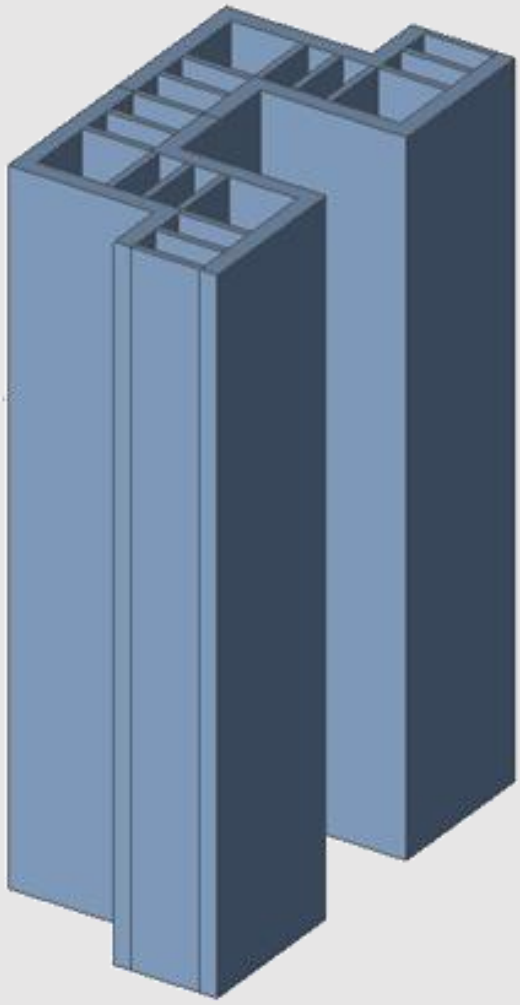
Delft University of Technology

A fibre reinforced polymer quay wall

Feasibility study

R. E. A. van der Valk



TU Delft

Gemeente Rotterdam

February 2017

A fibre reinforced polymer quay wall

Feasibility study

By

R. E. A. van der Valk

in partial fulfilment of the requirements for the degree of

Master of Science

in Hydraulic Engineering

at the Delft University of Technology,

to be defended publicly on Tuesday February 14, 2017 at 14:00.

Committee chair:	Prof. dr.ir. T. Vellinga	Delft University of Technology
Thesis committee:	Dr.ir. J. G. de Gijt	Delft University of Technology
	Dr.ir. H. M. Jonkers	Delft University of Technology
	Ir. H. E. Pacejka	Gemeente Rotterdam

An electronic version of this thesis is available at http://repository.tudelft.nl/.

"As for the future, your task is not to foresee it, but to enable it."

-Antoine de Saint Exupéry

Preface

This report contains the feasibility study of a fibre reinforced quay wall. This study is the final result of my master thesis and is part of my completion concerning the requirements for the degree Master of Science in Hydraulic Engineering at the Delft University of Technology.

This research could not have been realised without the guidance of my supervisors.

Special thanks to the engineering bureau of Gemeente Rotterdam (IGR) for providing me with all the necessary tools and a pleasant work environment. In particular I would like to express my thanks to Hans Pacejka for his daily support during my stay at IGR even outside of the office hours.

I also would like to thank my supervisors Tiedo Vellinga, Jarit de Gijt and Henk Jonkers from the Delft University of Technology for their advice, feedback and help with this research.

I would also like to thank Dirk Jan Peters and Henk Kolstein from Delft University of Technology for their support and advise during the start of my master thesis.

I would like to conclude with a big thank you to my family and friends for their support during my entire study. In particular I would like to express my appreciation for my parents who have provided me with the opportunities to develop myself as an engineer.

R. E. A. (Ramon) van der Valk

Delft, January 2017

Table of contents

Abstract

The first quay walls were constructed in 2400 BC and were constructed with bricks. Since then, the quay walls have developed a lot. The quay walls nowadays are constructed out of steel, concrete or a combination of both materials. An upcoming material in the field of hydraulic engineering is fibre reinforced polymer (FRP). FRP has several advantages such as the low weight to strength ratio. Bridges and lock gates have already been constructed out of FRP.

This thesis is a feasibility study to an FRP quay wall. An FRP quay wall is designed based on a reference project provided by the engineering bureau of Gemeente Rotterdam. CUR96 has been applied as a design guideline. The focus of this feasibility study is the retaining wall. The anchors and superstructure are not included in this study.

A variant study has been conducted concerning 4 types of quay walls. The cantilever quay wall, an anchored quay wall, an L-wall and a caisson. For each of the variants a preliminary design has been made. As a result of the variant study the anchored quay wall is proven to be the most suitable type of quay wall. This recommendation is based on the forces and moments on the structures as well as the amount of required material.

The maximum allowable deformation is based on the heeling angle and draught of the governing vessel as well as the distance between the moored vessel and quay wall. The quay wall from the case study has been modelled with the program 'D sheet Piling'. The stiffness of the quay wall has been lowered until the deformation of the quay wall was more or less equal to the maximum deformation.

The Young's modulus of FRP is a static variable while the moment of inertia is a dynamic variable. The moment of inertia depends on the cross section of the quay wall profile. The cross section of the FRP quay wall is similar to Z-profiles used for steel sheet piling. The FRP quay wall consists of 2 skin laminates, web laminates which connect the skin laminates with one another and foam that fills the space between the skins when no web is present. The foam has no significant mechanical properties and will therefore be neglected.

The skin laminate has been designed as an anisotropic laminate while the web laminate has been designed as a quasi-isotropic laminate. The laminates are designed with the programs 'eLamX^2' and 'Kolibri'. According to CUR96 the strain in the laminates may not be higher than 0,27%. With a hand calculation, a 2D model in 'SCIA' and a 3D FEM model in 'SCIA' the strains in the laminates have been calculated for the governing loading combination.

A Z-profile with a skin thickness of 50 millimetres, a web thickness of 20 millimetres, a width equal to 650 millimetres, the height equal to 725 millimetres and a spacing equal to 200 millimetres between the skin laminates remains below the strain criteria as given by CUR96.

The skin and web laminates have been checked for the following failure mechanisms: buckling, interlaminar shear stress, wrinkling and shear stress between the laminates. From these checks it can be concluded that neither the skin nor the web laminate is sensitive to these failure mechanisms.

Common checks related to quay walls have been performed as well. The anchor capacity, overall stability, bearing capacity and deadweight of the quay wall have been checked.

The joints of the FRP quay wall have been discussed quantitatively. This concerns the joints between 2 Z-profiles as well as the connection between the skin and web laminate. It is stated in design guidelines that the design of the joint between the skin and web laminate has to be verified with testing and a numerical calculation with a FEM. Literature shows that FRP is already being applied as shoring equipment but with a limited retaining height. The type of connection used in the shoring equipment is similar to the connections used with steel sheet piling.

The required installation technique for an FRP quay wall has been researched as well. Driving, vibrating and pressing the FRP quay wall to the required depth is not feasible. A diaphragm wall installation technique is therefore assumed to be the best alternative.

An LCA has been performed for the steel combi wall from the case study as well as for the FRP quay wall. The carbon footprint and environmental impact has been calculated for both quay walls and are compared to one another. The results are the same, the impact of the FRP quay wall is way bigger than the impact of the steel combi wall. The LCA has been performed for a running meter quay wall.

The FRP quay wall has been compared to the steel combi wall based on the LCA. Both structures have also been compared to one another based on a cost estimation for a running meter quay wall. The result of this estimation is that the FRP quay wall is far more expensive than the steel combi wall.

The conclusion of this feasibility study is that an FRP quay wall is technically feasible. However, the environmental impact as well as the cost estimation shows that a steel combi wall still is a better alternative.

It is recommend to perform more feasibility studies to FRP in large hydraulic structures to see whether the material is a suitable alternative for steel and concrete.

Keywords

Glass fibre, FRP, fibre reinforced polymer, polyester resin, quay wall, LCA and Life Cycle Analysis.

Executive summary

The first quay walls were constructed in 2400 BC and were constructed with bricks. Since then the quay walls have developed a lot. The quay walls nowadays are constructed out of steel, concrete or a combination of both materials. An upcoming material in the field of hydraulic engineering is fibre reinforced polymer (FRP). FRP has several advantages such as their low weight to strength ratio. Bridges and lock gates have already been constructed out of FRP.

The research objective of this feasibility study is defined as:

"Design an FRP quay wall based on a case study with the current regulations and standards."

A case study provided by IGR will function as reference for the design conditions of the FRP quay wall. A literature study has been conducted concerning information about the fields of applications, material characteristics, codes and guidelines and the cost of FRP material.

By means of a variant study the most suitable type of quay wall has been selected. The variant study considered a cantilever quay wall, an anchored quay wall, an L-wall and a caisson. For each type of quay wall a preliminary design has been made. As a result of the variant study the anchored quay wall is put forward as the most suitable type of quay wall. This recommendation has been based on the forces and moments on the structures as well as the amount of required material.

The maximum allowable deformation of the quay wall was calculated. The maximum allowable deformation is based on the heeling angle and draught of the governing vessel as well as the distance between the moored vessel and quay wall. With the program D-sheet a quay wall was modelled similar to the quay wall of the case study. The stiffness of this quay wall was then decreased until the deflection of the quay wall almost became 400 millimetres, the maximum allowable deformation. The stiffness of this altered quay wall is the starting point of the design of the FRP quay wall. Based on the cost, available guidelines and literature the quay wall will be designed with a glass fibre reinforcement and polyester resin.

The Young's modulus of FRP is a static variable while the moment of inertia is a dynamic variable. The moment of inertia depends on the cross section of the quay wall profile. The cross section of the FRP quay wall is similar to the Z-profiles used for steel sheet piling. The FRP quay wall consists of 2 skin laminates, web laminates which connect the skin laminates with one another and foam that fills the space between the skins when no web is present. The foam has no significant mechanical properties and will therefore be neglected.

The skins of the quay wall profile are constructed from an anisotropic laminate with 55% of the fibres in the vertical direction, the so called 0-direction, and 15% of the fibres are located in the other directions. The webs are constructed as a quasi-isotropic laminate which means that 25% of the fibres are placed in each of the 4 main directions. The skin laminate has a thickness of 26 millimetres while the web laminate is 12 millimetres thick. The webs are 100 millimetres high. This profile has a stiffness similar to the stiffness of the altered quay wall and thus meets the deflection criteria. This design is named design 1.

According to CUR96 the strain of the laminates will have to be checked since they may not be higher than 0.27% for a structure that is permanently located in water. The moment and normal force acting on the FRP quay wall have been obtained from the D-sheet model. With a hand calculation and a 2D SCIA model the strains in the skin and web laminates have been checked. The strains appeared to be above the limit of 0.27%. The highest strain in design 1 is equal to 0.99%.

The cross section of the FRP quay wall has been altered. The thickness of the skin laminate has been increased to 50 millimetres while the thickness of the web laminate has been increased to 20 millimetres. The height of the webs has been increased to 200 millimetres. This design is named design 2. Design 2 has a stiffness that is roughly 5 times the stiffness of design 1. The deflection of design 2 is equal to 79 millimetres which is far less than the deflection criteria.

The forces and moments have been obtained from the D-sheet model. The strains in the skin and web laminate were checked by means of hand calculation, a 2D SCIA model and the combination of a 3D FEM SCIA model with the program Kolibri. From these checks it can be concluded that the cross section of the FRP quay wall according to design 2 does fulfil the strain criteria of 0.27%. For the normal load combinations the strain will not be bigger than 0.21%. The exceptional loading case of an impact load results in a strain of maximum 0.25%.

The skin and web laminates have been checked for the failure mechanisms buckling, interlaminar shear stress, wrinkling and shear stress between the laminates. From these checks it can be concluded that neither the skin nor the web laminate is sensitive to these failure mechanisms.

The anchor capacity has been checked with the Kranz method while the overall stability of the FRP quay wall has been checked with the Bishop method. The bearing capacity of the FRP quay wall has been determined according to NEN6740. Only the point resistance has been taken into account for the bearing capacity. The bearing capacity has been determined for the cross section consisting of 2 Z-profiles. Since there are uncertainties concerning the installation method a conservative bearing capacity has been calculated. It is assumed that the FRP quay wall will be installed with a soil removing technique. This conservative calculation of the bearing capacity results in a point resistance equal to 615 kN while the vertical load on the quay wall has a maximum value of 512 kN. The bearing capacity of the profile is thus sufficient.

The deadweight of the FRP quay wall has been calculated per 2 Z-profiles. This has been done to check whether the profile can resist the uplift force generated by the groundwater. This check resulted in the conclusion that the profile is too light to withstand the uplift force. It is recommend to apply a foam in the FRP structure with a volumetric weight of approximately 300 kg/m^3 to prevent uplifting of the structure during the installation phase.

The joints of the FRP quay wall have been discussed quantitatively. This concerns the joints between 2 Z-profiles as well as the connection between the skin and web laminate. It is stated in design guidelines that the design of the joint between the skin and web laminate has to be verified with testing and a numerical calculation with a FEM. Literature shows that FRP quay walls are already being applied but with a limited length and retaining height. The type of connection used in these quay walls is similar to the connections used with steel sheet piling. Further analysis of the connections is necessary.

For the installation technique of the FRP quay wall with the cross section according to design 2, several options have been considered. Vibrating and pressing of the FRP quay wall seems not feasible based on the characteristics of both techniques. A driving analysis has been performed with the program Allwave PDP. With this program the compression stress in the quay wall element can be calculated. However, due to a possible error in the CPT file of the case study implausible results were obtained. By applying a CPT of the 'Amazonehaven' realistic results were obtained. The program showed that the compression stress was around 34 MPa but with peaks of 101 MPa. Based on the driving analysis and literature that discusses the driveability of an FRP Superpile it is concluded that driving of the FRP quay wall is not feasible. The FRP quay wall will therefore have to be installed with the diaphragm wall technique.

An LCA has been performed for the steel combi wall from the case study as well as for the FRP quay wall. The carbon footprint has been calculated for the phases manufacturing elements, transport to building site, construction and end of lifetime. The carbon footprint of the FRP quay wall is roughly 10 times as big as the carbon footprint of the steel combi wall. The environmental impact of both quay walls have been calculated for 10 impact categories. These 10 impact categories together form the environmental impact. The environmental impact has been obtained by normalizing the impact categories with the so called shadow costs. The environmental impact of the steel combi wall has been calculated at €2000,- while the normalized environmental impact of the FRP quay wall is equal to €12.000,-. The total environmental impact of the structures has only been calculated for the materials used in the quay wall itself since the literature does not provide the data to calculate the environmental impact for the other phases such as the transport to the building site. The LCA has been performed for one running meter quay wall.

The FRP quay wall was then compared to the steel combi wall based on the LCA. Both structures have also been compared to one another based on a cost estimation for a running meter quay wall. The cost estimation takes into account the cost of the materials as well as the installation technique. The cost of 1,0 meter quay wall constructed out of steel in the case study is equal to €5.500,- while a running meter FRP quay wall roughly costs €68.000,-. The difference between the two materials is a factor 12.

It can be concluded from this feasibility study that an FRP quay wall has been designed with the current standards and guidelines. Although a design has been made for an FRP quay wall it seems unlikely that an FRP quay wall will be able to compete with a steel or concrete quay wall.

Perhaps if a uniform standard and guideline for FRP in hydraulic structures could be realised it will result in a competitive FRP quay wall design. If the expertise concerning the FRP is shared this might result in a decrease of the material prices. It is advised to perform more feasibility studies for FRP in hydraulic structures so that a better insight can be gained in the potential of FRP for the field of hydraulic engineering. If these feasibility studies are performed the focus should not only be on the design but on the LCA and cost estimation as well.

List of symbols

Roman symbols

a	Distance between the normal centres	[mm]
A	Area	[mm2]
b	Width	[mm]
d	Thickness	[mm]
e	Eccentricity	[m]
E	Young's modulus	[kN/m^2]
E_1	Young's modulus in the principal direction	[MPa]
E_2	Young's modulus perpendicular to the principal direction	[MPa]
E_c	Young's modulus of the core	[MPa]
EG_{ss}	The self-weight of the superstructure	[kN/m']
EG_{quay}	The self-weight of the quay wall	[kN/m']
EI	The stiffness of the quay wall	[kNm^2]
E_s	Young's modulus of the skin	[MPa]
F_{adh}	The negative adhesion	[kN/m']
F_{anchor}	The anchor force	[kN]
F_b	Critical buckling load	[kN]
$F_{d,anchor}$	The anchor load in the ULS	[kN/m']
g	Gravitational acceleration	[m/s^2]
G	The shear modulus	[MPa]
G_{12}	Shear modulus	[MPa]
G_c	Shear modulus of the core	[MPa]
h	Height	[m]
I	The moment of inertia	[m4]
L_b	Buckling length	[m]
M	Moment	[kNm]
$M_{D\text{-}sheet,SLS}$	The maximum moment in the SLS	[kNm]
M_{max}	Maximum moment	[kNm]
N	Normal force	[kN]
R_R	The representative value	[-]
S_a	The statical moment of the sheared part	[m^3]
S_d	The design value	[-]
t	Thickness	[mm]
u_{max}	Maximum deflection	[mm]
u_{total}	Total deflection	[mm]
V	The shear force	[kN]
$V_{concrete}$	The weight of the anchored concrete block	[kN/m']
V_{ss}	The vertical load on the superstructure	[kN/m']

Greek symbols

α	The angle of the anchor with the horizontal	[°]
γ_{12}	Shear strain	[%]
γ_{cf}	The fatigue factor	[-]
γ_{ck}	The creep factor	[-]
γ_{ct}	The temperature factor	[-]
γ_{cv}	The moisture factor	[-]
γ_{m1}	The material factor	[-]
γ_{m2}	The manufacturing factor	[-]
ε_1	Strain in the principal direction	[MPa]
ε_2	Strain perpendicular to the principal direction	[MPa]
ν_1	Poisson ratio in the principal direction	[-]
ν_2	Poisson ratio perpendicular to the principal direction	[-]
π	Pi	[3,14]
ρ	Density	[kg/m^3]
σ	stress	[MPa]
σ_1	Stress in the principal direction	[MPa]
σ_2	Stress perpendicular to the principal direction	[MPa]
σ_x	Stress in the x-direction	[MPa]
σ_y	Stress in the y-direction	[MPa]
σ'_s	Skin wrinkling stress	[MPa]
τ	Shear stress	[MPa]
τ_{12}	Shear stress	[MPa]
τ_d	Design shear stress	[MPa]

List of abbreviations

BC	Before Christ
CO_2	Carbon dioxide
CPT	Cone Penetrometer Test
CUR	Civieltechnisch Centrum Uitvoering Research en Regelgeving
FEM	Finite Element Method
FRP	Fibre Reinforced Polymer
GPa	Gigapascal
GWP	Global Warming Potential
IGR	Ingenieursbureau Gemeente Rotterdam
kg	kilogram
kN	Kilonewton
kNm	Kilonewtonmetre
LCA	Life Cycle Analysis
MPa	Megapascal
NAP	Normaal Amsterdams Peil
NEN	Nederlandse Norm
SLS	Serviceability Limit State
ULS	Ultimate Limit State

1. Introduction

1.1 Quay walls

History tells us that there has been transport over water as far back as 6000 BC when the Egyptians used the river Nile for the transportation of cargo. The first quay walls were constructed in Lothal, India in 2400 BC. These quay walls were constructed with bricks. Since those days the quay walls have developed a lot. Nowadays quay walls are being constructed with an overall length of over 30 meters. Current quay walls are being constructed out of concrete, steel or a combination of these materials.

An upcoming product in the field of hydraulic engineering is fibre reinforced polymer. These so called plastics are used for the construction of bridges as well as lock gates [Valk, 2016]. Advantages of these plastics according to experts are the weight to strength ratio, freedom of shaping, fatigue strength, damage tolerability and low maintenance. It is also stated by the experts that due to these advantages the lifetime of a structure constructed with these plastics is longer than a comparable structure constructed with concrete or steel.

Since the fibre reinforced polymers are a new material in the field of hydraulic engineering there is not a lot of information available concerning the applicability of FRP for large hydraulic structures.

This feasibility study will research the applicability of FRP for a quay wall.

1.2 Research objective

This feasibility study will present the first design cycle for a quay wall constructed out of FRP. The research objective is defined as:

"Design an FRP quay wall based on a case study with the current regulations and standards."

To be able to fulfil the research objective several research questions have been formulated. These questions are listed below.

- What kind of composites are there?
- What are the characteristics of the composites?
- What are the requirements of the quay wall?
- Which composite is most suitable?
- Is a quay wall constructed out of composite CO2 neutral?
- What are the costs of a composite quay wall and is it financially attractive to construct a composite quay wall?
- How are plastic quay walls connected to each other?
- What is a suitable installation technique?

1.3 Research methodology

To be able to answer the research questions and fulfil the research objective a literature study has been conducted. In the literature study the information concerning the fields of application, FRP materials, codes and guidelines and quay walls in general is concluded.

A case study provided by IGR is the starting point of this feasibility study. The design conditions are derived from the case study. The design of the FRP quay wall will eventually be compared to the design of the case study. The design of the FRP quay wall has been conducted with the regulations and standards as they are available at the moment of writing.

The case study has been picked based on the design conditions. Literature shows that FRP is already being applied as shoring equipment but with a limited retaining height. The quay wall from the case study has a retaining height of approximately 10 meters. This retaining height is far more than the retaining height of shoring equipment.

For the design of the FRP quay wall several programs will be used. The programs are listed below.

- eLamX^2 2.3, a program of the university of Dresden used for the design of FRP,
- Kolibri 3 Demo version, a program of Lightweight Structures BV used for the design of FRP,
- D-Sheet Piling 14.1, a program of Deltares used for quay wall calculations,
- SCIA Engineer 14, a program of SCIA used for FEM calculations,
- Allwave PDP, a program of Allnamics used for driveability studies.

1.4 Assumptions and limitations

As already mentioned in the previous paragraph the design of the FRP quay wall will be based on a case study. The relevant requirements and design conditions are derived from the case study. The requirements and design conditions are elaborated into detail in chapter 2 Case study Alblasserdam.

This feasibility study is limited to the quay wall itself. Other aspects of the design such as the anchor and superstructure are not part of this feasibility study.

1.5 Report structure

The content of this study has been divided into several sections. The chapters in this report are:

1. Introduction. This chapter gives an overview of the motivation of this research as well as the research objective and questions.

2. Case study Alblasserdam. This chapter discusses the case study that was used for this feasibility study. The relevant requirements and design conditions are included in this chapter. Appendix A shows the CPT profile corresponding to the case study

3. Brief introduction to FRP. The third chapter of this report provides a brief introduction to FRP. The characteristics of the materials that form an FRP are presented in this chapter. The introduction is a brief summary of the literature study.

4. Variant study. This chapter discusses the quay wall variants that have been taken into account during this research. Appendix B contains the elaborated calculations of the variant study.

5. Design of the quay wall. This chapter contains the design process of the FRP quay wall. The appendices C to I contain valuable information for the design process.

6. Joints. The joints of the FRP quay wall are discussed qualitatively in this chapter.

7. Installation technique. The installation technique suited for an FRP quay wall is elaborated into detail in this chapter.

8. Life Cycle Analysis. The LCA of the FRP quay wall as well as the steel combi wall of the case study is discussed in this chapter. Appendix J provides background information related to this chapter.

9. Cost estimation. This chapter contains the cost estimation of the FRP quay wall and a comparison between the quay wall of the case study and the FRP quay wall.

10. Conclusion. The conclusions of this feasibility study are presented in this chapter.

11. Discussion. This chapter contains a discussion concerning the design of the FRP quay wall as well as the feasibility study as a whole.

2. Case study Alblasserdam

2.1 Chapter contents

This chapter will introduce the case study of the container transferium in Alblasserdam. The important and governing requirements will be provided before the used soil parameters will be presented. The chapter is concluded with an overview of the design that has been made for the container transferium by IGR.

2.2 Introduction

The container transferium is located in Alblasserdam. The project includes the design of the main wall and two wing walls. The main wall is divided in 4 sections. This partition was made based on the soil parameters and the differences in cross section. The partition of the main wall is given in Figure 1. The first two parts of the main wall are also characterized by already present anchors of the old quay wall. These anchors will have to be removed and the removal of those anchors influences the soil parameters.

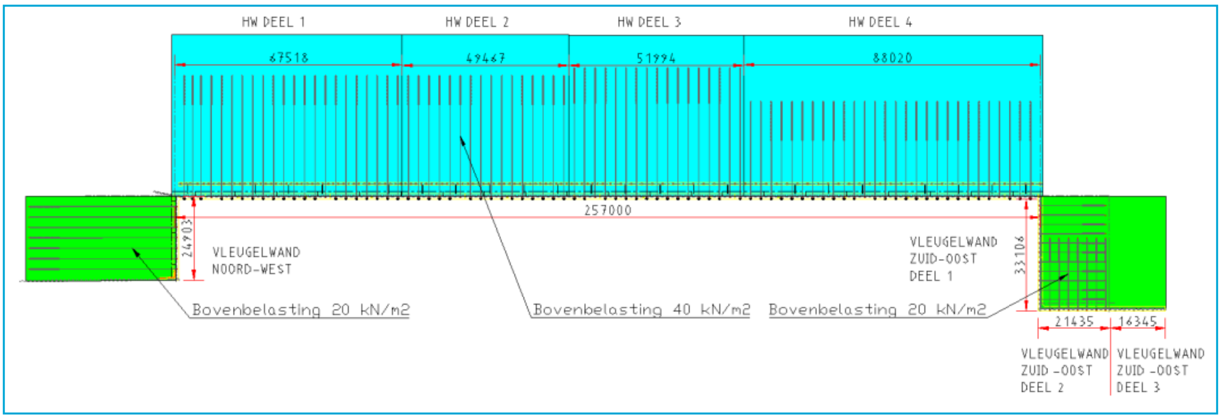


Figure 1: The various sections of the quay wall

The design of the FRP quay wall is based on the design that was made for part 4 of the main wall. This part of the main wall has the smallest retaining height as well as the best soil profile to minimalize the forces on the quay wall. Since this study is a feasibility study, the starting point of the design will be the most favourable situation. If it turns out that an FRP quay wall can be designed for the most favourable situation, one can try to design an FRP quay wall for more unfavourable situations.

2.3 Requirements

The requirements are related to the water levels, vessels that will moor at the quay wall and other categories. This section will provide the most important requirements which are stated in the original program of requirements for the case study. Table 1 presents the general data while Table 2 shows the water levels that have to be applied. Table 3 gives an overview of the important levels for the surfaces and other important layers. The loads acting on the quay wall are presented in Table 4.

Table 1: General data as stated in the program of requirements

Design life	50 years
Safety class[1]	CUR class II
Governing vessel combination	2 times a vessel of class V moored in the length of each other to the quay wall which will require a length of approximately 255 m. Length of 1 vessel: 110 m 1.2*L + 1.1*L=253 m which is rounded up to 255 m Heeling angle of 2° Draft of 4 meters

Table 2: Water levels as stated in the program of requirement

Low water level	NAP -0.35 m
Groundwater	NAP +0.15 m

Table 3: Important levels for the design of the quay wall as stated in the program of requirements

Top level	NAP +4.00 m
Contract depth	NAP -6.00 m
Top of the bottom protection	NAP -7.00 m
Construction depth	NAP -7.80 m
Stir boundary	NAP -8.30 m

Table 4: Loads on the quay wall as stated in the program of requirements

Surface load	40 kN/m^2
Bollard	
Capacity	600 kN
Level	NAP +4.00 m
Centre to centre distance	15 to 20 m
Mooring pin	
Capacity	300 kN
Level	NAP +2.00 m
Centre to centre distance	15 to 20 m
Transferred crane load	
Load	40 kN
Level	NAP +2.50 m
Impact load	
Load	600 kN
Level	NAP +3.15 m

[1] Nowadays the safety class should be according to the Eurocode. CUR class II however, will be maintained as safety class so that the comparison between the composite quay wall and the classic quay wall will not be influenced by the safety class.

2.4 Geotechnical data

The geotechnical data that has been used in the case study will be presented in this paragraph. The governing CPT is S012 for which the reader is referred to Appendix A: CPT S012. Based on this CPT the different layers are distinguished. The soil profile is presented in Table 5 along with characteristics of the soil layers.

Table 5: Composition of the soil based on CPT S012

Top of the layer [m NAP]	Soil type	Saturated and unsaturated weight [γ_{sat} / γ_{unsat}]	Cohesion [kPa]	Angle of internal friction [°]	Wall friction angle [°]	Modulus of subgrade reaction K_1 [kN/m²]	K_2 [kN/m²]	K_3 [kN/m²]
3,5	Peat	11 / 11	1	15	0	1000	500	250
0	Clay	16 / 16	5	25	8,33	8000	4000	2000
-4	Sand	20 / 20	0	28	18,67	16000	8000	4000
-27	Clay humeus	14 / 14	5	18	6	6000	4000	2000
-29	Peat	11 / 11	1	15	0	1600	800	500
-30	Sand	20 /20	0	28	18,67	16000	8000	4000
-34	Clay	18 / 18	5	22	7,33	6000	4000	2000
-36	Sand	20 / 20	0	30	20	20000	10000	5000

2.5 The design of the reference quay wall

The design of the case study that will act as a reference design is presented in this paragraph. An impression of the design is shown in Figure 2.

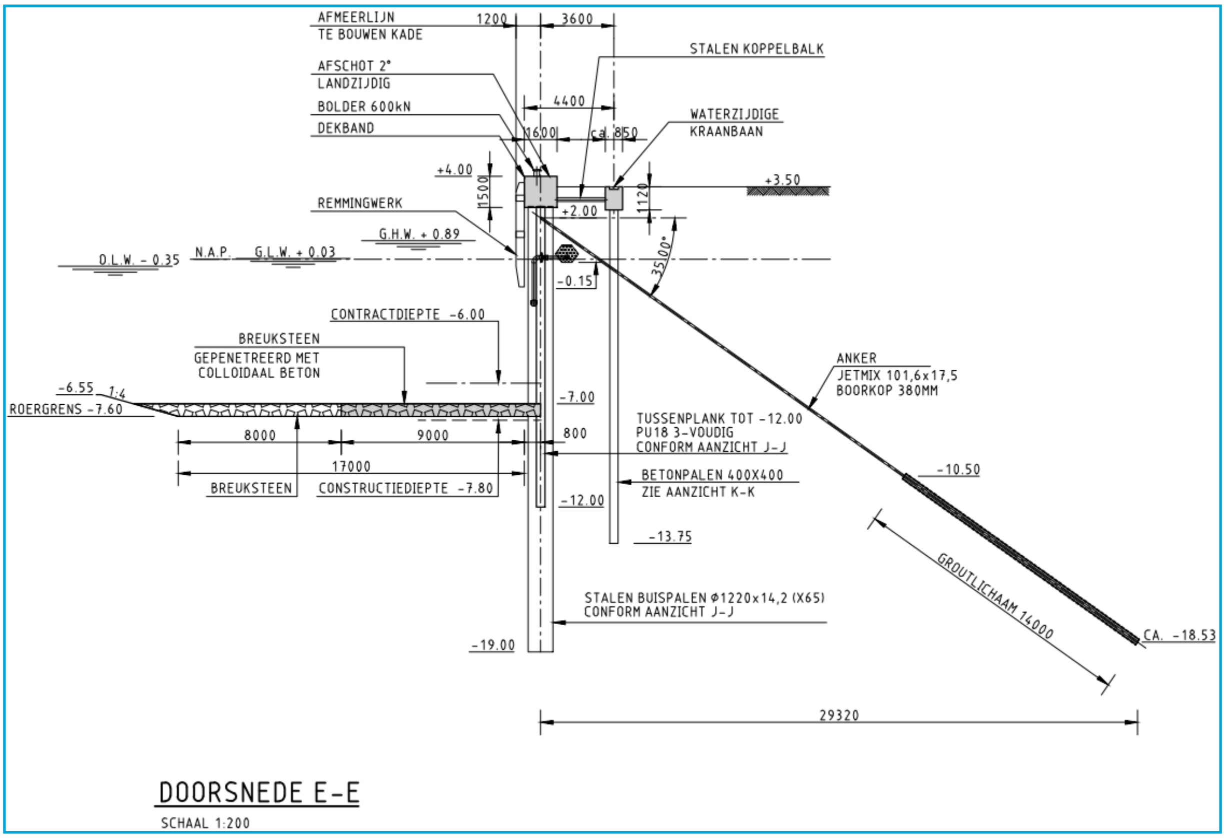


Figure 2: Cross section of the design of the quay wall

The design consists of a steel combi-wall with a grout anchor. The combi-wall consists of a steel tubular pile and three steel PU18 planks between the piles. The steel tubular piles are installed to a depth of NAP -19 m and the planks are installed to a depth of NAP -12 m. The grout anchor is installed with an angle of 35 degrees with respect to the horizontal. Properties of the tubular pile, the planks and the anchor are presented Table 6. The mechanical properties of the combi wall and the tubular pile are presented in Table 7.

Table 6: Details of the combi wall and anchor used in the case study

Tubular pile		
Diameter	1220	mm
Wall thickness	14.2	mm
Top level	+2.55	m NAP
Installation depth	-19.00	m NAP
Steel quality	X 52	
Plank		
Width	1800	mm
Steel section	294000	mm^2/set
Mass	230.7	kg/m/set
Moment of inertia	$642.4 \cdot 10^6$	mm^4/set
Moment of resistance	$2495 \cdot 10^3$	mm^3/set
Top level	+2.55	m NAP
Installation depth	-12.00	m NAP
Anchor		
Cross section	4623.6	mm^2/anchor
Yield stress steel	515	N/mm^2
Angle w.r.t. the horizontal	35	°
Pretension force	360	kN/m

Table 7: Mechanical properties of the combi wall

Mechanical properties			
Young's modulus	E	$2.1 \cdot 10^8$	kN/m^2
Bending stiffness combi wall	EI_{combi}	$7.105 \cdot 10^5$	kNm^2/m
Bending stiffness tubular pile	EI_{pile}	$6.667\ 10^5$	kNm^2/m
Surface tubular pile	A_{pile}	17460	mm^2/m
Moment of resistance	$W_{el,combi}$	$5.546\ 10^6$	mm^3/m

3. Introduction to FRP

3.1 Chapter content

This chapter will provide a brief introduction to the fibre reinforced polymers. The chapter will discuss different types of reinforcement, resins and cores as well as discussing briefly the sandwich construction. For more detailed information concerning FRP the reader is referred to [Valk, 2016].

3.2 Introduction

A composite consists of two or more materials. The properties of the composite material are not equal to the properties of the different components. FRP composites consist of load-bearing fibres and a matrix in which they are embedded. The fibres are the reinforcement and the matrix is a resin. The matrix should hold the fibres in their position and prevent them from buckling. The matrix should also protect the fibres against humidity and it transfers the loads to the fibres.

The mechanical properties are influenced by the material design. Not only the type of reinforcement and resin that will be used but also the amount of fibres, the angle of the reinforcement with the loading direction and the adhesion between the fibres and the matrix will be of influence on the mechanical properties of FRP [Kolstein, 2008].

3.3 Fibres

The reinforcement in a composite consists of fibres. They are used within resin systems to improve the mechanical properties of cured resin and provide usable components. The most applied fibre is glass fibre. Glass fibre is the cheapest fibre among the most used fibre options within the construction industry. Other fibres are high strength carbon fibres and polyaramid fibres. The high strength and stiffness-to-weight ratio of carbon and polyaramid fibres make them particularly attractive for the manufacture of lightweight structural components.

The stress-strain curves of reinforced fibres are presented in Figure 3 where it can be seen that glass fibres have higher elongation before failure than carbon and polyaramid fibres, but lower strengths and moduli. An overview of different reinforcement properties will be given in Table 8. It should be noted that the values are approximate values.

The abbreviations 'HM' stands for high modulus where 'HT' stands for high tenacity and the abbreviation 'SM' stands for standard modulus. [Kolstein, 2008]

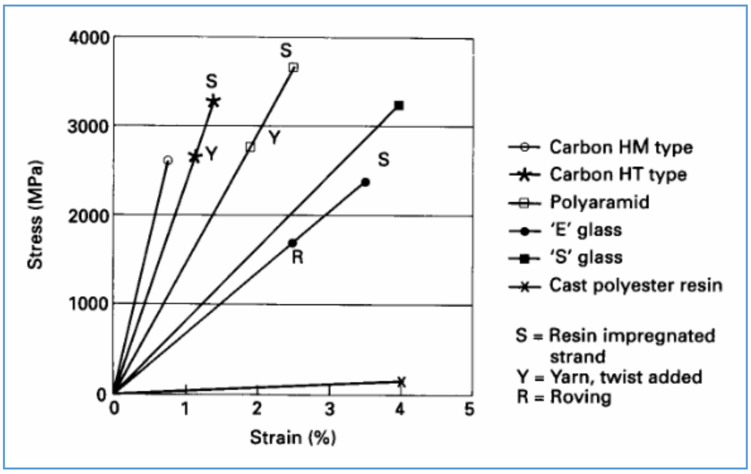


Figure 3: Stress-strain curves of reinforced fibres

Table 8: Approximate properties of fibres [Nijhof, 2004]

Fibre	Density, ρ [kg/m³]	Elastic modulus, E [GN/m²]	Tensile strength, [GN/m²]	Max. elongation, ε [%]
Glass				
E	2540	70	1.7 - 2.7	2.4 – 3.7
S	2490	85	2 – 3	2.3 – 3.5
Polyaramid				
HM	1450	130	3.0 – 3.5	2.3 – 2.6
Carbon				
SM	1800	220 - 240	3.5 – 4.5	1.5 – 1.9
HT	1800	250 - 300	4.4 – 5.0	1.5 – 1.8
HM	1850	360 - 420	2.0 – 3.0	0.5 – 0.7

3.4 Resins

At first it seems illogical to partially offset the high strength and stiffness of the fibres by mixing it with another material. Bundles of parallel fibres are of little use in a load-bearing structure. Bundles of parallel fibres have structural integrity in tension but their structural potential cannot be harnessed unless the bundles can be joined. This is the same when it comes to shear and compression. Without means of distributing load across a series of fibre bundles, the material is of no use for structural applications. The only way the fibres can then be used is as a rope, which works in cable tension. To construct a material that can be used for structural applications a resin has to be added to the fibres. The main functions of the resin are [Nijssen, 2013]:

- Binding the fibres and wrap the fibres with the resin so that it can withstand a higher compression,
- Distributing the load to the fibres by means of shear stresses,
- Adding buckling capacity to the material,
- Protecting the fibres from external influences such as ultraviolet light.

The types of resins will be divided into two categories, namely thermosetting and thermoplastic resins. When heated, thermoplastics will become soft and workable. When the thermoplastic is cooled it returns to a solid form. The advantage of this characteristic is that it can be formed into almost any shape. However, It is also a disadvantage when structural purposes are taken into account. Thermosets are produced with a polymerization reaction or hardening. A thermosets does not become soft when heated but will burn eventually. The most important resins are polyester, vinyl ester and epoxy which are thermosetting resins [Kolstein, 2008]. The advantages and disadvantages of the most important resins are presented in Table 9.

Table 9: Advantages and disadvantages of the most applied resins [Cripps, 2016]

	Advantage	Disadvantage
Polyester	Easy to use	Only moderate mechanical properties
	Lowest cost of available resins	High styrene emissions in open moulds
		High cure shrinkage
		limited range of working times
Vinylester	Very high chemical/environmental resistance	Post curing generally required for high properties
	Higher mechanical properties than polyesters	High styrene content
		Higher cost than polyesters
		High cure shrinkage
Epoxy	High mechanical and thermal properties	More expensive than vinyl ester
	High water resistance	Critical mixing
	Long working times available	Corrosive handling
	Temperature resistance can be up to 140 °C wet / 220 °C dry	
	Low cure shrinkage	

3.5 Cores

The purpose of a core is to increase the laminate's stiffness by effectively thickening it with a low-density core material. The flexural stiffness of a panel is after all proportional to the third power of its thickness. With the application of a core a sandwich construction will be created. The FRP materials will form the outer skins and the core will form the connection between the skins. When the sandwich is then subjected to a bending load, the skins will be put in compression and tension while the core is put into shear, as can be seen in Figure 4.

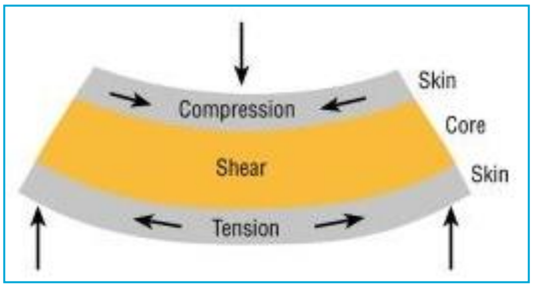


Figure 4: Sandwich construction in bending [Cripps, 2016]

3.6 Skins

The skin of a so called sandwich structure is called a laminate and consists of the chosen FRP mixture. A laminate is formed by a number of layers with a constant thickness. These layers are called lamellae. The properties of a lamellae depend on the alignment and orientation of the fibres within the lamellae. In order to have good mechanical properties in all directions, most laminates are built up by stacking lamellae onto each other. The angle between consecutive lamellae differs to provide the good mechanical properties in all directions for the laminate. An example of a stacking order is shown in Figure 5.

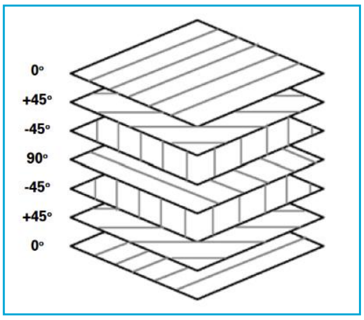


Figure 5: Example of a stacking order [Cripps, 2016]

4. Variant Study

4.1 Chapter content

There are different types of quay walls that can be applied. The most suitable type of quay wall for the case study was found by means of a variant study. The variant study contained 4 different types of quay walls. The external forces and moments have were calculated for each variant and where required balance checks have been performed. This chapter will present the variant study. The considered variants will be discussed before the types of calculations and checks will be elaborated into more detail. This chapter will be concluded by a recommendation based on the calculations. This recommendation will then be elaborated into more detail in the following chapters of this report.

4.2 The variants

Nowadays there are so many different quay walls that it is convenient to categorize them. There are 4 types of quay walls, namely gravity type quay walls, sheet-pile type quay walls, piled quay walls and quay walls with a special foundation. This classification is based on the mechanical behaviour of the different types of quay walls. The variants that are taken into account for this feasibility study are the cantilever sheet pile, an anchored sheet pile, a caisson and an L-wall structure. These variants belong to the categories gravity type quay walls and the sheet pile type quay walls. The quay walls with a special foundation are not taken into account due to the fact that the case study does not require a special foundation based on the geotechnical data. The piled quay walls are not presented in the variants because this type of quay wall is the most difficult quay wall to design merely out of composite. The piled type quay wall is loaded in compression and tension and FRP has a high tensile strength but the compression strength is not remarkable.

4.3 Dimensions of the variants

All the variants are dimensioned for the same soil profile and loading scheme. The soil profile is as mentioned in 2.4 Geotechnical data and the water level and groundwater level in accordance with Table 2. Only the surface load as mentioned in Table 4 is taken into account for the variant study.

To dimension the variants the horizontal earth pressure must be calculated. The horizontal earth pressure is calculated by multiplying the vertical earth pressure with an active or passive coefficient.

4.3.1 Sheet pile type quay walls

The length of the sheet pile quay walls is determined with different methods since there are two different types of sheet pile quay walls. The cantilever sheet pile is calculated with the following steps;

1. The resulting horizontal soil stresses are calculated for the active and passive side,
2. The embedded depth is determined with the moment equilibrium around the toe of the sheet pile. The toe of the sheet pile is at a yet unknown depth. By stating that the moment around the toe should be zero the embedded depth can be calculated,
3. To find the maximum moment in the sheet pile the depth of the point where the shear force is zero is calculated,
4. The moment is then calculated around the point where the shear force is zero.

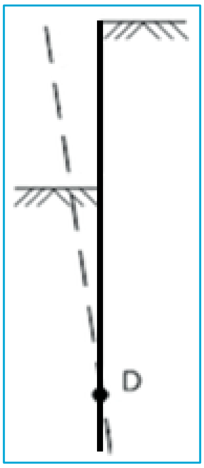


Figure 6: Cantilever sheet pile

The detailed calculations of the cantilever sheet pile are shown in Appendix B: Elaborated calculations of the variant study. A schematization of a cantilever sheet pile is shown in Figure 6 in which point D resembles the rotation point.

The anchored sheet pile quay wall is calculated according to the method of Blum. With the method of Blum there are 3 unknown parameters that have to be calculated, namely the embedded depth, the anchor force and the substitute force. The method of Blum thus consists of the following steps;

1. Compute the theoretical embedded depth using the condition that the horizontal displacement at the level of the anchor must be zero. Due to the schematisation of the method of Blum the theoretical depth is multiplied with 1.2 for compensation,
2. Compute the equilibrium of moments around the toe of the sheet pile in order to determine the anchor force,
3. Compute the substitute force from the horizontal equilibrium, which must be equal to zero,
4. The resulting horizontal stress diagram is used to calculate the maximum moment in the anchored sheet pile. From the effective horizontal stress the shear force can be calculated. The moment diagram then follows from the shear force diagram where the maximum moment will be at the point where the shear force is zero.

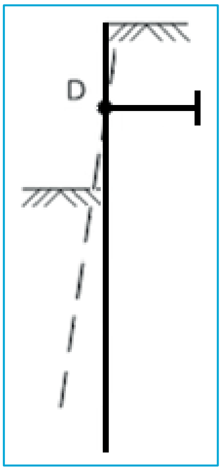


Figure 7: Anchored sheet pile

For the elaborated calculations the reader is referred to Appendix B: Elaborated calculations of the variant study. A schematization of a cantilever sheet pile is shown in Figure 7 in which point D resembles the rotation point.

4.3.2 Gravity type quay walls

The gravity type quay walls are checked for their horizontal, vertical and rotating stability. For both gravity type quay walls a density of 1300 kg/m^3 for the FRP is assumed. The two variants are checked for two situations, namely for the situation where the surface load is present and the situation where the surface load is absent. These situations are distinguished due to the fact that the surface load might be favourable for the stability of the gravity structures. For the caisson as well as the L-wall an embedded depth of 1 meter is assumed. The thickness of the FRP elements of the caisson and L-wall are set at a thickness of 1 meter.

The structures are checked for their horizontal, vertical and rotational stability. Piping has also been checked. The development of the water pressure along the seepage length for a structure that is embedded in the soil does not develop in a linear way. The rectification of the development of the water pressure has been taken into account for the stability calculations. Due to the small difference between the ground water level and the water level in the river, this rectification will not have a large influence on the outcome of the stability checks.

The maximum acting load on the soil should not be higher than 500 kN/m^2. This is a rule of thumb for densely packed sand. Soil cannot cope with tensile forces and the minimum acting load on the soil should therefore be larger than zero.

The calculations to perform the stability checks are presented in Appendix B: Elaborated calculations of the variant study. Figure 8 shows a schematization of an L-wall as well as a caisson.

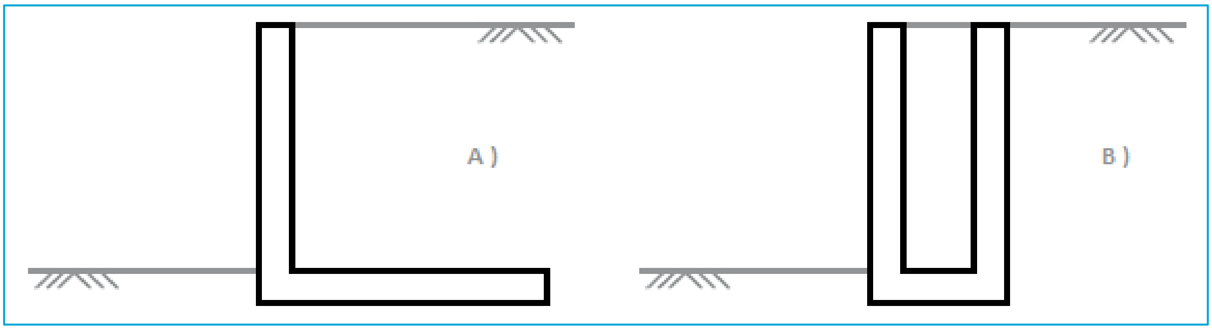


Figure 8: A) representation of an L-wall; B) representation of a caisson

4.4 Analyses of the variants

Based on the beforehand discussed checks and calculations the maximum moments for the variants have been calculated. For the sheet pile quay walls also the length of the sheet pile has been calculated. The results of these calculations and other important parameters and outcomes are presented in Table 10 and Table 11. A positive moment in Table 11 works anticlockwise.

Table 10: Governing properties of the sheet pile type quay walls

	Cantilever sheet pile	Anchored sheet pile
Length	24.58 m	20.38 m
M_{max}	4695.2 kNm	1119.7 kNm
F_{anchor}[2]	Not relevant	264.62 kN

Table 11: Governing properties of the gravity type quay walls

	L-wall	Caisson
Retaining height	12 m	12 m
Length of the bottom slab	12 m	9.35 m
Moment rotation point	-12355 kNm	-5769 kNm
Moment geometric centre of gravity	-4371 kNm	1858 kNm

The cantilever sheet pile requires a larger length than the anchored sheet pile. The moment in the cantilever sheet pile is roughly 4,5 times the moment in the anchored sheet pile. A restriction for a sheet pile is the displacement at the top of the sheet pile. The cantilever sheet pile will displace significant more than the anchored sheet pile. The anchored sheet pile is therefore favourable.

The caisson and L-wall are designed for the same retaining height. The length of the bottom slab is not equal for both gravity type quay walls. The caisson has a smaller bottom slab but on the contrary, the caisson has two vertical walls while the L-wall only has one vertical wall. The external moments working on the gravity structures are smaller for the caisson than for the L-wall. The caisson is therefore favourable.

Now that the cantilever quay wall and the L-wall have been eliminated there are two remaining options. A disadvantage of a caisson is that it is a kind of structure that will need additional ground work. The additional ground work consists of excavating and backfilling. To install the caisson it might be necessary to install an additional sheet pile so that the caisson can be placed on its final location. All of these factors will make the project more expensive. Taking into account that just the price of the material FRP is higher when compared to the price for the materials steel and concrete it is not favourable to pick the caisson as the most suitable variant.

Table 12 is a summary of the above. For the categories: material, moment and deformation a score from 1 to 4 has been given to each variant. Where 1 indicates that it is the best option while the score 4 indicates that it is the least suited option.

Table 12: Decision matrix

	Material	Moment	Deformation	Total
Cantilever sheet pile	2	4	4	**10**
Anchored sheet pile	1	1	3	**5**
L-wall	3	3	2	**8**
Caisson	4	2	1	**7**

The result of Table 12 is that the anchored sheet pile quay wall is recommended as most suitable FRP quay wall for the case study. The anchored sheet pile type quay wall will be designed in FRP in the following chapters.

[2] The anchor is assumed at 0,5 meters above the ground water level without an angle with respect to the horizontal

5. Design of the quay wall

5.1 Chapter content

The most suitable type of quay wall was indicated in the previous chapter and the design of the quay wall will be discussed in this chapter. First the external loads on the quay wall are indicated and discussed. Based on the data presented in 2.4 Geotechnical data a D-sheet model has been built. This model will be discussed briefly in this chapter after the first paragraph. The discussion of the D-sheet model is followed by the paragraph that presents the determination of the maximum allowed deflection of the quay wall which will be used as a starting point for the design. This chapter is concluded with paragraphs concerning general checks for a quay wall and a summary of the design.

The design of the quay wall is related to the retaining wall of the quay wall. The superstructure and anchor are not incorporated in the design of the FRP quay wall. It is assumed that the superstructure will have the same dimensions as in the case study. Only the volumetric weight of the superstructure is adjusted to the volumetric weight of FRP.

5.2 External forces

In the previous chapter 'Variant Study' the forces that have been taken into account are the horizontal earth pressures, water pressures and the surface load. Figure 9 shows the active and passive horizontal ground pressure as well as the resulting horizontal ground pressure, the shear force diagram and the moment diagram of the anchored sheet pile due to these forces.

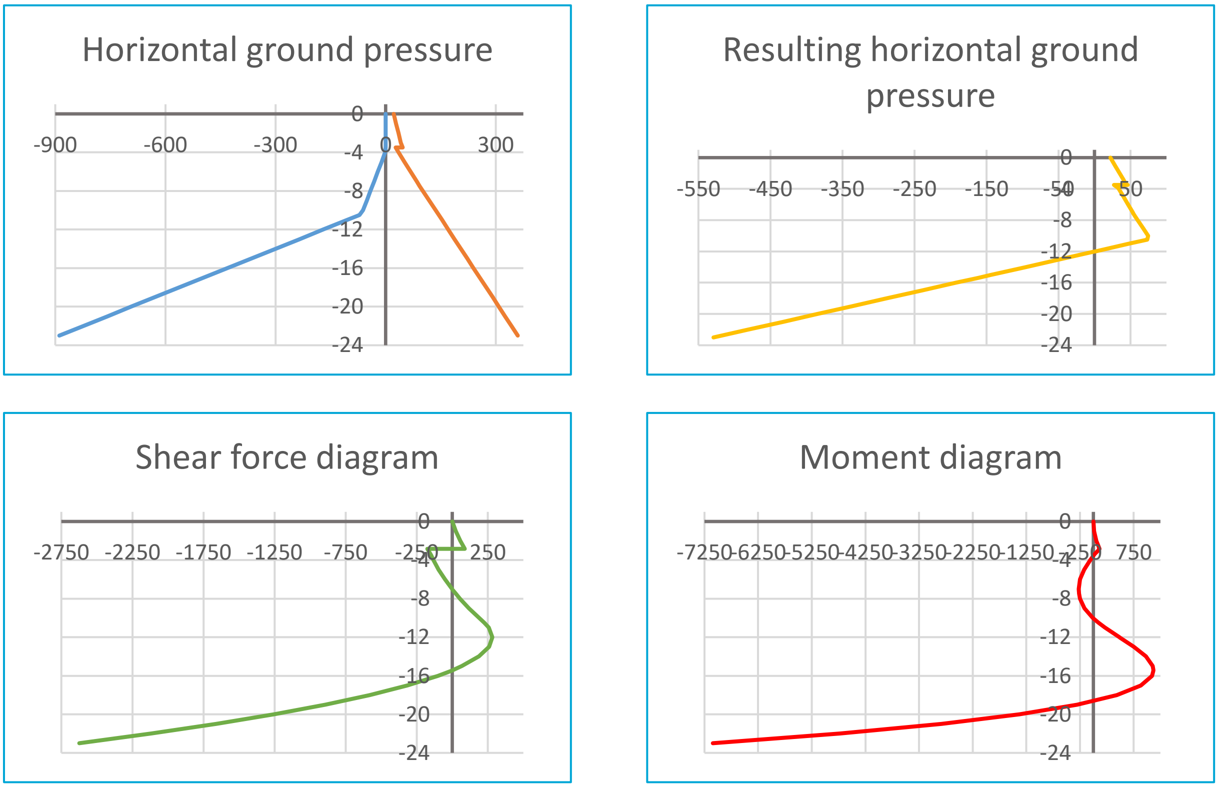


Figure 9: Diagrams belonging to the anchored sheet pile quay wall

However, as mentioned in Table 4, there are more external forces that will influence the moment distribution and thus the deflection of the quay wall. These forces will have to be taken into account for the detailed design of the FRP quay wall. The external forces are input for the D-sheet model. The D-sheet model will give the moment distribution and deflection of the quay wall which will be used for the cross section design of the quay wall. The required input for D-sheet must be entered per running meter sheet pile and thus the external forces have to be rewritten as forces per running meter. The transformation of these forces will be discussed in Appendix C: Transformation of the external loads. This paragraph only presents the transformed values and the load combinations.

5.2.1 *Transformed external forces*

The values of the external forces per running meter sheet pile are presented in Table 13. These values are obtained from the design report of the case study. These forces are not altered for the design of the FRP quay wall. The derivation of the values per running meter are based on centre to centre distances which are not influenced by the design of the quay wall.

Table 13: External loads per running meter quay wall

	Force [kN/m]	Working point or area
Scour protection	5	At NAP -7.00 m over a length of 10 meters from the quay wall
Negative Adhesion	42, 96, 100	At NAP -0.35 m, the point where the moment is maximum and the point where the shear force is zero.
Crane load	12	NAP +2.50 m
Mooring load	60	NAP +4.00 m
Impact load	60	NAP +3.15 m
Mooring pin load	60	NAP +1.00 m

5.2.2 *Load combinations*

The external forces discussed so far will not act on the quay wall all at the same time. The external forces will act on the quay wall in so called load combinations. When a vessel is moored at the quay wall by means of the bollards the mooring pin load cannot occur at the same time. These load combinations are shown in Table 14. The crane load can work in two directions, landwards and to the water. This is caused by the wind load on the crane and since the wind can come from multiple angles these two directions are taken into account for the design of the quay wall.

The impact load and landwards directed crane load are favourable loads. The most feared failure for a quay wall is namely the deflection directed towards the water. When this failure mechanism occurs the soil behind the quay wall will slide away and cause huge settings of the hinterland. Combination 2 and 3 are therefore not taken into account for the beginning of the design process.

Table 14: Load combinations

	Combination 1	Combination 2	Combination 3	Combination 4
Surface load	X	X		X
Bed protection	X	X	X	X
Mooring force	X			
Impact load		X	X	
Mooring pin load				X
Crane load landwards		X	X	
Crane load directed to the water	X			X

The vertical load of the crane is taken by the foundation of the crane rail itself. The crane rail has its own foundation which is coupled to the quay wall by means of horizontal rods. The horizontal forces induced by the crane are taken by the quay wall.

5.3 D-sheet model

Based on the data presented in 2.3 Requirements and 2.4 Geotechnical data a D-sheet model has been created and calibrated for the design of the case study. The anchor used in the D-sheet model is the same anchor that has been applied in the case study. This model is used to calculate the deflection, maximum moment and maximum shear force in the quay wall. The data that have been used to create the D-sheet model is presented in Appendix D: D-sheet model. Figure 10 gives a representation of the D-sheet model without external forces.

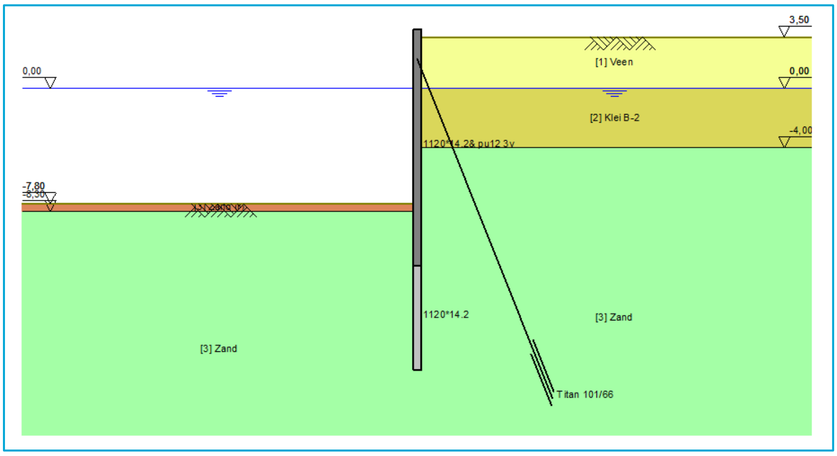


Figure 10: Representation of the D-sheet model

5.4 Maximum deflection

The design of FRP structures is, according to design handbooks, dominated by the maximum allowed deflection of the structure. In the program of requirements a maximum deflection of the quay wall has not been incorporated. The maximum deflection must therefore be determined based on a design rule. The allowed deflection is determined with the heeling angle of the design vessel. The heeling angle is given in Table 1 and is equal to 2 degrees. When the vessel is moored to the quay wall the maximum heeling angle is thus 2 degrees. The distance between the quay wall and the vessel with a heeling angle of 2 degrees at the bottom of the vessel will be the maximum allowed deflection of the quay wall. Figure 11 shows how the maximum deflection has been determined.

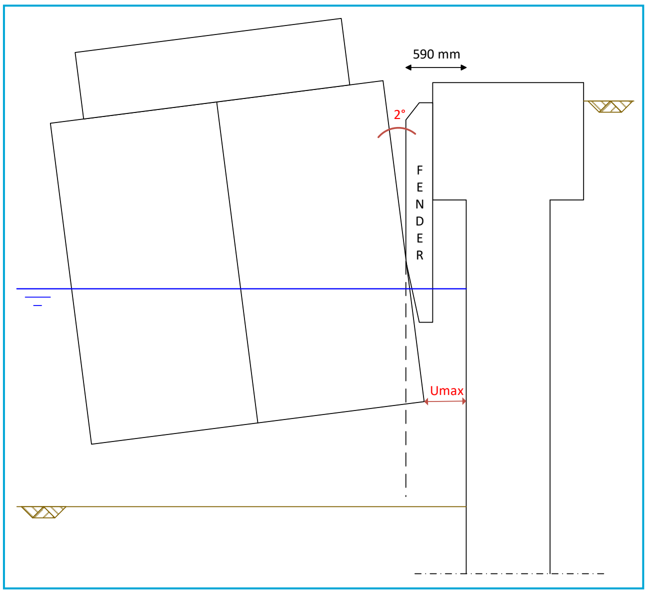


Figure 11: Determination of the maximum deflection. Not to scale

The distance between the outer line of the fender and the tubular pile of the quay wall is 590 millimetres according to a cross section of the design of the case study. The superstructure of the quay wall as well as the fender are kept the same for the design of the FRP quay wall and therefore these dimensions can be used to determine the maximum allowed deflection of the quay wall. The design vessel has a draft of 4 meters. With an angle of 2 degrees this results in a horizontal displacement of 140 millimetres of the vessel. The distance between the quay wall and the vessel is then 450 millimetres. The maximum deflection of the quay wall is assumed to be equal to 90% of the distance between the displaced vessel and quay wall. The maximum allowed deflection of the quay wall is equal to 400 millimetres.

5.5 Designing the FRP quay wall

The design of the FRP quay wall focuses on the cross section of the quay wall. The cross section determines the moment of inertia. The moment of inertia in combination with the Young's modulus determines the stiffness of the quay wall and therefore the displacement and moment distribution.

The following steps have been taken to design the cross section of the FRP quay wall;

Step 1. Pick a fibre and resin,
Step 2. Lower the stiffness of the quay wall so that the maximum deflection is reached,
Step 3. Determine the required moment of inertia,
Step 4. Check whether the quay wall can be designed as a straight sandwich structure,
Step 5. Adjust the cross section so that the required moment of inertia is obtained,
Step 6. Design the laminates,
Step 7. Check the cross section for the maximum strain criteria.

These steps are presented in a flowchart in Figure 12. Each step will be elaborated into more detail in the remaining of this paragraph.

The [CUR96, 2003] is used in the design process of the FRP quay wall. The most important features of the [CUR96, 2003] are summarized in Appendix E: CUR96.

The length of the quay wall is held in the first instance equal to the length of the case study, which is 23 meters.

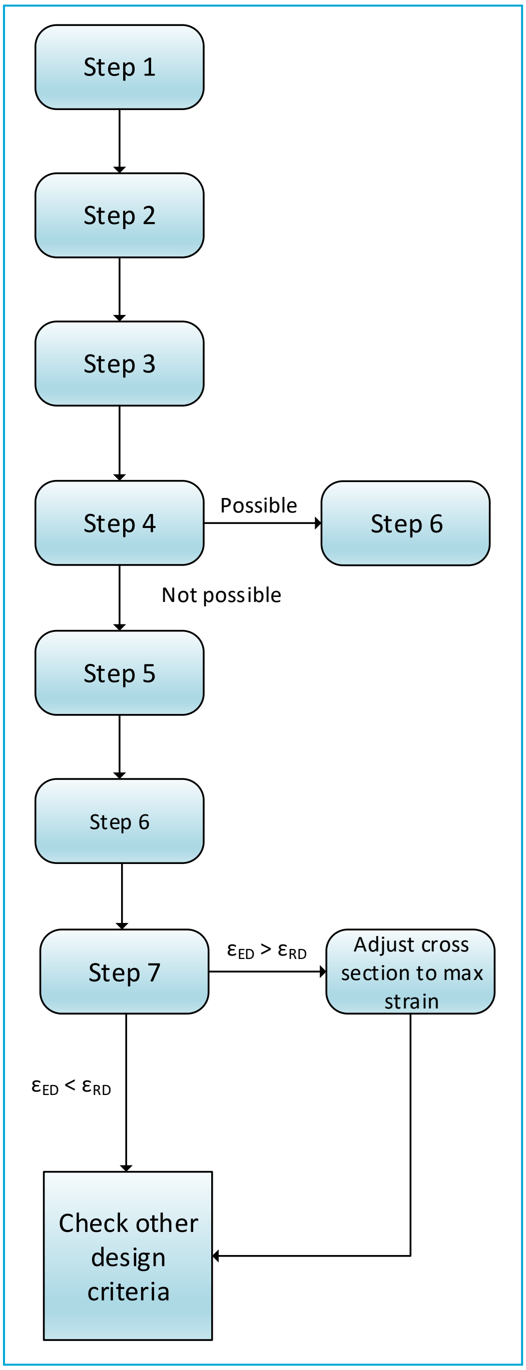


Figure 12: Flowchart of the design steps

5.5.1 *Step 1: The used fibre and resin*

[Valk, 2016] shows that the price of FRP is higher than steel and to be able to compete with a steel sheet pile quay wall the cost of an FRP quay wall should be kept as low as possible. There are 3 options to choose as a fibre, namely glass fibre, carbon fibre and polyaramid. Of these three options the carbon fibre has the highest Young's modulus and glass fibre the lowest. The Young's modulus of carbon is approximately 3 times higher than the Young's modulus of glass fibre. The price ratio between glass fibre on one side and polyaramid and carbon fibre on the other side is approximately 6 to 8.

The stiffness of a quay wall is determined by the Young's modulus and the moment of inertia. The moment of inertia depends on the cross section of the quay wall. The Young's modulus is a static variable while the moment of inertia is a dynamic variable. To reach the required stiffness one can gain more profit from altering the dimensions of the cross section than chancing the Young's modulus.

Despite that the Young's modulus of carbon is roughly 3 times higher than that of glass fibre, a glass fibre reinforcement will be used in the design of the FRP quay wall.

An argument to back the decision for glass fibre is that the design codes and handbooks that are available are only applicable for glass fibre reinforcements.

The choice for resin is merely cost driven and a polyester resin is therefore chosen to be applied in the design of the FRP quay wall.

5.5.2 Step 2: Lowering the stiffness

The maximum allowed deflection of the quay wall is used as a starting point for the design of the cross section. The stiffness of the quay wall that has been designed in the case study is used as a starting point. The stiffness of the quay wall is divided by the Young's modulus of steel. With this step the moment of inertia of the steel combi wall has been obtained. This moment of inertia is then multiplied with the Young's modulus of FRP. The Young's modulus of a glass fibre polyester is assumed to be equal to 25.8 GPa. This first assumption is based on conversations with FRP experts and [CUR96, 2003].

In the D-sheet model the stiffness of the quay wall is reduced until the maximum deflection of the quay wall is reached. The reduction of the stiffness has been done in steps of 10% of the current stiffness.

The results of the reduction of the stiffness is shown in Table 15 and it can be concluded that a stiffness of 2.421 x 10^4 kNm^2 is required to come close to the maximum deflection criteria. Since this is the first step in the design cycle the required stiffness will not be altered until it perfectly fits the deflection criteria.

Table 15: Required stiffness

EI [kNm^2]	$M_{max,ULS}$ [kNm]	$U_{max,SLS}$ [mm]
9,524E+04	746,7	109,9
8,572E+04	747,7	119,2
7,715E+04	749,1	129,5
6,943E+04	750,8	141,1
6,249E+04	753,0	154,0
5,624E+04	755,4	168,4
5,062E+04	758,0	184,4
4,555E+04	761,2	202,6
4,100E+04	764,9	222,8
3,690E+04	769,2	245,5
3,321E+04	774,2	271,2
2,989E+04	780,1	300,2
2,690E+04	786.8	332.9
2,421E+04	**794,6**	**369,4**
2,179E+04	803,7	411,1

5.5.3 Step 3: The required moment of inertia

The required moment of inertia is easily obtained by dividing the required stiffness by the Young's modulus of FRP. In this step an anisotropic laminate is assumed. Anisotropic means that the properties of the material are not equal in each direction. This anisotropic laminate consists of 55% of the fibres in the main direction and 15% of the fibres in the other three directions. The fibre volume percentage is 50%. The Young's modulus that is relevant for the bending of the quay wall is then equal to 25.8 GPa [CUR96, 2003].

The value of 25.8 GPa for the Young's modulus of a glass fibre polyester laminate cannot be used without applying a material factor and conversion factor according to [CUR96, 2003]. The Young's modulus according to [CUR96, 2003] is a representative value. The representative value must be transformed to a design value by means of a material factor and conversion factor. The material factor is composed with 2 separate factors while the conversion factor consists of 4 separate factors. The values for these factors and when they must be applied can be found in Appendix E: CUR96.

The design value of the Young's modulus is now calculated according to the following formula;

$$S_d = \frac{R_R}{\gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{ct} \cdot \gamma_{cv} \cdot \gamma_{ck} \cdot \gamma_{cf}} \quad \textbf{Eq. 1}$$

In which;
- S_d = The design value
- R_R = The representative value
- γ_{m1} = The material factor
- γ_{m2} = The manufacturing factor
- γ_{ct} = The temperature factor
- γ_{cv} = The moisture factor
- γ_{ck} = The creep factor
- γ_{cf} = The fatigue factor

The design value for the Young's modules becomes therefore 12 GPa instead of the 25.8 GPa. Dividing the required stiffness by this design value of the Young's modulus one obtains the required moment of inertia. The required moment of inertia is 2.02 x 10^{-3} m^4.

5.5.4 Step 4: A flat sandwich quay wall

As mentioned before the cross section of the quay wall determines the moment of inertia. The required moment of inertia has been calculated in the previous step. A sandwich construction will be used for the FRP quay wall since the lightweight core of a sandwich construction increases the stiffness significantly. Another motivation to apply a sandwich structure is that FRP construction experts have stated that the thickness of a laminate cannot be more than 10 centimetres. The thickness of the laminate is limited due to the capacity with regard to the interlaminar stresses of the laminate according to FRP experts of Gemeente Rotterdam.

A single laminate cannot reach the required moment of inertia without the use of enormous dimensions. The thickness of a single laminate must be 29 centimetres to achieve a moment of inertia of 2.02 x 10^{-3} m^4.

A sandwich quay wall consists of two skins and a foam core. The foam core has almost no structural properties and will therefore be neglected for the design of the cross section. The skins will contribute the most to the moment of inertia. The CUR96 describes that structures which are exposed to water pressure from groundwater or surface water cannot exceed a nominal strain limit of 0.27% due to the risk of moisture infiltration which will have a negative influence on the mechanical properties of the laminate.

FRP only has a linear elastic behaviour and no plasticity capacity. The linear elastic stress is given by Hooke's law;

$$\sigma = E \cdot \varepsilon \quad \textbf{Eq. 2}$$

The strain in the outer fibre cannot be more than 0.27% and this will be used to determine the maximum allowed stress in the outer fibre with equation 2. The stress is namely the only unknown in this equation since the design value of the Young's modulus has been determined in the previous step.

The stress in the outer fibre is calculated with;

$$\sigma = \frac{M \cdot e}{I} \quad \textbf{Eq. 3}$$

In which;				
	M	=	The moment according to D-sheet	[kNm]
	e	=	0.5*h	[m]
	h	=	Height of the sandwich structure	[m]
	I	=	The required moment of inertia	[m^4]

Combining equation 2 and 3 gives the height of the sandwich structure which must be 246 millimetres.

The thickness of the skins can be calculated now that the height of the sandwich structure is known. The sandwich structure resembles an I-profile. The moment of inertia of an I-profile consists of the contribution of the flanges and the web. The largest contribution to the moment of inertia is delivered by the flanges. By means of simplification the web is therefore neglected. This results in the following formula for the moment of inertia;

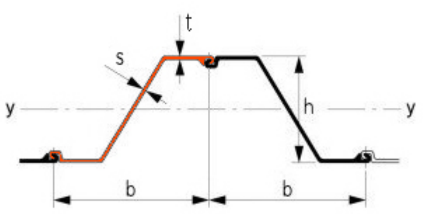
$$I = 2 \cdot (\frac{1}{12} bd^3 + Aa^2) \qquad \textbf{Eq. 4}$$

In which;				
	b	=	The width of the flange	[mm]
	d	=	The thickness of the flange	[mm]
	A	=	The area of the flange	[mm^2]
	a	=	e – 0.5d	[mm]

With the required moment of inertia equal to 2.02 x 10^{-3} m^4 this leads to a required thickness of 228 millimetres for a single skin. The skin of a sandwich structure is a laminate and it has been mentioned that the thickness of a laminate is limited to 10 centimetres or 100 millimetres. It is therefore concluded that the FRP quay wall cannot be designed as a straight quay wall with a sandwich structure as cross section. A more complex cross section has to be designed to reach the required moment of inertia. This will be elaborated into more detail in step 5.

5.5.5 *Step 5: Design of the cross section*

Now that a straight forward sandwich construction cannot be used for the design of the cross section a more complex cross section must be developed. Inspiration for the more complex form has been taken from the Z-profiles that are used for steel sheet piles. These sheet piles have a high moment of inertia due to the fact that the flanges are located far from the gravity centre of the cross section. Figure 13 shows the cross section of a Z-profile of ArcelorMittal along with various sections and the corresponding dimensions and properties.



Section	Dimensions				A	G_{sp}	G_w	I_y	$W_{el,y}$
	b	h	t	s					
	mm	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^4/m	cm^3/m
AZ 18[1)]	630	380	9,5	9,5	150	74,4	118,1	34 200	1 800
AZ 18 10/10	630	381	10,0	10,0	157	77,8	123,4	35 540	1 870
AZ 26[1)]	630	427	13,0	12,2	198	97,8	155,2	55 510	2 600
AZ 46	580	481	18,0	14,0	291	132,6	228,6	110 450	4 595
AZ 48	580	482	19,0	15,0	307	139,6	240,6	115 670	4 800
AZ 50	580	483	20,0	16,0	322	146,7	252,9	121 060	5 015

Figure 13: Cross sections of ArcelorMittal Z-profiles [ArcelorMittal, 2016]

A Z-shaped cross section is assumed as a starting point for the design. The web of the Z-profile stands perpendicular to the flanges in contradiction with the Z-profiles of for example ArcelorMittal. The common shape of a Z-profile is determined by the fabrication technique. Perpendicular angles are not easy to roll into a steel plate. An FRP structure however is constructed with the help of a mould and perpendicular angles are therefore not a restriction. The inclined web also requires more material to cover the same distance than a perpendicular web. As mentioned before the material costs of FRP are much higher than the costs of steel and should therefore be kept as low as possible to be able to compete with steel.

Figure 14 shows the cross section that has been used to design the more complex form of the cross section. Only the skins of the sandwich structure are shown. The webs that will connect the skins are neglected in the moment of inertia calculation.

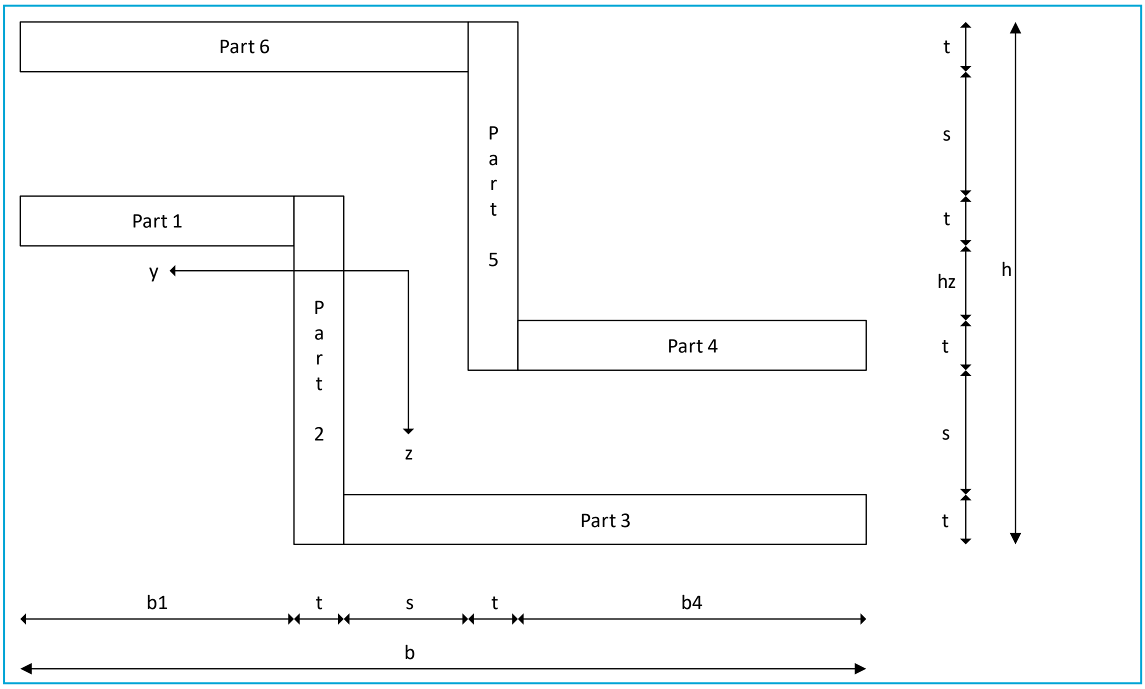


Figure 14: Schematization of the cross section

The dimensions of the cross section are determined by the parameters t, s, hz, b1 and b4. Where t is the thickness of the skin, s is the spacing between the skins, hz is the vertical distance between part 1 and part 4, b1 is the width of part 1 and b4 is the width of part 4. The skins are divided into 3 parts each to simplify the calculation. For each individual part the moment of inertia is calculated and the total moment of inertia is the sum of the moment of inertias of all parts.

Table 16 gives the estimated dimensions of the first design. With this first estimation the moment of inertia is calculated per Z-profile. The width b of the Z-profile with the dimensions as in Table 16 is 502 millimetres. The input for the D-sheet model is per running meter of quay wall. The moment of inertia will therefore be calculated for 2 coupled Z-profiles. The width of 2 coupled Z-profiles then becomes 1004 millimetres which is more or less a running meter of quay wall. If the dimensions of the cross section have to be altered, the moment of inertia of 2 coupled Z-profiles will have to be transferred to a value per running meter of quay wall.

Table 16: Dimensions of design 1

		Dimensions [mm]
Variables	t	26
	s	100
	hz	175
	b1	175
	b4	175
Elements	h1	26
	h2	353
	h3	26
	h4	26
	h5	353
	h6	26
	b1	175
	b2	26
	b3	301
	b4	175
	b5	26
	b6	301

With the dimensions of this first design named design 1, the moment of inertia becomes 1.02 x 10^{-3} m^4 for a single Z-profile. Per running meter of quay wall the moment of inertia is then 2.04 x 10^{-3} m^4 which is slightly more than the required moment of inertia. This design will be checked in step 7 for the maximum strain criteria of 0.27% and other failure mechanisms.

Alternative shapes for the cross section have been considered as well. Triangular, corrugated sheet and a circular profile have all been considered. Based on the standard formulas that can be applied to calculate the moment of inertia, the Z-profile has been chosen as the best fit to this problem.

The choice for the shape of the cross section is driven by the moment of inertia of the cross section. There are several rule of thumbs to calculate the moment of inertia for a certain cross section. The moment of inertia for a rectangular shape is given by $^{1}/_{12}bh^3$ while the moment of inertia for a triangular shape is given by $^{1}/_{36}bh^3$. So a rectangular shape with the same width and height obtains a moment of inertia that is 3 times bigger than a triangular shape with the same width and height.

The moment of inertia for a tubular profile is equal to $\pi R^3 t$ and the moment of inertia for a thin walled semicircle is equal to $0.298R^3t$.

The required moment of inertia per running meter quay wall is equal to 2.02×10^{-3} m^4 as calculated at step 4. When the width or radius of a certain shape is set to 1000 millimetre, which is equal to a running meter quay wall, the other dimensions of the shapes can be determined.

A rectangular shape requires a height of 289 millimetres and a triangle requires a height equal to 417 millimetres. A tubular shape with a radius of 500 millimetres requires a thickness of 5 millimetre. The semicircle with a diameter equal to 1 metre must have a thickness of 54 millimetres.

A point that has to be taken into account for the determination of the shape of the cross section are the joints. A quay wall cannot be designed out of merely tubular profiles. These profiles must be connected to one another to provide a water and soil retaining structure. A tubular profile cannot be applied without some other shape between two tubular profiles. This will result in a so called combi wall.

Another point for the tubular and semicircle shape is the possibility of torsion. To design the tubular and semicircle shape in such a way that it can withstand torsion the laminates have to be designed as a quasi-isotropic laminate. When a laminate is designed as a quasi-isotropic laminate the Young's modulus will be less in comparison with an anisotropic laminate. If the Young's modulus reduces the moment of inertia must be increased to obtain the required stiffness.

When the above mentioned remarks are taken into account it seems that the rectangular shapes and thus the Z-profile is the best option as shape for the cross section of the FRP quay wall.

5.5.6 Step 6: Designing the laminates

In the previous steps an anisotropic laminate, which is presented by the CUR96, has been used to determine among others the required moment of inertia. This step will focus on the design of the laminates for the skin as well as the webs that will be constructed to connect the skins with one another and provide more stiffness to the cross section. The design of the web laminate will be presented after the design of the skin laminate.

For the design of the laminates some indices have been used to describe properties in certain directions. The used definitions for the axes will be elaborated before the properties of the fibre and resin will be presented.

Figure 15 shows the coordinate system that will be used to describe properties in certain directions. Parallel with the fibre runs the 1-axis while the 2-axis runs perpendicular to the fibre. Figure 15 shows the coordinate system for a single fibre at the top while at the bottom of the figure the coordinate system for multiple fibres placed in the same direction is presented. Parallel placed fibres are called a unidirectional lamella.

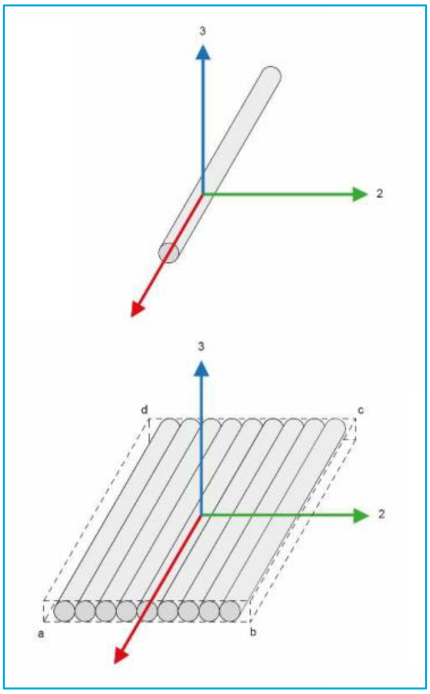


Figure 15: Reference axes of a fibre and lamella [CUR96, 2016]

The coordinate system for a laminate is shown in Figure 17. This coordinate system is linked to the strongest direction of the laminate, the so called 0°-direction. The x-axis corresponds with the 0°-direction of the laminate and the y-axis corresponds with the 90°-direction of the laminate.

One of the parameters that influences the mechanical properties of the laminate is the stacking order. The stacking order is not only defined by the thickness and number of lamellae but also by the angle of a lamellae. This angle indicated in which directions the fibres are placed in that specific lamellae with respect to the 0°-direction of the laminate. The angles that have been used in this study are 0°, +45°, -45° and 90°. These directions are shown in Figure 16.

The properties of the fibre and resin that have been used in the design of the laminate will be discussed first before the design of the skin and web laminate will be elaborated into more detail.

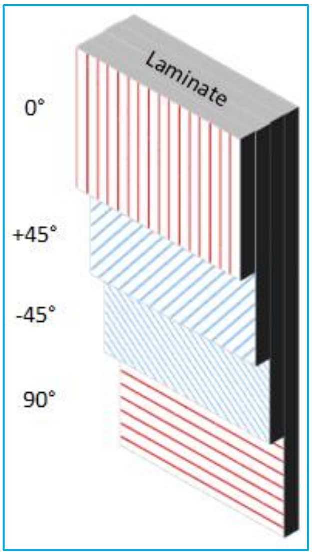


Figure 16: Applied fibre directions

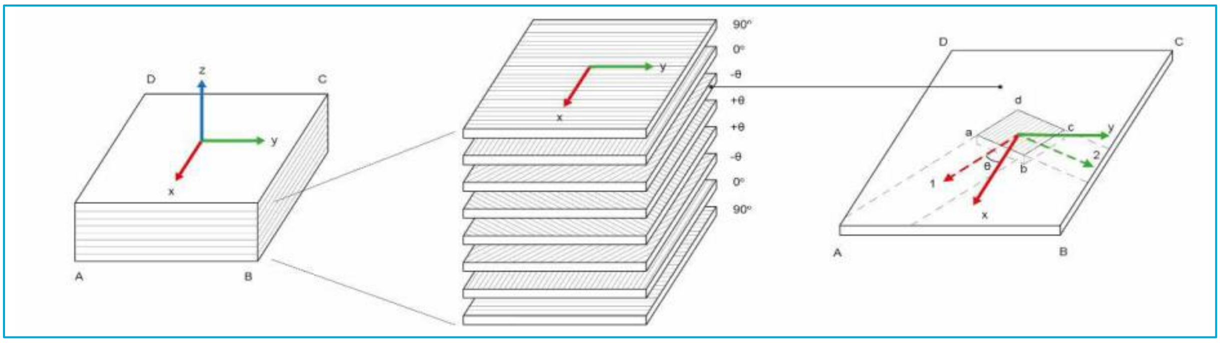


Figure 17: Reference axes of a oriented lamella abcd (1,2) with respect to global laminate axes (x,y) (plate ABCD) [CUR96, 2016]

5.5.6.1 Used fibre and resin properties

Step 1 of the design flowchart clarified the choice for the glass fibre reinforcement and polyester resin. With the program eLamX^2 lamellae can be designed. The program is developed by the Technical University of Dresden and can be downloaded freely. Part of the program is a material database. In this database several types of laminates are included which are taken from literature.

In the design of the FRP quay wall it is chosen to start with the properties of the fibre and resin and combine the two to a lamella. With these lamellae the laminate is constructed which will be clarified later on. The used properties of the fibre and resin are shown in Table 17. With the rule of mixture, which has been explained in [Valk, 2016], the properties of the lamella can be calculated. These calculations as well as the stacking order and the mechanical properties of the laminate are discussed in the remaining of this subparagraph. This has been done for the skin laminate as well as the web laminate since they are not the same.

Table 17: Properties of the applied fibre and resin [CUR96, 2016]

Property				Fibre	Resin
	Density	ρ	[kg/m^3]	2570	1200
Tension parallel to the fibre direction	Young's modulus	E_1	[MPa]	73100	3550
	Strain	ε_1	[%]	3.8	1.8
	Stress	σ_1	[MPa]	2750	55
	Poisson ratio	ν_1	[-]	0.24	0.38
Tension perpendicular to the fibre direction	Young's modulus	E_2	[MPa]	73100	3550
	Strain	ε_2	[%]	2.4	1.8
	Stress	σ_2	[MPa]	1750	55
	Poisson ratio	ν_2	[-]	0.24	0.38
Compression parallel to the fibre direction	Strain	ε_1	[%]	2.4	1.8
	Stress	σ_1	[MPa]	1750	55
Shear	Shear modulus	G_{12}	[MPa]	30000	1350
	Strain	γ_{12}	[%]	5.6	3.8
	Stress	τ_{12}	[MPa]	1700	50

5.5.6.2 *Skin laminate*

The skin laminate will be designed as an anisotropic laminate which means that the properties of the laminate are not the same in each direction. To be able to use the uniform strain criteria as given in the CUR96, the laminate design must fulfil a certain requirement. It is stated in the CUR96 that in each direction at least 15% of the fibres is present. This leaves 40% of the fibres that can be placed in any desired direction. The skins of the sandwich are merely loaded by tension or compression around the vertical axis of the quay wall. The remaining 40% of the fibres are therefore placed in the 0°-direction. A fibre volume fraction of 55% is taken as a first estimate. This means that a lamella consists for 55% of fibres and the remaining 45% is the chosen resin.

As can be seen from Table 16, the thickness of the outer skin is set to 26 millimetres. 55% of the fibres are placed in the vertical direction, the 0°-direction. So 55% of the thickness of the skin will be unidirectional lamellas with fibres in the 0°-direction. The 90°-direction, +45°-direction and -45°-direction will each be 15% of the total thickness of the skin. To minimalize the interlaminar stresses it is useful to design the stacking order of the lamellas in such a way that each lamella has a different fibre direction. It is also preferable to construct the laminate of many thin lamellae [Achtergrondrapport CUR96, 2003].

Table 18: Engineering constants of the skin laminate

E_1	[MPa]	28806,3
E_2	[MPa]	16140,6
G_{12}	[MPa]	6209,8
ν_{12}	[-]	0,301
ν_{23}	[-]	0,169
ρ	[kg/m^3]	1953.5

The stacking order of the skin laminate can be found in Appendix F: Design of the FRP quay wall. The engineering constants are shown Table 18.

The engineering constants are input for the hand calculation in the next step of the flowchart as well as for the FEM program SCIA. The engineering constants have been calculated with the program Kolibri. This program uses the classical laminate theory to determine the engineering constants.

5.5.6.3 *Web laminate*

In contrast to the skin laminate, the web laminate is designed as a quasi-isotropic laminate. The properties of the laminate are therefore more or less equal in every direction. The web laminate is designed as a quasi-isotropic laminate since the web of the sandwich is mainly loaded with shear stress. 25% of the fibres are therefore placed in each direction making the laminate quasi-isotropic. A fibre volume fraction of 50% has been applied to the web laminate. The higher the volume fraction the higher the Young's modulus will be.

So if the outcome of step 7 is that the laminate does not fulfil the strain criteria one of the options to improve the laminate is by increasing the fibre volume fraction.

The shear modulus of a fibre reinforced polymer is an important parameter with respect to the shear deformation. The shear modulus can be so low that the deflection due to shear can be normative. The total deflection due to bending and shear is given by the following formula;

$$u_{total} = \frac{M_{D-sheet,SLS}}{EI} + \frac{M_{D-sheet,SLS}}{GA} \qquad \textbf{Eq. 5}$$

In which;	$M_{D\text{-}sheet,SLS}$	=	The maximum moment in the SLS	[kNm]
	EI	=	The stiffness of the quay wall	[kNm^2]
	G	=	The shear modulus	[MPa]
	A	=	Area of the cross section	[m^2]

To determine the required amount of webs per running meter of quay wall the shear deflection is assumed to be 5% of the total deflection. Furthermore it is assumed that the deflection to the shear will be taken by the webs only.

With a shear modulus of approximately 7 GPa for a quasi-isotropic laminate according to [CUR96, 2003] the thickness of the webs per running meter quay wall can be calculated. The value of 7 GPa is a nominal value for the shear modulus. This value must be transformed to a design value by applying equation 1. With the calculated moment of inertia for the cross section designed in step 5 the maximum moment and deflection in the SLS can be calculated with the D-sheet model. The maximum moment is equal to 514.1 kNm and the deflection is 321.1 millimetres. The outcome of the calculation is that per running meter quay wall, the required amount of area for the webs is equal to 0.011 m^2. The height of the webs are equal to the spacing between the skins which, for this design is equal to 0.1 meters. The required amount of web thickness is obtained through dividing the area by the height. The required web thickness is equal to 110 millimetres per running meter quay wall. Therefore it is estimated that a single web has a thickness of 12 millimetres and the centre to centre distance between two webs is set to 100 millimetres.

The required thickness for a web is now known and the laminate can be designed with the program Kolibri. A distribution of 25% of the total thickness per direction will lead to a thickness of 3 millimetres per direction. As advised in the CUR96 the laminate will be stacked by many thin lamellae. The thickness of a single lamellae is set to 0.75 millimetres. The stacking order of the web laminate is presented in Appendix F: The design of the FRP quay wall. The program Kolibri calculates the engineering constants of the laminate with the classical laminate theory. This theory has been explained in [Valk, 2016]. Table 19 shows the engineering constants of the laminate that will be used as input for the FEM program SCIA and the hand calculations for the next step of the design flowchart.

Table 19: Engineering constants of the web laminate

E_1	[MPa]	18312,6
E_2	[MPa]	18312,6
G_{12}	[MPa]	7020,8
ν_{12}	[-]	0,304
ν_{23}	[-]	0,304
ρ	[kg/m^3]	1885

5.5.7 Step 7: Strain criteria check

5.5.7.1 Design 1

The mechanical properties for the skin and web laminate can be used to calculate the maximum strain in the outer fibres of the laminates. The strain criteria check has been performed with different methods. First a simple and quick hand calculation has been made with equation 6. The next step is a simple 2D model made with the FEM program SCIA. Both methods are elaborated into more detail in the remaining of this paragraph.

Hand calculation

For the hand calculation the following formula has been used;

$$\sigma = \pm \frac{M \cdot e}{I} \pm \frac{N}{A} \qquad \textbf{Eq. 6}$$

In which;	M	=	The moment	[kNm]
	e	=	Eccentricity of the place of interest	[m]
	I	=	Moment of inertia	[m^4]
	N	=	Normal force	[kN]
	A	=	Area	[m^2]

The strain criteria check is performed in the ultimate limit state and serviceability limit state. Safety factors have to be applied in the ULS. The safety factors are incorporated in the D-sheet model. Explanation of the D-sheet model can be found in Appendix D: D-sheet model. Table 20 shows the maximum deflection in the SLS as well as the maximum moment and the anchor force in both the SLS and ULS. This data is needed to calculate the maximum stress in the outer fibre of the skin and web laminate.

Table 20: Relevant data obtained with the D-sheet model for design 1

	SLS			ULS	
	U_{max} [mm]	M_{max} [kNm]	F_{anchor} [kN]	M_{max} [kN]	$F_{d,anchor}$ [kN]
Per running meter	321.1	514.1	377.6	763.1	453.12
2 Z-profiles	321.1	516.16	377.6	766.15	453.12

The normal force on the structure is related to the self-weight of the structure, the vertical derivative of the anchor force and the vertical load on the quay wall that is related to the surface load. The normal forces for the SLS and ULS are in formula form as follows;

$$N_{SLS} = EG_{quay} + V_{ss} + EG_{ss} + V_{concrete} + F_{anchor} \cdot \sin\alpha + F_{adh} \qquad \textbf{Eq. 7}$$

$$N_{ULS} = EG_{quay} \cdot 1.2 + V_{ss} \cdot 1.5 + EG_{ss} \cdot 1.2 + V_{concrete} \cdot 1.2 + F_{d,anchor} \cdot \sin\alpha + F_{adh} \qquad \textbf{Eq. 8}$$

In which;	EG_{quay}	=	The self-weight of the quay wall	[kN/m’]
	V_{ss}	=	The vertical load on the superstructure	[kN/m’]
	EG_{ss}	=	The self-weight of the superstructure	[kN/m’]
	$V_{concrete}$	=	The weight of the anchored concrete block	[kN/m’]
	F_{anchor}	=	The anchor load in the SLS	[kN/m’]
	$F_{d,anchor}$	=	The anchor load in the ULS	[kN/m’]
	α	=	The angle of the anchor with the horizontal	[°]
	F_{adh}	=	The negative adhesion	[kN/m’]

The negative adhesion has been assumed to be the same as the negative adhesion that has been calculated for the case study design. Since the cross section of the FRP quay wall is still being designed and has no influence on the outcome of the negative adhesion this is a valid assumption.

Also, due to the fact that the point where the maximum moment occurs in the FRP design is more or less equal to the maximum moment point of the case study. The value for the negative adhesion is 100 kN, see Appendix C: Transformation of the external loads. This appendix also shows the derivation of the vertical load on the superstructure which is equal to 20 kN/m’. The angle between the anchor and the horizontal is 35 degrees, just as in the design of the case study.

The self-weight of the quay wall has been calculated by multiplying the area of FRP in the cross section with its volumetric weight of 1953.5 kg/m^3 for the skins and the length of the quay wall. The force due to the self-weight is then 38.7 kN. The self-weight of the superstructure is roughly estimated. The dimensions of the superstructure are kept the same as in the case study. In the case study the superstructure is made of steel and concrete. The self-weight of the superstructure as mentioned in the case study is scaled to the volumetric weight of the FRP resulting in a self-weight of 20 kN/m’. The weight of the concrete anchor block is kept the same as in the case study, 18 kN/m’.

With the above mentioned data as input the stress in the outer fibres can be calculated. The eccentricity is equal to 0.5 times the total height of the cross section. Since the cross section is symmetrical, the normal centre of the cross section will be located in the middle of the cross section.

Table 21: Stresses and strains in the outer fibres of the skins and webs for design 1 according to the hand calculation

	SLS		ULS	
	Stress [MPa]	Strain [%]	Stress [MPa]	Strain [%]
Skin	-52.3	-0.39	-79.1	-0.59
Web	-45.7	-0.54	-69.4	-0.82

Although the webs are thus far neglected the strain in the webs can be estimated by calculating the stress at 0.5h-t from the normal centre since that is the place where the webs will be connected to the skins. For the calculation of the strain in the outer fibre of the web the Young’s modulus of the web must be used instead of the Young’s modulus of the skin. It can be concluded from Table 21 that the strains in both the SLS and ULS for the skin and web laminate are bigger than the allowed 0.27%. However, in the calculation so far the webs of the cross section are neglected for the calculation of the moment of inertia. In the 2D model in SCIA the webs will no longer be neglected.

SCIA 2D model

In the FEM program SCIA the FRP quay wall has been modelled as a column with at the bottom a fully clamped edge and at the top a free end. The moment and normal force are applied at the top of the column. With this model the moment distribution in the column will be constant. The column is modelled with a height of 1 meter. Figure 18 shows a representation of the 2D model.

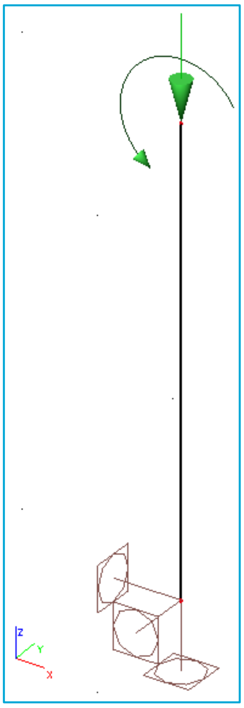


Figure 18: 2D SCIA model

In the cross section modulus of SCIA the cross section has been drawn including the webs. The cross section of two linked Z-profiles has been drawn so that the axis of the cross section coincide with the axis of the program. This has been done to prevent that the modelled moment and normal force will cause a torsional moment or other undesired forces. The mechanical properties of the skins and webs are input for the model. SCIA can calculate the stresses in the cross section which can be transformed to strains by dividing the stress with the respective Young’s modulus.

The cross section modulus of SCIA also calculates the moment of inertia of the cross section. This moment of inertia includes the webs of the cross section. The moment of inertia according to SCIA is 2.53 x 10^{-3} m^4 while the calculated moment of inertia for the cross section in step 5 was 2.04 x 10^{-3} m^4.

This is a difference of 24% between the two values and therefore the D-sheet model has been run again with the newly gained input data. The maximum deflection in the SLS and the maximum moment and anchor force for both the SLS and ULS are presented in Table 22.

Table 22: Relevant data obtained with the D-sheet model for design 1 with SCIA influence

	SLS			ULS	
	U_{max} [mm]	M_{max} [kNm]	F_{anchor} [kN]	M_{max} [kN]	$F_{d,anchor}$ [kN]
Per running meter	257.8	511.3	375.6	757.6	450.72
2 Z-profiles	257.8	513.35	375.6	760.63	450.72

With this data the strains in the outer fibres for the skin and web laminate can be calculated with equation 6.

Table 23: Stresses and strains in the outer fibres of the skins and webs for design 1 according to the 2D SCIA model

	SLS		ULS	
	Stress [MPa]	Strain [%]	Stress [MPa]	Strain [%]
Skin	-52	-0.39	-75.7	-0.69
Web	-46.9	-0.55	-68	-0.98

The strains are slightly increased compared to the strains obtained with the hand calculation. However, the strains are far above the allowed value of 0.27%. The design of the cross section must therefore be altered to fulfil the maximum strain criteria.

By taking a closer look to equation 6 it can be concluded that there is one parameter that influences the outcome of the stresses in the material the most and that is the moment of inertia. The shape of the cross section therefore remains unaltered. The dimensions of the cross section are adjusted to obtain a higher moment of inertia. As stated in the beginning of the design process, the skins contribute the most to the moment of inertia. The thickness of the skins is therefore increased. Also the spacing between the skins is increased. The other variables are kept constant.

Figure 19 shows the cross section of design 1 with the webs in green and skins in red. A 3D image of the quay wall with the dimensions according to design 1 is presented in Figure 20.

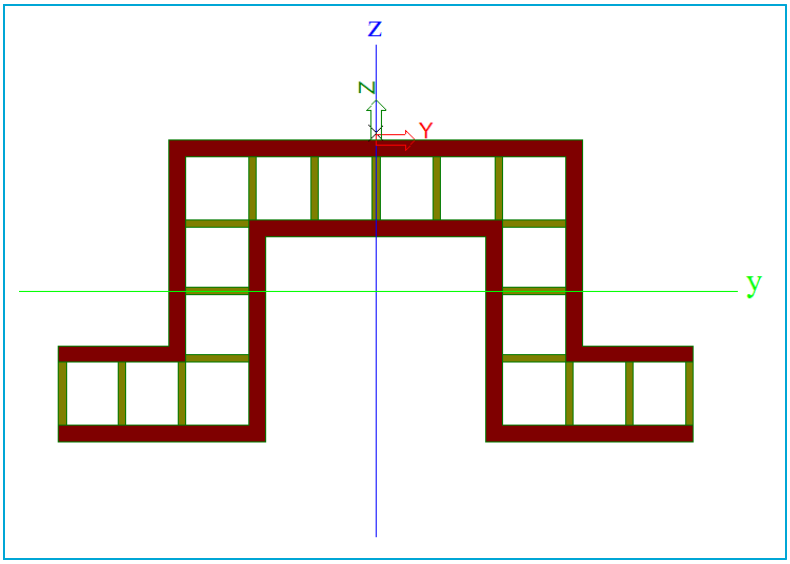


Figure 19: Cross section of design 1

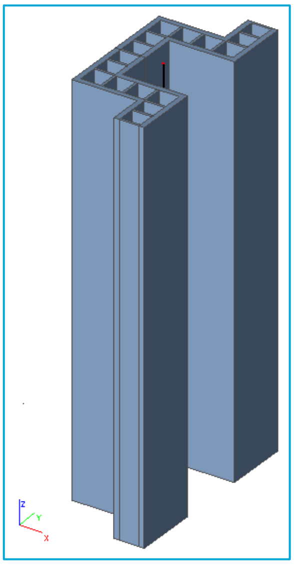


Figure 20: 3D image of design 1

5.5.7.2 Design 2

Step 5, 6 and 7 from the flowchart have been done again for design 2. Design 2 has dimensions as shown in Table 24. With these dimensions the moment of inertia becomes 6.72 x 10^{-3} m^4 for a single Z-profile. The moment of inertia per running meter quay wall becomes 1.03 x 10^{-2} m^4 without taking the webs into consideration. For comparison, this was 2.03 x 10^{-3} m^4 per running meter quay wall with the dimensions of design 1.

The skin laminate now has a thickness of 50 millimetres. The required thickness of the webs is calculated with equation 5. With the moment of inertia belonging to design 2 the deflection of the quay wall is 84 millimetres and the maximum moment is 506.7 kNm/m' according to the D-sheet model. The area of the webs per running meter quay wall is equal to 0.037 m^2. With a height of 0.2 meters this comes to a thickness of 0.185 m per running meter quay wall. With a centre to centre distance of 100 millimetres the thickness of a single web is set at 20 millimetres.

Due to the increase in thickness of the skins and webs the stacking order of the laminates will be different than the stacking order of the laminates corresponding with design 1. The mechanical properties however do not change with the increased thickness because the same distribution per direction is required for the anisotropic and quasi-isotropic laminate.

Table 24: Dimensions of design 2

		Dimensions [mm]
Variables	t	50
	s	200
	hz	175
	b1	175
	b4	175
Elements	h1	50
	h2	525
	h3	50
	h4	50
	h5	525
	h6	50
	b1	175
	b2	50
	b3	425
	b4	175
	b5	50
	b6	425

The stacking order of the skin and web laminate are presented in Appendix F: Design of the quay wall. The mechanical properties of the laminates have not been altered compared to design 1. Only the thickness of the laminate has increased which does not influence the mechanical properties of a laminate.

The same procedure has been followed as for design 1. The moment of inertia of design 2 is firstly calculated for the skins only, the webs are neglected. As already mentioned the moment of inertia is equal to 1.03 x 10^{-2} m^4 per running meter quay wall. The D-sheet model has been used to calculate the corresponding moments, deformations and anchor forces. Table 25 shows this information.

Table 25: Relevant data obtained with the D-sheet model for design 2

	SLS			ULS	
	U_{max} [mm]	M_{max} [kNm]	F_{anchor} [kN]	M_{max} [kN]	$F_{d,anchor}$ [kN]
Per running meter	84.1	507.4	367.7	746.5	441.24
2 Z-profiles	84.1	659.62	367.7	970.45	441.24

The data of the D-sheet model has been used to calculate the stresses and strains in the outer fibres of the skin and web laminate. The stresses and strains have been calculated with equation 6. The outcome of the stress and strain calculation is presented in Table 26.

Table 26: Stresses and strains in the outer fibres of the skins and webs for design 2 according to the hand calculation

	SLS		ULS	
	Stress [MPa]	Strain [%]	Stress [MPa]	Strain [%]
Skin	-15.1	-0.11	-24.6	-0.25
Web	-12.7	-0.15	-21	-0.33

With a 2D model in SCIA the stresses and strains have been checked. The same set up for the 2D model has been used for design 2 as that has been used for design 1. The moment of inertia according to SCIA is equal to 1.47 x 10^{-2} m^4. The deformation, maximum moment and anchor force have recalculated with the D-sheet model for the moment of inertia that has been obtained by SCIA. The outcome of the recalculation is shown in Table 27.

Table 27: Relevant data obtained with the D-sheet model for design 2 with SCIA influence

	SLS			ULS	
	U_{max} [mm]	M_{max} [kNm]	F_{anchor} [kN]	M_{max} [kN]	$F_{d,anchor}$ [kN]
Per running meter	79.2	507	367.4	746.6	440.88
2 Z-profiles	79.2	659.1	367.4	970.58	440.88

The stresses for the SLS and ULS according to the 2D model in SCIA are presented in Table 28. This table contains the corresponding stresses as well.

Table 28: Stresses and strains in the outer fibres of the skins and webs for design 2 according to the 2D SCIA model

	SLS		ULS	
	Stress [MPa]	Strain [%]	Stress [MPa]	Strain [%]
Skin	-18.7	-0.14	-26.9	-0.25
Web	-16.4	-0.19	-15	-0.22

It can be concluded that design 2 does fulfil the strain criteria check. The strain in the outer fibres of the skin and web laminate do not exceed the 0.27% boundary.

Design 2 will be modelled as a 3D plate model with the program SCIA Engineer. This model can calculate N_x, N_y, N_{xy}, M_x, M_y and M_{xy}. These forces are input for the laminate analysis with the program Kolibri. The 3D model and the laminate analysis will be discussed in the following subparagraphs.

The cross section of design 2 is presented in Figure 22 where the skins are pictured with the red colour and the webs with the green colour. Figure 21 shows a 3D impression of design 2.

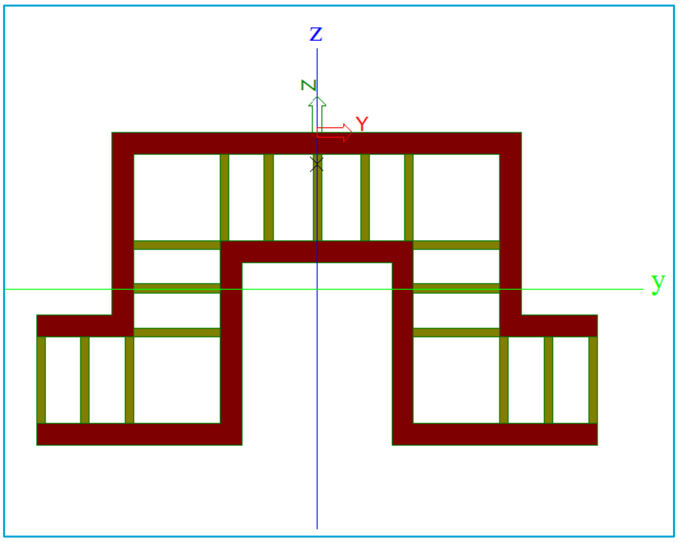


Figure 22: Cross section of design 2

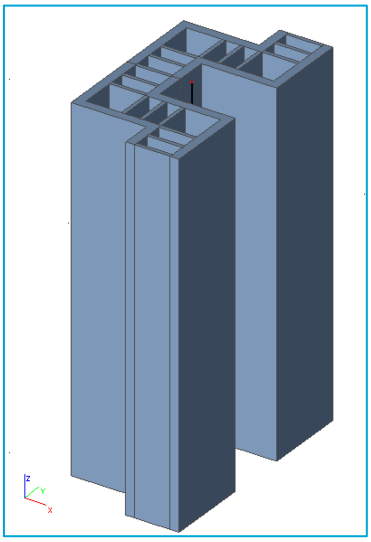


Figure 21: 3D impression of design 2

5.5.7.3 3D model

A 3D model has been made to take into account the orthotropy of the laminates. With SCIA Engineer a 3D plate model has been created. The orthotropic characteristics of the laminates are assigned to the plates which function as the skin and web laminates. This orthotropic character of the laminates is obtained with Kolibri. The stacking order of the laminate and the mechanical properties of the lamellae determine the so called ABD matrix. The ABD matrix for the skin and web laminate are shown in Figure 23 and Figure 24.

How the ABD matrix can be calculated is explained in [Valk, 2016].

Laminate Stiffness Matrix

$$[ABD] = \begin{bmatrix} 1.5174\cdot10^{9} & 2.5594\cdot10^{8} & 0 & 0 & 0 & 0 \\ 2.5594\cdot10^{8} & 8.5020\cdot10^{8} & 0 & 0 & 0 & 0 \\ 0 & 0 & 3.1049\cdot10^{8} & 0 & 0 & 0 \\ 0 & 0 & 0 & 3.3330\cdot10^{5} & 51653 & 1451.4 \\ 0 & 0 & 0 & 51653 & 1.6328\cdot10^{5} & 1451.4 \\ 0 & 0 & 0 & 1451.4 & 1451.4 & 63018 \end{bmatrix} \text{N, m}$$

Figure 23: The ABD matrix belonging to the skin laminate

Laminate Stiffness Matrix

$$[ABD] = \begin{bmatrix} 4.0359\cdot10^{8} & 1.2276\cdot10^{8} & 0 & 0 & 0 & 0 \\ 1.2276\cdot10^{8} & 4.0359\cdot10^{8} & 0 & 0 & 0 & 0 \\ 0 & 0 & 1.4042\cdot10^{8} & 0 & 0 & 0 \\ 0 & 0 & 0 & 16513 & 3980.4 & 491.43 \\ 0 & 0 & 0 & 3980.4 & 10616 & 491.43 \\ 0 & 0 & 0 & 491.43 & 491.43 & 4569 \end{bmatrix} \text{N, m}$$

Figure 24: The ABD matrix belonging to the web laminate

The skins and web laminates are thus modelled as plates with orthotropic characteristics. Design 2 will be checked for the maximum moment and not for the entire moment distribution as calculated with the D-sheet model. If the design is strong enough to resist the maximum occurring moment it will obviously be strong enough to resist the lower moments.

Not only a moment will act on the quay wall, there is also a normal force that works on the structure. This normal force can be calculated with equation 7 and 8. The normal force for the SLS is equal to 596.5 kN and in the ULS the normal force is equal to 720.7 kN. The value of the moment in the SLS is 507 kNm per running meter quay wall according to the D-sheet model and for the ULS the moment is equal to 746.6 kNm, see Table 27. These values will have to be converted to values that are in correspondence with the width of 2 Z-profiles and not a running meter quay wall. The values will therefore have to be multiplied with the width of 2 Z-profiles which is equal to 1.3 meter. The moment in the SLS than becomes 659.1 kNm and in the ULS the moment is equal to 970.58 kNm.

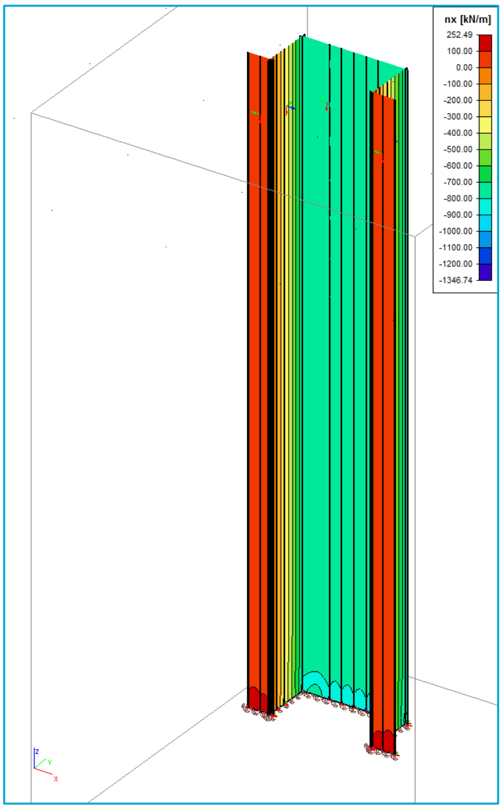


Figure 25: N_x distribution in the SLS

The normal force and moment must be applied at the normal force centre of the cross section. The normal force centre does not lay within a part of the cross section but in a hollow space between the 2 Z-profiles. To be able to apply the normal force and moment at the normal force centre a vertical plate has been modelled on top of the quay wall section. This plate has been modelled as a plate with infinite stiffness. Therefore, the plate itself will not deform. The normal force and moment are applied in the middle of the plate, The plate transfers the forces and moments to the section of the quay wall. The section of the quay wall is modelled as a cantilever beam. One end is fully clamped while the other end is free. The same reasoning is valid for this model as for the 2D model. The quay wall section has been modelled with such a height that the effects of the vertical plate and the supports are faded.

With the 3D model the forces and moments in the x, y and xy direction can be calculated. As output the model gives images as shown in Figure 25 and Figure 26. With these images in SCIA, the values for the forces and moments are determined at the middle of the height of the section so that the effects of the vertical top plate and supports are faded.

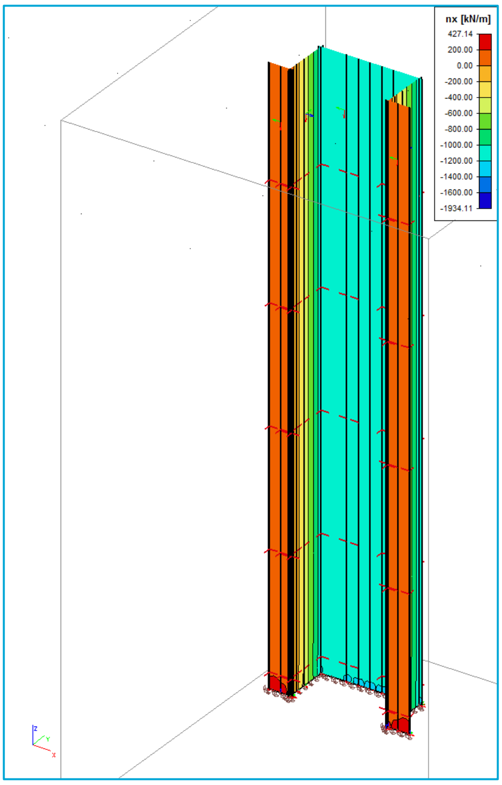


Figure 26: N_x distribution in the ULS

The verification of the model and the output of the model by means of images such as presented in Figure 25 and Figure 26 can be found in Appendix F: Design of the FRP quay wall.

The quay wall section has been divided in three parts to be able to distinguish the forces and moments working on the different orthotropic plates. The quay wall is divided in the two skins and the webs. In the 3D model these parts have been identified within the model as a separate layer so that the output can be generated for each part individually. The forces and moments that are required as input for the laminate analysis with the program Kolibri are shown in Table 29 to Table 34. The forces and moments obtained with the SCIA model do not yet include the material and conversion safety factors. The obtained forces and moments are therefore multiplied with the relevant factors resulting in the forces and moments as shown in the tables. Which factors must be applied are elaborated into detail in Appendix E: CUR96. The forces and moments for the ULS are relevant for the strength of the laminate. The stability of the laminate will be discussed in the following subparagraph.

Table 29: SLS, the outer skin

N_x	-1699,97	kN/m
N_y	-0,08607	kN/m
N_{xy}	0,06455	kN/m
M_x	0,96833	kNm/m
M_y	0,02151	kNm/m
M_{xy}	0	kNm/m

Table 31: SLS, the webs

N_x	-432,525	kN/m
N_y	0	kN/m
N_{xy}	-0,17215	kN/m
M_x	0	kNm/m
M_y	0	kNm/m
M_{xy}	0	kNm/m

Table 33: SLS, the inner skin

N_x	1235,17	kN/m
N_y	-0,08607	kN/m
N_{xy}	-0,43037	kN/m
M_x	0,96839	kNm/m
M_y	0	kNm/m
M_{xy}	0	kNm/m

Table 30: ULS, the outer skin

N_x	-2997,45	kN/m
N_y	-0,13205	kN/m
N_{xy}	0,10564	kN/m
M_x	1,76942	kNm/m
M_y	0,05282	kNm/m
M_{xy}	0	kNm/m

Table 32: ULS, the webs

N_x	-765,868	kN/m
N_y	0	kN/m
N_{xy}	-0,26409	kN/m
M_x	0	kNm/m
M_y	0	kNm/m
M_{xy}	0	kNm/m

Table 34: ULS, the inner skin

N_x	2302,87	kN/m
N_y	-0,15846	kN/m
N_{xy}	-0,71305	kN/m
M_x	1,71660	kNm/m
M_y	0	kNm/m
M_{xy}	0	kNm/m

The outer skin refers to the skin that consists of part 4 to part 6 as shown in Figure 14. The inner skins consists of parts 1 to part 3 as shown in the same figure.

It is stated in [CUR96, 2003] that when a laminate is loaded with a normal force and a moment that at least the outer and middle lamellae must be checked individually for the strain criteria of, in this case, 0.27%. The output of the laminate analysis with Kolibri shows the strains and stresses for each lamella individually. The strains of the outer and middle lamellae are shown in Table 35 to Table 40. In these tables the abbreviation 't' stands for top of the lamella and 'b' for bottom of the lamella. From these tables it can be concluded that the strains in the skin and web laminates do not exceed the 0.27% boundary in both SLS and ULS.

According to [CUR96, 2003] the shear strain should not be bigger than 1.6%. As can be concluded from Table 35 to Table 40 the occurring shear is not even near the boundary of 1.6%.

The strains obtained with SCIA and Kolibri are almost the same as the strains obtained with the hand calculations and the 2D SCIA model. It can therefore be stated that the 3D model has been modelled correctly.

The laminates have now been checked for the strength criteria's in the SLS and ULS. Other checks will have to be performed as well. These checks will be discussed in the following subparagraph.

Table 35: Strains in the SLS, outer skin

layer		51	26	1
ε_1	t	-0,11%	-0,12%	-0,13%
	b	-0,11%	-0,12%	-0,13%
ε_2	t	0,03%	0,04%	0,04%
	b	0,03%	0,04%	0,04%
γ_{12}	t	-1,06E-06	1,83E-07	1,43E-06
	b	-1,01E-06	2,33E-07	1,48E-06

Table 36: Strains in the SLS, webs

layer		16	8	1
ε_1	t	-0,12%	0,04%	-0,12%
	b	-0,12%	0,04%	-0,12%
ε_2	t	0,04%	-0,12%	0,04%
	b	0,04%	-0,12%	0,04%
γ_{12}	t	-1,23E-06	-1,23E-06	-1,23E-06
	b	-1,23E-06	-1,23E-06	-1,23E-06

Table 37: Strains in the SLS, inner skin

layer		51	26	1
ε_1	t	0,09%	0,09%	0,08%
	b	0,09%	0,09%	0,08%
ε_2	t	-0,03%	-0,03%	-0,02%
	b	-0,03%	-0,03%	-0,02%
γ_{12}	t	-2,59E-06	-1,41E-06	-2,31E-07
	b	-2,54E-06	-1,36E-06	-1,83E-07

Table 38: Strain in the ULS, outer skin

layer		51	26	1
ε_1	t	-0,19%	-0,21%	-0,22%
	b	-0,20%	-0,21%	-0,22%
ε_2	t	0,06%	0,06%	0,07%
	b	0,06%	0,06%	0,07%
γ_{12}	t	-2,02E-06	2,93E-07	2,61E-06
	b	-1,93E-06	3,88E-07	2,70E-06

Table 39: Strains in the ULS, webs

layer		16	8	1
ε_1	t	-0,21%	0,06%	-0,21%
	b	-0,21%	0,06%	-0,21%
ε_2	t	0,06%	-0,21%	0,06%
	b	0,06%	-0,21%	0,06%
γ_{12}	t	-1,88E-06	-1,88E-06	-1,88E-06
	b	-1,88E-06	-1,88E-06	-1,88E-06

Table 40: Strains in the ULS, inner skin

layer		51	26	1
ε_1	t	0,17%	0,16%	0,15%
	b	0,17%	0,16%	0,15%
ε_2	t	-0,05%	-0,05%	-0,04%
	b	-0,05%	-0,05%	-0,04%
γ_{12}	t	-4,43E-06	-2,34E-06	-2,49E-07
	b	-4,34E-06	-2,25E-06	-1,64E-07

5.5.8 Laminate analysis

The strength of the laminate has been checked in the previous subparagraph. Besides the strength of the laminate the stability, interlaminar shear strength and wrinkling of the skin should be checked as well. The shear stress between the skin and web laminate will be checked before the laminates will be checked for the exceptional loading case with an impact load.

5.5.8.1 Stability

The stability of the laminate is related to the buckling load. The buckling load has been determined with the theoretical buckling load according to Euler. The critical load is calculated with the following formula;

$$F_b = \frac{\pi^2 \cdot EI}{{L_b}^2} \qquad \textbf{Eq. 9}$$

In which L_b is the buckling length of the quay wall. This buckling length is estimated at 11.75 meters. This is the length between the two zero points in the moment distribution of the quay wall in the ULS. See Appendix F: Design of the FRP quay wall for the moment distribution of the quay wall obtained with D-sheet. The critical buckling load is then equal to 10825 kN. The largest normal force is equal to 3297.19 kN and is therefore smaller than the critical buckling load. Buckling of the skins will therefore not occur.

The buckling check has been performed for the webs as well. The webs have a height of 0.2 meters. This is the buckling length that will be taken into account. The axial loading is caused by the shear force. The shear force has a maximum value of 292.9 kN per running meter quay wall according to the D-sheet calculation. There are 17 webs per 1.3 meter of quay wall. The maximum shear force for a single web then becomes 40.2[3] kN. With equation 9 the critical buckling load can be calculated. The stiffness of a single web can be calculated with the standard formula for a rectangular shape. The Young's modulus of the web is equal to 1.83 x 10^7 kN/m^2. The moment of inertia is calculated with $^1/_{12}bh^3$ and is equal to 1.33 x 10^{-5} m^4. The critical buckling load for a single web is 60246 kN. It can therefore be concluded that buckling of a single web will not occur.

5.5.8.2 Interlaminar shear strength

The representative value of the interlaminar shear strength is given in [CUR96, 2003] for the three most applied resins. The representative interlaminar shear strength of polyester is 20 MPa. The interlaminar shear strength can be checked with Kolibri. Kolibri shows that the interlaminar shear strength has a maximum value of 12.8 MPa and delamination of the lamellae will therefore not occur.

5.5.8.3 Wrinkling

Skin wrinkling is a failure mechanism that must be checked for a sandwich type structure. Skin wrinkling for a standard sandwich construction, which only consists of two skins and foam in between, is shown in Figure 27.

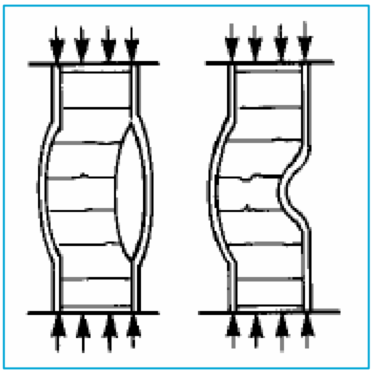


Figure 27: Skin wrinkling [Kolstein, 2008]

Skin failure occurs between the skins of a sandwich and the core and is a form of instability. So far the foam has been neglected for the design since the foam does not have a significant influence on the design and the design checks performed thus far. As core material a rigid PMI foam, type 71S is chosen. The properties of the foam are of importance for the skin wrinkling check and are therefore given in Table 41.

3 The material and conversion safety factor equal to 1.7932 for stability is included in this value.

Table 41: Properties of PMI foam core type 71S [Clarke, 2005]

Density [kg/m^3]	E_c [MPa]	σ_c [MPa]	Compression [%]	E_t [MPa]	σ_t [MPa]	Elongation [%]	τ [MPa]	G [MPa]
75	73	1.53	8	157	1.97	4.1	2.28	34

To check if skin failure will happen the maximum allowed skin wrinkling stress will have to be determined. According to [Kolstein, 2008] the skin wrinkling stress is based on buckling of a plate on an elastic foundation, the skin wrinkling stress σ'_s is given as;

$$\sigma'_s = \frac{1}{2}(E_s E_c G_c)^{\frac{1}{3}} \qquad \text{Eq. 10}$$

In which;
- E_s = Young's modulus of the skin [MPa]
- E_c = Young's modulus of the core [MPa]
- G_c = Shear modulus of the core [MPa]

The skin wrinkling stress is equal to 207.5 MPa. The maximum compression stress on the skin laminate is equal to 65.9 MPa and skin wrinkling will therefore not occur.

Since the cross section of the FRP quay wall is not a standard sandwich construction it is not likely that skin wrinkling will cause failure. The webs will function as a rigid support for the skin laminates. The skin might wrinkle in the section between two consecutive elements but as shown the maximum compression in the skin laminate is smaller than the skin wrinkling stress.

5.5.8.4 Shear stress

The allowable shear stress is determined by multiplying the shear modulus with the allowable shear strain. According to [CUR96, 2003] the shear strain is equal to 1.6%. The allowable shear stress for the skin laminate is therefore equal to 99 MPa and the allowable shear stress for the web is equal to 112 MPa. The highest shear stress is calculated with the following formula;

$$\tau = \frac{V \cdot S_a}{A \cdot I} \qquad \text{Eq. 11}$$

In which;
- V = The shear force [kN]
- S_a = The statical moment of the sheared part [m^3]
- A = The area of the sheared part [m^2]
- I = The moment of inertia of the structure [m^4]

As a first approximation the shear force obtained with D-sheet is transformed to a shear force per web so that a so called I-beam can be considered for the shear stress calculation.

The shear force per web is equal to 22.4 kN. The statical moment has a value of 6.25 x 10^{-4} m^3. The area of the sheared part is equal to 0.005 m^2 and the moment of inertia is equal to 1.717 x 10^{-4} m^4. The shear stress can be calculated with this data and is equal to 16.3 MPa. The allowable shear stress between the skin and web laminate is 99 MPa. However, the material and conversion safety factor have not yet been applied within this calculation. The allowable shear stress with taken into consideration the material and conversion safety factor is equal to 55 MPa. The occurring shear stress is lower than the allowable shear stress and shear failure will therefore not occur.

5.5.8.5 *Impact load*

The impact load is caused by a collision with a vessel. According to the program of requirements this impact load will be schematised as a point load of 60 kN/m', see Appendix C: Transformation of the external loads. In a worst case scenario the collision will take place while there is no surface load. The presence of a surface load will have a positive influence on the moment distribution of the quay wall. This scenario has been implemented in the D-sheet model as well as the 3D SCIA model. All the checks for the SLS and ULS have been performed in the same manner as for the normal loading combinations.

The deformation, moment distribution and anchor force have been calculated with the D-sheet model for load combination 3 as mentioned in Table 42.

Table 42: Relevant data obtained with the D-sheet model for design 2 for load combination 3

	SLS			ULS	
	U_{max} [mm]	M_{max} [kNm]	F_{anchor} [kN]	M_{max} [kN]	$F_{d,anchor}$ [kN]
Per running meter	77.8	662.7	211.7	881.2	254.08
2 Z-profiles	77.8	861.51	211.7	1145.56	254.08

From the table it can be concluded that the anchor forces are lower when compared to load combination 4. However, the moments in both the SLS and ULS are increased. The same checks have been performed for load combination 3 as has been done for load combination 4. From these checks it can be concluded that the FRP quay wall can withstand an impact load as schematized with load combination 3.

5.6 Quay wall checks

The quay wall now has been checked for relevant FRP failure mechanisms. There are also failure mechanisms related to quay walls in general. The quay wall design has been checked for the anchor stability, overall stability and bearing capacity. The deadweight of the quay wall will also be checked to check whether the uplift force can be withstand.

5.6.1 *Anchor capacity*

The anchor capacity of the anchored quay wall has been checked with the Kranz method. This check has been included in Appendix G: Anchor stability. The design of the anchor has not been altered with respect to the case study. The anchor capacity seems to be over-dimensioned but the design of the anchor is not part of this feasibility study.

5.6.2 *Overall stability*

The overall stability of the quay wall has been checked with the Bishop method. The overall stability has been checked for every construction phase of the FRP quay wall. The overall stability has been elaborated into detail in Appendix H: Overall stability. The lowest stability factor has been obtained for load combination 4 and is equal to 1.93. To prevent failure of the quay wall the stability factor should be at least equal to 1. It can therefore be concluded that the design of the quay wall is safe with regard to the overall stability.

5.6.3 *Bearing capacity*

The bearing capacity of the quay wall has been calculated according to NEN 6740. The details of the bearing capacity determination are included in Appendix I: Vertical stability. The bearing capacity is determined by the resistance of the quay wall and the vertical forces on the quay wall. The resistance consists of shaft friction and pile point resistance.

The pile point resistance is 615 kN and the sum of the vertical forces acting on the quay wall is equal to 612 kN when the negative adhesion is taken into account. Whether the negative adhesion should be taken into account or not depends on the applied installation technique. When the FRP quay wall is driven into the soil the negative adhesion will have to be taken into account. If it turns out that the FRP quay wall cannot be driven into the ground a soil removing installation technique must be applied. The negative adhesion does not occur when a soil removing installation technique will be applied.

5.6.4 *Deadweight*

The groundwater will result in an uplifting force on the quay wall. This uplifting force is equal to the displaced volume times the volumetric weight of water, which is equal to 10 kN/m^3. The total displaced volume of the quay wall is equal to 0.645 m^3 for two Z-profiles and with a height of 1 meter. The foam that will function as core between the skins has a neglectable volumetric weight and will therefore not be taken into account.

The skin laminates have a combined volume of 0.225 m^3. With the volumetric weight equal to 19.535 kN/m^3 this results in a downward force of 4.40 kN. The webs have a combined volume of 0.068 m^3 and their volumetric weight is equal to 18.85 kN/m^3. This results in a downward force equal to 1.28 kN. The uplift force is equal to 6.45 kN. This results in an effective upwards force of 0.77 kN.

The deadweight of the quay wall itself is therefore not sufficient to prevent the structure from floating. Before the construction of the superstructure and the placement of the anchors there will not be an additional vertical force acting downwards to prevent the structure from floating. It is therefore necessary to increase the deadweight of the structure This can be achieved by applying a foam with a higher density.

There is a volume of 0.352 m^3 foam present in the cross section that consists of two Z-profiles. To create an extra margin it is assumed that the foam will have to have deadweight equal to 1 kN so that the balance will turn from 0.77 kN upwards to 0.23 kN downwards. The required volumetric weight of the foam should therefore be 2.84 kN/m^3.

Another alternative is to adjust the composition of the skin and web laminate so that the volumetric weight of the laminates will increase. When the costs of the laminates are taken into consideration it seems not feasible to adjust the composition of the laminates since an increase in volumetric weight can only be achieved by increasing the fibre volume fraction or increase the thickness of the laminates.

5.7 Summary of the design

An FRP quay wall has been designed with a profile similar to the Z-profiles of steel sheet piles. An impression of the cross section of the quay wall is presented in Figure 28. The skin laminates are shown in red while the web laminates are shown in green.

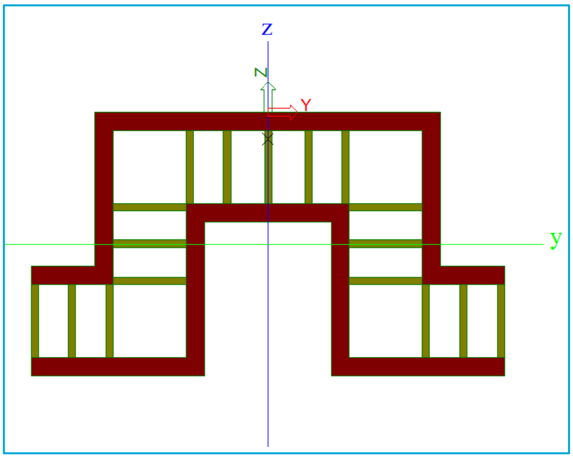


Figure 28: Cross section of design 2

The skin laminate is designed as an anisotropic laminate with a fibre volume percentage equal to 55%. 55% of the fibres are placed in the vertical direction of the quay wall, the so called 0°-direction. 15% of the fibres are located in the -45°-, 45°- and 90°-direction. The skin laminate has a thickness of 50 millimetres

The web laminate is designed as a quasi-isotropic laminate which means that 25% of the fibres is located in each of the 4 main directions. The web laminate are designed with a fibre volume percentage equal to 50%. Each web laminate has a thickness of 20 millimetres and a height of 200 millimetres.

The cross section as shown in Figure 28 has a width of 1300 millimetres and a height of 725 millimetres.

The deflection of the FRP quay wall is equal to 79.2 millimetres and the maximum occurring strain is 0.25% and is obtained with the exceptional loading combination of an impact load. The strain is thus not larger than the maximum allowable strain of 0.27%.

The skin and web laminates have been checked for the following failure mechanisms: buckling, interlaminar shear stress, wrinkling and shear stress between the laminates. From these checks it can be concluded that neither the skin nor the web laminate is sensitive to these failure mechanisms.

Common checks related to quay walls have been performed as well. The anchor capacity, overall stability, bearing capacity and deadweight of the quay wall have been checked.

6. Joints

6.1 Chapter content

The Z-profiles as designed in the previous chapter will have to be connected to one another to provide the watertight and soil retaining function. The skin and web laminate will have to be attached to one another as well. The elements of the FRP quay wall will be made with the vacuum injection method. The types of joints will be addressed briefly before a joint will be designed for the connection between the skin and web laminate. The first two paragraphs of this chapter are related to the connection between the skin and web laminate. The connection between two Z-profiles will be discussed before this chapter is concluded.

6.2 Types of joints

There are 3 types of joints that will be considered namely a mechanical joint, a bonded joint or a combined joint. With the combined joint a mechanical as well as a bonded joint will be applied. Each type of joint has their own characteristic. The most important characteristics concerning a quay wall are shown in Table 43.

Table 43: Characteristics of the three type of joints

Characteristic	Mechanical joint	Bonded joint	Combined joint
Stress concentration at joint	high	medium	medium
Water tightness	no	yes	yes
Smooth joints	bad	good	bad
Sensitive to peel loading	no	yes	no
Inspection	easy	difficult	difficult
Tooling costs	low	high	low

A mechanical joint is unwanted since this type of joint does not provide water tightness. The combined joint combines adhesive bonding and mechanical fastening. This combination will not improve the joint strength compared to that of a well-designed bonded joint. Since a structural adhesive provides a much stiffer load path than fasteners, the adhesive carries the load almost entirely and no load sharing between the adhesive and the bolts occurs [Kolstein, 2008]. A bonded joint will therefore be applied between the skin and web laminate.

6.3 Bonded joint

The skin and web laminate will be combined with a bonded T-joint. Figure 29 shows a representation of a T-joint. [Clarke, 2005] a T-joint must be verified with testing and primary structural connections should be calculated with a FEM. In this paragraph a simplified method will be used to estimate the dimensions of the joint.

The potential failure modes of a bonded T-joint are;

- Peeling of the ends of the bonding angle from the adherent members,
- Failure in the adhesive layer due to shear,
- Failure in the adherent members.

The thickness of the bonding angle is estimated at half of the thickness of the thicker of the two adherents. The skin laminate has a thickness of 50 millimetre. The thickness of the bonding angle will therefore be estimated at 25 millimetre.

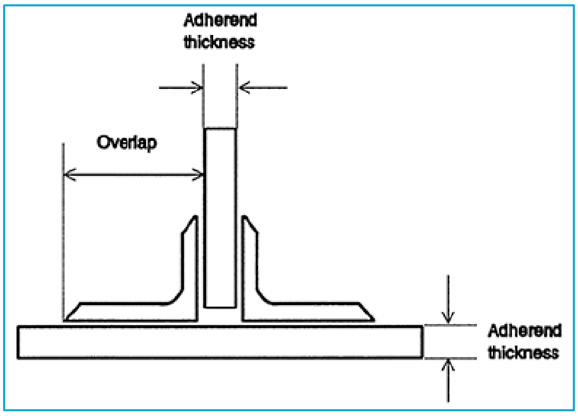


Figure 29: Representation of a T-joint

The length of the overlap should be at least 10 times the thickness of the bonding angle. The overlap length is therefore estimated at 250 millimetre.

The mechanical properties of the adhesive that will be applied are presented in Table 44. The design values of the adhesive have been calculated with the material factor as defined in [Clarke, 2004]. The material factor consists of 4 individual factors that have to be multiplied with one another to obtain the material factor. The first contributor to the material factor is related to the source of the adhesive properties and is equal to 1.25. The second contributor is depends on the application method of the adhesive. This contributor is equal to 1.25. The third contributor depends on whether the loading is long-term or short-term loading. The value for this contributor is equal to 1.5. The last contributor to the material factor depends on the environmental conditions and is equal to 1. The material factor is therefore equal to 2.34.

Table 44: Mechanical properties of the adhesive [Clarke, 2004]

Adhesive	Description	τ_{ult} [MPa]	τ_d [MPa]	σ_{ult} [MPa]	σ_d [MPa]
Araldit AV 138	Epoxy 2-part paste	18.4	7.85	17.7	7.42

The forces that occur in the joint are given in Table 45.

Table 45: Forces occurring in the joint

N_x	-2997,45	kN/m
N_y	-0,13205	kN/m
N_{xy}	0,105637	kN/m
M_x	1,769419	kNm/m
M_y	0,052818	kNm/m
M_{xy}	0	kNm/m

The minimum required overlap length can be calculated by assuming that the normal force will be transferred in the joint by shear stress. The minimum overlap length is then equal to:

$$l_{overlap,min} = \frac{N_x}{2 \cdot \tau_d} = \frac{2997.45}{2 \cdot 7.85} = 191 \text{ mm} \qquad \textbf{Eq. 12}$$

The overlap length is 250 millimetre as based on the rule of thumb as mentioned before. The stress in the x-direction is then equal to:

$$\sigma_x = \frac{N_x}{2 \cdot l_{overlap,min}} = 6 \text{ MPa}$$

The normal force in the y-direction will give a shear force in the skin joint of the bond and a tension force in the web joint. The stress in the y-direction will be:

$$\sigma_y = \frac{N_y}{l_{overlap,min}} = 5.28 \cdot 10^{-4} \text{ MPa}$$

The shear stress in the bond will be equal to:

$$\tau_{xy} = \frac{N_{xy}}{2 \cdot l_{overlap,min}} = 2.1 \cdot 10^{-4} \text{ MPa}$$

The force in the bond due to M_x should be rewritten to a force. A moment can be rewritten to forces by creating a couple of two forces which will have a certain arm to the concerned point. The value of the moment is per meter height. It is assumed that the acting point of the forces is located at a quarter of the height with respect to the bottom and top of the considered section of the quay wall. The distance between the two forces that work as a couple on the considered section is then equal to 0.5 meter. The stress in the x-direction due to the moment is then equal to:

$$\sigma_x = \frac{M_x/a}{2 \cdot l_{overlap,min}} = 7.07 \cdot 10^{-3}\ \mathrm{MPa}$$

All these stresses are much lower than the design value of the stress as mentioned in Table 44. According to this simplistic check there will be no failure due to the shear stress in the adhesive layer. However, as already stated, the design of the joint should be checked with testing and a FEM calculation.

6.4 Connection between two Z-profiles

The connection between two Z-profiles will be discussed qualitatively. It is recommended that the design of the connection will be done by a FEM calculation and verification by means of testing.

From [Bergschenhoek, 2016] it can be concluded that they use a standard shape for the connection. This type of connection is also applied with steel sheet piling. A representation of the connection is shown in Figure 30.

This type of connection prevents movement of the profile in the lateral direction as well as the perpendicular direction. Since the FRP structure is made with a mould the connections could be incorporated in the mould.

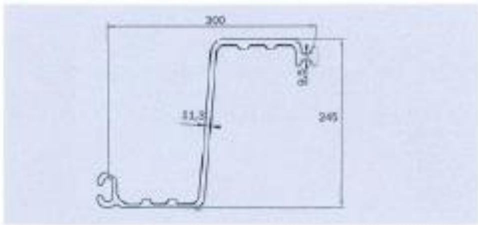


Figure 30: Example of a connection profile [Bergschenhoek, 2016]

The governing forces for the connections will occur during the installation of the profiles. Although it is assumed that the profile will be installed with the diaphragm wall technique, see chapter 7 Installation techniques, the entire FRP quay wall will not be installed as a continuous wall. Quay wall elements will have to be interlocked into elements that are already installed causing forces on the elements and especially the connections of the elements.

A result of the design with the FEM and verification with testing might be that the shape of the connection as shown in Figure 30 does not suffice the requirements. Other forms for the connection will have to be studied then to find the most feasible solution. An alternative form for the connection might be in the shape of an anchor. Other alternatives are shown in Figure 31.

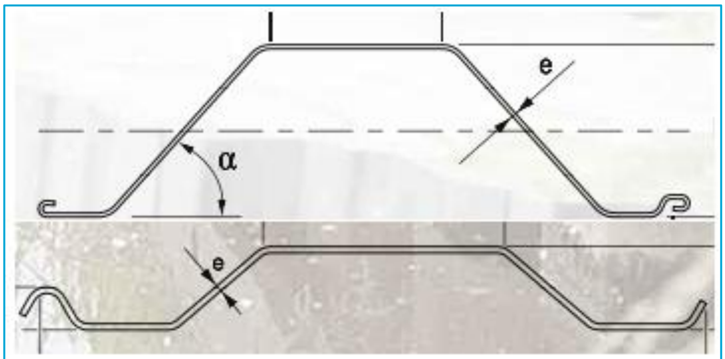


Figure 31: Alternative shapes for the connection [ArcelorMittal, 2008]

The connection between the two Z-profiles is a location in the quay wall where sand particles might be able to be washed out. This should be avoided. It is on the other hand favourable if the water pressure in the pores, induced by the fluctuations in the water level, could adjust itself to the fluctuations, The prevention of washing out of sand particles and adjustable water pressure in the pores could be obtained by applying a filter at the connection between the profiles.

7. Installation techniques

7.1 Chapter content

The FRP quay wall has now been designed. Another important aspect of this feasibility study is the installation technique of an FRP quay wall. Different installation techniques will be addressed in this chapter. For each installation technique a reasoning will be presented whether the technique can be applied for an FRP quay wall or not. Whether a technique can be applied or not depends on several factors. These factors will be addressed first before the installation techniques will be elaborated into more detail.

7.2 Factors

The factors that influence the applicability of a certain installation technique are the following;

- The installation depth of the quay wall,
- The retaining height of the quay wall,
- Soil conditions,
- Obstructions,
- The surroundings at the construction site.

These factors will be discussed for every possible installation technique. The possible installation techniques are driving of the quay wall elements, installing the elements by means of vibrations, pushing of the elements into the soil or the diaphragm wall technique. Under certain conditions, driving, vibrating and pressing of the elements can only be achieved with the help of jetting. Jetting will be discussed as well. The techniques driving, vibrating and pressing are soil displacements techniques while the diaphragm wall is a soil removing technique

7.3 Driving

With the driving of quay walls a very high, pulse-like load is repeatedly applied on the top of the quay wall. The repeatedly applied load is caused by a hammer that is attached to a crane. The driving installation technique causes noise disturbance for the surrounding as well as vibrations. There are different types of hammers that can be applied. The possible hammers are diesel hammers and drop hammers.

7.3.1 Diesel hammer

The diesel hammer principally consists of a cylinder, a piston and an impact block at the bottom of the cylinder. To start the hammer, the piston is lifted to a pre-set height and automatically released. The falling piston compresses the air in the compression chamber and activates the fuel pump to spray fuel on top of the impact block. The impact of the piston on the impact block atomizes the diesel fuel, which ignites in the highly compressed air. This explosive energy throws the piston upwards thus driving the pile downward and restarting the hammer cycle. [ArcelorMittal, 2004]

Diesel hammers perform well in cohesive or very dense soil layers. Driving caps are necessary to protect the top of the quay wall. Often wood is applied as a driving cap.

7.3.2 Drop hammer

There are three main types of drop hammer namely the cable operated drop hammer, the steam drop hammers and hydraulic drop hammers. The cable operated drop hammer consists of a machine lifted weight which is then released into a free fall to drive the quay wall. With the steam drop hammer the cylinder represents the falling weight which is lifted by steam pressure. The steam pressure is controlled by a valve and by lowering the steam pressure the cylinder will fall to the top of the quay wall. A hydraulic drop hammer consists of a segmental ram which is guided by two external supports. The ram is lifted by hydraulic pressure to a certain height and allowed to free fall on to the driving cap. With the hydraulic hammer it is also a possibility to add a downward acceleration to the ram so that the impact energy will be higher than that of a free fall. [ArcelorMittal, 2004]

Drop hammers can be used for all soil conditions as well as above and below the water level. Driving caps have to be applied for drop hammers as well.

7.4 Vibrating

Vibratory pile drivers apply vibrations to the piles to enable them to penetrate the soil. The principle of vibrating is to reduce the friction between the quay wall and the soil. The vibrations will temporarily disturb the soil around the quay wall causing minor liquefaction of the soil. The liquefaction of the soil results in a noticeable decrease in resistance between the quay wall and soil. This enables the quay wall element to be driven into the ground with little added load. [ArcelorMittal, 2004]

The vibrating technique is suited for sand, gravel and soft soils. Soils with firm consistency are much less suited. It is also found that dry soils give greater penetration resistance than soils that are moist, submerged or even fully saturated. A risk of the vibration method is that the soil might be further compacted than the original situation which will lead to an increase of the penetration resistance.

Vibrating has the following advantages compared to driving [Tol, 2006];

- The speed with which an element can be installed. For a suitable combination of soil and vibrator this is mostly just a matter of minutes,
- The liquification of the soil reduces the resistance of the soil resulting in smaller forces on the quay wall. Due to the smaller forces the likelihood of deformation or the risk of successive sections not being interlocked will be low,
- When there is doubt the element can be pulled back, inspected and reinstalled,
- The working speed and the liquification of the soil limits the hindrance for the surroundings.

The main disadvantage of this method is that an element cannot be vibrated deeper into the ground when there is no more penetration. If the element is not installed at the required depth at that time the element can be driven until the required depth is reached.

7.5 Pressing

With the pressing technique, as the name suggest, the elements will be pressed into the soil. With this technique there will be no noise or vibrating hindrance for the surroundings. Pressing is especially suited for the use in cohesive soils. This method requires more time than driving or vibrating.

The machines are hydraulically operated and take most of their reaction force from the friction of the previously driven elements. The hydraulic cylinders are connected to the piles and by pressurizing two hydraulic cylinders, whilst the others are locked, enables the elements to be pushed into the ground, two at a time, to the full extent of the hydraulic cylinders. When all the hydraulic cylinders are extended they are retracted simultaneously causing the crosshead and power pack to be lowered and the cycle is then repeated to completion. [ArcelorMittal, 2004]

When the top layers of the subsoil are weak to very weak this method is not applicable. A very weak type of soil is for example peat. The soil on either side of the top of the already placed element then results in a to low counter pressure to prevent a possible deflection perpendicular to the plane of the element. The already installed element will therefore tend to leave the straight line. This scenario will be the case in the final phase of the insertion when large penetration resistance must be overcome. [Tol, 2004]

7.6 Jetting

Under certain conditions the above mentioned installation techniques alone are not enough. To be able to install the elements at the required depth jetting might be needed. This also prevents overloading of the installation machine, damage to the elements and reduces the vibrations of the soil so less hindrance for the surroundings. The objective of this assistance method is to locate a pressurized water jet at the toe of the element connected by a pipe to a supply pump on the ground surface.

The water pressure loosens the soil and removes loose material. The toe resistance of the element is reduced and the rising water reduces surface and interlock friction. Whether or not jetting will be effective depends on the density of the soil, the available water pressure and the number of jetting pipes.

There are different types of jetting [ArcelorMittal, 2004];

- Low pressure jetting,
- High pressure jetting.

Low pressure jetting is mainly used in dense non-cohesive soil. The pressure produced by the pumps varies from 7 to 20 bar. The large volume of water may cause settlements and a reduction in soil characteristics can be expected in the short term. Low pressure jetting may also be used for ground pre-treatment prior to driving of the elements. When low pressure jetting is combined with the vibrating technique very dense soils can be penetrated.

High pressure jetting can be applied for extremely dense soil layers. When the surroundings are sensitive for settlements, when there are for example monumental buildings located next to the construction site, high pressure jetting is preferred to low pressure due to the reduced amount of water being used. The pump pressure for high pressure jetting varies between 250 to 500 bar.

7.7 Diaphragm wall

The diaphragm wall technique is a soil removing technique. This technique is often applied for the construction of tunnel walls. The new train tunnel in Delft has been constructed with the diaphragm wall technique. This technique consists of 3 steps.

Step 1 is the construction of the guide wall. The guide walls are constructed in-situ as lightly reinforced concrete elements. The guide walls are used for the alignment as well as for the support of the upper soil layer.

Step 2 is the excavation of the soil. While the soil is excavated, the trench will be filled with bentonite. Bentonite is a suspension that prevents the collapse of the trench.

Step 3 is the step where the elements are placed in the excavated trench.

This technique is normally applied for reinforced concrete walls. The volumetric weight of concrete is higher than that of the bentonite suspension. The volumetric weight of bentonite is approximately 2150 kg/m^3. When the reinforcement is then lowered into the trench and the concrete will be poured the bentonite suspension will be pushed towards the surface. The bentonite is extracted from the trench and cleaned so that it can be reduced.

7.8 Driving analysis

With the program Allwave-PDP from Allnamics a driving analysis has been performed to research whether the FRP quay wall can be driven into the soil. The CPT file of the case study is required as input for the program. Other input for the program are the Young's modulus of the quay wall, the cross section, the volumetric weight and the circumference. A hammer must be selected as well. A diesel hammer has been selected as a first estimation.

The output of the program is very comprehensive. The blow count, stress, static driving resistance and number of blows are just a few examples. After several runs of the program the output has not been reliable. In many runs the quay wall would reach the installation depth with just one blow of the diesel hammer.

The driving analysis has therefore also been done with a different CPT file. This CPT file is related to the “Amazonehaven” in Rotterdam. With the same input related to the FRP quay wall the outcome of the analysis seems reliable. The FRP quay wall will not reach the installation depth with just one blow of the diesel hammer.

The compression stress in the quay wall during the installation is around the 34 MPa but there are also registered values of 101 MPa. The maximum allowable compression of the FRP quay wall is related to the strain criteria of 0.27% as stated in [CUR96, 2003]. The strain criteria can be translated in a maximum compression stress. The maximum allowable compression stress can be calculated with Hooke’s law. The maximum allowable compression stress is 43 MPa.

7.9 Conclusion

7.9.1 Applied installation technique

From the above discussed installation techniques one should be chosen for the FRP quay wall. Due to the installation depth of 22.5 meters the pressing technique seems not applicable.

The maximum allowable compression stress is equal to 43 MPa while the driving analysis with Allwave PDP gives a compression stress around the 34 MPa with peaks of 101 MPa. There are formulas to estimate the compression stress in the quay wall. According to these formulas the compressions stress is determined by the Young’s modulus of the quay wall, the volumetric weight and the impact speed of the hammer. In formula form [Tol, 2004];

$$\sigma_d = \sqrt{E\rho} \cdot 10^{-3} \cdot \sqrt{2gh} \qquad \text{Eq. 13}$$

In which;	E	=	Young’s modulus of the quay wall	[MPa]
	ρ	=	Volumetric weight of the quay wall material	[kg/m^3]
	g	=	Gravitational acceleration	[m/s^2]
	h	=	Fall height of the hammer	[m]

With a fall height of 1.5 meters this results in a compression stress of 41 MPa which is less than the allowable compression stress according to [CUR96, 2003]. However, this estimation of the compression stress does not take into account the soil conditions and thus the soil resistance. It can therefore not be said that the quay wall element will penetrate the soil when a fall height of 1.5 meters is used. If the profile does not penetrate the soil, the fall height will have to be increased. To be exactly on the limit of the allowable compressions stress a fall height of 1.67 meters should be applied. The fall height is therefore limited to 1.67 meters.

Figure 32: Splitted pile head [ACE, 2012]

Atlantic Coast Engineering has performed a dynamic test and drivability analysis of a so called Superpile [ACE, 2012]. The Superpiles are constructed with a polyurethane resin containing E-glass fibre reinforcements. The Superpile has a Young’s modulus equal to 39.6 GPa and the volumetric weight is 1938 kg/m^3. The applied fibre volume fraction is higher than 50%. ACE has performed two driving analysis with two different hammers. The diesel hammer is the one that is interesting for this study. With the diesel hammer the Superpile has been driven to an embedment depth of 15.2 meters. At the end a fall height of 1.7 meters has been used. A day later the Superpile was redriven to 15.8 meters, where the pile heads split into several vertical pieces. The damaged pile is shown in Figure 32 and Figure 33.

The pile was driven at a blowcount in the range of 110 to 125 blows per 0.3 meter for 0.6 meter prior to the pile head splitting. The compressions stress in the pile when the pile was redriven was recorded to be between 76 and 103 MPa.

Figure 33: Damaged pile head after removal of the hammer [ACE, 2012]

The soil at the testing site consists of a sand/gravel layer with a thickness of approximately 3 meters. This layer is followed by a soft clay layer with a thickness of approximately 7 meters. The soft clay layer is followed by a layer consisting of loose clay and fine sand with a thickness roughly equal to 2.5 meters. Underneath the fine sand layer the dense sand layer with a thickness of approximately 10 meters is located

So the Superpile has been driven into the dense sand layer when the pile head splits into several vertical pieces.

The FRP quay wall has to be driven 22.5 meters into the ground. Based on the Allwave PDP analysis and the driveability test of a fibre reinforced pile it is concluded that driving of the FRP quay wall to the required depth is not feasible. Another method will have to be applied.

The remaining installation techniques are vibrating and the diaphragm wall technique. Vibrating the FRP quay wall segment to a depth of 22.5 meters seems highly unlikely. The steel combi wall of the case study is installed by a combination of driving and vibrating. The first few meters the combi wall is vibrated into the soil. The combi wall is then driven into the soil to the required depth. Installing the combi wall with merely the vibrating technique was not feasible.

The diaphragm wall technique shall therefore be assumed to be the right installation technique for the FRP quay wall. A trench will have to be excavated which corresponds with the shape of the FRP quay wall profile.

The above mentioned arguments are summarized in Table 46. The possible installation techniques are given a score from 1 to 4 for the categories: compression, installation depth and nuisance. The score 1 indicates that it is the best alternative while the score 4 indicates that it is the least suitable alternative.

Table 46: Decision matrix

	Compression	Installation depth	Nuisance	Total
Driving	4	2	4	**10**
Vibrating	2	3	3	**8**
Pressing	3	4	1	**8**
Diaphragm wall	1	1	2	**4**

The category 'compression' is related to the compression stress in the quay wall element while the category 'installation depth' is related to chance that the required installation depth will be reached. 'Nuisance' takes into account the nuisance for the surroundings at the construction site.

7.9.2 Remarks

A small remark to the driving technology is that as an alternative a steel profile, which is exactly the same as the profile of the FRP quay wall, could be used. The steel profile is then attached to the FRP profile and they are driven into the ground at the same time causing the impact of the hammer to be spread over the steel and FRP profile. The following FRP quay wall segment will then be driven into the soil where the steel profile has already been driven into the soil. The soil resistance will therefore be smaller which might lead to a lower compressive stress in the FRP quay wall segment. This will have to be elaborated into more detail by means of testing and if possible computer simulations.

The above mentioned conclusions concerning the installation techniques driving, vibrating and pressing are supported by a folder of the company Bergschenhoek. During my stay at IGR they gave a presentation about their ROwat sheet piling. This is a sheet pile constructed out of PVC also known as uPVC-M. They acknowledge that the sheet piling can be installed by driving, vibrating and pressing but that the techniques are limited by the soil conditions and required installation depth. Driving of their sheet pile to a depth of 8 meters has been done with a steel sheet auxiliary profile. [Bergschenhoek, 2016] The depth of 8 meters is just a quarter of the required depth for the FRP quay wall.

A remark concerning the FRP quay wall is that the volumetric weight of the FRP is less than that of the bentonite. The bentonite will therefore not automatically be pushed towards the surface by the FRP quay wall. A solution is to add extra pump capacity so that the bentonite can be extracted from the trench.

The installation techniques driving, vibrating and pressing have not been considered with jetting as an alternative. [Tissink, 2012] gives an interesting insight in the possibilities of a combination with pressing and jetting. Sheet piles are 18 meters pressed into the soil with the assistance of bentonite jetting. The bentonite does not only reduce the resistance of the soil but it also functions as a lubricant between the soil and the sheet pile resulting in lower required pressures to press the sheet pile into the soil.

8. Life Cycle Analysis

8.1 Chapter content

This chapter will focus on the LCA of the FRP quay wall as well as the steel combi wall. Some general information regarding the LCA will be presented before the LCA is determined for both types of quay walls. The LCA will be discussed step by step. The goal and scope definition will be presented. By means of a flowchart the relevant processes will be indicated involved in the life cycle of the quay walls. The flowcharts are followed by an impact assessment for several categories. This chapter is concluded with an evaluation of the LCA of both structures.

8.2 General information

Climate change is a continuous subject for nowadays politics. Everybody is aware of the climate changes around the world although there is not a mutual understanding for the causes of the climate changes. The main accepted idea is that the climate changes are induced by the greenhouse effect. There are also sceptics who state that the climate change is a natural phenomenon referring to the ice ages.

The six most important greenhouse gases according to NEN-ISO 14064-1 are carbon dioxide, methane, nitrous oxide, hydrofluorocarbons, perfluorocarbons and sulphur hexafluoride. For each of these gases the potential contribution to the global warming can be determined. The Global Warming Potential is a relative indicator with carbon dioxide as a reference. With the Global Warming Potential the so called carbon footprint of a structure can be calculated.

There are several databases developed by companies, universities or other institutions. A few examples are NIBE, Ecoinvent and “Stichting Bouwkwaliteit”.

NIBE is the Dutch institute for building biology and ecology. They research and advise in the fields of environment, health and building/maintenance. The database that can be accessed on their website does not contain the relevant materials for this study. [NIBE, 2016]

Ecoinvent is founded by institutes of the ETH Domain and the Swiss Federal Office. This database is used in several LCA programs as for example SImaPro. It contains international industrial life cycle inventory data on energy supply, resource extraction, material supply, chemicals, metals, agriculture, waste management services and transport services. This database can only be accessed when a licence is purchased.

“Stichting Bouwkwaliteit” is not a database in itself. The goal of SBK is to stimulate and promote quality management in the building sector and to harmonize all national guidelines and certification. This resulted in a ‘Nationale Materialen Database” which contains databases for products as well as processes and discard scenarios. The Database is incorporated in several LCA software such as the program DuboCalc.

To keep the LCA transparent it is not favourable to use a program to calculate the environmental impact of the structures.

The LCA in this feasibility study will be based on a database that has been provided by Dr.ir. H. Jonkers of the TU Delft. He is the chair-leader of the research chair sustainability of the Materials and Environment department. The database contains approximately 500 materials as well as several types of transportation and is subtracted from the ‘Stichting Bouwkwaliteits Nationale Milieudatabase’.

A life cycle analysis gives insight in the environmental impact of a structure. It is a systematic way to evaluate the environmental impacts of products or actions. This will be achieved by applying the cradle-to-cradle approach. An LCA consists of 4 steps [RIVM, 2016].

Step 1: Goal and scope definition
In this step the functional unit will be identified. The functional unit is the unit for which each relevant material will be taken into account. It is also important to identify the initiator of the assessment as well as the boundaries of the assessment. The output of this step is a descriptive text with the subject and the structure of the study.

Step 2: Inventory analysis
This step is also known as the Life Cycle Inventory. The inventory of the materials and relevant processes will be done by means of a flowchart. This flowchart will include all phases of the structures or products that are taken into account. It starts with the mining of the raw materials and will end with the demolishing or recycling of the structure or product. The output of this step is therefore a flowchart that indicates all the relevant processes.

Step 3: Impact assessment
The impact assessment is also known as Life Cycle Impact Assessment. The characterization is the core of the impact assessment. The characterization multiplies the impact categories with the corresponding characterization factors. This will result in an overview of the impact of the structure per impact category. To be able to compare structures or products with one another a normalization or weighing can be applied.

Step 4: Evaluation
The evaluation contains an analysis of the previous steps. Output of this step is for example an indication for which impact category the structure or product should be redesigned to decrease the environmental impact.

Each step will be elaborated into detail in the following paragraphs.

8.3 Goal and scope definition

The goal of the LCA will be determined along with the scope of the analysis in this paragraph.

The goal of the LCA is:

> "*Compare the environmental effects of the steel combi wall and FRP quay wall which have been designed for the same functional and technical requirements as defined in this feasibility study. The environmental effects of both quay walls will be calculated for the functional unit.*"

The functional unit that will be taken into account is 1 meter quay wall also named a running meter quay wall. Quay wall facilities such as fenders, cranes, bollards and anchors will not be taken into account since they are the same for both designs. The bottom protection that has been designed in the case study will be neglected for the same reason.

The following impact categories will be taken into consideration for the LCA;

1. Global Warming Potential (GWP),
2. Ozone Depletion Potential (ODP),
3. Abiotic depletion potential (ADP),
4. Human Toxicity Potential (HTP),
5. Fresh Water Aquatic Ecotoxicity Potential (FAETP),
6. Marine Aquatic Ecotoxicity Potential (MAETP),
7. Terrestrial Ecotoxicity Potential (TETP),
8. Photochemical Oxidation Potential (POCP),
9. Acidification Potential (AP),
10. Eutrophication Potential (EP).

The lifetime of the structure is, according to the program of requirements, set at 50 years. This will be the time span for which the LCA will be performed.

8.4 Inventory analysis

The inventory analysis has been done by means of flowcharts. The flowcharts for the steel combi wall and the FRP sandwich structure can be found in Appendix J: Life Cycle Analysis. The flowcharts are composed of processes related to the quay wall. These processes are manufacturing of the elements, construction, operational and end of lifetime. The input and output of each process is included in the flowchart. An example of input is the action transport and an example for output is the waste that remains after a product has been produced.

8.5 Impact assessment

The flowchart gives a clear vision of the processes and their input and output. These processes must be quantified so that the impact of the structure can be calculated for every impact category. Some assumptions have been made to be able to quantify the processes. These assumptions will be elaborated into detail in the following subparagraphs. The normalization will be discussed as well.

8.5.1 Production

For the production of the elements of the quay walls the required kilograms materials have to be calculated. From the designs of the quay walls the required volume of a certain material can be calculated. Multiplying the required volume with the density will result in kilograms of materials.

8.5.2 Transportation

The distance the elements must be transported from the factory to the construction site must be determined. These distances are based on a report of IGR. This report is a study to the CO_2 footprint of quay walls. In this study the carbon footprint of 3 existing quay walls has been determined. The used data for the determination of the carbon footprint is obtained from the Ecoinvent 2.0 database and the IVAM LCA Data 4. Both databases are not freely accessible. In the report the transport distances for several materials are given and they will function as guidance in this LCA.

The transport distances are [Rotterdam, 2010];

- Steel: Luxembourg, 280 km per axle;
- Concrete: Europort Rotterdam, 45 km per axle;
- FRP: Rotterdam, 20 km per axle;
- Bentonite: Overseas, 3500 km.

8.5.3 Construction

The construction of the quay walls will be done with equipment such as cranes, pumps, aggregates etc. The equipment will run for several hours resulting in emissions as well. The required equipment and the number of hours the equipment must run is determined for each construction stage.

8.5.4 Lifetime

During the lifetime of the structure maintenance might be required as well as inspection of the structure. The materials and use of equipment during the maintenance and inspection over 50 years must therefore be determined. Maintenance work such as replacement of the fenders and mooring systems will not be taken into account since they are equal for both quay walls.

The steel combi wall will have to be protected against corrosion. This can be done with cathodic protection or increase the thickness of the steel cross section so that corrosion of the cross section can be tolerated. The last option has been applied for the design of the steel combi wall in the case study. The corrosion during the lifetime is determined at 0.9 mm and this has been taken into account in the strength checks of the combi wall.

8.5.5 End of lifetime

When the structure has served for 50 years it has reached its end of lifetime. There are now several scenarios possible for the structure. The structure can be reused, recycled, incinerated or dumped. Each scenario requires certain equipment for several hours. Thus energy will be needed in this phase of the structure resulting in emissions.

The data that will be used to determine the environmental effects comes from the database provided by Dr.ir. H. Jonkers. Every waste post in the flowchart results in material that will not be used. To account for the waste a certain percentage can be added to the amount of required materials. Since this percentage will be the same for every material this will not make a difference and the posts waste are therefore not taken into account.

8.5.6 Normalization

When the environmental impact has been calculated for each impact category, the results will be normalized so that all categories can be added and a total score of the structures can be obtained and compared with one another. Each impact category will be expressed in euros. The normalization will be done with the so called shadow costs. These shadow costs are included in the database provided by Dr.ir. H. Jonkers and are shown in Table 47.

Table 47: Shadow costs for the impact categories

Impact category	Normalization [€/kg]
Global Warming Potential (GWP),	0.05
Ozone Depletion Potential (ODP),	30
Abiotic depletion potential (ADP),	0.16
Human Toxicity Potential (HTP),	0.09
Fresh Water Aquatic Ecotoxicity Potential (FAETP),	0.03
Marine Aquatic Ecotoxicity Potential (MAETP),	0.0001
Terrestrial Ecotoxicity Potential (TETP),	0.06
Photochemical Oxidation Potential (POCP),	2
Acidification Potential (AP),	4
Eutrophication Potential (EP).	9

8.5.7 Results

The carbon footprint has been calculated with the database and the report of IGR. The report has been used for the transport and construction technology while the database has been used for the materials. The carbon footprint is equal to the impact category GWP. The carbon footprint has been determined for the steel combi wall as well as for the FRP quay wall. The entire life cycle of the structures has been taken into account.

The carbon footprint of the steel combi wall and the FRP quay wall is presented in Figure 34. The GWP has been calculated for the manufacturing of the elements of the quay wall, transportation of the elements to the construction site, constructing the quay wall at the construction site and the end of lifetime. From the figure it can be concluded that the manufacturing of the elements produces the most GWP. The GWP in the construction phase of the FRP quay wall is also significant. This is caused by the bentonite that has to be used with the diaphragm wall installation technique. The GWP value for the steel combi wall is equal to 6076 kg CO_2 equivalent while the value for the FRP quay wall is 61363 kg CO_2 equivalent. The difference between the GWP of the steel combi wall and the FRP quay wall is a factor 10. The GWP has been calculated for a running meter quay wall.

The bill of quantities that has been used to calculate the values of the impact categories can be found in Appendix J: Life Cycle Analysis.

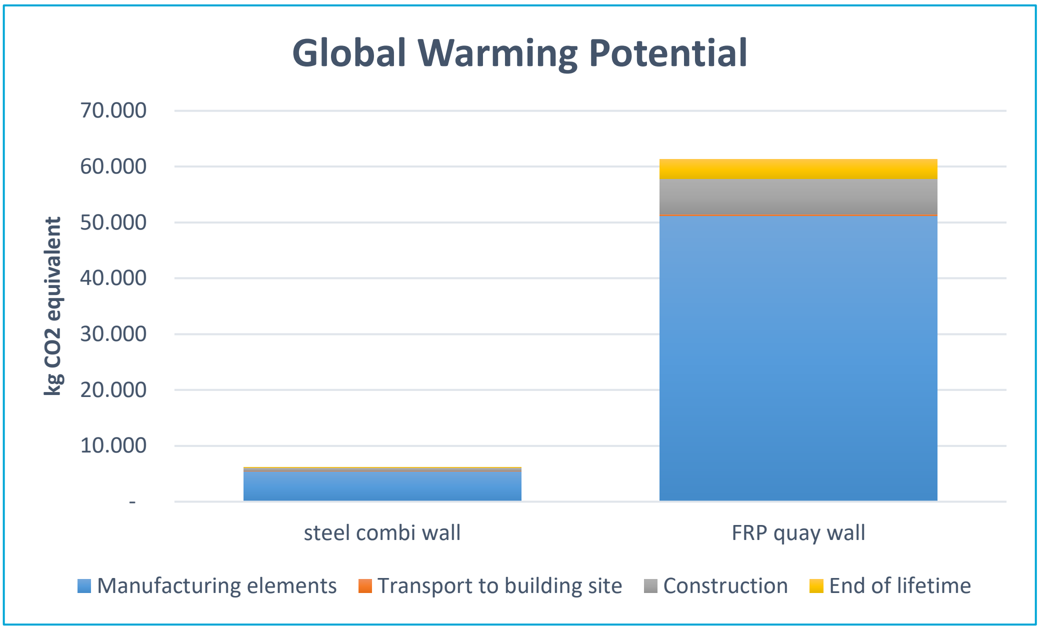


Figure 34: Global Warming Potential of the quay walls

The GWP has also been calculated for each phase as already mentioned. The GWP for the phase 'manufacturing elements' is shown in Figure 35. The GWP is again calculated for one meter running quay wall. The difference between the steel combi wall and FRP quay wall for the phase 'manufacturing elements' is again roughly a factor 10.

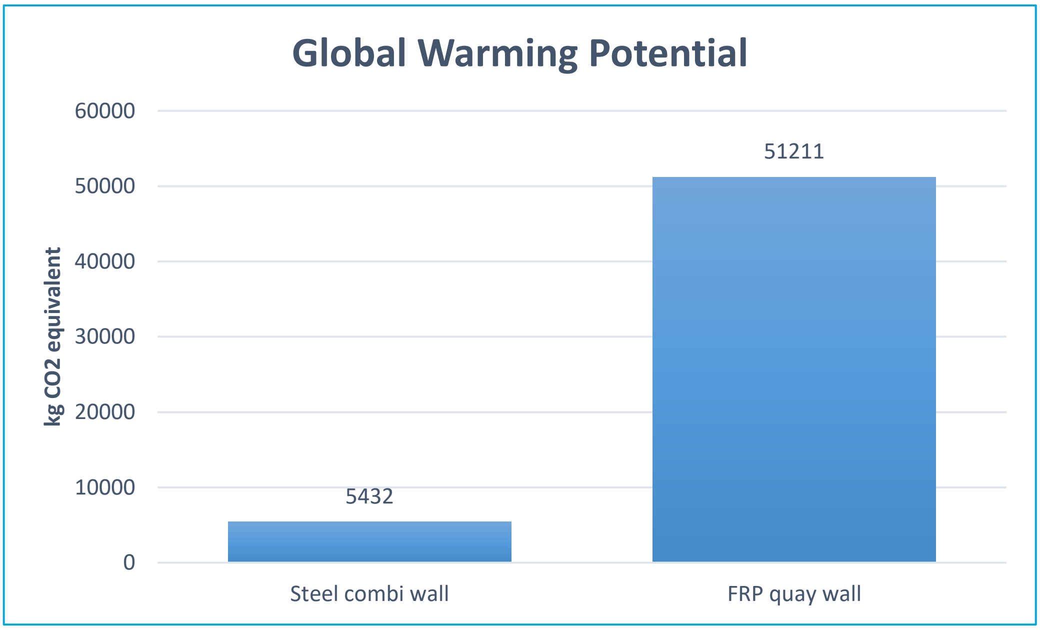


Figure 35: Global Warming Potential for the phase 'manufacturing elements'

The values for the impact categories as mentioned in 8.3 Goal and scope definition have been calculated for both quay walls. The result of each impact category has been normalized with the shadow costs as mentioned in Table 47. The normalized environmental impact for both type of quay walls is shown in Figure 36. The environmental impact for both type of quay walls has been calculated only for the phase 'manufacturing elements'. The other phases could not be calculated since the report of IGR does only provide values for the GWP.

The total normalized environmental impact for the steel combi wall is equal to €1.948,05 while the FRP quay wall has a value of €11.972,91. The factor between the two types of quay walls is roughly 6. Graphs of the individual impact categories are included in Appendix J: Life Cycle Analysis.

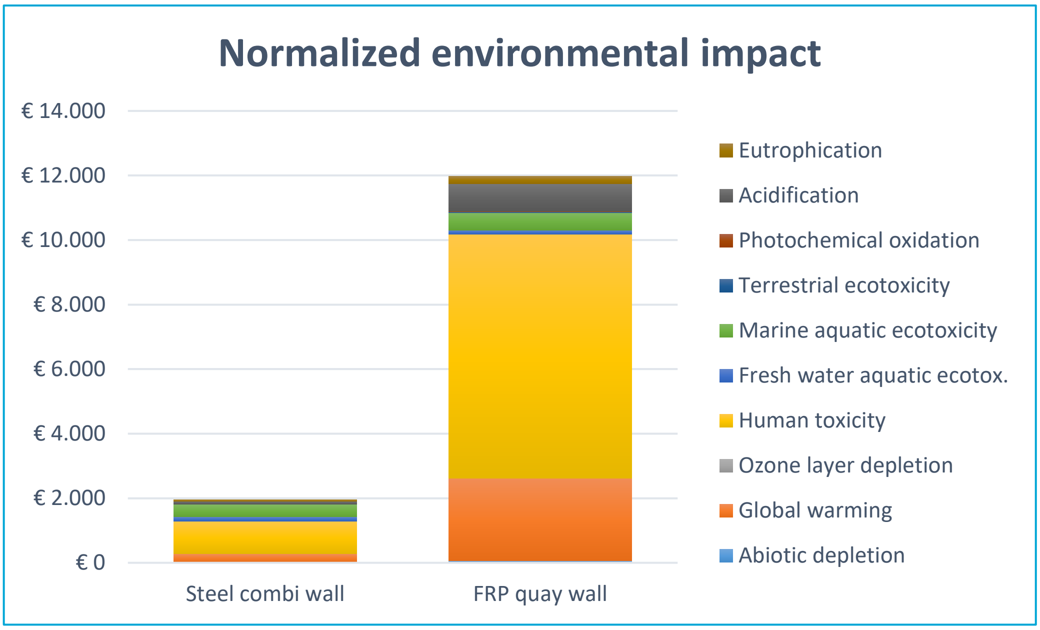


Figure 36: Normalized environmental impact for both types of quay walls

From Figure 36 it can be concluded that the categories GWP and HTP deliver the largest contributions to the normalized environmental impact of both structures.

The FRP quay wall has for almost every impact category a higher environmental impact than the steel combi wall. However, there is one exception. For the impact category 'Fresh water aquatic ecotoxicity' the steel combi wall has a slightly higher impact than the FRP quay wall, which can be seen in Figure 37.

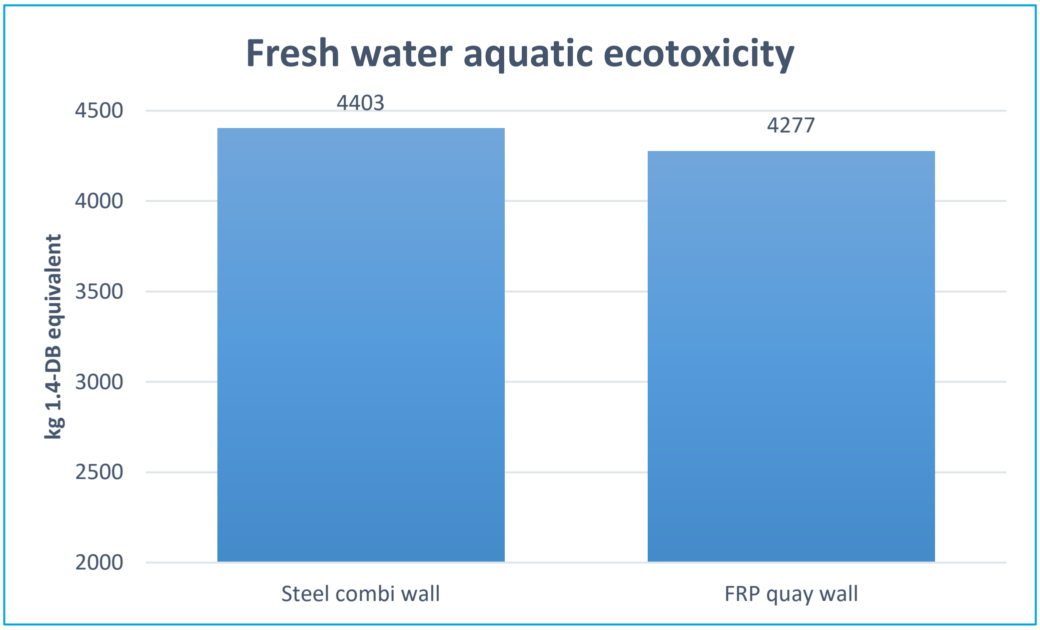


Figure 37: Results for the impact category 'Fresh water aquatic ecotoxicity'

8.6 Evaluation

It can be concluded from the impact assessment that the FRP quay wall has a far higher environmental impact than the steel combi wall. The largest environmental impacts are obtained in the phase 'manufacturing elements'. If the environmental impact of the structures has to be lowered it is in this phase where the biggest gains can be achieved.

Looking at the FRP quay wall there is also space for improvement in the 'construction' and 'end of lifetime' phase. The contribution in the 'construction' phase is caused by the bentonite that has to be applied. It is therefore recommendable to study the feasibility of other installation techniques.

The 'end of lifetime' phase has been based on estimated percentages for recycling, incineration and waste. It is estimated that 50% of the FRP quay wall can be reused, 25% will be incinerated and 25% will be waste and has to be dumped. Studies and research will have to show If the percentages of the 'end of lifetime' scenarios are realistic or not. Since FRP is an upcoming material in civil engineering there are yet no large hydraulic structures constructed out of FRP which have served their lifetime and reached the 'end of lifetime' phase.

Table 48 shows the ratio between the normalized environmental impacts of both quay walls. The steel combi wall has been taken as a reference to determine the comparison factor.

Table 48: Ratio between normalized environmental impact with the steel combi wall as reference

	Total weight [kg]	Total costs [€]	Ratio [€/kg]	Factor [-]
Steel combi wall	2969,3	1.948,05	0,66	1
FRP quay wall	11170,9	11.972,91	1,07	1.63

If the FRP quay wall could be designed with a lighter profile the environmental impact would decrease. With the strain limit as given in [CUR96, 2003], the profile of the FRP quay wall as designed in chapter 5 is almost as light as possible. However, in the revision of CUR96 the strain limit of 0.27% is not mentioned for FRP structures permanently situated in a humid environment. The only strain criteria mentioned in the revised version of CUR96 is 1.2%. This is an increase of 344 % for the strain criteria. The maximum calculated strain for design 1 was equal to 0.98% in the ULS for the web laminate. The area of the skin laminate for design 1 is equal to 0.086 m^2 per running meter while the area of the skin laminate for design 2 is equal to 0.173 m^2 per running meter. This is a difference of 50%. For just the skin laminate a reduction of 50% material can be achieved if the strain criteria is increased from 0.27% to 1.2%. The reduction of 50% material will have a significant impact on the environmental impact of the FRP quay wall.

However, even if the profile of the FRP quay wall becomes 50% lighter it will still have a bigger environmental impact than the steel combi wall.

A big variable for the LCA are the boundaries. The obtain the raw materials that are required to construct steel and FRP equipment is required. This equipment will run for several hours resulting in emissions. The equipment that is required to mine the raw materials will be manufactured in a factory. The factory will produce emissions as well. And so it can go on and on into the smallest details. All these emissions will influence the result of the LCA.

The material database provided by Dr.ir. H. Jonkers does not state which processes have been taken into account in the characterizations factors of each impact category.

9. Cost estimation

9.1 Chapter content

The costs of the steel combi wall and FRP quay wall will be estimated in this chapter. The cost of the steel combi wall are obtained from the bill of quantities from the contract party of the case study. The cost of the FRP quay wall will be estimated with data from [Valk, 2016]. [Valk, 2016] provides prices for the resins as well as the fibres. The labour costs for FRP products are also obtained from [Valk, 2016]. The steel combi wall will be discussed first. Before the chapter is concluded with a comparison between the two quay walls the cost of the FRP quay wall will be presented.

9.2 Steel combi wall

The price of the steel combi wall will be calculated for a running meter quay wall. A separation will be made between the material cost and installation cost. The combi wall consists of 2 tubular sheet piles and 3 PU18 sheet profiles. The total width of this combined system is equal to 4.24 meters. To derive a price of the steel combi wall per running meter quay wall, the total price for 2 tubular profiles and 3 PU18 sheet profiles is divided by 4.24 meter. As mentioned in the introduction of this chapter, the prices for the elements are obtained from the bill of quantities from the party that won the contract for the case study project. The derivation of the price for 1,0 meter combi wall is presented in Table 49.

Table 49: Price of 1,0 m steel combi wall

Steel combi wall			
Element	**Material**	**Installation**	**Total**
2 tubular piles	€ 9.000,00	€ 1.175,00	€ 20.350,00
3 PU18 sheet piles	€ 2.731,06	€ 215,80	€ 2.946,86
Combi wall (4,24 m)	€ 20.731,06	€ 2.565,80	€ 23.296,86
Combi wall, 1,0 m	**€ 4.889,40**	**€ 605,14**	**€ 5.494,54**

In [Valk, 2016] a price for steel per kilogram has also been given. The price of steel per kilogram is equal to €1,40. From the bill of quantities as shown in Appendix J: Life Cycle Analysis the amount of kg steel per 1,0 meter steel combi wall can be obtained. For 1,0 meter steel combi wall 2969 kilogram steel is required. This results in a price of €4.157,04 for 1,0 meter steel combi wall. The price as mentioned in Table 49 will be considered governing.

9.3 FRP quay wall

The price of the FRP quay wall will also be calculated for 1,0 meter quay wall. The price indication as mentioned in [Valk, 2016] will be used. According to [Cripps, 2016] the price of the polyester resin is €2.56 per kilogram while the price for the glass fibre is equal to €2.56 per 300 gram woven fabric. [Valk, 2016] also mentions costs of FRP construction, so constructions where the resin and fibre are already combined to a construction. The fibre volume is estimated at 65% and the resin volume is equal to 35%. The material cost, labour cost and total cost of a glass fibre polyester construction can be found in Table 50.

Table 50: Costs related to a glass fibre polyester construction [Valk, 2016]

Fibre and resin	Material costs [€/kg]	Mean base material cost [€/kg]	Labour costs [€/kg]	Total costs [€/kg]
E-glass and polyester	1.3 – 2.5	1.9	2 – 5	4 – 7

Table 51: Price of 1,0 m FRP quay wall

FRP combi wall					
	Volume material [m³]	mass [kg]	price per m³ [€/m³]	price per kg [€/kg]	Total
FRP skin	3,76	7353,80	-	€ 7,00	€ 51.476,60
FRP web	1,14	2144,55	-	€ 7,00	€ 15.011,85
Foam	5,89	1672,54	€ 312,50[4]	-	€ 1.840,38
FRP combi wall (1,0 m)	-	-	-	-	**€ 68.328,84**

Table 51 shows the price of 1 meter FRP quay wall. The price of the FRP quay wall is approximately 10 times higher than the steel combi wall. The required mass of FRP and foam per running meter quay wall has been obtained from the bill of quantities as presented in Appendix J: Life Cycle Analysis.

For the diaphragm wall installation technique there is no literature available to provide some kind of insight in the cost of the technique. The literature that is available provides information for the construction of a diaphragm wall. [Bodemrichtlijn, 2017] states that the cost of a diaphragm wall with a length between 15 and 30 meters varies between €155,- / m^2 and €303,- / m^2 depending on the thickness of the diaphragm wall. The diaphragm wall is constructed with reinforced concrete. From projects of IGR a price for reinforced concrete has been derived. The price of 1 m^3 reinforced concrete is estimated at €115,-.

The price for the diaphragm wall technique will therefore be estimated at €115,- per m^3. This is the average value for the reinforced concrete diaphragm wall minus the cost for reinforced concrete. The excavation of the trench and the bentonite are incorporated in this indicator.

[Bodemrichtlijn, 2017] states that the price for soil removal varies between €9,- and €20,- per m^3. The price for soil removal is therefore estimated at €14,50 per m^3.

With these two indicators for the installation technique an estimation for the construction technique for the FRP quay wall has been made. The results are presented in Table 52.

Table 52: Estimation of the price for the diaphragm wall installation technique for 1,0 m FRP quay wall

FRP quay wall			
Action	Volume [m³]	price per m3 [€/m³]	Total
Soil removal	11,01	€ 14,50	€ 159,60
Excavation and bentonite	10,79	€ 115,00	€ 1.241,00
Diaphragm wall technique (1,0 m)	-	-	**€ 1.400,61**

The difference between the diaphragm wall installation technique and the driving and vibrating of the sheet combi wall is more than a factor 2.

The total cost of the FRP quay wall consisting of the material costs and the installation costs is presented in Figure 38.

[4] [isolatie-weetjes, 2017]

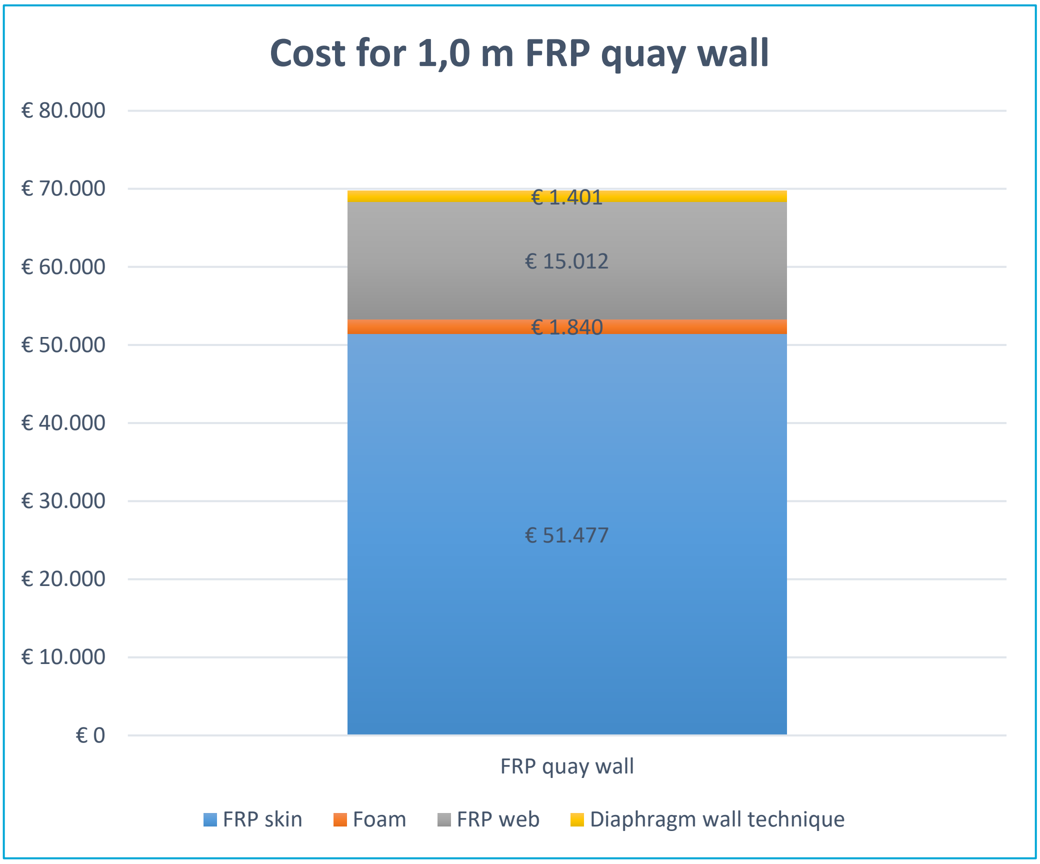


Figure 38: Cost estimation of 1,0 m FRP quay wall

9.4 Conclusion

The biggest difference between the FRP quay wall and the steel combi wall is in the cost of the materials, as was expected based on [Valk, 2016]. The costs for the steel combi wall and the FRP quay wall are presented in Figure 39.

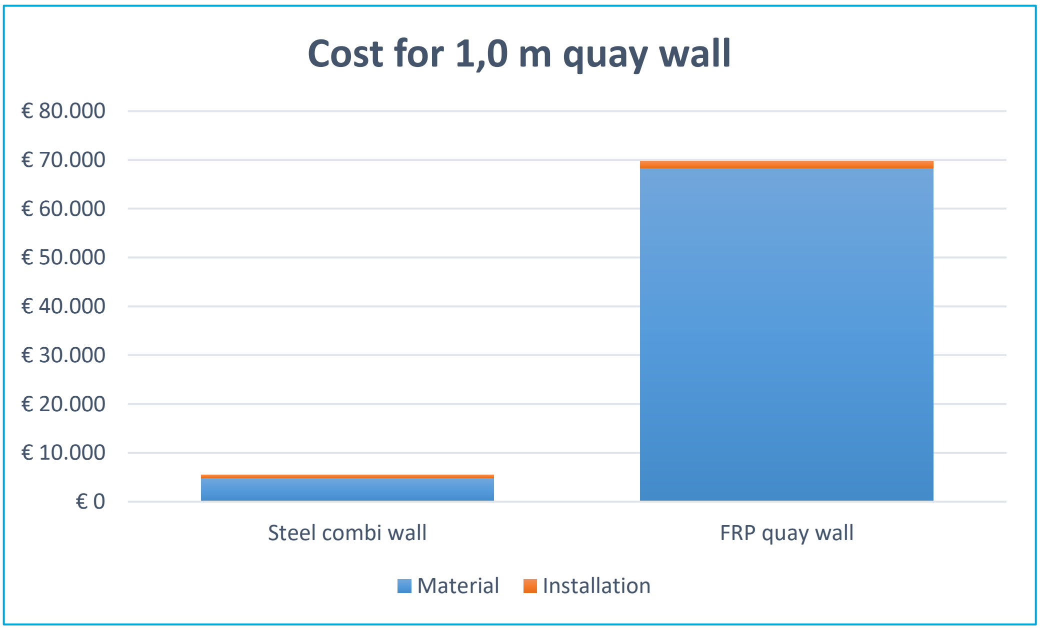


Figure 39: The cost for 1,0 m quay wall for both materials

The FRP quay wall differs a factor 12 to 13 compared to the steel combi wall.

It is clear that the materials contribute the most to the price of 1,0 meter quay wall. If the FRP quay wall could be designed with a lighter profile it would become more competitive with respect to the steel combi wall. Most of the projects nowadays are awarded to the party with the lowest tender price.

If the FRP quay wall could be designed with a lighter profile the cost would decrease. With the strain limit as given in [CUR96, 2003], the profile of the FRP quay wall as designed in chapter 5 is almost as light as possible. However, in the revision of CUR96 the strain limit of 0.27% is not mentioned for FRP structures permanently situated in a humid environment. The only strain criteria mentioned in the revised version of CUR96 is 1.2%. This is an increase of 344 % for the strain criteria. The maximum calculated strain for design 1 was equal to 0.98% in the ULS for the web laminate. The area of the skin laminate for design 1 is equal to 0.086 m^2 per running meter while the area of the skin laminate for design 2 is equal to 0.173 m^2 per running meter. This is a difference of 50%. For just the skin laminate a reduction of 50% material can be achieved if the strain criteria is increased from 0.27% to 1.2%. The reduction of 50% material will have a significant impact on the cost of the FRP quay wall.

Even if the price of the FRP quay wall could be reduced with 50% it would still be 6 times more expensive than the steel combi wall.

Another argument in favour of FRP is that it does not require a lot of maintenance during its lifetime. In the case study the corrosion of the steel combi wall is incorporated in the design. In the case study it is stated that the corrosion of the steel tubular pile will be 0.90 millimetre in 50 years. The remaining maintenance is then the replacement of fenders and mooring facilities. This maintenance will also have to be done for the FRP quay wall. It is therefore not likely that the FRP quay wall will close the investment gap with the steel combi wall based on the maintenance assumption of the materials itself.

However, it should be noted that the used data to estimate the costs of both structures are not constant data. The used data are variables with a certain deviation. The variables are used to estimate the costs of the structures meaning that several variables are added to one another which does not contribute to the reliability of the outcome. It is assumed that the data for the steel combi wall has a margin of 20% while the data used for the FRP quay wall has an assumed margin of 50%.

10. Discussion

10.1 Chapter content

During this feasibility study I have encountered information that can have a significant impact on the conclusions from this study. This additional information will be discussed in this chapter.

10.2 Standards and guidelines

Since the use of FRP is new in the field of hydraulic engineering a clear guideline for using FRP in designs of hydraulic structures such as quay walls is yet to be developed. The CUR96 that has been used in this feasibility study is mainly focused on the design of FRP bridges. There is also confusion related to the maximum allowed strain in an FRP structure that is in permanent contact with water.

With the strain limit as given in [CUR96, 2003], the profile of the FRP quay wall as designed is almost as light as possible. However, in the revision of CUR96 the strain limit of 0.27% is not mentioned for FRP structures permanently situated in a humid environment. The only strain criteria mentioned in the revised version of CUR96 is 1.2%. This is an increase of 344 % for the strain criteria. During the design cycle for the FRP wall, the first design, design 1, had the same profile as the evaluated design. The thickness of the skin laminate of design 1 is equal to 26 millimetres while the thickness of the web laminate is equal to 12 millimetres. The maximum calculated strain for design 1 was equal to 0.98% in the ULS for the web laminate. The area of the skin laminate for design 1 design is equal to 0.086 m^2 per running meter while the area of the skin laminate for the evaluated design is equal to 0.173 m^2 per running meter. This is a difference of 50%. For just the skin laminate 50% reduction of the material can be achieved if the strain criteria is increased from 0.27% to 1.2%. The reduction of 50% will have a significant impact on the cost of the FRP quay wall as well as the environmental impact.

The normalized environmental impact for design 1 is presented in Figure 40. Comparing the environmental impact of the FRP quay wall with dimensions according to design 1, Figure 36, with the environmental impact of the evaluated design shows a decrease of approximately 50%.

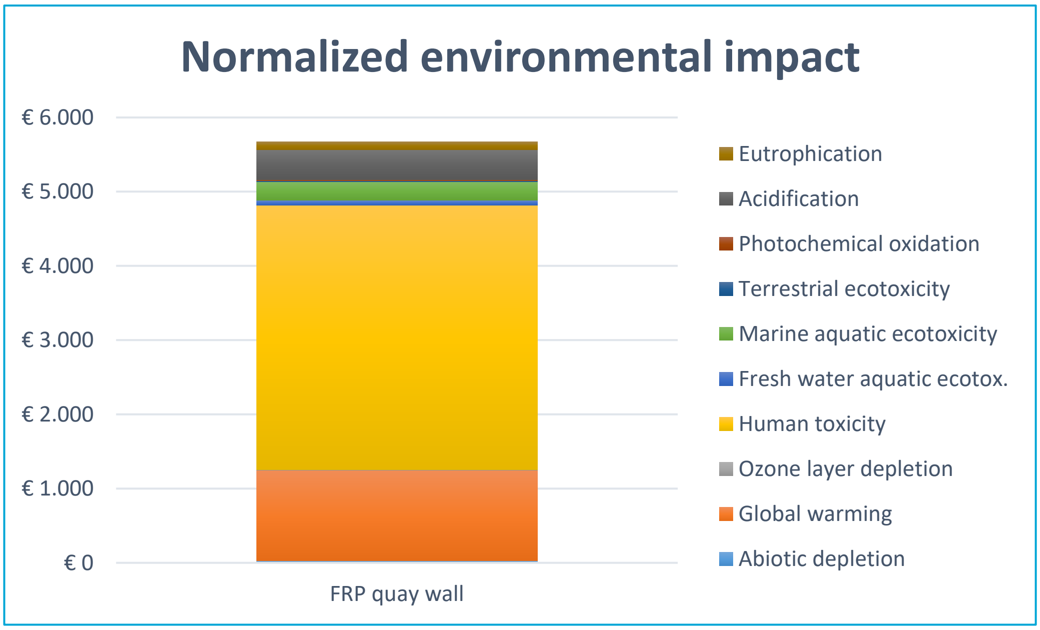


Figure 40: Normalized environmental impact for an FRP quay wall with dimensions corresponding to design 1

The material cost as well as the cost for the installation technique for an FRP quay wall with dimensions according to design 1 is shown in Figure 41. When these costs are compared to the costs of the evaluated design, Figure 38, it can be concluded that a reduction of approximately 50% has been achieved.

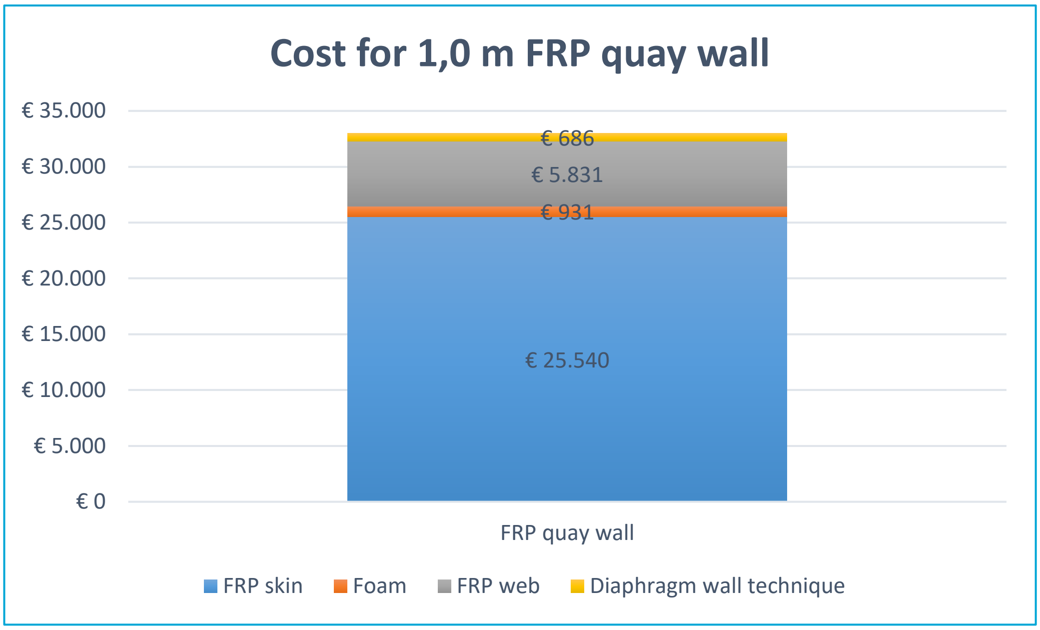


Figure 41: Cost for 1 meter FRP quay wall with dimensions corresponding to design 1

10.3 Superstructure

The superstructure of the quay wall is not incorporated in this feasibility study. To be able to design a quay wall merely out of FRP, the superstructure has to be designed in FRP as well. Difficulties are expected at the connection between the bollards and the superstructure. The local forces at the bollard are high which will have to be distributed along the superstructure. Therefore, the connection between a bollard and the superstructure requires a design that can withstand the high local forces.

10.4 Life Cycle Analysis

My supervisor Henk Jonkers provided an interesting example on this matter. He referred to an LCA that compared wood with other structural materials. In order to have enough wood for a construction, a forest will have to be harvested. This forest will use a certain area of land. In the LCA the impact category 'land use' had not been taken into account. This resulted in the conclusion that wood was one of the most environmentally friendly structural materials. However, if the impact category 'land use' would have been taken into account the result would be different. But because it was clearly stated in the goal and scope definition of the LCA it was a valid LCA.

What I aim to demonstrate with this example is that an LCA is very susceptible to fraud. If you state clearly in the goal and scope definition which impact categories you take into account and which one you neglect, you have a valid LCA. So if you know which impact categories have a negative effect on the outcome of your product, you could just state in the goal and scope definition that that specific impact category will not be taken into account.

The LCA that has been performed in this study calculated the environmental impact only for the materials itself. The transport of the materials to the construction site as well as the fabricating of the elements and the installation technique at the construction site are not taken into account. Although it is not likely that these phases will alter the outcome of the analysis, for a complete LCA they should be incorporated in the analysis. Unfortunately, as already mentioned in the chapter concerning the LCA, the database provided by Dr.ir. Henk Jonkers does not contain all relevant data regarding these phases.

10.5 Applied data in the LCA and cost estimation

The applied data in both the LCA and cost estimation are most likely mean values. Every mean value has a certain deviation. By adding and multiplying these mean values the deviation will become bigger and bigger. How big the deviation is for the outcome of the analysis and estimations cannot be said with certainty. The deviation of the data is not mentioned in literature. Assumptions have been made for the margins of the cost related data. How the data has been determined is not known and therefore there is a certain uncertainty within the outcome of the LCA as well as the cost estimation.

10.6 Eurocode

The design of the quay wall has been designed according to 'CUR166: Damwandconstructies' with the corresponding safety philosophy. Nowadays, the design of a quay wall must be made according to the Eurocode safety standard. The difference between the Eurocode and the CUR are the applied safety factors. The model of the FRP quay wall in D-sheet has been used to calculate the deformation and maximum moment according to the Eurocode.

The maximum deflection of the quay wall is equal to 79.2 millimetres and the maximum moment in the ULS is equal to 876.5 kNm with the Eurocode standard. For the CUR standard the deflection is equal to 79.2 millimetres while the maximum moment in the ULS is equal to 746.6 kNm.

The deflection of the quay wall is the same for both safety standards. The maximum moment in the ULS is higher with the Eurocode safety standard than the CUR safety standard. The difference between the two safety standards is more than 100 kNm. Therefore It cannot be said with certainty that the design of the FRP quay wall will be applicable according to the Eurocode safety standard.

10.7 Reference projects

At the start of this thesis there were no reference projects available for FRP quay walls. At the end of 2016 Deltares has tested a vertical composite sea wall designed by the Australian company Armour Group [Dutch Water Sector, 2016]. The construction of the sea wall consists of a double row of interlocked composite sheet piles. The distance between the two rows is 4 meters and the space between them is filled with sand. The two rows are held together by a system of geogrids which are inserted in the sand. The composite sheet piles are specially coated to resist very demanding marine environment.

This project can be used as a reference for further optimization of the design of the quay wall.

10.8 Additives

The design of the FRP laminate in this feasibility study is limited to the fibre reinforcement and resin. Due to the fact that the FRP quay wall has a permanent interaction with a humid environment, an additive has to be added to the design of the laminate. A possible additive is a gel coat. Gel coats improve the durability of components, protect the laminate from the environment, reduce the fibre pattern, provide a smooth aesthetic finish and eliminate the need for painting. The application of a gel coat has structural and aesthetic benefits and it reduces the required maintenance.

The impact of an additive on the mechanical properties must be determined by means of tests.

11. Conclusion

The research objective of this feasibility study is defined as:

"Design an FRP quay wall based on a case study with the current regulations and standards."

From this feasibility study it can be concluded that an FRP quay wall is technically feasible. The FRP quay wall has been designed based on the design conditions and requirements from the case study Alblasserdam. A quay wall profile consisting of 2 skins, webs to connect the 2 skins and a foam core has been realized in this study. The profile will have a Z-profile similar to a steel sheet pile. The quay wall has an overall height of 21.75 meters. Glass fibre reinforcement and a polyester resin form the unidirectional lamellae. Lamellae will be stacked in a certain order to obtain the laminates for the skins and webs.

The skins of the quay wall profile are constructed from an anisotropic laminate with 55% of the fibres in the vertical direction, the so called 0-direction, and 15% of the fibres are located in the other directions. The webs are constructed as a quasi-isotropic laminate which means that 25% of the fibres are placed in each of the 4 main directions. The skin laminate has a thickness of 50 millimetres while the web laminate is 20 millimetres thick.

The deflection of the quay wall is 79 millimetres for the normal load combinations and 78 millimetres for the exceptional loading case of an impact load. This is well below the maximum allowed deflection of 400 millimetres. The governing criteria for the FRP quay wall is the strain criteria according to CUR96.

The strain of the FRP quay wall should be lower than 0.27% in the SLS as well as in the ULS. The laminate strains have been calculated both manually and by means of a 2D model. The forces in the laminates have been calculated by using a 3d FEM SCIA model. Subsequently, the strains for the lamellae were calculated by using Kolibri. From these calculations it can be concluded that the strain of the FRP quay wall does not become bigger than 0.21% for the normal loading combinations. The exceptional loading case of an impact load results in a strain of maximum 0.25%.

The FRP quay wall has been checked for the characteristic failure mechanisms of buckling, interlaminar shear strength, wrinkling and shear stress failure. These checks have been performed for the normal loading combination as well as for the exceptional loading case of an impact load. The results of all these checks is that the FRP quay wall does not fail for either one of the failure mechanisms.

The anchor capacity, overall stability and bearing capacity of the FRP quay wall have been checked. From the calculations it can be concluded that the anchor capacity, overall stability and bearing capacity are sufficient. The bearing capacity of the FRP quay wall has been calculated for the point resistance only. The shaft resistance has been neglected since the installation technique that will be applied is a soil removing technique.

The joints of the FRP quay wall have been discussed theoretically. There are examples available for the connection between two Z-profiles. These examples of the connections are also applied for steel sheet profiles. The connection between the skin and web laminate is evaluated as a bonded joint. A mechanical joint will penetrate the laminates increasing the risk of moisture intrusion into the laminates which is not desirable due to the decrease of the mechanical properties. The design of a joint should be verified by testing and a numerical calculation with a FEM according to the design standards.

For the applicable installation technique an analysis has been performed. The installation techniques driving, vibrating, pressing and diaphragm wall have been discussed. With the program Allwave PDP a driving analysis has been performed for the FRP quay wall. The driving analysis with the program with the GEF file from the case study resulted in implausible results. A driving analysis with a GEF file from the 'Amazonehaven' resulted in more realistic values for the compression stress in the FRP quay wall. The maximum allowable compression stress is equal to 43 MPa.

The driving analysis showed a compression stress in the quay wall approximately equal to 34 MPa but there were also peaks of 101 MPa. Based on the driving analysis and literature it is advised to use the diaphragm wall installation technique. Vibrating and pressing of the quay wall 21.75 meters into the ground seemed not feasible as well based on the characteristics of both installation techniques.

The LCA of the steel combi wall and FRP quay wall shows that the FRP quay wall has a carbon footprint that is higher than the carbon footprint of the steel combi wall. The carbon footprint has been calculated for the entire lifetime of both structures. From the carbon footprint of the FRP quay wall it can be concluded that besides the manufacturing of elements, the construction phase has a significant contribution to the carbon footprint as well. This is caused by the bentonite that has to be used for the diaphragm wall installation technique. Besides the carbon footprint the environmental impact of both structures has been calculated as well. The environmental impact has been normalized with so called 'shadow prices'. The normalization of the environmental impact results in a price for the structures. The environmental impact has been calculated for the applied materials only. The environmental impact shows that the FRP quay wall has a higher impact than the steel combi wall. The LCA has been performed for one running meter quay wall.

The FRP quay wall is also compared to the steel combi wall based on the costs of the structures. The costs for the structures has been calculated for 1,0 meter quay wall. The costs for the steel combi wall are retrieved from the case study. It is assumed that the costs from the case study have a 20% margin. The costs for the FRP quay wall has been calculated with data found in literature. The margin of this data is assumed to be 50%. From the cost estimation for both structures it can be concluded that the FRP quay wall requires a larger investment than the steel combi wall. The cost of 1,0 meter quay wall constructed out of steel is equal to €5.500,- while a running meter FRP quay wall roughly costs €68.000,-. The difference between the two materials is a factor 12.

Even if the price of the FRP quay wall could be reduced with 50% it would still be 6 times more expensive than the steel combi wall. Another argument in favour of FRP is that it does not require a lot of maintenance during its lifetime. In the case study the corrosion of the steel combi wall is incorporated in the design. In the case study it is stated that the corrosion of the steel tubular pile will be 0.90 millimetre in 50 years. The remaining maintenance is the replacement of fenders and mooring facilities. This maintenance will also have to be done for the FRP quay wall. It is therefore not likely that the FRP quay wall will close the investment gap with the steel combi wall based on the maintenance assumption of the materials itself.

12. Recommendation

12.1 Chapter content

During the research I have encountered several problems. The unsolved problems will be discussed in this chapter. This chapter will also provide a critical view on this feasibility study as well as on related subjects.

12.2 Standards and guidelines

The 2003 version of CUR96 is only applicable for glass fibre reinforcement. The other types of reinforcement, carbon and polyaramid, are not included in the scope of CUR96.

It is therefore advised that the FRP experts, companies and factories join forces to create a guideline that can be applied to hydraulic structures in general and does include all the possible reinforcement and resins.

12.3 The case study

During my research I encountered several uncertainties within the design of the case study. So was the governing deformation of steel combi wall equal to approximately 60 millimetres. In itself this is not a strange result. However, the governing deflection was caused by the exceptional load case of an impact load and occurred at the top of the quay wall into the soil mass. So the quay wall deformed 60 millimetres into the soil mass.

Another point of the case study that does set of the alarm bells is the design of the anchor. From the anchor stability check it can be concluded that the anchors are heavily over dimensioned.

Despite these uncertainties within the case study the design of the FRP quay wall has not been affected by these uncertainties.

It is recommended that more feasibility studies will be performed for the application of FRP within the field of hydraulic engineering. These feasibility studies should not only focus on the design itself but also on a cost estimation as well as an LCA. The subjects of these studies should cover all possible structures within the field of hydraulic engineering. It is recommended to perform multiple feasibility studies for each subject so that they can be compared to one another.

12.4 Cooperation

During my research I encountered that FRP specialised companies and experts are not particularly willing to cooperate. On the one hand it is understandable that a company does not share all the ins and outs of its specialization. The material FRP is new in the field of hydraulic engineering and if you have an advantage on your competitors due to your specific expertise it is likely that you will be contracted for FRP related projects.

On the other hand, this research is a feasibility study to an FRP quay wall. These type of researches and studies might open a new sector of the market if it turns out that FRP is an excellent alternative for steel and concrete. One cannot expect that on beforehand a client such as IGR can tell whether or not quay walls will be constructed out of FRP in the nearby future.

So to all experts and companies specialized in FRP, I would like to recommend you all to be willing to guide students in the future. It even might open new markets for your product or specialization.

12.5 Life Cycle Analysis

Concerning the LCA I would like to recommend the following:

- Create one specific database that contains all materials and processes.
- Create a standard format for how the LCA should be executed.

Regarding the recommendation concerning one specific database I would like to point out the following; as mentioned in chapter 8: Life Cycle Analysis there are many different databases and programs that are being used to perform LCA. The data in the databased are all based on different information leading to a diversion of the data for the same object or material. It is therefore recommended to use one database so that the results from every LCA can be compared to one another.

12.6 Connections with FRP

As stated in chapter 6: Joints, the design of connections for FRP materials should be validated with testing and a numerical calculation with a FEM. The connections of two consecutive Z-profiles as well as the connection between the skin and web laminate has only be discussed quantitatively in this feasibility study. It is therefore recommended that the connections will be studied qualitatively.

FiberCore Europe has a patented technique for the connection between the skin and web laminates. They produce the lamellae in such a way that they run from the upper skin, through the web into the lower skin resulting in a continuous connection between the skin and web laminate.

12.7 Installation technique

The diaphragm wall installation technique is assumed to be the most suitable alternative for the FRP quay wall. This assumption is only based on a driving analysis from Allwave PDP and literature. Further research is recommended to determine which installation technique is the most suitable option for an FRP quay wall.

12.8 Dynamic behaviour of FRP

All load combinations in this feasibility study have been considered static. However, the exceptional loading case of an impact load is in fact a dynamical loading. How the FRP behaves under a dynamical loading is not known from literature and it is therefore recommended to conduct research into this matter. A fellow student is currently performing research to the behaviour of an FRP lock gate when this is loaded by collision with a vessel.

12.9 Optimization of the design

The evaluated design that has been obtained in this feasibility study is not an optimized design. With several design cycles the design of the FRP quay wall must be optimized. The influence of an additive as mentioned in 10.8 Additives must be included in the optimization. Important point for the optimization is the design standard that has to be used with the strain criteria.

Bibliography

Books

Bergschenhoek. (2016). *ROwat beschoeiingen en Damwanden*. Bergschenhoek.

Clarke, J. (2005). *Structural Design of Polymer Composites.* England: Taylor & Francis e-library.

CUR & Havenbedrijf der Gemeente Rotterdam & Gemeentewerken Rotterdam. (2003). *Handboek Kademuren.* Gouda: Stichting CUR.

CUR. (2003). *Vezelversterkte kunststoffen in civiele draagconstructies, Achtergrondrapport bij CUR-aanbeveling 96.* Gouda. ISBN 90 3760 352 1.

Hartsuijker, C. (2008). *Toegepaste Mechanica, Deel 2, Spanningen, vervormingen, verplaatsingen.* Den Haag: Sdu Uitgevers.

Kolstein, M. H. (2008). *Fibre Reinforced Polymer (FRP) Structures.* Delft: Delft University of Technology.

Nijhof, A. H. J. (2004). *Vezelversterkte Kunststoffen–Mechanica en ontwerp*. Delft: VSSD.

Nijssen, R. P. L. (2013). *Composieten Basiskennis.* 2nd print. Delft: Hogeschool Inholland.

Tol, A. F., van & Everts, H. J. (2006}. *Damwandconstructies*. Delft: Delft University of Technology.

Internet

ArcelorMittal. (2016). *Products, Z Sections*. Retrieved September 9, 2016, from: http://sheetpiling.arcelormittal.com/

Bodemrichtlijn. (2017). Visited on January 1, 2017. http://www.bodemrichtlijn.nl/

Cripps, D. (2016). *Guide to Composites*. Retrieved January 11, 2016, from: http://www.netcomposites.com/

Dutch Water Sector. (2016). *Deltares puts vertical composite sea wall to the test in world's biggest wave flume.* Visited on December 28, 2016. http://www.dutchwatersector.com/

Isolatie-weetjes. (2017). Visited on January 1, 2017. http://www.isolatie-weetjes.nl/

NIBE. (2016). Visited on December 15, 2016. http://www.nibe.org/

Rijksinstituut voor Volksgezondheid en Milieu. (2016). Retrieved December 15, 2016 from: http://www.rivm.nl/

Tissink. A. (2012). *Project gesmeerd damwand drukken met bentoniet.* Publication date April 6th 2012. http://www.cobouw.nl/

PDF files

ArcelorMittal, (2004). *Steel sheet piles, installation*. Retrieved December 20, 2016, from: http://sheetpiling.arcelormittal.com/

ArcelorMittal, (2008). *Cold formed sheet piles, Edition 2008.* Retrieved December 20, 2016, from: https://jsteeluat-public.sharepoint.com/

Atlantic Coast Engineering. (2012). *Summary of dynamic testing & driveability analysis of superpiles*. United States, Virginia beach. Provided by fellow student R. Winter.

CUR. (2003). *Aanbeveling 96, Vezelversterkte kunststoffen in civiele draagconstructies.* Provided by W. Schutte of IGR.

CUR. (2016). *CUR-AANBEVELING 96, Vezelversterkte kunststoffen in bouwkundige en civieltechnische draagconstructies.* 1e herziene uitgave. Provided by W. Schutte of IGR.

Report

Ingenieursbureau Gemeente Rotterdam. (2010). *CO_2 Footprint Kademuren, De uitstoot van CO_2 bij de bouw en onderhoud van kademuren.* Rotterdam.

Study

Valk, R. E. A., van. (2016). *Feasibility study for a composite quay wall, literature study.*

List of figures

List of tables