Flexible Quay Wall Structures for Container Vessels

MSc Thesis

Faculty of Civil Engineering and Geo Sciences
Department: Hydraulic Engineering, Ports and Waterways

By Paul Buring
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TU Delft Delft University of Technology

Royal HaskoningDHV Enhancing Society Together

Lifting Global Trade APM TERMINALS

Master Thesis

Flexible Quay Wall Structures for Container Vessels

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Colophon

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Preface

This report is the result of the graduation thesis written by Paul Buring, student of the Delft University of Technology. The thesis is written in order to receive the title Master of Science in Civil Engineering, track Hydraulic Engineering. For a specialization in the field of Ports and Waterways research for the application of flexible quay wall structures for container vessels was carried out in cooperation with Royal HaskoningDHV and APM Terminals.

This study is carried out under supervision of the committee members of the Delft University of Technology, Royal HaskoningDHV and APM Terminals, which are listed on the previous page of this report. I would like to thank all members of the committee for their support and guidance during my master thesis and the companies of Royal HaskoningDHV and APM Terminals for their cooperation. I would also like to thank all Royal HaskoningDHV colleagues, who assisted me during my thesis as well. Last but not leased I would like to thank my lovely girlfriend, family and friends for their support during my entire study career.

I hope the reader of this report may find this thesis interesting and useful. I hope that the contents shall be used for further research and shall eventually lead to the construction of a flexible quay wall structure in reality.

Paul Buring
Delft, March 2013

Summary

Handling of container vessels requires an advanced and expensive quay wall structure. These structures are often very well anchored in the ground and are very hard to reuse or relocate. Flexibility of quay wall structures for container vessels is therefore very low, but in practice it turns out that flexibility of these quays is desirable in some cases. Traditional quay walls that become inadequate or useless are often upgraded, demolished or wasted in the ground. Upgrading is usually done by building a new structure in front of the existing quay. By upgrading a quay it can be utilized again, but upgrading is still very costly, although cheaper than constructing an entire new quay wall.

First of all, one can distinguish two different types of flexibility. On the one hand side there is flexibility at a fixed location, which means that the structure can be adapted to changes in retaining height, increased crane loads and new quay dimensions. On the other hand side there is flexibility over different locations, which means that the structure can be mobilized and transported to a different location, when it becomes inadequate or useless at its original location. This thesis focusses on flexibility over different locations.

The technical lifespan of a quay wall structure is in the order of 50 years, but in practice they often become inadequate or useless far before the end of their lifespan. The main reasons for this are changes in the container shipping market, new generations of container vessels and equipment, and conflicts or war.
A new generation of container vessels may require a larger retaining height, because of the increased draught. Flexibility at a fixed location seems therefore more desirable for this issue, but it can also drive the desire to relocate a quay for this reason. The vessels that used the original quay wall remain operational, but the demand for these vessels shifts to different ports.

The objective of this master thesis is to design a flexible quay wall structure for container vessels that can be transported and rebuilt within a shorter construction period, compared to traditional quay wall structures. A faster construction period results in a quicker start of making revenues, which is an important factor that influences the financial feasibility of the flexible quay wall. A higher residual value and a reduced environmental impact are other factors that have a positive influence on the financial feasibility comparison with traditional container quays, since the initial construction costs of the flexible structure are higher. The largest advantage of the flexible structure however, is that it can be utilized during its entire lifespan of 50 years.

When a quay wall can be constructed within a short construction period, it could possibly also serve for different purposes. One could for instance consider using such a structure as a temporary quay wall during maintenance at existing quay walls or after a natural disaster, when many supplies are needed and local ports are damaged.
It could also be used as a so-called trial quay to give terminal operators the opportunity to explore the possibilities at a certain location. Based on their experience they can decide to construct their own structure or to leave the location, after the trail period at the flexible quay wall.

The investment risk of investing in a fixed quay wall structure plays an import role in the final decision making to start construction, especially when the continuity of a port is doubtful. A flexible structure could therefore be a realistic and effective solution.
A brief market demand analysis was carried out in cooperation with APM Terminals. It turned out that there could certainly be a demand for such a flexible structure, but the real demand basically depends on its financial feasibility compared to a traditional quay wall.

The structure has a retaining height of 22 meters and is not particularly designed for a certain location, since it must be applicable at many different locations.
The flexible container quay is designed to handle two panamax vessels with a maximum capacity of 5.000 TEU at a time and creates just enough space to accommodate ship-to-shore (STS) cranes with a rail span of 30,5 meters. The entire quay length is 700m and consists of two berths. No space for container storage is available on the quay, so it offers space of STS cranes only. Other equipment will ride on the terminal pavement, which is directly connected to the caissons. Container handling is done by STS cranes in combination with ordinary trucks and Rubber Tired Gantry (RTG) cranes. The required number of equipment is estimated by means of the so-called queuing theory.

In part III of this report, 11 possible alternatives for the design of a flexible quay wall structure for container vessels are described. Five basic alternatives, being a caisson quay wall, a sheet piled combi-wall, a mass concrete block-work wall, a deck on piles and a floating quay wall were assessed by means of a multi criteria analysis, without making too many calculations yet.
The caissons quay wall turned out to be most promising, but a floating quay scored very well too. Six new alternatives were established, based on the conclusions of the multi criteria analysis, being a ballasted floating quay, a hydraulic jack-up quay, a floating quay with spud piles, an immersed deck on piles, caissons placed on top of each other and a pre-tensioned floating quay, which is pulled partly under water.
In contradiction to the five basic alternatives, many calculations are carried out to investigate the behavior and technical feasibility of the six new alternatives. An overview of the calculations made, can be found in table 19.2 on page 83.
Calculations showed that stability of a floating quay cannot be guaranteed and motions are very likely to exceed the PIANC guidelines for efficient handling of container vessels. Efficient handling and good operability are important criteria for a container terminal. To keep the flexible quay comparable to a stable traditional quay, it’s decides not to use a flexible quay wall that floats during operation.

After analyzing the advantaged and disadvantages of all designed alternatives, it is decides to use a flexible quay wall structure, as illustrated in figure I.I.

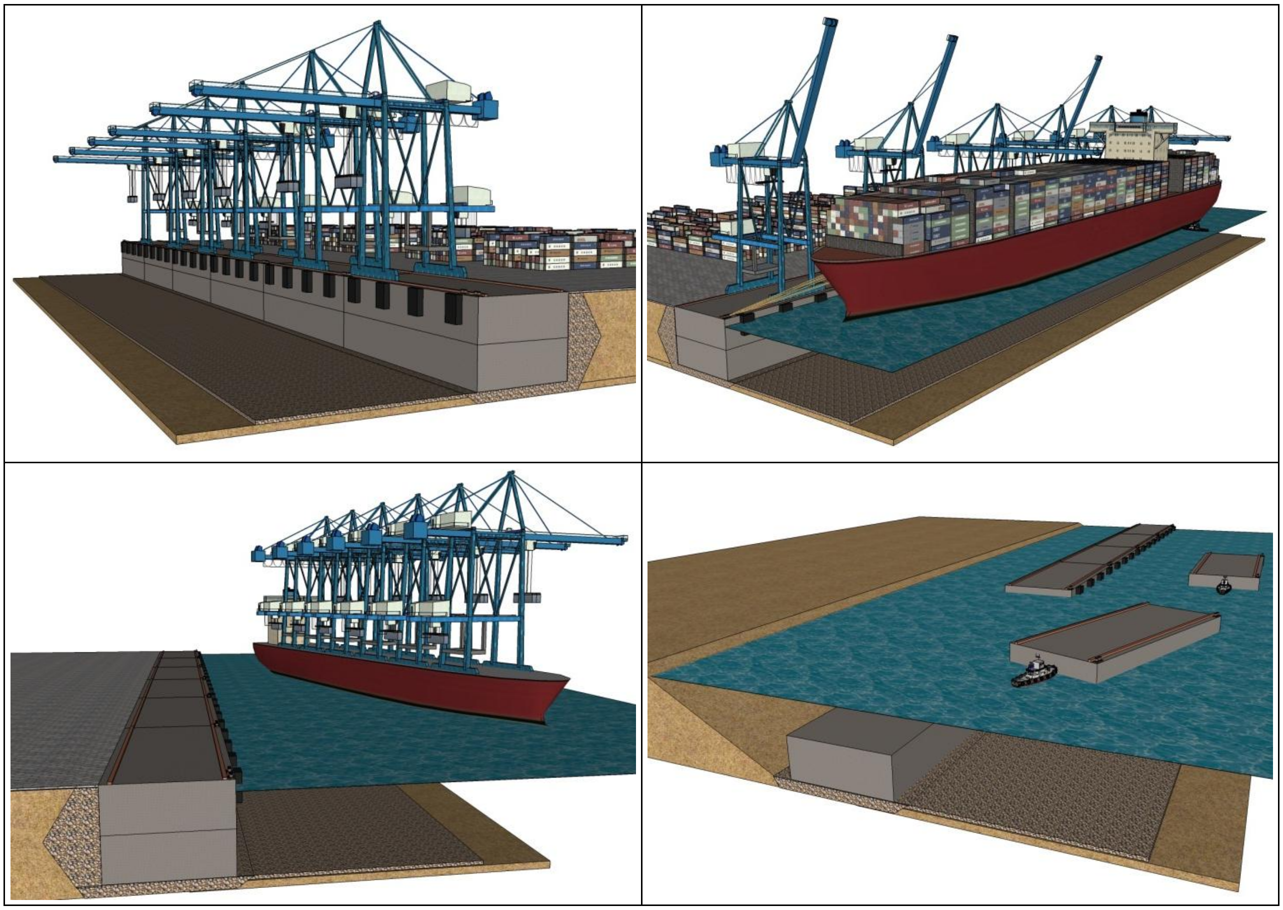


Figure I.I – illustrations of proposed flexible quay wall structure

The two berths quay wall consists of 14 caissons of each 100m in length, 33m wide and 11m high. Two caissons are placed on top of each other to reduce the height and the draught. The caissons are designed with 4 inner walls in longitudinal direction and 12 inner walls in cross direction. The quay is a gravity structure, so stability against sliding, overturning and bearing capacity is satisfied by means of mass and dimensions only.

The final unity check values for satisfaction of the failure mechanisms are: 0,95 for both sliding and bearing capacity and 0,77 for overturning. The empty caissons have a metacenter height of 15,57m in floating conditions and 14,87 during immersion. The minor difference is a result of the large number of inner walls. The natural floating oscillation period for roll motions was determined at 5,58 seconds and the structure requires a minimum bearing capacity of 400kN/m^2 under the bottom slab. In the worst case scenario, it can resist an earthquake acceleration of at least 0,2g, but more ballast can be applied to achieve a higher earthquake resistance.
A more detailed description of the structural design and the acting loads can be found in part IV of this report and the calculated results can be found in the appendix.

There are no connections between the caissons and the lower caissons are filled with ballast water, which can be pumped out to make them floating again. The upper caissons are ballasted with sand to create more weight. Transportation towards a new destination is done by tug boats for each caisson and a gravel bed is required to create a permeable foundation and a solid support. Dredging equipment is therefore needed to execute preparation works, before the elements can be immersed.

A financial feasibility study was carried out, where a comparison is made between a traditional quay wall and the flexible quay wall structure. The flexible structure is utilized during its entire lifespan of 50 years, but needs to be relocated within this period for a variable number of times. The traditional quay wall is non-flexible and will be utilized for the same period as the flexible quay at one location, with some difference because of the difference in (re)construction period. After this period, a new traditional quay wall has to be constructed at the new location.

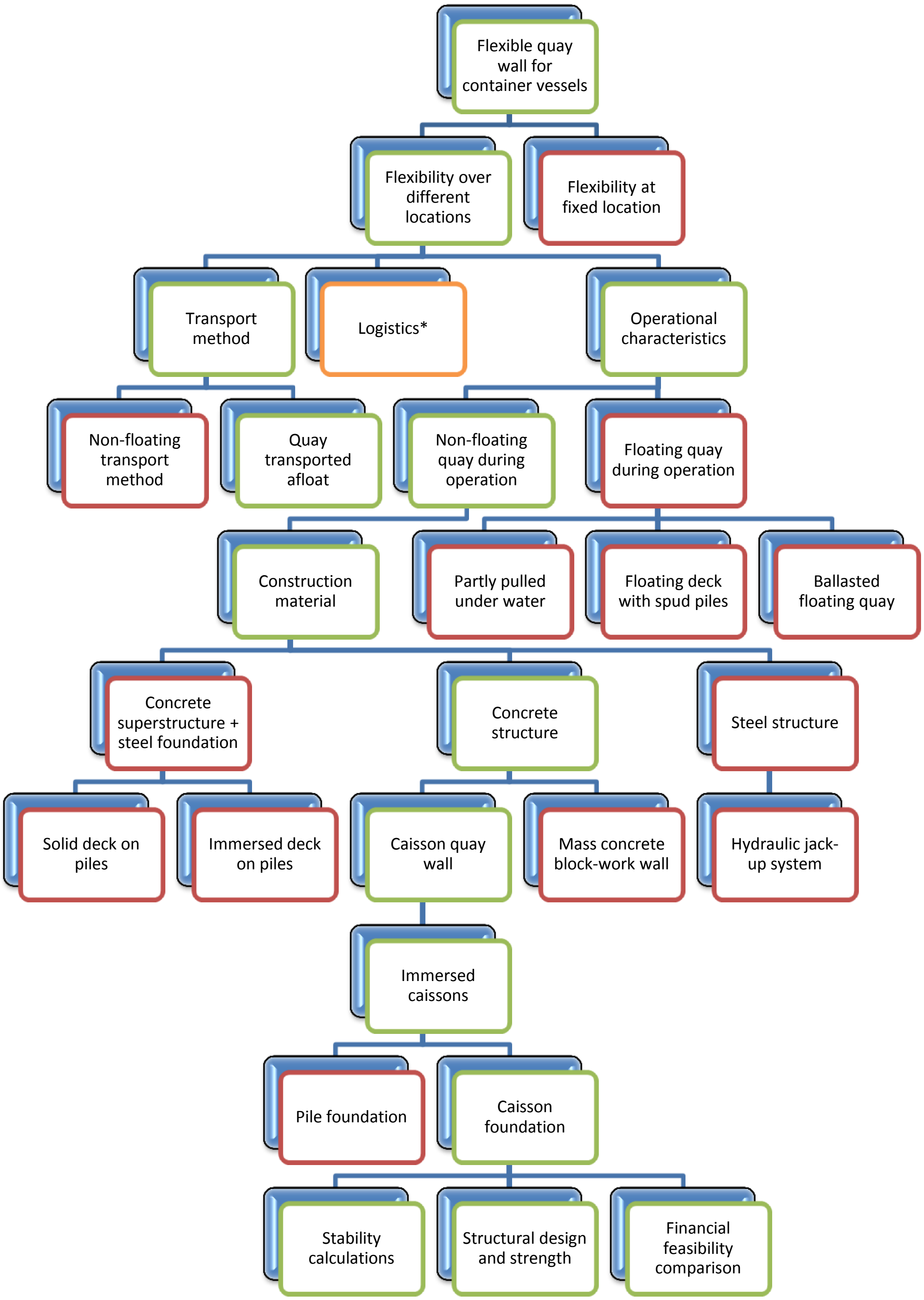
The relocation frequency has a large influence on the financial feasibility of the flexible quay wall. This parameter was therefore taken variable and the results with respect to financial feasibility were plotted as an over-all costs balance in time. The reduced reconstruction time and the reduced reconstruction costs of the flexible quay wall, turn out to have the most beneficial influence on the comparison with a traditional fixed quay. The biggest issue for using this structure in reality is the initial construction costs, which are about 64% higher than the average construction costs of a traditional quay wall that is constructed in situ. The relocation costs however, are about 28% less expensive than the construction of a new traditional quay wall.

It can be concluded from the financial feasibility comparison, that the proposed structure could be financially feasible for different scenarios. When the structure is used by the same owner during its entire lifespan of 50 years, it should at least be relocated twice, in order to make it financially more attractive than using three traditional quay wall structures. When the flexible structure is sold after a certain period, it can already be sold after using it at one single location, to make it financially feasible. The structure could probably even be a feasible alternative to constructing a quay wall in situ and upgrading it after a certain period. Although the required residual value is about 3,5% higher than the construction costs of a new traditional quay wall, it can be operational within a short period and it has a larger residual value.
Graphs of the results from the financial feasibility study can be found in figures 31.1 to 31.3 in part V of this report.

Designing the most suitable type of flexible quay wall structure was an important part of this master thesis, which took quite some time. A schematized overview of the design procedure is given in figure I.II on the next page. After selecting the caisson quay wall as most suitable, this structure was further elaborated with respect to stability, structural design and financial feasibility.

This graduation topic has a very wide range of aspects to be analyzed before construction in reality could be possible. Therefore, some aspects are beyond the scope of this work or should be elaborated into more detail. At the end of this report an overview of recommendations for further research on this topic can be found.

A schematized overview of the steps in the design procedure is given in figure I.II.



* = basic logistics of handling equipment by means of queuing theory only

Figure I.II – schematization of steps in design procedure

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1. Introduction

The handling of container vessels requires an advanced quay wall structure, which is a large investment. Flexibility of these types of structures often remains a problem. Flexibility of container quays can be divided into flexibility at a fixed location and flexibility over different locations. Considerations with respect to future deepening of the harbor basin, larger crane loads and sea level rise refer to flexibility at a fixed location, whereas the other reasons drive the desire to relocate a quay wall at a different place. This thesis will focus on flexibility over different locations and not on flexibility at a fixed location. Motivations for this choice can be found in paragraph 6.10.

Note that larger vessels and container cranes can also drive the desire to relocate a quay wall, since inadequate quays on a certain location can still suffice in a different port. For example, 7.000 TEU container vessels were the world's biggest 15 years ago, only calling at major ports in the world. Nowadays, these vessels also call at various medium size ports all over the world. This explains that there is still a demand for quay facilities for these vessels, but the demand as shifted from the world's largest ports, towards smaller medium size container ports.

Quay wall structures are large and heavy and are often very well anchored in the soil. Moving and reconstructing such a structure at a different location is therefore often impossible. However, in some cases it might be desirable to move the container quay to a new location.

Fast growing markets could for instance be a reason to move a flexible quay walls structure to a different location. With such a structure it is possible to react faster in response of new growing markets. A flexible quay wall structure can be transported in parts or as a whole, which makes the construction time in the port relatively short. Another advantage of a short construction time is a short period of possible traffic hindrance, due to construction in the port. Besides, constructing a quay wall faster, results in a quicker start of making revenues.

Changes in the existing market could drive the desire to relocate a quay wall as well. Especially the continuity of transshipment hub ports is sensitive to price variations. These are ports at intersecting shipping routes, where containers are moved from one vessel to another. Competition of other hub ports may lead to a shift towards another location. With a flexible quay wall structure, one can respond faster to these changes.
Transfer of container terminals to countries with lower wages is another issue that causes changes in the container shipping market. Many container terminals moved to China in the past couple of years. However, shifts to countries with lower wages like Bangladesh and Cambodia can be observed nowadays.
Another aspect that could cause changes in the existing shipping market is the transfer of production processes to different countries.

International conflicts and wars could also be a reason to relocate a quay wall. It is unattractive to call at ports in war zones and conflicts may lead to blockage or closure of shipping routes.

A flexible quay wall could also offer a solution in case of maintenance or adaptations to existing quay walls. Here it can serve as a temporary quay. Such a temporary structure can also be applied in case of natural disasters or large events. Many supplies are needed after a natural disaster, but local ports may be severely damaged or may have insufficient capacity. Constructing a traditional quay wall is not an option since it takes too much time. A flexible quay wall that can be operational within a short period would be a good solution. Such a quay can also be applied when a large amount of extra goods has to be transshipped during a short period of time. This is for instance the case around large events like the Olympic Games.

Possibilities for transport routes along the Northern coast of Russia are still being researched. If this route becomes navigable in the summer period, flexible quay wall structures could be attractive to serve as a seasonal solution.
Quay wall structures are usually designed for a lifespan of approximately 50 years and must become profitable after 8 – 15 years. Construction of such a structure is therefore a long term investment that needs to be well considered. When investors have doubts about investment in a new container terminal, a flexible quay wall could offer a solution. Clients can hire a flexible structure for a certain trial period to explore the local possibilities. Based on their experience they can either decide to invest in their own structure after the trial period, or leave the location because of disappointing business.

A flexible quay wall structure is likely to be more expensive than a traditional quay, but because of its flexibility, it can be utilized very well during the entire lifespan. The investment risk is therefore lower, compared to traditional quay walls. Traditional quays often become inadequate before the end of their lifespan, because of changes in the container market, vessel size and handling equipment.

Container vessels and handling equipment are becoming bigger and bigger, but traditional fixed quay walls are hard to modify to new requirements. Deepening of the basin in front of a quay wall is often not possible, because stability of the structure becomes a problem due to the increase in retaining height. Larger container cranes might be required to handle bigger vessels, but these cranes are often too heavy. The structure cannot cope with the extra load, leading to instable situations. This also hampers the flexibility of traditional quay walls.
A new structure is often built in front of the existing one when such a structure doesn't allow deepening. Removing or adapting the old quay wall is difficult and expensive and therefore it is often left in the ground. This old structure will not serve as a wall anymore, but it still has a positive contribution in decreasing the loads on the new structure. Upgrading an inadequate quay by constructing a new quay in front of it is therefore cheaper than construction an entire new quay wall.

Removing quay wall elements and reconstructing them at a different location could also be an option. However, elements are large and heavy which makes reconstruction expensive and unfeasible, especially for large retaining heights.

Predictions of sea level rise vary, but a certain sea level rise is expected in the future. The freeboard of existing quays may become insufficient or quays may even be flooded depending on the degree of sea level rise. Raising the quay seems to be an obvious solution, but this is not always possible, since it leads to similar stability problems as for increasing the water depth.

Table 1.1 summarizes the main reasons which may drive the desire to relocate a quay wall. It makes a distinction between high and low frequencies of relocation, where high is assumed to be more than once in two years and low is assumed to be less than that.
A more detailed market demand analysis is given in chapter 5.

Possible reasons for relocation of a container quay, summarized	***Relocation Frequency***
Changes in container shipping market	Low
New booming markets	Low
Shift to a different transshipment hub	Low
Shift to countries with lower wages	Low
Production processes move to different countries	Low
Conflicts and war	Unknown
Quay becomes insufficient for new generation of vessels	Low
Seasonal shipping route along the Northern coast of Russia	High
Temporary quay during maintenance at existing quay	High
Trial quay to explore possibilities at new locations	High
Sea level rise	Low
Temporary quay after natural disaster	High
Temporary quay during large international events	High

Table 1.1 – possible reasons for relocation of a container quay

A selection of the most relevant reasons used for the design of the flexible quay wall structure can be found in chapter 7.10, where assumptions for the frequency of relocation are made.

In 2011, Royal HaskoningDHV has organized a brainstorm session in response of problem definitions provided by APM Terminals and the Port of Rotterdam. Students of the Delft University of Technology have tried to design preliminary solutions for flexible quay wall structures during this meeting.
In response to this brainstorm session a graduation topic on flexible quay wall structures became available within Royal HaskoningDHV. In this graduation thesis various types of flexible quay wall structures are judged on the basis of criteria such as; flexibility, costs, stability, and construction period.

The structure that appears to be most suitable will be elaborated in part IV of this report. The more detailed design of the construction will focus mainly on loads, strength, stability, construction, and costs.

2. Problem Definition

As stated in the introduction, flexibility of traditional quay wall structures for container vessels can be problematic in some cases, since they are often hard to relocated and adjust. However, relocation of such a structure could be desirable in some cases, as mentioned in table 1.1 and in the introduction.

Traditional fixed quay walls are designed for a lifespan of approximately 50 year, but hardly ever reach this age, since they often become inadequate on their location much earlier. Instead, a flexible quay wall structure can be utilized during its entire lifetime, because it can be transported to a new location.
When a flexible structure can be rebuilt at a different location within a relatively short construction period, one can also react faster to changes in the market and start making revenues earlier.

In this thesis it will be investigated what kind of structure is most suitable with respect to flexibility over different locations. The design will be elaborated and a feasibility study is done to determine whether it is an attractive alternative or not. The construction costs of such a flexible structure are likely to be higher than those of a traditional quay, but the utilized lifespan can be much longer.

To summarize the problem definition in one sentence:
Traditional quay wall structures for container vessels are often very hard to reuse and relocate. However, relocation could be desirable for various reasons.

3. Objectives

The objective of this master thesis is to design a flexible quay wall structure for container vessels, which can be transported to a different location. In the first place it can be used to provide a response to changes in the container shipping market. Other reasons that may require relocation of a quay wall can be found in table 1.1 and each of them is described in the introduction.

The structure will not be designed for a specific location, since it must be possible to locate the quay at a wide range of ports, all over the world. General boundary conditions and requirements for the flexible quay are therefore selected and established.

Several possible types of flexible quay wall structures are being established and judged on various criteria. The structure that turns out to be the most promising alternative will be further elaborated with respect to failure mechanisms, structural design and costs. The number and type of container handling equipment will also be determined, by means of the queuing theory and corresponding waiting times.

Flexibility with respect to future deepening of the harbor basin and heavier handling equipment will not be taken into account during the design of the quay wall structure, because this thesis mainly focusses on flexibility over different locations. However, as described in the introduction, inadequacy for new generations of vessels and equipment can also drive the desire to relocate a container quay.

A financial feasibility study will also be part of this thesis. The costs for construction and relocation of the flexible structure are being compared to construction costs and periods of traditional quay walls. Based on these costs, a conclusion will be drawn on the financial feasibility of reusing a flexible structure instead of building a new quay wall. Relocation frequencies have a severe influence on the feasibility. Therefore, it will be investigated which frequencies are feasible.

In the end it can be concluded if such a flexible quay wall structure is a feasible alternative to building new structures with a shorter lifespan than the flexible structure.

To summarize the objectives of this thesis in one sentence:
Designing a reusable quay wall structure for container vessels, which can be rebuilt at a different location, within a short reconstruction period compared to the construction period of a traditional quay wall.

4. Structure of the report

This report is divided into six parts and starts with a summary and an introduction to the graduation topic and a market demand analysis.

The first part of the report consists of a short summary of previous research and master theses on relevant topics.
The requirements and boundary conditions for the quay wall are established in Part II. Besides, it includes results of the queuing theory, applied to the selected type of handling equipment.
In Part III a number of possible flexible quay wall structures are designed and assessed on multiple criteria.
Part IV describes the structural design of the quay wall type that appears to be the most suitable alternative and checks all failure mechanisms of the structure.
In Part V a financial feasibility study is carried out, were relocation of the flexible quay is compared to building a new fixed quay wall.
Part VI summarizes the conclusions and recommendations of this master thesis and the final part consists of several appendices about various topics.

5. Market demand analysis

In order to investigate the market demand for a flexible quay wall structure, a brief market demand analysis is carried out in cooperation with APM Terminals. The mentioned possible reasons which could drive the desire to relocate a quay wall structure are discussed on relevance and expected level of occurrence. Some examples of situations in which relocation is desirable are described in this chapter. The outcome is summarized in table 5.1.

5.1. Recouping the construction costs

A very import aspect that determines the market demand for a flexible structure are the costs and benefits involved. A flexible type of structure is likely to be more expensive in construction costs, so one should research opportunities to recoup this investment. The two main possibilities to realize this are a significantly reduced reconstruction period in the port and the residual value of the quay wall components themselves.
If (re)constructing a flexible container quay saves one or two years compared to the construction period of a traditional quay wall, one starts to make revenues much earlier. A flexible structure also has a much larger residual value after serving a certain terminal for a couple of years, since it can be fully reused at a new location. This is another serious advantage compared to traditional quays. As mentioned in the introduction, traditional quays are usually either upgraded or wasted in the ground. In fact, they might even cost additional money for complete demolition.
Whether the saved construction time and the residual value of a flexible quay wall can balance the increased construction costs or not, will be investigated in part V of this report.

Another advantage of reusing a flexible structure is its positive influence on the environmental impact, because the use of raw materials and polluting equipment is likely to be less, compared to upgrading or constructing a few traditional quay walls.

5.2. Real life examples of demand for flexibility

Figure 5.1 shows that 15 years ago 7.000 TEU container vessels were the biggest is the world, only serving the world's major container ports. Quay facilities that were designed for these vessels are completely inadequate to accommodate today's largest vessels, since their capacity is more than twice as large. However, those 7.000 TEU vessels are still operational nowadays, since they are just 15 years old. These vessels are now being handled at various large and medium size ports all over the world, whereas they were only handled at the world's largest ports in 1997. This example clarifies that quay walls will become inadequate for new generations of container vessels sooner or later. However, they can still be used at a different place, since the vessels themselves will remain operational.

Emma Maersk is with about 15.000 TEU, currently the largest operational container vessel, which now only calls at the largest ports in the world, including the Port of Rotterdam. This vessel will not be the largest vessel in 2030 and probably also calls at medium sized container ports, by then. In that case it might become interesting to sell a flexible quay wall structure that accommodates Emma Maersk in for instance Rotterdam, to medium size surrounding container terminals like those in Felixstowe, Le Havre or Zeebrugge. These ports might still have a demand for vessels like Emma Maersk, whereas the Port of Rotterdam is probably interested in larger vessels by that time. Note that figure 5.1 was published in 2008 and that 18.000 TEU vessels are expected to be operational in 2014.

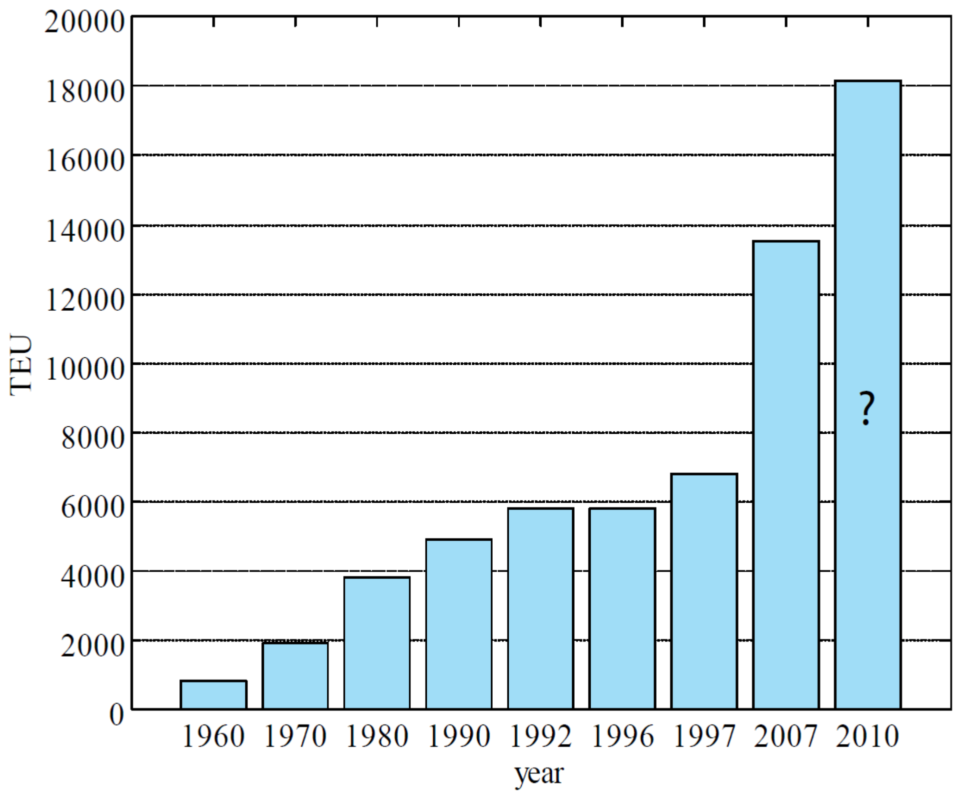


Figure 5.1 – container vessel evolution [A History of Quay Walls, J.G. de Gijt (2008)]

Changes in the container shipping market, as already mentioned in the introduction, turn out to occur in reality quite often. Quay wall operations basically depend on the clients of their clients. Manufacturers of export products determine the shipping routes of container vessels and these shipping routes determine the required location of container terminals. In practice it occurs frequently that manufactures move to countries with lower wages. Consequently, the shipping companies and terminal operators follow them sooner or later. Terminal operators can respond faster to these changes with a flexible quay wall and the downtime of their revenues is shorter, because of the faster reconstruction period.

The investment risk of investing in a fixed container quay plays a very important role in the final decision to start construction at a certain place. When investors have doubts about the continuity of a port, construction of a fixed quay is risky, since the residual value is very low compared to the investment. A flexible structure would make the decision for construction much easier, because the terminal operator can reuse the structure or sell it, when the revenues turn out to be disappointing.

A flexible quay wall also offers the possibility to explore the market by means of a trial quay. Terminal operators could rent such a trial quay to experience the opportunities in new markets themselves. Depending on the results, investors can either decide to invest in their own structure or to leave the location. Such a system could work very well, since the investment risk and uncertainties of market assumptions play a very import role in decision making in reality.

Leaving a country because of war is a realistic scenario, but moving towards a country after a war also turns out to be very attractive. Urban life and infrastructure are usually severely damaged after a war and hence many supplies are needed. Ports are a strategic target during war and are therefore often seriously damaged. The market demand for a quay wall that can be constructed in a very short period is high in these cases. The same principle yields in case of a natural disaster like a devastating earthquake.

The demand for a temporary quay during maintenance or upgrading of an existing quay seems a realistic scenario, but the operational period of the temporary structure is probably too short to make a flexible quay feasible for this purpose. The transport costs and reconstruction costs are probably too high, when the quay is used for just a couple of months. In part V of this report, it will be investigated which transport frequencies and travel distances can be financially feasible.

Using a flexible quay wall structure for a seasonal shipping route along the Northern coast of Russia is not assumed to be a very likely scenario. Although possibilities of using this route are still under investigation, ice loads and frozen waterways are problematic. The frequency of moving the structure twice a year is also likely to be too high to make it feasible. The market demand for this purpose is therefore nil.

Table 5.1 gives an overview of the possible reasons which could drive the market demand for a flexible quay wall structure. The term “likely” only states that there could be a market demand for this reason. It does not state that a flexible quay is likely to be feasible for this purpose. A feasibility study is carried out in a later stage of this thesis to judge about this issue.

Possible reasons for relocation of a container quay, summarized	***Expected Occurrence and Demand***	***Relocation Frequency***
Changes in container shipping market	Likely	Low
Conflicts and war	Likely	Unknown
Quay becomes insufficient for new generation of vessels	Likely	Low
Seasonal shipping route along Northern coast of Russia	Unlikely	High
Temporary quay during maintenance at existing quay	Likely	High
Trial quay to explore possibilities at new locations	Likely	High
Sea level rise	Scale Dependent	Once
Temporary quay after natural disaster	Likely	High
Temporary quay during large international events	Likely	High

Table 5.1 – results of market demand analysis

5.3. Conclusion of market demand analysis

After executing a basic market demand analysis, it can be concluded that there certainly is a demand of a flexible quay wall structure, but the real demand mainly depends on the financial feasibility compared to a traditional quay wall.
Whether such a flexible structure is financially feasible or not, will be determined in part V of this thesis.

Part I
Literature Review

6. Previous master theses on relevant topics

The next few paragraphs describe the scope of master theses on relevant graduation topics, which were carried out in the near past.

6.1. The container terminal of the future

In June 2004, C. Paus completed his master thesis of the so called container terminal of the future. Since it was written in the Dutch language, it can only be found on the internet with the title “De Container Terminal van de Toekomst”, the Dutch translation. Paus designed a floating steel pontoon, which can be used as an additional extra quay wall to reduce the handling time of large container vessels. The steel pontoon is self-propelled and can be moored along the vessel using a combination of mooring lines and electro magnets. The vessel can be handled from both sides, by means of this extra floating quay, which is moored along the vessel after the vessel itself is moored at the fixed quay wall. Mooring forces on the floating quay are not taken into account, since the fixed quay wall is assumed to resist these forces. The pontoon is not used for storage of containers, but for handling equipment only. A hinged ramp, supported by a separate pontoon connects the floating quay with the terminal area. Short descriptions about the various constructions are given, but strength calculations were beyond the scope of the thesis.

Several aspects are considered, with various solutions listed with both advantages and disadvantages, on which a conclusion was drawn. The main aspects are: alignment and dimensions of the quay, position keeping, ballast system, type of handling equipment, length of the crane boom, productivity, mobilization method and maneuverability.

In order to counteract the movements of the pontoon, Paus proposed a ballast system inside the floating quay. High capacity water pumps are installed in order to compensate tilting moments that are caused by crane movements the handling of containers.

Paus considered several possibilities for handling equipment, traffic lines and numbers of equipment, in order to reduce the handling time. Handling is done by rail mounted gantry cranes (RMG) and automatic guided vehicles (AGV). In his master thesis, he proved that application of the extra floating quay wall will reduced the handling time of 8.000 – 15.000 TEU vessel with 6,5 – 8 hours.

A detailed cost-benefit-analysis was made to research the financial feasibility of the structure. The extra investment costs are estimated at about 31 million euro and the extra operational cost at about 5 million per year. In order to make the structure feasible, Paus determined the required price per move. He concluded the an increase of 2,50 euro per move is necessary to recoup the extra costs. This seems to be an attractive solution, since the reduced handling time saves about 3,80 euro per move over the entire process.

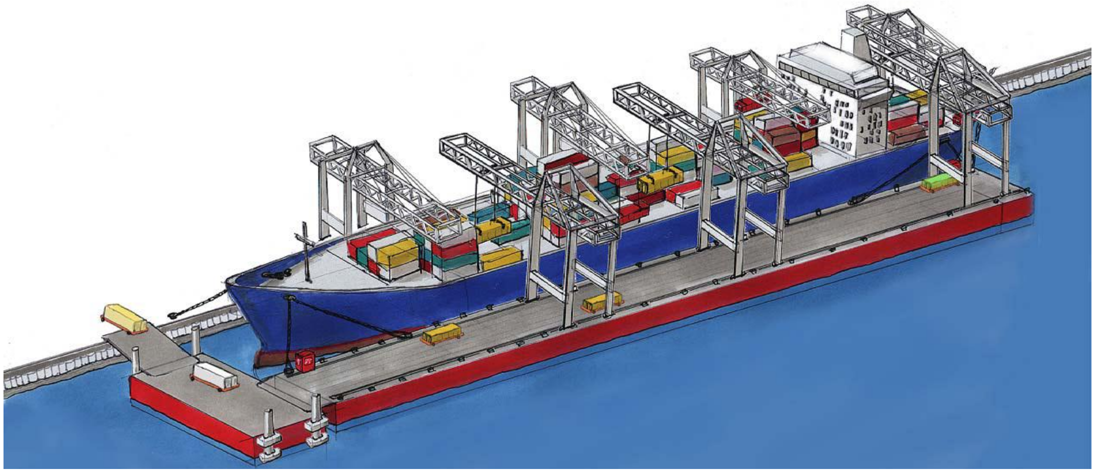


Figure 6.1 – container terminal of the future by C. Paus

6.2. Floating container transshipment terminal

A. Ali made a master thesis on a floating container transshipment terminal in January 2005. An offshore transshipment terminal with an annual throughput of one million TEU is designed at a distance of about 5 kilometers from the shoreline and a water depth of 100m is taken into account. The final design has a length of nearly 1200 meters and a width of 240m, but consists of several smaller concrete pontoons that together create a rigid body. The floating structure accommodates space for container storage as well and the required number of berths is derived using the queuing theory. Facilities and accommodations at the terminal are also taken into account and AGV's and RMG's are used as container handling equipment. A vertical wall around the structure is constructed to reduce wave overtopping.

The structure is located in an unprotected area and will therefore be exposed to severe waves during both service limit state (SLS) and ultimate limit state (ULS) conditions. A significant wave height of 10,25m is derived for the ULS and 3m for SLS. Main objective of the master thesis is to research the operability, with respect to motion tolerances of such a structure in unprotected waters. The offshore floating container terminal is considered to be a possible option for port expansion in case of lack of space.

Ali carried out detailed hydrodynamic computer modeling using the computer programs DELFRAC and SEAWAY. These models are used to determine the motions of the structure and the resulting wave heights at the lee side of floating terminal. This is an important part of the thesis, since the structure also functions as a floating breakwater for the moored vessels behind it. Detailed models are made to determine the response between the vessels and the terminal and the oscillation frequencies. These motions are compared to the allowable motions from PIANC guidelines to define the maximum operational period per year.

Ali proposed four different types of pontoons, of which a simple rectangular shaped concrete pontoon was selected to be most suitable. Several alignments for the berths are assessed as well. Berths in line were eventually chosen to be most effective.

Multiple systems for position keeping of the entire structure are described, of which a combined DP thrusters-turret mooring system was selected to be most suitable for the large water depth. This type of mooring system was also selected, because it allows the structure to rotate bow-on to the waves during high seas.

To determine the financial feasibility of the terminal, a cost-benefit-analysis is carried out. This analysis shows that the structure can only be feasible when transshipment fees are raised.

The absorption of forces by the structure is briefly illustrated, but strength calculations were beyond the scope of this master thesis.

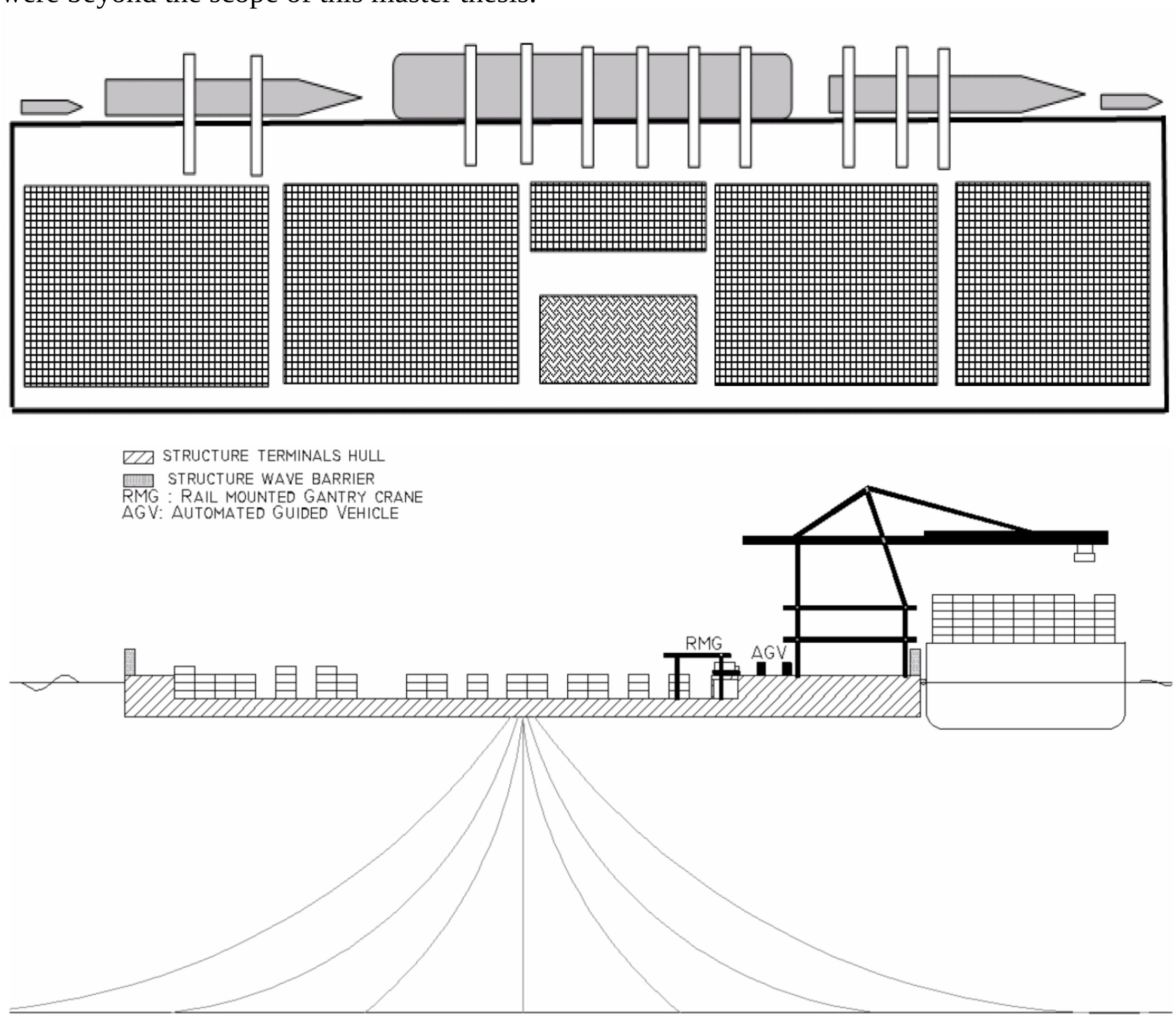


Figure 6.2 – floating container transshipment terminal by A. Ali

6.3. A very large floating container terminal

In April 2006, G.V.P de Rooij finished her master thesis on a very large floating container terminal as well. The Western Scheldt, in The Netherlands, is used as a case study to design a floating port expansion near the city of Vlissingen. Therefore, the design is best suitable for shallow water conditions with restricted wave and wind loads. The terminal consists of 120 pontoon shaped floating concrete elements that are rigidly connected together in a stretched bond configuration, in order to distribute the connection forces over the structure. Each single pontoon is 180m in length, 50m in width and 16m in height. The draught of the caissons can be controlled by ballast water, which has a positive effect on the stability. The entire terminal dimensions are 2100m in length and 500m in width and an access bridge creates the connection with the shore. The construction is kept in position by means of a large number of tubular piles with a sliding connection along the sides of the terminal.

De Rooij analyzed the hydrodynamic behavior of the entire rigid body using DELFRAC. The connection forces between the floating elements are determined carefully during exposure to the waves, by means of a spectral analysis from DELFRAC. The output of DELFRAC is translated into static loads, which are used to determine the required strength of the connections.

Special attention is paid to the constructive design of the structure. The pontoon elements are all pre-stressed using a post tensioning method in two directions and have a wall thickness of 800mm. A very detailed structural design is carried out for the connection between the elements. Several types are described, of which finally a vertical needle versus peg connection with discontinuous trapezoidal concrete studs was selected. Each stud is pre-stressed with 6 curved tendons in order to introduce the loads to the inner walls and the top and bottom slab. These tendons are anchored at a cantilever beam behind an inner wall. The connection is designed in such a way that pontoons can easily be disconnected if necessary.

Apart from the very detailed design of the connections, attention is also paid to the construction method and the types of pontoon structures, deck structures, quay dimensions, alignment of the floating elements, and stability.

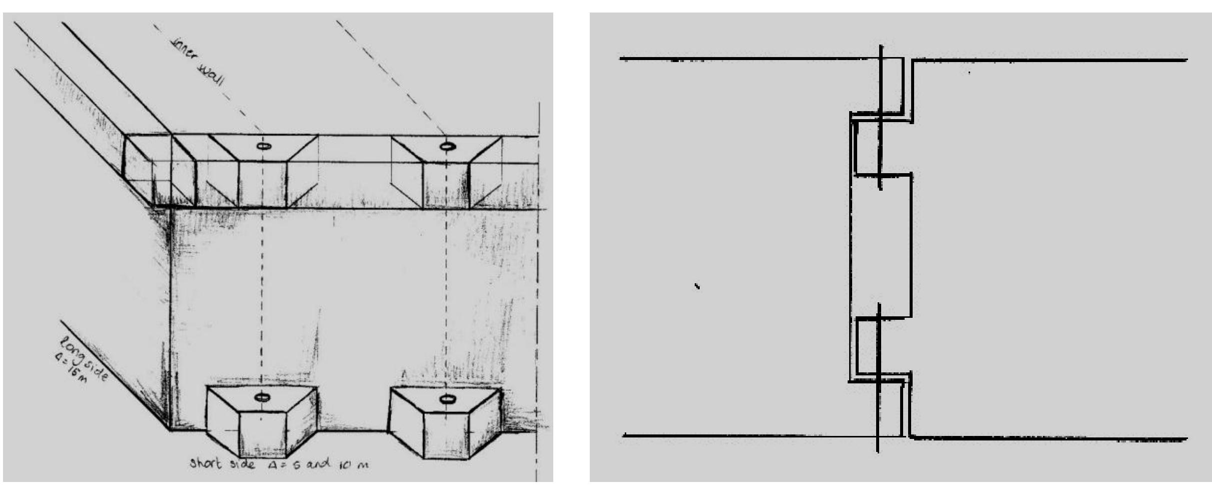


Figure 6.3 – connection method proposed by De Rooij

6.4. The use of a floating quay of container terminals

M. van der Wel researched possibilities for a floating quay wall for container vessels in 2010, in order to keep handling times of the largest container vessels below 24 hours. Two designs were made to accommodate the largest generation of vessels. One design consists of three concrete pontoons that are rigidly connected together. The other design consists of one single concrete pontoon of 480m long, 40,5m wide and 15,5m high. The floating quay wall doesn't have space to store containers and AGV's are used for container transport towards the stack.

The scope of this study mainly focusses on the static and dynamic stability of the structure and its hydrodynamic behavior. Again the hydrodynamic behavior is computed by schematizing the structure as a mass-spring-system in DELFRAC. The computer program delivers a value for the 6 degrees of freedom for quay motions being: yaw, pitch, roll, sway, surge, and heave as illustrated in figure 6.4.

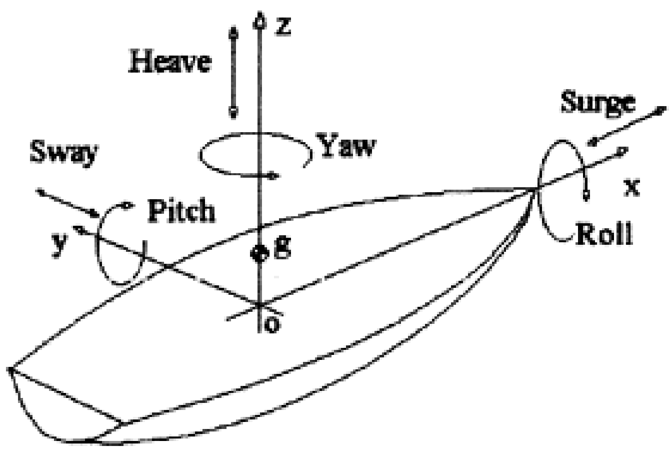


Figure 6.4 – six degrees of freedom for a floating structure [www.sciencedirect.com]

Equipment on the quay and handling of containers result in a tilting moment at the quay wall, which is calculated with the Scribanti method. Calculations show that rolling of the quay results in a vertical movement of 3m for containers that are lifted at the end of the crane booms. This movement is far above the maximum allowable movement of 1m, which is described by PIANC guidelines. The natural oscillation frequency of the quay wall was also determined by Van der Wel and was calculated at 0,70 rad/s.

After the investigation of the quay motions, it is concluded the handling of container vessels is impossible on the floating structure.

The reason why other MSc reports draw different conclusions is caused by the significant difference in width of quay wall, compared to the master theses of Ali and De Rooij. Calculations of Van der Wel show that effective container handling is impossible at a floating structure of only 40,5m wide.

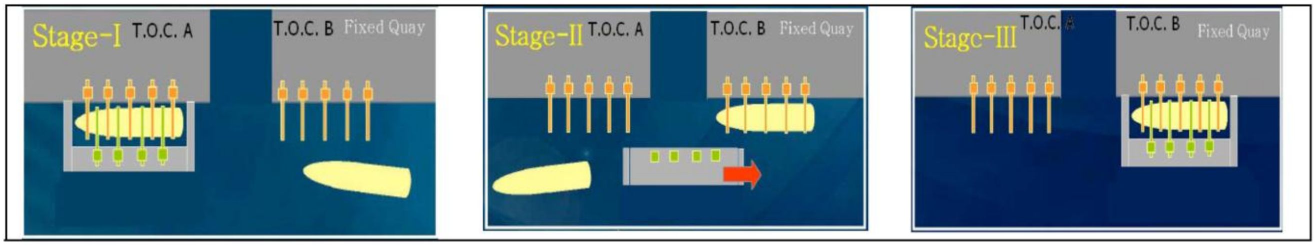


Figure 6.5 – floating quay wall for container terminals by Van der Wel

6.5. Connecting modular floating structures

M. Koekoek did a master thesis on the connections between modular floating structures in September 2010. The construction of floating structures is considered to be a possible solution in cities with lack of space. The thesis is written in order to graduate on a specialization in structural design and therefore mainly focusses on structural aspects. The design of a modular floating pavilion in an abandoned harbor for inland waterways in Rotterdam is taken as a case study. The pavilion of 46 x 24 meters has already been constructed, but was built as one piece. Koekoek researched the possibilities to construct the structure out of modular connected elements. Floating elements can be coupled and uncoupled by means of the connections, which makes a designed structure much more flexible with respect to future adaptations or relocation.

Especially the connection between the floating bodies is elaborated into detail, but the pavilion itself has been calculated on structural strength as well. A combination of EPS and concrete is found to be the best construction method for the floating structure. Connections can be used to restrict motions, but this will increase the internal forces. The other way around, allowing motions can decrease internal forces considerably.

Several different types of connections are mentioned and assessed on various criteria. A distinction is made between floating modular elements with intermediate spacing and without, but the thesis mainly describes a rigid connection for the later one. Research resulted in a connection that consists of trapezoidal ridges for self-alignment and shear force, a vertical pen as tension connector in de lower area and a longitudinal bolt on the top side. This bolt can also be used to pre-tension the connection and an elastic material is used in between the connection parts to provide damping and avoid small relative movements.

The report also describes the shortcomings of general building codes, when they are applied for floating buildings. Aspects like buoyancy force, waves, stability and required freeboard are not included in current building codes and are considered in the report of Koekoek. Natural oscillation periods are determined as well and seem to be in the same range as the wave period for single modular elements. However, connecting them rigidly together eliminates this problem.

The structure is design for a significant wave height of 1,5m and a wave period of nearly 12s, both estimated using the Bretschneider equations for wind wave development.

Figure 6.6 – floating modular pavilion by Koekoek

6.6. Design of a floating structure for container terminal activities

In January 2011, an interdisciplinary project on the design of a floating structure for container terminal activities in the Port of Rotterdam was carried out by E. Bijloo, J. Breukink, J. Donkers and C. Paquel. The scope of this study is limited since it is not a master thesis, but an interdisciplinary project. Furthermore, only the appendix of the report was available for this literature study and the report itself was nowhere to be found. The project mainly focusses on the logistic part of a floating container terminal, because the research is done for a specialization in transport and planning.

A SWOT analysis (Strength, Weakness, Opportunities and Threats) was done for both the Port of Rotterdam and a floating container terminal. A list of stakeholders was made including their interests, power and needs. A morphological chart was composed for aspects like transport to hinterland, handling equipment, distance to the shore, berth configuration, position keeping, mobilization of the terminal and dimensions. The proposed options are assessed by means of a multi criteria analysis and a sensitivity analysis, resulting in the final design.

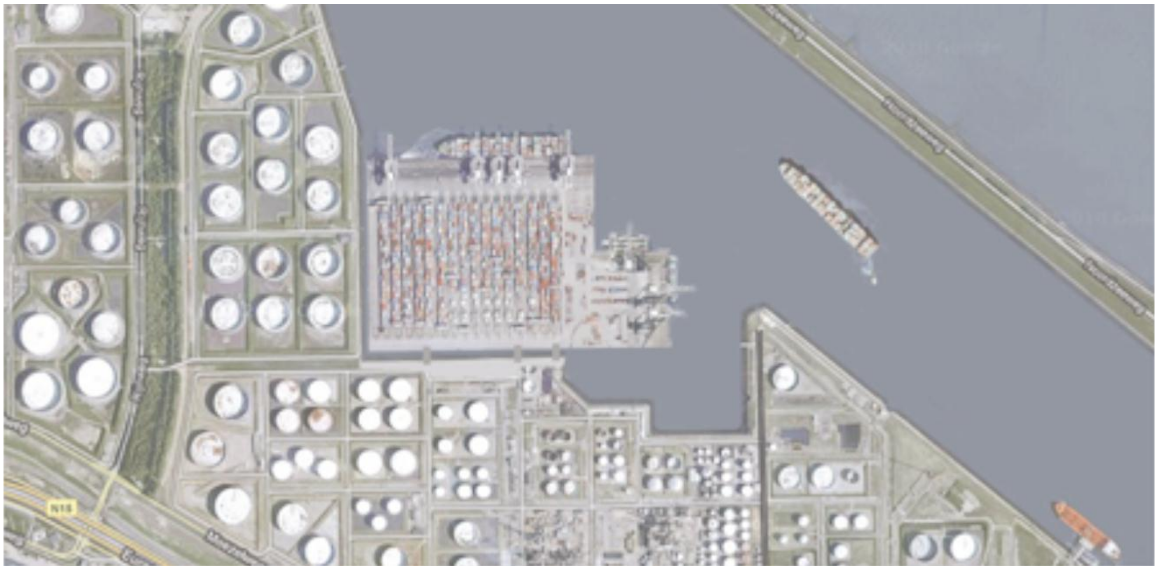


Figure 6.7 – floating structure for container terminals activities by E. Bijloo, J. Breukink, J. Donkers and C. Paquel

6.7. The floating construction method

In December 2011, R. Hendriksen graduated on the so called floating construction method. This master thesis was done for a specification in hydraulic structures and doesn't consider a quay wall structure. It describes the construction method of an underwater parking garage at the Oosterdok in Amsterdam, without the use of a construction pit or a dry dock. Previous research on this structure concluded that such a parking garage was financially unfeasible because of the high construction cost. Hendriksen researched a different construction method in order to reduce the construction costs and make the realization feasible.

The concrete floor of the parking garage is casted on a floating flexabase floor consisting of fiber glass reinforced EPS, which has sufficient buoyancy to carry the weight of the concrete floor. The parking place is nearly 130 long, about 75 meters wide, has two parking decks and offers space for 750 cars.

The parking garage still floats after construction of the walls and bottom and top slabs, and is finally immersed by means of ballast tanks. Once the structure is positioned on its foundation, grout anchors are drilled trough the bottom slab into the subsoil. The grout anchors absorb all tensile stresses, when the ballast tanks are emptied, keeping the entire structure submerged.

Since this master thesis was carried out to graduate for the specialization in hydraulic structures, the scope mainly focusses on the structural part. The new construction method seems to reduce the construction costs, compared to the traditional construction method, making it technically and financially feasible.

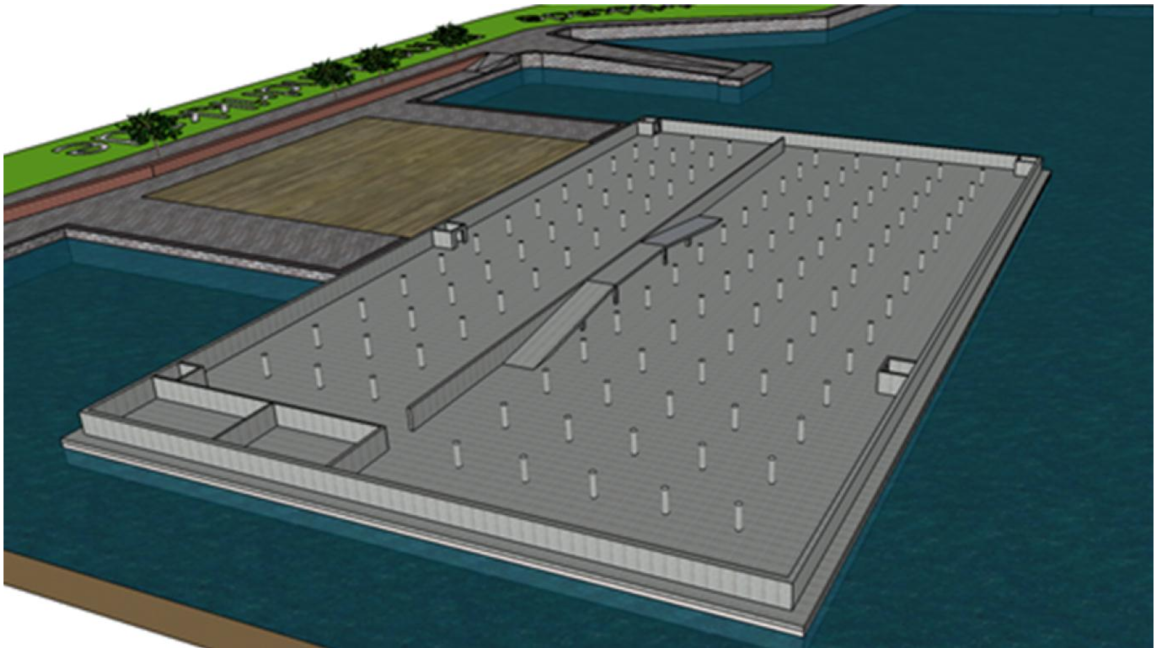


Figure 6.8 – floating construction method by R. Hendriksen

6.8. Floating quay applications for container terminals

In April 2012, W. Pradityo finished his master thesis on the design of a floating quay wall for a container terminal in Manuas, Brazil. This city is located 1450km landward in the confluence of the Rio Negro and the Rio Solimões in the Amazon basin. Due to seasonal differences there is a water level variation of 15m between high and low river discharges, which seems to make a floating quay a sound solution.

Pradityo considered several different solutions for various aspects such as berth configuration, types of pontoon structures, position keeping of the quay, handling equipment, connection with terminal area, construction method, connection between pontoon elements and environmental impact. Each possible solution is assessed by means of either a multiple criteria analysis, or a comparison of advantages and disadvantages, resulting in the final design.

The quay is designed to handle panamax vessels with a capacity of about 5.000 TEU and a call size of 2.000 TEU, so no large modern container carriers. Equipment is selected in such a way that motions of the floating quay stay within the allowable tolerances. In order to reduce to tilting moment, slewing cranes are used instead of heavy and fast handling STS cranes. The quay wall consists of rigidly connected concrete pontoons and is kept in position by means of heavy anchors on fiber rope anchor lines. Two access ramps to the terminal area are supported by floating pontoons and an expansion joint is used to deal with the water level difference.

A structural analysis of the pontoon structure was made after investigation of the main load combinations acting on the structure. The required bar reinforcement and pre-stressed tendons are determined in case of a concrete structure and the material thickness is estimated in case of a steel structure. For the later, deformation of the deck appeared to be problematic for effective operability of the quay.

The final part of Pradityo's master thesis consists of a financial feasibility study in which a comparison is made between a steel and a concrete structure. Prices for construction, maintenance, demolition and operational cost are taken into account, finally resulting in a concrete structure. Distinction is made between short and long term scenarios and attention is also paid to possibilities to reuse the structure for other purposes. A jetty structure on piles is calculated to be more financially feasible for a short term scenario, whereas the concrete floating structure is favorable for the long term scenario.

6.9. Conclusions regarding literature study

After studying previous master theses by other students, one can conclude that research on floating quay wall structures for container terminals is not completely new. The feasibility of a large floating container terminal that also accommodates space for the container storage yard, handling equipment and logistical processes has already been studied by Ali and De Rooij. Ali designed it for deep water at a distance of about 5 km off the shoreline and mainly focused on the motions of the terminal and the moored vessels in exposed wave conditions. De Rooij researched the feasibility of such a structure in protected shallow water conditions and elaborated the connections between the pontoon elements. Both master theses were meant to research new methods for port expansion when space on land is not available or scarce.

Koekoek did a detailed study on the connection between floating elements as well, but his thesis considers a floating pavilion and not a quay wall. Hendriksen elaborated a new floating construction method to save the investment in a construction pit or a dry dock and the interdisciplinary project by Bijloo, Breukink, Donkers and Paquel focusses on the field of transport and logistics of the floating container terminal activities.

Van der Wel, Paus and Pradityo considered a floating quay wall structure that only accommodates handling equipment and doesn't offer space for container storage. Consequently, the width of these structures is considerably smaller than those studied by Ali and De Rooij. Pradityo offered an attractive solution at a location with 15m of water level differences, whereas Paus and Van der Wel designed an additional mobile quay to handle vessels from both sides, in order to reduce the handling time.

The study of Pradityo proved that handling of medium size container vessels is possible with a slender floating structure using medium size handling equipment in an inland port in the Amazon basin. However, Van der Wel proved that effective handling of the largest container vessels is impossible by means of such a slender floating quay wall. Motion tolerances of container cranes are small and motions due to container lifting only, already exceed the PIANC guidelines amply. Paus designed a stabilization system by means of high capacity ballast water pumps inside the floating quay. First estimates suggest that the ballasted motions are of the same magnitude as the maximum allowable motions, but not all sources of motion are taken into account. The structure proposed by Paus was made of steel, whereas all other designs resulted in a concrete structure.

6.10. Opportunities for new research

Flexibility of quay walls can be split into two different types of flexibility. On the one hand side, flexibility over different locations. This means that the structure can be transported and reconstructed at a different location. On the other hand side, flexibility at a fixed location, which means that the quay wall can be adapted to changes in a later stage after construction.

Adaptations on the retaining height of quay walls are often desirable, because of the continuously increased dimensions of container vessels over the past decades. Detailed research on flexibility with respect to future deepening of the harbor basin and increased loads on a quay wall structure has therefore been carried out before. A quay wall that is designed for a certain retaining height simply doesn't suffice anymore for a larger retaining height. Some retaining structures were designed to make future deepening possible, but their feasibility is very doubtful. In practice it turns out to be cheaper to upgrade the existing quay wall by building a new structure in front of it. The old structure is usually left in the ground, since demolition costs are too high and reuse is financially unfeasible. Nevertheless, the old structure does not become completely useless, since it reduces the load on the new structure that is built in front. Upgrading an existing quay by constructing a new one in front of it is therefore cheaper than building a new quay in normal soil.
Solutions for flexibility at a fixed location have been studied by many experts before, but the alternatives to solve this problem turned out to be very limited.

During this master thesis, the focus will therefore be on flexibility over different locations, instead of flexibility at a fixed location. However, in the preliminary design stage, flexibility at location will be taken into account as well, trying to design a structure in which both types of flexibility can be combined. If a combination of both flexibilities turns out to be impossible or unfeasible, only flexibility over different locations will be further investigated.

Since this master thesis is about designing a flexible quay wall structure, the design doesn't necessarily have to consist of a floating structure. However, discussions between different groups of students during the so called Denkfabriek about this topic already concluded that a floating structure might be an attractive solution. Whether this is indeed the case, shall be further investigated in a later stage of this report.

Floating structures are flexible in the sense that they could possibly be moved to a different location, but none of the previous master theses designed a structure for this specific purpose. Ali and De Rooij considered an entire container terminal, whereas this master thesis is restricted to the design of only the quay wall itself. The width of the structure is therefore likely to be in the same order of magnitude as the quay walls considered by Paus, Van der Wel and Pradityo.

Analyzing and optimizing the logistical processes around a floating container terminal has already been done by Bijloo, Breukink, Donkers and Paquel. Besides, the logistical aspects are less relevant, since this thesis is written for the track hydraulic engineering and not for the track transport and planning.

The problems that were faced by Van der Wel and Paus offer a possible field of new research. It could be interesting to study the possibilities of stable flexible quay wall structures for large container vessels. Van der Wel already proved that a floating structure doesn't satisfy the tolerances with respect to quay motions and Paus also faced uncertainties in dynamic behavior and quay motions. Research can therefore be done on stabilizing techniques for floating structures, but another type of non-floating flexible structure could offer a new solution as well. Analyzing the behavior of floating structures in waves and resulting connection forces between modular elements is less interesting, since this has already been done by Ali and De Rooij.

When designing a quay wall structure, one cannot ignore the structural design, since the required strength is closely related to things like wall thickness, weight and allowable spans in the structure. A general structural analysis of the quay wall structure will therefore be part of this master thesis. All other failure mechanisms of the quay wall will be investigated and the final design should satisfy these criteria.

Finally a cost comparison analysis is carried out in order to investigate whether the flexible quay wall structure is a feasible alternative for building new quays or not. Relocation frequencies play a very important role in this.

Part II
Requirements, Boundary Conditions & Equipment

7. Requirements

7.1. Terminal lay-out

The flexible quay wall structure will be part of the container terminal and will not function as container storage yard. This means that in case of a floating structure, the width is mainly determined by the span of the ship-to-shore crane rails and some additional space for traffic lines and mooring facilities. A reduced width has a positive effect on the construction cost, but has a negative influence on floating stability. For a soil retaining structure, the width is somewhat less, since the terminal pavement can be used for accommodating traffic lines.

7.2. Vessel size

Panamax vessels are the largest vessels that will be handled at the flexible quay wall structure and have a length of 294m and a maximum capacity of about 5.000 TEU.

Vessels in this category are already large, but the latest generations are considerably larger. Emma Maersk is the largest operational vessel at the moment with a length of 397m and a capacity of about 15.000 TEU. Maersk is currently developing its triple E-class generation, which is 400m in length. Despite the minor increase in length, this new generation can carry up to 18.000 TEU. Vessels with a 22.000 TEU capacity are being designed at the moment, but are not operational yet.

The quay wall structure is not designed for the latest generation of container vessels, since these vessels only call at the largest container ports in the world. Continuity of these ports is assumed to be stable, so the demand for a flexible quay that can be moved to a different place is low. Panamax vessels are still quite large, but call at many ports all over the world. The demand for a flexible type of quay wall is therefore higher. Having many ports that handle these vessels also reduces the expected transport distance of relocation.

Figure 7.1 – panamax vessel "Tokyo Express" [commons.wikimedia.org]

Figure 7.2 – Emma Maersk, world's largest operational container vessel in 2012 [www.navegando.eu]

The typical dimensions of panamax vessels are given in table 7.1.

Panamax Vessel Properties	
Length	294,13 m
Beam	32,31 m
Draught*	12,04 m
Max capacity	5.000 TEU

** = Maximum allowable draught to pass the Panama Canal lock complex. The draught of a panamax vessel can be somewhat larger at maximum cargo capacity.*

Table 7.1 – typical dimensions panamax vessels [Wikipedia / www.hapag-lloyd.com]

7.3. Throughput quantity and call sizes

The quay wall is designed for an annual throughput of 1.000.000 TEU per year. Due to some exceedance of the serviceability limit state conditions, an operational period of 8.400 hours per year is assumed. This equals 350 days of 24 working hours, so 15 non-operational days per year. An average call size of 1.500 TEU is taken into account, which is delivered by approximately 667 container vessels per year. A TEU-factor of 1,6 is taken into account, since cargo consists of both TEU and FEU.

7.4. Handling equipment

Ship-to-shore cranes (STS) will be used to move the containers from ship to shore, with an average net productivity of 30 moves per hour. This type of crane is very common for container terminals, but is not appropriate to transfer containers directly to the storage yard.

Figure 7.3 – ship-to-shore crane [en.wikipedia.org]

For container transport between the quay apron and the container stack, one has several options. A simple tractor-trailer system can for instance be used in combination with a reach stacker. The STS crane places the container on a trailer, which is transported towards the storage yard, where it is unloaded by a reach stacker. This method is relatively cheap, but requires quite some maneuvering space for the reach stacker. Another disadvantage is the fact that they cannot be used in each process.

Figure 7.4 – tractor-trailer & reach stacker system [www.forkliftactions.com]

Straddle carriers could also be used for handling of containers within the storage yard. A smaller maneuvering area is needed in comparison with a reach stacker, and handling can be done quite accurately. A disadvantage is the need for qualified operators and the limited stacking height. The costs of a straddle carrier system are higher than a tractor-trailer system in combination with a reach stacker. An advantage of the straddle carrier is the fact that they can handle containers directly for the yard towards the vessel and vice versa. In case of transport by trucks, an additional type of equipment is required to (un)load the trucks at the storage yard. Storage on chassis could also be applied, but this needs an enormous storage space.

Figure 7.5 – straddle carrier [www.forkliftactions.com]

The most efficient method for container storage at the yard is a Rail Mounted Gantry crane (RMG). This method requires trucks as well, because the RMG cannot be directly (un)loaded by a STS crane. An RMG has a high productivity and is capable to handle a high stacking height. The required terminal area is therefore minimal. A stacking height of five to six containers is usually considered as a maximum with respect to efficiency. Storing them higher will lead to a reduced productivity, since a lot of handling maneuvers are required to reach the containers at the bottom.
A Rubber Tired Gantry crane (RTG) is basically the same piece of equipment, but does not need steel rails, since it drives on rubber tires.

Container transport between a STS and a RMG can either be done by trucks or by Automatic Guided Vehicles (AGV). Since this equipment is automatically controlled, the labor costs are very low. Instead, the equipment itself is very expensive and works complicated. Maintenance costs are also high and AGV’s need a large driving area. Therefore, this method is assumed to be less interesting for the flexible quay wall structure.

RTG’s in combination with ordinary manned trucks are assumed to be the most suitable handling method for container transport between storage area and vessel. This system is very flexible, because all handling equipment in the storage yard drives on rubber tires. Using RTG’s saves a lot of preparation works on the storage yard compared to the rail mounted RMG. However, due to the lower production some more pieces of equipment are required to achieve to same productivity.

Figure 7.6 – AGV and RMG system [www.flickriver.com]

7.5. Number of berths and ship-to-shore cranes

The required number of berths and STS cranes can be roughly estimated with a simple equation that is mentioned below.

$$N = \frac{C_s}{C_b}$$

Equation 7.1

Where: N = number of berths at the quay [-]
C_s = annual throughput [TEU/year]
C_b = berth production [TEU/berth/year]

$$C_b = p \cdot f \cdot N_b \cdot t_n \cdot m_b$$

Equation 7.2

Where: p = crane production [moves/hour]
f = TEU-factor [-]
N_b = average number of cranes in use during operation [-]
t_n = operational hours [hours/year]
m_b = first estimate of utilization rate of the quay [-]

According to these two equations, four STS cranes are needed to suffice with two berths, using a utilization rate of 35% as first estimate. The queuing theory is a more accurate calculation method, which will be used to investigate whether four STS cranes are sufficient or not. This will be done in paragraph 6 of this chapter.

7.6. Queuing theory

7.6.1. Number of berths and ship-to-shore cranes

Whether two berths and four STS cranes will indeed suffice with respect to waiting times of vessels and handling times can be determined by means of the so-called queuing theory.

It is assumed that average waiting times for container vessels may not exceed 1 hour. Based on the average call size of 1.500 TEU and the average handling speed of the STS cranes, an average service time of nearly 10 hours can be estimated. This includes about 2 hours for berthing and departure. The waiting times found by the queuing theory may therefore not exceed 10 % is the service time, which is less than 1 hour.

Vessel inter-arrival times are assumed to have a negative exponential distribution and service times are assumed to be E_2-distributed. The berth utilization rate can be calculated by dividing the annual throughput by the maximum possible annual throughput in case of 100% utilization. This results in a utilization rate of 31%. According to the queuing theory, the expected waiting time for container vessels will mount to 8,3% of their service time in case of two berths, which equals 49 minutes. This satisfies the criterion for the waiting times, so two berths with each four STS cranes can be applied.

7.6.2. Number of trucks

A similar kind of calculation can be carried out for the expected time that a STS crane has to wait for a truck to arrive. The trucks are assumed to do net 8 moves per hour, which is much less compared to the STS cranes. The inter arrival times and service times are both assumed to be E_2-distributed and the maximum allowable average waiting time of a STS crane is set to 5 seconds. Using the $E_2/E_2/n$ table of the queuing theory, one finds a total number of 6 trucks to be applied to each STS crane. This corresponds with a utilization rate of 63% and an average maximum waiting time of 5 seconds, which satisfies the criterion. Using only 5 trucks results in an average maximum waiting time of 20 seconds for the STS cranes, so 6 is the minimum number to be used.

7.6.3. Number of RTG

It was determined in the previous paragraph that 6 trucks are needed to serve one STS crane in order to limit the waiting time, for a truck to arrive. The number of required RTG can be calculated in a similar way, to avoid truck delays when the RTG's are occupied. This is important, since truck delay leads to STS crane delay.

Each RTG is assumed to have a handling speed of net 16 moves per hour and again both service and arrival times are assumed to be E_2-distributed. The maximum average waiting time of a truck for a RTG to arrive is assumed to be 15 seconds. This time is more than the waiting time of a STS crane. Although it is part of the same cycle, there are much more trucks than STS cranes. Therefore, a longer waiting time is allowed. When determining the utilization rate of the RTG, one must not forget to reduce the truck production by their utilization rate as well.

For practical reasons it would be a good idea to allocate a number of RTG's to only one STS crane, so to 6 trucks. However, the queuing theory shows that 5 RTG per STS crane are required for this system, which seems rather much. When a gang of two STS cranes and 12 corresponding trucks is allocated to a certain number of RTG's, it turns out that 7 RTG's per gang are sufficient. This means a reduction of 12 RTG's for the entire quay. Allocating a larger number of STS cranes and trucks to a certain amount of RTG's, doesn't decrease the required number of RTG much further. Even if all STS cranes and trucks of one berth are allocated to a certain number of RTG's, it turns out that still 10 RTG per berth are required. This is caused by the logistical chaos and the reduced productivity. Applying such a system is therefore often not a good idea.

Using the system illustrated in figure 7.7 leads to 7 RTG's per gang of 2 STS cranes and 12 trucks. The corresponding maximum average waiting time of a truck, for a RTG to arrive was determined at 11 seconds according to the queuing theory.

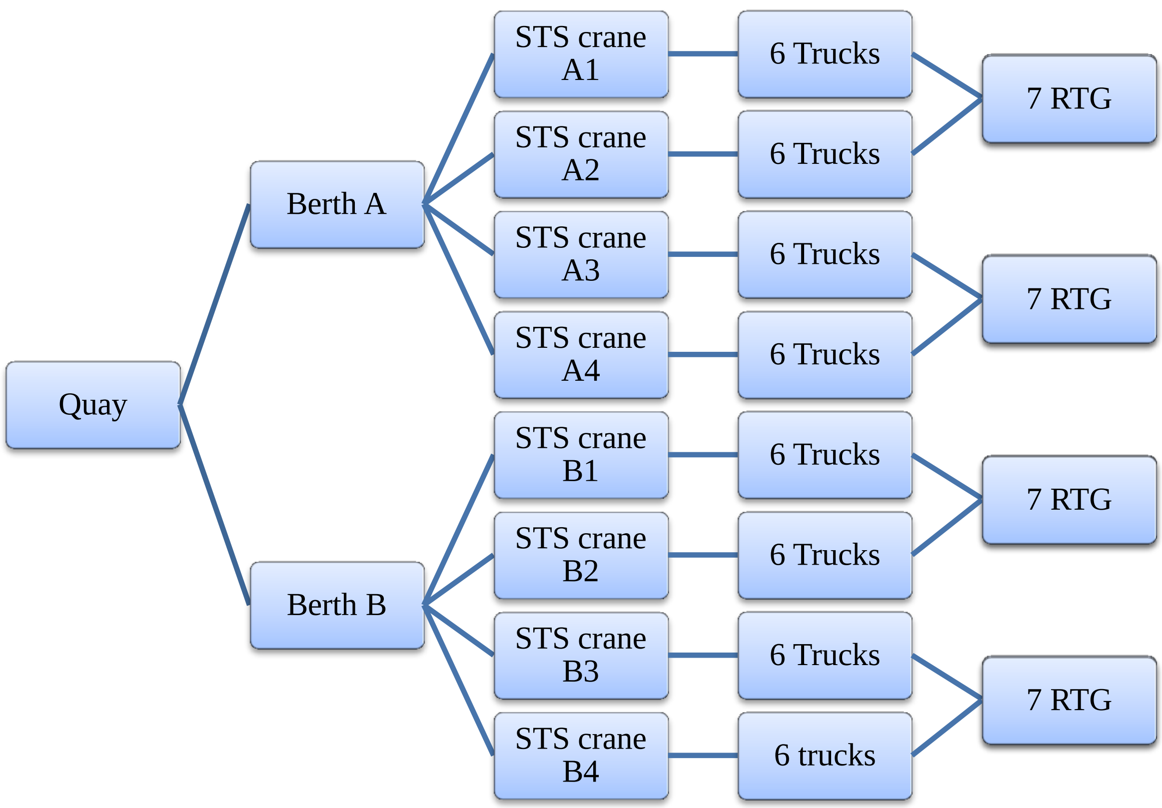


Figure 7.7 – handling logistics at the quay wall

After checking the first estimates by means of the queuing theory, a total number of 2 berths, 8 STS cranes, 48 trucks and 28 RTG were determined to serve the container terminal. Note that it would be a wise idea to have a couple of spare trucks, in case of maintenance or breakdowns.

A more detailed analysis of the waiting times could be obtained by means of computer simulation programs, but this is beyond the scope of this thesis. The results of the queuing theory are assumed to provide a sufficient realistic estimate.

Calculations and tables of the queuing theory can be found in appendix A, B and C.

Also note that vessels are kept up to date on the occupancy of a berth in reality. Arriving vessels reduce their sailing speed when a quay is still occupied and try to arrive when the quay is available.

7.7. Dimensions of the container quay

The length of the quay wall can be determined from the length of the vessels and the number of berths that is required to achieve the throughput quantities. Some additional space is required for mooring lines and space between two moored vessels. The length of the quay wall can be estimated by means of the following equation.

$$L_{quay} = 1{,}1 \cdot N \cdot (L_{vessel} + 15) + 15 \qquad \textit{Equation 7.3}$$

Where: L_{quay} = length of the quay [m]
L_{vessel} = length of the design vessel [m]

Using a design vessel of 294m long and two berths, results in a required quay length of approximately 695m.

The width of the quay wall is mainly determined by the span of the STS crane rails and space for mooring facilities. In case of a floating structure some additional space for trucks is required as well. The final width of the structure will be carefully determined in a later stage of this report, but the order of magnitude is likely to be around 50 meters, in case of a floating structure. In case of a non-floating structure that is directly connected to the terminal area, the width is reduced to about 35 meters, since the span of the crane rails in 30,5m.

The height of the quay wall structure mainly depends on the type of flexible structure that is used. In case of a floating structure, the height is determined by the required freeboard plus the draught that results from the total dead weight. However, the height can be completely different for another type of structure. Therefore, the height will also be determined in a later stage of this master thesis.

More detailed information on the final design and dimensions of the structure can be found in part IV of this report.

7.8. Element sizes

Element sizes of the structure are of importance for transportability of the quay and wave loads in open water. When self-floating caissons are used, the width-length ratio should be around 1:3 in order to achieve good navigational properties. When elements have to be constructed in a construction dock, the element sizes are restricted by the dock dimensions. The final element sizes can be found in part IV as well.

7.9. Serviceability Limit State conditions (SLS)

The Serviceability Limit State (SLS) conditions determine the operational hours of a terminal per year. There will be no activities when the SLS conditions are exceeded. Wind, waves, tide and current are the main factors that determine the SLS conditions of a container terminal.

Wind speed causes trouble for container lifting operations and for the overturning moment on STS cranes. STS cranes must be tied down during a storm, in order to prevent overturning.
Waves result in ship motions, which may exceed the tolerances for STS crane handling. The wave period and direction are usually more important than the wave height, since these parameters determine the response of the vessel to the waves.
A tidal window could limit the serviceability of a port as well. When the water depth is insufficient for vessels to reach the container terminal at low tide, a vertical tidal window is present. A horizontal tidal window describes the situation in which vessels are not able or allowed to enter the port, because of too strong currents.

In this thesis it is assumed that the wind speed is governing for the serviceability of the container terminal in any case, because of a sheltered location with respect to waves. This is a reasonable assumption, because most container terminals are very well protected against wave penetration into the harbor basin.
The SLS wind speed is assumed to be 20m/s or 8 beaufort, which is a common value of container lifting by STS cranes. It is assumed that this wind speed is exceeded during 15 days per year and tidal windows are assumed to be absent. The down time due to exceedance of the SLS-wind conditions can be estimated quite well by means of an extreme value distribution applied to location bound wind data. However, this method is less relevant for this thesis, since the structure will be transported to different locations during its lifetime.

7.10. Frequency of relocation and transport distance

The frequency of relocation is an important factor for the financial feasibility of the flexible quay wall structure, just like the distance over which it has to be transported to the new location. The transport distance has an influence on the costs for relocating and the reconstruction time, and the frequency of relocation determines the number of shifts during its lifetime.

The most favorable type of quay wall structure also depends on the relocation frequency. A structure that has to be relocated each few months requires different properties than a structure that is mobilized every 10 years.

The required frequency of relocation and the transport distance are almost impossible to determine, since future developments in the container shipping market cannot be predicted accurately. In fact, this is the main reason which drives the demand for a flexible quay wall structure.

Nowadays, countries like Bangladesh and Cambodia are becoming more favorable than China for the container transport market, because of the low wages. Therefore, a shift from China towards these countries is observed. Whether these countries are still attractive in 2020 or not is uncertain, since one cannot predict the future.

The same yields for the transport distance from one location to another. When the quay has to be transported after 10 years, one cannot already predict the travel distance during the design stage of the quay wall.

For this thesis, a relocation frequency between 5 – 15 years is chosen as a first assumption. The main reasons which may drive the desire to relocate a quay wall for such a period could be:

- Changes in the container shipping market
- Inadequacy of quay walls for larger vessel generation
- Conflicts and war
- Sea level rise

The other reasons mentioned in the introduction are more likely to require a shorter period between different locations.

A shift to a different transshipment hub and conflicts and war are likely to require a shorter transport distance than changes in the container shipping market, but since neither of these reasons can be predicted well, the transport distance remains a guess.

The distance is first assumed to be between 1000 – 5000 nautical miles, which is simply a guess.

In part V of this thesis, a feasibility study will be carried out. The construction and relocation costs of the flexible quay will be compared to construction costs of a traditional quay wall and the difference in construction period. Based on the results of this analysis, one can determine which relocation frequencies and travel distances are feasible. The assumptions made in this paragraph are therefore just a first estimate that needs to be done to select the most suitable type of structure. Whether these assumptions are feasible or not, will be concluded at the end of part V.

Figure 7.8 shows the primary en secondary shipping routes. One can clearly see the large variation in distances between container terminals over the world.

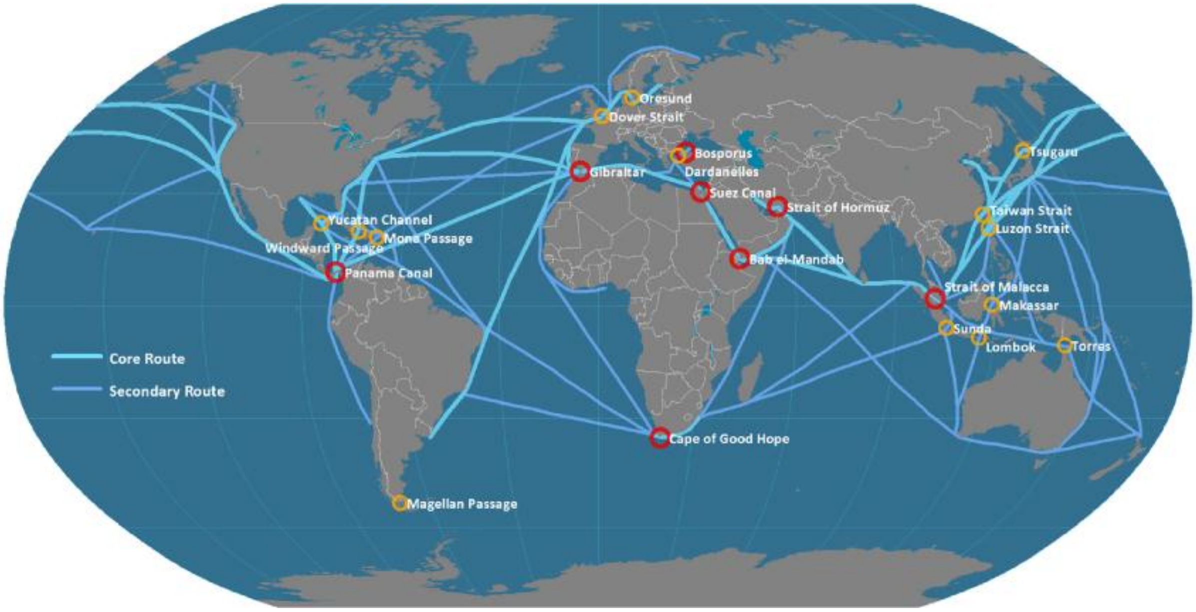


Figure 7.8 – core and secondary shipping routes [www.people.hofstra.edu]

7.11. Lifespan

The quay wall is designed for a lifespan of 50 years, which is a common period with respect to the sustainability of such a structure. However, traditional container quays are usually designed to become profitable within a period of 8 – 15 years, depending on the investment risk of the location. This is because a traditional quay wall often becomes inadequate before the end of its structural lifespan.

Since a flexible quay wall structure can be relocated, the quay can be utilized well during its entire lifespan. The period to become profitable can therefore be longer. Transport costs have a negative influence on the total feasibility of the structure, but a reduction in reconstruction time at the new location has a positive effect. When the construction period can be reduced by one year, compared to the construction of a traditional quay wall, one starts to make money one year earlier.

A lifespan of 50 years and a relocation frequency between 5 – 15 years, results in 3 to 9 shifts during the entire lifetime. Whether this is feasible of not, will be investigated in part V of this report.

7.12. Properties summarized

Summary of container terminal properties	
Vessel type	Panamax (294m x 32m x 12m)
Vessel capacity	5.000 TEU
Average call size	1.500 TEU / call
Annual Throughput	1.000.000 TEU / year
Number of vessels	667 / year
Operational hours	8.400 hours / year
Non-operational days	15 / year
Quay length	695m
Number of berths	2
TEU - factor	1,6
Number of STS cranes	4 / berth (30 moves/hour)
Number of trucks	6 / STS crane (8 moves/hour)
Number of RTG	14 / berth (16 moves/hour)
Average service time	10 hours / vessel
Arrival time distribution function	Vessels: Negative exponential (M)
Service / arrival time distribution functions	STS cranes: Erlang 2 (E_2)
	Trucks: Erlang 2 (E_2)
	RTG: Erlang 2 (E_2)
Average waiting times	Vessel for berth: 49 minutes
	STS crane for truck: 5 seconds
	Truck for RTG: 11 seconds
SLS wind conditions	Max 20m/s or 8 bft
Assumed frequency of relocation	5 – 15 years
Assumed travel distance for relocation	1000 – 5000 nautical miles
Lifespan	50 years

Table 7.2 – summary of quay wall properties

8. Boundary conditions and applicable area

The flexible quay wall structure is not designed for one specific location, since it can be relocated during its lifetime. Therefore, a description of the applicable area is given in this chapter, by means of formulating boundary conditions for various criteria.

8.1. Wave climate

As stated in chapter 7.9, the wave climate is not governing for the operability of the flexible quay wall. It is assumed that the location of the quay is sheltered, with respect to wave penetration into the harbor basin. This is a realistic assumption, since container terminals are nearly always very well protected against waves. There are no limitations on the prevailing wave climate at a specific location, for this reason.

8.2. Wind climate

The wind climate plays an important role in the horizontal loads and overturning momentum at the quay wall structure. Moored vessels exert a severe load on their mooring lines, when strong winds affect the hull of the vessel and its cargo.

The SLS conditions were determined at a wind speed of 20m/s, as described in chapter 7.9. However, survival of the structure during extreme events, determines the required strength and stability of the quay wall. Such an extreme event has a very low frequency of occurrence. For example, an Ultimate Limit State (ULS) condition that has an expected occurrence of once in 1000 years, leads to an occurrence chance of 5%, within a period of 50 years. Numbers like these can be determined by means of equation 8.1.

$$P_f = 1 - (1 - f_{per\ year})^{T_{life}} \qquad \textit{Equation 8.1}$$

Where: P_f = probability of failure within lifetime [-]
$f_{per\ year}$ = frequency of occurrence of extreme event [1/year]
T_{life} = lifetime of the structure [years]

The ULS condition is different for each location, but a limit of 42m/s is used as ULS condition for the flexible quay wall structure. In practice, this means that the structure may fail when these conditions are exceeded.
The wind speed of 42m/s was found on the website of liebherr, a well-known manufacturer of STS cranes. This wind speed was given as a typical design value for STS crane failure, so crane collapse above 42m/s. Note that STS cranes in areas that are vulnerable for strong hurricanes, may have a heavier construction.

Figure 8.1 – STS crane collapse after typhoon Maemi (up to 215km/h) in Port Busan, South Korea [www.cargolaw.com]

Table 8.1 gives an overview of the beaufort scale and corresponding wind speeds.

Wind speed classification					
Beaufort number	Description	Wind speed [knots]	[m/s]	[km/h]	**STS status**
0	Calm	< 1	0 - 0,2	0 - 0,7	**Operational**
1	Light air	1 - 3	0,3 - 1,5	0,8 - 5,4	**Operational**
2	Light breeze	4 - 6	1,6 - 3,3	5,5 - 11,9	**Operational**
3	Gentle breeze	7 -10	3,4 - 5,4	12,0 - 19,4	**Operational**
4	Moderate breeze	11 - 16	5,5 - 7,9	19,5 - 28,4	**Operational**
5	Fresh breeze	17 - 21	8,0 - 10,7	28,5 - 38,5	**Operational**
6	Strong breeze	22 - 27	10,8 - 13,8	38,6 - 49,7	**Operational**
7	Near gale	28 - 33	13,9 - 17,1	49,8 - 61,6	**Operational**
8	Gale	34 - 40	17,2 - 20,7	61,7 - 74,5	**Operational**
9	Strong gale	41 - 47	20,8 - 24,4	74,6 - 87,8	**Non-operational**
10	Storm	48 - 55	24,5 - 28,4	87,9 - 102,2	**Non-operational**
11	Violent storm	56 - 63	28,5 - 32,6	102,3 - 117,4	**Non-operational**
12	Hurricane	64 - 71	32,7 - 36,9	117,5 - 132,8	**Non-operational**
13	Hurricane	72 - 80	37,0 - 41,4	132,9 - 149,0	**Non-operational**
14	Hurricane	81 - 89	41,5 - 46,1	149,1 - 166,0	**Collapse**
15	Hurricane	90 - 99	46,2 - 50,9	166,1 - 183,2	**Collapse**
16	Hurricane	100 - 109	51,0 - 56,0	183,3 - 201,6	**Collapse**
17	Hurricane	109 - 118	56,1 - 61,2	201,7 - 220,3	**Collapse**

Table 8.1 – wind speed classifications [whale.wheelock.edu]

The frequency of occurrence of an extreme event can be determined with extrapolated data, by means of an extreme value distribution for each specific location. Depending on the governing ULS conditions, the actual structure may need additional strength and resistance. The other way around, it could be less strong when the ULS condition is below the mentioned 42m/s, like in The Netherlands for instance.
When a gravity structure is used, this can easily be done by adding or subtracting mass.
A detailed determination of the wind load on STS cranes can be found in appendix P.
Also note that the wind direction plays a role as well.

8.3. Tidal difference and retaining height

The tidal difference is an important parameter for the determination of the retaining height of the structure. A tidal difference of 4m is assumed as maximum value for operability of the quay wall.

Panamax vessels have a maximum draught of about 13m and require an under keel clearance of say 1,5m. The freeboard of the quay during High High Water Spring (HHWS) is about 3,5 meters, so the total height of the structure will be 22m. An illustration of the retaining height, the tide, freeboard and water depth is given in figure 8.2.

For a smaller tidal difference, one can either decide to bury the lower caisson into the bottom or to create additional buoyancy force on the upper caisson, in order to place it on top of the lower one.

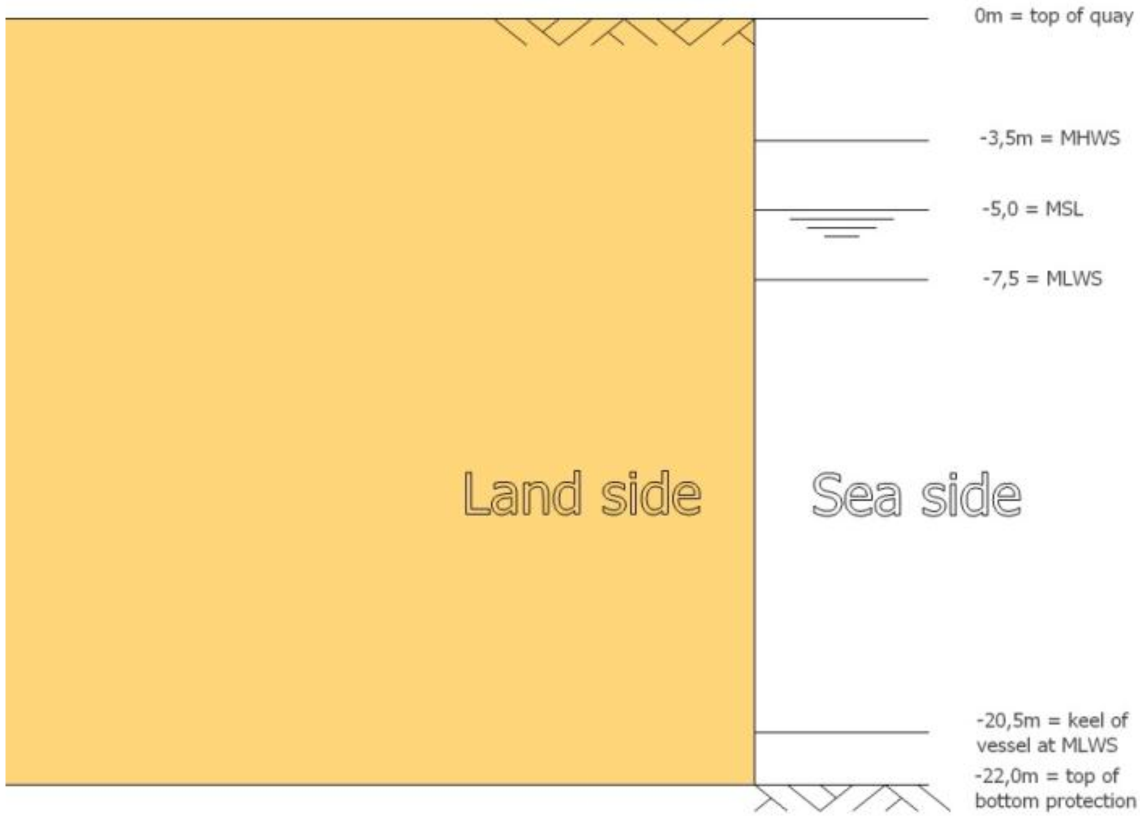


Figure 8.2 – illustration of retaining height, tide, freeboard and water depth.

Wave overtopping is also related to the freeboard of the structure, but because of the sheltered location of the container terminal, it is not governing for the required freeboard. The European overtopping manual gives an allowable overtopping discharge of 0,4 l/m/s to prevent damage to equipment on a quay wall. However, this value seems rather low, depending on the situation. For the operability of highly automatized systems with AGV's driving close to the edge of the quay, this might be a relevant value, but a truck and RTG system is likely to accept more overtopping. In practice, some quay walls are even flooded during extreme events.

The overtopping discharge can be calculated with equation 22.27 in chapter 22 of this report. The overtopping discharge and wave height are plotted in a graph, for a freeboard of 3,5 meter, as illustrated in figure 8.2. It can be concluded that there is hardly any overtopping for wave heights below 1m. Since larger wave heights are not likely to occur in a sheltered harbor basin, wave overtopping is not governing for the determination of the quays' freeboard.

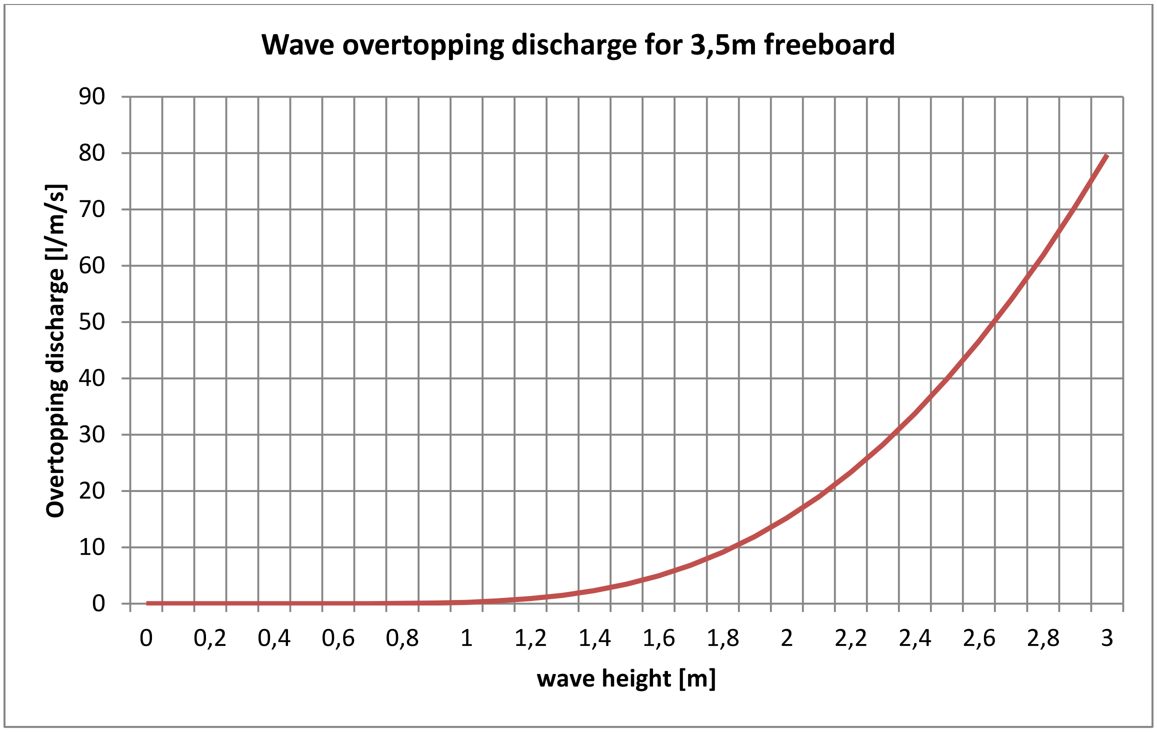


Figure 8.3 – overtopping discharge and wave height for 3,5m freeboard.

8.4. Soil properties

Soil properties are important for the bearing capacity of the quays' foundation and for the stability of soil retaining structures. However, one can modify soil conditions by improving the soil properties around the structure. This can for instance be done by applying a gravel bed foundation under a gravity structure to create more bearing capacity. Rubble material can also be applied behind a soil retaining structure to reduce the horizontal soil pressure and to provide a proper drainage.

Sandy soils are chosen as the most suitable subsoil for the flexible quay wall structure, but it must be applicable in slightly rocky soils and soft soils to some extent as well. For rocky subsoil, this may require excavation works and for very soft soils, a soil improvement may be required. The flexible quay wall structure is not applicable in very hard rock or in extremely soft soil conditions.

8.5. Earthquake zones

Quite some container ports around the world are located in areas that are vulnerable for earthquakes. To make the flexible quay wall structure appropriate for these areas, the structure will be able to cope with earthquake loads, to some extent.

The influence of earthquake loads depends on the type of quay wall structure. In chapter 22 this influence in described, when the type of structure is determined.

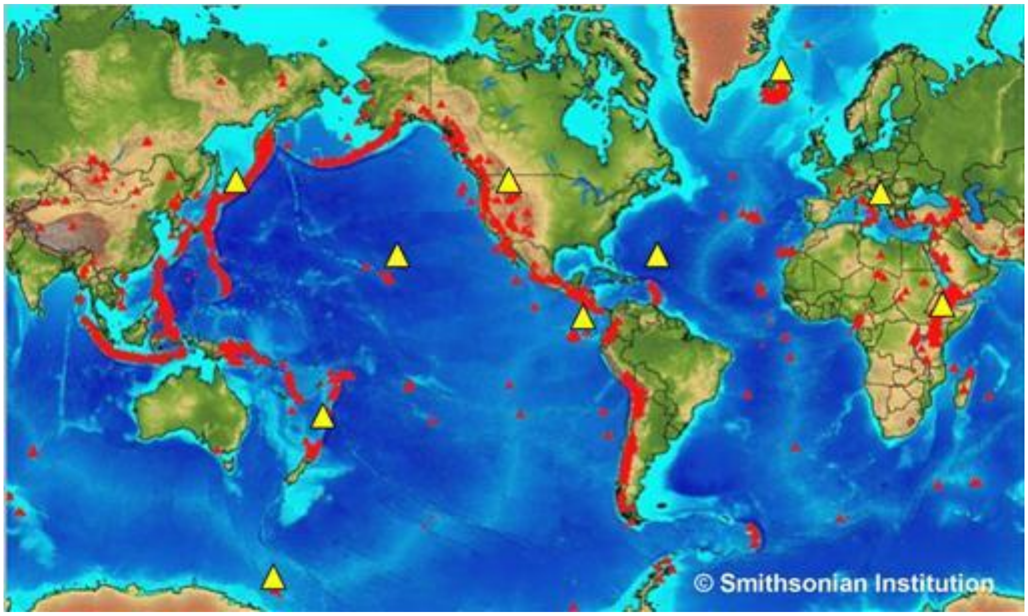


Figure 8.4 – global earthquake zones [www.earthquakeusgs.org]

8.6. Boundary conditions summarized

Boundary conditions flexible quay wall structure, summarized	
Wave climate	No limitations, because of sheltered location
Wind climate	< 42m/s wind speeds (1/1000 year probability)
Tidal difference	0m – 4m
Maximum retaining height	22m
Soil properties	No hard rock or extremely soft soils
Earthquake proof	At least PGA = 2 m/s^2 *

Table 8.2 – boundary conditions summarized

* = The actual minimum resistance against earthquake loads, was determined in the final design of the quay wall structure in chapter 27.

Part III
Quay Wall Structures

9. Introduction

In part III of this graduation thesis several quay wall structures are considered. After considering 5 possible solutions for a flexible container quay, each type of structure will be assessed on criteria such as; flexibility to relocate, stability, costs, reconstruction period etc., by means of a multi criteria analysis. The first 5 alternatives are all quay wall structures that have already been used in practice. After the multi criteria analysis, some variants on these structures will be established in order to obtain the best properties. The best alternative will be elaborated in part IV of this master thesis.

10. Location bound properties

Most properties of a quay wall structure are largely dependent on the local conditions. Governing failure mechanisms and construction costs can differ for each location. In fact, this is the reason why so many different types of quay walls do exist.

Soil conditions are a very important parameter. A cheap type of quay wall structure for rocky subsoil may be far too expensive or even impossible to apply in soft soil conditions and vice versa.

Availability of raw materials has a large influence as well. When a concrete factory is nearby, it becomes very attractive to design a concrete structure. Steel structures may be cheaper when concrete has to be transported over large distances, although steel is a much more expensive material.

Figure 10.1 shows the construction costs relative to the retaining height and figure 10.2 shows the countries of which data was used for the graph. One can clearly see the large differences in construction costs. Note that this graph doesn't only consider quay walls for container terminals, but includes quays for other cargo types as well.

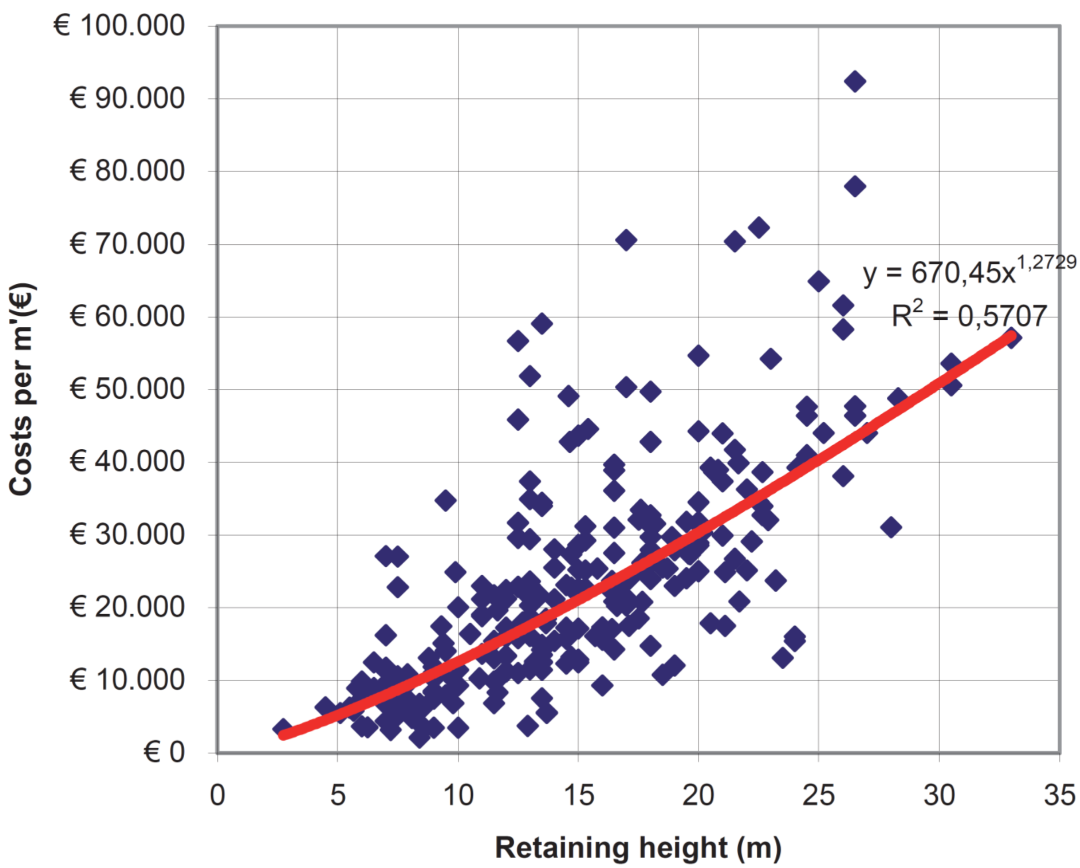


Figure 10.1 – quay wall costs per running meter relative to the retaining height
[A History of Quay Walls, J.G. de Gijt (2008)]

Note: information in figure 10.1 and 10.2 is for traditional quay wall structures in general, so not in particular for a flexible quay wall structure for container vessels.

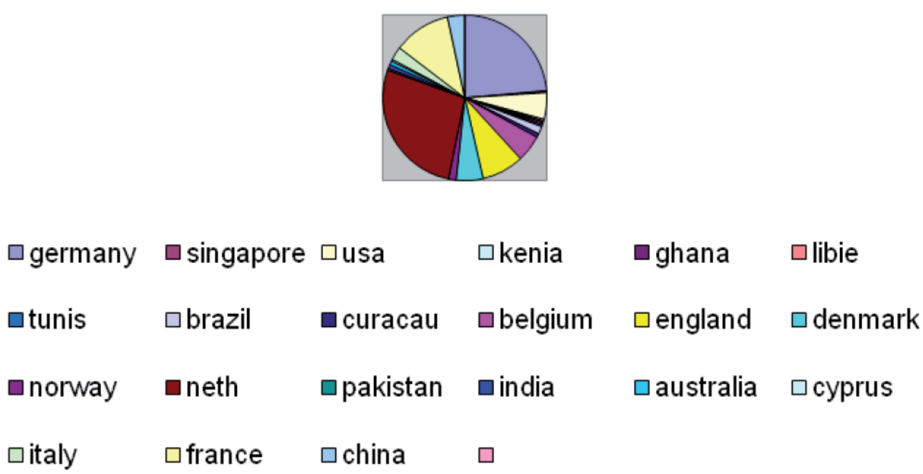


Figure 10.2 – data distribution of counties used for figure 10.1
[A History of Quay Walls, J.G. de Gijt]

11. Construction material

The retaining height and the construction material are by far the most important factors influencing the price of a quay wall structure in Rotterdam. Since the retaining height is a given parameter for each specific location, the construction material is one of the most import issues with respect to the costs of a flexible quay wall structure. Concrete and steel are the most suitable construction materials for a quay wall, but differ a lot in price and properties.

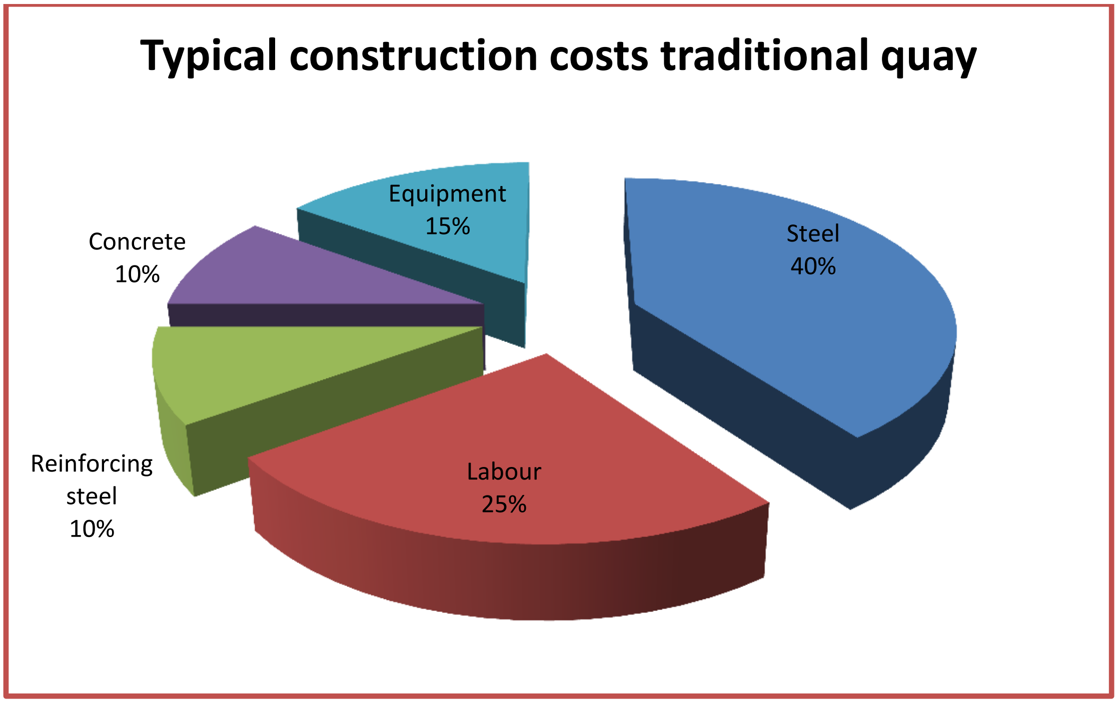


Figure 11.1 – analysis of construction costs of quay walls in Rotterdam [Handboek Kademuren]

Note that the diagram in figure 11.1 contains information of traditional quay wall structures, so non-flexible ones.

The density of steel is about three times more than that of concrete. However, depending on the shape and type of structure, steel structures often need less material, often making the structure as a whole lighter. A steel caisson would for instance be much lighter than a concrete caisson, but steel caissons are not very common. Instead, piles can be made of both steel and concrete, depending on the application.
Concrete consists of gravel, sand, cement and water and is therefore much cheaper than steel. With respect to costs, concrete is nearly always more favorable that steel, but both materials have their limitations.

Concrete cannot or hardly deal with tensile forces and needs reinforcement steel or pre-stressed steel tendons in this case. Instead, it has a good compressive strength.
The compressive and tensile strength of steel are both high, so steel can be used for both tensile and compressive forces very well. However, compressive forces may cause buckling. Combinations of normal forces, shear forces, momentum and torsion may cause various failure mechanisms for both steel and concrete.

Steel elements can be connected by bolts or welding, whereas concrete elements are usually connected by joints, cementing them together or constructing them as one piece. Connections between concrete elements often need a lot of reinforcement steel.
Steel needs more maintenance than concrete to prevent corrosion, especially in salt water conditions. Nevertheless, concrete is affected by salt and chemicals in the water as well, reducing the lifetime compared to applications in fresh water conditions.
When cracks appear in concrete structures, salt water may reach the reinforcement steel. The steel will expand due to corrosion and the concrete surface gets damaged, making the problem worse.
Steel is a bit more affected by temperature variations causing it to shrink or expand, but the difference with concrete is nil. Therefore, temperatures in reinforced concrete hardly lead to trouble caused by expansion differences between concrete and the reinforcement.
To avoid cracks in the concrete structure, one can also increase the strength of concrete by pre-stressing it. Pre-stressing can either be done by pre-tensioning or post-tensioning. In case of pre-tensioning, tendons are pre-stressed before casting of the concrete. The tendons are released once the concrete has reached sufficient strength, introducing a compressive force into the concrete, which is absorbed by adhesion between the tendons and the concrete. The post-tensioning method uses curved tubes that are poured in the concrete body. Tendons are pre-stressed inside these tubes when the concrete has reached its ultimate strength, introducing an additional compressive pressure in the concrete. The aim of pre-stressing concrete is to eliminate all tensile forces. As a result the compressive forces will increase.

Soil can be seen as a construction material too, because it provides resistance, as well as it poses loads on a structure. Soil properties are therefore also very important for the quay wall design. Local soil properties should be investigated and if necessary, modified by applying a soil improvement.

Objective of this master thesis is to find a feasible solution for a flexible quay wall structure. In order to keep the construction costs low, the objective is to design a flexible quay wall that doesn’t need a large amount of steel.

12. Alternative 1: caisson quay wall

One possibility is to build a flexible quay wall structure with caisson elements. Caissons are usually made of reinforced or pre-stressed concrete and are built in a construction dock under dry conditions. The construction dock is located next to a waterway and the floor of the dock is below local water level. When the construction of a caisson is finished, the dock is filled with water and the floating caisson can be towed to the construction site of the quay. Once the caisson is brought into position, its chambers are filled with water, sinking the caisson down to its gravel bed foundation. The area behind the caissons will be backfilled up to terminal elevation. The top slab of the caisson will remain well above mean sea level and will function as deck of the container quay.

The caissons must have sufficient strength to resist the load of the container cranes, the containers and other equipment on top of the caisson. During transport they must have sufficient floating stability and the walls should be strong enough to resist the static and dynamic water pressure. Once in operation, the system of caissons should be able to absorb the mooring energy of a container vessel and resist its hawser forces. A bottom protection is needed in front of the caissons to prevent scour holes as a result of propeller wash from the vessels. Furthermore, it must stay in position withstanding forces caused by soil, wind, waves and water pressure. Failure mechanisms like tilting, sliding, stability against buoyancy, settlements, scour holes, bearing capacity of the soil, structural strength, piping, and water tightness should be carefully checked.

If the length of the quay wall requires adjustments, caissons could be added or removed. Relocating the entire structure can be done by pumping the ballast water out of the chambers, making the caissons floating again. However, this will cause serious damage to the terminal area, because the soil retaining structure is now removed.
Deepening of the basin in front of the quay is problematic for the stability of the structure, which hampers the flexibility of a caisson quay wall at a fixed location.

Figure 12.1 – illustration caisson quay wall

Figure 12.2 – illustration operational caisson quay wall

12.1. Advantages

- Relatively easy to relocate the quay wall at a different place
- Good connection with terminal area
- Short construction time in the harbor reducing hindrance to traffic in the port
- No strong connection between elements needed
- Construction material mainly concrete
- Low maintenance costs
- Position keeping by mass of the structure

12.2. Disadvantages

- Non-flexible for deepening harbor basin, because of instability
- Construction only possible in a construction dock or a slipway
- Settlements may cause a non-smooth surface of the quay
- Ballast water inside the structure may reduce the lifetime of concrete
- Large amount of concrete needed
- Relatively high construction costs
- Preparation of a gravel bed trench and foundation
- (Collapse of the soil after removing the structure)

13. Alternative 2: sheet piled combi-wall

Another flexible solution for a quay wall structure could be a sheet piled combi-wall. This type of structure consists of a combination of sheet piles and steel tubes. Both are driving deep into the subsoil to guarantee horizontal and rotational stability. To reduce the length of the elements, the combi-wall is connected to an anchor wall. The top of the combi-wall is covered with concrete to create a smooth surface and the crane rails are positioned straight above the combi-wall and the supporting anchor wall.

The structure must have sufficient strength to resist the horizontal soil pressure and may not tilt to the sea side of the quay. The passive zone of the construction elements is very important for this. In addition, the quay wall must be capable to deal with the hawser forces of the vessels. Furthermore, it should be able to provide enough support for the crane load and must be sand tight to prevent settlements at the terminal side of the quay. To prevent erosion by propeller wash, the sea bed will be covered with a bottom protection.

The length of the quay wall can be adjusted if enough space for expansion is left. Rebuilding the structure at a different location is possible, but removing it from its original location is very difficult. The top of the combi-wall is cemented together with concrete and at a certain depth it is connected to the supporting anchor wall. Pulling the tubes and sheet piles out of the soil takes time and requires noisy heavy equipment. Reusing the materials is therefore often not feasible.

Increasing the water depth in front of the structure is only possible if the construction is over dimensioned for the original situation. If the original situation is used as governing load, stability of the combi-wall will be insufficient when the harbor basin is deepened.

Figure 13.1 – illustration sheet piled combi-wall

Figure 13.2 – illustration operational sheet piled combi-wall

13.1. Advantages

- Elements are easy to transport by ship
- Less vulnerable to ship collision
- Good connection with terminal area
- Elements are not hard to connect

13.2. Disadvantages

- Steel structure needs maintenance
- Relocating the structure is difficult and expensive
- Expensive construction materials
- Difficult to apply in rocky subsoil
- Deepening only possible in case of over dimensioning
- Construction may hamper traffic in the port
- (Collapse of the terminal area when the quay is removed)

14. Alternative 3: mass concrete block-work wall

Quay walls could also be built by placing several concrete blocks on top of each other. The blocks can be dimensioned and positioned in such a way, that no reinforcement steel or connection is needed. The stability of the structure is achieved by giving all blocks sufficient mass. Blocks usually differ in size over the retaining height of the structure and the top layer is often casted as one piece to create a smooth surface. Only the crane rails on the sea side is located above the block wall. The inner crane rail is mounted on a separate foundation.

Each block must deliver sufficient vertical pressure to its subsurface in order to avoid sliding due to the horizontal soil pressure. To avoid a water level difference at both sides of the structure, a permeable backfill and foundation is preferable. When the blocks are not positioned straight above each other, one should take care that no tensile forces occur. No part of the quay may tilt to the sea side and should therefore be checked at each level. Reinforcement is required in the top layer to resist hawser forces of the vessels. Furthermore, the soil should have sufficient bearing capacity to support the entire structure including the loads.

One of the main disadvantages of a mass concrete block-work wall is its very large element size for large retaining heights. An enormous amount of concrete is needed and building the quay wall requires heavy equipment. The reinforced top layer, which is constructed as one piece, also causes trouble for the flexibility of the quay wall. Reusing the elements is possible, but often unfeasible.
However, especially in case of small retaining heights, such a structure may be a good flexible solution.

Deepening of the harbor basin in front of a block-work wall is hard. It could be done in combination with a spacer between the vessel and the quay and applying an underwater slope. Often this is not an option, because the reach of the container cranes becomes insufficient. Flexibility of this type is construction, with respect to future deepening of the basin is therefore doubtful.

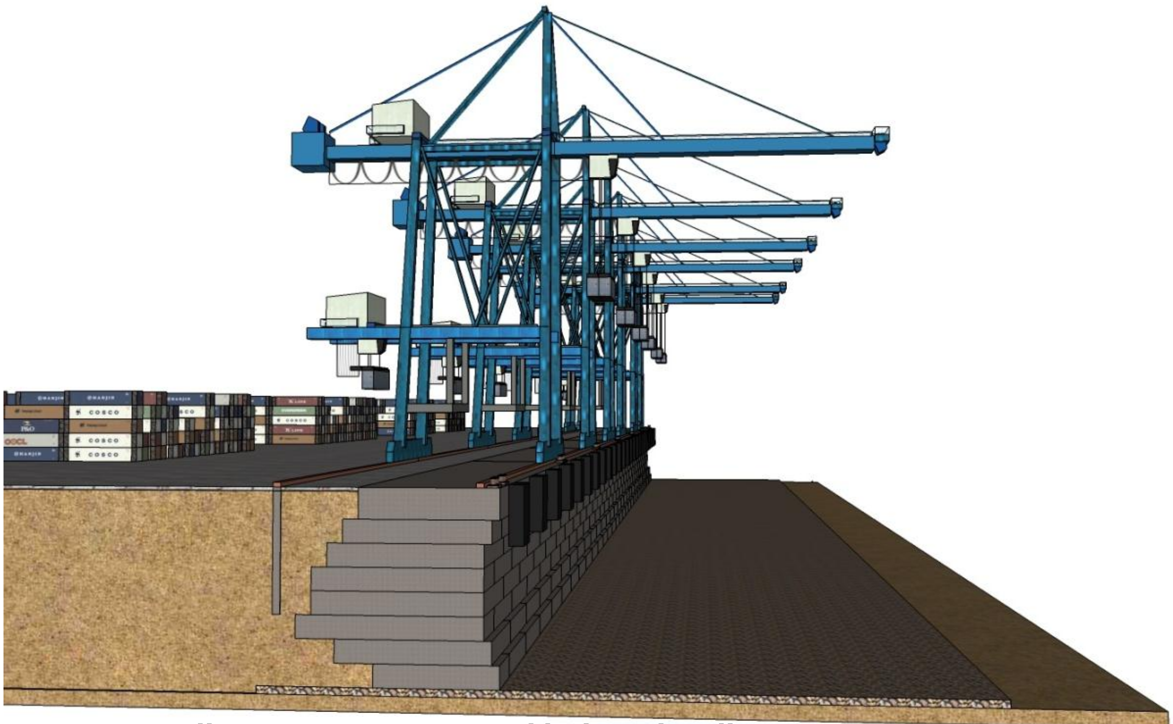


Figure 14.1 – illustration mass concrete block-work wall

Figure 14.2 – illustration operational mass concrete block-work wall

14.1. Advantages

- Cheap construction material
- Elements can be casted on site on beforehand
- No connection between under lying elements
- Simple equipment suffices for small retaining heights
- Very cheap to maintain
- Good connection with terminal area
- Position keeping by mass of the structure

14.2. Disadvantages

- Large element sizes for large retaining heights
- Relocation unfeasible for large elements
- Deepening of the basin not possible
- Elements are large and heavy, especially at the bottom
- Large amount of concrete needed
- Top of the quay wall cements under lying blocks together
- Differential settlement may cause trouble
- (Collapse of the terminal area when the quay is removed)

15. Alternative 4: deck on piles

A deck on piles could also be used as a quay wall structure for container vessels. A reinforced concrete deck is often casted at location and is supported by a pile foundation. Both crane rails are located straight above a row of piles, to decrease the load on the deck. Some piles can be placed under a certain angle to absorb horizontal forces.

The deck of the structure must be strong enough to resist to loads of the cranes and the containers. Driving fewer piles leads to a heavier deck construction and vice versa, so an optimum number of piles, their dimensions and their position can be determined. The piles must create sufficient capacity to support the deck and must be able to deal with the governing horizontal loads. For stability of the pile and the supporting soil, it should be driven deep enough into the subsoil. It could be wise to drive the piles a bit deeper into the soil than necessary to allow deepening of the harbor basin in the future. Like in all cases, a bottom protection is required to guarantee stability of the structure and prevent the development of scour holes.

Relocating the structure can be done by driving new piles at the new location or reusing the old steel piles. A prefabricated deck could be transported by barges, but a new deck structure can also be casted at the new location. Increasing the length of the quay wall can be done by driving new piles next to the quay wall and placing new deck elements on top.

Figure 15.1 – illustration deck on piles

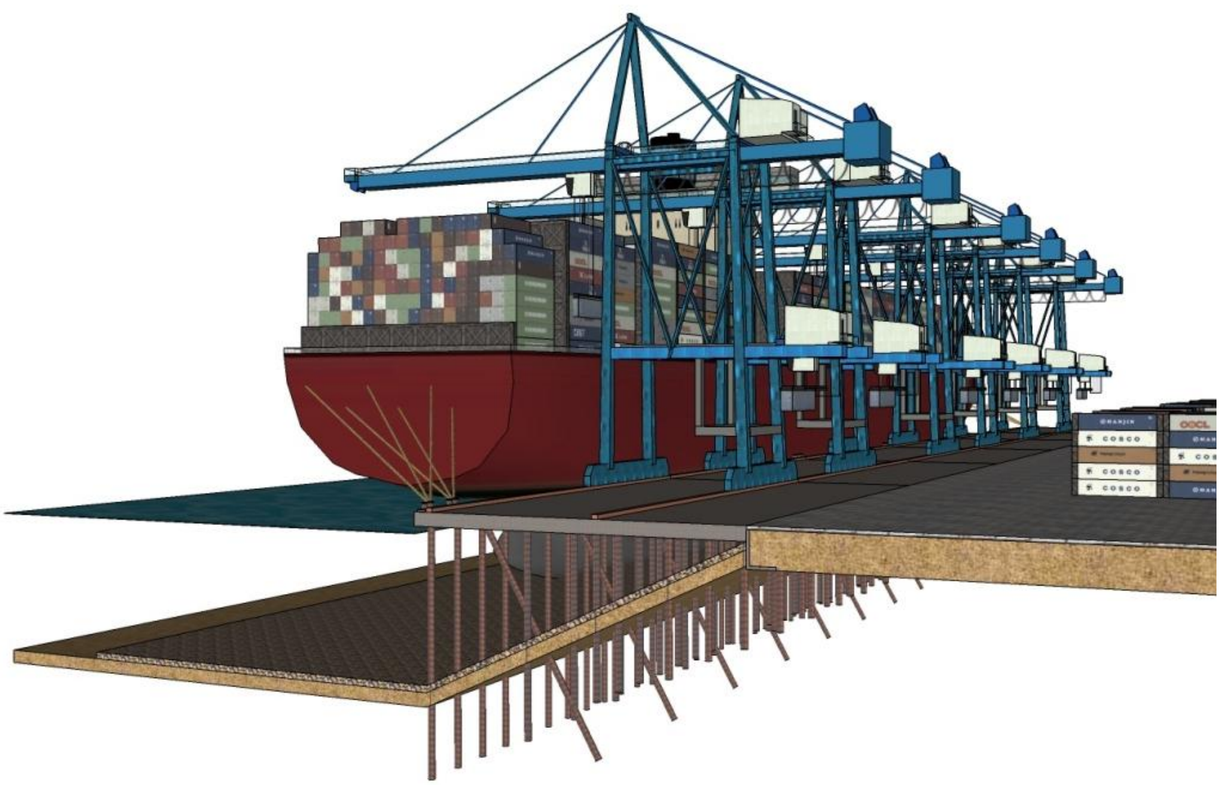


Figure 15.2 – illustration operational deck on piles

15.1. Advantages

- Increasing the water depth is possible to some extent
- Elements could be reused at another location
- Good connection with terminal area
- (No collapse of terminal area after removal)

15.2. Disadvantages

- Piles are hard to remove
- Needs special equipment in rocky subsoil
- Absorption of horizontal loads
- Long piles needed for large retaining heights
- More sensitive to ship collision
- Vibrations and horizontal displacements of the deck may occur

16. Alternative 5: floating quay wall

A floating quay wall structure is a very flexible solution. Floating caissons can be connected to each other creating a platform for container cranes and handling equipment. The quay wall is connected to the terminal area, which is built on land. It can be disconnected from its mooring system and the floating elements can be towed to a different location. However, a floating structure has some serious disadvantages.

The main problem of a floating structure is its dynamic behavior in waves, wind and currents. Motions of the quay wall may reduce the productivity of the quay and threaten the handling time of container vessels. Tolerances in motions are small for container cranes and a floating structure is therefore not a wise alternative for ports with severe wave conditions.
Just like the quay wall itself, the container vessel is affected by waves as well. Waves reflect on the hull of the vessel and the quay wall, and motions of the quay will generate new waves. These motions could cause resonance between the vessel and the quay, making the situation very complicated and undesirable.

Apart from the motions caused by external forces, the floating quay will also move as a result of crane motions and lifting containers. These forces can be calculated accurately and could be compensated by ballast water or counterweights to a certain extent. However, water pumps with a high capacity are required and creating space for counterweights is difficult.

Position keeping of a floating quay wall creates another problem. The structure is affected by wind and currents and would drift away without a proper mooring system. Anchors are a possible solution, but these are attached to chains, which are an obstacle for traffic in the port.

Besides horizontal motions, the structure is exposed to vertical motions as well. The terminal area has a fixed elevation and handling equipment is restricted to a certain maximum slope. Therefore, the distance off the shoreline is determined by the water level fluctuations and the water depth. The connection bridge to the terminal storage area should be capable to resist the berthing energy of container vessels.

Connecting the elements of the floating platform requires careful research. The dynamic behavior of the structure introduces large forces on the connections. Forces should be determined properly and a suitable connection has to be designed.

An important advantage of a floating structure is the possibility to increase the water depth without influencing the stability of the structure. In combination with its flexibility with respect to relocating the quay wall, it could become an attractive alternative for ports with a large tidal difference and a very calm wave climate. But waves generated by passing ships and motions caused by container lifting may remain a problem.

Some more information on floating quay walls can be found in the literature review, since research on floating structures has been carried out before.

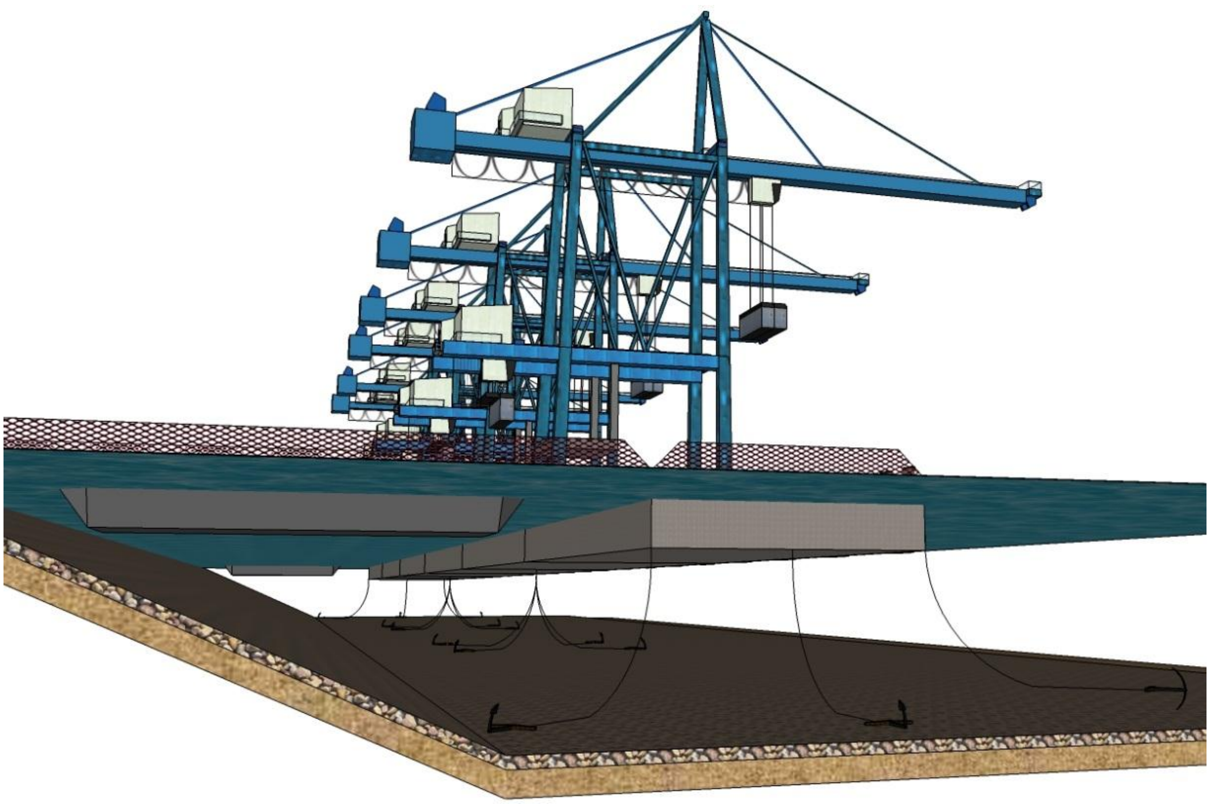


Figure 16.1 – illustration floating quay wall

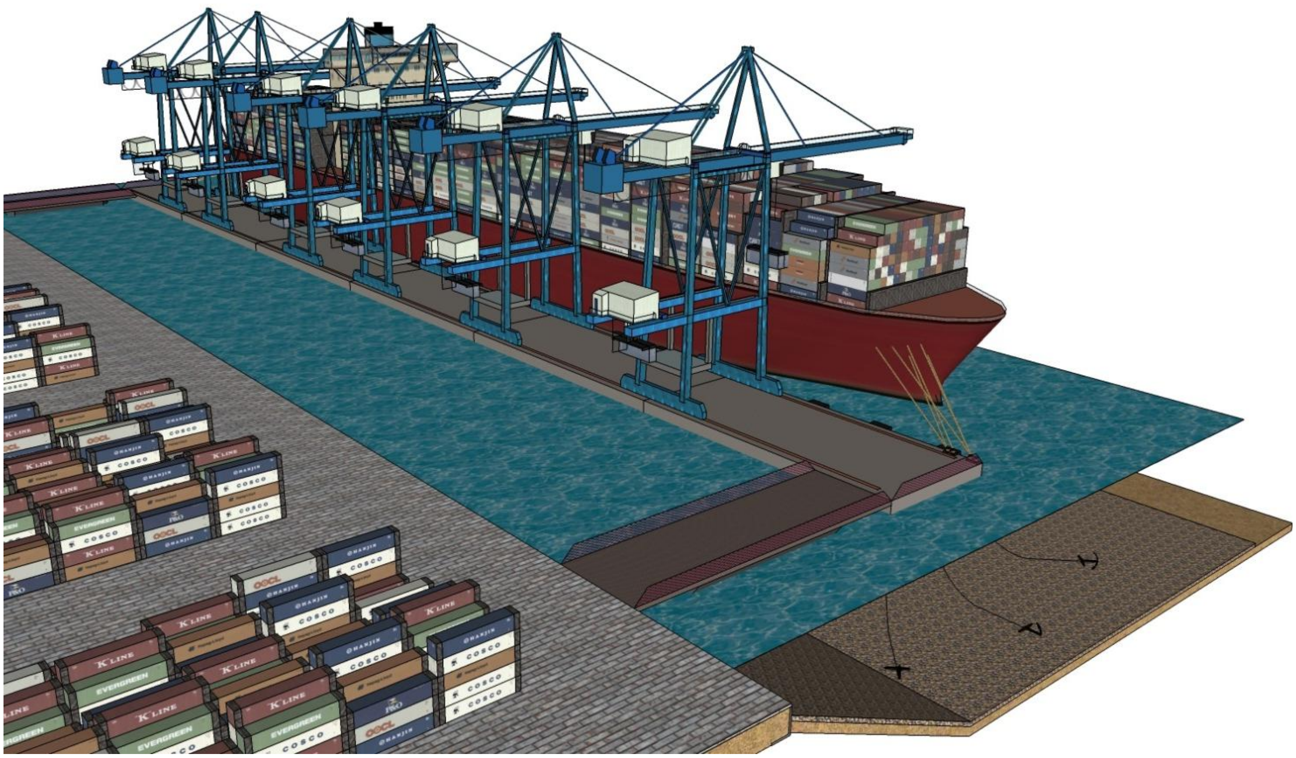


Figure 16.2 – illustration operational floating quay wall

16.1. Advantages

- Easy to relocate the entire structure
- Deepening the harbor basin in the future is possible
- Short construction time at quay wall location
- Attractive for large water level differences
- (No collapse of terminal area after removal)

16.2. Disadvantages

- Motions due to waves, wind, current and handling equipment
- Uncertain productivity due to uncertain motions
- Connection bridge towards terminal area required
- Structure requires a mooring system
- Relatively large investment costs
- Elements need to be constructed in a dock
- Introducing berthing energy to the shore
- Needs strong connection with terminal area

Stability calculations on floating quay walls can be found in appendix H and I.

17. Multi Criteria Analysis

The five alternatives that were mentioned in de previous chapters are now being assessed by means of a multi criteria analysis. The objective of this thesis is to design a quay wall structure that can be reused and relocated. Other properties like (re)construction time and costs, residual value and environmental impact are related to this. Stability of the quay wall is very important too, because an instable quay wall is useless.

The criteria of the multi criteria analysis are listed below:

- Flexibility to relocate
- Stability during operations
- Reconstruction time
- Reconstruction costs
- Initial construction time
- Initial construction costs
- Residual value of structure
- Stability of soil after removal
- Connection with terminal area
- Environmental impact
- Possibility of deepening port basin
- Earthquake resistance

For each criterion a score of 1, 2 or 3 points can be obtained by every single alternative, where 1 represents a poor score, 3 a good score and 2 an intermediate score. The scores of the first multi criteria analysis are presented in table 17.1.

	Caisson quay wall	Sheet piled combi-wall	Mass concrete block-work wall	Suspended deck on piles	Floating quay wall
Flexibility to relocate	3	1	2	2	3
Stability during operations	3	3	3	3	1
Reconstruction time	3	1	2	1	3
Reconstruction costs	3	1	2	2	3
Initial construction time	1	3	2	2	1
Initial construction costs	1	3	3	2	1
Residual value of structure	3	1	3	2	3
Stability of soil after removal	1	1	1	3	3
Connection with terminal area	3	3	3	2	1
Environmental impact	3	1	3	2	3
Possibility of deepening port basin	1	1	1	2	3
Earthquake resistance	2	2	2	3	3
Total score	**27**	**21**	**27**	**26**	**28**
Percentage of maximum score [%]	*75*	*58*	*75*	*72*	*78*

Table 17.1 – scores multi criteria analysis without weight factors

The basic multi criteria analysis in table 17.1, shows that a floating quay wall has the highest score. However, the criteria are not of equal importance in reality.

17.1. Weight factors

Because the importance of the criteria is not equally distributed, a weight factor is applied to each criterion. In this way, important aspects have more influence on the final score than aspects of minor importance.

The weight factors are determined by comparing the importance of each criterion, with respect to all other criteria. If one criterion is more important than another, it scores a 1 in its row and when it is less important it scores a 0. The final score of each criterion is found by the sum of its own row, plus the number of zeros in its column. Finally, the weight factor is determined by normalizing the score on a scale of 1 to 10. The determination of the weight factors is displayed in table 17.2.

	Flexibility to relocate	Stability during operations	Reconstruction time	Reconstruction costs	Initial construction time	Initial construction costs	Residual value of structure	Stability of soil after removal	Connection with terminal area	Environmental impact	Possibility of deepening port basin	Earthquake resistance	**Total score**	**Normalized weight factor**
Flexibility to relocate	-	0	1	1	1	1	1	1	1	1	1	1	10	**9,18**
Stability during operations		-	1	1	1	1	1	1	1	1	1	1	11	**10,0**
Reconstruction time			-	0	1	1	1	1	1	1	1	1	8	**7,55**
Reconstruction costs				-	1	1	1	1	1	1	1	1	9	**8,36**
Initial construction time					-	0	0	1	1	0	1	1	4	**4,27**
Initial construction costs						-	0	1	1	0	1	1	5	**5,09**
Residual value of structure							-	1	1	1	1	1	7	**6,73**
Stability of soil after removal								-	0	0	0	0	0	**1,00**
Connection with terminal area									-	0	0	0	1	**1,82**
Environmental impact										-	1	1	6	**5,91**
Possibility of deepening basin											-	0	2	**2,64**
Earthquake resistance												-	3	**3,45**

Table 17.2 – determination of weight factors for multi criteria analysis

Sorting the criteria based on their importance leads to the ranking given below. Keeping the objectives of this thesis in mind, the ranking seems to make sense. An instable quay wall is useless in any case and criteria 2 to 6 are directly related to the main objectives of this thesis.

1) Stability during operations
2) Flexibility to relocate
3) Reconstruction costs
4) Reconstruction time
5) Residual value of structure
6) Environmental impact
7) Initial construction costs
8) Initial construction time
9) Earthquake resistance
10) Possibility of deepening port basin
11) Connection with terminal area
12) Stability of soil after removal

Since this thesis is about designing a quay wall structure that can be reused at a different location, flexibility to relocate comes in the second place.
The reconstruction costs and reconstruction time are related to this. When reconstruction costs are very high, it might be more attractive to build a new structure, so reconstruction costs are a bit more important than reconstruction time.
The residual value of the structure is of importance to make relocation worth the effort and to make selling the structure a possible alternative.
Environmental impact is always important and has the trend to become more and more important in the future.
The initial construction costs and construction time are of medium importance, because the higher scoring criteria are mentioned to recoup the initial construction costs in the long term.
Earthquake resistance is important to avoid damage during an earthquake, but a quay wall structure can be designed to cope with this additional load. Therefore, vulnerability for earthquakes is less important.
A good connection with the terminal area reduces the required number of handling equipment to achieve a certain production, but is of minor importance.

The importance of the possibility to deepen the harbor basin in the future and stability of the soil after removing the quay wall is taken low in this multi criteria analysis, but can vary for a different situation. In this thesis, it is assumed that collapse of the terminal area is not a big issue, because the old location is abandoned.
As mentioned in the introduction, this thesis focusses on flexibility over different locations and not on flexibility at a fixed location. Therefore, future deepening of the harbor basin has a low score, although it would be a desirable property of a flexible quay wall structure.

The new scores including weight factors are presented in table 17.3 on the next page.

	Applied weight factor	Caisson quay wall	Sheet piled combi-wall	Mass concrete block-work wall	Suspended deck on piles	Floating quay wall
Flexibility to relocate	**9,18**	3	1	2	2	3
Stability during operations	**10,0**	3	3	3	3	1
Reconstruction time	**7,55**	3	1	2	1	3
Reconstruction costs	**8,36**	3	1	2	2	3
Initial construction time	**4,27**	1	3	2	2	1
Initial construction costs	**5,09**	1	3	3	2	1
Residual value of structure	**6,73**	3	1	3	2	3
Stability of soil after removal	**1,00**	1	1	1	3	3
Connection with terminal area	**1,82**	3	3	3	2	1
Environmental impact	**5,91**	3	1	3	2	3
Possibility of deepening port basin	**2,64**	1	1	1	2	3
Earthquake resistance	**3,45**	2	2	2	3	3
Total score		**169**	**112**	**158**	**139**	**156**
Percentage of maximum score [%]		*85*	*56*	*80*	*70*	*79*

Table 17.3 – scores of multi criteria analysis including weight factors

According to this more detailed analysis, a caisson quay wall has the highest score, but scores of a floating quay and a block-work wall are still quite close. It can be seen from the table that properties of the alternatives differ a lot on various criteria.

Sensitivity to the assumption for stability of the soil after removal of the quay and the choice to consider flexibility over different locations only, is check in the next paragraph. Since the importance of these two criteria can differ, the weight factors are also determined for the situation in which future deepening is desirable too and stability of the terminal area remains important after relocation.

17.2. Sensitivity

When future deepening of the harbor basin is also considered as an important flexible property and when stability of the soil after relocation is assumed to be important, the weight factors are determined as given in table 17.4.

	Flexibility to relocate	Stability during operations	Reconstruction time	Reconstruction costs	Initial construction time	Initial construction costs	Residual value of structure	Stability of soil after removal	Connection with terminal area	Environmental impact	Possibility of deepening port basin	Earthquake resistance	**Total score**	**Normalized weight factor**
Flexibility to relocate	-	0	1	1	1	1	1	1	1	1	1	1	10	**9,18**
Stability during operations		-	1	1	1	1	1	1	1	1	1	1	11	**10,0**
Reconstruction time			-	0	1	1	1	1	1	1	0	1	7	**6,73**
Reconstruction costs				-	1	1	1	1	1	1	0	1	8	**7,55**
Initial construction time					-	0	0	0	1	0	0	1	2	**2,64**
Initial construction costs						-	0	0	1	0	0	1	3	**3,45**
Residual value of structure							-	1	1	1	0	1	6	**5,91**
Stability of soil after removal								-	1	0	0	1	4	**4,27**
Connection with terminal area									-	0	0	0	0	**1,00**
Environmental impact										-	0	1	5	**5,09**
Possibility of deepening basin											-	1	9	**8,36**
Earthquake resistance												-	1	**1,82**

Table 17.4 – determination of new weight factor in order to determine sensitivity

1) Stability during operations
2) Flexibility to relocate
3) Possibility of deepening port basin
4) Reconstruction costs
5) Reconstruction time
6) Residual value of structure
7) Environmental impact
8) Stability of soil after removal
9) Initial construction costs
10) Initial construction time
11) Earthquake resistance
12) Connection with terminal area

When flexibility at a fixed location is combined with flexibility over different locations and when the terminal area must remain stable, the following scores are obtained.

	Applied weight factor	Caisson quay wall	Sheet piled combi-wall	Mass concrete block-work wall	Suspended deck on piles	Floating quay wall
Flexibility to relocate	**9,18**	3	1	2	2	3
Stability during operations	**10,0**	3	3	3	3	1
Reconstruction time	**6,73**	3	1	2	1	3
Reconstruction costs	**7,55**	3	1	2	2	3
Initial construction time	**2,64**	1	3	2	2	1
Initial construction costs	**3,45**	1	3	3	2	1
Residual value of structure	**5,91**	3	1	3	2	3
Stability of soil after removal	**4,27**	1	1	1	3	3
Connection with terminal area	**1,00**	3	3	3	2	1
Environmental impact	**5,09**	3	1	3	2	3
Possibility of deepening port basin	**8,36**	1	1	1	2	3
Earthquake resistance	**1,82**	2	2	2	3	3
Total score		**159**	**102**	**145**	**141**	**164**
Percentage of maximum score [%]		*80*	*52*	*73*	*71*	*83*

Table 17.5 – new scores in order to determine the sensitivity

As a result of combining both types of flexibility, the floating quay wall has become more favorable, because it allows future deepening of the harbor basin and it is not a soil retaining structure. The most flexible structure with respect to both types of flexibility is therefore probably a floating quay wall.
However, its floating stability and the connection with the terminal area must be carefully determined, since these are the main drawbacks of such a structure compared to a caisson structure.

17.3. Conclusions regarding quay wall properties

First of all, one can conclude that a sheet piled combi-wall is not a suitable quay wall type for flexible purposes. It is very well anchored in the soil, which makes it simply unfeasible to relocate. Combi-walls and other structures that are anchored in the soil are therefore not likely to offer an attractive flexible alternative.

A mass concrete block-work wall could be a very attractive alternative for small retaining heights, but for large container vessels it is less flexible. The large retaining height results in very large and heavy blocks, since it is a gravity structure. Heavy equipment is required and the blocks can only be transported on vessels or barges. The reinforced top layer is casted on site and cements the under lying blocks together. This is necessary to resist hawser forces, but it makes it much harder to relocate the blocks.

A deck on piles could be a possible solution, unless the required number of piles turns out to be very high. Driving steel piles is time consuming and costly and the feasibility of reusing the piles is doubtful. The needed number of piles will therefore be investigated in a later stage of this report, but this alternative is not likely to offer the best flexible solution. A benefit of a deck on piles is the possibility of future deepening of the harbor basin to some extent.

A floating structure seems to be a very flexible solution, since it can easily be disconnected from its mooring system and can be towed to a new location. This alternative can also be combined with flexibility at location, since future deepening of the harbor basin is not a problem. If such a structure is stable enough to guarantee an acceptable operability, it would be a very promising alternative. Further research on stability of a floating quay will therefore certainly be part of this thesis.

Constructing a quay wall with caissons seems to be a very good alternative as well. Caissons can also be towed to a different location, just like a floating quay. A disadvantage compared to a floating quay is collapse of the soil after removal and the increased height, since it is immersed on the bottom. However, an advantage in comparison with a floating quay is its stability. Especially if the stability of a floating structure turns out to be problematic, a caisson structure is a very attractive alternative.

17.4. Conclusion regarding multi criteria analysis

It can be concluded from the multi criteria analysis that the caisson quay wall has the best scores, but in the sensitivity analysis, the floating quay turns out to have a slightly higher score.

From the matrices, it can also be observed that the alternatives differ a lot in properties, which gives a wide range of scores in the matrix. Therefore it would be a good idea to combine the aspects that lead to desirable properties of a flexible quay wall structure. Stabilizing a floating structure would be an interesting solution to combine the flexibility of a floating structure with the stability of a fixed one. A structure that scores well on both stability and transportability will achieve a higher score.

In chapter 18 it is tried to establish six new alternatives which combine the desired properties. In contradiction to the 5 alternatives of the multi criteria analysis, many calculations on the technical feasibility of these new alternatives will be carried out. Results can be found in the appendix.

18. Steps in design procedure

18.1. Flexibility of the structure

As stated in the first part of this report, one can distinguish between flexibility over different locations and flexibility at a fixed location. During this thesis the focus will be on flexibility over different locations. Arguments for this choice are given in the last paragraph of the literature review.

18.2. Construction material

In chapter 11 this report, the main properties of steel and concrete are listed with both advantages and disadvantages. A more detailed consideration can be found in that chapter, where concrete is considered to be the most suitable construction material, mainly based on construction costs and maintenance.

18.3. Method of transportation

Flexibility over different locations means that the quay wall can be transported to a different location and can there be reconstructed within a relatively short construction period. Since quay wall elements are large and heavy, transport over water would be the easiest and cheapest way to move the quay to a different place.

Transportation over water can either be done by transporting non-floating quay elements on barges and vessels, or by making quay elements self-floating and towing them by tug boats. When the first method is used, heavy lifting equipment will be required to place the elements on barges. All elements should also be designed with connection points for hoisting equipment, and the strength of the elements needs to allow hoisting as well.

A design of self-floating quay elements is chosen to be the most suitable transportation method. However, one must realize that towing the quay elements to a new location might be time consuming, depending on the sailing distance. One can consider a streamlined bow that can be attached to the front of a floating element to reduce the drag force. Whereas non-floating elements have to be designed for hoisting forces, floating elements have to be designed for larger wave forces during transport and tug forces.

The STS cranes on the quay wall will be transported by means of a suitable vessel. It is calculated that large floating quay wall elements provide sufficient stability to carry the STS cranes themselves. However, for practical reasons they will be transported apart from the quay wall.

Figure 18.1 – transportation method of ship-to-shore cranes [www.cochinsquare.com]

18.4. Operational stability

Stability of the quay wall is crucial for the operability and productivity of the quay. Motions should therefore be analyzed in detail when a floating quay wall is used. It can be seen in the literature review that detailed research on the dynamic behavior of a floating quay and moored vessels has already been carried out, but the interaction between quay and vessel turned out to be very hard to model. Results on governing movements are therefore doubtful.

18.5. Stabilization methods

To stabilize a floating deck structure, six possible solutions could be considered. Each of them will be described in the next subparagraphs. Table 18.1 shows motion tolerance for a container vessel moored at a fixed quay, recommended by PIANC guidelines. Definitions of the six motions can be found in figure 6.4 in the literature review of this report.

Efficiency	*Surge [m]*	*Sway [m]*	*Heave [m]*	*Yaw [°]*	*Pitch [°]*	*Roll [°]*
100%	*0.5*	*0.3*	*0.3*	*0*	*0*	*1*
50%	*1.0*	*0.6*	*0.6*	*0.75*	*0.5*	*3*

Table 18.1 – motion tolerances for container vessel moored at a fixed quay [PIANC]

18.5.1. Jack-up system

The jack-up system is a well-known stabilization method in the world of offshore engineering. A ship or platform is lifted completely out of the water by means of at least three steel legs that are lowered to the sea bed. These legs can consist of either hollow steel piles, or a steel framework.

Figure 18.2 – jack-up vessel with hollow pile legs [www.windmanagement.co.uk]

The advantage of a jack-up system is the fast shift between mobilization and stabilization. For this reason, the system is very appropriate for construction methods that required a continuous shift between these two conditions. The construction of offshore wind turbines for instance.

A disadvantage is the extremely strong deck structure that is required to resist the loads in the stabilized situation. The buoyancy force is completely eliminated and the structure can now be seen as a deck structure on only a few supports. Jack-up platforms are nearly always steel structures for this reason.
A jack-up system could certainly be a solution to stabilize the floating deck structure of a flexible quay wall. However, the frequency of mobilization and transportation of a quay wall is assumed to be quite low. A cheaper stabilization method by means of fixed piles is likely to be cheaper and would probably suffice as well.

In order to reduce the compressive force on the jack-up legs and the bending moments in the deck, one can decide to lift the quay wall not completely out of the water, but still the span between the jack-up legs remains problematic for the strength of the deck. An important issue regarding a jack-up system for quay facilities is the absorption of horizontal forces. Berthing energy of container vessels and wind loads are both severe forces that need to be absorbed by the jack-up legs. One must also realize that the presence of jack-up legs may not hamper the movement of handling equipment on the quay.

Figure 18.3 – jack-up vessel with framework legs [www.knudehansen.com]

Figure 18.4 – illustration operational floating quay wall with jack-up system

18.5.2. Ballast water and counter weights

Stabilizing a floating quay wall by means of ballast water tanks or counterweights is possible to a certain extent, but the force due to container lifting is very hard to counteract. When a ship-to-shore crane lifts a container at the end of its boom, a severe tilting moment is exerted on the floating structure. This momentum is established very quickly and is almost impossible to counteract. The result is a certain tilt angle of the quay and a significant vertical displacement of the crane boom tips. The thesis of Van der Wel already proved that the displacement due to container lifting exceeds the PIANC guidelines with about factor three for very large container vessels.

When ship-to-shore cranes move along their rails, the tilting moment in longitudinal direction changes. These changes occur much slower and are therefore easier to counteract. A mobile counterweight could for instance be placed in the hollow area inside the floating structure, or high capacity water pumps could be used.

The motions that are mentioned so far, all originate from movements of equipment on the quay itself. However, the structure is also exposed to external forces like wind, waves and currents. Interactions of ship motions and motions of a floating quay wall are very hard to model and might result is significant amplifications when oscillations approach the natural oscillation period.

This method also needs strong connections between the floating quay elements. The use of a stabilized floating structure by means of ballast systems and counterweights is assumed to be only possible in a very calm wave climate for small to medium size equipment. Therefore, this method of stabilization doesn’t seem to be a good solution for the scope of this thesis.

18.5.3. Partly pulled under water

Another method to make a floating structure more stable is pulling it partly under water by means of chains connected to ground anchors. The buoyancy force increases when a floating structure is pulled down, making it more stable. This extra buoyancy force is divided over the number of chains that pulls the quay wall down. A horizontal load on the pre-tensioned floating body, will result in a horizontal displacement. The chains now make a certain angle with the vertical plane, which results in a horizontal component of the tensile force in these chains. This component counteracts the horizontal load that acts on the structure and becomes larger when the floating body is pulled deeper under water.

When tension chains are located at each corner of the quay wall elements and the floating body is pulled further under water than the wave amplitude, motions are significantly restricted. Pitch, roll and heave are completely eliminated when each tension chain remains tight and tensioned. Yaw, sway and surge are not eliminated, but each of these motions is dampened by the resulting horizontal component of the tension chains.

Mentioned properties sound very promising, so the design of this stabilization method has been investigated by calculation the magnitude of acting forces with an Excel sheet. The dimensions of the elements and the distance they are pulled down play a major role in the resulting forces, just like the number of tension chains used.

The main problem that arises for this stabilizing method is creating a ground anchor that offers sufficient tensile capacity. There are some types of suction anchors that could possibly deal with the forces, but still many anchors are required. Besides, suction anchors cannot be applied in each type of subsoil.

The tide creates another problem. Rising water levels result in a massive increase of the tensile forces. However, this problem could be solved by making the chain length variable or by using ballast water inside the floating body to increase the natural draught.

This stabilization method also requires a connection between the floating elements, to avoid relative motions of quay elements. Just like the quays with a jack-up and ballast system, it needs a connection bridge towards the container storage yard on land.

Calculations on acting forces and displacements of a pre-tensioned floating quay wall can be found in appendix D.

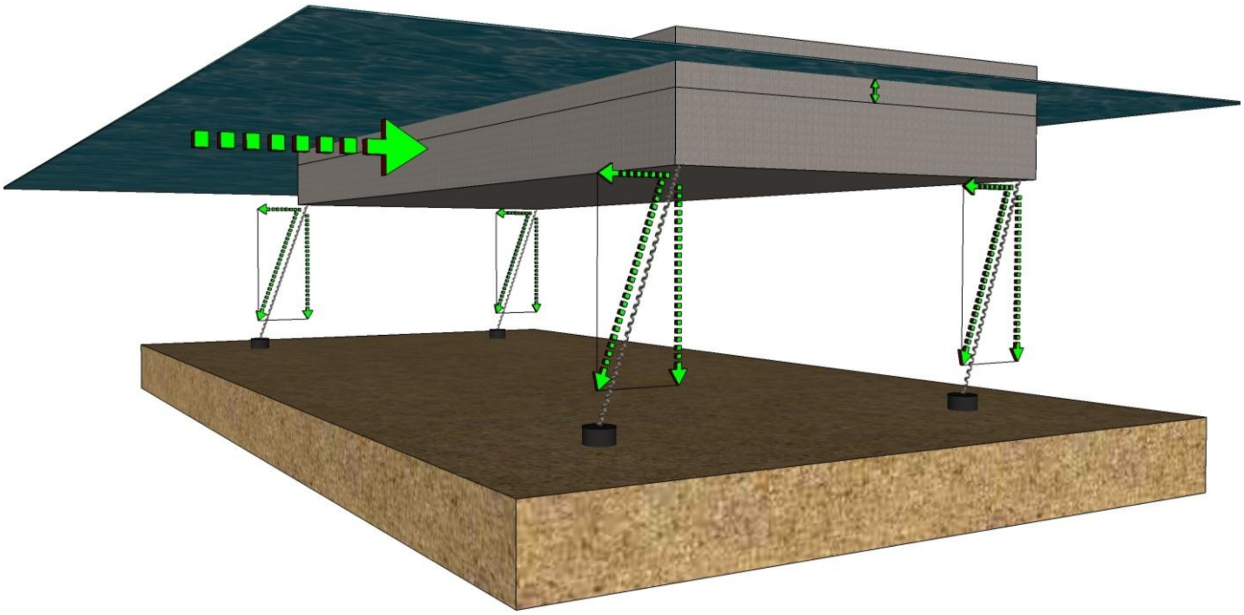


Figure 18.5 – principle of pre-tensioned floating quay wall

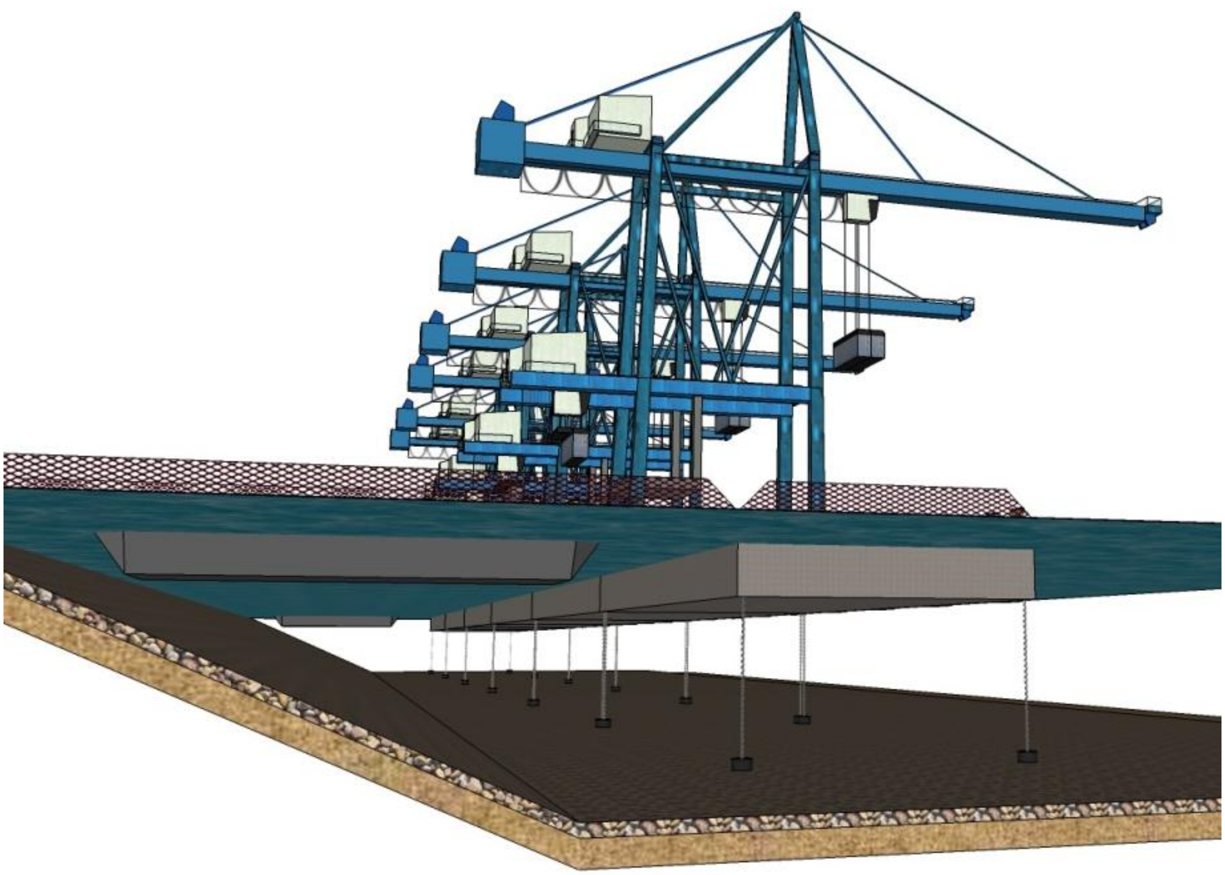


Figure 18.6 – illustration pre-tensioned floating quay wall

18.5.4. Immersed deck on pile foundation

Because this thesis is about designing a flexible solution for quay wall structures, floating elements are a good alternative to traditional quay wall structures, because they are relatively easy to mobilize.

Motions of floating elements remain a major problem, but can be eliminated by immersing the structure on a solid foundation. The bottom of the harbor basin could function as such a foundation, but this would make future deepening of the harbor basin almost impossible.

A possible solution could be a foundation on piles, just like alternative 4. Caissons can be brought in position above the piles and can then be immersed on the piles by filling them with ballast water. No connection has to be applied if there is sufficient friction and downward pressure between the piles and the deck, but one should check whether the pile heads punch through the caissons or not. The entire quay can be constructed with a number of caissons, which do not need a connection either, when they are all stably immersed on the piles.

When the structure has to be moved to a different location, the ballast water can be pumped out of the caissons, making it a floating structure again. The caissons of the quay wall can now be towed to their new location, separately.

Steel piles could be pulled out of the soil again, but financial feasibility of reusing them is doubtful. Concrete piles are cheaper, but are also weaker and usually cannot be reused. If the length of the piles is insufficient at the new location, new piles have to be driving anyway. Therefore the number of piles needed is a very important issue. If just a couple of piles per caisson will suffice, this might be an interesting alternative. If lots of piles are needed, this solution does not sound very promising.

Piles placed under a certain angle or abutments on land can be applied if the absorption of horizontal forces on the quay appears to be a problem. The piles and abutments are not that flexible with respect to relocating the quay wall somewhere else. However, this alternative does allow future deepening of the harbor basin, to some extent.

The connection between the quay and the terminal area may need a small sheet pile wall, because of the draught of the caissons. This can be a simple wall with a retaining height, somewhat higher that the height of the caissons.
A cheaper solution would be to backfill the soil underneath the terminal pavement directly to the caissons. In that case the caisson must also be able to absorb the horizontal soil pressure and the soil itself must be stable to avoid settlements at the terminal side. Note that removing the quay will cause serious damage to the pavement of the terminal, when the soil is directed backfilled against the caissons.

A large tidal difference will cause trouble for this type for structure. During construction, the caissons are brought into position above the pile heads during high tide. Once the floating elements are immersed on the piles, they must have enough vertical downward pressure on the piles during both high and low water. The pressure on the pile heads will become much higher at low tide and could exceed the bearing capacity of the piles, because the upward buoyancy force is dramatically decreased. This problem can be solved by installing water pumps inside the caissons, so one can adjust the ballast water level to compensate for the varying buoyancy force. Using the tide for this will be impossible, since the surrounding water level will remain higher than the water level inside the caissons in both conditions. A more simple method is to use more piles or increase their bearing capacity.

Figure 18.7 – illustration immersed floating deck on pile foundation

Several calculations on foundations piles can be found in appendix J.

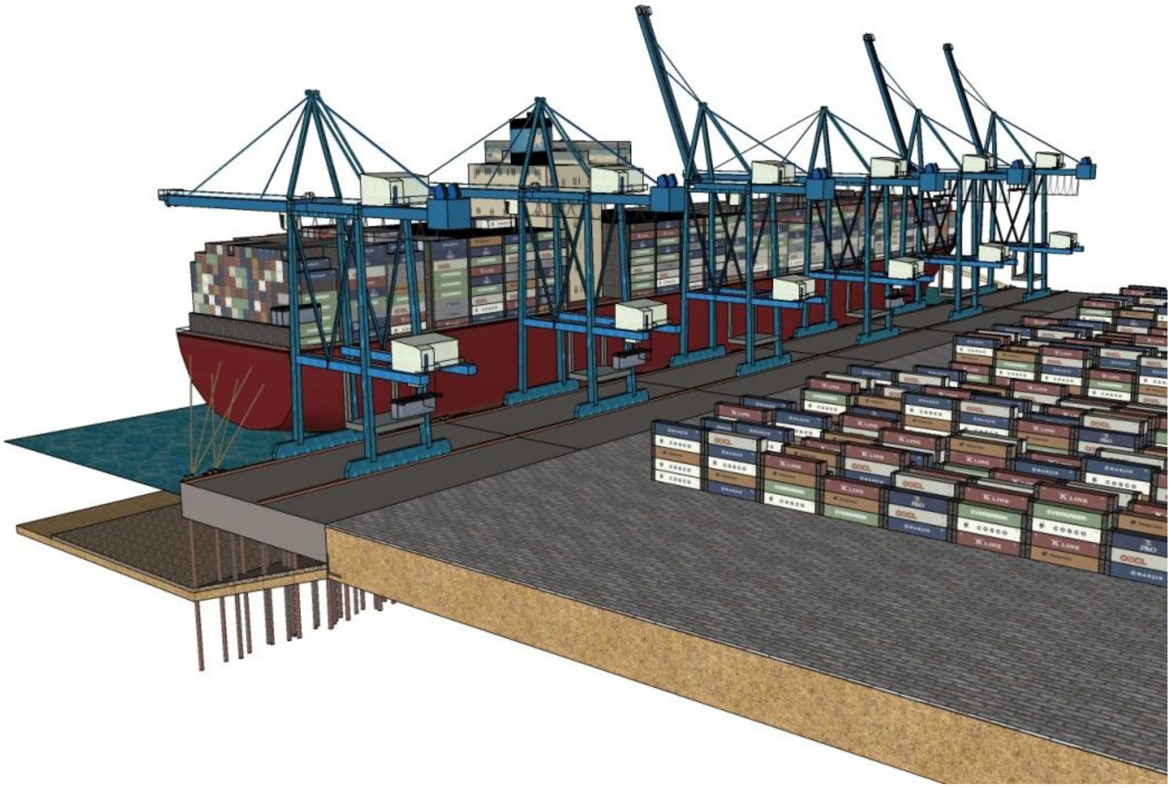


Figure 18.8 – illustration operational immersed floating deck on a pile foundation

18.5.5. Floating deck with spud piles

It is also possible to absorb the horizontal forces on a floating deck, by means of four large spud piles in the corner of each floating element. Spud piles are piles that are dropped through a hole in the deck and penetrate into the bottom by means of their weight and velocity. When frequent shifts of the structure are required, a hydraulic system is often used to lift the spuds out of the soil again. This method is very common in dredging technology on cutter suction dredgers and backhoes. When the frequency of shifts is much lower, spud piles can be dropped and lifted by external equipment.

When spud piles are used to absorb the horizontal forces, one can decide to use a connection bridge towards to terminal area. By applying such a bridge, the terminal area doesn't collapse when the quay is transported to a different location.

The length of the spuds is of importance, in order not to hamper the crane operations on the quay wall. When the length above the deck is too large, crane movements may be restricted. When the piles are cut to a certain length after installation, they could be too short when the quay has to be relocated. The same problem yields for jack-up piles.

One must realize that spud piles are not connected to the deck, so the structure can still move up and down. The roll angle can only be reduced when the spuds fit very tightly trough the deck, but this increases the loads on both piles and deck significantly.

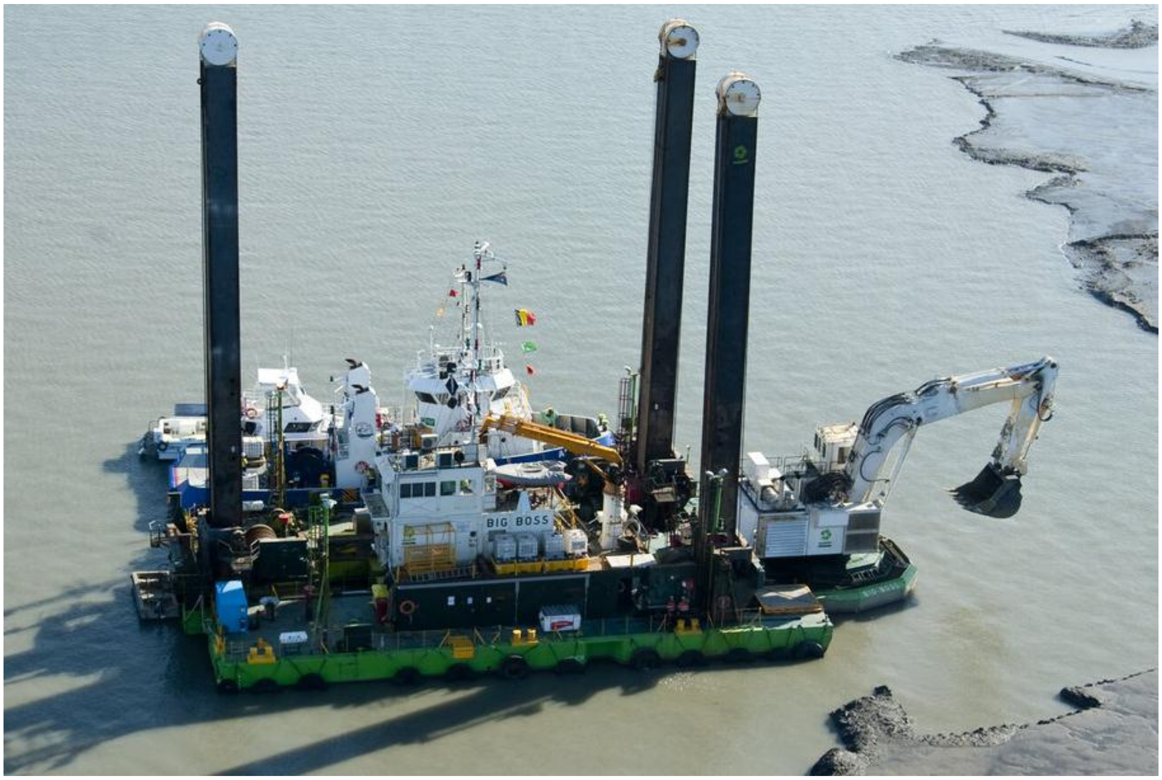


Figure 18.9 – backhoe dredger with spud piles [www.dredgingpoint.org]

Figure 18.10 – illustration operational floating deck with spud piles

18.5.6. Caissons on top of each other

Another alternative is to place two caissons on top of each other. No foundation piles are needed in this case, which makes the structure more flexible. Drilling many piles into the subsoil takes time and removing them when the structure is relocated is also time-consuming and costly.

Figure 18.11 – illustration operational structure with two caissons on top of each other

By immersing the caissons with ballast water, they can be mobilized again, by pumping the water out. Disadvantage of filling the caissons with water, is the fact that water is much lighter than concrete or rubble material. This will result in a reduced weight of the structure, which has a negative effect on the soil retaining stability. Filling the caissons with sand is also possible, but pumping the sand in and out is a bit more complicated compared to ballast water.

Since the caissons serve as a retaining wall, the horizontal ground pressure may not lead to overturning or sliding aside. Both caissons do not need any connection if sufficient downward pressure is present, to resist the horizontal forces. However, the total pressure may not exceed the soil's bearing capacity. The horizontal water pressure is reduced by applying a permeable backfill just behind and under the caissons, so water level differences between land and sea side are minimal. A filter layer must be applied to prevent sediment seepage, which will result in settlements at the land side.

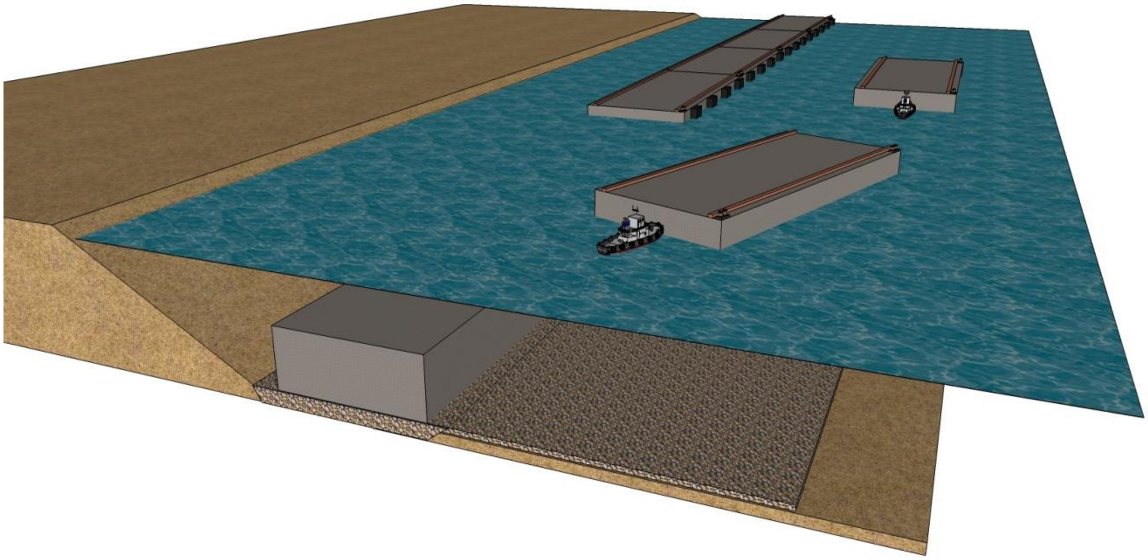


Figure 18.12 – illustration of relocating a quay with two caissons on top of each other

Applying a caisson foundation instead of a pile foundation may result in less reinforcement steel and a reduced wall thickness of the caissons, since the bending moment caused by the ship-to-shore cranes is significantly reduced. However, in practice it might turn out that the minimum reinforcement percentage or the transportation over water is governing for the required strength. Whether this is indeed the case or not, will be investigated in part IV of this report.

Using two caissons on top of each other results in a significantly reduced draft of each caisson, compared to the situation with only one single caisson per unit of retaining height, as mentioned in chapter 12. This may be favorable with respect to transportability. However the draft of one single caisson is still likely to be less than that of a container vessel. So if a container vessel can reach a certain location, the quay elements can reach it as well.

It might even be more favorable to use one single caisson per unit of retaining height, due to dynamic wave loads during transport. Such a caisson is capable to resist larger wave loads. Inner walls are present to provide more strength and to support the crane rails of STS cranes on the quay.

A disadvantage of using a caisson foundation instead of a pile foundation is the loss of flexibility with respect to future deepening of the harbor basin. Besides, a trench needs to be dredged, which is later filled with gravel to provide a good foundation. This gravel bed foundation is also needed to make caissons float again for relocation. When the caissons are positioned on an impermeable soil layer, they will stick to the seabed when the ballast is pumped out.

19. Chosen stabilization method

The previous chapter described six possible alternatives for flexible quay wall structures. This chapter will judge which of these six is most suitable as a flexible quay. First the six alternatives are summarized in table 19.1.

Possible alternatives for flexible quay wall structure
Jack-up quay wall
Counter ballasted floating quay
Pre-tensioned floating quay
Immersed deck on piles
Floating deck with spud piles
Caissons on top of each other

Table 19.1 – established quay wall alternatives summarized

To selected one of these alternatives one has to research if each of the alternatives is possible in the first place. Therefore, calculations have been made to investigate the technical feasibility of each alternative.

19.1. Calculations made

Table 19.2 summarizes the calculations that have been made for each of the proposed quay wall types. Results of these calculations can be found in the appendix of this report.

	Jack-up quay wall	Counter ballasted floating quay	Pre-tensioned floating quay	Immersed deck on piles	Floating deck with spud piles	Caissons on top of each other
Static stability of floating body	x	x	x	x	x	x
Natural oscillation period for dynamic stability	x	x	x	x	x	x
Static stability during immersion				x		x
Tilt angle caused by crane load and movement		x				
Tilt angle caused by container lifting		x				
Momentum on deck caused by crane load and spans	x			x		
Wall thickness and reinforcement of bending moment				x		x
Horizontal soil pressures				x		x
Horizontal stability against sliding	x			x		x
Resulting momentum on entire structure	x	x	x	x	x	x
Stability against overturning						x
Influence of ballast water inside structure		x	x	x	x	x
Required bearing capacity of the soil	x			x		x
Required freeboard for wave overtopping discharge	x	x	x	x	x	x
Bottom protection against scour of propeller wash	x	x	x	x	x	x
Horizontal loads caused by tidal current	x	x	x	x	x	x
Vertical and horizontal wind load on structure	x	x	x	x	x	x
Berthing energy of vessels and fender reaction	x	x	x	x	x	x
Number of piles needed for foundation				x		
Location and magnitude of bending moment in piles				x		
Driven pile depth and absorbable momentum by soil				x		
Occurring stress in foundation piles				x		
Pile stiffness and quay wall displacements				x		
Anchor forces in subsoil			x			
Displacements of pre-tensioned floating body			x			
Angle of anchor lines and required strength			x			
Wave load in operational and floating conditions	x	x	x	x	x	x

Table 19.2 – overview of calculations made for established quay wall alternatives

Note that not each specific situation is attached to the appendix separately, in order to keep the number of appendices acceptable. Since all calculations were made in Excel, one can just adapt some value in order to calculate the results for a somewhat different situation. The Excel sheets in the appendix provide all calculation methods that are needed to obtain results for each situation given in table 19.2. The equations used in the Excel sheets are either described in part IV of the report or in the appendix itself.

19.2. Unattractive alternatives after calculations

Based on calculations on various aspects, one can already eliminate some of the alternative.

19.2.1. Main arguments against ballasted floating quay wall

Stabilizing a floating structure by means of counter weights or ballast water turns out to be very difficult. Especially the tilting momentum caused by container lifting acts so suddenly, that it is almost impossible to counteract. Calculations show that the vertical movement of the crane tip already exceeds the PIANC guidelines mentioned in table 18.1. Since the calculated tilt angle is based on the situation in which all STS cranes lift a container at the end of their boom at the same time, the occurrence of this specific situation will be very low. However, this calculation excludes ship motions and motions of the floating quay itself.

The natural oscillation period of a ballasted floating quay wall is determined at 5 to 6 seconds, which is not an unlikely wave period to occur inside a harbor basin. Trouble with respect to response motions to waves can therefore be expected. The wave direction is an important factor which influences the response of the structure to waves. Container terminals are often very well protected against direct wave penetration from wind waves, but wave reflection on vertical walls inside the port plays an important role. Waves generated by passing ships are another wave source inside a harbor basin. Note that container lifting operations could also cause trouble with respect to the quay's natural oscillation frequency.
Because of the calculated tilt angle for container lifting and the uncertainties in quay motions, it is decides that a ballasted floating container quay in not a reliable alternative for flexible quay wall structures for panamax container vessels.

19.2.2. Main arguments against immersed deck on piles

An immersed deck on piles would have been a nice alternative which would also allow future deepening of the harbor basin to some extent. However, calculations proved that many piles are needed to support the deck structure. The required strength of the deck and the bearing capacity of all piles in the subsoil are the most important factors for this. Depending on the local conditions, 4 to 8 piles can be driven in one day per piece of equipment. For a spacing of 6 meters between the piles, one needs about 900 piles for a quay with two berths, so driving piles is costly and takes a lot of time.
The flexibility of a pile foundation is also doubtful, since reusing the piles is likely to be more expensive than applying new ones at a different location.

Applying an immersed deck on a pile foundation turns out to be possible, but with respect to flexibility over different locations, it is not likely to be the best solution.

19.2.3. Main arguments against jack-up system

A jack-up system requires an extremely strong deck structure to allow STS cranes to drive along the span between the jack-up legs. Construction is therefore only possible with steel, which makes the structure much more expensive than the other alternatives. Applying more jack-up legs reduces the load on the deck, but these legs are very expensive as well.
Issues with respect to overturning moment can also be expected, but costs and construction material are the main reasons for abandoning this alternative.

19.2.4. Main arguments against floating quay with spud piles

Using spud piles on a floating structure could be a solution, but very strong piles are required to absorb the horizontal forces on the quay wall. When the spuds fit very tightly in the holes, they will reduce the roll angle of the quay, which is required as provided by the calculated tilt angle. However, this stabilization method introduces enormous forces on the deck as well.
An alternative with spud piles will therefore not be elaborated.

19.2.5. Main arguments against pre-tensioned floating body

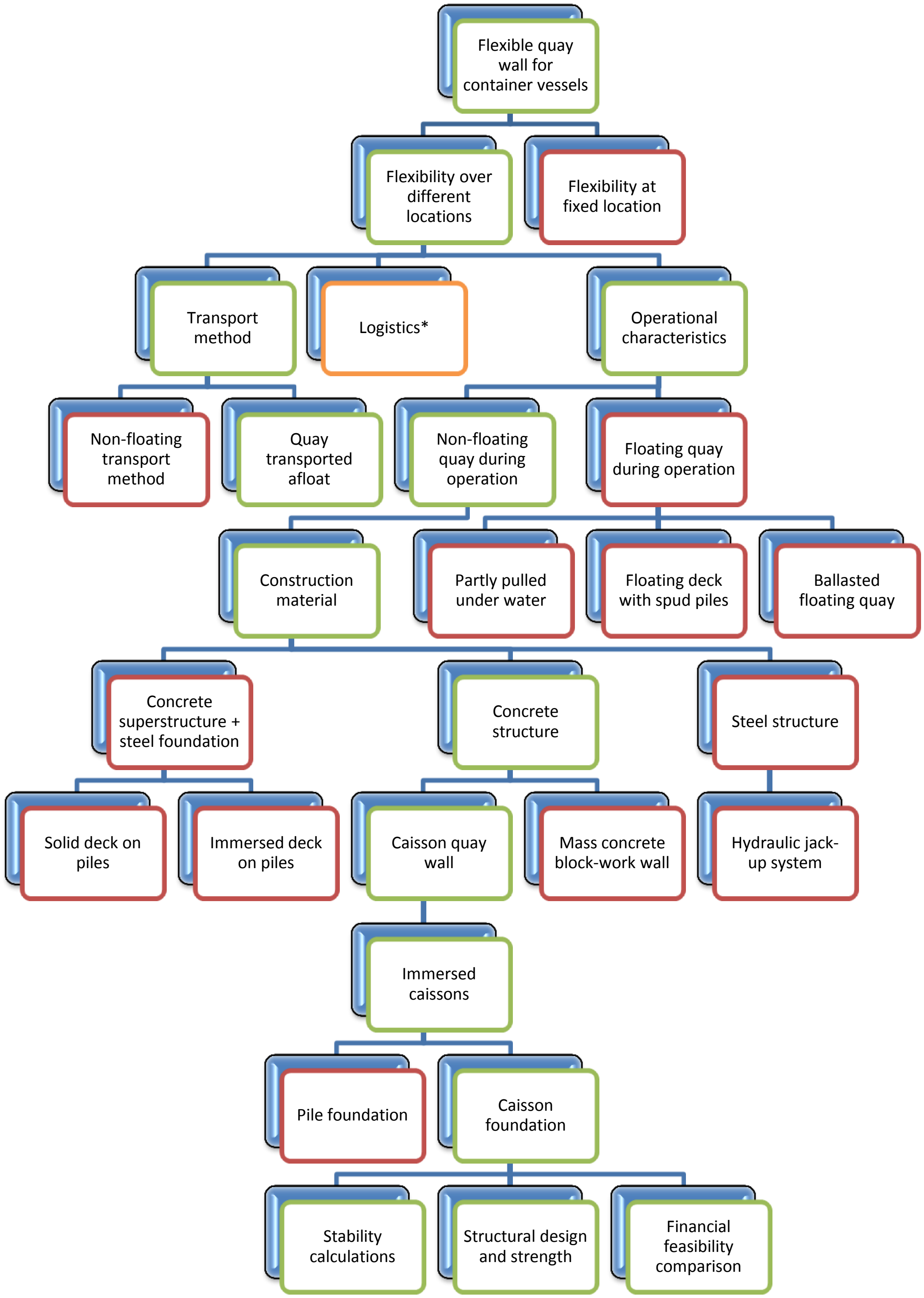
The properties of a pre-tensioned floating body are very promising, but unfortunately, the corresponding anchor forces turned out to be massive. Creating anchors in the sea bed that provide sufficient capacity is therefore problematic.
A ballast system or winches are needed to compensate for the tidal differences, unless there is no tide at all. Applying such a system is risky, since failure of the system could lead to serious damage of the quay.
Pre-tensioning a floating body will also result in a shorter natural oscillation period. This frequency has not been calculated for the pre-tensioned floating quay, but it could lead to trouble as well.

19.3. Selection of best alternative to be elaborated

After many considerations and calculations on possible flexible quay wall structures for container vessels, the alternative with two caissons placed on top of each other, seems the most promising structure. In fact, this structure can be seen as a combination of a caisson quay and a mass concrete block-work wall as mentioned in chapters 12 and 14. The main advantages and disadvantage were already described in chapter 18.5.6. Looking back to the outcome of the multi criteria analysis in chapter 17 and the poor stability of a floating quay wall, this conclusion seems to make sense.

The final design of this quay wall type will be elaborated in part IV of this report.

20. Choice tree of design procedure



* = basic logistics of handling equipment by means of queuing theory only

Figure 20.1 – choice tree of design procedure

Part IV
Structural Design

21. Use of Excel sheets

In part IV of this master thesis the structural design of the flexible quay wall structure will be investigated. First all loads acting on the quay have to be determined, before the required strength and dimensions can be calculated.

To calculate the required dimensions, the loads and the required strength, Excel sheets were made. The sheets contain all formulas and calculate new results, when new input values are given. First it takes a while to create such an Excel sheet, but it will save a lot of time when more and more aspects are taken into account.

Dimensions, loads and strengths are directly related to each other. If one changes for instance the dimensions of the caissons, nearly all other values are affected. Some criteria that did suffice for the previous dimensions may now be insufficient and vice versa. Before reading the next chapters on loads, stability and strength, it might be useful to already know the final caisson dimensions in advance. In chapter 27.1, the final caisson dimensions were determined at 100m in length, 33m in width and 11m high. Motivations for this choice can be found in chapter 27.1 as well. Obviously, these dimensions were determined after studying the loads, stability and strength.

An Excel sheet is also a handy tool to optimize the input values by attuning them to the related results. Many of the Excel sheets can be found in the appendix of this report.

22. Loads acting on the structure

This chapter describes the loads on the flexible quay wall structure and how to determine them. The main loads to be considered are listed below:

- Weight of the structure itself
- Static water pressure
- Soil pressure
- Surface loads at landside
- Wind load and bollard pull
- Berthing energy and fender reaction force
- Wave load during transport
- Wave load during operation
- Propeller wash
- Wave overtopping
- Equipment on the quay
- Tug boat forces during transport
- Earthquake loads

22.1. Weight of the structure

The weight of the structure itself is a load that acts on the foundation and creates stability against sliding and overturning. However, a too large weight may cause a slide circle in the subsoil. The weight also determines the draught in floating conditions from which the floating stability can be derived.

The total weight of the structure itself can easily be determined by multiplying the total volume of concrete and the density of concrete.

$$m_{concrete} = \rho_{concrete} \cdot V_{concrete} \qquad \textit{Equation 22.1}$$

Where: $m_{concrete}$ = mass of concrete [kg]
$\rho_{concrete}$ = density of concrete [kg/m^3]
$V_{concrete}$ = volume of concrete structure [m^3]

To calculate the vertical force downward, the buoyancy force generated by the displaced water has to be subtracted from the total weight of the structure.

$$F_{vertical;structure} = (m_{concrete} \cdot g) - F_{buoyancy} \qquad \textit{Equation 22.2}$$

Where: $F_{vertical;structure}$ = downward vertical force by the structure [kN]
$m_{concrete}$ = mass of concrete [ton]
g = gravitational acceleration [m/s^2]
$F_{buoyancy}$ = upward force generated by the displaced water [kN]

The equation of the buoyancy force is given in the paragraph “static water pressure”. Note that the buoyancy force is not constant along the width of the structure when there is a water level difference between both sides. As a result, there will also be an overturning momentum present in this case.

22.2. Static water pressure

Static water pressure acts on all structures that are built in the water. The pressure increases linear with the water depth and can easily be calculated by equation 22.3.

$$p_w = \rho_w \cdot g \cdot h \qquad \textit{Equation 22.3}$$

Where: p_w = water pressure at depth h [kN/m^2]
ρ_w = density of water [ton/m^3]
h = water depth [m]

The resulting horizontal force can be calculated by means of equation 22.4 and acts at $^2/_3$ of h below the water level.

$$F_{w;hor} = ½ \cdot \rho_w \cdot g \cdot h^2 \qquad \textit{Equation 22.4}$$

Where: $F_{w;hor}$ = resulting horizontal force [kN/m]

The buoyancy force generated by the displaced volume of water can be calculated with equation 22.5 and acts in vertical upward direction.

$$F_{buoyancy} = \rho_w \cdot g \cdot h \cdot A_{bottom\ slab} \qquad \textit{Equation 22.5}$$

Where: $F_{buoyancy}$ = upward force generated by the displaced water [kN]
$A_{bottom\ slab}$ = surface of bottom slab caisson [m^2]

22.3. Soil pressure

When the structure is used to retain the soil, this will cause an extra horizontal load on the structure. To determine this horizontal load, one must first determine the vertical pressures. The vertical pressure from the soil can be calculated in the same way as described in equation 22.3, by using the density of the soil instead of the water density. The horizontal soil pressure of dry soil can then be determined by multiplying it with a horizontal soil coefficient, as stated in equation 22.6.

$$p_{soil;hor;dry} = \rho_{soil\,;dry} \cdot g \cdot h_{soil} \cdot k_{hor} \qquad \textit{Equation 22.6}$$

Where: $p_{soil;\,hor;\,dry}$ = horizontal soil pressure of dry soil [kN/m^2]
$\rho_{soil\,;dry}$ = density of dry soil [ton/m^3]
h_{soil} = thickness of the soil layer [m]
k_{hor} = horizontal soil coefficient [-]

This calculation is somewhat different in case of wet soil, since horizontal and vertical water pressures are equal. One first determines the grain stress by subtracting the water pressure as determined in equation 22.3. The horizontal soil pressure follows from the grain stress multiplied by the horizontal soil coefficient. To total horizontal pressure is defined by the sum of this horizontal grain pressure and the water pressure.

$$p_{soil;hor;wet} = [(p_{soil;vert;\,wet} - p_w) \cdot k_{hor}] + p_w \qquad \textit{Equation 22.7}$$

Where: $p_{soil;\,hor;\,wet}$ = horizontal pressure caused by wet soil [kN/m^2]
$p_{soil;\,vert;\,wet}$ = vertical wet soil pressure [kN/m^2]

Pressures increase linearly with the layer thickness, but one must distinguish between wet and dry layers and layers with different soil properties. The resulting horizontal forces can be calculated by multiplying the pressures with the area, keeping in mind that the pressure distribution has either a rectangular or a triangular shape.

For caissons, the neutral horizontal soil coefficient must be used, which is given by:

$$k_{hor} = 1 - \sin(\varphi) \qquad \textit{Equation 22.8}$$

Where: φ = angle of internal friction [degrees]

The overturning momentum that results from the retained soil must be determined by calculating the main attachment point of all horizontal forces first. This is done in a comparable way as the determination of a joint center of gravity, as described by equation 23.2 in a later stage of this report.

$$\overline{RC}_{total} = \frac{\sum(F_{diagram;i} \cdot \overline{RC}_{diagram;i})}{\sum F_{diagram;i}}$$

Equation 22.9

Where: $\overline{RC}_{total}$ = distance between rotation center and joined attachment point of all horizontal components together [m]
$F_{diagram;i}$ = resulting horizontal force of pressure diagram i [kN]
$\overline{RC}_{diagram;i}$ = distance between horizontal force of diagram i and rotation center [m]

The overturning momentum can now easily be determined by multiplying the total resulting horizontal force by the distance between its attachment point and the rotation center.

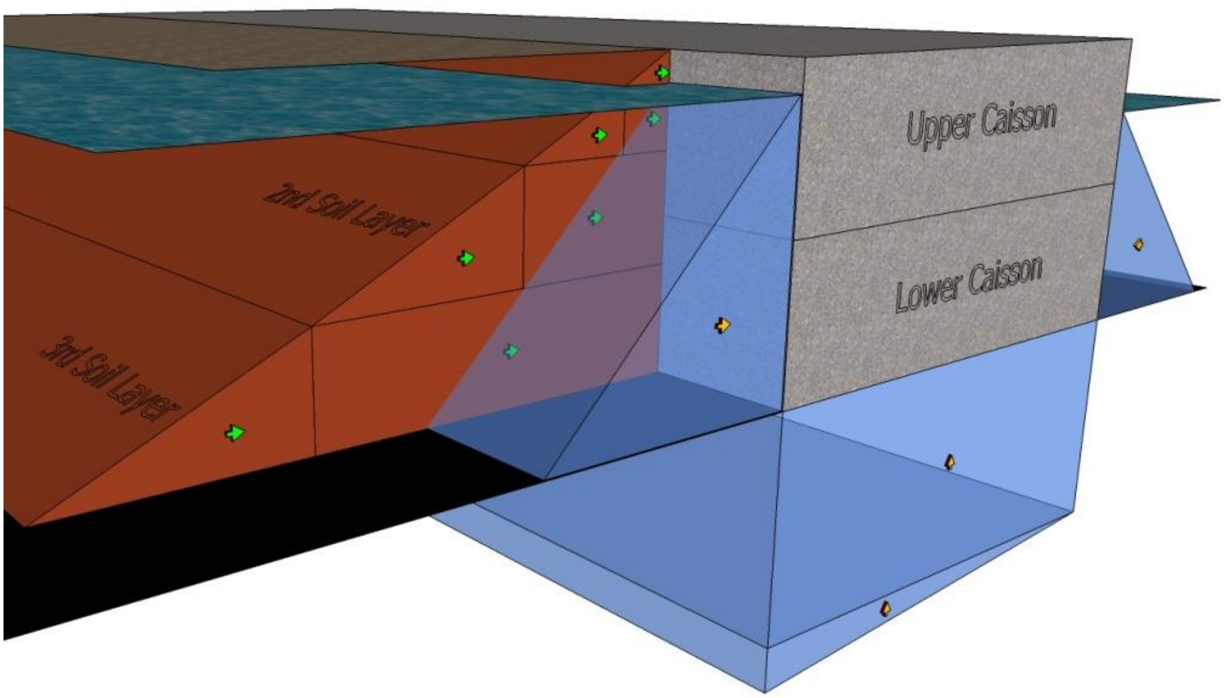


Figure 22.1 – horizontal soil and water pressure distribution (not to scale)

22.4. Surface loads at land side

The container stack, the terminal pavement and handling equipment exert an additional load that is introduced into the subsoil. The extra weight results in an increased soil pressure at the landside of the quay wall. The spread angle of surface loads and the effect on the horizontal and vertical soil pressure is illustrated in figure 22.2.

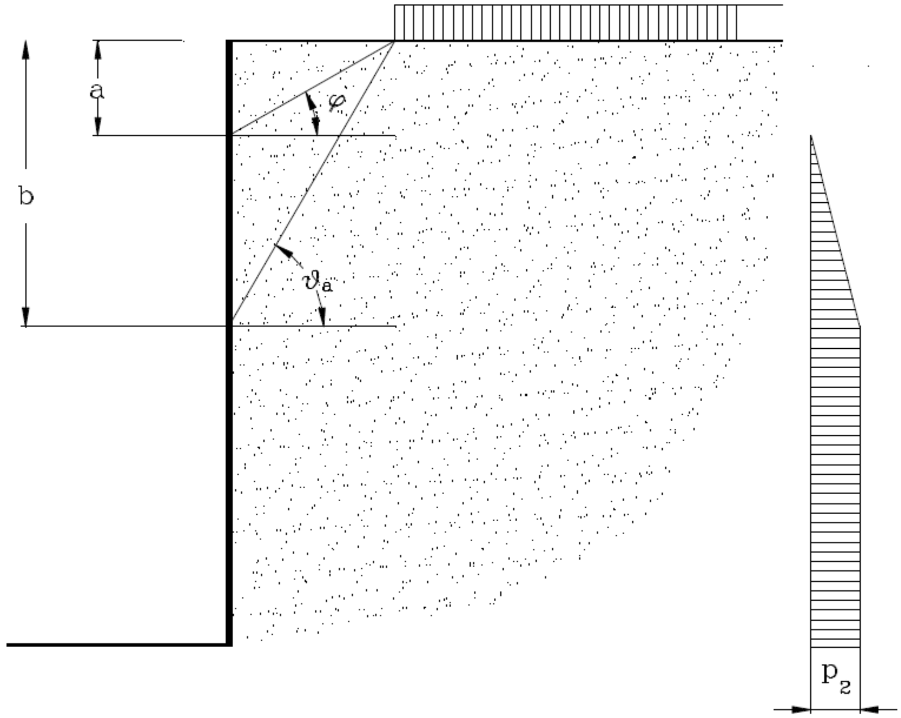


Figure 22.2 – effect of surface load on additional soil pressure

The spread angle of the surface load where it reaches the constant value p_2 can be determined by means of equation 22.10.

$$\tan(\vartheta_a) = \tan(\varphi) + \sqrt{\frac{(1 + \tan^2(\varphi))\tan(\varphi)}{\tan(\varphi) + \tan(\delta_{soil})}}$$

Equation 22.10

Where: ϑ_a = spread angle of surface load to reach maximum value [degrees]
φ = angle of internal friction [degrees]
δ_{soil} = angle of external friction [degrees]

The angle of external friction can be estimated using $\delta = {}^{2}/_{3}\,\varphi$.

The value of p_2 in figure 22.2 is determined by the weight of the surface load in Newton, multiplied with the horizontal soil coefficient k_{hor}.

22.5. Wind load

Forces generated by the wind are an important load on the quay wall structure, especially when vessels with a large air draught are moored at the quay. Although the density of air is very small, wind speed can reach severe levels. Due to the quadratic increasing wind resistance and the large surface of container vessels, this can results in very large forces. The wind force perpendicular to a certain surface can be calculated with the following formula.

$$F_{wind} = ½ \cdot \rho_{air} \cdot C_D \cdot A_{vessel} \cdot v_{wind}^2 \qquad \textit{Equation 22.11}$$

Where: F_{wind} = force generated by the wind [kN]
ρ_{air} = density of air [ton/m^3] ($\approx 1{,}28_E^{-3}$)
C_D = wind force coefficient [-]
A_{vessel} = surface of vessel perpendicular to the wind direction [m^2]
v_{wind} = wind velocity at 10 meter elevation [m/s]

Figure 22.3 shows the drag force coefficients of various shapes. A determination of the wind loads on the container vessel and the STS cranes can be found in the appendix. Especially the wind load and corresponding overturning moment on the STS cranes is hard to determine and specific information is very hard to find. A detailed calculation of the governing wind direction and the corresponding wind loads on STS cranes can be found in appendix P. A highly detailed digital 3D scale model of a STS crane is used to determine the wind surface in all directions, by means of a computer program. An impression of the scale model is illustrated in appendix P as well.

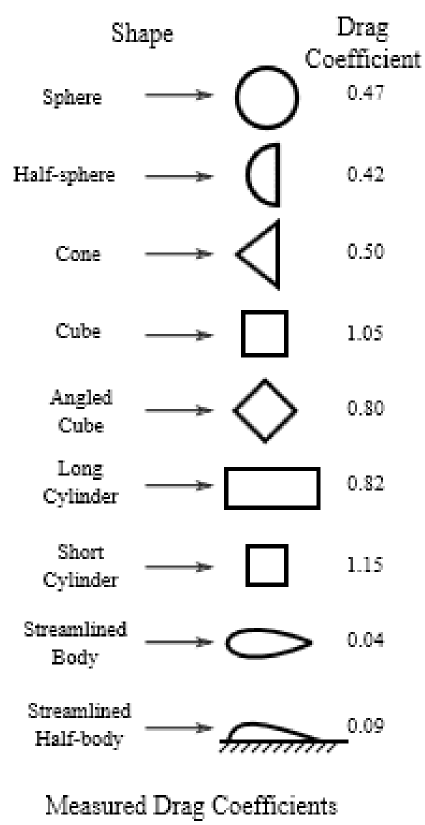


Figure 22.3 – drag force coefficients of various shapes [www.wikipedia.org]

Vessels moored at the quay are connected to bollards which have to absorb to increased forces during a storm. Since mooring lines are connected under a certain angle with the horizontal plane, wind forces result in a vertical and a horizontal component.
The components are determined with the next equations.

$$F_{wind;vert} = F_{wind} \cdot \sin\alpha \qquad \textit{Equation 22.12}$$
$$F_{wind;hor} = F_{wind} \cdot \cos\alpha \qquad \textit{Equation 22.13}$$

Where: $F_{wind;vert}$ = vertical component wind force [kN]
$F_{wind;hor}$ = horizontal component wind force [kN]
α = angle of mooring lines with the horizontal plane [degree]

22.6. Berthing energy and fender reaction force

Berthing vessels transfer a large amount of energy onto the quay wall. Depending on the energy that has to be absorbed, a certain type of fender has to be selected that is capable to deal with this energy. When the fender absorbs the berthing energy, it defects and transfers a reaction force onto the quay wall. The corresponding reaction force is provided by the manufacturer of the fender.

The berthing energy of a vessel depends mainly on its mass and berthing velocity and is given by:

$$E_{kin} = ½ \cdot m_{vessel} \cdot v_{vessel}^2 \cdot C_H \cdot C_E \cdot C_S \cdot C_C \qquad \textit{Equation 22.14}$$

Where: E_{kin} = kinetic berthing energy [kNm]
m_{vessel} = mass of the vessel [ton]
v_{berth} = berthing velocity perpendicular to the quay wall [m/s]
C_H = added mass coefficient for water moving along with the ship [-]
C_E = eccentricity coefficient [-]
C_S = stiffness coefficient [-]
C_C = berth configuration coefficient [-]

Under normal berthing conditions this formula can be simplified:

$$E_{kin} = 0{,}35 \cdot m_{vessel} \cdot v_{vessel}^2 \qquad \textit{Equation 22.15}$$

22.7. Wave load during transport

When the caissons are transported by tug boats, wave motions result in large bending moments on the entire caisson, especially when the wave length equals the length of the caissons. Due to the wave steepness, the corresponding wave height in open water is about 2% of the wave length. The maximum bending moment can be estimated by means of equation 22.16.

$$M_{wave} = \frac{g \cdot H \cdot L^2}{4 \cdot \pi^2}$$

Equation 22.16

Where: M_{wave} = momentum caused by waves during transport [kNm/m]
H = wave height [m]
L = wave length [m]

More specific information on dynamic wave loads during transport can be found in appendix U.

22.8. Wave load during operation

Waves do exert a load on the quay wall structure as well. A distinction of the type of waves has to be made first, before the load can be calculated. Breaking waves are not expected, because of the water depth in front of the structure.

The method of Sainflou can be used to make an estimate of the maximum wave pressure on the structure. This method is based on Stokes' second order theory and non-breaking waves with a reflection coefficient of 1. This is a reasonable assumption for quay walls, because of the large water depth compared to the wave height and the high reflection coefficient of a vertical wall.

First the height increase of the middle level by the waves can be determined by equation 22.17.

$$h_0 = ½ \cdot \mathrm{k} \cdot H_i^2 \cdot \coth(kd)$$ *Equation 22.17*

With: $k = \frac{2\pi}{\mathrm{L}}$ *Equation 22.18*

and: $L = \frac{g}{2\pi} T^2$ if $\frac{h}{L} > ½$ *Equation 22.19*

$L = \frac{g}{2\pi} T^2 \cdot \tanh(kh)$ if $\frac{1}{20} < \frac{h}{L} < ½$ *Equation 22.20*

Where: h_0 = increase of middle water level [m]
k = wave number [m^{-1}]
H_i = incoming wave height [m]
d = water depth between surface and point d [m]
h = water depth (bottom) [m]
L = wave length [m]
T = wave period [s]

Now, the wave pressures at the structure can be determined with the following equations:

$$p_1 = \rho_w \cdot \mathrm{g} \cdot H_i$$ *Equation 22.21*

$$p_0 = \frac{\rho_w \cdot g \cdot H_i}{\cosh(kd)}$$

Equation 22.22

Where: p_1 = pressure at mean water level [kN/m^2]
p_0 = pressure at depth d [kN/m^2]

In figure 22.4 a schematization of the pressure distribution according to Sainflou is given.

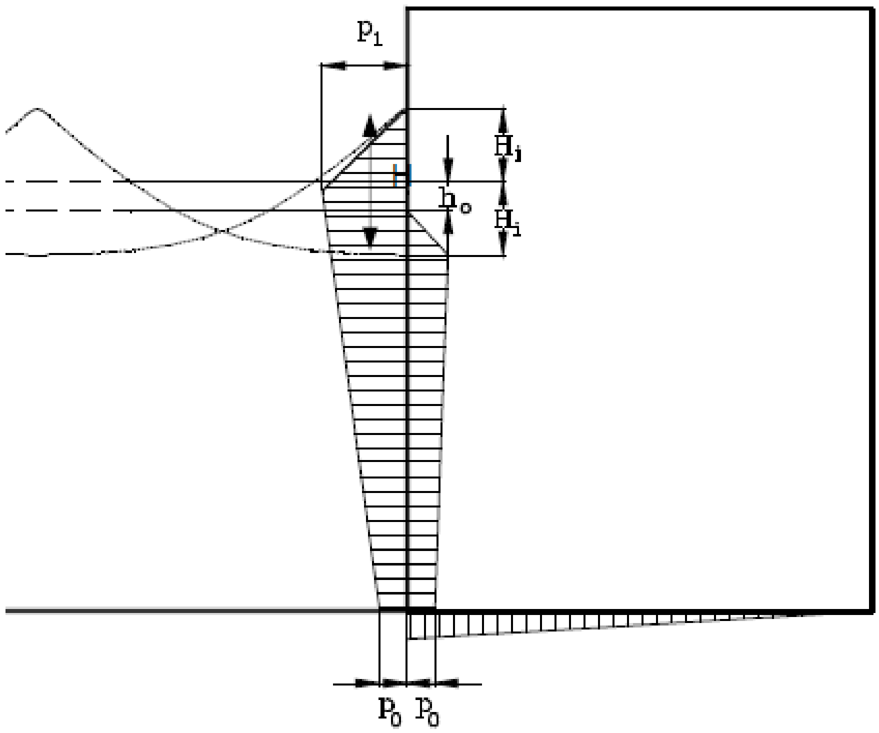


Figure 22.4 – wave pressure distribution according to Sainflou
[Lecture Notes – manual hydraulic structures]

The pressure between p_1 and p_0 is assumed to be linear. Therefore, this method will overestimate the wave pressure for steep waves. The wave force acting on the entire structure will be overestimated as well when the wave force per running meter is multiplied with the total length of the quay. This is because it is not likely that a wave crest will hit the structure at the same time over the entire length.

Nevertheless, Sainflou's method gives a safe upper boundary of the maximum wave force.

Note that the wave load will also lead to an upward force when p_0 is not equal to zero, at the bottom slab of the caisson. This pressure will reduce the downward pressure, but at the same time provides a turning moment in the beneficial direction.

The wave pressure will approach the hydrostatic pressure in case of a very large wave length as illustrated in figure 22.5. The figure also makes clear that the pressure between p_0 and p_1 is not linear in case of a short wave length.

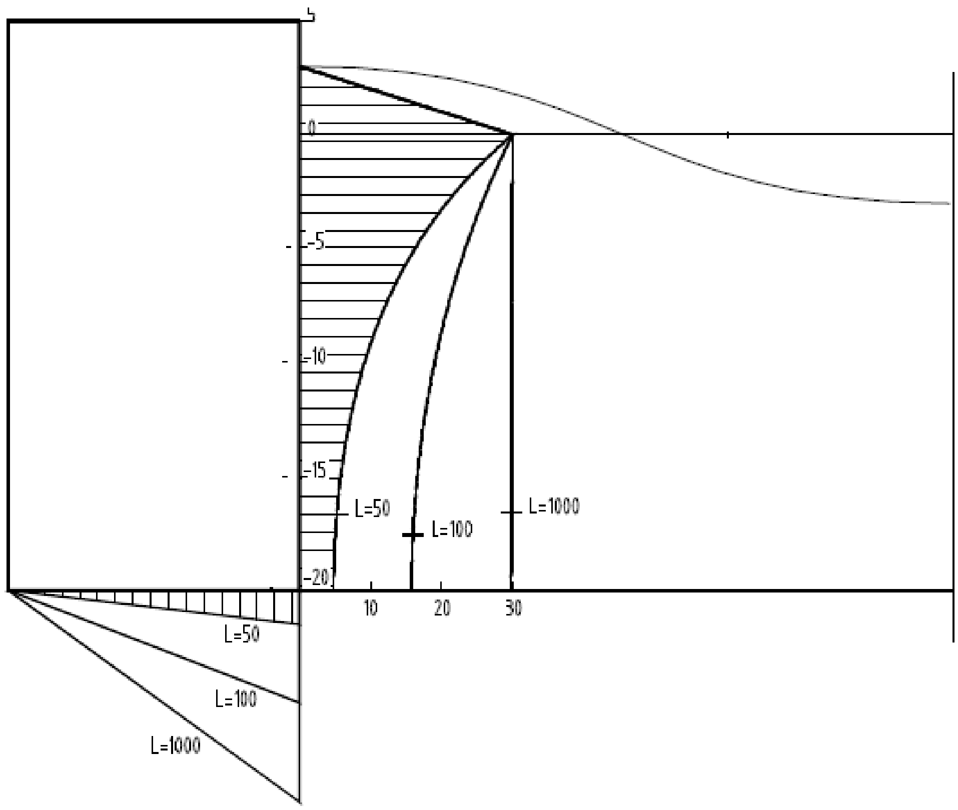


Figure 22.5 – pressure distribution depending on wave length
[Lecture Notes – manual hydraulic structures]

22.9. Propeller wash and scour holes

A bottom protection is required in front of the quay to prevent the development of scour holes. Propeller washes create high flow velocities, which will lead to erosion of an unprotected sea bed. Scour holes undermine the stability and strength of the foundation and the underwater slope. Bottom protection must therefore be designed is such a way that scour holes develop either at a sufficient large distance from the quay, or do not develop at all.

An estimate of the flow velocities due to ship propellers can be made, using the next equations.

$$u_0 = 1{,}15 \cdot \left(\frac{P_{vessel}}{\rho_w \cdot d_{eff}^2}\right)^{1/3}$$

Equation 22.23

$$u_{b-max} = 0,3 \cdot u_0 \cdot \frac{d_{eff}}{z_b}$$

Equation 22.24

Where: u_0 = flow velocity behind the propeller [m/s]
P_{vessel} = vessel's propeller power [Watt]
ρ_w = density of the water [kg/m^3]
d_{eff} = effective propeller diameter [m]
$u_{b\text{-}max}$ = flow velocity at the bed [m/s]
z_b = distance between bottom and propeller axis [m]

Keep in mind that container vessels do not use full power of their main propeller during berthing and departure and that the effective propeller diameter is about 70% of the real diameter. Berthing of container vessels is guided by tug boats, which usually do use their full power. However, the distance between the propeller axis and the bottom is much larger and the installed power is less.

Once the flow velocity at the sea bed is calculated, one can determine the required stone size of the bottom protection, using an Izbash type of formula. Tables can be used to select the appropriate stone grading for each value of d_{n50}. The last term in the equation represents the effect of a bottom slope, hence this term is one for $\alpha = 0°$.

$$d_{n50} = \frac{2,5}{\Delta} \cdot \frac{u_{b-max}^2}{2g} \cdot \frac{1}{\sqrt{1 - \frac{\sin^2 \alpha_{bottom}}{\sin^2 \varphi}}}$$

Equation 22.25

Where: d_{n50} = nominal stone diameter [m]
Δ = relative under water density of stone material [-]
α_{bottom} = under water slope of the bottom protection [deg]
φ = angle of internal friction of rubble material [deg]

The stone gradation that follows from the mentioned equations corresponds with the top layer of the bottom protection. In order to establish a proper transition to the original sea bed, one or more filter layers must be applied. These can either be open or closed granular filters or a geotextile.

In geometrically open filters the grains do fit through the pores of the upper layer. However, the layer thickness is designed in such a way that flow velocities in each layer remain small enough to prevent the grains to move through the upper layer. Geometrically closed granular filters must satisfy the filter rules of Terzaghi, which in fact state that the pores of a granular layer must be smaller than the grain size of the under lying layer. Properties of geotextiles are often provided by the manufacturer.

The filter rules of Terzaghi are listed in the following equations.

$$\frac{d_{15F}}{d_{85B}} < 5 \qquad \frac{d_{15F}}{d_{15B}} > 5 \qquad \frac{d_{60}}{d_{10}} > 10$$

Equation 22.26

Where: d_{15F} = d_{15} of filter layer [m]
d_{85B} = d_{85} of above lying layer [m]
d_{15B} = d_{15} of above lying layer [m]
d_{60} = d_{60} of layer [m]
d_{10} = d_{10} of layer [m]

22.10. Wave overtopping and freeboard

Waves in the harbor basin might overtop the quay wall if the freeboard is insufficient with respect to the wave height. Wave overtopping is a very random process with high variations in overtopping volumes per individual wave. An average overtopping discharge, however, is often used as a design condition. To avoid damage to equipment and cargo an average overtopping discharge of 0,4 l/m/s could be tolerated, as described in the Overtopping Manual.

The amount of overtopping is mainly determined by the local wave height and the freeboard of the structure. For vertical walls, the average wave overtopping discharge can be estimated by equation 22.27.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0{,}04 \cdot exp\left(-1{,}8 \cdot \frac{R_c}{H_{m0}}\right)$$

Equation 22.27

$$if: \qquad 0{,}1 < \frac{R_c}{H_{m0}} < 3{,}5 \qquad and \qquad \frac{h}{H_s} \cdot \frac{h}{L_0} > 0{,}3$$

Where: q = mean overtopping discharge [l/s/m]
H_{mo} = significant wave height of a wave spectrum [m]
R_c = freeboard of the vertical wall [m]
H_s = significant wave height [m]
L_0 = deep water wave length [m]
h = water depth [m]

Note that wave overtopping is not the only criterion that determines the quay's freeboard. The vertical reach of STS cranes and container vessel dimensions also play a role in this. The significant wave height can be derived from a wave spectrum by calculating the zero order moment.

$$H_{m0} = \sqrt{4m_0}$$

Equation 22.28

and:

$$m_0 = \int_0^{\infty} E(f)df$$

Equation 22.29

Where: m_0 = zero order moment of wave spectrum [m^2]
E(f) = wave spectrum

Note that the effect of wind is not taken into account in the equation. Overtopping under q =1 l/m/s might increase by up to 4 times the calculated discharge due to a strong onshore wind, especially when much of the overtopping is spray.

The required freeboard that follows from the equation is not necessarily the height of the quay apron. If a large freeboard is required, one can decide to construct a vertical wall around the sea side of the deck. This will reduce the overtopping discharge and will hardly affect the draught of the entire structure in floating conditions. However, there must be a proper drainage system to get rid of water on the quay, caused by rainfall and overtopping.

22.11. Equipment on quay apron

The container handling equipment on the quay apron exerts forces on the deck and the walls of the caisson. The highest load originates from the STS cranes that ride on their rail, which is located straight above the walls of the caisson.
Container lifting operations of STS cranes exerts an overturning moment on the quay wall. Crane booms are very long and FEU can weigh on to 30 tons. Accelerating such a weight vertically at the end of the crane boom, results in a large overturning momentum.

The deck of the structure must be strong enough to resist the loads exerted by the axes of handling equipment and TEU container units. The larger the distance between the inner walls of the caisson, the heavier the deck construction becomes.

Acceleration and deceleration of vehicle and cranes delivers an extra horizontal load on the deck which can easily be calculated by equation 22.30. This equation can also be used to determine the extra vertical acceleration force of container lifting operations.

$$F_{equipment} = m_{equipment} \cdot a_{equipment}$$ *Equation 22.30*

Where: $F_{equipment}$ = force due to moving equipment [kN]
$m_{equipment}$ = mass of the equipment moved [ton]
$a_{equipement}$ = acceleration of the equipment [m/s^2]

22.12. Tug boat loads during transport

When the caissons are transported, tug boats exert a certain force on the caissons. This force can be estimated in the same way as the calculations of a wind or current load.

$$F_{tug} = ½ \cdot \rho_w \cdot C_D \cdot A_{tot} \cdot v_{current}^2 \qquad \textit{Equation 22.31}$$

Where: F_{tug} = force generated by the sailing velocity of tug boat [kN]
ρ_w = density of water [ton/m3]
A_{tot} = total surface perpendicular to the current [m^2]
$v_{current}$ = current velocity ≈ sailing velocity [m/s]

One must determine the surface of the caissons perpendicular to their sailing direction and the effective velocity through the water. A drag coefficient of 0,82 must be applied for a long cylindrical shape, as given in figure 22.3. Note that this is not exactly correct, since the caisson is not completely submerged.

Tug boats can either push or tow the caissons. Pushing seems to be the best solution, to avoid large tensile forces on the concrete structure. In practice this is also done for steel push barges, with even larger dimensions, so pushing should be possible.
Note that towing the structure also leads to towing line force components in different directions.

22.13. Earthquake loads

Since some container ports are located in areas that are vulnerable for earthquakes, the caisson quay wall structure must be able to resist earthquake loads to some extent.

The main failure mechanisms of a caisson structure during an earthquake are caused by liquefaction. The effective grain stresses can be reduced to nearly zero, making the retained soil behave like a high density fluid, as illustrated in figure 22.6.
The increased horizontal load may lead to sliding aside of the structure. Another earthquake effect that contributes to this is the horizontal and vertical acceleration. Due to these accelerations, the vertical downward pressure is reduced, during a short period and the horizontal pressure is increased. Earthquake accelerations lead to an increase of the active horizontal soil coefficient and a decrease of the passive coefficient. Liquefaction of the subsoil underneath the caissons may lead to differential settlements.
A third effect, which is directly related to liquefaction, is an increased ground water level at the land side of the quay wall.

The increased loads, because of liquefaction can be resisted by applying a rubble material back fill behind and underneath the caissons. The mass and angle of internal friction are sufficiently large to avoid liquefaction. The width of the rubble material layer must be large enough to translate the additional horizontal forces of the liquefied soil behind it, towards the subsoil.
The soil underneath the gravel bed foundation has usually sufficient effective stress and compaction to prevent liquefaction underneath the caissons.

The final resistance of the caisson structure was determined by setting all safety factors back to 1.0 and determine the corresponding unity check values. Based on these values, the allowable increase of the horizontal soil coefficient was calculated, which was used to determine the corresponding maximum Peak Ground Acceleration (PGA) value.

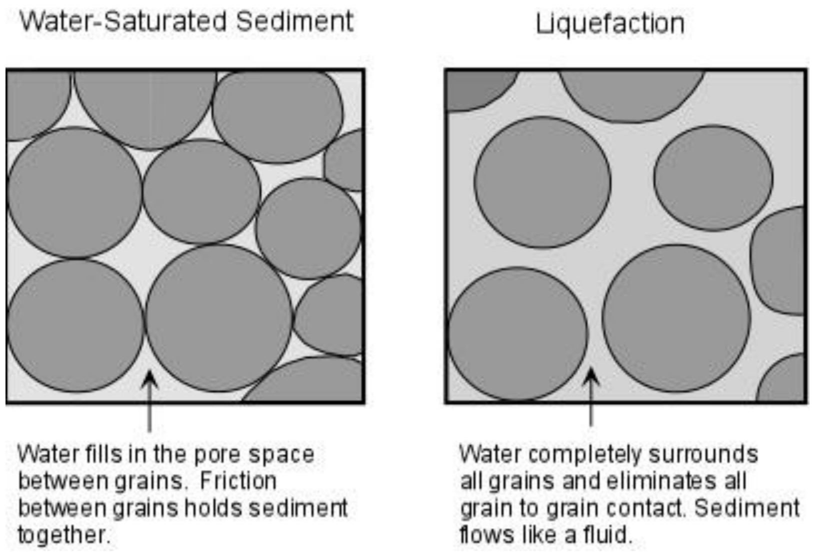


Figure 22.6 – principle of liquefaction [www.e-pao.net]

An illustration of the failure mechanisms caused by earthquakes on a caisson quay wall, is given in figure 22.7. As stated in the figure, an earthquake may also trigger a tsunami, but tsunami loads are not taken into account during this thesis.

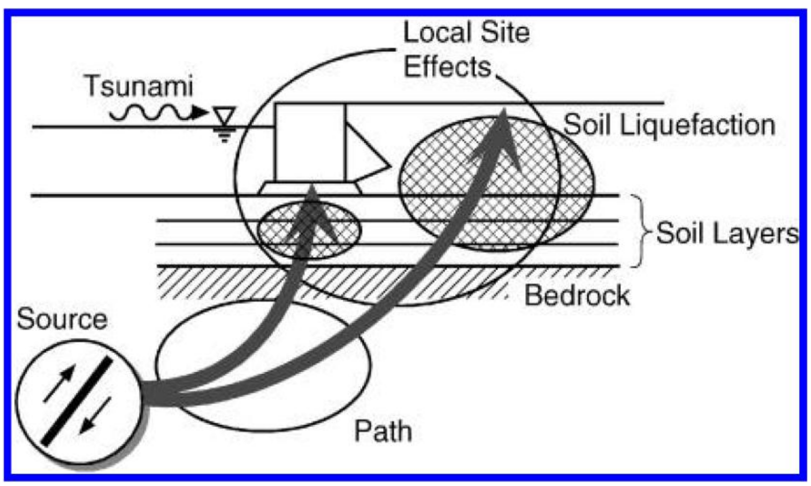


Figure 22.7 – earthquake loads on a caisson quay wall [PIANC]

22.14. Ice load

Ice loads can exert a severe force on a quay wall structure, but for this master thesis it is assumed that ice formation does not play a role. Ice loads are therefore not taken into account.

22.15. Temperature differences

Differences in temperature in the structure will lead to additional stresses. Analyzing the temperature in the structure and the resulting stresses is beyond the scope of this report and will therefore neither be taken into account.

23. Static stability in floating conditions

The caissons are constructed in a dock and are towed to the construction site of the quay wall. During transport, the caissons must have sufficient stability in floating conditions. To check the stability, the draught of the caisson must be determined first. This can easily be done with the law of Archimedes. It states that de weight of the total structure is equal to the weight of the displaced water by the structure, as described in equation 23.1.

$$D_{caisson} = \frac{m_{caisson}}{\rho_w \cdot A_{bottom;slab}}$$

Equation 23.1

Where: $D_{caisson}$ = draught of the caisson [m]
$m_{caisson}$ = mass of the caisson [ton]
ρ_w = density of water [ton/m^3]

Obviously, the draught should never be larger than the height of the caisson. If this is the case, the caisson doesn't float at all and will sink.

Now, the vertical position of the center of gravity has to be determined. This can be done by multiplying the weight of each element by the position of its center of gravity. The sum of all elements divided by the total weight of the structure results in the position of the center of gravity.

$$\overline{KG} = \frac{\sum(m_{element;i} \cdot \overline{KG}_{element;i})}{m_{caisson}}$$

Equation 23.2

Where: $\overline{KG}$ = vertical distance between bottom of bottom slab to center of gravity caisson [m]
$m_{element;i}$ = mass of element i [ton]
$\overline{KG}_{element;i}$ = vertical distance between bottom of bottom slab to center of gravity element i [m]

In order to calculate the height of the metacenter point of the caisson, one must now determine the moment of inertia of the area that intersects the water surface. In case of a rectangular caisson this can be calculated by the next equation:

$$I_{caisson} = \frac{1}{12} \cdot L_{caisson} \cdot B_{caisson}^3$$

Equation 23.3

Where: $I_{caisson}$ = moment of inertia of the caisson [m^4]
$L_{caisson}$ = length of the caisson [m]
$B_{caisson}$ = width of the caisson [m]

The distance between the center of buoyancy and the metacenter point is determined by dividing the moment of inertia by the volume of displaced water, as started in de next formula.

$$\overline{BM} = \frac{I_{caisson}}{L_{caisson} \cdot B_{caisson} \cdot D_{caisson}}$$

Equation 23.4

Where: $\overline{BM}$ = distance between center of buoyancy and metacenter point [m]

Finally, the metacenter height can be calculated by:

$$h_m = \overline{BM} + ½D_{caisson} - \overline{KG}$$

Equation 23.5

Where: h_m = metacenter height of the floating caisson [m]

Theoretically the caisson is stable if $h_m > 0$, but in practice it can be considered stable if $h_m > 0{,}5$m. If this criterion doesn't suffice, measures have to be taken to increase the stability. This can be done by changing the dimensions of the caisson. Increasing the width of the caisson significantly increases the stability, due to the third power in the formula.

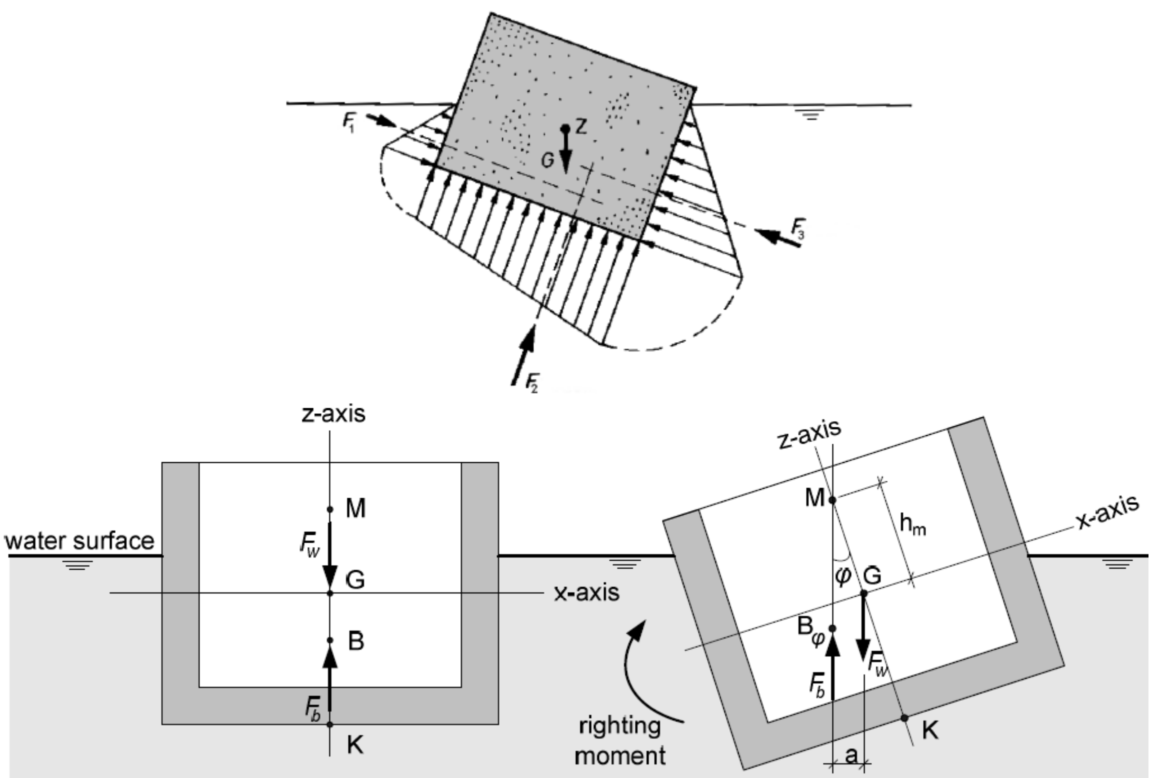


Figure 23.1 – stability of a floating caisson
[Lecture Notes – hydraulic structures - Caissons]

23.1. Tilt angle of floating structure

When the total center of gravity is not located in the middle of a floating structure, a tilting moment is present. A tilting moment can be caused by equipment on the quay or by an asymmetric design of the caisson itself. It is also possible to design the caisson in such a way that the eccentricity of the crane load is counteracted by an asymmetric design of the caisson, in case of a floating structure with cranes on top. However, lifting of containers will always result in a tilting moment, since this force is variable and appears and disappears very rapidly.
Movement of the STS crane along its beam will result in a tilting moment in the longitudinal direction. The tilt angle can be determined using the equation of Scribanti, which is only valid for angles lower than 10°.

$$M_{tilt} = \sin\theta\,\{h_m + [½ \cdot BM \cdot \tan^2\theta]\} \cdot \rho \cdot g \cdot D_{caisson} \cdot L_{caisson} \cdot B_{caisson}$$

Equation 23.6

Where: M_{tilt} = tilting moment [kNm]
θ = tilting angle [degree]

When the tilt angle is determined, one can determine the vertical movement at a certain distance from the rotation center. This can be done by multiplying the tangent of the tilt angle with the distance to the rotation center.

$$\Delta x_{vert} = \Delta x_{hor} \cdot \tan\theta$$ *Equation 23.7*

Where: Δx_{vert} = vertical displacement [m]
Δx_{hor} = horizontal distance to rotation center [m]

23.2. Static stability during immersion

A special case with respect to stability occurs when a floating caisson is immersed by opening gates in the wall. The draught of the structure increases during immersion, which results in a higher location of the center of buoyancy, point B. Despite the lowered center of gravity, the caisson will be much less stable, due to a dramatically decreased moment of inertia of the area that intersects the water surface.

The moment of inertia during immersion of a rectangular caisson without inner walls can be calculated by subtracting the moment of inertia of the area inside the caisson from the moment of inertia of the entire caisson.

$$I_{caisson} = \left(\frac{1}{12} \cdot L_{caisson} \cdot B_{caisson}^3\right) - \left(\frac{1}{12} \cdot (L_{caisson} - 2t_{w;h}) \cdot (B_{caisson} - 2t_{w;s})^3\right)$$

Equation 23.8

Where: $t_{w;h}$ = wall thickness head walls [m]
$t_{w;s}$ = wall thickness side walls [m]

In case the caisson is constructed with inner walls, these will have a positive effect on the static stability during immersion. The total moment of inertia that intersects the water surface can now be determined by subtracting the sum of all moments of inertia of the areas enclosed by the inner walls from the moment of inertia of the entire caisson.

$$I_{caisson} = \left(\frac{1}{12} \cdot L_{caisson} \cdot B^3_{caisson}\right) - \sum I_{area}$$ *Equation 23.9*

Where: I_{area} = moment of inertia of area enclosed by inner walls [m^4]

In all cases it is assumed that immersing the caisson in done by opening gates or valves in the outer walls of the caisson. When the caisson is immersed by dumping ballast into it, the moment of inertia of the area that intersects the water surface doesn't change. Consequently the stability is much less affected. It will even improve, because of the lowered center of gravity.

24. Dynamic stability in floating conditions

Besides static stability, the caisson must satisfy requirements for dynamic stability as well. To determine the dynamic stability, one must calculate the natural oscillation period of the caisson. In order to prevent large rotations, the caisson's natural oscillation period should be much larger than the prevailing wave periods.

The first step in determining the dynamic stability of a floating caisson is the calculation of the polar moments of inertia around both z and x-axis.

$$I_{xx;polar} = \frac{1}{12} \cdot H_{caisson} \cdot B^3_{caisson} - (I_{xx;hollow})$$ *Equation 24.1*

and

$$I_{zz;polar} = \frac{1}{12} \cdot B_{caisson} \cdot H^3_{caisson} - (I_{xx;hollow})$$ *Equation 24.2*

Where: $I_{xx;polar}$ = area moment of inertia around x-axis [m^4]
$I_{zz;polar}$ = area moment of inertia around z-axis [m^4]
$H_{caisson}$ = height of the caisson [m]
$I_{xx;hollow}$ = moment of inertia hollow area around x-axis [m^4]
$I_{zz;hollow}$ = moment of inertia hollow area around z-axis [m^4]

Note that the equations only apply in case of a symmetric caisson. If the caisson is asymmetric, Steiners theorem has to be used to determine $I_{xx;polar}$ and $I_{zz;polar}$, as described in equation 24.3.

$$I_{tot} = \sum I_{element;i} + \left(A_{element;i} \cdot \overline{GG}^2_{element;i}\right)$$

Equation 24.3

Where: I_{tot} = total polar moment of inertia [m^4]
$I_{element;i}$ = polar moment of inertia element i [m^4]
$A_{element;i}$ = area cross section element i [m^4]
$\overline{GG}^2_{element;i}$ = distance between total center of gravity and center of gravity of element i [m]

The entire polar moment of inertia is defined as the sum of the polar moments around the z and x-axis.

$$I_{polar} = I_{xx;polar} + I_{zz;polar}$$ *Equation 24.4*

Where: I_{polar} = the total polar moment of inertia [m^4]

Next step is the determination of the polar inertia radius, which can be calculated with the following formula:

$$j = \sqrt{\frac{I_{polar}}{A_{cross}}}$$

Equation 24.5

Where: j = polar inertia radius [m]
A_{cross} = area of cross section caisson [m^2]

Note that the polar moment of inertia is not equal over the entire length and width of the caisson, due to the presence of head walls and inner walls. To determine the final polar inertia radius, one must average it over the length of the caisson, making a distinction between cross sections with inner walls and cross sections without.

Now, all parameters to calculate the natural oscillation period are known. The natural period is determined by equation 24.6.

$$T_0 = \frac{2\pi \cdot j}{\sqrt{h_m \cdot g}}$$

Equation 24.6

Where: T_0 = natural oscillation period [s]

As stated before, this period should be larger than the prevailing wave periods to prevent natural oscillations of the caisson.

25. Failure mechanisms

Once all loads on the structure are determined, all possible failure mechanisms should be checked. These failure mechanisms are described in the remaining part of this chapter.

25.1. Horizontal sliding

A very important failure mechanism of a caisson quay wall is horizontally sliding aside, as illustrated in figure 25.1.

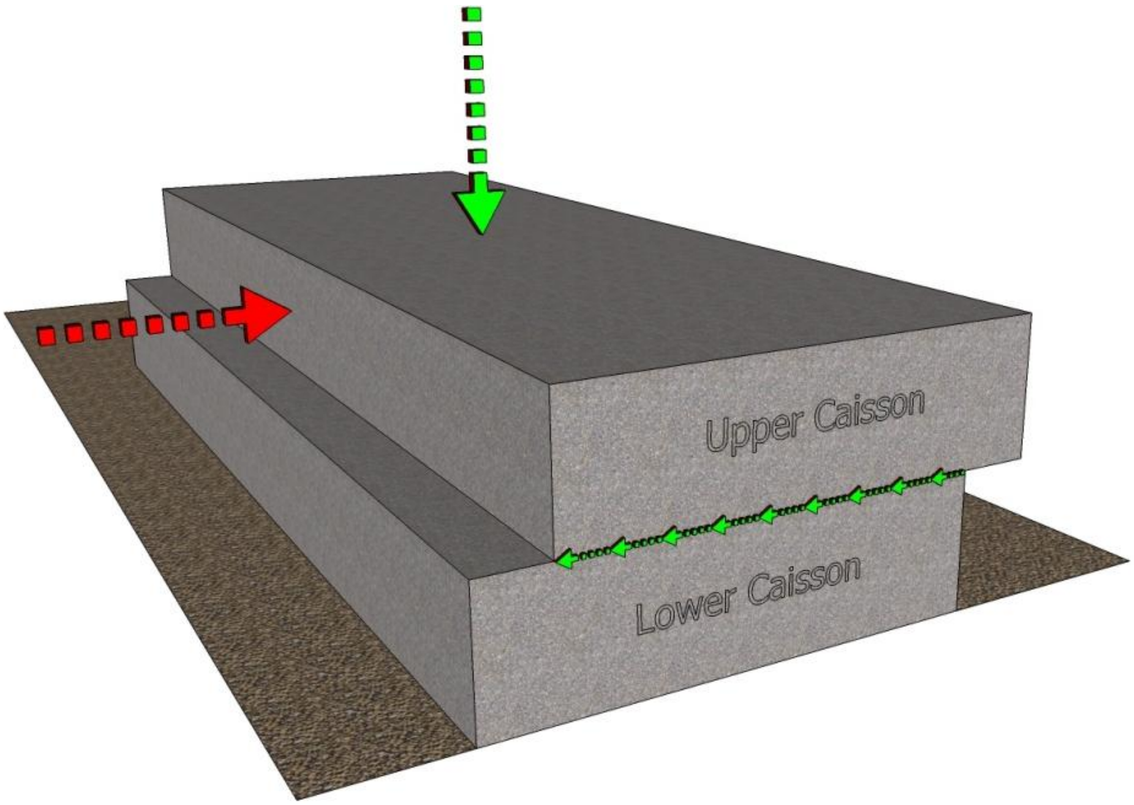


Figure 25.1 – illustration failure mechanism, horizontal sliding of upper caisson

To prevent sliding aside of the caissons due to the horizontal pressure, it must have sufficient downward pressure on its foundation, since position keeping is realized by friction between these two surfaces.

$$F_{hor;max} \leq F_{vert} \cdot c_{fr} \qquad \textit{Equation 25.1}$$

Where: $F_{hor;max}$ = maximum allowable horizontal force on structure [kN]
F_{vert} = net vertical downward force [kN]
c_{fr} = friction coefficient [-]

Especially when the caissons are filled with water, this might become problematic, because of the reduced weight of the structure. If ballast water turns out to provide insufficient weight, the caissons can also be filled with sand. A liquefaction pump must then be used to pump the sand out again, which makes relocation a bit more complicated, especially when many inner walls are present inside the caisson.

The equation to check stability against sliding aside is rather straight forward, but determination of the governing load combination is the hardest part of it. The horizontal pressure of the retained soil is the most important factor, but wind load on container vessels plays a role in satisfaction of this criterion too. Mooring lines have a certain angle with the horizontal plane, which results in a vertical and horizontal component of the hawser forces. Water level variations at both sides of the caisson have a severe contribution as well. Results of calculation can be found in appendix K.

Note that a distinction is made between sliding aside of the upper caisson on its own and sliding aside of both caissons together. The situation in which the lower caisson is pushed out by the horizontal load is also considered, but this won't be governing in reality.

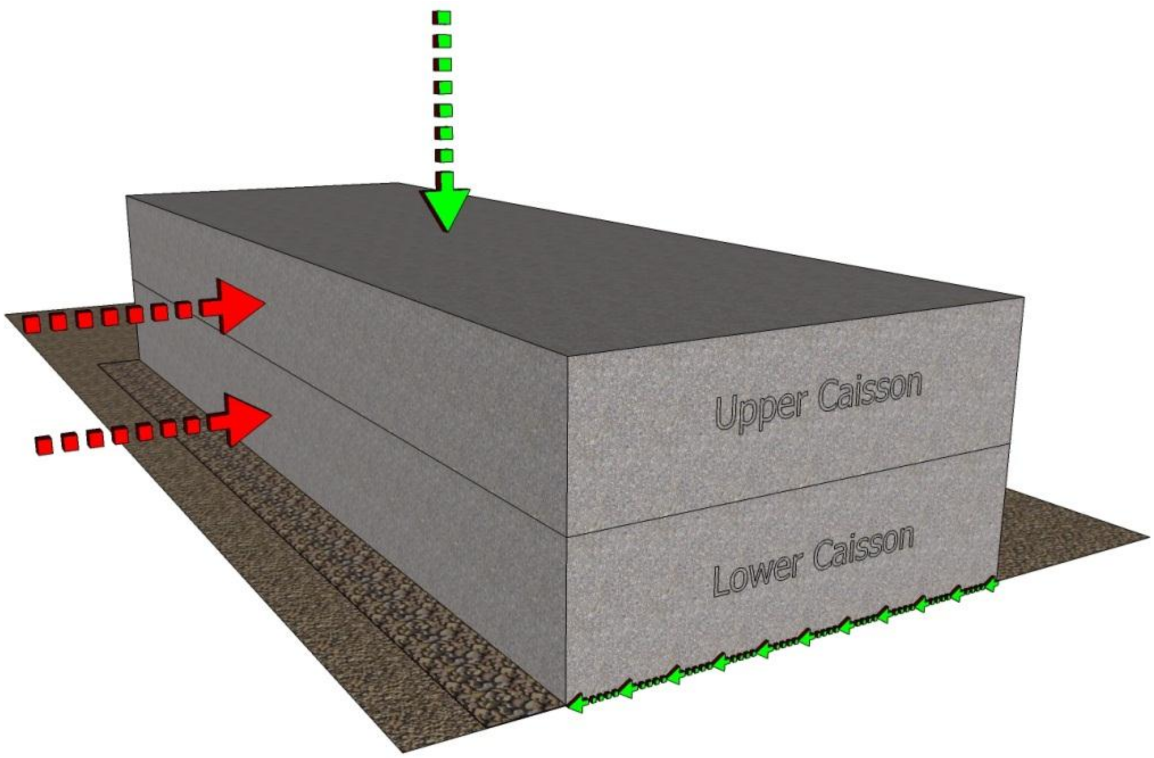


Figure 25.2 – illustration failure mechanism, horizontal sliding of both caissons together

25.2. Overturning moment

Overturning of the caissons due to the overturning moment is another failure mechanism that should be checked carefully. Many load combinations can occur during the lifespan of the structure and the governing scenario must be determined. Note that wind forces on STS cranes and container lifting operations also exert an overturning moment on the quay wall. However, both loads don't act together in the ULS condition, since no container lifting operations take place for that scenario.

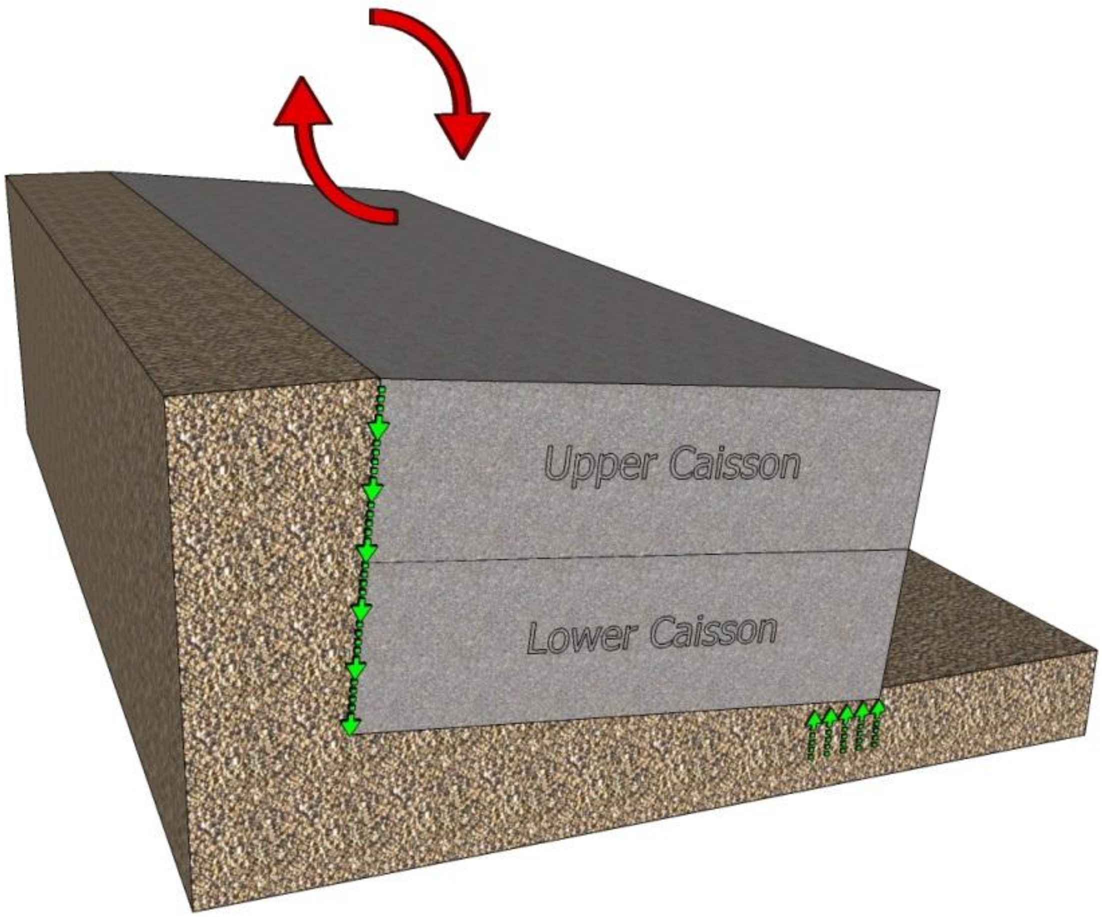


Figure 25.3 – illustration failure mechanism, overturning of both caissons

The criterion for stability against overturning of the caisson is determined by equation 25.2.

$$\frac{\sum M}{\sum V} \leq \frac{1}{6} \cdot B_{caisson}$$

Equation 25.2

Where: $\sum M$ = sum of all moments on the caisson [kNm]
$\sum V$= sum of all vertical forces on the caisson [kN]

Again, one can distinguish two basic scenarios for overturning of the caisson quay wall, since two caissons are positioned on top of each other. The upper caisson could turn over while the lower one remains in position and both caisson could turn over together. Note that the rotation point is different for each scenario and that overturning of the upper caisson only, is very unlikely.

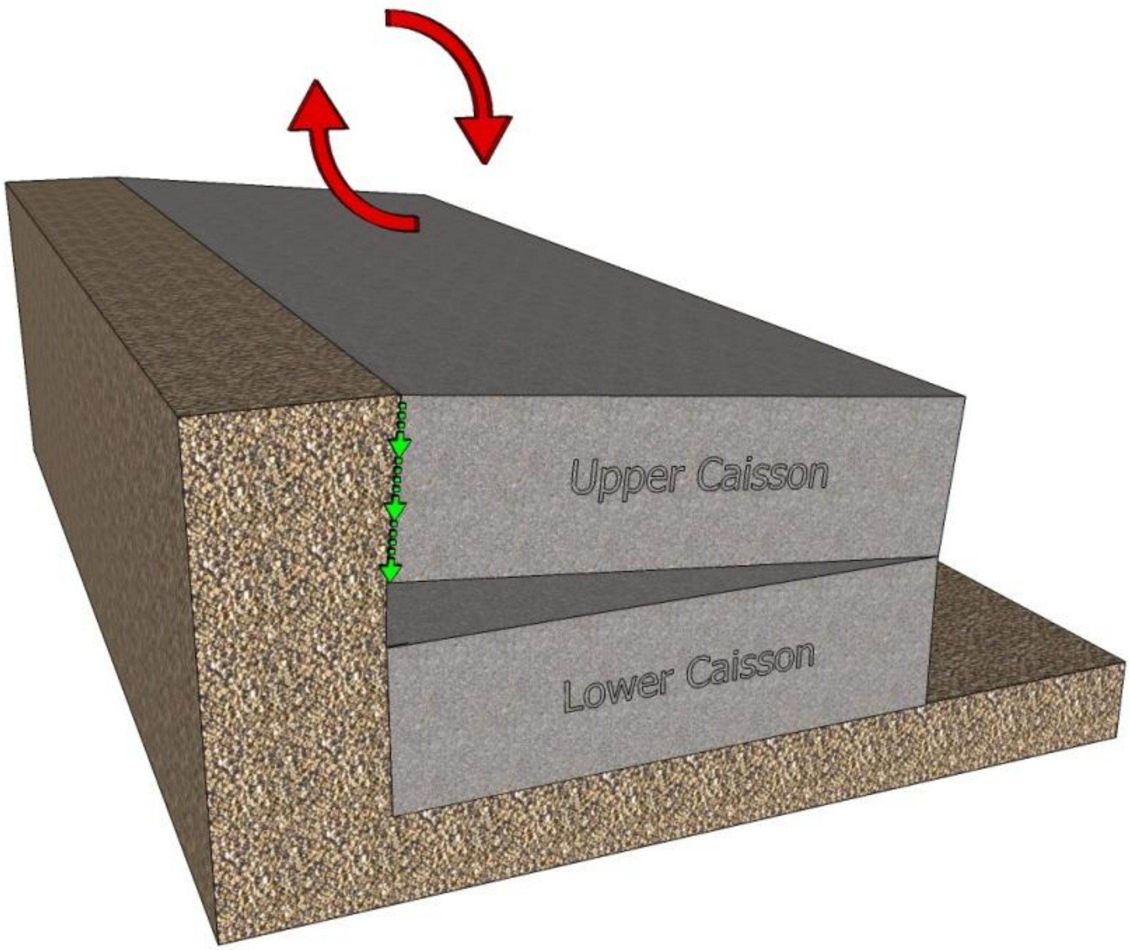


Figure 25.4 – illustration failure mechanism, overturning of upper caisson

25.3. Bearing capacity and slide plane

The vertical downward pressure of the quay wall structure may not exceed the bearing capacity of the soil. When the bearing capacity is exceeded, the whole structure will collapse along a slide circle in the soil, as illustrated in figure 25.5. When the bearing capacity doesn't satisfy the criterion, one can consider improving the soil properties by backfilling a trench with for instance gravel.

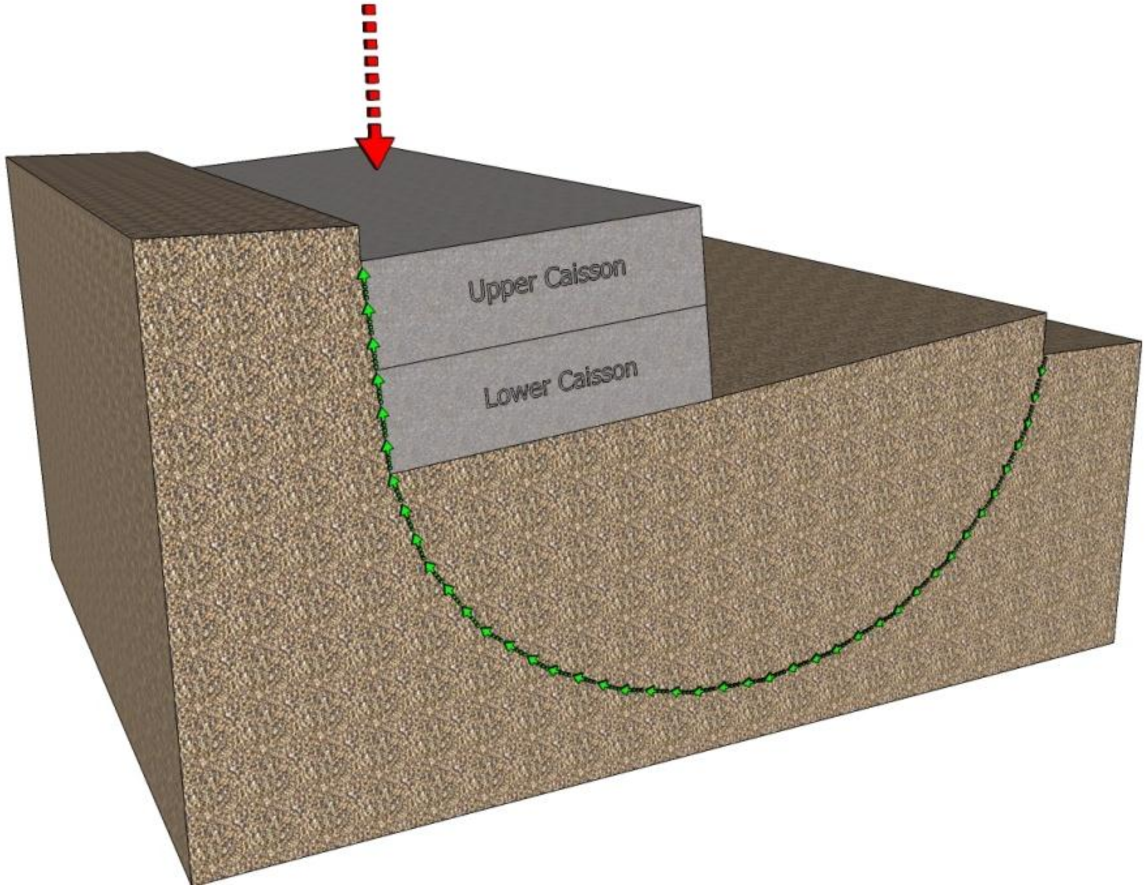


Figure 25.5 – illustration failure mechanism, slide circle and bearing capacity

The required bearing capacity can be determined by the following equation.

$$\sigma_{bear;req} = \frac{\sum V}{B_{caisson} \cdot L_{caisson}} + \frac{\sum M}{\frac{1}{6} \cdot L_{caisson} \cdot B_{caisson}^2}$$

Equation 25.3

Where: $\sigma_{bear;\,req}$ = required bearing capacity of the soil [kN/m^2]

This equation assumes that pressure can build up under the caisson. However, in reality this is not really true for a caisson on a gravel bed foundation. Equation 25.4 can be used to determine the required bearing capacity without building up of pressure under the caisson.

$$\sigma_{bear;req} = \frac{2}{3} \cdot \frac{\sum V}{L_{caisson} \cdot \left(\frac{1}{2} B_{caisson} - \frac{\sum M}{\sum V}\right)}$$

Equation 25.4

The results of the calculations in appendix K show that both equations have more or less the same results, so distinction between both scenarios does not have a large effect on the final design.

Since a gravel bed foundation is applied to support the caissons, the actual required bearing capacity is somewhat less. The vertical pressure spreads under an angle 45 degrees through the gravel bed layer, reducing the pressure at the original soil underneath it. But keep in mind that the weight of the gravel bed itself should also be taken into account for the required bearing capacity of the original soil.

A fourth failure mechanism that could occur is the criterion for floating stability during transport and immersion. This has already been described in chapter 23.

26. Required strength of the structure

The mentioned failure mechanisms all consider stability of the caissons, but obviously, the caissons themselves should be capable to deal with the loads as well. The required strength of the reinforced concrete is described in this chapter. Calculations can be found in appendix Q, R, S, U and V, whereas the final results are given in chapter 27.

26.1. Strength of deck, walls and bottom slab

Inner walls are located in both longitudinal and transversal direction, in order to reduce the load on the walls and slabs. These outer walls can now be seen as a plate that has a fixed connection at all four sides. Once the load on these sections is calculated, one can use the tables presented in appendix V, to determine the shear force and the bending moment. The values given in these tables are the values for triangular shaped loads, but they can also be used for rectangular shaped loads. Simply adding two triangular loads, results in a rectangular one, but keep in mind that the values to be used are mirrored for both triangles.

26.2. Shear force and wall thickness

For a more simple construction method, it would be nice if no shear reinforcement is required. In that case, the walls must be designed in such a way that they can resist the shear force by means of their thickness and the applied reinforcement for bending moments. To calculate the required wall thickness without applying shear reinforcement, equation 26.1 to 26.6 of Eurocode 2 must be used.

$$V_{Rd;c} = \left[C_{Rd;c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] b_w \cdot d_{con}$$

Equation 26.1

With a minimum of:

$$V_{Rd;c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_{con}$$

Equation 26.2

With:

$$k = 1 + \sqrt{\frac{200}{d_{con}}} \quad \leq \quad 2{,}0$$

Equation 26.3

And:

$$\rho_1 = \frac{A_{sl}}{b_w \cdot d_{con}} \quad \leq \quad 0{,}02$$

Equation 26.4

And:

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} \quad < \quad 0{,}2 \cdot f_{cd}$$

Equation 26.5

And:

$$v_{min} = 0{,}035 \cdot k^{\frac{2}{3}} \cdot \sqrt{f_{ck}}$$

Equation 26.6

Where:
$V_{Rd;c}$ = absorbable shear force by reinforced concrete [N/m]
$C_{Rd;c}$ = 0,18 / γ_C [-]
γ_C = material factor for concrete strength (=1,5) [-]
ρ_1 = ratio of reinforcement steel in concrete cross section [-]
f_{ck} = characteristic compressive concrete strength [N/mm^2]
k_1 = 0,15 [-]
σ_{cp} = normal stress in cross section [N/mm^2]
b_w = per unit width [mm]
d_{con} = effective height of concrete cross section [mm]
A_{sl} = applied reinforcement area for bending [mm^2/m]
N_{Ed} = normal force at concrete cross section [N]
A_c = area of concrete cross section [mm^2]
f_{cd} = design value of concrete tensile strength [N/mm^2]

When the required wall thickness turns out to be very large or uneconomic, shear reinforcement must be applying. To determine the amount of shear reinforcement, equations 26.7 to 26.11 of Eurocode 2 must be used. The prevailing shear force is determined with the tables in appendix V and the pressure distributions calculated in appendix K. Calculations for shear reinforcement can be found in appendix Q. Note that the calculated shear force is a maximum value. To optimize the amount of shear reinforcement, one should determine the distribution of the shear force over the entire slab and calculated the corresponding shear reinforcement for different parts of each slab.

$$V_{Rd;s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot (\cot\theta_{strut} + \cot\alpha)\sin\alpha$$

Equation 26.7

With a maximum possible value of:

$$V_{Rd;max} = \alpha_{cw} \cdot b_w \cdot z_{slab} \cdot v_1 \cdot f_{cd} \cdot \frac{\cot\theta_{strut} + \cot\alpha_{steel}}{1 + \cot^2\theta_{strut}}$$

Equation 26.8

And a maximum effective reinforcement area of:

$$\frac{A_{sw;max} \cdot f_{ywd}}{b_w \cdot s_{bar}} \leq \frac{\alpha_{cw} \cdot v_1 \cdot f_{cd}}{2 \cdot \sin\alpha_{steel}}$$

Equation 26.9

With:

$$v_1 = 0{,}6\left[1 - \frac{f_{ck}}{250}\right]$$

Equation 26.10

And:

$$f_{cd} = \frac{f_{ck}}{\gamma_C}$$

Equation 26.11

Where:
$V_{Rd;s}$ = absorbable shear force by shear reinforcement [N/m]
A_{sw} = shear reinforcement area [mm^2/m]
s_{bar} = center to center distance of shear reinforcement bars [mm]
z_{slab} = lever arm of concrete slab [mm]
f_{ywd} = yield stress of shear reinforcement steel [N/mm^2]
θ_{strut} = angle of struts [degree]
α_{steel} = angle of shear reinforcement [degree]
$V_{Rd;max}$ = maximum possible shear force for slab thickness [N/m]
α_{cw} = factor for stress in pressurized edges (= 1,0) [-]
v_1 = strength reduction factor for shear in concrete [-]
$A_{sw;max}$ = maximum applicable reinforcement area [mm^2/m]

26.3. Bending moment and required reinforcement

The bending moment can be determined in the same way as the shear force and reinforcement steel is required to absorb this momentum. Tables and calculation results can be found in appendix Q and V, whereas the final design is given in chapter 27.

Once the prevailing bending moment is determined, one can calculate the required amount of reinforcement steel by means of equation 26.12.

$$A_s = \frac{M_{Ed} \cdot 10^6}{f_{yk} \cdot 0{,}85 d_{con}}$$

Equation 26.12

Where: A_s = required amount of reinforcement steel in cross section [mm^2/m]
M_{Ed} = bending moment in cross section [kNm/m]
f_{yk} = steel yield stress [N/mm^2]
d_{con} = effective height of concrete cross section [mm]

It is important to note that this method neglects the presence of normal stresses. In reality, the walls and slabs will be exposed to a combination of bending moment and normal stresses. Nevertheless, equation 26.12 gives a good estimate of the required amount of reinforcement steel and the structural feasibility of the caisson design, because the contribution of normal stresses is very small.

Also note that a minimum reinforcement percentage of 0,2% is required for concrete class C30/37. The actual percentage can easily be determined by the ratio of reinforcement area and the concrete area in the cross section.

26.4. Strength of the entire caisson during transport

It is also important to consider the caisson as a whole, instead of considering all separate sections apart. When the caisson is transported, wave loads exert forces on the entire caisson. Especially when the wave length equals the length of the caisson, a large bending moment occurs.

This situation is described and illustrated in appendix U, where equations to determine the magnitude of these loads are given as well.

An illustration of the caissons' cross section is given in figure 26.1. Inner and outer walls in transversal direction are not displayed in this image.

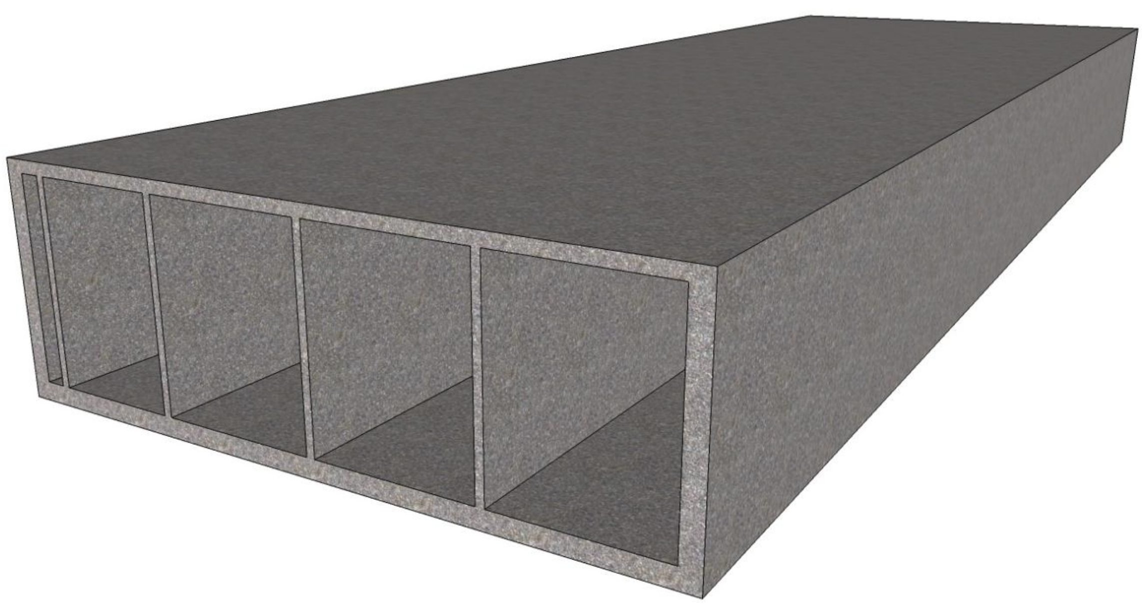


Figure 26.1 – cross section of caisson

Since the entire top and bottom slabs are located alternately in the tensile zone of the cross section, the required amount of reinforcement steel can be calculated by means of equation 26.13.

$$A_s = \frac{M_{wave} \cdot 10^6}{f_{yk} \cdot z}$$

Equation 26.13

Where: A_s = required amount of reinforcement steel in cross section [mm^2/m]
M_{wave} = momentum caused by wave load during transport [kNm/m]
z = distance between center of top and bottom slab [mm]

The required percentage of reinforcement steel to absorb the tensile stress caused by wave loads can now be calculated by means of equation 26.14.

$$\rho_{min} = \frac{A_s}{10 \cdot d_{slab}}$$

Equation 26.14

Where: ρ_{min} = minimum required reinforcement percentage to absorb bending moment caused by wave load [%]
d_{slab} = slab thickness [mm]

Note that in this case, the calculated reinforcement percentage represents the sum of the applied reinforcement at the inner and outer side of the slabs, in longitudinal direction.

As the wave length equals the length of the caisson, the corresponding wave height will be about 2% of the wave height, according to the wave steepness in open water.

26.5. Local tensile and compressive forces

The compressive forces that originate from local forces, such as fenders and crane loads are not elaborated in this thesis. The rail gauges of the STS crane are located straight above walls in longitudinal direction and fenders are located in front of inner walls in transversal direction. The walls are assumed to be thick enough to absorb the compressive forces, since the compressive strength of concrete is much higher that the tensile strength.

The rail gauge connection of the STS crane is exposed to tensile forces during a storm, when the cranes are tied down on their rails. A detailed check of the introduction of this tensile force into the concrete is beyond the scope of this thesis.
The required reinforcement to resist the local bollard force is beyond the scope as well. Bollards are located at the intersection of inner walls in transversal direction and the outer wall of the caisson.

27. Final design

27.1. Caisson dimensions

There are a couple of aspects that need to be taken into account for the determination of the caisson dimensions. The most important aspects are listed below:

- Width-length-ratio of 1:3
- STS crane rails span
- Stability criteria of failure mechanisms
- Transportability
- Draught
- Constructability

The quay wall is designed to be transported to different locations during its entire lifespan, so transportability is an important criterion for the dimensions of the caissons. Transport is done by towing the caissons by means of a tug boat, but when the caissons are small enough, one could also consider lifting them on a barge or a vessel. This requires additional hoisting equipment, but the sailing speed can be higher.
Keep in mind that the caissons cannot be too small, because of their stability against sliding and overturning. Position keeping in done by gravity and a reinforced top layer that cement the upper caissons together should be avoided. Such a top layer is usual for a mass concrete block-work wall, but hampers flexibility. The caissons must therefore be heavy enough to satisfy all stability criteria by means of their mass and dimensions only.

When large caissons are towed by tugs, they must be capable to resist the dynamic wave forces during transport. These forces are larger for longer caissons, when both ends are supported by a wave trough and the intermediate part by one single wave crest.

For good navigational properties, a width-length ratio of 1:3 should be applied. The crane rails of nearly all STS cranes have a span of 30,5 meter, but the width of the quay wall is somewhat more. It can be seen in appendix O, that STS cranes for panamax vessels may have a span of just 16m between their crane rails. However, all cranes for larger vessels have the same span of 30,5 meters. To make the flexible quay wall suitable for theses cranes as well, the width is determined by the accommodation of a crane rail with a span of 30,5 meters. Besides, a width of about 18m for a panamax crane turns out to be insufficient to guarantee stability of the structure.

When the width of the caissons is about 32 to 35 meters, the length should be in the order of 100 meters, which is very large.

One could also consider using much smaller caissons, which are rotated 90 degrees. Now the length is about 32 to 35 meters and the corresponding width should now be about 11 meters. Since two caissons are positioned on top of each other, the height will be in the order of 11 meters as well, for both “small” and very large caissons.

Another possible alternative is to place two caissons next to each other over the width of the quay. The caisson dimensions for this situation are about 17 meters in width, 11 in height and 50 meters in length.

An illustration of the three proposed caisson dimensions can be found in respectively figure 27.1, 27.2 and 27.3.

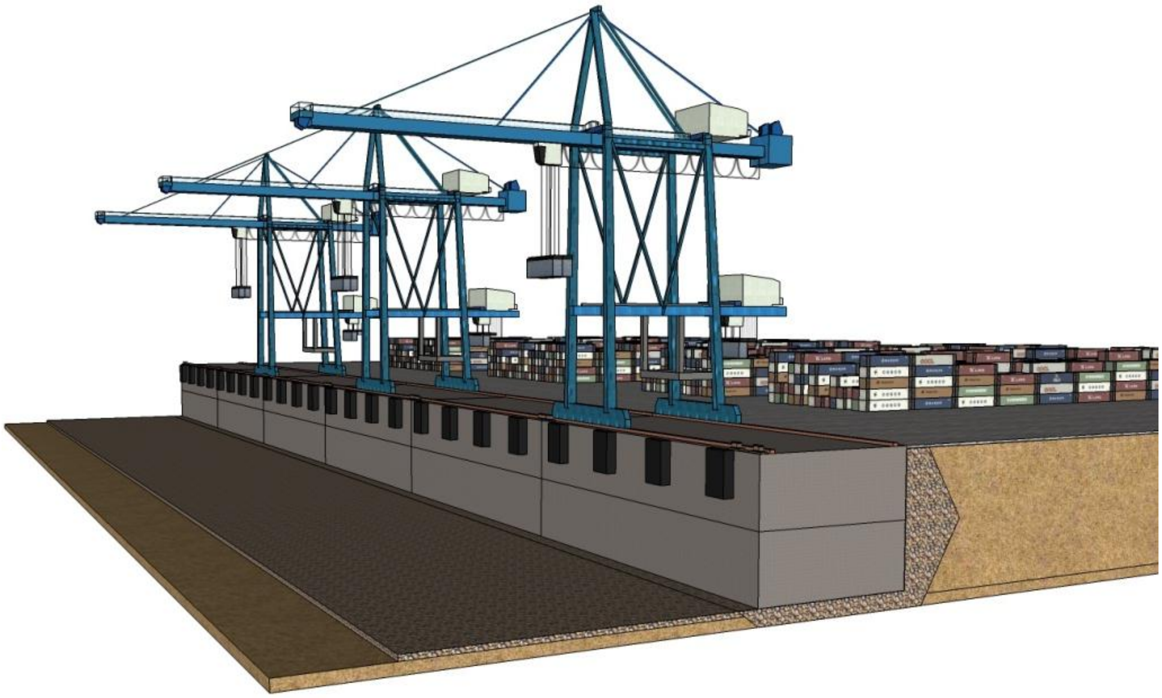


Figure 27.1 – very large caissons: 100m x 33m x 11m

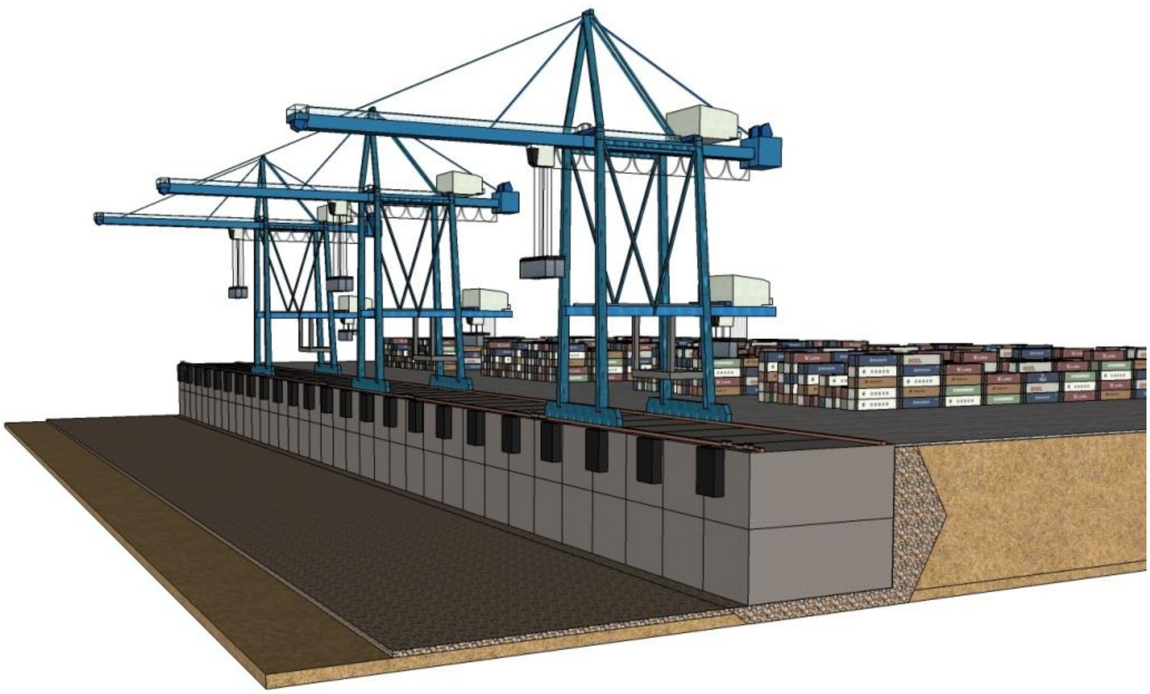


Figure 27.2 – smaller caissons 90 degrees rotated: 33m x 11m x 11m

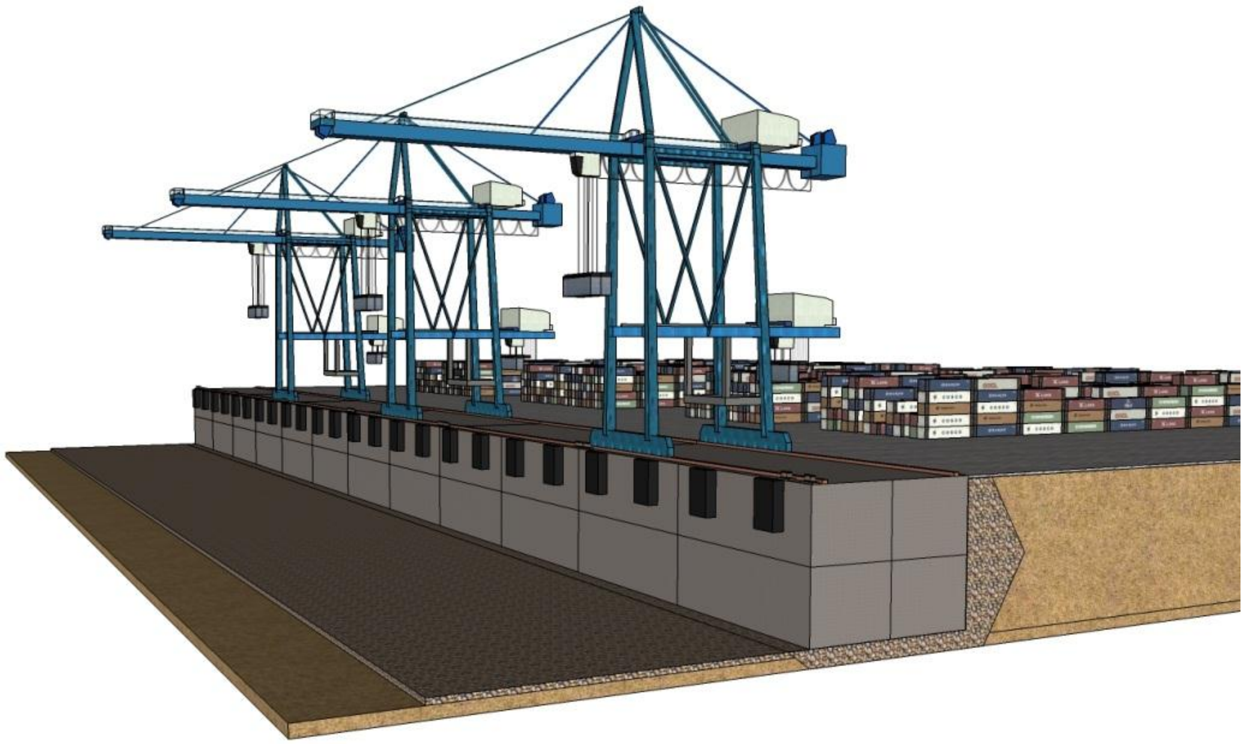


Figure 27.3 – two caissons next to each other: 50m x 17m x 11m

A calculation sheet to determine the satisfaction of the failure mechanisms was established in Excel and can be found in the appendix. Calculations show that stability against overturning is not satisfied for the alternative illustrated in figure 27.3, because of the reduced width of a single caisson. This problem could be solved by applying a proper connection between the caissons, but it is preferred to use caissons that don't need any connection with each other.

The alternative in figure 27.1 turns out to be the most stable solution, because of the large caisson dimension. Depending on the percentage of the total hawser force that is absorbed by one single caisson, the alternative in figure 27.2 seems to be possible as well.

A length of 32 to 35 meter and a height and width of both 11 meters are more usual dimensions for a caisson, but there are some serious disadvantages of using these dimensions, compared to the very large caissons.

In the first place, much more caissons are needed, which makes relocation more complicated and costly. Towing a few very large caissons is easier than towing lots of smaller ones.

Another disadvantage is the increased draught of the smaller caissons, compared to the very large caissons. An increased draught means that more construction material is needed for the alternative illustrated in figure 27.2. This is because outer walls need to be thicker than inner walls.

Another disadvantage is the differential settlement of the caissons. Much more caissons are needed when smaller caissons are used, which means more transitions from one caisson to another.
An advantage of the smaller caissons is the reduced wave load during transport.

Caissons are usually constructed in a construction dock, although construction on a slipway or a floating construction method is also possible. The size of the caissons may therefore also be limited by the dimensions of an existing construction dock or a new construction dock has to be build.

Based on the mentioned advantages and disadvantages it is decided to use caissons of 100m in length, 33m wide and 11m high, as illustrated in figure 27.1.

27.2. Applied safety factors

Safety factors must be applied to guarantee the stability and structural strength of the quay wall. The Manual Hydraulic Structures was used for safety factors with respect to the caisson's stability, whereas the Eurocode was used for material factors for soil, concrete and reinforcement steel. An overview of the applied safety factors is given in table 27.1.

Applied safety factors			
	Safety factor		**Material factor**
	Non-beneficial	Beneficial	
Permanent loads	1,2	0,9	-
Variable loads	1,3	0,9	-
Accidental loads	1,0	1,0	-
Soil angle of internal friction	-	-	1,25
Steel yield stress	-	-	1,15
Concrete tensile strength	-	-	1,5

Table 27.1 – applied safety factors

27.3. Load combinations

Table 27.2 gives an overview of the load combinations for each failure mechanism. These loads are determined using Excel sheets in which all required equations are programmed. The input and output of these sheets can be found in the appendix of the report, whereas a description of the equations is given in the report itself. The magnitude of all relevant loads can be found in several appendices.

	Horizontal sliding SLS	Overturning moment SLS	Bearing capacity SLS	Floating stability and transport	Horizontal sliding ULS	Overturning moment ULS	Bearing capacity ULS
Weight of single caisson	x	x	x	x	x	x	x
Ballast weight in upper caisson	x	x	x		x	x	x
Ballast weight in lower caisson	x	x	x		x	x	x
Horizontal soil pressure land side	x	x	x		x	x	x
Horizontal water pressure sea side	x	x	x		x	x	x
Upward buoyancy force	x	x	x		x	x	x
Weight of STS crane		x	x			x	x
Lift force of containers		x	x				
Wind force at STS crane in SLS	x	x	x				
Wind force at STS crane in ULS					x	x	x
Fender reaction force							
Vertical component mooring lines SLS	x	x	x				
Horizontal component mooring lines SLS	x	x	x				
Vertical component mooring lines ULS					x	x	x
Horizontal component mooring lines ULS					x	x	x
Wave load during operation	x	x	x		x	x	x
Wave load during transport				x			
Tugboat force during transport				x			

Table 27.2 – load combinations for stability

An upper and lower limit is determined for the loads per failure mechanism, because some loads that are beneficial for one criterion are unfavorable for another. For example, a heavy weight is favorable for sliding and overturning, whereas it is unfavorable for the required bearing capacity. Distinction between different scenarios is included in the Excel sheet in appendix K.

27.4. Wall thickness and reinforcement steel

The determination of the wall thickness and required reinforcement steel is of importance, since it has a direct influence on the weight of the structure. An overview of the various loads acting on the structure is illustrated in figure 27.4.

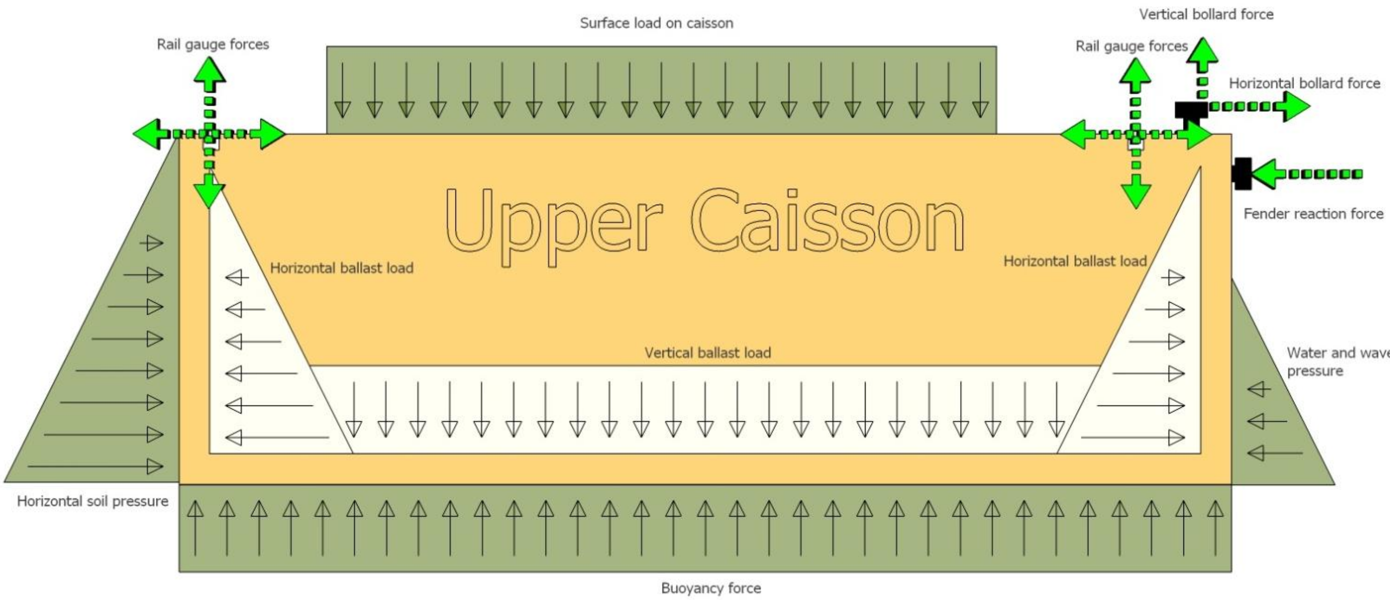


Figure 27.4 – origin of various loads on upper caisson (not to scale)

Note that the image in figure 27.4 does not display the governing load combinations, but just describes the various loads acting on the upper caisson. Also note that the tug boat forces and drag force during transport are not included in this picture. A top view of figure 27.4 is given in figure 27.5, where the tug boat force and drag force are illustrated.

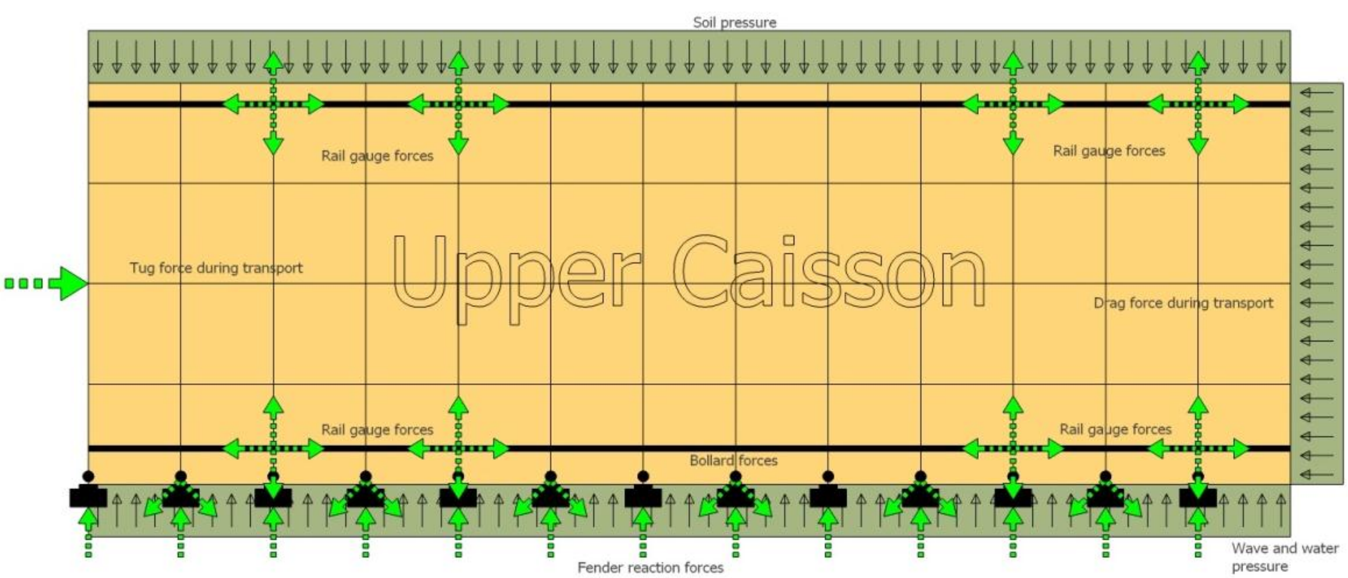


Figure 27.5 – top view of various loads on upper caisson (not to scale)

There are 12 inner walls in transversal direction inside the caissons and 4 in longitudinal direction. Since the caissons are 11 meters high, this results in sections of about 9,5m x 7,2m for the walls in longitudinal direction and sections of 7,2m x 7,6m for both bottom and top slabs. Consequently, the side walls have sections of about 9,5m x 7,6m. Adding more inner walls has a relatively small influence on the magnitude of the shear forces, since the height remains about 9,5m. More inner walls reduce the width of the sections, but also increase the height-width ratio given in the tables in appendix V. Applying an inner floor would decrease the shear force significantly, but this becomes problematic for both construction and removal of ballast material. Applying more inner walls does have an influence on the bending moment in the wall sections, since it increases quadratic. However, calculations showed that the required amount of reinforcement steel is already close to the minimum reinforcement percentage. All caissons are constructed with a C30/37 concrete class.

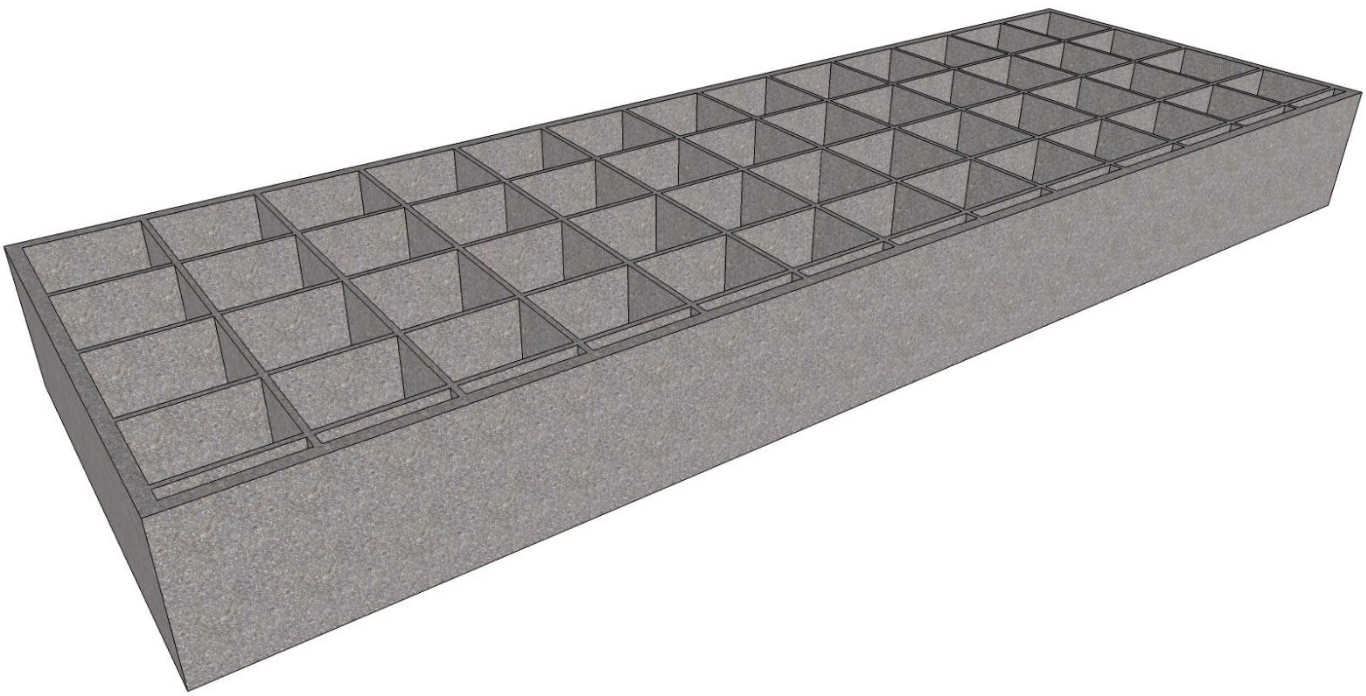


Figure 27.6 – illustration of caisson design, without displaying the top slab

First it was determined whether or not the walls can be designed in such a way that no shear reinforcement is required, using equation 26.1 to 26.6. Using the maximum shear forces according to the tables in appendix V, the minimum required wall thickness was calculated in appendix S. The results are presented in table 27.3, but the calculated thicknesses seemed to be uneconomic. Besides, the corresponding draught would make it very difficult to place the upper caissons on top of the lower ones, because the draught is larger than the water depth above an immersed caisson. Using more inner walls was considered, but this would not reduce the amount of concrete to an acceptable level.

Using shear reinforcement therefore seems unavoidable, so equations 26.7 to 26.11 were used to determine the amount of shear reinforcement for a chosen wall thickness. The chosen wall thickness and the corresponding reinforcement for both bending and shear are given in table 27.3 to 27.6. Calculations can be found in appendix Q, which contains an extensive Excel sheet. Note that the caissons both have the same dimensions and wall thickness for this first design.

The characters A to E are used to indicate the governing situation for the loads on the walls and slabs.

A = maximum of 11m water pressure in floating conditions
B = temporary container storage and equipment on the quay wall
C = pumping ballast water out of immersed caisson (air inside and water pressure on top)
D = ballast sand inside caisson and outside water level at low tide
E = horizontal retained soil pressure (outside) minus water pressure at low tide (inside)

Wall thickness of C30/37 concrete caissons (first design)				
	Minimum required wall thickness without shear reinforcement [mm]			
Caisson section	**Upper caisson**		**Lower caisson**	
Bottom slab	1140	[A]	1140	[A]
Top slab	660	[B]	835	[C]
Outer walls in longitudinal direction	1115	[A / D]	1575	[E]
Outer walls in transversal direction	1115	[A / D]	1575	[E]
Inner walls in longitudinal direction	300		300	
Inner walls in transversal direction	300		300	
	Chosen wall thickness [mm]			
Caisson section	**Upper caisson**		**Lower caisson**	
Bottom slab	900		900	
Top slab	600		600	
Outer walls in longitudinal direction	900		900	
Outer walls in transversal direction	900		900	
Inner walls in longitudinal direction	300		300	
Inner walls in transversal direction	300		300	

Table 27.3 – wall thickness of caissons for first calculation

Required shear reinforcement in concrete walls and slabs (first design)		
	Applied shear reinforcement [mm] – [mm]	
Caisson section	**Upper caisson**	**Lower caisson**
Bottom slab	Ø 12 – 275	Ø 12 – 275
Top slab	Ø 12 – 280	Ø 12 – 255
Outer walls	Ø 12 – 275	Ø 12 – 240
Inner walls	None	None

Table 27.4 – required shear reinforcement in caisson walls and slabs for first calculation

Required reinforcement in concrete walls and slabs for bending (first design)		
	Applied bending reinforcement [mm] – [mm]	
Caisson section	**Upper caisson**	**Lower caisson**
Bottom slab longitudinal outer side	Ø 25 – 270	Ø 25 – 270
Bottom slab longitudinal inner side	Ø 25 – 270	Ø 25 – 270
Bottom slab transversal outer side	Ø 25 – 270	Ø 25 – 270
Bottom slab transversal inner side	Ø 25 – 270	Ø 25 – 270
Top slab longitudinal outer side	Ø 20 – 260	Ø 20 – 260
Top slab longitudinal inner side	Ø 20 – 260	Ø 20 – 215
Top slab transversal outer side	Ø 20 – 260	Ø 20 – 260
Top slab transversal inner side	Ø 20 – 260	Ø 20 – 215
Outer walls longitudinal outer side	Ø 25 – 270	Ø 25 – 270
Outer walls longitudinal inner side	Ø 25 – 270	Ø 25 – 270
Outer walls transversal outer side	Ø 25 – 270	Ø 25 – 270
Outer walls transversal inner side	Ø 25 – 270	Ø 25 – 260
Inner wall longitudinal outer side	Ø 12 – 180	Ø 12 – 180
Inner wall longitudinal inner side	Ø 12 – 180	Ø 12 – 180
Inner wall transversal outer side	Ø 12 – 180	Ø 12 – 180
Inner wall transversal inner side	Ø 12 – 180	Ø 12 – 180

Table 27.5 – bending reinforcement in caisson walls and slabs for first calculation

Color legend of table 27.5:

Black = Bending moment caused by soil and/or water pressure is governing
Red = Minimum reinforcement percentage for tensile stress is governing
Green = Minimum reinforcement percentage for bending moment is governing

Applying the minimum reinforcement percentage in the bottom slab, leads to an over dimensioning by factor 1,5. This is desirable because of possible irregularities in the gravel bed foundation that may increase the load on the bottom slab.

Table 27.6 contains the same information as table 27.5, but in different units.

Required reinforcement in concrete walls and slabs for bending (first design)				
	Applied bending reinforcement [mm] – [mm]			
	Upper caisson		**Lower caisson**	
Caisson section	**kg/m^3**	**[%]**	**kg/m^3**	**[%]**
Bottom slab longitudinal outer side	15,86	0,20	15,86	0,20
Bottom slab longitudinal inner side	15,86	0,20	15,86	0,20
Bottom slab transversal outer side	15,86	0,20	15,86	0,20
Bottom slab transversal inner side	15,86	0,20	15,86	0,20
Top slab longitudinal outer side	15,81	0,20	15,81	0,20
Top slab longitudinal inner side	15,81	0,20	19,12	0,24
Top slab transversal outer side	15,81	0,20	15,81	0,20
Top slab transversal inner side	15,81	0,20	19,12	0,24
Outer walls longitudinal outer side	15,86	0,20	15,86	0,20
Outer walls longitudinal inner side	15,86	0,20	15,86	0,20
Outer walls transversal outer side	15,86	0,20	15,86	0,20
Outer walls transversal inner side	15,86	0,20	16,47	0,21
Inner wall longitudinal outer side	16,44	0,21	16,44	0,21
Inner wall longitudinal inner side	16,44	0,21	16,44	0,21
Inner wall transversal outer side	16,44	0,21	16,44	0,21
Inner wall transversal inner side	16,44	0,21	16,44	0,21

Table 27.6 – bending reinforcement in caisson walls and slabs for first calculation in kg/m^3 and %

The applied reinforcement in the bottom and top slabs is able to absorb the total bending moment at the entire caisson during transport as well. The actual absorbable moment is somewhat higher than the values given in appendix Q, since some reinforcement in the walls is in the tensile zone too.

Table 27.7 represents the costs of the construction materials, required for the first design of the caisson quay wall.

Costs of construction material only (first design)			
Construction material	**Quantity used**	**Unit price**	**Price**
Concrete in [m^3]	130.746 m^3	€150	€19.611.900
Reinforcement steel for bending in [kg]	8.040.983 kg	€1,25	€10.051.229
Shear reinforcement steel in [kg]	467.072 kg	€1,25	€583.840
Framework for casting top slab [m^2]	3.566 m^2	€200	€713.200
Framework for casting other sections [m^2]	19.340 m^2	€100	€1.934.000
Total price construction material	-	-	€32.894.169
Construction material price per meter quay	-	-	**€46.992**

Table 27.7 – costs of construction material for first design

27.5. Optimizing the wall thickness and reinforcement

It becomes interesting to optimize the amount of concrete and reinforcement steel, in order to save money on the construction materials.

A final calculation was carried out, where a distinction is made between the loads on the upper caisson and on the lower one. For the lower caisson, a distinction between the outer walls on the land side and those on the sea side was made as well.
The required amount of reinforcement steel is also determined by making a distinction between negative and positive bending moments in both x and y-direction.

The optimal wall thickness was determined by plotting the wall thickness and the corresponding costs of the construction material in a graph. An example of such a graph is given in figure 27.7, which applies for the outer walls of the upper caissons. For a wall thickness between 300mm and 900mm the required amount of reinforcement steel was calculated for five criteria, being: positive and negative bending moments in both x and y-direction and shear reinforcement. The graphs and tables of all other wall sections can be found in appendix R. Each graph consists of 39 values that were all calculated separately by means of the Excel sheet in appendix Q. However, appendix Q only contains the final values, in order to keep the amount of appendices within a reasonable quantity.

It can be seen from the figure that the steel price increases, for a larger concrete thickness, at a certain moment. This is because the minimum reinforcement percentage becomes governing for a large wall thickness. By increasing the wall thickness further, one must apply more reinforcement steel too. Consequently, the actual strength of the wall is far more than the required strength in that case. The graphs are not perfectly

smooth and have some small bends in it. This is because of shifts to different diameters of the reinforcement bars.
The calculated shear reinforcement is a maximum value that is only required at some places in the wall sections. Other sections need less shear reinforcement and some parts don't need it at all. Hairpins are used as shear reinforcement and the average price of hairpins per cubic meter of concrete was assumed to be 30% of the maximum price in the governing section. The contribution of shear reinforcement remains small, compared to the weight of reinforcement steel of bending moments.
A concrete price of 150 €/m^3 and a steel price of 1,25 €/kg were used for the determination of the prices in figure 27.7.

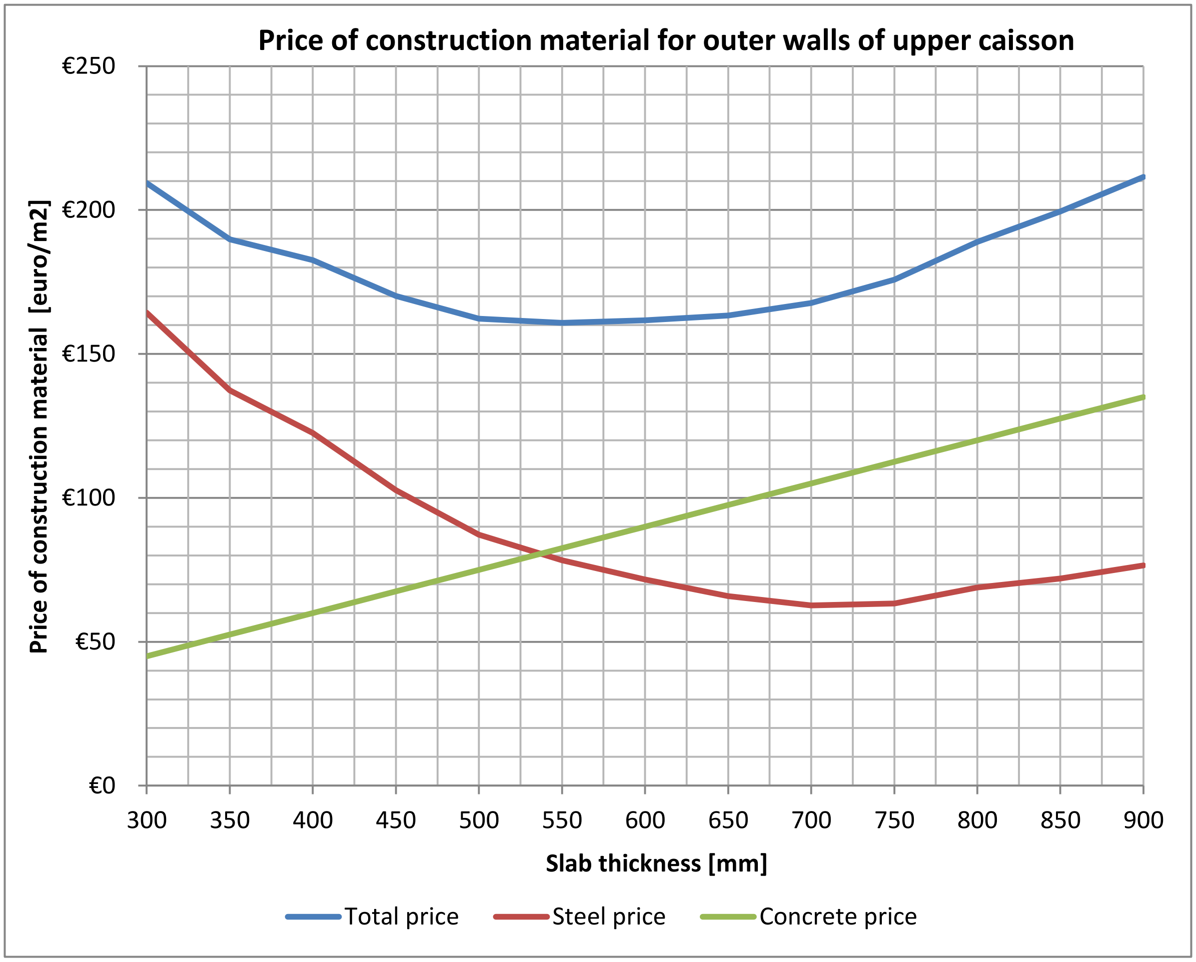


Figure 27.7 – method for optimization of the wall thickness related to material costs

The results of the final design are displayed in tables 27.8 to 27.12, whereas the calculation sheet can be found in appendix Q. The numbers in this sheet are those of the final calculation, but the same sheet was also used for the first calculation. It can be concluded that an optimization of the used construction materials, reduces the price significantly.

Wall thickness of C30/37 concrete caissons (final design)		
	Optimized wall thickness [mm]	
Caisson section	**Upper caisson**	**Lower caisson**
Bottom slab	450	550
Top slab	350	400
Outer wall in longitudinal direction land side	550	550
Outer wall in longitudinal direction sea side	550	400
Outer walls in transversal direction	550	550
Inner walls in longitudinal direction	300	300
Inner walls in transversal direction	300	300

Table 27.8 – optimized wall thickness of caissons for final calculation

Note that the bottom slab of the upper caisson is thinner than the bottom slab of the lower one. The upper caisson is placed in top of the lower one, which provides a smooth foundation. The lower caisson is placed on a gravel bed foundation, so an extra safety margin of 1,5 for possible irregularities was applied.

Required shear reinforcement in concrete walls and slabs (final design)		
	Applied shear reinforcement [mm] – [mm]	
Caisson section	**Upper caisson**	**Lower caisson**
Bottom slab	Ø 16 – 260	Ø 16 – 235
Top slab	Ø 16 – 285	Ø 16 – 280
Outer walls land side	Ø 16 – 290	Ø 16 – 250
Outer walls sea side	Ø 16 – 290	Ø 16 – 280
Inner walls	None	None

Table 27.9 – required shear reinforcement in caisson walls and slabs for final calculation

Required reinforcement in concrete walls and slabs (final design)		
	Applied reinforcement	
Caisson section	**Upper caisson**	**Lower caisson**
Bottom slab longitudinal outer side	Ø 16 – 220	Ø 20 – 285
Bottom slab longitudinal inner side	Ø 25 – 185	Ø 32 – 255
Bottom slab transversal outer side	Ø 16 – 220	Ø 20 – 285
Bottom slab transversal inner side	Ø 25 – 185	Ø 32 – 255
Top slab longitudinal outer side	Ø 16 – 260	Ø 16 – 240
Top slab longitudinal inner side	Ø 25 – 215	Ø 25 – 210
Top slab transversal outer side	Ø 16 – 285	Ø 16 – 250
Top slab transversal inner side	Ø 25 – 215	Ø 25 – 210
Outer walls longitudinal outer side land side	Ø 25 – 255	Ø 20 – 250
Outer walls longitudinal inner side land side	Ø 20 – 205	Ø 32 – 260
Outer walls transversal outer side land side	Ø 25 – 255	Ø 20 – 285
Outer walls transversal inner side land side	Ø 20 – 205	Ø 32 – 245
Outer walls longitudinal outer side sea side	Ø 25 – 255	Ø 16 – 250
Outer walls longitudinal inner side sea side	Ø 20 – 205	Ø 25 – 210
Outer walls transversal outer side sea side	Ø 25 – 255	Ø 16 – 250
Outer walls transversal inner side sea side	Ø 20 – 205	Ø 25 – 210
Outer side walls in longitudinal direction outer side	Ø 25 – 255	Ø 20 – 250
Outer side walls in longitudinal direction inner side	Ø 20 – 205	Ø 32 – 260
Outer side walls in transversal direction outer side	Ø 25 – 255	Ø 20 – 285
Outer side walls in transversal direction inner side	Ø 20 – 205	Ø 32 – 245
Inner wall longitudinal outer side	Ø 12 – 180	Ø 12 – 180
Inner wall longitudinal inner side	Ø 12 – 180	Ø 12 – 180
Inner wall transversal outer side	Ø 12 – 180	Ø 12 – 180
Inner wall transversal inner side	Ø 12 – 180	Ø 12 – 180

Table 27.10 – optimized bending reinforcement for final calculation

Color legend of table 27.10:

Black = Bending moment caused by soil and/or water pressure is governing
Red = Minimum reinforcement percentage for tensile stress is governing
Green = Minimum reinforcement percentage for bending moment is governing

Note that the wave load during transport is not governing. The bottom and top slabs are able to absorb the additional bending moment caused by waves and the vertical walls can resist the extra shear force as well.

The same results are given in table 27.11, but different units are used.

Required reinforcement in concrete walls and slabs (final design)				
	Applied reinforcement			
	Upper caisson		Lower caisson	
Caisson section	**kg/m³**	**[%]**	**kg/m³**	**[%]**
Bottom slab longitudinal outer side	15,94	0,20	15,73	0,20
Bottom slab longitudinal inner side	46,29	0,59	45,01	0,57
Bottom slab transversal outer side	15,94	0,20	15,73	0,20
Bottom slab transversal inner side	46,29	0,59	45,01	0,57
Top slab longitudinal outer side	17,34	0,22	16,44	0,21
Top slab longitudinal inner side	51,21	0,65	45,87	0,58
Top slab transversal outer side	15,82	0,20	15,78	0,20
Top slab transversal inner side	51,21	0,65	45,87	0,58
Outer walls longitudinal outer side land side	27,47	0,35	17,94	0,23
Outer walls longitudinal inner side land side	21,87	0,28	44,15	0,56
Outer walls transversal outer side land side	27,47	0,35	15,73	0,20
Outer walls transversal inner side land side	21,87	0,28	46,85	0,60
Outer walls longitudinal outer side sea side	27,47	0,35	15,78	0,20
Outer walls longitudinal inner side sea side	21,87	0,28	45,87	0,58
Outer walls transversal outer side sea side	27,47	0,35	15,78	0,20
Outer walls transversal inner side sea side	21,87	0,28	45,87	0,58
Outer side walls in longitudinal direction outer side	27,47	0,35	17,94	0,23
Outer side walls in longitudinal direction inner side	21,87	0,28	44,15	0,56
Outer side walls in transversal direction outer side	27,47	0,35	15,73	0,20
Outer side walls in transversal direction inner side	21,87	0,28	46,85	0,60
Inner wall longitudinal outer side	16,44	0,21	16,44	0,21
Inner wall longitudinal inner side	16,44	0,21	16,44	0,21
Inner wall transversal outer side	16,44	0,21	16,44	0,21
Inner wall transversal inner side	16,44	0,21	16,44	0,21

Table 27.11 – optimized bending reinforcement for final calculation in kg/m³ and %

Costs of construction material only (final design)			
Construction material	**Quantity**	**Unit price**	**Price**
Concrete in [m^3]	92.470 m^3	€150	€13.870.500
Reinforcement steel for bending in [kg]	8.649.967 kg	€1,25	€10.812.459
Shear reinforcement steel in [kg]	1.308.580 kg	€1,25	€1.635.725
Framework for casting top slab [m^2]	3.031 m^2	€200	€606.200
Framework for casting other sections [m^2]	23.363 m^2	€100	€2.336.300
Total price construction material	-	-	€29.261.184
Construction material price per meter quay	-	-	**€41.802**

Table 27.12 – final costs of construction materials used

Comparing table 27.12 and table 27.7, it can be concluded that the optimization of the wall thickness and reinforcement steel has led to a reduction of the construction material costs of more than 3,6 million euros.

27.6. Final quay wall stability

During operation, the lower caissons will completely be filled with water and the upper ones will be filled with 9,0m of sand. Satisfaction of the quay wall's stability is checked with a final unity check. This value is obtained by dividing the actual value of a stability criterion, by the maximum allowable one. Since all safety and material factors are already included in the calculation, a value equal or lower than 1.0 suffices. The results of this unity check values are listed in table 27.13, just like the properties for floating stability. The values correspond with the worst case scenario for each criterion.

Stability criterion	*Unity check*	*Metacenter height*	*Natural oscillation period*
Horizontally sliding	0,95	-	-
Overturning moment	0,77	-	-
Bearing capacity / slide circle	0,95	-	-
Static floating stability	-	15,57 m	-
Floating stability during immersion	-	14,87 m	-
Dynamic floating stability	-	-	5,58 s

Table 27.13 – final satisfaction of stability criteria

Note that the optimal unity check values and the absorbable earthquake load can be achieved for each specific location by determining the amount of ballast material in the caissons. This can be done by means of the calculation sheets in appendix K.
The final design is capable of resisting earthquake loads of at least PGA $\approx$ 2m/s^2 and requires a minimum bearing capacity of 400kN/m^2. If a higher resistance against earthquakes is desired, one can apply more ballast material, but this will also increase the required bearing capacity.

Part V
Costs & Financial Feasibility Comparison

28. Introduction

This part basically determines whether the proposed structure could be a financially feasible alternative to traditional quay walls or not. A comparison is made between an in situ constructed quay wall that becomes inadequate or useless before the end of its lifetime and the flexible quay wall structure that is utilized during its entire lifespan of 50 years. This will be done by plotting the over-all costs balances of both quay wall structures in time. The comparison will investigate which relocation frequencies could be financially feasible and whether the structure is financially feasible at all, or not.

The reconstruction costs of the flexible structure are lower than the construction costs of a new in situ constructed quay wall, which is favorable for its financial feasibility. Another important advantage of the flexible quay is the faster reconstruction period at a new location, compared to the construction of a new in situ constructed quay wall. By constructing a quay wall faster, one can respond faster to changes and starts to make revenues earlier. In case of selling the flexible quay wall before the end of its lifetime, the higher residual value is an important aspect too.

The possibilities to recoup the higher initial construction costs of the flexible structure are researched for different scenarios. Some scenarios are approached from a point of view, in which the structure is used by the same owner during its entire lifespan. Other scenarios are approached from a point of view, in which the flexible structure is sold after using it for a certain period.

29. Traditional, in situ constructed quay wall structure

29.1. Construction costs

The first problem already arises when the term "traditional, in situ constructed quay wall" is used, because what is a traditional quay wall? There are many different types of quays, which differ a lot in price and properties. Figure 29.1 shows the costs of gravity quay wall structures only, relative to their retaining height. Since the data in this figure is for gravity structures in particular, it offers more appropriate data than figure 10.1, for a comparison with the flexible caisson quay wall. However, the predicted price for 22m retaining height is more or less the same.

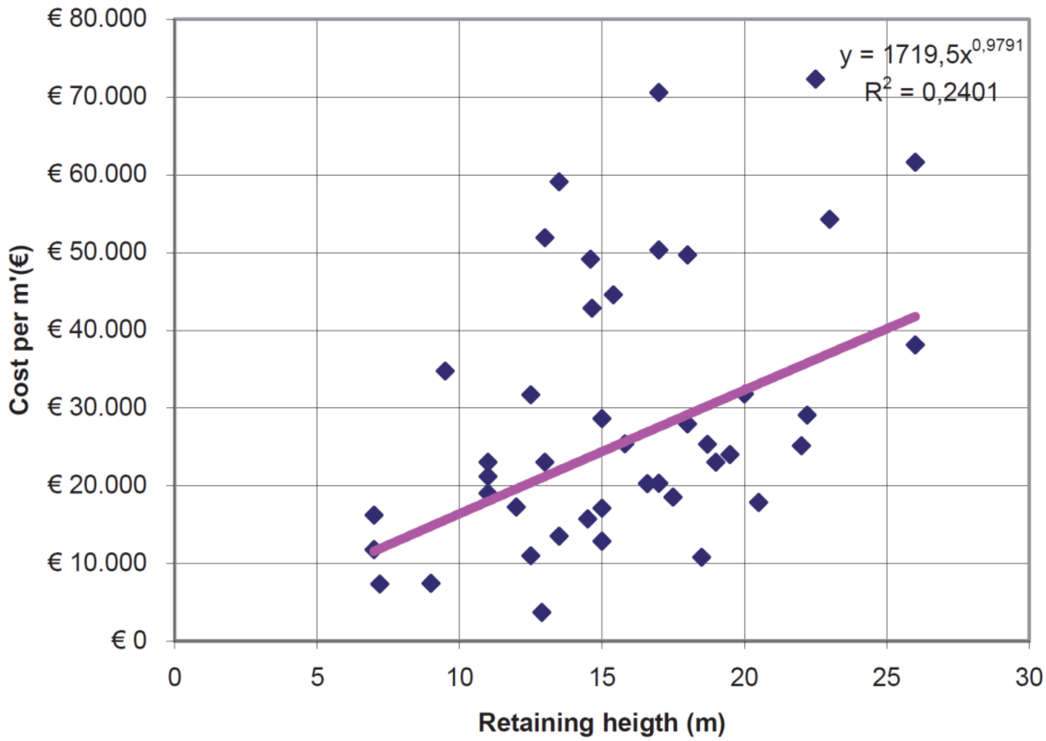


Figure 29.1 – gravity quay wall costs per running meter relative to the retaining height [A History of Quay Walls, J.G. de Gijt (2008)]

The figure shows that there is a large variety in construction costs, which makes it very hard to determine one price for the construction costs of a traditional quay wall structure. Note that the data in figure 29.1 is not for container quays in particular, but contains data of bulk terminals as well. Obviously, there is some difference between the two, but local conditions, like for instance soil properties, have a larger influence on the final construction costs of a quay wall.

The trend line in figure 29.1 is used for the financial feasibility comparison in this master thesis. Because the data consists of 2008 values, a correction is made to obtain 2013 values, by assuming a 2,5% costs inflation per year. This results in an average construction price of 40.000 €/m, for an in situ constructed quay wall, with a retaining height of 22m.

29.2. Upgrade costs

As mentioned before, upgrading an existing quay wall that has become inadequate is a possibility to make it useful again. However, an upgraded quay wall has different properties than the original one, which makes it hard to compare with the flexible structure. An upgraded structure can for instance handle larger, or more vessels and can therefore make more revenues. Since the fixed structure needs to have the same properties for a fair comparison with the flexible structure, upgrading traditional quay walls is only taken into account, until the moment of upgrading. The period after upgrading is not considered, because the quay wall properties have changed.
The construction costs of an upgraded quay wall are assumed to be 70% of the price in figure 29.1, with a correction for the price inflation. This is because the existing old quay wall reduces the loads and therefore reduces the price with about 30%.

29.3. Construction period

Just like the construction costs, the construction period of a traditional quay wall varies for each type of structure and location. Based on knowledge and experience within Royal HaskoningDHV and APM Terminals, an average construction period of 18 months is used in this comparison.

29.4. Residual value

The residual value of a fixed quay wall that has become useless is very low. In fact, it might even cost money to demolish it. In this comparison study, the demolition costs are assumed to be 10% of the initial construction costs in figure 29.1. These costs are included in the price of a new in situ constructed quay wall, because it is assumed that the previous one is demolished. Furthermore, it is assumed that the residual value of the traditional quay wall is negligible.

29.5. Utilized lifespan

The utilized lifespan is almost impossible to determine on beforehand. The uncertainty of this period is basically the reason which drives the demand for a flexible quay wall structure. The utilized lifespan of the fixed quay wall will therefore be variable between 5 and 15 years during this feasibility study. An explanation of this choice can be found in chapter 7.10.

A comparison is made between using a traditional quay wall for a certain period and relocating a flexible quay wall structure after the same period. The difference in costs mainly originates from the difference between relocation costs of a flexible quay and construction costs of a traditional quay wall structure. The difference in (re)construction time has an influence as well. Obviously, the initial construction costs of the flexible quay wall are a very important factor too.

30. Flexible quay wall structure

30.1. Construction costs

The costs of the construction material for the caissons were already determined in chapter 27 and are displayed in table 27.12. The final costs of the flexible quay wall are higher, since things like dredging works, gravel materials and transport costs were not taken into account so far. Table 30.1 summarizes the costs estimates of the most relevant aspects for building the flexible caisson quay wall, but one must realize that aspects like dredging works and travel distance are very much location dependent. All prices are a rough estimate, based on experience and knowledge within the companies of Royal HaskoningDHV, APM Terminals and TU Delft.

Description	*Quantity*	*Unit price*	*Total price*
Concrete for caisson construction	92.500 m^3	150 €/m^3	€ 13.875.000
Reinforcement steel in caissons	10.000.000kg	1,25 €/kg	€ 12.500.000
Framework for caisson construction	23.500 m^2	100 €/m^2	€ 2.350.000
Framework for top slab of caissons	3.000 m^2	200 €/m^2	€ 600.000
Construction dock	1 pcs	30% of caisson	€ 900.000
Trench dredging works	435.000 m^3	5 €/m^3	€ 2.175.000
Dredger (de)mobilization costs	1 pcs	800.000 €/pcs	€ 800.000
Immersing caissons	14 pcs	100.000 €/pcs	€ 1.400.000
Back fill (sand)	250.000 m^3	5 €/m^3	€ 1.250.000
Back fill (gravel)	300.000 m^3	15 €/m^3	€ 4.500.000
Bottom protection	45.000 ton	25 €/ton	€ 1.125.000
Geotextile filter layer	80.000 m^2	12 €/m^2	€ 960.000
Travel distance	300 nm	1.000 €/nm	€ 300.000
Tug boat mobilization costs	14 pcs	20.000 €/pcs	€ 280.000
Gravel bed foundation	40.000 ton	20 €/ton	€ 800.000
Bollards	60 pcs	3.000 €/pcs	€ 180.000
Fenders	60 pcs	30.000 €/pcs	€ 1.800.000
Total price per meter of quay wall	**700 m**	**65.421 €/m**	**€ 45.795.000**

Table 30.1 – overview of relevant costs of flexible quay wall structure

The initial construction costs of the flexible quay wall structure are estimated at €45.795.000, which corresponds with 65.421€/m. As expected, this price is higher than the average price of a traditional quay wall that is constructed in situ. Based on knowledge and experience within the companies of Royal HaskoningDHV and APM Terminals, the initial construction period is estimated at 2,5 years.

The assumptions that were made to determine the quantities in table 30.1 are illustrated in figure 30.1 to 30.3. These quantities strongly depend on local conditions, like original bottom profile and soil properties. The mentioned quantities are a rough estimate of the average quantities. The amount of geotextile is determined by the area between gravel layers and sand and a spacing of 12m is assumed for bollards and fenders.

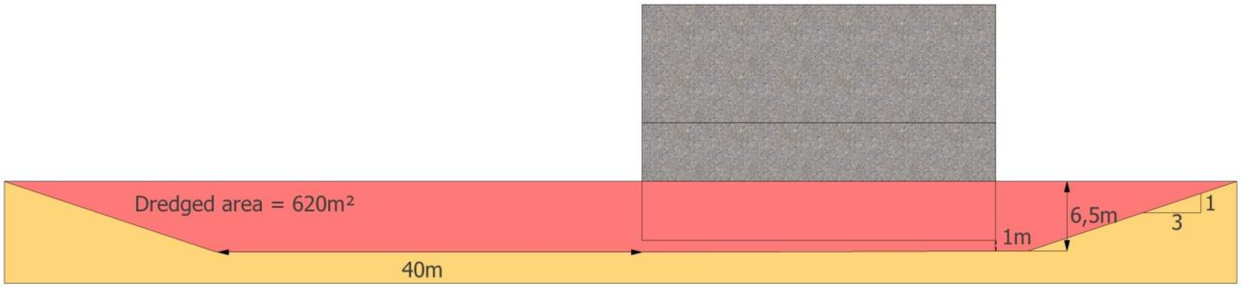


Figure 30.1 – quantity of dredged material per meter

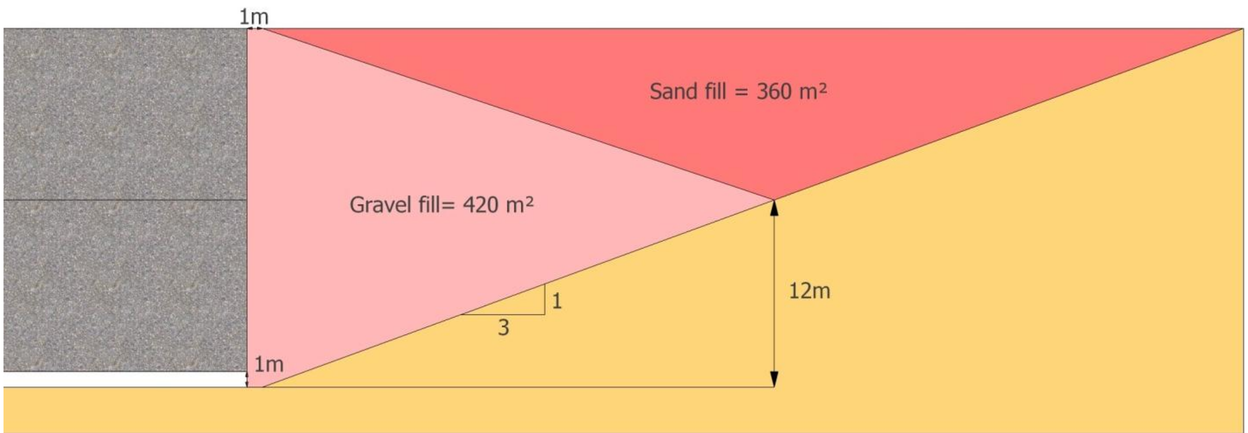


Figure 30.2 – quantity of back fill material per meter

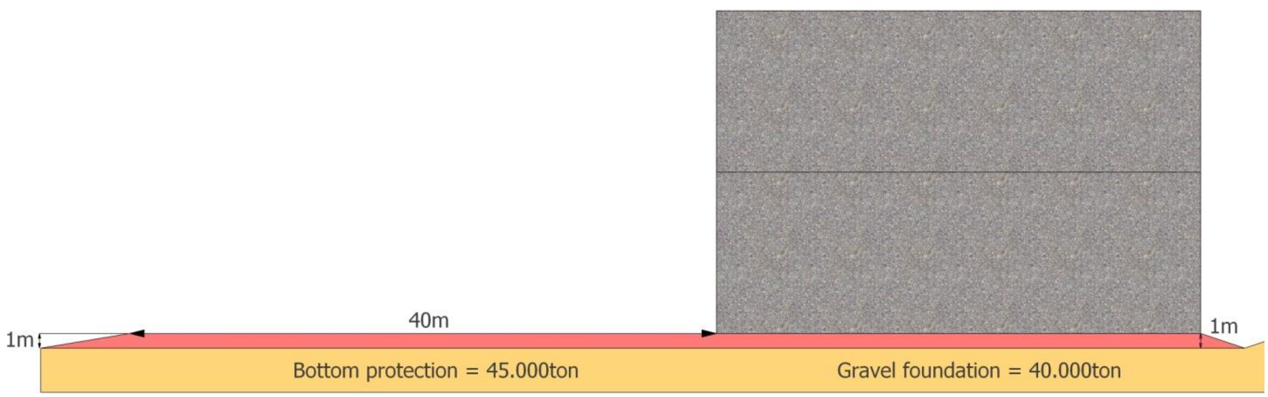


Figure 30.3 – total quantity of bottom protection and gravel bed foundation

30.2. Relocation costs and reconstruction time

Since the flexible quay wall is design for reusability, the relocation costs and the reconstruction time are very important. Together with the residual value, these are the three main parameters that could possibly recoup the initial construction costs in the long term. Determination of the relocation costs is done in a comparable way as the initial construction costs. The most relevant proceedings for relocation are listed in table 30.2.

Description	*Quantity*	*Unit price*	*Total price*
Old location			
Mobilizing STS cranes	8 pcs	100.000 €/pcs	€ 800.000
Excavation works	350.000 m^3	5 €/m^3	€ 1.750.000
De-ballasting the caissons	14 pcs	100.000 €/pcs	€ 1.400.000
Tug boat mobilization costs	14 pcs	20.000 €/pcs	€ 280.000
New Location			
Travel distance	3.000 nm	1.000 €/nm	€ 3.000.000
Trench dredging works	435.000 m^3	5 €/m^3	€ 2.175.000
Dredger (de)mobilization costs	1 pcs	800.000 €/pcs	€ 800.000
Immersing caissons	14 pcs	100.000 €/pcs	€ 1.400.000
Back fill (sand)	250.000 m^3	5 €/m^3	€ 1.250.000
Back fill (gravel)	300.000 m^3	15 €/m^3	€ 4.500.000
Bottom protection	45.000 ton	25 €/ton	€ 1.125.000
Geotextile filter layer	80.000 m^2	12 €/m^2	€ 960.000
Gravel bed foundation	40.000 ton	20 €/ton	€ 800.000
Relocation costs per meter quay wall	**700 m**	**28.914 €/m**	**€ 20.240.000**

Table 30.2 – overview of relocation costs of flexible quay wall structure

Relocation costs of the caisson quay wall are estimated at €20.240.000 for the entire quay. This price is below the construction costs of a new traditional quay wall structure, so relocation could be interesting. Relocating the flexible structure also takes less time than building a new traditional structure. Based on knowledge and experience within the companies of Royal HaskoningDHV and APM Terminals, the relocation time is estimated at 6 months.

30.3. Residual value

The residual value of the flexible structure is of importance too, but is quite hard to determine. The structure can fully be reused, but determination of the price will be a matter of negotiating between seller and client. Therefore, the residual value is not an input, but an output parameter is this financial feasibility comparison. The required residual value will be determined for different scenarios, to make them feasible. Based on these values, it can be judged whether the required residual value is reasonable or not.

30.4. Utilized lifespan

The utilized lifespan of the flexible quay wall is equal to its technical lifespan, so 50 years. The structure will be transported to a different location during this period, a couple of times. Because the utilized period of a traditional quay wall structure at the same location was chosen between 5 to 15 years, the corresponding relocation frequency of the flexible structure will be between 3 to 9 times, during its entire lifetime.

31. Financial feasibility comparison

First of all, it is important to realize that this financial feasibility comparison is based on a deterministic approach, which means that fixed values and chances are used. The chance at a certain event or scenario and the distribution of prices are of importance too, so a probabilistic approach would give a more reliable result. Also note that the prices have a constant value over time, which makes the absolute values less reliable for the long term. However, the principle of recouping the relatively high initial construction costs, by means of a faster reconstruction period, less reconstruction costs and a higher residual value, is clearly visible in a deterministic approach with constant values as well.

An Excel sheet was established to plot the over-all costs balances in a graph over time. The following aspects are programmed as a variable parameter, because they are different for each scenario and have a large influence on the difference between the over-all costs balance of both structures.

<u>Flexible quay wall structure:</u>

- Initial construction period
- Initial utilized period*
- Reconstruction period
- Travel distance for relocation
- Utilized period after relocation
- Net profit per year (equal for both structures)

<u>Traditional quay wall structure:</u>

- (Re)construction period
- Utilized period
- Net profit per year (equal for both structures)

* = By defining an initial utilized period for the flexible structure, it is possible to obtain the same moment of relocation for both structures, so it can be used as a correction for the different initial construction periods.

The costs of table 30.1 and 30.2 were used for the flexible structure and a reconstruction price of €30.800.000 is used for a traditional quay wall of 700m, including 10% demolition costs. A complete overview of the input parameters can be found in appendix T.

With the Excel sheet, numerous scenarios can be investigated. Using the advantage of such a program, a few interesting scenarios are selected to display in this report.

A distinction is made between two different points of view. The first point of view considers the situation in which the structure is used by the same owner, during the entire lifespan. The other point of view refers to the situation in which the structure is sold to a different owner after a certain period.

31.1. Scenario 1a

The first scenario considers 5 periods of 10 years for both structures. In this scenario, the moment of relocation is equal for both structures, because it is caused by the same event. Possible examples of such an event are mentioned in the introduction of this report, in table 1.1. For the flexible structures, the 10 years include 6 months for relocation and for the traditional structure it includes 1,5 years for (re)construction. An initial construction period of 2,5 years and a relocation distance of 1.000 nautical miles are taken into account for the flexible quay wall.
The net profit per year is determined by the assumption that the initial costs of a traditional quay wall are recouped in 9 years, which results in a net profit of 4 million euros per year, for both structures.
Furthermore, it is assumed that both structures are owned by the same person or company, during the entire period of 50 years.

Figure 31.1 represents the over-all costs balance in time, of both structures. The graphs start at the initial construction price. After the initial construction period, the quay becomes operational and revenues start. At the end of the 10 year period, the flexible quay wall is relocated or a new traditional structure is built at a different location. Therefore, the graphs drop by respectively the relocation costs of a flexible quay and the construction costs of a new traditional one. So, each interval between two peaks in the graph represents a different location. The flexible quay starts to make revenues earlier than the fixed one, since its relocation time is shorter than the construction time of the traditional quay wall.

Looking at figure 31.1, it can be concluded that the flexible quay wall structure becomes more attractive on the long term, whereas the traditional quay wall offers more profit on the short term. However, it nearly equals the costs of the traditional quay wall, after relocating the structure once.
After relocating the structure twice, so at the third location, the flexible quay wall certainly has a higher costs balance than the in situ constructed quay wall. Note that the residual value is higher, so in fact, it could already be more favorable after the first relocation.

To make the flexible quay wall structure financially feasible over a period of 50 years, it must at least be relocated twice, as illustrated in figure 31.1. Relocating the structure more often will make it more profitable, compared to a structure that is constructed in situ. The other way around, a flexible structure is likely to be financially unfeasible for ports with a very high certainty of their future continuity and forecasts. This is because it is probably not required to relocate the structure, in that situation.

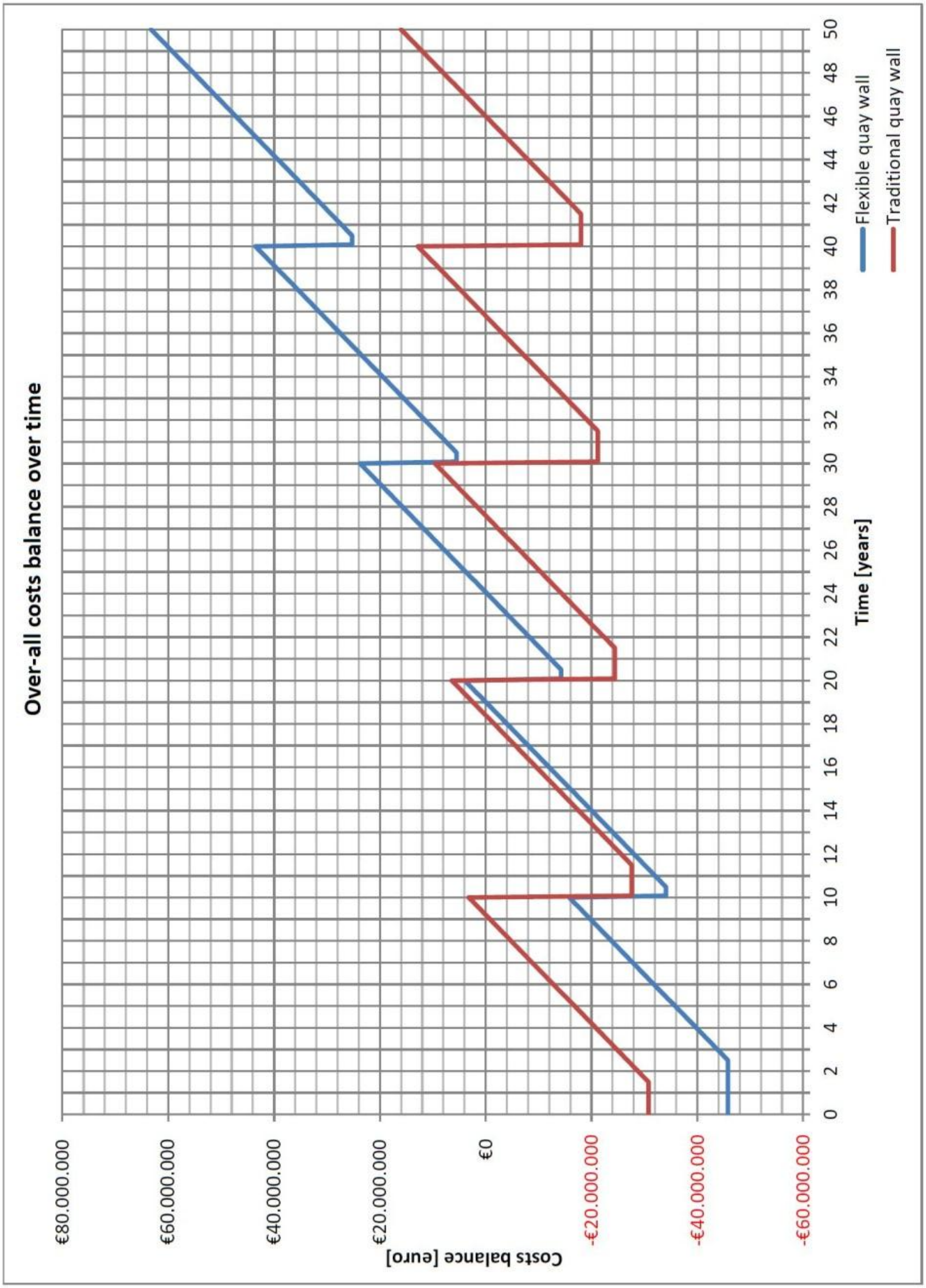


Figure 31.1 – financial feasibility comparison for scenario 1a

31.2. Scenario 1b

Scenario 1b is based on scenario 1a. The only difference is a transport distance of 5.000 nautical miles, instead of 1.000.

Comparing figure 31.1 and 31.2, one can observe a slight increase of the relocation price of the flexible quay wall. However, the impact of a larger travel distance doesn't have a large influence on the total relocation costs.

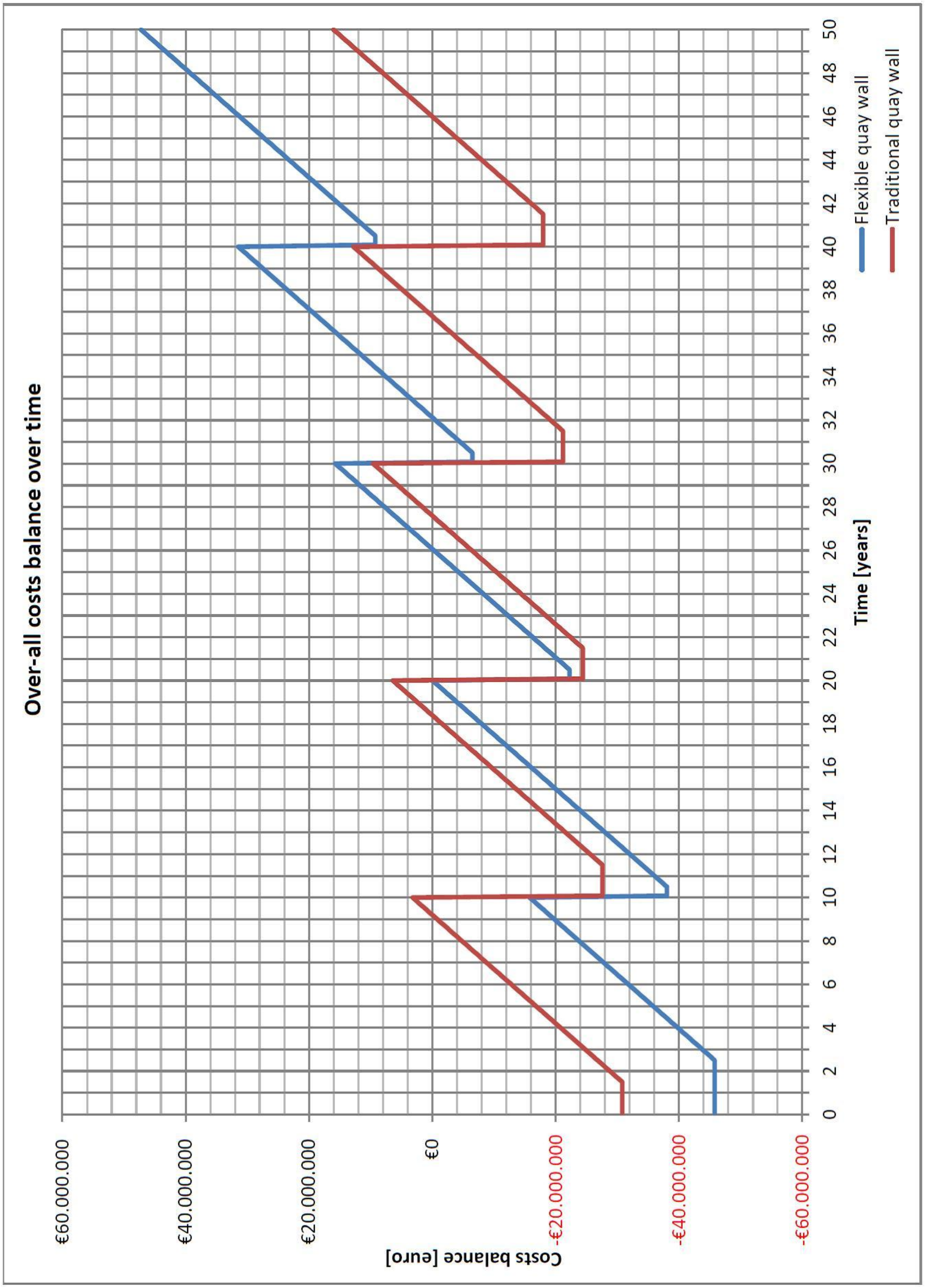


Figure 31.2 – financial feasibility comparison for scenario 1b

31.3. Scenario 2a

In scenario 2, the flexible quay wall structure is sold after a period of 13 years, including 2,5 years of construction. This means that this scenario is approached from a different point of view, compared to scenario 1.

This scenario is meant to investigate if it would be financially feasible to invest in a flexible structure, use it for 10,5 years, and then sell it to someone else. After the structure is sold, a fixed quay wall with 26m retaining height is constructed in situ to handle larger container vessels.
This situation is compared with first constructing a 22m fixed quay wall in situ, and upgrading it to a retaining height of 26m after a period of 13 years.
Since one specific location is considered, the graphs stop after the upgrade.

Since 2013 prices are used over the entire period, the construction costs of a 26m in situ constructed quay wall are estimated at €33.250.000. Consequently, upgrading a 22m fixed quay wall to a 26m fixed quay wall costs about €23.275.000, which is 70% of the construction costs of a completely new quay wall.

Looking at figure 31.3, the difference between both lines is about €28.970.000, after 13 years. This would mean that the flexible quay wall structure should be sold for at least this price to make if financially feasible for this scenario. The required residual value of €28.970.000, is slightly more than the construction costs of a new in situ constructed quay wall of 22m, which costs €28.000.000.

Although the price is 3,5% higher than a new traditional structure, buying the second hand quay wall would be more favorable for the new owner, since the construction time is shorter. The influence of a short reconstruction time on the costs balance can clearly be observed in figure 31.1 and 31.2. Also realize that the new owner could sell the flexible quay wall to a third owner again, which is more difficult for a traditional quay wall.

31.4. Scenario 2b

This scenario is basically the same as scenario 2a, but the first owner makes a different decision.

When the first owner decides to stop business and just sell the flexible quay wall structure, the difference between having used a traditional structure is €18.995.000. This price is 32% less than the price of a new in situ constructed quay wall, so this scenario is likely to be financially feasible. For scenario 2b, the difference between the peaks in figure 31.3 is used, whereas the difference between the troughs was used for scenario 2a.

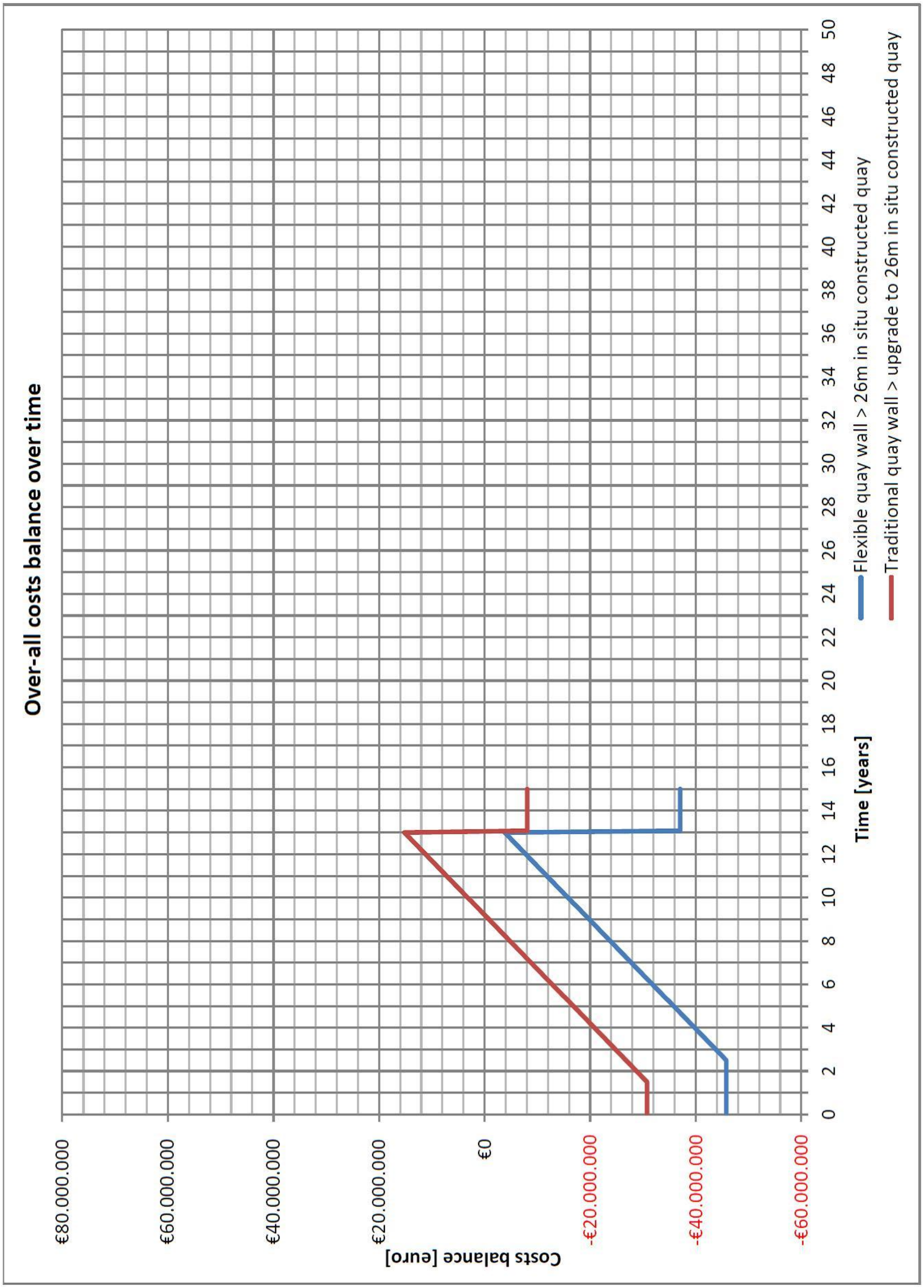


Figure 31.3 – financial feasibility comparison for scenario 2a and 2b

32. Conclusions regarding financial feasibility comparison

From this financial feasibility comparison, it can be concluded that the flexible quay wall structure could be a financially feasible alternative to traditional quay wall structures that are constructed in situ.

The initial construction costs of the flexible quay wall structure are estimated at about 65.421€/m, which is 64% more expensive than the average price of a traditional quay wall structure. However, the relocation costs are estimated at €20.240.000 for the entire quay wall, which is 28% less expensive than the construction costs of a traditional quay wall. Besides, the reconstruction period was estimated at 6 months, which is about one year faster than the construction of a traditional quay wall, but the initial construction period is about one year longer.

When the structure is used by the same owner, during its entire lifespan of 50 years, it should be relocated twice, in order to make it financially more attractive than building three new traditional quay walls in the period of 50 years.
A large travel distance has some influence on the reconstruction costs, but the quay still needs to be relocated at least once, to make it feasible.

When the structure is sold after a certain period, it could possibly already be sold after the first utilized period. The required residual value to make this more attractive than upgrading a traditional quay is 3,5% higher than the price of a new traditional quay wall. However, the fast reconstruction period and the possibility to sell it to a third user again, could make it more attractive to buy the second hand flexible quay wall.
When the first owner decides to sell the flexible quay and leave the market, the required residual value is about 32% lower than the construction costs of a new in situ constructed quay wall, so this scenario is likely to feasible too.

Again note that this financial feasibility comparison is based on a deterministic approach with constant values in time. A detailed probabilistic approach, combined with variable values in time is recommended to obtain a more reliable result of the absolute values in the long term of figure 31.1 to 31.3.

Part VI
Conclusions & Recommendations

33. Conclusions

Based on this master thesis on flexible quay wall structures for container vessels, the following main conclusions can be drawn:

- First of all, it can be concluded that flexibility of traditional quay walls is very low and that a flexible quay wall structure could be desirable. Reusability and a faster (re)construction time are both valuable for quay wall structures.

- It can be concluded that stability of a floating quay wall for container vessels cannot be guaranteed. The tilt angle and corresponding vertical displacements caused by container lifting operations and the natural oscillation period of roll motions are the most important factors. Motions are very likely to exceed the PIANC guidelines for container lifting activities.
- Stabilizing a floating quay wall by means of counterweights or ballast tanks is very difficult, because the tilting moment caused by container lifting appears and disappears very fast.
- Stabilizing a floating quay wall by pulling it partly under water results in desirable properties, but the corresponding anchor forces on the sea bed are massive.
- A jack-up system could be used to stabilize a floating structure, but a heavy steel structure is required to resist the large bending moments in the deck, caused by the container cranes and the span between the jack-up legs.
- Reusability of a deck on piles is very doubtful, because many piles are required and reusing them is likely to be more expensive than using new piles. Besides, driving hundreds of piles into the subsoil is time consuming and has a negative influence on the (re)construction time of the quay.
- Reusing a mass concrete block-work wall is also difficult, because the reinforced upper layer is casted on site and cements the under lying blocks together. Elements become very heavy for large retaining heights and reusing them requires heavy hoisting equipment and is time consuming.
- Having investigated the poor stability of a floating quay wall, a caisson quay wall seems to be the most appropriate flexible quay wall structure for panamax container vessels.

- A final quay wall design with two caissons on top of each other was selected to be the most suitable flexible quay wall structure. The caissons are 100m long, 33m wide and 11m high. The width of the STS crane rails and the desirable width-length-ratio of 1:3 for good navigability are the most important factors that played a role in the determination of the dimensions. Each caisson has 12 inner walls in transversal direction and 4 in longitudinal direction.
- Filling both caissons with ballast water only, provides insufficient downward pressure to satisfy the stability criteria. The lower caissons are filled with water only, which makes it relatively easy to pump the ballast out again. The upper caissons are filled with sand, so a liquefaction pump must be used to pump the ballast material out. Since the upper caissons remains well above the water level, this should not be a problem.

- Relocation of the quay wall is done by digging the retaining soil free from the caissons and pumping the ballast material out. The caissons start to float again and will be transported by tug boats.
- The final unity check values for satisfaction of the failure mechanisms are: 0,95 for both sliding and bearing capacity and 0,77 for overturning. The empty caissons have a metacenter height of 15,57m in floating conditions and 14,87 during immersion. The minor difference is a result of the large number of inner walls. The natural floating oscillation period for roll motions was determined at 5,58 seconds and the structure requires a minimum bearing capacity of 400kN/m^2 under the bottom slab. In the worst case scenario, it can resist an earthquake acceleration of at least 0,2g, but more ballast can be applied to achieve a higher earthquake resistance.

- Strength calculations proved that an uneconomic amount of concrete is required to avoid shear reinforcement.
- Optimizing the wall thickness with respect to the price of required construction materials leads to a significant price reduction. The optimum wall thickness was determined by plotting the costs of concrete and reinforcement steel in a graph. This was done for a wall thickness between 300mm to 900mm for each wall of slab section. The optimum wall thickness of the outer walls and slabs varies between 350mm and 550mm.
- The applied amount of reinforcement steel in the top and bottom slabs of the caissons is able to absorb the bending moment on the entire caisson, caused by wave loads during transport.

- The initial construction costs of the flexible quay wall structure are estimated at about 65.421€/m, which is 64% more expansive than the average price of a traditional quay wall structure.
- The relocation costs are estimated at €20.240.000 for the entire quay wall, which is 28% less expensive than the construction costs of a traditional quay wall. The transport distance has some influence, but other costs are governing for relocation.
- The reconstruction period was estimated at 6 months, which is about 12 months faster than the construction of a traditional quay wall, but the initial construction period is about 12 months longer.
- From a financial feasibility comparison, it can be concluded that the flexible quay wall structure could be a financially feasible alternative to traditional quay wall structures that are constructed in situ, for different scenarios.
- When the flexible structure is used by the same owner during its entire lifespan of 50 years, it must at least be relocated twice, in order to make it financially more attractive than using different fixed structures during the same period.
- When the first owner decides to sell the flexible structure, it could already be possible after using it at one single location.
- Using the flexible structure for a certain period, selling it, and building a larger traditional structure could possibly be a feasible alternative to constructing a fixed structure and upgrading it after the same period. The required residual value is 3,5% higher than the construction costs of a new traditional quay wall in that case.

Since the structure can be reconstructed 12 months faster and can also be sold to a third owner again, it might be a financially feasible alternative too.

34. Recommendations

This graduation topic has a very wide range of aspects to be considered, before the flexible quay wall structure for container vessels can be constructed in reality. This chapter summarizes the most important parts, with respect to this report, that were beyond the scope of this master thesis or may need further research or elaboration.

- The feasibility study in this report is just a brief investigation of the financial feasibility of the flexible quay wall structure compared to a certain traditional quay wall structure. The values and costs in this feasibility study could be optimized and can be divided into several subcategories. The feasibility study can also be seen as an opportunity for an entire new thesis, which focusses on costs and finance only. Using a probabilistic approach instead of a deterministic one, will be a good improvement. Converting prices and revenues as a function in time will also lead to more reliable values.
- A pre-tensioned floating quay wall was one of the alternatives proposed in part III of the report, but the resulting anchor forces turned out to be very large. Because the properties of such a structure were quite promising, one could try to find a solution to resist the massive anchor forces. For instance by applying suction anchor, which are common in offshore engineering. However, even for suction anchors, the forces are very high. Making the caissons smaller is an option, but this will increase the number of caissons to be transported and the number of anchors to be installed.
- Looking at figure 27.2, one could consider using this caisson configuration and make the width and height of the caissons somewhat different. When the width and height are say, 13m and 11m, one could possibly create a flexible retaining height between 22 and 26 meters. By turning both caissons, or a single caisson 90 degrees, the retaining height can be different at the new location. But, keep in mind that many more caissons need to be transported, because of the reduced caisson dimensions. It might be interesting to study the possibility of transporting the quay wall elements on barges, so many elements can be transported at a time.
- It could also be interesting to design the caisson quay wall in such a way, that it doesn't retain the soil. Connection bridges must be applied between the caissons and the terminal area. The caissons can be placed on a gravel bed foundation, which is connected to the terminal area with a protected slope. An advantage is the reduced horizontal load on the structure and a stabile terminal area after removal of the quay. Disadvantage is an increased caisson width, because of required traffic lines for container trucks on the caissons. Also keep in mind that the slope of the shoreline must be protected with rubble material and filter layers to avoid erosion and that the connection bridges must be around 50m long.
- Using caissons of 22m in height instead of using two caissons of 11m could be considered for the construction of a flexible quay wall. The draught and drag force will increase, but the number of caissons to be transported is halved. Also note that the hydrostatic forces increase significantly, because of the doubled height.

- One could consider using a steel bow that can be attached to the caissons during transport, in order to reduce the drag force and to safe fuel.
- The proposed number and type of container handling equipment can be optimized by applying a computer simulation program, instead of the queuing theory only.
- A more extensive market demand analysis could be carried out to investigate the demand for such a structure and possible investors in it.
- The structural design of the flexible quay wall could be extended by taken into account: a crack width calculation, normal stresses, temperature differences and ice loads. Calculations of an increased static load were made for wave loads during transport, but dynamic computations may be needed to determine these loads more carefully. One can also consider pre-stressed concrete instead of reinforced concrete and a detailed execution plan could be made for construction of the flexible quay wall.

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List of abbreviations

AGV	=	Automatic Guided Vehicle
FEU	=	Forty Feet Equivalent Unit
HHWS	=	High High Water Spring
LLWS	=	Low Low Water Spring
MSc	=	Master of Science
MSL	=	Mean Sea Level
MTS	=	Multi Trailer System
PGA	=	Peak Ground Acceleration
RMG	=	Rail Mounted Gantry crane
RTG	=	Rubber Tired Gantry crane
SLS	=	Serviceability Limit State
STS	=	Ship-to-shore crane
SWOT	=	Strength, Weakness, Opportunities and Threats
TEU	=	Twenty feet Equivalent Units
ULS	=	Ultimate Limit State

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List of tables

List of equations

List of symbols

$A_{bottom\ slab}$	=	surface of bottom slab caisson
A_c	=	area of concrete cross section
A_{cross}	=	area of cross section caisson
$A_{element;i}$	=	area cross section element i
$a_{equipement}$	=	acceleration of the equipment
A_{pile}	=	surface of pile tip
A_s	=	required amount of reinforcement steel in cross section
A_{sl}	=	applied reinforcement area for bending
A_{sw}	=	shear reinforcement area
$A_{sw;max}$	=	maximum applicable reinforcement area
A_{tot}	=	total surface perpendicular to the current
A_{vessel}	=	surface of vessel perpendicular to wind direction
$B_{caisson}$	=	width of the caisson
$\overline{BM}$	=	distance between center of buoyancy and metacenter point
b_w	=	per unit width
C_b	=	berth production
C_C	=	berth configuration coefficient
C_D	=	drag force coefficient
C_E	=	eccentricity coefficient
c_{fr}	=	friction coefficient
C_H	=	added mass coefficient
C_S	=	stiffness coefficient
C_s	=	annual throughput
d	=	water depth between surface and point d
d_{10}	=	d_{10} of layer
d_{15B}	=	d_{15} of above lying layer
d_{15F}	=	d_{15} of filter layer
d_{60}	=	d_{60} of layer
d_{85B}	=	d_{85} of above lying layer
$D_{caisson}$	=	draught of the caisson
d_{con}	=	effective height of concrete cross section
d_{eff}	=	effective propeller diameter
d_{n50}	=	nominal stone diameter
d_{pile}	=	diameter or width of the pile
d_{slab}	=	slab thickness
E	=	elasticity modulus
E(f)	=	wave spectrum

E_{kin}	=	kinetic berthing energy
f	=	TEU-factor
$F_{abutment}$	=	force exerted on the abutment
$F_{bear;shaft(z)}$	=	pile shaft bearing capacity
$F_{bear;tip}$	=	pile tip bearing capacity
$F_{buoyancy}$	=	upward force generated by displaced water
f_{cd}	=	design value of concrete tensile strength
f_{ck}	=	characteristic compressive concrete strength
$F_{diagram;i}$	=	resulting horizontal force of pressure diagram i
$F_{equipment}$	=	force due to moving equipment
F_{hor}	=	acting horizontal force
$F_{hor;max}$	=	maximum allowable horizontal force
$F_{hor;soil}$	=	maximum absorbable horizontal force by the soil
$f_{per\ year}$	=	frequency of occurrence of extreme event
$F_{shaft;neg}$	=	negative pile shaft friction
F_{tug}	=	force generated by sailing velocity of tug boat
F_{vert}	=	acting vertical force
$F_{vertical;structure}$	=	downward vertical force by the structure
$F_{w;hor}$	=	resulting horizontal force
F_{wind}	=	force generated by the wind
$F_{wind;hor}$	=	horizontal component wind force
$F_{wind;vert}$	=	vertical component wind force
f_{yk}	=	steel yield stress
f_{ywd}	=	yield stress of shear reinforcement steel
g	=	gravitational acceleration
$\overline{GG}^2_{element;i}$	=	distance between total center of gravity and center of gravity of element i
h	=	water depth
h_0	=	increase of middle water level
H	=	wave height
$H_{caisson}$	=	height of the caisson
H_i	=	incoming wave height
h_{layer}	=	layer thickness
h_m	=	metacenter height of the floating caisson
H_{mo}	=	significant wave height of a wave spectrum
h_{pile}	=	height of the pile above the soil
H_s	=	significant wave height
h_{soil}	=	thickness of the soil layer
I	=	moment of inertia of pile cross section
I_{area}	=	moment of inertia of area enclosed by inner walls
$I_{caisson}$	=	moment of inertia of the caisson
$I_{element;i}$	=	polar moment of inertia element i
I_{polar}	=	the total polar moment of inertia
I_{tot}	=	total polar moment of inertia
$I_{xx;hollow}$	=	moment of inertia hollow area around x-axis
$I_{xx;polar}$	=	area moment of inertia around x-axis

$I_{zz;hollow}$	=	moment of inertia hollow area around z-axis
$I_{zz;polar}$	=	area moment of inertia around z-axis
j	=	polar inertia radius
k	=	wave number
$\overline{KG}$	=	vertical distance between bottom of bottom slab to center of gravity caisson
$\overline{KG}_{element;i}$	=	vertical distance between bottom of bottom slab to center of gravity element i.
k_{hor}	=	horizontal soil coefficient
k_{pile}	=	spring stiffness of one single pile
$K_{o;rep}$	=	neutral soil coefficient
K_p	=	passive soil coefficient
L	=	wave length
L_0	=	deep water wave length
$L_{caisson}$	=	length of the caisson
L_{quay}	=	length of the quay
L_{vessel}	=	length of the design vessel
m_0	=	zero order moment of wave spectrum
m_b	=	first estimate utilization rate of the quay
$m_{caisson}$	=	mass of the caisson
$m_{concrete}$	=	mass of concrete
M_{Ed}	=	bending moment in cross section
$m_{element;i}$	=	mass of element i
$m_{equipment}$	=	mass of the equipment moved
$M_{max;hor}$	=	maximum momentum as a result of horizontal force
M_{tilt}	=	tilting moment at caisson
M_{wave}	=	momentum caused by wave load during transport
m_{vessel}	=	mass of the vessel
N	=	number of berths at the quay
n	=	number of piles
N_b	=	average number of cranes in use during vessel operation
N_{Ed}	=	normal force at concrete cross section
O_s	=	circumference of the pile shaft
O_{tip}	=	circumference of pile tip
p	=	crane production
p_0	=	pressure at depth d
p_1	=	pressure at mean water level
P_f	=	probability of failure within lifetime
$p_{soil; hor; dry}$	=	horizontal soil pressure of dry soil
$p_{soil; hor; wet}$	=	horizontal pressure caused by wet soil
$p_{soil; vert; wet}$	=	vertical wet soil pressure
P_{vessel}	=	vessel's propeller power
p_w	=	water pressure
q	=	mean overtopping discharge
q_c	=	cone resistance
$q_{c;I;avg}$	=	average cone resistance section I

$q_{c;I;avg}$	=	average cone resistance section II
$q_{c;I;avg}$	=	average cone resistance section III
R_c	=	freeboard of the vertical wall
$\overline{RC}_{diagram;i}$	=	distance between horizontal force of diagram i and rotation center
$\overline{RC}_{total}$	=	distance between rotation center and joined attachment point of all horizontal components together
s	=	shape factor for the cross section of the pile
s_{bar}	=	center to center distance of shear reinforcement bars
T	=	wave period
t	=	pile driven depth
T_0	=	natural oscillation period
t_o	=	depth where the moment of the ideal load is zero
T_{life}	=	lifetime of the structure
t_n	=	operational hours per year
$t_{w;h}$	=	wall thickness head walls
$t_{w;s}$	=	wall thickness side walls
u_0	=	flow velocity behind the propeller
u_{b-max}	=	flow velocity at the bed
v_1	=	strength reduction factor for shear in concrete
v_{berth}	=	berthing velocity perpendicular to the quay wall
$V_{concrete}$	=	volume of concrete structure
$v_{current}$	=	current velocity
$V_{Rd;c}$	=	absorbable shear force by reinforced concrete
$V_{Rd;max}$	=	maximum possible shear force for slab thickness
$V_{Rd;s}$	=	absorbable shear force by shear reinforcement
v_{wind}	=	wind velocity at 10 meter elevation
X_m	=	depth below height above the soil, where the bending moment is at its maximum
z	=	distance between center of top and bottom slab
z_b	=	distance between bottom and propeller axis
z_{slab}	=	lever arm of concrete slab
α	=	angle of mooring lines with the horizontal plane
α_{bottom}	=	under water slope of the bottom protection
α_{cw}	=	factor for stress in pressurized edges
α_p	=	pile class factor
α_S	=	factor depending on method of realization
α_{steel}	=	angle of shear reinforcement
β	=	shape factor for foot of the pile
γ'	=	under water weight of the soil
γ_C	=	partial safety factor for concrete
Δ	=	relative under water density of stone material
Δx_{hor}	=	horizontal distance to rotation center
Δx_{vert}	=	vertical displacement
δ	=	horizontal displacement of the pile head
δ_{rep}	=	angle of external friction between soil and pile

δ_{soil}	=	angle of external friction
φ	=	angle of internal friction
θ	=	tilting angle of caisson
θ_{strut}	=	angle of struts
ρ_1	=	ratio of reinforcement steel in concrete cross section
ρ_{air}	=	density of air
$\rho_{concrete}$	=	density of concrete
ρ_{min}	=	minimum required reinforcement percentage to absorb bending moment caused by wave load
$\rho_{soil;dry}$	=	density of dry soil
ρ_w	=	density of water
$\sigma'_{v;avg}$	=	average effective vertical soil pressure
$\sigma_{bear;req}$	=	required bearing capacity of the soil
σ_{cp}	=	normal compressive stress in cross section
$\sum M$	=	sum of all momentums on the caisson
$\sum V$	=	sum of all vertical forces on the caisson
ϑ_a	=	spread angle of surface load to reach maximum value

Appendix

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Use of colors in Excel sheets

(grey)	= constant values
(green)	= input value
(orange)	= output value
(light blue)	= obtained from table
(yellow)	= solved iteratively
(blue)	= satisfaction of failure mechanism

Note that not all input parameters in the Excel sheets correspond with the final design. This is because some Excel sheets were used to make calculations on different quay wall designs, described in part III of the thesis.

Appendix A: Queuing Theory – Excel sheet

Quay logistics and dimensions		
Annual throughput	1.000.000	[TEU/year]
Average crane productivity	30	[moves/hour]
Number of cranes per berth	4	[-]
Operational hours per year	8.400	[-]
TEU-factor	1,6	[-]
vessel length	294	[m]
Average call size	1.500	[TEU]
Time for berthing and departure	2	[hour]
Average productivity truck	8	[moves/hour]
Average productivity RTG	16	[moves/hour]
First estimate of quay utilization	35	[%]
Rule of thumb for number of berths	1,77	[-]
Chosen number of berths	2	[-]
Number of trucks per STS	6	[-]
Number of trucks per gang	12	[-]
Number of RTG per gang	7	[-]
Number of gangs per berth	2	[-]
Required quay length	695	[m]
Handling time per vessel	9,81	[hours]
Number of calls	667	[calls/year]
Utilization rate of quay	**31**	[%]
Number of service points (M/E2/n)	**2**	[-]
Utilization rate of trucks	**63**	[%]
Number of service points (E2/E2/n)	**6**	[-]
Utilization rate of RTG	**54**	[%]
Number of service points (E2/E2/n)	**7**	[-]

	Waiting time in units service time	Service time	Waiting time	
Average waiting time vessel for berth	0,0830	9,81	**49**	minutes
Average waiting time STS for truck	0,0399	2,00	**5**	seconds
Average waiting time truck for RTG	0,0240	7,50	**11**	seconds

Appendix B: Table $M/E_2/n$ average waiting times queuing theory

Average waiting time of ships in the queue $M/E_2/n$
(In units of average service time)

utilisation (u)	number of servers (n) 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.10	0.08	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.15	0.13	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.20	0.19	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25	0.25	0.05	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.30	0.32	0.08	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.35	0.40	0.11	0.04	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.40	0.50	0.15	0.06	0.03	0.02	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.45	0.60	0.20	0.08	0.05	0.03	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.50	0.75	0.26	0.12	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00
0.55	0.91	0.33	0.16	0.10	0.06	0.04	0.03	0.02	0.02	0.01	0.01	0.01	0.00	0.00	0.00
0.60	1.13	0.43	0.23	0.14	0.09	0.06	0.05	0.03	0.03	0.02	0.02	0.01	0.01	0.01	
0.65	1.38	0.55	0.30	0.19	0.12	0.09	0.07	0.05	0.04	0.03	0.03	0.02	0.02	0.02	
0.70	1.75	0.73	0.42	0.27	0.19	0.14	0.11	0.09	0.07	0.06	0.05	0.04	0.03	0.03	
0.75	2.22	0.96	0.59	0.39	0.28	0.21	0.17	0.14	0.12	0.10	0.08	0.07	0.06	0.05	
0.80	3.00	1.34	0.82	0.57	0.42	0.33	0.27	0.22	0.18	0.16	0.13	0.11	0.10	0.09	
0.85	4.50	2.00	1.34	0.90	0.70	0.54	0.46	0.39	0.34	0.30	0.26	0.23	0.20	0.18	
0.90	6.75	3.14	2.01	1.45	1.12	0.91	0.76	0.65	0.56	0.50	0.45	0.40	0.36	0.33	

Appendix C: Table $E_2/E_2/n$ average waiting times queuing theory

Average waiting time of ships in the queue $E_2/E_2/n$
(In units of average service time)

utilisation (u)	number of servers (n) 1	2	3	4	5	6	7	8	9	10
0.1	0.0166	0.0006	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.2	0.0604	0.0065	0.0011	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.3	0.1310	0.0235	0.0062	0.0019	0.0007	0.0002	0.0001	0.0000	0.0000	0.0000
0.4	0.2355	0.0576	0.0205	0.0085	0.0039	0.0019	0.0009	0.0005	0.0003	0.0001
0.5	0.3904	0.1181	0.0512	0.0532	0.0142	0.0082	0.0050	0.0031	0.0020	0.0013
0.6	0.6306	0.2222	0.1103	0.0639	0.0400	0.0265	0.0182	0.0128	0.0093	0.0069
0.7	1.0391	0.4125	0.2275	0.1441	0.0988	0.0712	0.0532	0.0407	0.0319	0.0258
0.8	1.8653	0.8300	0.4600	0.3300	0.2300	0.1900	0.1400	0.1200	0.0900	0.0900
0.9	4.3590	2.0000	1.2000	0.9200	0.6500	0.5700	0.4400	0.4000	0.3200	0.3000

Appendix D: Forces and displacements of a pre-tensioned floating body – Excel sheet

Pre-tensioned floating body		
Caisson width	45	[m]
Caisson length	120	[m]
Extra height pulled under water	**0,7**	[m]
Horizontal force on structure	8.000	[kN]
Water density	1030	[kg/m3]
Gravitational acceleration	9,81	[m/s2]
Cable length	10	[m]
Number of cables per caisson	**4**	[-]
Extra buoyancy force	38.194	[kN]
Angle of cables with vertical plane	**9,825**	[deg]
Horizontal force equilibrium (must be zero!!!)	0	[kN]
Horizontal displacement of quay	1,71	[m]
Additional draught due to displacement	146,66	[mm]
Additional buoyancy force due to displacement	8.002	[kN]
Resulting force in cable	**11.721**	[kN]
Horizontal component cable force	2.000	[kN]
Vertical component cable force	11.549	[kN]

Tensile strength of steel	235	[N/mm2]
Minimum required cross sectional area	0,0499	[m2]

Appendix E: Bottom protection against scour holes – Excel sheet

Propeller wash and bottom protection		
Propeller diameter	5	[m]
Vessel power	40.000	[kW]
Power used for berthing / departure	15	[%]
Distance between propeller axis and bottom	3,50	[m]
Bottom slope	0	[deg]
Angle of internet friction rubble material	35	[deg]
Flow velocity behind propeller	8,08	[m/s]
Flow velocity near the bottom	2,42	[m/s]
Required Dn50	**0,45**	[m]

Appendix F: Freeboard for overtopping – Excel sheet

Overtopping & Freeboard		
Quay's freeboard	**3,50**	[m]
Wave height near quay	1,08	[m]
Overtopping discharge	0,40	[l/m/s]

Appendix G: Quick and dirty estimate of reinforced concrete beam and spacing between foundation piles – Excel sheet

Reinforcement of concrete beam, quick & dirty		
Height of beam	9000	[mm]
Width of beam	700	[mm]
Concrete cover	50	[mm]
Reinforcement diameter	**50**	[mm]
Number of reinforcement bars	**5**	[-]
Compressive strength concrete	25	[N/mm2]
Yield stress reinforcement	435	[N/mm2]
Length of span between piles	**25**	[m]
Point load of crane (in middle of beam)	1500	[kN]
Freeboard of quay	4,5	[m]
Safety factor permanent loads	1,1	[-]
Safety factor variable loads	1,3	[-]
Safety factor beneficial loads	0,9	[-]
Cross section area reinforcement	9.817	[mm2]
Effective height	8.925	[mm]
Reinforcement percentage	0,157	[%]
Spacing between reinforcement bars	87,5	[mm]
Maximum absorbable bending moment	37.573	[kNm]
Maximum occurring bending moment	35.673	[kNm]

Appendix H: Static and dynamic stability of a floating and an immersing caisson – Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of concrete	2550	[kg/m3]
Density of water	1030	[kg/m3]

Dimensions caisson		
Height	11,00	[m]
Width	33,00	[m]
Length (> width)	100,00	[m]
Number of inner walls in transversal direction	12	[-]
Number of inner walls in longitudinal direction	4	[-]
Thickness bottom slab	0,50	[m]
Thickness deck	0,38	[m]
Thickness side wall transversal direction	0,55	[m]
Thickness side wall longitudinal direction	0,51	[m]
Thickness inner walls in transversal direction	0,30	[m]
Thickness inner walls in longitudinal direction	0,30	[m]
Number of caissons used in entire quay structure	14	[-]

Static stability in floating conditions		
Total volume of caisson	36.300	[m3]
Solid volume of caisson	6.605	[m3]
Empty volume in caisson	29.695	[m3]
Bottom slab surface empty volume	2.933	[m2]
Total weight of one caisson	16.842	[ton]
Draught	4,96	[m]
Total vertical center of gravity with respect to bottom of bottom slab	5,22	[m]
Horizontal eccentricity of center of gravity in transversal direction	0,00	[m]
Height of metacentre point	20,79	[m]
Metacentric height	**15,57**	[m]
Dynamic stability in floating conditions		
Polar moment of inertia around x-axis	5.359	[m4]
Polar moment of inertia around x-axis at inner walls in transversal direction	32.942	[m4]
Polar moment of inertia around z-axis	895	[m4]
Polar moment of inertia around z-axis at inner walls in transversal direction	3.660	[m4]
Concrete area cross section at location of inner wall in transversal direction	363	[m2]
Concrete area cross section	51	[m2]
Total averaged polar moment of inertia	8.132	[m4]

Averaged polar inertia radius	10,98	[m]
Natural oscillation period caisson	**5,58**	[s]
Static stability during immersion		
Height of ballast inside caisson	0,10	[m]
Draught including ballast water	5,04	[m]
Vertical center of gravity during immersion	5,14	[m]
Height of metacentre point during immersion	20,00	[m]
Metacenter height during immersion	**14,87**	[m]

Appendix I: Tilt angle resulting from tilting moment on a floating body – Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of concrete	2550	[kg/m3]
Density of water	1030	[kg/m3]
Dimensions caisson		
Height	9,00	[m]
Width	45,00	[m]
Length (> width)	120,00	[m]
Number of inner walls in transversal direction	12	[-]
Number of inner walls in longitudinal direction	3	[-]
Thickness bottom slab	0,90	[m]
Thickness deck	0,60	[m]
Thickness side wall transversal direction	0,70	[m]
Thickness side wall longitudinal direction	0,70	[m]
Thickness inner walls in transversal direction	0,40	[m]
Thickness inner walls in longitudinal direction	0,40	[m]
Number of caisson used in entire quay structure	6	[-]
Additional weight of equipment		
Additional weight of one STS crane per caisson	800	[ton]
Vertical center of gravity additional weight with respect to deck of quay	30,00	[m]
Horizontal eccentricity of center of gravity crane weight in cross direction	8,00	[m]
Length of crane boom	37,00	[m]
Number of ship-to-shore cranes	8	[-]
Tilt angle due to crane's dead weight (non-ballasted caissons)		
Total volume of one caisson	48.600	[m3]

Solid volume of one caisson	12.412	[m3]
Empty volume in one caisson	36.188	[m3]
Bottom slab surface empty volume of one caisson	4.825	[m2]
Total weight including additional load of one caisson	32.450	[ton]
Average draught including additional load	5,83	[m]
Total vertical center of gravity with respect to bottom of bottom slab	4,92	[m]
Horizontal eccentricity of center of gravity in transversal direction	0,20	[m]
Height of metacentre point	31,84	[m]
Metacentric height	26,92	[m]
Tilting moment at entire quay caused by cranes dead weight	502.272	[kNm]
Calculated tilting moment with tilt angle (solve iteratively)	502.272	[kNm]
Tilt angle (solve iteratively)	**0,559738**	[deg]
Vertical displacement of quay corner in transversal direction	0,22	[m]
Vertical displacement of crane boom tip by cranes dead weight	**0,58**	[m]
Container lifting operations		
Maximum hoisting weight	50	[ton]
Maximum vertical acceleration	0,5	[m/s2]
Number of lifting container cranes	8	[-]
Tilt angle by container lifting (ballasted for crane's dead weight)		
Tilting moment caused by container lifting	245.378	[kNm]
Calculated tilting moment with tilt angle (solve iteratively)	245.378	[kNm]
Tilt angle (solve iteratively)	**1,640380**	[deg]
Vertical displacement of quay corner in transversal direction	0,64	[m]
Vertical displacement of crane boom tip by container lifting only	**1,70**	[m]

Appendix J: Foundation piles

During this master thesis it is carefully checked whether a pile foundation is an attractive solution or not. Since the final design does not contain foundation piles, the calculation method is displayed in this appendix. Appendix J ends with the Excel sheet of the calculation, but the methods and equations used in this sheets are described in appendix J.1. to J.10.

J.1. Number of piles

Whether a pile foundation is a feasible solution or not, mainly depends on the number of piles that is needed. When it appears that many piles have to be driven into the ground, this will reduce the flexibility of the quay wall structure. Although the quay wall itself can be transported as a floating structure, the piles have to be pulled out of the soil and driven back into it at the new location. Or if this turns out to be unfeasible, new piles must be driven at the new location.

The number of piles is also related to the required strength of the deck. Decreasing the number of piles will increase the distance between the piles. The increased span will result in higher bending moments caused by the container cranes and the weight of the deck itself.

The total number of piles that is needed to stabilize the quay wall depends on many criteria that are listed in the next paragraphs of this appendix.

J.2. Connection between deck and piles

In order not to hamper the flexibility of the structure, there is no connection between the piles and the immersed deck of the quay. This is only possible if the vertical downward force is large enough to avoid horizontal sliding due to the maximum horizontal force. In other words: the vertical force times the friction coefficient between the piles and the deck must be larger than the horizontal force, in any case.

$$F_{hor;max} \leq F_{vert} \cdot c_{fr} \cdot n \qquad \textit{Equation J.1}$$

Where: $F_{hor;max}$ = maximum allowable horizontal force on structure [kN]
F_{vert} = net vertical downward force [kN]
c_{fr} = friction coefficient [-]
n = number of piles [-]

Note that the contact surface between deck and piles doesn't influence the allowable horizontal force. However, punching through the deck should be checked carefully. Also note that the piles must be capable to resist the shear force.

Water level variations and vertical components of hawser forces result in a variable vertical pressure on the piles. Therefore it should be carefully checked which situation is governing for horizontal sliding of the deck.

The absence of a connection between deck and piles is a great benefit for the quay wall's flexibility. If ballast water is pumped out of the caissons, the deck will automatically float free from the pile heads and can be mobilized.

J.3. Compressive force on piles

Once the quay wall is immersed on the piles, it exerts a vertical downward force onto the piles. This force is equal to the total weight of the structure including ballast water and surcharges, minus the upward buoyancy force generated by the displaced amount of water. It is assumed that the deck of the quay behaves like a rigid body and that all pile heads have the same elevation. According to this schematization, one can determine the compressive force on a single pile by dividing the total vertical force by the total number of piles.

To avoid penetration through the deck and large bending moments in the bottom slab, the piles are located straight under the inner walls of the caissons.

J.4. Bearing capacity and pile settlement

The bearing capacity of compression piles is determined by the sum of the tip bearing capacity and the shaft bearing capacity. Both can be determined by the equations of Koppejan.

$$F_{bear;tip} = A_{pile} \cdot ½ \cdot \alpha_p \cdot \beta \cdot s \cdot \left(\frac{q_{c;I;avg} + q_{c;II;avg}}{2} + q_{c;III;avg}\right)$$

Equation J.2

$$F_{bear;shaft(z)} = O_{tip} \int_0^{\Delta L} (\alpha_s \cdot q_c) dz$$

Equation J.3

Where:
$F_{bear;tip}$ = pile tip bearing capacity [kN]
A_{pile} = surface of pile tip [m^2]
α_p = pile class factor [-]
β = shape factor for foot of the pile [-]
s = shape factor for the cross section of the pile [-]
$q_{c;I;avg}$ = average cone resistance section I [kN/m^2]
$q_{c;I;avg}$ = average cone resistance section II [kN/m^2]
$q_{c;I;avg}$ = average cone resistance section III [kN/m^2]
$F_{bear;shaft(z)}$ = pile shaft bearing capacity [kN]
O_{tip} = circumference of pile tip [m]
α_S = factor depending on method of realization [-]
q_c = cone resistance [kN/m^2]

Note that the negative shaft bearing capacity has to be subtracted from the total bearing capacity. This capacity is calculated by the next formula.

$$F_{shaft;neg} = O_s \cdot \sum (h_{layer} \cdot \sigma'_{v;avg}) \cdot K_{o;rep} \cdot \tan(\delta_{rep})$$

Equation J.4

Where:
$F_{shaft;neg}$ = negative pile shaft friction [kN]
O_s = circumference of the pile shaft [m]
h_{layer} = layer thickness [m]
$\sigma'_{v;avg}$ = average effective vertical soil pressure [kN/m^2]
$K_{o;rep}$ = neutral soil coefficient [-]
δ_{rep} = angle of external friction between soil and pile [degree]

Determining all parameters to calculate the bearing capacity is a bit complicated and makes use of a sounding. For a preliminary design however, one can make reasonable assumptions for this.

Settlement of piles can result in much larger bending moments in the bottom slab and inner walls of the caissons. Therefore the pile heads may not differ too much in elevation. Notice that higher pile heads receive a higher load than lower ones, creating more settlement. This mechanism will level the pile heads, because of the quay wall's weight.

Pile settlements can be considered small and can be estimated by a simple rule of thumb. Settlements are about 2 to 3% of the width of the pile tip in case of a ratio of about 1.5 between the actual load and the maximum allowable load on the pile.

J.5. Absorbable horizontal force by the soil

The quay wall structure will be exposed to horizontal loads caused by wind on moored vessels, waves, ship berths and crane accelerations on the deck. First of all the friction between the deck and piles must be large enough to avoid horizontal sliding of the deck, as described in one of the previous paragraphs.

Apart from this, the piles must be driven sufficiently deep into the subsoil, so the soil is able to absorb the horizontal forces.

The maximum horizontal force on the pile heads that can be absorbed by the soil can be calculated with equation J.5.

$$F_{hor;soil} = \gamma' \cdot K_p \cdot \frac{t_o^3}{24} \cdot \frac{t_o + 4d_{pile}}{t_o + h_{pile}}$$

with:

$$t_o = \frac{t}{1,2}$$

Equation J.5

Where: $F_{hor;soil}$ = maximum absorbable horizontal force by the soil [kN]
γ'= under water weight of the soil [kN/m^3]
K_p = passive soil coefficient [-]
t_o = depth where the moment of the ideal load is zero [m]
t = pile driven depth [m]
d_{pile} = diameter or width of the pile [m]
h_{pile} = height of the pile above the soil [m]

The minimum pile driven depth to avoid breaking out of the soil follows from the maximum horizontal force acting on the piles. The area of the soil that contributes to the stability is schematized in figure J.1. It can be seen from the figure, that the total absorbable force cannot be determined by adding the forces of all single piles together if they are positioned that close to each other so the areas overlap.

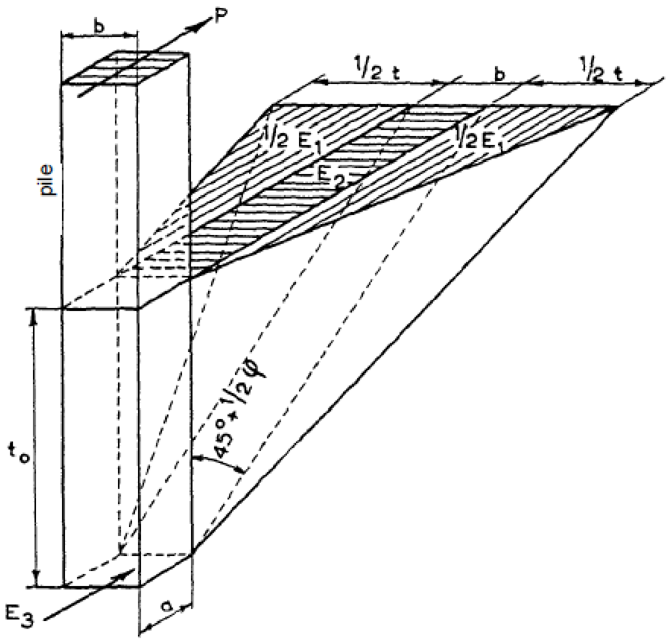


Figure J.1 – area contributing to stability of the soil around horizontally loaded piles [Lecture Notes – Manual Hydraulic Structures]

J.6. Horizontal displacement of pile head

The horizontal displacement of the pile heads, caused by the horizontal load can be calculated with Blum's formula.

$$\delta = \frac{F_{hor}(h_{pile} + 0{,}65t)^3}{3EI}$$

Equation J.6

Where: δ = horizontal displacement of the pile head [mm]
F_{hor} = acting horizontal force [N]
h_{pile} = length of the pile above the soil [mm]
t = pile driven depth [mm]
E = elasticity modulus [N/mm^2]
I = moment of inertia of pile cross section [mm^4]

Displacement of the piles results in displacement of the deck of the quay. Since container handling equipment and operating personnel is working on the quay, displacements should be limited.

J.7. Bending moment in piles

A bending moment will occur in the piles, due to the horizontal load on the pile head. This momentum cannot simply be calculated by force and length above the soil, because the soil will deform. The location of the maximum bending moment in the pile should be determined first, before one can calculate the bending moment itself.

This location must be solved iteratively by equation J.7.

$$X_M^2(X_M + 3d_{pile}) = \frac{t_o^3}{4} \cdot \frac{t_o + 4d_{pile}}{t_o + h_{pile}}$$

Equation J.7

Where: X_m = depth below height above the soil, where the bending moment is at its maximum [m]

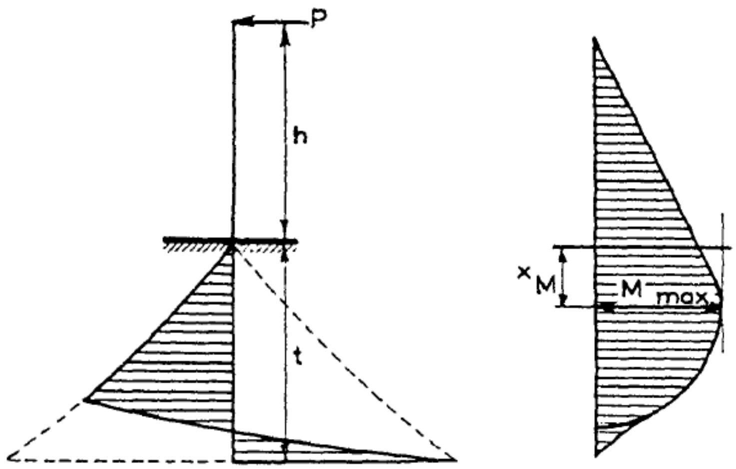


Figure J.2 – location of maximum bending moment in horizontally loaded pile in soil [Lecture Notes – Manual Hydraulic Structures]

Now the maximum momentum itself can be determined by:

$$M_{max;hor} = F_{hor}(h_{pile} + X_M) - \gamma' \cdot K_p \left(\frac{d_{pile} \cdot X_M^3}{6} + \frac{X_M^4}{24} \right)$$

Equation J.8

Where: $M_{max;hor}$ = maximum momentum as a result of horizontal force on the pile head [kNm]

Note that the total bending moment will be larger, due to the horizontal displacement of the pile head in combination with the vertical downward force on the piles.

J.8. Torsion

When the quay wall is immersed on the piles only and has no strong connection to the shore, torsion will also play a role. Vessels berthing on the tips of the quay wall introduce torsion onto the piles. To reduce the problems caused by horizontal forces one could connect the quay wall's deck to abutments on land.

J.9. Absorbing horizontal loads by abutments

To avoid horizontal displacements of the entire structure, one could connect the quay wall to abutments on land. These abutments must be able to absorb the horizontal forces acting on the structure. Therefore they should be anchored in de soil very well to resist the wind load acting on moored vessels.

Using abutments leads to a concentration of loads at the connections between the quay wall caisson and the abutment. The caisson must be able to resist this force and the forces must be transferred properly onto the walls of the caissons.

Horizontal forces can act in all directions of the horizontal plane. Abutments should either be applied on one side of the quay, given that they are capable to resist sufficient shear force, or must be applied on two perpendicular sides of the quay, so they will be exposed to normal forces only.

J.10. Pile stiffness and anchor force on abutments

When abutments are used to prevent horizontal movement, one must calculate the force that the abutments need to absorb. In order to do so, the stiffness of the pile foundation has to be determined. Since the horizontal displacement of a pile head could be calculated using to equations mentioned before, one can determine the force in the connection between the deck and the abutments by using the spring stiffness. The stiffness of a single pile in one layer of soil can be determined by the next equation.

$$k_{pile} = \frac{3EI}{(h_{pile} + 0{,}65t)^3}$$

Equation J.9

Where: k_{pile} = spring stiffness of one single pile [kN/m]

Now, the force in the connection between the quay wall and the abutment can be calculated using equation J.2.

$$F_{abutment} = k_{pile} \cdot \delta \cdot n$$

Equation J.10

Where: $F_{abutment}$ = force exerted on the abutment [kN]
δ = displacement of a free moving pile head [m]
n = number of piles [-]

The abutments should be capable to resist the force that follows from the mentioned equations. The soil behind the abutment must be able to absorb the compressive force and the ground anchors should be able to absorb the tensile forces. If the abutments are 90 degrees rotated with respect to each other in the horizontal plane, no shear forces have to be absorbed.

J.11. Foundation piles – Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of soil	1900	[kg/m3]
Density of water	1030	[kg/m3]
Foundation pile properties		
Number of piles per caisson	80	[-]
Pile diameter	700	[mm]
Wall thickness steel pile	25	[mm]
Steel yield stress	235	[N/mm2]
Length above the bottom	8,0	[m]
Length into the soil	6,0	[m]
Modules of elasticity steel	2,10E+05	[N/mm2]
Cross sectional area of pile	53.014	[mm2]
Moment of inertia [I]	3,02E+09	[mm4]
Moment of resistance [W]	6,71E+06	[mm3]
Soil properties		
passive soil pressure coefficient	3	[-]
Loads on foundation piles		
Compressive force per pile	800	[kN]
Horizontal force on pile head	50	[kN]
Normal pressure in pile	15,09	[N/mm2]
Maximum occurring shear force in pile due to horizontal force	0,94	[N/mm2]
Maximum absorbable horizontal force by the surrounding soil	349	[kN]
Maximum pile head displacement due to horizontal force	44	[mm]
Depth of maximum bending moment in pile with respect to the bottom	2,1105	[m]
1 st calculated value with iteration (must equal 2nd)	18,75	[m3]
2 nd calculated value with iteration (must equal 1st)	18,75	[m3]
Maximum bending moment in pile	492	[kNm]
Maximum occurring stress in pile due to bending moment	**73**	[N/mm2]
Stiffness of all piles under one caisson	90.427	[kN/m]
Force on abutment per caisson	4.000	[kN]

Appendix K: Two caissons on top of each other – Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of water	1030	[kg/m3]
Density of concrete	2550	[kg/m3]
Reverence level = container terminal surface	0	[m]
Safety factor beneficial loads	0,90	[-]
Safety factor non-beneficial loads	1,20	[-]
Safety factor angle of internal friction of the soil	1,25	[-]
Soil properties		
Ground water level	**-6,5**	[m]
Sea water level	**-7,5**	[m]
Top level upper soil layer	0	[m]
Upper soil layer: density of dry soil	1835	[kg/m3]
Upper soil layer: density of wet soil	2040	[kg/m3]
Upper soil layer: angle of internal friction	35	[degree]
Upper soil layer: horizontal soil coefficient	0,54	[-]
Top level intermediate soil layer	**-6,5**	[m]
Intermediate soil layer: density of dry soil	1935	[kg/m3]
Intermediate soil layer: density of wet soil	2140	[kg/m3]
Intermediate soil layer: angle of internal friction	37,5	[degree]
Intermediate soil layer: horizontal soil coefficient	0,51	[-]
Top level lower soil layer	**-13**	[m]
Lower soil layer: density of dry soil	1935	[kg/m3]
Lower soil layer: density of wet soil	2140	[kg/m3]
Lower soil layer: angle of internal friction	37,5	[degree]
Lower soil layer: horizontal soil coefficient	0,51	[-]
Minimum required bearing capacity of the soil	400	[kN/m2]
Top level upper caisson	**0**	[m]
Bottom level upper caisson	**-11**	[m]
Top level lower caisson	**-11**	[m]
Bottom level lower caisson	**-22**	[m]
Surface load		
Additional surface load on land side	7,5	[kN/m2]
Resulting pressures at land side		
Vertical pressure at ground water level *	124,51	[kN/m2]
Vertical pressure at bottom of upper soil layer	124,51	[kN/m2]
Vertical pressure at bottom of intermediate soil layer	260,97	[kN/m2]
Vertical pressure at bottom of lower soil layer	449,91	[kN/m2]

Horizontal pressure at ground water level *	67,38	[kN/m2]
Horizontal pressure at bottom of upper soil layer	67,38	[kN/m2]
Horizontal pressure at bottom of intermediate soil layer	165,86	[kN/m2]
Horizontal pressure at bottom of lower soil layer	307,07	[kN/m2]
Horizontal pressure at top of upper caisson	0	[kN/m2]
Horizontal pressure at bottom of upper caisson **	166,50	[kN/m2]
Horizontal pressure at top of lower caisson **	166,50	[kN/m2]
Horizontal pressure at bottom of lower caisson	307,07	[kN/m2]
Upper caisson		
Horizontal force from land side on upper caisson	74.519	[kN]
Momentum caused by horizontal force from land side on upper caisson	247.654	[kNm]
Horizontal force from sea side on upper caisson	-6.189	[kN]
Momentum caused by horizontal force from sea side in upper caisson	-7.220	[kNm]
Upward buoyancy force on upper caisson	-133.377	[kN]
Momentum caused by upward buoyancy force on upper caisson	91.697	[kNm]
Net horizontal force on upper caisson	**68.330**	[kN]
Net momentum in non-operational conditions at upper caisson	**332.130**	[kNm]
Lower caisson		
Horizontal force from land side on lower caisson	246.054	[kN]
Momentum caused by horizontal force from land side on lower caisson	1.194.741	[kNm]
Horizontal force from sea side on lower caisson	-100.033	[kN]
Momentum caused by horizontal force from sea side on lower caisson	-438.106	[kNm]
Upward buoyancy force on lower caisson ***	-366.786	[kN]
Momentum caused by upward buoyancy force on lower caisson ***	0	[kN]
Net horizontal force on bottom caisson	**146.022**	[kN]
Net momentum in non-operational conditions at bottom caisson	**756.636**	[kNm]
Both caissons together		
Horizontal force from land side on both caissons together	320.573	[kN]
Moment from horizontal force from land side on both caissons	2.262.100	[kNm]
Horizontal force from sea side on both caissons together	-106.221	[kN]
Moment from horizontal force from sea side on both caissons	-513.404	[kNm]
Upward buoyancy force on both caissons together	-500.163	[kN]
Momentum caused by upward buoyancy force on both caissons together	91.697	[kN]
Net horizontal force on both caissons together	**214.351**	[kN]
Net momentum in non-operational conditions at both caisson together	**1.840.393**	[kNm]
Dimensions caisson		
Height	11,00	[m]
Width	33,00	[m]
Length (> width)	100,00	[m]
Number of inner walls in transversal direction	12	[-]
Number of inner walls in longitudinal direction	4	[-]

Thickness bottom slab	0,55	[m]
Thickness deck	0,40	[m]
Thickness side wall transversal direction	0,55	[m]
Thickness side wall longitudinal direction	0,48	[m]
Thickness inner walls in transversal direction	0,30	[m]
Thickness inner walls in longitudinal direction	0,30	[m]
Friction coefficient between caisson and subsoil	0,5	[-]
Friction coefficient between both caissons	0,5	[-]
Thickness of gravel bed foundation	1,0	[m]
Height of ballast material in upper caisson	9,00	[m]
Density of ballast material upper caisson	18	[kN/m3]
Additional weight in lower caisson	0	[kN]
Height of ballast water in lower caisson ***	10,05	[m]
Caisson properties		
Total volume of caisson	36.300	[m3]
Solid volume of caisson	6.753	[m3]
Hollow volume in caisson	29.547	[m3]
Bottom slab surface of hollow volume	2.940	[m2]
Dead weight empty caisson	168.928	[kN]
Draught of empty caisson	5,07	[m]
Upper caisson non-operational		
Weight of ballast material in upper caisson	476.281	[kN]
Total weight of upper caisson including ballast material	645.209	[kN]
Upward buoyancy force on upper caisson	-133.377	[kN]
Net vertical force of upper caisson	511.832	[kN]
Net horizontal force on upper caisson	81.996	[kN]
Maximum absorbable horizontal force against sliding off	230.325	[kN]
Net momentum in non-operational conditions at upper caisson	332.130	[kNm]
M / V ratio	0,87	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Stability against horizontal sliding	**YES**	[-]
Stability against overturning	**YES**	[-]
Lower caisson non-operational		
Weight of ballast water in lower caisson	298.552	[kN]
Total weight of lower caisson including ballast water	467.481	[kN]
Upward buoyancy force on lower caisson	-366.786	[kN]
Net vertical force of lower caisson	100.695	[kN]
Net horizontal force on lower caisson	175.226	[kN]
Maximum absorbable horizontal force against pushing out	505.962	[kN]
Stability against pushing out of lower caisson	**YES**	[-]
Both caissons together non-operational		

Net vertical force of both caissons together	612.527	[kN]
Net horizontal force on both caissons together	257.222	[kN]
Maximum absorbable horizontal force against sliding of both caissons	275.637	[kN]
Net momentum in non-operational conditions at both caissons together	1.840.393	[kNm]
M / V ratio	4,01	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Stability against horizontal sliding	**YES**	[-]
Stability against overturning	**YES**	[-]
Additional loads during storm, with equipment		
Dead weight of STS crane	7.000	[kN]
Horizontal center of gravity of STS crane, relative to rotation point	2	[m]
Additional weight on upper caisson	5.000	[kN]
Horizontal force caused by wind friction on STS crane in SLS	227	[kN]
Horizontal force caused by wind friction on STS crane in ULS	1.003	[kN]
Height of reattachment wind force point relative to top of upper caisson	29,87	[m]
Wind force on container vessel in SLS	1.800	[kN]
Wind force on container vessel in ULS	7.850	[kN]
Angle of mooring lines	25	[degree]
Percentage of wind load on vessel per caisson	33	[%]
Bollard height	1	[m]
Bollard distance from edge of caisson	1,5	[m]
Crane boom length	37	[m]
Maximum weight of one hoist operation	500	[kN]
Vertical acceleration of container lifting	0,5	[m/s2]
Vertical component of wind load on vessel per caisson SLS	-251	[kN]
Horizontal component of wind load on vessel per caisson SLS	538	[kN]
Vertical component of wind load on vessel per caisson ULS	-1.095	[kN]
Horizontal component of wind load on vessel per caisson ULS	2.348	[kN]
SERVICEABILITY LIMIT STATE (SLS)		
Upper caisson during SLS conditions		
Net vertical force of upper caisson (upper limit)	531.632	[kN]
Net vertical force of upper caisson (lower limit)	511.581	[kN]
Net horizontal force on upper caisson (upper limit)	83.186	[kN]
Net horizontal force on upper caisson (lower limit)	82.642	[kN]
Maximum absorbable horizontal force against sliding (upper limit)	239.235	[kN]
Maximum absorbable horizontal force against sliding (lower limit)	230.212	[kN]
Momentum caused by dead weight of STS crane	14.000	[kNm]
Momentum caused by wind force on STS crane	9.277	[kNm]
Momentum caused by wind force on container vessel	2.695	[kNm]
Momentum on upper caisson caused by container lifting	28.113	[kNm]
Total net momentum on upper caisson (upper limit)	437.606	[kNm]

Total net momentum on upper caisson (lower limit)	334.825	[kNm]
M / V ratio (upper limit)	1,10	[1/m]
M / V ratio (lower limit)	0,87	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Stability against horizontal sliding (upper limit)	YES	[-]
Stability against horizontal sliding (lower limit)	YES	[-]
Stability against overturning	YES	[-]
Both caissons together during SLS conditions		
Net vertical force of both caissons together (upper limit)	632.327	[kN]
Net vertical force of both caissons together (lower limit)	612.276	[kN]
Net horizontal force on both caissons together (upper limit)	258.412	[kN]
Net horizontal force on both caissons together (lower limit)	257.868	[kN]
Maximum absorbable horizontal force against sliding (upper limit)	284.547	[kN]
Maximum absorbable horizontal force against sliding (lower limit)	275.524	[kN]
Momentum caused by dead weight of STS crane	14.000	[kNm]
Momentum caused by wind force on STS crane	11.774	[kNm]
Momentum caused by wind force on container vessel	8.616	[kNm]
Momentum on both caissons together caused by container lifting	28.113	[kNm]
Total net momentum on both caissons together (upper limit)	1.956.785	[kNm]
Total net momentum on both caissons together (lower limit)	1.849.009	[kNm]
M / V ratio (upper limit)	4,13	[1/m]
M / V ratio (lower limit)	4,03	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Required bearing capacity of foundation (buildup of pressure)	332	[kN/m2]
Required bearing capacity of foundation (no buildup of pressure) upper limit	378	[kN/m2]
Required bearing capacity of foundation (no buildup of pressure) lower limit	363	[kN/m2]
Stability against horizontal sliding (upper limit)	YES	[-]
Stability against horizontal sliding (lower limit)	YES	[-]
Stability against overturning	YES	[-]
Stability against slide plane in the subsoil	YES	[-]
ULTIMATE LIMIT STATE (ULS)		
Upper caisson during ULS conditions		
Net vertical force of upper caisson (upper limit)	529.738	[kN]
Net vertical force of upper caisson (lower limit)	510.738	[kN]
Net horizontal force on upper caisson (upper limit)	87.220	[kN]
Net horizontal force on upper caisson (lower limit)	84.813	[kN]
Maximum absorbable horizontal force against sliding (upper limit)	238.382	[kN]
Maximum absorbable horizontal force against sliding (lower limit)	229.832	[kN]
Momentum caused by dead weight of STS crane	14.000	[kNm]
Momentum caused by wind force on STS crane	40.993	[kNm]
Momentum caused by wind force on container vessel	145.923	[kNm]

Momentum on upper caisson caused by container lifting	0	[kNm]
Total net momentum on upper caisson (upper limit)	588.039	[kNm]
Total net momentum on upper caisson (lower limit)	478.054	[kNm]
M / V ratio (upper limit)	1,48	[1/m]
M / V ratio (lower limit)	1,25	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Stability against horizontal sliding (upper limit)	**YES**	[-]
Stability against horizontal sliding (lower limit)	**YES**	[-]
Stability against overturning	**YES**	[-]
Both caissons together during ULS conditions		
Net vertical force of both caissons together (upper limit)	630.432	[kN]
Net vertical force of both caissons together (lower limit)	611.432	[kN]
Net horizontal force on both caissons together (upper limit)	262.446	[kN]
Net horizontal force on both caissons together (lower limit)	260.039	[kN]
Maximum absorbable horizontal force against sliding (upper limit)	283.695	[kN]
Maximum absorbable horizontal force against sliding (lower limit)	275.145	[kN]
Momentum caused by dead weight of STS crane	14.000	[kNm]
Momentum caused by wind force on STS crane	52.026	[kNm]
Momentum caused by wind force on container vessel	37.577	[kNm]
Momentum on both caissons together caused by container lifting	0	[kNm]
Total net momentum on both caissons together (upper limit)	2.010.021	[kNm]
Total net momentum on both caissons together (lower limit)	1.877.970	[kNm]
M / V ratio (upper limit)	4,25	[1/m]
M / V ratio (lower limit)	4,10	[1/m]
Maximum M/V ration against overturning	5,50	[1/m]
Required bearing capacity of foundation (buildup of pressure)	335	[kN/m2]
Required bearing capacity of foundation (no buildup of pressure) upper limit	381	[kN/m2]
Required bearing capacity of foundation (no buildup of pressure) lower limit	364	[kN/m2]
Stability against horizontal sliding (upper limit)	**YES**	[-]
Stability against horizontal sliding (lower limit)	**YES**	[-]
Stability against overturning	**YES**	[-]
Stability against slide plane in the subsoil	**YES**	[-]

Worst case scenario unity check for sliding	**0,95**	[-]
Worst case scenario unity check for overturning	**0,77**	[-]
Worst case scenario unity check for slide circle	**0,95**	[-]

Appendix L: Loads on quay wall structure and required bearing capacity in case of pile foundation– Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of concrete	2550	[kg/m3]
Density of water	1030	[kg/m3]
Density of air	1,28	[kg/m3]
Dimensions caisson		
Height	11,00	[m]
Width	33,00	[m]
Length (> width)	100,00	[m]
Number of inner walls in transversal direction	12	[-]
Number of inner walls in longitudinal direction	3	[-]
Thickness bottom slab	0,90	[m]
Thickness deck	0,60	[m]
Thickness side wall transversal direction	0,90	[m]
Thickness side wall longitudinal direction	0,90	[m]
Thickness inner walls in transversal direction	0,40	[m]
Thickness inner walls in longitudinal direction	0,40	[m]
Number of caisson per berth	3	[-]
Additional weight of equipment		
Additional weight of cranes containers etc. per caisson	2000	[ton]
Ballast water inside caisson		
Height of ballast water inside caisson	2,60	[m]
Maximum possible height of ballast water	9,50	[m]
Freeboard	7,50	[m]
Freeboard in floating condition without ballast water	3,15	[m]
Loads in operational conditions		
Dead weight of container vessel	120.000	[ton]
Berthing speed perpendicular to the quay	0,15	[m/s]
Reaction force of fender	2.000	[kN]
Surface area of wind force of container vessel	7.000	[m2]
Wind speed perpendicular to vessel ULS	42	[m/s]
Wind speed perpendicular to vessel SLS	20	[m/s]
Water flow velocity	2,0	[m/s]
Maximum bollard force during berthing (not for ULS)	2.000	[kN]
Angle of mooring lines with horizontal plane	25	[degree]
Number of bollards used for berthing	0	[-]
Friction factor between quay and foundation	0,5	[-]

Caisson properties		
Total volume of caisson	36.300	[m3]
Solid volume of caisson	9.681	[m3]
Empty volume in caisson	26.619	[m3]
Bottom slab surface empty volume	2.802	[m2]
Total weight including additional load*	26.687	[ton]
Draught including additional load*	7,85	[m]
Loads on structure		
Weight of ballast water per caisson	7.504	[ton]
Vertical force on foundation per caisson**	**218.702**	[kN]
Absorbable horizontal force against sliding**	109.351	[kN]
Berthing energy of container vessel	945	[kNm]
Horizontal component bollard force during berthing	1.813	[kN]
Vertical component bollard force during berthing	845	[kN]
Horizontal force due to wind on hull of entire vessel in ULS	7.162	[kN]
Vertical force due to wind on hull of entire vessel in ULS	3.340	[kN]
Horizontal force due to wind on hull of entire vessel in SLS	1.624	[kN]
Vertical force due to wind on hull of entire vessel in SLS	757	[kN]
Horizontal force caused by water current in transversal direction	238	[kN]

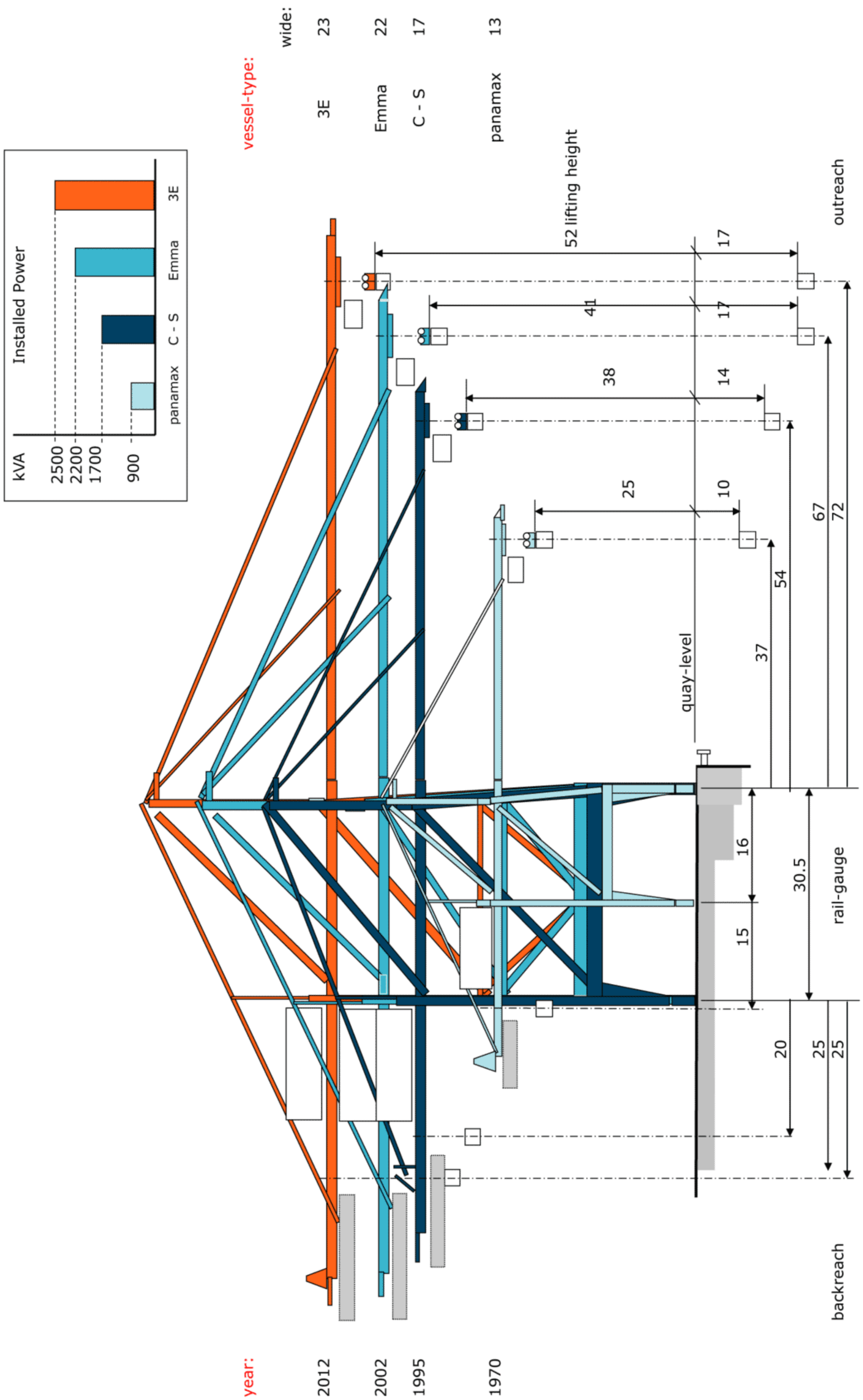
Appendix M: Tug boat force during transport – Excel sheet

Constant values		
Water density	1030	[kg/m3]
Gravitational acceleration	9,81	[m/s2]
Caisson properties		
Width of caisson	33	[m]
Draught of caisson	10	[m]
Drag force coefficient	1,15	[m]
Sailing velocity without wind and current	4	[m/s]
Total drag force during transport	3.127	[kN]
Load on side wall (bow of caisson)	9,48	[kN/m2]

Appendix N: Wave load on vertical wall – Excel sheet

Constant values		
Density of water	1030	[kg/m3]
Gravitational acceleration	9,81	[m/s2]
Wave properties and corresponding forces		
Freeboard of upper caisson	3,5	[m]
Height of upper caisson	11	[m]
Height of lower caisson	11	[m]
Width of caissons	33	[m]
Length of caissons	100	[m]
Wave height	2	[m]
Wave period	8	[s]
Water depth	15	[m]
Wave length (deep water)	99,92	[m]
Wave number	0,073	[1/m]
Wave length (transitional water)	79,95	[m]
Increase of middle water level	0,18	[m]
Peak wave pressure at water line	**20,21**	[kN/m2]
Wave pressure near bottom	**12,12**	[kN/m2]
Total wave force on upper caisson	**15.948**	[kN]
Height of attachment point relative to bottom of upper caisson	4,21	[m]
Total wave force on lower caisson	**15.979**	[kN]
Height of attachment point relative to bottom of lower caisson	5,20	[m]
Total wave force on both caissons together	**31.927**	[kN]
Total height of reattachment point relative to bottom of lower caisson	10,20	[m]
Additional upward buoyance force caused by wave pressure	**20.002**	[kN]
Resulting overturning momentum		
Overturning momentum caused by additional buoyance force	110.010	[kNm]
Overturning momentum at upper caisson	**67.194**	[kNm]
Overturning momentum at lower caisson	**83.036**	[kNm]
Overturning momentum at both caissons together	**435.668**	[kNm]

Appendix O: Ship-to-shore crane dimensions [APM Terminals]



Appendix P: Determination of wind load on ship-to-shore crane

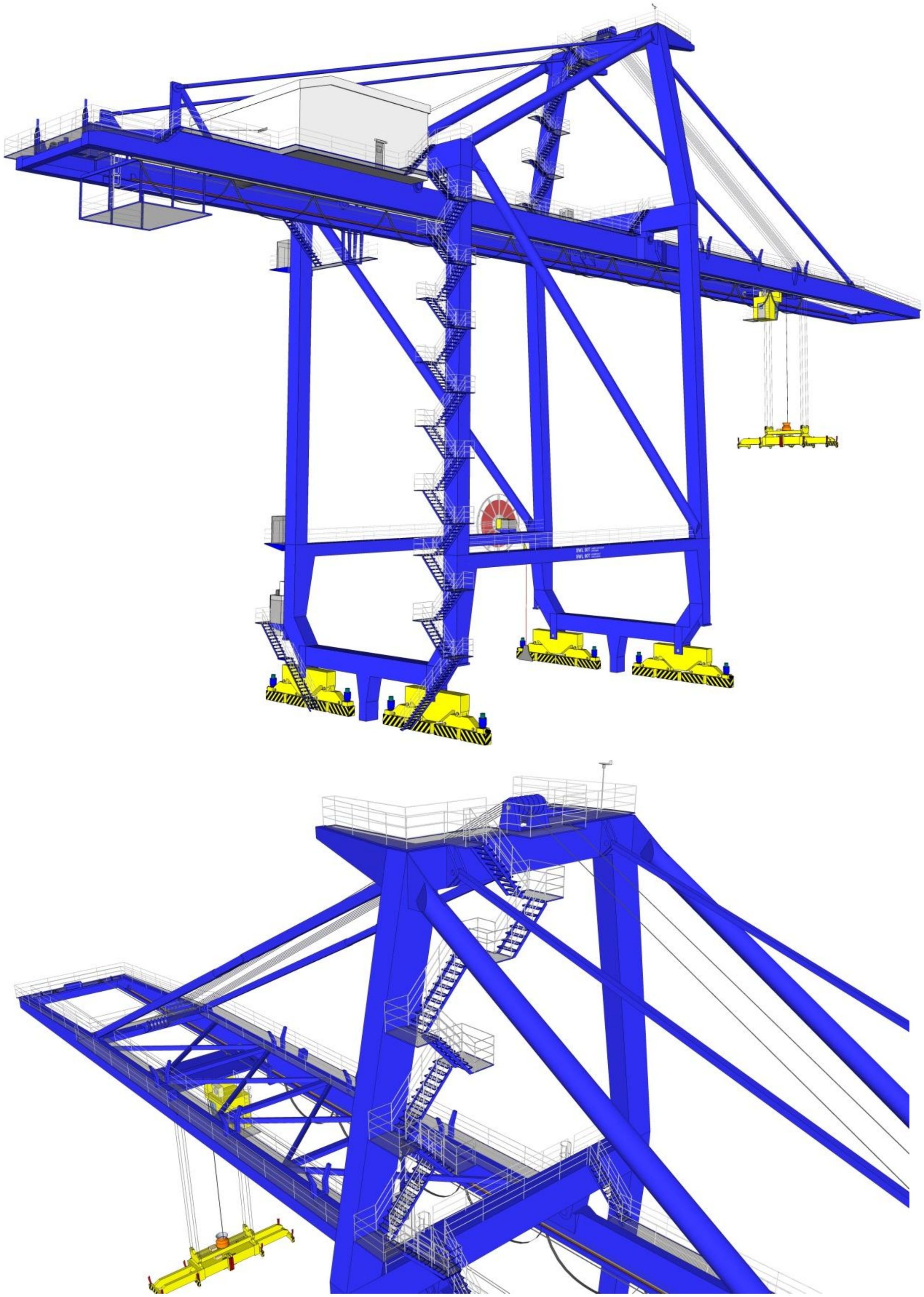


Figure P.1 – impression of highly detailed scale model of a post panamax STS crane, used to determine the governing wind surface and wind load.

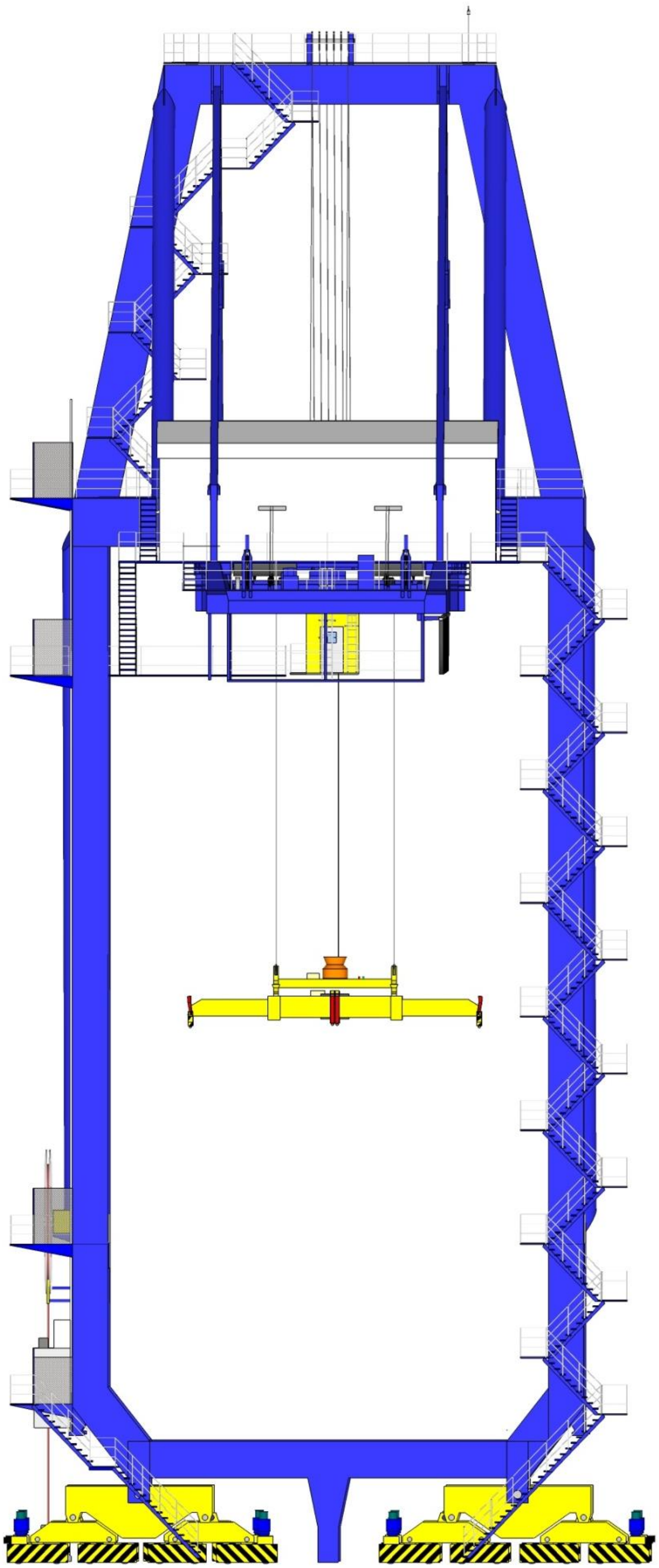


Figure P.2 – backside view of STS crane in parallel perspective

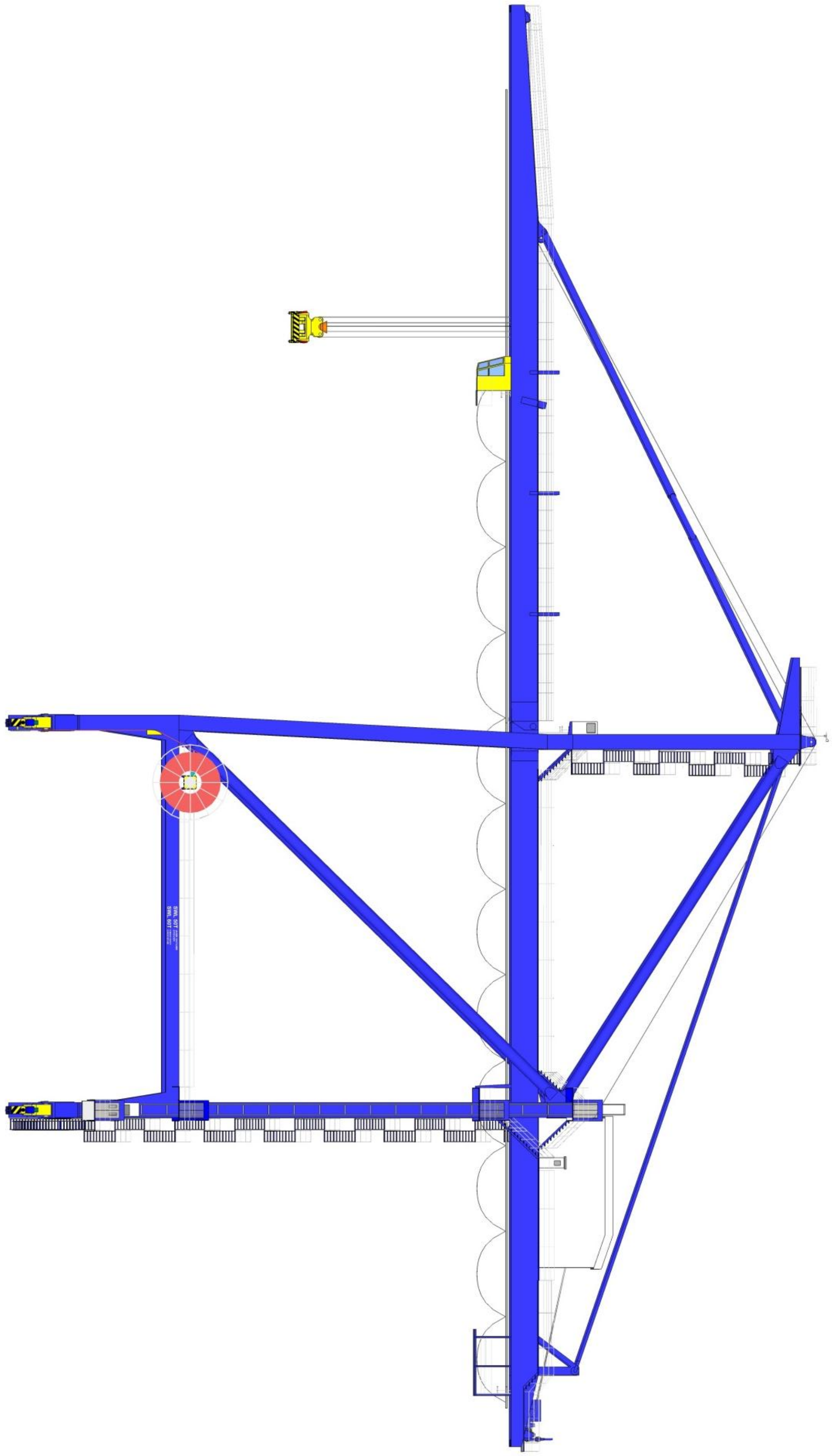


Figure P.3 – side view of STS crane in parallel perspective

Figure P.4 – illustration of governing wind load surface for overturning criterion of the caissons

Constant values		
Density of air	1,28	[kg/m3]
Gravitational acceleration	9,81	[m/s2]
STS crane and wind load properties		
Wind load surface at 90/270 degrees cross wind on STS	1200	[m2]
Height of attachment point above quay wall surface	37	[m]
Wind load surface at 0/180 degrees tail or head wind on STS	800	[m2]
Height of attachment point above quay wall surface	27	[m]
Drag force coefficient	1,15	[-]
Wind speed in SLS	20	[m/s]
Wind speed for STS crane failure	42	[m/s]
Angle of wind direction (range from 0 - 90 degrees)	**15**	[deg]
SERVICEABILITY LIMIT STATE (SLS)		
Wind force on STS crane	245	[kN]
Wind load component parallel to quay wall (90/270 degrees)	91	[kN]
Wind load component perpendicular to quay wall (0/180 degrees)	227	[kN]
Height of attachment point wind force, relative to top of quay	29,87	[m]
Overturning momentum on quay wall caused by wind force on STS	**6.795**	[kNm]
STS CRANE FAILURE		
Wind force on STS crane	1.081	[kN]
Wind load component parallel to quay wall (90/270 degrees)	403	[kN]
Wind load component perpendicular to quay wall (0/180 degrees)	1.003	[kN]
Height of attachment point wind force, relative to top of quay	29,87	[m]
Overturning momentum on quay wall caused by wind force on STS	**29.964**	[kNm]

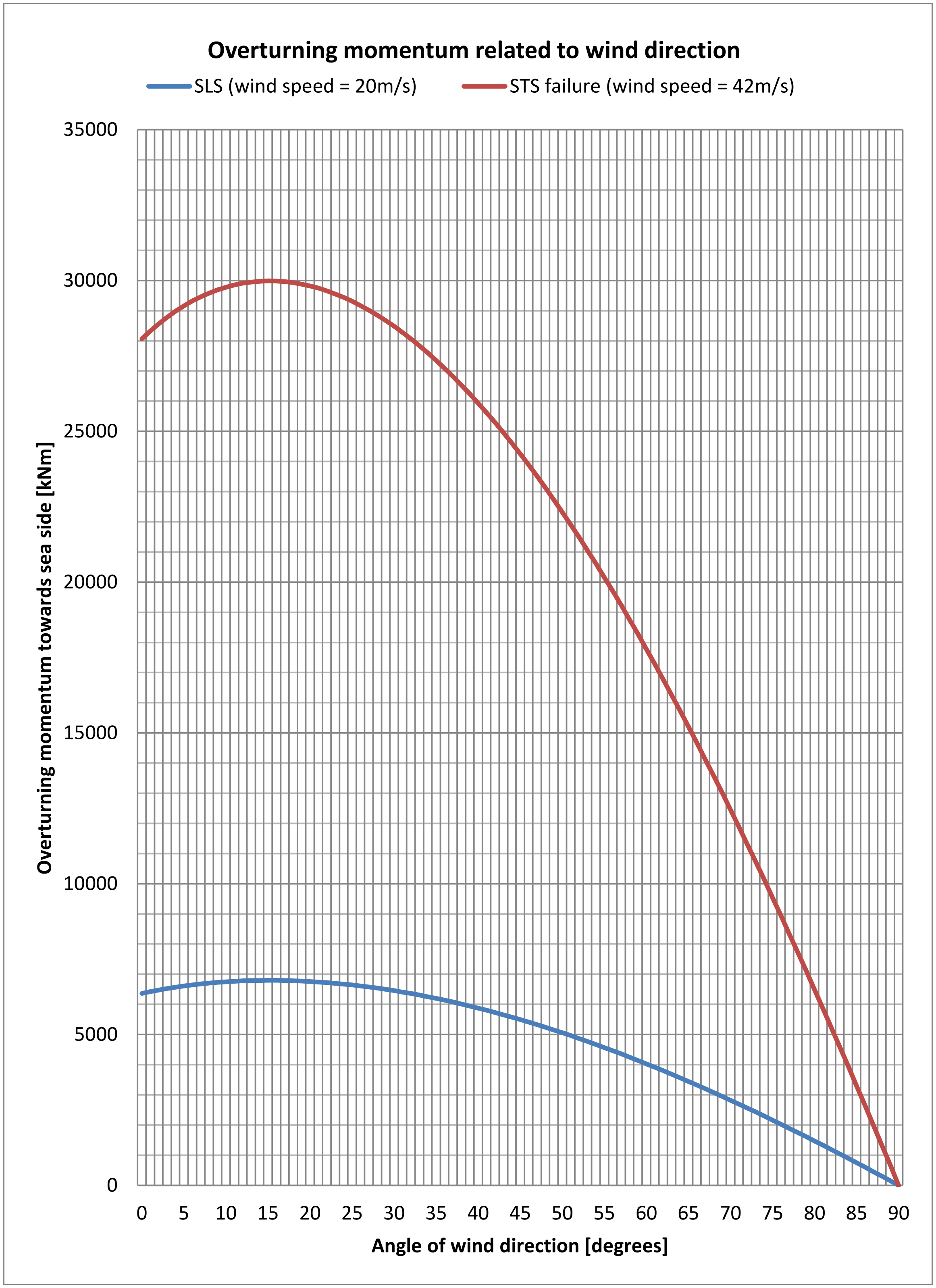


Figure P.5 – graph of governing wind load direction of overturning moment (15 degrees)

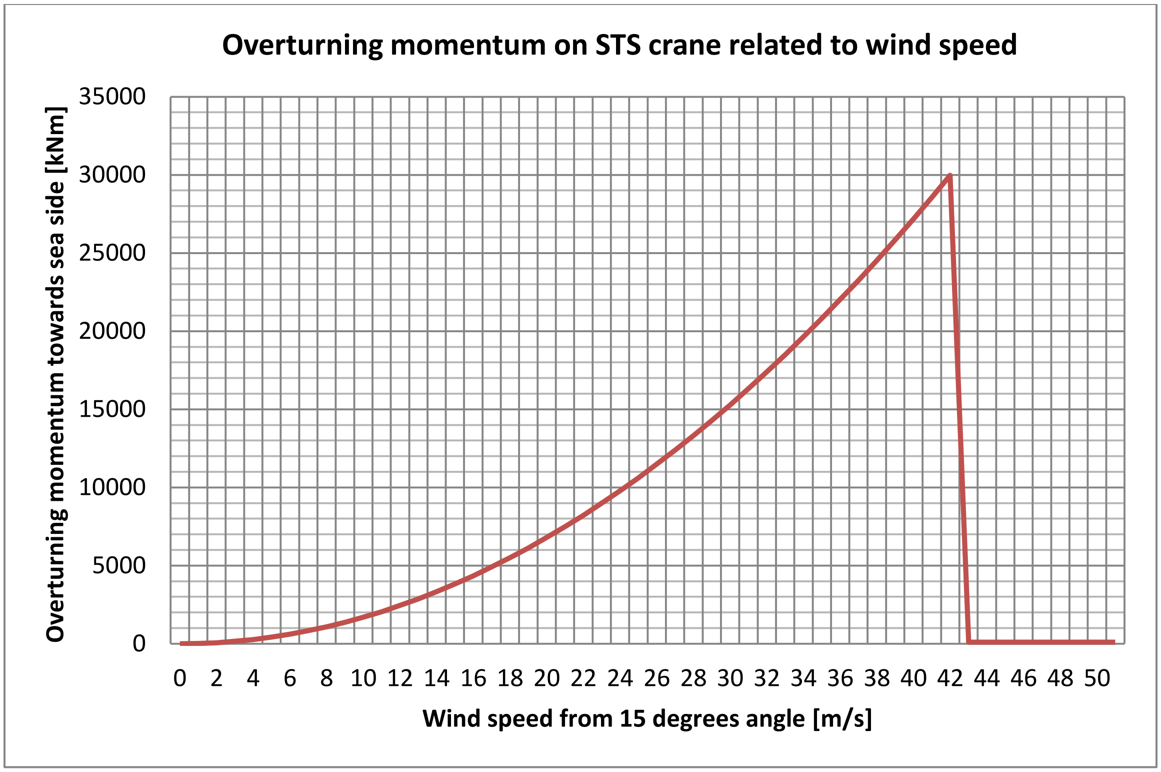


Figure P.6 – graph of overturning moment on STS crane, related to wind speed

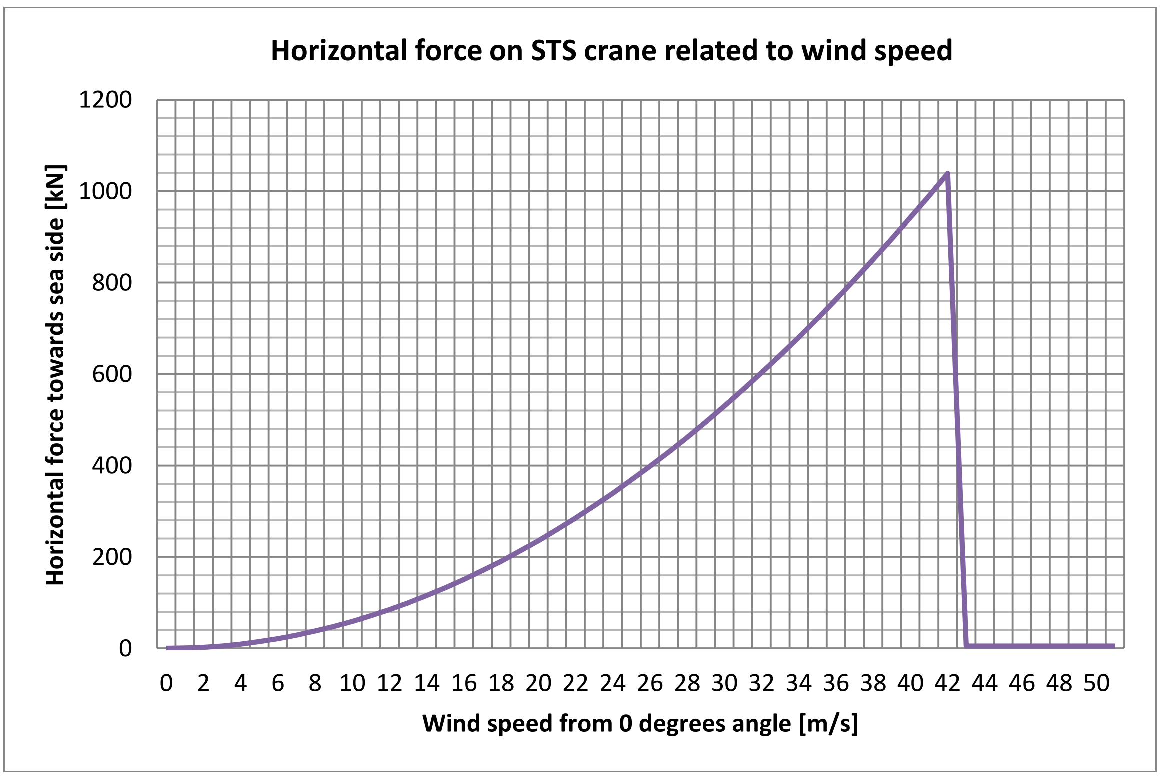


Figure P.7 – graph of horizontal force towards sea side, related to wind speed

Appendix Q: Wall thickness and reinforcement – Excel sheet

Constant values		
Gravitational acceleration	9,81	[m/s2]
Density of water	1030	[kg/m3]
Density of concrete	2550	[kg/m3]
Safety factor for permanent load	1,2	[-]
Safety factor for variable loads	1,3	[-]
Material factor steel yield stress	1,15	[-]
Material factor concrete tensile strength	1,5	[-]
Concrete properties		
Compressive strength	30	[N/mm2]
Tensile strength 5% failure (C30/C37)	2,0	[N/mm2]
Concrete cover	50	[mm]
Design value tensile strength	1,33	[N/mm2]
Reinforcement steel yield stress	500	[N/mm2]
Design value steel yield stress	435	[N/mm2]
Wave load during transport		
Wave length	100	[m]
Wave height	2	[m]
Maximum shear force in bottom slab	156	[kN/m]
Maximum bending moment at entire caisson	4.970	[kNm/m]
Maximum shear force caused by waves during transport	156	[kN/m]
Design value of maximum momentum caused by long waves during transport	213.205	[kNm]
Required amount of reinforcement steel in top and bottom slab to absorb wave load	1486	[mm2/m]
Wave load during operation		
Maximum wave pressure at vertical wall	35	[kN/m]
Caisson properties		
Caisson length	100	[m]
Caisson width	33	[m]
Caisson height	11	[m]
Number of inner walls in longitudinal direction	3	[-]
Number of inner walls in transversal direction	12	[-]
Inner wall thickness	300	[mm]
Width of outer wall sections	7,2	[m]
Height of outer wall sections	9,6	[m]
Width of bottom and top slab sections	7,6	[m]
Length of bottom and top slab sections	7,2	[m]

UPPER CAISSON		
Vertical longitudinal outer walls of upper caisson		
Maximum net pressure acting from inner side towards outer side*	-166	[kN/m2]
Maximum net pressure acting from outer side towards inner side**	133	[kN/m2]
Maximum shear force in cross section	436	[kN/m]
Maximum positive bending moment in x-direction (100m length)	344	[kNm/m]
Maximum negative bending moment in x-direction (100m length)	-276	[kNm/m]
Maximum positive bending moment in y-direction (11m length)	344	[kNm/m]
Maximum negative bending moment in y-direction (11m length)	-276	[kNm/m]
Chosen wall thickness	**550**	[mm]
Reinforcement bar diameter in x-direction (100m length) inner side	**20**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) inner side	**205**	[mm]
Maximum absorbable momentum in x-direction (100m length) inner side	278	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) inner side	**20**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) inner side	**205**	[mm]
Maximum absorbable momentum in y-direction (11m length) inner side	278	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) inner side	0,279	[%]
Actual reinforcement percentage in y-direction (11m length) inner side	0,279	[%]
Reinforcement bar diameter in x-direction (100m length) outer side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) outer side	**255**	[mm]
Maximum absorbable momentum in x-direction (100m length) outer side	347	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) outer side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) outer side	**255**	[mm]
Maximum absorbable momentum in y-direction (11m length) outer side	347	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) outer side	0,350	[%]
Actual reinforcement percentage in y-direction (11m length) outer side	0,350	[%]
Minimum reinforcement percentage in both directions	0,2	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**290**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	693	[mm2/m]
Maximum allowable amount of shear reinforcement	3.522	[mm2/m]
Maximum possible absorbable shear force for wall thickness	2.222	[kN/m]
Actual absorbable shear force by applied shear reinforcement	437	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) outer side	**YES**	[-]

Satisfaction of minimum reinforcement criterion	**YES**	[-]
Vertical transversal outer walls of upper caisson		
Maximum net pressure acting from inner side towards outer side*	-166	[kN/m2]
Maximum net pressure acting from outer side towards inner side**	133	[kN/m2]
Maximum shear force in cross section	436	[kN/m]
Maximum positive bending moment in x-direction (33m length)	344	[kNm/m]
Maximum negative bending moment in x-direction (33m length)	-276	[kNm/m]
Maximum positive bending moment in y-direction (11m length)	344	[kNm/m]
Maximum negative bending moment in y-direction (11m length)	-276	[kNm/m]
Chosen wall thickness	**550**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**20**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**205**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	278	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) inner side	**20**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) inner side	**205**	[mm]
Maximum absorbable momentum in y-direction (11m length) inner side	278	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,279	[%]
Actual reinforcement percentage in y-direction (11m length) inner side	0,279	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**255**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	347	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) outer side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) outer side	**255**	[mm]
Maximum absorbable momentum in y-direction (11m length) outer side	347	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,350	[%]
Actual reinforcement percentage in y-direction (11m length) outer side	0,350	[%]
Minimum reinforcement percentage in both directions	0,2	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**290**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	693	[mm2/m]
Maximum allowable amount of shear reinforcement	3.522	[mm2/m]
Maximum possible absorbable shear force for wall thickness	2.222	[kN/m]
Actual absorbable shear force by applied shear reinforcement	437	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion	**YES**	[-]

Bottom slab of upper caisson		
Maximum net pressure acting from outer side towards inner side**	133	[kN/m2]
Maximum shear force in cross section	444	[kN/m]
Maximum positive bending moment in x-direction (33m length)	106	[kNm/m]
Maximum negative bending moment in x-direction (33m length)	-374	[kNm/m]
Maximum positive bending moment in y-direction (100m length)	132	[kNm/m]
Maximum negative bending moment in y-direction (100m length)	-374	[kNm/m]
Chosen wall thickness	**450**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**185**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	380	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) inner side	**185**	[mm]
Maximum absorbable momentum in y-direction (100m length) inner side	380	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,590	[%]
Actual reinforcement percentage in y-direction (100m length) inner side	0,590	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**220**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	132	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) outer side	**220**	[mm]
Maximum absorbable momentum in y-direction (100m length) outer side	132	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,203	[%]
Actual reinforcement percentage in y-direction (100m length) outer side	0,203	[%]
Minimum reinforcement percentage in x-direction (33m length)	0,2	[%]
Minimum reinforcement percentage in y-direction (100m length, both sides)	0,33	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**260**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	773	[mm2/m]
Maximum allowable amount of shear reinforcement	3.157	[mm2/m]
Maximum possible absorbable shear force for wall thickness	1.818	[kN/m]
Actual absorbable shear force by applied shear reinforcement	445	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion for bending	**YES**	[-]
Satisfaction of minimum reinforcement criterion for tension	**YES**	[-]

Top slab of upper caisson		
Maximum net pressure acting from outer side towards inner side	78	[kN/m2]
Maximum shear force in cross section	281	[kN/m]
Maximum positive bending moment in x-direction (33m length)	66	[kNm/m]
Maximum negative bending moment in x-direction (33m length)	-238	[kNm/m]
Maximum positive bending moment in y-direction (100m length)	83	[kNm/m]
Maximum negative bending moment in y-direction (100m length)	-238	[kNm/m]
Chosen wall thickness	**350**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**215**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	243	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) inner side	**215**	[mm]
Maximum absorbable momentum in y-direction (100m length) inner side	243	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,652	[%]
Actual reinforcement percentage in y-direction (100m length) inner side	0,652	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**285**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	76	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) outer side	**260**	[mm]
Maximum absorbable momentum in y-direction (100m length) outer side	83	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,202	[%]
Actual reinforcement percentage in y-direction (100m length) outer side	0,221	[%]
Minimum reinforcement percentage in x-direction (33m length)	0,2	[%]
Minimum reinforcement percentage in y-direction (100m length, both sides)	0,42	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**285**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	705	[mm2/m]
Maximum allowable amount of shear reinforcement	3.461	[mm2/m]
Maximum possible absorbable shear force for wall thickness	1.414	[kN/m]
Actual absorbable shear force by applied shear reinforcement	288	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion for bending	**YES**	[-]
Satisfaction of minimum reinforcement criterion for tension	**YES**	[-]

LOWER CAISSON		
Vertical longitudinal outer walls on the land side of lower caisson		
Maximum net pressure acting from outer side towards inner side*	204	[kN/m2]
Maximum shear force in cross section	587	[kN/m]
Maximum positive bending moment in x-direction (100m length)	226	[kNm/m]
Maximum negative bending moment in x-direction (100m length)	-547	[kNm/m]
Maximum positive bending moment in y-direction (11m length)	144	[kNm/m]
Maximum negative bending moment in y-direction (11m length)	-577	[kNm/m]
Chosen wall thickness	**550**	[mm]
Reinforcement bar diameter in x-direction (100m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) inner side	**260**	[mm]
Maximum absorbable momentum in x-direction (100m length) inner side	553	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) inner side	**245**	[mm]
Maximum absorbable momentum in y-direction (11m length) inner side	587	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) inner side	0,562	[%]
Actual reinforcement percentage in y-direction (11m length) inner side	0,597	[%]
Reinforcement bar diameter in x-direction (100m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) outer side	**250**	[mm]
Maximum absorbable momentum in x-direction (100m length) outer side	228	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) outer side	**285**	[mm]
Maximum absorbable momentum in y-direction (11m length) outer side	200	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) outer side	0,228	[%]
Actual reinforcement percentage in y-direction (11m length) outer side	0,200	[%]
Minimum reinforcement percentage in both directions	0,2	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**250**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	804	[mm2/m]
Maximum allowable amount of shear reinforcement	3.036	[mm2/m]
Maximum possible absorbable shear force for wall thickness	2.222	[kN/m]
Actual absorbable shear force by applied shear reinforcement	588	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length)outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion	**YES**	[-]

Vertical longitudinal outer walls on the sea side of lower caisson		
Maximum net pressure acting from outer side towards inner side*	133	[kN/m2]
Maximum shear force in cross section	332	[kN/m]
Maximum positive bending moment in x-direction (100m length)	99	[kNm/m]
Maximum negative bending moment in x-direction (100m length)	-285	[kNm/m]
Maximum positive bending moment in y-direction (11m length)	70	[kNm/m]
Maximum negative bending moment in y-direction (11m length)	-285	[kNm/m]
Chosen wall thickness	**400**	[mm]
Reinforcement bar diameter in x-direction (100m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) inner side	**210**	[mm]
Maximum absorbable momentum in x-direction (100m length) inner side	292	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) inner side	**210**	[mm]
Maximum absorbable momentum in y-direction (11m length) inner side	292	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) inner side	0,584	[%]
Actual reinforcement percentage in y-direction (11m length) inner side	0,584	[%]
Reinforcement bar diameter in x-direction (100m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in x-direction (100m length) outer side	**250**	[mm]
Maximum absorbable momentum in x-direction (100m length) outer side	102	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) outer side	**250**	[mm]
Maximum absorbable momentum in y-direction (11m length) outer side	102	[kNm/m]
Actual reinforcement percentage in x-direction (100m length) outer side	0,201	[%]
Actual reinforcement percentage in y-direction (11m length) outer side	0,201	[%]
Minimum reinforcement percentage in both directions	0,2	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**280**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	718	[mm2/m]
Maximum allowable amount of shear reinforcement	3.400	[mm2/m]
Maximum possible absorbable shear force for wall thickness	1.616	[kN/m]
Actual absorbable shear force by applied shear reinforcement	341	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (100m length)outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion	**YES**	[-]
Vertical transversal outer walls of lower caisson		
Maximum net pressure acting from outer side towards inner side**	204	[kN/m2]

Maximum shear force in cross section	587	[kN/m]
Maximum positive bending moment in x-direction (33m length)	226	[kNm/m]
Maximum negative bending moment in x-direction (33m length)	-547	[kNm/m]
Maximum positive bending moment in y-direction (11m length)	144	[kNm/m]
Maximum negative bending moment in y-direction (11m length)	-577	[kNm/m]
Chosen wall thickness	**550**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**260**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	553	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) inner side	**245**	[mm]
Maximum absorbable momentum in y-direction (11m length) inner side	587	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,562	[%]
Actual reinforcement percentage in y-direction (11m length) inner side	0,597	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**250**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	228	[kNm/m]
Reinforcement bar diameter in y-direction (11m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in y-direction (11m length) outer side	**285**	[mm]
Maximum absorbable momentum in y-direction (11m length) outer side	200	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,228	[%]
Actual reinforcement percentage in y-direction (11m length) outer side	0,200	[%]
Minimum reinforcement percentage in both directions	0,2	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**250**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	804	[mm2/m]
Maximum allowable amount of shear reinforcement	3.036	[mm2/m]
Maximum possible absorbable shear force for wall thickness	2.222	[kN/m]
Actual absorbable shear force by applied shear reinforcement	588	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (11m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion	**YES**	[-]
Bottom slab of lower caisson		
Maximum net pressure acting from outer side towards inner side**	133	[kN/m2]
Maximum shear force in cross section	444	[kN/m]
Maximum positive bending moment in x-direction (33m length)	106	[kNm/m]

Maximum negative bending moment in x-direction (33m length)	-374	[kNm/m]
Maximum positive bending moment in y-direction (100m length)	132	[kNm/m]
Maximum negative bending moment in y-direction (100m length)	-374	[kNm/m]
Chosen wall thickness	**550**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**255**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	564	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) inner side	**32**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) inner side	**255**	[mm]
Maximum absorbable momentum in y-direction (100m length) inner side	564	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,573	[%]
Actual reinforcement percentage in y-direction (100m length) inner side	0,573	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**285**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	200	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) outer side	**20**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) outer side	**285**	[mm]
Maximum absorbable momentum in y-direction (100m length) outer side	200	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,200	[%]
Actual reinforcement percentage in y-direction (100m length) outer side	0,200	[%]
Minimum reinforcement percentage in x-direction (33m length)	0,2	[%]
Minimum reinforcement percentage in y-direction (100m length, both sides)	0,27	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**235**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	856	[mm2/m]
Maximum allowable amount of shear reinforcement	2.854	[mm2/m]
Maximum possible absorbable shear force for wall thickness	2.222	[kN/m]
Actual absorbable shear force by applied shear reinforcement	666	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion for bending	**YES**	[-]
Satisfaction of minimum reinforcement criterion for tension	**YES**	[-]
Top slab of lower caisson		
Maximum net pressure acting from outer side towards inner side	102	[kN/m2]
Maximum shear force in cross section	341	[kN/m]
Maximum positive bending moment in x-direction (33m length)	81	[kNm/m]

Maximum negative bending moment in x-direction (33m length)	-288	[kNm/m]
Maximum positive bending moment in y-direction (100m length)	104	[kNm/m]
Maximum negative bending moment in y-direction (100m length)	-288	[kNm/m]
Chosen wall thickness	**400**	[mm]
Reinforcement bar diameter in x-direction (33m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) inner side	**210**	[mm]
Maximum absorbable momentum in x-direction (33m length) inner side	292	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) inner side	**25**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) inner side	**210**	[mm]
Maximum absorbable momentum in y-direction (100m length) inner side	292	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) inner side	0,584	[%]
Actual reinforcement percentage in y-direction (100m length) inner side	0,584	[%]
Reinforcement bar diameter in x-direction (33m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in x-direction (33m length) outer side	**250**	[mm]
Maximum absorbable momentum in x-direction (33m length) outer side	102	[kNm/m]
Reinforcement bar diameter in y-direction (100m length) outer side	**16**	[mm]
Center to center distance of reinforcement bars in y-direction (100m length) outer side	**240**	[mm]
Maximum absorbable momentum in y-direction (100m length) outer side	106	[kNm/m]
Actual reinforcement percentage in x-direction (33m length) outer side	0,201	[%]
Actual reinforcement percentage in y-direction (100m length) outer side	0,209	[%]
Minimum reinforcement percentage in x-direction (33m length)	0,2	[%]
Minimum reinforcement percentage in y-direction (100m length, both sides)	0,37	[%]
Shear reinforcement diameter	**16**	[mm]
Shear reinforcement center to center distance	**280**	[mm]
Angle of compressive diagonals (theta)	45	[degree]
Angle of shear reinforcement bars (alpha)	90	[degree]
Applied amount of shear reinforcement	718	[mm2/m]
Maximum allowable amount of shear reinforcement	3.400	[mm2/m]
Maximum possible absorbable shear force for wall thickness	1.616	[kN/m]
Actual absorbable shear force by applied shear reinforcement	341	[kN/m]
Satisfaction of resistance against shear force	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) inner side	**YES**	[-]
Satisfaction of resistance against bending moment in x-direction (33m length) outer side	**YES**	[-]
Satisfaction of resistance against bending moment in y-direction (100m length) outer side	**YES**	[-]
Satisfaction of minimum reinforcement criterion for bending	**YES**	[-]
Satisfaction of minimum reinforcement criterion for tension	**YES**	[-]
Inner walls		
Chosen wall thickness	**300**	[mm]
Reinforcement bar diameter in x-direction	**12**	[mm]

Center to center distance of reinforcement bars in x-direction	**180**	[mm]
Actual reinforcement percentage in x-direction	0,209	[%]
Reinforcement bar diameter in y-direction	**12**	[mm]
Center to center distance of reinforcement bars in y-direction	**180**	[mm]
Actual reinforcement percentage in y-direction	0,209	[%]
Minimum reinforcement percentage	0,2	[%]
Satisfaction of minimum reinforcement criterion	**YES**	[-]

Appendix R: Optimized wall thickness for material costs

Optimization of slab thickness and material costs – outer walls of upper caisson			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 209,32	€ 164,32	€ 45,00
350	€ 189,86	€ 137,36	€ 52,50
400	€ 182,59	€ 122,59	€ 60,00
450	€ 170,18	€ 102,68	€ 67,50
500	€ 162,24	€ 87,24	€ 75,00
550	**€ 160,84**	**€ 78,34**	**€ 82,50**
600	€ 161,68	€ 71,68	€ 90,00
650	€ 163,37	€ 65,87	€ 97,50
700	€ 167,63	€ 62,63	€ 105,00
750	€ 175,78	€ 63,28	€ 112,50
800	€ 188,89	€ 68,89	€ 120,00
850	€ 199,50	€ 72,00	€ 127,50
900	€ 211,54	€ 76,54	€ 135,00

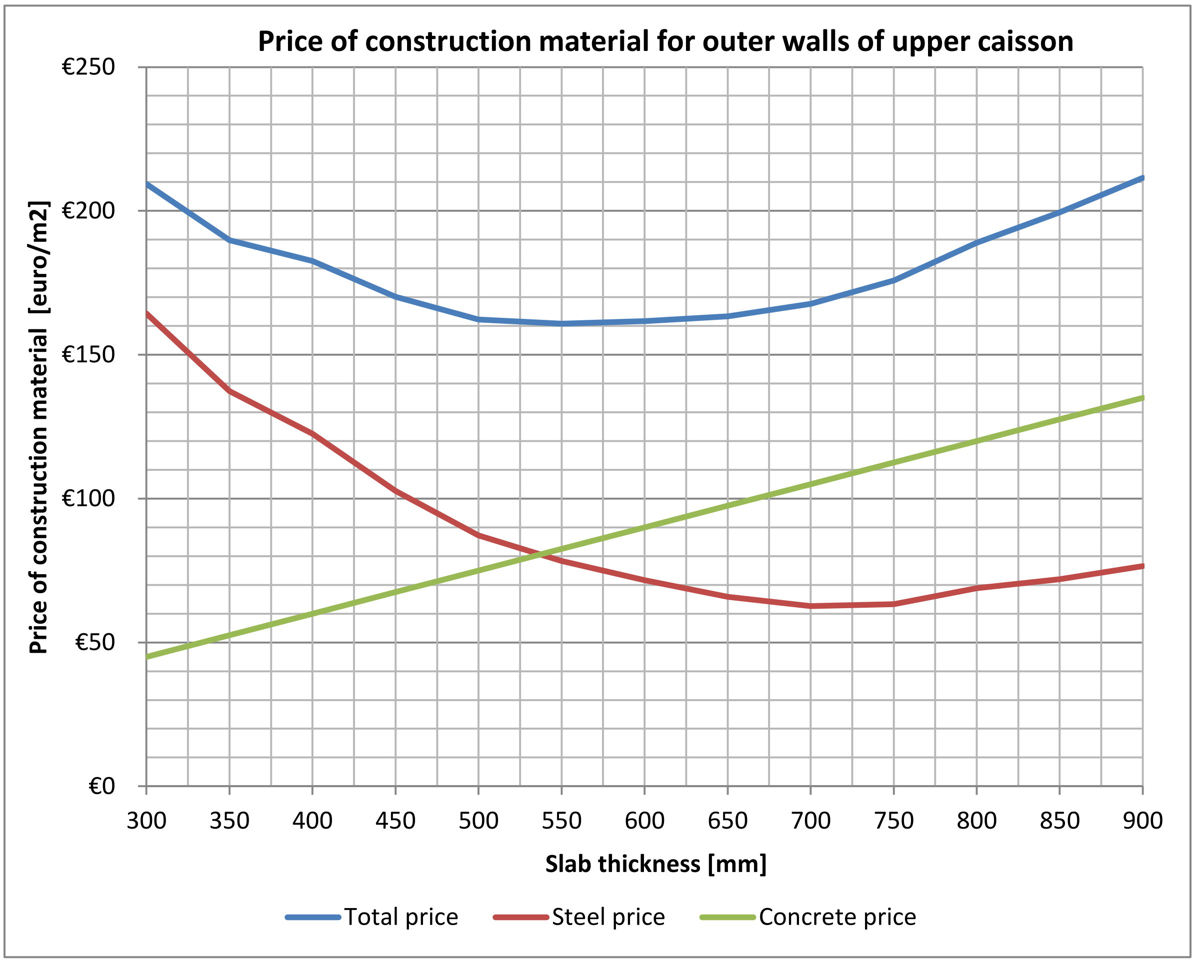


Figure R.1 – optimized wall thickness and material costs of outer walls of upper caisson

Optimization of slab thickness and material costs - bottom slab of upper caisson			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 180,10	€ 135,10	€ 45,00
350	€ 165,17	€ 112,67	€ 52,50
400	€ 151,19	€ 91,19	€ 60,00
450	**€ 150,30**	**€ 82,80**	**€ 67,50**
500	€ 152,70	€ 77,70	€ 75,00
550	€ 155,95	€ 73,45	€ 82,50
600	€ 160,63	€ 70,63	€ 90,00
650	€ 164,11	€ 66,61	€ 97,50
700	€ 171,24	€ 66,24	€ 105,00
750	€ 178,78	€ 66,28	€ 112,50
800	€ 188,89	€ 68,89	€ 120,00
850	€ 200,63	€ 73,13	€ 127,50
900	€ 211,54	€ 76,54	€ 135,00

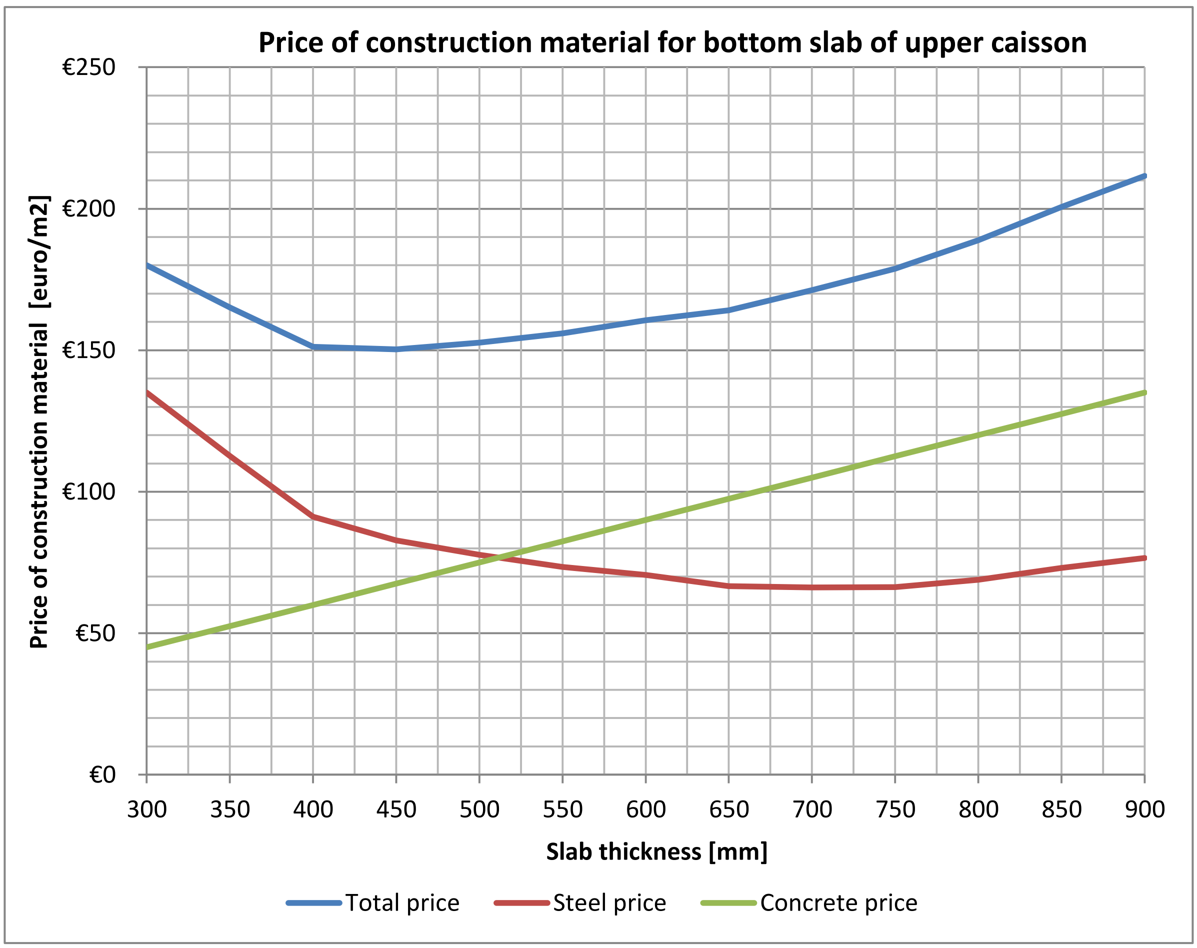


Figure R.2 – optimized wall thickness and material costs of bottom slab of upper caisson

Optimization of slab thickness and material costs - top slab of upper caisson			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 128,26	€ 83,26	€ 45,00
350	**€ 122,64**	**€ 70,14**	**€ 52,50**
400	€ 123,14	€ 63,14	€ 60,00
450	€ 127,20	€ 59,70	€ 67,50
500	€ 131,25	€ 56,25	€ 75,00
550	€ 135,71	€ 53,21	€ 82,50
600	€ 142,44	€ 52,44	€ 90,00
650	€ 153,45	€ 55,95	€ 97,50
700	€ 165,36	€ 60,36	€ 105,00
750	€ 176,73	€ 64,23	€ 112,50
800	€ 187,03	€ 67,03	€ 120,00
850	€ 198,77	€ 71,27	€ 127,50
900	€ 209,84	€ 74,84	€ 135,00

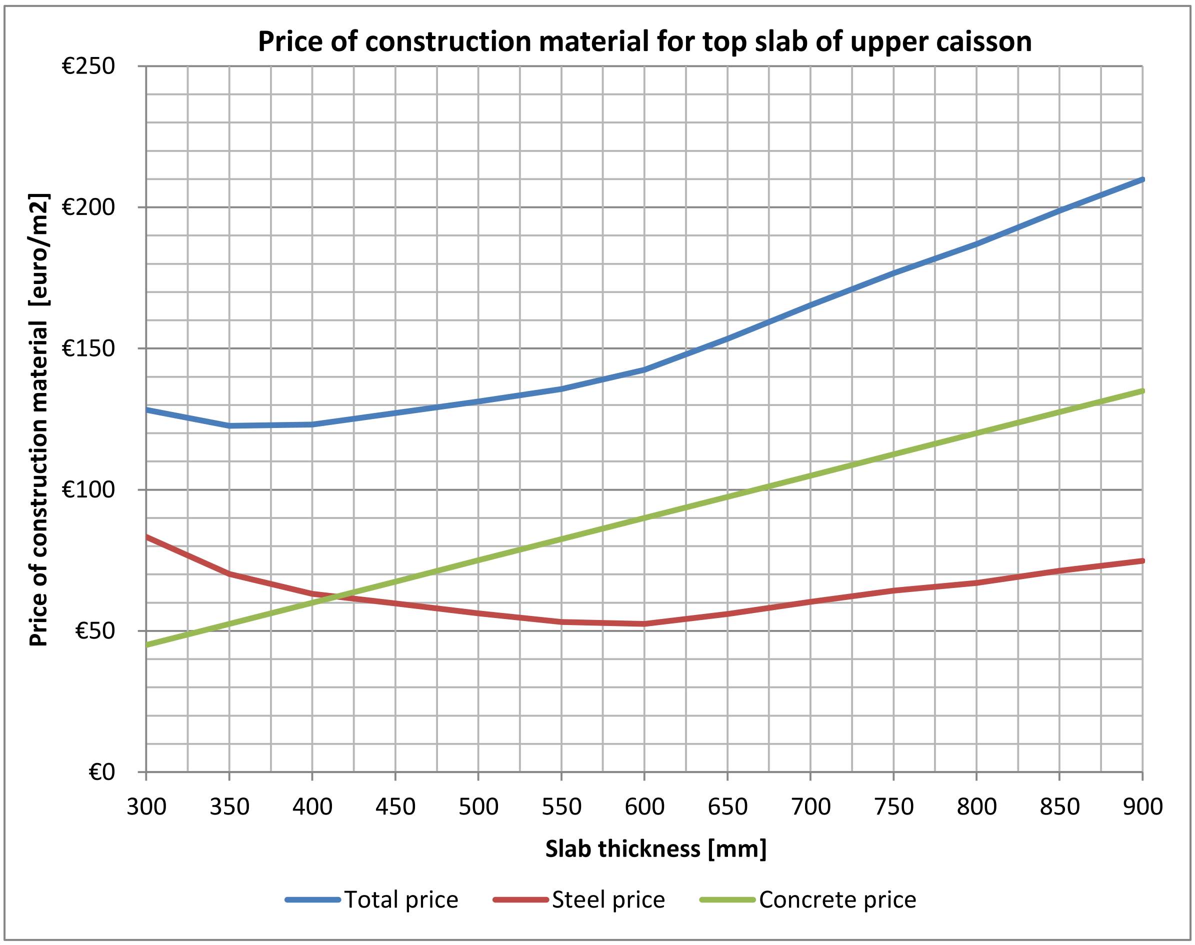


Figure R.3 – optimized wall thickness and material costs of top slab of upper caisson

Optimization of slab thickness and material costs - outer walls of lower caisson on land side			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 241,37	€ 196,37	€ 45,00
350	€ 215,60	€ 163,10	€ 52,50
400	€ 198,24	€ 138,24	€ 60,00
450	€ 187,21	€ 119,71	€ 67,50
500	€ 183,48	€ 108,48	€ 75,00
550	**€ 181,95**	**€ 99,45**	**€ 82,50**
600	€ 183,22	€ 93,22	€ 90,00
650	€ 183,81	€ 86,31	€ 97,50
700	€ 188,43	€ 83,43	€ 105,00
750	€ 194,42	€ 81,92	€ 112,50
800	€ 200,00	€ 80,00	€ 120,00
850	€ 206,60	€ 79,10	€ 127,50
900	€ 213,64	€ 78,64	€ 135,00

Figure R.4 – optimized wall thickness and material costs of outer walls of lower caisson on the land side

Optimization of slab thickness and material costs - outer walls lower caisson on sea side			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 143,87	€ 98,87	€ 45,00
350	€ 134,45	€ 81,95	€ 52,50
400	**€ 132,83**	**€ 72,83**	**€ 60,00**
450	€ 134,62	€ 67,12	€ 67,50
500	€ 136,76	€ 61,76	€ 75,00
550	€ 141,56	€ 59,06	€ 82,50
600	€ 147,45	€ 57,45	€ 90,00
650	€ 154,22	€ 56,72	€ 97,50
700	€ 166,11	€ 61,11	€ 105,00
750	€ 178,59	€ 66,09	€ 112,50
800	€ 187,74	€ 67,74	€ 120,00
850	€ 199,18	€ 71,68	€ 127,50
900	€ 210,44	€ 75,44	€ 135,00

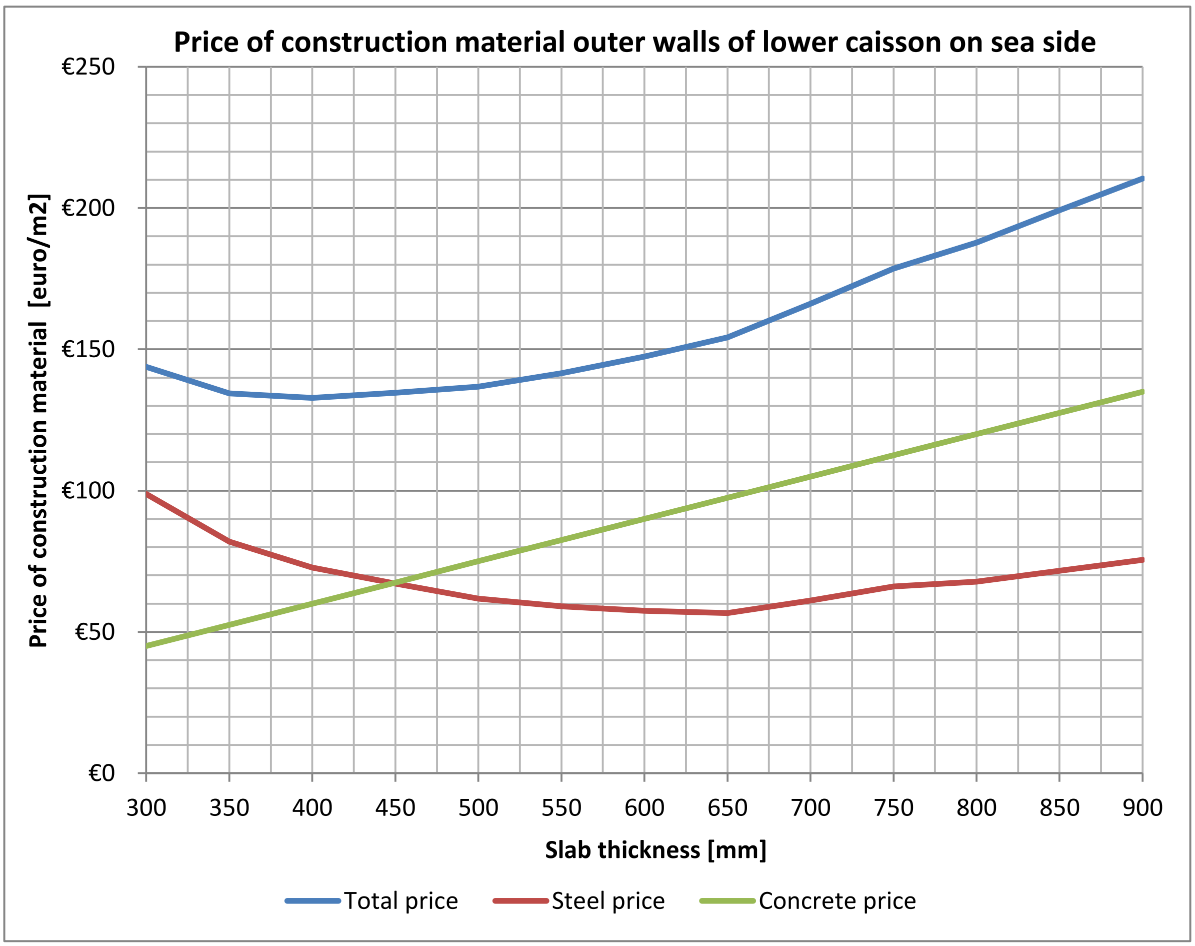


Figure R.5 – optimized wall thickness and material costs of outer walls of lower caisson on the sea side

Optimization of slab thickness and material costs - bottom slab of lower caisson*			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 251,00	€ 206,00	€ 45,00
350	€ 220,79	€ 168,29	€ 52,50
400	€ 204,74	€ 144,74	€ 60,00
450	€ 188,02	€ 120,52	€ 67,50
500	€ 182,95	€ 107,95	€ 75,00
550	**€ 181,41**	**€ 98,91**	**€ 82,50**
600	€ 184,34	€ 94,34	€ 90,00
650	€ 187,14	€ 89,64	€ 97,50
700	€ 192,37	€ 87,37	€ 105,00
750	€ 198,30	€ 85,80	€ 112,50
800	€ 203,76	€ 83,76	€ 120,00
850	€ 210,33	€ 82,83	€ 127,50
900	€ 216,23	€ 81,23	€ 135,00

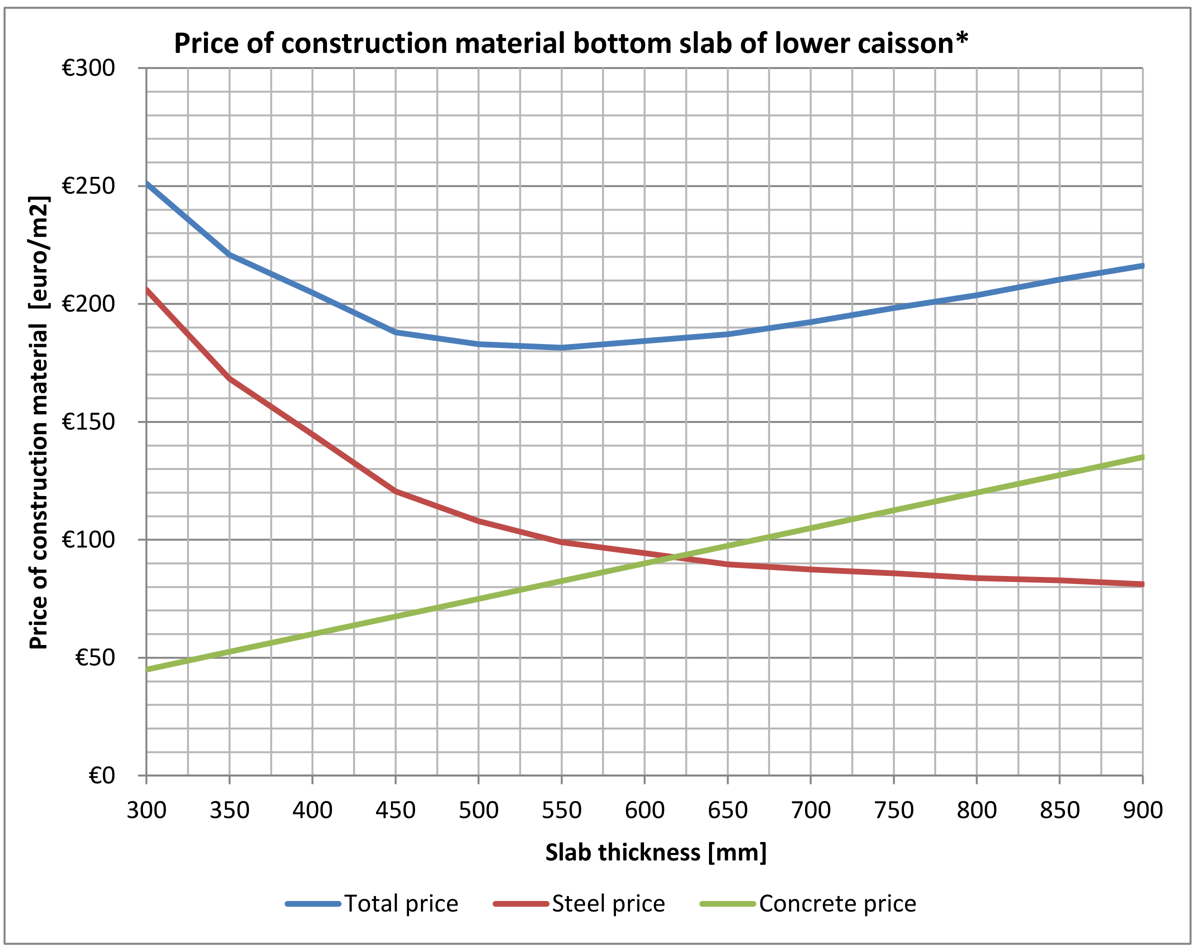


Figure R.6 – optimized wall thickness and material costs of bottom slab of lower caisson

* = over dimensioned by factor 1,5 for possible irregularities in gravel bed foundation

Optimization of slab thickness and material costs - top slab of lower caisson			
Slab thickness [mm]	Total price [€/m2]	Steel price [€/m2]	Concrete price [€/m2]
300	€ 146,39	€ 101,39	€ 45,00
350	€ 136,92	€ 84,42	€ 52,50
400	**€ 133,16**	**€ 73,16**	**€ 60,00**
450	€ 135,75	€ 68,25	€ 67,50
500	€ 139,06	€ 64,06	€ 75,00
550	€ 142,10	€ 59,60	€ 82,50
600	€ 148,31	€ 58,31	€ 90,00
650	€ 154,95	€ 57,45	€ 97,50
700	€ 166,22	€ 61,22	€ 105,00
750	€ 177,51	€ 65,01	€ 112,50
800	€ 187,74	€ 67,74	€ 120,00
850	€ 199,42	€ 71,92	€ 127,50
900	€ 210,44	€ 75,44	€ 135,00

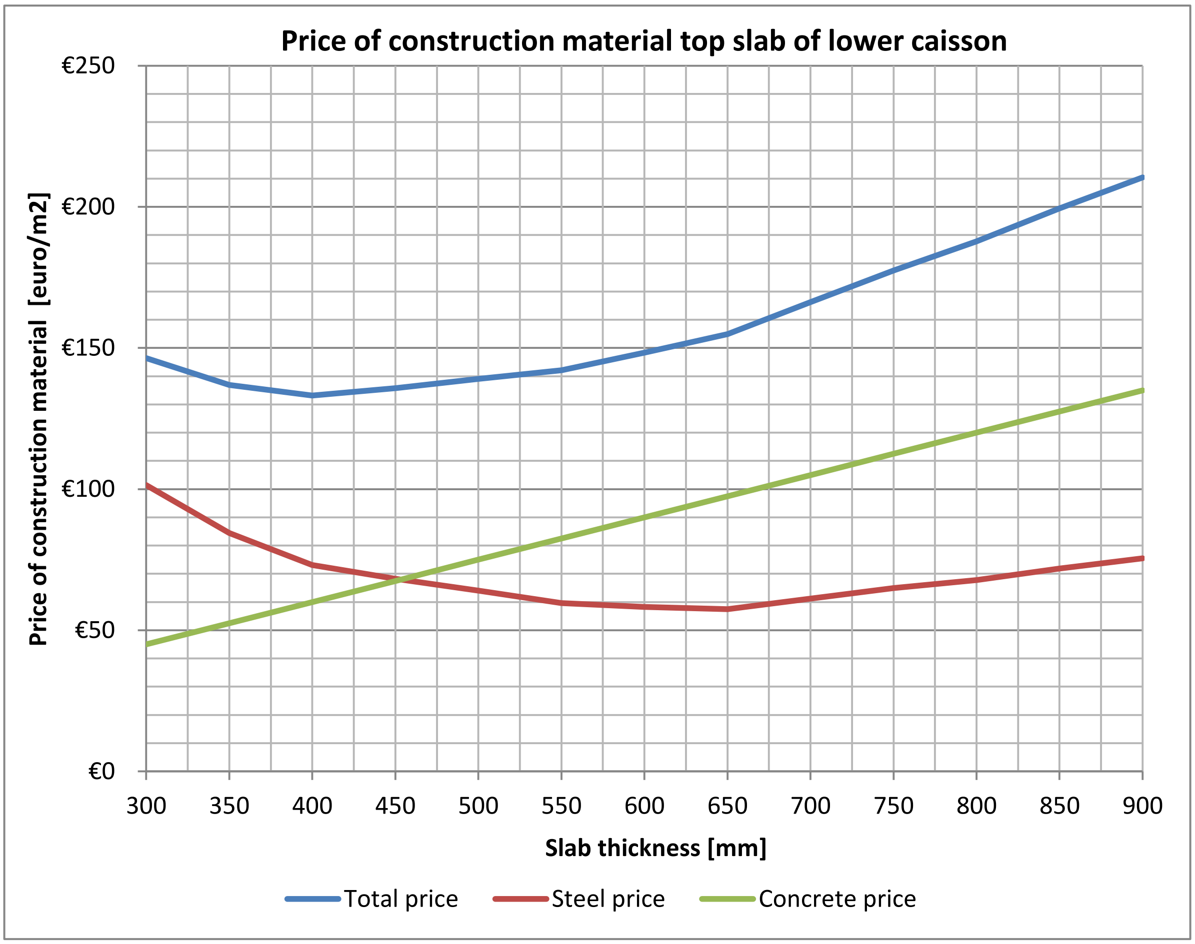


Figure R.7 – optimized wall thickness and material costs of top slab of lower caisson

Appendix S: Required wall thickness without shear reinforcement – Excel sheet

Bottom slab upper and lower caisson		
Minimum required slab thickness without shear reinforcement	**1140**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]
Absorbable shear force without shear reinforcement	**444**	[kN/m]
Lower limit of absorbable shear force without reinforcement	69	[kN/m]
Shear stress in cross section	0,39	[N/mm2]
Top slab upper caisson		
Minimum required slab thickness without shear reinforcement	**660**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]
Absorbable shear force without shear reinforcement	**281**	[kN/m]
Lower limit of absorbable shear force without reinforcement	46	[kN/m]
Shear stress in cross section	0,43	[N/mm2]
Top slab lower caisson		
Minimum required slab thickness without shear reinforcement	**835**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]
Absorbable shear force without shear reinforcement	**342**	[kN/m]
Lower limit of absorbable shear force without reinforcement	54	[kN/m]
Shear stress in cross section	0,41	[N/mm2]
Vertical outer walls of upper caisson		
Minimum required slab thickness without shear reinforcement	**1115**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]
Absorbable shear force without shear reinforcement	**436**	[kN/m]
Lower limit of absorbable shear force without reinforcement	68	[kN/m]
Shear stress in cross section	0,39	[N/mm2]
Vertical outer walls of lower caisson at land side		
Minimum required slab thickness without shear reinforcement	**1575**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]

Absorbable shear force without shear reinforcement	**587**	[kN/m]
Lower limit of absorbable shear force without reinforcement	89	[kN/m]
Shear stress in cross section	0,37	[N/mm2]
Vertical outer walls of lower caisson at sea side		
Minimum required slab thickness without shear reinforcement	**810**	[mm]
Concrete compressive strength	30	[N/mm2]
Total reinforcement percentage	0,004	[-]
material factor for concrete	1,5	[-]
Absorbable shear force without shear reinforcement	**333**	[kN/m]
Lower limit of absorbable shear force without reinforcement	53	[kN/m]
Shear stress in cross section	0,41	[N/mm2]

Figure S.1 – required wall thickness and shear force without shear reinforcement

Appendix T: Feasibility and comparison with traditional quay – Excel sheet

Flexible quay wall structure (initial construction costs)		
Amount of concrete used	92.500	[m3]
Amount of reinforcement steel used	10.000.000	[kg]
Amount of framework for casting concrete top slabs per caisson	3.000	[m2]
Amount of framework for casting other concrete sections per caisson	23.500	[m2]
Concrete price per cubic meter	150	[€/m3]
Reinforcement steel price per kg	1,25	[€/kg]
Framework price per square meter for top slabs	100	[€/m2]
Framework price per square meter for other sections	200	[€/m2]
Total concrete price	13.875.000	[€]
Total reinforcement steel price	12.500.000	[€]
Total framework price	2.950.000	[€]
Total price of construction material	29.325.000	[€]
Price of construction dock	900.000	[€]
Tug boat mobilization costs	280.000	[€]
Transport costs per nautical mile	1.000	[€/nm]
Initial sailing distance	**300**	[nm]
Total transport costs	580.000	[€]
CSD (de)mobilization costs	800.000	[€]
Dredging works	435.000	[m3]
Price per cubic meter	5	[€/m3]
Total price of dredging works	2.975.000	[€]
Amount of bed protection against scour holes	45.000	[ton]
Price of bed protection per ton	25	[€/ton]
Total price of bed protection	1.125.000	[€]
Amount of back fill material (sand)	250.000	[m3]
Price of back fill material (sand) per cubic meter	5	[€/m3]
Total price of back fill material	1.250.000	[€]
Amount of gravel for back fill	300.000	[m3]
Price of gravel material per cubic meter	15	[€/m3]
Total price of gravel material	4.500.000	[€]
Amount of rubble material for caisson foundation	40.000	[ton]
Price of rubble material per ton	20	[€/ton]
Total price of rubble material for caisson foundation	800.000	[€]
Amount of geotextile filter layer	80.000	[m2]
Price of geotextile per square meter	12	[€/m2]
Total price of geotextile	960.000	[€]
Total price of bollards	180.000	[€]

Total price of fenders	1.800.000	[€]
Total price of equipment for immersing the caissons	1.400.000	[€]
Initial quay wall costs	**45.795.000**	[€]
Initial costs per meter quay wall	65.892	[€]
Flexible quay wall structure (relocation costs)		
Relocation distance	**1.000**	[nm]
De-ballasting and mobilizing caissons	1.400.000	[€]
STS crane mobilization costs	800.000	[€]
Amount of excavation works	350.000	[m3]
Price per cubic meter excavation works	5	[€/m3]
Total costs of excavation works before de-ballasting	1.750.000	[€]
Total relocation costs	**18.240.000**	[€]
Initial construction time	**2,5**	[years]
Relocation time	**0,5**	[years]
Utilized period	**9,5**	[years]
Initial utilized period	**7,5**	[years]

Fixed quay wall structure		
Construction costs per meter quay wall	40.000	[€]
Construction costs for entire quay length	28.000.000	[€]
Demolition costs	2.800.000	[€]
Total costs	**30.800.000**	[€]
Utilized period	**8,5**	[years]
Construction period	**1,5**	[years]

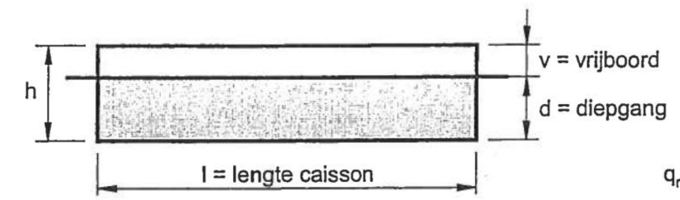
Net profit (equal for both structures)	4.000.000	[€/year]

Appendix U: Bending moment on entire caisson during transport

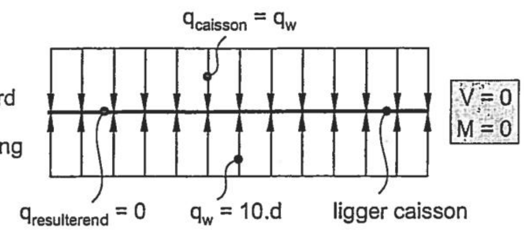
Golfbelastingen op caisson **1.12.2**

Opmerking:
T.b.v. indicatie golfkrachten
is aangehouden: lengte caisson l = golflengte L

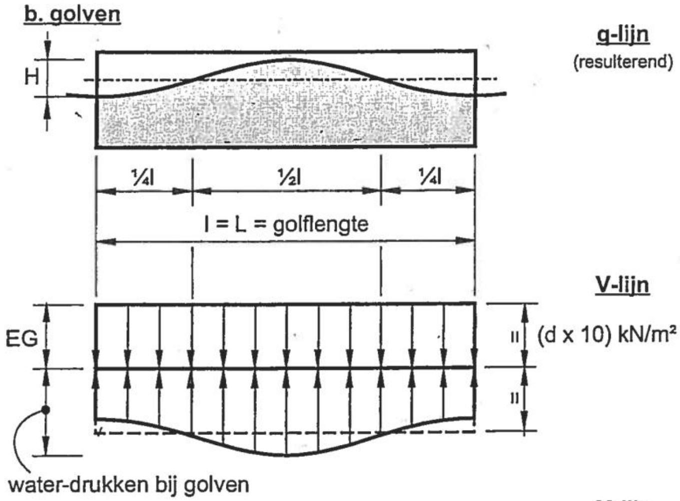
a. geen golven

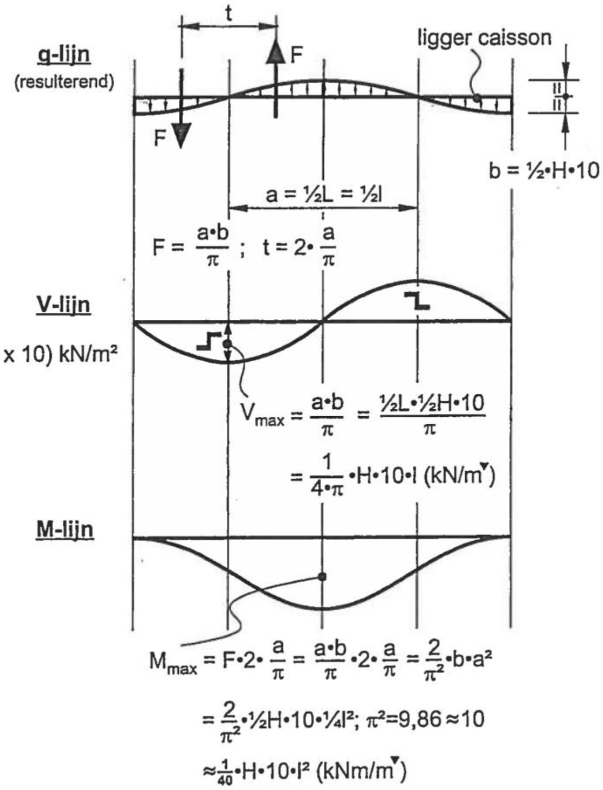


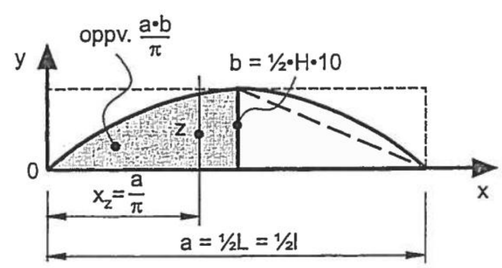
Mechanica-modellen



b. golven







sinusfunctie: $y = b \cdot \sin \frac{\pi}{a} x$

oppervlak: $O = \frac{a \cdot b}{\pi}$

zwaartepunt: $x_z = \frac{a}{\pi}$

- **sinusvormige golf:** $F = \frac{a \cdot b}{\pi}$; $t = 2 \cdot \frac{a}{\pi}$

  $M_{max} = F \cdot t = \frac{2}{\pi^2} \cdot b \cdot a^2$

- **rechthoekige golf:** $F = ½ \cdot a \cdot b$; $t = ½ \cdot a$

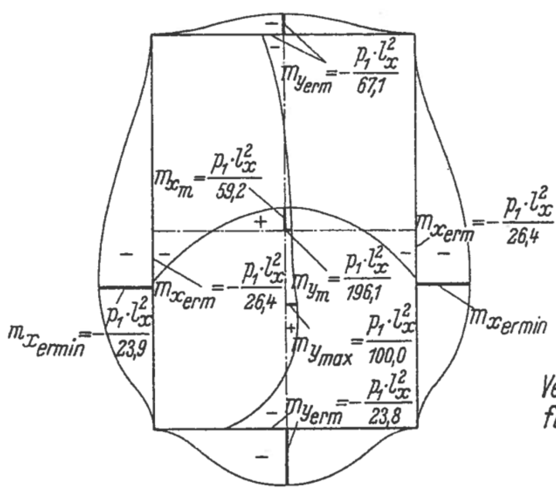
  $M_{max} = ¼ \cdot b \cdot a^2 = \frac{1}{32} \cdot H \cdot 10 \cdot l^2$

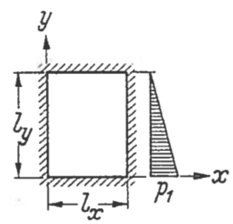
- **driehoekige golf:** $F = ¼ \cdot a \cdot b$; $t = \frac{2}{3} \cdot a$

  $M_{max} = \frac{1}{6} \cdot b \cdot a^2 = \frac{1}{48} \cdot H \cdot 10 \cdot l^2$

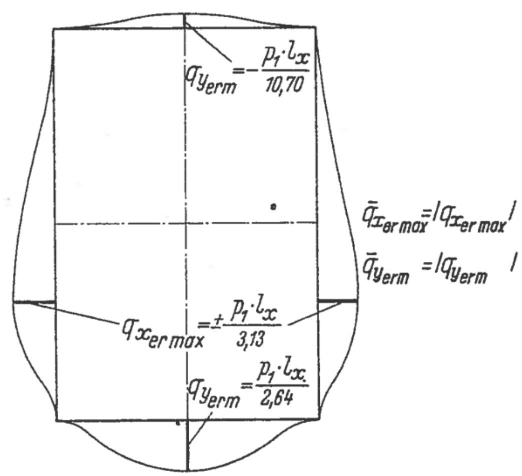
Appendix V: Shear stress and bending moment in caisson walls

$l_y : l_x$			1,0	1,1	1,2	1,3	1,4	1,5	2,0
$m_{x\,\mathrm{ermin}}$	$= -$	$p_1 \cdot l_x^2$:	36,9	33,1	29,8	27,5	25,6	23,9	20,2
$m_{x\mathrm{m}}$	$=$	$p_1 \cdot l_x^2$:	113,6	91,7	78,7	69,9	63,7	59,2	50,0
$m_{y\,\mathrm{erm}\,(y=0)}$	$= -$	$p_1 \cdot l_x^2$:	30,0	27,5	26,1	25,0	24,1	23,8	21,9
$m_{y\,\mathrm{erm}\,(y=l_y)}$	$= -$	$p_1 \cdot l_x^2$:	56,2	55,5	57,2	59,5	62,9	67,1	92,5
$m_{y\max}$	$=$	$p_1 \cdot l_x^2$:	98,0	98,0	98,0	99,0	99,0	100,0	100,0
$q_{x\,\mathrm{ermax}\,(x=0\ \mathrm{u.}\ l_x)}$	$= \pm$	$p_1 \cdot l_x$:	4,14	3,86	3,59	3,39	3,25	3,13	2,83
$q_{y\,\mathrm{erm}\,(y=0)}$	$=$	$p_1 \cdot l_x$:	3,07	2,90	2,80	2,74	2,70	2,64	2,49
$q_{y\,\mathrm{erm}\,(y=l_y)}$	$= -$	$p_1 \cdot l_x$:	8,25	8,40	8,77	9,40	9,99	10,70	14,29
f_m	$= \dfrac{p_1 \cdot l_x^4}{E \cdot d^3} \cdot$		0,0076	0,0091	0,0104	0,0115	0,0124	0,0132	0,0152



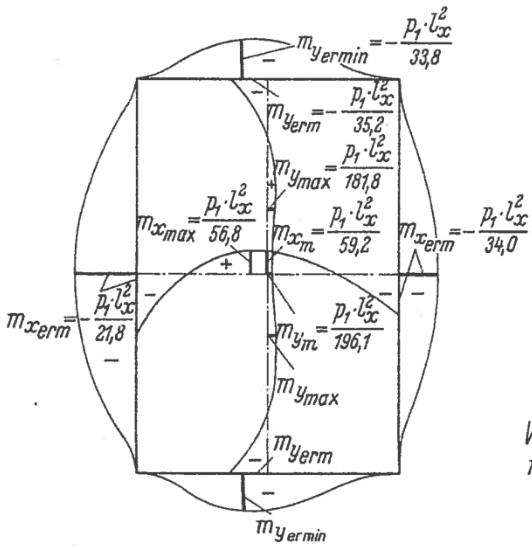


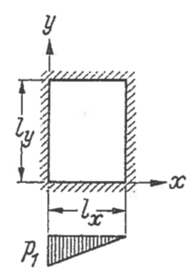
Verlauf der Schnittgrößen
für das Seitenverhältnis
$l_y : l_x = 1,5$



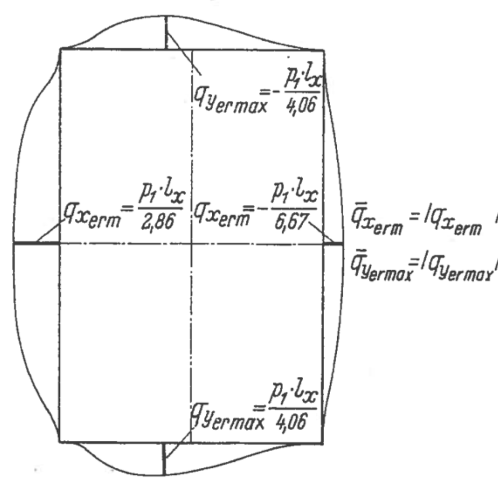
2.3.6 Starre Einspannung aller vier Ränder

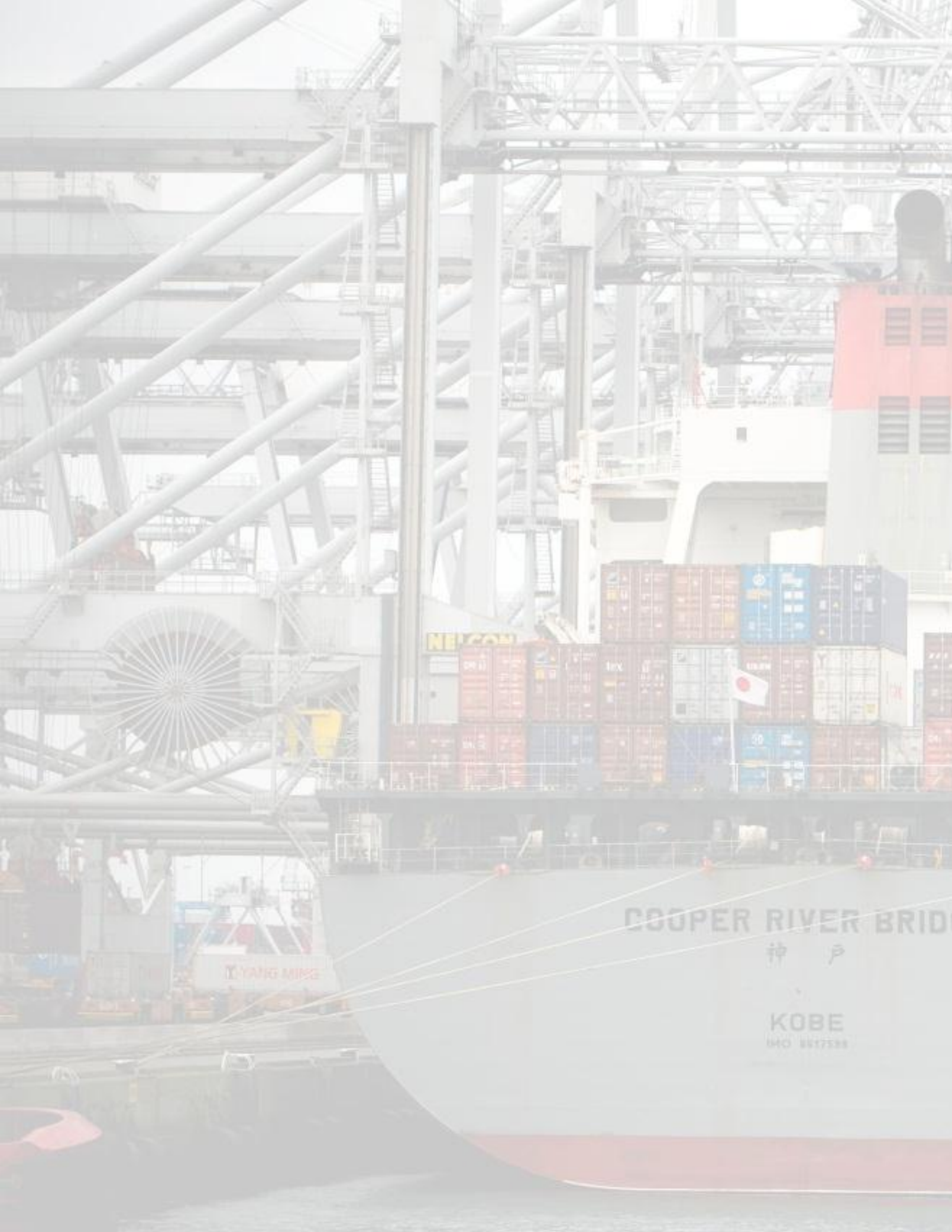
$l_y : l_x$			1,0	1,1	1,2	1,3	1,4	1,5	2,0
$m_{x\mathrm{erm}\,(x=0)}$	$= -$	$p_1 \cdot l_x^2 :$	30,0	26,7	24,7	23,3	22,2	21,8	20,2
$m_{x\mathrm{erm}\,(x=l_x)}$	$= -$	$p_1 \cdot l_x^2 :$	56,2	47,1	41,7	38,1	35,5	34,0	30,4
$m_{x\max}$	$=$	$p_1 \cdot l_x^2 :$	98,0	82,6	73,0	65,8	60,6	56,8	48,5
$m_{y\mathrm{ermin}}$	$= -$	$p_1 \cdot l_x^2 :$	36,9	36,0	35,1	34,6	34,4	33,8	33,8
$m_{y\max}$	$=$	$p_1 \cdot l_x^2 :$	113,6	120,4	131,5	147,0	166,6	181,8	212,6
$q_{x\mathrm{erm}\,(x=0)}$	$=$	$p_1 \cdot l_x :$	3,07	2,93	2,86	2,86	2,86	2,86	2,86
$q_{x\mathrm{erm}\,(x=l_x)}$	$= -$	$p_1 \cdot l_x :$	8,25	7,35	6,67	6,67	6,67	6,67	6,67
$q_{y\mathrm{ermax}\,(y=0\text{ u. }l_y)}$	$= \pm$	$p_1 \cdot l_x :$	4,14	4,12	4,08	4,07	4,07	4,06	4,08
f_m	$= \dfrac{p_1 \cdot l_x^4}{E \cdot d^3} \cdot$		0,0076	0,0091	0,0104	0,0115	0,0124	0,0132	0,0152





Verlauf der Schnittgrößen für das Seitenverhältnis $l_y : l_x = 1{,}5$




YANG MING
COOPER RIVER
神 戸
KOBE