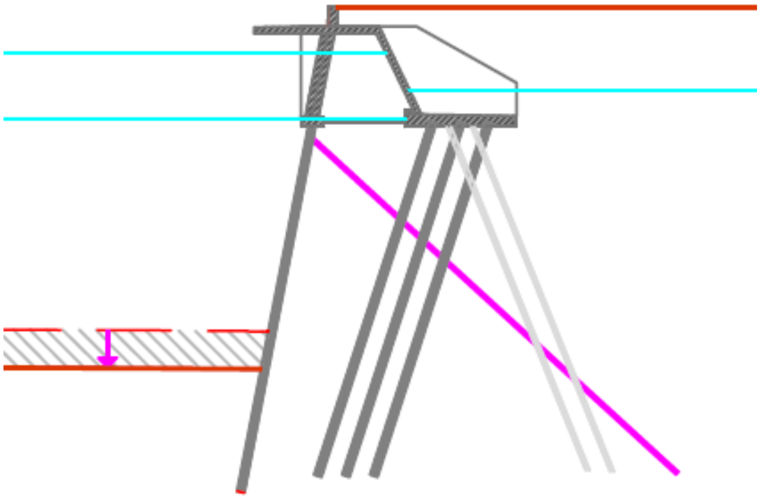


Figure 25 solution 22

3.6.4.2 Description

Solution 22 is also a variation of solution 20. The additional anchorage will be add at the top of the front wall. The predictable effect of this solution is less in compare with solution 20 and 21.

3.6.4.3 Advantages

The advantages of this solution are:

- Short execution time;
- Long life time.

3.6.4.4 Disadvantages

The disadvantages of this solution are:

- Local weakening of the front wall by the anchorage connections;
- Less effect on the front wall.

3.7 Remaining solutions

3.7.1 Solution 23: additional anchor on the relieving platform

3.7.1.1 Illustration of the solution

See Figure 26 for the illustration of solution 23.

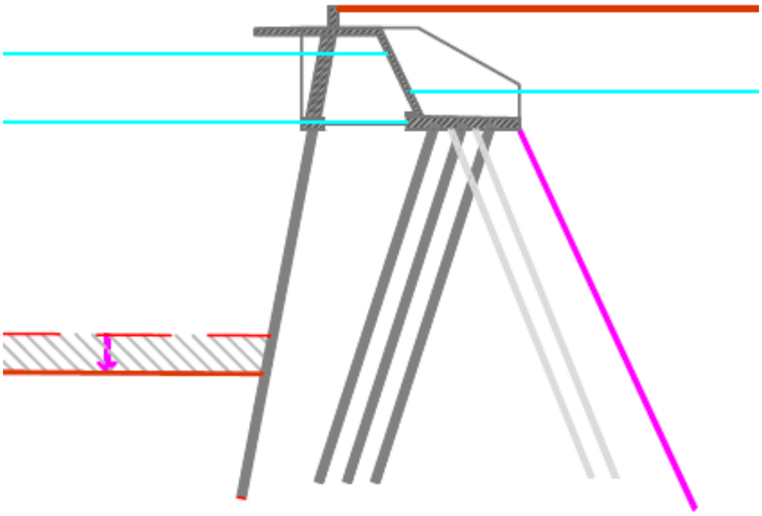


Figure 26 solution 23

3.7.1.2 Description

Solution 23 is the same solution as 22. The difference is the additional anchorage is not connected to the front wall, but to the relieving platform. The effect of this solution is small is mentioned during the interview with the experts.

3.7.1.3 Advantages

The advantages of this solution are:

- No adjustment to the current structure and geometry;
- Additional horizontal force.

3.7.1.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Difficult to apply;
- Less effect on the front wall.

3.7.2 Solution 24: new wall with connection to the current structure

3.7.2.1 Illustration of the solution

See Figure 27 for the illustration of solution 24.

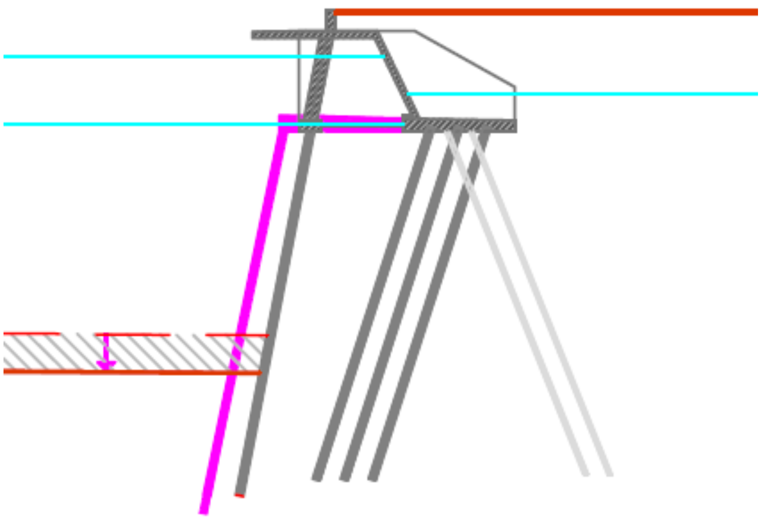


Figure 27 solution 24

3.7.2.2 Description

The solution of an additional new front wall can will be seen as a total new structure. The new front wall will be connected to the current relieving platform.

3.7.2.3 Advantages

The advantages of this solution are:

- More deepening can be reached;
- Does solve all the failure mechanism.

3.7.2.4 Disadvantages

The disadvantages of this solution are:

- Difficult execution method because of the current structure;
- New structure instead of upgrade.

3.7.3 Solution 25: soil nailing of the ground

3.7.3.1 Illustration of the solution

See Figure 28 for the illustration of solution 25.

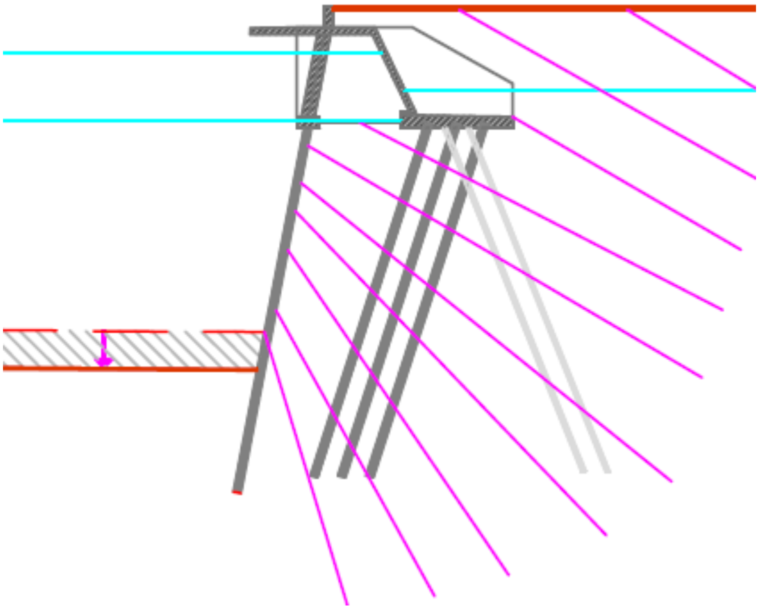


Figure 28 solution 25

3.7.3.2 Description

This solution is based on the technique of the dike improvements. The soil nails prevent the dike, so the structure for the sliding of the bishop overall stability. The soil nails are small grout anchorage.

3.7.3.3 Advantages

The advantages of this solution are:

- Approved technique for dike improvements;
- Small anchorage possible.

3.7.3.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Difficult execution method;
- Long execution time;
- Local weakening of the front wall by the anchorage connections.

3.7.4 Solution 26: piping protection geotextile

3.7.4.1 Illustration of the solution

See Figure 29 for the illustration of solution 26.

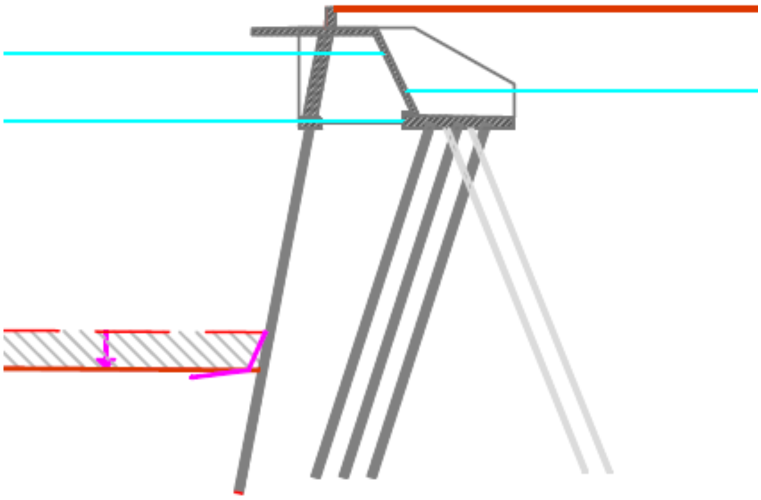


Figure 29 solution 26

3.7.4.2 Description

This solution is often just in the dike improvements. The heave and piping of the sand is prevented by the geotextiles. These textiles are water open but ground tight. This solution only prevents the piping failure mechanism.

3.7.4.3 Advantages

The advantages of this solution are:

- Piping prevention;
- No adjustment to the current structure and geometry.

3.7.4.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Additional solutions needed.

3.7.5 Solution 27: waterglass ball screen

3.7.5.1 Illustration of the solution

See Figure 30 for the illustration of solution 27.

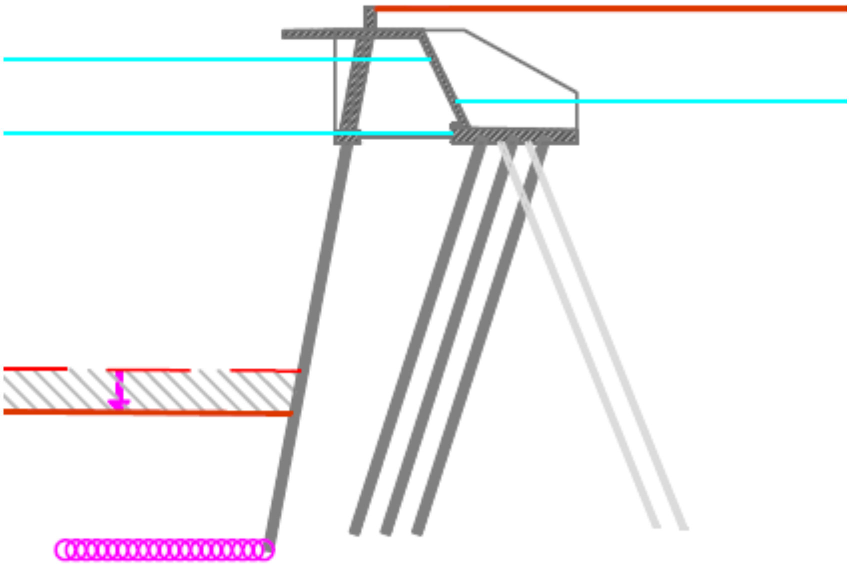


Figure 30 solution 27

3.7.5.2 Description

Instead of an geotextile and waterglass screen will be add for solution 27. This waterglass screen prevents the waterflow. Heave and piping will be prevented by this solution.

3.7.5.3 Advantages

The advantages of this solution are:

- Short execution time;
- Piping prevention;
- Easy to apply.

3.7.5.4 Disadvantages

The disadvantages of this solution are:

- Does not solve all the failure mechanism;
- Temporary solution.

4 Selection criteria

The longlist of alternatives will be made according to executed projects in the past, brainstorm and interviews sessions with experts and literature research. These alternatives will be filtered to a shortlist by a selection. The selection will made by the minimum criteria. The solutions which comply with all or the most of the selection criteria are considered to continue to the short list. The ranking and the assessment of the solutions is done in cooperation with the supervisors. The solutions will be ranked by the following criteria:

4.1 At least 2 meter deepening

The minimum deepening of 2 meter need to be achieved by the application of the solution. The solution will be ranked with a Yes, if the deepening of 2 meters can be arranged. The solution will be ranked with an No is the 2 meters deepening not can be arranged. This criteria is ranked in cooperation with the supervisor and the experts. The solutions with less influence to the structure are ranked with an no.

4.2 Multidisciplinary solution

The solutions can be project specific of multidisciplinary. This research will focus on multidisciplinary solutions. Multidisciplinary solutions are solutions for several type of quay wall with relieving platforms. The multidisciplinary solutions solve multipole failure mechanism, project specific solve 1 type of failure mechanism as piping. The project specific solutions will be ranked with a No and multidisciplinary solutions will be ranked with a Yes.

4.3 Technical feasibility

The technical feasibility of the solution will be ranked with, Yes or no. The ranking of the feasibility of the solution will be done by the interviews with the expert and the experience of the student. The solution will proceed with a Yes, if the solution is feasible to perform. The solution will be ranked with a No, if the upgrade is not feasible to perform.

4.4 New structure or upgrade

Solution can be an upgrade or a total new structure. This research focuses on the upgrade of the structure. The solution will proceed with a Yes, if the solution is an upgrade. The solution will be ranked with a No, if the solution is a new structure.

5 Ranking solutions

The ranking of the solution is done by the selection criteria mentioned in chapter 5 The ranking of the solution is checked and adjusted by the supervisor Alfred Roubos and experts Harm Kortman, Maarten Meijler and Johan Plugge. The validation of the ranking is added in appendix A.

The Table 1 below presents the summary of the ranking of the solution.

Table 1 score summary longlist

Nr	Solution	At least 2 meter deepening	Multidisciplinary solution	Structural feasibility	New structure or upgrade	Total score
3	Excavation below the relieving floor	yes	yes	yes	yes	4
6	Grout injection behind the retaining wall	yes	yes	yes	yes	4
9	Grout injection at the toe of the retaining wall	yes	yes	yes	yes	4
10	Inject the ground in front of and behind the wall with grout to fixate the wall	yes	yes	yes	yes	4
13	Additional wall with concrete connection in the toe of the retaining wall	yes	yes	yes	yes	4
16	Additional sheet pile with full grout connection	yes	yes	yes	yes	4
20	Additional low underwater anchorage	yes	yes	yes	yes	4
21	Additional middle underwater anchor	yes	yes	yes	yes	4
8	Additional high relieving platform	yes	yes	yes	yes	4
19	Multiple anchorage	yes	yes	no	yes	3
1	Refill ground behind wall with light-weight material	yes	no	no	yes	2
2	Refill the ground above the structure with light-weight material	no	no	yes	yes	2
7	Additional low relieving platform	yes	no	no	yes	2
11	Add heavy material at the toe of the structure	no	yes	no	yes	2
14	Extend the current wall	yes	no	no	yes	2
15	Additional wall with corbelling of the current structure	yes	no	no	yes	2
18	Add additional steel to the front wall	no	no	yes	yes	2
22	Add high underwater anchor	no	no	yes	yes	2
24	Add additional wall in front of the existing wall with connection to the relieving platform	yes	no	yes	no	2
25	Add soil nails through the bishop sliding surface	yes	no	no	yes	2
26	Add piping prevention screen	no	no	yes	yes	2
4	Soil mix wall behind the structure	no	no	yes	no	1
5	Add extra sheet pile behind the structure	no	no	yes	no	1
12	Additional jet grout wall behind the wall	no	no	yes	no	1
17	Cutter soil mix wall in front of the wall	no	yes	no	no	1
23	Add an additional anchor at the relieving structure	no	no	no	yes	1
27	Waterglass ball screen for piping prevention	no	no	no	yes	1

5.1 Pre-selection assessment

The solutions with a score four out of four are selected. See Table 2 for the selected solutions with a score four out of four.

Table 2 top 9 solutions

Nr	Solution	At least 2 meter deepening	Multidisciplinary solution	Structural feasibility	New structure or upgrade	Total score
3	Excavation below the relieving floor	✓	✓	✓	✓	4
6	Grout injection behind the retaining wall	✓	✓	✓	✓	4
9	Grout injection at the toe of the retaining wall	✓	✓	✓	✓	4
10	Inject the ground in front of and behind the wall with grout to fixate the wall	✓	✓	✓	✓	4
13	Additional wall with concrete connection in the toe of the retaining wall	✓	✓	✓	✓	4
16	Additional sheet pile with full grout connection	✓	✓	✓	✓	4
20	Additional low underwater anchorage	✓	✓	✓	✓	4
21	Additional middle underwater anchor	✓	✓	✓	✓	4
8	Additional high relieving platform	✓	✓	✓	✓	4

Several solutions are more or less the same. The same solutions are combined as main solutions categories. The main solution categories with proceeded the pre-selection are:

- Excavation below the relieving floor;
- Grout injection behind the retaining wall;
- Grout injection at the toe of the retaining wall;
- Additional wall with concrete connection in the toe of the retaining wall;
- Additional underwater anchorage;
- Additional high relieving platform.

The remaining solutions are proceeded to the final trade-off selection. See **Fout! Verwijzingsbron niet gevonden.** and Figure 31for the solutions which proceed to the final selection.

Solution	At least 2 meter deepening	Multidisciplinary solution	Structural feasibility	New structure or upgrade	Total score
Excavation below the relieving floor	✓	✓	✓	✓	4
Grout injection behind the retaining wall	✓	✓	✓	✓	4
Grout injection at the toe of the retaining wall	✓	✓	✓	✓	4
Additional wall with concrete connection in the toe of the retaining wall	✓	✓	✓	✓	4
Additional underwater anchorage	✓	✓	✓	✓	4
Additional high relieving platform	✓	✓	✓	✓	4

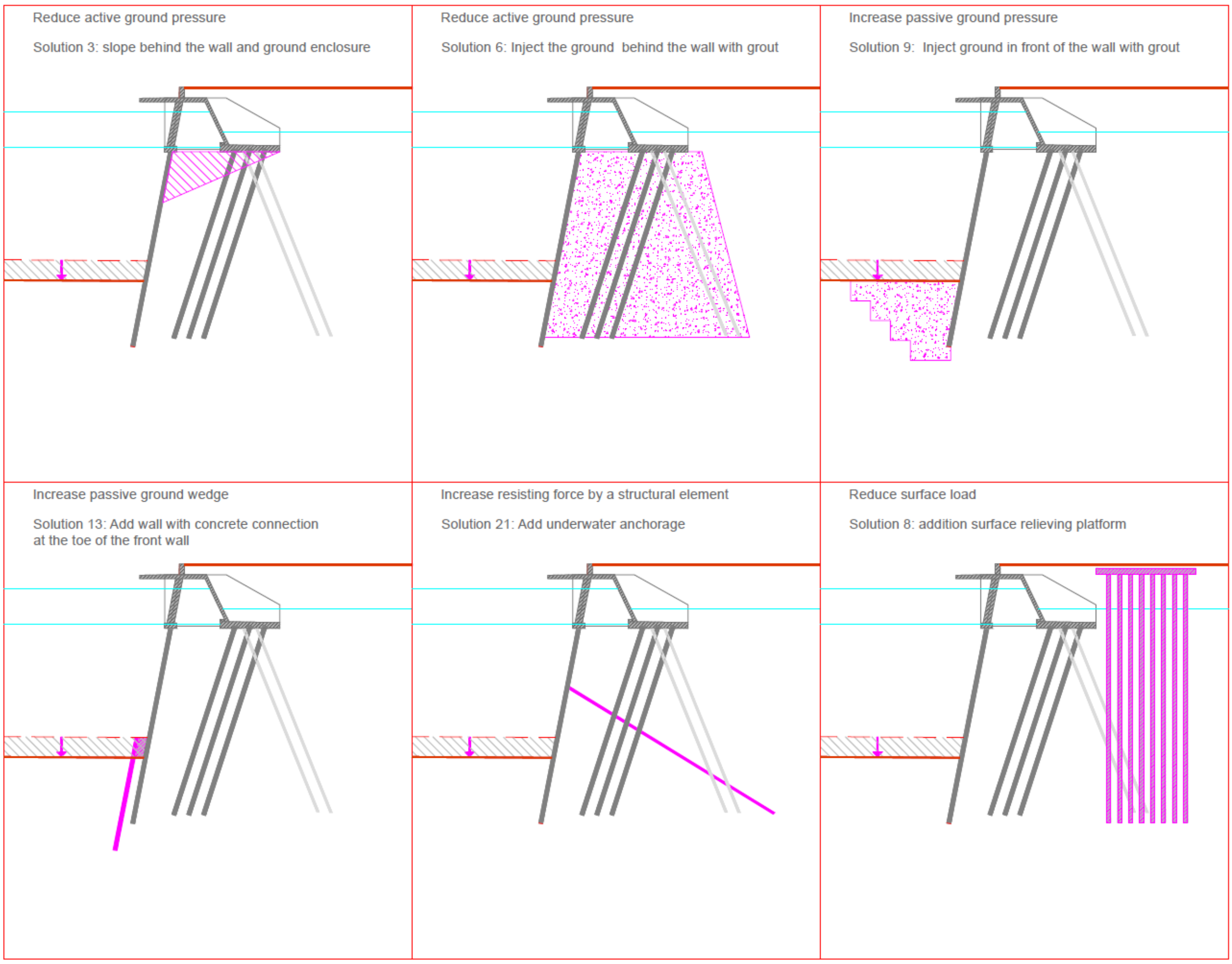


Figure 31 Selected solutions of the pre-selection

6 Conclusion

According to the ranking of the solutions the six best solutions of the pre-selection provides to the shortlist. The solutions of the short list be modulated in Plaxis. The solutions of the shortlist will be elaborated and compared to determine the most feasible solution in a Multi Criteria Analysis (MCA). The following solutions will proceed to the shortlist.

- Excavation below the relieving floor;
- Grout injection behind the retaining wall;
- Grout injection at the toe of the retaining wall;
- Additional wall with concrete connection in the toe of the current wall;
- Additional underwater anchorage;
- Additional high relieving platform.

See Figure 32 for the illustration of the solutions which proceed to the shortlist.

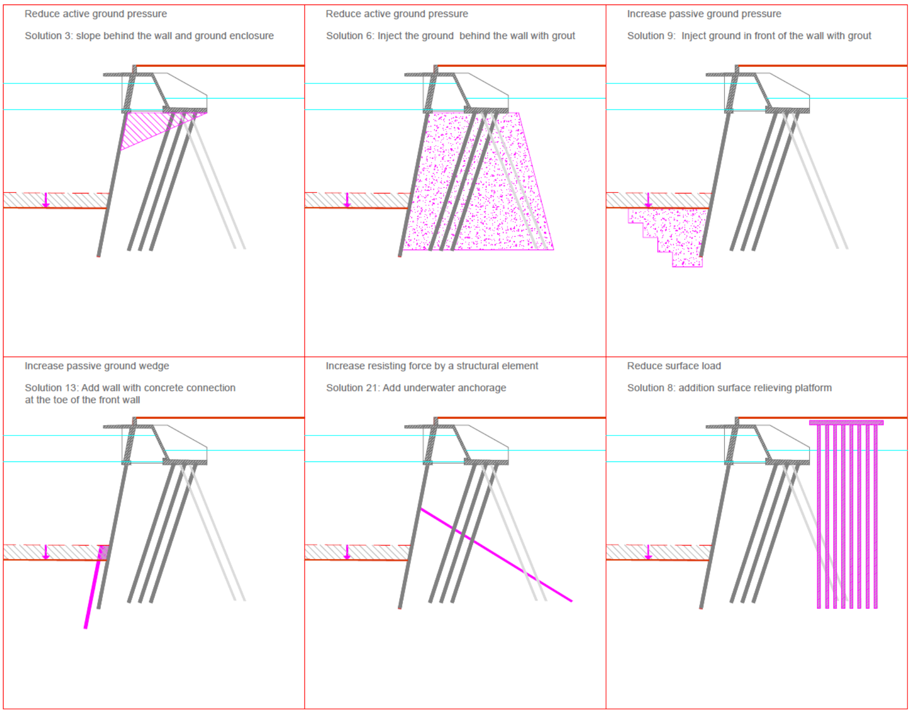


Figure 32 Selected solutions of the pre-selection

7 Bibliography

SBRCURnet. (2014). *Publication 221E "Quay walls, Second Edition".* Rotterdam, The Netherlands: CRC Press/Balkema.

6 Trade-off selection

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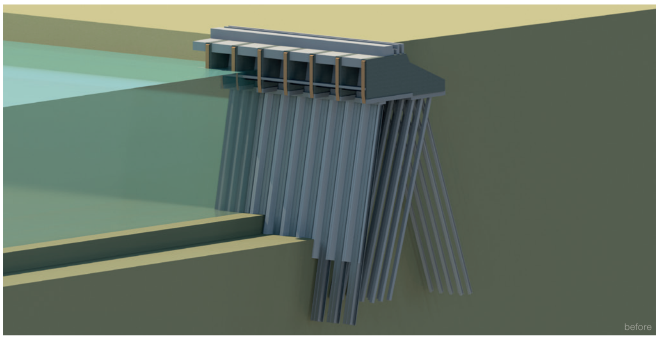

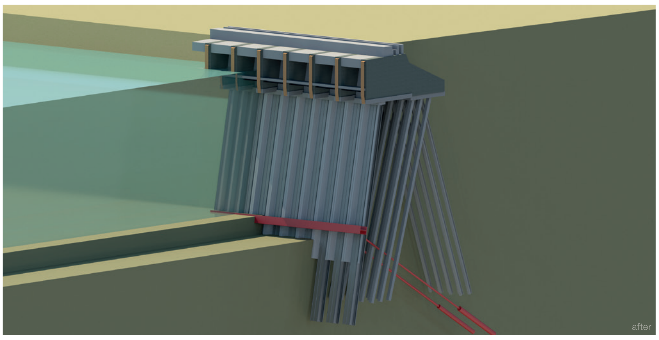

DEEPENING OF AN EXISTING COMBI WALL

TRADE-OFF SELECTION

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Deepening of an Existing Combi Wall

Trade-off Selection

12 June 2017, Rotterdam, The Netherlands

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Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	0.20
Study year:	2016-2017

Summary

The preselection of the solution provides six solution which proceeded to the final trade-off. The substantiation of the solutions is performed in this report and result in the trade-off selection by trade-off criteria, which are:

- Costs;
  - Execution costs;
- Function;
  - Execution time;
  - Lifetime extension;
  - Execution difficulty;
- Technical requirements;
  - Safety factor increasing effect;
  - Bending moment reduction effect;
  - Piping/ insufficient intermediate pilling depth prevention.

According to the score of the solutions by the trade-off criteria the following ranking of the solutions is arranged.

1. Additional underwater anchorage;
2. Additional wall with concrete connection in the toe of the current wall;
3. Grout injection behind the retaining wall;
4. Grout injection at the toe of the retaining wall;
5. Additional high relieving platform;
6. Excavation below the relieving floor.

The additional underwater anchorage is preferential because of the reduction of the maximum stresses of the front wall, the equal safety factor and the execution method without much hindrance.

Besides of the underwater anchorage, the additional sheet pile wall is also preferential, because the safety factor increases, the maximum stress remain equal and the execution method is without much hindrance. The additional sheet pile wall scored more or less the same as the underwater anchorage in the trade-off matrix.

However the construction costs of the additional underwater anchorage are expected to be lower compared to the additional sheet pile wall, so the value of the underwater anchorage is higher related to the additional sheet pile wall. For that reason, the underwater anchorage is determined as most preferential solution.

Table of contents

1 Introduction

This document is made according to the primary question of the graduation thesis: “What is the most optimal solution for a combi-wall in respect to deepening the construction depth 2,8 meters in front of the existing combi-wall structure, without reducing the current reliability?”.

The most optimal solutions are selected in the pre-selection by the minimum criteria. The pre-selection is checked and approved by the expert of the Port of Rotterdam Authority. The solutions which are selected in the pre-selection are:

- Excavation below the relieving floor;
- Grout injection behind the retaining wall;
- Grout injection at the toe of the retaining wall;
- Additional wall with concrete connection in the toe of the current wall;
- Additional underwater anchorage;
- Additional high relieving platform.

The solutions of the pre-selection are work out in the same way in the following chapters. The solutions are worked out as the following aspects:

1. Design description;
2. Construction;
    2.1. Dimensions;
    2.2. Parameters;
    2.3. Plaxis modulation;
    2.4. Plaxis results;
3. Execution difficulty;
4. Time;
    4.1. Downtime;
    4.2. Lifetime extension;
5. Costs;
6. Summary.

The solutions are compared and ranked in a trade-off matrix The main criteria of the final trade-off matrix are:

1. Safety factor increasing;
2. Bending moment reduction;
3. Piping prevention;
4. Execution risk;
5. Downtime/hinder;
6. Lifetime extension;
7. Costs.

2 Excavation below the relieving floor

2.1 Design description

This solutions is about the excavation of the soil below the relieving floor. This soil is removed by an underwater dredger. The excavation of the soil is removed to relieve the pressure to the front wall. The solutions is underpinned in consultation with expert Maurice Krul of W. Smit Dive- & salvage company. See Figure 1 for the illustration of solution 3.

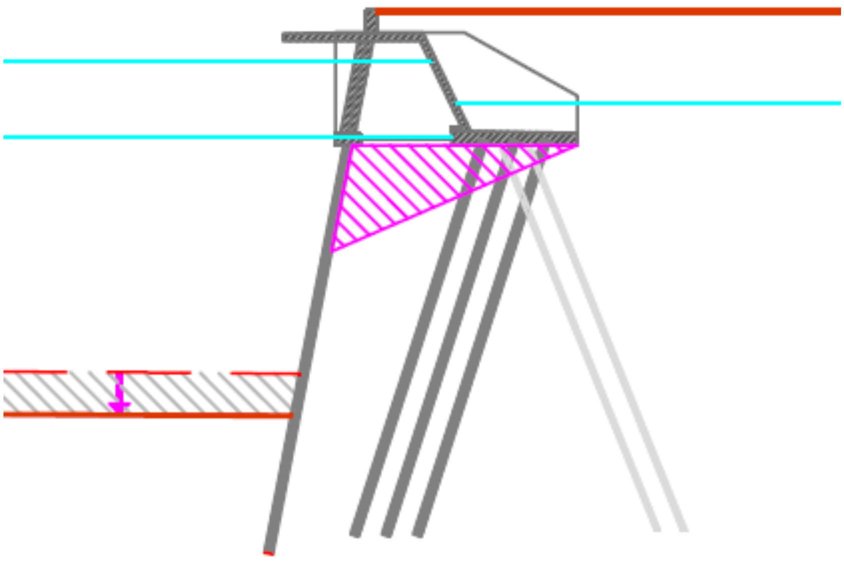


Figure 1 principle of the excavation below the relieving floor (Schutte, Inventory and preselection, 2017)

2.2 Construction

This solution does not provides additional materials. The slope under the relieving structure consists of the digging out the soil behind the wall and under the superstructure.

2.2.1 Dimensions

The slope under the structure is modulated in the range of internal friction angle, which is the parameter φ. The clay layer of the structure is representative of the total soil as lowest value, which concludes a slope of the digging off of 25°.

The height (x) of the digging out is the angle of internal friction times the length of the front wall to the end of the relieving platform (l).

$$x = \tan(\varphi) * l = \tan(25) * 11$$

$$x = 5{,}1\ m$$

The ground is been dig off until -7,1 m in a slope of 25 ° to the end of the relieving platform. Which concludes a total volume material which need to be digging out of 0.5*5.1*11*1=28 m3 per meter quay.

2.2.2 Parameters

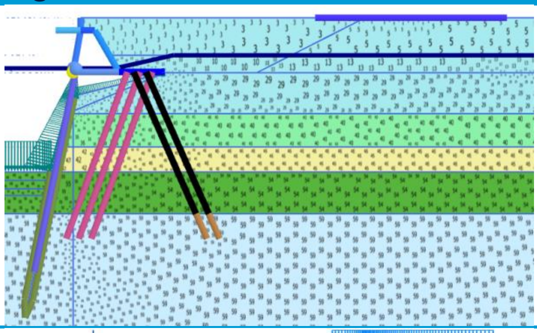
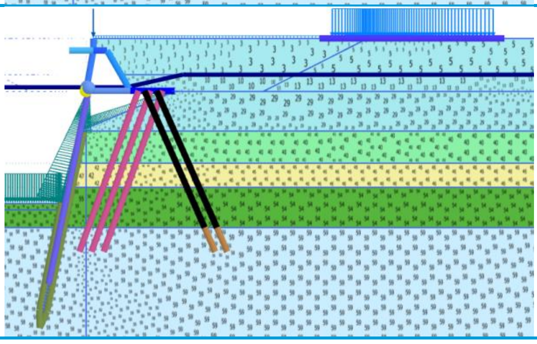
The parameters of the plaxis model are the same as the reference model. The digging of is modulated in two phases into plaxis. These two phases are figured in the next paragraph 1.2.3.

2.2.3 Plaxis modulation

The modulation of the solution into plaxis is done in the made reference plaxis model. The load combination 4 is the dominant load combination of this research, which is determined in the structural engineering of the reference model. The solutions are also modulated into plaxis with load combination 4. These load are switched of in the plaxis model because of the

The modulation of this solution is done by deactivation the several soil elements in the angle of internal friction. The soil digging off is modulated into two stages as shown in the Table 1 below.

Table 1 plaxis phases and schematisation of the modulation solution (Schutte, Plaxis calculations deepening solutions, 2017)

Plaxis model	Figure
Before deepening	
After deepening after digging off	

2.2.4 Plaxis results

The results of the plaxis calculation of the solution are compared to the reference model before deepening. The dominant load combination 4 is used for this compare. The critical structure elements and the most common values of the plaxis calculation are shown in the Table 2.

Table 2 result of the excavation compared to quay wall before deepening

Phase	Annotation	Load combination 4 before deepening	Excavation below the relieving floor	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.168	-6.26%
Structural				
Bending moment front wall	kNm/m	935	1131	21.00%
Shear force front wall	kN/m	229	246	7.67%
Normal force front wall	kN/m	799	789	-1.25%
Normal force bearing pile 3	kN	595	559	-6.05%
Bending moment bearing pile 3	kNm/m	92	121	31.52%
Normal force tension pile 1	kN	95	123	29.47%
ULS				
Deformations x top quay wall	m	0.13	0.19	46.15%

The solution does not provide an improvement of the piping prevention.

2.3 Execution methods

The execution of the digging out the materials below the relieving floor is uncommon. The normal execution methods cannot be done because of the superstructure and the pile configuration. Example of the common execution methods are removal of the material by divers with underwater suction equipment, airlifting of the materials or excavation with an excavator. The materials under the structure need to be removed by a suction pile which can be applied behind the relieving structure. The materials have to mixed with water to be extracted by suction. This method and specify equipment is uncommon, so this solution is ranked as new method or equipment in the trade-off matrix.

2.4 Time

Time is important for the clients of the port of Rotterdam because the turnover depends on the time of the availability of the berthing facilities and the surface of the quay structure. The aspect time is deviated into two subjects downtime of the berthing facilities and the surface of the quay structure and the lifetime extension of the total quay structure.

2.4.1 Downtime

The downtime of the berthing facilities and the surface of the quay structure depends on the execution time. That time is estimated by a standard pump capacity of 15 m3 per hour. This capacity is not the exact capacity because of the difficult execution method, so the capacity is halved. The replacement of the nozzles and suction pile provides extra decrease of the capacity. The total assumed production is 2 m3 per hour. The berth facilities does not have downtime because of the execution method of the sucking out of the material from behind the structure. The execution time of the solutions is the downtime of the surface of the quay structure.

The estimated execution time is 28/2=14 hours per m1 quay structure. That 14 hours does not include preparations and discharge, so the downtime per m1 quay structure is assumed as 3 day per m1 quay structure.

2.4.2 Lifetime extension

The lifetime extension of this solution depends on the structure elements of the quay wall structure. The front wall is more exposed to corrosion, so the front wall will degrade more. The limited lifetime of the front wall provides the estimation of the lifetime extension of 15-20 years.

2.5 Costs

The expected costs per meter are arranged with the following assumptions:

- Production 4 days per m1 quay;
- Unknown execution method provides specialized equipment, which is assumed of €5,000 per including discharge, additional costs and personnel.

The total costs per meter of this solution is estimated as shown in Table 3 .

Table 3 cost estimation of the excavation solution

<table>
<tr><th>Excavation below the relieving floor</th><th>Amount</th><th>Unit price</th><th>Price</th><th>Uncertainty factor</th><th>total price</th></tr>
<tr><td>Materials</td><td>m3</td><td>€</td><td>€</td><td rowspan="4">1.35</td><td rowspan="4">€ 27,000.00</td></tr>
<tr><td>-</td><td></td><td></td><td>€ 0.00</td></tr>
<tr><td>Equipment</td><td>days</td><td>€</td><td>€</td></tr>
<tr><td>special equipment</td><td>4.00</td><td>€ 5,000.00</td><td>€ 20,000.00</td></tr>
</table>

2.6 Summary

The summary of important specifications of the solutions, excavation below the relieving floor, is displayed in Table 4.

Table 4 summary of the specifications of the excavation solution

Criteria	Description or value
Costs per meter	€ 27,000.00
Execution time	5.5 hours per 1 m1 quay
Execution difficulty	New method or equipment
Lifetime extension	15-20 years
Plaxis results safety factor	1.168
Plaxis results bending moment	1131 kNm
Piping prevention	No influence

3 Grout injection behind the retaining wall

3.1 Design description

This solution is based on the executed deepening project in Yokohama, Japan. The ground behind the structures is injected with grout because of the earthquake resistance of the overall structure with the grout injection. The grout mixture is injected with high pressure and replaces the soil. See Figure 2 for the illustration of solution of the grout injection behind the retaining wall.

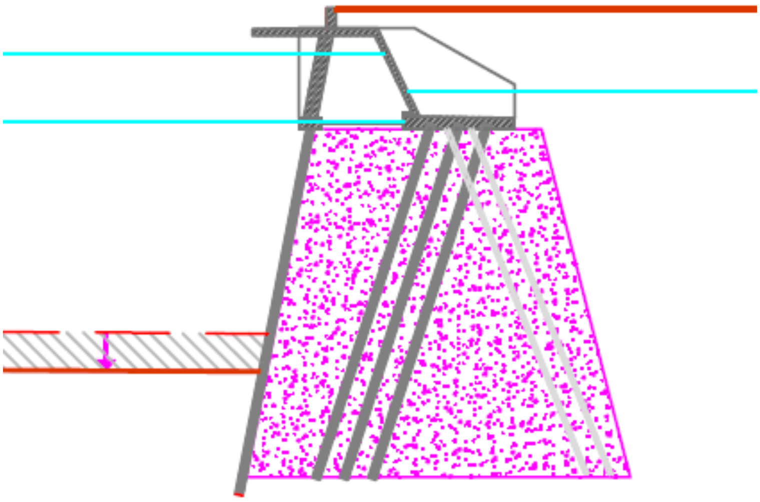


Figure 2 solution 6 (Schutte, Inventory and preselection, 2017)

3.2 Construction

The parameters and the implementation of the grout into the soil is arranged in consultation with expert Rob Selhorst.

3.2.1 Dimensions

The soil of the structure is replaced by grout in the plaxis model. That grout implemented into the model in 5 steps, so the grout can settle. The grout injection is add to the structure from the front wall to the end of the relieving structure. That total area from below the relieving structure to the bottom of the bearing pile is injected with grout injection.

3.2.2 Parameters

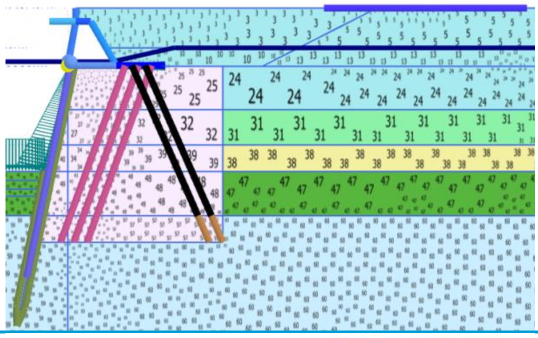
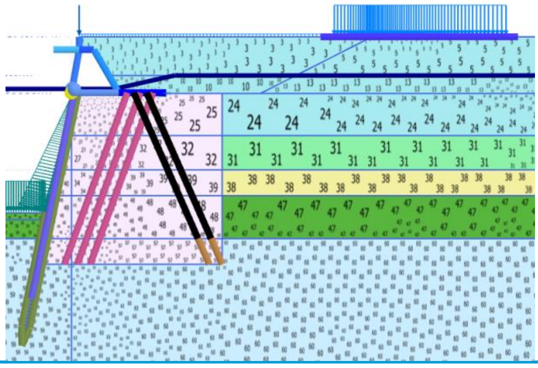
The most common grout inject is Supergrout 70 because of the high strength and the workability of the mixture. According to the material sheet of Supergrout 70 (Grouttech, 2017), the following parameters assumed:

- Strength class K50;
- $42\ N/mm^2$compressive strength after 1 day;
- $E = 22{,}250 + 250 * 42 = 32{,}750\ N/mm^2 = 32{,}750{,}000\ kN/m^2$;
- Strenght application of 50% grout and 50% soil because of the application method

3.2.3 Plaxis modulation

The modulation into plaxis is arrange in cooperation with Henk Brassinga. The parameters are as the parameters of Grouttech 70. The total volume of the soil under the relieving floor and the tension piles are replaced by grout. The plaxis model, with the application of the grout injection, before deepening and after deepening is shown in Table 5.

Table 5 plaxis phases and schematisation of the grout injection behind the wall

Plaxis model	Figure
Before deepening	
After deepening after digging off	

3.2.4 Plaxis results

The results of the plaxis calculation of the solution are compared to the reference model before deepening. The critical structure elements and the most common values of the plaxis calculation are shown in the Table 6.

Table 6 the results of the structural assessment of the solutions grout inject behind the retaining wall

Phase	Annotation	Load combination 4 before deepening	Grout injection behind the retaining wall	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.36	9.15%
Structural				
Bending moment front wall	kNm/m	935	911	-2.59%
Shear force front wall	kN/m	229	293	28.33%
Normal force front wall	kN/m	799	709	-11.32%
Normal force bearing pile 3	kN	595	493	-17.14%
Bending moment bearing pile 3	kNm/m	92	88	-4.35%
Normal force tension pile 1	kN	95	95	0.00%
ULS				
Deformations x top quay wall	m	0.13	0.19	46.15%

The grout injection of the grout provides a big improvement of the piping prevention because the soil cannot washout in the grout, so piping and washout cannot arise.

3.3 Execution method

Grout inject in obstacle free spaces are very common. This method consist of a concrete pump, nozzle and a specialized team. The execution method under the relieving floor is a challenge. The biggest challenge is the approval if the mixture is in place, nowadays it is difficult to verify where the mixture in placed.

3.4 Time

Time is important for the clients of the port of Rotterdam because the turnover depends on the time of the availability of the berthing facilities and the surface of the quay structure. The aspect time is deviated into two subjects downtime of the berthing facilities and the surface of the quay structure and the lifetime extension of the total quay structure.

3.4.1 Execution time

The production of grout injection is in common circumstances and in object free soil about 30 until 50 m3 per day. The execution of the grout inject under the relieving floor is not common and the not object free, so the production is halved compared to the common execution method. This production is validated by Third-party validation by market parties.

The assumed production is 15 m3 per day, the amount of grout is more or less assumed of 80 m3, so the execution time per 1 meter quay is 5 days. This occurs in a lot of down time because of the use of the space at the quay side.

3.4.2 Lifetime extension

The replacement of soil to grout provides an improvement of the soil behind the retaining wall. That structure The estimation of the lifetime extension of 15-50 years, because of the uncertainty of the grout injection application.

3.5 Costs

The expected costs per meter are arranged with the following assumptions:

- 50 m3 per meter quay structure;
- Consumption= 1.6 ton/m3, 80 ton;
- €1,155 per ton;
- Production of 15 m3 per day.

The price of the deepening solution is estimated per m1 quay structure. The estimation is arranged with an uncertainty factor of 1,3. That uncertainty factor increase the price with 30%, so the general cost, profit and risk of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. The total cost per meter is excluding dredging costs, scout protection costs, engineering costs, project management of the Port of Rotterdam and costs for additional project specific adjustments. That global cost estimation of the solutions is shown in Table 7.

Table 7 cost estimation of the grout injection behind the wall solution

<table>
<tr><th>Grout injection behind the retaining wall</th><th>Amount</th><th>Unit price</th><th>Price</th><th>Uncertainty factor</th><th>Total price</th></tr>
<tr><td>Materials</td><td>ton</td><td>€</td><td>€</td><td rowspan="6">1.35</td><td rowspan="6">€ 164,052.00</td></tr>
<tr><td>Grout</td><td>80.00</td><td>€ 1,155.00</td><td>€ 92,400.00</td></tr>
<tr><td>Equipment</td><td>days</td><td>€</td><td>€</td></tr>
<tr><td>Small equipment</td><td>5.33</td><td>€ 160.00</td><td>€ 853.33</td></tr>
<tr><td>Pump team incl pump</td><td>5.33</td><td>€ 800.00</td><td>€ 4,266.67</td></tr>
<tr><td>Dive team</td><td>5.33</td><td>€ 4,500.00</td><td>€ 24,000.00</td></tr>
</table>

3.6 Summary

The summary of important specifications of the solutions, grout injection behind the retaining wall, is shown in Table 8.

Table 8 summary of the grout injection behind the wall solution according to the criteria

Criteria	Description or value
Costs per meter	€ 164,052.00
Execution time	5 days on land
Execution difficulty	New method or equipment
Lifetime extension	15-20 years
Plaxis results safety factor	1.36
Plaxis results bending moment	911
Piping prevention	Better piping prevention

4 Additional high relieving platform

4.1 Design description

The additional high relieving platform structure consist off bearing piles and an relieving platform. These bearing piles and the relieving floor are concrete. The bearing piles are prefab and the relieving floor is cast in situ. These combined structure elements provide an relieving of the surface load to the subsoil and so the relieving of the front wall.

This solution is arranged in consultation with Henk Brassinga. See Figure 3 for the illustration of the additional high relieving platform.

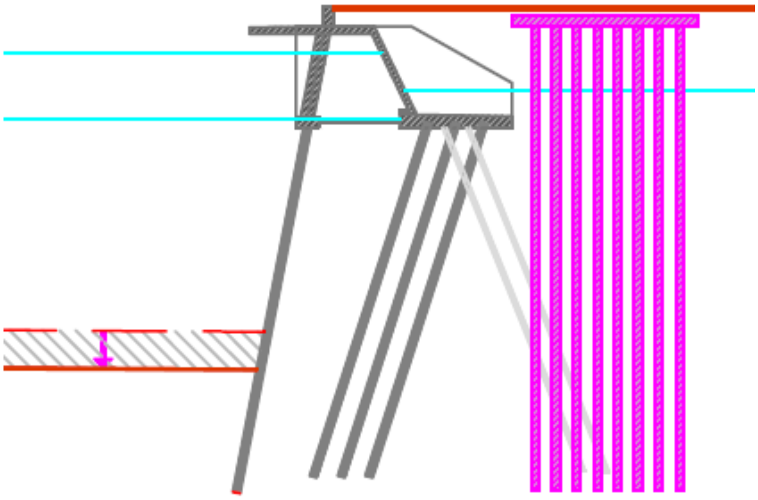


Figure 3 solution 8 (Schutte, Inventory and preselection, 2017)

4.2 Construction

The additional high relieving floor is conservative modulated into Plaxis with common materials and easy executable dimensions.

4.2.1 Dimensions

The dimensions of the high relieving platform are assumed and validated by expert Henk Brassinga. The bearing pile are assumed as 400x400 mm square piles. these piles are very common to pile and are easy to handle. The concrete floor is assumed as 0.8 m thick with reinforcement. This thickness is an experience assumption. The grid of the piles is 3x3 m, so the execution is easy to execute compared to a smaller grid.

To relieve the surface load, the additional high relieving platform is assumed right under the high surface load.

4.2.2 Parameters

The same parameters as the reference model are used for the structural assessment of the effect of the relieving floor solution. The parameters of the floor are:

$\gamma = 24\ kN/m^2$

$E = 29{,}750{,}000\ kN/m^2$

$I = \frac{1}{12} * b * h^3 = \frac{1}{12} * 1 * 0.8^3 = 0.04\ m^4$

$A = b * h = 1 * 0.8 = 0.8\ m^2$

$w = A * \gamma = 0.8 * 24 = 19.2\ kN/m$

$EI = 29{,}750{,}000 * 0.04 = 1{,}190{,}000\ kNm^2/m$

$EA = 29{,}750{,}000 * 0.8 = 23{,}800{,}000\ kN/m$

The parameters of the bearing piles are :

$\gamma = 24\ kN/m^2$

$E = 30.000.000\ \ kN/m^2$

$A = 0.4 * 0.4 = \ 0.16\ m^2$

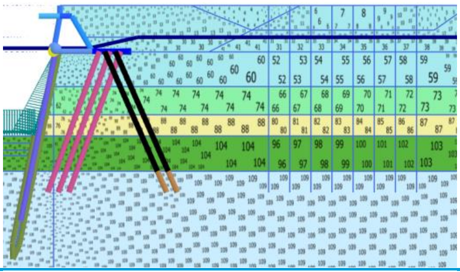
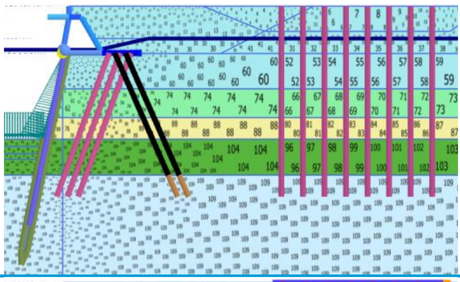
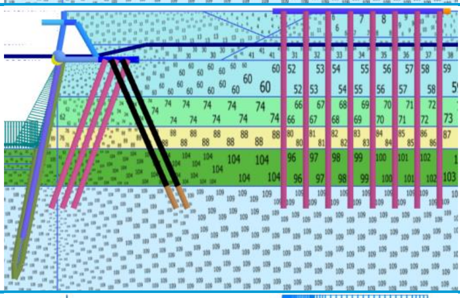
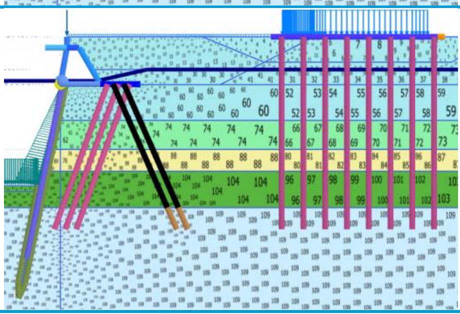
$Lspacing = 3\ m$

4.2.3 Plaxis modulation

The bearing piles are modulated into Plaxis the same as the bearing pile of original relieving structure. These bearing piles are modulated as embedded beam row. The parameters remain the same as the original calculation. The new relieving floor starts 30 meters out of the front of the quay structure. The length of the new relieving floor is 24 meter, so the relieving floor supports the whole surface load.

The construction of the relieving floor is done by excavation a small trench to pile the bearing piles and construct the relieving floor. See Table 9 for the phases of the execution of the solution.

Table 9 modelling steps of the high relieving floor solution

Plaxis model	Figure
Before deepening without loads	
Application bearing pile	
Application relieving floor	
After deepening after deepening	

4.2.4 Plaxis results

The critical structure elements and the most common values of the plaxis calculation are shown in the Table 10.

Table 10 result of the structural assessment of the additional high relieving platform solution

Phase	Annotation	Without adjustments before deepening	Additional high relieving platform	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.19	-4.49%
Structural				
Bending moment front wall	kNm/m	935	924	-1.14%
Shear force front wall	kN/m	229	228	-0.42%
Normal force front wall	kN/m	799	830	3.93%
Normal force bearing pile 3	kN	595	561	-5.71%
Bending moment bearing pile 3	kNm/m	92	90	-2.17%
Normal force tension pile 1	kN	95	74	22.11%
ULS				
Deformations x top quay wall	m	0.13	0.16	23.08%

4.2.5 Execution method

Before the additional high relieving floor can be executed, the building site on the quay surface area need to be cleared. After the construction side is clear, the execution of the relieving floor can be started. First of all the bearing piles are driven onto the thick fundamental sand layer by a specialised pilling foundation machine. After the bearing piles are driven onto the right depth, the formwork for the concrete floor is constructed and the reinforcements placed. Afterwards the concrete is casted.

4.3 Time

Time is important for the clients of the port of Rotterdam because the turnover depends on the time of the availability of the berthing facilities and the surface of the quay structure. The aspect time is deviated into two subjects downtime of the berthing facilities and the surface of the quay structure and the lifetime extension of the total quay structure.

4.3.1 Execution time

The production of the foundation machine is more or less 6 piles per day, so the production per meter quay is 8/3=2.66 piles per meter quay. These piles take more or less 4 hours to be piled. The preparations and the casting of the concrete also take a day, so the estimated execution time is 2 days on land side, with much hinder because of the execution method.

4.3.2 Lifetime extension

The additional wall in front of the retaining wall provides estimated an lifetime extension of 30 until 50 years.

4.4 Costs

The estimation of the costs of the construction of the solution is established trough assumptions, which are presented in the bullet points below:

- The production of 6 bearing piles per day;
- Production of the concreter is 150 m3 per day.
- Bearing pile length 30 meters, grid 3x3 m, 8 piles per 3 meters, so 8/3 pile per meter;
- 21 meter length relieving floor, 0.8 meter thick, so 16.8 m3 concrete per meter quay;
- Foundation machine includes the personnel, pile driving equipment and other small equipment.

The price of the deepening solution is estimated per m1 quay structure. The estimation is arranged with an uncertainty factor of 1,3. That uncertainty factor increase the price with 30%, so the general cost, profit and risk of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. The total cost per meter is excluding dredging costs, scout protection costs, engineering costs, project management of the Port of Rotterdam and costs for additional project specific adjustments. That global cost estimation of the solution is shown in Table 11.

Table 11 cost estimation of the high relieving platform

Additional high relieving platform	Amount	Unit price	Price	Uncertainty factor	Total price
Materials				1.35	€ 20,772.00
Bearing piles 400*400	77.33 m1	€ 50.00	€ 3,866.67		
Reinforced concrete C25/30	16.80 m3	€ 150.00	€ 2,520.00		
Equipment					
Foundation machine	**days**	**€**	**€**		
Concrete pump	2.00	€ 3,500.00	€ 7,000.00		
Concrete formwork	2.00	€ 500.00	€ 1,000.00		

4.5 Summary

The summary of important specifications of the solutions, additional high relieving platform, is displayed in Table 12.

Table 12 summary of the additional high relieving floor solution according to the criteria

Criteria	Description or value
Costs per meter	€ 20,772.00
Execution time	2 days on land
Execution difficulty	Common method or equipment
Lifetime extension	15-20 years
Plaxis results safety factor	1.19
Plaxis results bending moment	924
Plaxis results normal force tension pile 1	74

5 Grout injection at the toe of the retaining wall

5.1 Design description

This solution is based on the executed deepening of the Waalhaven, Rotterdam. The ground in front of the structure is replaced by grout injection. This grout injection is executed before the deepening works and in segments. The grout replaces the soil and provides a solid structure.

This solution is arranged in consultation with grout injection expert Rob Selhorst. See Figure 4 for the illustration of grout injection at the toe of the retaining wall.

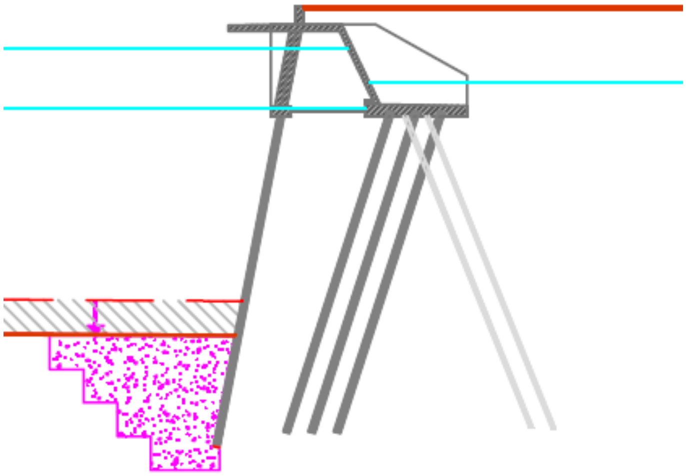


Figure 4 principle cross section of the grout injection at the toe of the structure (Schutte, Inventory and preselection, 2017)

5.2 Construction

The grout injection at the toe of the structure is arranged with the same assumptions as the grout injection behind the wall. The dimensions and the parameters are described below.

5.2.1 Dimensions

The ground in front of the structure is replaced by a grout injection mixture. This mixture replace the first 2.5 meters at the toe of the structure from -16 untill -28. The effect of the grout injection is determined by this modulation.

5.2.2 Parameters

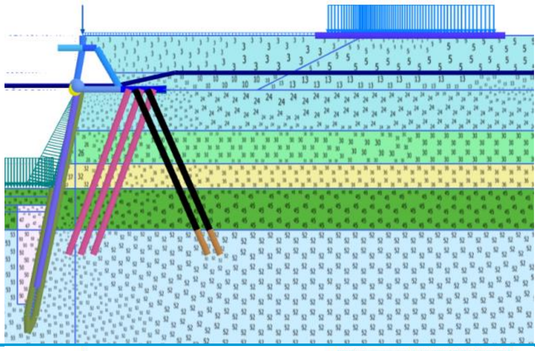
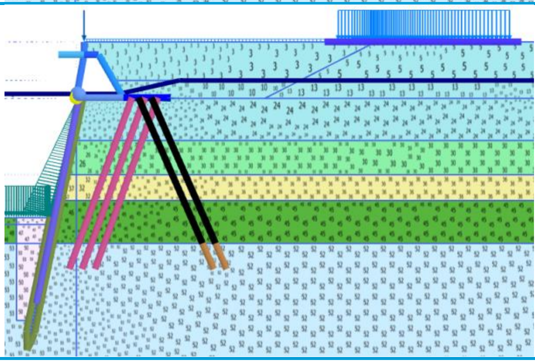
The most common grout inject is Supergrout 70 because of the high strength and the workability of the mixture. According to the material sheet of Supergrout 70 (Grouttech, 2017), the following parameters assumed:

- Strength class K50;
- $42\ N/mm^2$compressive strength after 1 day;
- $E = 22{,}250 + 250 * 42 = 32{,}750\ N/mm^2 = 32{,}750{,}000\ KN/m^2$;
- Strenght application of 50% grout and 50% soil because of the application method.

5.2.3 Plaxis modulation

The modulation into plaxis is arrange in cooperation with Henk Brassinga. The parameters are as the parameters of Grouttech 70. The total volume of the soil under the relieving floor and the tension piles are replaced by grout. The plaxis model, with the application of the grout injection, before deepening and after deepening is shown in Table 5.

Table 13 modelling of the grout injection at the toe of the wall solution

Plaxis model	Figure
Application of the grout injection before deepening	
After deepening after digging off	

5.2.4 Plaxis results

The critical structure elements and the most common values of the Plaxis calculation are shown in the Table 14.

Table 14 result of the structural assessment of the grout injection at the toe of the retaining wall solution

Phase	Annotation	Load combination 4 before deepening	Grout injection at the toe of the retaining wall	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.337	7.30%
Structural				
Bending moment front wall	kNm/m	935	980	4.78%
Shear force front wall	kN/m	229	268	17.23%
Normal force front wall	kN/m	799	854	6.88%
Normal force bearing pile 3	kN	595	610	2.52%
Bending moment bearing pile 3	kNm/m	92	94	2.17%
Normal force tension pile 1	kN	95	121	27.37%
ULS				
Deformations x top quay wall	m	0.13	0.19	46.15%

This solution provides a big increase of the piping prevention, because of the impermeable specifications of the grout mixture after the hardenings period.

5.3 Execution method

The execution method of the application of grout underwater is more of less the same as above water, which is the common execution method. The application is not done by a grout specialist but by a diver, which is instructed by the grout specialist. To test the right application of the grout tests with a rod.

The application starts with the application of the grout nozzle into the soil. Afterwards the grout mixture is pumped into the nozzle and the nozzle is pulled back out of the soil to make an grout column. The application of the grout column is repeated until the whole area is filled with columns. The testing of the right application is the most challenging part of the complete operation. It is uncertain to declare if the grout injection is applied in the exact right place as planned.

5.4 Time

Time is important for the clients of the port of Rotterdam because the turnover depends on the time of the availability of the berthing facilities and the surface of the quay structure. The aspect time is deviated into two subjects downtime of the berthing facilities and the surface of the quay structure and the lifetime extension of the total quay structure.

5.4.1 Execution time

The production of grout injection is in common circumstances and in object free soil about 30 until 50 ton per day. The execution of the grout inject under the relieving floor is not common and the not object free, so the production is halved compared to the common execution method. The assumed production is 15 ton per day, the amount of grout is more or less assumed of 36 ton, so the execution time per 1 meter quay is 2.4 days. The execution method is on the waterside with less downtime.

5.4.2 Lifetime extension

The replacement of soil to grout provides an improvement of the soil behind the retaining wall. That structure The estimation of the lifetime extension of 15-50 years, because of the uncertainty of the grout injection application.

5.5 Costs

The estimation of the costs of the construction of the solution is established trough assumptions, which are presented in the bullet points below:

- 22.5 m3 per meter quay structure;
- Consumption= 1.6 ton/m3,so 36 ton;
- €1,155 per ton;
- Production of 15 ton per day, .

The price of the deepening solution is estimated per m1 quay structure. The estimation is arranged with an uncertainty factor of 1,3. That uncertainty factor increase the price with 30%, so the general cost, profit and risk of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. The total cost per meter is excluding dredging costs, scout protection costs, engineering costs, project management of the Port of Rotterdam and costs for additional project specific adjustments. That global cost estimation of the solution is shown in Table 15.

Table 15 cost estimation of the grout injection at the toe of the structure

<table>
<tr><th>Grout injection at the toe of the retaining wall</th><th>Amount</th><th>Unit price</th><th>Price</th><th>Uncertainty factor</th><th>total price</th></tr>
<tr><td>Materials</td><td>ton</td><td>€</td><td>€</td><td rowspan="7">1.35</td><td rowspan="7">€ 73,476.72</td></tr>
<tr><td>Grout</td><td>36.00</td><td>€ 1,150.00</td><td>€ 41,400.00</td></tr>
<tr><td>Equipment</td><td>days</td><td>€</td><td>€</td></tr>
<tr><td>Small equipment</td><td>2.40</td><td>€ 160.00</td><td>€ 384.00</td></tr>
<tr><td>Pump team incl pump</td><td>2.40</td><td>€ 768.00</td><td>€ 1,843.20</td></tr>
<tr><td>Dive team incl equipment, vessel and decompresiong tank</td><td>2.40</td><td>€ 4,500.00</td><td>€ 10,800.00</td></tr>
</table>

5.6 Summary

The summary of important specifications of the solutions, grout injection at the toe of the retaining wall is displayed in Table 16.

Table 16 summary of the grout injection at the toe of the structure solution according to the criteria

Criteria	Description or value
Costs per meter	€ 73,480
Execution time	On water, < 2 days
Execution difficulty	Common method or equipment
Lifetime extension	15-20 years
Plaxis results safety factor	1.34
Plaxis results bending moment	980
Plaxis results normal force tension pile 1	95

6 Additional sheet pile wall

6.1 Design description

This solution is based on an extra sheet pile wall in front of the current quay wall structure. The sheet pile will be piled vertical The connection between the current wall and the new wall will be done by concrete so the connection is ground tight and moment fixed.

See Figure 5 for the illustration of the solution of the additional wall.

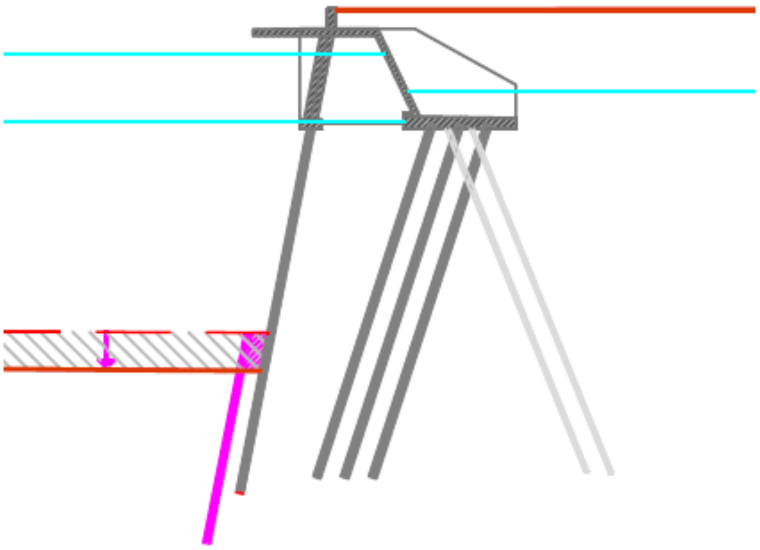


Figure 5 principle cross section of the additional wall with concrete connection (Schutte, Inventory and preselection, 2017)

6.2 Construction

The construction of the additional sheet pile wall is assumed to be executed with common materials and equipment.

6.2.1 Dimensions

The dimensions of the additional sheet pile wall are assumed as AZ26, this is a very common sheet pile, which is much is stock at the suppliers. The AZ26 pile are determined by the parameter of Arcelormittel. (Arcelore Mittel, 2017)

Piling of the additional sheeppile wall is assumed to NAP-32 meters. This the new sheetpile wall penetrate the Circular slip surface of Bischop to improve the overall safety. The sheet piles are assumed to be pilled vertical in case of the technical feasibility of the exection. A concrete connection of the material of the groutinjection is used to connect the new wall to the retaining wall. This connection provides an

6.2.2 Parameters

The parameters of the AZ26 sheet pile wall are shown in Table 17.

Table 17 parameters of the AZ26 sheet piles (Arcelore Mittel, 2017)

	Sectional area (A)	Mass per m	Moment of inertia (I)	Section modulus
	cm^2	kg/m	cm^4	cm^3
Per m of Wall	198,0	155,2	55510	2600

The parameters which are used in the plaxis calculation are shown in the following list:

$E = 210{,}000\ N/mm^2 \ = 210{,}000{,}000\ kN/m^2$

$EA = 210.000.000 * 198/10^4 = 4{,}16\ * 10^6 kN$

$EI = 210.000.000 * 55{,}510/10^8 = \ 0.12 * 10^6 kNm^2$

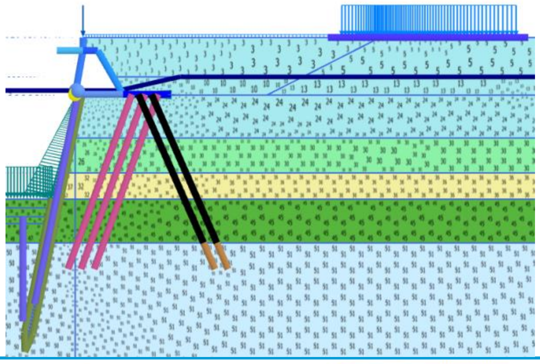
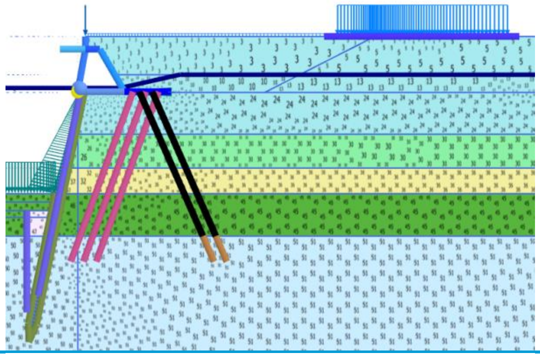
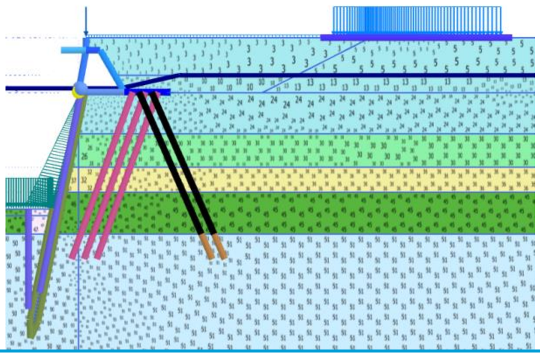
$W\ = 155{,}2 * 9{,}81 = 1520\ N = 1.52\ kN$

$V\ = 0$

6.2.3 Plaxis modulation

The modelling of the additional sheet pile wall solution into Plaxis is shown in Table 18.

Table 18 schematisation of the additional sheet pile wall into Plaxis

Plaxis model	Figure
Application of the front wall	
Application of the grout connection	
After deepening after digging off	

6.2.4 Plaxis results

The critical structure elements and the most common values of the plaxis calculation are shown in the Table 19.

Table 19 result of the Plaxis calculation of the additional sheet pile wall

Phase	Annotation	Without adjustments before deepening	Additional wall with concrete connection in the toe of the current wall	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.29	3.53%
Structural				
Maximum stress front wall	N/mm2	212	216	1.85%
Bending moment front wall	kNm/m	953	967	1.43%
Normal force at maximum BM	kN/m	737	771	4.61%
Shear force front wall	kN/m	229	315	37.82%
Normal force bearing pile 3	kN	595	602	1.18%
Bending moment bearing pile 3	kNm/m	92	94	2.17%
Normal force tension pile 1	kN	95	116	22.11%
ULS				
Deformations x top quay wall	m	0.13	0.15	15.38%

This solution improve the piping prevention, because of the increase of the additional length of the piping length.

6.3 Execution method

The normal execution of the application sheet pile wall in above water. The execution underwater can be executed with the same equipment, but the equipment need the be adjusted. This adjustment contains on the watertight of the drilling machine. Besides of the adjustment of the equipment , does the method also be adjusted. The guidance of the single sheet piles cannot be done underwater, so the guidance frame have to be suitable to guide the single sheet pile.

Sheet pile are installed by a crane with a vibrating hammer. This combination is placed on a pontoon because of the execution location on the water side. The vibrating hammer is connected to the single sheet piles after the sheet pile is lifted vertical. Afterwards the sheet pile is placed in position and drilled to the contract depth. The interlocking of the sheet pile is checked by a lock guidance during the execution. If the sheet pile are piled on the right depth, the vibrating hammer is disconnected and the grout is injected between the existing pile and the new piles by divers.

6.4 Time

Time is important for the clients of the port of Rotterdam because the turnover depends on the time of the availability of the berthing facilities and the surface of the quay structure. The aspect time is deviated into two subjects downtime of the berthing facilities and the surface of the quay structure and the lifetime extension of the total quay structure.

6.4.1 Execution time

The execution time underwater is halve of the execution time above water, which is validated by Hans Schutte of Dimco. The production is above water 8 meters per day, so underwater 4 meters per day. The execution time per m^1 quay is 0,25 day, so 2 hours per m^1.

6.4.2 Lifetime extension

The replacement of additional sheet pile provides a estimation of the lifetime extension at least 50 years. It should be noted that the condition of the existing wall and the degradation of the additional wall could result in a lower lifetime extension.

6.5 Costs

The estimation of the costs of the construction of the solution is established trough assumptions, which are presented in the bullet points below:

- Steel price of €975 per ton;
- Grout injection price of €975 per m^3;
- Price pontoon including crane, vibrating hammer and additional small equipment €6,000 per day;
- Grout injection pump price including team of € 768.00;
- Dive team price incl small equipment, vessel and decompression tank of € 4,500 per day;
- Production of 4 meters per day including dive activities and the application of the grout;
- 14 m^1 sheetpile, with a weight of 155 kg per meter;
- 8 m^3 grout per per m^1.

The price of the deepening solution is estimated per m1 quay structure. The estimation is arranged with an uncertainty factor of 1,35. That uncertainty factor increase the price with 35%, so the general cost, profit and risk of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. That global unit price estimation of the solutions is shown in Table 20.

Table 20 cost estimation of the additional sheet pile wall

Additional sheet pile wall	Amount	Unit price	Price	Uncertainty factor	total price
Materials		**€**	**€**	1.35	€ 19,133.21
AZ 26	2.17 ton	€ 975.00	€ 2,115.75		
Grout injection	8.00 m3	€ 1,155.00	€ 9,240.00		
Equipment	**days**	**€**	**€**		
Pontoon incl crane, hammer and small equipment	0.25	€ 6,000.00	€ 1,500.00		
Pump team incl grout pump	0.25	€ 768.00	€ 192.00		
Dive team incl equipment, vessel and decompression tank	0.25	€ 4,500.00	€ 1,125.00		

6.6 Summary

The summary of important specifications of the solutions, additional wall with concrete connection in the toe of the current wall is displayed in table X.

Table 21 summary of the additional wall with concrete connection solution according to the criteria

Criteria	Description or value
Costs per meter	€ 19,130
Execution time	On water, <2 days
Execution difficulty	Adjusted common method or equipment
Lifetime extension	15-20 years
Plaxis results safety factor	1.29
Plaxis results bending moment	967
Plaxis results normal force tension pile 1	116

7 Additional underwater anchorage

7.1 Design description

This solution is based on the executed deepening project of Ravenna, Italy. The solution consist of an additional underwater anchorage. This solution add the anchorage at the current port bed level (Lenzi, et al., 2011).

This solution is arranged in consultation with expert Marinus de Heus. See Figure 6 for the illustration of underwater anchorage.

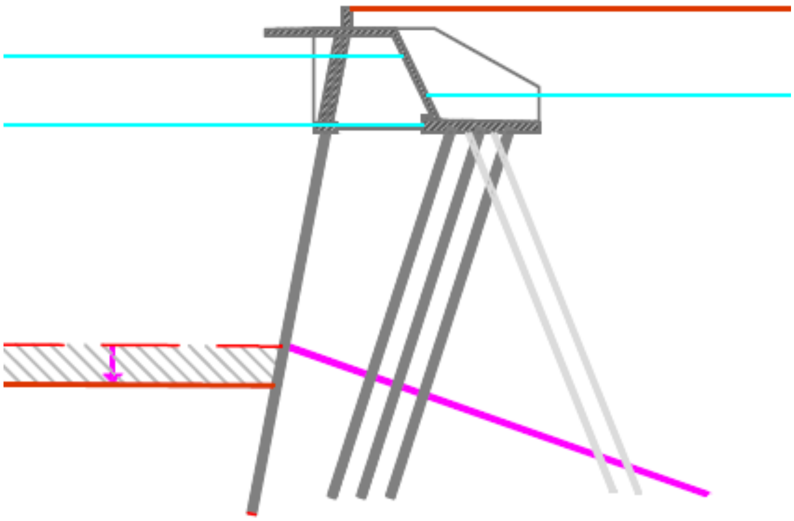


Figure 6 principle cross section of additional underwater anchorage (Schutte, Inventory and preselection, 2017)

7.2 Construction

The construction of the underwater anchorage is assumed and validated by an anchorage calculation of Jetmix.

7.2.1 Dimensions

The application of the anchorage is assumed to be on the maximum bending moment of the front wall to obtain the maximum bending moment reduction. The position of the maximum bending moment is based on the reference Plaxis calculation. The bending moment max is at NAP -14.00 m. At that depth the wall is not exposed to oxygen, so the wall is not much degradation. The anchorage length is assumed as the maximum of 10 meters.

The force of the anchorage is divided and spread out by a purlin. The anchorage bars are connected to this purlin and go through the wall at the intermediate piles. For that reason is the space between 2 anchorage bars, 3 meters.

The anchorage is applied in an angle of 30°, so the additional normal force is less as possible. Beside of the angle is the anchorage also pre-tensioned with a force of 750 kn. This pretension reduces the deformation of the anchorage grout body after application. So the retaining wall does not deform much more. The application does not provide an contra deformation of the wall, so the wall does not deform back to the ground bodies.

Before the drilling of the hole for the application of the anchorage bars could the ground behind the wall be injected with soil improvement to prevent the soil to wash-out.

The assumption of the anchorage are validated by a third-party company. The validation of the anchorage assumption is positive, the chosen anchorage type is suitable for the situation. See Appendix A for the validation anchorage calculation by the third-party.

7.2.2 Parameters

The parameters of the underwater anchorage as determined in the Plaxis model. The anchorage is applied without pretension, so the anchorage force is determined. Afterwards the revealing anchorage is chosen with a conservative approach to be sure the anchorage not fail. The anchorage type is the Jetmix 7 anchorage with diameter of 60,3 mm and a thickness of 16,0 mm. the maximum force of the anchorage is 986 kN. In the Plaxis model is the anchorage pre-tensioned with 75% of the maximum anchor force. For that reason, the pre-tension force is 750 kN.

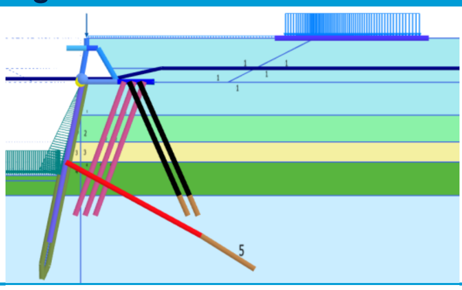
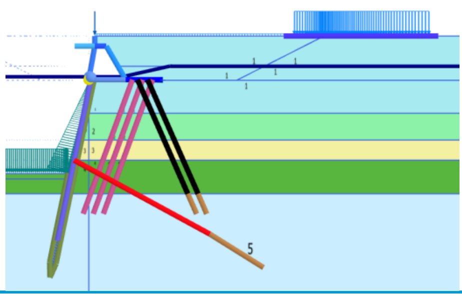
The modelling of the anchorage is done by an node-to-node anchor with an embedded beam row end piece. The influence of the ground on the anchorage is zero and the space between the 2 anchorage is calculated. The following parameters are the basis of the additional underwater anchorage in the Plaxis calculation:

- Diameter grout is 0.3 meters;
- $EA = 340{,}000\ kN$;
- $L_{spacing} = 3.0\ m$;
- Grout length 10 meters.

7.2.3 Plaxis modulation

The modelling of the underwater anchorage solution into Plaxis is shown in Table 22

Table 22 schematisation of the modelling of the underwater anchorage

Plaxis model	Figure
Application of the underwater anchorage	
After deepening with the underwater anchorage applied	

7.2.4 Plaxis results

The critical structure elements and the most common values of the plaxis calculation are shown in the Table 23.

Table 23 result of the structural assessment of the underwater anchorage

Phase	Annotation	Without adjustments before deepening	Additional underwater anchorage	Deviation
ULS				
Geotechnical				
Safety factor	-	1.246	1.234	-0.96%
Structural				
Maximum stress front wall	N/mm2	212	197	-6.77%
Bending moment front wall	kNm/m	953	882	-7.42%
Normal force at maximum BM	kN/m	737	718	-2.58%
Shear force front wall	kN/m	229	225	-1.54%
Normal force bearing pile 3	kN	595	576	-3.19%
Bending moment bearing pile 3	kNm/m	92	92	0.00%
Normal force tension pile 1	kN	95	110	15.79%
ULS				
Deformations x top quay wall	m	0.13	0.17	30.77%

The additional underwater anchorage does not influence the piping prevention.

7.3 Execution method

On behalf of the application of the grout injection anchorage the first anchor rod is placed in the machine and the drill chuck is attached. The drill master positions the drilling machine in front of the drilling hole in the existing structure. The positioning of the drilling machine in front of the drilling is guided by a diver. The angle of the anchorage is checked by the drill master with a spirit level. Afterwards the drillmaster start the engines of the pump and the drill engine as the anchorage is started to apply the anchorage.

During the application of the anchorage, the drilling hole is being keep opened by the sluice of water or a thin grout mixture. The water is used during common circumstances and the grout mixture is used if the drilling hole can became instable or the grout injection opening shuts. The grout is being injected by turning movements of the anchor rods. The grout is being injected as 5 to 20 Bar.

In advance of the completed application of the anchorage, that particular anchorage is clasped in the bottom of the drilling machine by the hydraulic fastener installation. The drill master reload the drilling machine and applies the next anchorage. The reloading of the drilling machine should be done automatically underwater. The process of application and the reloading of the anchorage is continued until the anchorage is on the right depth.

After application of the grout injection anchorage need the hardening time of 14 days to be considered before tensioning the anchors. The anchorage should be tensioned by a pretension installation and is guided by a diver.

7.4 Time

The time aspect is deviated into two parts, the execution time and the lifetime extension estimation.

7.4.1 Execution time

The production and so the downtime per meter is estimated and this production is third-party validated by market parties. The estimated execution time is 0.5 day per anchorage, so the production per meter is 0.25 days, so more of less 2 hours.

The application of the underwater anchorage is flexible because of the simplicity of the drilling machine. The drilling machine does not provides much space on the water side. So the downtime of the solutions in minimum.

7.4.2 Lifetime extension

The lifetime extension of this solution depends on the soil circumstance and the corrosion of the additional anchorage. That anchorage is add to the best part of the front wall, which is not much degraded. For that reasons, the lifetime extension is approximately 30 to 50 years.

7.5 Costs

The estimation of the costs of the construction of the solution is established trough assumptions, which are presented in the bullet points below:

- The production of 90 m1 anchorage per day;
- Additional cost to modify the equipment to work underwater of €2500 per day;
- Anchorage unit price includes couplings, bolts, drill chuck and the anchorage bar;
- Grout injection pump includes a storage silo, grout mixer and small jet equipment;
- 22 meters anchorage and 7 meters grout;
- 0,5 tons of grout per 1 meter grout injection;
- Prefab purlin for the connection is €1500 per meter quay
- Dive team includes 4 divers, decompression tank, support vessel and small equipment;
- 4 anchorages per 10 meter, so 1 per 2,5 meters, so 0.4 anchorage per m1 quay.

The price of the deepening solution is estimated per m1 quay structure. The estimation is arranged with an uncertainty factor of 1,35. That uncertainty factor increase the price with 35%, so the general cost, profit and risk of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. That global unit price estimation of the solutions is shown in Table 24.

Table 24 cost estimation of the additional underwater anchorage

Additional underwater anchorage	Amount	Unit price	Price	Uncertainty factor	total price
Materials	**ton**	**€/ton**	**€**	1.35	€ 14,215.50
Anchorage bar	2.16	€ 1,750.00	€ 3,780.00		
Grout	2.00	€ 250.00	€ 500.00		
Prefab purlin	1.00	€ 1,500.00	€ 1,500.00		
Equipment	**days**	**€/day**	**€**		
Drilling machine	0.50	€ 2,500.00	€ 1,250.00		
Dive team	0.50	€ 4,500.00	€ 2,250.00		
Modification drilling equipment	0.50	€ 2,500.00	€ 1,250.00		

7.6 Summary

The summary of important specifications of the solutions, additional underwater anchorage is displayed in Table 25.

Table 25 summary of the additional underwater anchorage solution according to the criteria

Criteria	Description or value
Costs per meter	€ 14,215.50
Execution time	On water, <2 days
Execution difficulty	Adjusted common method or equipment
Lifetime extension	50 years
Plaxis results safety factor	1.23
Plaxis results bending moment	882
Plaxis results normal force tension pile 1	110

8 Trade-off criteria

The chapters above describes the solutions in more detail. This chapter measures out the most feasible solution by a trade-off matrix. Three different categories of criteria are determined: Costs, Function and Plaxis results. The seven criteria of this trade/off matrix arranged by category are:

- Costs;
    - Execution costs;
- Function;
    - Execution time;
    - Lifetime extension;
    - Execution difficulty;
- Technical requirements;
    - Safety factor increasing effect;
    - Bending moment reduction effect;
    - Piping/ insufficient intermediate pilling depth prevention

8.1 Global execution costs

The total costs of the upgrade of the current quay wall structure is the first criteria of the trade-off. The costs of every individual solution is estimated in an global costs estimation. The estimations are arranged and are approved by the cost accountants of the Port of Rotterdam.

The cost of the solutions is implemented in the trade-off as the determination of the value of the solutions.

8.2 Functional

The functional category is separated in execution time, execution difficulty and lifetime extension.

8.2.1 Downtime/hinder

Downtime is important for the clients of the Port of Rotterdam. The time aspect is ranked per meter quay. The fastest solution is faster 2 days execution time and is executed on the water side. The slowest solution and the solution which provides much space on the quay land side are the solutions with the lowest score. The scores in between is deviated in steps of above the 2 days execution time and the location of the execution. The score and description of the score are shown in Table 26.

Table 26 score and description of the downtime criteria

Score	Downtime/Hinder
1	On water, < 2 days
0.75	On water, >2 days
0.5	On land and water
0.25	On land, < 2 days
0	On land, >2 days

8.2.2 Execution risk

The execution method of the solution is also ranked. This criteria is ranked because of the importance of the execution risks of a solution. The easiest solution with common work methods and common equipment is scored with an 5. These solutions are the solutions with the lowest execution risk. The work methods with needs adjustment or equipment needs adjustments are scored with an 3 and are more risk to execute. The solutions which provides new work method or new equipment is scored with an 1 are a major risk to execute because of the complexity and the new work method or equipment. The score and the description of the execution difficulty are shown in Table 27.

Table 27 score and description of the execution risk criteria

Score	Execution risk
1	Common method or equipment
0.5	Adjusted common method or equipment
0	New method or equipment

8.2.3 Lifetime extension

The expected lifetime extension is difficult to predict, but the lifetime extension of the solutions is determined in consultation with experts. The highest predictable life time extension is above 50 years and is ranked with an 5. The lowest predictable lifetime extension is below 15 years and is ranked with an 1. The life time extension between the 15 and 50 years is ranked with an 3. The score and corresponding lifetime extension are shown in Table 28.

Table 28 score and description of the life time extension criteria

Score	Lifetime extension
1	> 50 years
0.5	15-50 years
0	<15 years

8.3 Technical requirements

The structural assessment concludes failure mechanism and the critical structure elements . These critical structure elements do have a value before deepening. The most feasible deepening solution must meet the value of the structure element before deepening, to be feasible without reducing the reliability. The values are assumed as acceptable within a range around the value before deepening, because of the uncertainty of the model, the conservative modulation of the solutions and the inventory purpose of this research. The values of the critical structure elements are considered to be acceptable if the value after deepening is in between the lower limit and upper limit, which are shown in Table 29. These value are in 95% of all the cases, so the range of 5% is assumed as variation.

Table 29 lower limit and upper limit of the requirements

Lower limit	Requirement	Upper limit
-2.5%	0	2.5%
1.22	1.25	1.28
207	212	217
223	229	235

The deepening solutions must meet the following requirements to be considered as feasible:

- Safety factor, $1.22 < \mathrm{SF} \leq 1.28$;
- Front wall, $207\ N/mm^2 \leq\ \sigma_{max}\ \leq 217\ N/mm^2$;
- Shear force, $223\ kN \leq\ F_s\ \leq 235\ kN$;
- Prevention of eroding/piping of the soil between the primary piles.

If the solutions not meet the requirement, so are ranked with a score of 0, this solution is eliminated as possibility of most feasible solution.

8.3.1 Safety factor increasing effect

The ranking of the criteria is done by the minimum requirements. The minimum safety of the solutions must be $1.22 < \mathrm{SF} \leq 1.28$. The score is deviated in 5 scores with safety factor results result. The lowest score does not meet the requirement. The best score is a safety factor above 1.37. The scores between are deviated into steps of 0.05 increase of the safety factor. See the Table 30 below of the ranking and corresponding the safety factor value.

Table 30 score of the safety factor criteria

Score	Safety factor (SF)
1	SF ≥ 1.37
0.75	1.32 ≤ SF < 1.37
0.5	1.28 ≤ SF < 1.32
0.25	1.22 ≤ SF < 1.28
0	SF <1.22

8.3.2 Maximum stress front wall

The minimum requirement of the maximum stress of the front wall is $207 < \sigma \leq 217$ N/mm^2, so the lowest score is the bending moment result above 217 N/mm^2. The other scores are deviated in step of the decreasing of 10 N/mm^2. The highest score is the bending moment of the front wall below 187 N/mm^2. See maximum stress of the front wall

Table 31 for the ranking and the corresponding maximum stress of the front wall

Table 31 score of the maximum stress front wall criteria

Score	Maximum stress front wall
1	$\sigma < 187$ N/mm^2
0.75	$187 < \sigma \leq 197$ N/mm^2
0.5	$197 < \sigma \leq 207$ N/mm^2
0.25	$207 < \sigma \leq 217$ N/mm^2
0	$\sigma > 217$ N/mm^2

8.3.3 Prevention of piping and local geotechnical stability

The existing does not provide piping prevention, because of the lower layer thickness became piping critical. The solutions can increase the piping safety by as example injection of grout or an additional wall. beside of the increase the piping safety can also decrease, as example the temporary removal of the clay layers. The score deviation of the piping prevention criteria is shown in Table 32.

Table 32 score of the piping prevention criteria

Score	Piping prevention
1	Improvement
0.5	No Influence
0	Deteriorate

9 Trade-off matrix

To arrange the most optimal solution to deepening the construction depth 2 meters a trade-off matrix is made. This chapter describes the weight factors of the trade-off matrix, the matrix itself and the sensitivity of the matrix.

9.1 Weight factor

The weight factors of the criteria are ranked by the researcher to arrange the final most feasible solution. A recommendation is to rank the criteria by clients and other stakeholders to get a better view of the interest of the different stakeholders. In the thesis the criteria are compared to each other and ranked compared to each other. The overview of the compare and the weight factor of ever criteria is shown in Table 33.

Table 33 N2 matrix for the determination of the weight factor

	Lifetime extension	Execution risk	Downtime/hinder	Safety factor ΣMSF (GEO)	Maximum stress front wall (STR)	Piping prevention (HYD)	Sum	Weight factor w_i
Lifetime extension	x	1	1	1	1	1	5	5
Execution risk	0	x	1	1	1	1	4	4
Downtime/hinder	0	0	x	1	1	1	3	3
Safety factor ΣMSF (GEO)	0	0	0	x	1	1	2	2
Bending moment (STR)	0	0	0	0	x	1	1	1
Piping prevention (HYD)	0	0	0	0	0	x	0	1

The criteria piping prevention(HYD) does score a sum of 0. The weight factor of the piping prevention (HYD) is the same as the lowest other criteria which concludes the weight factors in right column 'Weight factor'.

9.2 Trade-off matrix with weight factors

The trade-off matrix consists of 2 parts, the weighted average of the criteria and the determination of the value to divide the weighted average and the costs partial. The weighted average and the value are calculated as the following formulas.

$$Weighted\ arithmetic\ mean = \frac{\sum w_i * score}{\sum w_i}.$$

$$Costs\ partial = \frac{cost_i}{\sum cost_i}.$$

$$Value = \frac{Weighted\ arithmetic\ mean}{costs\ partial}.$$

The results of the trade-off matrix are shown in Table 34.

Table 34 trade-off matrix results

Ranking of the solutions	Lifetime extension	Execution risk	Downtime/hinder	Safety factor ΣMSF (GEO)	Maximum stress front wall (STR)	Piping prevention (HYD)	Weighted arithmetic mean	$cost_i$ per meter (x1000)	Cost partial	Value
Weight factors w_i	5	4	3	2	1	1	**16**			
a) Excavation below the relieving floor	0.5	0.5	0	0	1	0.5	**0.38**	€ 27.00	0.08	**4.44**
b) Grout injection behind the retaining wall	0.5	0.5	0.25	0.75	1	1	**0.55**	€ 164.05	0.51	**1.07**
c) Grout injection at the toe of the retaining wall	0.5	0.5	1	0.75	0	1	**0.63**	€ 73.48	0.23	**2.72**
d) Additional high relieving platform	1	1	0.25	0	0	0.5	**0.66**	€ 20.77	0.06	**10.10**
e) Additional sheet pile wall	1	0.5	1	0.5	0.25	1	**0.77**	€ 19.13	0.06	**12.79**
f) Additional underwater anchorage	1	0.5	1	0.25	0.50	0.5	**0.72**	€ 15.23	0.05	**15.09**

It should be noted that three solutions score of a 0, because these solutions does not meet the technical requirements, in the blue rectangle. These three solutions are shown because of an exploratory understanding. The score of 0 means in fact that these solutions appeared to be not feasible, because of the negative effect on the structure which is not acceptable. According to the score above the following sequence is arranged:

1. Additional underwater anchorage;
2. Additional sheet pile wall;
3. Additional high relieving platform;
4. Grout injection at the toe of the retaining wall;
5. Excavation below the relieving floor.
6. Grout injection behind the retaining wall;

On the base of the trade-off criteria analyses can be concluded that the additional underwater and additional sheet pile wall are the preferential solutions. These solution score more of less the same, but the underwater anchorage provides a bigger decrease maximum stress of the front wall and the additional sheet pile wall provides a higher safety factor. The costs of the solutions does tip the scale to the underwater anchorage, so the underwater anchorage solution could be indicated as final preferential solution. The sensitivity of the trade-off matrix concludes the additional sheet pile wall and underwater anchorage as best solutions.

9.3 Trade-off sensitivity

The sensitivity of the trade-off is tested in difference deviations.

9.3.1 Ranking without cost and without excluding

The ranking without the cost is as the following Figure 7.

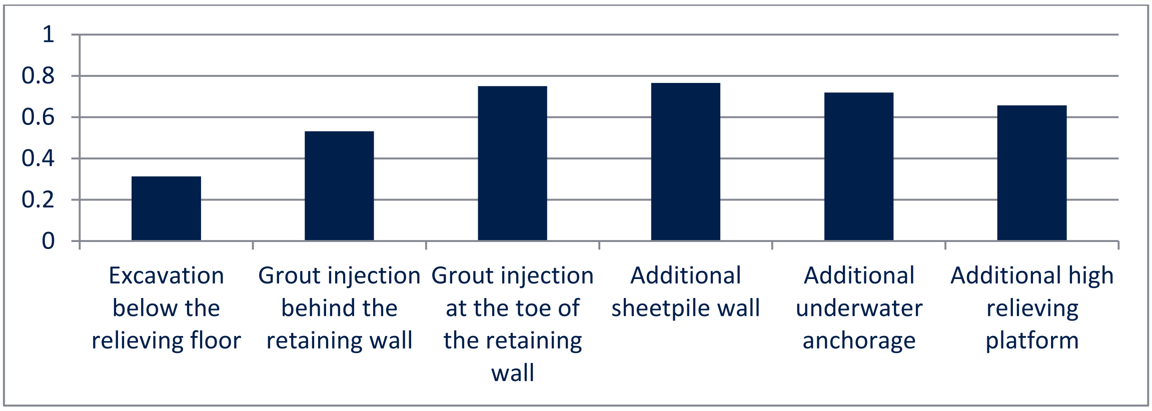


Figure 7 ranking of the solutions without cost partial

9.3.2 Ranking without weight factor

Results of the trade-off without weight factor is displayed in Table 35.

Table 35 ranking of the solutions with weight factor 1.0

Ranking of the solutions	Lifetime extension	Execution risk	Downtime/hinder	Safety factor ΣMSF (GEO)	Maximum stress front wall (STR)	Piping prevention (HYD)	Weighted arithmetic mean	$cost_i$ per meter (x1000)	Cost partial	Value
Weight factors							6			
Excavation below the relieving floor	0.5	0.5	0	0	1	0.5	0.42	€ 27.00	0.08	4.93
Grout injection behind the retaining wall	0.5	0.5	0.25	0.75	1	1	0.67	€164.05	0.51	1.30
Grout injection at the toe of the retaining wall	0.5	0.5	1	0.75	0	1	0.63	€ 73.48	0.23	2.72
Additional high relieving platform	1	1	0.25	0	0.25	0.5	0.50	€ 20.77	0.06	7.70
Additional sheet pile wall	1	0.5	1	0.5	0.25	1	0.71	€ 19.13	0.06	11.84
Additional underwater anchorage	1	0.5	1	0.25	0.5	0.5	0.63	€ 15.23	0.05	13.12

9.4 Trade-off conclusion

The sensitivity analyses of this trade-off matrix approve the most feasible solution of the trade-off matrix. The additional underwater anchorage is the most feasible because of the highest value of the solutions.

10Conclusion

The preferential solutions in obtained by a trade-off matrix with trade-off criteria. Figure 8 illustrate the additional underwater anchorage, which is the preferential solutions to deepen the construction depth at least 2 meters. The additional underwater anchorage is preferential because of the reduction of the maximum stresses of the front wall, the equal safety factor and the execution method without much hindrance.

Besides of the underwater anchorage, the additional sheet pile wall is also preferential, because the safety factor increases, the maximum stress remain equal and the execution method is without much hindrance. The additional sheet pile wall scored more or less the same as the underwater anchorage in the trade-off matrix.

However the construction costs of the additional underwater anchorage are expected to be lower compared to the additional sheet pile wall, so the value of the underwater anchorage is higher related to the additional sheet pile wall. For that reason, the underwater anchorage is determined as most preferential solution.

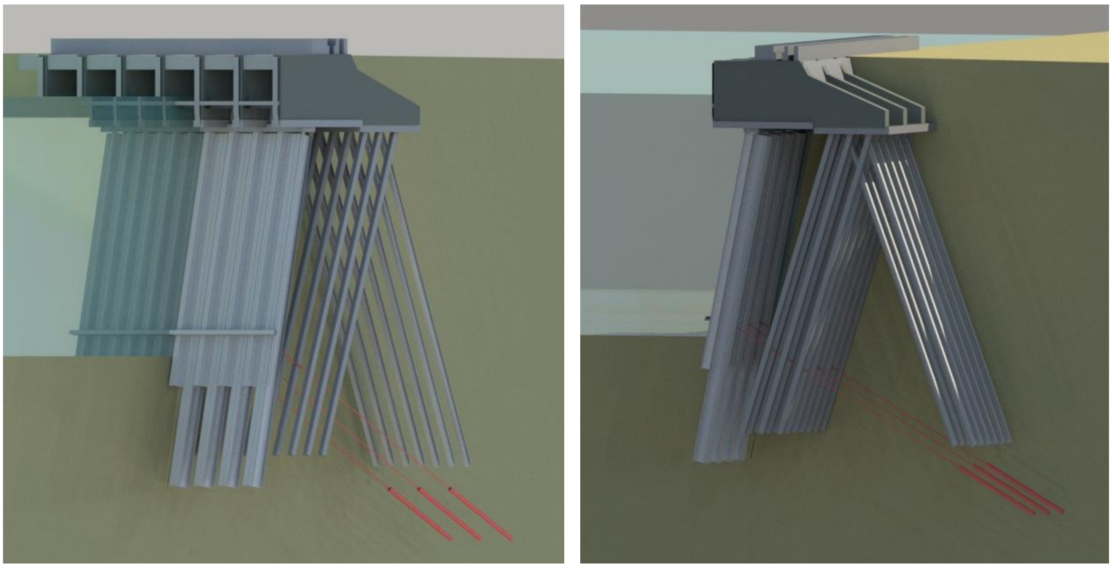


Figure 8 visualisation of the additional underwater anchorage (red) in the existing quay wall

11Bibliography

Arcelore Mittel. (2017). *AZ 26*. Retrieved may 9, 2017, from Sheet piling: http://sheetpiling.arcelormittal.com/section/more/id/53/referer/zsections/name/zsections

Grouttech. (2017). *SuperGrout 70, Krimparme gietmortel.* Nunspeet: Grouttech.

Jetmix. (2017). *Anchorage table Jetmix groutinjection achorages.* Product sheet, Werkendam.

Lenzi, M., Valmori, F., Maletti, F., Sciacca, L., Melegari, P., Campana, P., et al. (2011). THEORETICHAL AND EXPERIMENTAL INVESTIGATION. *PIANC E-magazine n° 143, July/juillet*, 11-28.

Peiner Träger GmbH. (2017). *Sheet piling sections*. Retrieved February 17, 2017, from Peiner Träger GmbH: http://www.peiner-traeger.de/en/products/product-range/sheet-piling-sections.html

SBRCURnet. (2014). *Publication 221E "Quay walls, Second Edition".* Rotterdam, The Netherlands: CRC Press/Balkema.

Schutte, J. J. (2017). *Inventory and preselection.* Rotterdam.

Schutte, J. J. (2017). *Plaxis calculations deepening solutions.* Rotterdam.

Schutte, J. J. (2017). *Structural engineering reference structure.* Rotterdam.

Technology, C. f. (2012). *Recommendations of the Committee for Waterfront Structures Harbours and Waterways – EAU 2012.* Hamburg: Ernst & Sohn GmbH & Co.

Appendix A: anchorage validation

Jetmix FUNDERINGSTECHNIEK

Doorsnede: 1 (S1) Ontwerpnr. 01

Project	17.000 Haven Rotterdam		**Rekensheet versie 2.0 d.d. 20-10-2016**
Opdrachtgever	Havenbedrijf Rotterdam		
Onderdeel	Ankerontwerp		
Versie	0.1	**Datum** 12-05-2017	

Te verankeren constructie

Damwandtype	:	n.t.b.	Trajectlengte	:	30,00 m^1
Bovenkant wand	:	+ 0,00 m NAP	Aantal secties	=	10,00 st
Onderkant wand	:	- 20,00 m NAP	Eindankers tbv ankeruitval	:	1 st
Ankerniveau	:	- 14,00 m NAP	Praktisch aantal ankers	:	11 st

Bepaling belastingen

Hoek tov hor. (D-sheet)	:	30 °				
$P_{max;ax}$ UGT (D-sheet)	:	167 kN/m^1	-->	$P_{max;hor}$ = $P_{max;ax}$ x cos (α_h)	=	144 kN/m^1
$P_{vsp;ax}$ Voorspanning (D-sheet)	:	kN/m^1	-->	$P_{vsp;hor}$ = $P_{vsp;ax}$ x cos (α_v)	=	0 kN/m^1
α_h (Ankerhoek tov hor.)	:	30 °		Hart-op-hart afstand	:	3,00 m^1
α_v (Ankerhoek tov vert. (=offset))	:	0 °		P_{max}	=	500 kN

Anker

Ankersysteem	:	Jetmix groutinjectieankers			
Ankertype	:	Ø 60,3x16,0	A_{staal} vóór corrosie	:	2194 mm^2
Corrosie	:	0,012 mm/jaar	A_{staal} na corrosie	=	1971 mm^2
Levensduur	:	100 jaar			
$P_{d;staal}$ = P_{max} x 1,25	=	625 kN			
$R_{t;y}$ = (A_{staal} x f_y) / 1,0	=	986 kN	f_y (vloei)	:	500 N/mm^2
$R_{t;u}$ = (A_{staal} x f_u) / 1,4	=	986 kN	f_u (breuk)	:	700 N/mm^2
$R_{t;d}$	=	986 kN	**Unity Check**	=	**0,63**

Toets groutlichaam

$P_{d;grout}$	=	550 kN	α_t	:	0,0150 %
Maaiveld	:	+ 1,00 m NAP	Boorkop diameter	:	260 mm
Bovenkant zandlaag	:	- 19,00 m NAP	Oppersing	:	40 mm
Bovenkant grout	:	- 20,00 m NAP	Diameter groutlichaam	=	300 mm
Check 5,00 m gronddekking	:	OK	Omtrek groutlichaam	=	0,942 m
Check h.o.h. afstand min. 8D	:	OK			
L_{grout} + extra overlengte	=	5,50 m	Onderkant grout	=	- 22,75 m NAP
$Q_{c;red;gem}$	=	9,10 MPa			
$R_{a;d}$ = (α_t x O x L_{grout} x $Q_{c;gem}$) / γ_a	=	590 kN (γ_a = 1,2)	**Unity Check**	=	**0,93**

Jetmix FUNDERINGSTECHNIEK

Rekensheet versie 2.0 d.d. 20-10-2016

Project 17.000 Haven Rotterdam
Opdrachtgever Havenbedrijf Rotterdam
Onderdeel Ankerontwerp
Doorsnede 1 (S1) **Ontwerpnr. 01**
Versie 0.1 **Datum** 12-05-2017

Bodemopbouw

Toegepaste sondering : S1
Maaiveld : + 1,00 m NAP
Bovenkant grout : - 20,00 m NAP

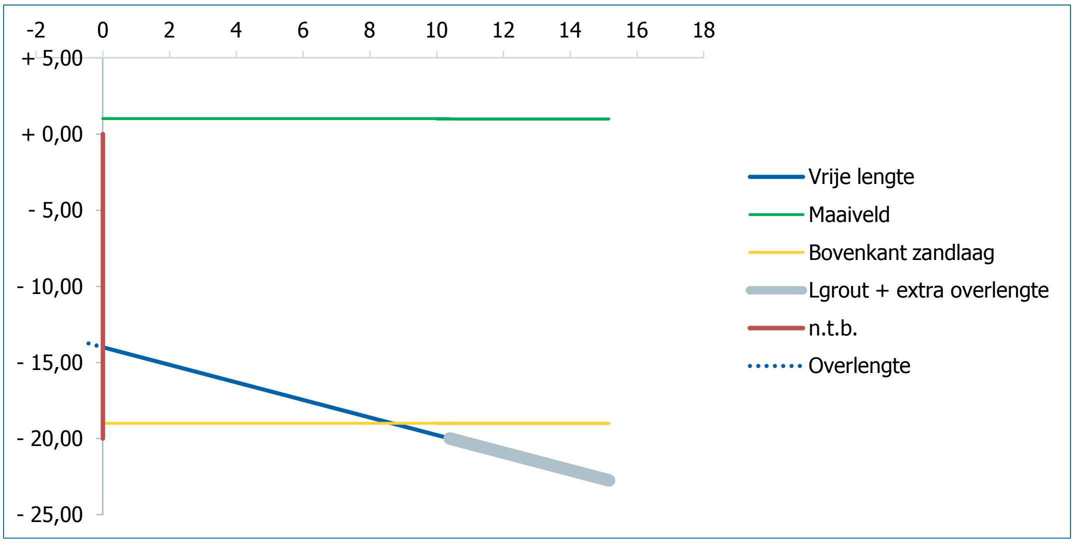
van [m NAP]	tot [m NAP]		Q_c [MPa]	dR_d [kN]	$dL_{grout;}$ [m]	R_d [kN]
- 20,00	- 21,00		4,0	94	2,00	94
- 21,00	- 22,00		12,0	283	2,00	377
- 22,00	- 23,00		13,0	173	1,13	550
- 23,00	- 24,00		13,0	0	0,00	0
- 24,00	- 25,00		14,0	0	0,00	0
- 25,00	- 26,00			0	0,00	0
- 26,00	- 27,00			0	0,00	0
- 27,00	- 28,00			0	0,00	0
- 28,00	- 29,00			0	0,00	0
- 29,00	- 30,00			0	0,00	0
- 30,00	- 31,00			0	0,00	0
- 31,00	- 32,00			0	0,00	0
- 32,00	- 33,00			0	0,00	0
- 33,00	- 34,00			0	0,00	0
- 34,00	- 35,00			0	0,00	0
- 35,00	- 36,00			0	0,00	0
- 36,00	- 37,00			0	0,00	0
- 37,00	- 38,00			0	0,00	0
- 38,00	- 39,00			0	0,00	0
- 39,00	- 40,00			0	0,00	0
- 40,00	- 41,00			0	0,00	0
- 41,00	- 42,00			0	0,00	0
- 42,00	- 43,00			0	0,00	0
- 43,00	- 44,00			0	0,00	0
- 44,00	- 45,00			0	0,00	0
- 45,00	- 46,00			0	0,00	0
- 46,00	- 47,00			0	0,00	0
- 47,00	- 48,00			0	0,00	0
- 48,00	- 49,00			0	0,00	0
- 49,00	- 50,00			0	0,00	0
Totaal				**550**	**5,13**	

Rekensheet versie 2.0 d.d. 20-10-2016

Project	17.000 Haven Rotterdam		
Opdrachtgever	Havenbedrijf Rotterdam		
Onderdeel	Ankerontwerp		
Doorsnede	1	(S1)	**Ontwerpnr. 01**
Versie	0.1	**Datum**	12-05-2017

Bepaling ankerlengte

Vrije ankerlengte	=	12,00 m	Overlengte tbv ankerkop	:	0,50 m
Ankerlengte theoretisch	=	17,63 m	L_{app} fictieve ankerlengte	=	14,75 m
L_{totaal} (ankerlengte praktisch)	=	18,00 m	Horizontaal ruimtebeslag	=	15,16 m



Samenvatting ankerontwerp

Jetmix groutinjectieanker	:	Ø 60,3x16,0
L_{totaal} (ankerlengte praktisch)	:	18,00 m
L_{grout}	:	5,50 m
L_{over}	:	0,50 m
Boorkop diameter	:	260 mm
Ankerhoek tov hor.	:	30 °
Ankerhoek tov vert. (offset)	:	0 °
H.o.h. afstand	:	3,00 m^1
Trajectlengte	:	30,00 m^1
Aantal ankers	:	11 st

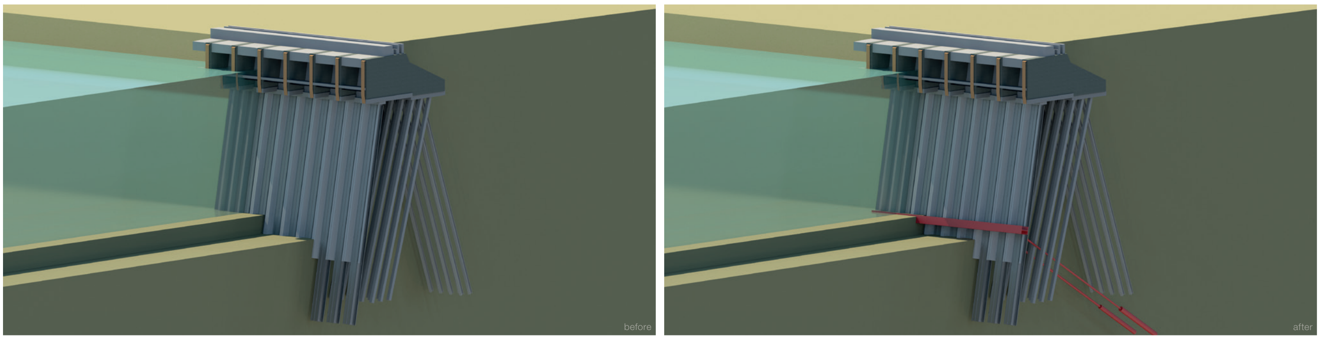
Testbelastingen

$P_{d;test}$	= $P_{d;grout}$	=	550 kN
	P_i (initiële kracht)	=	55 kN
	40% P_{test}	=	220 kN
	55% P_{test}	=	305 kN
	70% P_{test}	=	385 kN
	85% P_{test}	=	470 kN
	100% P_{test}	=	550 kN
	P_{vsp} voorspanning	=	0 kN
Max. $R_{t;d}$	(staal)	=	997 kN

(vóór corrosie en uitgaande van γ=1,1)

7 Documents for the Rotterdam University of Applied Sciences

Port of Rotterdam



DEEPENING OF AN EXISTING COMBI WALL

DOCUMENTEN VOOR HOGESCHOOL ROTTERDAM

ROTTERDAM UNIVERSITY
UNIVERSITY OF APPLIED SCIENCES

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Deepening of an Existing Combi Wall

Documenten voor de Hogeschool Rotterdam

12 juni 2017, Rotterdam, The Netherlands

Module:	**CIVAFS40**
Author:	**Jordy Schutte**
Supervisor company:	**A. A. Roubos MSc**
Supervisor school:	**H.J. Dommershuijzen MSc**

Educational institution:	University of Applied Sciences Rotterdam
Study:	Civil Engineering
Version:	1.0
Study year:	2016-2017

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Inhoudsopgave

1 Introductie

Dit document is opgesteld om de formulieren te verschaffen die worden gevraagd van uit de opleiding Civiele techniek aan de Hogeschool Rotterdam. Dit document is tevens opgesteld in het Nederlands in verband met de invul formulieren die in het Nederlands zijn verschaft. Dit document omvat de competentie verantwoording en het publicatieverklaring voor de hbo kennisbank.

2 Competentie verantwoording

De competentie verantwoording is toegevoegd aan dit rapport op de volgende bladzijde. The beschrijving van de niveaus is toegevoegd in de onderstaande tabel.

Tabel 1 beschrijving van niveaus

Niveau	Factoren	Beschrijving
1	taak	Eenvoudig, gestructureerd, bekende methoden direct toe te passen
	context	Bekend, eenvoudig, monodisciplinair
	zelfstandigheid	Sturende begeleiding (docent gestuurd)
2	taak	Complex, gestructureerd, bekende methoden in wisselende situaties toe te passen
	context	Bekend, complex, monodisciplinair, praktijkprojecten onder begeleiding
	zelfstandigheid	Coachende begeleiding
3	taak	Complex, ongestructureerd, aan de situatie aangepaste methoden toe te passen
	context	Onbekend, complex, multidisciplinair, in de praktijk
	zelfstandigheid	Zelfstandig, begeleiding indien nodig (student gestuurd)

Competenties & Leerdoelen	Niveau	Criterium	Dat het leerdoel is behaald blijkt uit:
1. Initiëren en sturen 1.1 Signaleren en/of analyseren van (de behoefte aan) een civieltechnisch project in de gebouwde omgeving. 1.2 Ontwikkelen van een Programma van Eisen voor een te maken Civiel Technisch project.	2	B.1, B.3	1.1 De behoefte aan een civieltechnisch project is gesignaleerd in **Interim report 1: Project plan** en in hoofdstuk 1 **Introduction**. Hierin staat beschreven waarom er behoefte is en wat de aanleiding is voor de behoefte. Het vooronderzoek van de verdiepingsprojecten en de bepaling van het referentie model zijn een toegevoegde waarde voor de probleemstelling. Deze documenten zijn te vinden in hoofdstuk 2 Preliminary investigation en **Interim report 2: Executed deepening projects** en **Interim report 3: Reference structures Botlek area** 1.2 het referentie model is gemodelleerd in Plaxis, daarmee zijn de kritische constructie onderdelen blootgesteld. De begin waarden van deze kritische onderdelen zijn als eisen gesteld aan de verdiepingsoplossingen. Hiermee zijn de oplossingen getoetst en beoordeeld. De bepaling van de eisen is te vinden in hoofdstuk 3 structural engineering en **Interim report 3: Structural engineering reference structure**
2. Ontwerpen 2.1 Ontwerpen van oplossingsvarianten in de vorm van bv. Schema's, tekeningen en/of berekeningen voor Civiel Technische (deel) problemen. 2.2 Oplossingsvarianten beoordelen en de meest passende kiezen 2.3 Inventariseren en verzamelen van gegevens.	3	B.1, B.2, B.4	2.1 De effecten van het verdiepen op een bestaande kade constructie met de oplossingen bepaald aan de hand van een eindige elementen methode programma. De uitgangspunten en de resultaten zijn te vinden in hoofdstuk 5 en 6 en **Interim report 6: Trade-off selection.** 2.2 om de voorkeursvariant te bepalen is er een voorselectie gedaan en een uiteindelijke trade-off matrix met verschillende weegfactoren. De inventarisatie is opgesteld en uitgewerkt in hoofdstuk 5 en **Interim report 5: Inventory and preselection.** De uiteindelijke afweging van de voorkeursvariant is gemaakt in het hoofdstuk 6 en **Interim report 6: Trade-off selection.** 2.3 De basis van het referentie model is opgesteld aan de hand van de originele berekening. De uitgangspunten van deze berekening zijn beoordeeld en geïmplementeerd in het referentie model. De oplossingen om te verdiepen zijn bepaald aan de hand van brainstorm sessies en interviews met deskundige van uit het werkveld. Voor de onderbouwing zie **Interim report 5: Inventory and preselection en Interim report 3: Structural engineering reference structure**

3. Specificeren 3.1 Schematiseren van de werkelijke situaties in een rekenkundig of fysisch model. 3.2 Detailleren en/of berekenen en tekenen van een (deel) Civieltechnisch ontwerp.	3	B.5	3.1 De effecten van het verdiepen op een bestaande kade constructie is bepaald aan de hand van een eindige elementen methode programma. De uitgangspunten en de resultaten zijn te vinden in hoofdstuk 3 en **Interim report 3: Structural engineering reference structure.** 3.2 voor deze eis geld het zelfde als bovenstaand. Dit staat uitgebreid beschreven in **Interim report 3: Structural engineering reference structure**

Competenties & Leerdoelen	Niveau	Criterium	Dat het leerdoel is behaald blijkt uit:
6. Monitoren, toetsen en evalueren 6.1 Hanteren van plan-do-check-act cyclus. 6.2 Omgevingsbewust en maatschappelijk verantwoord handelen.	3	A.2, B.1, B.2, B.3, B.5	6.1 voorafgaande aan het afstuderen is er een plan van aanpak opgesteld met een daarbij horende planning. Gedurende het afstuderen is er de plan-do-check-act cycles toegepast door de planning te bekijken en bij te sturen. Wekelijk en dagelijks zijn er to-do-listen opgesteld om de voortgang te blijven controleren. Daarnaast is document meerdere malen opgesteld, gecheckt, actiepunten opgesteld en verwerk. De beoordeling van de bedrijfsbegeleider dient als onderbouwing. 6.2 gedurende de afweging van de voorkeursvariant is er omgevingsbewust en maatschappelijk gekozen, door de eisen van verschillende stakeholders in te schatten. De hinder van een oplossing is meegenomen wat zorgt voor omgevingsbewust handelen.
7. Onderzoeken 7.1 Uitvoeren van onderzoek.	3	A.1, A.2, B.2, B.3, B.4, B.5	7.1 dit leerdoel is behaald gedurende de gehele uitvoering van het onderzoek. Aan het begin is de probleemstelling vast gesteld, een plan van aanpak opgesteld en is er gestart met een literatuur onderzoek. In navolging daarop zijn er gesprekken geweest met marktpartijen om een zo groot mogelijk areaal van oplossingen te verschaffen. Voor het onderzoek zijn dan ook verschillende soorten onderzoek toegepast, zoals interviews en literatuur onderzoek. Voor de onderbouwing zie het gehele rapport.

8. Communiceren en samenwerken 8.1 Verwoorden en verbeelden van informatie. 8.2 Functioneren in teams.	3	C.2, C.3, C.4	8.1 de verwoording van het eindrapport in het Engels was een uitdaging. Het niveau van het schrijven is tijdens het afstuderen zeer verbeterd. De verbeelding van resultaten en informatie is gedaan in de rapporten. Zie het hoofd rapport voor de onderbouwing van dit leerdoel. 8.2 tijdens het onderzoek is er veel contact geweest met experts uit het werkveld. Door dit contact is er veel validatie geweest en is er duidelijk gewerkt in teams en samengewerkt. Naast de experts uit het werkveld is er ook veel contact geweest met personen intern bij het Havenbedrijf Rotterdam. De bedrijfsbeoordeling dient als bewijslast.
9. Managen en innoveren 9.1 Regie voeren over eigen leerproces. 9.2 Projectmatig werken en processen aansturen.	2	A.2, B.2, C.1, C.2	9.1 het onderzoek is zelfstandig uitgevoerd met geringe bijsturing van de begeleiders. De planning en het uiteindelijke resultaat is continue in het vizier gehouden wat heeft gezorgd voor een goede leercurve gedurende het afstuderen. 9.2. gedurende het onderzoek bleek dat het optimaliseren van de uiteindelijke voorkeursvariant niet mogelijk was i.v.m. de tijd, waardoor er is gestuurd in de planning en het proces. Daarnaast is er continue feedback gevraagd en verwerkt in het proces. De onderbouwing is de bedrijfsbeoordeling en het gehele rapport.

3 Publicatie verklaring

De publicatie verklaring is toegevoerd aan dit document op de volgende pagina’s.

HOGESCHOOL ROTTERDAM

Onderwijs en
Ontwikkeling

Kennis & Informatie / Mediatheek

Toestemmingsformulier tot opname en beschikbaarstelling afstudeerscriptie in een digitale kennisbank

Datum	9-6-2017
Naam student	Jordy Schutte
Studentnummer	0877616
Naam instituut	IGO
Opleiding	Civiele techniek
Afstudeerrichting	Bachelor
Titel scriptie	Deepening of an Existing Combi Wall
Toestemming	Ja / [redacted]
Emailadres	jordyschutte@hotmail.com

Digitale kennisbank
De Hogeschool Rotterdam heeft een digitale kennisbank opgezet waarin de Hogeschool scripties die door studenten in het kader van hun studie aan de Hogeschool hebben geschreven, toegankelijk worden gemaakt voor derden. Hierdoor wordt het proces van creatie, verwerving en deling van kennis binnen het onderwijs mogelijk gemaakt en ondersteund.

De in de kennisbank opgenomen scripties worden toegankelijk gemaakt voor potentiële gebruikers binnen en buiten de Hogeschool. Om opname en beschikbaarstelling mogelijk te maken dient dit toestemmingsformulier.

Bij opname en beschikbaarstelling in de digitale kennisbank behoudt de student zijn of haar auteursrecht. Daarom kan hij of zij de toestemming tot het beschikbaar stellen van haar / zijn afstudeerscriptie intrekken.

Paraaf voor gelezen en akkoord:	

Rechten en plichten student
De student verleent aan de Hogeschool kosteloos de niet exclusieve toestemming om zijn afstudeerscriptie op te nemen in de digitale kennisbank en om deze beschikbaar te stellen aan gebruikers binnen en buiten de Hogeschool. Hierdoor mogen gebruikers de afstudeerscriptie geheel of gedeeltelijk kopiëren en bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie en/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.

De student geeft de Hogeschool het recht de toegankelijkheid van de afstudeerscriptie te wijzigen en te beperken indien daar zwaarwegende redenen voor bestaan.

De student verklaart dat de stagebiedende organisatie dan wel de opdrachtgever van de afstudeerscriptie geen bezwaar heeft tegen opname en beschikbaarstelling van de afstudeerscriptie in de digitale kennisbank.

Verder verklaart de student dat hij of zij toestemming heeft van de rechthebbende van materiaal dat de student niet zelf gemaakt heeft om dit materiaal als onderdeel van de afstudeerscriptie op te nemen in de digitale kennisbank en aan derden binnen en buiten de Hogeschool beschikbaar te stellen.

Daarnaast verklaart de student dat hij of zij de scriptie heeft geanonimiseerd: namen, adressen, telefoonnummers en e-mailadressen zijn uit de scriptie verwijderd.

Rechten en plichten Hogeschool
De door de student verleende niet-exclusieve toestemming geeft de Hogeschool het recht de afstudeerscriptie aan gebruikers binnen en buiten de Hogeschool beschikbaar te stellen.
De Hogeschool mag verder de afstudeerscriptie voor gebruikers binnen en buiten de Hogeschool vrij toegankelijk maken voor een gebruiker van de digitale kennisbank en mag deze gebruiker toestemming geven om de afstudeerscriptie te kopiëren en te bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie_n/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.
De Hogeschool zal ervoor zorgen dat vermeld wordt wie de schrijver(s) is/zijn van de afstudeerscriptie waarbij zij tevens aangeeft dat bij gebruik van de afstudeerscriptie de herkomst hiervan duidelijk vermeld moet worden. De Hogeschool zal duidelijk maken dat voor ieder commercieel gebruik van de afstudeerscriptie toestemming van de student nodig is.

De Hogeschool heeft het recht de toegankelijkheid van de afstudeerscriptie te wijzigen en te beperken indien daar zwaarwegende redenen voor bestaan.

Rechten en plichten gebruiker
Door dit Toestemmingsformulier mag een gebruiker van de digitale kennisbank de afstudeerscriptie geheel of gedeeltelijk kopiëren en/of geheel of gedeeltelijk bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie_en/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.

Naam:	Handtekening:
Jordy Schutte	

6-6-17