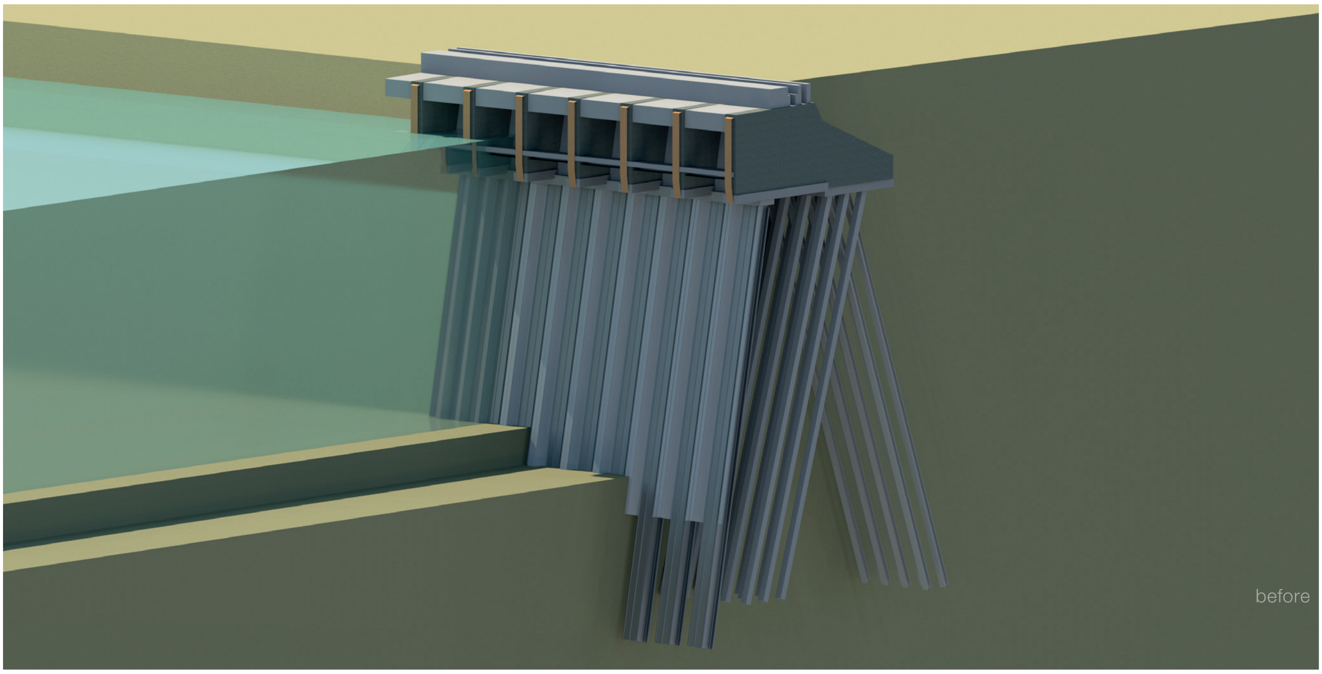
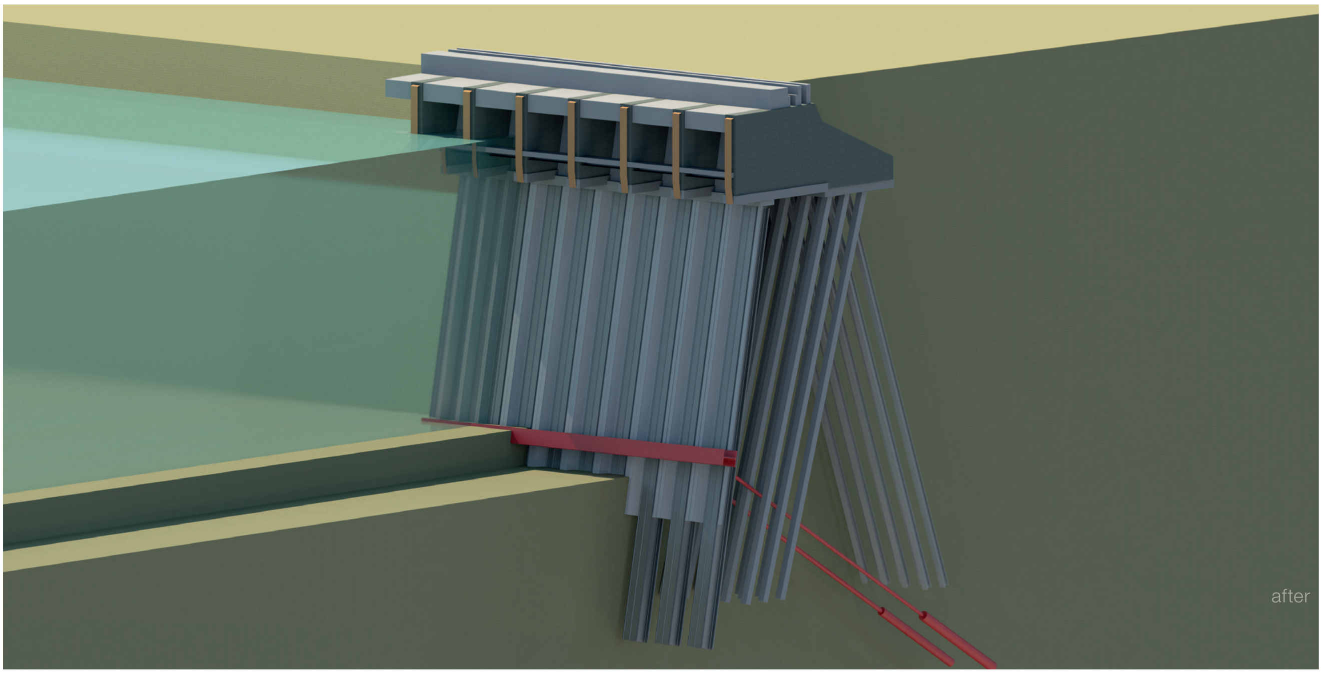
DEEPENING OF AN EXISTING COMBI WALL

FINAL REPORT





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Date 12 June 2017

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Deepening of an Existing Combi Wall

Final report

12 June 2017, Rotterdam, The Netherlands

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

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Preface

This bachelor thesis is the last phase of my study Civil Engineering of the University of Applied Sciences Rotterdam. This thesis project verifies the expertise and competences which I must obtain to finish my study. This thesis investigates the most preferred solution to deepen the construction depth in front of an existing quay wall, at least 2 meters while the reliability needs to be equal.

The Port of Rotterdam Authority is the supervisory company and Witteveen+Bos is the advisory company for my thesis. I performed this bachelor thesis since February 2017 until June 2017. I have experienced this graduation period as very interesting and educational. Initially of the thesis, I had limited knowledge about quay structures. However, during this thesis, I have learned a lot about quay walls, such as upgrading techniques of quay walls and critical elements of a quay walls structure. The modulation into Plaxis and the final selection of the most preferred solution were the biggest challenges for me.

I want to thank my supervisor of the Port of Rotterdam, Alfred Roubos and my supervisor of the University of Applied Sciences Rotterdam, Harry Dommershuijzen for their guidance, advice, feedback and support during the execution of this thesis. During the phases, I validated my assumptions and result by experts of the Port of Rotterdam Authority, expert of contractors and lectors at the University of Applied Sciences Rotterdam.

Regarding the modelling and validation of the Finite Element Method calculations, I would like to thank Henk Brassinga of the Port of Rotterdam Authority, Dirk-Jan Jaspers Focks of Witteveen+Bos, Rob Vinks of Dimco B.V. and Awni Sedra of the University of Applied Sciences Rotterdam. In case of the inventory and the advice of the substantiation of the solutions, I want to thank Hein van Laar of Hakkers, Marco van der Berg of De Klerk, Marinus de Heus of Jetmix, Maurice Krul of W. Smit Dive- & salvage company, Rob Vinks of Dimco b.v., Willem-Jan Nederlof of Dimco b.v. and Rob Selhorst of Combinatie Civiele Technieken BV.

I gratefully acknowledge Harm Kortman, Maarten Meijler and Johan Plugge of the Port of Rotterdam Authority and Hans Schutte of Dimco b.v. for the validation of the arranged preselection. For the cost estimation, I gratefully acknowledge Marinus de Heus of Jetmix and Maarten Smiet of the Port of Rotterdam Authority. Besides of the validation and advice, I gratefully acknowledge Johan Plugge of the Port of Rotterdam Authority and Dirk-Jan Jaspers Focks of Witteveen+Bos for their feedback on this thesis. Finally, I want to thank my family and friends for their support and their review of my documents.

Jordy Schutte

Rotterdam, 12 June 2017

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Summary

The Port of Rotterdam Authority is interested in the possibilities to upgrade existing quay walls, before constructing new structures to receive vessels with larger dimensions. The global growth of the size and draft of the seagoing vessels are the reason of the growing demand for deeper berths. To achieve this increased demand, port beds have to be deepened. The most important question of this study is based on fulfilling this demand and is formulated as:

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

To get familiarised with the subject and select the reference quay structure for this thesis, a preliminary investigation is performed. This investigation includes an inventory of executed deepening project around the world and a selection of a reference structure. The combi wall of the Sint Laurenshaven is chosen as reference structure is this thesis. It should be noted that 38% of the total structures in the Port of Rotterdam are combi walls, which makes the Sint Laurenshaven combi wall a suitable reference structure. This reference structure is used to determine the effect of the deepening to the structure.

The critical elements and the failure mechanism, because of the deepening, are determined by a structural assessment with Finite Element Method software. The results of this structural assessment, such as bending moment and global safety factor, were used to formulate requirements for new deepening solutions to guarantee equal reliability.

As a result of the structural assessment, an inventory of twenty-seven solutions is arranged by an inventory of executed deepening projects in the past, brainstorming and interview experts and literature research. These solutions are filtered in a preselection with preselection criteria, such as multidisciplinary application possibilities and the technical feasibility. In figure 1 are the preselected solutions presented.

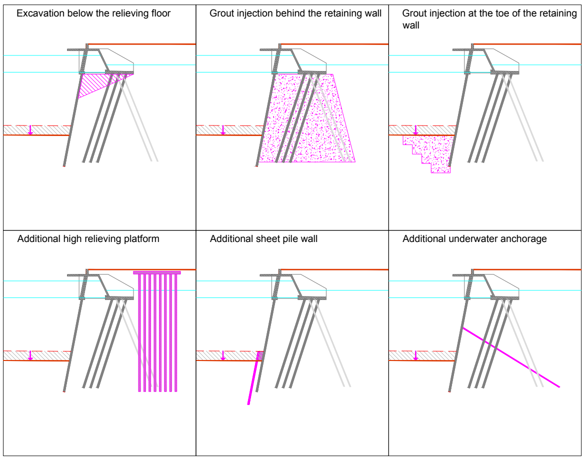


Figure 1 preselected solutions

The preferred solutions in obtained by a trade-off matrix with trade-off criteria. Figure 2 illustrate the additional underwater anchorage, which is the preferred solutions to deepen the construction depth at least 2 meters. The additional underwater anchorage is preferred because of the reduction of the maximum stresses of the front wall, an equal safety factor and the execution method without much hindrance.

Besides of the underwater anchorage, the additional sheet pile wall is also preferred, because of the safety factor increases, the maximum stress remains 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 preferred solution.

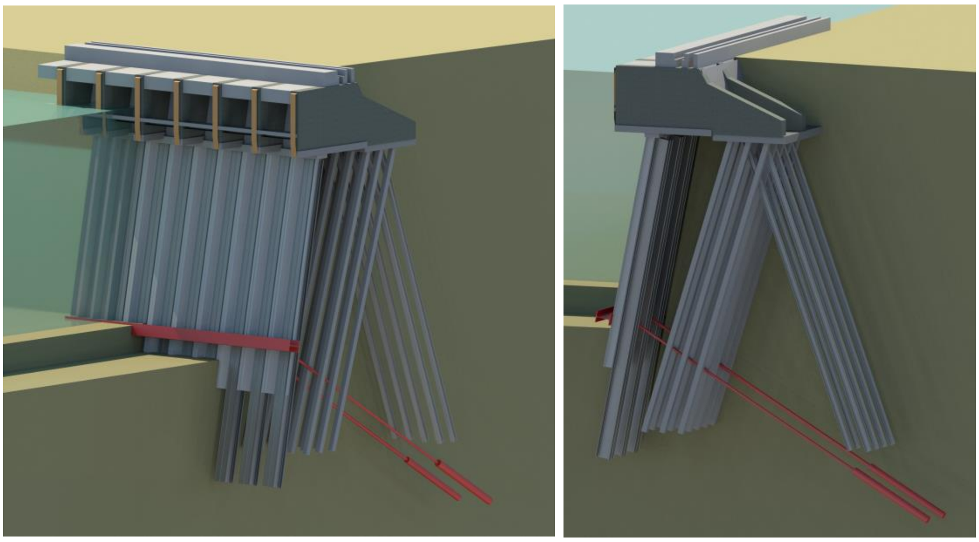


Figure 2 visualisation of the additional underwater anchorage (red) in the existing quay wall

As a result of the additional underwater anchorage as preferred solution, are the main recommendations formulated as the bullet point below.

- Perform a design engineering of the additional underwater anchorage with project specific ground and structural element parameters and with advanced soil-structure interface;
- Run a pilot for the application of the underwater anchorage in order, to acquire insight in the effects of the solution into reality;
- Perform a detailed engineering of the additional sheet pile wall and also perform a detailed engineering of a combination of the additional wall and the additional underwater anchorage to specify the exact effects of the solutions, if the results are positive, also run a pilot;
- Investigate the opportunities of the grout injection and share knowledge with other countries, such as Japan and Spain, because the result of the Plaxis calculation is promising for lifetime extension, but the price is extremely high.

Samenvatting

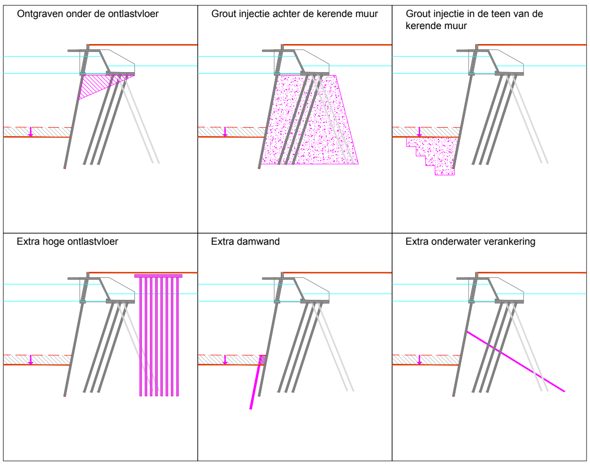
Het Havenbedrijf Rotterdam is geïnteresseerd in de mogelijkheden om bestaande kademuren te upgraden, in plaats van het bouwen van geheel nieuwe kademuren. De aanleiding van deze interesse is de wereldwijde groei van de scheepsafmetingen van de zeegaande schepen die afmeervoorzieningen met meer diepgang vragen.. Door deze groeiende vraag moeten de havenbodems worden verdiept. In verband met de verdiepingsvraagstelling is de hoofdvraag van dit onderzoek als volgt geformuleerd.

Wat is de voorkeursoplossing om de constructiediepte van een bestaande combiwand constructie 2 meter te verdiepen, zonder het huidige veiligheidsniveau te verlagen?

Om vertrouwd te raken met het onderwerp en het selecteren van het referentie model is er een vooronderzoek uitgevoerd. Dit vooronderzoek bevat een inventarisatie van uitgevoerde verdieping projecten en de keuze voor een referentie kade constructie. Het referentie model is een combiwand, omdat 38% van het totale areaal van het havenbedrijf Rotterdam combiwanden zijn. Om deze reden zijn de combiwand constructie het meest voor de hand liggend. De combiwand in de Sint Laurenshaven is gekozen als referentie kade constructie. Dit referentie model is gebruikt om de effecten van het verdiepen op de kademuur te bepalen.

De kritieke elementen en de kritieke faal mechanisme van de referentie constructie, door het verdiepen, zijn bepaald aan de hand van een constructieve toetsing met Eindige Elementen Model software. Het resultaat van deze toetsing, zoals buigend moment en veiligheidsfactor, zijn gebruikt om eisen op te stellen voor oplossingen om te kunnen verdiepen met behoud van het huidige veiligheid niveau.

In opvolging op de constructieve toetsing is een inventarisatie van zevenentwintig mogelijke oplossingen opgesteld door middel van uitgevoegde verdiepingen in het verleden, brainstormen en interviews met experts en literatuur onderzoek. Deze oplossingen zijn gefilterd in een voorselectie, met als resultaat de oplossingen die aan de preselectie criteria, zoals multidisciplinaire toepassingsmogelijkheden en technische haalbaarheid, hebben voldaan. In Figuur 1 zijn de oplossingen weer gegeven die zijn voorgeselecteerd.

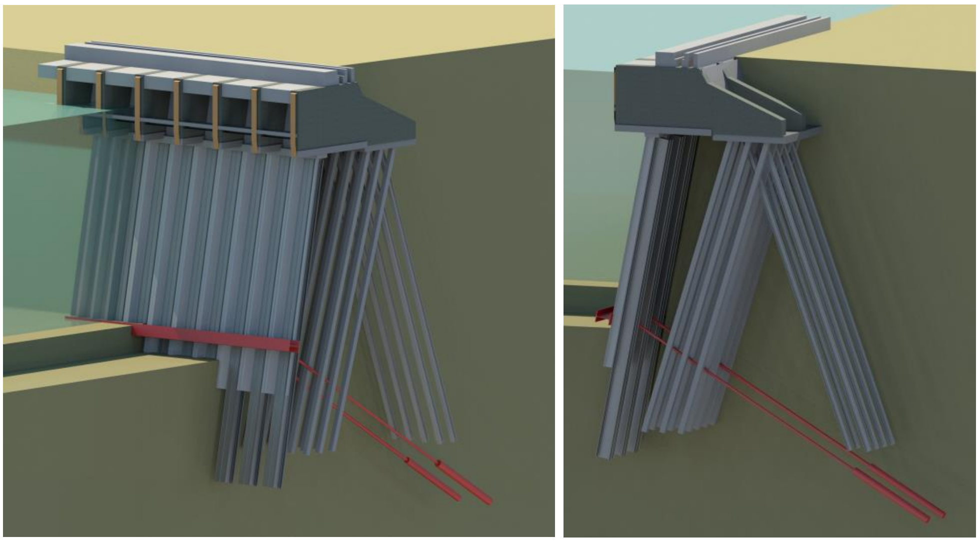


Figuur 1 voorgeselecteerde oplossingen

De voorkeursoplossing op bepaald aan de hand van een multi criteria analyse. Op figuur 2 is de extra onderwater verankering te zien, wat de voorkeursoplossing is om de constructie diepte 2 meter te verdiepen. De extra onderwater verankering is de voorkeursoplossing, omdat de spanningen in de combiwand afnemen, het veiligheidsniveau gelijk blijft en de uitvoeringsmethode weinig hinder oplevert.

Naast de extra onderwater verankering is het extra damwand ook een voorkeursoplossing, omdat het veiligheidsniveau toeneemt, de spanningen in de combiwand gelijk blijven en de uitvoeringsmethode ook niet zorgt voor veel hinder. De scores in de multi criteria analyse zijn ongeveer gelijk als de extra onderwater verankering.

Doordat de extra onderwater verankering oplossing goedkoper is in vergelijking met de extra damwand, is de waarde van de extra onderwater verankering hoger dan de extra damwand. Om deze reden is de extra onderwater verankering bepaald als voorkeursoplossing.



Figuur 2 visualisatie van de onderwater verankering

Naar aanleiding van de conclusie van de onderwater verankering als voorkeursoplossing om 2 meter te verdiepen, zijn er aanbevelingen opgesteld:

- Uitvoeren van een ontwerp berekening van de onderwater verankering met project specifieke eigenschappen van de grond en de constructie onderdelen met een geavanceerde grond constructie interactie programma;
- Uitvoeren van een proef van de upgraden van een bestaande constructie met onderwater verankering, om de effecten van de oplossing te zien in de praktijk;
- Uitvoeren van een ontwerp berekening van de extra damwand en een ontwerp berekening van een combinatie van de extra damwand met onderwater verankering. Als deze resultaten positief uitvallen ten opzichte van het onderwater anker ook een proef uitvoeren;
- Onderzoek naar de mogelijkheden van grout injectie en het delen van kennis met andere landen, zoals Japan en Spanje, omdat de resultaten van de Plaxis berekening veelbelovend zijn, maar de kosten uitzonderlijk hoog zijn.

Table of contents

Separately provided reports

In addition to this final thesis report, the following interim reports and documents for the Rotterdam University of Rotterdam are separately provided.

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. *Documenten voor de hogeschool Rotterdam:*
    - *Verantwoording behaalde competenties;*
    - *Publicatieverklaring.;*

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Symbol	Description	Unity
General		
G	Free fall acceleration	m/s^2
γ_w	Weight density of water	kN/m^3
CG	Centre of gravity	-
Concrete		
f_{ck}	Characteristic cube tension stress	N/mm^2
E	Modules of elasticity of soil	kN/m^2
R_{inter}	Interface factor in FEM calculations	$-$
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	Moment of resistance of the cross-sectional area	m^3
E	Modules of elasticity of soil	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^2
Soil		
γ_{sat}	Saturared weight density of the soil	kN/m^3
γ_{unsat}	Unsaturared weight density of the soil	kN/m^3
$E_{50,ref}$	Modules of elasticity of soil at a 50% deviatoric stress	kN/m^2
$E_{oed,ref}$	Oedometric stiffness modulus	kN/m^2
$E_{ur,ref}$	Unloading reloading stiffness modulus	kN/m^2
ψ	Angle of dilatancy	°
φ	Angle of internal friction	°
δ	Structure-ground interface friction angle	°
C'	Effective cohesion in drained conditions	kN/m^2
υ	Coefficient for Lateral contractions or Poisson-factor	$-$
σ'_h	Horizontal pore pressure	N/mm^2

1 Introduction

Global standards of ship dimensions of seagoing vessels keep increasing however, the water depths of the port basins remain equal. The global growth of the size and draft of the seagoing vessels are the reason of the growing demand for mooring facilities with more draft. To meet the increasing demand for deeper mooring facilities the port beds have to be deepened. The assets, such as quay walls and jetties, need to be adjusted without compromising the reliability level of the existing assets.

The Port of Rotterdam Authority is aware of the increasing demand for deeper berth facilities. For that reason, the feasibility of deepening the port basins are initiated. These projects include the deepening of the port basins and upgrading of the existing assets. One of these deepening projects is the deepening of the New Waterway and the entire Botlek area the upcoming years. The biggest investment considers the dredging of the New Waterway and the Botlek. The Botlek port basins will be deepened with approximately 2 meters to NAP -15.90 meters (Port of Rotterdam Authority 3, 2016). The renewal of the assets, such as quay walls have to be performed because of the planned deepening.

The Botlek area is one of the largest 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. Besides of the petrochemical industry, the Port of Rotterdam also focuses on the energy transition in the Botlek. That energy transition consists of the exploration of new markets, such as offshore wind and bio-based energy (Port of Rotterdam Authority 5, 2017). To explore these new markets the interests of deepening the Botlek increases. The location of the Botlek area is presented in figure 1 in the blue rectangle.

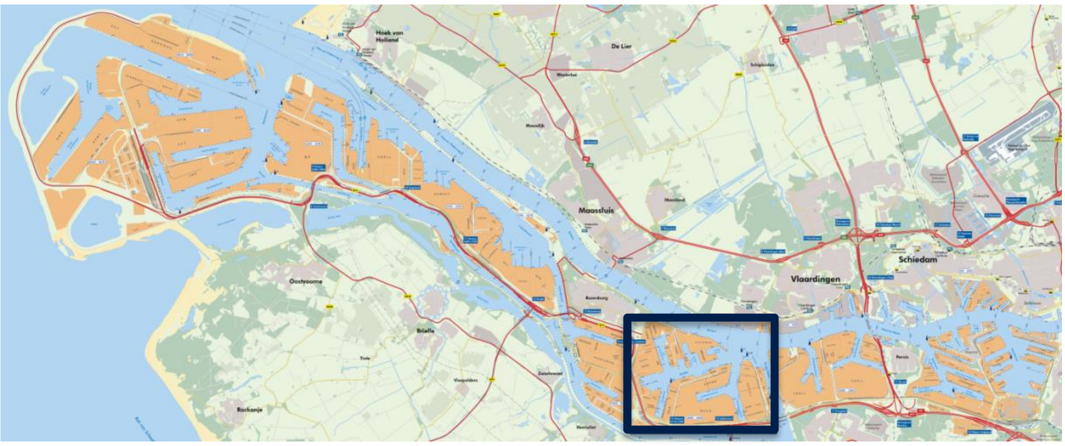


Figure 1 Port of Rotterdam overview with the framed Botlek area (Port of Rotterdam Authority 1, 2017)

After the deepening of the Botlek area is completed, vessels with a draft of 15 meters, such as New Panamax with DWT of 120,000, can be facilitated in the Botlek. The deepening of the harbour brings engineering challenges, such as influence on the strength of the existing quay walls and jetties. Safety is crucial, so the strength of these assets has to be sufficient after deepening.

Most of the quay walls in the Botlek area are constructed around 1960. For that reason, the assets are close to the end of the design lifetime. It should be noted that end of design lifetime does not automatically mean the end of service life. Most of the quay walls consist of anchored retaining walls (combined and single sheet pile), with or without a relieving platform. Over time these combi walls are improved, but the principle of the combi wall remained the same; primary piles with intermediate sheet piles. These primary piles were in 1960 Peiner piles as shown in figure 2 (Peiner Träger GmbH, 2017). At present, the primary piles are tubular steel piles as also shown in figure 2.

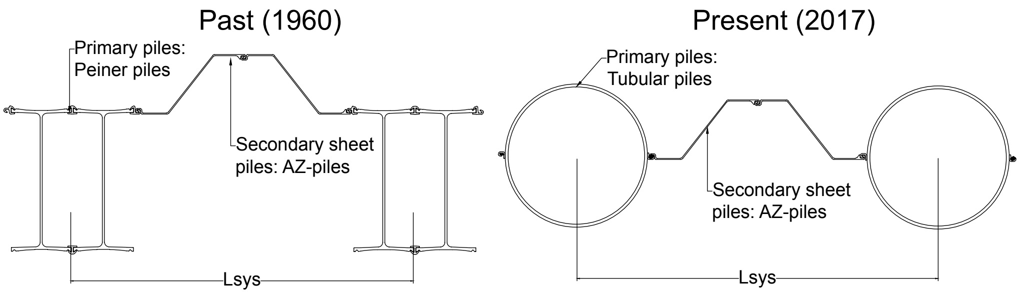


Figure 2 typical combi wall type in the Port of Rotterdam

The division of quay wall types in the port of Rotterdam is various, as shown in Figure 3. In that figure, it is shown that the combi walls are 38% of the total quay constructions in the Port of Rotterdam. For that reason, the combi walls are assumed to be representative of main quay wall type of the Port of Rotterdam.

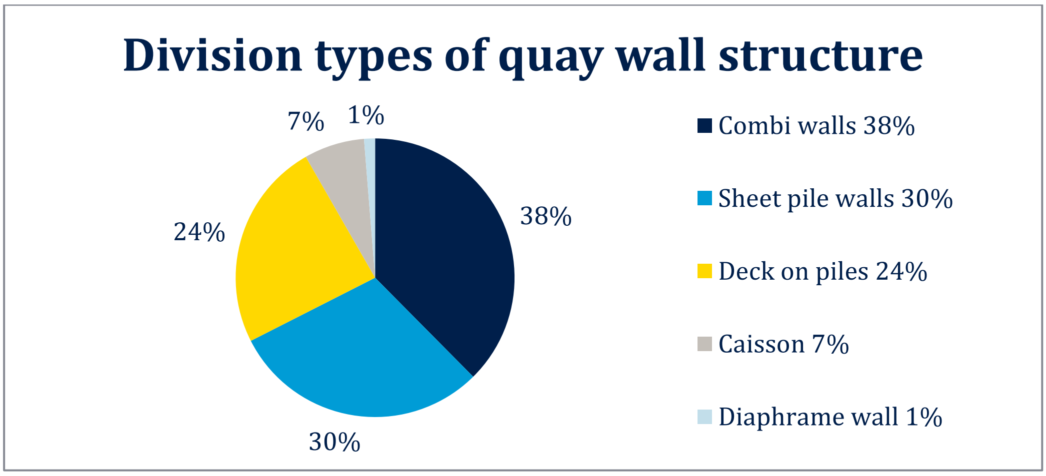


Figure 3 deviation of quay wall types in the Port of Rotterdam (Port of Rotterdam Authority 4, 2017)

Prevention of downtime of port activities for terminals is very important to guarantee a safe environment to work and invest in the Port of Rotterdam. Existing terminals have to be able to continue their business activities without a lot of hindrance and interruptions. The construction of a new quay structure generally interrupt the business activities of the existing terminals. To date is not clear if it is possible to upgrade existing quay walls without interrupting terminal activities. Terminals can then continue their operations during these upgrades without major downtime. Besides of the lower downtime the costs of upgrades could be less compared to renewals or complete renovations. For these reasons, the Port of Rotterdam Authority is interested in possibilities for upgrading existing quay walls

1.1 Research objective

The aim of the research is to define a suitable solution for deepening a reference combi-wall. This solution should preferably be applicable to other types of quay wall as well. The insights of the effects of the solution have to be acquired by modelling the soil-structure interface as close as possible.

The following main research question is formulated:

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?

The secondary research questions are formulated as:

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?

1.2 Success factors of the research

This research succeeds if the following goals are accomplished:

- The Port of Rotterdam Authority intends to apply the thesis results to multiple types of quay walls;
- Inventory of solutions could be implemented in future projects;
- Lifetime extension and deepening combined in one solution;
- Realistic modelling of soil-structure interaction in a Finite Element Method model;
- Technically feasible solution to deepen at least 2 meters in front of combi walls;
- Third-party validation by market parties.

1.3 Method of approach

The research consists of seven phases, during these phases different research methods are used. The phases are listed on the flowchart on Figure 4. The left side of the figure presents the phases, and the right side presents the phase results.

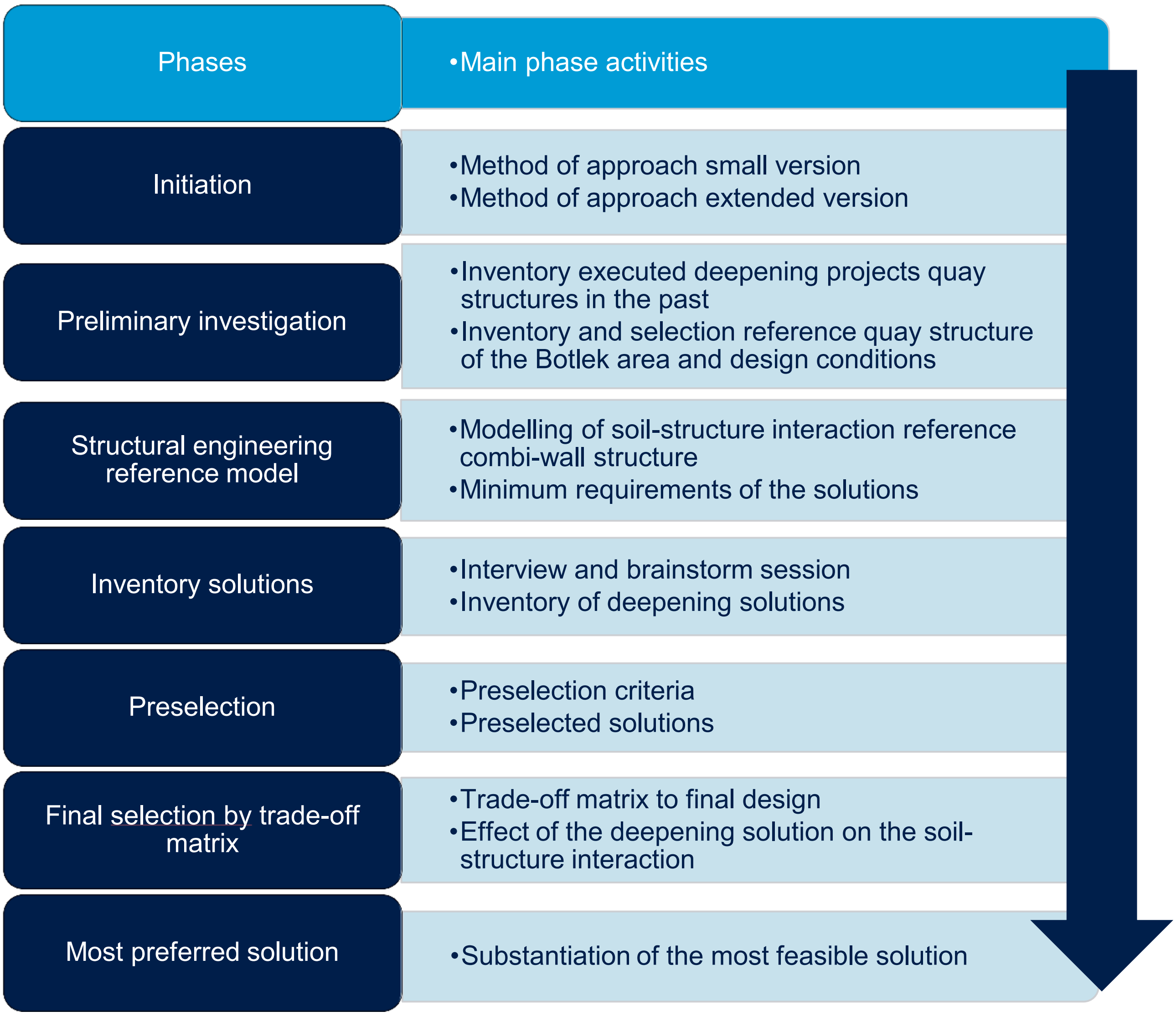


Figure 4 phases graduation thesis

1.4 Demarcation

This thesis includes the following constraints:

- This thesis focuses preliminary on the technically feasible solution to deepen existing quay wall, and all other aspects, such as assumptions of the reference model, are subordinate;
- This thesis is an exploratory research, so soil conditions are derived from limited investigation. The representative values were defined through practical formulas of the CUR166 and the experience of experts of the Port of Rotterdam and Witteveen+Bos;
- During the execution of the research, a reference Plaxis model is arranged which is checked by experts of Witteveen+Bos. A sensitivity analyses with the most important parameters is performed which derives the same values. For these reasons the model is assumed as acceptable model;
- The arbitrary selection of the solutions in the pre-selection, this selection is arranged with experts and the supervisor, so is assumed as a validated selection;
- Degradation of the structure elements was not taken in account into this research, because the degradation is project specific and this is a multidisciplinary research;
- The business case of the deepening solutions are not part of the research;
- The schematization of the reference model into the Plaxis model is representative of the reference model with parameters without safety factor and with a conservative approach. In order to these constraints the model schematization is established and validated by experts of the Port of Rotterdam, Witteveen+Bos and the University of Applied Sciences Rotterdam.

1.5 Layout of the report

The layout of this report is as follows. After this introductory chapter, the preliminary investigation is presented in chapter 2. That investigation includes an inventory of executed deepening project around the world, an inventory of quay wall structures in the Botlek area and the selection of the reference quay structure for this thesis.

Subsequently, in chapter 3 the structural engineering of the reference model is presented. That chapter 3 includes the validation of the model, the basics of design e.g. ground parameters, loads and structural modulation and the structural assessment of the reference model before deepening and after deepening. That structural assessment results in critical structural members and critical failure mechanism of the structure and finally the minimum requirement which the deepening solutions must meet.

Chapter 4 is about the inventory of deepening solutions, which are arranged by interviews with experts, brainstorm sessions and the preliminary investigation of executed deepening projects. A Preselection is arranged to filter the solution by minimum criteria. These preselected solutions proceed to the trade-off selection which is presented in chapter 5. This chapter also includes the substantiation of the selected solutions, the trade-off criteria and the final trade-off selection. This trade-off selection results in the most preferred solution. This most preferred solution is substantiated in chapter 6.

In chapter 7 the conclusions are given, and afterwards the recommendations are presented in chapter 8. The bibliography of the consulted literature is presented in chapter 9. Finally, the appendixes are enclosed to this thesis.

2 Preliminary investigation

In order to get familiarised with the subject and select the reference quay structure for this thesis the preliminary investigation is performed. This preliminary investigation discusses the first and the second secondary research questions. The result of this chapter is an inventory of executed deepening projects around the world, an inventory of the type of quay walls in the Botlek area and the selection of the reference quay wall structure for this thesis.

The first secondary question: “What adjustments have been made in the past, to deepened combi wall structures in the Port of Rotterdam and other ports?”, is answered in the preliminary investigation, see separately provide report 2. Executed deepening project. The executed projects are used to determine the solutions to deepen the port bed. The deepening projects which are found and investigated are:

- Sint Laurenshaven, Port of Rotterdam, The Netherlands (Port of Rotterdam Authority 2, 2017);
- Pier 6, Waalhaven, Port of Rotterdam, The Netherlands (Tijssen & Veldhuijzen, 1998);
- Port of Felixstowe, United Kingdom (Tijssen & Veldhuijzen, 1998);
- Port of Ravenna, Italy (Mollahasani, 2014);
- Port of Kaohsiung, Taiwan (ArcelorMittal, 2009);
- Port of Yokohama, Japan (PARI, 2017).

The deepening of the port beds is executed next to several types of quay structures, such as 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 of 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.

2.1 Type of quay wall structures

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 transhipment of freights, carry loads, crane loads and storage loads;
- Mooring: the structure must provide water depth for the bigger vessels, enough berth and mooring facilities.

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

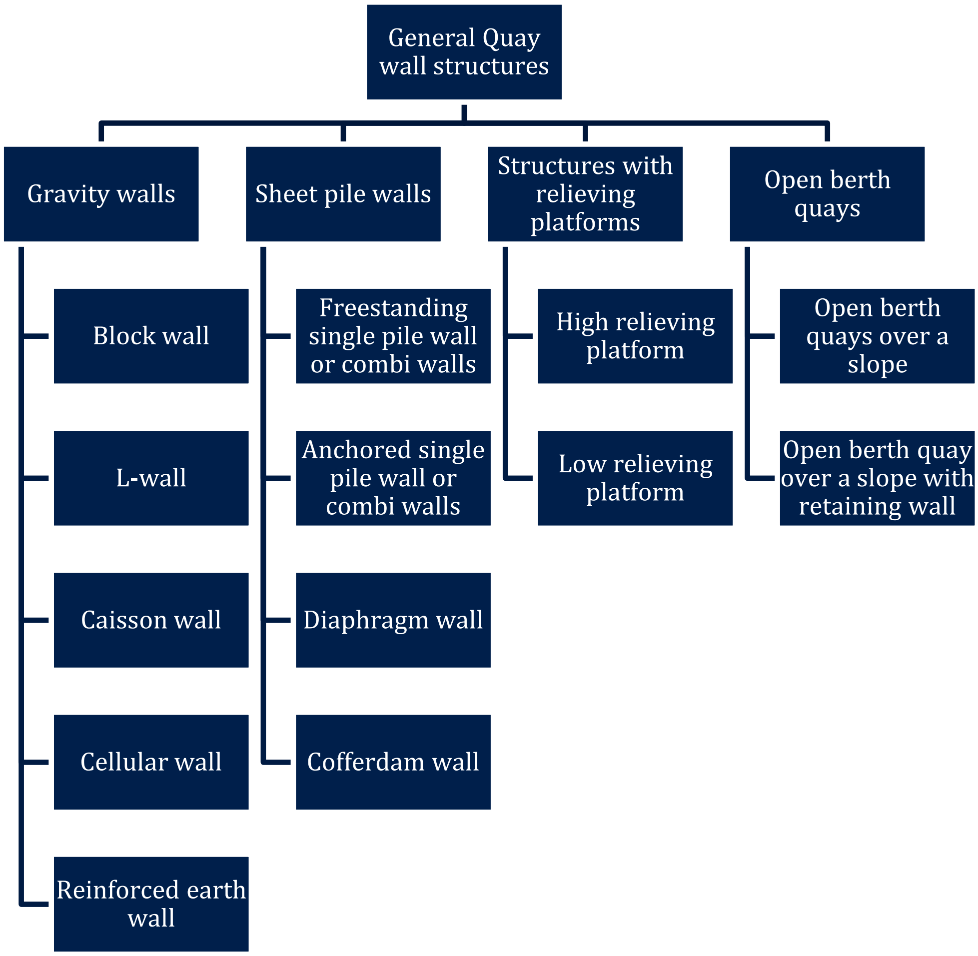


Figure 5 main types of quay wall structures

This thesis focuses on structures with relieving platforms because these structures are constructed in 38% of the total areal in the Port of Rotterdam, as shown in figure 3.

The relieving platform reduces the horizontal load on the front of the wall. These loads on the quay surface are directly transferred to the subsoil by the concrete platform. The entire structure consists of a retaining combi wall, a concrete relieving floor and bearing piles. Relieving platform structure can be divided into two types, which are described in the following bullet point.

- 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 6.

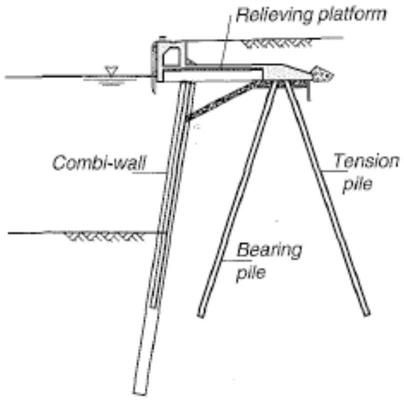


Figure 6 principle of a high relieving platform (SBRCURnet, 2014)

- 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 of the combi wall and on the land on the bearing piles and one row of tension piles. Cast iron saddles between the relieving platform and the combi wall create a hinge, so the vertical forces are not transferred to the wall. The principle of a low relieving platform is pictured in Figure 7

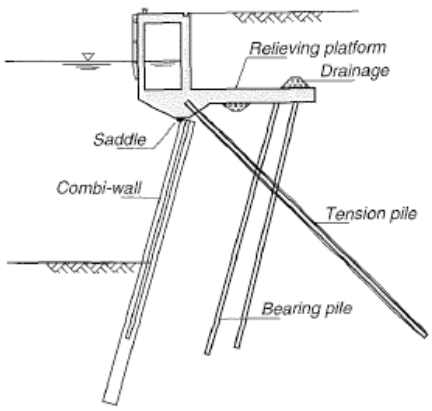


Figure 7 principle of a low relieving platform (SBRCURnet, 2014)

2.2 Reference quay wall Botlek area

The second secondary question “What is a representative combi-wall of the Botlek area?” is investigated in the preliminary investigation. That investigation concludes the reference structure for this thesis. See separated provided report 3: Reference structures Botlek area for the investigated structures.

The types of quay walls in the Botlek area are numerous. The reference model is chosen because of the executed deepening before, the challenging soil-structure interface, challenging inclined angle of the front wall and the Peiner piles combi-wall which are frequently used 50 years ago. The reference structure for this thesis is the quay wall structure of the Sint Laurenshaven, which is shown in Figure 8. In Figure 9 the principle cross section of the reference model with the applied deepening solutions of additional asphalt matrasses is presented.

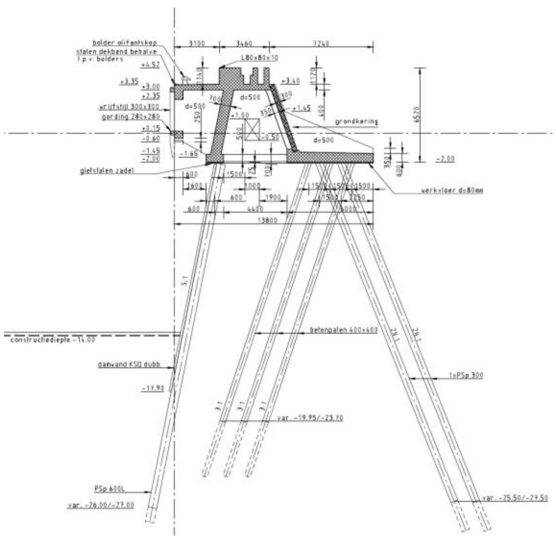


Figure 8 principle cross section reference without adjustment

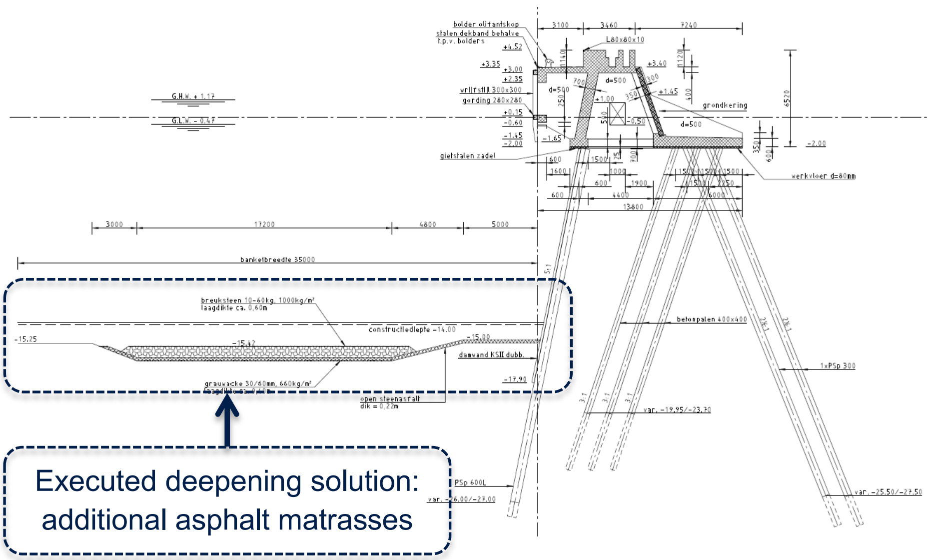


Figure 9 principle cross section reference structure with adjustment

3 Structural assessment existing quay wall

The structural assessment is described in this chapter, which includes the basis of design, the assessment before deepening, the assessment after deepening, the effects before and after deepening leads to the requirements which the deepening solutions must meet. These requirements are related to the critical structure elements and corresponding critical failure mechanism.

The critical structure elements and corresponding critical failure mechanism are determined by modelling the reference quay wall in a Finite Element Method (FEM) model. That FEM model is chosen in this thesis because of the following reasons:

- The geometry of the structure, in order to model the inclination of the combi wall;
- The opportunity to model the possible solutions for deepening, such as local grouting;
- The advanced soil-structure interaction (Technology, 2012);
- The advanced behaviour of the clay layers because of the different stiffness parameters into Plaxis.

The following aspects are considered as the uncertainty of the Plaxis model:

- The structural assessment is done with a conservative approach and with partial factors of 1.0 because the purpose is to derive the effect of the deepening and the effect of the solutions and not to design and optimise a deepening solution. The results can deviate, if the parameters are different or if the partial factors are considered as the NEN9997-1:2012, which are based on the probabilistic design approach;
- Extremely high surface load;
- The inaccuracy margin of ± 30% of Plaxis deformations according to the CUR 166 (SBRCURnet, 2008);
- Schematisation of the structure into Plaxis.

A principle cross section of the reference quay structure before and after deepening with a Panamax vessel is presented in Figure 10.

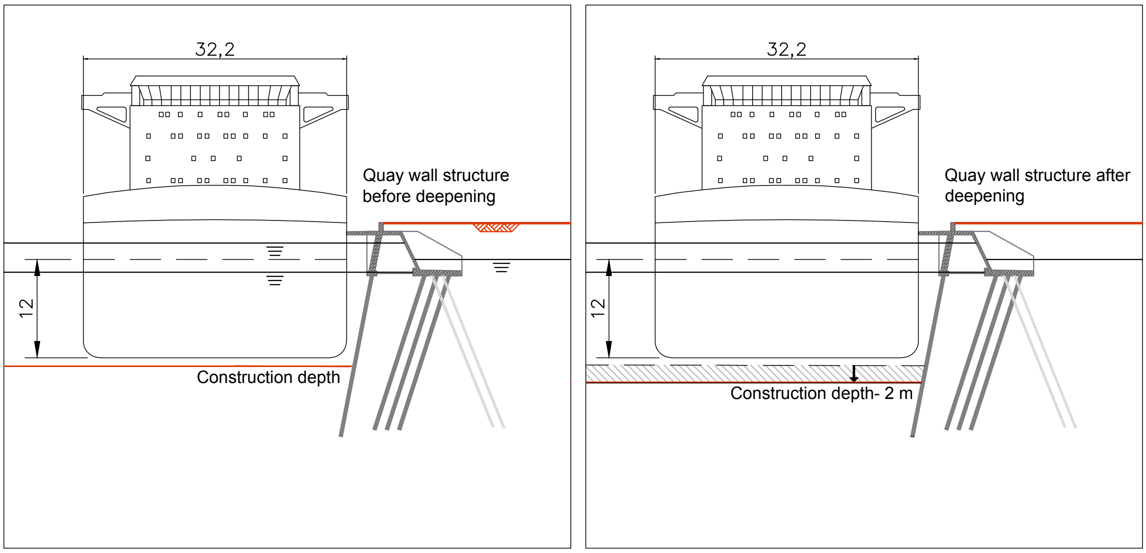
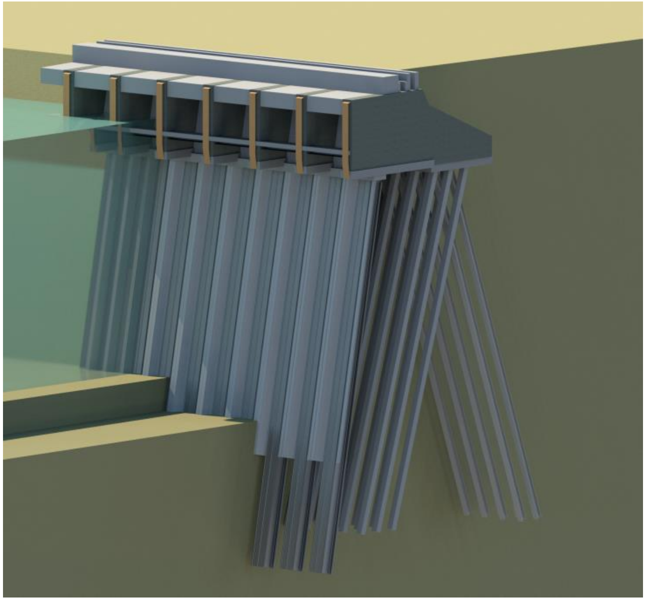


Figure 10 principle cross section before deepening (left side), and after deepening (right side)

3.1 Basis of design

The structure consists of a foundation and a superstructure. This superstructure can be divided into the light grey parts of the concrete superstructure and the dark grey parts counterforts as shown in Figure 11. The front view and cross section of the superstructure are shown in Figure 12. The total description and the basis of the design are provided in the separately provided report 4: Structural engineering reference structure.



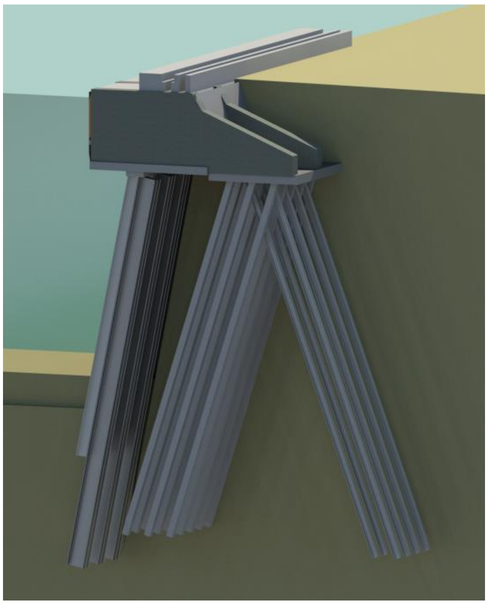


Figure 11 overview of the structure

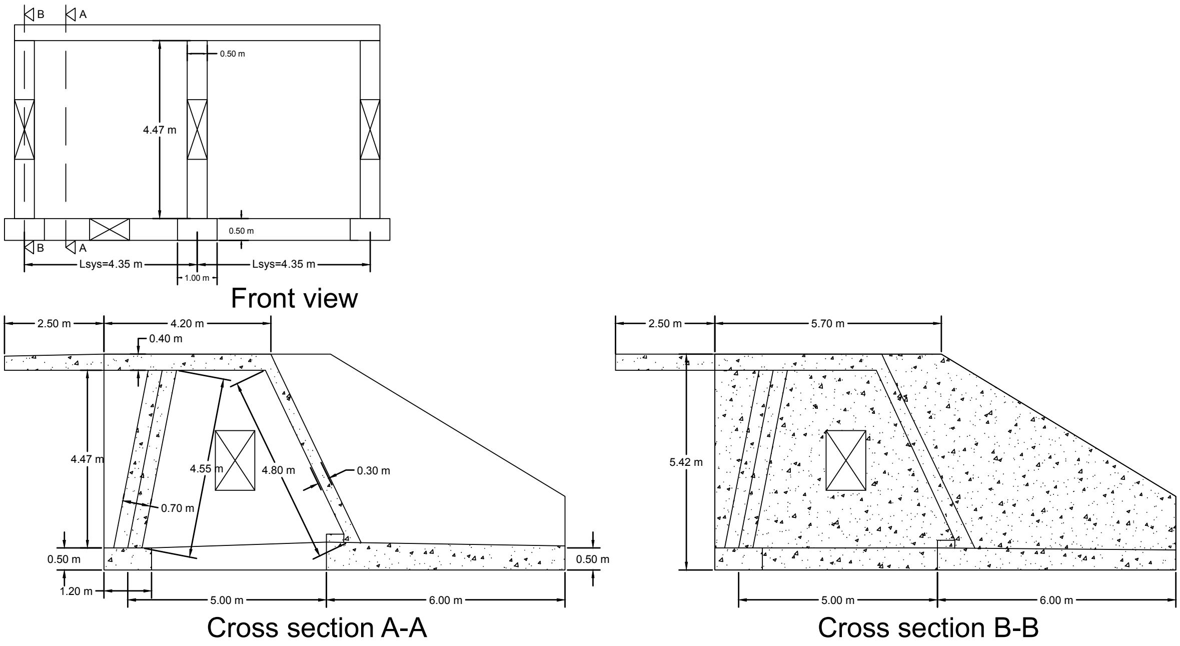


Figure 12 overall overview of the superstructure front view and cross section A-A and B-B

3.1.1 Modelling into Plaxis

Soil-structure interaction can be applied into Plaxis by different soil models. The hardening soil model includes the stiffness and the deformations of the soil, so the results of the model are more realistic (Plaxis B.V., 2016). For those reasons, the hardening soil model is the most suitable soil model for relieving structure because of the advanced soil-structure interface.

3.1.1.1 Ground

The ground parameters are determined by the original calculation and with the formulas of the CUR 2003-7 (SBRCURnet, 2003). The parameters are determined according to the cone penetration test pictured on Figure 13. The soil parameters of the Plaxis model are shown in Table 1.

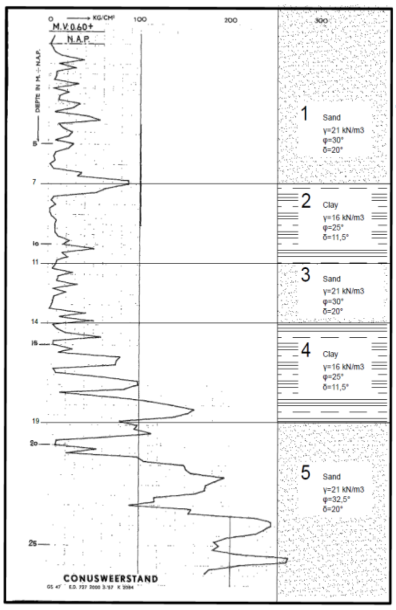


Table 1 soil parameters Plaxis

Soil parameter		1 Sand	2 Clay	3 Sand	4 Clay	5 Sand
Symbol	Unit					
γsat	kN/m3	21	16	21	16	21
γunsat	kN/m3	18	6	18	6	18
E50 ref	kPa	28600	6000	18400	6000	31700
Eoed ref	kPa	28600	3000	18400	3000	31700
Eur ref	kPa	85800	15000	55200	15000	95100
ψ	°	30	0	30	0	32.5
φ	°	30	25	30	25	32.5
Rinter	-	0.8	0.66	0.8	0.66	0.8
C'ref	kPa	0	10	0	10	0
m	-	0.5	1	0.5	1	0.5

Figure 13 cone penetration test results

3.1.1.2 Loads

The modulation of the loads is arranged according to the original calculation of the reference quay wall. The schematisation of the present loads is shown in Figure 14.

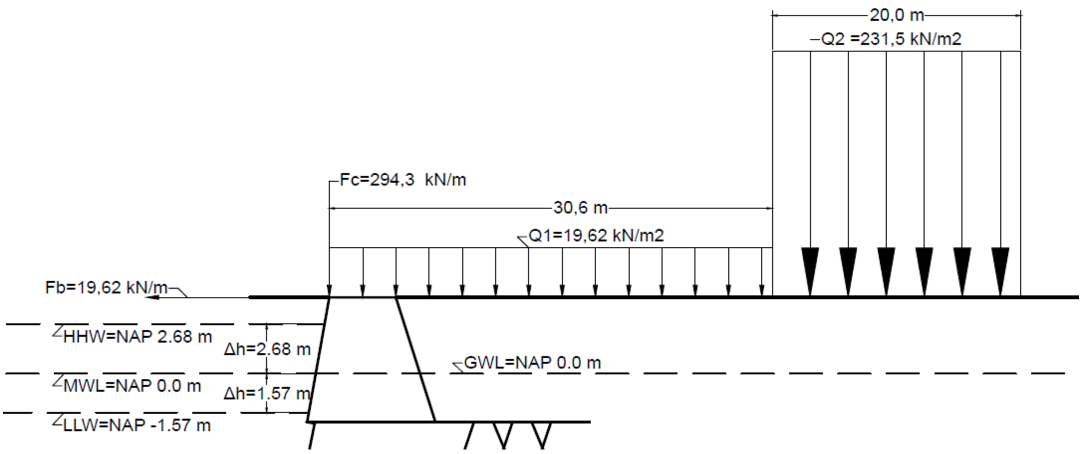


Figure 14 modulation of the loads

The different loads are modulated into Plaxis in various load combination to determine the dominant load combination for this research. The different load combinations which are compared in the Plaxis model are shown in Table 2. The different load combinations before deepening are compared with each other in the structural assessment. Load combination 4 is found to be the dominant load combination since it provides the highest forces and moments and the lowest safety factor. For these reasons load combination 4 is used in the next phases of this thesis. Load combination 4 is used to determine the effect of the deepening and to identify effects of the solutions according to this reference model. The dominant load combination is framed in dark blue in Table 2.

Table 2 load combinations

load combination	Groundwater level	Waterlevel	Q1	Q2	Fb	Fc	ΔH
0	NAP 0.00 m	NAP 0.00 m					0.00 m
1	NAP 0.00 m	NAP 0.00 m	x		x	x	0.00 m
2	NAP 0.00 m	NAP 0.00 m	x	x	x	x	0.00 m
3	NAP 0.00 m	NAP -1.57 m	x		x	x	- 1.57 m
4	NAP 0.00 m	NAP -1.57 m	x	x	x	x	- 1.57 m
5	NAP 0.00 m	NAP 2.68 m	x		x	x	2.68 m
6	NAP 0.00 m	NAP 2.68 m	x	x	x	x	2.38 m

3.1.1.3 Structure modulation

The modulation of the superstructure is arranged in cooperation with Plaxis experts. The modulation of the complete structure is done according to the schematization as Figure 15. The summary of the parameters corresponding to the schematization of Figure 15 is shown in Table 3.

Table 3 material parameter Plaxis

Plate parameters			
nr.	Name	EA (kN/m)	EI (kNm/m)
1	Horizontal beam 1	$10.7 * 10^6$	$1.0 * 10^5$
2	Horizontal beam 2	$11.9 * 10^6$	$1.6 * 10^5$
3	Vertical beam 1	$20.8 * 10^6$	$8.5 * 10^5$
4	Vertical beam 2	$8.9 * 10^6$	$0.7 * 10^5$
5	Counterfort full	$18.7 * 10^6$	$427 * 10^5$
6	Counterfort reduced	$26.9 * 10^6$	$106 * 10^5$
7	Front wall (Psp60L)	$5.2 * 10^6$	$3.5 * 10^5$
Node-to-Node parameters			
nr.	Name	EA (kN/m)	Lspacing (m)
8	Tension pile (Psp30)	$3.25 * 10^6$	1.5
Embedded beam row parameters			
nr.	Name	A (m2)	Lspacing (m)
9	Bearing pile	0.16	1.5
10	Tension pile end piece (Psp30)	0.0155	1.5

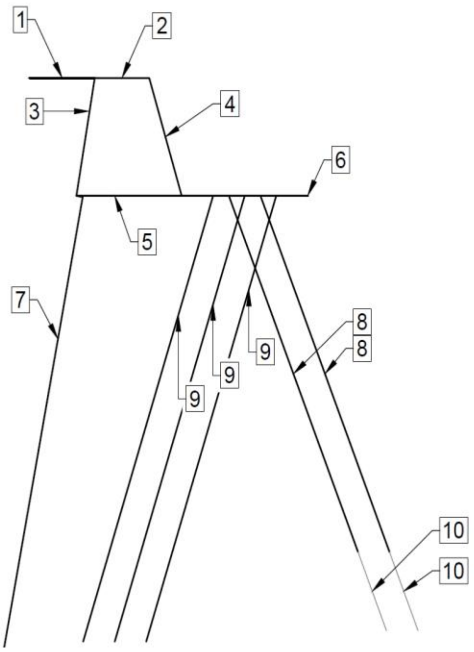


Figure 15 number of the structural elements

3.2 Structural assessment existing structure

The results of the structure before deepening and after deepening are determined in the Plaxis model and are shown in the sub-chapters below. The modelling of the structure into Plaxis is done as Figure 16.

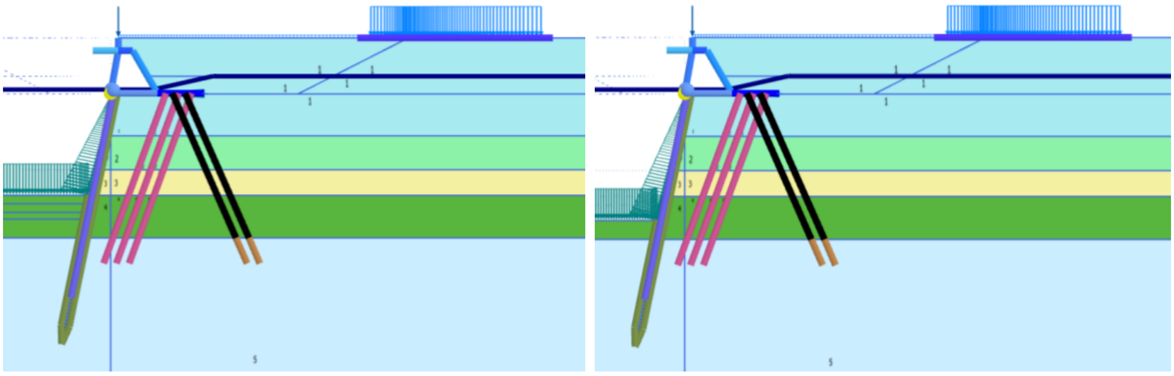


Figure 16 Plaxis model before deepening (left side) and after deepening (right side)

3.2.1 Compare before deepening and after deepening

To determine the critical structure elements and corresponding critical failure mechanism after deepening the reference structure is deepened 2 meters. This deepening is modulated in the dominant load combination 4. The total displacements of the model before and after deepening are shown in Figure 17. Afterwards, the total displacement of the Plaxis model after the φ-C reduction to determine the $\sum Msf = \frac{\tan \varphi_{input}}{\tan \varphi_{reduced}} = \frac{c_{input}}{c_{reduced}}$ are shown in Figure 18

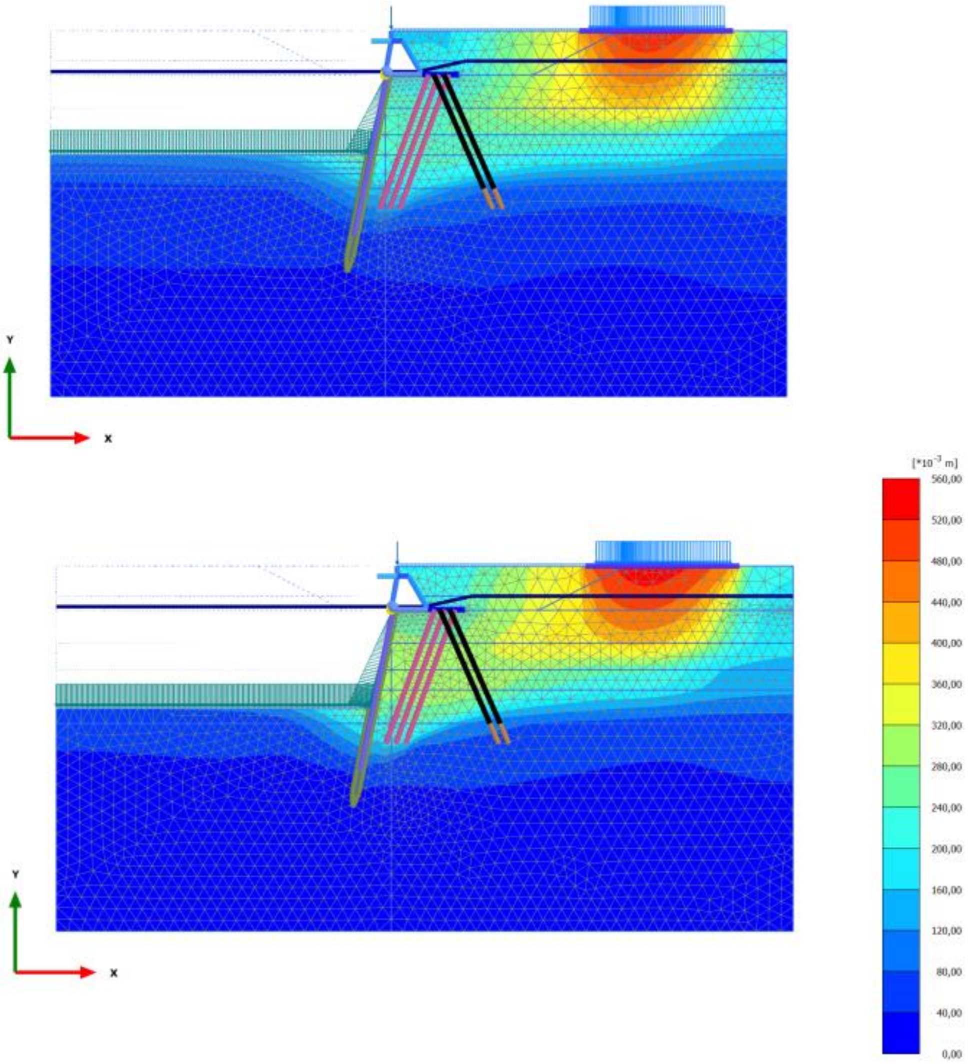


Figure 17 total displacements before deepening (upper figure) and after deepening (bottom figure)

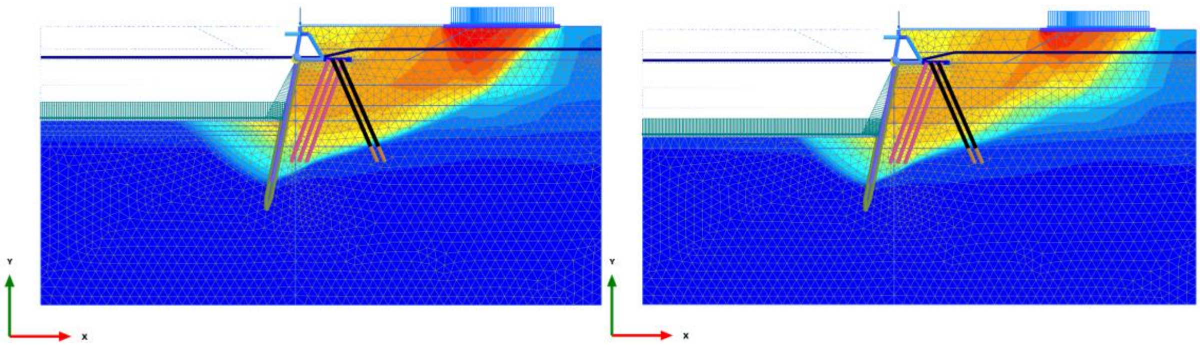


Figure 18 total displacements $\sum Msf$ before deepening (left figure) and after deepening (right figure)

The results of the deepening on the forces within the front wall are shown and compared in Figure 19.

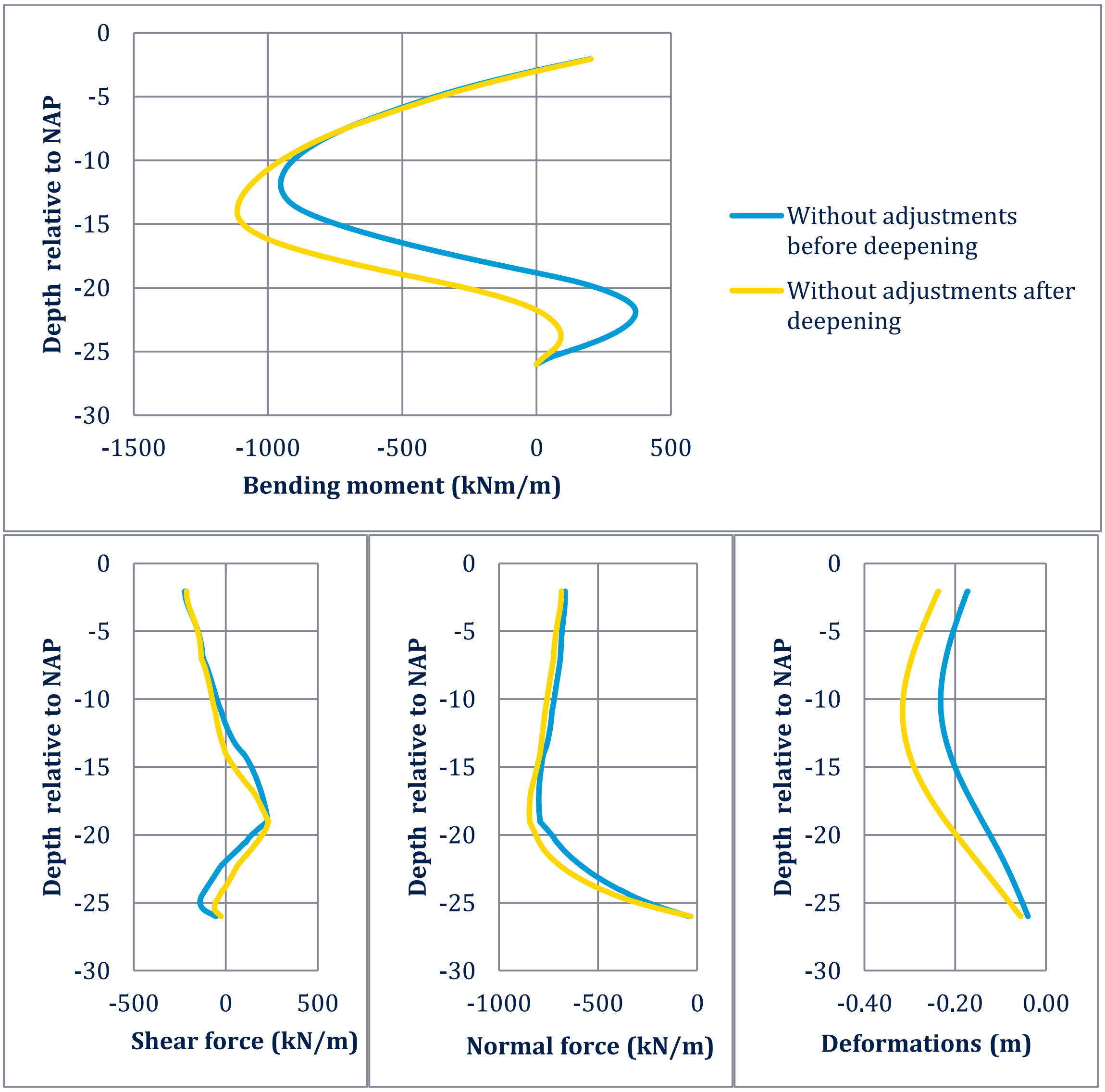


Figure 19 results of the front wall: bending moment, shear force, normal force and deformations, before (blue) and after deepening (yellow)

3.3 Relative effect of deepening

The summary of the deviation and the effect of the deepening on the structure is shown in the table below. The deviation is calculated after deepening to before deepening. The results of the comparison are shown in Table 4.

Table 4 summary of the values and the effect of the deepening to the structure elements

Phase	Annotation	Without adjustments before deepening	Without adjustments after deepening	Deviation	Maximum value
ULS					
Geotechnical					
Safety factor	-	1.246	1.121	-10.03%	-
Structural					
Front wall $\sigma_{max} = \frac{M}{W} + \frac{N}{A}$	N/mm2	212	245	15.71%	235
Shear force front wall	kN/m	229	232	1.49%	6110
Normal force bearing pile 3	kN	595	599	0.67%	1600
Bending moment bearing pile 3	kNm/m	92	119	29.35%	213
Normal force tension pile 1	kN	95	119	25.26%	1190
ULS					
Deformations x top quay wall	m	0.13	0.19	40.25%	-

According to the results of the Plaxis calculation the following structural members are significantly influenced by the deepening, so these structural members are critical:

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

Deformations are not critical because the structure must deform to be functional. The following failure mechanisms are critical according to deepening the port bed with 2.0 meters. The visualisation of the failure mechanism is shown in Figure 20 and Figure 21.

- Structural:
  - Failure of front wall (1);
  - Failure of tension piles (2);
- Geotechnical:
  - Insufficient passive resistance (3);
  - Piping/Local failure of geotechnical stability between the primary piles (4);
  - Failure of anchor/pile tension resistance (5).

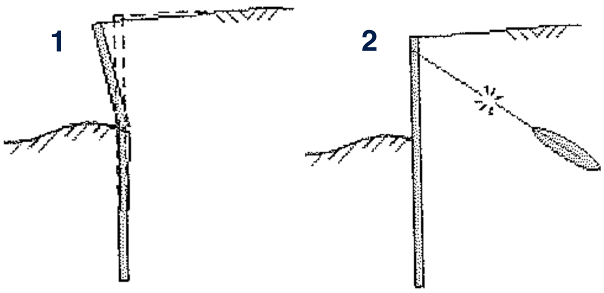


Figure 20 visualisation of the structural failure mechanism (SBRCURnet, 2014)

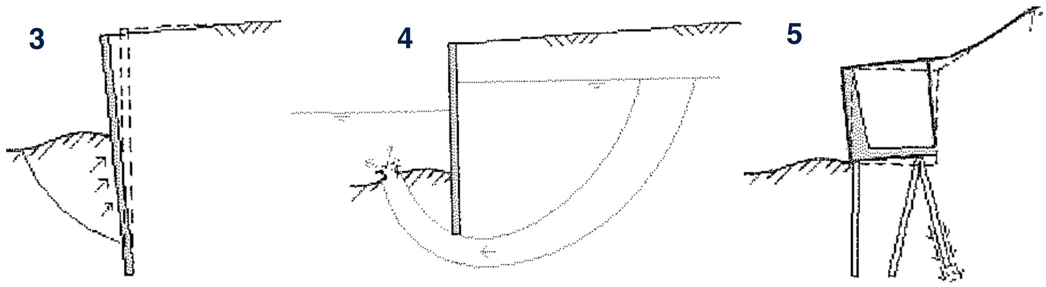


Figure 21 visualisation of the geotechnical failure mechanism (SBRCURnet, 2014)

The structural assessment concludes which failure mechanism and which structure elements are critical. These critical structure elements do have a reliability level, expressed in stresses, forces of $\sum Msf$ factor, before deepening. The most preferred deepening solution must meet the requirements, which are the value of the structural members before deepening, to be preferred without reducing the reliability. The requirements are assumed to be 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 (-2.5%) and upper limit (+2.5%), which are shown in Table 5.

Table 5 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.

4 Inventory and preselection

The inventory of alternatives is arranged according to executed deepening projects in the past, brainstorm and interview sessions with experts and literature research (Douairi & De Gijt, 2013). These methods of research provided twenty-seven ideas for deepening solutions. These solutions are evaluated by conducting a preselection, on the basis of minimum criteria. The result of this chapter is an inventory of solutions and a preselection leading to preferred solutions which proceed to the detailed trade-off analysis.

4.1 Type of solutions

The solutions are divided into different type of solutions; typed included of this research and types excluded from this research. This deviation is presented in Figure 22. The light blue hatched types are excluded from this research and the dark blue hatched types are part of this research.

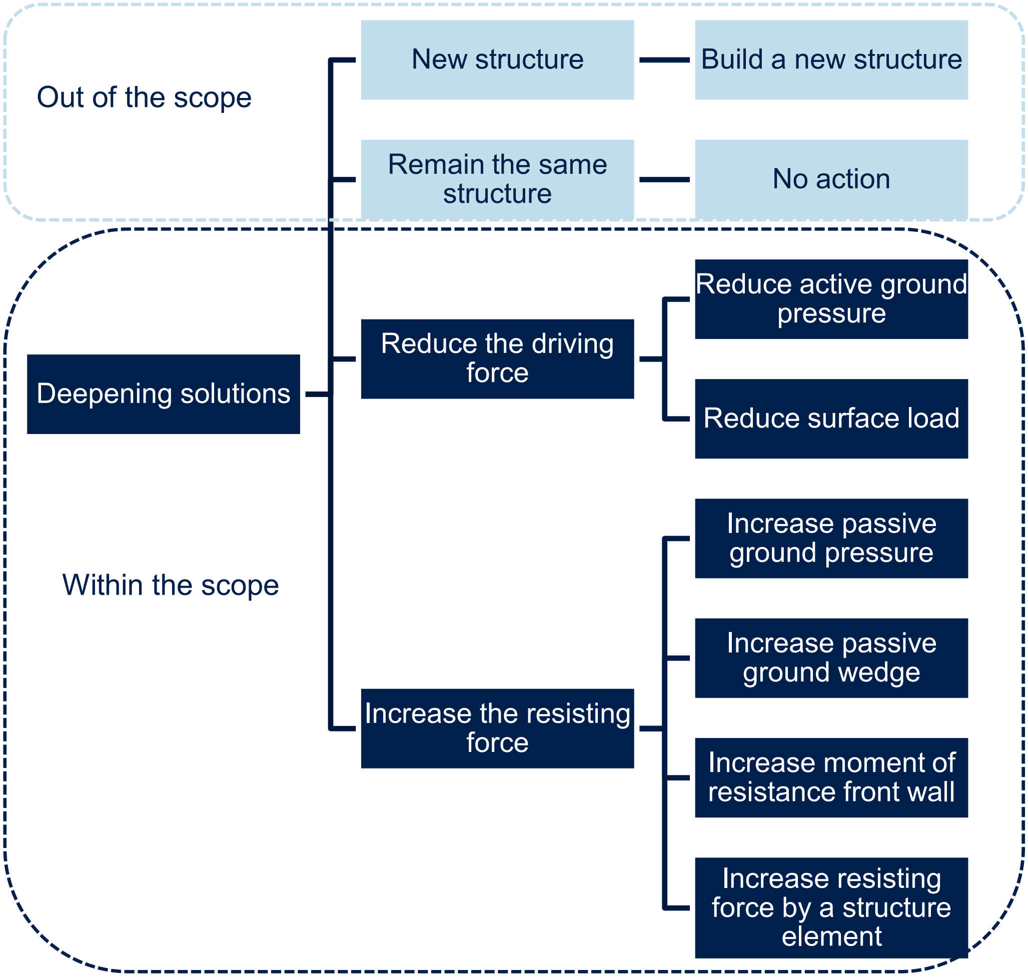


Figure 22 type of upgrading solutions

4.2 Inventory

The inventory of possibilities for deepening is derived by combining the preliminary investigation, brainstorm session, interviews with expert and literature research. The twenty-seven solutions which are derived are shown in Figure 23 and Figure 24.

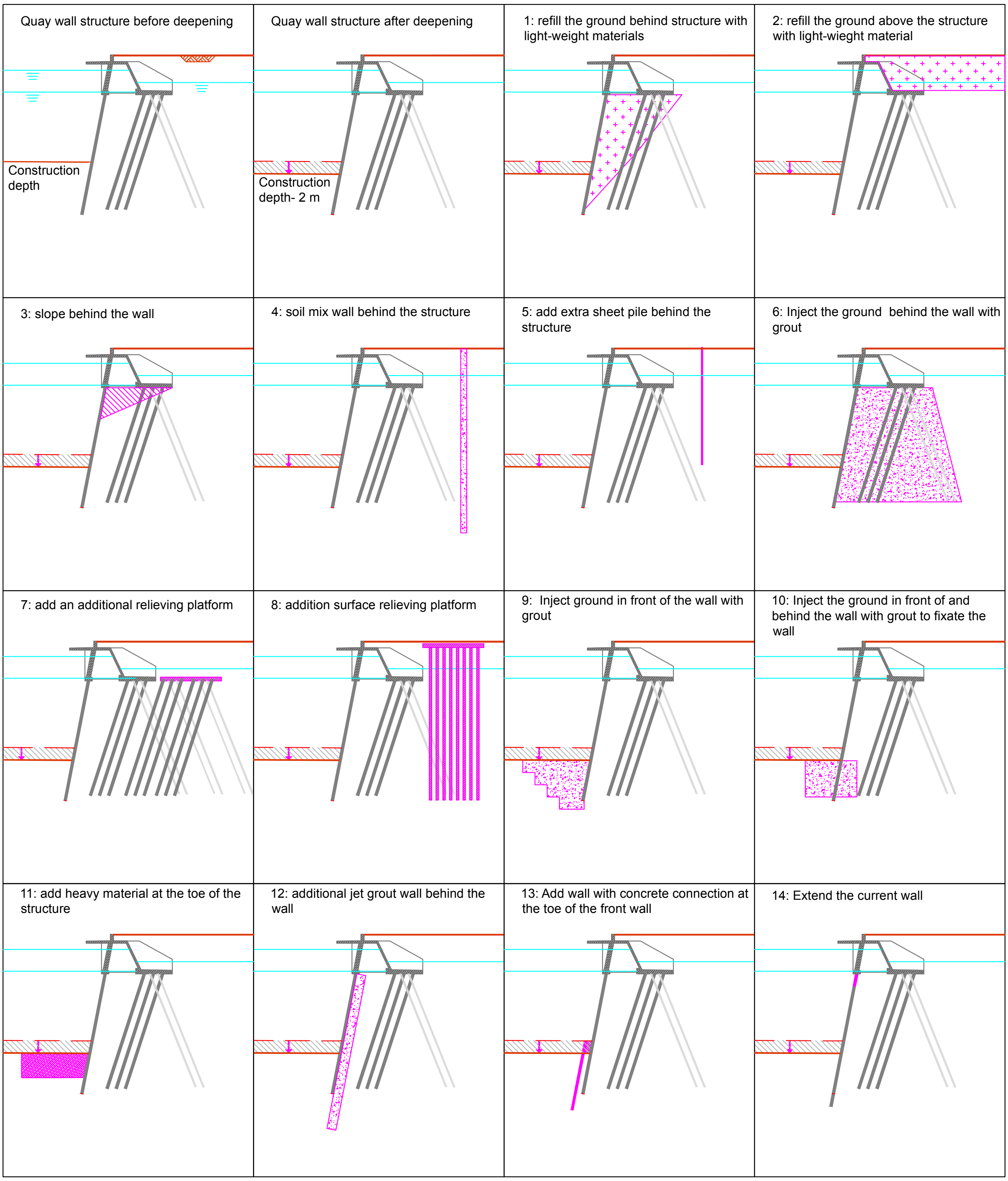


Figure 23 inventory of deepening solutions

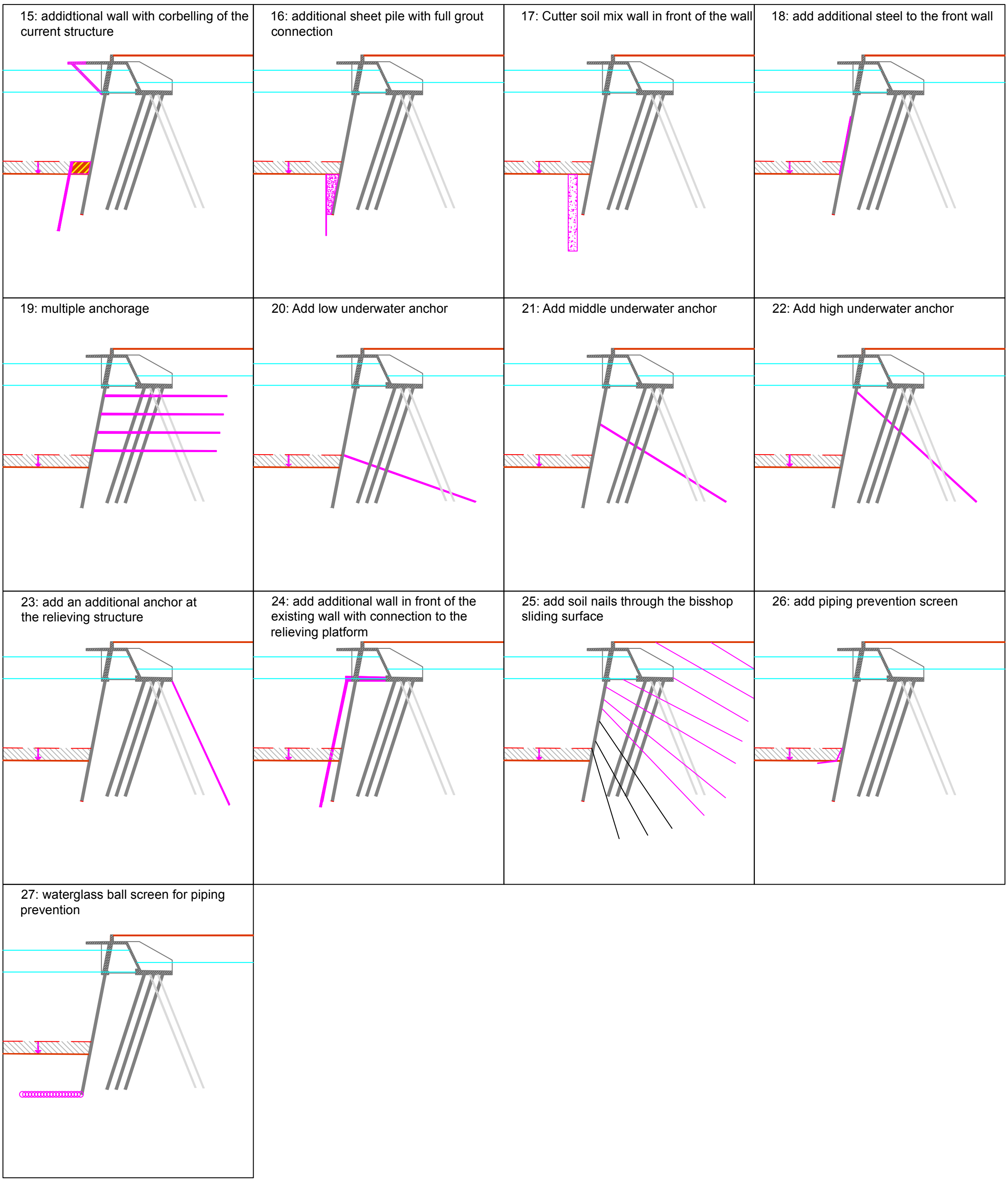


Figure 24 inventory of deepening solutions

4.3 Preselection

The selection is performed by evaluating minimum criteria. The solutions which comply with all of the minimum selection criteria are subjected to a detailed trade-off analysis.

4.3.1 Preselection criteria

The solutions are ranked by the following preselection criteria:

- At least 2 meters deepening;
    - The minimum deepening of 2 meters needs to be achieved by the application of the solution. The solution is ranked with a Yes, if the deepening of 2 meters can be accomplished. The solution is ranked with a No is the 2 meters deepening not can be achieved;
- Multidisciplinary solution;
    - The solutions can be project-specific or multidisciplinary. This research focuses on multidisciplinary solutions. These solutions are applicable to several types of quay walls with relieving platforms. The multidisciplinary solutions solve multiple failure mechanisms compared to project specific solutions which solve one type of failure mechanism such as piping. The project-specific solutions are ranked with a No and multidisciplinary solutions are ranked with a Yes;
- Technical feasibility;
    - The technical feasibility of the solution is also ranked with, Yes or no. The ranking of the feasibility of the solution is performed by interviewing the experts. The solutions proceed with a Yes, if the solutions are assumed to be feasible to perform. The solutions are ranked with a No, if the solutions are not assumed to be feasible to perform;
- New structure or upgrade
    - A solution can be an upgrade or a totally new structure. This research focuses on the upgrade of the structure. The solutions proceed with a Yes, if the solutions are an upgrade of the existing structure diversely the solutions are ranked with a No, if the solutions are a total new structure.

4.4 Preselection

The selection of the solutions is arranged and validated by experts of the Port of Rotterdam Authority. Only the solutions complying with the preselection criteria proceed to the trade-off analysis. These solutions are shown in Table 6 and Figure 25. The inventory of the solutions provides in particular solutions with more or less the same adjustment. For that reason, these solutions are ordered by solutions category. The solution numbers which match with the solution categories are shown in the second column of Table 6. A detailed score of the solutions is attached in Appendix A.

Table 6 ranking of the preselected solutions

Solution category	Solutions number	At least 2 meter deepening	Multidisciplinary solution	Structural feasibility	New structure or upgrade	Total score
A. Excavation below the relieving floor	3	✓	✓	✓	✓	4
B. Grout injection behind the retaining wall	6	✓	✓	✓	✓	4
C. Grout injection at the toe of the retaining wall	9,10	✓	✓	✓	✓	4
D. Additional high relieving platform	8	✓	✓	✓	✓	4
E. Additional sheet pile wall	13,16	✓	✓	✓	✓	4
F. Additional underwater anchorage	20,21	✓	✓	✓	✓	4

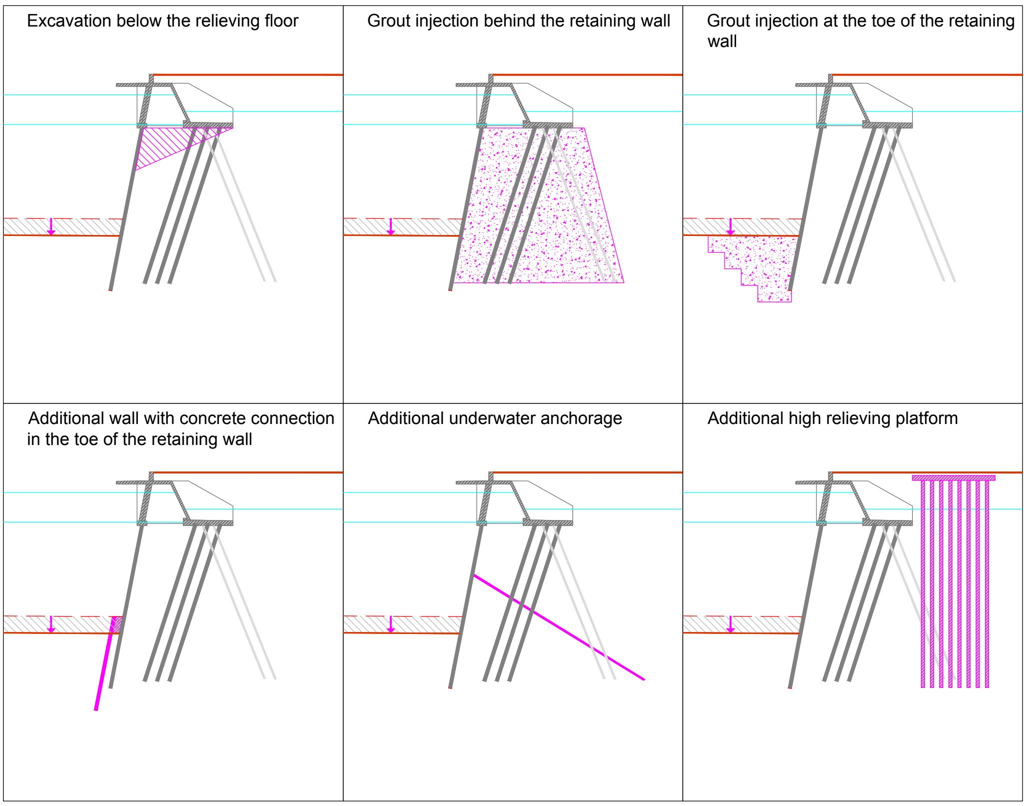


Figure 25 preselected solutions

5 Trade-off selection

The solutions which proceeded the preselection are established and ranked by trade-off criteria. The result of this chapter is the most preferred solution. The criteria ranking, trade-off and founding of solutions is done according to the criteria of the following bullet points:

- Functional assessment criteria;
- Technical assessment criteria;
- Costs of alternatives.

5.1 Solutions overview

The solutions which proceeded the pre-selection are individual substantiated. The results of the solutions substantiation are shown in the sub-chapters below. The extensive overview and assumptions of the solution are presented in separately provided report 6: Trade-off selection.

5.1.1 Lifetime extension

The lifetime extension of a solution is essential to the business case of deepening projects. For that reason is the lifetime extension estimated in cooperation with experts of the Port of Rotterdam of the project development department and with experts of third-party contractors. The overview of the estimated lifetime extension is shown in Table 7 below.

Table 7 overview of the lifetime extension of the solutions

Solutions	Lifetime extension
a) Excavation below the relieving floor	15-50 years
b) Grout injection behind the retaining wall	15-50 years
c) Grout injection at the toe of the retaining wall	15-50 years
d) Additional high relieving platform	> 50 years
e) Additional sheet pile wall	> 50 years
f) Additional underwater anchorage	> 50 years

5.1.2 Downtime/hindrance

The lack of income of the Port of Rotterdam and the clients of the quay structure depends on the downtime and hindrance of the adjustments. Solutions executed from the waterside and less execution time does lead to less downtime and hindrance. Conversely, solutions which are constructed on the land side with high execution time does have a lot of downtime and hindrance. The overview of the downtime and hindrance of the solutions to the total quay is shown in Table 8.

Table 8 overview of the downtime/ hindrance of the solutions

Solutions	Downtime/hindrance
a) Excavation below the relieving floor	On land, > 2 days
b) Grout injection behind the retaining wall	On land, > 2 days
c) Grout injection at the toe of the retaining wall	On water, < 2 days
d) Additional high relieving platform	On land, >2 days
e) Additional sheet pile wall	On water, < 2 days
f) Additional underwater anchorage	On water, < 2 days

5.1.3 Execution risk

Common work methods and common equipment do not require any additional execution risk, on the other hand, a new work method or an unknown or uncommon equipment results in a high execution risk. In between of these execution risks are the execution risk for the adjusted work method or adjusted equipment which provide an increased risk but not unacceptable. The overview of the execution risk per solution is shown in Table 9.

Table 9 overview of the execution risk of the solutions

Solutions	Execution Risk
a) Excavation below the relieving floor	Adjusted common method or equipment
b) Grout injection behind the retaining wall	Adjusted common method or equipment
c) Grout injection at the toe of the retaining wall	Common method or equipment
d) Additional high relieving platform	Common method or equipment
e) Additional sheet pile wall	Adjusted common method or equipment
f) Additional underwater anchorage	Adjusted common method or equipment

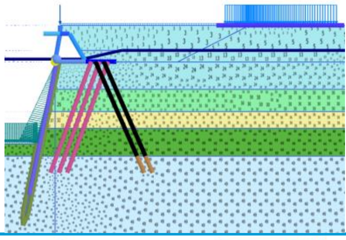
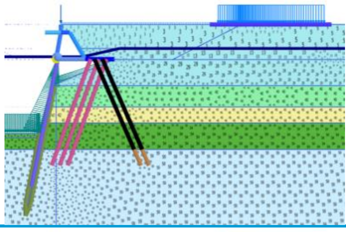
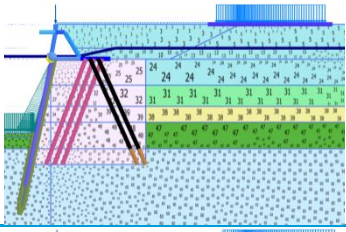
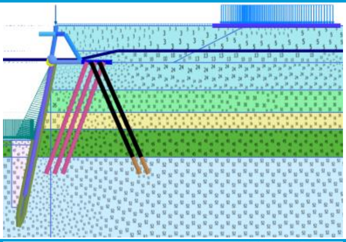
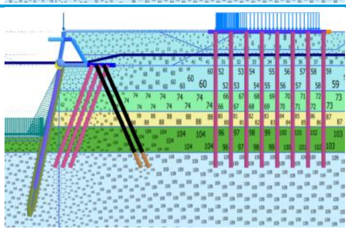
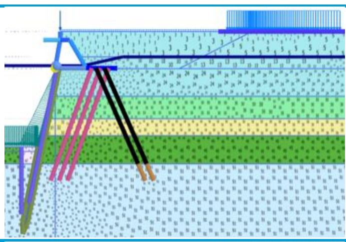
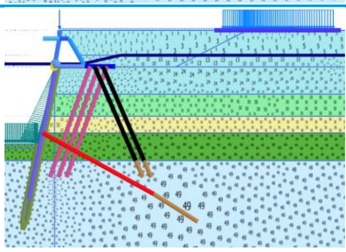
5.1.4 Technical requirements

To investigate the effect of a solution to the deepening of the structure the modelling of the solutions into the Plaxis model is established. The solutions are conservative modulated with assumptions of common materials in consultation with experts of the Port of Rotterdam Authority and with experts of third-party contractors.

5.1.4.1 Assumptions of the modulation

The materials and parameters are presented in the third column ‘Most important assumptions’ of Table 10. This table also contains the schematisation of the solutions into Plaxis. These solutions are activated in the reference model and afterwards, the 2 meter deepening is achieved in the model. The schematisations of the model are of the applied solutions and after the achieved deepening. The schematisations of the different solutions into Plaxis are shown in the second column in Table 10.

Table 10 schematisations and main assumption of the solutions after the deepening

Plaxis model	Schematisation	Most important assumptions
Without adjustment		• No adjustments;
a) Excavation below the relieving floor		• Excavation of the soil in the internal friction angle of 25 °, because of the prevention of the local failure of the soil layers;
b) Grout injection behind the retaining wall		• Supergrout 70 as grout type (Grouttech, 2017); • Total replacement of the soil by grout injection; • $E = 0.5\,E_{grout} + 0.5\,E_{soil}$;
c) Grout injection at the toe of the retaining wall		• Supergrout 70 as grout type (Grouttech, 2017); • Total replacement of the soil by grout injection; • $E = 0.5\,E_{grout} + 0.5\,E_{soil}$;
d) Additional high relieving platform		• Relieving floor 0,8 m thick; • Bearing piles 400 mm x 400 mm; • Pilling grid of 3 m x 3 m. •
e) Additional sheet pile wall		• AZ26 sheet pile (Arcelore Mittel, 2017); • Pilling depth -30 meters NAP; • Length 14 meters; • Supergrout 70 as connection (Grouttech, 2017);
f) Additional underwater anchorage		• Jetmix anchorage type 6, ø 60.3*16,0 mm (Jetmix, 2017); • Application at the maximum bending moment; • 30 ° drilling angle; • Spacing 3 meters; • 22 meter anchorage length; • 7 meter grout length;

5.1.4.2 Results of the structural assessment

The results of the front wall of the Plaxis calculation are shown in Figure 26.

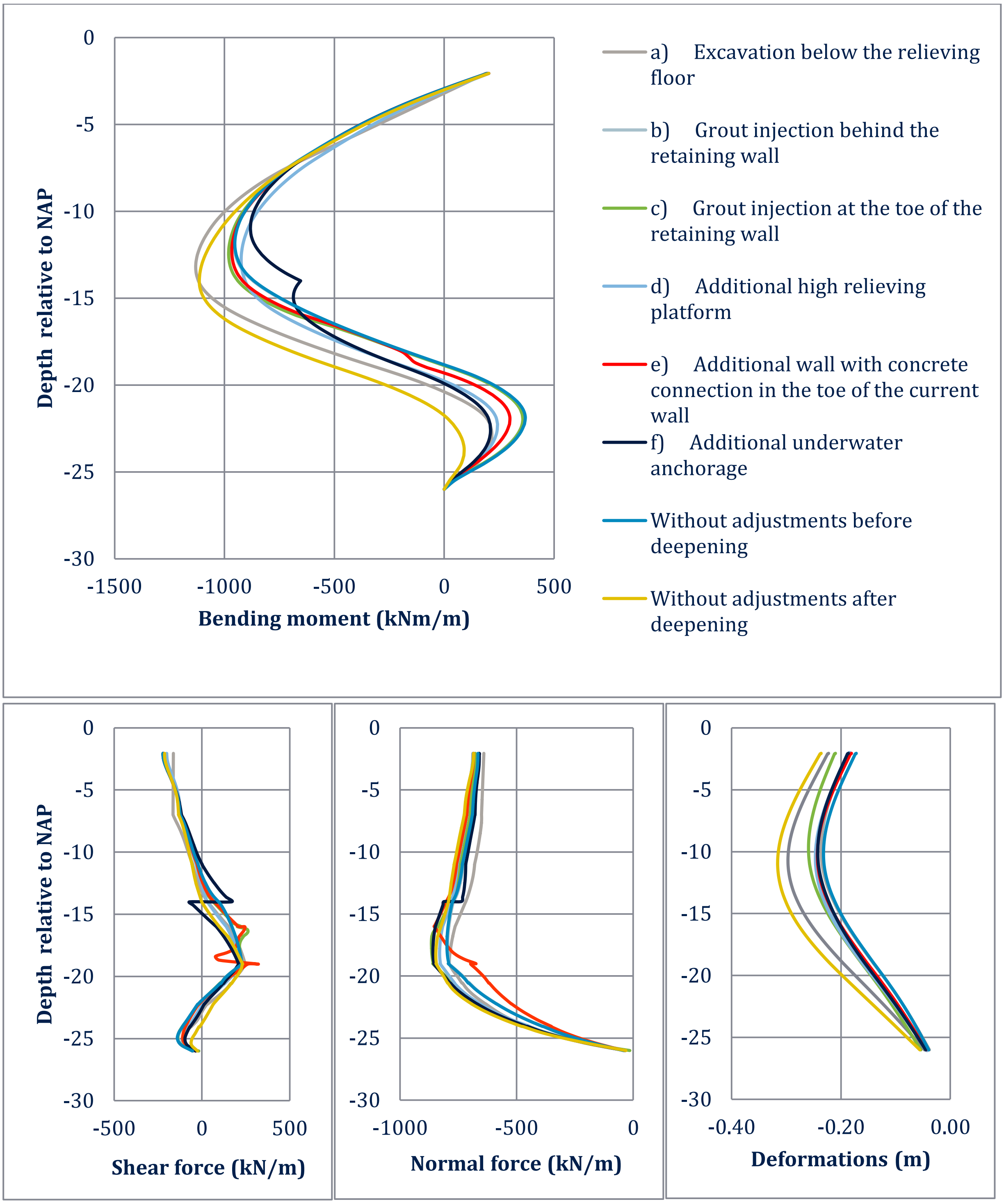


Figure 26 results of the bending moment, shear force and normal force of the front wall

The total results of the different solutions in the Plaxis calculation are shown in Table 11. The values which not meet the requirements are highlighted in red and the values which meet the requirements are highlighted in green. The values which not meet the requirement, but are also not critical according to the structural assessment of the reference model are orange highlighted.

Table 11 overview of the results of the technical requirements

Phase	Annotation	Without adjustments before deepening	Without adjustments after deepening	a) Excavation below the relieving floor	b) Grout injection behind the retaining wall	c) Grout injection at the toe of the retaining wall	d) Additional high relieving platform	e) Additional sheet pile wall	f) Additional underwater anchorage
ULS	-	-							
Geotechnical									
Safety factor	-	1.25	1.12	1.17	1.37	1.33	1.19	1.29	1.23
Structural									
Front wall $\sigma_{max} = \frac{M}{W} + \frac{N}{A}$	N/mm2	212	245	245	213	218	207	216	197
Shear force front wall	kN/m	229	232	246	229	262	228	315	225
Normal force tension pile 1	kN	95	119	123	95	121	74	116	110
ULS	-	-							
Deformations x top quay wall	m	0.13	0.19	0.19	0.19	0.19	0.16	0.15	0.17

Do not meet the requirement
Do not meet the requirement, but are not critical according to chapter 3.3
Do meet the requirement

5.1.5 Cost

The costs of every individual solution are estimated in a global costs estimation. This estimation is arranged per meter quay structure, which is arranged in cooperation with the expert of the third-party contractors. Besides of the knowledge of the expert, the cost estimations are checked and approved by the cost accountants experts of the Port of Rotterdam. The overview of the cost per meter per solutions is presented in Table 12.

Table 12 overview of the costs of the solutions

Solution	Cost per meter
a) Excavation below the relieving floor	€ 27,000
b) Grout injection behind the retaining wall	€ 164,050
c) Grout injection at the toe of the retaining wall	€ 73,480
d) Additional high relieving platform	€ 20,770
e) Additional sheet pile wall	€ 19,130
f) Additional underwater anchorage	€ 15,230

5.2 Trade-off criteria

The most preferred solution is determined by using a trade-off matrix. Three different categories of assessment criteria are used: functional, technical requirements and global execution costs.

5.2.1 Functional assessment criteria

The functional assessment criteria are separated in downtime/hindrance, execution difficulty and lifetime extension.

5.2.1.1 Downtime/ hindrance

Downtime of port activities when the assets are a challenge for the Port of Rotterdam Authority and their clients. The downtime on the land site provides a bigger economical risk as the downtime on the water side. For that reason, the ranking of the downtime/ hindrance criteria is done by the execution time and the execution method on the water of on the water side. Table 13 present the score and the corresponding description of the downtime criteria.

Table 13 score and description of the downtime/ hindrance criteria

Score	Downtime/ hindrance
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

5.2.1.2 Execution risk

The execution risk is ranked because of the importance technical feasibility of a solution. The easiest solution with a common work method and common equipment is scored with an 1. These solutions are the solutions with the lowest execution risk. The work methods which need adjustment or equipment need adjustments are scored with a 0.5 and are more risk to execute. The solutions which provide entirely new work method or completely new equipment is scored with a 0. These solutions are a significant risk to execute because of the complexity and the new work method or equipment. The score and the description of the execution risk are shown in Table 14.

Table 14 score and description of the execution risk criteria

Score	Execution risk
1	Common method or equipment
0.5	Adjusted method or equipment
0	Completely new method or equipment

5.2.1.3 Lifetime extension

The expected lifetime extension is hard to predict. The expected lifetime extension of a certain solution is therefore determined in consultation with experts. When the predictable lifetime extension of a solution is above 50 years, then this solution is ranked with an one. When the lowest predictable lifetime extension of the solutions is below 15 years, then this solution is ranked with a zero. The lifetime extension between the 15 and 50 years is ranked with a 0.5. Table 15 shows the score and the description of the lifetime extension criteria.

Table 15 score and description of the lifetime extension criteria

Score	Lifetime extension
1	> 50 years
0.5	15-50 years
0	<15 years

5.2.2 Technical assessment criteria

In advance, the effects of the solutions were unpredictable, because of the challenging soil-structure interface. The effects of the solutions are derived by the implementation of the solutions in the reference Plaxis model. The modelling of the solutions into Plaxis are mentioned in the chapter above. The effect of the solution is verified by checking the influence of the solution on the main requirements. The effect of the solution into the Plaxis calculation are determined by the minimum requirements mentioned in chapter 3.3.

If a solution does not meet the requirements, this solution is ranked with a score of 0. In that case, this solution needs to be eliminated as potential as preferred solution.

5.2.2.1 Safety factor increasing effect

The minimum safety of the solutions must be 1.25 with a tolerance of 2.5%, which is the safety factor of the reference model before deepening. 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 in between are deviated into steps of 0.05 increase of the safety factor. The score and the description of the safety factor criteria are shown in Table 16.

Table 16 score and description of the safety factor increasing effect criteria

Score	Safety factor (SF)
1	$SF \geq 1.37$
0.75	$1.32 \leq SF < 1.37$
0.5	$1.28 \leq SF < 1.32$
0.25	$1.22 \leq SF < 1.28$
0	$SF < 1.22$

5.2.2.2 Maximum stress front wall effect

The requirement of the maximum stress of the front wall is 212 with a tolerance of 2.5% 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. Table 17 describes the score and the corresponding description of the maximum stress of the front wall criteria.

Table 17 score and description of the Maximum stress front wall effect criteria

Score	Maximum stress front wall
1	σ <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	σ >217 N/mm^2

5.2.2.3 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 for an example the temporary removal of the clay layers. The score and the description of the corresponding score are shown in Table 18

Table 18 score and description of the piping prevention criteria

Score	Piping prevention
1	Improvement
0.5	No Influence
0	Deteriorate

5.2.3 Global execution costs

The total costs of the upgrade of the current quay wall structure is an individual criteria of the trade-off. The costs of every individual solution is estimated in a global costs estimation. The estimations are approved by the cost accountants market parties and cost accountants of the Port of Rotterdam Authority.

The cost of the solutions is implemented in the trade-off as the determination of the value of the solutions.

5.3 Weight factors

A weight factor for the criteria is arranged to rate the criteria different according to the importance compared to each other. The weight factors of the criteria are ranked to arrange the final most preferred 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. The overview of the compare and the weight factor of each criteria is shown in Table 19.

Table 19 N2 matrix for the determination of the weight factor

	Lifetime extension	Execution risk	Downtime/ hindrance	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/ hindrance	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

It should be noted that the criteria piping prevention(HYD) scored a theoretically equals zero, but is assumed to be 1 in order to account for the reliability of the trade-off matrix. The weight factor of the piping prevention (HYD) is the same as the lowest other criteria which conclude the weight factors in right column ‘Weight factor’.

5.4 Results of the trade-off matrix

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 by 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 20.

Table 20 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 do 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 preferred, 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 preferred 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 do tip the scale to the underwater anchorage, so the underwater anchorage solution could be indicated as final preferred solution. The sensitivity of the trade-off matrix concludes the additional sheet pile wall and underwater anchorage as best solutions. These solutions score both as best, but the value of the underwater anchorage is in all the sensitivity analyses the highest. The sensitivity analyses of the trade-off matrix are attached to this report as Appendix B.

6 Preferred solution

The overview of the preferred solution, the additional underwater anchorage, is described in this chapter. The substantiation includes the cost estimation, work method description and downtime and hindrance explanation.

6.1 Costs

The estimation of the cost of the construction of the solution is established by the determination of the dimensions, the quantity of the materials and the unit rates of the materials and the equipment. These dimensions, quantity and unit rates are validated by third-party contractors, because of the new techniques and the adjusted work methods. The cost estimation is and trough assumptions. These assumptions are arranged in cooperation with anchor experts. The assumptions of the costs are presented in the bullet points below:

- The production of 90 m^1 anchorage per day;
- Additional cost to modify the equipment to work underwater of €2,500 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 tonnes of grout per m^1 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 m^1 quay structure. The estimation is established with an uncertainty factor of 1,35. That uncertainty factor increases the price with 35%, so the general costs, profit and risks of the contractor, inflation, unforeseen cost and unit price rate deviations are enclosed. The total costs per m^1 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 21.

Table 21 cost estimation underwater anchorage

Additional underwater anchorage	Amount	Unit rate	Price	Uncertainty factor	total price
Materials	**ton**	**€/ton**	**€**		
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	1.35	€ 15,228.00
Pontoon	0.50	€ 1,500.00	€ 750.00		
Dive team	0.50	€ 4,500.00	€ 2,250.00		
Modification drilling equipment	0.50	€ 2,500.00	€ 1,250.00		

6.2 Work method description

The work method description is based on the work method of the application of anchorage on the land site (Jetmix 2, 2017). In deviation from the common execution method of the application of anchorage on land, the execution method of the anchorage underwater results in adjustments to existing equipment. These adjustments consists of realisation of the automatic reload of the anchor rods, the water tightness of the drilling machine, the water tightness of the testing equipment and the extension of the length of the drilling machine boom.

Additional activities of the application of the underwater anchorage compared to the common method are the service of the divers. These divers are acting to drill the hole in the wall to apply the anchorage, the process control of the anchorage application and the tension capacity test of the anchorage with underwater testing equipment.

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 kept open 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 become 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 hold in place. 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 at the right depth. After application of the grout injection anchorage needs 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.

6.3 Downtime

Besides of the work method, the downtime and hindrance of the solution to the terminal activities is also estimated. 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.

6.4 Solution overview

An overview of the solution applied to the existing quay wall is shown in Figure 27.

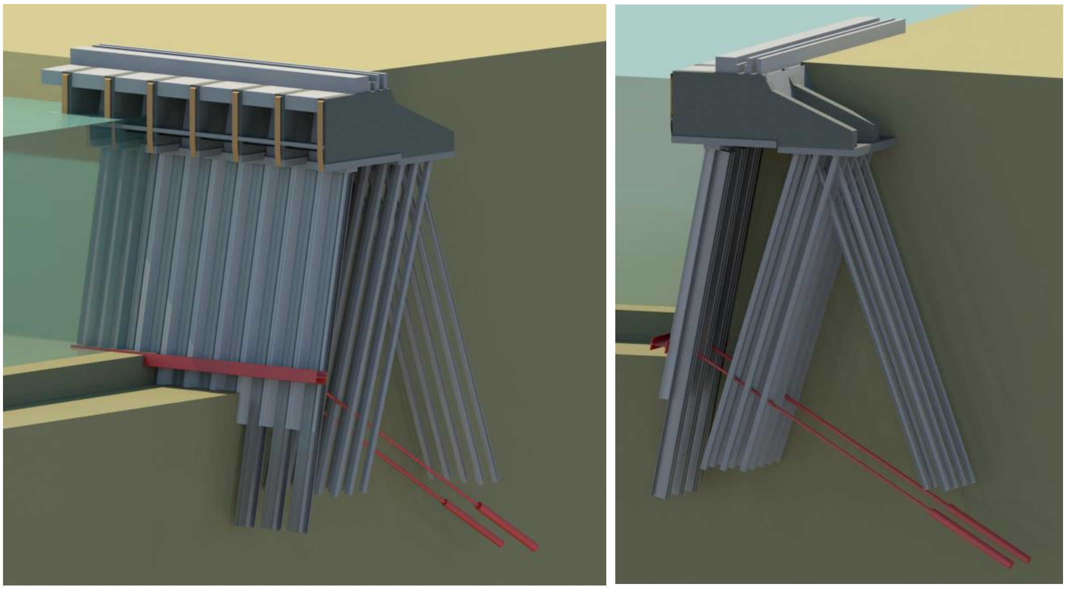


Figure 27 schematisation of the application of underwater anchorage

7 Conclusion

Port beds have to be deepened, in order to meet the increasing demand for deeper mooring facilities. This research concerns feasibility of deepening existing combi walls. The main research question of this thesis is:

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

7.1 Secondary research questions

Before answering the main research question, the secondary questions are answered to support the answer of the most important question.

1: “What adjustments have been made in the past, to deepened combi wall structures in the Port of Rotterdam and other ports?”

Studying the executed deepening project in the world shows multiple types of deepening solutions. During this study different types in different countries were found, such as additional asphalt matrasses in front of the existing structure, injection of the ground in front of the existing structure with grout, additional wall in front of the existing structure and additional a low underwater anchor near to the port bed. These results are used to determine the representative combi wall of the Botlek area.

2: “What is a representative combi wall of the Botlek area?”

An inventory of the quay wall type shows that there are numerous types of quay wall are constructed in the Botlek area. Combi walls are constructed in 38% of cases. The combi walls are considered to be the main type of quay structure in the Port of Rotterdam. The combi wall of the Sint Laurenshaven, which is the representative combi wall, was already deepened with asphalt matrasses, is constructed with Peiner piles combi-wall which are frequently used 50 years ago, so are at the end of design lifetime. Beside of the executed deepening and the end of design life consists the representative combi wall on a challenging soil-structure interface and an inclination of the front wall. The reference combi wall is used to determine the effect of the deepening.

3: “What are the failure mechanism and critical structural members of the reference combi wall structure?”

The failure mechanism and critical structural members are determined by modelling the reference combi wall in a FEM model. The modelling of the reference combi wall is firstly performed without deepening. Afterwards the modelling of the reference combi wall is performed with a deepening of 2 meters, which exposed the following critical failure mechanism and critical structural members, such as failure of the front wall, failure of tension piles,an insufficient passive resistance of the front wall and local failure of geotechnical stability between the primary piles. These failure mechanisms and critical structural members are used to determine requirements, which the preffered solutions must meet.

4: "What are the preferred solutions for deepening a combi-wall structure?"

An inventory of solutions to deepen combi walls is made by studying executed deepening projects in the past, brainstorming and interviews with experts and a literature research. These methods of research provided twenty-seven ideas for deepening solutions. These solutions are evaluated by performing a preselection, on the basis of minimum selection criteria, such as multidisciplinary application possibilities and the technical feasibility. The preselected solutions are the excavation below the relieving floor, grout injection behind the retaining wall, grout injection at the toe of the wall, additional sheet pile wall, additional underwater anchorage and an additional high relieving platform. A schematisation these solutions are shown in Figure 28.

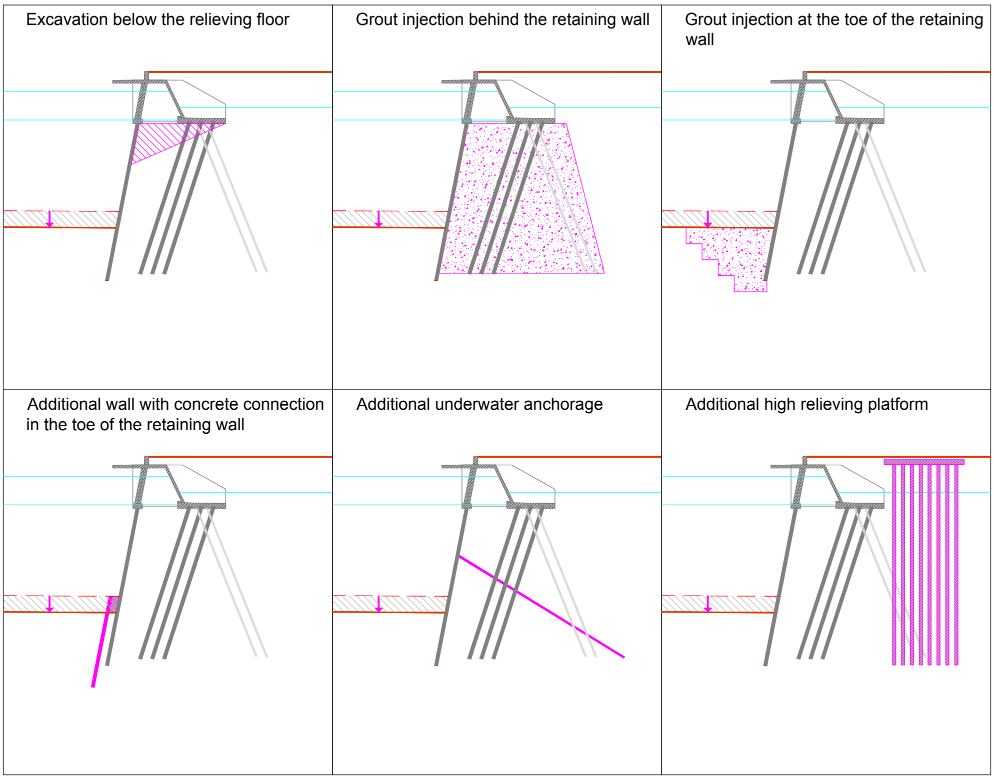


Figure 28 preselected best solutions to deepen a combi wall structure

7.2 Main research question

To meet the demand of deepening port beds, the main research question of this thesis is answered in this section. The preferred solutions in obtained by a final trade-off matrix with trade-off criteria. Figure 29 illustrate the additional underwater anchorage, which is the preferred solutions to deepen the construction depth at least 2 meters. The additional underwater anchorage and the additional wall are the solutions which are both preferred, because the score of the trade-off matrix are more or less the same. 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 preferred solution.

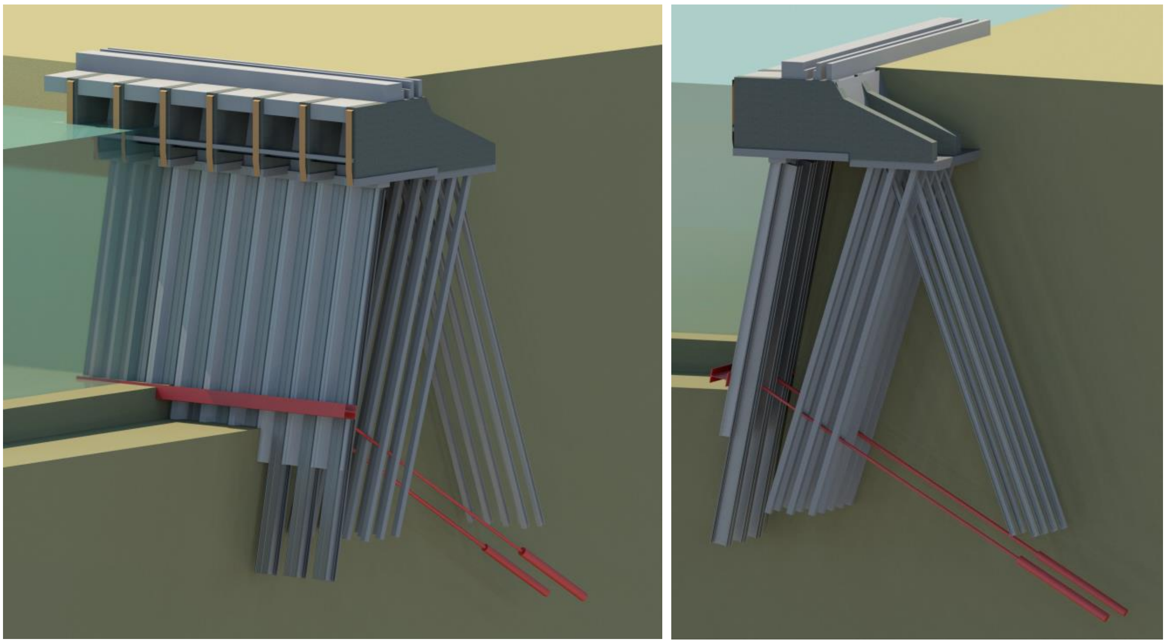


Figure 29 visualisation of the additional underwater anchorage (red) in the existing quay wall

8 Recommendations

The additional underwater anchorage was found to be the preferred solution. It should be noted that the structural assessment is established on the basis of limited soil investigation and assumptions with partial factors of 1.0. The underwater anchorage cannot immediately be constructed. The design solution should be modelled in project specific circumstances and with the prevailing guidelines. Besides of that uncertainty, the solutions are also not optimised, which also should be done before application of one of these solutions.

The recommendations for future extension of this research are:

- Perform a design of the underwater anchorage with project specific ground parameters, dimensions of structural elements and with an advanced soil-structure interface;
- Run a pilot for the application of the underwater anchorage in order, to acquire insight into the effects of the solution in reality;
- Perform a detailed structural engineering assessment of the additional sheet pile wall and a combination of the additional wall and anchorage to specify the exact effects of the solutions, if the results are positive, also run a pilot;
- Investigate the opportunities of the grout injection and share knowledge with other countries, such as Japan and Spain, because the results of the Plaxis calculation are promising for lifetime extension, but the price is extremely high;
- It is recommended to estimate the cost of the most preferred solution in more detail with project specific parameters and local prices;
- Involve different stakeholders for the determination of the weight factors used in the trade-off matrix;
- Investigate the demand of application of scour protection or other eroding prevention at the toe of the quay wall to prevent insufficient length of the intermediate piles;
- Investigate the demand of application of scour protection solutions to protect the port bed against propeller induced loads.

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Appendix A: Total ranking of the preselection

The total ranking of the solutions in the preselection is shown in Table 22

Table 22 total ranking of the preselection

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

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Appendix B: Sensitivity analyses trade-off matrix

The sensitivity of the trade-off is tested in difference deviations.

Ranking without cost and without excluding

The ranking without the cost is as the following Figure 30.

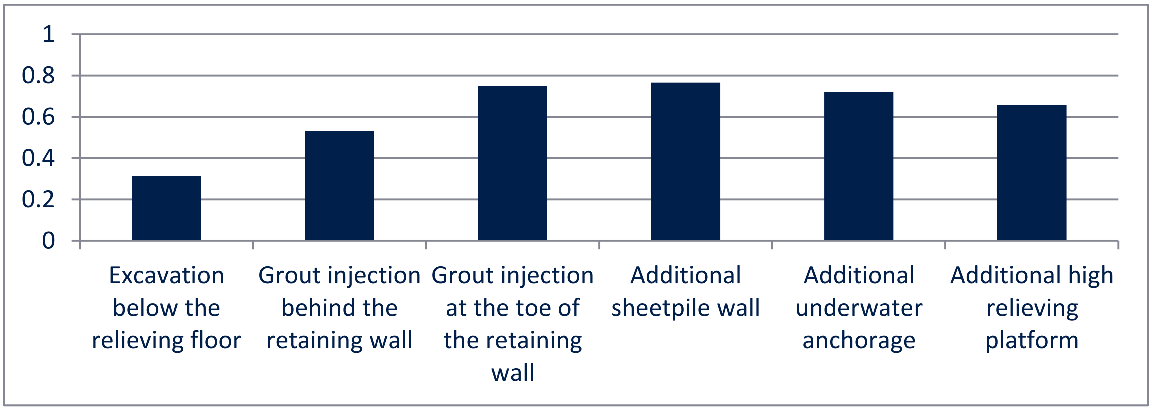

Figure 30 ranking of the solutions without cost partial

Ranking without weight factor

Results of the trade-off without weight factor is displayed in Table 23.

Table 23 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

Trade-off conclusion

The sensitivity analyses of this trade-off matrix approve the most preferred solution of the trade-off matrix. The additional underwater anchorage is the most preferred because of the highest value of the solutions.

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