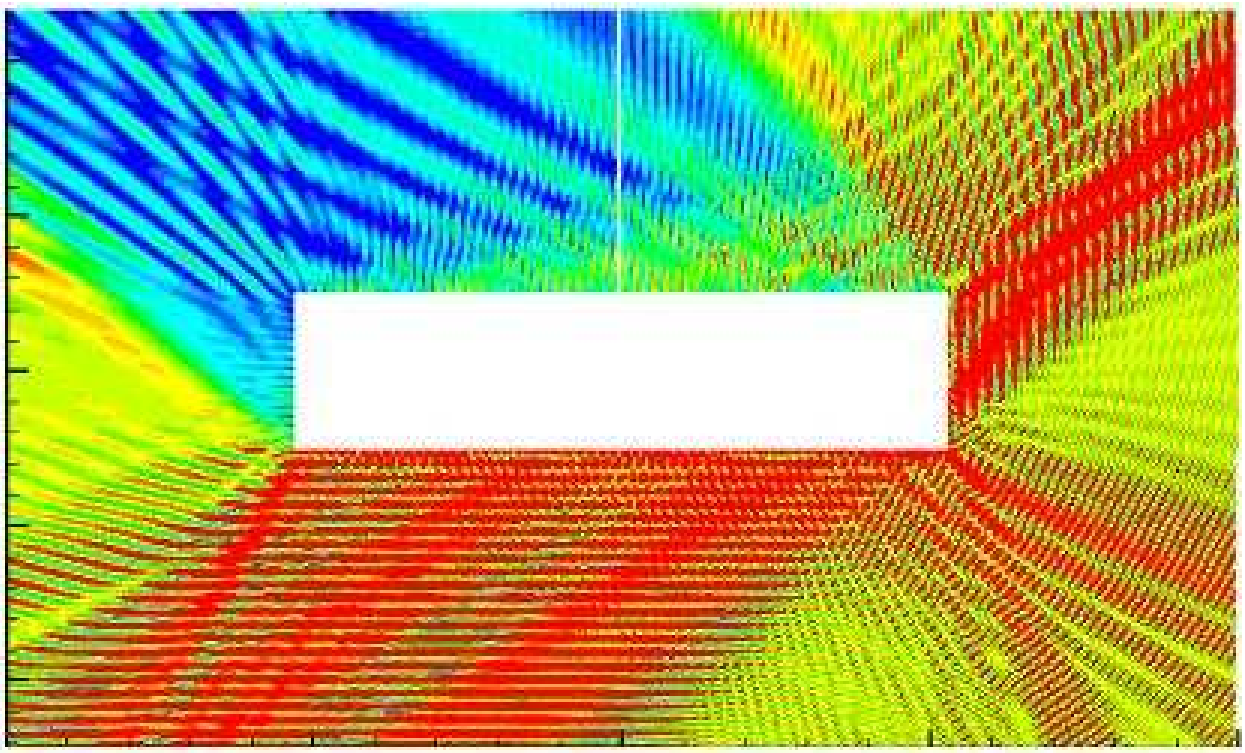
A Very Large Floating Container Terminal

Feasibility study

Application possibilities for the situation in the Netherlands



Master Thesis
Ing. G.V.P. de Rooij
April 2006

TU Delft

Van Hattum en Blankevoort

A Very Large Floating Container Terminal

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Master Thesis – Final version
Ing. G.V.P. de Rooij
April 2006

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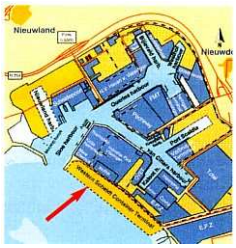
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Abstract: A Very Large Floating Container Terminal

By Gina de Rooij

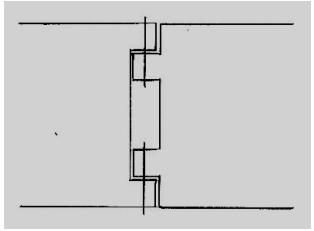


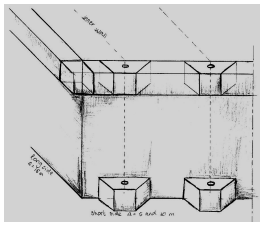
Floating structures may bring a flexible solution to changing requirements regarding the location, size and function of the port infrastructure. During this thesis the technical feasibility of a floating container terminal has been researched in co-operation with Van Hattum en Blankevoort. A conceptual design has been made of the floating container terminal and its floating elements. Special attention has been given to the hydrodynamics of the terminal and to the connections between the floating elements. The conceptual design is based on a case study of the Western Scheldt Container Terminal. This location has a tidal difference of about 4 m. Furthermore the location is characterized by strong tidal movements, currents and mainly short waves. The main loads on the terminal are wave loads, container storage loads and gantry crane track loads. The terminal may be moved to a near-shore location in the course of its lifetime.

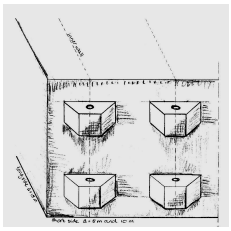
The terminal consists of 120 rectangular pontoon shaped elements with dimensions 180 * 50 * 16 m, resulting in a terminal of 2100 * 500 m. The pontoons are connected rigidly to each other in a stretching bond configuration. The concrete pontoons are composed of rectangular compartments. Inner walls are positioned underneath the crane tracks.

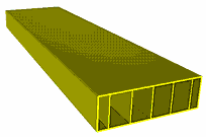
The hydrodynamic behaviour of the terminal and the single elements has been studied using the computer application DELFRAC. The terminal has been modelled as a rigid single-body platform. Response motions, wave forces, wave elevations round the terminal and connection forces between the pontoons have been determined with spectral analysis of the results from DELFRAC. There is a slight possibility that standing waves will occur in front of the terminal. The wave transmission towards the lee side of the terminal is small. The response motions of the terminal are very small due to the large dimensions of the structure.

The corner pontoon of the quay directly facing diagonal wave attack has to resist the largest loads. Therefore, this pontoon has been designed. The design is based on simple manual calculations of the internal forces of the pontoon. All structural parts will be prestressed in two directions. A rigid connection between the pontoons has been developed. The selected concept is a vertical needle vs. peg connection with discontinuous trapezoid concrete studs. The studs (ca. 3000*3000*2500 mm) are constructed on the sides of the pontoon and located on the inner walls.









The results from DELFRAC have been translated into static loads. The studs are prestressed with 6 curved tendons, which are distributed over the height of the stud. Consequently, no sensitive anchors are installed at the front of the connectors. The tendons introduce the connection forces into the inner walls and the top and bottom slab of the pontoon. The tendons are anchored at a cantilever beam behind an inner wall. The connection is tensioned by jacks, which are constructed at the sides of the pontoon. Thus, the pontoons can be easily disconnected. The pontoons will be towed from the construction dock to the site and coupled using a temporary structure. During coupling, the pontoon will be de-ballasted and towed into position horizontally.

The designed terminal is considered to be technically feasible in the Western Scheldt. The terminal is most suited for locations with shallow waters with restricted wave and wind loads.

Foreword

This Thesis, called A Very Large Floating Container Terminal, is written in order to receive the title of Master of Science in the field of Civil Engineering at Delft University of Technology. The project was carried out in cooperation with Van Hattum en Blankevoort. The report describes my research into large floating objects, the hydrodynamics of a large floating platform and the structural design of the connections between the floating elements of the terminal.

Van Hattum en Blankevoort provided me with a place to work and put at my disposal the required resources to carry out the project and the case study, for which I am very thankful. I would also like to thank my colleagues for their flexibility in working hours during the last month of my graduation project. Many persons supported me in different ways throughout the period of my graduation project. I would especially like to thank Hans Galjaard for his support and guidance. I would like to thank Jan van Kessel for all his time and his patience in explaining ship hydromechanics to me.
Furthermore I would like to thank my professors and mentor for their help and advice in their different areas of expertise:

- Ir. W.F. Molenaar
- Prof. drs. Ir. J.K. Vrijling
- Ir. W.J.M. Peperkamp
- Prof. dr. Ir. J.C. Walraven

Finally, I would like to thank my mother for helping me with the final corrections and Machiel Mastenbroek, my father and my sister for their never-ending moral support. I would definitely like to thank Machiel for his endless patience with me.

Gina de Rooij

Delft, January 2006



Abstract

Various research projects have been initiated around the world to study very large floating structures. A research project at Delft University of technology focuses on floating port structures. Floating structures may bring a flexible solution to changing requirements regarding the location, size and function of the port infrastructure. During this thesis the technical feasibility of a floating container terminal has been researched in co-operation with Van Hattum en Blankevoort. A conceptual design has been made of the floating container terminal and its floating elements. Special attention has been given to the hydrodynamics of the terminal and to the connections between the floating elements. The conceptual design is based on a case study of the Western Scheldt Container Terminal, which has been provided by Van Hattum en Blankevoort.

The terminal will be located in the estuary of the Western Scheldt, directly bordering on the existing port area of Vlissingen. This location in the Western Scheldt has a tidal difference of about 4 m. Furthermore the location is characterized by strong tidal movements, currents and mainly short waves with a maximum amplitude (1/100 yrs) of 2,37 m. The main loads on the terminal are wave loads, container storage loads and gantry crane track loads. The terminal may be moved to a near-shore location near the mouth of the Western Scheldt in the course of its lifetime.

The terminal will be connected to the shore by entrance bridges. It is moored by a large number of tubular piles with a sliding connection along the sides of the terminal. The terminal consists of 120 rectangular pontoon shaped elements with dimensions 180 * 50 * 16 m, resulting in a terminal of 2100 * 500 m. The pontoons are connected to each other to create a continuous deck structure for the terminal. The pontoons are connected in a stretching bond configuration. As a result, the connections will be well distributed over the platform. The freeboard of the terminal during operation should be larger than the height of the design wave. Therefore a draught of the pontoons of about 13,5 m will be necessary. The draught will be controlled by the loads on the terminal and by a ballast system. It is suggested to construct the pontoons in a construction dock on the 2nd Maasvlakte. Consequently, the pontoons have to be towed to the site in the Western Scheldt. The static stability of the rectangular pontoons is sufficient both during transport and during operation.

The concrete pontoons are composed of rectangular compartments. Inner walls are positioned underneath the crane tracks. The outer compartments therefore have dimensions of 15* 5 m and the inner compartments are 15*10 m.

The hydrodynamic behaviour of the terminal and the single elements has been studied using the computer application DELFRAC. The terminal has been modelled as a rigid single-body platform. The pontoons will be rigidly connected to each other. Response motions, wave forces, wave elevations round the terminal and connection forces between the pontoons have been determined with spectral analysis of the results from DELFRAC. There is a slight possibility that standing waves will occur in front of the terminal. The wave transmission towards the lee side of the terminal is small. The response motions of the terminal are very small due to the large dimensions of the structure. The wave loads and consequently the connection forces of the rigid connection between the pontoons are high. The response motions of a single pontoon (during transport) are also small.

The corner pontoon of the quay directly facing diagonal wave attack has to resist the largest loads. Therefore, this pontoon has been designed. The design is based on simple manual calculations of the internal forces of the pontoon. The internal forces have been calculated for the governing load combination during operation. The outer shell of the pontoon (walls and top and bottom slab) will have a thickness of about 800 mm. All structural parts will be prestressed in two directions. The slabs will be post-tensioned with straight tendons, which are located in the centre of the slabs.

A rigid connection between the pontoons has been developed. The best alternative has been selected from an extensive Multi Criteria Analysis. The selected concept is a vertical needle vs. peg connection with discontinuous trapezoid concrete studs. The studs are constructed on the sides of the pontoon and located on the inner walls. Two large studs will be locked at the top of the side and two studs will be locked at the bottom of the side of the pontoon by a peg. The top and bottom pegs are steel tubular piles constructed in the shape of a nail.

The results from DELFRAC have been translated into static loads. These design connection forces have been checked with calculations and have been distributed over the discrete connectors. The connector has been calculated, both the concrete stud and the steel peg. The studs are prestressed with 6 curved tendons, which are distributed over the height of the stud. Consequently, no sensitive anchors are installed at the front of the connectors. The large dimensions
(ca. 3000*3000*2500 mm) of the studs are necessary to resist the connection forces and the prestressing forces. Furthermore, the studs provide sufficient space for the prestressing steel. The tendons introduce the connection forces into the inner walls and the top and bottom slab of the pontoon. The inner walls of the outer compartment have a thickness of 1000 mm to provide sufficient space for the prestressing and for the splitting reinforcement. Large haunches enable a favourable profile of the tendons. The tendons are anchored at a cantilever beam behind an inner wall. The connection is tensioned by jacks, which are constructed at the sides of the pontoon. Thus, the pontoons can be easily disconnected.

The compartments of the pontoons will be constructed using a special rectangular framework for the entire compartment. The top slabs of the compartments will be prefabricated and installed upon the inner walls of the pontoon. The pontoons will be coupled with the help of towing boats and a temporary structure. The pontoon will be ballasted. During coupling, the pontoon will rise and the freeboard will increase. After that, the pontoon will be shoved into position horizontally. The connection will be locked by the steel peg, which will be lowered into the studs from the top.

The type of floating terminal that has been described in this thesis may be applied in other situations. However, application at the open ocean is not recommended. The terminal is most suited for shallow waters with restricted wave and wind loads. For deep but protected waters, another mooring system may be developed.
The designed terminal is considered to be technically feasible in the Western Scheldt.

List of abbreviations

CEM	=	Cement class
DPS	=	Dynamic Positioning System
F	=	Freeboard [m]
FBT	=	Floating Bulk Terminal
GWL	=	ground water level
HHWS	=	high high water spring
LAT	=	lowest all times
LLWS	=	low low water spring
MOB	=	Mobile Offshore Base
MBM	=	Multibuoy Mooring system
MHWS	=	mean high water spring
MHW	=	mean high water
MLW	=	mean low water
MLWS	=	mean low water spring
RAO	=	Response Amplitude Operator (transfer function)
SLS	=	Serviceability Limit State
SPM	=	Single Point Mooring system
SW	=	southwest (wind)
SSW	=	south southwest
T	=	Draught [m]
TEU	=	Twenty feet Equivalent Units (8 ft x 8ft x 20 ft)
ULCS	=	Ultra Large Container Ships
ULS	=	Ultimate Limit State
VLFS	=	Very Large Floating Structure
WCT	=	Western Scheldt Container Terminal
WL	=	water level
WS	=	Western Scheldt
WSW	=	west southwest

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1. Introduction

The graduation project has been initiated to research the technical feasibility of floating port structures. The project has been done in co-operation with Van Hattum en Blankevoort. Especially the structural concept of the floating elements and the connections between the elements are examined. The project is expanded to incorporate the hydrodynamic analysis of a rigid platform. The hydrodynamics of a multi-body platform consisting of flexibly connected elements is beyond the scope of this thesis due to the limited period of time. A conceptual design is made of the pontoons and the connectors.
The design is made within the framework of a case study.
The case study regarding the Western Scheldt Container terminal has been provided by Van Hattum en Blankevoort.

1.1 Problem analysis

Context

Harbour structures have a long technical lifetime. Their design often does not meet the requirements anymore after a period of time. The economical lifetime is therefore shorter than the technical lifetime, which means that returns on a structure are not being optimised. The steady growth in cargo and specifically container transport and in ship dimensions creates a demand in harbour expansion with new or larger terminals.
Design alternatives are judged on the physical, economical and social-environmental boundary conditions of the location. Environmental conditions such as deep water, little space ashore, soft soil, a large tidal window or an unstable estuary/delta may complicate the construction of a land-based port. Economical boundary conditions, as just described, may include the prolongation of the average economical lifetime of a terminal, decreasing maintenance costs for dredging and maximising returns on the harbour structure. In that case, floating port arrangements may bring a solution. Floating port structures may be designed moveable and adaptable.

Problem definition

Would a floating container terminal create a technologically feasible solution to a need for transhipment, storage and transport of containers from Ultra Large Container Ships, considering the optimisation of the economical lifetime of movable and adaptable harbour structures?

Objective

The objective of the thesis is to realise a technological feasible conceptual design and a functional lay-out of a floating container terminal. A case study is made for a port expansion at Vlissingen harbour in the Netherlands.

1.2 Case study

The concept of a floating terminal can be applied in the Western Scheldt. Van Hattum en Blankevoort made the land-based design of a large L-shaped retaining wall combined with a landfill near the eastern harbour of Vlissingen. All the information on design requirements, environmental conditions and project history were made available by Van Hattum en Blankevoort.
Since the “Raad van state” put a stop to the Western Scheldt Container Terminal (WCT) project, new alternatives are very interesting to research. The WCT project may be restarted within time.

1.3 Boundary conditions

The scope of the graduation project is very wide. It contains an overall view of the floating terminal, its hydrodynamic behaviour and a conceptual design of the pontoons and connections. The design calculations are therefore not detailed in this thesis. Furthermore, A multi-body terminal could not be researched. Also only concrete and pontoon shaped elements of the terminal were researched.

1.4 Structure of the report

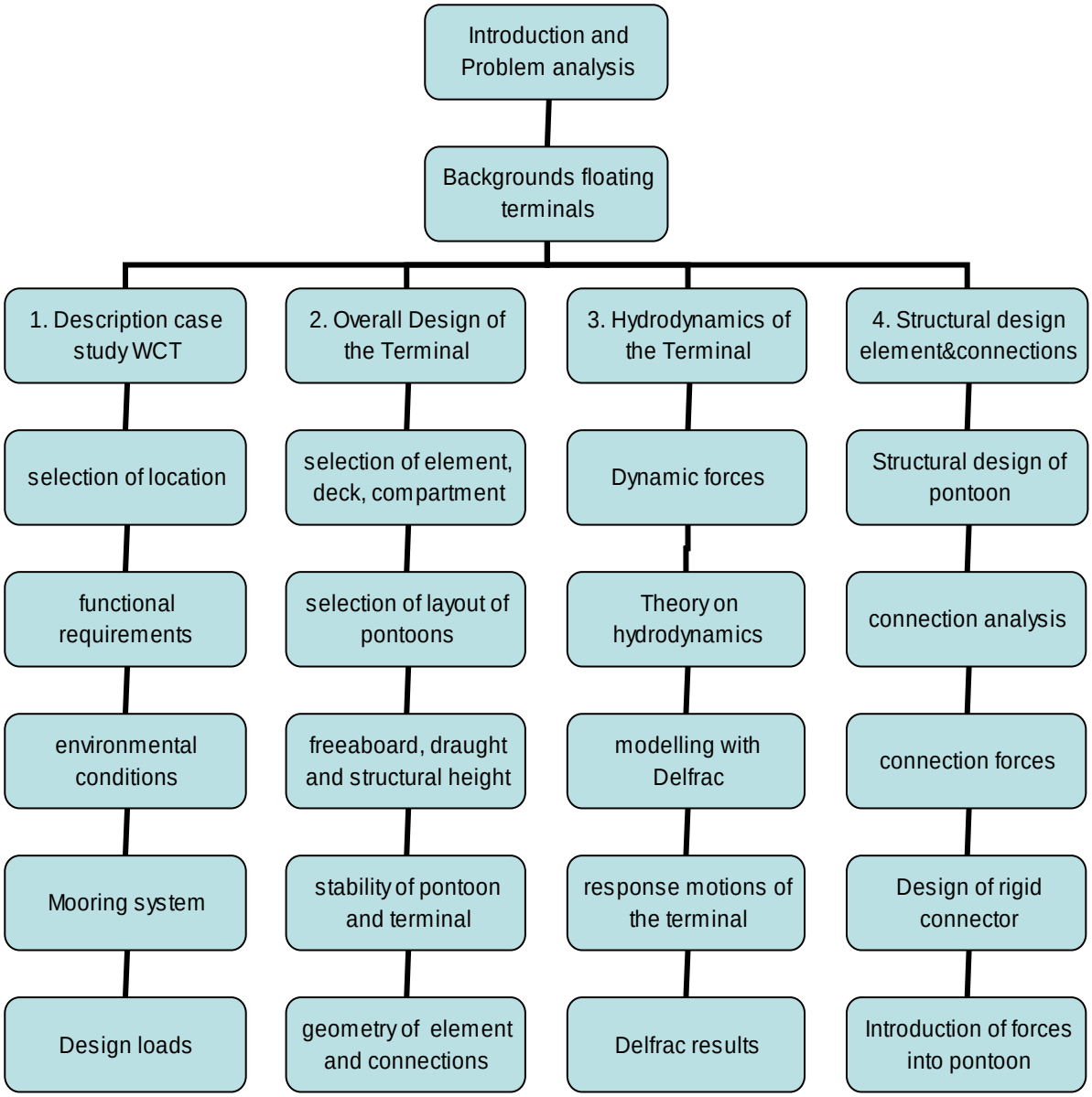
The framework of the project is formed by the case study of the Western Scheldt Container Terminal. Therefore, the first part of this report describes the case study. This part is preceded by an outline of general backgrounds of floating port structures.

The second part of this thesis describes the overall design of the terminal. Several design choices have been made, based on a study of alternatives. This overall design contains the selection of the type of structural floating element, the deck structure and the configuration of the pontoons, as well as the geometrical design of the compartments of the pontoons and the connections.

The third part of the thesis presents the hydrodynamic analysis of two single pontoons and a rigid single-body terminal. The terminal has been modelled with Delfrac. The results are examined and interpreted.

The last part of the thesis contains the conceptual design of a quayside pontoon and of a connection between pontoons. The dynamic forces are translated into static loads and the results from Delfrac are used to determine the connection forces. A connector is developed and the introduction of the connection forces into the pontoon is considered.
This last part is followed by the conclusion and recommendations of the project. All accompanying calculations can be found in the appendices. The plan of the report is shown in the figure on the next page:

Figure 1.1: Scheme of the thesis



2. Backgrounds on Floating Terminals

Transport by ship has grown steadily over the years and more growth is expected in the future. Not only growth in the amount of cargo to be transported, but also in ship dimensions. These factors may lead to necessary harbour expansion.

When a new harbour project or a harbour expansion is initiated, the most important boundary conditions concern minimum costs, maximum return, a maximum economical and technical lifetime and usability.
The design alternatives may be analysed using a life cycle cost analysis.
Several alternatives, from traditional ones to very innovative options, may be introduced.
One of the major controversies is the one between bottom-fixed and floating marine structures.

The initiative of a harbour expansion is followed by research into suitable technical alternatives. During this research, the following aspects should be taken into account:
The type of cargo, the location of the harbour structures, income and costs determined by an economic analysis of future developments of port and ship dimensions and boundary conditions of that particular project of physical, mechanical, political, economical or environmental nature.

The alternatives for harbour structures can roughly be divided in land-based or floating port facilities. The best alternative depends on the location, the Program of Requirements and the function and usability during the lifetime of the structure. A number of scenarios can be considered for each alternative during the lifetime of the structure. It would be optimal to tune the design of the structure to the applications and potential functions of the structure during its lifetime.

To consider the total costs and revenues of a structure during its lifetime, a life cycle approach to the system can be used. The construction costs (investment), maintenance costs, inspection and repair, demolition and the risk of failure are analysed. Then the total costs are minimised by finding a type of structure with a defined investment, an amount of necessary maintenance and an acceptable risk of failure to be expected. Very often, the economic lifetime proves to be smaller than the technical lifetime. Taking this into account, the total costs should be minimised.
A way to do this is to prolong the economic lifetime, or to decrease the investment or maintenance costs. It depends on the type of structure and the distribution of the costs over several aspects, which method will prove most effective. Furthermore the way in which the revenues are divided over the lifetime of the structure influences this choice.

2.1 Moveable marine structures

In Rotterdam it seems that the economic lifetime of piers has been declining over the years, while the technical lifetime has not. The technical lifetime may even increase. This causes the harbour structures to be out of use or demolished before the end of their technical lifetime.
Increasing ship dimensions cause the harbour activities to move further from the city along the coast or, when possible, to move along the estuary towards the sea. Older terminals

can not handle the new ship lengths and can only be expanded with great difficulty in their present location. Therefore, the activities of the terminal move and the pier or terminal is left behind empty. It can be used again for smaller cargo ships or for other ends, like passenger ships or recreational shipping. When the city grows and the harbour is moved towards the sea, the old pier may be used as a boulevard with offices or shops or as a quay for villas with their own ships or as a pier for houseboats. All these are examples of the scenarios a structure may follow during its lifetime.

A design able to follow these changes in use or function of the structure should be flexible. At best, the structure should be expandable, movable and adaptable. These three factors may be reason to explore floating marine structures, especially floating terminals or piers. A trend seems to be initiated to design a structure on its usability and invest a larger building sum to ensure a longer economical lifetime. Another choice, which has often been made in the past, would be to build a terminal at minimum costs and not to consider life cycle costs, being maintenance, demolition and return from a long economic lifetime.

The mobility of a floating terminal built-up out of floating elements is better than that of a fixed structure, because the elements can be disconnected and towed to a new location, where port operations will become more efficient. A floating terminal may be moved in elements and, if considered during the first design, expanded (to new ship length) by extra elements. If this is to be expected, the structure should be over-dimensioned at first. As the dimensions of the structure increase, so will the loads from mooring and from bearing on wave tops etc. Expansion is one thing, but also flexibility in element configuration is an advantage. The configuration of the terminal can be changed, if most elements are structurally equal and if the heavier elements are used in the right place.

The reasons for moving a structure can vary.
The first reason to move the terminal out of the city towards (perhaps deeper harbour basins) the sea, has been considered above. Other reasons can be found in politics, company policies or physical features of the location. For example, the policy of a company, such as an oil company, can be such that a terminal would be out of use in one location within several years, while it is needed elsewhere in the same period of time. This may be due to the location of the oil field or for example ore mines, which become exhausted. Another possible reason is political instability of the country, where the oil field or mines are located. Port structures may be left behind and the investment will be lost. Movable, floating structures can be retrieved and towed to another location.

Large developed harbours like Rotterdam are being kept to depth by maintenance dredging each year. The costs of this maintenance dredging decrease profit for harbour participants. However, returns are adequate or sufficient. Harbours in less developed countries for instance, may also need much maintenance dredging. Estuaries or deltas are seldom stable in their position. Erosion and aggregation (sedimentation) play an important role, because the location of the estuary may change in a certain direction over the years. The amount of dredging needed may far outrun the returns on harbour activity, especially if the natural depth is much less than the depth required for large ocean going vessels. In that case, dredging is not an option. A harbour may silt up and be rebuilt in another place, but moving the terminals altogether would be a cheaper option. This only holds if the costs of constructing new or extra infrastructure to the new location of the harbour from the hinterland are acceptable. In short: the sedimentation of a harbour could cause fewer ships to call and the port could get out of use.

Then again if fewer ships are calling at the port, less money will be available for dredging, so that sedimentation will increase.

2.2 Floating marine structures

The physical boundary conditions of ports vary around the world. The design of a new harbour or a harbour expansion will be made to fit the local conditions. Difficulties have been found in traditional pier structures, which create reasons for floating harbour structures:
A large tidal difference complicates the loading and unloading activities.
Earthquakes and tsunamis endanger the port structures.
A lot of ice in a port can complicate berthing and loading and will exercise great loads on a non-moving, bottom-fixed structure.
A coast consisting of soft soil may provide an insufficient foundation for large concrete structures while soil improvement or other foundation structures are expensive. Landfill structures may experience large settlements in soft soil. The greater the water depth, the larger the landfill load on the bottom.
When space ashore is limited, land reclamation can be a solution. In great water depth, land reclamation is difficult and a floating structure may be cheaper.
Another problem may arise with land reclamation when the appointed (large amount of) soil to be used for landfill is unsuitable or contaminated.
A coast formed by rocks or cliffs may provide little space for harbour structures. The water in front of the cliffs is often deep.
At some locations, changes in the surroundings may only little affect the environment. A harbour expansion will always affect the environment, but some alternatives will cause more problems (e.g. landfill in the water or closing off a bay by a barrier sluice) than others. A landfill means loss of sea life and the present ecological system, which has to be widely compensated for.

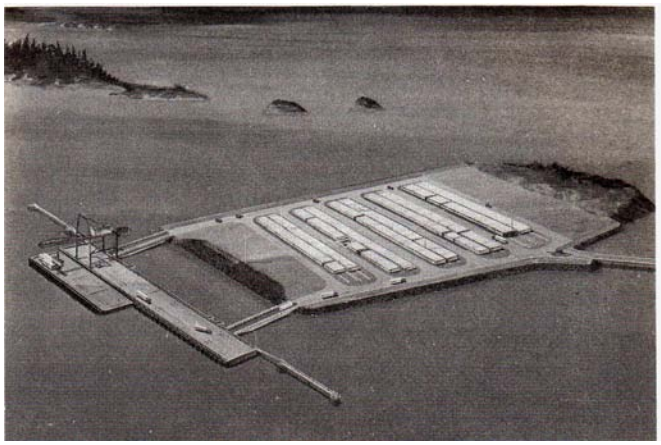


Figure 2.1: Combination of a land-based terminal and a floating pier in Alaska

A bottom-fixed port structure may be designed to bear the extra loads from many of the above influences, but this is expensive. A floating port brings a solution to most of these problems:
Floating structures can move vertically with the tide and possibly horizontal movements are allowed under influence of wind, current, waves or ice.

The consequences of an earthquake on a structure floating on water are smaller, tsunamis will have small consequences as long as the water is deep (offshore) and the bottom is not sloping towards the shore.
A floating structure will only be moored or fixed to the bottom in some way. As such, problems concerning soft soil can be prevented.
A floating structure is not founded on soil. Nevertheless it is a kind of land reclamation.
A floating structure does affect the environment, but does not destroy everything on the specific site. Consequences are dealt with later, but seem to be small.

Depending on the number of problems and the impact of the building project, and regarding the complete life cycle of the harbour, the costs of a bottom-fixed design should be compared to a floating design. Based on this comparison, the choice between a floating or a bottom-fixed structure can be made.

Deep-sea ports

The Hub-and-Spokes model provides a guideline for a future scenario.
It describes a system of large ports all over the world and smaller ports per continent, connected by ship liners. The large ports are called upon by large vessels; tankers, bulk carriers, Ultra Large Container Ships etc. They transport cargo between the continents. Every large port has its own network of connections to smaller ports. Smaller sea ships called feeders will transport cargo from the large port to several smaller ports along the ‘near’ coast. A feeder may be a general cargo ship or a typical ‘coaster’ or ‘short sea’ vessel.

This model can be used to envision a number of offshore deepwater ports over the world, constructing a network of container liners and bulk carriers. These floating offshore ports would replace the now existing large ports on the coast in their transhipment function, thus providing more space ashore for other activities. The port activities regarding storage, connecting to the hinterland and production of goods could remain on land. As removing those facilities and replacing them with the same facilities on a floating platform would prove to be very expensive, the large port ashore would keep its function. However, maintenance dredging for Ultra Large vessels would decrease. Maintenance and deepening of approach channel and harbour basins is expensive, especially when the soil in the harbour basins is contaminated.
The Hub-and-Spokes model with deep-sea ports shows that, as distances will be smaller and large vessels do not have to call at harbours ashore, the line service across the oceans may be faster. Moreover, an offshore port may lessen the pressure on several ports ashore, as the intensity of the shipping traffic on the wet infrastructure is very high in many ports at present. With little space available for port expansion, capacity problems may be expected in the near future.

Another function of a deep-sea port could be an offshore security port before a large metropolitan city, for example near the US. Danger to civilians caused by terrorists etc can be minimised when the cargo in containers is checked offshore. At present only a small percentage of the container throughput is checked, as this cannot be done onboard.

If a floating terminal is constructed in view of the above function: a deep-sea port for very large vessels, then the capacity of the terminal should be such that future developments can be met. In that case, the design should especially take into account a larger future design ship (length and draught) and a large throughput and storage of the port. The type

of design ship and its dimensions could be researched, but it can also be assumed to be an Ultra Large Container Ship (ULCS) of 12500 or 13500 Twenty feet Equivalent Units (TEU). This is of less importance in a specific case study.

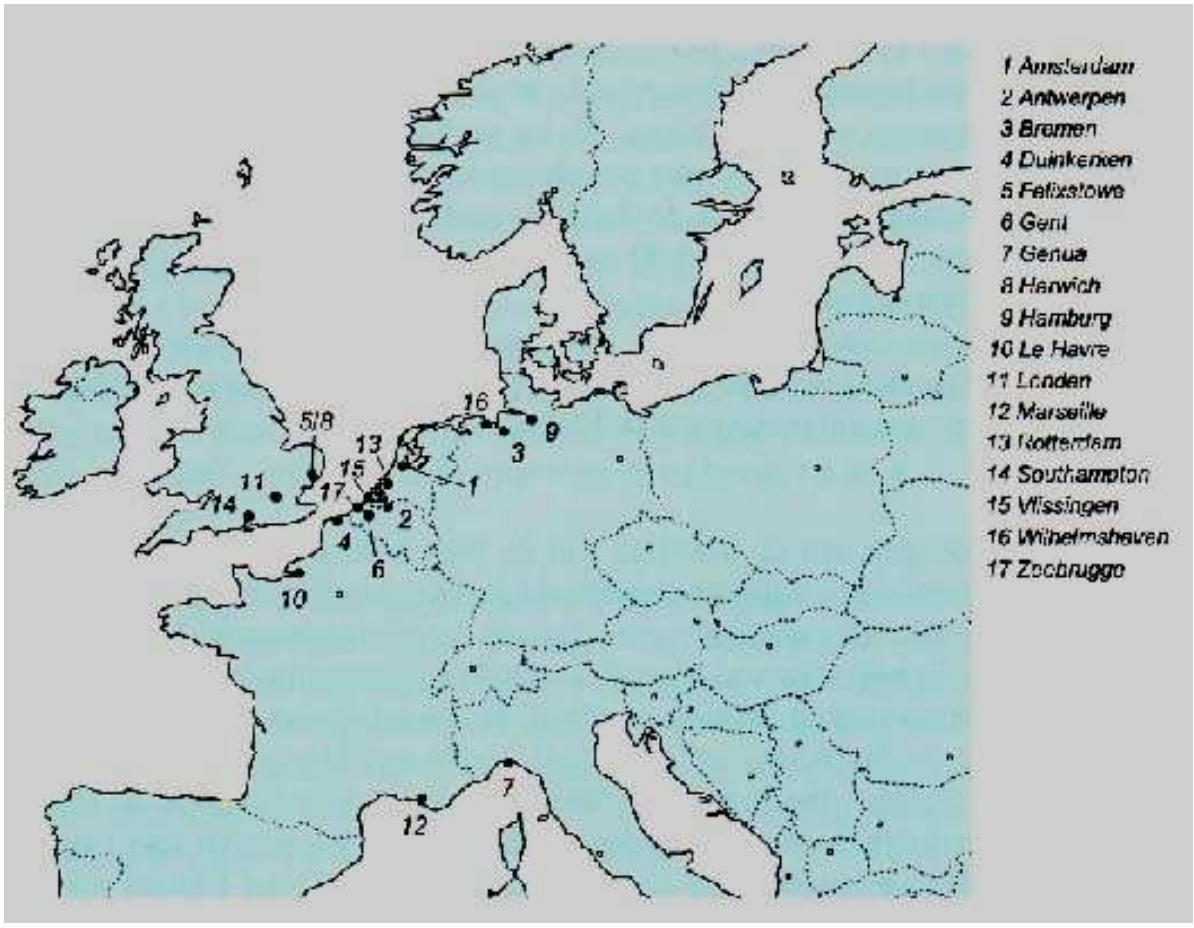


Figure 2.2: The large on-shore ports of Western Europe

Summarised, the principle of a floating terminal is:
That it will be moveable, changeable and will therefore lengthen the economical lifetime of the terminal structure
That it is easy to reach by ULCS and therefore is more exposed to the waves, being situated in less protected waters
That it may be less expensive to build in a certain environment, due to the costly adaptations needed for a land-based design.

Part 1:

Case Study Western Scheldt Container terminal (WCT)



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1 The Western Scheldt Container Terminal (WCT)

1.1 Situation outline

The ports of Zeeland are badly equipped to handle container ships. The cargo transhipment mainly consists of liquid and dry bulk. However, the container cargo branch is growing steadily. In 1999, Zeeland Seaports expressed the need for a container terminal.
The area around the Western Scheldt forms the business centre of Zeeland, concentrated at the harbours of Vlissingen and Terneuzen, where growth is stimulated by the Westerschelde tunnel. A new container terminal would further stimulate the growth of the economy.

The harbour of Vlissingen contains mostly bulk terminals. The port area has been developed for the main part. The port is situated quite near to the city and near to infrastructure. There is remaining space available for new infrastructure. From the port at Vlissingen, cargo can be very well transported further over land.



Figure 1.1: Aerial view and map of the port of Vlissingen

The initiative for a new container terminal was developed in co-operation with the Port of Rotterdam and Hessenatie from Antwerp. The new container terminal would support both Rotterdam and Antwerp. Considering the present capacity of the container terminal in the Netherlands and Belgium, competition is not expected to become a problem.

Three alternative locations were elected for the Western Scheldt Container terminal.
All three locations would have a different capacity. These locations are described in *[Appendix Part 1]*. The harbour expansion would either be attain by reorganising an old terminal or by land reclamation.

The following are the original requirements for the Western Scheldt Container Terminal:

- The terminal area should be larger than 100 ha.
- The terminal location should be directly located at deep navigational waters.
- The quay length for Ultra Large Container Ships (ULCS) should be larger than 1200 m.
- Inland shipping should have its own pier facilities.
- There should be a possibility for a railway connection.

A summary of the functional and technical requirements of the terminal is presented in *[Appendix Part 1]*.

The only location that did completely meet the requirements was in the Western Scheldt between the eastern harbour dam and the cooling water discharge canal from the nuclear plant at Borssele. The space available is nearly 3 km in length. The terminal would be created by large land reclamation of 120 ha, directly bordering on the existing port area.

Several designs have been made for the pier, such as a design with caissons. The winning design was a large L shaped retaining wall with struts, which would be prefabricated elsewhere and transported over water. This design was made by Van Hattum en Blankevoort.



Figure 1.2: Location of the Western Scheldt Container Terminal

1.2 Project history

Environmental associations started a lawsuit to prevent the nature reserve in the Western Scheldt from disappearing. In 2003 the WCT plans were stopped by the “Raad van State”, because the economical need for this project had not been proved well enough to sacrifice the European nature reserve in the Western Scheldt, even if the nature lost would be compensated elsewhere. Too few alternatives had been developed and researched for the container terminal by the authorities, so that the damage to be done to the environment by this project could not be justified.
The verdict of the “Raad van State” was followed by extra research in order to prove the necessity of the harbour expansion for the economy of Zeeland. Moreover, new alternatives for the terminal had to be found to complete the research.

Further research in the last few years has proved that the traditional terminal at Vlissingen, would not be profitable if its length would be smaller than 2000 m (5 berths). Therefore, a present boundary condition is that the length of the terminal should be larger than 2000 m. The costs and returns of the terminal, infrastructure, hinterland connections, settlement area for companies and port facilities should be optimised.

1.3 A floating terminal in the Western Scheldt

This thesis describes research into very large floating objects.
There are several reasons to consider a large floating terminal rather than a land-based terminal in the Western Scheldt. After the verdict of the of the “Raad van State” on the project of WCT, the possibilities of a harbour expansion in the Western Scheldt should be fully explored. If a harbour expansion near Terneuzen or Vlissingen is seen as the best option to stimulate the economy of Zeeland and to strengthen the competitive position of the Dutch ports, alternative harbour structures should be considered.

Floating or partly floating harbour structures can be very practical, because they can be moved to a new location. Harbour structures that consist of modular elements can simply be changed or expanded. A combination of these properties results in a flexible harbour structure, that can have different functions, different dimensions and moreover, different locations during its lifetime. As such, it may replace several other structures or it may have a longer economical lifetime than other land-based structures. As a result, the life cycle costs may be lower.

A harbour expansion can be obtained in several ways.
A number of floating harbour structures can be applied in the Western Scheldt:

- A floating pier in front of a land-based storage yard
  (either on land or made by land reclamation).
- A floating transhipment terminal; a floating pier or quay wall with a small storage facility, and build the remaining facilities such as the storage yard on land.
- An entire floating terminal; the pier, storage yard and offices are all situated on the floating terminal, which is connected to land.

Depending on the space available on shore, the size of the floating part of the terminal can vary. A pier with limited dimensions fixed in a certain location, has a limited economical lifetime. A storage yard may be easier to reorganise or rebuild in a new location. The economical lifetime can be considered of less importance to the storage yard than to a pier structure. Therefore, part of a new container terminal can be constructed on shore. Depending on the available area, only a part of the storage yard such as “empties” or the complete storage yard can be developed on land.

In this thesis, the concept of a complete floating terminal is researched. There is little space available ashore, whereas there are various possibilities in and near the water of the Western Scheldt. An entire floating terminal can be transported and replaced in 50 years time to

another location, nearer to the sea. For this reason, it is more appropriate to design a complete floating terminal.

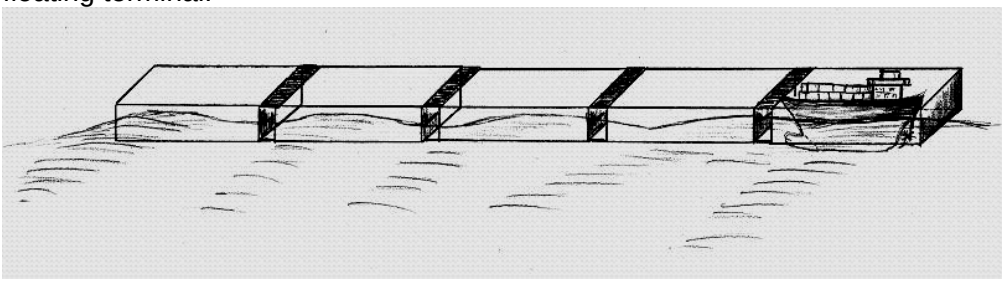


Figure 1.3: Floating pier

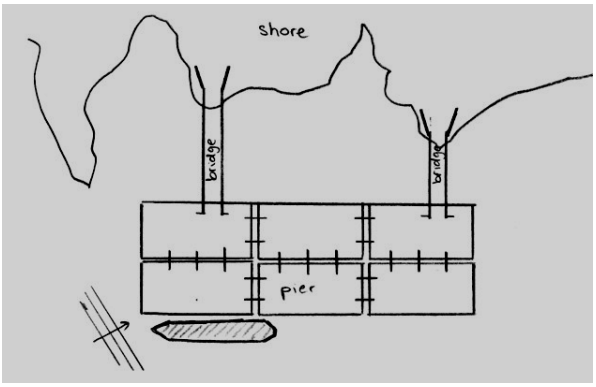


Figure 1.4: Floating pier and partial floating storage yard

A floating terminal may, after for example 30 to 50 years, be used elsewhere to build a near-shore or offshore deep-sea container transhipment terminal. The structure's technical lifetime will be 100 years. The application of a floating terminal would be a solution to the earlier mentioned question of prolongation of the economic lifetime of the harbour structures. This is the main reason for constructing a floating, moveable and adaptable WCT.

The second reason for a floating terminal is the environmental problem of a WCT made by land reclamation in the Western Scheldt. This reason is less strong than the first, because the advantage of a floating platform with regard to a landfill terminal has not been proved yet. The environmental analysis on the Mega Float platform near Tokyo had a very positive result, but although the location and hydraulic conditions of both floating structures are comparable, the width of the (test) landing strip was much smaller than that of the terminal under consideration. It is expected that the environmental effect of the floating terminal on the vegetation, on sea life and on the shallows in the Western Scheldt will be smaller than the effect of a landfill. A landfill will destroy all local life-forms and will drastically affect the currents etc. in the river, which means that the future of the shallows would become uncertain.

Floating harbour structures may also offer a solution in various situations with complex environmental conditions. Examples are a large tidal window, very deep water (too deep for landfill) or very soft soil. The planned landfill in the western Scheldt is very large and will

therefore be expensive. However, the depth of the Western Scheldt is limited and it is therefore assumed that a concrete floating structure will not be less expensive than a landfill.

If a floating terminal would be credited with bringing a better solution to the environmental question and if it would prove to be technically and commercially feasible, then a Very Large Floating Structure would become a serious alternative to restart the political process of the container terminal in Zeeland. If research can deliver enough data proving the necessity of the harbour expansion for the economy of Zeeland, (especially when combined with a new, less environmentally threatening alternative) the “Raad van State” may revoke their decision and allow the WCT project to continue.

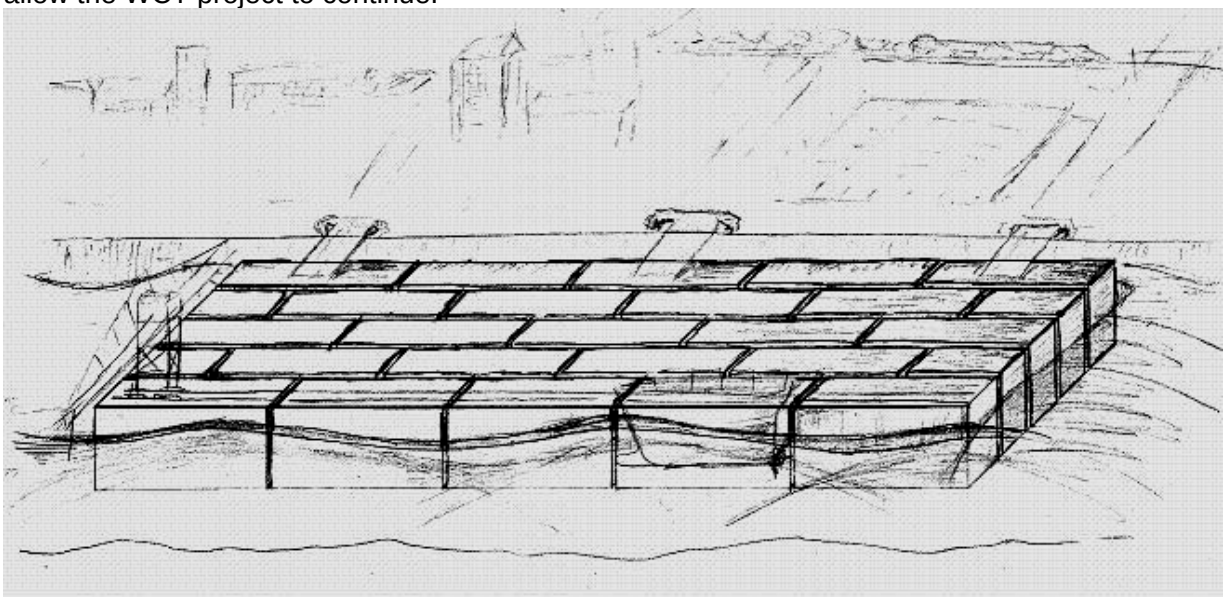


Figure 1.5: 3D impression of the floating terminal

1.4 Location of a floating Western Scheldt Container Terminal

A floating container terminal should meet the same requirements regarding size and capacity as would the land-based terminal in the Western Scheldt. In addition to that, a floating terminal has certain boundary conditions regarding its location:

- A construction dock is needed for construction.
- An easy guiding channel during construction is necessary.
- Extensive anchorage of the terminal takes up space under water.
- The water should not be too shallow.
- The navigation channel should be wide enough near the terminal and away from the mooring lines.
- The terminal has a great dynamic response and should be located in the right direction or protected from the waves.

- The floating terminal should serve its purpose, which is to lengthen the economical lifetime of the terminal or reduce its costs. In other words it is no use to construct a floating platform when land reclamation is not useful.



Figure 1.6: Aerial photograph of the location of the Western Scheldt Container Terminal

The space considered as available for a new container terminal is to be sought near Vlissingen or Terneuzen, as these two cities are the industrial and commercial centre of Zeeland.
The shortage of container transhipment in Zeeland and the local need for a container terminal, is combined with the national (Dutch) need for a new terminal for very large and deep container ships. This drives the container terminal from the Rotterdam area towards the Western Scheldt, where the terminal can compete with or assist Antwerp harbour.

The Western Scheldt will not be deep enough for the future Ultra Large Container Ships. Consequently, solutions should be sought for the future harbour activities at Antwerp. One of these possibilities is to continue maintenance dredging in the Western Scheldt, which will become more complicated and expensive as the ships' draught will increase further. Another option is to keep the Western Scheldt up to depth for the present large ships and to develop a transhipment terminal for future ULCS near the mouth of the river. Ships with a smaller draught may carry the containers up the river to Antwerp. The best location for a deep sea port is near to the sea, either at Vlissingen or at the beginning of the estuary.

In the Western Scheldt are actually two shipping channels with shallows between them in the middle of the river. Unless one of the flats or shoals would be dredged away, a floating terminal will not be possible in the centre of the river. The first is not recommendable in relation to the natural habitat in the Western Scheldt, the present currents and the present location of the tidal channels.

At the south side of the river, a narrow channel passes the port of Terneuzen. To build a terminal within this channel would take up too much space.. The infrastructure between Terneuzen and Vlissingen (the tunnel) would normally make a terminal at Terneuzen feasible. Most of the space available in the Western Scheldt is situated along the north bank of the river. Further upstream from Vlissingen the river becomes narrower and the shallows in the centre grow. The tidal window increases and the tidal channels are more pronounced, being smaller

and deeper. It is of no use to locate the terminal in a narrow passage upstream, where all ULCS will have difficulty or will waste time in reaching the terminal.

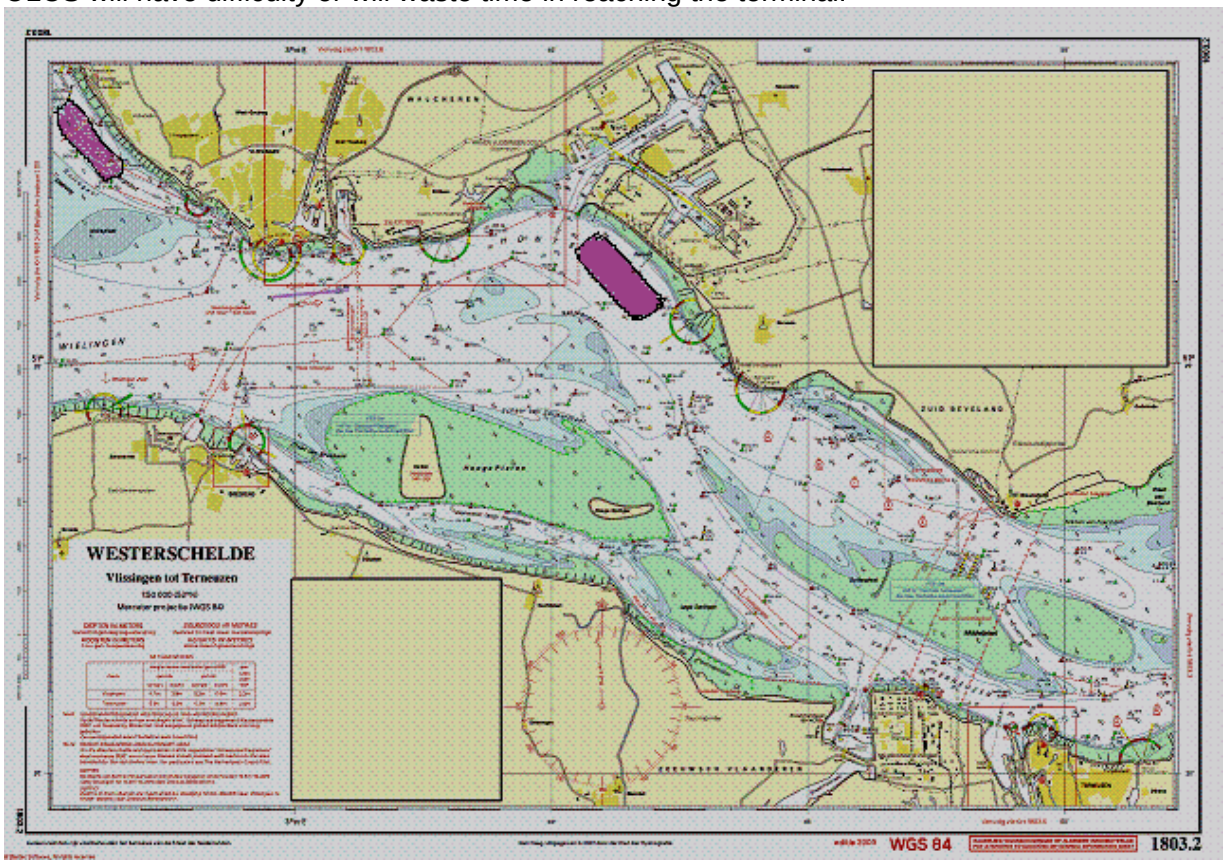


Figure 1.1: Map of the estuary if the Western Scheldt with 2 possible locations of the floating terminal

Concluding, only the following locations in the area of the Western Scheldt are considered suitable for a large floating terminal:

1. The same location as the land-based design WCT near the eastern harbour of Vlissingen, located in the Western Scheldt.
2. A front shore location west of Vlissingen, near the mouth of the Western Scheldt. The location could be near shore or offshore.

The first alternative has already been discussed. A floating terminal could replace the original plans of land reclamation, while space and depth are sufficient. Moreover, the structure will not be entirely exposed to the North Sea wave and wind loads. The technical feasibility of a floating WCT depends on the terminal responses to the incoming waves, in a rather exposed location.

The near-shore location will offer more space along the shore, somewhere between Westkapelle, Zoutelande and Vlissingen. The water depth is larger. The terminal would be exposed to longer waves than at Vlissingen, but the wave conditions near-shore are less

severe than offshore. The disadvantage of a near-shore terminal is the lack of infrastructure. At present, the highway and railway infrastructure does not reach the West coast of Walcheren. Vlissingen already has hinterland connections, although these will have to be expanded in case of a harbour expansion. Lengthening the highway and railway up to, for instance, Zoutelande, would bring many extra costs. The hinterland connections at Vlissingen are easier to expand, also keeping in mind the already-built tunnel under the Westerschelde.

For the present or near future, the location in the Western Scheldt at Vlissingen is the best alternative. In this thesis, the terminal will be designed for that location.

The terminal may be moved from Vlissingen to the near-shore location (near Westkapelle) in 30 to 50 years, when the ULCS will be larger and have a larger draught, and when Vlissingen harbour is crowded. By then, infrastructure may already have expanded or expanding it to Zoutelande or Westkapelle will be worth the costs. The terminal at Westkapelle will be easier and faster to reach by large ocean going vessels. The location will either have sufficient depth, in which case no maintenance dredging is necessary, or will be kept up to depth more easily than the Western Scheldt. The location near the mouth of the estuary will result in heavier wave and wind loads on the terminal. These should be taken into account in the design.



2 Environmental conditions

The Western Scheldt has a width of 500 m at the Belgian border. It is 5 km wide at the mouth at Vlissingen. The length of the Western Scheldt is 60 km. The tide difference at Vlissingen is about 4 m and the strong tidal movements at Vlissingen cause a local depth of about 20 m. The mouth of the estuary is connected to the North Sea by 4 shipping channels, the main one being the Wielingen [part 1 of the Wateralmanak].
The width of the main navigational channel is ca. 500 m.

The local environmental conditions in the Western Scheldt will be taken into account in the design of the terminal in addition to the functional and technical requirements of the terminal itself. The following presents a summary of environmental conditions and useful data from among others the Program of Requirements of the Western Scheldt Container Terminal.

The data on water levels from the Program of Requirements is from Vlissingen. This data should be interpreted before it can be used for the Western Scheldt Container Terminal.
From the report “Hydraulische randvoorwaarden voor het ontwerp” by DHV (and Van Hattum en Blankevoort) and “Hydraulisch en nautisch onderzoek Haven Vlissingen Oost” by Delft Hydraulics:

Water level Frequency of exceedance	**Water level WCT at start of construction**	**water level WCT at end of lifetime**
HHW:		
1/10000 yrs	NAP + 5,60 m	NAP + 5,85 m
1/1000 yrs	NAP + 5,05 m	NAP + 5,30 m
1/100 yrs HW	NAP + 4,50 m	NAP + 4,75 m
1/10 yrs HW	NAP + 3,95 m	NAP + 4,20 m
1/1 yrs HW	NAP + 3,40 m	NAP + 3,65 m
LLW:		
1/1 yrs LW	NAP – 2,80 m	NAP – 2,55 m
1/10 yrs LW	NAP – 3,05 m	NAP – 2,80 m
1/100 yrs LW	NAP – 3,35 m	NAP – 3,10 m

Water level (Tide)	Water level WCT at start of construction	water level WCT at end of lifetime
MHWS	NAP + 2,58 m	NAP + 2,83 m
MHW	NAP + 2,20 m	NAP + 2,45 m
MHWN	NAP + 1,55 m	
Average sea level	NAP – 0,01 m	
MLWN	NAP – 1,47 m	
MLW	NAP – 1,81 m	NAP – 1,56 m
MLWS	NAP – 2,04 m	NAP – 1,79 m
LLWS	NAP – 2,32 m	NAP – 2,07 m
LAT	NAP – 2,54 m	NAP – 2,29 m

According to the Program of Requirements, the water levels at the start of construction are already corrected with 10 cm for deviations of the WCT location instead of Vlissingen East; and with 5 cm for the future sea level rise.

Current

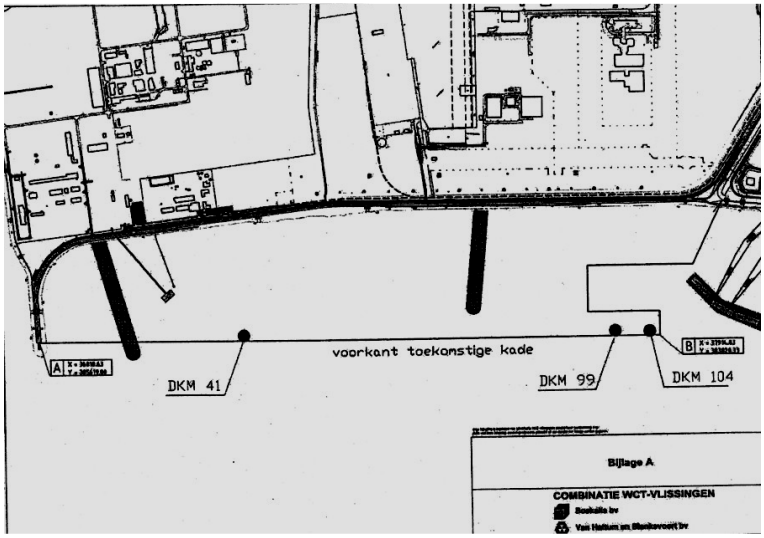
[ref. 20; p.12]: results from the calculations of the Waterloopkundig Laboratorium in Delft (1999) and used in “Hydraulische randvoorwaarden” by DHV.
Location near to the new terminal:

- Maximum flood current : 1,5 m/s
- Maximum ebb current : 1,2 m/s
- Maximum flood current further away in channel : 1,65 m/s
- Maximum ebb current further away in channel : 1,35 m/s

Wind in the Western Scheldt

- Wind waves are caused by a landward wind
- A seaward wind does not result in wave attacks on the terminal
- The prevailing wind direction is south-west, but the direction varies from west to south.

West



East

Wind speed (m/s)	Repetition period (year)
17,8	0,5
18,9	1
19,9	2
21,1	5
22	10
22,8	20
23,8	50
24,5	100
25,3	200
26,2	500

Exceeding frequency	Wind direction (°N)											
	0	30	60	90	120	150	180	210	240	270	300	330
	Wind speed (m/s)											
1/1000 yrs	20,6	18,6	19,4	18,7	17,3	18,6	22,1	27,2	30,0	31,2	29,5	23,8
1/100 yrs	17,7	16,3	17,2	16,6	15,3	16,5	19,7	24,1	26,3	27,0	25,2	20,5
1/10 yrs	14,4	13,8	14,6	13,9	13,0	13,9	16,9	20,5	22,0	22,2	20,1	16,7
1/1 yrs	12,0	12,0	13,0	12,0	12,0	13,0	16,0	17,0	19,0	18,0	16,0	14,0

Waves

To analyse the movements of the terminal-elements and the moored ships, knowledge of the wave spectre: wave period, wave height and direction at the WCT location is essential. Most long waves from sea do not reach the location at Vlissingen, so that the wave spectre contains mainly short waves.

There are two types of waves generated by wind: locally generated wind waves and swell. Swell can travel over longer distances while not under influence of the local wind anymore, and therefore it has a longer wave period. At the WCT the locally generated waves are most important. Their average wave period is defined by:

$T_{m0} = 2\pi * m_0/m_1$

$T_p = 1{,}1/0{,}69 * T_{m0}$ *[Combinatie WCT, 2003]*

$T_s = 0{,}9 * T_p$ *[Battjes, 1982]*

The occurring waves at WCT are: *[Combinatie WCT, 2003]*

The highest waves occur from the south to southwest (sector 210-270°N)

The direction perpendicular to the pier is ca. 225°N. The wave directions 195-255°N generate waves perpendicular to the pier due to refraction *[Waterloopkundig laboratorium, 1999]*

- Wind waves from landward wind
- No waves with seaward wind
- Waves are lower than out at sea.
- Waves from passing ships in the access channel of the Western Scheldt. The ships from and to Antwerp travel at 13-16 knots.

Offshore wave conditions:

1/100 years Hm0 = 7,07 m.
1/10 years Hm0 = 6,22 m.
1/1 years Hs = 5,8 m, which is the average of 1/3 of the highest waves in a period.
Hm0 is the average wave height as determined from the energy spectre Hm0= $4\sqrt{m0}$
With m_0 the area under the energy spectre (curve). The significant wave height Hs is assumed equal to $H_{1/3}$; the average wave height of the highest 1/3 part of the waves in a time interval.

The short waves occurring at the WCT are mainly caused by the wind and by the fetch of the wind. The following table presents the wind direction and the relating angle of the incoming waves perpendicular to the terminal. Head or stem waves on a ship are presented as a direction of 180°.

Wind direction (°N) Max 360°	**Angle of incoming waves (to the normal of the terminal=90°)**	**Modelled wave direction (°)**
195	-26	90-26 = 64
225	+11	90+11 = 101 (beam waves)
255	+29	90+29 = 119

Hydraulic boundary conditions at harbour Vlissingen East [ref. 27, 28]:

Exceeding Frequency (yrs)	**High water level to N.A.P. (m)**	**Low water level to N.A.P. (m)**	**Design wave height H_s (m)**	**Wave period T_p (s)**
10	2,90	-2,54	1,20	4,7
1	3,45	-2,80	1,50	5,2
0,1	4,00	-3,05	1,80	5,7
0,02	4,35	-3,35	-	-

HHW		**Wave heights and periods**			
Exceeding frequency	**Direction to north °N**	**Hs (m)**	**T_{peak} (s)**	**T_s = 0,9*T_p (s)**	**ω (r/s)= 2π/T_p**
1/100 yrs	195 (S SW)	1,52	4,8	4,3	1,309
	225 (SW)	**2,13**	5,7	5,2	1,102
	255 (W SW)	**2,37**	6,2	5,6	1,013
	285 (W NW)	2,26	6,0	5,4	1,047
1/10 yrs	195	1,28	4,5	4,1	
	225 (SW)	**1,82**	5,4	4,8	
	255	**2,06**	5,8	**5,2**	
	285	1,93	5,7	5,1	
1/1 yrs	195	1,11	4,3	3,9	
	225 (SW)	**1,55**	5,1	4,6	
	255	**1,77**	5,5	**4,9**	
	285	1,59	5,4	4,9	

LLW		Wave heights and periods			
Exceeding frequency	**Direction to north °N**	**H_S (M)**	**T_{peak} (s)**	**T_s (s)**	**ω (rad/s)**
1/100 yrs	195	1,32	4,5	4,0	
	225 (SW)	1,78	5,1	4,6	
	255	2,03	5,4	4,9	
	285	1,87	5,4	4,8	
1/10 yrs	195	1,10	4,2	3,8	
	225 (SW)	1,48	4,7	4,3	
	255	1,69	5,1	4,6	
	285	1,53	4,9	4,4	
1/1 yrs	195	0,96	4,0	3,6	1,745
	225 (SW)	1,24	4,4	4,0	
	255	1,41	4,8	4,3	
	285	1,27	4,6	4,2	

Wave data low water level, SLS:

Frequency of Exceedance	**Design wave height H_s (m)**	**Wave period T_p (s)**	**Direction to north°**
1/1	1,24	4,0	225°
1/1	0,69	3,1	135° corner element
1/100	1,78	4,6	224°
1/100	0,93	3,5	135°

Wave data high water level, during construction:

Exceeding Frequency (yrs)	**Design wave height H_s (m)**	**Wave period T_p (s)**	**Direction to north°**
1/10	1,82	4,8	225°
1/10	0,81	31	135°

The design waves are short waves. Long waves do not form a part of the occurring wave spectrum within the Western Scheldt.

The significant wave height H_s and the average of 1/3 of the highest waves $H_{1/3}$ could be determined through analysis of a wave scatter diagram of the Western Scheldt. However, this is beyond the scope of this thesis, and the available wave data of the location of the WCT will be used. The significant wave height is the highest 1/100 years wave: H_s = 2,37 m.

Swell waves are explained in Part 3 of this report. Assumptions regarding the boundary conditions due to swell waves are also discussed in Part 3.

Passing ships

The WCT is located in the Western Scheldt, parallel to the main navigation channel towards Antwerp.
Many ships pass at an average speed of 15 knots.
Passing ship speed: 13-16 knots
Next to these ships, mooring or leaving ships pass the pier.
Passing ships cause a local decrease in water level at the pier. The size of the water level drop depends on the size of the ship passing, its speed and the geometry of the navigational channel. The report “Hydraulische randvoorwaarden” by DHV presents the findings of a research into the water level drop, done by Waterloopkundig Laboratorium Borghout. The described design ship was used as a normative ship. The results are:
A maximum decrease in water level occurs at low water (GLWS = NAP – 1,92 m) and at a minimal distance between ship and pier. The distance varies from 230 to 320 m, as this is the present distance from the future terminal to the navigational channel.
The governing decrease in water level is 0,47 m at the sea quay.

3 Site and layout of the floating container terminal

The land-based design of the Western Scheldt Container Terminal is of a rather rectangular shape. It contains a dock (harbour basin) for barge berths (inland shipping).
For a floating platform, dynamic behaviour is important. Also, construction demands a simple shape of the platform that will be built by assembling elements. Therefore, a rectangular platform shape is chosen. A rectangular shape will increase the simplicity of the dynamical model. No harbour basin will be applied. Furthermore, the rectangular shape will lead to a simple and efficient layout of the terminal, with a large and straight storage yard

Present views are followed in the functional requirements of the floating Western Scheldt Container Terminal. There will be sufficient space for 5 deep-sea berths with a total length of ca. 2100 m. The width of the terminal will be 500 m, so that the capacity is ca. 105 ha. Another 30 ha was already planned ashore, this can be maintained in the design of the floating terminal. The upstream short side of the platform will be used as barge pier for inland shipping. This pier will have a length of 500 m.
The long side of the terminal towards the coast can also be used as pier, for instance for tugboats, but this is not taken into account.
The terminal may be expanded in the future to the original required size of 2615x500 m.

The layout of the container terminal contains the following parts:
- quayside
- Storage of containers for short duration
- Storage of containers for longer duration
- Storage of empty containers
- Storage area of inland ships
- Infrastructure (roads, rails)
- Facilities such as offices and parking place
- Entrance and infrastructural connection to train

The number of movements with cargo and the time needed per movement should be minimised. Therefore, containers with a short storage duration time, should be stored near to the quay. Empties should be stored at the back of the terminal, bordering the parking place and the offices The facilities of the terminal should be closest to the entrance (bridge to the shore). Last, the storage areas should be large and of a simple shape (linked yards), but enough roads should accommodate the traffic on the terminal.
The sketches below show a possible simple layout of the terminal.

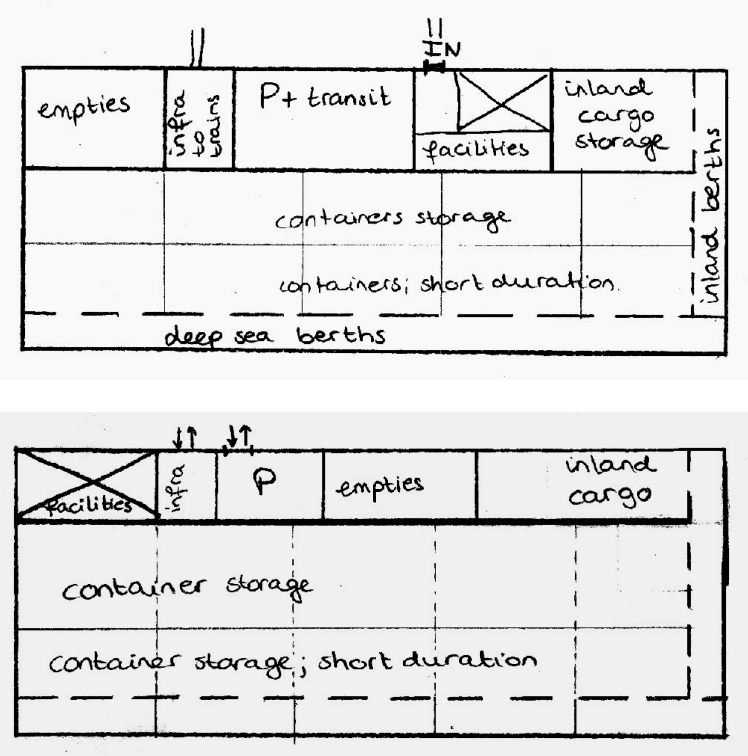


Figure 3.1: layout alternatives 1 and 2 for the floating terminal



Figure 3.2: Example of the layout of the Maersk terminal in the Europahaven, Rotterdam

4 Positioning of the terminal

A mooring system or anchorage takes care of the fixation of the floating structure to the bottom; horizontal motions are limited, to prevent the structure floating off location.
The structure will have to be able to move with the tide and waves, but will stay in place within boundaries. Different mooring systems are described in *[Appendix Part 1].* For the floating terminal in the Western Scheldt, either a Single Point Mooring system (SPM) or a Multibuoy mooring system (MBM) can be applied. A dynamic positioning system with a propeller on every floating element is assumed to be too expensive and unnecessary in the Western Scheldt. Mooring system components include anchors, sinkers, anchor chains, buoys and mooring lines. The type of mooring to be used depends on the location, the water depth, the type of structure, current and wave loads and other influencing factors.

4.1 Requirements and environmental conditions

The following is a situation sketch:
A benign wave climate dominates the Western Scheldt estuary. The space available in the Western Scheldt is limited. The currents form a load on the anchorage system of the terminal; these loads will be quantified elsewhere.
At the mouth of the estuary at Vlissingen, the Western Scheldt is 5 km wide.
The shipping traffic towards and from Antwerp is busy. The shipping channels comprise ca. 1000 m of the width of the river, in the middle and just outside the centre of the river. The width of the terminal is about 500 m, and an estimated width for harbour activities such as turning and manoeuvring of the ships is 450 m (>1,1*L_{OA} design ship).
This leaves ca. 2500 – 500 – 500 – 450 = 1150 m width. This distance is smaller than the length of the terminal; 2100 m. The terminal will not be allowed to turn and weathervane with the waves or wind, because there is no space. Therefore, a Single Point Mooring system will be out of the question and a Dynamic Positioning is considered to be useless. This system would only be effective when the terminal can rotate freely, such as when located offshore. The shipping channel will have to stay in the same location as it is at present, because of maintenance dredging and the location of the shallows.

Possible types of moorings for the large floating terminal are:
1. Gravity based structures; pontoons ballasted to the bottom, concrete blocks
2. Anchor cables, connected to rock anchors or a foundation block.
3. Anchor cables, connected to piles or suction piles
4. Piles with a sliding connection (limited water depth)
5. Several large dolphins, pylons or tower-shaped structures, to which the floating object can be connected with a vertically sliding connection.

These types can be divided in two main groups:
1. Flexible elements (cables and lines)
2. Rigid elements (structures)

The terminal will be situated in the longitudinal direction of the river, seemingly parallel to the bank. This direction does not differ much from the prevailing current and wind or wave direction. Types 1 to 4 (and possibly 5) are all spread moorings or MBM.
Option 1 is possible in the Western Scheldt, but because of the limited water depth, the blocks should actually be dug into the bottom soil or placed well outside shipping lanes.
Option 3 will be very well possible, as the bottom of the Western Scheldt consists of soft soil. For the same reason option 2 will not be a very plausible solution.
Piles with a sliding connection (option 4) may be an expensive solution. Many piles will be needed to bear the mooring forces of a large platform, or the piles structures will have to be very large, which would lead to option 5, which is a small number of dolphins or tower-shaped structures instead of many piles. These dolphins will improve the dynamic behaviour of the platform. Considering all the alternatives, the options 3, 4 and 5 seem the most plausible solutions. The terminal will be located as much as possible in the prevailing direction of the waves. The maximum mooring forces of the MBM will be large due to an oblique wave attack, as the structure cannot weathervane. The mooring system to be applied should be chosen on minimal costs, maximum safety and practicability, for instance how difficult it proves to be to introduce the mooring forces into the structure. The mooring system has to be able to bear the mooring forces. The constructability decreases when too many tubular (suction) piles are needed to bear the load. At the same time, the costs will rise. In that case, larger mooring structures such as dolphins may bring a solution.
The mooring force of the terminal will be equal to the sum of the external forces, caused by waves. The mooring forces will be estimated using the wave drift forces.

4.2 Applied mooring system

A multibuoy system will be applied at the Western Scheldt Container terminal.
Four alternative mooring systems are considered:

I. Three to five large dolphins along the side of the terminal without berths
II. Large piles or dolphins with a sliding connection in the centre longitudinal axis of the terminal
III. Many tubularl (suction) piles along the sides of the terminal without berths
IV. A large pile with a sliding connection for every pontoon element of the terminal, all piles in the centre of the pontoon

A multi criteria analysis has been used to select the mooring system in the Western Scheldt. The analysis is presented in *[Appendix Part 1]*.
Alternative III is selected as the best applicable mooring system, based on the construction method of the system, the local environmental conditions and the costs. A large number of tubular piles will result in a well distributed load on the terminal due to mooring. There is sufficient space along the long side of the terminal to install a large number of piles.

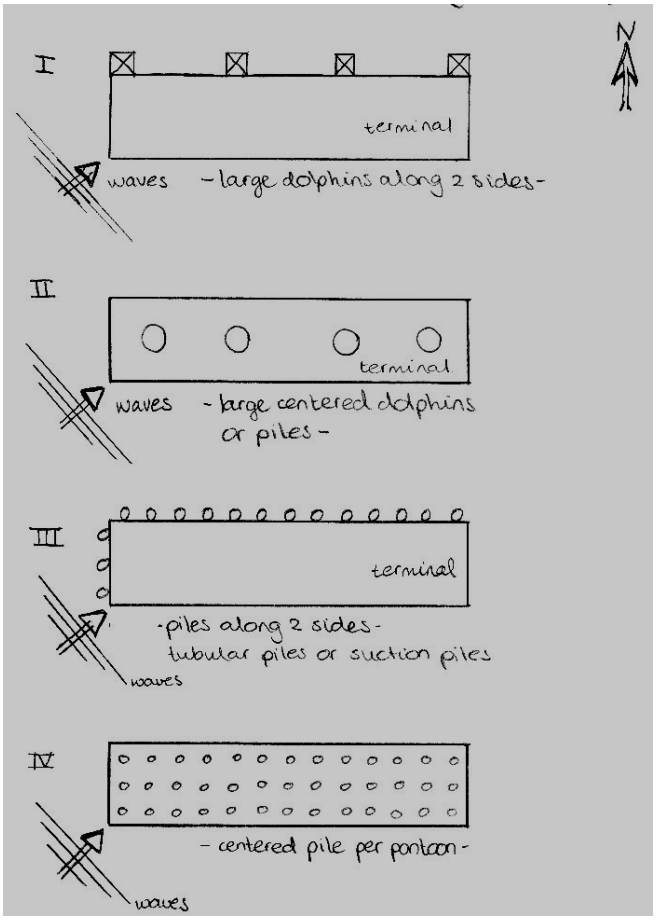


Figure 4.1: Alternative mooring systems

Due to the wave loads, the distribution of the mooring forces may be very unfavourable. Dependant on the location of a wave top, the full mooring forces may be transferred to the apron elements. As a realistic model of the connected elements is complicated, the entire mooring force per element will be taken into account for the apron elements. This connection force per element is distributed under 45° from the piles through the concrete and connections. The load will be transferred to the apron pontoon through the distributed connections.

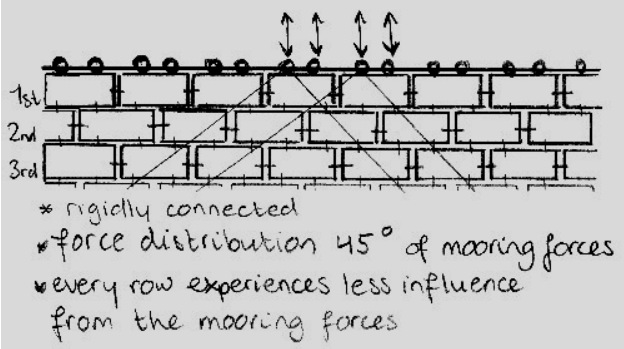


Figure 4.2: Distribution of mooring forces

The dimensions of the large tubular piles have been calculated in *[Appendix Part 1]*.
A pile with a diameter of 2,20 m and a thickness t of 50 mm will be placed every 90 m along the long side of the terminal (facing the shore). The pile will have a length of ca. 24 m above the bottom. The total length of the pile would have to be determined using soil data.

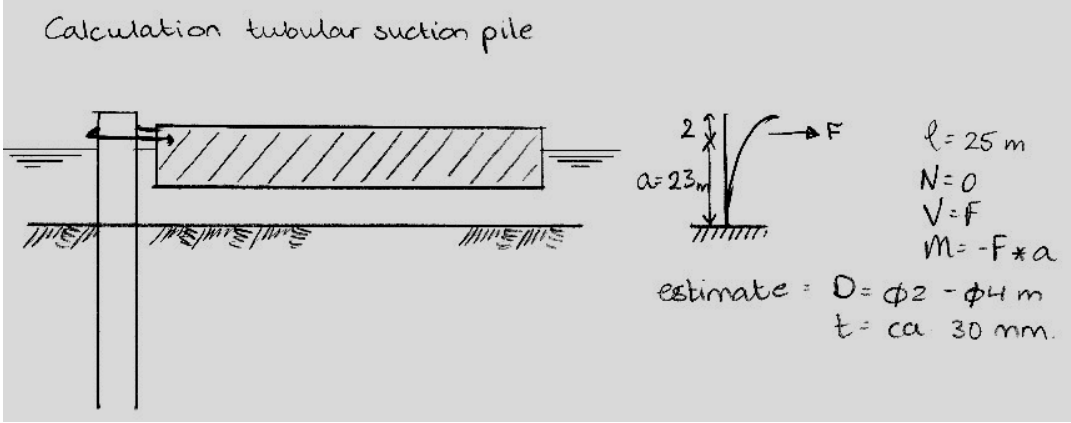


Figure 4.3: Calculation of mooring pile

5 Loads on the terminal

Permanent loads	**Variable loads**
Dead weight	Ballast load
Water pressure and buoyancy	Storage load
Deck load	Traffic and equipment load
	Crane track loads
	Bolder mooring forces
	Fender load
	Wave load
	Current load
	Wind load
	Wave drift forces
	Water level depression
Shrinkage and creep	Temperature load

The load combinations are put together from the correlation between wind and wave direction and amplitudes (storm situation), bolder force (wind force, wave height) and operational loads. The loads are quantified in the appendices and a full overview is shown in the structural design of the pontoon *[Part 4]*.

- **Self weight**

The dimensions of the concrete structure will specify the self weight. The concrete is assumed to be prestressed and reinforced γ_c = 25 [kN/m^2].

- **Water pressure**

A hydrostatic pressure on the outer walls and the floor of the pontoon is a static load. The buoyancy force causes pressure on the pontoon floor.
Maximum pressure on the walls q = ρg * T [kN/m^2] in which T = draught of the pontoon
Maximum force on the walls F = ½ * ρg * T^2 [kN/m^1]
Pressure on the bottom slab q = ρg * T [kN/m^2]
Maximum force on the slab F = ρg * T * L * B [kN]

- **Deck load**

This load represents vertical loads of structures, materials or facilities on deck; 1,5 kN/m^2

- **Shrinkage and creep**

Concrete shrinkage and creep grow in importance as the structure dimensions increase. They will be determined during the concrete or prestressing design.



- **Ballast load**

A ballast sytem will control the system's freeboard. This ballast will consist of wet sand or water. The ballast will be pumped into the pontoon's compartments, upon the floor. Heavy iron ore ballast is not necessary, as there is plenty of space for more lightweight ballast, which is easier to handle and will cause less damage to the inner walls of the elements. The maximum ballast load will be about as large as the total maximum deck load, as those loads compensate each other.

- **Container storage load**

From Program of Requirements: p = 60 kN/m^2 (stacked 5 containers high)
The terrain load and traffic load do not occur at the same place at the same time;
The location is either infrastructure or storage area.
The emergency storage in the apron area is assumed at 40 kN/m^2 (stacked 3 containers high).

- **Traffic and equipment load**

- VOSB class 60 [NEN 6723 VBB]
  3 kN/m^2 and 3 axes loads of 200 kN, distributed over four wheels per axes.
- Mobile crane: maximum strut load 2600 kN/ 6m^2, location at quay near crane track is possible.
  This load is governing, located at a corner with its length parallel to the pier length.
- Straddle carrier: maximum wheel load 270 kN x 2 wheels per corner. Straddle carriers can move up to and underneath the container crane.
- Fork lift truck and reach stacker: maximum axis load 1000 kN

- **Crane track loads**

A container gantry crane causes a distributed load per wheel, which can either be modelled as a system of point loads for every wheel system, or as a line load. Next to the vertical load, horizontal loads parallel and perpendicular to the rails have to be taken into account. The crane also causes a bending moment on its foundation (the crane track).
The pontoon shaped element to be designed is an quayside element.
The concentrated crane loads are expected to cause large internal forces. In total, the vertical load on the terminal due to stored containers is much larger than the crane loads. The quay element will be designed for the specific WCT design super Post-Panamax crane from the Program of Requirements. A maximum of 3 cranes per quay element is assumed.

The maximum distance between front side of the fender and centre of the crane rails is 5,50 m. The distance between the two crane rails is 30 m. A gantry crane has four legs and one wheel system per leg. The rails are located 30 m apart, but the columns with the wheel system are 18 m apart to the centre. The wheel system has a length of 9 m for 8 wheels and a load distribution under a bit less than 45° up to the centre of the upper part of the structure.

Total crane weight : 25600 kN
Crane track : Vertical 800 kN/m, horizontal 80 kN/m
In general: horizontal crane track load = 10% x vertical crane track load
The crane loads depend on the wind direction and wind force.

Buffer load crane track
Horizontal collision force : 700 kN
Located 1,5 m above upper side of the rail, parallel to the quay.
The dynamic collision factor S = 1,2 is taken into account for all horizontal crane track loads.

- **Mooring forces on bolder**

Landward wind and high waves produce a small bolder force. Another combination will cause the maximum bolder force. Fender and bolder load will not occur at the same time.

Bolder force	:	2000kN over 30 m (70 kN/m)
Distance between bolders	:	max. 30 m
Distance between fenders	:	max. 30 m, bottom side NAP –2,80 m.

- **Fender load**

F = 5500 kN / fender set
Centre distance of the fenders = ca. 15 m.
Toggle hole: F = 150 kN per bollard

- **Wave load**

Wave loads are estimated according to Sainflou and the linear theory. These estimations are afterwards compared with the results from Delfrac. The wave loads are computed from a dynamic analysis, but are translated into static design loads.

- **Current load**

The currents in the Western Scheldt cause quasi static load on the terminal, mainly on the upstream face. Uniform current causes a drag force on a fixed body in the direction of the current and it causes a lift force on the body perpendicular to the direction of the current.
The container terminal is a floating body that is fixed within boundaries in its horizontal location. Water can flow underneath the terminal. Therefore, drag and lift forces will be smaller than on a fixed object such as a bridge pile (cylinder). The terminal is assumed to be a very large flat plate with its longitudinal axis rather parallel to the direction of the current.
An estimate of the resulting force on the body according to the Morison equation is: *[Van Baars, 2002]*

$$F = \rho g \frac{u^2}{2g} = \frac{1}{2}\rho u^2 = \frac{1}{2}.1025.1,65^2 = 1395,3N/m^2$$

Marine growth on the pontoons has not been taken into account.
The empirical formulas for the drag and lift force are:

$F_D = \frac{1}{2}\rho u^2 (C_D + C_D^{'}).A$ with C_D and $C_D^{'}$ the static and dynamic drag coefficient respectively

$F_L = \frac{1}{2}\rho u^2 (C_L + C_L^{'}).A$ with C_L and $C_L^{'}$ the static and dynamic lift coefficient respectively

$A=500*13,11=6555\ m^2$ Area of the body perpendicular to the flow

The static coefficients depend on the shape of the body, which can be expressed in the Reynolds number:

$$\text{terminal } \mathrm{Re} = \frac{ud}{\nu} = \frac{1,65.500}{1.10^{-6}} = 825.10^{6} > 1.10^{4} \text{ turbulent flow}$$

The estimated C_D for a flat plate = 2
The terminal is assumed to have a small angle with the direction of the current; α=10°
L/D = 2100/500 = 4,2
C_L is estimated at 0,3.

The dynamic current load is caused by vortex shedding in time behind an object in the stationary flow.
The vortex shedding frequency is:

$$f_s = \frac{uS}{D} = \frac{1,65.0,2}{500} = 6,6.10^{-4} \text{ (is fs>fn there is no resonance expected)}$$

The dynamic drag coefficient is estimated at $C'_D \approx 0{,}10$ and $C'_L \approx C'_D$ is assumed
F_D = 19207 kN
F_L = 3658 kN

- **Wind load**

The wind load consists of a static wind pressure part, dependent on the height and location of the structure, and a dynamic part. The dynamic part consists of wind gusts in various directions. The wind spectra can be used to analyse the phenomenon. The wind speed data have already been discussed. As the floating structure is not very high, mainly the static wind pressure is important. The container cranes and the containers stacks are expected to be ca. 30 m high upon the deck.
Wind pressure:

$$p_w = \frac{1}{2}\rho u^2 . C_D = \frac{1}{2}.1,25.31,2^2.0,7 = 426 N/m^2$$

The wind pressure is normalised in the Netherlands *[Van Baars, 2002],* the Western Scheldt is situated in area II *[Tabellen voor bouw en waterbouwkunde].*
At a height of less than 7 m in an open area, the wind pressure is 0,78 kN/m^2
At a height of less than 30 m in an open area, the wind pressure is 1,24 kN/m^2

- **Wave drift forces**

Wave drift forces cause a force in the mooring system of the terminal, which fixes its horizontal position.
These forces can be determined with a dynamic analysis, but they will be estimated using a hand calculation.

- **Water level depression**

Water level depression (also during coupling of elements) = 0,47 m. It results in a smaller buoyancy force; a reduction of 4,7 kN/m^2.

- **Temperature load**

The coupled floating pontoons will be warmed up by the sun on the deck, but the water temperature will remain low. The concrete will want to deform unevenly due to this temperature difference. This should not lead to cracking of concrete. Temperature loads occur on single pontoons, but more severely on rigidly connected pontoons.

5.1 Load combinations in general

Load combinations are taken into account for the Serviceability Limit State and the Ultimate Limit State.
The load combinations are based on two main situations:
1. transport and coupling at normal waves (1/1 year) and wind
2. Operational phase during storm (1/100 years)

As the floating terminal will move with the tide and water level, the water level has no influence on the draught of the pontoons and therefore the buoyancy remains the same. The water depth of the Western Scheldt will have to be sufficient. High storm waves will cause high wave loads and thus produce the highest connection and mooring forces.
For a floating structure, a low water level and low waves are favourable. Therefore only the high water level with storm waves will be taken into account in the load combinations.

Aspects determining the governing load combination:
- Storm and high water
- Water level depression
- Accident; collision
- wind towards land or sea
- Wave direction
- Apron activities; either emergency storage or traffic and mobile cranes.

As the storage load is higher than the evenly distributed traffic load, the storage load will be taken into account in the hand calculations

The following load combinations have to be taken into account:

LC	name	Description
I	Construction	Construction and transport: Normal waves, wind and current, empty deck, full ballast
II	Operation: Emergency storage	Maximum waves, wind and current, full deck storage load, empty ballast, full apron crane and emergency storage load, full fixation, full fender and mooring load
III	Operation: apron cargo transport	Maximum waves, wind and current, full deck storage load, empty ballast, full apron crane/traffic/mobile crane load, full fixation, full fender and mooring load

A quantification of these combinations can be found in Part 4 of the thesis.

Part 2:

Overall Design of the Terminal

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1 Structure of the floating terminal

The floating terminal consists of structural elements. The first step of the design of the terminal is to select the best type and shape of floating element, the deck structure and the layout of the floating elements of the terminal. In the following paragraphs, the alternatives are described and the best combination of the structural parts of the terminal is selected. The process of selection is a design spiral of the terminal, which takes into account the elements and the mooring system of the terminal and also the connections between the floating elements.

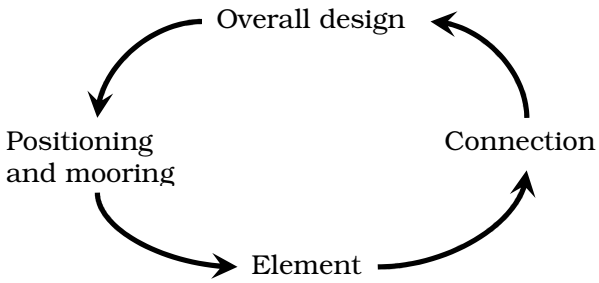


Figure 1: design spiral of the floating terminal

1.1 Type of element

Four alternatives are considered for the floating elements ("floaters") of the terminal:

I. Semi-submerged structure (Mobile Offshore Base-concept).
II. Pontoon-shaped structure, with various possible connections and mooring systems
III. Tension-leg floating structure.
IV. Pneumatic floater (use of air pressure)

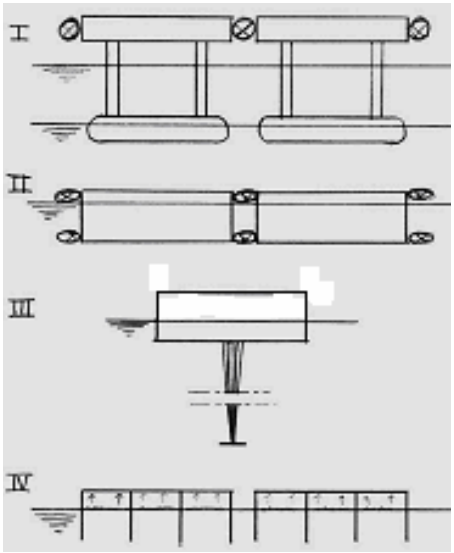


Figure 1.1: Alternative floaters

The alternatives for the elements of the terminal are weighed on aspects of:
The building costs and life cycle costs, constructability, static and dynamic stability, behaviour during transport and during operation, efficiency during operation, construction time. The pontoon shaped elements are assumed to provide the best solution for the permanent structure of the terminal in the shallow waters of the Western Scheldt.

For the design of the floating element, several aspects are taken into account:
- The type of structure; a semi-permanent terminal, which will not be moved often.
- The loads; crane loads etc are heavy, but the intended location for the WCT is not offshore,
  so the wave climate is not severe.
- The dynamic behaviour of the type of structure.
- The stability during transport and during operation of the structure.
- (Re)movability and adaptability of the structure.
- Damage stability requirements; the “floater” must survive and remain afloat and stable should any two adjacent cells be punctured below the waterline.
- A relatively small water depth of about 20-25 m in the Western Scheldt.
- Wave protection for vessels.

Pontoon shaped structures have a large water plane area and a low centre of gravity and metacentre. They are mostly very stable. However, they do experience a large wave load. Pontoon shaped structures can be applied in deep and shallow water. However, the pontoon-structure is less suited for separate use at the open ocean, even if they are long enough. Fast coupling and decoupling may be difficult, but as the terminal is a semi-permanent structure this may not be a problem. The pontoon-shaped concept can be selected as best alternative on financial aspects, stability and vessel protection. The hydrodynamic behaviour is worse than that of the semi-submersible, but the Western Scheldt has a mild wave climate. Its stability may be better during transport. The pontoon-shaped Mega-Float that is fixed to the bottom by dolphins instead of cables has a better hydrodynamic behaviour, which could be a solution for the floating terminal. A pontoon-shaped structure is chosen for the floating container terminal based on financial aspects and the environmental conditions in the Western Scheldt. The other alternatives are shortly described below, but they are mainly assumed to be too expensive and too large for the situation at hand, for various reasons.

In general, the stability of a floating object is proportional to the water plane area of the object. The hydrodynamic behaviour is dependent on the wave pressure on the structure, which is larger when the water plane area is larger. The hydrodynamic behaviour of the semi-submersible is favourable, because the water plane area is small during operation (the structure is ballasted at site). The wave pressure is therefore smaller. The stability of the two-legged semi-submersible is sufficient at the same time, because of the low centre of gravity due to the floaters under water.
During transport, the water plane area of the floaters is larger, which will increase the wave load, but also increase stability. However, the very high centre of gravity above the water causes more instability.

Tension-leg structures are only applied in deep water. In shallow water, other mooring systems will be less expensive. The Mobile Offshore Base-concept is designed especially for temporary

use: fast coupling and disassembling of the independent floating elements, followed by immediate transport. The expected use is on the ocean, in a severe wave climate.

1.2 Type of deck structure

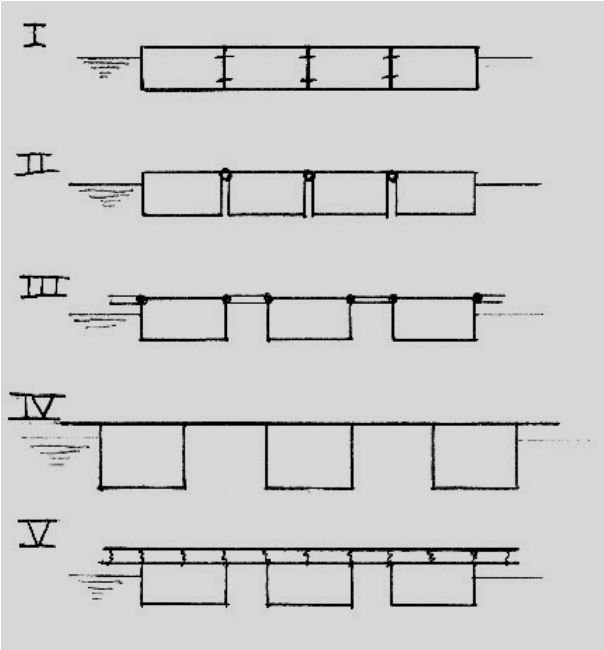
The deck structures can be divided into three groups:

1. The deck is formed by the top of the floaters
2. The deck is formed by the top of the floaters and cantilevering deck structures
3. The terminal consists of floaters and a separate deck structure

In the first group, the pontoons have both the function of floater (bearing the top load and creating sufficient buoyancy force) and of the deck slab. In group 3 these functions are separated. Group 2 is a combination of the two.

The following alternatives for the deck structure are considered:

I. Large pontoons joined rigidly.
II. Large pontoons connected by pivots (hinged).
III. System of individual pontoons linked with each other through articulated deck sections (bridges)
IV. System of individual pontoons spanned by a continuous deck slab.
V. System of individual pontoons, linked by a deck supporting system that allows uniform load distribution between two adjacent pontoons and spanned by a continuous deck slab.



The first two alternatives can be classified into the first group. Alternative III consists of floaters with cantilever deck structures. Alternative IV and V both represent separate deck structures.

Figure 1.2: Structural alternatives

The deck structures are judged on their structural strength, the necessary material use, behaviour during operation, construction time and costs.

The first group of alternatives will experience a large buoyancy force and will thus be able to bear large loads. A very rigid connection between the pontoons will ensure small relative motions and will comply with the operational criteria of the crane track. Another advantage of alternatives I and II, and to a lesser extent alternative III, is that they can be coupled and decoupled easily without a spanning deck structure. This is valid during construction of the terminal, but also for maintenance or disassembling of the terminal. For this reason, alternative I and II are selected as the best solution.

Separate floaters with a deck structure will experience a smaller buoyancy force. These structures are mostly applied to save material and costs, when the loads are relatively small.
A separate deck structure will have less structural strength and stiffness, unless a very large structural height is applied with a large material use. As especially the crane track loads are large and a continuous storage yard is preferable, pontoons joined by bridges or short-span deck sections are not desirable.

A completely separate deck structure such as group 3 has to be very stiff to distribute the loads to the floaters. A stiff steel deck with large dimensions can be very expensive, while a concrete deck is very heavy. The dead load of the deck necessitates enlargement of the pontoons while the stiffness of the deck between the pontoons may be too small regarding strict crane track requirements. For this reason, alternative III, IV and V do not seem preferable from a capital-cost point of view. The construction time of the terminal is lengthened when a deck structure still has to be constructed when all pontoons are already coupled. It may prove to be very difficult to disassemble a terminal built up out of floaters with a continuous deck structure.

The most economical solution would be one stiff structure (one long pontoon), but this is only feasible for a pier structure. The dimensions at hand are too large. A very large stiff floating platform also has the maximum achievable stability and a good dynamic behaviour and it can fulfil the requirements of the crane-track performance. The material consumption will be large, as the concrete pontoons are large and high.

Alternative V presents a very interesting system to keep the floating platform levelled horizontally under loads and perhaps movements. The system may partly replace a dynamic ballast system. It does however seem to be an option for structures with smaller loads and a smaller area. To construct a deck system stiff enough for these loads and build it over the entire area of the terminal will be very expensive indeed.

In this thesis, the simplest solution is researched. The terminal will be built up out of pontoons, which are either rigidly or flexibly connected to each other. The deck structure of the terminal consists of the connected top slabs of the pontoons.
This alternative will best meet the requirements of stability, dynamic behaviour and low costs, while large loads can be distributed well.

1.3 Horizontal structure

The floating structure will consist of several types of modular elements. The number of element types will be minimised for cost and construction optimisation.



The terminal will roughly have dimensions 2100 x 500 m. A single-body model will be computed in Delfrac to interpret the behaviour of a rigid terminal. The number of elements used and their configuration depends on the element dimensions. The effects of element sizes and configuration within the terminal could be examined by comparing two separate models. This should be done with multi-body models, one with many smaller elements and one with a few large elements. However, this is beyond the scope of this thesis.

The terminal can be constructed from several types of elements with different dimensions. The configuration of these elements can be varied:

A. All equal elements pointing in x-direction of the terminal
B. An outer boundary of quay pontoons with an inner storage yard (x-direction)
C. An outer boundary of quay pontoons and other square elements for the storage yard.
D. All equal pontoons pointing in y–direction of the terminal
E. An outer boundary of quay pontoons with an inner storage yard (y-direction)
F. Stretching bond (as a half brick wall)

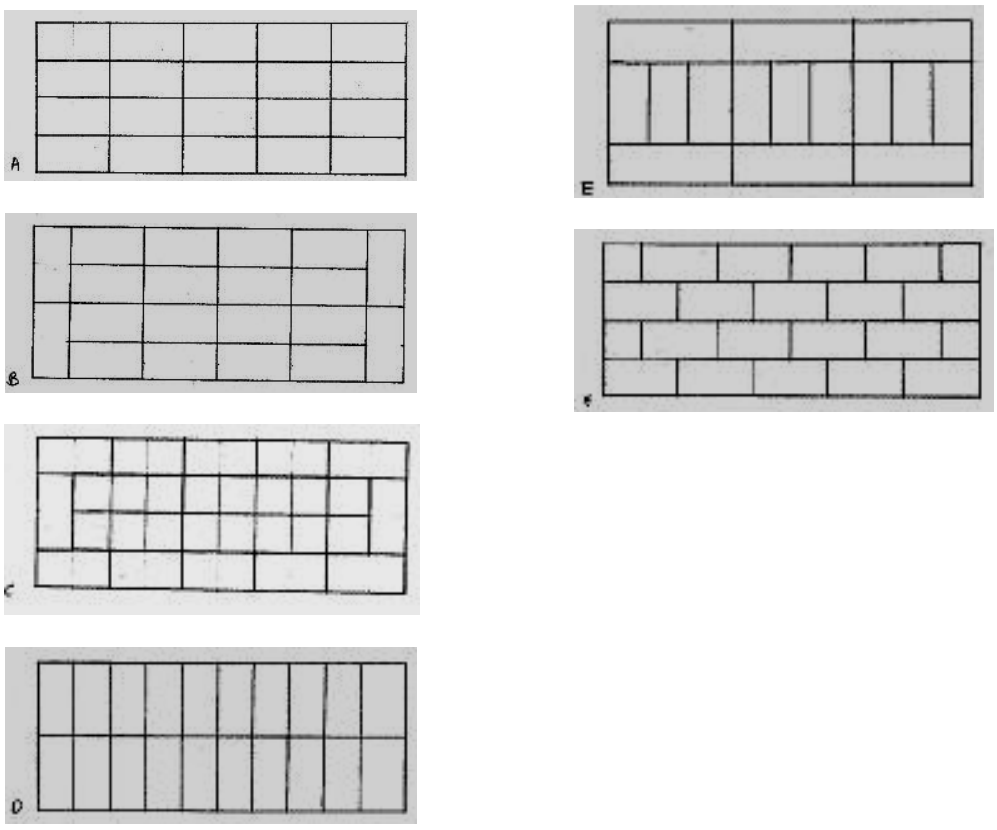


Figure 1.3: Alternative configurations of the pontoons in the terminal

The element configuration and direction within the terminal is variable. However, computing various layouts of the elements of the terminal would make the research too complex.

Alternatives A and F are based on the same principle. The configuration is simple and only one standard pontoon is needed. Configuration F, the stretching bond, is selected because it has the great advantage that the connections are better distributed over the platform. In that case the response motions of four elements are not concentrated in one point. For the outer elements, several half standard elements will be constructed. The hydrodynamic study will be done with one type of element and all elements pointed in the same direction; in the longitudinal axis of the terminal.

1.4 Vertical structure

The shape and dimensions of the standard pontoon have to be designed.
The design will take the following stages of the lifetime of the pontoon into account:

1. Construction stage
   - Production of the pontoons
   - Transport of the pontoons
   - Assembly of the pontoons (floating)
2. Operational stage
3. Disassembly (for moving or demolition of the terminal)

A desirable width of the quayside pontoon is discussed. The combination of a practical length and a necessary structural height of the pontoon will result in the selected dimension of the standard pontoon. The hydrodynamic behaviour and stability of the pontoon depend on the dimensions and the ratio of the dimensions of the pontoon. Therefore, the selection of these dimensions is influenced by the resulting hydrodynamic behaviour of the pontoon, which is very important.

The elements should be designed, bearing several aspects in mind:

- The element height influences the buoyancy and the dynamic behaviour of a pontoon.
- The larger the construction height the larger the material costs, because the terminal area remains the same.
- The bending stiffness of the pontoon will increase due to a larger structural height. A large stiffness positively affects the dynamic and structural behaviour of the terminal and the independent pontoons.
- The larger the number of small pontoons the larger the number of connections, however the smaller the connection forces.
- Extra concrete thickness results in extra costs. In addition, a larger structural height will be needed for the buoyancy.
- The dynamic behaviour of a rectangular concrete pontoon depends on the l/b ratio; the sailing properties of a rectangular pontoon will be better than of a squared pontoon.
- Connections are very expensive structural parts.
- Large elements will have a longer construction time and larger costs per element and are difficult to handle in shallow coastal water or rivers during transport and coupling.
- A small number of connectors results in large connection forces. As a result the connection will be more expensive or more difficult to construct.

The main starting point for the design of the pontoon is that the number of pontoons and connections is minimised, while the dimensions of both the pontoons and the connections are

optimised. In other words the maximum dimensions are applied, which will still cause allowable connection forces. Furthermore, large pontoons have the advantage that response motions due to waves are smaller, because the water plane area and self weight of the pontoon are large.

There are a number of limiting factors to the dimensions of the pontoons. They are listed here:
- The availability of a construction dock or the space for a new construction dock.
- The maximum depth in the dock or during transport of the pontoons.
- The maximum length or width in the construction dock or during transport.
- The manoeuvrability during transport and coupling of the pontoons.

Most submerged tunnel segments built in the Netherlands have a length smaller than 250 m. Ship docks can be longer, but are not very wide.

Large pontoons will limit the number of transports and coupling handlings.
The costs for transport, connectors and perhaps finishing construction on site will increase with the element dimensions. These costs should be compared to the costs for a greater number of transports and coupling handlings and connectors. There will be an optimal length per element. Transport can be a truly limiting factor in dimensions, if shallow waters do not provide sufficient manoeuvring space for the vessel, or if the maximum number of tugboats and other facilities is not sufficient to keep the vessel under control.

The design of the pontoon will be made for a quayside pontoon. The width of this element is very important. There should be enough room for a crane track, infrastructure, mooring facilities for the ships at berth and other pier facilities (cables and pipelines etc.) The width of the pontoon should not be too large, because then the crane track would be far outside the centre of gravity, and this results in a large eccentric concentrated load. The effect on the transversal stability could be compensated by permanent ballast in the outer compartment at the other side of the pontoon. However, these measures should only be taken when absolutely necessary. The quayside does not have to be that wide.

The crane track has an average width of 30-35 m. The pier facilities in front of the crane will require ca. 5 m of space and a road of 6 m wide would be sufficient. Other means of container transport from and to the storage yard require space next to the crane, while loading. An important design requirement is the possibility of emergency storage of containers on the quayside. From the functional point of view, a width of 50 m width will be sufficient and it is also a practical size within the configuration of the terminal. 40 m would be rather small.

The necessary structural height depends on the length, width and the dead load of the pontoon. The buoyancy force is defined as:
$F_{upwards} = \rho*g*(L*B*T)$ in kN
A larger length will result in a smaller structural height, but sufficient bending stiffness of the bending beam (pontoon) should be ensured.

The selection of the optimal length of the pontoon is interrelated with the structural principle of the terminal and mainly the quay. The length of the quayside pontoons depends on the connections to be applied regarding the crane track performance. The cranes have to move

alongside the moored ship. Either the cranes are located on one pontoon about the size of the largest ship, or they have to cross several pontoons. Summarised:

1. Medium large pontoons will be rigidly connected.
2. Very large pontoons will be connected flexibly.

The following alternative element dimensions are considered:

A. Small
   L = 50 - 100 m
   B = 50 m
   H = ca. 20 m
   Pontoon A can be constructed in Barendrecht and is easy to handle, but transport is problematic because of the construction height. It cannot float from the construction dock and should therefore be finished on site. A square element is difficult to transport.

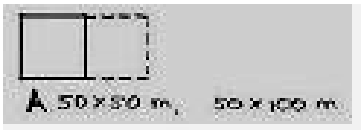


Figure 1.4: Dimensions of small pontoon

B. Medium sixed
   L = 180 m
   B = 50 m
   H = 16 m
   This element is quite large, but it can still be towed reasonably well. The element would have to be constructed at a new construction dock at the second Maasvlakte or in the Western Scheldt, just as the other larger elements.
   The width is the best structural alternative.

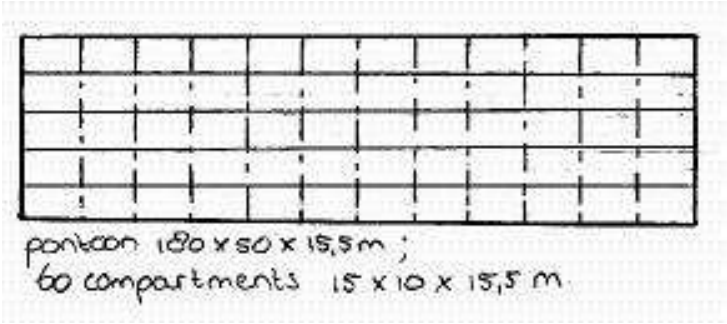


Figure 1.5: Estimate of dimensions of medium sized pontoon

C. Design ship or MOB dimensions
   L = 420 m
   B = 50 or75 m
   H = 15 m
   Pontoon E has a length comparable to the Mobile Offshore Base elements. The length is chosen, because it is equal to the design berth length. If one berth is situated on one quay element, then the crane tracks on the quay elements do not have to be rigidly connected.

The disadvantage of the size is that it will be difficult to handle and tow the element. It may be necessary to equip the element with a propellor or drive of its own. A large number of tugboats during transport will be cheaper.

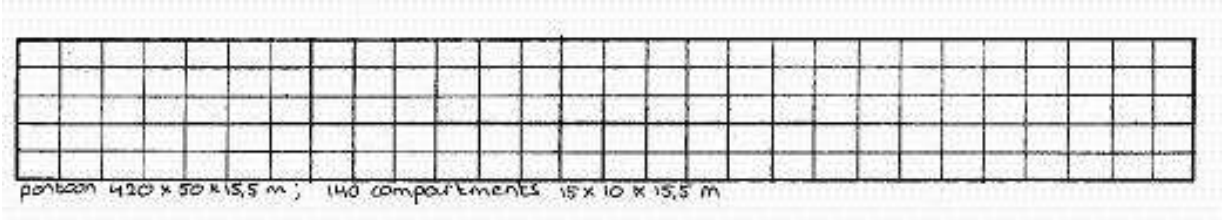


Figure 1.6: Estimate of dimensions of a large pontoon

Choice of best alternative pontoon:

The alternatives are weighed in a multi criteria analysis, which is shown in the appendices. The constructability and the hydrodynamic behaviour of the terminal are the most important criteria. The project costs are very important, but technical feasibility comes first. The constructability is mainly determined by the construction and coupling procedure of the pontoons.
Transport is an activity of short duration and will take place only once or twice and would thus be less important. However, transport aspects can be very important to construction costs. Also, disassembling and moving the platform will then not be done easily and may therefore be postponed too long.

Both pontoon B and C can be applied.
The medium sized pontoons will be rigidly connected to each other. The large pontoons can be flexibly connected to each other, while only one pontoon for each berth is needed.

A number of 5 berths at the sea pier are assumed. The terminal length will be ca. 2100 m or 2160 m (dependent on the pontoon type). The terminal width remains 500 m.

1.5 Overview of the terminal

From the floating element alternatives, two pontoon types have been selected:

B. An medium sized pontoon 180 x 50 x 16 m
C. A very large pontoon with MOB-dimensions 420 x 50 x 15 m

The last comparison with the American Mobile Offshore Base-project is explained in the appendices.

Pontoon C is rather slim. Both alternatives have their own structural advantages and disadvantages. To check their dynamic behaviour, they are both modelled in Delfrac. Based on the output and on the structural considerations (concerning construction, connection forces and costs), one of the two pontoons is chosen as the standard pontoon.

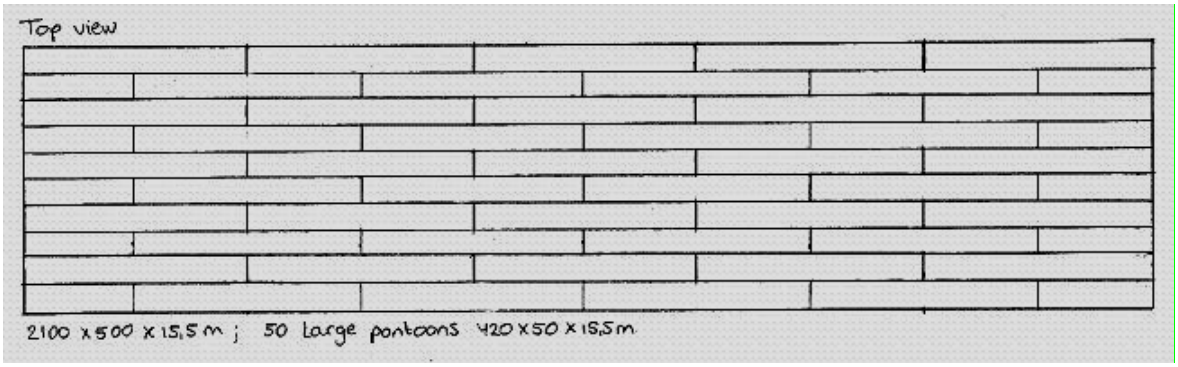


Figure 1.7: Top view of configuration of large pontoons in the terminal

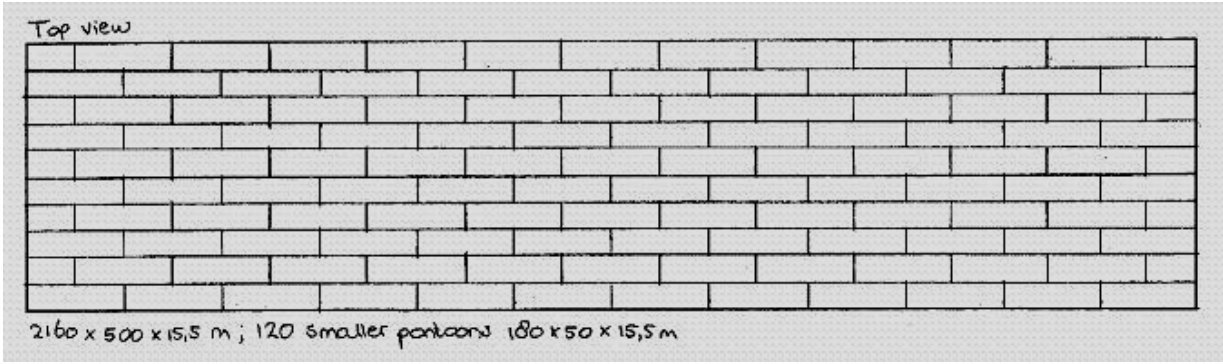


Figure 1.8: Top view of configuration of medium sized pontoons in the terminal

1.6 Limitations due to construction

The construction method of the concrete pontoon shaped elements influences many aspects of the conceptual design. Alternatives B and C from the last paragraph are studied:

- Concept B: 120 pontoons with dimensions 180 x 50 x 16 m
- Concept C: 50 pontoons with dimensions 420 x 50 x 15 m

The elements will be built in a construction dock and then towed to the location of the terminal to be coupled with the other pontoons.
The construction method within the construction dock will be discussed in part 4 of this thesis, which deals with the structural design of the pontoons.
The chosen alternatives for the overall design of the terminal will be checked by the practical limitations from the construction stage.
The construction methods will have consequences for:

- The overall design of the terminal
- The structural design of the pontoons
- Transport of the pontoons
- The coupling procedure of the pontoons

There are several possibilities regarding the location of construction of the pontoons:

1. Construct the pontoons in a dry dock or floating dock for ships.
2. Construct the pontoons in the existing (land-based) construction dock at Barendrecht. Possibly finish the pontoon at site (mainly in height)
3. Construct the pontoons at the site of the Western Scheldt Container Terminal.
4. Construct the pontoons in a existing (land-based) construction dock near Antwerp. Construct the pontoons in a new, larger land-based construction dock on the 2nd Maasvlakte in Rotterdam.

A large number of pontoons will have to be constructed. The construction, flooding and transporting cycle should be well organised. It will be expensive to rent a ship dock for a long period of time. The choice of dock should be based on the costs, location and dimensions of the dock. Every construction dock or dry dock will result in design requirements considering the dimensions of the pontoons. A ship dock has smaller dimensions than a construction dock that is normally used for immersed tunnel elements. Only one element a time can be constructed in a ship dock and especially the possible length and width of the pontoons will be restricted. A ship dry dock is assumed to be either too small or too expensive to rent for a long time.

A large land-based construction dock that is quite near to the location of the container terminal, is preferred. This leaves the construction docks in Barendrecht and Antwerp or a new construction dock on the 2nd Maasvlakte.
Construction dock Barendrecht has mainly (if not only) been used for the construction of immersed tunnels. The pontoons will be larger than tunnel elements. The construction dock is quite near to Zeeland and the Western Scheldt, but the elements will have to be transported to the sea, underneath a bridge, over sea and then into the Western Scheldt.
The following limitations have to be kept in mind:

- The dimensions of the Barendrecht construction dock are 300 x 150 x 10 m. 10 m is the average estimated height for an element, to be able to float up from the bottom of the construction dock without additional devices being used.
- The span of the bridge to be passed (max. width) is 40 m.
- A length larger than 150 m is difficult to transport through the canal bends from Barendrecht to the North Sea.

The construction dock near Antwerp has had a similar purpose as the one at Barendrecht and has similar dimensions.



Figure 1.9: Construction dock at Barendrecht

These conditions are restrictions to the maximum width and height and even of the element. The height of the vessel is very important in flooding the construction dock and towing the element out of the construction dock. A large height (or length x width) is needed to acquire sufficient buoyancy.

Another option is to finish the height and top slab of the pontoon after transport, in the Western Scheldt. The dead weight of the pontoon will be less during transport and it will be easier to lift the pontoon from the bottom to let it float.

The location of the terminal in the Western Scheldt is large. There is sufficient space to construct the terminal without causing too much hindrance to the shipping traffic. Transport of the elements would not be necessary. The cycle of construction and coupling could be easier. However, there is not so much space available on land, e.g. for a construction dock and infrastructure. The bottom slab and the walls of the pontoon should be constructed in a dry construction dock. The top slab could be finished while floating in the Western Scheldt. A dock could be made in the Western Scheldt, but this would be expensive as it is only meant for one particular project and as it has to be large. Traffic and construction work may be a nuisance to the shipping traffic in the Western Scheldt. Furthermore, logistics of materials, machinery and people to and from the pontoons during the construction of the top slabs will be complex and expensive.

Since the 2nd Maasvlakte is planned to be built within the same time interval as the Western Scheldt Container terminal, it can be assumed that a new construction dock can be developed there, which can be used for more future projects. There is plenty of space available. This assumption allows larger pontoon dimensions and easier transport, as the construction site is very near to sea.

For the development of concept B: 180 x 50 x 16 m, it is assumed that a new construction dock on the 2nd Maasvlakte will be used.

2 Conceptual and functional design of pontoons

2.1 Design procedure

A conceptual design of the terminal (and the pontoon shaped element) is made as preparation for the hydrodynamic analysis, because the geometry of the structure has to be known to do that analysis. An estimation of the loads on the structure and the dead load of the structure will lead to a calculation of the terminal's freeboard and static stability.
After analysis and evaluation of the hydrodynamics, the elements of the terminal will be designed further.
The concrete thickness of the pontoon walls and floors has been estimated. All calculations are done for both pontoon alternatives and for a single-body rigid terminal; they are included in the appendices. In Part 4 of this thesis, the results of the hydrodynamic analysis will be translated into structural loads and a conceptual design will be made of the pontoons.

2.2 Freeboard determination and ballast system

The draught and freeboard of a floating structure are part of the functional and structural requirements. They also follow from the dimensions of the structure and the applied amount of loads and ballast on or in it. The main requirement regarding the vertical position of the pontoons is that the freeboard of the pontoons has to be larger than the design wave. The deck of the terminal has to remain almost horizontal, within the operational criteria of the container crane and the activities on the terminal.

Buoyancy force: $F_{upwards} = \rho*g*(L*B*T)$ in kN
The larger the underwater volume of the structure the larger the buoyancy force. The freeboard of a floating structure can be controlled by a combination of deck loads and a ballast system based on sensor readings or an alarm system.
When considering one compartment, a ballast system has both advantages and disadvantages:

Advantages ballast system	**Disadvantages ballast system**
- good control of freeboard and angle of the pontoons.	- costs and construction time of pipelines, alarm system, pumps.
- levelling connected pontoons with different loads and thus decreasing connection forces.	- ballast water decreases the durability of the concrete of the inside of the pontoon
- smaller bending moments in bottom slab due to buoyancy force if compartment is never assumed to be empty.	- the inner space of the pontoons cannot be used for other purposes.
	- maintenance is more difficult.

For the functionality of the floating terminal, it would be preferable not to apply ballast. Also the cost for a very large ballast system will be high. However, structural requirements lead to the

assumption that a ballast system will be needed to keep the connected pontoons on an equal vertical level, irrespective of the difference in (storage and traffic) loads per pontoon, thus ensuring a horizontal continuous deck. If no ballast is applied, the connections between the pontoons will have to bear these forces. The connection forces will already be large due to the response motions to the waves. It is assumed the connectors will not be able to bear these loads. Therefore, a ballast system will be applied. The connections of the pontoons and the pontoons themselves will not be designed to withstand the forces due to the difference in buoyancy of a fully loaded pontoon next to an empty pontoon.

This ballast system will react too slowly to compensate movements due to container processing, but it will be able to keep the different pontoons at the same level. The pontoon dimensions will be determined for the situation of maximum loads (the governing load) and empty ballast tanks.

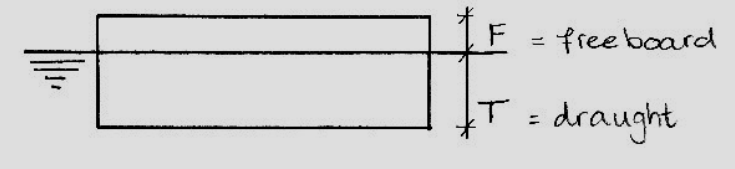


Figure 2.1: Draught and freeboard of a pontoon

Different stages and situations in the structure's lifetime affect the freeboard and draught;

- Initial freeboard F, when floating up from the construction dock, only loaded by self weight.
- Freeboard during transport of the pontoon is small (1to 10% of the structural height); ballasted, but with small loads.
- Freeboard during operation is fixed and defined by H_s, and mooring conditions for the ships at berth; the freeboard is determined by loads and ballast and their proportions. Quayside loads due to the crane track are permanent. If storage loads decrease temporarily, more ballast can be applied.
- For a possible transport situation at a shallow river or ship lock the transport draught should be decreased by losing ballast.

To achieve a large freeboard and a transversely stable structure under maximum loading, the pontoon should have a large underwater volume and a small self weight; reasonably small concrete thickness and large inner space. Small dimensions like length and width would have to be compensated for by a large height.
The pontoon does not have to be ballasted during transport, if the initial static stability (GM) is sufficient. However, a lower centre of gravity will increase stability; the floating structure may also be easier to control. However, the larger the draught during transport, the larger resistance under water, which means the tug boats will need more power to tow the structure. The conclusion would be to choose a larger freeboard that combines with a sufficient amount of power of the tug boats.

The minimum operational freeboard during construction will be 2,06 m, the 1/10 years design wave. The maximum design wave of 1/100 years has a wave height of 2,37 m.

The terminal will be designed for the 1/100 years wave. Since standing waves ($H = 2 * H_i$) in front of the quay may be possible, the entire wave height is taken into account instead of the amplitude: a factor 2. A retaining wall on one side of the quayside pontoon would cause an eccentric load on an already eccentrically loaded quay element.

With a fixed freeboard F of 2,37 m and the governing storage load, the necessary draught T is calculated. The structural height H = T + F

The following dimensions are the results of the calculations
for the alternative pontoons A and B:

- Alternative B: medium-sized
  L x B x H = 180 x 50 x 16 m
- Alternative C: MOB-dimensions
  L x B x H = 420 x 50 x 15 m

The freeboard and height calculations were made for:

1. A quay element with the crane track load and traffic load
2. A storage element with maximum terrain storage load and traffic load
3. A single-body rigid terminal with total vertical loads

The crane load was assumed to be governing, but the maximum total weight Q of the stored containers is much larger and determines the draught needed for sufficient buoyancy.
The storage elements form the largest part of the terminal. They need a large draught.
The necessary height of the quayside pontoons is smaller than the height of the storage pontoons. Either different pontoons are applied with equal freeboard but different draught, or one standard pontoon is chosen for all elements. One standard pontoon is chosen, because that will result in a practical and simple standard construction method of the pontoons. Furthermore, the advantage of a large draught is a large static stability and a large hydrodynamic mass causing a better hydrodynamic stability. These are important advantages for the front pontoons, which directly face the incoming waves.

The initial freeboard without ballast is calculated for a single medium-sized and a single large pontoon *[Appendix Part 2]* :

- Medium-sized pontoon B: 180 * 50 m

F = 7,52 m
T = 8,48 m

- Large pontoon C: 420 * 50 m

F = 7,29 m
T = 8,21 m

2.3 Section properties of the pontoons

The elements of the terminal are concrete pontoons. These pontoons are designed symmetrical in x, y and z-direction. Symmetry in relation to the z-axis is important for the static stability, especially in transverse direction.
The structure's freeboard and vertical deck load affect the vertical position of the centre of gravity of the structure. The position of the centre of gravity is important for the static stability.
The pontoon shaped element is a rectangular body, which is assumed to be symmetrical in 3 axes for the hydrodynamic analysis. The exact dimensions and concrete thickness are determined in *[Part 4: Structural design]* of this thesis. The horizontal position of the centre of gravity is in both directions (x,y) at the middle of the cross section, because of the pontoon's symmetry.

Vertical Centre of Gravity:
Initial Centre of gravity = ½ * H from own mass M [kN]
Loads:

- Storage: 60 kN/m^2
  c.o.g. = H + 6,02 m
- Crane: 800 kN/m
  c.o.g. = H + 20 m
  a container crane probably has 1/2 of its weight in the lowest 2/3 of the height. The crane itself has an estimated height of 30 m, the Centre of Gravity is located at 2/3 of the height.
- Traffic = 3 kN/m^2 + point loads several traffic and crane types
  c.o.g. = H + 2 m

The vertical position of the Centre of Gravity is determined by S=A*y. These values are shown in the appendices.

The following section properties of the medium-sized pontoon B and the large pontoon C are determined in Appendix Part 2:

- Structure dimensions
- Compartments
- Cross section area
- Concrete volume
- Moments of Inertia I of pontoons and platform
- Radii of gyration k of pontoons and platform

This information is mainly needed to do the hydrodynamic analysis in Part 3 of this thesis.

3 Hydrostatic stability of the terminal and the pontoons

The hydrostatic stability of the alternatives is calculated in the appendices of this report. The static stability in transverse direction is most important. In longitudinal direction stability is mainly important for ships when the cargo load is changed. The concrete pontoon is expected to be loaded or ballasted constantly in longitudinal direction. It will however be researched. The stability analysis is preceded by weight- and draught/freeboard calculations. The draught value and GM distance are used as input for the dynamic analysis.

3.1 Motions due to static loads

Several translations and rotations of a floating structure are a result of static loads, which bring the structure out of its balance. The motions are:

- Offset: a horizontal translation
- Heel: a rotation around the structure's longitudinal x-axis (through CoG)
- Trim: a rotation around the structure's transversal y-axis (through CoG)

These motions are non-oscillatory, which means that the structure will only return to its equilibrium position if the static load is changed. Angles of trim and heel of a ship are mainly caused by the uneven loading of cargo. This is also applicable to the pontoon shaped elements. A horizontal translation can be caused by the following loads:

- Current
- Wind
- Anchorage forces
- Mean wave drift forces

A vertical translation or rotation can be caused by the following loads:

- Deck loads
- Ballast in compartments

3.2 Static stability

The structural layout of the pontoon will be selected preceding to the load distribution and structural performance. The shape and dimensions of the compartments affect the stability of the pontoon, when they are filled with ballast water. The water plane will move with the pontoon. Small compartments will restrain large movements of the water volume. They will allow precise application of ballast and smaller influence of liquid ballast on stability.

Stability should be inspected for several stages during the lifetime of the structure:

1. Initial stability of the pontoons
2. Stability during transport of pontoon (including ballast)
3. stability during operation of pontoon (including ballast)
4. stability during operation of a rigid single-body terminal



5. Stability affected by movements causing trim and heel of a pontoon during operation
6. Stability in survival conditions; when 2 pontoon compartments are flooded due to collision damage

The following aspects affect the static stability of a single pontoon:

1. Load and ballast determine draught and freeboard, during transport and operation
2. Container processing; the change or shift in mass of containers or ballast (as compensation) may lead to a shift, removal or addition of masses onboard the pontoon. Consequently, the centre of gravity of the pontoon changes.
3. Quay/stacks cranes; the movements of portainers or Rail Mounted Gantry cranes (RMG) cause a shift in mass on the pontoon. These motions may cause trim or heel of the structure

The second and third aspects are not taken into account in the calculations, as the quay pontoons will be connected to the pontoons behind the first row. These loads will be too small to have much consequence on the stability of the entire terminal.

Initial static stability depends on the structure's geometry and dead weight only. The water plane area of the pontoon has an influence on the static stability and the hydrodynamic rolling motion of the pontoon. The squared moment of area of the water plane area of a statically stable structure is large.
The stability calculation is concentrated on process number 3 (during operation). However, it is assumed that the movements due to container processing will not affect the stability of the pontoons very much for two reasons:

- The movements are rather slow
- The processing of several containers at once causes a shift in weight that is small compared to the weight of the large pontoons.

When floating at even keel (the water line and the floating element both being horizontal), the Centre of Buoyancy B (the centre of the replaced volume of water) is positioned above the Centre of Gravity G in a vertical line through G. The pontoon is statically stable in longitudinal and transverse direction, when the maximum caused trimming and heeling moments are counteracted by a returning moment that drives the structure back into its equilibrium position. Thus the element will not capsize due to activities onboard.
The returning moment is largest when the distance between the Centre of Gravity G and the Metacentre M, called GM-value or the height of the Metacentre h_m, is large. However, too large a value for GM will cause the structure to be over-stabilised, which will cause sea-sickness.

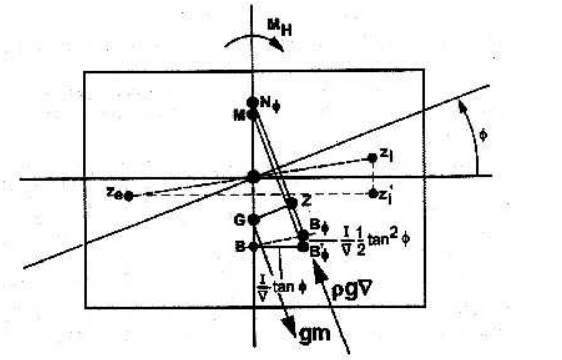


Figure 3.1: Transverse stability calculation

A stability requirement is found as a compromise between the following competing influences:

- For stability in water, GM should be large
- For manoeuvrability and a stable behaviour in the water, the draught T should be large, thus h_m should be small.
- Resonant rolling motions should be prevented so as to reduce the risk of capsizing; thus h_m should not be too large
- A small T can cause the body to be difficult to control and to ship large quantities of water. Therefore, T should be large and m should be small.

Recommended values for concrete pontoons or tunnel segments in literature are found mostly between GM = 0,20 and 1,0 m *[J.G. de Gijt, 2003]*
Both the pontoons and the single-body terminal have a metacentric height of many metres *[Appendix Part 2]*. The longitudinal value of GM is much larger, because of the water plane inertia moment. The long rectangular pontoons are therefore very stable in the water. However, the larger the pontoons, the more difficult the handling and the steering manoeuvrability decreases. Resonant rolling motions will occur. The chance of capsizing is reduced due to the large hydrodynamic mass of the pontoon.
When necessary, other structural measures may be applied to improve the floating stability (behaviour), such as adding ballast, changing the water plane area of the structure or using buoyancy aids.

3.2.1 Calculation of transverse and longitudinal stability

The initial stability is in fact calculated, because only small angles of trim etc are taken into account.
Basis: $\Sigma H = 0$ $\Sigma V = 0$ $\Sigma M = 0$
Draught $T = M / (\rho_w * B * L)$
$GM = KB + BM - KG$ in which

K = Keel
B = Centre of Buoyancy
G = centre of Gravity
KB = ½* T
$KG = ½ * H_c + \Delta$

BG = KG – KB positive if G lies above B
Δ = vertical delta position Centre of Gravity
∇ = underwater volume
KG is calculated from $G_{concrete}$, $G_{ballast}$ and G_{load}

Condition: KG > KB
$BM_T = I_T / \nabla = 1/12*L*B^3 / T*L*B = 1/12*B^2 / T$ transverse
Determining the metacentric height:
Overturning moment $M_{list} = M_k = G*Z_\varphi*\rho*g*\nabla$ of which $\rho*g*\nabla = \gamma*\nabla$ = underwater mass
The Scribanti-formula can be applied for (max. trimφ) angles smaller than the deck immersion angle. The dimensions of the pontoon cause the value of GM to be very large. The deck immersion angle will be very large, as the pontoon is over-stable. The allowable trim angles will be negligible compared to the deck immersion angle, therefore the Scribanti-formula will not be used.

When taking the ballast water surface inside the pontoon into account, the calculation method changes to:
$GM = 1/\nabla * (I_{wp} - \Sigma I_{bw}) – BG$. These calculations have been done in the appendices. It is shown, that the ballast has a negligible influence on the GM-value and the pontoon stability. Therefore, it will also be negligible for the entire terminal.

The longitudinal stability is calculated in exactly the same way as the transversal stability. The only difference is the water plane inertia moment. The size and weight of the pontoon structure cause the longitudinal GM-value to be very large. The calculation is mainly useful as input for the dynamic analysis and the estimation of a natural frequency of the structure.
Condition: KG > KB
GM= KB + BM - KG
$BM_L = I_L / \nabla = 1/12*L^3*B / T*L*B = 1/12*L^2 / T$ longitudinal
Determining the metacentric height:
Overturning moment $M_{list} = M_k = G*Z_\varphi*\rho*g*\nabla$ of which $\rho*g*\nabla = \gamma*\nabla$ = underwater mass
These are the main results from the calculations in the appendices:

structure	**Initial GM [m]**	**GM transport incl liquid ballast [m]**	**GM operation [m]**	**GM operation incl liquid ballast [m]**
Pontoon 180 * 50 m				
Transversal	21,7	13,2	24,0	23,1
Longitudinal	318,0	180,1	320,2	318,4
Pontoon 420 * 50 m				
Transversal	22,7	16,1	11,5	11,0
Longitudinal	1794,8	1018,7	1075,5	1074,4
Terminal 2100 * 500 m				
Transversal	2925,3		2923,2	
longitudinal	50073		50071	

3.3 ULS survival condition

At the beginning of this chapter, 6 phases were mentioned in which stability is an issue. In the sixth mentioned situation, under survival conditions, separate requirements are applied for the terminal stability. The pontoon shaped element should remain afloat and not capsize for a design condition of a 1/100 yrs wave height Hs = 2,37 m, when two compartments are damaged and flooded with sea water, e.g. by ship collision.
The same kind of requirement should go for the entire terminal; when two quay elements (most vulnerable) are damaged and flooded at the front side by a ship, the terminal platform should remain afloat and stable. At this design stadium, the check calculation will be done for one quay pontoon shaped element:

$M_{concrete}$ element 180 x 50 m = 780445 kN
G_{loads} = 202783 or 490513 (storage) kN
$M_{concrete}$ element 420 x 50 m = 1758689 kN
G_{loads} = 239431 (quay) or 1141393 (storage) kN

V compartment = 15 x 10 x 14,7 m = 2205 m^3
$\rho_w g_{\ sea\ water}$ = 10,25 kN/m^3

$G_{2\ flooded\ comp}$ = 2 x 2205 x 10,25 = 45202,5 kN
Δ T = ΔM / ρ_wg*B*L = 0,49 m = Δfreeboard
remaining freeboard F = H – (T + ΔT) = 2,14 m < 2,37
The Δfreeboard should be smaller than the determined freeboard. The determined freeboard is equal to the design wave height. The platform deck will thus remain above water.
Compared to the self weight of the pontoon and the storage load on the deck, the weight of the compartments filled with water is rather small. Therefore, the effect on the freeboard is limited. The pontoon will not sink when only two compartments are flooded.

$M_{concrete}$ terminal (from 180 x 50 m elements) =91497238 kN
G_{loads} = 49281500 kN

$V_{inner\ pontoon}$ = 2205 x 12 x 2 elements = 132300 m^3
$G_{2\ flooded\ pontoons}$ = 2 x 132300 x 10,25 = 2712150 kN

Δ T = ΔM / (ρ_wg*B*L) = 0,25 m
remaining freeboard F = H – (T + ΔT) = 1,85 m < 2,37
The larger the pontoon dimensions and therefore a smaller amount of pontoons, the greater the effect of one pontoon sinking. The survival stability conditions for the entire terminal are more difficult to satisfy when large pontoons are applied.

4 Geometry of the element and the connections

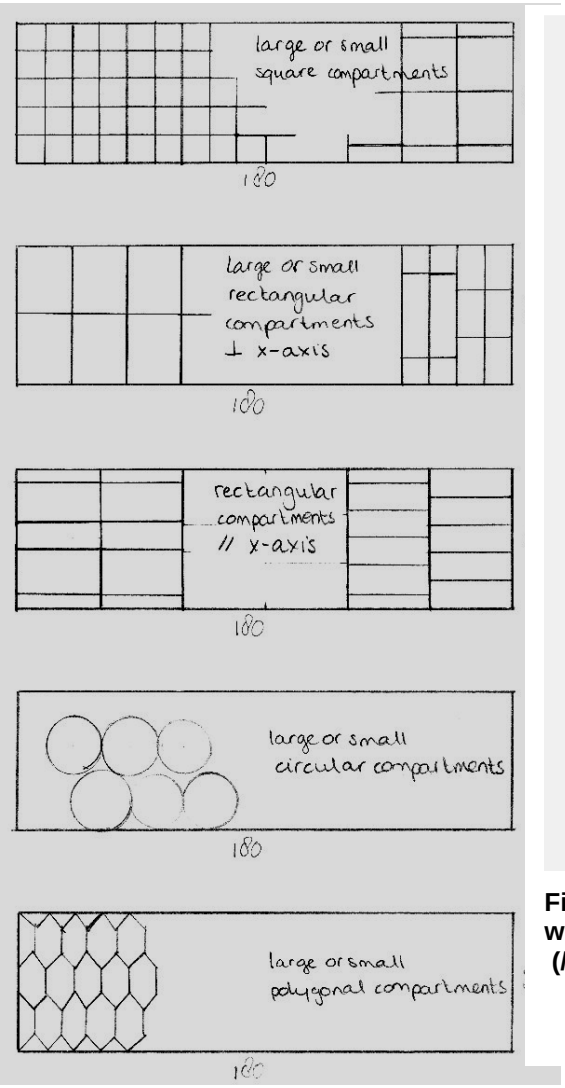
The floating terminal consists of pontoons. The layout of these pontoons within the platform has already been chosen. The geometry of the structural members and the connectors of one pontoon shaped element should be designed and optimised.

Process analysis:
The main part of the geometry of a pontoon is the compartment distribution. The dimensions of the compartments and inner walls have to be chosen, as well as the thickness of the outer shell of the pontoon. The geometry of the inner walls has to be considered in relation to the geometry of the connectors on the outside of the pontoon, which determines the distribution and introduction of the connection forces between the pontoons. Several boundary conditions follow from earlier made choices in this thesis, such as the shape and layout of the elements of the terminal. The choices from the dynamic analysis in this thesis initiate the working method of first choosing the pontoon geometry and then developing the connector geometry in accordance. Several starting points have been defined on the basis of limited time during this Master thesis. More alternatives could have been researched, but a seemingly practical structure has been further developed.

Functional analysis:
Every pontoon shaped element is supposed to carry its own deck load and the connection and mooring forces, transferred through the connectors. The connectors restrict the motions of the elements and as a result, impose loads on the hull structure of every pontoon. A pontoon consists of compartments, which distribute the load over several structural members.
The compartments have the following functions, regarding the cell geometry:

- The walls stiffen the element and ensure a good force distribution.
- Distributing the deck load over inner walls.
- Controlling the bending moments in the deck plates.
- Introducing connection forces and point loads such as bolder and fender forces into the hull structure.
- Distributing the connection forces over inner walls.
- Evenly distributing the ballast water (or sand) within the element and preventing dashing of ballast water.
- The compartments, if watertight, can ensure the floating stability of a damaged (flooded) element.
- The compartments may (if the element is designed for that) be used for storage of goods or materials, or other space-use.

The shape of the compartments can be rectangular, squared, circular, triangular or polygonal. These alternatives are shown in fig. 4.1. Since the pontoon element has a rectangular shape and as this shape is easy to construct, it is assumed in this thesis that rectangular or squared compartments would provide the best solution. The straight walls are very useful for the distribution of for instance the crane track load. Construction time will be shorter if a simple shape of the walls is applied.



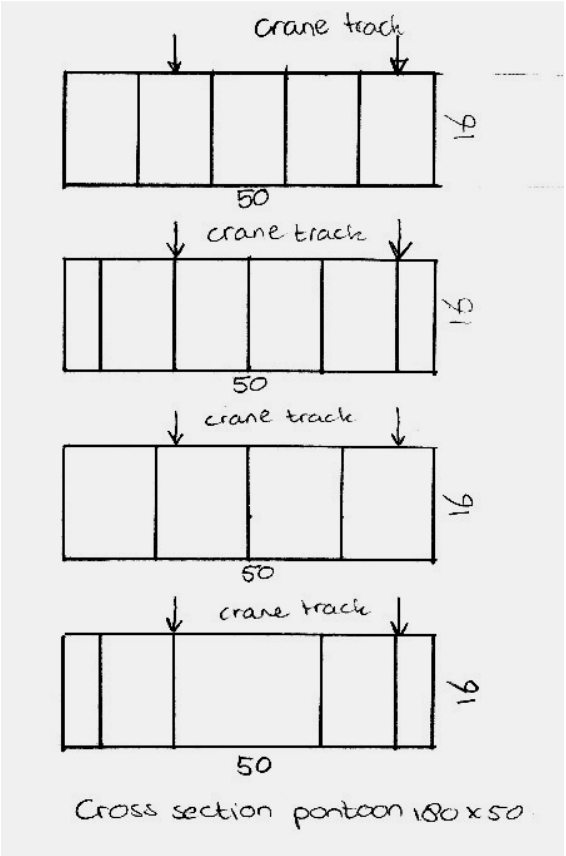


Figure 4.1.b: Alternative cross sections of a pontoon with rectangular compartments in longitudinal direction (// X-axis)

Figure 4.1.a: Top view of alternative geometrical designs of compartments: squared, rectangular // X-axis or Y-axis, circular, polygonal

The geometry and the dimensions of the compartments and the geometry of the connections between the pontoons are interrelated. The dimensions and geometry of both are selected using a design spiral.

The connectors between the elements have the following functions:

- To restrict the response motions of the elements (and cause a certain stiffness)
- To ensure a continuous deck
- To transfer mooring forces

The geometry of compartments is dependent on:

- The maximum span of plates regarding the internal forces from the deck load
- The maximum dead weight of the pontoon regarding the buoyancy force
- The preferred stiffness of the pontoon, when considered as a beam (interaction with the waves and elastic behaviour)
- The geometry of the connectors and preferred introduction of connection forces
- The location of concentrated deck loads, preferably on top of a structural member such as an inner wall
- The preferred number of rigid connections of structural members of the pontoon

The dimensions of the compartments can be optimised, while the height has already been determined from the first design on buoyancy and static and dynamic stability. The following table presents the (dis)advantages of small and large compartments in the pontoon:

Cell type	advantages	disadvantages
Small cells	Small spans and small internal forces	Large self weight dependent on wall thickness
	Large concrete area to introduce connection forces	More walls means difficult construction and longer construction time
	Large pontoon stiffness	
Large cells	Large inner space	Large internal forces in plates
	Small self weight dependent on wall thickness	Large loads from connections; less area in cross section
	Shorter construction time	Smaller pontoon stiffness

An optimum in cell dimensions and wall thickness will have to be found. Small compartments which ensure a small deck span and a large concrete cross section would preferably be combined with a small wall thickness.

The following assumptions lead to the selection of the dimensions of the compartments, because the time for research is limited during this thesis:

- The pontoon to be rigidly connected has dimensions 180x50 m.
- Rectangular cells parallel to the longitudinal axis of the pontoon are assumed to be the most favourable type of compartment.
- The location of the inner walls in the cross section of the pontoons is partly determined by the location of the crane tracks, as two inner wall should be placed beneath the crane tracks because of load distribution. The first crane track is located 5 m from the front, the next 35 m from the front of the quay.
- The outer shell of the pontoon has a large thickness, estimated at 900 mm, because of expected bending moments and torsion.



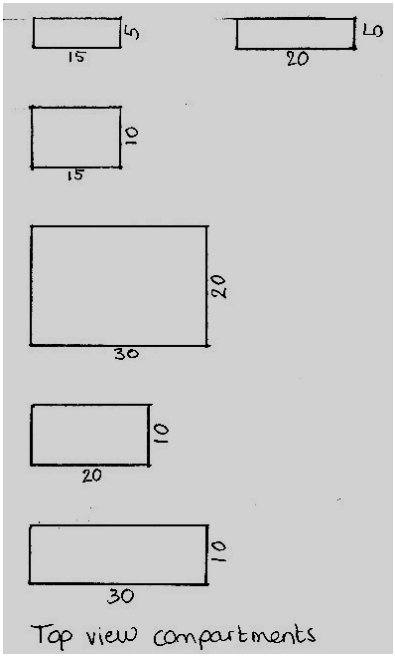


Figure 4.1: Top view of possible dimensions of compartments

The dimensions of the compartments will be 15 x 10 m and 15 x 5 m in the outer row. The length of the compartments is chosen parallel to the longitudinal x-axis of the pontoon. This geometry is shown in fig. 4.3. The same cross section is recommended for a storage pontoon.

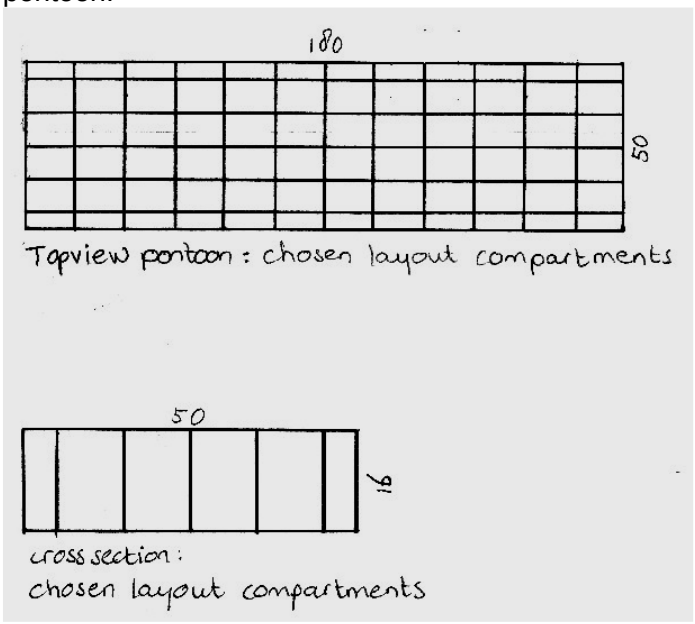


Figure 4.2: Top view and cross section of the geometrical design of the compartments of the pontoon

Part 3:

Hydrodynamics of the Floating Terminal

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1 Dynamic forces

Dynamic forces can be considered as a summation of sinusoidal loads.
Fluctuating forces cause a dynamic response.
They cause:

- a load [kN/m^2] on the floating structure
- an excitation of the structure into oscillatory response motions.

Dynamic forces can be divided in:

- first order wave forces
- second order low frequency wave drift forces
- wave forces from swell
- wave forces from waves generated by passing ships
- Turbulence (assumed small, not modelled)

Dynamic forces excite all types of structure into the same, defined modes of motion. The two main types of structure; a rigid and an elastic floating platform, will suffer the same type of response motions from a certain dynamic load.
An overview:

Dynamic forces	***Result in***	***Oscillatory motions of rigid structure***	***Oscillatory motions of elastic structure***
first order wave forces	- load [kN/m^2] on terminal - oscillatory response motions	- heave - roll - pitch	- heave - roll - pitch
Second order low frequency wave drift forces	- load [kN/m^2] on terminal - oscillatory response motions	- surge - sway - yaw	- surge - sway - yaw
Swell waves	- small load [kN/m^2] on terminal; small Hs - oscillatory response motions	- heave - roll - pitch	- heave - roll - pitch
Current (quasi-static)	- oscillatory response motions		

The defined modes of motion will be explained in the next chapter.
The dynamic forces are already mentioned for the greatest part, in the hydraulic boundary conditions of the Western Scheldt, but here the dynamic loads are summarised and quantified.



1.1 Wave frequency loads

The incoming type of waves at the WCT can be:
- wind waves; irregular waves divided in sea waves and long period swell waves
- Astronomical tides
- Short waves caused by passing ships; neglected for the time being

Wave frequency loads are mainly loads from wind waves with a rather short period. There is sufficient data on these waves in the Western Scheldt. There is little or no data on swell or ship waves.
These are the governing wave data:

Wave height Hs	***Wave period T_p***	***Wave frequency ω_p***
2,37	6,2	1,013
2,13	5,7	1,102
2,06	5,2	1,208
1,82	4,8	1,309
0,96	4,0	1,745
0,81	3,1	2,027

The water depth d in the Western Scheldt is ca. 20 m.
Through linear short wave theory, the wave length can be determined:
Amplitude a = ½ H
Wave frequency $\omega = 2\pi/T$
Wave speed $c = \omega/k = L/T$
Wave number $k = 2\pi/L$

Deep water:
Wave length in deep water $L_0 = gT^2 / 2\pi$
Wave length $L = 2\pi/k$

In shallow water:
Put d/L_0 in diagram and determine d / λ
In the Western Scheldt;
The longest occurring wave has T = 6 s and L = 58,7 m
The shortest occurring wave has T = 3,1 s and L = 15,0 m.

The first order wave forces will be determined with DELFRAC. These wave forces have been estimated using the Sainflou method in the appendices of part 3: q = 323,20 kN/m[1] (length of the terminal).

1.2 Swell

Swell waves are long period wind waves generated by local wind; however, they have travelled away from the local wind area. These long waves have long periods. In general, a floating body will have time enough to follow the movements of the water plane. Therefore the body will not experience great response motions due to swell. The occurrence of swell in the Western Scheldt is small. However, there is a possibility that swell waves from a storm on the North Sea or, for instance, near the United Kingdom, will travel towards the Dutch coast and into the Western Scheldt. These waves are assumed to have a period between 8 and 12 seconds, with a wave height varying from 0,2 to 0,5m.

T [s]	ω [rad/s]
8	0,7
12	0,5

As the swell frequency is smaller than the short wave frequency, it might be nearer to the natural frequencies of the pontoon shaped element, in which case excitation might result in large response motions. Therefore, response to these wave frequencies will be examined. Swell waves are taken into account in the hydrodynamic analysis with DELFRAC.

1.3 Low frequency wave drift forces

Low frequency wave drift forces in combination with certain spring characteristics of the mooring system of the terminal can cause oscillatory motions of the floating body, as this body presumably has low natural frequencies.
These drift forces may especially be decisive in determining the anchorage forces in the fixation of the terminal. These forces will be introduced into the elements of the terminal and distributed.
The mean drift forces are quantified in the appendices.
Regular waves:

$\bar{F}' = \frac{1}{2}\rho g.\zeta_{ar}^2$ [kN/m^1 length of the structure]

Considering a wave spectrum of irregular waves, the mean wave drift force becomes:

$\bar{F}' = \sum \frac{1}{2}\rho g.\zeta_a^2(\omega)$ with amplitude of the wave spectrum:

$$S_\zeta(\omega).d\omega = \frac{1}{2}\zeta_a^2(\omega) = \int_0^\infty S_\zeta(\omega).d\omega = m_0$$

$$\bar{F}' = \rho g.\int_0^\infty S_\zeta(\omega).d\omega = \rho g.m_0$$

Also: $H_{1/3} = 4\sqrt{m_0}$ so that $m_0 = \frac{1}{16} H_{1/3}^2$

$$\bar{F}' = \frac{1}{16} \rho g . H_{1/3}^2$$

The calculations are shown in the appendices.

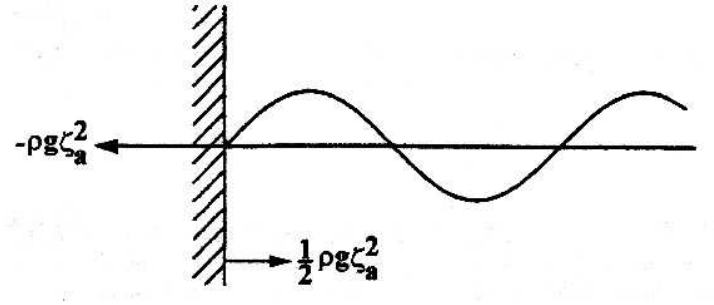


Figure 1.1: Wave drift forces

Low frequency wave drift forces are mainly important to flexible (mooring) structures such as a Single Point Mooring system. As this terminal is moored by a more rigid Multi Buoy Mooring system these forces are of less importance.
The drift forces can be calculated with DELFRAC, but as the scope of this thesis is not to design the mooring system of the terminal, the drift forces are estimated in a manual calculation in the appendices of part 3. The result is q = 28,24 kN/m^{1}over the length of the terminal.

2 Motions of a structure in waves

The description of response motions of a floating structure in waves is based on the definitions of motions in the six degrees of freedom:

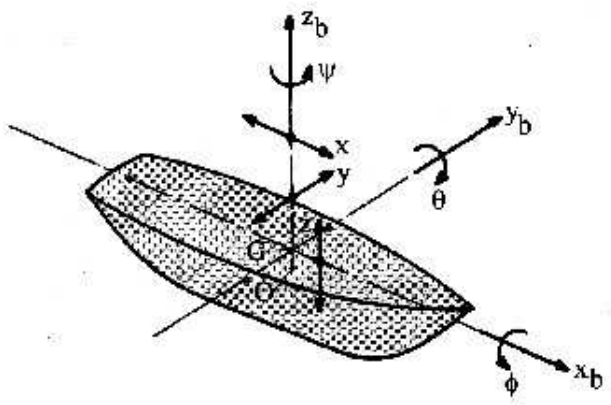


Figure 2.1: Response motions of a floating object

Three translations:
- Surge: a horizontal translation along the longitudinal x-axis of the body
- Sway: a horizontal translation along the transversal y-axis of the body
- Heave: a vertical translation along the vertical z-axis of the body.

Three rotations:
- Roll: a rotation around the longitudinal x-axis (φ)
- Pitch: a rotation around the transversal y-axis (θ)
- Yaw: a rotation around the vertical z-axis (ψ)

An anchored floating structure produces six equations of motion, one in every degree of freedom.
The six defined modes of motion each have their own natural frequency in a structure.
All of these oscillatory motions are mainly caused by the wave forces on the structure. The wave force is a dynamic; sinusoidal or oscillatory force itself. The dynamic wave force excites the floating structure in a certain frequency, different per direction of degree of freedom.
Three types of wave forces have been considered:
1. Wave forces with a frequency (ca.) equal to the incoming waves.
2. Low frequency wave drift forces.
3. Swell waves.

To attain a general idea of the motion response of a floating structure,
a simple analytical method can be used. However, this method has some problems:
The hydrodynamic mass and damping coefficients depend on the wave frequency for the roll motion and they are greatly influenced by the presence of a bottom in shallow waters. These

parameters are difficult to determine theoretically, but they are important because they influence the response motion of the structure. The simplification of the approach method comprises linearizations and assumptions, and an estimation of several parameters. One of the assumptions is, that the motion components in the different degrees of freedom are uncoupled and that the motions are made linear. This will further be discussed in the next chapter.

The terminal is mainly excited by short waves. No long waves are expected. Short waves have a relatively high frequency. The structure's natural frequencies for heave, roll and pitch response motions may coincide with a short wave frequency. These motions are the most important to examine. If their natural frequencies coincide with short wave frequencies, the amplitudes of these motions will be the largest. The natural frequency in the other motions surge, sway and yaw is expected to be lower, and may coincide with the low wave drift forces frequencies. These forces are important when determining the structure's anchorage force, but they have a very long period and have a small influence on the terminal motions. If the natural frequencies of surge and sway coïncide with swell wave frequencies while the other natural frequencies are lower than the short wave frequencies, then the surge and sway response motions may have the largest amplitudes. In this chapter, heave, roll and pitch will be further examined.

2.1 Approximation of natural frequency

With the data from the static stability calculation, the (first) natural frequency of the pontoons can be approximated. This could serve as a preparation for the hydrodynamic analysis and as a comparable value for the results.
The dynamic stability for roll motions is examined by calculating the natural period T of the pontoon.

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

The natural frequency is $1/T_0$.
The element is dynamically stable for roll motions if the natural frequency is much lower than the wave or swell frequency, because in that case the wave excitation on the element would only be very small.
If the wave frequency would be higher than the natural frequency of the element, the excitation of the element by the dynamic wave force would be much larger compared to the element motions.

In general the free equation of motion is:

$$(m+m_w)\frac{d^2x}{dt^2}+(k+k_w)\frac{dx}{dt}+(c+c_w)x=F(x,t)=0$$

A general equation for the natural frequency is

$$f=\frac{1}{2\pi}\sqrt{\frac{k}{m}(1-\frac{c}{c_c})^2}$$

By manual calculation the natural frequencies of the pontoon can only be approximated. The specific motion is assumed to occur only in the considered degree of freedom. The motions of

the pontoon are then uncoupled. The heave, roll and pitch motions are checked by hand to compare with the results from the dynamical model.

The equation of motion for roll is: $(I_{xx} + m_{\varphi\varphi})\ddot{\varphi} + N_{\varphi\varphi}.\dot{\varphi} + (\rho g \nabla . GM_T)\varphi = 0$

m_φ is the added mass coefficient for roll and ∇ is the underwater volume.
The first natural frequency is:

$$\omega_{\varphi,0} = \sqrt{\frac{\rho g \nabla GM_T}{I_{xx} + m_{\varphi\varphi}}}$$

The equation of motion for heave is:

$P = \rho g T . A_w$

$(m + a)\ddot{z} + b\dot{z} + cz = 0$ general form

$(M + m_{zz})\ddot{z} + Nzz.\dot{z} + (\rho g . A_w)z = 0$

The first natural frequency is:

$$\omega_{\varphi,0} = \sqrt{\frac{\rho g . A_w}{M + m_{zz}}}$$

The equation of motion for pitch is: $(I_{yy} + m_{\theta\theta})\ddot{\theta} + N_{\theta\theta}.\dot{\theta} + (\rho g \nabla . GM_L)\theta = 0$

The first natural frequency is:

$$\omega_{\varphi,0} = \sqrt{\frac{\rho g \nabla GM_L}{I_{yy} + m_{\theta\theta}}}$$

A few possible situations are kept in mind:

1. A low natural frequency causes great amplitudes of the response motions, when excited in the natural frequency; with a chance of motions with zero-crossings in roll. There is a chance of capsizing and resonance (under stable)
2. A high natural frequency causes a small response amplitude, and a very fast motion back to equilibrium position and in zero-crossings. Compensating the movements happens fast; the structure will vibrate in a roll motion.
3. An overstable floating body (high frequency, low natural period T) will have very short rolling motions (caused by waves), because the body is returned to equilibrium position quickly. These motions exceed the operating criteria of vertical and horizontal accelerations. The motions of an overstable floating body can easily cause seasickness, and therefore unworkable conditions.

The calculations of the natural frequencies are included in the appendices. The results are:

	ω P 420x50 m [rad/s]	ω P180x50 m [rad/s]
Roll	0.036	0.069
Heave	0.134	0.217
pitch	0.174	0.229

The wave spectrum in the Western Scheldt includes short waves between ω = 1,0 and 2,0 rad/s and swell waves between ω = 0,5 and 0,7.
The natural frequencies are more in the range of swell waves and long waves. The surge, sway and yaw natural frequencies will be smaller. There are little excitation and only small response amplitudes to be expected.

2.2 Damping

There are several types of damping. The most important are:
1. Hydrodynamic damping (by excited response waves)
2. Viscose damping (friction etc.), which is small compared to the first

The response motions of the terminal cause surface waves around the structure.
Hydrodynamic damping is caused by these response waves around the terminal, excited by the terminal itself (from the response motions), which dissipate energy from the moving structure. This wave field around the structure may counteract incoming waves.

The hydrodynamic damping becomes zero when the structure is totally submerged, so there has to be a freeboard for hydrodynamic damping to occur. A structure with large draught and large dimensions, but with a small sufficient freeboard, generates a lot of damping (in deep water). A large structural mass combined with a large part of the structure under water restricts the response movements, for instance during a storm. This will reduce anchor forces. Square and rectangular bodies produce more damping than do rounded bodies.

Viscose damping is caused by surface friction, vortices, flow separation etc.
Viscose damping is negligible in every almost every degree of freedom except roll (x).
Water is pushed from underneath the floating structure along the sides of the terminal when it rolls (in response motion).
The motions of the terminal are damped by the resistance caused by an attached volume of water, which acts as a boundary layer. This is called the added mass.
The hydrodynamic mass is a superposition of $M_{structure} + M_{added}$
The boundary layer of water around the structure offers resistance when the structure moves.

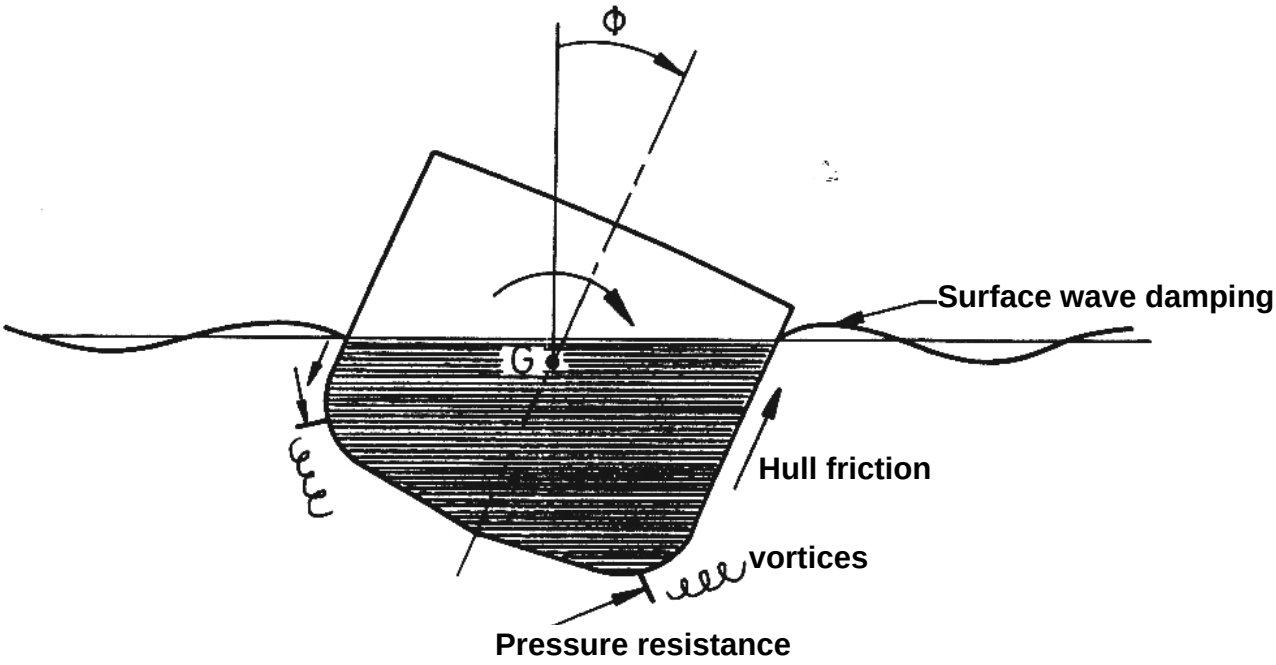


Figure 2.2: Components of damping

[Ref. Golven, scheepsbewegingen, sturen en manoeuvreren 1 by Prof. Ir. J. Gerritsma]

3 Approach method hydrodynamics of the terminal

3.1 Introduction hydrodynamics

The terminal will move in response to the dynamic forces acting on it. The elements of the terminal will act as one when connected entirely rigid. In reality, the elements will influence each other's motions. The motions of a berthed ship and an apron element will also be coupled.
The response motions to be studied are listed:

- Terminal motions as a whole
- Element motions, when connected flexibly
- Element motions during transport
- Berthed ship motions

The possible application of a breakwater is not taken into account for the time-being. No motions regarding a breakwater are calculated.
In the previous chapter a part of the theory of floating objects in waves has been discussed. The defined motions in six degrees of freedom have been mentioned. In the next chapters the expected response motions of the terminal are described. The motions and phase lag between the terminal and ship motions are neglected. The operating conditions of the terminal are taken into account. A short overview of the theory of short waves is the basis for further dynamic analysis. As the theory is in fact background information, it has been included in the appendices of this report.
In this chapter, the calculation method to be used will be discussed, the design conditions of the terminal and its elements are dealt with and a scheme is presented to research the influence of several aspects of the floating structure on its dynamical behaviour. The concentration points are the type and dimensions of the elements of the terminal, their configuration within the terminal and the element connections.
The structural alternatives generated earlier, will be considered again or evaluated from a hydrodynamic point of view.

3.2 Calculation method

The terminal motions will have to be determined as well as possible in this early design stage. In general, two approach methods can be used to determine the structures responses:

1. A **simplified approach** *[ref. I, k, 22]* method by hand may give a general idea of the motions. The method is based on the superposition principle of all uncoupled motions. The assumption of uncoupled motions is a strong simplification, as is omitting the influence of the fixation of the terminal. In reality, at least heave and pitch motions and also sway and roll motions are coupled. Heave, roll and pitch will probably be the most important motions for a large offshore structure. The fixation of the terminal to the bottom will have a favourable effect on the motions of the terminal, which is not taken into account. The size of the structure, the number of wave directions and frequencies and the possible number of elements of the terminal cause a very large amount of work. The real structure seems too



large and too complicated to use this method for realistic results. However, by describing the simplified approach, the basis for a more complex calculation is given and this will improve the understanding of the system's hydrodynamics.

2. An **advanced approach** method by using a diffraction model based on a three-dimensional potential theory and the panel method, would give more realistic results. However, a diffraction model also needs a number of assumptions and simplifications of reality. The program Delfrac is such a diffraction model. Modelling with Delfrac and a part of the program will be discussed in the next chapter, before input and output of the models are shown. The forces on the terminal, the terminal motions and element motions will be calculated with Delfrac for several alternatives.

3.3 Element connections

The element size and the connection type are interrelated.
Pontoon B can probably be coupled by either rigid or flexible connections. Rigid connections are preferable in relation to the crane track on the apron element and in relation to the internal transport of containers. The length of the large pontoon E is as long as one deep-sea berth should be. Therefore, the crane track may end at the end of the pontoon. Compliant connectors bring a solution to the connection forces.

The connection forces were planned to be examined in three models:
- The single-body rigid terminal model in Delfrac can produce data on retaining forces of an element in relation to the entire rigid body.
- A multi-body model can be run with a very low connector stiffness that would practically make the connection a pivot-connection.
- A second multi-body model will be run with an estimation of the connector stiffness, after a number of connector alternatives have been researched and a choice has been made (depending on the choice of pontoon).

Only the single-body run has been performed to find connection forces of rigid connectors.

3.4 Design requirements from hydrodynamics

The main design requirement is that the floating terminal should equal the conditions on a land-based terminal as much as possible. This means motions and dynamic forces on the terminal should be minimised. Motions of the terminal should be minimised, but also relative motions and motions of the berthed ships should be within boundaries.

Boundary conditions for a safe berth, when the berth is not at the lee side of the terminal (at WCT),are:
- No standing wave at the pier
- Limited reflection of the waves
- Minimum turbulence
- Limited breaking waves

Other aspects of influence at berth are wave diffraction and absorption. Wave transmission is only of importance at the lee side of the terminal.

Summary functional and structural requirements:
- Simple construction
- Repetition effect in construction
- Minimal maintenance
- Stability and minimal damage at survival conditions (storm)
- Guaranteed stability at survival condition of two flooded compartments

The following requirements are according to hydrodynamics:
- Shipping should not be hindered by reflected or standing waves
- Prevent erosion at berth from standing waves
- Small wave transmission
- Small heave and sway motions

Small reflection and diffraction at berth can be solved by wave absorption:
- Low natural frequency of the structure, smaller than the wave frequency
  Resonance would lead to large amplitudes in one or more of the defined motions and consequently to large anchoring (fixation) forces.
- Or: find a structure shape which creates a lot of damping (not a pontoon).

A low natural frequency can be obtained by the following requirements:
- Small transverse stability (small GM causes long T)
- Wave energy absorption in roll motion of apron elements
- Allow roll motion through small mass and a flexible fixation in the roll-d.o.f.
- Allow roll motion through a large radius of gyration k_{yy}.

Waves behind a floating structure (transmission) are mainly caused by sway and heave response motions and depend amongst other things on the water depth. Heave motions can occur rather freely, but sway and surge are horizontal translations and are limited by the terminal's fixation to the bottom to avoid the structure floating off (because of the horizontal component of the orbital movement in surface waves). Pitch and yaw for example will cause waves as well, but their amplitudes will be smaller.
- To decrease heave-motions of the floating structure, the hydrodynamic mass (=M+addedM) should be large. However, the motions have to be allowed, because of tidal movements. Therefore, the fixation should be flexible in z-direction.
- Sway motions are not preferred, as independent pontoons may damage each other. A large hydrodynamic mass will decrease sway and the fixation should limit horizontal translations.
- The fixation of the terminal should be flexible for most rotational movements, but should be rigid in horizontal translations.

Summarising: a wave-absorbing structure should be flexible and movements can be large;
A wave-reflecting structure can be rigid. It will bear high forces and experience small movements, while most movements are prohibited.

The conceptual structure is checked on the requirements above:
- The terminal has a large mass, but the hydrodynamic mass differs in the different degrees of motion. A large mass is not favourable for wave Absorption In roll motions, but it is favourable in decreasing other response amplitudes.

- The terminal has a flexible fixation that can meet the requirements.
- A rigid rectangular plate floating on the water surface generates a reasonable amount of damping. This is only valid when the structure height can be simulated as a plate thickness that is in deep water.

3.5 Operational criteria and Relative motions

Operational criteria:
The movements and hydrodynamics of the structure to be designed, are directly connected to the operability of the terminal or floating port. The operability is an important part of the commercial feasibility of the project. A large downtime of the port due to large response motions to the waves would cause income and returns to decrease. The terminal's motions partly define the Serviceability Limit State (SLS) of the structure by operational conditions. The Ultimate Limit State (ULS) will among other things be defined by relative motions between elements or element and ship and their possible damage, the deck immersion angle, the connection and fixation forces caused by element motion (and wave forces) etc.
The above shows that the study into hydrodynamics is not only done to define the technical feasibility of the project, but is also preferable to define the commercial feasibility of the terminal.

The operability of the terminal is described by operating conditions, mostly motions amplitudes, acceleration and speed etc. As surge, sway and yaw concern horizontal translation or rotation around the vertical axis (the motion being in the XY-plane), the natural frequencies of these response motions of the system depend on the type of fixation or anchorage of the terminal.

Operational criteria are used for moored ships:

The following criteria from PIANC are advisory for the (de)loading operations of a container vessel:

efficiency	Surge (m)	Sway (m)	Heave (m)	Roll (°)	Pitch (°)	Yaw (°)
100 %	1,00	0,60	0,80	3,0	1,0	1,0
50 %	2,00	1,20	1,20	6,0	2,0	1,5

The same loading operations criteria according to Hesse-Natie:

efficiency	Surge (m)	Sway (m)	Heave (m)	Roll (°)	Pitch (°)	Yaw (°)
100 %	0,12	0,11	0,10	0,24	0,20	0,26
50 %	0,24	0,22	0,20	0,48	0,40	0,52

PIANC also has criteria for the safety of moored vessels under movements:

Allowable motion amplitudes				
Type of ship	Surge (m)	Sway (m)	Yaw (°)	Heave (m)
Container ship	0,5	0,3	1	0,3

The limiting design wave heights from PIANC are for safe mooring of vessels.
For the tying up of tugboats, a limit H_s = 1,5 m.

	Limiting wave heights H_s in m	
Type of ship	0° (head or stern)	45°-90° (beam)
Container, Ro-Ro ship	0,5	-

Allowable accelerations are limiting criteria with regard to operation of equipment, cargo safety, personal safety and efficiency. Accelerations cause loads on cargo and equipment and are an important reason for seasickness.

In the operational phase, during (un)loading and (de)berthing, ships at berth should lie within sheltered waters. This means that waves or wind should not push the vessel towards the pier and that the amplitude of the waves present should be within boundaries.
At an offshore or near shore location, this condition can lead to two solutions, considering the already chosen layout of the terminal:

1. The terminal is positioned beam-on to the waves; either by weathervaning; or, in a location with a clear dominant (ca. 80%) wave/wind direction, by restricting the terminal's location and direction.
   The safe berths are located on one side of the terminal, not exposed to the incoming waves.
2. The terminal is (semi)permanently positioned with one side exposed to the waves. The waves will be transmitted through/under the terminal, but their amplitude will decrease. When leaving the terminal, at the sheltered berths at the lee side of the terminal, the wave-height will be much smaller. This assumption is checked in the hydrodynamic analysis with Delfrac.

The degree of transmission of waves underneath a floating platform is influenced by the shape and stiffness of the platform. A flexible platform (of hinged elements) will move with the waves and will transmit a larger part of the wave energy and height than will a rigid platform, as the energy is not dissipated. A rigid structure will take more of the wave load (e.g. heave force in kN) and move only little; the wave energy is dissipated.
The WCT deep-sea berths are planned at the exposed side of the terminal. It is not the aim of this study to do a downtime analysis of the WCT. Ref.[20] already provides a downtime-analysis of the traditional design of the WCT exposed to the incoming waves in the Western Scheldt (there is another one planned), which has a positive outcome. The downtime of the same, but floating WCT may prove to be longer.

Relative motions:
Relative motions are the sum of motions of independent structures, such as ship and quay element, which influence each other. If the independent structures are in close range of each other, or connected, their motions can become coupled.

The ship motions and (apron) element motions should be studied together, as one can say that a ship at berth is in its motions coupled to the apron elements, especially if these elements are hinged or flexibly coupled. This phenomenon is difficult to examine, even in Delfrac. If all elements of the floating platform are rigidly connected, the hydrodynamic mass will be very large compared to the ship's mass and the terminal and ship are not expected to move with

frequencies or amplitudes in the same range (damage through collision). Influence from terminal motions on ship motions can still occur.
Should the output of either be limited, then the worst scenario of a 180°phase lag may be a safe assumption to examine. Then apron element and ship are always moving in the opposite direction. The same can be assumed of the coupled motions of hinged elements. On this assumption, the amplitude of the relative motion is the summation of the element amplitude and ship amplitude, per defined motion (degree of freedom). Collision caused by a phase lag between the movements of ship and quay or two elements should be prevented.

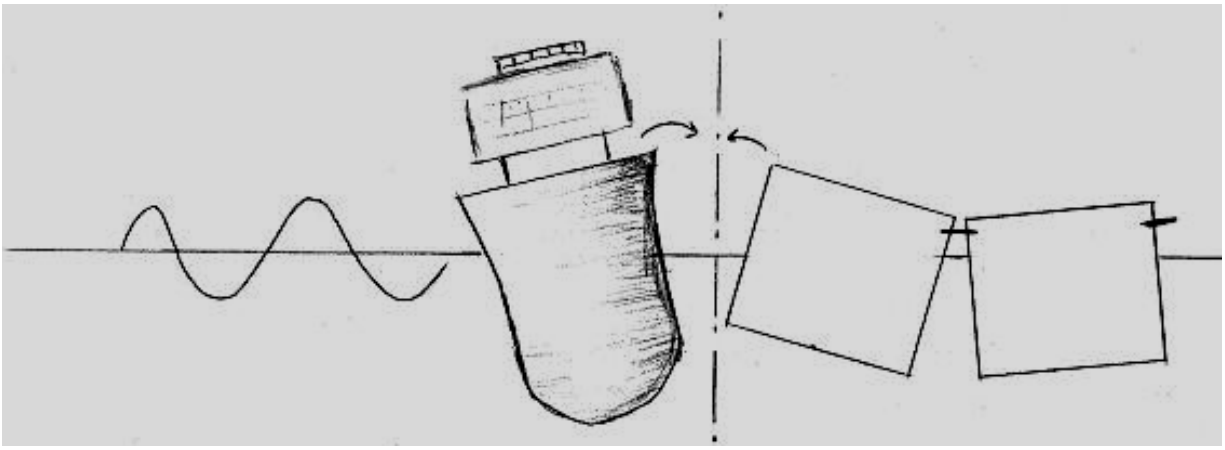


Figure 3.1: Relative motions

In further analysis, the influence on both terminal and berthed ship motions from waves caused by (fast) passing ships in the Western Scheldt, should be taken into account. Many ships will be passing the WCT in the Western Scheldt, heading for the port of Antwerp. The relative motions caused by these waves, can cause damage to ship and quay, if not reckoned with. These extra waves and wave forces can be modelled in Delfrac, but it would quite extend the dynamic analysis.

Relative motions can change with changing behaviour of one of the independent structures.
The hydrodynamic behaviour of a quay element or a ship may change, when several aspects regarding stability etc, are variable.
Aspects of influence on the hydrodynamics and stability of the system are:

1. The degree of loading of the moored ship influences the ship mass and through that:
   - The hydrodynamic properties of the ship such as the added mass, the damping and the hydrostatic stiffness.
   - The draught of the ship
   - The projected area above and under water and the under keel clearance, so that wind forces and current forces on the ship will change. Container ships are seldom unloaded, as they always sail within shipping liners. During loading and unloading the draught and the degree of loading will change constantly. Therefore an average degree of loading and an average draught should be used in modelling a moored ship.

2. The local water level influences the hydraulic conditions (waves and currents), the location of the ship towards the pier, the hydrodynamic properties of the ship and the keel clearance. The maximum wave force and maximum bolder force are calculated for the water level with the largest occurring wave height, because then the largest motions will occur.

4 Modelling with Delfrac

DELFRAC is a radiation/diffraction model program, based on a three-dimensional linear potential theory, including wave diffraction and drift forces acting on structures. A source-distribution technique is used to solve the linearised velocity potential problem. The potential theory of Delfrac will not be discussed in this intermediate report.
The panel method is used to describe the body of the model. Outward this method looks like the Finite Element Method, but Delfrac is not based on FEM. The panels are rectangular plane elements (a distribution of source singularities). A basic rule for the panel dimensions (for accurate results), is that the panel length should be about 1/5 of the shortest wave length to be computed.
The number of panels used and their length compared to the shortest wave length, affects the accuracy of the output of Delfrac.

Delfrac is used to analyse the interaction of surface waves with ship and offshore structures. It can calculate the wave loads and response motions of the floating container terminal, both as a free floating, or a fixed structure in regular waves. The program is applicable to deep and shallow water cases, so it can be used for the case study of the 20 m deep Western Scheldt. The internal forces in cross section of the terminal can be determined from the wave forces.

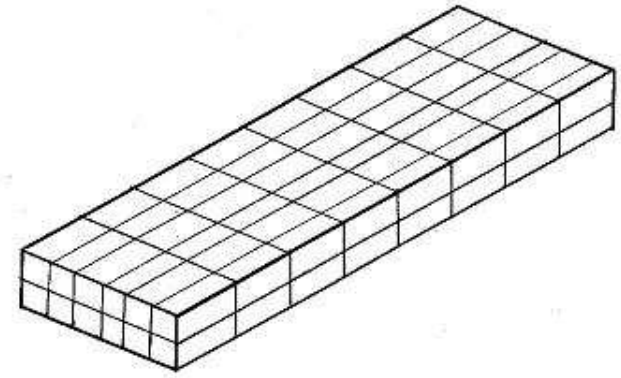


Figure 4.1: Body of the terminal modelled with panels

Delfrac has a number of simplifications of reality, caused by the following assumptions in theory:

- Fluid is assumed to be inviscid, homogeneous, irrotational and incompressible: the basic assumption of many theories.
- Delfrac is based on regular waves; linear theory of free surface waves.
- The linearisation of the velocity potential problem. This may limit the calculation of turbulence (as may occur at a certain type of reflecting breakwater). Delfrac can simulate and produce data on transmission, but not on turbulence. Turbulent liquid particle motion causes (wave) energy to dissipate.
- The linearisations also influence the simulation of the coupled motions of a ship at berth and the apron element. The relative motions are simplified.

Delfrac knows two types of models:

1. A single-body model (such as a ship hull or pontoon)
2. A multi-body model (such as a platform built from elements)

Both models generally need the same input parameters and create the same type of output. A multi-body model has more possibilities in output and reference points.

Delfrac input of variable parameters is:

- structure geometry in mesh co-ordinates
- number of panels
- waterline point: the under water body is modelled
- vertical position of centre of gravity
- matrix if mass coefficients and radius of gyration of the structure
- water depth (and thus, water level)
- wave direction (angle of incoming waves)
- wave frequencies
- water particle velocities
- co-ordinates of reference points: locations where output data on motions or forces are needed

A short manual (in Dutch) on the input models is written in the appendices of this report. Reference co-ordinates at connection point and at the lee side of the terminal are also given in the appendices.
The geometry is modelled according to the co-ordinate system, which is located on the free surface water plane. The z-axis is vertical and positive upward. The x-axis is in the longitudinal direction of the structure and the y-axis is in the transversal direction of the structure.
The present version of Delfrac can compute a maximum of 10000 panels. This can easily be extended in the future. A quarter of the WCT will be modelled with a maximum of 2500 panels. To be able to compute the shortest incoming wave at the large terminal, many more panels are needed. When the panel dimensions increase, the panel length will become larger than the shortest wave length. In that case, the output no longer seems reliable. Longer waves, with a smaller frequency and a longer period, can be computed.

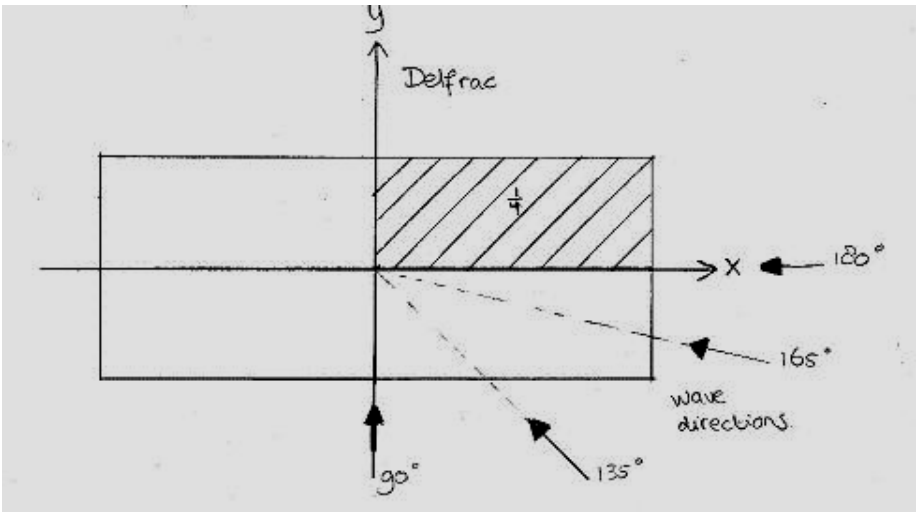


Figure 4.2: Wave directions and axes in Delfrac

Delfrac output:

1. six (d.o.f.) added mass and damping coefficients, by structure/fluid interaction calculated from hydrodynamic reaction forces and moments
2. amplitudes of six (first order) wave exciting forces and moments, calculated as total forces on the entire structure. Internal forces are taken at the middle transverse cross section for six degrees of freedom
3. amplitudes and phases of motions in all six degrees of freedom, calculated per wave direction and wave frequency. The amplitudes are numerical transfer functions (ROA) and still have to be determined in meters by hand
4. mean drift forces and moments for six degrees of freedom
5. amplitudes and phases of reference point motions and accelerations
6. amplitudes and phases of relative motions
7. panel data: area, hydrodynamic pressure and 3 components of relative fluid velocity at the centre
8. free-surface elevation

The length of de terminal is very large compared to the wave length. The flow will not separate from the structure like the flow around small tubes or the piles-pylon of a bridge. This means that the Van Morison equation is no longer valid. Drag forces are neglected. The wave force will have to be calculated according to the diffraction theory or the Froude-Krylov theory within the framework of the potential theory, if neither large separation nor large diffraction of the flow is expected. Delfrac uses the diffraction theory to simulate the expected diffraction of the waves. The terminal will reflect part of the incoming waves in different directions.
The wave directions of incoming waves are simulated as stem, oblique and beam waves as 180, 135, 165 and 90 degrees respectively.

The mean drift forces are secondary order forces with a large period, which means that they change slowly. These forces have little influence on the motions of the pontoons and terminal because of their long period. They are omitted in the models used to define the terminal element size and configuration and connector loads, because of their small significance and because of a very long computation time. Drift forces are only computed if water particle velocities are entered as input.
Drift forces determine the anchorage force in the fixation of the terminal. This may be approximated by hand, as it will be a summation of the other forces. The hand calculation will only be an estimation of the anchorage force of the terminal, because the complete calculation may be rather complex. The aim of this thesis is not to design the terminal fixation, therefore an estimation of the forces may suffice.
However, the accurate drift forces can be determined by Delfrac. This can be done for one model, to get a general idea of the forces. They will not be computed in every run, because of the long computation time.

Relative motions can be used to examine the motions of a berthed ship together with the apron element.
Reference points are used to attain an indication of:

- internal section forces at the location of expected element connection, within a rigid terminal.

- Motions and forces at the lee side of the terminal, where either the safe berths or the infrastructure and hinterland connections are situated. The wave transmission by the terminal is dependent on the water depth.
- Reference points together with free-surface elevation data will provide information about the transmitted wave height at the lee side of the terminal.

All amplitudes produced as output by Delfrac are Response Amplitude Operators (RAO), multiplication factors. This is valid for the amplitudes of the wave and connection forces, the motions and the wave heights at reference points.
An input wave spectrum (e.g. Bretschneider or Jonswap) can be transferred by a transfer function (a frequency characteristic) into an output spectrum of motion of the terminal structure. The force and motions transfer functions are calculated numerically by Delfrac. These functions are the amplitude factors-coefficients in the output (RAOs). These coefficients can be 'multiplied' with the input spectrum, and then the output spectrum of motion in meters is calculated.

Connection forces

Connection forces between elements can be determined with Delfrac, both in a multi-body model and in a single-body model. The single-body model of the terminal represents a floating platform of elements joined together rigidly. Internal forces in a certain cross section can not be determined.
Delfrac can compute the etaining force necessary for one part of the structure (a pontoon) to follow the motions of the rest of the platform, through an infinitely rigid connection. The connection can bear normal forces, shear forces and moments. This connection force is computed in one point (x,y,z). Again, the results are Response Amplitude Operators. To be able to calculate the real connection forces, the number of connectors, the type of connectors and their locations have to be known. Therefore the results of Delfrac have to be distributed over the pontoon dimensions.

The connection forces are computed during a 'run' of the same model that determined the wave forces and the repsonse motions, but an additional input-file is created and put into Delfrac. This input-file contains data on the location, centre of gravity and panel numbers of the pontoon to be examined. The pontoon in the corner directly facing the oblique wave attack has to bear the largest connection forces. This location and another corner and the centre of the pier have been researched in Delfrac; the input-file is shown in the appendices.

A multi-body analysis would give data on connection forces in more points than one, according to the input-model. In the multi-body analysis estimated connection stiffness can be taken into account.
The results of the rigid model are rather conservative, as the real connection will not be entirely rigid. The connection forces will decrease when more motions are allowed.

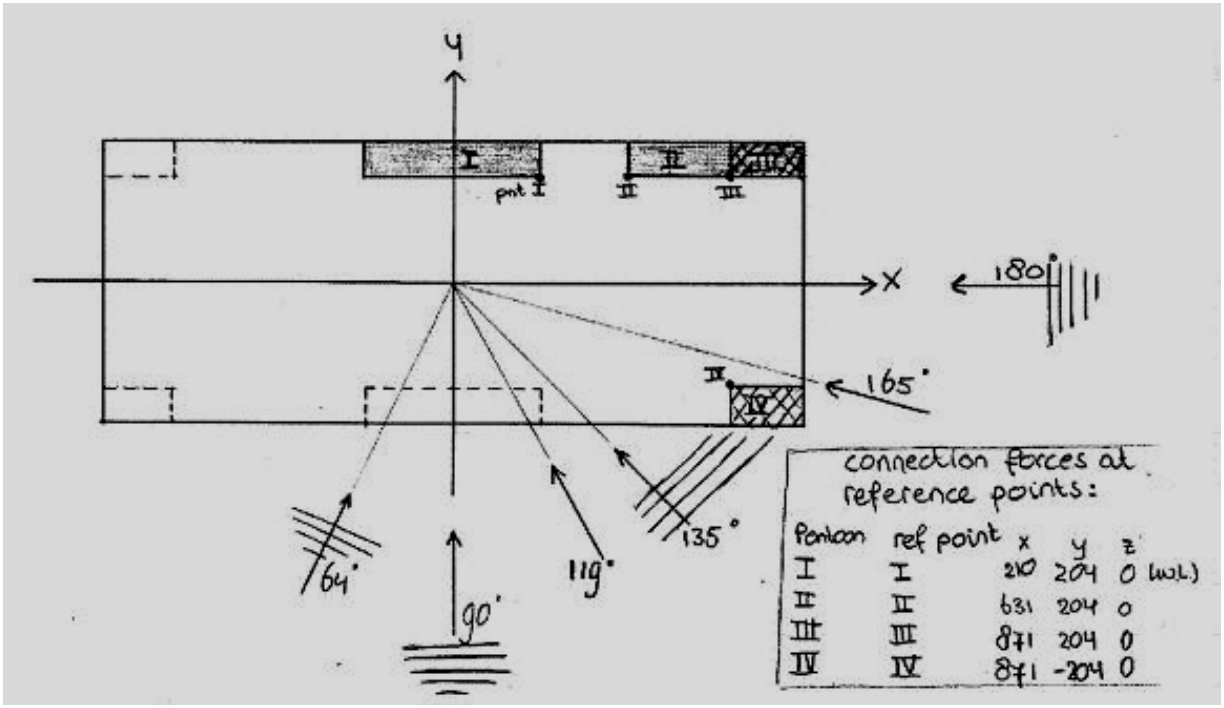


Figure 4.3: Wave directions and locations of connection forces in single-body terminal

5 Motions of the terminal

The usable output is shown in the appendices. The output is analysed in the same appendices. This chapter will include:

- calculations of the Response Amplitude Factor and the real amplitudes in m or kN
- an overview of the governing values per output-type.
- an explanation of the motions

The dynamic analysis is done for a range of wave frequencies. The occurring waves in the Western Scheldt have a frequency varying between 1,0 and 2,0 rad/s. This interval caused capacity problems in the software. The maximum capacity of the computer program is a number of 10000 panels for the total structure of the terminal. The capacities of the Haven3-program and the three-dimensional viewer lie in the range of 1500 – 2500 panels. Therefore, a quarter of the terminal is modelled with 2500 panels, partly with Haven3, partly by hand. Basic rule states that the maximum panel length should be 1-5 of the shortest wave length in the interval, to obtain reliable results. The minimum possible panel length proved to be ca. 15 m. In the following table it is shown that a part of the occurring wave frequencies in the Western Scheldt gave very unreliable results. The wave elevation amplitude factors for the shortest waves were much too high.

ω [rad/s]	T [s]	L_{wave} [m]	Max panel capacity	Rule of thumb: Lp=1/5*Lw [m]	Applied panel length	Wave type	output
0.1	62	6000	10.000	1200	13	Long	Reliable
0.4	15.7	385	10.000	77	13	Long	Reliable
0.5	12.6	248	10.000	50	13	Swell	Reliable
0.7	9.0	126	10.000	25	13	Swell	Reliable
1.0	6.2	60	10.000	12	13	Short	Reliable
1.1	5.7	51	10.000	10	13	Short	Reliable
1.4	4.5	32	10.000	6.4	13	Short	Reliable (checked)
1.5	4.2	27	10.000	5.4	13	Short	Not reliable
1.7	3.7	21	10.000	4.2	13	Short	Not reliable
2.0	3.1	15	10.000	3	13	Short	Not reliable

For the last model of the entire terminal, a choice was made to apply a different range of wave frequencies. As the accuracy of the output increases with the wave length, lower wave frequencies of longer waves were included. In these frequencies 0,05 - 1,45 rad/s, possible swell waves in the Western Scheldt are included. Longer waves generally cause higher wave forces. The statistical amplitude values computed from the output are calculated from the variance of the response spectrum. For all results from the model, a response spectrum has to be determined. As the response spectrum is equal to a constant multiplied by the wave spectrum, there is very little energy in these low wave frequencies. Therefore, the wave forces on the structure are not very different from those within the occurring wave spectrum.



5.1 Incoming waves

The incoming type of waves at the WCT can be:

- wind waves; irregular waves that can be divided in:
  - sea waves; train of waves generated by local wind
  - swell waves; long period wind waves generated by local wind, but which have travelled away from the local wind area
- Astronomical tides with a very long period
- Short waves caused by passing ships

For the time-being internal waves in ballast water are neglected.
Tsunamis etc are not expected.

The long waves such as the tides do not affect the terminal motions strongly, because the terminal body, and the possibly berthed ship will have time enough to follow the movements of the water plane. The bodies will not be excited near their natural frequencies.
Therefore, the terminal motions are mostly determined by the energy in shorter wind waves.
This energy in irregular wind waves can be quantified by an incoming wave spectrum S_ζ.
Three main methods can be used to determine the occurring wave spectrum:

1. Bretschneider wave spectrum
   (Spectral density S_ζ as function of H_s and T, especially suited for open sea areas)
2. JONSWAP wave spectrum
   (A spectrum formulation based on many measurements on the North Sea)
3. Pierson-Moskowitz wave spectrum (spectral density S_ζ as function of wind speed)

[From Journee,J. and Massie, W.; lecture notes Offshore hydromechanics, TU-Delft 2001]
The second method contains a lot of wave data from the North Sea. This data will be valid for a near-shore location before the coast of Walcheren, and it will be the best possible approximation of the wave climate in the Western Scheldt, which mouths into the North Sea. Therefore, the JONSWAP method will be applied. The JONSWAP spectrum is more peaked than the other spectra. The waves from passing ships are neglected for the time being, as they are not modelled in Delfrac either.

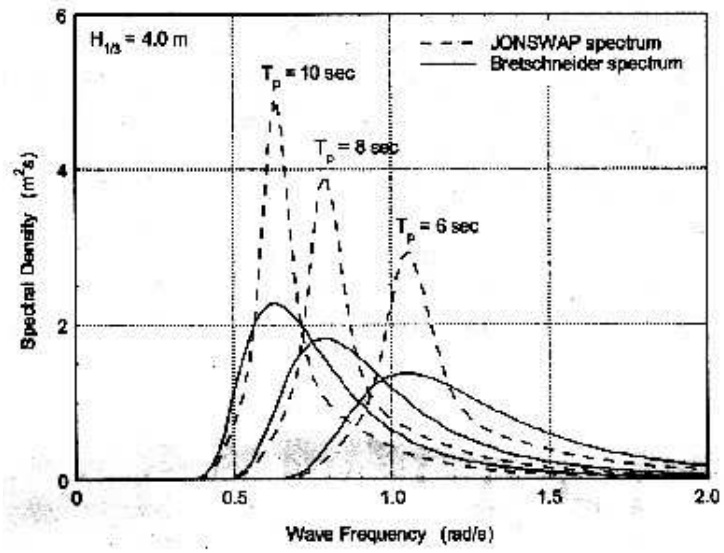


Figure 5.1: Jonswap and Bretschneider wave spectrum

$$\text{wave energy} = \frac{1}{2}\rho g \zeta_a^2$$

5.2 Spectral analysis

The real forces and the structure and the response motion amplitudes are determined by spectral analysis.
A wave amplitude can be expressed by a wave (energy) spectrum. A response amplitude in irregular waves can also be expressed by a response motions spectrum. The relation between the wave spectrum and the response spectrum is the transfer function.
Standard wave spectra $S_\zeta(\omega) = H_{1/3}^2 . f(\omega, T)$

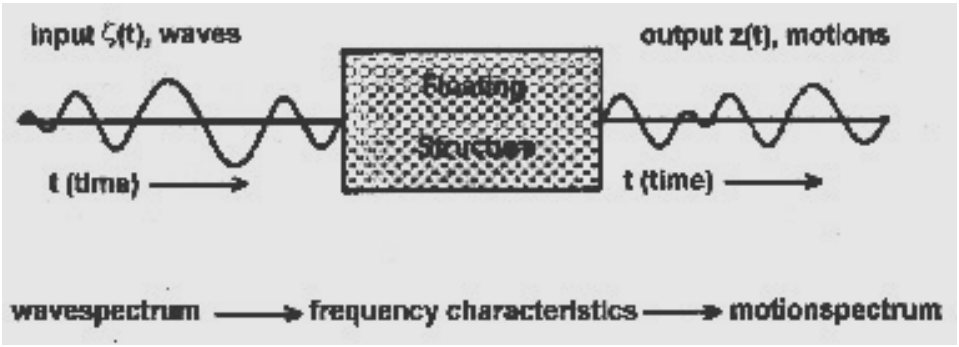


Figure 5.2: Spectral analysis

The JONSWAP input wave spectrum:

$$S_\zeta(\omega) = \frac{320.H_{1/3}^2}{T_p^4}.\omega^{-5}.e^{\left(\frac{-1950}{T_p^4}.\omega^{-4}\right)}.\gamma^{e^{\{-(\frac{\frac{\omega}{\omega_p}-1}{\sigma\sqrt{2}})^2\}}}$$

The wave amplitude can be described as:

$$S_\zeta(\omega).dw = \frac{1}{2}.\zeta_a^2(\omega)$$

The response motion amplitude can be described as:

$$S_z(\omega).dw = \frac{1}{2}.z_a^2(\omega)$$

$$S_z(\omega).dw = \left|\frac{z_a}{\zeta_a}(\omega)\right|^2.S_\zeta(\omega).dw$$

Therefore the relation between a wave spectrum and the response motion spectrum is:

$$S_z(\omega) = \left|\frac{z_a}{\zeta_a}(\omega)\right|^2.S_\zeta(\omega) \quad \text{wherein}$$

$$\left|\frac{z_a}{\zeta_a}(\omega)\right|^2 = RAO^2 \text{ (numerical transfer function from Delfrac)}$$

For the interval ω = 0,05 – 1,45 [rad/s], the values for the response spectrum have been determined according to the method above. This has been done for degrees of freedom for the wave forces, the response motions amplitudes, the added mass and the connection forces.
Then a response spectrum curve can be drawn for every magnitude in every degree of freedom.
The area under the curve represents the variance σ^2 or m0 of the response spectrum.
A number of statistical values can be determined. The moments of the response spectrum are given by:

$$m_{nz} = \int_0^\infty S_z(\omega).\omega^n.d\omega$$

For n=0 m_0 gives the area under the spectrum curve, and n=1 and n=2 provide the first and second moment of inertia of the spectral curve. The mean value of the highest 1/3 of the amplitudes and wave heights are:

$$z_{a1/3} = 2\sqrt{m_{0z}}$$

$$H_{z1/3} = 4\sqrt{m_{0z}}$$

$$z_{\max} = 1{,}86.z_{a1/3}$$

The statistical value used in all further calculations is z_{max}, which presents a maximum response amplitude in meter, a maximum wave force and a maximum connection force.



5.2.1 Delfrac output

The output-files are all separate text files containing:
Panel data, damping coefficients per wave direction, mass coefficients per wave direction, wave forces per wave direction, motions per wave direction, relative motions per wave direction and one file with hydromechanic data.
The panel data and hydromechanic data form a first check of the accuracy of the model output, after the .DAT-file and the 3Dviewer. The amplitude multiplication factors of the response motions are roughly expected to be in a range of 0,0 to 2,0. The same range is expected for wave height multiplication factors.
The wave forces are also multiplication factors in [kN/m]. These are forces on the total structure. To obtain a structural load on the outside of the structure, the wave force will have to be divided by the section area of the structure in the relating direction. It can easily be explained that a very long wave (such as the tidal wave) would have a large heave-wave force, as it lifts up the structure. Very short waves hardly affect the floating structure at all.
The output of the pontoons and the single-body terminal model are evaluated and explained in the next sub-paragraphs.

5.2.2 Qualification of Delfrac-output of pontoons

The two models of the alternative pontoons have the following panel dimensions (x,y):

a. 180 x 50 m pontoon: 18 x 5 panels = 10 x 10 m panel dimensions
b. 420 x 50 m pontoon: 42 x 5 panels = 10 x 10 m panel dimensions

Both pontoons are 16 m in height and have two panels in the z-direction: 8 m in length.
They are both comprised from less than 600 panels.
The following frequencies are analysed: 0,7 and 1,0 rad/sec.
The output of both pontoons is compared:

The static stability of both is sufficient. The transversal metacentric heights hardly differ, the GM-value of pontoon b is a bit larger. The longitudinal metacentric height of pontoon b is a lot higher, as the pontoon has a larger water plane area.

The wave forces (RAO; multiplication factors):
At 180 deg (x-direction), the surge and pitch motions have the largest amplitudes, the sway and roll motions are very small. Stem waves do not excite sway or roll motions because those motions are performed in another direction. The motion amplitudes are $*10^{-8}$. These values can be neglected (they are actually zero) because they are induced by numerical discretioning.
At 90 deg (y-direction), the sway and roll motions have the largest amplitudes, the surge and pitch motions are very small ($*10^{-8}$) and can be neglected. Beam waves do not excite surge or pitch motions.
At 135 deg (x-direction), all motions have rather large amplitudes, the sway motion is remarkably high. The difference between the two wave frequencies seems to be higher. The motions and wave forces should indeed be larger at oblique incoming waves, because of reflection and diffraction and because all 6 defined response motions are excited.

The yaw motion seems to have a small amplitude at 180 and 90 degrees, the heave motion is reasonably large at all times. All amplitudes are larger in the smaller wave frequency, as is to be expected; Longer waves will generate larger wave forces and larger motions. In general,

pitch can deliver higher amplitude factors than roll. Then again, the roll amplitude in m will be calculated in another way than the other motions.
The wave forces on the larger pontoon b are larger, because the surface area and the volume of the pontoon is larger. The force is a multiplication factor for the entire body.

The motions (RAO; multiplication factors):
The relations between the different motions seem to be the same as described above for the wave factors. The proportion of the multiplication factors is different; the factors are small, all smaller than 1, with sway, yaw and heave delivering the largest factors.
The amplitude factors will be multiplied with the input wave spectrum, which is the same for both pontoons. Therefore, also the amplitude factors etc can be compared, without first calculating the amplitudes.
The amplitude motion factors of both pontoons differ only slightly. In some defined motions, the amplitudes of the smaller pontoon a is somewhat higher, but the small difference will hardly influence the structure's behaviour in the water. Both pontoons are highly over-stabilised.

The added mass and damping coefficients:
Both pontoons have a high added mass and damping, the differences between the pontoons are again small. Pontoon b generates slightly more damping and a larger added mass, mainly in longitudinal directions, due to the structural length. The yaw motion seems clearly to be damped by the great length of the pontoon. This off course causes a larger wave force in the yaw motion.

Conclusion:
The hydrodynamic behaviour of the two alternative pontoons is different, but the differences are rather small. Both pontoons are large, heavy structures. The length of the large pontoon has some influence on several of the defined motions, but it does not have very bad effect on the hydrodynamic behaviour overall.
The choice of the best alternative can be made, based on motions, forces or the structural considerations discussed in the report. The large pontoon b will experience somewhat smaller motions, but larger wave forces.
The structural considerations seem to be of more significance to the choice of the best alternative pontoon.

5.2.3 Qualification of Delfrac-output of rigid terminal

The relations between the different defined motions seem to be the same as described for the single-body pontoons. Both forces and motions are larger because of the terminal dimensions.
The terminal is modelled with 2482 panels x 4 = 9928 panels; just below the program maximum of 10000 panels for maximum accuracy at these structure dimensions.
The output will shortly be evaluated per type:
The first check of the model is the panel data (such as GM-values etc), which can easily be calculated;
These match the expectations.
The second check is the wave elevation at the reference points. These values are easy to interpret and give valuable information.

Wave elevations

The wave elevations are given at the reference points. These points lie at the edge of the terminal and around the terminal at the berths.
The wave elevation factors lie in the range of 0 to 2,5.
The exposed sides of the terminal are those reference points with positive x and/or negative y coordinates.
From the locations of the reference points, the following conclusions can be drawn:

- The exposed side of the terminal reflects the incoming waves, even standing waves (multiplication factor ≥ 2) are not impossible.
- The wave heights are greater at most locations due to obliquely incoming waves.
- The wave height factors differ per wave direction and frequency. There does not seem to be a clear relationship between wave height and wave frequency. It should be kept in mind that the output might not be entirely reliable from wave frequencies over 1,0-1,2 rad/s.

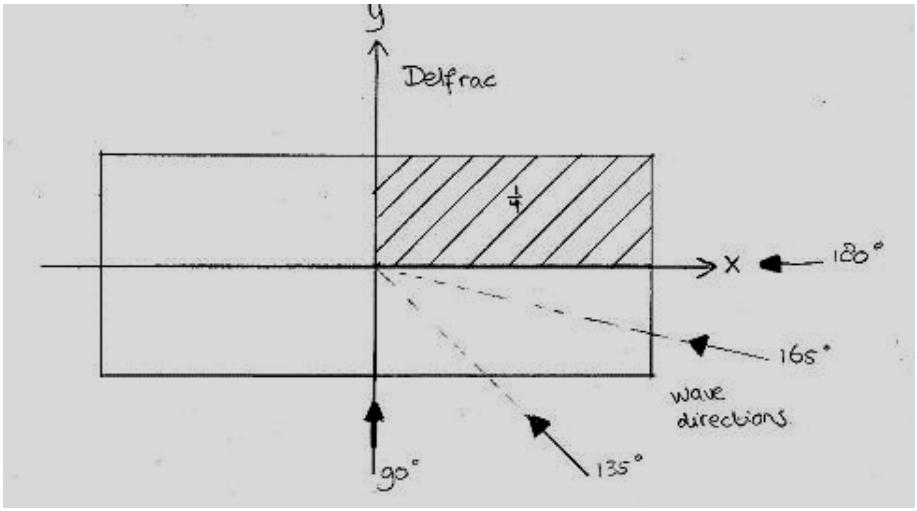


Figure 5.3: Wave directions and axes in Delfrac

A standing wave has two knots and two troughs at incoming waves of 90° and 180°. The wave height at the troughs of the standing wave is two times the wave height of the incoming waves. This is off course not wanted at a berth.

Stem waves(180°) and beam waves(90°) :

- At the exposed berth, the reflection under stem waves is somewhat smaller in the middle and symmetrically higher at the outside and at the berths.
- At the exposed berth, reflection under beam waves is somewhat higher at the terminal's centre and symmetrically lower towards the outer corners.
- The wave transmission to the lee side of the terminal is smallest in the centre of the terminal and increases towards the outer corners.
- The wave heights along the berth decrease from the exposed side to the lee side.
- The wave heights at the exposed edge of the terminal are higher than 28 m from the terminal edge at the berth centre.

- The transmission at beam waves (short cross section) is higher than at stern waves, because the transmission length underneath the terminal is larger.

Oblique waves:

- The wave height is smaller at the corners, whether the exposed or lee side of the terminal. This is valid for both the long and the short side of the terminal.
- The wave reflection is largest in the centre of the terminal, at the long side.
- The wave heights at the centre of the berth; at 28 m from the terminal edge, are higher than at the edge.
- At the exposed long side of the terminal, a standing wave is possible with the largest multiplication factors present.
- The wave transmission towards the long lee side of the terminal is small. Especially near the opposite corner of the incoming waves, the wave height is close to zero. This wave height increases along the lee side towards both corners (of the long and of the short side). Precisely at the opposite corner of the incoming waves, the wave height is higher, due to diffraction.

Motions

At 180° (x-direction), the surge and pitch motions have the largest amplitudes, the sway and roll motions are negligible; numerical discretioning causes the amplitudes to be non-zero.
At 90° (y-direction), the sway and roll motions have the largest amplitudes, the surge and pitch motions are negligible.
At 135° (oblique), all motions have rather large amplitudes, but the maxima for the different defined motions are found either at 90° or at 180°.
The direction of 165° (oblique) is decisive for the results as a whole. All defined motions are excited at 165 deg. All amplitude factors, including the wave elevation, seem to be high in this wave direction, but most maxima are definitely at stem or beam waves, when the other response motions are not excited.
At 165° only the yaw motion has its maximum.
The phase angles do not coincide in maximum with the motion amplitudes.
The heave motion is reasonably large at all times, the floating structure is lifted up by the waves. All amplitudes are larger in the smaller wave frequencies, as is to be expected from longer waves. However, the output of higher frequencies (all above 1.0 to 1.2) is to be questioned. For the frequencies up to 1.0, the panels of the model are small enough to compute with accuracy. In general, pitch can deliver higher amplitude factors than roll.

Wave forces

The amplitudes of the wave forces follow the motions.
The stem waves generate the highest surge and pitch wave forces, the beam waves cause the highest sway, heave and roll wave forces and the 165° oblique waves produce a high yaw wave force.
All wave forces seem to increase with the decreasing wave frequency or increasing period T and wave length.



Added mass and damping coefficients

The added mass coefficients and the damping coefficients both seem to increase with decreasing wave frequency; that is, with increasing period and wave length.
The coefficients depend on the wave frequency, but not on the wave direction. They do vary in all the defined motions.

Hydromechanic data

The maximum values are summarised in the hydromechanic data.
Also, the natural frequencies of the structure's defined response motions, are calculated per incoming wave frequency (not per wave direction).
The natural frequencies are in the range of 0,1 to 0,5 rad/s.

Connection Forces

The Delfrac output is a holding force that is necessary for the pontoon to follow the response motions of the rest of the structure. This result is given in one point and for every wave direction and wave frequency.
The total connection forces are about as large as the wave forces upon the entire structure
The maximum connection forces are found at oblique incoming waves, at the corner pontoon.
The wave attack under 135° is directed towards the corner of the terminal. Waves in the longitudinal or transversal direction of the terminal have less consequence.

5.2.4 Results of the single-body terminal

The Delfrac output of the single-body terminal is collected in a spreadsheet, presented in the appendices. As an example, the main results (maxima) of the single-body terminal are presented here for the frequency of the longest occurring wave in the Western Scheldt (L=60 m). It is important to consider that the length of the terminal is larger than the wave lengths.

Natural Frequencies Surge, sway, heave, roll, pitch, yaw
0.000E+00 0.000E+00 2.661E-01 2.686E-01 2.688E-01 0.000E+00

Maximum values at wave freq = 1,0	***Wave forces ROA***	***Phase wave forces***	***Motions ROA***	***Phase angle motions***
Stem waves 180°				
Surge	7.691e4	37.87	5.337e-3	218.4
Pitch	5.984e7	6.240	5.707e-5	218.7
Beam waves 90°				
Sway	3.138e5	97.44	2.169e-2	279.5
Heave	2.458e5	64.62	1.458e-3	277.6
Roll	5.838e7	247.4	1.005e-3	9.640
Oblique waves 165°				
Yaw	3.091e6	1.803	3.253e-5	199.9

The phase angles do not have a maximum value that coincides with the maximum amplitude. Therefore the phase angle coinciding with the maximum amplitude is considered.

The following response spectra cause the largest amplitudes of response motions and the maximum wave forces and connection forces. In the first graph the transfer function from Delfrac for sway is shown. All other results are shown in the appendices of part 3.

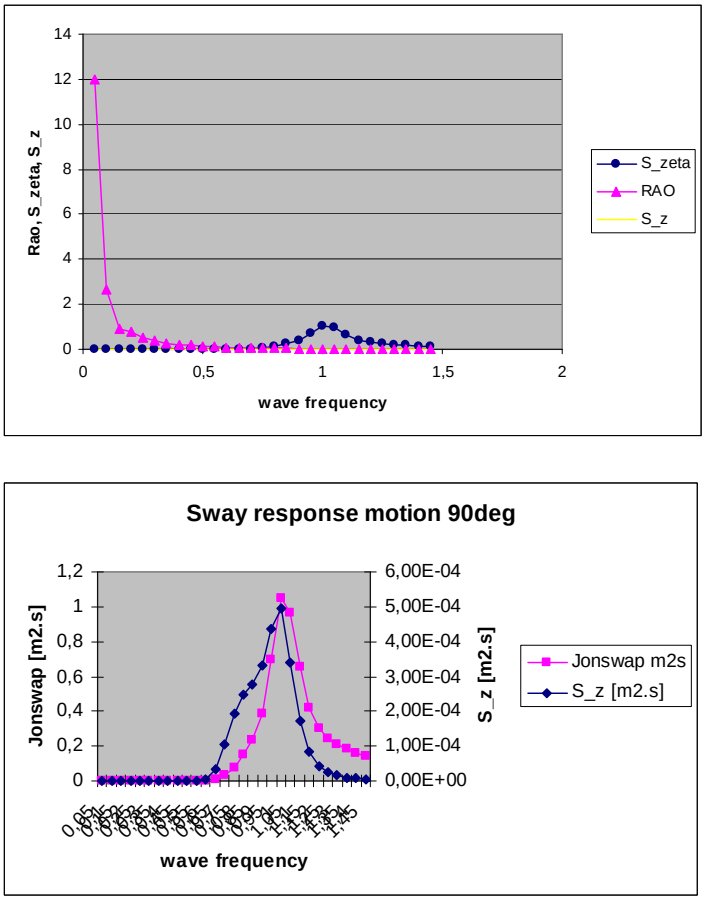


Figure 5.4: Spectral analysis of the results from Delfrac. The maximum values for response motions, wave forces and connection forces.

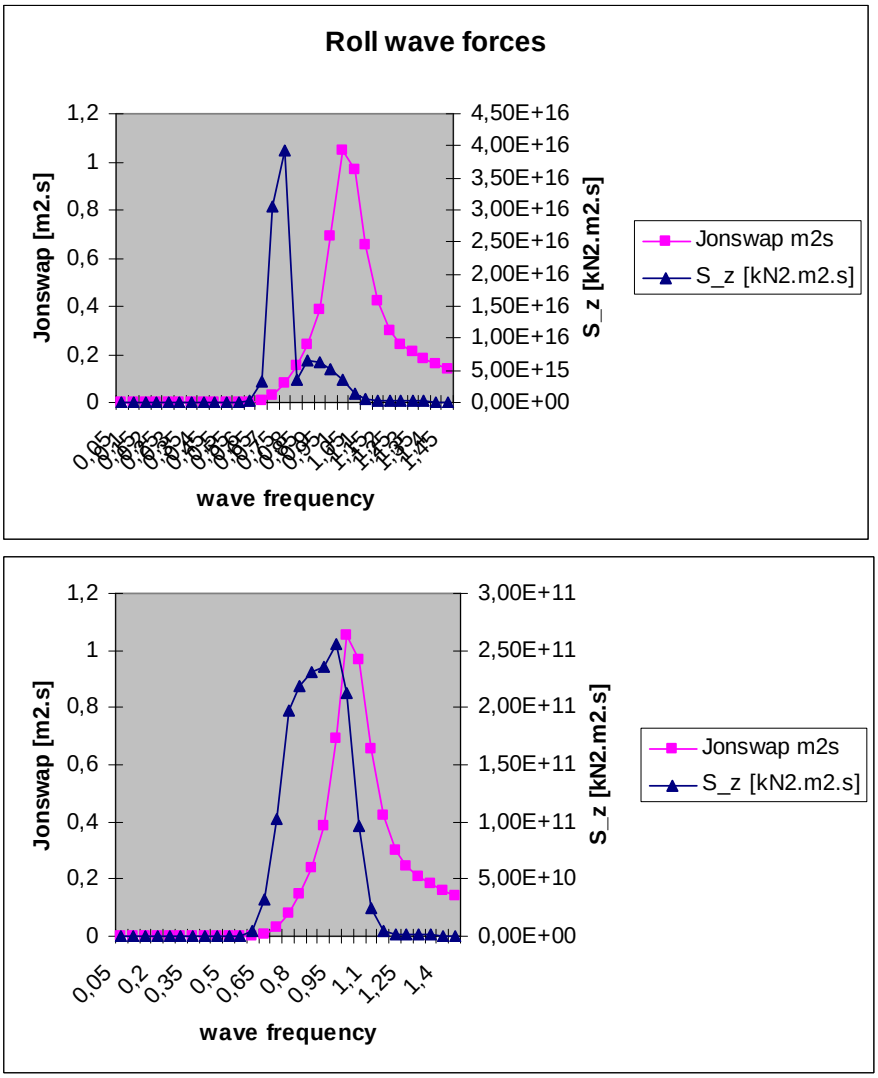


Figure 5.5: Spectral analysis of the results from Delfrac

Output summary

The following tables show the maximum values from the hydrodynamic analysis. These results will be used in the static structural design in Part 4 of the thesis.

Summary wave frequencies 0.05 - 1.45 rad/s

		Max motion ampl. [m or deg]	Max wave force [kN or kNm]	added mass [-]	connection force [kN or kNm]
m	**Surge**	1.08E-02	1.45E+05	3.87E+04	6.96E+03
m	**Sway**	4.42E-02	6.01E+05	1.59E+05	5.30E+03
m	**Heave**	4.02E-03	8.36E+05	2.88E+08	7.06E+03
deg	**Roll**	2.28E-03	2.60E+08	8.70E+12	1.81E+05
deg	**Pitch**	1.38E-04	1.84E+08	1.04E+14	1.06E+06
deg	**Yaw**	7.00E-05	5.99E+06	6.28E+10	4.32E+05

Computation of wave forces:

		Max wave force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	wave load [kN/m1 BorL]	Surface wave load [kN/m2]
kN	**Surge**	1.45E+05	2104.5	509	13.11	1.404E+07	285.69	21.79
kN	**Sway**	6.01E+05	2104.5	509	13.11	1.404E+07	285.55	21.78
kN	**Heave**	8.36E+05	2104.5	509	13.11	1.404E+07	397.03	0.78
kNm	**Roll**	2.60E+08	2104.5	509	13.11	1.404E+07	242.73	18.51
kNm	**Pitch**	1.84E+08	2104.5	509	13.11	1.404E+07	172.05	13.12
kNm	**Yaw**	5.99E+06	2104.5	509	13.11	1.404E+07	217.21	0.43
							[kNm/m2]	*[kNm/m3]*

Computation of connection forces pontoon IV: **180x50m; 4th quadrant end of pier**

		Maxconnection force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	conn. load [kN/m1 B]	distributed load [kN/m2]
kN	**Surge**	6962.42	180.9	50.9	15.5	1.427E+05	136.79	8.82
kN	**Sway**	5296.13	180.9	50.9	15.5	1.427E+05	104.05	1.89
kN	**Heave**	7056.40	180.9	50.9	15.5	1.427E+05	138.63	2.52
kNm	**Roll**	180685.31	180.9	50.9	15.5	1.427E+05		
kNm	**Pitch**	1057547.33	180.9	50.9	15.5	1.427E+05		
kNm	**Yaw**	432154.21	180.9	50.9	15.5	1.427E+05		

Small pontoon alternative to be applied in combination with rigid connections.
Location in 4th quadrant at the corner is most heavily loaded by the incoming waves.
These froces are decisive for the design of the smaller pontoon.



Computation of connection forces pontoon III: **180x50m; 1st quadrant end of pier**

		Maxconnection force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	conn. load [kN/m1]	distributed load [kN/m2]
kN	**Surge**	3.39E+02	180.9	50.9	15.5	1.427E+05	6.67	0.43
kN	**Sway**	2.85E+02	180.9	50.9	15.5	1.427E+05	5.61	0.10
kN	**Heave**	5.09E+02	180.9	50.9	15.5	1.427E+05	9.99	0.06
kNm	**Roll**	1.16E+04	180.9	50.9	15.5	1.427E+05		
kNm	**Pitch**	4.31E+04	180.9	50.9	15.5	1.427E+05		
kNm	**Yaw**	3.56E+04	180.9	50.9	15.5	1.427E+05		

Small pontoon at other corner (end of pier); no direct incoming waves and therefore smaller forces.

The larger the element structure (pontoon), the larger the connection forces will be
Fyi = The forces in point x necessary to follow the rest of the structure (the terminal platform),
the forces are concentrated in one point! (because only one point is fixed)
The result of very large forces means that a certain number of connectors per side of the element is necessary to distribute the connection force.
(smaller force per connector unit)

Computation of connection forces pontoon II: **420x50m; 1st quadrant end of pier**

		Maxconnection force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	conn. load [kN/m1]	distributed load [kN/m2]
kN	**Surge**	3.35E+02	420.9	50.9	15.5	3.321E+05	6.58	0.42
kN	**Sway**	7.20E+03	420.9	50.9	15.5	3.321E+05	141.45	1.10
kN	**Heave**	9.12E+02	420.9	50.9	15.5	3.321E+05	17.91	0.04
kNm	**Roll**	6.41E+04	420.9	50.9	15.5	3.321E+05		
kNm	**Pitch**	2.28E+05	420.9	50.9	15.5	3.321E+05		
kNm	**Yaw**	6.66E+05	420.9	50.9	15.5	3.321E+05		

Large pontoon, same location as pontoon nr III
The location does not produce the decisive connection forces, but it does give a good comparison with the smaller pontoon.

Computation of connection forces pontoon I: **420x50m; 1st & 2nd quadrant - centre of pier**

		Maxconnection force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	conn. load [kN/m1]	distributed load [kN/m2]
kN	**Surge**	1.49E+02	420.9	50.9	15.5	3.321E+05	2.93	0.19
kN	**Sway**	1.20E+04	420.9	50.9	15.5	3.321E+05	236.44	1.84
kN	**Heave**	2.64E+04	420.9	50.9	15.5	3.321E+05	519.53	1.23
kNm	**Roll**	6.41E+04	420.9	50.9	15.5	3.321E+05		
kNm	**Pitch**	3.62E+05	420.9	50.9	15.5	3.321E+05		
kNm	**Yaw**	2.53E+06	420.9	50.9	15.5	3.321E+05		

Very large pontoon in centre of pier; connection forces are larger than at end of pier in 1st quadrant.
This pontoon will not be applied in combination with rigid connections.
For flexible or pivot connections, this pontoons has to be researched in other locations (location of pontoon 4; 4th quadrant).



The wave field around the floating terminal has been researched. Standing waves are to be expected around the terminal, especially in direct wave attack. The lee side of the terminal hardly experiences any waves. The wave field is shown in the next figure.

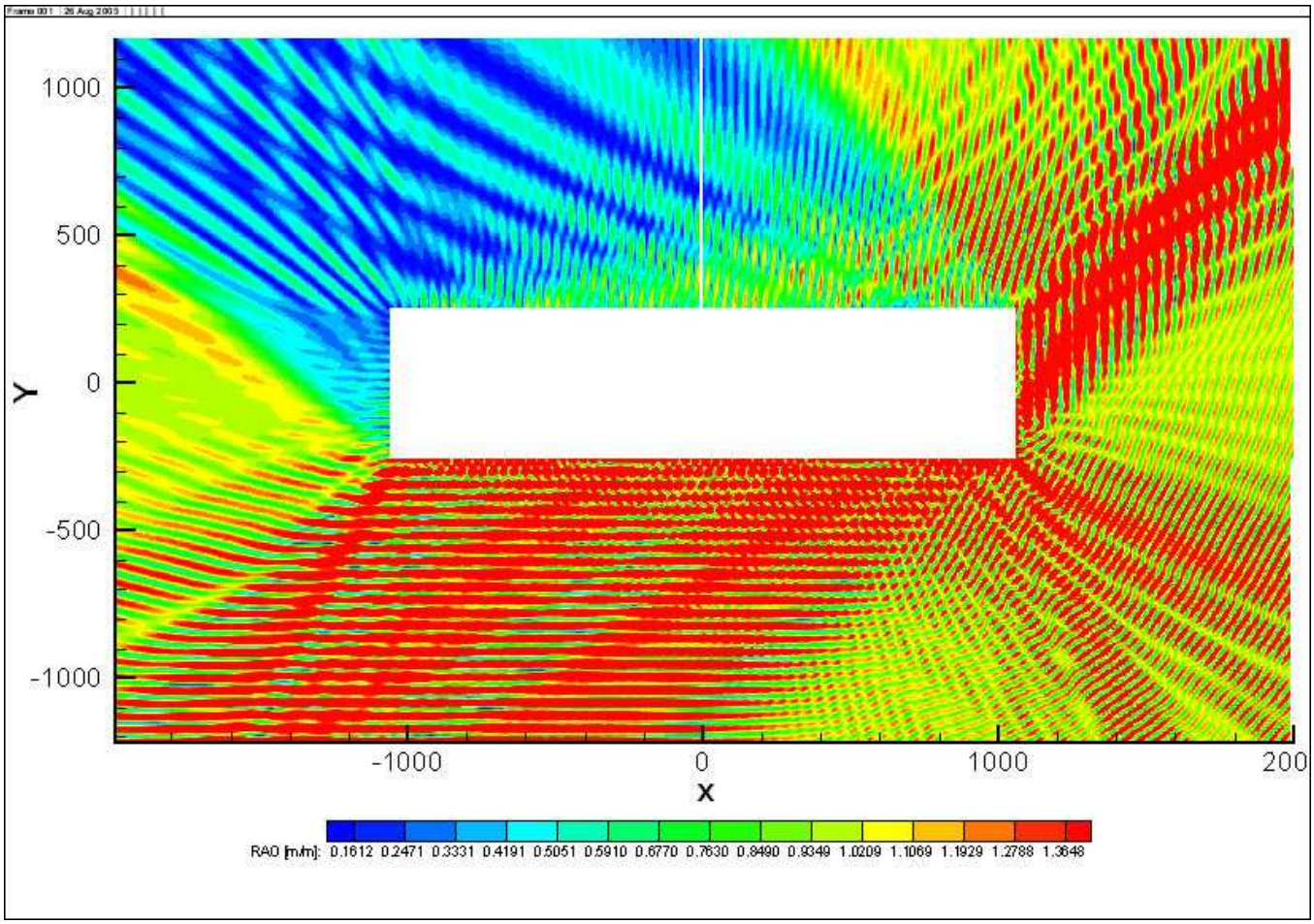


Figure 5.6: Wave field around the floating terminal at diagonal wave attack: blue colour presents the lee side (top left side of the figure).

The figure above shows that diagonally incoming waves (45°) also cause standing waves. The reflected waves (135°) produce 1 knot and 1 trough in stead of 2 at the interception points with the incoming wave field. The standing waves are largest at the berths of the terminal.

Conclusion regarding Hydrodynamic Analysis

The response motions of the terminal on a wave spectrum that includes short waves and swell waves are very small. The single-body rigid terminal practically does not move. However, the wave forces and the connection forces are large. Both the small movements and the large forces are a result from the rigid single-body model of the terminal.
Low frequency drift forces only influence the mooring forces of the terminal (in stead of the 1st order wave forces). The terminal is moored rather rigidly. However, some flexibility is assumed. The result of a manual calculation of the low frequency drift forces is q = 28,24 kN/m[1]over the length of the terminal.

There are two possibilities for further research:

1. If the connection forces from the results are allowable on the pontoon structure, the terminal will be designed with smaller pontoons 180x50 m and rigid connections.
2. If the connection forces are too large for the pontoon structure, the terminal will have to be designed with larger pontoons (420x50 m), which will be connected by compliant connectors. In that case, a multi-body analysis in Delfrac is recommended to research the response motions and connection forces of the elements. This is beyond the scope of this thesis. Large pontoons can also be connected rigidly. In principle, large pontoons will experience larger connection forces, but these can be distributed over a larger surface along the sides of the pontoon. Moreover, the connection forces will be restricted by the smaller response motions of the larger pontoons. The connections will in any case be assumed to be less strong and less rigid than the pontoons themselves.

In the last part of this thesis, the concept of smaller pontoons of 180 x 50 x 16 m with rigid connections is developed.



Part 4:

Structural Design

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1 Structural design of pontoon

1.1 Introduction

A conceptual design is used to consider the technical and economical feasibility of the project. It is a first structural design of shape and dimensions. The method for the conceptual design of this project should be a rather simple and fast method to calculate the internal forces of a structure by hand and to design the reinforcement or prestressing needed. The calculations are made according to a conservative approach of the structure. The safety margins in this design will allow for both structural and economical optimisation in the next detailed design.
From the conceptual design, the following can be deduced:

- Dimensions and shape of the structure
- Type of materials and material qualities
- Quantities of materials etc.

Several design methods are considered in the Literature study to determine the internal forces in the structure. The load distribution on the terminal and the calculation method are discussed in Part 4 of the Appendices. For the conceptual design of the rigidly connected pontoons, it will be assumed that the terminal is an infinitely stiff platform. The load is evenly distributed.

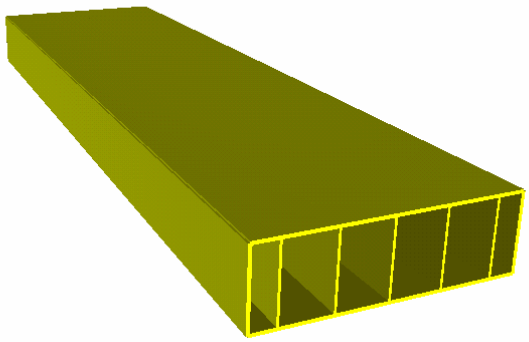


Figure 1.1: A 3D model of the quayside pontoon

As a first estimate, the pontoon is considered as a beam supported on the waves.
The maximum bending moment in the hull is approximated by the Still Water Bending Moment, mostly used for ships.
Further, the governing cross sections are chosen. The governing cross sections are considered as a framework and thus, the internal forces are estimated. However, the transversal walls in the pontoon can not be taken into account in a cross section. For a more accurate concrete design, the structure will be divided into pieces: Structural

members such as a floor, a deck and walls, and connections. The parts will be designed to form the whole structure. This design method is presented with drawings.

Longitudinal section:
Only a front section with the crane track and possible emergency storage is considered. The waves support the pontoon by an upward load. This is modelled as springs underneath the walls of the pontoon with a stiffness equal to the buoyancy force, combined with a distributed load upward.

Transversal cross section:
Most loads do not vary much in the longitudinal direction. Therefore, one centre cross section is considered with a full deck load or ballast load, in storm conditions. The waves can provide a hogging or sagging condition in the transversal direction. The buoyancy force is modelled as springs underneath the walls of the pontoon with a stiffness equal to the buoyancy force, combined with a distributed load upward.

Top slab:
The entire top slab should be considered as one statically indeterminate slab with many fields. Most fields are evenly loaded. However, for the conceptual design, the top slab of the pontoon is considered as the top slab of a single compartment. The plate is uniformly, rigidly supported by the walls of the compartment.

Bottom slab:
The bottom slab of the pontoon is casted in situ, and is evenly supported by the buoyancy force. The load is evenly distributed, but may vary per compartment. The bottom slab is considered as a bottom slab of one compartment, which is uniformly supported by the walls.

Outer wall slabs:
The outer wall slabs are supported every 10 or 15 m by the compartment walls. The hydrostatic load and the wave load are unevenly distributed.

Inner wall slabs:
The inner walls can be considered with the dimensions of the compartments. They are loaded in compression by the normal force due to the deck load and by the hydrostatic load from ballast water. The ultimate limit state load is a hydrostatic load from a flooded compartment (after collision/accident).

The plate design can be simplified by using the plate design tables from NEN 6720 (according to the linear theory of elasticity) for several standard cases with only an evenly distributed load.

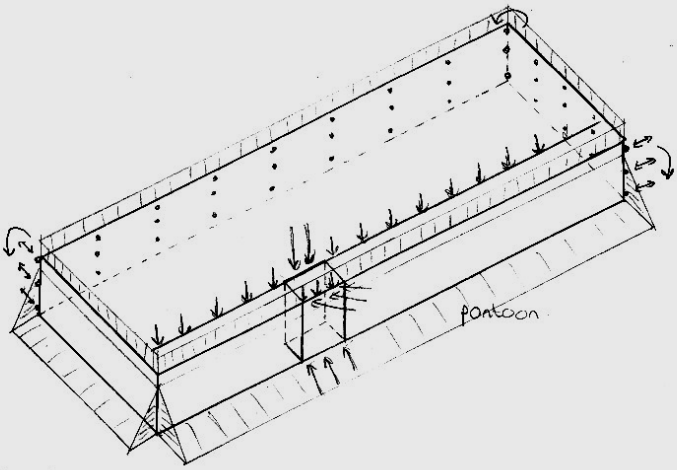
The entire shell of the pontoon may be loaded in torsion, caused by an obliquely incoming wave. The cross sections of the pontoon present a closed section, which can bear a torsional load.

The connector and pontoon design are integrated by introducing the connection forces into the pontoon hull structure. However, the connection forces are analysed locally in the pontoon and are thus treated separately from the general distribution of internal forces in the pontoon.

Van Hattum en Blankevoort

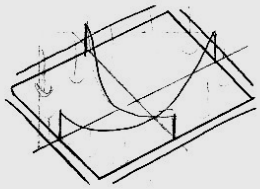
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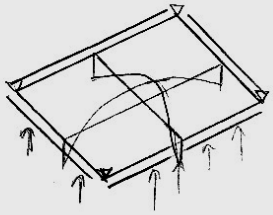
1 Rough force distribution of pontoon
Structural models:

1 deck



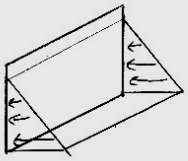
deck load (storage etc) supported by walls

2 floor



resulting upward buoyancy load (7- ballast)

3 Outer walls



hydrostatic water pressure + wave load

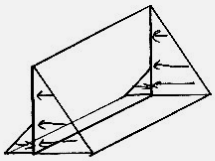
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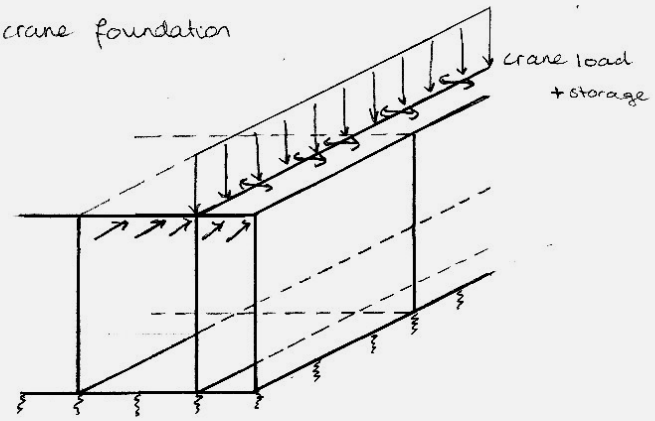
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4. Inner walls



ULS: survival condition of one flooded compartment
hydrostatic water pressure from flooded compartment & ballast

5 crane foundation



6 Longitudinal and transversal cross section ⇒ general force distribution

Figure 1.2: Calculation of connection forces

1.2 Load combinations

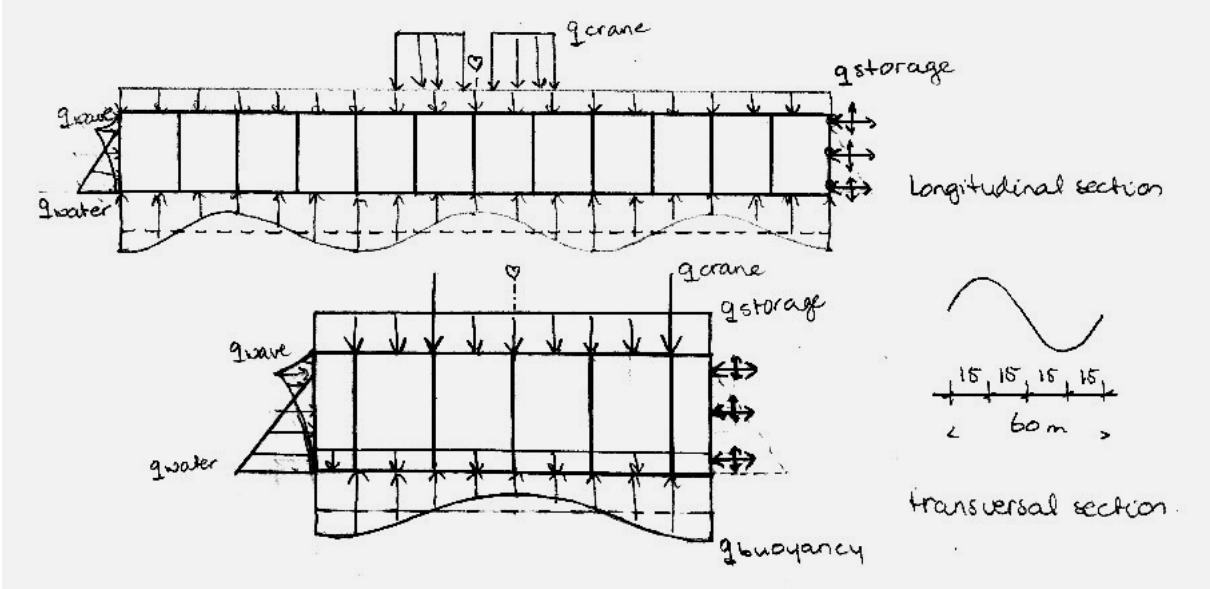


Figure 1.3: Loads on longitudinal and transversal cross section

The maximum wave loads and connection forces have been determined from several wave directions. These maximum values will be used in the pontoon design. The maximum connection (corner section) forces were found for a wave direction of 45° oblique on the terminal. The maximum wave forces differ in the wave direction per degree of freedom:
Wave loads from Delfrac:

Summary wave frequencies 0.05 - 1.45 rad/s

		Max motion ampl. [m or deg]	Max wave force [kN or kNm]	added mass [-]	connection force [kN or kNm]
m	**Surge**	1.08E-02	1.45E+05	3.87E+04	6.96E+03
m	**Sway**	4.42E-02	6.01E+05	1.59E+05	5.30E+03
m	**Heave**	4.02E-03	8.36E+05	2.88E+08	7.06E+03
deg	**Roll**	2.28E-03	2.60E+08	8.70E+12	1.81E+05
deg	**Pitch**	1.38E-04	1.84E+08	1.04E+14	1.06E+06
deg	**Yaw**	7.00E-05	5.99E+06	6.28E+10	4.32E+05

Computation of wave forces:

		Max wave force [kN or kNm]	L [m]	B [m]	T [m]	V [m3]	wave load [kN/m1 BorL]	Surface wave load [kN/m2]
kN	**Surge**	1.45E+05	2104.5	509	13.11	1.404E+07	285.69	21.79
kN	**Sway**	6.01E+05	2104.5	509	13.11	1.404E+07	285.55	21.78
kN	**Heave**	8.36E+05	2104.5	509	13.11	1.404E+07	397.03	0.78
kNm	**Roll**	2.60E+08	2104.5	509	13.11	1.404E+07	242.73	18.51
kNm	**Pitch**	1.84E+08	2104.5	509	13.11	1.404E+07	172.05	13.12
kNm	**Yaw**	5.99E+06	2104.5	509	13.11	1.404E+07	217.21	0.43
							[kNm/m2]	*[kNm/m3]*

The main load combinations are:

LC	name	Description
I	Construction	Construction and transport: Normal waves, wind and current, empty deck, full ballast
II	Operation: Emergency storage	Maximum waves, wind and current, full deck storage load, empty ballast, full quayside crane an emergency storage load, full mooring forces
III	Operation: quay cargo transport	Maximum waves, wind and current, full deck storage load, empty ballast, full quayside crane/traffic/mobile crane load, full mooring forces

During transport stage, the hydrostatic pressure on the walls will be governing, but the increased ballast load will equal the additional buoyancy due to a large draught. The mainly governing situation will occur during the operational stage. A full deck load or a large ballast load will eventually amount to the same load, equal to the buoyancy force, but either placed on deck or on the bottom of the pontoon. The conceptual design will be based on LC II, as time during this thesis is short to consider all load combinations. All load cases will be taken into account without momentary factors; $\psi = 1,0$

The dynamic analysis for the Ultimate Limit State has been done without different safety factors per load case. For the conceptual design, an overall safety factor of $\gamma = 1,5$ will be used. No distinction will be made between the permanent and variable loads

Loads	LC I Transport	notes
Self weight	4314 kN/m^1	= 86,3 kN/m/m
Buoyancy load	150 kN/m/m	
Hydrostatic load (walls)	150 kN/m^2	= 1125 kN
Ballast load	63,7 kN/m^2	
Wave load	286 kN/m^1 horiz	397 kN/m^1 vert
Wind load	0,78 kN/m^2	
Temperature	0	
Mooring forces	0	
Crane load	0	
Storage load	0	
Traffic load	0	
Mobile crane	0	
Other cranes	0	
Bolder load	0	
Fender load	0	
Connection forces	0	

Loads	**LC II: emergency storage**	**notes**
Self weight	4314 kN/m^1	= 86,3 kN/m/m
Buoyancy load	136 kN/m/m	
Hydrostatic load (walls)	136 kN/m^2	= 924,8 kN
Ballast load	1,2 kN/m^2	
Wave load	286 kN/m^1 horiz	397 kN/m^1 vert
Wind load	0,78 kN/m^2	
Temperature		
Mooring forces	19 kN/m^1 length	Via connectors
Crane load	800 kN/m1 vertical	80 kN/m^1 horiz
Storage load	40 kN/m^2	
Traffic load	0	
Mobile crane	0	
Straddle carrier	0	
Reach stacker	1000 kN	30 m
Bolder load	2000 kN	15 m
Fender load	1500 kN	
Connection forces	See following chapters	

LC II = $1,5*q_1 + 1,5*q_2 + 1,5*q_3 + \ldots. + 1,5*q_{16}$

Loads	**LC III: traffic and ballast**	**notes**
Self weight	4314 kN/m^1	= 86,3 kN/m/m
Buoyancy load	136 kN/m/m	
Hydrostatic load (walls)	136 kN/m^2	= 924,8 kN
Ballast load	42 kN/m^2	
Wave load	286 kN/m^1 horiz	397 kN/m^1 vert
Wind load	0,78 kN/m^2	
Temperature		
Mooring forces	19 kN/m^1 length	Via connectors
Crane load	800 kN/m1 vertical	80 kN/m^1 horiz
Storage load	0	
Traffic load	3 kN/m^2	3x200 kN
Mobile crane	2600 kN/ 6 m^2	4x
Straddle carrier	540 kN	4x
Reach stacker	1000 kN	2x
Bolder load	2000 kN	30 m
Fender load	1500 kN	15 m
Connection forces	See following chapters	

1.3 Conceptual design of pontoon

The conceptual manual design calculations of the pontoon are kept rather simple, because of the time available during this thesis. The following simplifications of the calculations are applied:

- Only one load combination is considered; LC II
- As the dynamic analysis did not include Ultimate Limit State safety factors, a general conservative safety factor $\gamma = 1,5$ is used.
- Only the Ultimate Limit State is considered; the SLS is omitted in the calculations
- Safety class II and environmental class IV are taken into account
- As bolder and fender forces during storm are omitted in the dynamic analysis, they will be omitted in the general design of the pontoon. The relatively local forces from fenders and bolders could be considered in a structural detailed design.
- The prestressing of the pontoon, especially of prefabricated parts, is supposed to have a straight horizontal or vertical profile.

The calculations are presented in the appendices.
The following is a summary of the results of the internal forces in the pontoon:

Summary internal forces
Results from calculations (Appendices, Part 4)
The results are the maximum ULS values including gamma = 1,5

Top slab			**Bottom slab**		
Mv;x	306	kNm	Mv;x	566,1	kNm
Mv;y	144	kNm	Mv;y	266,4	kNm
Ms;x	-675	kNm	Ms;x	-1248,8	kNm
Ms;y	-495	kNm	Ms;y	-915,8	kNm

Outer wall			**Inner wall**		
MxD+ ULS (span)	509,90	kNm	MxD+ ULS (span)	469,2	kNm
MyD+	535,80	kNm	MyD+	537,6	kNm
MxD- ULS (support)	1264,20	kNm	MxD- ULS (support)	1204,4	kNm
MyD-	1563,90	kNm	MyD-	1542,9	kNm
Vmax x =	685,35	kN	Vmax x =	678,45	kN
V max z =	907,65	kN	V max z =	915,75	kN

1.4 Design of reinforcement and prestressing steel

The type of prestressing is determined amongst others by the construction method of the pontoons. The construction method is discussed in Chapter 4 of this part of the thesis.
The pontoons will be prestressed with post-tensioned steel, with bond.
Tendons are placed in thin-walled (profiled) steel sleeves, which are used to create ducts in the concrete. These tendons will be tensioned and anchored as soon as the green concrete of the entire pontoon has developed sufficient compressive strength. The compression strength of the concrete structural parts should be larger than 12 N/mm^2 when the prestressing steel is tensioned. The tendons will be anchored and tensioned from two sides, because of the large dimensions of the pontoon. Coupling anchors can be used to lengthen the tendons if necessary. The space between the tendons and the ducts is injected with grout after the placement of the anchorages. The grout is injected through

various pipes along the length and width of the pontoon, because it should reach every part of the ducts inside the concrete, while the injection pressure should stay within boundaries. This grout will protect the corrosion sensitive prestressing steel. It will create a bond, which prevents an immediate loss of tension when the concrete starts cracking.

Considering the construction method and the rough design calculations, the tendons will have a straight profile in the walls and in the top and bottom slab of the pontoon, which are all statically indeterminate. The straight tendons are located in the centre of the slabs. They cause a compression force, but they do not cause a bending moment. The concrete cover on the sheaths is more than sufficient.
A prestressing cable outside the centre of the slab would increase either the field or the support moment of the statically indeterminate slab. Two tendons (one at either side) would compensate the bending moment caused by the other cable. They would only produce a higher compression force.
The central tendons of the slabs will reduce the tension stress in the slab, but will increase the compression stress. The slabs will be tensioned up to the maximum allowable compression stress.

The top slabs of the compartments are prefabricated. There are 2 types of top slabs:

1. l * b * h = 15 * 10 * 0,7 m for all inner compartments and the end compartments at the short sides of the pontoon
2. l * b * h = 15 * 5 * 0,7 m for 24 outer compartments at the long sides of the pontoon

The reinforcement of these slabs (∅ 16-100 in 2 directions) is sufficient to bear the dead load and transport load. The slab of 15 * 10 m is governing for the calculation of the internal forces. v The reinforcement is calculated in Part 4 of the appendices. The top slabs are rigidly connected to each other when the horizontal tendons in the deck of the pontoon are tensioned. The joints between the top slabs are casted in situ.

Allowed tensile stress at all times $\sigma_b < ½ * f_b = 1,08$ N/mm^2
There are no fixed rules in the NEN 6720 for this choice. However, as the structure is floating in water, cracking should be reduced to a minimum. As long as there will be space left between an outside crack and an inside crack, there will not occur a direct leak in the structure. The cracks should preferably be well distributed and small. This can be obtained by a good reinforcement.
Allowed compressive stress at tensioning $\sigma'_{bi} < 0,75*f'b = 0,75*39 = 29,3$ N/mm^2
Losses due to shrinkage, creep and relaxation are not researched at this stage. The losses are estimated at 15 % for centric straight tendons. For the top slab, haunches are taken into account, which provide an upward load that will counteract the deck load. Haunches are also taken into account for the outer walls, which are supported by the inner walls.

The crane tracks are located on top of two inner walls in the longitudinal direction of the pontoon. The crane mainly causes a normal force in these walls. Due to the distribution of the concentrated load in the concrete top slab, a small extra support moment is assumed in the transversal direction of the top slab. Extra reinforcement of ∅ 20-100 in stead of ∅ 16-100 is applied with a length of 5 m on either side of the inner walls. Two more strands could be applied in the sleeves of the tendons 19K15 in the top slab. However, this does not seem necessary and it will have a negative effect in other cross sections of the top slab.
All calculations are presented in the appendices.

The results of the calculations are presented in the following summary:

	thickness	Longitudinal prestressing	Transversal or vertical prestressing	reinforcement	Max prestres-sing level	check
Top slab (incl. haunches)	700 mm	14∅ 15,7 – 1000 mm = 2100 mm^2/m	14∅ 15,7 – 1000 mm = 2100 mm^2/m	∅ 16-100 = 2011 mm^2 2 directions, top & bottom	4 N/mm^2	Mu > Md $\sigma_b < 0,5*f_b$ = 1,08 N/mm^2
Bottom slab	800 mm	16∅ 15,7 – 500 mm = 4800 mm^2/m	19∅ 15,7 – 500 mm = 5700 mm^2/m	∅ 16-100 = 2011 mm^2 2 directions, top & bottom	9,6 N/mm^2	Mu > Md $\sigma_b < 0,5*f_b$ = 1,08 N/mm^2
Outer walls (incl. haunches)	800 mm	18∅ 15,7 – 1000 mm = 2700 mm^2/m	19∅ 15,7 – 1000 mm = 2850 mm^2/m	∅ 16-100 = 2011 mm^2 2 directions, top & bottom	4,8 N/mm^2	Mu > Md $\sigma_b < 0,5*f_b$ = 1,08 N/mm^2
Inner walls	800 mm	28∅ 15,7 – 1000 mm = 4200 mm^2/m	18∅ 15,7 – 500 mm = 5400 mm^2/m	∅ 16-100 = 2011 mm^2 2 directions, 2 sides	8,8 N/mm^2	Mu > Md $\sigma_b < 0,5*f_b$ = 1,08 N/mm^2

The thickness of the inner walls is equal to the thickness of the outer walls, because one compartment can be flooded. In that case, the inner walls have to bear the full hydrostatic pressure, but they experience no wave load.
The inner walls of the outer compartments are used to introduce the connection forces into the pontoon. They are further designed in the next paragraph. For practical reasons, a similar amount of tendons is applied in the inner walls of the outer compartments. A larger distance centre-to-centre is applied to provide space for the curved tendons in the outer compartments. The straight tendons are lengthened (coupled) through the entire inner walls. Haunches of 0,5 * 0,5 m are applied in the top corners of the walls and the top slab of the pontoon. Haunches are also applied at the intersection of the outer walls with other walls and the top and bottom slab.

The centre-to-centre distance of the tendons has been optimised regarding the anchorage of the tendons. The tendons will be tensioned from two sides.
The type of tendon 19K15 requires anchors of b * h = 218 * 218 mm and a minimum centre-to-centre distance of 380 mm. A centre-to-centre distance of 500 mm therefore meets the requirements. The minimum distance of the anchor to the end of the slab is 270 mm, which is equal to 379 mm from the centre of the outer tendon to the end of the slab.

1.5 Durability and concrete technology

The concrete composition has been researched according to the Dutch NEN 5980, as the research had already been concluded before the new guidelines NEN-EN 206-1 came into use in September 2005.

1.5.1 Situation analysis

The concrete pontoons will be constructed in a building.
During the operational stage of the terminal, the pontoons will be floating at all times.
The dimensions of the pontoons are large and the main part of the pontoon is casted in situ. The top slabs may be prefabricated and transported by ship, to be installed while the pontoon is floating. The top slabs may also be prefabricated on site in the construction dock and then placed on the pontoon structure.

The environmental conditions of main importance are:

- The terminal is always floating in salt water of limited depth, environmental class IV.
- The water line and spray zone vary, thus the concrete surface in that area will be changing between wet and dry.
- The spray zone reaches up to an average of 7 m (> F=2,4 m) above the water line in the Netherlands *[Ref. CUR-report 215]* and spray will therefore cover a part of the quay.
- The terminal is always subject to wave loads and response motions; a dynamic load causing fatigue. Wind loads are also dynamic.
- The water in which the terminal is floating is contaminated with salts and acid from port activities and industry.
- The Western Scheldt has a mild wave climate, but a rather strong current.
- The water carries rather a lot of sandy sediment.

The main mechanical or structural conditions are:

- The inner space is variably partly filled with ballast water.
- The individual pontoons have a long design lifetime of 100 years.
- During the lifetime minimum maintenance is required, maintenance activities will have to be carried out while the terminal is floating and in operation.
- The structural loads on the pontoons of the terminal are large, especially the concentrated loads.
- The front pontoons of the terminal are subject to erosion due to the loads from ship propellers and bow thrusters. They may also experience a ship collision.
- The concrete bottom slab and walls of the pontoons are casted in situ.
- The concrete deck plates are prefabricated, installed and connected (casted) on site.
- The reinforcement will be dense and prestressing cable tubes have to be included.

1.5.2 Durability of marine concrete structures

Marine structures mainly come in contact with salt, water and air. This combination can cause multiple forms of degradation. Most deterioration mechanisms were found not to determine the lifespan of the structure. According to *[CUR-report 215: Duurzaamheid mariene betonconstructies]* the most potential deterioration mechanism in concrete marine structures is chloride ingress, as was concluded from test results from several Dutch marine structures.

Expectable deterioration mechanisms are carbonation, leaching, alkali-silica reactions, freeze-thaw action, erosion or salt crystallisation. These seemed virtually absent from the structures that have been researched.
The most severe deterioration of concrete is expected around the water line and above it, in the splash zone, because the structure will be changing between wet and dry at periods of time. Carbonation is strongly reduced when concrete is wet for long periods of time; from the water line downwards. Differences in exposure will cause a large amount of scatter in the chloride profiles of different areas on one structure.

Most deterioration mechanisms and in this case especially carbonation and chloride ingress and the following corrosion initiation can be reduced through the choice of materials. The following aspects should be kept in mind when considering the choice of material:

- An Alkali Silica Reaction (ASR) can be suppressed for the main part by using little or no reactive aggregates.
- The application of blast furnace cement (CEM III/B) causes very dense concrete that is quite resistant against chlorides and it suppresses alkali silica reactions. However, in practice the critical threshold of chloride content for initiation of corrosion of the reinforcement in blast furnace cement is as yet unknown in practice. [*Ref. CUR-report 215]*
- A low water/cement ratio prevents freeze-thaw damage.
- High Strength Concrete develops into a dense mature concrete with high strength. Dense concrete is less permeable and chloride ingress and carbonation may be reduced.
- The carbonation depth should be reduced to less than the concrete cover on the reinforcement. Therefore a large concrete cover in marine structures is beneficial to the lifespan.
- The concrete cover should not crackle from the reinforcement in case of heat (fire) or collision.
- As the influence of early contact with chloride on concrete is not known *[Ref. CUR-report 215],* this contact may be useful to avoid, especially when concrete elements are prefabricated in a building dock anyway.

More governing failure mechanisms are discussed in Part 4 of the Appendices.

1.5.3 Alternatives for concrete composition

Requirements of the building material follow from the environmental and structural conditions of the Western Scheldt Container Terminal:

- The concrete composition should produce a dense and strong structural concrete.
- The concrete should have a minimum permeability because of possible chloride ingress, but it does not have to be watertight/water resistant.
- The concrete composition should ensure that the concrete, which is casted in situ, is well processible.
- The hardened concrete should have a small sensitivity to chloride ingress.
- The hardened concrete should not experience Alkali Silica Reactions.
- The hardened concrete should not be brittle at collision.
- The hardened concrete should not be brittle at fire load (less important in the open air).
- Prestressed concrete needs a better moisture control because of the sensitivity to oxidation of tendons (bars) and anchorage mechanisms (Very small cracks are allowed in the concrete hull).

Six alternative concrete compositions based on strength, durability and self weight are discussed in Part 4 of the Appendices. High strength concrete is preferred for this project. Very high strength concrete is thought to be too expensive considering the large amount of concrete needed. It also requires extra measures during construction and hardening.
The choice between B65 and B100 is based on the resistance of the concrete against degradation mechanisms and on costs.

Blast furnace cement has a high durability and resistance against chlorides. It ensures a dense pore structure and will therefore be very well suitable for the pontoon shaped elements, providing that the ultimate concrete strength is sufficient. The construction conditions will determine the measures to be taken to control the hardening process; the rate of temperature development, hydration and initial strength.
In practice, concrete with blast furnace cement will reach a maximum quality of B65. High strength concrete of a higher quality is made with Portland cement and fillers.

The concrete strength can be approximated according to the next formula:

$$f'_{cm}(n) = a * N_n + \frac{b}{wcr} - c$$ *[ref. Betonpocket 2003 by Enci and Mebin]*

For CEM II-V:
a = 0,85
b = 33
c = 62
B100: f'_{cm} (28 days) = 0,85*100+(33/0,4) – 62 = 105.5 N/mm^2
B65: f'_{cm} (28 days) = 0,85*65+(33/0,4) – 62 = 75,8 N/mm^2
For CEM III/B
a = 0,75
b = 18
c = 30
B100: f'_{cm} (28 days) = 0,75*100+(18/0,4) – 30 = 90 N/mm^2
B65: f'_{cm} (28 days) = 0,75*65+(18/0,4) – 30 = 63,8 N/mm^2
Application of additives allows a lower water cement ratio. As such, a concrete strength of 65 N/mm^2 can be realised. B100 however is not considered to be feasible when applying blast furnace cement.
Even with Portland cement and additives, it will prove to be difficult to reach a concrete quality of B100 when the concrete is casted in situ outdoors in stead of in a factory with climate control etc.

B100 will have a denser pore system than B65, which is preferable regarding chloride ingress. At the same time, the extra structural strength can be used to optimise the pontoon design. However, as blast furnace cement can not be applied in a B100 concrete composition, Portland cement will have to be used in the B100 composition. B100 is too expensive and can survive up to 200 years, which is not necessary for this project.

As blast furnace cement produces a dense concrete with small sensitivity to chlorides, the strongest possible type of concrete composition with blast furnace cement offers the best solution, which is B65. B65 with CEM III/B, rough sand, broken stone and small fillers (fly ash or silica fume) is chosen as the best concrete composition for the pontoons. A low water cement ratio is necessary. Additives such as a plasticizer can be used to ensure a processible mortar on site.

1.5.4 Protection and maintenance

A summary of protection and repair measures of concrete structures is presented in Part 4 of the Appendices.

The economical lifetime of a floating terminal is supposed to be higher than the average economical lifetime of present harbour structures. As such, the technical lifetime of structural elements is set on 100 years in stead of 50 years. Degradation is a process that develops through time; it may slow down or accelerate. A maintenance cycle follows a bathtub shape. A peak exists at the start of the lifetime of the structure; mostly caused by construction mistakes. The second peak is expected near the end of the structure's lifetime, when degradation has already been large.

The floating terminal is a very large structure. The most severe degradation may however be expected around the water line and in the spray zone. Coating (or a cover) may be applied in this zone, but this will be too expensive due to the large surface area. A coating should in average be applied every 15 to 20 years. A plastic or steel cover will also be too expensive.
Therefore, minimum maintenance of the concrete pontoons is required. The main part of the durability of the large concrete structure will be determined by the concrete composition, configured to a maximum chloride resistance.
Steel parts of the connections are maintained using external protection measures and may be replaced after a score of years.
Sacrificial anodes provide a good solution to prevent corrosion of reinforcement and other steel parts in the completely submerged part of the pontoons.

A number of protection and repair measures can be combined to limit costs.
Under water, sacrificial anodes will be applied, as they are not that expensive. They will be replaced every 10 to 20 years. After a part of the lifetime, one may choose not to renew the anodes. The degradation process has already been delayed enough.

A coating around the water line could be used for a certain interval of time; after that the concrete cover will be the main protection of the reinforcement.
If no coating or cover is applied, a monitoring program will offer a solution. The quality of the concrete surface is monitored and repaired when necessary. In that case, the durability depends on the concrete composition, and (large) costs are only caused by necessary repairs.

The combination of these maintenance or repair activities can be described as a maintenance program. The costs of a repair program should be compared with the costs of maintaining a coating or cover.

The conclusion for this thesis is that no protection of the pontoons will be applied. The combination of B65 CEM III/B with sufficient concrete cover will last the expected lifetime. Monitoring of the structure should be applied.

2 Connection analysis

This chapter involves a feasibility study of the type of connections. Many requirements are not quantified.
A spectrum of connector types is possible, because the relative motion between the pontoons consists of six components. Each of these degrees of freedom can be rigid, compliant or fully released. As a result there are merely eighteen possible choices of connector type by this classification category *[Ref. MOB Final Report].* Other marine connectors differ in type of force resistance or the methods of (dis)connection. Each connector concept has its upper limits of motion, which generate (dis)connecting procedures for that connector type.
The key issue for all connector design concepts will be the constructability of the connection and secondly, the distribution of the large connection forces from the connector into the concrete structure of the pontoon. The feasibility of the connector is among others determined by the largest size that can be built and by the difficulty of constructing several of these connectors near to each other.

Four general categories of connection schemes are:
1. Rigid connectors (for smaller elements)
2. Flexible, discrete connectors such as a hinge, a ball joint etc.
3. Flexible, distributed connectors such as a very long and flexible truss bridge between or outboard of the elements of the terminal.
4. Rigid connectors consisting of a combination of multiple flexible connectors
5. Dynamic positioning (for elements as large as a berth) without structural connection

2.1 Connections between floating structures

All connections can be considered on three levels:
1. Structural connector parts:
   A connector can be a pure hinge or it can be a flexible (or compliant) connector with a defined stiffness. A single flexible connector may be flexible in several degrees of freedom, but it does not have to be flexible in every direction. It can be hinged or rigid in a certain degree of freedom. The combination of multiple connectors, taking the locations of the connectors into account, will often lead to a rigid connection in nearly all degrees of freedom.
2. Connection system on one side of the pontoon:
   For the design of the connectors, the connections forces at each side of the pontoon are considered. For example, the connectors on the short side perpendicular to the longitudinal axis will transfer the largest part of the load due to pitch motions. Although one horizontal row of hinges will allow pitch motion, this motion will be restrained by a second horizontal row of similar hinges.
3. Complete three-dimensional connection system of all connected sides of the pontoon:
   To determine the relative motions of the pontoon and the occurring connection forces on the pontoon, the entire three-dimensional connection system has to be considered.

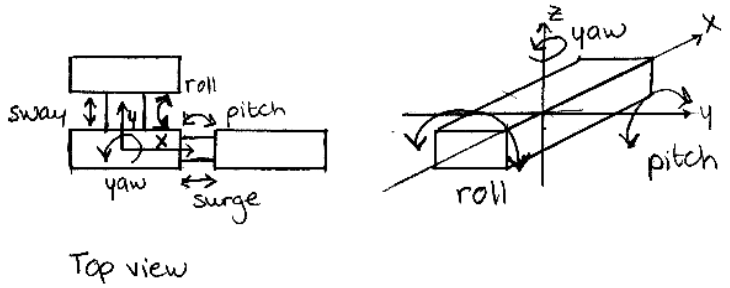


Figure 2.1: Relative motions of connected pontoons

A single compliant connector with a defined stiffness in every degree of freedom can be designed to allow a certain type of motion. The allowed motion is dependent on the type of connector and on its location. The following table shows three types of structural elements of connections:

Type & Direction of connector	**Allowed d.o.f. by 1 connector**	**Possibly allowed translation**
Centre point	All rotations	-
Vertical element	yaw	heave
Horizontal element short side	pitch	sway
Horizontal element long side	roll	surge

Most compliant connectors restrain the translations like surge, sway and heave, simply because these connectors keep the floating structures together. A margin may be left for small motions.
The following table shows what effect multiple compliant connectors have on allowed motions:

Results of combined compliant connectors, considered per side of the pontoon:

		Extra restrained degrees of freedom	
Type & Direction of connector	**Allowed d.o.f. by 1 connector**	**2 hinges, (hor) next to each other**	**2 hinges, (vert) above each other**
Centre point short face	All rotations	roll, yaw	pitch
Centre point long face	All rotations	pitch, yaw	roll
Vertical element long face	yaw	yaw	-
Horizontal element short face	pitch	yaw	pitch
Horizontal element long face	roll	yaw	Roll

In the horizontal plane, the pontoon will be connected at both a long and a short side. The combination of multiple compliant connectors of (only) one type at two or more sides of the pontoon, results in an overall connection with no unrestricted degrees of freedom. The margins in the connectors will determine the possible relative motions between the pontoons and thus and the connection stiffness. The forces in the connections are dependent on the connection stiffness.

		Remaining allowed d.o.f. 3D system	
Type & Direction of connector	**Allowed d.o.f. by 1 connector**	**2 hinges, (hor) next to each other**	**2 hinges, (vert) above each other**
Centre point short face	All rotations	-	-
Centre point long face	All rotations		
Vertical element short face	yaw	-	-
Vertical element long face	yaw		
Horizontal element short face	pitch	-	-
Horizontal element long face	roll		

From the tables can be concluded that a three-dimensional combination of flexible connectors generally results in a rigid connection.

When multiple hinges are combined to a rigid connection, it may prove to be rather difficult to construct a pure hinge or a very flexible connector. In practice, a flexible connection will mostly be constructed by a central connector per side of the pontoon. This combination results in two allowable motions; pitch and roll. Connection forces will be concentrated in one connector.

The design of the connections will be done according to the following principles:
1. Flexibility or rigidity
2. Modularity (standard connections, possibly divided in parts per degree of freedom)
3. Distribution of forces

2.1.1 Approach to the design of connections

The first phase of design comprises the design of the structural elements (members of the structure) to be connected. In this preliminary design, the connections can be modelled as infinitely stiff. When the connections are researched and assumed to be economically constructable, the connection strength and stiffness can be taken into account in a detailed design of the structural elements

The slender pontoon elements can be considered as beams which have to be connected. Compared to a hinged connector, the beams are infinitely stiff.
A connection between beams can be modelled as a bending moment- stiff connection or as a hinge. A hinge transfers normal forces and shear forces, but the bending moment-strength is zero.
In reality, connections are flexible, which means that their stiffness lies between zero and infinity. *[Ref. Staalbouwkundig genootschap 1999: Publicatie momentverbindingen]*

A connector is designed to have sufficient strength and stiffness to resist the loads from construction, operation and fatigue.

A “completely strong” connection would have a larger bending moment capacity than the adjoining structural members (beams; continuous junction). In general, this is not true. Connections usually have a smaller bending moment capacity than the adjoining structural

members. The moment strength of the connection affects the internal force distribution within the structure. Generally, stiff structural parts attract a larger part of the load.

The main difference between flexible connections and rigid connections is the transmitted bending moment. The largest forces in a hinged connector will be axial forces, torsion and mainly (vertical) shear forces, while the transmitted moments are negligible. The rigid connectors between the pontoons of the Western Scheldt Container Terminal will transfer large bending moments. The defined stiffness of the connectors and the small rotations or translations they may experience cause some flexibility in the otherwise rigid connections between the pontoons. The connectors will be loaded in fatigue due to the dynamic forces on the terminal and the relative response motions of the pontoons.

A connector may be modelled in a finite element method - software package with linear spring elements; one for each degree of freedom. Then the structural scheme will also be modelled. This is beyond the scope of this thesis. The connections will be designed by hand.

2.2 Connections applied in the Western Scheldt Container Terminal

2.2.1 Summary of requirements for design

The connections of the terminal in the Western Scheldt will have to meet general and more specific requirements. Several requirements have been based on *[Ref: MOB Final Rreport].*

Functional requirements:

- Connectors will introduce, transfer and distribute connection forces, caused by wave forces, anchoring forces, all structural loads and response motions.
- Connectors will restrict the relative motions of the coupled pontoons within the boundary conditions of the operational criteria of the terminal (Serviceability Limit State).
- Connectors should bear the connection forces and keep the pontoons together under survival loads (Ultimate Limit State).
- The elements should be connected and disconnected with comparable ease; Coupling and decoupling operations in normal weather and waves
- An optimum has to be found between the costs and good usability of a connector
- A crane track will have to be constructed over the quayside, parallel to the ship berths. Consequently a crane should be able to drive from one pontoon to another.

Technical requirements:

- The terminal should have a flat, continuous deck and heave and roll motions should be restricted.
- No damage from relative motions of other elements or moored ships; restricted roll and pitch motions
- The terminal should have a horizontally continuous deck. Surge and sway motions should be restricted.
- Connectors should be possible to manufacture or to construct. They should be maintainable (or durable) and preferably cost-effective.
- Excessive wear, brittle fracture, corrosion and degradation of strength and stiffness of connector materials should be reduced as much as possible (NB fatigue).
- Uncertainty regarding performance should be limited and a design safety factor should be assessed.
- Introduction of connection forces into a pontoon structure should be distributed as much as possible, or located at a part of the hull best determined for that specific load .(torsion, shear, bending). A concentrated connection transfers a concentrated load, which will be difficult to introduce into the pontoon structure without failure.
- Connectors must transfer the anchorage forces of the mooring system of the terminal.
- Connection forces are dependent on the connector stiffness. The maximum structural stiffness of the connector should be chosen as a limit of the connection forces to be introduced into the pontoon shaped elements.
- The elements of the terminal will have to survive storms in a connected state.

Environmental and structural conditions:

- The connector forces depend on the wave length and wave height.
- The connector forces are largest in the corner of the floating structure, facing direct diagonally incoming waves, nearly beam-on.



The requirements in this paragraph are included in the Multi Criteria Analysis of the alternatives for connections.

2.2.2 Practical considerations and solutions

Practical considerations regarding the construction of connectors:

- The connections are expected to be larger than present industry practice.
- Most connections between floating objects have been designed for steel structures. The WCT connectors may be combined concrete-steel structures.
- The maximum thickness of steel plate and rubber material may limit the connector size.
- Uncertainty on reliability and performance of key components of the connection.
- Uncertainty on constructability and construction time (schedule).
- Fatigue may lead to serious fatigue corrosion within a short time.
- Connection forces in the floating structure rise to a maximum in the middle of a maximum span between wave tops. This (static) bending moment may occur in the middle of the pontoon, but also at the connections.

Practical partial solutions:

- A load monitoring system can be installed and maintained as part of the connector system.
- A separation of functions may ensure simplicity of the connector system. The best example is a connection consisting of multiple groups of hinges, which all restrict one specific degree of freedom.
- A large number of smaller connections may be more expensive than a smaller number of larger connections. Therefore the number of connections should be minimised to the necessary number with acceptable connection forces.

2.2.3 System of connections

The first consideration is the assembly of the elements of the terminal and the geometry of the connections within the terminal. The layout of the terminal has been chosen earlier. In this Master Thesis, a terminal of equal rectangular pontoons is assumed. Therefore, the alternative of floaters with load tower structures on top, carrying a deck, is not considered. The connections between the pontoons can be located in several (realistic or more futuristic) ways:

1. All elements are connected similarly; connections are regularly distributed across the terminal area. Connectors should be as much standardised as possible, regarding constructability and costs.
2. The terminal can be covered by a large deck structure or other cover (e.g. a textile cover sustaining a vacuum), while the terminal elements are kept together at the perimeter by a ‘hoop’.
3. The front row quayside elements are the only pontoons that are truly required to be connected rigidly, because of the crane track. This outer ring of pontoons can be connected rigidly, while compliant connectors connect the inner pontoons (storage yard).

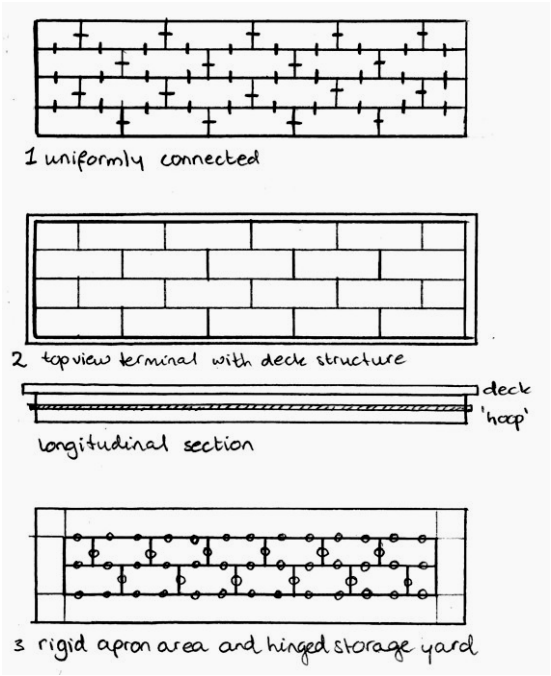


Figure 2.2: Geometry of connections within the terminal: 1. all equal 2. all kept together externally 3. rigidly connected quayside and hinged storage yard pontoons

From these alternatives the first one will be considered in this thesis. It can be combined with the earlier selected layout of the elements of the terminal and it may be a well feasible solution. There is not sufficient time to research every alternative and to compare the cost estimates.

When all pontoons are connected equally, the type of connection has to be selected. This choice is simplified to a rigid or a compliant connection. In the following table, the advantages and disadvantages of both connector types are listed:

Connector type	Positive aspects	Negative aspects
Rigid	Small or no motions	Large connection forces in heavy sea loads
	Complies with crane track requirements	Complex connection type
	Easy internal transport of containers etc	Difficult to decouple the connectors
	Easy support for access bridges from the shore	Shrinkage/creep
	One entire deck on the platform	
Flexible	Low connection forces	Large complicated connectors
	Low(tensile)stresses from shrinkage/creep in the summer	Many motions with large amplitudes
	Easier force distribution from connection into element	Crane movements
	Easy decoupling of connectors	Movements of stored containers

Connector type	Positive aspects	Negative aspects
		Complex internal transport
		A flexible deck with gaps (or short bridges) between pontoons
		Expensive?

In this thesis, rigidly connected pontoons regarding the quayside are developed *[Ref. Part 3]*. The crane track performance has been assumed to be a hard design requirement. No alternative crane types are applied in this thesis.

2.2.4 Limit states

Two limit states are taken into account during design:

1. The Serviceability Limit State (SLS)
2. The Ultimate Limit State (ULS)

The Serviceability Limit State represents the limit of maximum loads, and maximum allowed motions, deformations, accelerations etc. during operation. Actually, these limit values are based on the operational criteria of the terminal. The terminal will be set out of operation at a storm that is stronger than the 1/1 year storm. Loading and off-loading activities of ships will stop. Cranes will no longer be in operation. The Serviceability Limit State includes:

- Maximum wave height and wind speeds caused by a 1/1 year storm [Ref. Part 1].
- Maximum crane motions and accelerations [Ref. Part 2].
- Maximum relative motions of quay and ship [Ref. Part 2].

When a heavier storm occurs, the terminal will be put out of operation and ships will leave the berth to prevent damage from collision due to relative motions between ship and quay.

The Ultimate Limit State represents this survival situation.
The pontoon shaped elements of the terminal will not be disconnected at any time during its operational lifetime. The terminal is moored in one place and in one direction. Therefore, the maximum loads during storm are the design loads for the pontoons.
The Ultimate Limit State includes:

- Maximum wave height and wind speeds caused by a 1/100 year storm [Ref. Part 1].
- Maximum storage or ballast loads.
- Crane and equipment loads, excluding traffic or cargo movements (the terminal is no longer in operation).
- Maximum connection forces due to wave and response motions.

As a ballast system has been chosen for the entire terminal, the situation of one empty pontoon connected to one fully loaded pontoon is not considered. This would be a separate loading combination with very large heave connection forces. It has not been calculated in this thesis.

Non-linearity of the connector stiffness would be an issue for hinged connections, when a lower connector stiffness would be preferable during storms (larger motions, larger forces). However, the non-linearity principle is not considered feasible regarding rigid connections.

2.3 Alternatives for connection

A floating body that consists of rigidly joint elements can be approximated as a single rigid body. The hydrodynamic behaviour of a rigid body is favourable as wave induced motions will be small. The motions should be within the allowable range of motions and accelerations of the container gantry crane that travels the quayside over a crane track.

Different structural principles in stiff connectors:
- Fix all translations and rotations; a six degrees of freedom-connector
- Fix all translations and rotations, but install a compliant material in between connector-parts.
- Fix relative translations and allow only very small relative rotations by margins in the connector; not in all directions
- Allow only limited pitch by applying several connectors; mainly attach each face of the pontoon rigidly to the top deck edge. This type of connector allows a large amount of pitch.
- Separate functions of the connections in separate structural parts; this can lead to a connector easier to produce and construct. Every degree of freedom can be restrained, limited or entirely blocked individually. Multiple hinges ensure rigidity of the connection.

The design alternatives can be divided in the following applicable construction principles that may be considered either as realistic or more futuristic:
1. Tensioning the elements together
2. “clicking” the elements together (such as multiple hinges; in all degrees of freedom)
3. anchoring a concrete toothed structure
4. Using vacuum (air pressure) or magnetism or glue to construct one stiff structure; either sucking connection between two elements or a cover containing all elements (like coffee)
5. A packaging or wrapping around the entire terminal or a part of it such as the outer row of elements to limit the structure’s movements in every direction.

The connector will be designed on strength, constructability and durability, while costs should be minimised. These design criteria have been subdivided in secondary aspects. Six alternative connections will be considered; the choice of best alternative connector is based on a multi criteria analysis, which includes these aspects.
The criteria are clarified in the table below. The weighting factors in the matrix present the importance of a factor.

Main criterion	Secondary aspect	Remark/clarification	Range good - bad
1. Structural strength	Ultimate allowable force [2]	Dependent on type and dimensions of conn.	Large allowable force – small force
	Connection force distribution on pontoon face [5]	Concentrated or distributed connection loads on pontoon	Distributed well over concrete cross section – forces concentrated
	Location of connectors [4]	Favourable geometry regarding location of max loads; torsion, moment, shear etc	Connectors along outer shell and top/bottom side – connector concentrated in centre
	Load coinciding with strong axis [2]	Favourable type of loading; bending, shear or normal force	Preferably distributed normal force – concentrated shear or bending/torsion

Main criterion	Secondary aspect	Remark/clarification	Range good - bad
	Occurring peak stresses [3]	Shape of connector may cause peak stresses in concrete pontoon, causing damage	Distributed low stresses – concentrated peak stresses
2. Constructability	Complexity structural detail [1]	A combination of connector parts may be difficult top construct	Easy to design and construct – difficult to construct
	Watertightness [2]	Should connection be made watertight, can water enter the inner space of pontoon?	Watertight without extra measures – not watertight
	Coupling during wave motions [5]	Will wave motions cause damage or make coupling impossible?	No damage/easy coupling – coupling impossible without damage
	Coupling margins [3]	Are large structural margins necessary to make coupling possible?	Small margins – large margins
	Temporary coupling measures [3]	Extensive temporary clasping structure needed? Fast re-use possible?	No extra measures – extensive semi-permanent structure necessary
	Joining technique [4]	Independent joining per pontoon or join multiple pontoons at once; possibly add connector afterwards?	“Click” separate pontoons – join row of multiple pontoons to terminal at once
	Easy decoupling [2]	Independent “click” system or decouple multiple elements. Is part of connection destroyed in the process?	Independent decoupling/no damage to connector – multiple decoupling, connectors destroyed
3. Durability	Material durability in water [2]	Is connection fully exposed to water and what is the materials’ resistance?	Small exposure/ durable material – complete exposure/sensitive material
	Damage chance [4]	Damage chance during operation (collision) or construction	Very robust conn – connection, very sensitive to damage
	Fatigue chance [4]	Is the connection fatigue-sensitive?	Non fatigue-sensitive design – extra measures needed against fatigue
	Maintenance and repair [3]	Is connector easy to repair and maintain? (can it be reached during operation?)	Easy to maintain – difficult to reach and maintain
4. Costs	Material use [4]	Amount of material	Small – large material use
	Construction method [3]	Made on site or expensive prefabrication; shape	Simple, on site – complex and prefabrication

Main criterion	Secondary aspect	Remark/clarification	Range good - bad
	Number of connections [1]	Costs per connector *nr of connections	Small – large number of connectors
	Connector dimensions [2]	Costs influenced by dimensions; Possibility to manufacture a large scale	Small, distributed connector parts – large, concentrated connector
	Temporary coupling measures [2]	Extensive temporary clasping structures needed?	No extra measures – multiple temporary structures necessary

- Determining weighting factors:
  Four main criteria have been defined, with four to seven secondary aspects each. The secondary aspects are all defined with a weighting factor varying from 1 for “of smaller importance” to 5 for “very important”. This amount of points allows for a good differentiation between aspects, but it does not produce unnecessary many points. Only three points would not be sufficient to show the difference in importance between aspects, and more than 5 points would produce an unclear analysis.

- Distribution of points between main criteria:
  The points for the secondary aspects can be summed up to a distribution of points over the main criteria: 16 points for structural strength, 20 points for constructability of the connection, 13 points for durability and 12 points for the costs. Strength and constructability are by far the most important issues. The total amount of points in the weighting factors is 61.

- Assigning of grades:
  The secondary aspects are all weighed per alternative with a grade varying from 1 for “very bad” to 5 for “very good”. These grades will be multiplied by the weighting factors. Both weighting factors and grades have a distribution of 1 to 5 points, because 5 points provides the desired accuracy and point distribution.
  The score of the alternative is determined by

  $$\sum(weighting\ factor * grade) \quad \text{for all main criteria}$$

  The “best” alternative produces the highest score. The average grade can be determined by

  $$average\ grade = \frac{total\ score}{61}$$

The main ideas behind the multi criteria analysis are shown in the following table:

Criterion	Notes
1. Structural strength	Most important is that the connection forces can be introduced into the pontoon.
	Next, the connector has to bear the load and peak stresses are unwanted.
2. Constructability	Most important is a smooth coupling procedure without much damage.
	Preferably not many or large temporary structures and little work after coupling.
3. Durability	The connections should either be very durable or easy to maintain and repair.
	Fatigue may prove to be the most important failure mechanism of the connectors.
4. Costs	The costs of the connections are mainly influenced by the material use and the construction method of the connectors.
	The geometry of the connections should be taken into account in the design.

The elements have to be connected while floating. The wave induced motions strongly influence the coupling process and the necessary connector margins. These aspects are heavily weighed in the multi criteria analysis. The amount of possible damage due to coupling depends on the type of connector and the way forces are introduced into the structure. For example, large heave motions may fracture a horizontal protruding connector element.

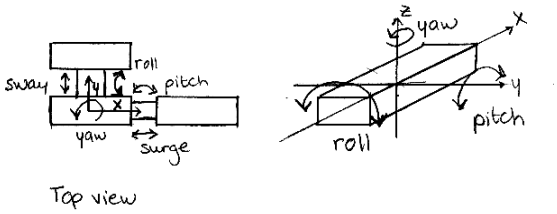
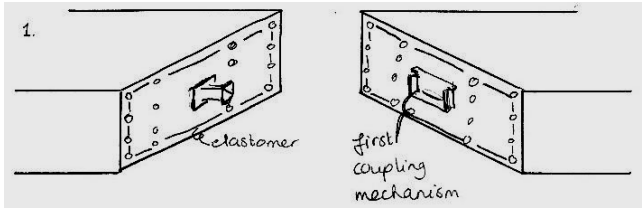
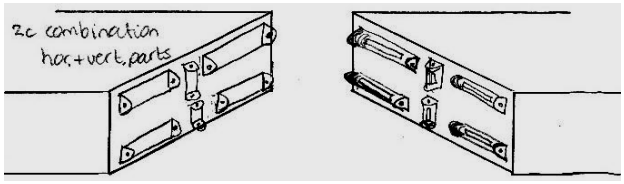


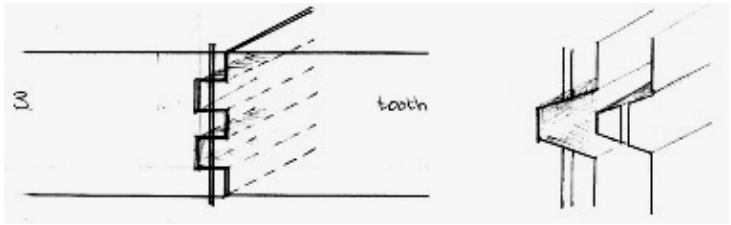
Figure 2.3: Motions during coupling procedure

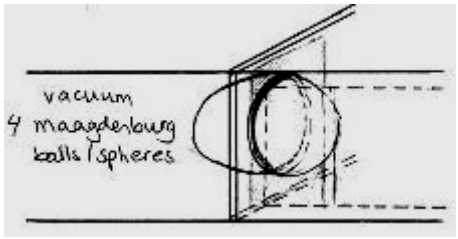
The alternatives for connection are:

1. Prestressing with post tension cables or prestressing bars
2. A horizontal or vertical joint with multiple steel piano hinges;
   partly above and partly under the water line.
3. A horizontal or vertical concrete toothed connection with a steel pin
4. A joint using vacuum; like the balls of Maagdenburg (partly internal)
5. A joint using vacuum; a vacuum pad (external)









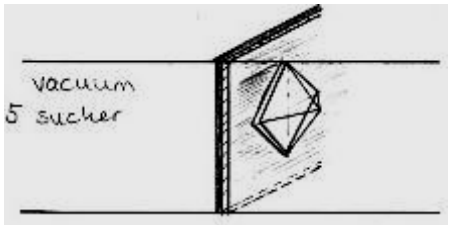


Figure 2.4: 5 alternative rigid connections

2.3.1 Tensioning

- Description:

To tension the pontoons to each other, prestressing elements should be led through ducts in the pontoon hull and tightened. In-between pontoons, a rubber or elastomer material will be installed (on the face of the pontoon). This will be combined with either a concentrated hinge (during coupling) or a clamping mechanism.
The large dimensions of several pontoons together cause problems. Either tendons would have to be lengthened and anchored or every two or three elements would have to be tensioned together and the last tensioned to the next group. This internal prestressing and anchoring within the element could more easily be gained by applying prestressing bars. The concrete hull would have to be adapted inside.
Prestressing is difficult to remove, therefore tendons should not be attached to the sleeves by grouting. However, this leaves the tendons not very well protected against corrosion save for the grease in the sleeves. A detail of the watertight closure at the end of the hull has to be designed.

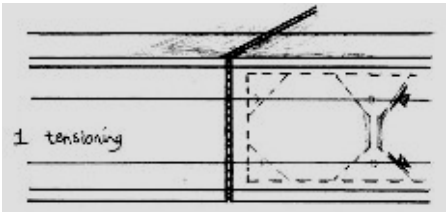


Figure 2.5: Alternative 1- tensioning

- Main characteristics:

The principle of this connection is simple. The connection is very rigid. A clamping structure may be needed for temporary coupling. The connection can be optimised considering the type of prestressing and construction.

- Applicability regarding requirements:

To tension the elements together would generate the most rigid connection available. It would generate the best distribution of the connection forces as well. The structural strength is large, but shear from e.g. heave motions may cause problems. Connecting the pontoons during wave motions will be somewhat difficult; firstly because the ducts for the tendons or bars are small and secondly, because the first prestressing would snap under the large connection force. Therefore a temporary structure is needed until all prestressing elements are installed. Special attention is needed to create a watertight connection and to ensure that the prestressing elements will not corrode. The costs of the connectors are small. However, to disassemble the terminal, the prestressing may be destroyed and a large part of the connections will have to be renewed.

2.3.2 Multiple steel hinges

- Description:

Two or three rows of horizontal piano hinges can be constructed in steel, at least one near the top slab and one near the bottom slab of the pontoon. The combination will restrict all relative motions.
Vertical rows of piano hinges can also be constructed in stead of the horizontal rows.

These could be constructed every 10 or 15 m (compartment dimensions).
The last option is a combination of horizontal and vertical flexible connectors. This is not necessary regarding the restriction of motions, but the different structural elements may be able to bear the load easier, especially because the vertical hinges will be heavily loaded by pitch in an undesired direction.

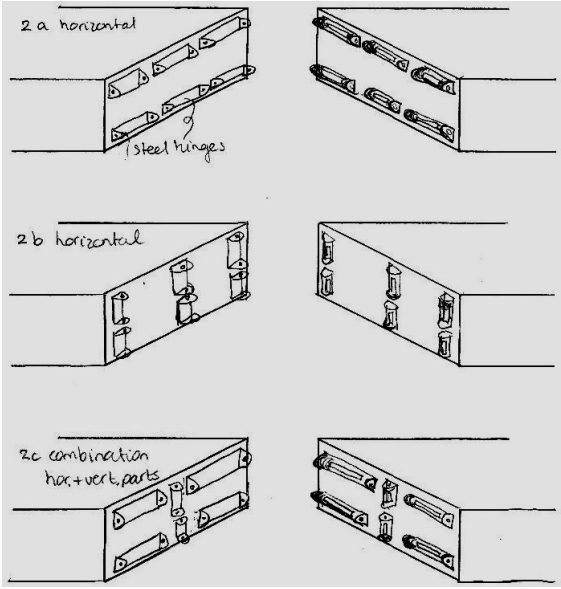


Figure 2.6: Alternative 2 - different types of multiple steel hinges

- Main characteristics:

This connection is based on the use of multiple hinges, either horizontal or vertical, which produce a rigid connection. It can be compared with the idea of the MOB compliant connections. The connection is made from steel parts. The steel parts are attached externally to the pontoons.
The connectors are distributed over the sides, to decrease point loads and to increase rigidity.
The compliant connectors will be locked by a steel bar (tube). This tube will be easier to insert into the coupled connection in the vertical direction.

- Applicability regarding requirements:

The large dimensions of the steel bar and the hinge may guarantee sufficient stiffness, but the margins of movement within the hinge are large. The protruding steel connectors can be considered as vulnerable parts that can be damaged easily. The distribution of one hinge at the top and one hinge at the bottom of the side of the pontoon is a good solution for transferring bending moments.
Heave motions will hinder the coupling procedure, especially when a bar must be put into the connection horizontally. A centre ball joint or centre tooth connection may be necessary to reduce these motions during coupling. Only part of the connection load should be concentrated at this location.
The steel connectors will be relatively expensive, compared to concrete or prestressing elements. As the connectors are attached externally to the pontoons, the connection is perfectly watertight. However, the connectors are hard to reach for inspection or maintenance and steel is not quite as durable as concrete. Especially at a low self weight, fatigue of the connectors may lead to early repairs or replacement. Replacement of the

steel connectors would be more expensive than replacing the prestressing elements when the terminal is disassembled.

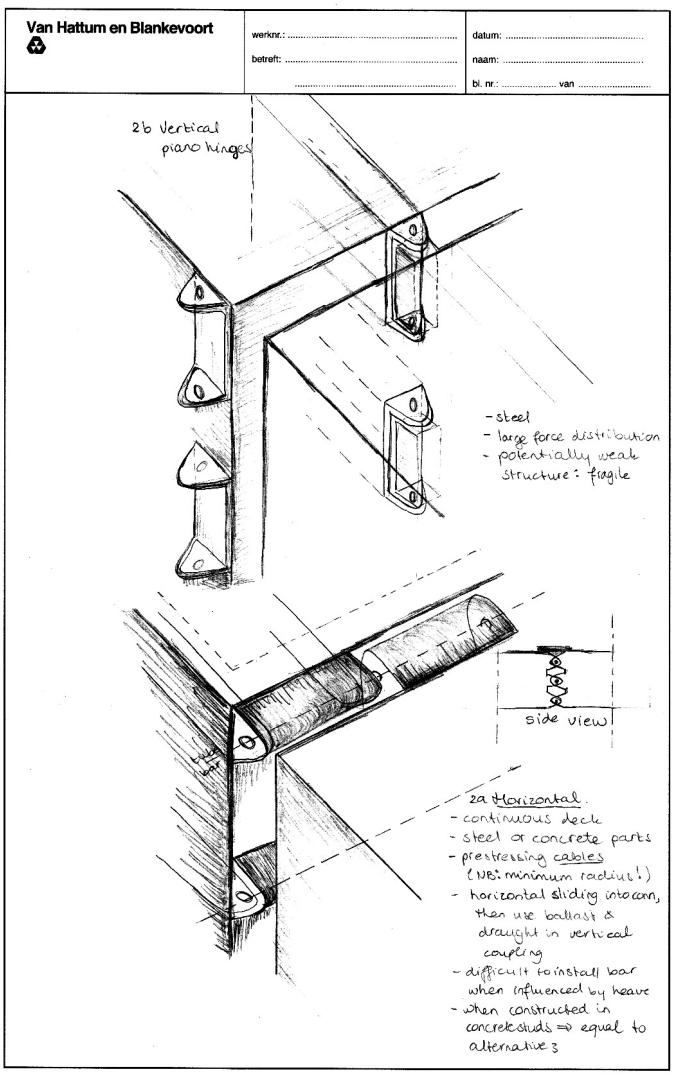


Figure 2.7: 3D impression of steel hinges

2.3.3 Concrete toothed connections

Description:
A concrete toothed connection can be constructed horizontally or vertically. The concrete teeth or studs can have different shapes; rectangular, trapezoidal or rounded. The studs can be continuous over the height or length/width of the pontoon, or they can be interrupted. The toothed connection will be locked by a steel bar. Several horizontal or vertical rows of studs can be applied. A continuous deck is ensured by a cantilever beam attached to the top slab of the pontoon.

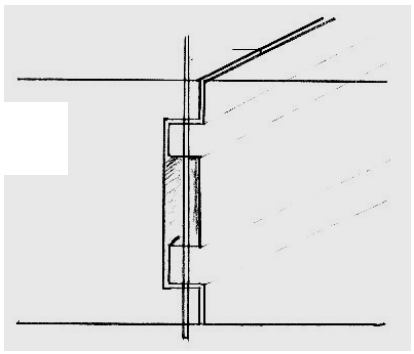


Figure 2.8: Alternative 3 - Concrete toothed connection

- Main characteristics:

The connection is a typical “knitting needle” connection. The principle will be much like alternative 2. However, it is a more (preferably simple) solid, sturdy connection; the connectors are heavily built and can introduce large forces. The protruding studs will have to deal with impacts from heave motions. The connection should be slid or shoved in place and locked, which will be an accurate job. A ballast system would be very useful to influence the vertical position of the pontoon.
The “knitting needle” can be applied externally from the pontoon. Shorter tubes may also be used in combination with a connection that can partly be constructed from the inside of the pontoon.

- Applicability regarding requirements:

The large protruding studs are not easily damaged during coupling, but they will hinder the coupling procedure. The easiest way would be to use vertical motions of the pontoons to shove the connector parts into place. Another possibility would be to construct a few connectors after coupling of the pontoons.
Again a vertical tooth connection would be simpler to couple than a horizontal one. Disassembling the terminal can be quite easy by pulling out the steel tube. A tooth connection in concrete can be a stiff connection when loaded in compression, but the margins needed to insert the steel bar during coupling, cause leeway that allows small rotations. After coupling, a kind of wedge or rubber profile can be driven in to ensure the connection’s rigidity. The dimensions of the connector and the introduction of connection forces through reinforcement and prestressing elements can prevent most peak stresses. The connectors will be able to bear high loads. Due to the high self weight of the connector parts, fatigue will be less of a problem. Also concrete is a rather durable material, even though the connectors are difficult to reach for maintenance or repair.

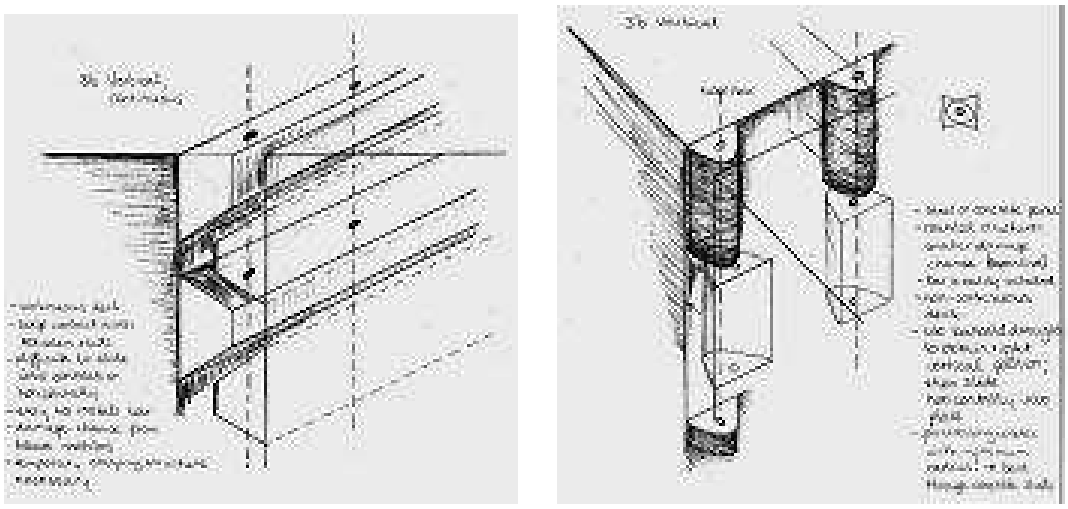


Figure 2.9: 3D impression of different types of concrete studs

2.3.4 Vacuum spheres

- Description:

Two large half spheres or balls may be constructed inside the elements at the small faces, and several more along the long faces. These balls would have to be made of steel or copper, like a large ball-joint. Next, the steel structure would have to be anchored in the concrete hull of the element and the faces would have to be completed watertight and including facilities to pump the water and air out of the connected spheres. Perhaps even external prestressing within the pontoon may be used, but the prestressing elements are difficult to attach to the sphere without welding the high strength steel.

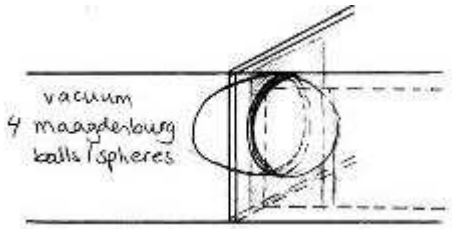


Figure 2.10: Alternative 4 - vacuum spheres

- Main characteristics:

The alternative is based on the principle of the spheres (balls) of Maagdenburg. The half spheres create a very stiff connection in all directions. All motions can be restrained by only one connector. The connection forces will be distributed over several connectors, because the vacuum spheres have their limits regarding structural strength and transferable forces.

- Applicability regarding requirements:

In theory the idea could work, but it seems much too expensive and perhaps not technically feasible. It would be very easy to connect and disconnect the pontoons and the connection would be rigid in all directions, but the feasibility of the connection also depends on the ultimate allowable force in the connector. As the connectors are internal for the largest part, the material will be easy to maintain. The face of the connectors does come into

contact with water, this part should be protected. The balls would have to be poured steel or bronze, which is very expensive, especially regarding the number of connectors for the terminal.

2.3.5 Vacuum pads or magnets

- Description:

Vacuum pads (or large electrical magnets) could be distributed over the pontoon face and anchored in the hull structure. The seam between the two pontoons will have to be protected from water by a rubber profile along the circumference of the pontoon face, from which water can be pumped out. The connectors will thus be attached externally to the pontoon face. The connection should be watertight and in the case of the magnets, the electrical circuit should be well protected from water and should have a backup circuit. The pontoons should not be able to disconnect suddenly, therefore an extra kind of lock on the magnet or vacuum pad connection would be advisable.

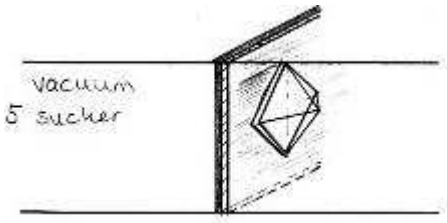


Figure 2.11: Alternative 5 - vacuum pads or magnets

- Main characteristics:

Vacuum pads or strong magnets form a rigid connection. They can be evenly distributed over the sides of the pontoons. It is like gluing the pontoons together.
The structural strength both of the pads and the magnets regarding bending moments may not be very large and depends on the type of structure and its construction.

- Applicability regarding requirements:

The suckers may have smaller dimensions and be more constructable and less expensive than half spheres, but the main question is whether the ultimate transferable force is sufficiently high to restrain the movements of the pontoons. The half spheres have an advantage, because they are circular and the stresses from motions in all directions will be transferred easily. This is not the case with rectangular vacuum pads. However, the smaller pads are distributed more evenly. The pads will be expensive connectors. They are protected from water, but their durability is questionable. Maintenance is almost impossible, because the connectors can not be reached.

2.3.6 Choice of alternative

The alternatives are awarded a grade per aspect, which is multiplied with the weighting factor.

		Alternatives				
Main criterion	***Aspect***	***1***	***2***	***3***	***4***	***5***
1. Structural strength	Allowable force [2]	5	3	4	2	1
[16]	Distribution of connection forces [5]	5	3	4	1	1
	Location of connectors [4]	5	4	3	2	3
	Load coinciding with strong axis [2]	4	2	4	4	2
	Peak stresses [3]	4	4	2	2	2
2. Constructability	Complexity structural detail [1]	4	2	4	1	2
[20]	Watertightness [2]	2	5	4	4	4
	Coupling influenced by wave motions [5]	2	1	4	2	2
	Coupling margins [3]	3	3	1	4	4
	Temporary coupling measures [3]	1	2	3	3	2
	Joining technique [4]	2	4	3	4	4
	Easy decoupling [2]	1	4	3	5	5
3. Durability	Durability in water [2]	3	1	4	3	3
[13]	damage chance [4]	3	1	2	2	3
	Fatigue[4]	5	1	4	4	4
	Maintainability and repairability [3]	3	2	5	3	2
4. Costs	Material use [4]	3	2	4	1	2
[12]	Construction method of connector parts [3]	4	2	5	1	1
	Number of connectors [1]	2	2	3	2	3
	Connector dimensions [2]	5	3	3	1	3
	Temporary coupling measures [2]	2	3	3	3	3
	Total: 61	198	161	209	153	159
	Average	**3,25**	**2,64**	**3,43**	**2,51**	**2,61**

5 very good; 4 good; 3 neutral; 2 bad; 1 very bad



Alternatives I, II and III seem feasible connections. The scores of alternatives I and III are close. The concrete toothed connection (III) is developed as the "best" alternative. It is assumed to be the strongest connection which can be applied to bear the high connection loads. The coupling procedure of this alternative is controllable and feasible.
Several sketches and drawings of all three feasible alternatives are included in the appendices.

2.4 Development of best rigid alternative

The main question for alternative III: the "knitting needle" connection is the shape of the connector studs and the direction, horizontal or vertical, of the locking bar.

Comparison of toothed connections: *[fig. 2.9 and 2.13]*

Direction of locking bar	**Type of connection**	**deck**	**Shape of studs**
3a. Horizontal	Continuous	Continuous	Rectangular stud (vert.)
	Non-continuous	Continuous	Trapezoid stud (vert.)
			Rounded stud (vert.)
3b. Vertical	Continuous	Continuous	Rectangular stud (hor.)
	Non-continuous	Discontinuous	Trapezium stud (hor.)
			Rounded stud (hor.)

The continuity of the deck will be ensured by a cantilever beam attached to the wall and top slab of the pontoons. The camber in the deck might also be solved by a filling structure, but this structure may not be strong enough to carry the live load on deck.

Comparison of properties of the shape if the studs: *[fig. 2.9 and 2.13]*

Aspect	**Rectangular stud**	**Trapezoid stud**	**Rounded stud**
Damage chance	--	0	+
Continuous deck 3b	+	++	--
free motion within connection	-	0	+
Necessary tolerance	-	0	+
Coupling activity	--	+	+
Constructability	++	+	--
Total	-	+	0

A trapezoid stud will be applied, because it is less expensive and more practical than a rectangular or a rounded stud. The rectangular studs take up too much space. The margins during coupling will be too small. The corners of the concrete connectors may easily crack.

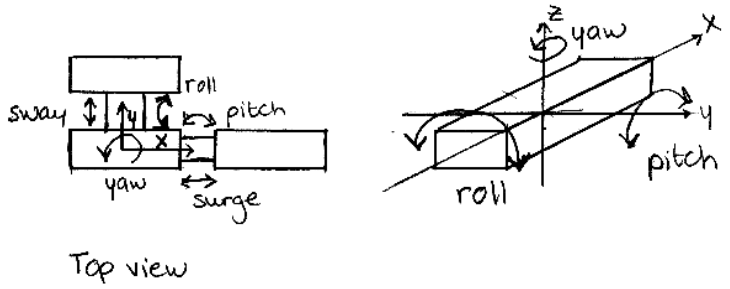
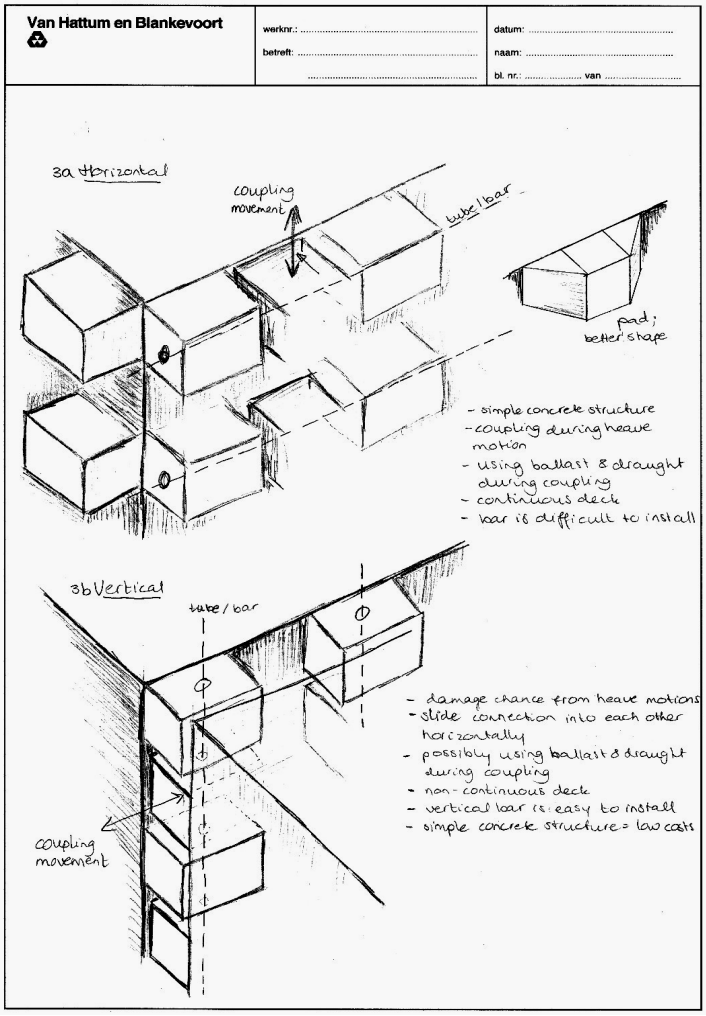


Figure 2.12: Motions of connected pontoons

Comparison of properties of the connections:

Aspect	3a. Horizontal	3b. Vertical	note
deck	continuous	discontinuous	Or use continuous hor. studs
Height (z-direction)	smaller	larger	
Surface contact area	Large sideways (hor.)	Large vertically	
Coupling motion	Float up/down	Slide	Sliding may cause damage
Install locking bar	Difficult	Easy	
Coupling damage from heave	Large	Small	Smaller height means more damage
Surge or sway	Compression studs or shear in bar	Compression studs or shear in bar	
Roll or pitch	Shear, Bending in bar, friction between studs	Shear in bar, normal force in studs	
heave	No contact area, shear in bar	Compression Contact area; normal force in bar	
yaw	Shear in bar, row of studs is twisted (friction)	Bending in bar, normal forces in studs	
Location of force introduction	Normal forces easy, shear difficult into stud; friction	Shear and normal forces easy into stud	
Total	0	+	

The vertically locked connection is assumed to offer the best solution. This connection will be designed. From structural point of view the vertically locked connection is preferable for the concrete studs. For the steel tube, a horizontal direction will cause smaller (mainly shear) loads. Coupling the pontoons by installing the bar in moving water is easier when it can be done from the top of the pontoon, while tugboats keep the structure in place horizontally.
The vertical concept has only one disadvantage: the direction of the coupling motion. Sliding a floating pontoon into place horizontally may cause damage. Larger tolerances will be necessary for sliding than for floating the pontoon into place vertically. This problem will be solved with a temporary clamping structure.



An extra bar through the studs may be designed perpendicular to the face plate of the pontoon (x or y-direction). These bars are mainly loaded in tension or compression by surge, sway, roll and pitch motions. This is more favourable than bending or shear.

3 Design of rigid connector

The design of the pontoon has been made for the outer pontoon in the west corner of the quayside, directly facing wave attack. The design of the connectors will be made for the same pontoon, as this is the governing location regarding wave forces and response motions.

3.1 Concept of connector

The concept of the best connector is a vertical “knitting needle” connection with discontinuous trapezoid concrete studs. The studs are constructed on the sides of the pontoon and located on the inner walls. The studs are not constructed at the outer corners of the pontoons, because local force distribution will cause problems in the outer walls.

- The tube and all other connector parts are attached externally of the pontoon.
- The space between the top studs will be filled by cantilever beams to ensure a continuous deck of the terminal.
- At the long side of the pontoon, there are 11 vertical rows of studs (11 inner walls).
- At the short side of the pontoon, there are 5 vertical rows of studs (5 inner walls).
- The distribution of the suds over the structural height of the pontoon is done according to the force distribution. Two large studs will be constructed at the top and at the bottom of the sides of the pontoon. The stresses caused by bending moments will be small at half of the height of the pontoon. No studs will be constructed in the centre of the height of the pontoon.
- The location of the force introduction is ensured by applying only two large studs at the top and bottom of the pontoon. There is minimal uncertainty about the location of the connection forces.
- The vertical space between the studs larger than the studs. The steel tube can be lowered into the bottom studs from this space.

Two final choice of stud system is dependent on the shear load on the steel tube. The entire normal force between the two pontoons is transferred through the steel tube as a shear force.
The figure to the right shows a more complicated shape of the connection, which provides a better distribution of the shear force on the steel tube.

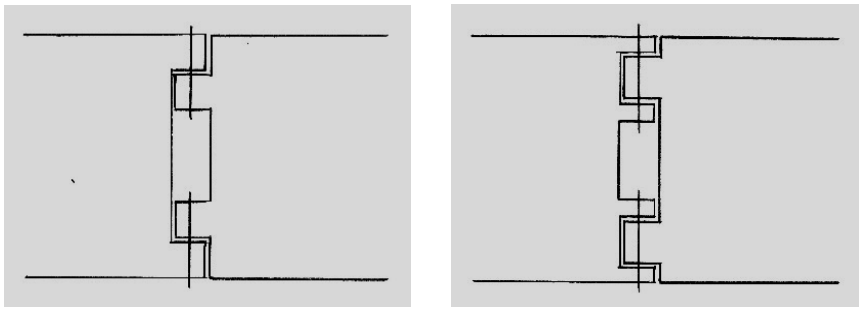


Figure 3.1: Left - a stud system with 1 governing section of the steel tube
Right – a stud system with 2 governing sections of the steel tube.

However, calculation *[Appendices, Part 4]* show that the left type of connection is feasible with a steel tube of 1m diameter and a thickness of 15 cm.

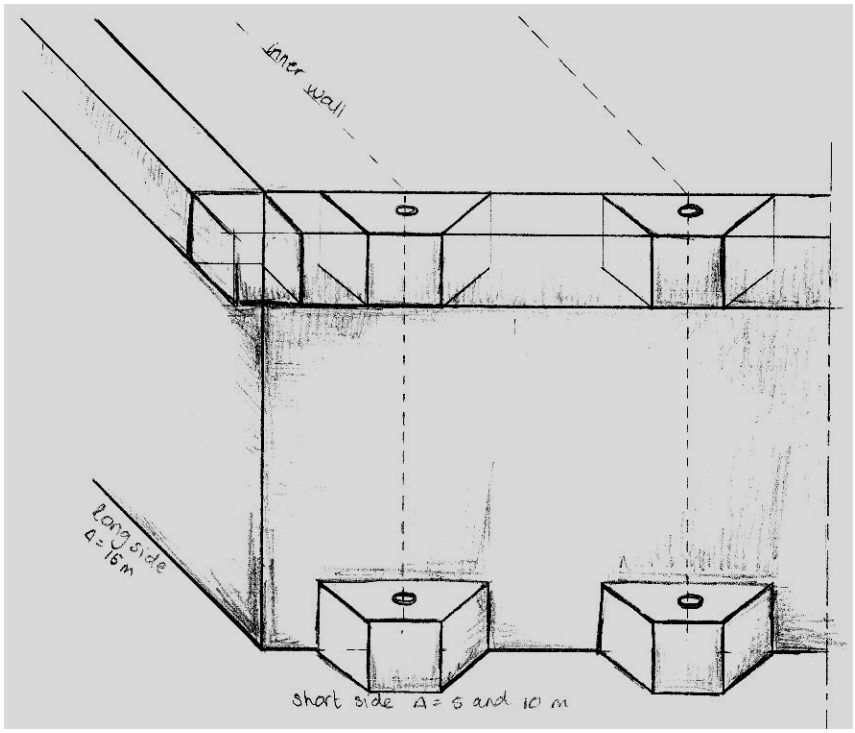


Figure 3.2: 3D impression of short side of a pontoon with top and bottom connectors and a cantilever beam at the top slab

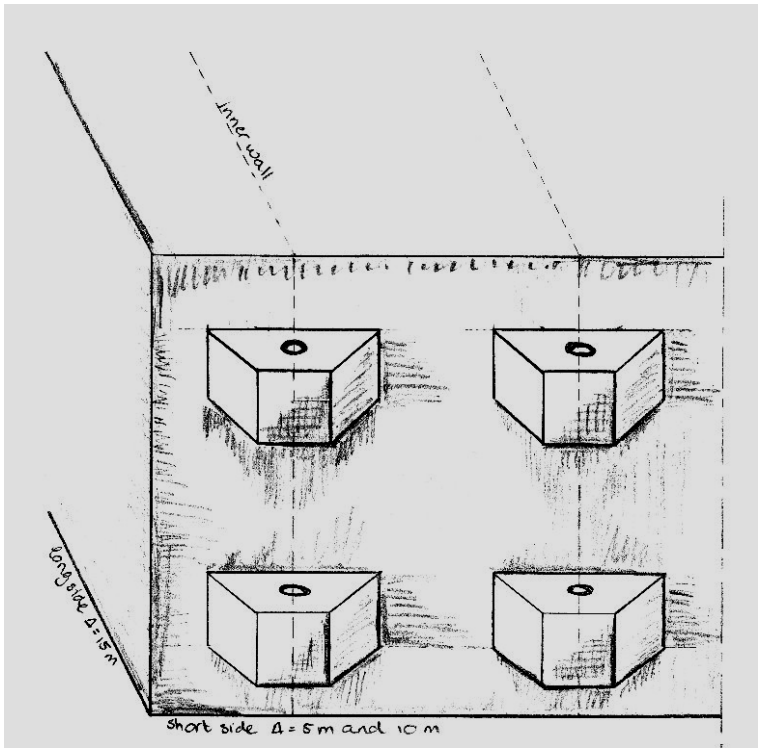


Figure 3.3: 3D impression of short side of the pontoon with two inner connectors

3.2 Connection forces

A number of starting points is formulated to simplify the calculation of the distribution of the connection forces over the various connectors.
The rigid type connector will have to transfer the following maximum loads. These loads occur at the front corner of the quay of the terminal, directly facing the dominant wave attack.

Motion type	Connection force		Load per length or width kN/ m[1]	Distributed load kN/m[2]
Surge	6962,42	kN	136,79	8,82
Sway	5296,13	kN	104,05	1,89
Heave	7056,40	kN	138,63	2,52
Roll	180685,31	kNm		
Pitch	1057547,33	kNm		
Yaw	432154,21	kNm		

The dimensions of the pontoon are 180*50*16 m.
The long side is parallel to the longitudinal axis of the pontoon. The small sides are parallel to the transversal axis of the pontoon.

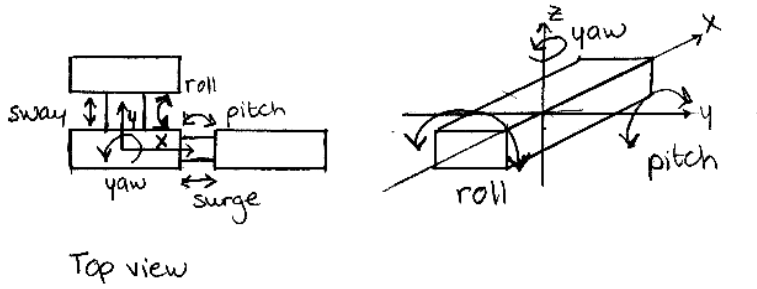


Figure 3.4: Motions of connected pontoons

The connection loads are calculated for a stud connection with two studs at the top and bottom of the side of the pontoon [fig.3.1] at every strand of the pontoon pattern (every wall).

Starting points:

- The corner pontoon of the quay is connected to other pontoons at two sides only. Therefore the connection forces are distributed over two sides, which is a governing situation.
- The connection forces are considered locally, separate from the general force distribution in the pontoon. The stresses caused by the connection forces will be taken into account in the concrete calculations of the pontoon.
- The connection forces are known in one corner point from Delfrac, in six degrees of freedom.
- These results are all maximum values, which do not cohere. Every maximum may occur at a different configuration of the waves. For this reason, the degrees of freedom are dealt with separately. In the worst case scenario, for example surge forces may not be present to counter a yaw motion.

- The 6 maximum values are used because of the large amount of possibly governing situations from Delfrac; the governing situation is hard to determine as 1*max + 5 coherent or perhaps 6*90% forces may be governing.
- The translations surge, sway and heave are evenly distributed as a normal force over the connectors at the short side, long side and both sides of the pontoon respectively.
- The bending moments in the vertical plane due to roll and pitch are distributed according to their distance to the centre of gravity of the pontoon. They are introduced as a compression and tension force on the pontoon (torque), divided by the centre-to-centre distance of the top and bottom connectors.
- The bending moment due to roll is introduced as a torque (normal forces) at the long side of the pontoon. It is introduced as a vertical shear force at the short side of the pontoon.
- The bending moment due to pitch is introduced as a torque (normal forces) at the long side of the pontoon. It is introduced as a vertical shear force at the short side of the pontoon.
- The bending moment due to yaw in the horizontal plane is introduced into the connectors as a horizontal shear force.
- The mooring forces are taken into account as a normal force per connector; 19 kN/m[1] pontoon length, divided by the number of connectors on the long side of the pontoon.
- The combination of normal forces and shear forces lead to a resultant discrete force on the connector. The governing situation is a top or bottom stud at the short side of the terminal.
- The connector is designed with regard to this resulting force.

The calculations and drawings are included in *[Appendix Part 4B]*.

The maximum bending moments, shear and normal forces in the hull structure of the pontoon, are caused by the buoyancy force and the waves. These bending moments, for instance due to a wave trough, can also occur at the connections between the pontoons. Therefore, the maximum bending moments due to waves are compared to the maximum connection forces from Delfrac. The connection will be designed for the resulting force on the connector of the governing situation.

Governing situation:
Top stud at the short side of the pontoon: discrete system.
Centre-to-centre distance studs h = 11 m

Load	Bending moment (horizontal or vertical)	Normal force (compression or tension)	Shear force (horizontal or vertical)
Surge		696,3 kN	
Sway		- (long side)	
Heave			705,6 kN vertical
Roll	13484,0 kNm		404,5 kN vertical
Pitch	105754,7 kNm	9614,1 kN	
Yaw	63552,1 kNm		423,7 kN horizontal
Mooring forces		- (long side)	
Gantry Crane	5120 kNm	465,5 kN	
Total		10775,9 kN	1110,1 kN

Resulting force: Nr = 10832,9 kN on top stud.

3.3 Design of connector

The concrete stud is designed with similar starting points as the pontoon *[chapter 1].*
The governing internal force is the resulting normal force from the last paragraph. This normal force can result in either compressive stresses in the stud or tensile stresses.
Allowed tension stress at all times $\sigma_b < ½ * f_b = 1{,}08$ N/mm^2
Allowed compressive stress in pontoon at tensioning $\sigma'_{bi} < 0{,}75*f'b = 0{,}75*39 = 29{,}3$ N/mm^2

A bending moment due to the dead weight of the stud is taken into account. The stud is therefore considered as a cantilever beam. The small bending moment is compensated completely by the prestressing force.

Fatigue should be taken into account in the design, but due to the limited available time during this thesis, fatigue has not been considered. Also several checks of the calculations have not been performed during this stage of the design, such as crack width control.

3.3.1 Dimensions of connector

The trapezoidal stud should give sufficient space to curved tendons around the hole for the steel tube with an outer diameter of 1 m. The width between the studs should be larger than the studs, because of the vertical coupling motion. The minimum radius of curved tendons is 1250 mm. The radius is designed at 1500 mm. Smooth steel sleeves will be applied in the prestressing ducts.

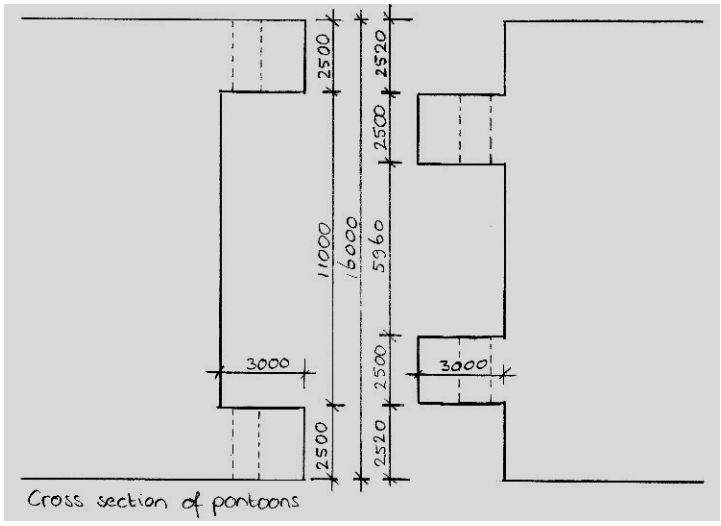


Figure 3.5: Cross section of pontoons with connectors

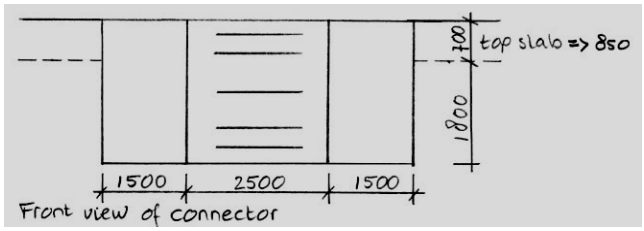


Figure 3.6: Front view with dimensions of the connector

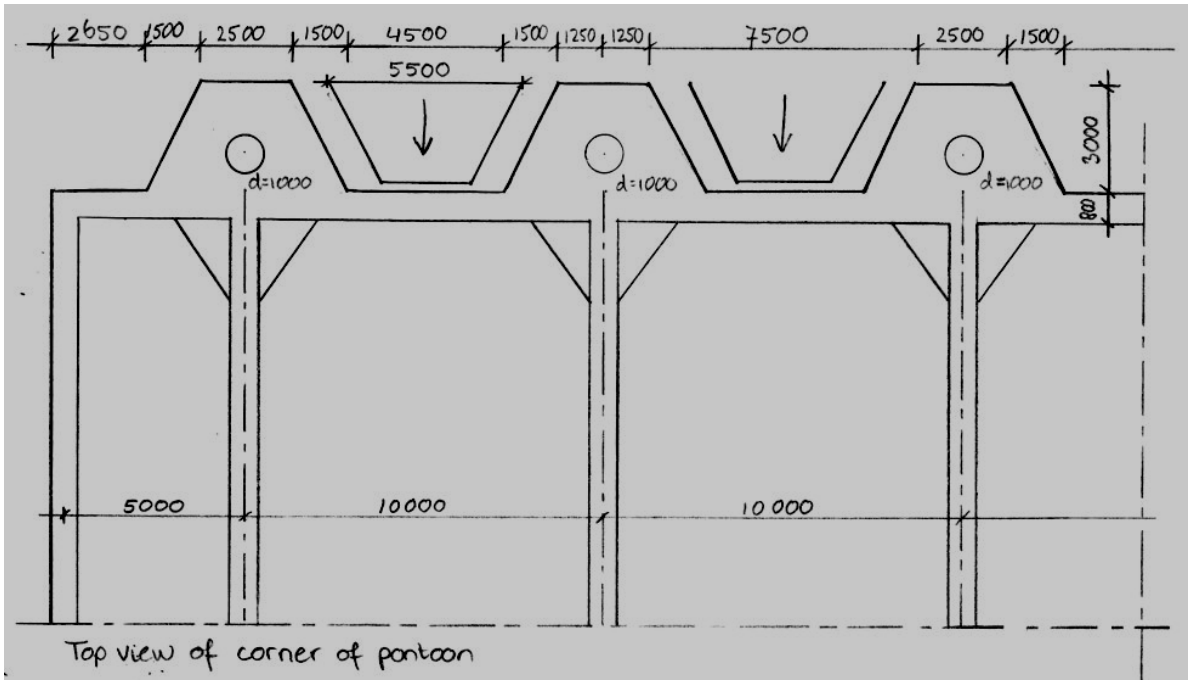


Figure 3.7: Top view of pontoon with connectors

The large studs provide sufficient space for the steel tube and the curved prestressing the horizontal plane. The height of the stud provides space for a maximum of 6 curved tendons. The large concrete volume within the curved tendons and around the tendons prevents high concrete stresses and provides space for splitting reinforcement.

3.3.2 Prestressing of connector

The best solution for the geometry of the prestressing in the connector is a curve. Straight tendons would have less effect in the area around the tube, where the load is initially introduced into the connector. Moreover, straight tendons would have to be anchored at the front of the connector. The sensitive anchors would be severely damaged during the coupling procedure of the pontoons.

The tendon should be longer than several metres, because of the introduction length of the prestressing force. Therefore, the curved tendons will not be anchored directly behind the studs. The tendons proceed trough the inner wall and they are anchored behind the inner wall that separates the outer compartment and an inner compartment. The length is ca. 19 m (short side connector) or 9 m (long side connector).

The prestressing in the connector causes a compressive force in the concrete stud and in the wall. In addition, an inwardly directed load q_p is caused by the curvature of the tendons. This load only causes a compressive force, not a bending moment in the stud.
The small radius in the tendons of 1500 mm causes large losses. The prestressing force during operation is therefore estimated at $F_{pw} = 0{,}5 * F_{pi}$.
However, within the curve of the tendon, the compressive stress is very large. The maximum allowable compressive stress of the concrete is assumed to be

$\sigma'_{bw} < 0{,}5*f'b = 0{,}5*39 = 19$ N/mm^2
The prestressing tendons are dimensioned to compensate the resulting normal force.

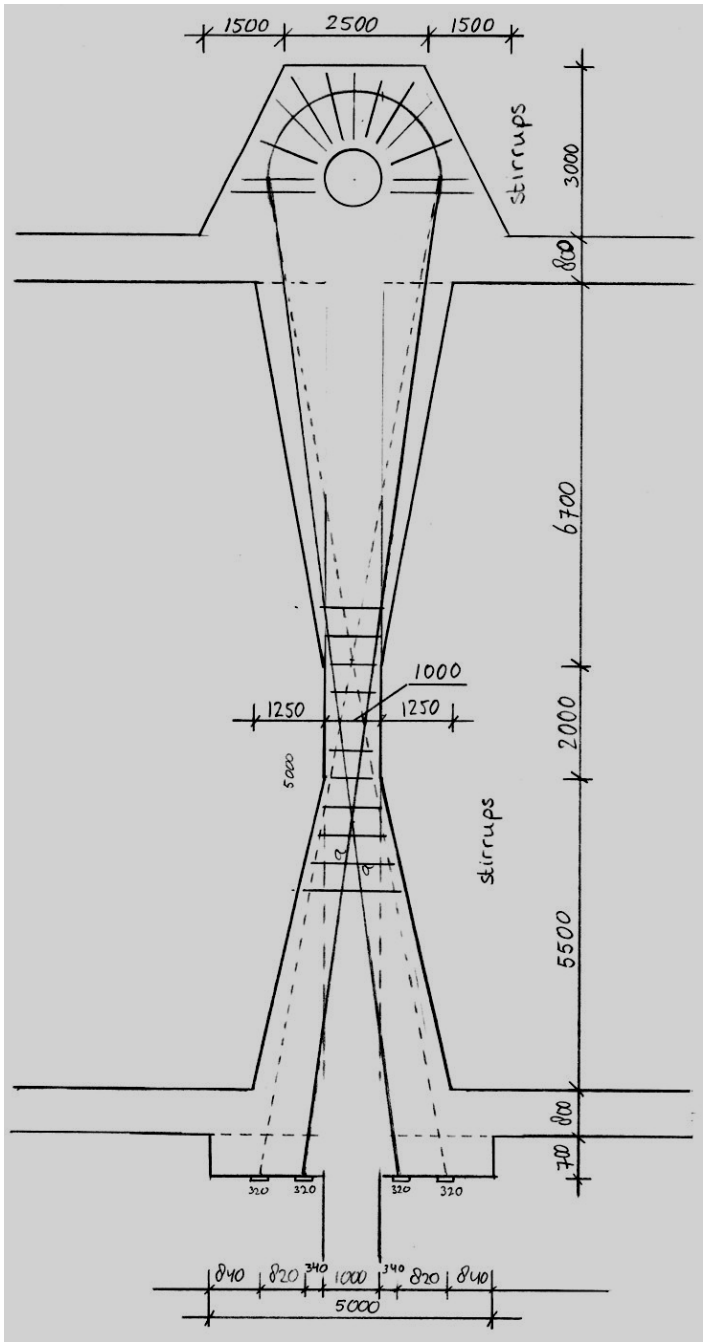


Figure 3.8: Top view of connector and outer compartment, including prestressing and anchorage

The tendons have to cross each other. Otherwise, sharp bends in the tendon profile would cause large outward forces. These forces would be too high for the inner wall. The tendons will be tensioned from both ends to reduce the losses of prestressing force.
6 tendons will be applied with 30 strands ∅ 15,7 FeP1860.
Per tendon: Ap = 30 * 150 = 4500 mm^2 and Fpi = 5850 kN
Fpw = 2925 kN per tendon.

qp = Fp/R with R = 1500 mm.
The fact that the profile of the tendons is not parallel to the wall is taken into account in the calculations. The initial prestressing force Fpi will be larger (resulting force) due to the angle between the axis of the wall and the tendons.
However, as a result of the large haunches of the inner wall, the angle has a maximum of 8°. Therefore, the effect is rather small. The prestressing in the wall is discussed in paragraph 3.5.

Two governing situations are checked in the connector:

1. The maximum compressive stress within the curve of the tendon, at tensioning (Fpi) and a compressive force from the connection (Nr).
2. The maximum tensile stress at the outside of the stud during operation (Fpw) and a tensile force from the connection (Nr).

Both situations meet the requirements regarding maximum stresses.
The calculations are presented (with sketches) in *[Appendix 4-B]*.

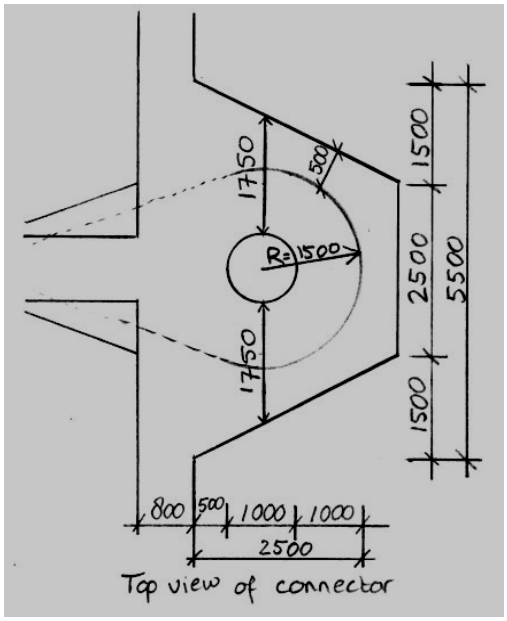


Figure 3.9: Top view of the connector

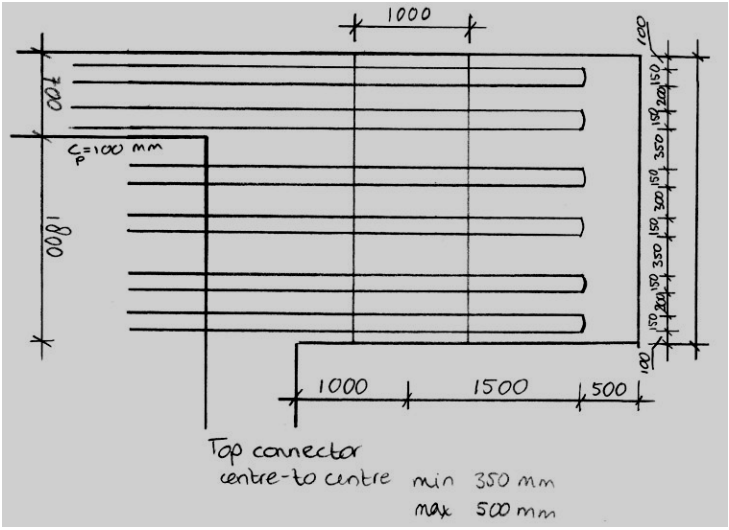


Figure 3.10: Distribution of curved tendons in top connector

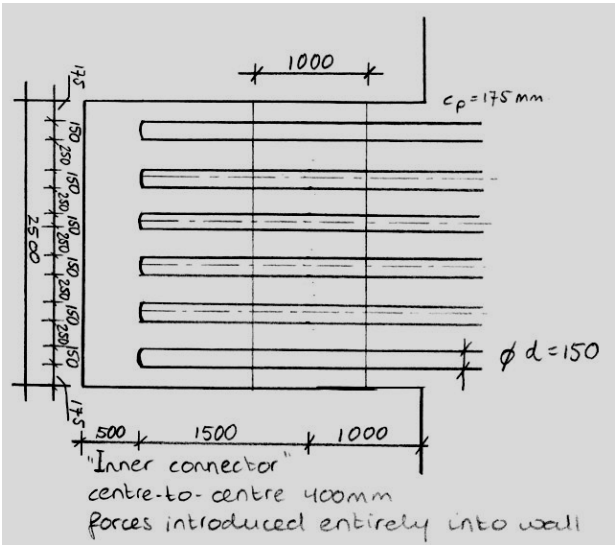


Figure 3.11: Distribution of curved tendons in inner connector

3.3.3 Design of steel tube

The steel tube introduces the connection forces into the stud through contact stresses at the perimeter of the tube. The calculations in *[Appendix 4-B]* show that a tubular pile with D = 1000 mm and t = 100 mm will suffice.

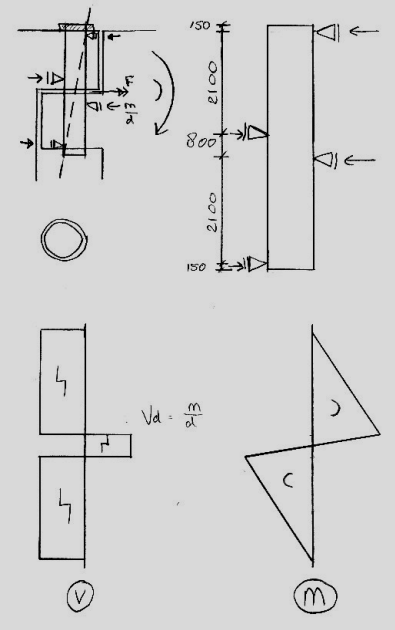


Figure 3.12: calculation of steel tube

The tubular pile has the shape of a large nail and will be lowered into the connection from the top. The margins in the holes in the stud can be several more than 1 cm. Therefore, the connection is not rigid once the tube is installed.
There are 2 ways to stiffen the connection:

1. Grouting of the space between the tube and the studs, which will ensure a very good distribution of the stresses.
2. Jacks will be placed at the sides of the pontoon, where the other stud will touch the pontoon. These jacks push the stud outward. Consequently, the connection is tensioned. The same could be done with rubber profiles in which air can be pumped, but it is assumed that these profiles will not be able to completely stiffen the connection.

The compression force of the jacks will be introduced into the pontoon. This has not been calculated. Because of the limiting tome during the thesis, the jacks are not designed. However, The jacks will cause a compressive force similar to the connection force in the stud. In the next paragraph is shown that this force can be introduced into the inner wall without problems. Only the height of the studs is needed for the distribution of the forces. Therefore it can be expected that the jack forces can also be introduced into the pontoon.

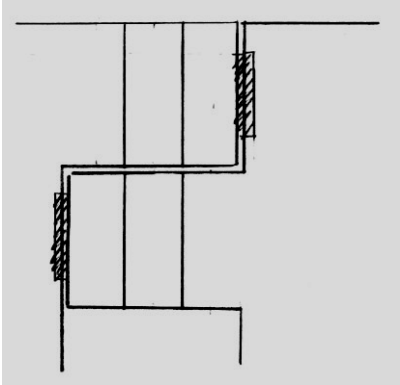


Figure 3.13: Jacks to tension the connection

3.4 Introduction of forces into pontoon

The design of the connector and the pontoon are combined. The inner walls and the top and bottom slab are all prestressed in two directions because of the internal forces due to the live load on the pontoon. The curved tendons are added to the slabs and walls to introduce the connection forces into the pontoon.

3.4.1 Introduction of connection forces

The connection forces are introduced into the pontoon via the concrete studs, the reinforcement and the prestressing elements. The method of stress fields is used to show the distribution of stresses into the pontoon. Peak stresses are prevented as much as possible. The top and bottom studs of one pontoon directly introduce part of their load into the top and bottom slab of the pontoon respectively. The rest of the load has to be introduced into the inner walls of the pontoon. The studs of the other type of pontoon, with only two “inner connectors” transfer the entire connection load to the inner wall behind the connector. This governing situation has been checked in the calculations. These inner walls have a larger thickness in the outer compartments of the pontoon. Large haunches at the crossing of the walls ensure a reasonable profile of the curved tendons, which are anchored at the first inner wall.
The calculation in *[Appendix 4-B]* shows that the combination of several prestressing forces, the connection force and the internal forces due to the live load of the pontoon result in a maximum concrete compressive stress and a maximum concrete tensile stress. The stresses meet the requirements, as mentioned in paragraph 3.3.
Behind this first inner wall, a cantilever beam gives space for the anchorage. The haunches can not be taken into account in the calculation of the prestressing in the walls, because they counteract each other.

The horizontal and vertical straight tendons in the inner walls of the outer compartments have a large centre-to-centre distance of 1000 mm. Consequently, there is more space available in the wall section for the crossing of the curved tendons from the connectors. The dimensions of the walls and the reinforcement and prestressing have been checked according to the guidelines of NEN 6720. The design of the connectors and walls meets the requirements. Furthermore, during the design of the prestressing, the anchorage of the tendons has been taken into account. There is sufficient space for the anchors.

The next figure shows the stress field within the connector. The tube introduces a connection forces into the stud. The concrete stress at the perimeter of the tube are distributed under 45° and then further transferred by the prestressing. The stresses are distributed horizontally in the top and bottom slab and also vertically in the wall. The wall thickness ensures a good stress distribution.

From the calculations can be concluded, that the best distribution of the tendons would be to introduce two tendons of the top stud into the top slab of the pontoon. However, a thickness of 700 mm is not sufficient. The thickness of the top slab should be increased to 850 mm.

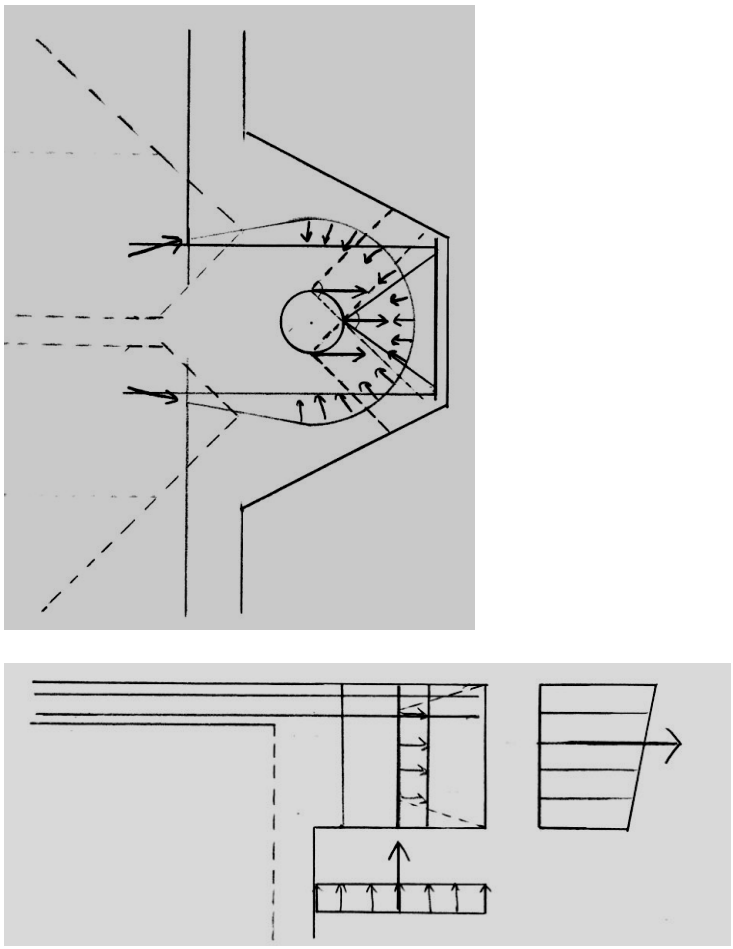


Figure 3.14: Force introduction in connector

The splitting reinforcement in the stud is determined in the appendices. A reinforcement of ∅ 20 – 100 mm will be applied in the curve.

3.4.2 Introduction of prestressing forces

The prestressing force is introduced near to the anchorage of the tendons. At that location, a large concrete section provides a good distribution of the stresses. The anchors are placed at a cantilever beam behind an inner wall. The anchors will be placed at the cantilever beam in vertical rows of three anchors. Thus, sufficient space is made available.

The inner wall has a large thickness. The haunches are very long. Splitting reinforcement will be applied at the smallest section of the wall. The splitting force is determined based on figures 3.15 and 3.16. The reinforcement will be ∅ 20 – 100 mm over a distance of 4 m.

The inner wall itself is prestressed in two directions. The distance between the centres of the tendons is 1000 mm. As a result, there is sufficient space for the crossing tendons.

	thickness	Longitudinal prestressing	Transversal or vertical prestressing	reinforcement	Max prestres-sing level	check
Top slab (incl. haunches)	700 mm	14∅ 15,7 – 1000 mm = 2100 mm^2/m	14∅ 15,7 – 1000 mm = 2100 mm^2/m	∅ 16-100 = 2011 mm^2 2 directions, top & bottom	4 N/mm^2	Mu > Md $\sigma_b < 0,5*f_b$ = 1,08 N/mm^2
Inner walls of outer compart-ment	1000 mm	20∅ 15,7 – 1000 mm = 3000 mm/m	28∅ 15,7 – 1000 mm = 4200 mm/m	∅ 16-100 = 2011 mm^2 2 directions, 2 sides	5,5 N/mm^2	Basis for further design

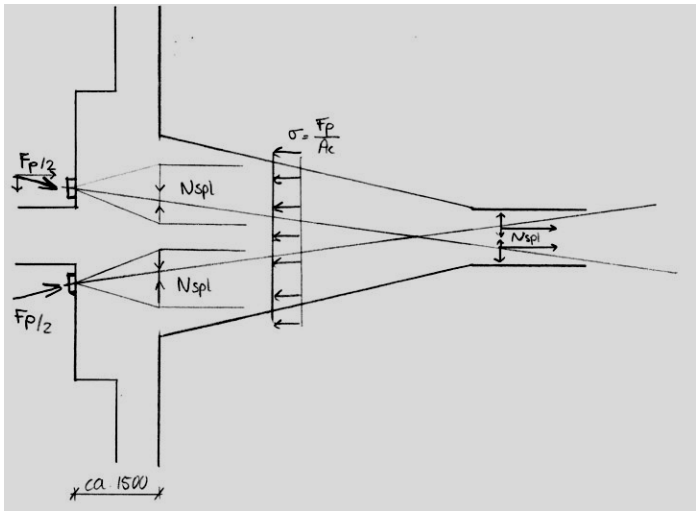


Figure 3.15: Introduction of prestressing forces at the anchorage, top view

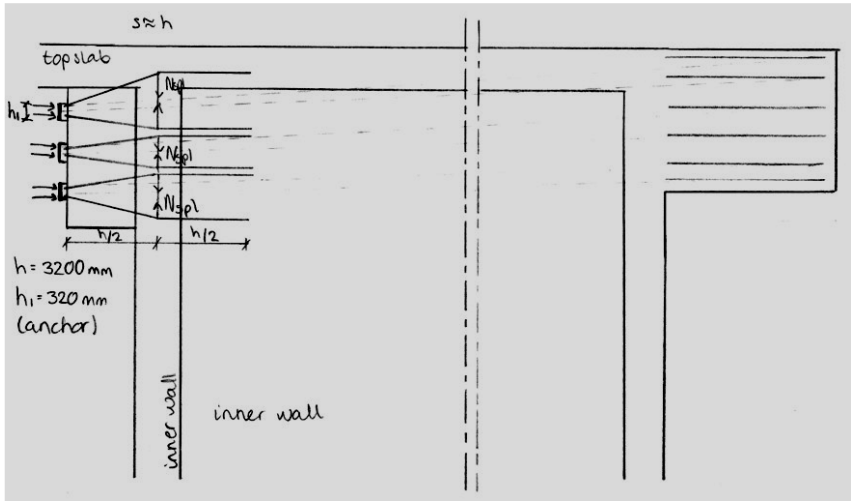


Figure 3.16: Introduction of prestressing forces at anchorage, side view

3.4.3 Distribution of connection forces in pontoon

The connection forces are introduced into the walls of the pontoon by prestressing forces. The prestressing forces have to be distributed through the pontoon beyond the prestressing anchor. The further distribution, mainly via diagonals in the walls towards the top slab and the bottom slab, is attained by other prestressing elements and reinforcement. Stress fields show a maximum distribution of (compressive) stresses under 45º.

The main structural requirement regarding the distribution of forces within the pontoon is internal equilibrium of the forces, or shortly:
$\Sigma H = 0$
$\Sigma V = 0$
$\Sigma M = 0$
The equilibrium of both horizontal and vertical forces (and bending moments) in the terminal is provided by all connectors of the different pontoons. The introduced connection forces are counteracted by the connection forces from other connectors, which are all rigid in the horizontal and vertical; translations. Eventually, the forces “reach” the tubular mooring piles, which provide the external equilibrium force. The mooring system provides a limited margin for horizontal and vertical translations.

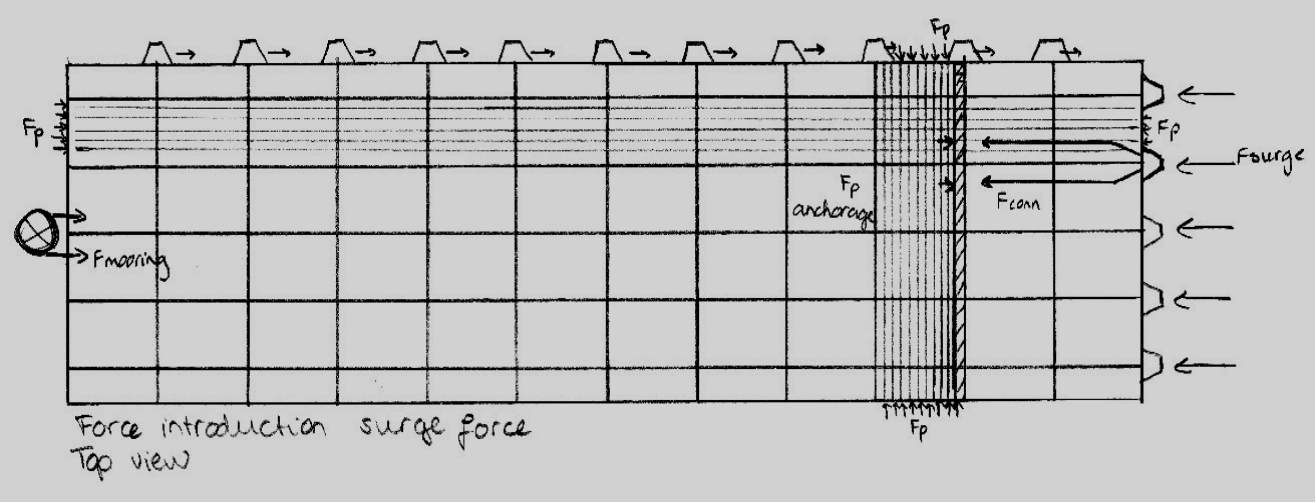


Figure 3.157: Force introduction and equilibrium of forces in pontoon, top view

The prestressing forces have to be distributed over the entire pontoon. In general, the tendons cause compression stresses in the walls and slabs of the pontoon, which distribute under 45°. The same stresses are found in the opposite direction, caused by the prestressing or connection forces from the connector or mooring system at the other side of the pontoon. Finally, the equilibrium force is provided by the adjoining pontoon or the tubular mooring pile.

Behind the prestressing anchor, the stresses distribute vertically towards the top and bottom slab of the pontoon and through the inner walls. In the top and bottom slab, the stresses distribute horizontally in both longitudinal and transversal direction.

The tendons are placed in the longitudinal and the transversal direction of the pontoon. Therefore the compression in the transversal direction will support a better distribution in

the other direction and will reduce the forces (and vice versa). An example of the top slab was shown in fig. 3.17. The next figure shows the equilibrium of forces in the longitudinal cross section of the pontoon. The forces include both loads and prestressing forces.

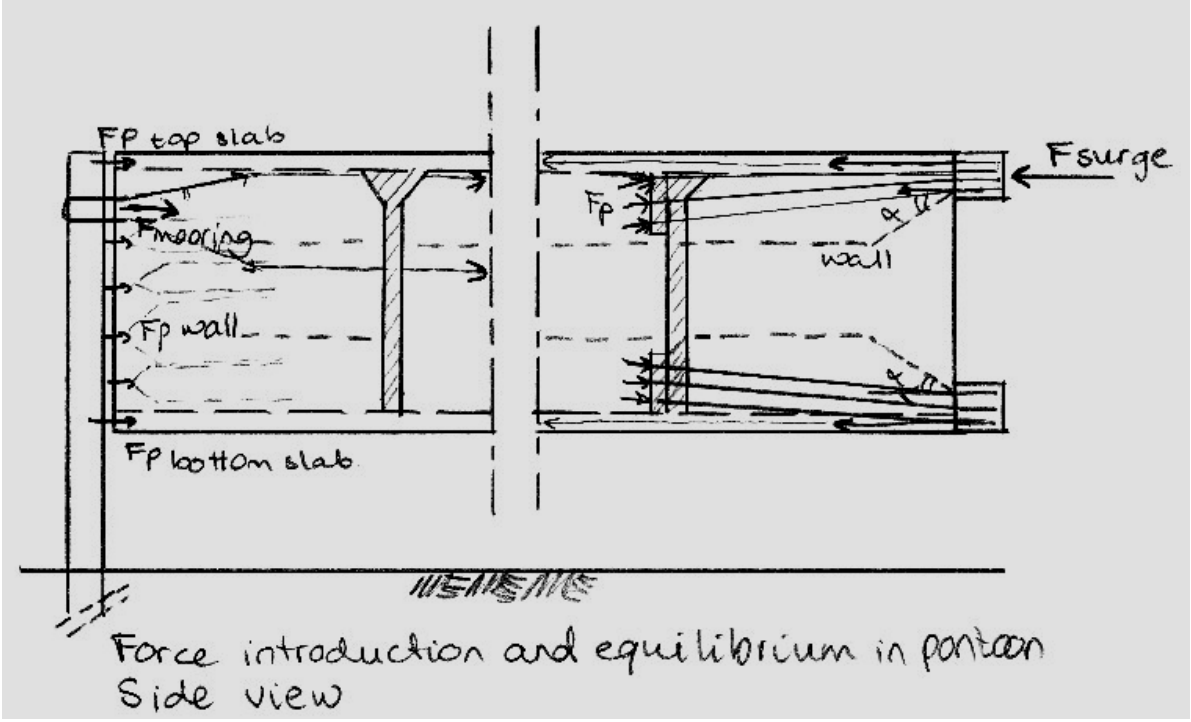


Figure 3.18: Introduction of prestressing forces and force equilibrium in pontoon, side view

The counteracting forces are prestressing forces in several directions and distributed forces from other connectors. These forces are concentrated at the (crossing of) inner walls.

In general, the internal forces in the pontoon and the mooring forces will provide equilibrium with the connection forces from the connectors. This is valid for all pontoons. The next figure shows the equilibrium within the terminal, including the mooring system.

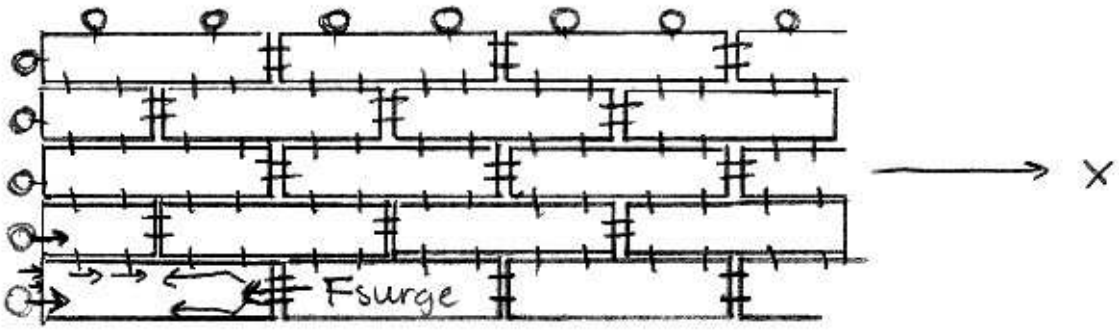


Figure 3.169: Force introduction and equilibrium of forces in terminal, top view

3.5 Construction stage

The type of connector influences the coupling procedure of the pontoons. The steps during coupling will be discussed. Several aspects are listed here:

- Every pontoon will be coupled individually to the terminal.
- The gaps between the inner studs are larger than the studs. In that way a vertical upward motion of the pontoon can almost put the pontoon into place. A short horizontal slide-shove will finish the position. The vertical motion during coupling is controlled by the ballast system of the pontoon.
- Two steel tubes are applied to lock the connection, one for the two top studs and one for the two bottom studs.
- The top vertical steel tube is installed during coupling from the top side of the pontoon.
- The bottom vertical steel tube in installed during coupling from the platform on the inner stud at the bottom. The tube is smaller than the distance between the inner studs, which is 6 m. Therefore it will fit between the inner studs and it will be lowered into the bottom studs.
- The steel tubes both have the shape of a large nail and they have a length of 5250 mm. They will be supported by the upper stud of the two studs.

The coupling procedure is as follows:

1. The first pontoon is in place at the terminal.
2. The second pontoon is ballasted low in the water
3. The second pontoon is towed close to the first pontoon and kept in place by the towing boats.
4. The second pontoon is partly de-ballasted and rises vertically upwards.
5. A temporary structure (with winches) helps keeping the second pontoon in place. The design of such a temporary structure is beyond the scope of this thesis.
6. The second pontoon is shoved (towed) to the side of the terminal. The connectors slide into place.
7. From the top and the middle the two steel tubes are inserted into the holes.
8. Jacks on the side of the pontoons are used to put pressure on the tubes. The connection is made rigid.

To disassemble the terminal, the coupling procedure has to be followed backwards:

1. Every pontoon is disassembled individually.
2. The pontoons are still fastened at two sides when they are to be uncoupled.
3. The uncoupling procedure deals with successive pontoons in the longitudinal direction of the terminal.
4. The tube will be pulled out of the connection from the top side of the pontoon.
5. A short horizontal shove will release the concrete studs from the connection
6. The ballast system will be used to let the pontoon drop far enough to remove the pontoon from the terminal without damage.

4 Construction method

4.1 Introduction

The construction and coupling of 120 pontoons requires a large effort. A commercially feasible project can be characterised by an optimised quality of the structure, made with minimal costs within a minimal construction time. Both costs and construction time can be reduced by applying a standardised work method for all the stages of construction. The size and number of the pontoons that have to be constructed gives sufficient reason to design a specific formwork or factory for this project. In such a large project, the initial extra costs may be recovered by saving personnel and decreasing building time.

The construction stage gives rise to many boundary conditions to the (geometrical) design of the pontoon and the connectors. The construction of the terminal can be divided in 4 stages:

1. Concrete construction work in construction dock
2. Transport of the pontoons
3. Completion while floating
4. Coupling of the pontoons

During the structure's lifetime, there will be a fifth construction stage, when the terminal is disassembled and transported to a new location. At the end of its lifetime, the terminal will be disassembled and demolished.
Every construction stage provides its own conditions regarding the shape and construction method of the pontoon and of the connectors. Maintenance and repair during the operational phase is not taken into account.

4.2 Construction of the pontoons

The design alternatives regarding the pontoon and the connectors are restricted by the construction method of the pontoon. Most important is the use of prefabricated concrete parts. The application of prefabricated structural members will decrease building time. Prefabricated parts are also expensive.
Construction alternatives have been considered for the bottom slab of the pontoon, the outer walls, the inner walls and the top slab of the pontoon. These are outlined in Part 4 of the Appendices.
Criteria in the selection of concrete construction method are:

- Sufficient concrete quality
- Limited construction time
- Optimised costs from type of concrete and type of construction method, building time and amount of concrete
- Minimal amount of lost formwork to reduce costs
- A high degree of repetition to increase a learning effect and reduce labour and construction time
- Sufficient space for the planned activities at all times

The pontoons will be built in a construction dock, as has been argued in part 1 of this thesis.

The bottom slab and outer walls of the pontoon will be casted in-situ. This will produce a closed outer shell of the pontoon with a minimum of construction joints. The pontoon will be sufficiently watertight. There is sufficient working space (for the formwork etc) available in the dock and the horizontal surface is not complicated to construct.
The restricted space within the compartments of the pontoon causes a need for a special framework to construct the inner walls. Several types of formworks are outlined in the appendices, which may provide a solution for the walls of the pontoon.

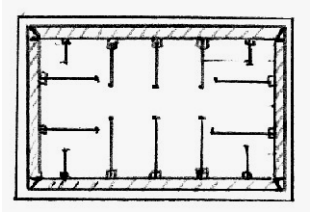


Figure 4.1: Wall system formworks with struts take up too much space in a compartment

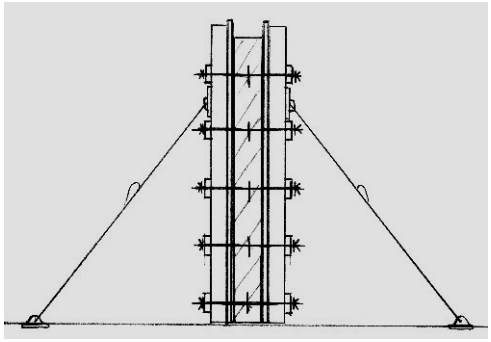


Figure 4.2: A wall formwork with struts

The walls will be constructed using a rectangular compartment formwork, which can be hoisted to each next compartment. Twelve of these formworks will be made as there are six compartments in the transversal direction of the pontoon. The construction cycle will proceed in the direction of the longitudinal axis of the pontoon. The outer side of the pontoon will be shuttered with a system of high wall frameworks.

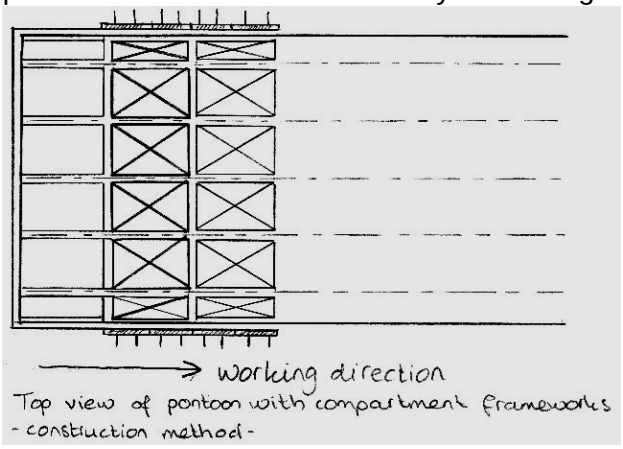


Figure 4.3: Special compartment frameworks and working direction

The top slab of the pontoon can be casted in-situ or it can be (partly) prefabricated.

The alternatives for the top slab are outlined in Part 4 of the appendices.
Prefabricated slabs, whether totally or only half prefabricated offer the best solution.
All prefabricated parts are expensive and too large to transport by road. The top slabs will be prefabricated at site, in a defined area of the building dock. This will be less expensive and the deck plates can be transported and installed by the heavy equipment (cranes) in the building dock during all of the construction time. A disadvantage is that the feasible concrete quality on site will be lower than in a factory. The necessary concrete quality is B65, which will be feasible to produce on site.

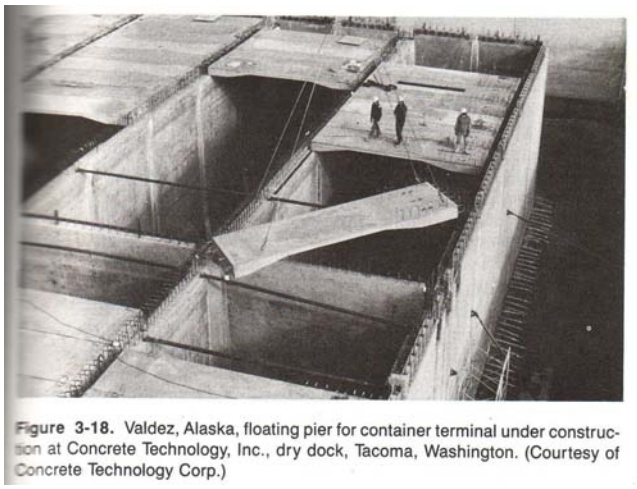


Figure 3-18. Valdez, Alaska, floating pier for container terminal under construction at Concrete Technology, Inc., dry dock, Tacoma, Washington. (Courtesy of Concrete Technology Corp.)

Figure 4.4: Example of prefabricated top slabs on a pontoon

It is assumed *[Ref. Part 1]* that a construction dock will be created at the 2nd Maasvlakte at Rotterdam. This dock will have dimensions of ca. 390 * 190 m and a depth of ca. 12 – 15 m (depending on future expected projects). There is space available for 6 pontoons at a time. Therefore, 6 * 12 special compartment frameworks will be made and used at the same time.

4.3 Transport and coupling of the pontoons

The design requirements regarding transport conditions have been discussed in Part 1 of this thesis.
The transport stage consists of the following activities:

1. Floating upwards from foundation bed in construction dock
2. transport out of the construction dock
3. Completion of top slab structure of pontoon (and possibly connectors) while floating
   Finishing of the construction is assumed to be carried out near the construction dock, considering available space in the water. It might however also be done in the Western Scheldt.
4. transport through inland waters (canal, harbour)
5. transport over sea (short distance)

A number of practical considerations regarding the transport stage are taken into account:

- A temporary clamping (clutching) structure is necessary to couple the pontoons. The temporary structure will have to be constructed and fastened on the pontoon before transport.
- A minimum of 4 tugboats and other equipment is necessary to keep pontoon under control.
- During the transport stage at sea, longer waves will occur than the waves to be assumed during the operational stage in the Western Scheldt.
- The pontoons have a bad manoeuvrability in currents and waves.
- A screw pump or jack will be needed to control the pontoon during coupling movements.

The coupling procedure of the elements of the terminal is very important to the design of the connectors. The decoupling procedure is less important, because the terminal will probably only be moved once or twice in its lifetime. The coupling procedure should be possible up to wave conditions with an occurrence of once a year. This is assumed to be decisive for the overall constructability of the floating terminal.

The following circumstances will affect the coupling procedure:

- Continuous heave (pitch, roll) motions caused by waves.
- The pontoon to be coupled is large and difficult to keep under control.
- Horizontal translations can be more easily restricted by tugboats and lines than rotations or vertical translations.

The connections between the pontoons should be watertight. Therefore, a completely external connector is preferred. It is difficult and labour-intensive to close a connection and make it watertight after coupling.

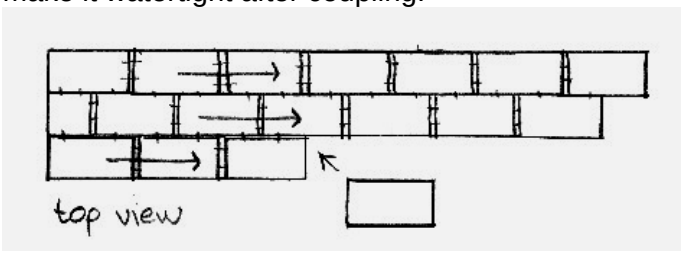


Figure 4.5: Top view of coupling procedure of an individual pontoon

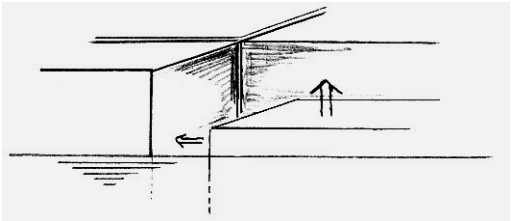


Figure 4.6: General coupling movement

Practical considerations from coupling procedure:

- Large structures need large connector margins (structural tolerances) to be coupled in this situation.
- Draught and freeboard can be influenced and used as a tool during coupling.
- Horizontal manoeuvring is difficult.

- Pontoons can be connected in full rows parallel to the longitudinal x-axis; however, a row of elements should not be floating unset on the Western Scheldt for a long period of time.
- Pontoons can be coupled independently per pontoon (ideal).
- Pontoons can be coupled in twos and then connected to the terminal.
- Protruding connectors prevent connection per separate pontoon, if the pontoon is to be connected at more than 1 side, unless the ballast system is used to create an upward vertical coupling movement.
- Fragile protruding connectors may easily be damaged during the joining operation.
- A small number of connectors might be constructed from the top side after coupling.
- Many connector concepts may need a temporary aligning or clasping/clutching structure on the two pontoons to be coupled (in a row). This will be more difficult if the independent pontoon will be installed in a corner, connected at two pontoons at its long and its short side.
- On the other hand, partly internal connectors may be repaired from the inside of the pontoon. Coupling from the inside may be simpler as well, but then it will be difficult to ensure a watertight connection.

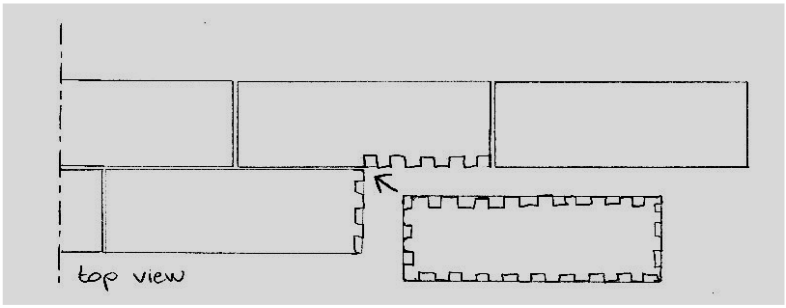


Figure 4.7: Top view of coupling procedure with protruding connectors

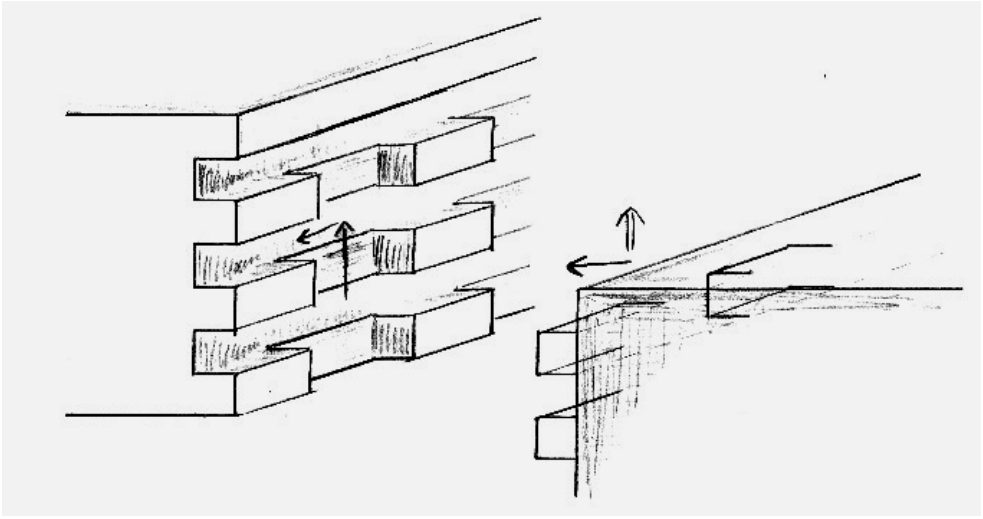


Figure 4.8: Horizontal and vertical coupling movement in case of protruding connectors

4.4 Disassembling and moving the terminal

The terminal may be disassembled after a part of its lifetime, to be moved to a new location.
This disassembling and reassembling of the terminal may happen once, twice or even tree times during its lifetime; or it may not happen at all. There is no complete certainty about this.

A very important boundary condition for the design of the connectors is a simple coupling operation that will not cause much damage to the connectors.
The disassembling operation is of less importance to the design. Disassembling should however be made possible with a minimum amount of activities and minor damage (and repair) to the pontoons. Connectors may be partly replaced afterwards.

Practical considerations regarding disassembling of the terminal:

- The terminal should be disassembled row by row. The rows of pontoons are decoupled into separate pontoons afterwards. This method is very practical if the entire terminal is being moved.
- It may be advantageous , if it is possible, to start decoupling from the inside of the pontoon
- Pontoons might be decoupled independently (ideal), so that the front row of quay elements may be decoupled for maintenance or repair (due to e.g. a collision).
- The layout of the elements of the terminal prevents decoupling of the intermediate pontoons (behind the front row) for maintenance.

Evaluation of the concept of the floating terminal

The general concept of a floating terminal can be applied in different situations. However, a terminal will always be designed according to local conditions. The main aspects that control a design are:

- The water depth of the location, which influences the mooring system of the terminal
- The tidal window of the location, which influences both the mooting system and the mainland connection of the terminal.
- The type and size of the loads on the structure, which influence the geometry and the structural dimensions of the terminal.
- The local wave-spectrum and the wave height, which influence the selection of the type of floating element (because of hydrodynamic behaviour) and the dimensions of the structure.

The concept that has been developed in this thesis is suitable for reasonably shallow waters. The main characteristics of the concept in this thesis are:

- The terminal is a rather permanent and very large structure that will only be disassembled once or twice during its lifetime.
- A simple floating element with a large waterplane area, made of concrete.
- A floating element with a rather small structural height, but with a large length and width
- A floating element of which the geometry and dimensions can be optimised to meet other requirements.
- A terminal consisting of rigidly connected pontoons, without a separate deck structure
- A mooring system with tubular piles that is suitable for shallow waters. The alternative mooring system with tubular suction piles can be applied in deeper waters.
- A rigid connection between pontoons that can be de-coupled.

This structure is not suitable to be moved often. A movable base or pier should be easier to couple and decouple and should be moved fast. These were the main criteria if the Mobile Offshore Base.

The concept of the floating Western Scheldt Container terminal can be applied elsewhere in the world. The location would have to be in somewhat protected or shallow waters. The main reason for this is the large waterplane area and small structural height of the pontoons. The selected floating elements will not be applied offshore at the open ocean, because the pontoons will experience very large wave loads. The pontoon height can be increased as can the wall thickness, but the hydrodynamic behaviour will not be sufficient. At open seas, a semi-submersible will be better equipped to survive the wave loads. Moreover, the large possible wave-height of ca. 15-20 m on the Atlantic Ocean will be too high for a pontoon structure.

The developed terminal may be adapted too withstand the wave loads of the North Sea. The mooring system can be designed for the North Sea as well. Other small seas or bays may provide a good location for a floating terminal consisting of concrete pontoons. A good

example of this is the coast of Japan. The loads are comparable with those in the Western Scheldt.

The developed terminal could be constructed in a deep-water location, if a good solution is found for the mooring system. That is beyond the scope of this thesis.
For example, a fjord is a deep-water location, protected from wind and waves. The pontoons could be applied in this case, because their design would only have to be adapted slightly to the different loads. However, the mooring system may cause large loads on the terminal. That should be researched.

Summarised, the terminal can not be applied at a location with a severe wave and wind climate. Not only would the wave loads be too large for this design, but also the hydrodynamic behaviour of the terminal would probably be insufficient.
The terminal as designed in this thesis can on the other hand be applied in small seas or bays or estuaries. Many of the future locations of near-shore ports for Ultra Large Ships belong in this category. Whether the concept of the terminal can be applied in a deep-water location such as a fjord depends on the possibilities regarding the mooring system.

Conclusion

A floating container terminal in the Western Scheldt near Vlissingen is technically feasible. It can provide a solution for the transhipment of containers from Ultra Large Container Ships. It also has sufficient storage capacity and cargo can be transported over land by the connected infrastructure (railway and road).

The terminal has dimensions 2160 * 500 * 16 m and it consists of 120 concrete pontoons with dimensions 180 * 50 * 16 m, which are rigidly connected to each other. The terminal has been designed as a rigid single-body terminal for the location in the Western Scheldt, bordering on the eastern harbour of Vlissingen. It will have a lifespan of 100 years and it may be moved to a deeper location after a score of years and reassembled near the mouth of the Western Scheldt, a near-shore location. The platform can also be adapted in size to larger future ships.

The pontoons are connected in a stretching bond configuration. The terminal has a freeboard of ca. 2,4 m. The independent pontoons are divided in rectangular compartments of 15 * 10 and 15 * 5 m. The hydrodynamic behaviour of the single pontoons during transport and of the rigid terminal during operation is good, mainly because of the large dimensions and dead weight of the structures. The response motions of the terminal due to the governing diagonal wave attack in the Western Scheldt are small. The amplitudes have a maximum of several centimetres.

The pontoons will be casted in-situ in a construction dock. The top slabs will be prefabricated, installed and post-tensioned at the same time as the rest of the pontoon. The top slabs will need a thickness of 850 mm instead of the earlier calculated 700 mm in order to introduce the connection forces into the pontoon. The inner walls require a similar wall thickness as the outer walls of 800 mm. The bottom slab is 850 mm thick.

The wave loads on the pontoons are large. Consequently the connection forces are large. The pontoons are rigidly connected by a concrete stud connection. The connection consists of two large prestressed concrete studs (ca. 3,0 * 2,5 m) at the top and bottom of the sides of both connected pontoons, which are locked by a steel tubular pile with a diameter of 1000 mm and a thickness of 10 cm. The tube is kept in place by jacks, because grouting would be a permanent connection.

The forces are introduced into the top and bottom slab and the inner walls of the pontoon through the stud shaped connectors. The walls in the outer compartments require a thickness of 1000 mm to introduce the connection forces and provide space for the prestressing steel. An optimisation of the calculations would lead to smaller compartments and a larger wall thickness of the pontoons. The larger dead weight and bending stiffness of the pontoon are favourable for the hydrodynamic behaviour.



Recommendations

The focus of this graduation project has been the development of an overall conceptual design of a floating terminal and its elements, especially the connections between the pontoons. Due to the wide scope of this thesis, several parts of the research are not much elaborated. When the project should actually come to be further developed, the following research should be elaborated:

- The output of the single-body terminal should in time be compared with the multi-body terminal results. The multi-body model can be expected to produce larger motions and different wave elevations due to different reflection and diffraction. The wave forces per element will be smaller than on the whole structure. The added mass and damping are also determined per element and will differ from the rigid terminal values.
- The alternative concept of flexibly connected pontoons with dimensions equalling a design ship (ca. 420 * 50 * 16 m) should be researched.
- The entrance to the terminal from the shore (e.g. a bridge) should be further developed and the loads should be taken into account in the design of the terminal.
- The drift forces should be determined more carefully in order to design a good mooring system.
- The present rigid single-body terminal should be modelled again with the actual larger dead weight and bending stiffness of the terminal. The motions response motions will be smaller and consequently the connection forces will decrease.

This thesis can be used as a basis for further research into floating port structures. The concept of the pontoon and the designed connector is considered feasible and practical. Therefore the conceptual design in this thesis should be optimised and detailed. The following aspects should be taken into account in a more detailed design of the pontoons and the connections:

- The dimensions of the compartments of the pontoon should be changed to 10 * 5 m in order to decrease internal forces and to facilitate the introduction of connection forces.
- The thickness of the present top slab should be 850 mm instead of 700 mm to introduce connection forces.
- The concrete connectors can be prefabricated and afterwards tensioned to the pontoons. Construction time would be shorter and the concrete quality may be higher.
- The conceptual design calculations should be elaborated with a crack width check, a fatigue calculation and a calculation of the jacks (force introduction into the pontoon).
- Furthermore creep, shrinkage and temperature loads on the pontoon should be checked. A calculation of torsion of the floating pontoon should be made.
- A detailed design should be made on the basis of a three-dimensional Finite Element model of the pontoon. The internal forces can be determined more accurately. All load combinations should be checked.
- The reinforcement and prestressing should be determined accurately for a detailed design, taking into account the losses due to shrinkage, creep and relaxation.

Finally, in order to finish the thesis in time, a number of assumptions have been made. These should be discussed for a more detailed design:

- Straight and centrally located tendons in the structural parts of the pontoon
- Prestressing without or without bond.



- Complete or partial prestressing.
- In the conceptual design, a stress σpi = 1300 N/mm^2 is assumed.
- The losses of prestressing force due to creep, shrinkage and relaxation have not been calculated. The work prestressing force has been estimated at 0,85*Fpi for straight tendons and at 0,5*Fpi for the strongly curved tendons.

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A Very Large Floating Container Terminal -

feasibility study

Application possibilities for the situation in the Netherlands

Literature study

Master Thesis G.V.P. de Rooij

TU Delft

Van Hattum en Blankevoort

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Introduction

This literature survey serves as a preliminary study for the Master Thesis titled
“A Very Large Floating Container Terminal”.
The literature survey is a report of a limited research into the most important subjects concerning the design and construction of floating structures. An inventory has been made about what has been constructed and, perhaps even more important, what has not been created or researched. The preliminary study has been limited in time to a period of a month, to assure not too much time will be lost in extensive research. As the project is fairly new, many aspects were dealt with in time with creativity and my own insight in the matter.
The plan of this report is as following:
The information acquired is described in several subjects, all briefly summarized. The chapters mainly follow the structure of the final report of this thesis. In the second part an evaluation from every peace of information is given of its usefulness and background and about the similarities with the subject(s) of the thesis. Which publications were used in further research into the subject, are stated in an overview.
As a conclusion of the study, a work plan and a time planning was made.

1. Applications of floating terminals

Ref1; Port Engineering: p.424
From experience it is evident that for locations where waterways are obviously unstable, possess a wide range of water levels, and lack good morphological and hydrological data, a floating wharf arrangement is an ideal approach to port design..
Although floating docks have long been used in port construction, waterfront structures traditionally are in built in the form of bottom fixed structures..
Port development has shown that floating terminals can be implemented successfully at inland ports and protected seaports and harbours as well as offshore. In general, for given specific conditions (e.g. deep water, strong current, short period suitable for construction, difficult and/or uncertain soil conditions, environmental concerns), construction of floating rather than bottom fixed terminals may be advantageous..
Floating docks are also very important in military applications, where speed of construction is the most challenging of engineering tasks.

Despite some obvious advantages of floating ports, in some cases they may have higher operating and maintenance costs than those for fixed facilities. This may especially be true at locations of heavy ice movements and in corrosive environments. Also, in areas exposed to significant wave action, the operating stability of a dock may be reduced, thus increasing its downtime.

The above summarises the main aspects of floating terminals in the last century.
The idea of a floating harbour is not new. Floating docks and piers have often been applied in different situations. In these cases, the surroundings or environmental conditions were such that a floating structure was easier to build or use than a traditional port. For example, a bottom-fixed structure may prove to be expensive in a deepwater location or on very soft soil. Also, land reclamation could be a reason to build a floating structure and retain space ashore.

A more recent development in floating harbour structures is the thought of elongation of their economical lifetime. The steel and concrete harbour structures already seem to have a long technical lifetime, but their economical lifetime could be lengthened by moving the terminal to a new location within or outside of a continuously developing port. These developments are hardly described in older literature.

The principle of an offshore port at deep water for very large ships is essentially based on the Hub and Spokes model. This model describes one transhipment port for large ships and several lines towards smaller ports. The cargo would be transported by feeders. These days, there are many large ports and as such, the Hub and Spokes model seems no longer valid. However, when the draught of the future Ultra Large Container Ships or Ultra large Crude Carriers becomes too large, deep-sea ports may become necessary. Feeders can do offloading to shore, in which case the depth of the port ashore may be less. Precious space ashore will remain available for a coastal city when a smaller area is reserved for port activities. Coasters or shortsea ships may transport the cargo from the large harbour to several other harbours along the continent (according to the hub and spoke model).
The very large crude carriers already load and unload at many floating facilities, but these facilities are limited in their dimensions to one or two ships, as yet. Several examples will be given in the next chapter.

The applications of a floating structure may be found in housing, greenhouses, plants, airports, emergency bases, recreation and accommodation facilities etc. Every function will entail its own specific loads. Some of the structures are rather permanent, but others, especially those in military applications, have a temporary function and may be transported more often. The application of a floating shipping terminal is very site-specific.
A port ashore is mostly well protected against current, wind and waves. It can provide a safe berth and entrance channel to ships. There is room ashore for all quay facilities, storage and infrastructure. A floating terminal will often be less well protected, but it can offer other advantages.

A floating port can be constructed near an existing harbour or in protected waters. Most of the existing floating structures have been constructed at sites with small physical loads from current and waves.
A floating terminal can be the best alternative in several circumstances. It is in fact a large floating artificial island, which saves space ashore. When the location is in deep water, landfill methods will be very expensive and uncertainty about settlements may arise. The physical conditions of a site area are therefore very important for the choice of structure. A steep rocky shore or very soft soil may also be a reason to build in water.

A floating structure will move with the tide and waves as do the berthed ships. In areas with a large tidal range or with ice on the water, this can be an advantage. Floating structures are sensitive to ice loads though.
The floating terminal is comparatively indifferent to earthquakes (and tsunamis when situated offshore) and it will be less destructive to the environment and less noisy than most landfill methods.
A Mega floater can easily be expanded with more elements, which means it can grow with future ship lengths, while activities on the platform may continue.

Much of the information on the applications of floating terminals can be found in [ref. 1 and 2], and more up to date information on several websites, accounted for in the reference list.

2. State-of-the-art on floating structures

In the last 50 years, different types of floating docks have been built in all corners of the world.
The principle of an entire floating port has come into view around the time that large floating platforms do not seem to be part of futuristic plans anymore. The research done into the Japanese “Mega Float” and the American “Mobile Offshore Base” prove that large floating platforms will be technically possible.

Floating structures can naturally have many functions.
Research has been done into floating greenhouses; floating houses have already been built. Many plans have been made for floating airports, in stead of artificial islands. A floating port or terminal would mainly be an enlargement in scale. A larger area in square metres will be needed and heavier loads will be on the platform. This enlargement in scale, together with the requirements considering durability and behaviour while floating, has many consequences for the structure of the platform. It means that another material may be preferred and that the platform has to be built from modular elements, which have to be large and strong at the quay. The connections between these elements are entirely another matter than they used to be.
Many of the projects already constructed are situated on a river or in other protected waters. This means that wave forces and connection forces will be restrained. At sea the structure will often be comprised out of one part only, so that the application of connections is prevented.
The Mega Float in Japan seems to be the first platform offshore built up out of many elements, joined together.

Several types of existing floating structures will now be dealt with, to give a clear overview of the state-of-the-art in this area of work. Many more floating structures have been found during this literature study, but the following seemed to be the most interesting structures for this thesis.
A number of (mostly relatively small) floating structures are described in [ref. 1 and 2] as cases, from which comparable parts may be used in the thesis. The older floating terminals are mainly built in wide rivers or other protected waters, not offshore. The best comparison can be found in the Floating Storage and Offloading vessels or actually floating liquid bulk transshipment terminals. Those structures are made from one large pontoon-shaped structure or semi-submersible however. The most up-to date information about multi-module offshore structures is on the Mega Float in Japan and the Mobile Offshore Base-concept in the United States.

2.1 Mulberry harbours in WWII

Many civil engineering structures have first been invented and applied for military ends. The floating Mulberry harbours are an example of this. The allied forces needed harbours in order to land all the men and supplies that they would need if Operation Overlord (D-Day) was to succeed.
The Mulberry harbours were two floating ports (one English and one American) for the transport of men and supplies towards the front.
The Mobile Offshore Base project in a sense has the same goals as the Mulberry harbours had in World War II; namely, the logistic support of the armed forces overseas.

From www.historic-uk.com:

The harbours, code-named ‘Mulberries’, would consist of 73 individual prefabricated concrete blocks which when assembled would make up the ports, breakwaters and pontoons where ships could tie-up and unload their precious cargoes. Floating ramps would be used as roadways to allow the lorries to be driven directly on to the beaches. The component sections of the harbours would be built in ports throughout the UK and towed across the Channel for final assembly off the Normandy coast.



Figure 3.1a-d: Mulberry Harbour in WWII and today, Arromanches

Parts of the artificial harbours were floating piers anchored to the sea floor by piles (Whale piers and Lobnitz piers, floating pontoons (beetles) as roadways and breakwaters; floating breakwaters (Bombardons), submerged pontoons (Phoenix caissons) as breakwaters and sunken ships (Gooseberries) as breakwaters.
Many of the steel pontoons of the Mulberry harbours were reused after the war, for instance in repairing the dikes in Zeeland, the Netherlands.

2.2 Passenger (cruise) terminals

Cruise ships up to 240 m long can since 2004 sail right into the heart of London. The floating terminal will be used by the Port of London Authority for cruise ships usually moored at Greenwich and at the Tower Bridge.
Ref.1; p.428: Small passenger docks:
A great number of efficient floating docks serving local passenger traffic have been constructed throughout the world in the past two to three decades. Three representative examples of such docks are the floating passenger docks at Kobe - Japan, Bangkok – Thailand and Kiev – Ukraine. Several floating docks are in service in England too.

Floating marinas for recreational shipping are becoming more and more commonplace around the world; different small pontoons systems with decks have been developed and introduced in Germany, Denmark, Norway and mostly the United States.

2.3 Floating terminals in Peru

Ref. 1;p.450
A number of floating docks have been built on the Amazon river and its tributaries. These facilities are part of the Trans Andean Transportation Corridor, which is studying the construction of several inland ports and road network to facilitate the industrial development of the area. Three significant projects are the ports of Iquitos on the Amazon river, Pucallpa on River Ucayali and Yurimaguas on the Huallaga River.

2.4 Floating terminals in Valdez, Alaska

Ref 1: p.441- / ref. 2

Valdez harbour is a natural deepwater fjord located in the north eastern part of Prince William Sound in the Gulf of Alaska. The harbour is protected on all its sides by steeply rising mountains. It remains ice-free throughout the year… Because the harbour is protected from the open sea, waves within the harbour are normally limited to the wind-generated (wind speed >30 km/h from any direction at 3,5% throughout the year) waves restricted by the available fetch.

Alaska Crude Oil Loading Port

The seabed at berth 1 consists of a very steeply sloping rock with little overburden. Because of very deep water at the design location of berth 1, this site was not economically suitable for construction of a bottom-fixed marine structure and, on the other hand, was ideal for construction of a floating dock. The dock was designed to receive tankers of up to 125 000 dwt. It comprises a floating pier and four bottom-fixed mooring dolphins. The pier is free to move vertically to accommodate tidal movements and wave actions, including pitch and roll, but is restricted by a system of booms from movement on the horizontal plane. Restraint is provided by three-dimensional truss-type rigid struts hinged at the berth structure and at the land-based abutments.
The dock consists of buoyancy tanks and a steel tubular space frame.

Floating pier of container terminal

This facility was constructed near the northern bank of Valdez harbour. The project was first envisaged as a bottom-fixed structure, but placement of the fill to form the yard indicated that a fixed structure could not be practical. As the 600 000 m^3 of fill settled to the bottom, the rock slope at the site of the pier slid into deeper water, providing a warning of what might happen during an earthquake. The follow-up studies indicated that a giant floating dock would cost no more than a fixed structure but would have a great many advantages. The floating dock was designed to handle a 40-ton container gantry crane and to accommodate container ships of up to 50 000 gross registered tons. It can handle both container vessels and ro/ro carriers.
The floating dock consists of three principal parts: the floating pier, anchor system and articulated access bridges.

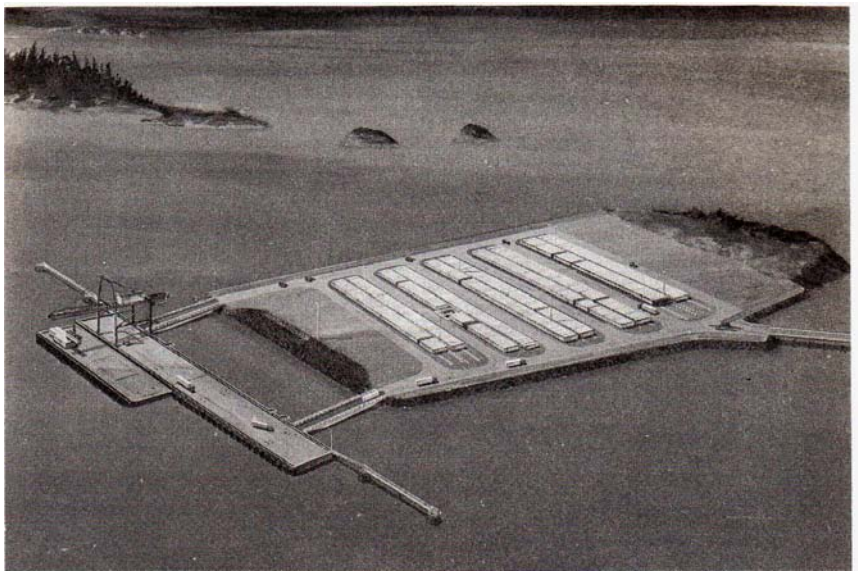


Figure3.2: Floating container terminal at Port Valdez, Alaska [ref.1]

Only the pier has been constructed as a floating structure from one pontoon. There are only hinterland connections.

2.5 Floating (Production) Storage and Offloading vessels (FPSO)

An FPSO is mostly a storage and transhipment facility for liquid bulk cargo; oil or gas.
Several have already been built from a pontoon or ship hull and are in use.

Besides building an FPSO, also an old tanker or crude carrier, extended with a central manifold, can be used as a storage vessel. A production unit can be placed upon the vessel. It will be moored with a single point mooring system.
One of the reasons for constructing FPSO's is that many oil derricks (production platforms) do not have a large storage unit.

2.6 Floating LNG terminal:

[Ref. 18] describes that the structure can be constructed as floating, semi-floating, anchored, gravity based or with landfill.
The purpose of a floating LNG terminal is storage and transshipment of Liquefied Natural Gas.
It is a floating steel or concrete structure, comprised of one pontoon only and moored with a single mooring point system. The safety requirements are very strict on a floating LNG-terminal. Near the coast of California, one has already been built.

2.7 Floating dry bulk terminal or Floating Transfer Station (FTS):

The purpose of a floating LNG terminal is storage and transhipment of dry bulk, such as ore.
An FTS can be positioned near to a mine and costs less than a land based terminal.
This type of floating terminal is, like the other types, made from one large pontoon and moored offshore with a single point mooring system, so that it can weathervane. Feeders can transport dry bulk to the terminal, where large bulk carriers can be loaded.

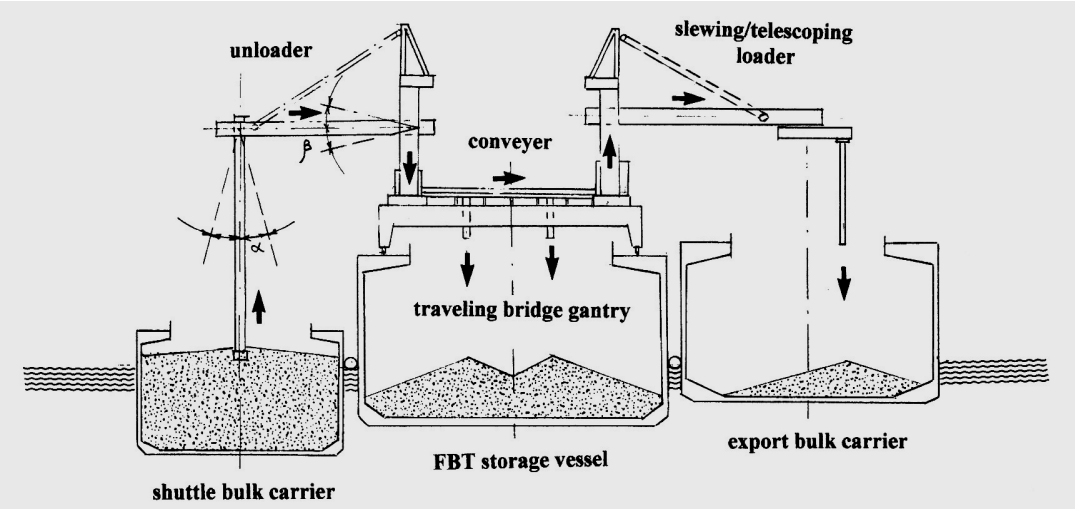


Figure 2.3: Floating bulk terminal [ref.1]

2.8 Floating quay wall, case study second Maasvlakte, Rotterdam

For the Maasvlakte II a number of studies have been done into quay walls, terminals and cargo handling systems of the future. To have an overview if the research performed in the Netherlands, a quick literature survey has been carried out on a number of graduation reports from the Delft University of Technology. Two of these; "Maasvlakte 2, vanuit historisch perspectief" by R.A. Hutter (2003) and "Maasvlakte II, capaciteit natte infrastructuur" by E.C.J. Goemans (2003) are not really usable for my thesis, but did give some insight in the project of the second Maasvlakte.
Ref. 14 is also a part of the project of quay wall of the future, but this deals with a block wall and not with any floating structure. Therefore, it will not be used further. In ref. 15 a container terminal of the future is described, with a floating and moveable quay. This is an interesting study and several of the findings may be used in describing the floating terminal in operation.

2.9 Floating naval pier for repair and loading of stores in Norfolk

Ref. 2: p.335

Navy docks are primarily used to carry out minor vessel repairs and supply vessels with ammunition, food, potable water. Consequently, the pier deck must accommodate such services as electricity, steam, compressed air, potable water and sewage. The deck also serves as a platform for contractors to refit vessels. Finally, the pier has to accommodate heavy pedestrian traffic. All this creates pier congestion. To overcome these problems, a double-deck floating pier has been developed for specific site conditions at the Charleston Navy Station USA.
Overall, the proposed pier has a length of 366 m and a width of 22,8 m. The pier's upper deck is connected to the shore by a centrally located articulated ramp.

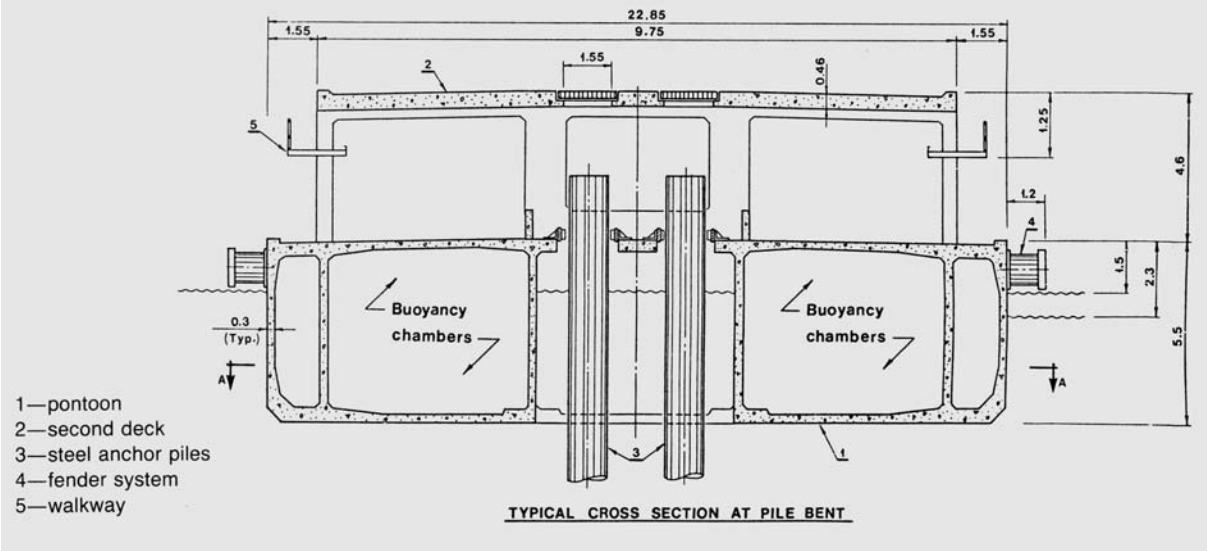


Figure 2.4: Floating pier at Navy port Norfolk [ref.2]

This structure consists only of a pier, not a storage yard. It is moored independently.

2.10 Mega-Float; Test platform floating airport in Tokyo Bay

Mega-Float is a very large artificial island floating in the sea; a Very Large Floating Structure (VLFS). The technology to design, fabricate and construct Mega-Float have been developed by Technological Research Association of Mega-Float (TRAM) comprised of seventeen leading shipbuilding companies and steel companies in Japan from 1995 to 2000.
[Ref. http://www.srcj.or.jp/html/megafloat_en/whatmega/what_index.html]

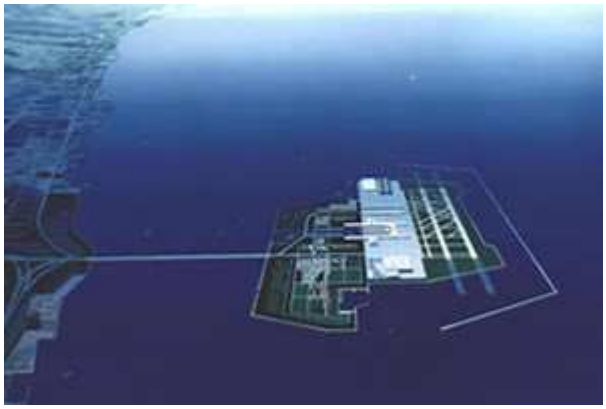




Figure 2.5: Floating offshore airport

In the second phase of the research and development of a Mega Float, a 1000 m x 60-121 m floating airport was constructed in Yokosuka Bay. Test flights (take-offs and landings) have been carried out.

Mega Float consists of a breakwater, the main floating body, a mooring unit and an access route to land. The Mega Float was originally moored by a Mono-pod fender-type mooring dolphin, which constrains the large floating body within safe limits. The water depth is limited.
The lifetime of the Mega-Float is expected to be 100 years and maintenance will have to take place at sea. Minimal maintenance is therefore preferred.

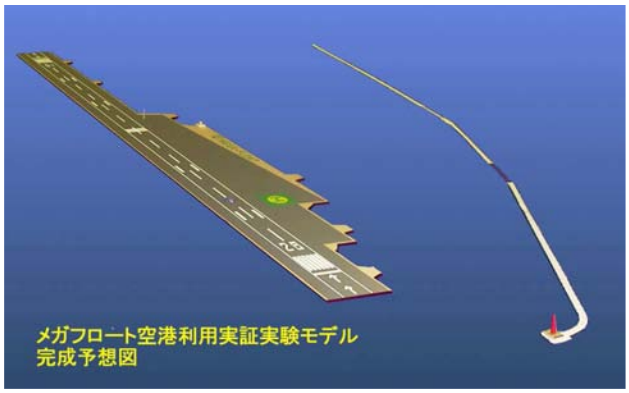




Figure 2.6: Mega-Float

The steel pontoons were fabricated at shipyards (max. pontoon dimensions were 300 x 60 m), towed to the site where the mooring device was being installed and then welded together and moored.

2.11 Floating greenhouses

[Ref. 31]; background study to floating greenhouses:
There is some information on the mooring and connection of the pontoons used as foundation for the floating greenhouses in this thesis, but little can be used for the design of the floating terminal. The analysis of dynamic behaviour is short and the location is on a lake in Friesland. The loads of floating greenhouses are much smaller than those of a terminal.

2.12 Floating houses; Amphibian living

A great number of architectural ideas have been put forward in this project, but they are of little consequence to this thesis. Floating elements have been designed in rectangular and hexagon shape. The connections have been designed both rigid and flexible. Mainly, the pontoons would contain one or a few houses and the pontoons would be moored individually. They could move in relation to each other, especially in vertical direction.

2.13 Mobile Offshore Base (MOB)

[Ref. 25]:
The present interest in offshore basing is motivated by the steady decline in long-term access to U.S. overseas bases such as airfields, shipping ports and logistics facilities. This affects a wide range of missions with military, diplomatic, political and economic implications.
A MOB is a self-propelled, modular, floating platform that can be assembled in lengths up to 2 kilometres, as required, to provide logistic support of U.S. military operations where fixed bases are not available or adequate. An MOB would accept cargo from aircraft and container ships and discharge resources to the shore via a variety of surface vessels and aircraft.
An assembled MOB would, as the Mega Float, be a Very Large Floating Structure (VLFS), as it consists of connected elements.

There are four alternative concepts in MOB: [ref. 25, p.25]
- Hinged steel semi-submersible modules.
- Semi-submersible steel modules with flexible bridges.
- Independent steel semi-submersible modules.
- Concrete semi-submersible modules.

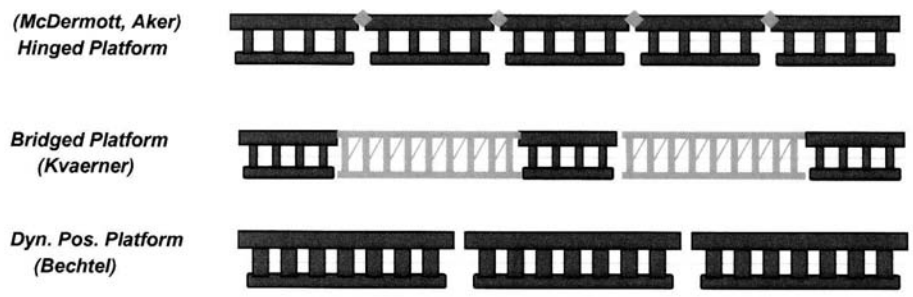


Figure 2.7: Three MOB alternatives [ref 25]

Each module consists of a box-type deck supported by multiple columns on two parallel pontoons. When transiting between operational sites, the module is deballasted and travels with the pontoons on the water surface much like a catamaran. When on site, the semi-submersible is ballasted down so that the pontoons are submerged below the surface zone, thereby minimizing the wave-induced dynamic motions [www.dt.navy.mil].

The MOB is especially designed for rapid transport, coupling and decoupling. The container terminal will be a semi permanent structure, which will be expected to be in only one or two places during its lifetime, as the Mega-Float is. Also, the MOB has been designed for very harsh conditions and a high sea state. The container terminal will not be located in the middle of the Atlantic Ocean. The MOB floating elements can be used independently and the demands of very high performance result in very large dimensions and high costs of the elements. The requirements for the terminal will be different.

MOB components are the following subsystems:
- Inter-module connectors.
- Alternative marine materials.
- Positioning concepts; mooring and dynamic positioning
- Response mitigation methods.
- Construction and repair methods.
- Open-sea cargo transfer techniques

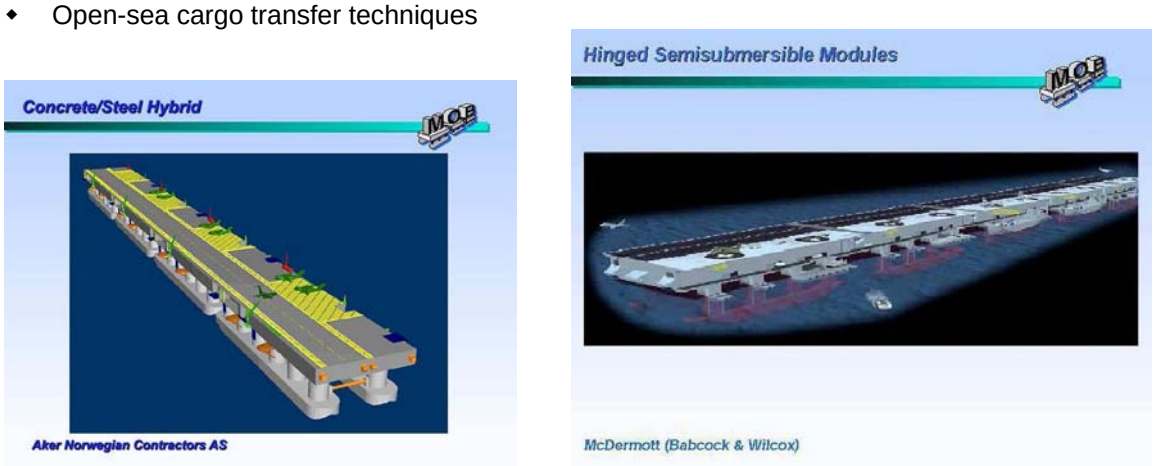


Figure 2.8: Two MOB alternatives visualized

Ref. 25 gives information on the design of the MOB, structural analysis and on dynamic analysis. These aspects will not be described here, but the information is used when considering the hydrodynamic behaviour of the floating terminal.

2.14 Monaco semi floating breakwater

One of the most interesting projects for this thesis next to MOB and Mega Float is the rather new semi floating breakwater in Monaco. The breakwater consists of a concrete pontoon 352 m in length that is connected to the platform area at the mainland by an articulated connection; a large ball-joint, so the connection is flexible. The breakwater is also designed to be used as a pier for cruise ships. The inner space of the pontoon has been used for parking space, a shopping centre, a restaurant etc.

The cruise ships will safely berthed at the lee side of the breakwater. Special measures have been taken to minimise the wave transmission underneath the breakwater. The larger the width of the terminal will be, the smaller the expected wave transmission. The pontoon dimensions are very interesting, because it shows that constructing and transporting very large pontoons is technically possible.

3. Environmental requirements and consequences

Ref 24: Marine structures 14: p.160
An environmental impact assessment of a Mega-Float in the Tokyo Bay was studied using a numerical model and the following conclusions were reached:

1. *When the Mega-Float alone is installed, the impact on the flow and water quality is minimal. The change in water temperature is especially small.*
2. *When breakwaters are installed with the Mega-Float, the environmental impact may slightly increase. The current technologies however, have the potential to assure environmental restoration.*

In the Westerschelde, the environmental aspects are very important. An environmental impact assessment of a floating platform should be made, but perhaps the EIA of the Tokyo Bay may serve as a comparison. It is clear from literature that a floating terminal will lead to a smaller environmental impact than a bottom-fixed terminal. However, the environmental loads caused by harbour activities and ship traffic will remain practically the same. Therefore, a floating terminal may be an environmentally friendly solution compared to a fixed terminal, but compared to the zero-alternative, this is another matter. The need for a harbour-expansion is the first thing that should be decided upon.

Ref 13; Integration of environmental issues in port planning and design: p.12
Port developments and environmental considerations
Port developments have a great impact on the surrounding areas. Though not always seen as an environmental issue it is worth while to recognise influences on erosion and accretion and increased salt wedge intrusion caused by a new layout of the port. Impacts on morphologic and hydrologic systems are relevant too. Other points to take into account are:

- *Impacts caused by capital dredging and disposal of dredged materials.*
- *Impacts caused by the construction of piers, breakwaters and other structures.*
- *Disturbance of landscape and natural environment.*
- *Loss of animal, plant or marine species or the natural habitats on which they depend.*
- *Relocation, loss of present land use of the area, involuntary resettlement.*
- *Social-cultural impact.*

Some of these impacts are reduced by constructing a floating port, which may primarily be designed to follow the movement of an unstable estuary and be towed to a new location after i.e. 30 years.
As a floating platform is built, the land area can be used for other ends. The influence on morphologic and hydrologic systems is not completely clear yet.

Operational activities and environmental considerations
The most relevant problems during operational activities of the port might arise from:
- Contamination from:
- ship spills.
- ship discharges of oily ballast, bilge water and sewage.
- uncontrolled run-off or waterfront drainage.
- uncontrolled waste disposal
- accidents
- Impacts caused by maintenance dredging and disposal of polluted dredged materials
- Impacts caused by industrial activities and traffic burdens.
- Nuisance from noise, vibration, light, waste, dust.
- Risks of hazardous materials and cargoes.
Environmental impact from operational activities will hardly change in the case of a floating port.
An environmental impact assessment (EIA) is an environmental analysis which is conducted to determine whether a proposed undertaking would significantly affect the environment, resulting in a report.
The idea behind EIA is simple: to draw up a list of expected impacts from a proposed project on the environment and describe a number of possible alternatives, the purpose being to provide decision makers with the information needed to arrive at a balanced judgement.

An intimate ecological survey was operated to investigate the influence of Mega-Float. From previous study, it was appeared that Mega-Float did not affect against native current, and, living things like Fish, shell, planktons and so on were not so much affected than what had been thought.
[Ref. www.srcj.or.jp/html/megafloat_en/whatmega]:

Current:
According to the result of the corroborative experiment of Mega-Float(hereafter called The Experiment) it is confirmed that the current is reduced a little just beneath the bottom of the floating structure and that in a little bit lower zone apart from the bottom it is almost unchanged from that around it, and also confirmed that due to the existence of the adequate current under the floating structure the current around it is unchanged.
Furthermore, little impacts to the current are found around the floating structure of several thousand meters in the current simulation analysis.

Water Quality:
According to the result of The Experiment little difference of the various factors of the water quality can be found between those under and around the floating structure except the small reduction of the dissolved oxygen (DO) just adjacent to the floating structure due to the activity of the attached organism, and no impacts due to in the installation of the floating structure to the water quality can be found.

Attached organisms:
According to the result of The Experiment it is confirmed that seaweeds and blue mussels are dominant species as the attached organism on the sidewall of the floating structure and blue mussels are on the bottom. Furthermore, the distribution of the attached organism is found to be same as that on an old breakwater in the kinds and the amount.ccording to the result of The Experiment it is confirmed that seaweeds and blue mussels are dominant species as the attached organism on the sidewall of the floating structure and blue mussels are on the bottom. Furthermore, the distribution of the attached organism is found to be same as that on an old breakwater in the kinds and the amount.

Fish:
The space under the floating structure is where fishes can stay comfortably and the attached organism on the sidewall and bottom is useful for the growth of fishes.

Sound in water and fishes:
Through the observation and investigation on the behavior of fishes to the various kinds of sound under the floating structure in The Experiment, it is confirmed that little impacts are found in the behavior of fishes by the sound in the water which diminishes with the distance from the floating structure except just adjacent to it.

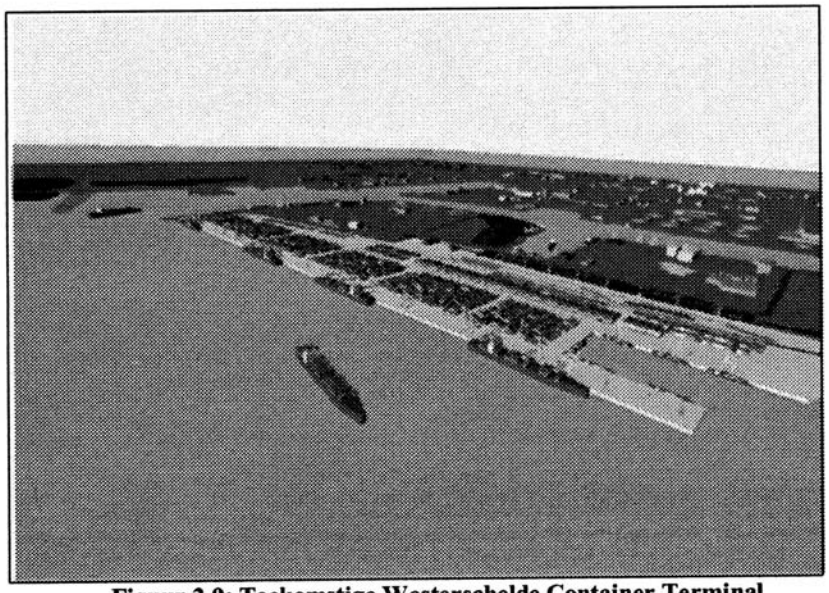
Plankton:
Phytoplanktons cease their photosynthesis in the shade zone under the floating structure and recover the activity after passing through it in the current and they never die out. Therefore, there are almost no impacts on zooplanktons which feed on phytoplanktons then there are little impacts on the ecosystem.

4. Natural boundary conditions and physical features of case studies

Three separate case studies will be described. The Westerschelde case study near Vlissingen is the most important one, as the design in the thesis will completely be made for that situation.
The other case studies are both a floating terminal at a deepwater location, one in the protected waters of a fjord and the other completely offshore, for instance in front of the coast of Japan. The offshore floating port will probably have the largest physical or environmental loads, but each case study has its' own characteristics.

4.1 Westerschelde Container Terminal

The boundary conditions of the situation at the Westerschelde will be based on the Program of requirements of the WCT [ref. 27], and also on [ref. 28] and [ref.19]. This Master thesis [19] comprises a study of alternatives of bottom-fixed (traditional) methods of building a pier and terminal in the Westerschelde. The thesis includes a project description with local and social backgrounds of the project and the requirements for the port expansion. This information will be used in the case-study.



Figuur 2.9: Toekomstige Westerschelde Container Terminal

Figure4.1: Future Westerschelde Container Terminal [ref. 20]

[Ref. 19]: "Westerschelde Container Terminal":
In June 1999 Zeeland Seaports declared to have the intention of realizing a container terminal in the port area of East Vlissingen. This Westerschelde Container Terminal will have a capacity of ca. 1,5 million containers a year (in 2015). The area of the terminal terrain will include about 150 hectares. The quay length comprises about 2650 metres.

[Ref. 20]; Downtime analyse van een "exposed" container terminal:
Due to the growing need for deepsea ports and especially deepsea container terminals, more and more terminals are being constructed in "exposed" locations. To sufficiently protect such a harbour, expensive breakwaters are needed to ensure the ships can be moored at berth in calm waters and that loading and unloading activities will not be disturbed. Costs may be spared if a terminal in an exposed location can operate to satisfaction without breakwaters being constructed.
From the seaside the container terminal will be connected to deepsea and shortsea (by feeders etc.) transport. The port expansion with a container terminal is crucial for the future development of the Zeeland ports.

The Westerschelde container terminal will, according to the plans of a bottom-fixed terminal, be constructed on a site exposed to higher waves and stronger currents than most harbours. The protection of the harbour from the sea is small, as the entrance of the Westerschelde is very wide and there are few measures taken. This could lead to a higher downtime of the terminal, whilst a short turn-around time of the ship at berth would be optimal.
For a floating terminal this exposed site may prove very important because of the dynamical behaviour of the structure. However, the boundary conditions in the Westerschelde will still be less severe than on a location offshore.

Choice of location [ref.19]:
An extensive analysis has been made about the best location of the new container terminal. This location, to the east of Vlissingen, may not be the best location for a floating terminal, although many of the boundary conditions concerning space, infrastructure, transport and economics will stay the same. The analysis in [ref.19] can be used as a guide in the choice of a new location; among other things the same alternatives can be considered.

Loads [ref. 20, 27, 28]:
- *Wind waves from landward wind*
- *No waves with seaward wind*
- *Waves are lower than out at sea*

Wind speed (m/s)	*Repetition period (year)*
17,8	0,5
18,9	1
19,9	2
21,1	5
22	10
22,8	20
23,8	50
24,5	100
25,3	200
26,2	500

Offshore wave conditions:
1/100 years H_{m0} *= 7,07 m.*
1/10 years H_{m0} *= 6,22 m.*
1/1 years H_s *= 5,8 m, which is the average of 1/3 of the highest waves in a period.*

Hydraulic boundary conditions at harbour Vlissingen East [ref. 27, 28]:

Frequency (yrs)	High water level to N.A.P. (m)	Low water level to N.A.P. (m)	Design wave height H_s (m)	Wave period T_p (s)
10	2,90	-2,54	1,20	4,7
1	3,45	-2,80	1,50	5,2
0,1	4,00	-3,05	1,80	5,7
0,02	4,35	-3,35	-	-

[ref. 20; p.12]: results from the calculations of the Waterloopkundig Laboratorium in Delft (1999).
Location near to the new terminal:
- Maximum flood current: 1,5 m/s
- Maximum ebb current: 1,2 m/s
- Maximum flood current further away in channel: 1,65 m/s
- Maximum ebb current further away in channel: 1,35 m/s

Primary parts of the Program of Requirements of the Westerschelde Container Terminal [ref. 27] can be used for the design of the dimensions of the terminal:
General configuration of the quay:
Existing ground level: NAP+5,20 m
Topside quay wall: *NAP+7,25 m*
Lifetime: *minimum 50 years and safety class 3*
Length sea side quay: *2615 m*

Length quay inland shipping: 930 m.

Cranes :
Front of the quay wall is the foundation of the front crane rails
(whether this is feasible for a floating terminal remains to be seen)
Maximum distance between front side fender and centre of the crane rails is 5,50 m.
The distance between the two crane rails is 30 m

Ship data [ref. 19, 27, 28]:

Capacity:	*12500 TEU*
Length b.p.:	*382 m*
Length:	*400 m*
Beam:	*57 m (22 boxes)*
Actual depth:	*14,2 m (normal; Lloyds)*
Max. depth:	*17 m*
Height incl. boxes:	*54 m*
Water moved:	*220 000 mtons*
Block coefficient:	*0,686*
Design speed:	*23-25 knots*
Machinery:	*single / hybrid*
Berth depth [ref.27]:	*NAP – 20 m by Lloyds register [ref. 27, appendix 3]*

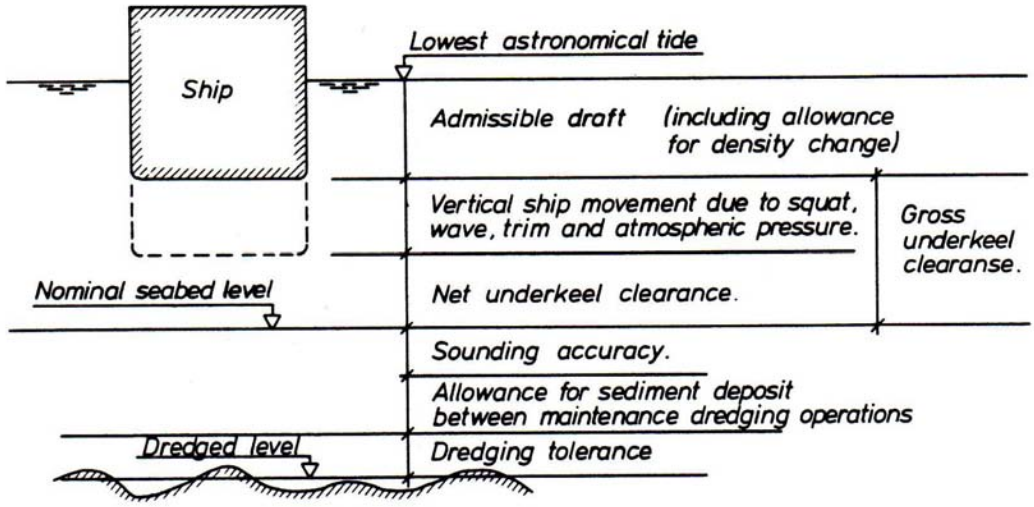


Figure 4.2: Design and contract depth [ref. 41]

Mechanical Loads:
Container storage load: q = 60 kN/m^2

Crane track:	*Vertical 800 kN/m, horizontal 80 kN/m*
Bolder force:	*2000kN over 30 m (70 kN/m)*

Distance between bolders:	*max. 30 m*
Distance between fenders:	*max. 30 m, bottom side NAP –2,80 m.*

The WCT includes a terminal for ULCS with four berths and a quay and apron area terminal for inland ships and feeders, the “Lichterhaven” with two berths; north and south.
The Lichterkade will have to have a length of 300 m with a contract depth of 12 m, an extra head pier has a length of 100 m where small inland ships and tug boats can berth.
If the layout of the floating terminal is different from the fixed terminal, a solution will have to be sought for these three berths along the pier.

Applicable bottom-fixed designs are: [ref. 28, 19]
- *L-wall*
- *Anchored combi-wall with relieving floor*
- *Caisson structure*

- *Cell dam structure*

This shows that the idea of prefabricated elements has been considered before, but the pontoons would be submerged.

4.2 Deepwater location in Norwegian fjord

In a fjord several types of loads can be expected. The water can be presumed to be deep and cold. Ice loads will certainly be of great importance. As the fjord is protected from open sea by an entrance and often by mountains along the shore, the wave loads and wind load will be limited compared to an offshore location. The dynamic behaviour of the large floating structure will be less of a problem, when mainly short waves can enter the harbour basin, as long waves are kept out. The significant wave height will be less in the harbour than at sea, which is positive for the concrete structure.
About existing currents in a fjord, not much has been researched.
The waves will mostly come from the dominant wind- or current direction, which is probably the landward wind from sea. The other side of the terminal is protected by the shore.
Hinterland connections will be necessary in this case and a complicating factor may be the steep, rocky shore on which one side of the Connection Bridge, ramp or other has to be founded.

Ref. 3; Marine structures engineering: p.136
Ice forces on structures
Ice acts as a medium to transmit such environmental forces as winds, currents and tides to the structure. If the driving forces are sufficiently high, the structure may be fully enveloped in the ice feature. In this case, the contact width is at its maximum value and subsequently the ice load exerted upon the structure reaches its maximum value. The ice failure mode in this case is crashing.
Two other failure modes in case of smaller driving forces are buckling and bending of ice, as ice can not take tension stresses.

In [ref.3] a method is explained to determine the ice load in all the different failure modes of the ice. Also dynamic ice forces could be of importance (p.158):
Drifting ice floes can move with considerable speed under the action of environmental driving forces, especially in spring during ice break-up. In this case, the velocity of the ice is such that the failure of the ice occurs in the brittle or transition zone. The failure modes that may be involved are crushing, spalling, ice cracking and ice wedging.
The considerations in this book on ice might be needed when the different case studies in the thesis are compared and more will be said about the loads and construction in a fjord.

[Ref. 23] is a Master thesis on arctic terminals in Russia, which may partly be used in considering a different design and construction method for a floating terminal in a fjord.

4.3 Deepwater location offshore near Japan [ref. 22]

A deepwater port offshore may be located a few kilometres from the shore or from the port of the hinterland. In this case, the floating port will mainly have a transhipment function. Ultra Large Container Ships could be loaded or unloaded at several of these deep ports, where their cargo will be loaded upon coasters or feeders. Those smaller ships will bring the cargo ashore or to smaller ports along the coast, which do not have enough depth to accommodate ULCS.
This way, as they visit only a few ports per continent, large sea ships need less time to visit all the continents in one voyage.

The loading on the floating structure will be severe in this case. The wave spectrum may vary from long to short waves, the significant wave-height may be large and the dominant wave- or wind direction may vary (e.g. with the season), while no protection from the wind or waves is at hand.
It will be beneficial for the floating structure if it can move astern the waves, so that the mooring forces will be less.

Deep water should be assumed in this case and a heavy wind load.
The hinterland connection is formed by the feeders' ship line service, assuming container cargo.
Liquid bulk could be easily transported by pipeline.
Dry bulk could perhaps be transported by conveyor belt.

As a ramp can only suffice for short distances, a hinterland connection could be designed as a long floating bridge or a floating tunnel, perhaps with a founded bearing structure in between, located at limited depth. This could be a gravity structure or anchored structure (from offshore technology), or even a small artificial island. Naturally the need for such a bearing is dependent on the length of the hinterland connection.

A deepwater offshore terminal might be located at the North Sea, in front of the port of Rotterdam. Such a floating port will only be feasible if the largest future containerships can be loaded and unloaded reasonably fast. A large container crane may be needed for that purpose, such as the crane within the requirements of the new Euromax quay wall in Rotterdam. This crane may achieve an average of 50-60 moves per hour. As yet, the foundation and wheel dimensions and therefore the loads of the crane on the quay are uncertain, as the crane puts a very high load and tension on the steel wheel surface.

When viewing Ultra Large Container ships in combination with a very large crane, perhaps it would be better to use a design ship with a capacity of 13500 TEU.

The application of a floating harbour may be limited to parts of the oceans, as a floating port will hardly be able to function in extreme conditions such as in very cold seas with wave heights of around 20 m. Arctic engineering is an entirely different part of hydraulic engineering. Depending on its dimensions, ship types and other things, a floating port may be operational only under circumstances up to a determined wave height or amount of wind or ice.

5. Functional Harbour Layout

Port planning concerns an entire harbour. A terminal will only be part of such a harbour.
Port planning is described in {Ref. 9,13], but on a detailed level it may be beyond the scope of this thesis.

The dimensions of the terminal are determined by the number of berths required, the length of the design vessel and the cargo-handling equipment to be used. The port layout and the berthing facilities are designed to accommodate the future expected ships and amount of cargo turnover. The layout of the port depends on the type of port operation; for example container, liquid bulk or dry bulk transport. The layout of the roads, offices and storage area will vary with the type of cargo. Especially the cargo-handling equipment at the terminal and therefore the required infrastructure and traffic is determined by the type of cargo.

Ref.2; Floating ports: p.3
The location selected for a floating port should be based on the:

1. *Sufficiency of the port basin area for ship operation.*
2. *Accessibility to point of need (e.g. terminal).*
3. *Protection from open-sea exposure.*
4. *Required water depths.*
5. *Required vessel manoeuvring room.*
6. *Prevailing wind, wave, current and ice conditions.*

The shape of a floating pier depends on the site conditions and the type of ships and cargo to be expected:

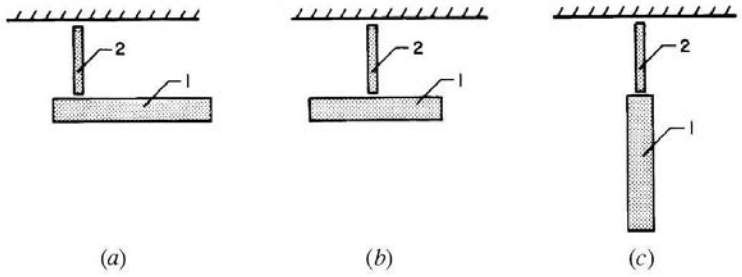


Figure 7.48 Typical layouts of floating docks: (*a*) L-shaped pier; (*b*) T-head pier; (*c*) finger pier. 1, floating pier; 2, access bridge (From Tsinker and Vernigora, 1980.)

Figure 5.1: [ref. 1]

The first boundary condition is the space in the harbour basin (the finger pier is rather old-fashioned, because large port productivity makes internal transport difficult). Secondly, the amount of traffic and cargo-handling equipment, whether automated or not, determines the optimal width of the pier and length towards the nearest connection to the mainland. The number of connections is also variable. These considerations hold not only for a floating pier, but also for a floating terminal.

In [Ref. 22] four different alternatives are considered for the lay-out of a floating transhipment container terminal offshore. The most important aspects considered are the operational and structural aspects the costs (and surface area). Connected aspects are the protection of the moored ships against the waves, the mooring system of the terminal and the dynamic behaviour of the structure. From this study, the rectangular variant with several berths alongside was selected as the best alternative. This type of layout can also be a starting point for this Master Thesis.
The layout of the storage yard of a floating terminal does have to be very different from that of a land-based terminal. Simplicity of shape, plans and infrastructure is preferable, considering the construction of the terminal with large concrete floating elements.

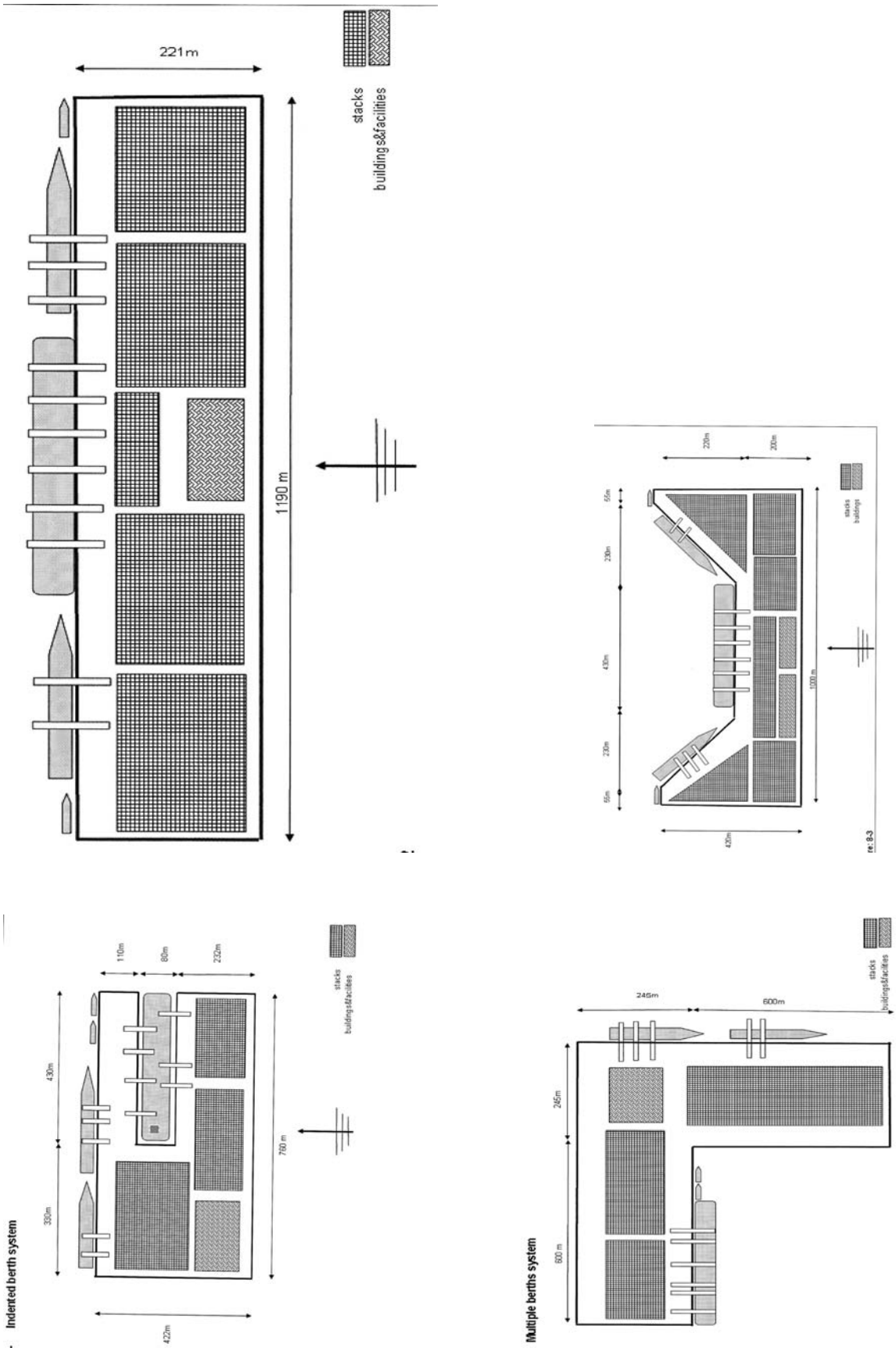


Figure 5.2 a-d: Four alternative layouts of a floating container terminal [ref. 22]

[Ref. 9] gives a lot of information on port planning and layout, as do the lecture notes on ports and terminals [ref. 13-1] from Delft University of Technology, but this can only partly be used for a floating terminal. Therefore, the information is supplemented with the functioning of a floating pier [ref. 1,2], which is off course not entirely applicable to a terminal.

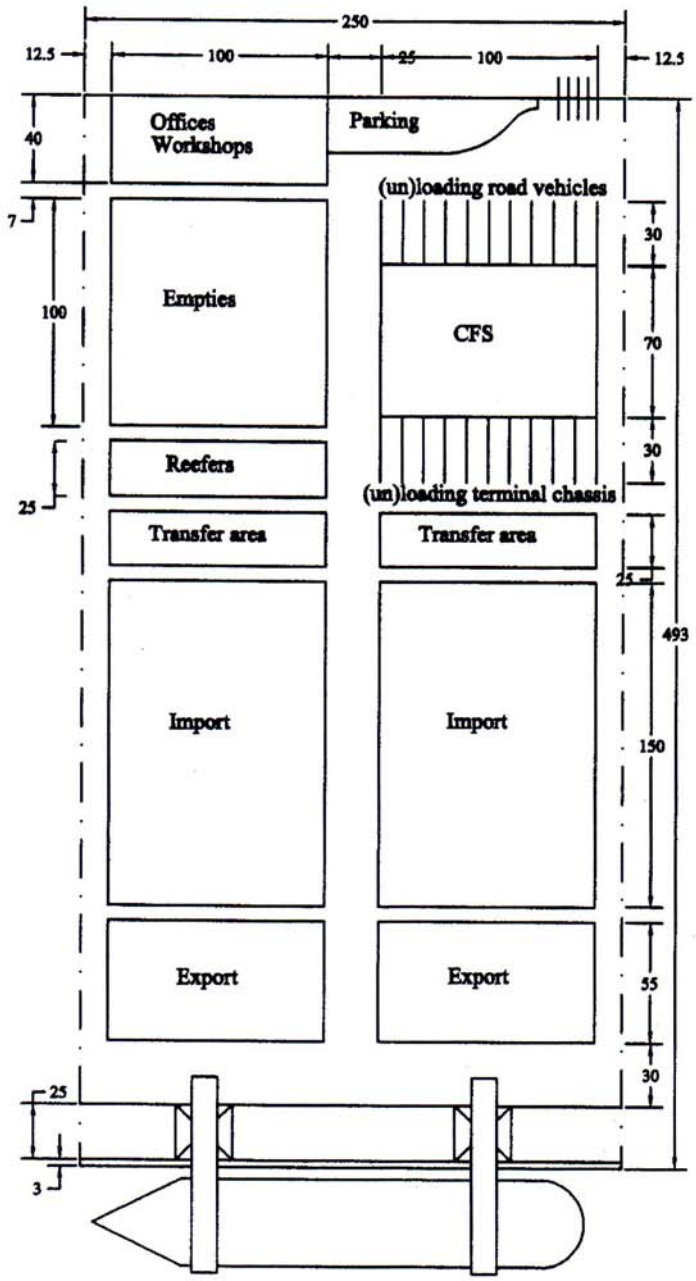


Figure 5.3: A functional layout of a terminal according to [ref. 13-1]

This figure shows the basic layout of a (container) terminal. Because of its simplicity (rectangular parts), this may very well be applicable for a floating terminal.
The port operations and queuing theory are mainly omitted, because the aim of this thesis is not to examine port management etc. Several boundary conditions and starting points will be taken from [ref. 22] and [ref. 13-1].

Ref 9; Port design: p.26 provides some starting points for the layout of the floating terminal:
For the majority of ports, the storage time in the port area is one of the most critical factors in evaluating the port capacity. Today, with faster loading and unloading of ships, it is very seldom the

berthing capacity (the number of berths) that reduces the port efficiency. As a rule it is the limitation of the storage area behind the quay which is the determining factor.
It may prove difficult to find a suitable area for port expansion ashore.
The following determines the terminal area needed:

The length of the berth will depend on the type of ship and/or cargo to be expected. The number of berths will depend on the berth occupancy (ratio).
High berth occupancy factors can seem attractive because this yields the highest berth utilization, but it is usual to assume a ration of the average waiting time or congestion time to the average berth service time(depending on type of berth, ship and cargo) not higher than betweeen5 to 20%.

Today for example, the capacity per hour for a typical container crane varies between 10 and 50 containers, with an average capacity of about 25 containers per hour. If the average crane capacity is increased to about 50 or more containers per hour, the total terminal capacity, i.e. the area needs, storage and delivery capacity etc. has to be increased tremendously.
This would be the case at the new Euromax terminal in Rotterdam. Such a crane is not yet in use.

The width of the apron will vary between 15 to 50 m depending on the loading and deloading equipment, trucks, cranes etc. The yard or area behind the apron may be divided in a primary yard (adjacent to the apron, storing of inbound and outbound cargo) and a secondary yard (empties and equipment) with entrance facilities.
The yard area should have a depth of preferably about 700 m behind the apron for a modern container terminal. Tthe land requirements are related to the storage density and the time the cargo stays in port. As a rule of thumb the area required for a container terminal is about 4 to 20 ha per berth depending on the generation of the container ship. These figures include areas for offices, sheds, workshops, roads etc. The storage area should have an additional area of between 25 and 40% as reserve capacity for the future.

The terminal must be efficiently used. Functional requirements are comprised of a retaining, bearing, protecting and mooring function of a pier or quay wall.
The functional requirements concern Area requirements, structural requirements (bearing all internal and external loads), berth facilities and other facilities and equipment. [Ref. 22].

Area requirements:
- Storage yard (import, export, reefers and empties).
- Apron area.
- Intern transport infrastructure.

Berth facilites:
- Quay.
- Mooring system for ships
- Fender system.

Facilities:
- Buildings (main administration and others).
- Utilities (fire fighting system, waste disposal system, electricity and water supply).

Equipment:
- Loading/unloading equipment
- Horizontal transport equipment
- Supply boats, tugboats and other marine equipment

The operational requirements concern motions, accelerations, stability of the structure and a maximum wave height (boundary conditions). The operational aspects of the terminal layout are mainly concerned with the number of berths (and cranes), the quay length and the surface area of the apron and storage area. The number of berths needed could be calculated from a queuing theory analysis and a calculation of the productivity per berth. However, these matters will not be examined any further, because the literature on the Westerschelde Container Terminal is clear about those requirements.

The breakwaters or other facilities needed to protect the harbour entrance and approach channel are part of the port layout. The entrance should be protected against the dominant wave, wind or current direction, while ships should easily be able to enter the approach channel, as can be seen in the figure below [ref.13].
Breakwaters can be constructed according to different concepts. In general, a breakwater is a very expensive structure. The necessity of breakwaters around a floating terminal could be researched. In protected waters, a breakwater might not be necessary at all. The site conditions and the design (strength) of the specific terminal determine the combination of the kind of terminal and the type of possible breakwater. In a deepwater location, a combination of a floating harbour and a floating breakwater offer the best solution. In the case of strong requirements regarding moveability, both a floating terminal and a floating breakwater could be easily moved to another location.

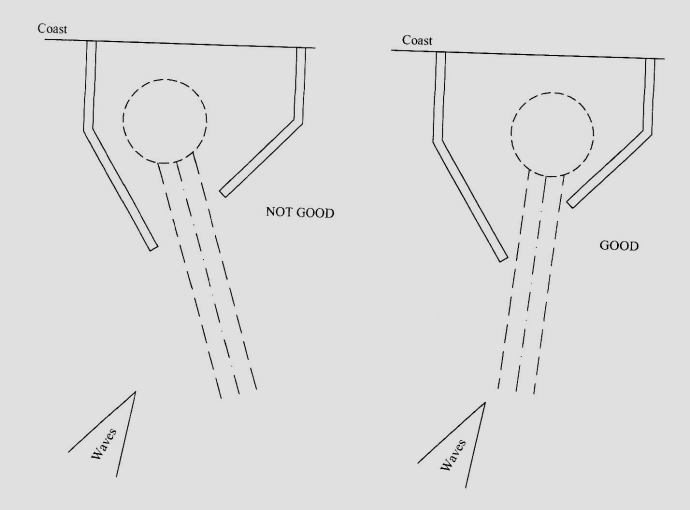


Figure 5.4: direction of breakwaters and approach channel [ref. 13-1]

Ref 1; Port Engineering: p.673
Experience has shown that the use of floating breakwaters can be structurally and economically feasible under certain conditions:
- *Deepwater conditions result in excessive construction costs for bottom-fixed structures.*
- *Foundation conditions may present high costs for corrective action.*
- *A bottom-fixed structure impedes the free interchange of water in a basin, resulting in minimized water quality parameters.*
- *A fixed structure causes "visual pollution" in the area.*

Furthermore, the experience has confirmed the following:
- *Floating breakwaters offer a feasible and economic alternative to conventional breakwaters in deep water when exposed to moderate wave conditions (i.e. < 2m).*
- *Floating breakwaters can be designed to reduce waves to acceptable levels as long as those waves are not long. A practical upper limit is on the order of 4 to 6 s; in most practical cases, floating breakwaters have been designed for wave periods of less than 4 s.*
- *Floating breakwater structural units and their mooring systems are vulnerable to catastrophic failure during severe storms when they are needed the most.*
- *Floating breakwaters require a relatively high level of maintenance compared to conventional fixed structures.*

6. Connections to the mainland

The floating terminal has to be connected to the mainland, if it is not a transhipment terminal. In case of container cargo, a pipeline or conveyor belt is out of the question. An automated cargo-handling system or vehicle should cross the connection towards the mainland.
A number of principles concerning a shore connection are mentioned below. Several alternatives have been constructed before. Others are more futuristic. Only a few of the possibilities are suited for a long distance connection, because in this case, the connection needs its own foundation or mooring system and can't be founded on the floating structure and the shore only [ref. 1].

Alternative connections are:
1. An articulated bridge (or ramp) for reasonable short distances.
2. A vertical lift-bridge for short distances.
3. A floating bridge connected to a pivot ashore or at the riverbank.
4. A floating bridge over the slope.
5. A floating pontoon and mobile wedge.
6. A floating tunnel for very long distances.

Ref. 2; Floating ports: p.23
An access bridge is the link between the pier and the shore or riverbank. For efficient dock operation, the means of access must provide effective circulation and the shortest possible distance from the pier to storage area. This can be achieved in several ways, depending on local conditions..
The type of bridge used in floated dock construction is selected on the basis of economic, structural and functional considerations; the cost of bridge maintenance and possible replacement should also be taken into account. The bridge must be convenient and safe in operation. In some cases aesthetic considerations may also affect the selection.

A floating terminal near the shore would most easily be connected by articulated ramps or short bridges, either floating or not.
The emphasis of this thesis will be on the design of the floating elements of the terminal and the connections between these elements. The hinterland connection could be a very interesting study, but it will only be discussed shortly.
The width and number of the hinterland connections is important to the port operations regarding transport speeds.


Figure 6.1: Floating bridge, United States

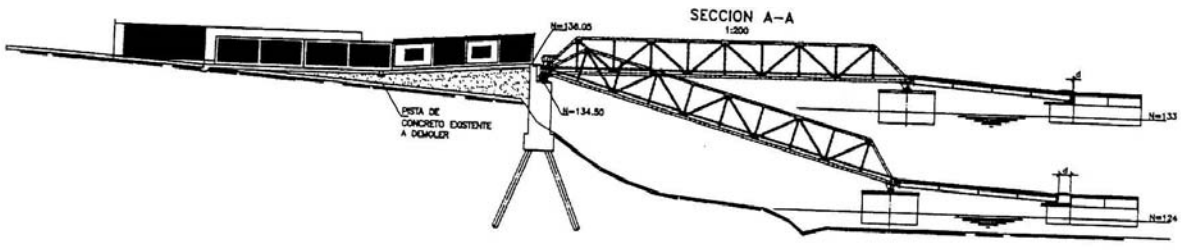


Figure 7.35 Plans and typical cross section, port of Yurimaguas, Peru floating terminal.

Figure6.2: [ref.1]

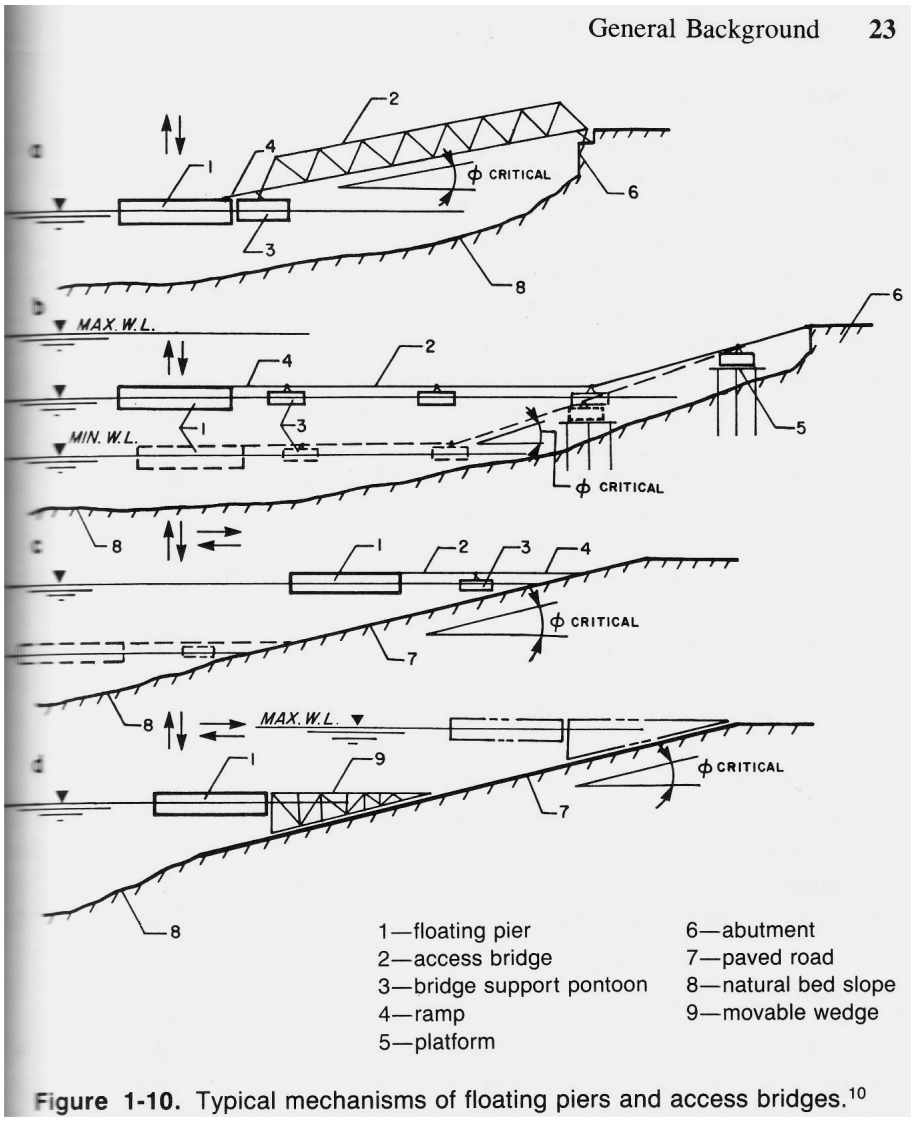


Figure 1-10. Typical mechanisms of floating piers and access bridges.[10]

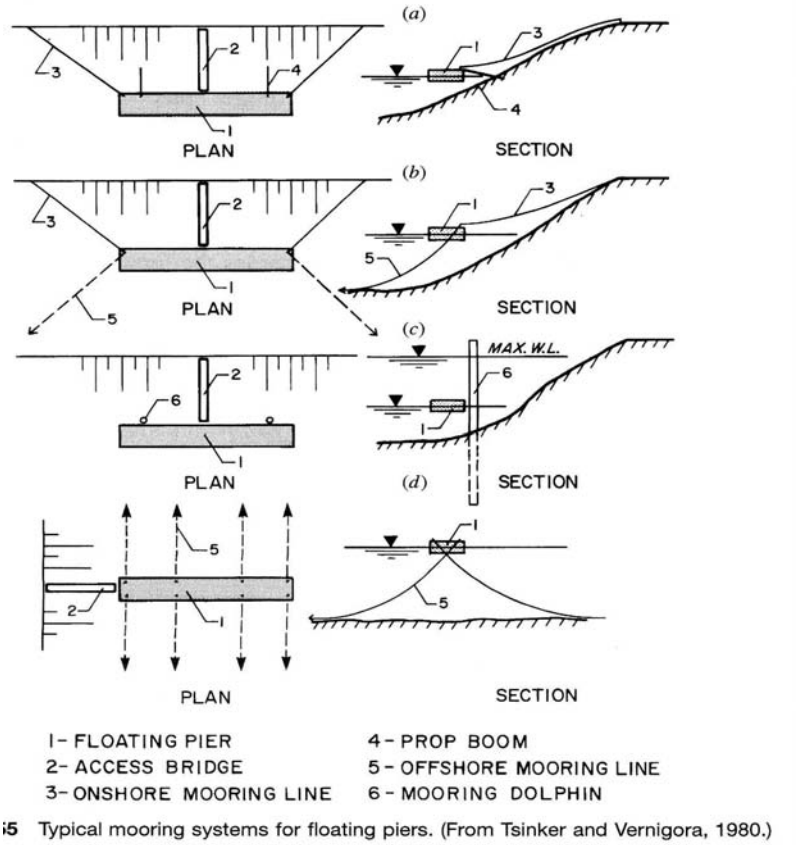
Figure 6.3: [ref. 1]

7. Mooring system of the terminal

A mooring system or anchorage takes care of the fixation of the floating structure to the bottom; horizontal motions are limited, so that the horizontal place (co-ordinates) is fixed.
The structure will have to be able to move with the tide and waves, but will stay in place within boundaries.
Mooring systems can generally be divided into the following systems [ref.1]:

- Single point mooring (SPM):
  Vessels are secured by a single line or structure.
  The floating object is allowed to weathervane; that is, swing around the in order to align itself with prevailing wind, wave and current conditions. This alignment tends to reduce the load on the mooring system. However the mooring forces enter the structure at one point, which will have to endure a very large force. An SPM requires a lot of space.
- Multibuoy moorings (MBM) or Spread moorings:
  An MBM holds a vessel in a relatively fixed position, and the vessel cannot turn head-on to the prevailing waves. As a result, an MBM can experience relatively high loads if wind, currents or waves act at an angle to mooring. The floating body cannot move and rotate freely, but the mooring forces are distributed better over the structure. Smaller forces are easier to introduce into a structure.
- Dynamic Positioning System:
  Next to a mooring system a Dynamic Positioning System could be employed, mainly to relieve the structure of some of the loads by turning it beam on to the waves [ref. 22]. This system would only be effective when the terminal can rotate freely, such as when located offshore. A fixed location near the shore would not permit such movements; therefore a DPS might be useless.

Mooring system components include anchors, sinkers, anchor chains, buoys and mooring lines.



i5 Typical mooring systems for floating piers. (From Tsinker and Vernigora, 1980.)

Figure 7.1: Different mooring systems [ref. 1]

Possible types of moorings are:
1. large ship anchors (limited structure weight)
2. Gravity based structures; pontoons, concrete blocks
3. Anchor cables, connected to piles or for instance rock anchors or a foundation block.
4. Anchor cables and suction piles
5. Piles with a sliding connection (limited water depth)
6. One (Single point mooring) or several large pylons, pillars or tower-shaped structures, to which the floating object can be connected with a vertically sliding connection.
7. Single point mooring by turret mooring
8. Single point mooring by a Calm buoy.

Single point mooring systems are often applied in Floating Storage and Offloading vessels and the like. Offshore floating structures have enough space to weathervane. Floating oil platforms (semi-submersibles) can be moored in various ways. Floating structures in more narrow, protected waters are not always allowed to weathervane, for instance because of the shipping traffic. Luckily, the mooring forces will mostly be smaller in these waters than offshore.

Ref 1: Port engineering: p. 472 Site criteria and dock layout
Normally, a floating dock is oriented to minimize mooring forces and ship impact. It is usually done by aligning the dock's longitudinal axis with the dominating current direction or the prevailing wind direction. This will reduce ship berthing impact..

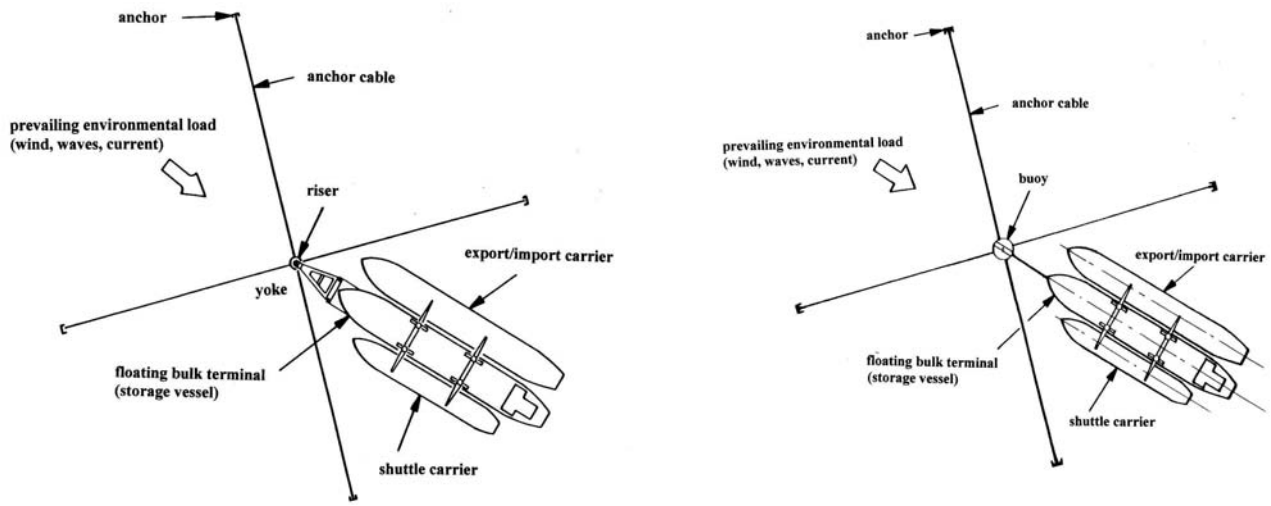


Figure 7.2: Single Mooring Point systems with riser or buoy [ref. 1]

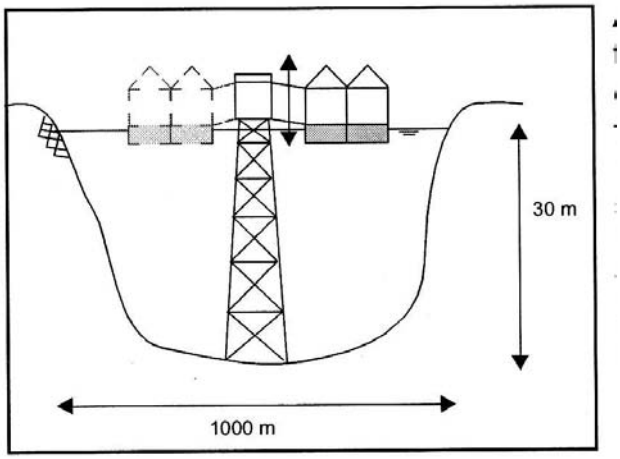


Figure 7.3: Single Point Mooring System for floating greenhouses [ref. 31]

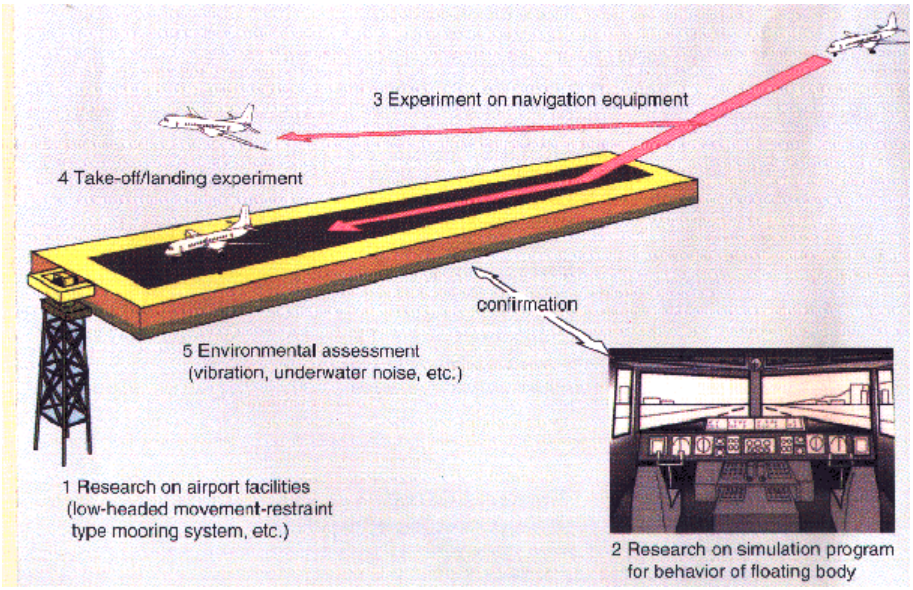


Figure7.4: Mega Float test plan with SPM system

8. Hydraulic design of a floating structure

Ref. 2; Floating ports: p.21
The typical floating port consists of five principal elements:

1. *A floating pier*
2. *Access bridges*
3. *The mooring system*
4. *The fender system*
5. *Mooring accessories*

In the case of this Master thesis the whole terminal is expected to be floating; the entire storage and office area, not just the apron area of the pier.
Every floating object should be classified by Lloyds and so should this floating terminal [ref.1].
The hydraulic design of the terminal would have to include a study into:

1. The shape of the floating elements of the terminal.
2. The type of deck structure of the terminal.
3. The geometry and configuration of the floating elements.
4. The geometrical design of the dimensions of the elements.
5. The static design of the elements, including freeboard, ballast and stability.

In [Ref. 22]; Floating transhipment container terminal, the best applicable concept for a floating element was chosen from four alternatives:

1. semi-submerged structure (Mobile Offshore Base concept)
2. Pontoon-shaped structure
3. Mega Float, a pontoon-shaped floating structure supported by mooring dolphins (or piles/pillars)
4. Tension leg floating structure

The pontoon-shaped structure was selected as best alternative, winning mainly on flexibility in water depth and financial aspects, but also as second (behind the Mega-Float) on stability and vessel protection. The hydrodynamic behaviour is less than that of the semi-submersible, but its stability is better. The hydrodynamic behaviour can be influenced by the type of mooring system, as the Mega float fixed to the bottom by dolphins in stead of cables, has a better hydrodynamic behaviour.
This thesis will deal with the same type of VLFS-concept; a pontoon-shaped structure. The mooring system however is still to be considered.

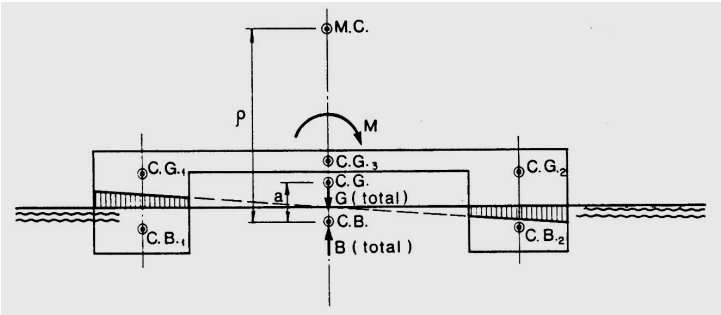


Figure 8.1: semi-submersible stability [ref. 1]

The structural scheme of the terminal consists of a combination of floating elements and a deck structure and is discussed in [Ref 2]:
There are four principal structural schemes for floating piers (Fig):

1. *One long pontoon.*
2. *Several large pontoons joined by pivots.*
3. *A series of small pontoons spanned by a number of single-span decks.*
4. *A series of small pontoons spanned by a continuous deck.*

From the capital-cost point of view, the last two alternatives are the least desirable because of the extra weight of the deck, which necessitates enlarging the pontoons. Structurally, pontoons could be built from concrete or steel, but steel is used more frequently.
However, from the material-consumption point of view, the continuous deck span is more economical than the pier consisting of a number of single-span decks. The most economical solution is the one with maximum stability, the one long pontoon. Alternative 2 is ranked from the cost point of view somewhere between alternatives 1 and 4. Alternatives 2, 3 and4 are preferable in view of construction, operations and maintenance, since removal of one pontoon or even an entire section of the pier for repair or maintenance would not significantly interrupt dock operations.

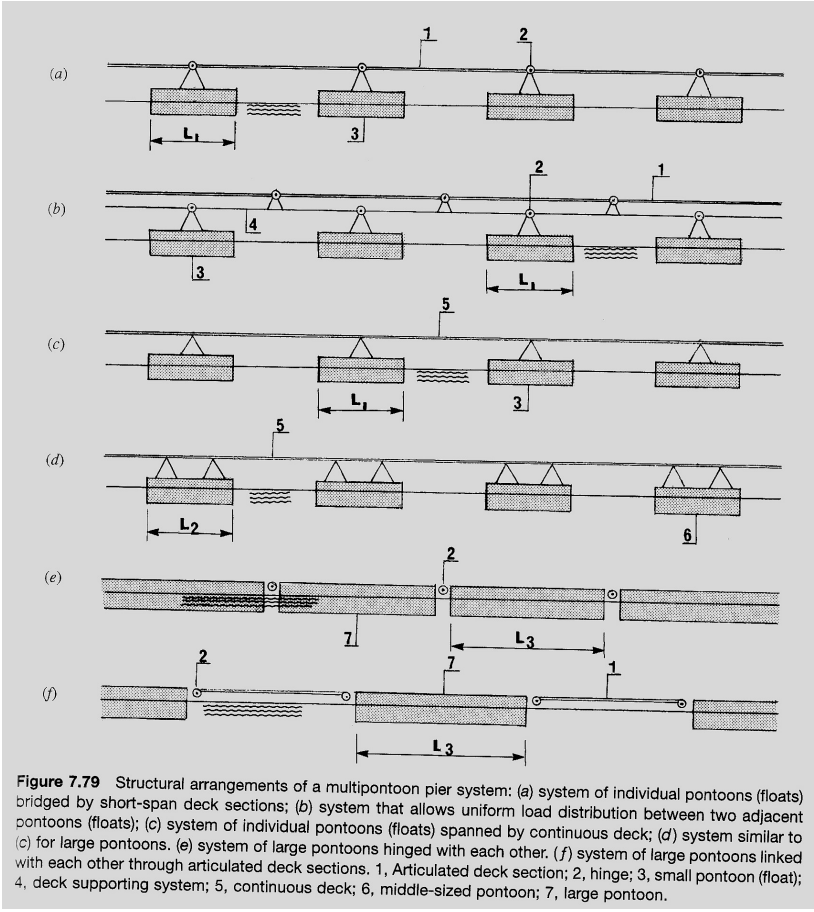


Figure 7.79 Structural arrangements of a multipontoon pier system: (*a*) system of individual pontoons (floats) bridged by short-span deck sections; (*b*) system that allows uniform load distribution between two adjacent pontoons (floats); (*c*) system of individual pontoons (floats) spanned by continuous deck; (*d*) system similar to (*c*) for large pontoons. (*e*) system of large pontoons hinged with each other. (*f*) system of large pontoons linked with each other through articulated deck sections. 1, Articulated deck section; 2, hinge; 3, small pontoon (float); 4, deck supporting system; 5, continuous deck; 6, middle-sized pontoon; 7, large pontoon.

Figure 8.2: six alternative Very Large Floating Structures [ref.1]

Over the years more structural schemes for floating platforms haven been designed:

- Mega Float; several large pontoons joined rigidly. Steel rectangular pontoons are connected stiffly to each other.
- Mobile Offshore Base; semi-submersible structures are independent or coupled with flexible connections.
- Small pontoons joined either by pivots or by rigid connections, without a separate deck structure.
- Pontoons coupled by a stiff plate as deck and as bottom, being more a kind of beam, in cross section consisting of many I-profiles. The concrete pontoons are tensioned together [ref. 16].

A continuous deck is a kind of rigid connection between the elements, single-span decks will be connected by pivots. The alternative structural schemes with a deck and possibly a bottom connecting the pontoons, generally have less structural height than the coupled pontoons.
[Ref. 26]: Floating structures in coastal zone – Part 2: Forces and response, can give useful information on the structural mechanics of a Very Large floating structure.

8.1 Basic design considerations

Ref 1; Port Engineering: p.531:
A concrete floating pier is usually built in the form of a closed box type of cellular structure. Two basic structural systems of concrete pier hull are usually considered:

1. *A cellular system with framing consisting of a modular arrangement of longitudinal and transverse watertight bulkheads.*
2. *A system consisting of a double-walled hull with a stiffening of framelike longitudinal and transverse walls.*

The pier structure should meet the following criteria: (implicit requirement of low cost)

1. *A sufficient number of watertight cells to satisfy the damage stability requirements; the pier must survive and remain afloat and stable should any two adjacent cells be punctured below the waterline.*
2. *Minimize construction time and in-situ concreting, maximize the use of precast plate panels.*
3. *Construction must be simplified by allowing for as much repetitious use of formwork as possible.*
4. *The hull shape must permit pier construction in the dry dock or building dock and must possess sufficient buoyancy to permit its safe launch, delivery and operation.*
5. *The pier must be robust, adequately stable and require minimum maintenance.*

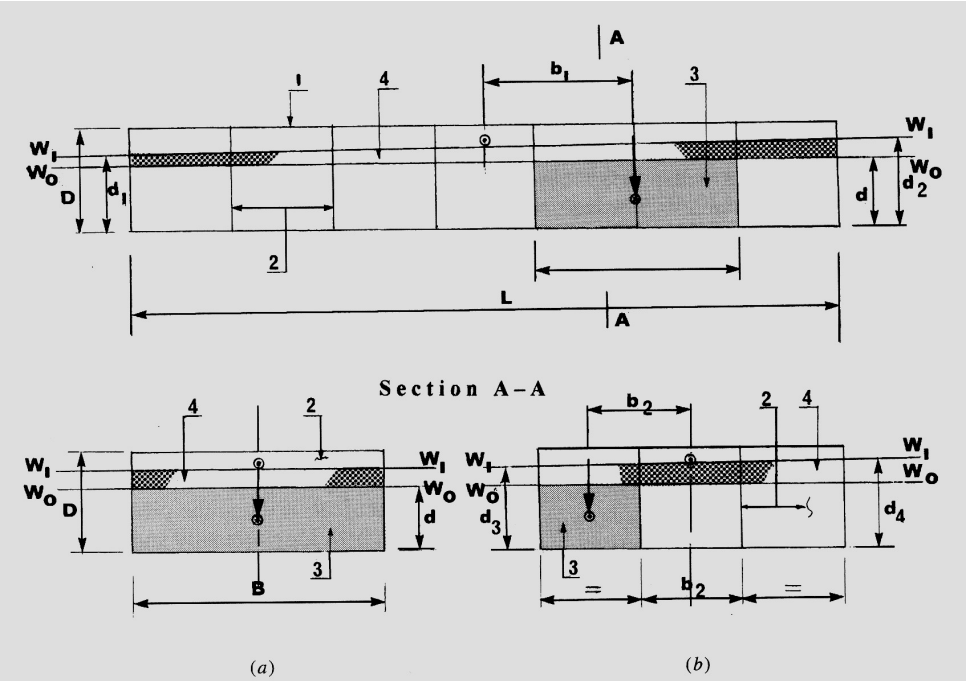


Figure 7.77 Stability of a damaged pier: (*a*) symmetric flooding; (*b*) asymmetric flooding. 1, Pier; 2, watertight bulkhead; 3, flooded area; 4, additional buoyancy to compensate for that lost.

Figure 8.3: Stability of a damaged pier [ref.1]

8.2 Static stability of the floating elements

The initial static stability is determined by the properties of the floating body; the dimensions and the self weight, and is not affected by external loads or excitations. [Ref. 5]; Ship stability for master and mates will be used to calculate the stability in transverse and longitudinal direction of the pontoons. The transverse direction is most important, because the metacentre M is mostly lower than in longitudinal direction and this also determines the dynamic behaviour of the floating body.
The static stability during operation can be influenced by loads on the structure or for example, by movements of cargo. The stability of an apron element of the terminal should be considered with

respect to cranes loaded with a suspended mass [ref. 5]. The suspended mass can increase the angle of the floating element in transversal direction.

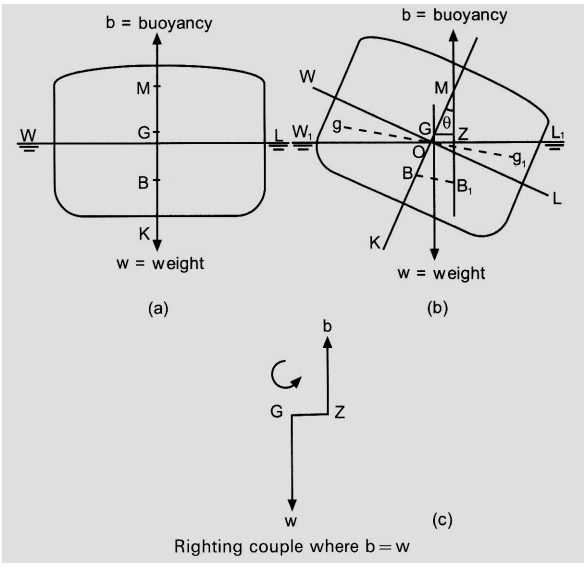


Figure 8.4: Transverse static stability [ref. 5]

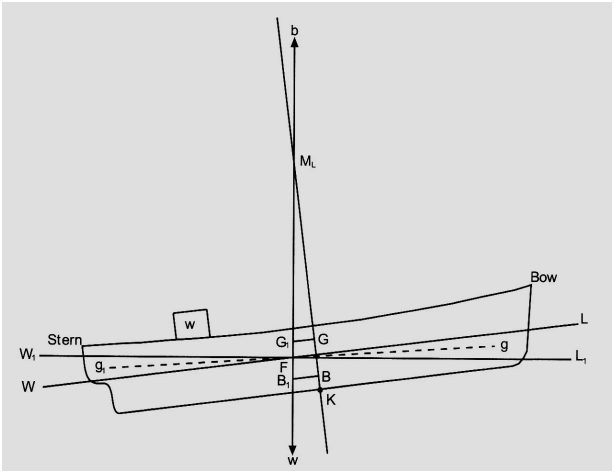


Figure 8.5: Trim [ref.5]

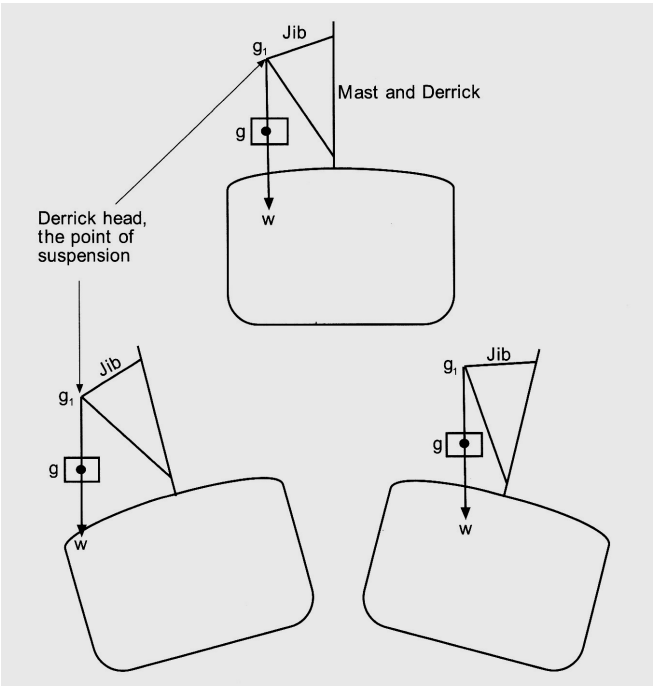


Figure 8.6: Static stability including suspended mass [ref. 5]

9. Hydromechanics of a floating structure

Hydromechanics are dealt with in ship mechanics, in offshore structures and in other floating structures. Several studies have already been done into the field of mega floaters or MOB's.
Most of these studies concerning fluid mechanics or hydroelasticity are about stiff platforms and describe mainly the wave patterns and the behaviour of currents etc.
Also the behaviour in stability and dynamics of a large floating hinged platform has been studied. However, not much is said about the real response of a steel or concrete structure to these hydrodynamic conditions. A number of studies, for instance by Prof. Hermans from Delft University of Technology, are mainly aimed at giving a mathematical explanation of the hydrodynamics concerning a large floating platform.
As yet it is not certain whether the connections between the pontoons will be entirely stiff, but they can't be entirely hinged either. Those studies can partly be used for a better understanding of the behaviour of a platform in the water.
An important aspect is the water depth. One article by Prof. Hermans [ref. 36] deals with wave interaction in shallow or deep water, which makes all the difference to the floating structure.

Several of the studies are too far fetched for this thesis, as the aim of this thesis is not to describe the mathematical applications in designing a floating platform. Therefore, if used, mainly their conclusions will be presented.
Other studies are performed just outside the thesis boundaries, as they are scientific, but not applicable to the situation at hand. Their boundary conditions are simply very different.

On the subject of hydromechanics, also a number of books have been found. They are accounted for, together with the articles used, in the reference list. As the time for this literature study was short, it is noted that further study will be done into hydromechanics and hydrodynamics of floating structures during the thesis. The books and articles mentioned will be used in this study.
The aims of the thesis in this part are to find out more about the dynamical behaviour of the floating terminal when loaded by waves and currents. In this respect, the influence of the number of elements in the floating platform and of the type of connections can be examined. The wave pattern around the structure will be looked at to a limited extent. All these eventually lead to describing a serviceability limit state for the design of the (front) structure, with external loads and dynamic response from the structure.

The lecture notes on Offshore Hydromechanics [ref. 10] are very extensive. They are used during all parts of the dynamic and hydromechanical analysis of the floating terminal.
Both [ref. 20] and [ref.22] are used as a start in the methods of approach of the hydromechanics of the structure, the first mainly for dynamic loads etc. The theory needed on the subject of dynamic behaviour is used from the lecture notes 'Offshore Hydromechanics' [ref. 10] and, if necessary, from maritime engineering 'Ship hydromechanics' [ref.46] and 'Geometrie en stabiliteit' [ref. 43]. Theory on the occurring waves in the Western Scheldt is found in the lecture notes 'Short waves' [ref.42].

9.1 Motions of a floating element

The motions of a floating structure are caused by the dynamic forces on the structure. A single floating element follows the motions of the tides and the waves, especially the vertical heave motions. The secondary movements are the response motions of the floating element, which are caused by the combinations of several dynamic forces. The theories of short waves and of hydromechanics provide the methods to determine the dynamical behaviour of a floating structure. These theories can be found in [Ref. 42 and 47]; lecture notes of Delft University of Technology. No specific part of these lecture notes is mentioned in this literature study, as the theory used is discussed in the hydrodynamic analysis and those appendices. The theories of hydrodynamics are studied from [Ref. 6 and 10].

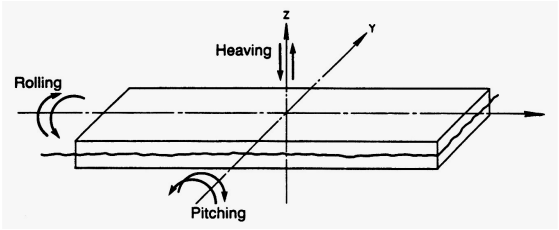


Figure 9.1: pontoon motions [ref.2]

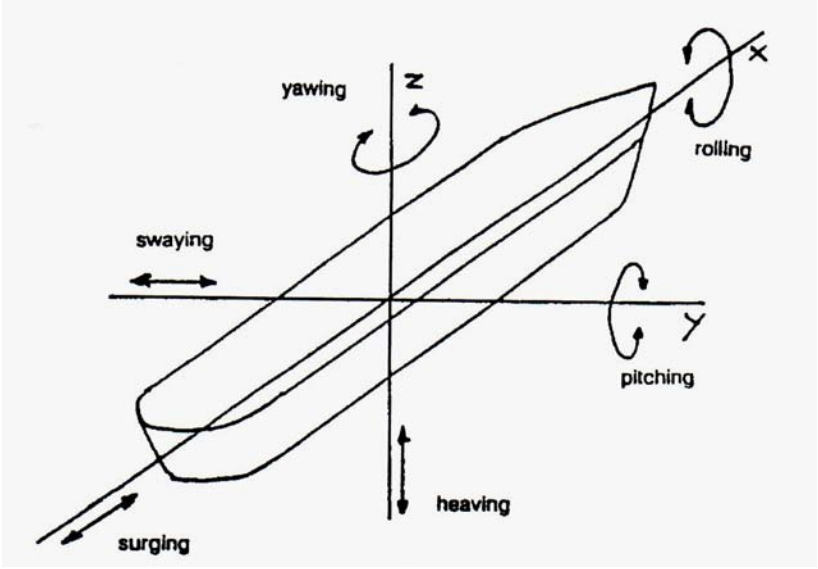


Figure9.2: Definition of ship motions in six Degrees of Freedom [ref. 5]

Three translations:
- Surge: a horizontal translation along the longitudinal x-axis of the body
- Sway: a horizontal translation along the transversal y-axis of the body
- Heave: a vertical translation along the vertical z-axis of the body.

Three rotations:
- Roll: a rotation around the longitudinal x-axis (φ)
- Pitch: a rotation around the transversal y-axis (θ)
- Yaw: a rotation around the vertical z-axis (ψ)

Any ship motion is built up from these basic motions. Another important type of motion is the vertical relative motion, defined as the vertical wave elevation minus the local vertical motion of the ship [ref. 10]. Relative motions between two coupled floating elements are very important. These could be two apron elements of the terminal or an apron element and a moored ship.

In general the free equation of motion is:

$$(m+m_w)\frac{d^2x}{dt^2}+(k+k_w)\frac{dx}{dt}+(c+c_w)x=F(x,t)=0$$

A general equation for the natural frequency is

$$f=\frac{1}{2\pi}\sqrt{\frac{k}{m}(1-\frac{c}{c_c})^2}$$

9.2 Dynamic forces

Both [ref. 20] and [ref.22] give a start in the approach method of the hydrodynamic analysis of the structure.
Dynamic loads can be considered as a summation of fluctuating, sinusoidal loads. They cause a (static) load on the structure and also an excitation of the structure into oscillatory response motions, which depend on the natural frequencies of the structure [Ref. 10].

Ref. 22; The FTCT: p.21 from PIANC: Criteria for movements of moored ships, a practical guide (report working group 24, 1995)
The wave periods of storm and swell waves are far from the natural periods of surge, sway and yaw of medium and large ships. Therefore, horizontal motions of significance are normally not occurring due to short waves. The natural periods of heave, pitch and roll of large ships are typically within the large range of short wave periods and consequently these motion modes can be excited.

Long waves are in contrast more difficult to dissipate. Their periods are close to the natural periods of surge, sway and yaw for medium and large ships.

The wind effect can be decomposed into a static (constant wind or slow variation in intensity) and a dynamic action (gusty wind, intensity blusters and changing direction). For calculation of steady wind forces, methods such as British Ship Research Association or Oil Companies International Marine Forum can be used. For the terminal the static wind load is most important. However, the high container canes will experience gusts of wind.

Current forces are caused by pressure drag. Under certain circumstances, a current can induce lateral oscillations due to 'flatter'. Flatter occurs when the arm of the moment exerted by the ensemble of external forces relative to the centre of gravity of the ship, including the added mass, reaches a value close to radius of gyration. Mostly, current forces are modelled as quasi static loads.

The astronomical tide does not exert forces of importance on moored ships, but it causes vertical rise and fall of the vessel. The use of constant tension winches avoids manual adjustments of the moorings. Another aspect to be taken into account is the position of the ship relative to the fenders at different tide levels. The heave motions from tides are of main importance to the mooring system of the terminal, but the motions are too slow (very long wave) to be considered as dynamic.

A passing ship generates a pressure pulse and a wave system. The pressure pulse is caused by the displacement of the water, which has to flow from the stem to the stern, resulting in a water level depression alongside the ship. The pressure pulses may inconvenience moored ships as the ship may be set into motion.

In [ref. 20]; downtime analysis of an exposed container terminal] information can be found on dynamic forces and operational criteria of a quay wall and apron area. This thesis will certainly be used in the analysis of dynamic loads, especially as the research was done for a case study of the Westerschelde container terminal. Many of the local boundary conditions are examined.
The study deals with a bottom-fixed container terminal however, so that the dynamic behaviour of the structure will be entirely different. A downtime analysis would be too elaborate to perform completely during this thesis, but operational criteria are very important.

9.3 Dynamic stability and behaviour

The dynamic stability and behaviour should be considered for a single floating element and for the entire floating terminal.
The natural frequencies of both floating structures can be calculated. These can be compared to the excitation frequencies from the waves and afterwards, with the results from the hydrodynamic analysis using a program such as DELFRAC. This is only a starting point for the consideration on the dynamic forces and dynamic behaviour.

The first simplified calculations of the Mega Float were done according to the beam theory for the structural analysis, and the hydrodynamic analysis was done according to the strip method. Then, more complex methods were used.

The dynamic analysis of the MOB can be found in [ref. 26].

A clear overview of the motions and dynamic responses of a floating terminal is presented in the figure below:

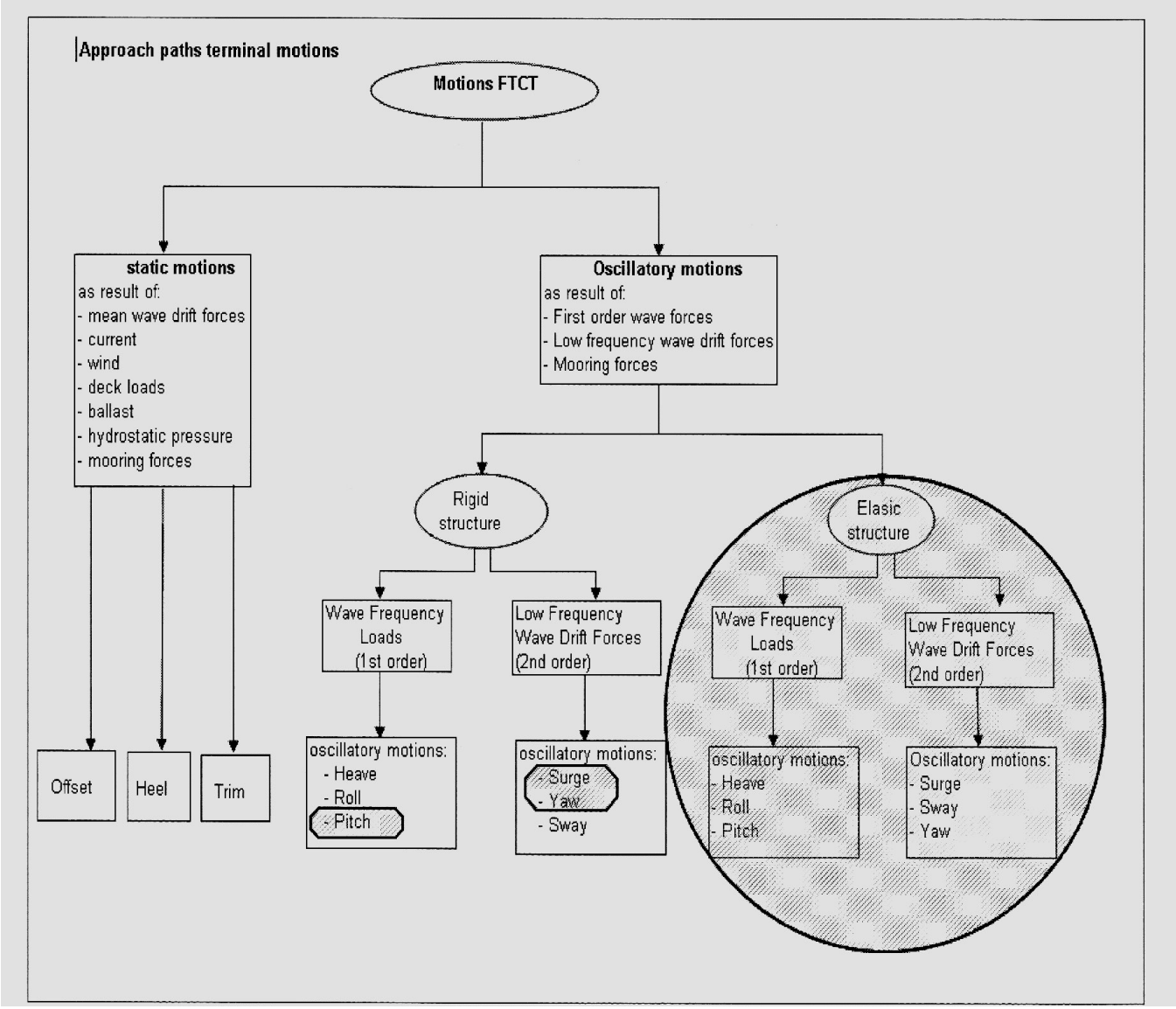


Figure 9.3: Overview of motions of a floating terminal [ref. 22]

When all loads and boundary conditions are clear, the dynamic behaviour of the floating structure can be analysed with software, such as the simulation models SEAWAY or DELFRAC.
Seaway can be used to simulate the response and potential coefficients of floating structures on wave loads. SEAWAY is a frequency-domain ship motions PC program, based on both the ordinary and the modified strip-theory, to calculate the wave-induced loads and motions with six degrees of freedom of mono-hull ships and barges in seaway. Seaway is especially concentrated on single ship hulls.
DELFRAC has more possibilities. It can be used for all kinds of (small or large) floating bodies and even more important, it can compute both single- body and multi-body models. In the multi-body analysis, several bodies can be coupled. Therefore, DELFRAC seems to be the better program to use in this thesis. A study of DELFRAC is done during the hydrodynamic analysis, and a description of the program will be found elsewhere in the appendices.

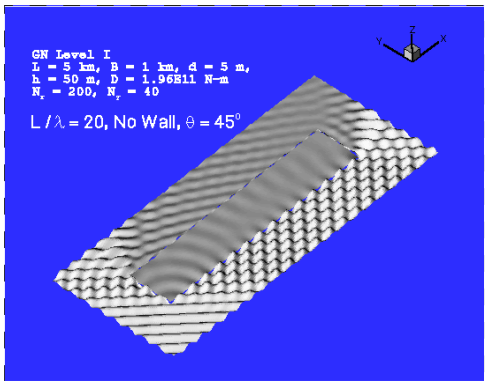


Figure 9.4: An example from dynamic analysis of the Mega Float

9.4 Operating criteria of the terminal

The terminal will be designed for two limit states: The serviceability limit state (SLS) and the ultimate limit state (ULS). During the SLS the terminal is in operation. The ULS is a survival condition, for example a heavy storm. The terminal will no longer be in operation by then, there will probably be no ships at berth. The ULS produces boundary conditions about strength and stability of the terminal; the structure shall have to bear the loads in a connected state and remain afloat.
The SLS produces operational criteria for the terminal regarding:

- Safe berths for ships.
- Workable conditions for employees (sea sickness), both on deck and in the cranes.
- Allowable (relative) motions (amplitudes, angles, accelerations) for a container crane in operation.

Several of the operational criteria mentioned above can be found in [Ref. 22].

The floating terminal will be fixed to the bottom to ensure a horizontally fixed location within boundaries. Movements of the floating structure will only partly be prevented; the horizontal translations are restrained, but the rotations of the floating structure are much harder to control.. In the vertical direction, the structure may be practically free of restrictions in motions.
The relative motions are of great importance; the relative motions of two coupled pontoons, but perhaps even more important, the relative motions between the front pontoons and a berthed ship. Both ship and floating terminal have their own operational requirements, as shown in [Ref. 13] and [Ref.22].

When the operating criteria are crossed, the terminal can no longer operate. This may be a few days per year, according to the defined expected crossing frequency. To find the frequency of non-operation, a downtime analyses can be carried out. This has been done for the (bottom-fixed) Westerschelde Container Terminal in [ref.22]; a Master thesis.

[Ref. 22]: p.9 Operational and technical requirements
(from PIANC 1995: Criteria for movements of moored ships)
The allowable motions are limiting criteria for vessel movements under working conditions:

	Allowable motion amplitudes			
Type of ship	Surge (m)	Sway (m)	Yaw (°)	Heave (m)
Tankers	2-3	2-3	1	1.5
Bulkers	0.5-1.5	0.5-1.0	-	0.3-0.5
Container ship	0.5	0.3	1	0.3
Ro/Ro ship (at the ramp)	0.3	0.2	0	0.1

Allowable accelerations are limiting criteria with regard to operation of equipment, cargo safety, personal safety and efficiency. Accelerations cause loads on cargo and equipment and are an important reason for seasickness:

	Merchant ships	Naval vessels	Fast small craft
Vertical accelerations at forward Perpendicular (RMS-value)	0.275g(L≤100m) 0.05g(L≥300m)[a]	0.275g	0.65g
Vertical accelerations at bridge (RMS-value)	0.15g	0.2g	0.275g
Lateral accelerations at bridge (RMS-value)	0.12g	0.1g	0.1g
Roll (RMS-value)	6.0 deg	4.0 deg	4.0 deg
Slamming criteria (probability)	0.03(L≤100m) 0.01(L≥300m)[a]	0.03	0.03
Deck wetness criteria(probability)	0.05	0.05	0.05

[a] *The limiting criterion for lengths between 100 and 330 m varies almost linearly between the values L= 100 m and 330m.*

The limiting design wave heights are for safe mooring of vessels.
For the tying up of tugboats, a limit H_s = 1,5 m.

	Limiting wave heights H_s in m	
Type of vessel	0° (head or stern)	45° - 90° (beam)
General cargo	1.0	0.8
Container, Ro/Ro ship	0.5	
Dry bulk (30,000-100,000); loading	1.5	1.0
Dry bulk (30,000-100,000); unloading	1.0	0.8 - 1.0
Tankers 30,000 dwt	1.5	
Tankers 30,000 - 200,000 dwt	1.5 - 2.5	1.0 - 1.2
Tankers > 200,000 dwt	2.5 - 3.0	1.0 - 1.5

More extensive motion criteria are:

Ship type	Cargo handling equipment	Surge (m)	Sway (m)	Heave (m)	Yaw (°)	Pitch (°)	Roll (°)
Fishing vessels	Elevator crane LO-LO Suction pump	0.15 1.0 2.0	0.15 1.0 1.0	0.4	3	3	3
Freighters, Coasters	Ship's gear Quarry cranes	1.0 1.0	1.2 1.2	0.6 0.8	1 2	1 1	2 3
Ferries, RO-RO General cargo	Side ramp Dew/storm ramp link span Rail ramp	0.6 0.8 0.4 0.1 2.0	0.6 0.6 0.6 0.1 1.5	0.6 0.8 0.8 0.4 3	1 1 3 - 2	1 1 2 1 2	2 4 4 1 5
Container Vessels	100% efficiency 50% efficiency	1.0 2.0	0.6 1.2	0.8 1.2	1 1.5	1 2	3 6
Bulk carriers	Cranes Elevators / Bucket-wheel Conveyor belt	2.0 1.0 5.0	1.0 0.5 2.5	1.0 1.0	2 2 3	2 2	6 2
Oil Tankers	Loading arms	3.0	3.0				
Gas Tankers	Loading arms	2.0	2.0		2	2	2

Ship size (DWT)	Surge (m)	Sway (m)	Heave (m)	Yaw (o)	Pitch (o)	Roll (o)
1,000	0.6	0.6	-	2.0	-	2.0
2,000	0.4	0.4	-	1.5	-	1.5
3,000	0.3	0.3	-	1.0	-	1.0

Further operational requirements are:

- The floating structure must be statically and dynamically stable. The requirement of stability also applies when two elements of a large floating structure are damaged and partly filled with water.
- The structure must be able to withstand all acting internal and external loads.

10. Structural design of the floating element

10.1 Loads from harbour and apron activities

In general, a floating pier is subjected to the following loads:

1. Deck loads.
2. Wave load during construction, transport and final phase.
3. Local hydraulic pressures.
4. Ship berthing impact.
5. Loads of thermal origin.
6. Loads associated with pier construction, launch and tow.
7. Environmental loads such as wind, waves, ice and current force.

A quay wall or terminal will be designed to withstand the following loads:
Permanent loads from self weight, soil (horizontal and vertical loads) and water (horizontal water pressure and groundwater pressure and upward pressure), ship loadings from mooring, collision and bollard force, and the traffic and storage loads.
This last category is originated from the terminal activities. The storage of containers causes a terrain load, as does the traffic on the pier (transport of cargo). The largest concentrated load is caused by the container crane on a crane track.

Loads are caused by the usage of the structure and therefore they are linked to the structures function. From [ref. 22] a clear overview was taken of the functions of an offshore container terminal:

Functions	Sub-functions	(Possible) Components
Processing containers	- Loading/unloading. - Intern transport of containers. - Storage of containers. - Maintenance	- Cranes. - Chassis, fork lifts, AGV's, etc. - Storage yard, apron area and traffic lanes. - Maintenance workshop
Processing vessels	- Tugging vessels. - Mooring vessels. - Supplying vessels.	- Tug boats. - Mooring lines/fenders - Supply boats/systems
Processing Personnel	- Transport of personnel to/from fast land. - Hosting personnel	- Boat, helicopter, etc. - Offices, restaurants, hostels, etc.
Supplying terminal	- Electricity and light - Fuel supply - Food and drink water	- Light posts, generator station, electricity station. - Fuel tanks - Storage, supply boat
Protection of terminal and vessels	- Against waves, currents and wind	- Terminal (functioning as breakwater). - Additional facilities: floating breakwater, submerged breakwater, conventional breakwater, etc. - Horizontal fixation of the terminal

The system consists of the floating terminal and the vessels calling on it.
In the case of the Westerschelde, a near shore terminal, the hinterland connections and their traffic are also part of the system.

Research into the type of (future) crane and the requirements of the pier may prove useful. While in operation, the crane structure has strict demands on the foundation structure and surroundings. Several of these can be found in NEN-ISO guidelines.
The present largest container crane causes a limited load on the quay wall. That crane was applied for the design of the land-based Western Scheldt Container Terminal. The future Euromax cranes however, cause a tremendous load [ref 29, 41], about 3 to 4 times the load of the WCT cranes. Ref.4 seems to equal the WCT crane data. The old estimations about the crane load being only 2-3 % of the total load on the structure may not be applicable anymore with this crane. The Euromax crane is not yet complete in its design and the dimensions of the wheel foundations are still questionable, therefore it is not yet certain if application of these loads on a floating structure is a wise thing to do.

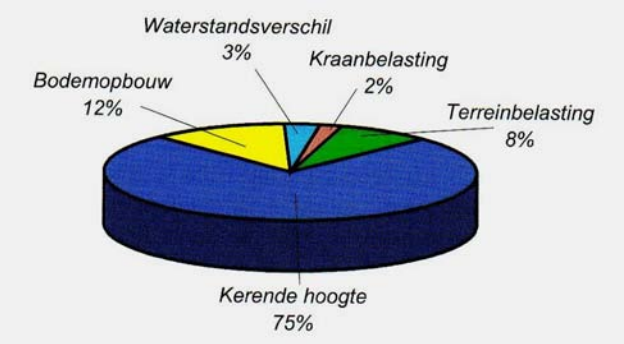


Figure10.1: cost distribution quay wall:
12% soil features
3% water level difference
2% crane loads
8% terrain loads
75% retaining hight

Guidelines for average crane loads are:

Loads for Harbour Cranes

	Rotating cranes	Container cranes and other transhipment gear
Bearing capacity [t]	7–50	10–80
Dead weight [t]	180–350	200–1200
Portal span [m]	6–19	9–45
Clear portal height [m]	5–7	5–13
Max. vertical corner load [kN]	800–3000	1200–8000
max. vertical wheel surcharge load [kN/m]	250–600	250–700
Horizontal wheel load		
transverse to the direction of the rail	up to approx. 10 % of vertical load	
in the direction of the rail	up to approx. 15 % of the vertical load of the braked wheels	
Claw load [1] [kN]	Mobile crane up to 4800	

[1] Prerequisite is a zone of 40 m^2 subject to no other loads; the claw load can be taken as distributed over 4 m^2.

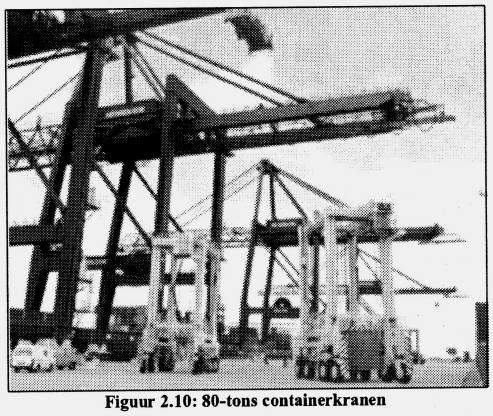
Container crane loads from Euromax [ref. 29, Gemeentewerken Rotterdam]:

Containerkraan.			
Afstand water- landzijdige poten.	30,48 meter (100 voet).		
Afstand tussen buffers.	27,20 meter.		
H.o.h. afstand wielstellen.	17,25 meter.		
Wielen.	8 wielen per poot h.o.h. 1,05 meter.		
Operationele gemiddelde windsnelheid.	25 m/s. [overeenkomstig FEM eisen voor Rotterdam].		
Belasting (rep.) verticaal:	**Per wiel [kN]**	**Hoeklast [kN]**	**Last per meter [kN/m']**
Landzijde verticaal in operatie.	2.500	19.700	2.700
Waterzijde verticaal in operatie.	2.000	15.700	2.200
Landzijde verticaal bij storm.	2.000	16.000	2.200
Waterzijde verticaal bij storm.	1.400	10.400	1.400
Tie-down belasting bij storm	1.700 kN per lash		
Belasting (rep.) horizontaal ⊥ op de kraanbaan:	**Per wiel [kN]**	**Hoeklast [kN]**	**Last per meter [kN/m']**
Landzijde horizontaal in operatie.	30	240	40
Waterzijde horizontaal in operatie.	30	240	40
Landzijde horizontaal bij storm.	90	660	100
Waterzijde horizontaal bij storm.	80	630	90
Belasting (rep.) horizontaal // met de kraanbaan:	**Per wiel [kN]**	**Hoeklast [kN]**	**Last per meter [kN/m']**
Landzijde horizontaal in operatie.	60	430	60
Waterzijde horizontaal in operatie.	50	370	50
Landzijde horizontaal bij storm.	1.900	15.300	2.100
Waterzijde horizontaal bij storm.	1.600	12.500	1.700

The Program of Requirements of the Euromax quay wall [ref.29] mentions the same design ship as does the Program of Requirements of the Westerschelde Container Terminal, namely a maximum capacity of 12500 TEU; the Southhampton ++ or Suezmax class. These ships are larger than the PostPanamax class.

In determining the loads on the apron area, [ref. 4] and [ref.27] will be used, for the storage area also [ref 1 and 9] give useful information.
The elements of the floating terminal would best be designed on the crane loads according to the Program of Requirements of the Westerschelde container terminal. The most important data is presented in the figures below.
Two guidelines are found that can define the cranes' operating criteria and other requirements:

- NEN-EN 15011 Cranes – bridge and gantry cranes.
- NEN-EN 13852-2 Cranes – Offshore cranes – Part 2: floating cranes.



Figuur 2.10: 80-tons containerkranen

Figure10.2: 80 ton container cranes [ref. 4].

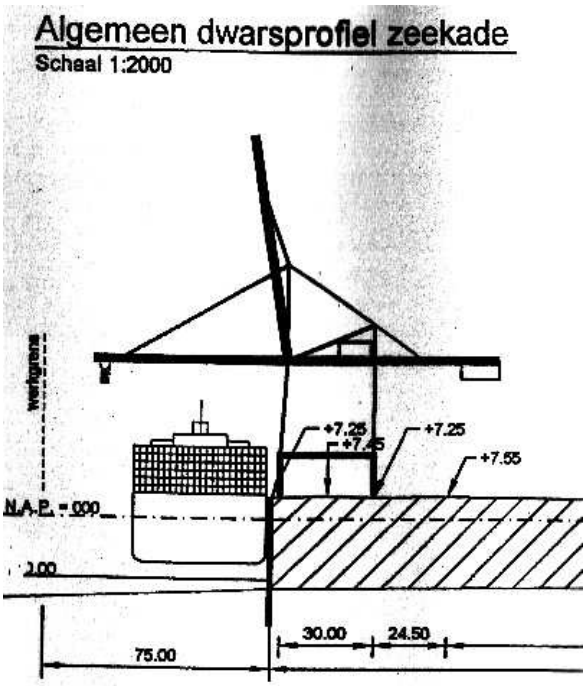


Figure10.3: Cross section of the pier.

Crane loads:
The following tables and figures show the crane loads of the WCT super Post Panamax crane. Also the configuration of the wheel system and of other mobile cranes etc are included [Combinatie WCT, 2003].

Tabel 13: Bepaling maatgevende verticale kraanbelasting

Scenario			Wielbelasting		Pootbelasting		Som	Blokbelasting	
	Maximaal	Wind	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]	[kN/m]
			links	rechts	links	rechts		links	rechts
1	WZ	Geen	675,1	712,1	5401	5697	**11098**	600	633
2	LZ	Geen	288,9	304,8	2311	2438	4749	257	271
3	WZ	Aflandig	620,3	800,1	4962	**6400**	**11362**	551	711
4	LZ	Aanlandig	347,9	197,3	2783	1578	4361	309	175
5	WZ	Parallel	754,3	632,9	6034	5063	11097	670	563
6	LZ	Parallel	194,4	399,4	1555	3195	4750	173	355
7	Geen	Aflandig	186,3	668,9	1490	5351	6841	166	595
8	Geen	Aanlandig	520,6	78,7	4165	630	4795	463	70
9	Geen	parallel	92,3	635,0	738	5080	5818	82	564

Tabel 14: Blokbelasting WZ kraanbaan

scenario	blokbelasting WZ kraanbaanbelasting [kN/m']	
	links	rechts
Aanlandige wind	600	633
Wind parallel en aflandig	551	711

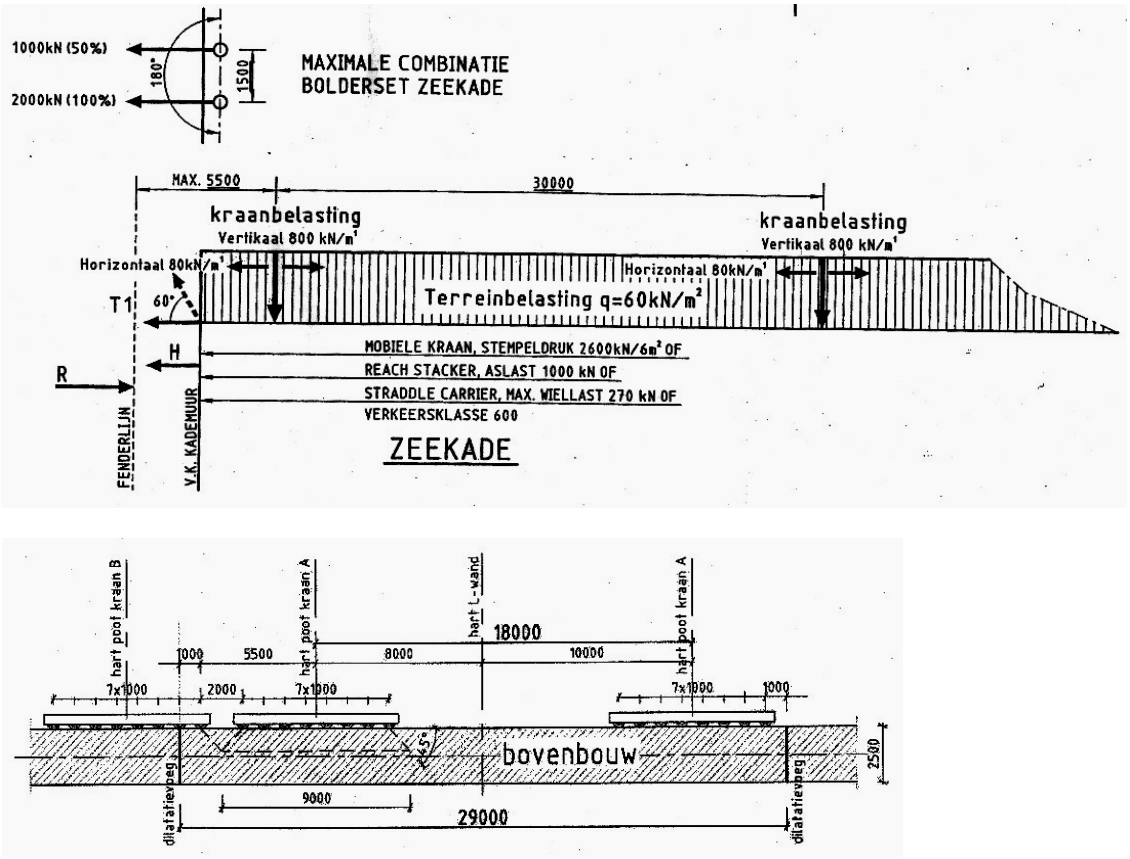


Figure10.4: 80 Loads on the pier.

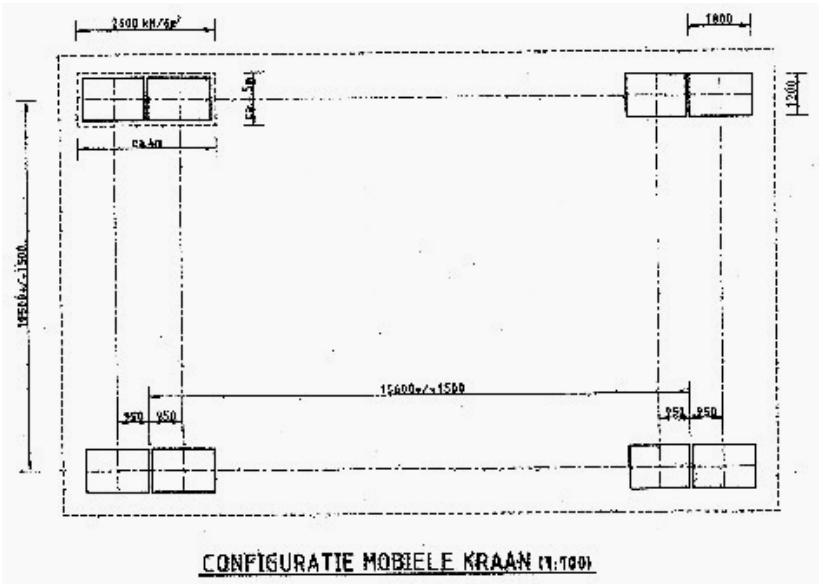


Figure10.5: Configuration of the loads of caused by a mobile crane.

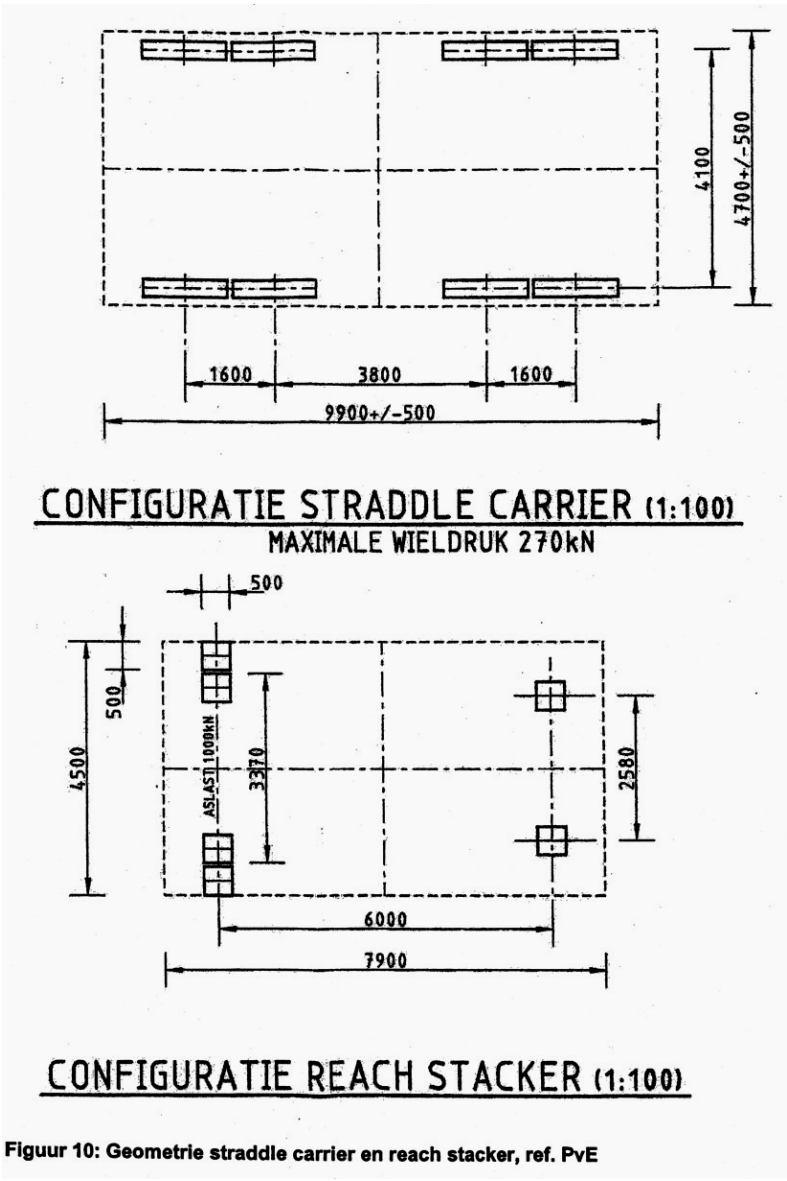


Figure10.6: Configuration of the loads caused by a straddle carrier and a reach stacker.

10.2 Design methods

Mostly, little research is done into the subject of mechanics, force distribution in a pontoon and concrete calculations. As the aim of this thesis is to provide a simple method for a rough conceptual design of the floating element, theories are compared that can be used for a hand calculation of the distribution of internal forces.
The pontoon-shaped structure could be calculated as a 3D structure with the help of FEM-software (e.g. ESA Prima Win), but it is preferred to start with a conceptual hand-method.

The design method to be used should be rather simple and should have firm safety margins. According to the Dutch VBC, the linear, quasi linear and non linear theory of elasticity and the theory of plasticity may be used to design a structure. The following methods are considered:

- Linear theory of elasticity (LE)
- Theory of plasticity
- Method of balance or Strip method
- Strut and tie or Stress field method

The linear theory of elasticity is mostly used for first design calculations. There is a considerable safety margin. The nonlinear and quasi linear theories of elasticity are too complicated to use in a conceptual design. The theory of plasticity is used to optimise a design to economy. An actual failure load is determined and to this load the structure is designed with a smaller safety factor. The actual failure load is found by considering all the possible failure mechanisms of the structure according to the upper bound theorem of the theory of plasticity. The governing failure mechanism is the one occurring at the lowest load on the structure. When using the theory of plasticity, the structure is designed as lightly as possible. There is a chance of designing on the wrong failure mechanism, causing the safety factor to be smaller than one.
The yield line theory uses the theory of plasticity in the design of plates and slabs. It is actually a method for check of strength. The theory of plasticity will not be used for this conceptual design.

The strip method is based on the lower bound theorem of the theory of plasticity. This theorem reformulated: Find a moment distribution which fulfils the equilibrium equations. Design the reinforcement for these moments. The chosen moment distribution does not have to fulfil any other conditions such as the relation between the moment and the curvature, which means that one distribution has to be chosen from an infinite set of possible distributions. As the strip method uses the lower bound theorem of the theory of plasticity, it creates an average between the linear theory of elasticity and the theory of plasticity (as would the nonlinear theory of elasticity). It can be used to optimise to economy, but the safety factors are reasonably large. It is a design method rather than a method to check strength. Reinforcement can be designed practically by averaging it over a large width of a plate or slab, when using the strip method. The linear theory of elasticity would lead to more reinforcement and more locally concentrated reinforcement.

The simple strip method is based on the following principle:
The plate load is assumed to be carried by strips that run in the direction of the reinforcement. No torsional moments acts in these strips. The strips are considered as beams, dividing lines are assumed to be lines of zero shear force in all directions (and thus maximum or minimum moments). Punching is also neglected. According to the strip method, the entire load is carried by bending moments in the direction of the coordinate axes of the plate.

As the strip method is based on the theory of plasticity, the load on the structure is assumed to be a constant, full load. A predominant varying live load or even dynamic load, may complicate using the strip method. It may influence the results somewhat. Also it is true that the plastic properties of a plate decrease with an increasing amount of reinforcement and plate thickness. However, the strip method still seems to provide a safe design. [Ref.]
According to NEN 6720, a plate can be divided in strips to determine the stiffness of the T- or L-beams carrying the plate (or the walls) and to determine the loads on these line shaped supports. The strip method would nicely coincide with this theory and, as it is based on the theory of plasticity, it is allowed by the Dutch NEN 6720.

10.2.1 Comparison of characteristics of design methods

The plate structure is reasonably complicated, as it is a composition of many smaller rectangular fields with a few small holes, rigidly supported. The loads are evenly and unevenly distributed and several concentrated loads occur.

The design calculation can easily be made according to the strip method, but the linear theory of elasticity and the theory of plasticity would also suffice. The linear theory of elasticity will actually be the only theory with which more complicated plates can be designed fairly easy. The theory of plasticity is not suitable for these kinds of structures. Simple plate structures can be designed by all three methods. The following table shows the advantages and disadvantages per method [Ref.CUR report 79; Platen]. “Reinforced Concrete Fundamentals” by P.M. Ferguson deals with the strip method for slab design.

Design method or theory	**advantages**	**disadvantages**
Linear theory of elasticity		
	+standard cases in NEN 6720	-hand calculations are long/difficult
	+fast numerical methods available	-reinforcement plan is relatively complicated
	+Model research gives good results	
	+the method of determining reinforcement from reinforcement moments is very clear	
Theory of plasticity		
	+standard cases are expressed in formulas	-Very long calculation in case of complicated plates
	+ Through the yield line pattern, the internal forces are determined	-limited certainty of choice of right failure mechanism
	+very simple and practical reinforcement is possible	-moment distribution in fracture sections is unknown
	+low amount of reinforcement steel needed	- ending of reinforcement bars difficult to determine with sufficient safety
	+only basic check method of bearing capacity of plates	
Method of balance/Strip method		
	+simple design calculation	- method is less suited to calculate plates without rigid line shaped supports
	+The chosen load distribution determines the moment distribution and therefore the ending of the reinforcement bars	-Minimal reinforcement causes the reinforcement bars to be distributed unevenly.
	+ a simple reinforcement is possible	-The assumptions of the method in load distribution can cause extra reinforcement
		- Method not suited for check of bearing capacity

10.2.2 Introduction of connection forces

The last sub-paragraph showed methods to make a general conceptual design of the floating elements of the terminal. The internal forces have to be considered on a more detailed level as well. At concentrated loads, it is important to find out how the force will be introduced into the hull structure of the floating element. The most important concentrated loads on the floating elements are the connection forces, as they are transmitted through the connectors between the elements. The force distribution would have to be considered locally, using a method of hand calculations. As it is still a conceptual design, no Finite Element Models will be used.

The strut and tie method (STM) or stress field method may bring a solution here. The basis of the strut and tie method is to define internal forces in tension and compression (ties and struts) and to (in reinforced or prestressed concrete structures) specify steel in areas of tension.
The major steps in the Strut and Tie Method design process are visualized in the figure below:

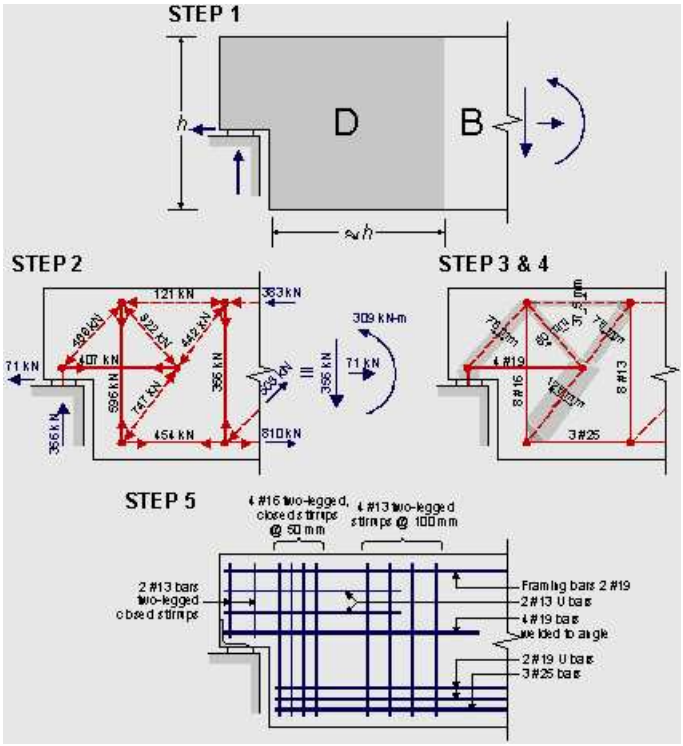


Figure10.7: Five major design steps in STM

Some theory on the method has been found in several articles, but most information has been found in [Ref.: Design of structures with stress fields by Muttoni, Schwartz and Thürlimann]

…

There are several programs available either based on the Strut-and-tie method or the stringer-panel method (which is an average between STM and FEM), but for this thesis, hand calculations are applied.

10.3 Durability of marine structures and the materials used

Engineers have often chosen steel as building material for floating offshore structures because of the weight and therefore buoyancy characteristics of the structures [ref. 1]. Steel can be very well protected in a marine environment, but maintenance is costly. More concrete or hybrid structures, being a combination of steel and concrete) have been put into practice [ref. 1].

Ref 3; Marine structures engineering: Ch.1
There are five distinct zones that concern marine structure deterioration: atmospheric, splash zone, tidal zone, zone of continuous immersion and seabed zone..
A marine environment is one of the most severe and aggressive on earth. Despite general evidence of long-term durability of concrete structures in a marine environment and their generally outstanding performance in the ocean, in some cases serious concrete deterioration has been reported.
The general cause of deterioration was noted to be cracking of concrete, resulting in corrosion of embedded reinforcing steel. Corrosion occurs in the presence of oxygen, moisture and an electrolyte.

Damage to a structure is done by mechanical actions, chemical and biological actions from the environment or physical actions. Degradation of concrete structures can be caused by the presence of (exoansive) salts, acids or by the corrosion of the reinforcement due to several causes.
[Ref. Durability of engineering structures by Jan Bijen]

Causes of deterioration (mostly cracking) are as follows:
1. Chemical reactions such as acid attack, alkali-silica reactions, aggressive water or sulphate attack.
2. Construction errors such as adding too much water to the mixture, improper consolidation or curing, moving the formwork too soon or occurring vibrations on the young, unhardened concrete.
3. Corrosion of embedded metals (e.g. from salt in the aggregates, but also carbonation and pitting corrosion due to chlorides).
4. Design errors.
5. Erosion corrosion by ice and sediment movements.
6. Freezing and thawing.
7. Settlement and movement.
8. Shrinkage.
9. Temperature changes.
10. Corrosion of the reinforcement

[Ref. 3,p.27] Damages from dock operation can be attributed to the following factors: ship impact forces exceed design values at docking, effects of cargo handling and hauling systems, ship propeller induced scour which may threaten to undermine the dock structure, or a combination of all these.

Corrosion of the reinforcement in concrete can be caused by:
- Passivation of the alkaline concrete environment
- Chloride ingress causing pit corrosion
- Carbonation causing uniform corrosion
- Corrosion due to stray currents
- Stress corrosion (in case of high tension stresses, such as in prestressing steel).

According to *[CUR-report 215: Duurzaamheid mariene betonconstructies]* the most potential deterioration mechanism in concrete marine structures is chloride ingress, as was concluded from test results from several Dutch marine structures.
Expectable deterioration mechanisms are carbonation, leaching, alkali-silica reactions, freeze-thaw action, erosion or salt crystallisation.

[Ref. 26]- Part 6 deals with materials and maintenance in floating marine structures. This information may be more extensive or more up-to-date than the information acquired from ref, 1, 2 and 3. Information especially on the durability of concrete should best be found in [ref. 47].



Figure 10.8: Corroded sheet piling in the port of Rotterdam [ref. 4]



Figure 10.9: Erosion corrosion by water with sediments [ref.2]

11. Construction methods

There are four phases in the lifetime of a floating element:
1. Construction of the concrete elements in a dry dock or at site
2. Transport and coupling of the pontoons at site
3. Operational phase of the terminal
4. Disassembling the terminal in order to move it to a new location or for demolition

The following aspects have to be taken into account for the construction methods of the floating elements:
Location: [ref. 12]
- construction in a ship dry dock (fixed or floating).
- Use an existing construction dock or construct a large new dock adjacent to a waterway.
- Construct the pontoons at the site of the final location of the terminal.

The dimensions of the construction dock in Barendrecht are 350 * 150 * 10 m. Many elements of submerged tunnels have been constructed there.



Figure 111.1:Mega Float element in shipyard and towed to site

Concrete construction methods [ref.1]:
- Casting of the whole pontoon at the site of the dock traditionally.
- Apply prefabricated walls on an in-situ casted floor, cast the conections.
- Apply prefabricated deck plates, cast the connections.
- Use a sliding formwork and cast the whole pontoon in situ in the dock.
- Use modern formwork systems to cast as many walls as possible at once with minimal work.

It is important to find a good cycle in the construction of reinforcement, formwork and casting of concrete, because this cycle will improve speed and quality of the work as the learning aspect increases.

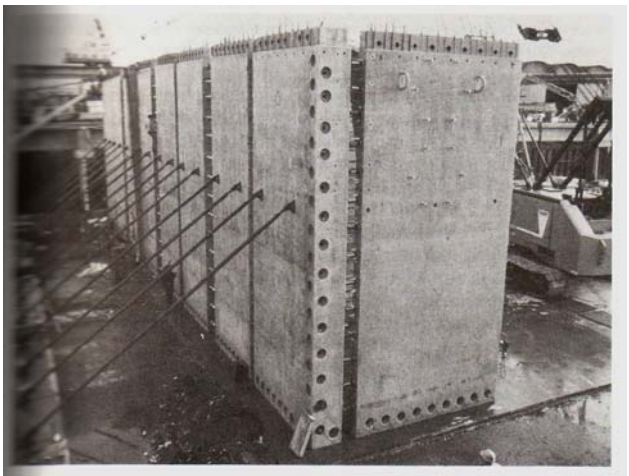
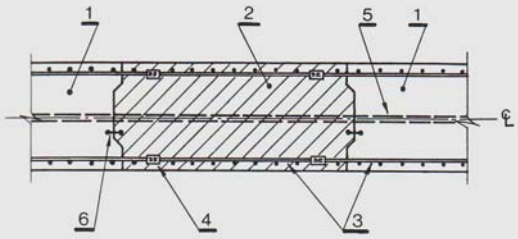


Figure 3-19. Valdez, Alaska, container terminal joint between two precast panels. (Courtesy of Concrete Technology Corp.).



1—precast panel
2—cast-in-place concrete
3—rebars
4—coupling
5—post-tensioned tendon
6—standard waterstop

Figure 3-20. Typical joint between two precast panels.

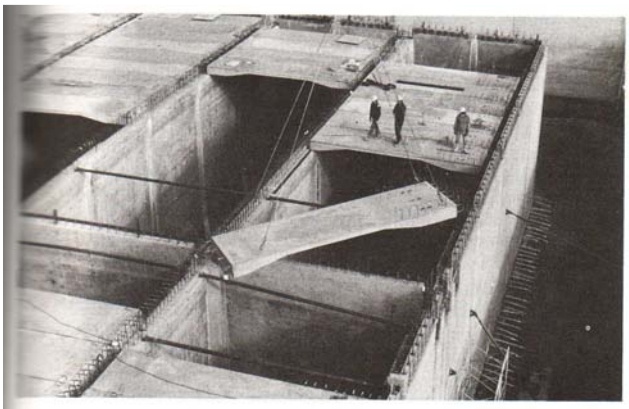


Figure 3-18. Valdez, Alaska, floating pier for container terminal under construction at Concrete Technology, Inc., dry dock, Tacoma, Washington. (Courtesy of Concrete Technology Corp.)

Figure 11.2: Construction of concrete pontoons for the Valdez floating container terminal; with prefabricated walls and deck plates [ref. 1]

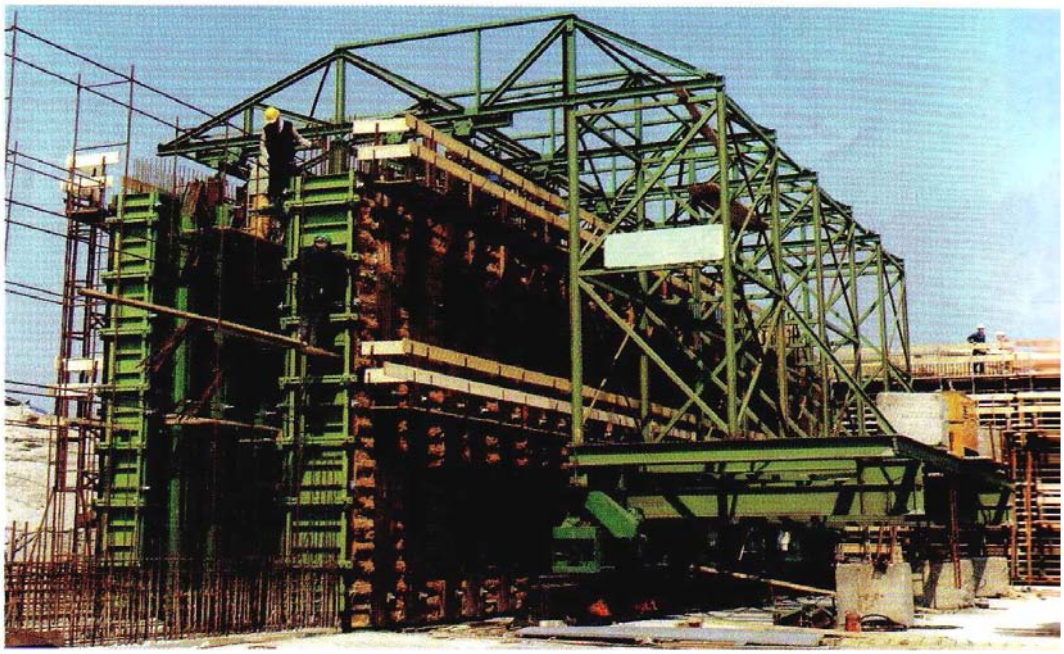


Figure 111.3: A sliding formwork

[Ref. 26] – Part 5 considers the construction of floating platforms. This information will be useful.

12. Connections of floating structures

A connection between the floating elements of the terminal will be developed. The pontoons should be relatively easy to couple and to disassemble. Depending on the design, the connections should be either as rigid as possible, or flexible and even hinged in a certain direction.

One of the aims of this literature survey was to make an inventory of the possible connections between floating pontoons or caissons, either hinged or stiff. This proved to be difficult, as there are only a few working connections and those are mostly constructed in steel. Hardly anything could be found about non-traditional (as in a submerged tunnel) concrete connections between floating objects. The concrete floating elements of the terminal will bear resemblance to the MOB and Mega-float elements. The alternatives of connection of the Mobile Offshore Base are considered to be most promising for a new kind of connection of the floating Western Scheldt Container Terminal.

Types of rigid connections (the figures are presented below):

A. From a European concrete submerged tunnel: tooth connection, stiff and wet-formed (concreted).
B. From an American submerged tunnel; a steel bulkhead with holes outside and connectors.
C. Tensioned together by cables and concreted [ref. 16].
D. Steel pontoons tensioned together by cables [ref. 45], small marinas.
E. Mega Float system; the steel pontoons are welded together.

The connections listed above, can be divided into several connection principles:

1. a “wet-formed” connection, constructed after coupling
2. tensioning
3. a steel, plastic or other cover around the elements
4. “glue” two elements together, either by welding, magnetism or vacuum

The first type of connection will be hard to disconnect again. The other rigid connections may be applicable for the new terminal, although material will be lost when the terminal is disassembled.

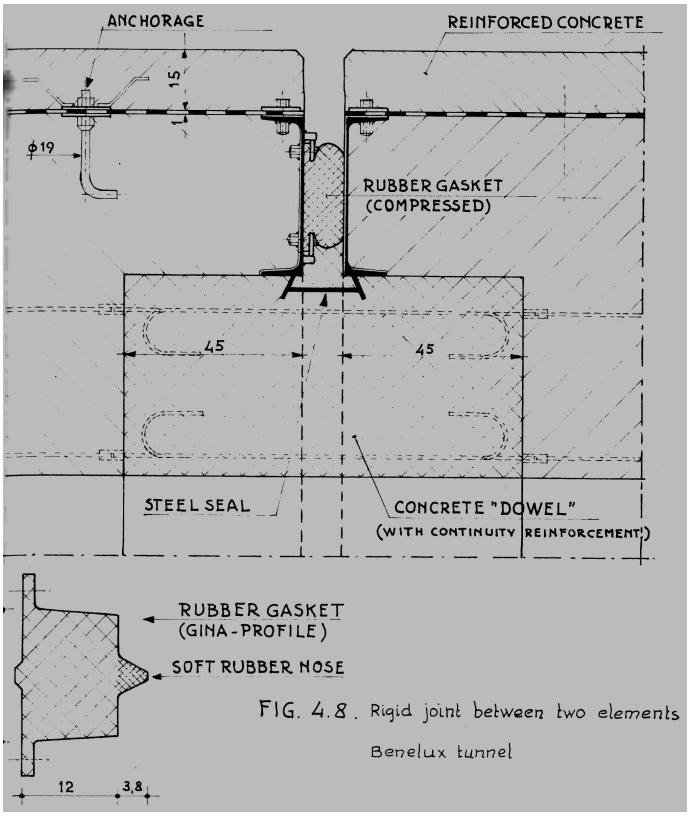


Figure 12.1 a-b: A. stiff (concrete casted) joints of submerged tunnels in the Netherlands [ref. 12, 44]

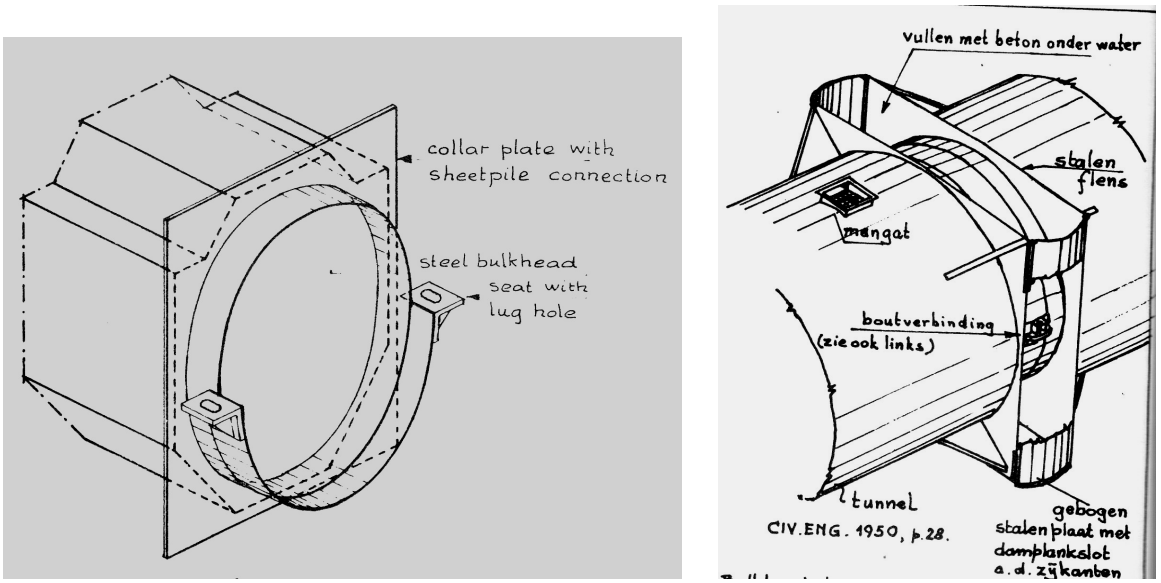


Figure 122.2: B. a rigid connection of an American submerged tunnel; concrete is cast in between the steel collar plates [ref 44]

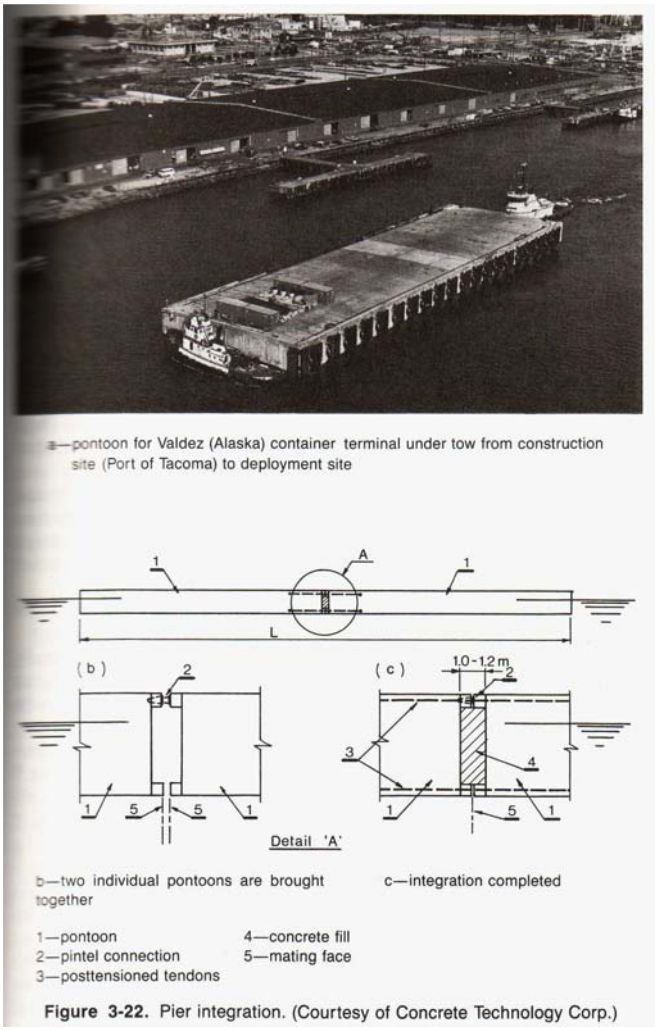


Figure 122.3: C. pier integration of concrete pontoons, Valdez [ref. 1]

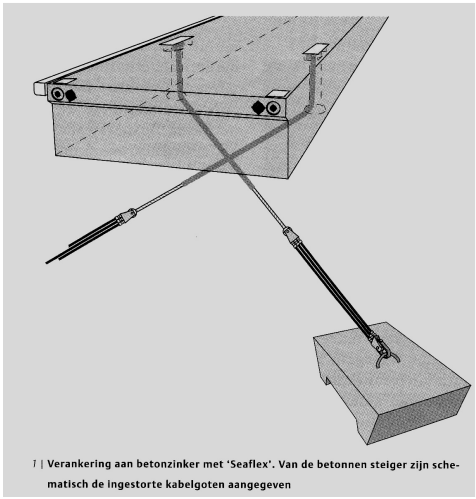


Figure 12.4: D. tension-cable system on concrete pontoons, anchored with a "Seaflex" system



Figure 12.5: E. Coupling of Mega Float

[Ref. 16] is a Master thesis from 1972 about a floating airport in the Netherlands. The structure would be made out of concrete elements with the following dimensions: 2 x 2 x 2 m, filled with polystyrene. Those pontoons would be tensioned together with prestressing cables and the joints would be cast in concrete, so that a connection is created. The dimensions of the structure are very different from what is expected from a floating terminal and many demands are set for the functioning of the connectors. This thesis provides little information usable, as it is mainly a calculation of a concrete pontoon and few other matters are considered. However, the clamping construction with coupling bars at the casted connections could be interesting. Also it is stated that the anchorage could be done with pontoons (gravity structure) and that floating breakwaters would only function rightly in short waves. The long waves seem to cause a bending moment larger than the airplanes.

Types of flexible connections are (figures below):

A. Tug-and-barge (ocean faring ships)
B. Flexifloat
C. Several small pontoons systems for steel and plastic.
D. Mulberry system:
   The Mulberry pontoons were independently anchored to the sea bottom by piles. They were connected to each other flexibly, chains would be most probable.
E. Mobile Offshore Base: compliant connectors, but some alternatives are more flexible than others. These are discussed in the next sub-paragraph.
F. A ball-joint such as of the gates of the Maasland barrier.

These connections are based on the following principles (figures below):

1. A clicking connection with studs or a circular bus, with or without a locking device. The clicking connection mainly bears compression, while the lock can bear tension. These large steel connectors are expensive
2. A kind of piano hinge, either horizontal or vertical.
3. A “knitting needle” connection.
4. Connection with external ropes, lines, chains (based on a moored or towed ship).

Lines and ropes are mainly used for temporary connections,

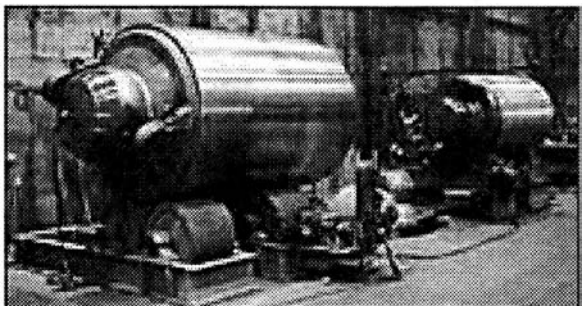



Figure 122.6: A,1. tug-and-barge system for sea ships; ship and coupling structure

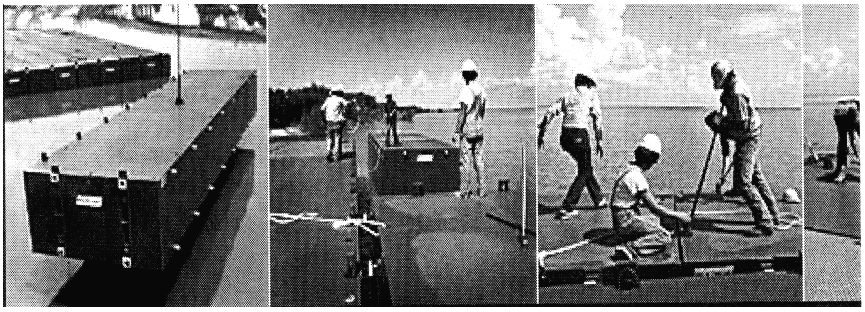
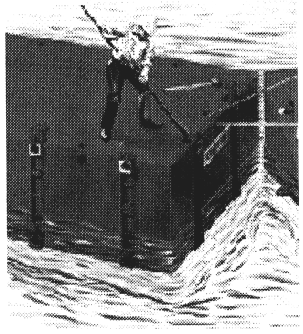



Figure 122.7 a,b : B,1. Flexifloat system; hinged

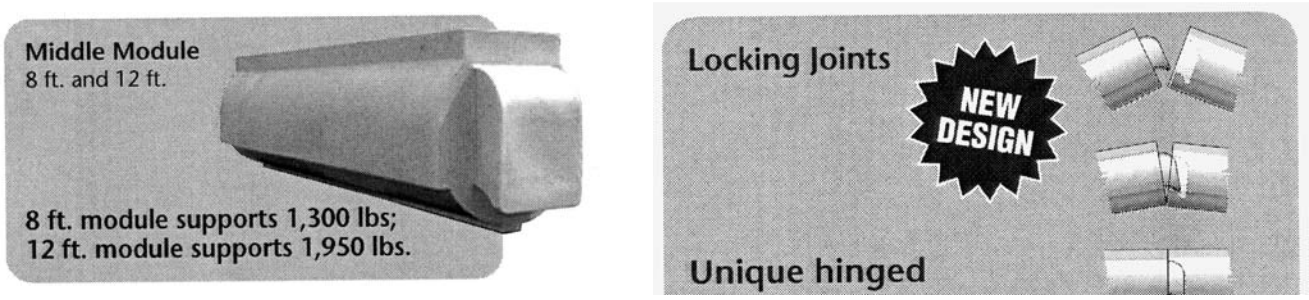
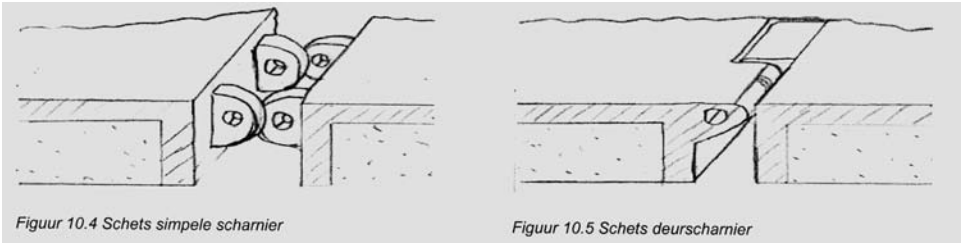


Figure 122.8: C,1. plastic pontoon hinged system with studs; small marinas

In [ref. 17] a few ideas on possible hinged and rigid connectors of pontoons are mentioned. These will be taken into account, but they can only be used if elaborated. The pontoon structures are rather different from the aimed structures in this thesis, so further information from ref. 17 will probably not be used.



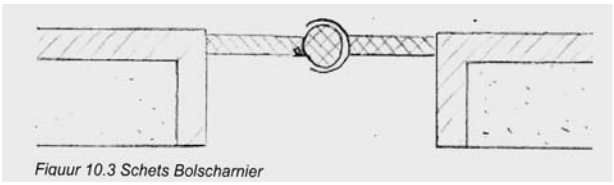


Figure 122.9: 2,3 "knitting needle" connection and piano hinge; Three types of hinges [ref. 17]

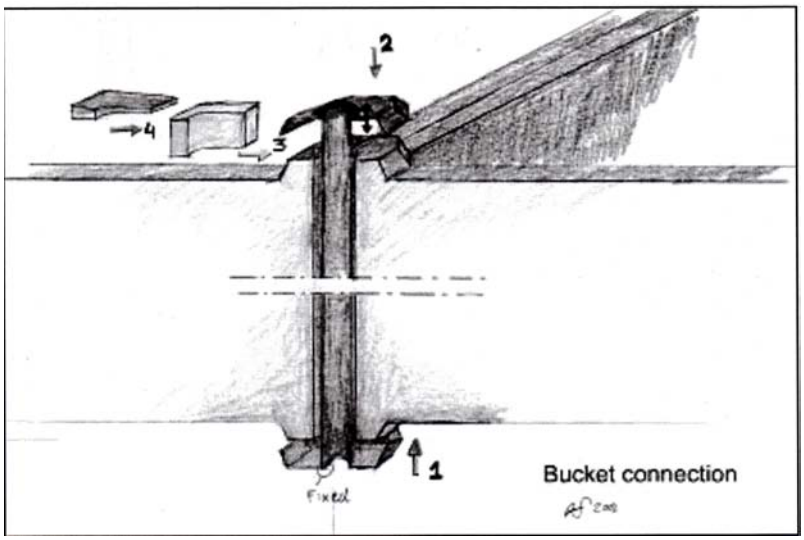


Figure 122.10: C. Hinged connection between floating greenhouses [ref. 31]

12.1 Comparison between WCT and MOB

Similarities:
- Both projects concern a Very Large Floating Structure
- Both projects need connectors larger than present industry
- MOB is originally designed as a floating airport, but can also be used as a floating port; WCT is also a floating port

Differences:
- MOB is built up out of semi-submersibles; WCT elements will probably be pontoon shaped.
- MOB is designed for the most severe environmental conditions; heavy sea states on the Atlantic ocean including hurricanes and typhoons
- WCT is designed for the location in a benign wave-climate of coastal waters
- MOB is optimised for deep waters
- WCT is situated in reasonably shallow waters of ca. 20 m depth
- MOB is optimised to be used in a location anywhere for short duration and be disconnected and moved fast; WCT will stay in its location for years (semi-permanent structure) before being moved only once or twice during its life time
- MOB has larger elements, with a larger height
- WCT has a larger area

Connection forces are for the main part cause by wave loads (per unit length), the dynamic response motions and the anchorage forces from the mooring system of the floating structure/
Th Mobile Offshore Base concept is made from semi-submersibles, which have a small water plane area during operation. The wave loads per unit length will therefore be smaller than the wave loads working on a pontoon shaped structure. The semi-submersible will then experience less wave-induced motions. The connection forces will increase with the dimensions of the element, as the wave load on an element increases. These forces will be distributed over a number of connectors. The larger the number of connectors, the smaller the connection forces per connection.

There are five alternative MOB concepts, which can be characterised by their main particulars:

Main particulars Per concept	**McDermott McMOB**	**Kvaerner SeaBase**	**Bechtel IMMOB**	**Aker Concrete MOB**	**Brown&Root**
Type of element	Semi-submersible	Semi-submersible	Semi-submersible	Semi-submersible	Semi-submersible
Size of element			Very large		180 m (small)
Number of elements	5	3	3	4	6
Material	Steel	Steel	Steel	Concrete	Steel
Deck type		Flattest and straightest of all	Continuous, but vertical bumps	Steel deck	
Connectors	Flexible, compliant; centerline ball joint	Flexible truss bridges damp resonance, fixed conn	Soft; or independent element	Flexible; elastomer and post-tensioned cables	Semi-rigid, at 4 points
Location of connectors	Above waterline; central				2 above and 2 below waterline
Allowed motions	All rotations		All; small due to inertia		
Connection forces	reduced	small			Too large
Machinery	Propulsion + 5*DP		Propulsion + 3*DP		DP 2 elements

[Ref. Ir. J. van Kessel]

Especially the concrete Aker-concept is interesting to compare with the WCT design. However, other concepts may also have usable connector concepts.

Another Very Large Floating Structure is the test Mega-Float airport in Japan. This floating test structure was built from steel pontoons, constructed in ship docks and welded together on site (floating). No connectors were designed.
The following is a summary of the method of construction:
1. Producing steel unit in ship yard
2. Transporting the unit to site by tugboats
3. Mooring unit to dolphin
4. Drawing the new unit to the already assembled part
5. Connecting and locking the new unit onto the assembled part
6. Evacuating water from the space around the weld joining the units
7. Welding

As welding is not an option for the large concrete elements of the WCT, the Mega-Float is of no real interest to this thesis study.

-Several figures from MOB connection technology are presented at the next page-

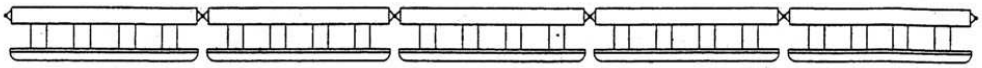


Figure 1: McDermott MOB Concept

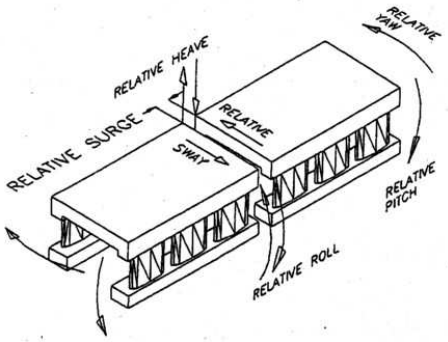


Figure 4. MOB Joint Degrees of Freedom

Table 2. MOB Joint Requirements

MOB Joint DOF	Requirement
Relative Surge	Soft
Relative Heave	Stiff
Relative Sway	Stiff
Relative Pitch	Soft
Relative Yaw	Stiff
Relative Roll	Stiff

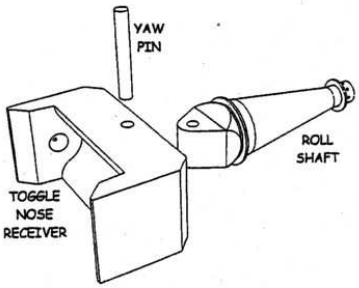


Figure 5: Centerline Ball Joint Detail

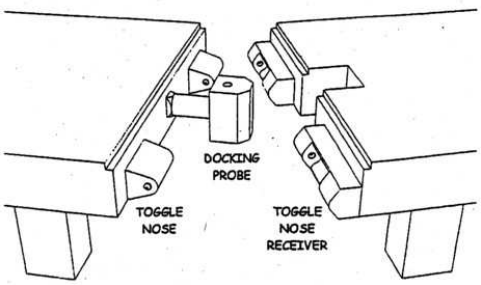


Figure 2: Simple Hinge Concept

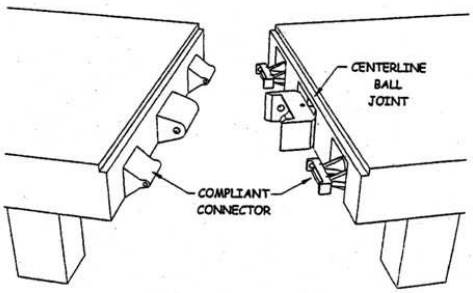


Figure 4: Compliant Connector Arrangement

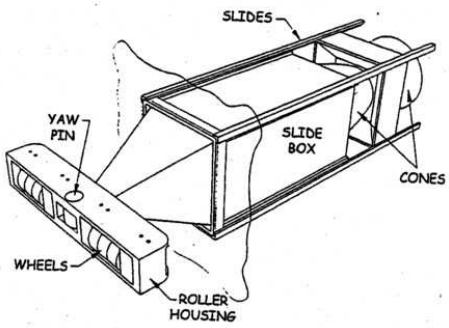


Figure 9: Roller Connector

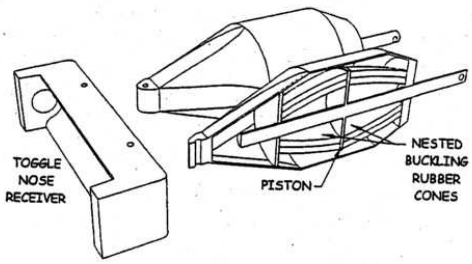


Figure 6: Compliant Connector Detail

Fig 12.11: Original MOB and more recent rigid hinge connectors by McDermott; docking centre ball joint and two outboard compliant hinges

Conclusion - Evaluation of the literature survey

This Master Thesis will be concentrated on the design of a floating terminal, under the circumstances of the case study of the Westerschelde Container Terminal.
A more conceptual view on an offshore deepwater port will be developed.
This means that the aspects and assumptions concerning an offshore floating object, mainly about hydromechanics, will be initially limited to the simpler case study of a floating terminal near shore and in partly protected, shallow waters.

The main features of the thesis will follow a design process of a floating structure A number of important subjects will be explored more elaborately and more theoretically. These subjects are the hydromechanics, the behaviour of flexibly or rigidly connected elements, the structural design of a front element bearing the crane load and the design of a connection between the elements of the terminal. A few other subjects are studied less extensively. These are the layout and operation of the terminal, the hinterland connections, an environmental analysis and, though considered during the design of the terminal, the durability of the structure. The mooring systems will be considered during the planned preliminary study into hydrodynamics, but will not be designed itself.

To summarize, the table below presents whether enough information has been obtained or not per chapter of the literature study:

Chapter	Information available	References, remark
2. Applications of floating structures	Sufficient, as introduction	[ref. 1,2]
3. State-of-the-art on floating structures	Sufficient, as introduction	[ref. 1,2,15-18,22,25,26
4. Floating structure design considerations	Sufficient, should be integrated with Ch.14	[ref. 1,2,4]
5. Hydromechanics of a floating structure	Preliminary study needed, a lot of information available	[ref. 5-8,10,11,20,22,25, 26,31-40]
6. Functional harbour layout and logistics	Enough info, but little on a floating terminal	[ref. 1,2,4,9,13]
7. Connections to the mainland	Sufficient, but not for every case study	[ref. 1,2] + websites
8. Mooring systems; fixation	Various systems, but little on a‘tower-shaped’ dolphin structure	[ref. 1,2,10,22]
9. Loading from apron activities	Sufficient in loads, but little on crane requirements on VLFS	[ref. 4,15,19,10,27,28, 29]
10. Durability of marine structures and the materials used	Sufficient	[ref. 1,3, expected 48]
11. Environmental requirements and consequences	Sufficient for this thesis	[ref. 13,24] website
12. Construction methods	Sufficient	[ref. 1,2,12,26
13. Connections of floating structures (connections between floating pontoons)	Many steel systems, little on concrete; hinged or partly rigid and removable	[ref. 1,2,12,16,25,26, 32,34,35,44,45] +websites
14. Design concrete element	More info needed on several hand methods	[ref. 58-61]+websites, expected [ref. 49-57]
15. Natural boundary conditions and physical features of case studies	Sufficient in WCT, more specific info needed for other case studies	[ref. 3,19,20,23,27,28,29

One of the boundary conditions of the case study is the concept of a container terminal, rather than another type of cargo. Therefore, initially only a container terminal will be considered. As many of the loads for a container terminal are larger than for other cargo, the terminal design may well do for other cargo types, though the requirements of the terminal might be different in each case.

References preliminary study

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alternatieven voor de zeekade		
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[42] Korte Golven	Prof. Battjes	Lecture notes ct4230, Delft University of Technology
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[46] Ship hydromechanics – part 1: Introduction	J.M.J. Journée and Jacob Pinkster	Lecture notes mt519, Delft University of Technology
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Title	Author	Publisher or remark
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[50] Plates and slabs	Prof. dr. ir. Blauwendraad	Lecture notes ct 3130, Delft University of Technology
[51] Structural dynamics part 1 Structural vibration	Prof. dr. ir. Blauwendraad	Lecture notes ct 4140, Delft University of Technology
[52] Constructief ontwerpen in beton	Ir. W.C. Vis & ing. R. Sagel	Enci media, 4th edition 1997
[53] Voorgespannen beton	Prof. dr. ir. J.C. Walraven, ir. J.C. Galjaard	Betonpraktijkreeks, ENCI media
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Internet address	Site owner or Author	remark
www.go2war2.nl		Mulberry harbours
www.historic-uk.com		Mulberry harbours
www.flexifloat.com	Flexifloat	Flexifloat pontoons
www.intercon.com	Intercon	Tug-barge couplers
www.plasticpontoon.com	Wilson distribution, Inc.	
www.globalsecurity.org/military/		MOB
www.dt.navy.mil/	USA navy	MOB
www.pianc-aipcn.org		
www.worldcargonews.com		
www.nmri.go.jp/ocean/		Mega Float
www.jfe-holdings.co.jp/		Mega Float
www.mlit.go.jp/english/maritime/mega_float.html		Mega Float
www.srcj.or.jp/html/megafloat_en/		Mega-Float
http://web-japan.org/atlas/technology/		Mega-FLoat
www.dianet.or.jp/Mega-Float/		Mega-Float
www.porttechnologies.org		
http://www.mechanics.citg.tudelft.nl/spancad/spm.html:		Stringer-panel method
www.cee.uiuc.edu/kuchma/		Strut and Tie method
www.geocities.com		Strut and Tie method
www.amfibischwonen.nl		

Codes, standards and guidelines to be used:

Code	Title	Publisher or remark
NEN 6702	Belastingen en vervormingen	
NEN 6720	Voorschriften beton TGB 1990	
PIANC	Criteria for movements of moored ships, a practical guide	Report of working group 24, 1995
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NEN-EN 13852-2	Cranes – Offshore cranes Part 2: Floating cranes	Nov. 2004

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Ir. J. van Kessel; PHd at the Faculty of Mechanical Engineering and Maritime Technology, Delft University of Technology
Ir. I. Drummen; PHd at Trondheim University, Norway
Ir. G. Lagers, associated professor Offshore Engineering, Delft University of Technology

A Very Large Floating Container Terminal -

feasibility study

Application possibilities for the situation in the Netherlands

Final report

APPENDICES

Master Thesis G.V.P. de Rooij

TU Delft

Van Hattum en Blankevoort

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Appendix Part 1:

Case Study Western Scheldt Container terminal (WCT)

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1. Location of the Western Scheldt Container Terminal

Location screening for a land-based container terminal:

Alternative locations were selected, about 5 years ago, along the Western Scheldt. Three locations seemed most promising for development:

1. Reorganising the Quarles terminal

An existing general cargo, roll-on roll-off (and neo-bulk) terminal, originally developed for container cargo. The terminal will have to be enlarged with terrain of the Bijleveld harbour basin to the east. The terminal would have an area of 135 ha and a deep-sea quay length of 1775 m.

The Quarles terminal lies within the existing harbour; there is no area available for a floating platform, which would be useless at this location.

2. Nieuwland terminal

The Nieuwland harbour is the most western part of the existing port area and still offers space for a new terminal. The location borders the same harbour basin as the Quarles terminal. Land reclamation combined with the excavation of a new harbour basin will provide a small terminal, which does not meet the requirements regarding size and capacity. Within the existing harbour basins, it can be assumed there is insufficient space available and use of a floating terminal cannot be justified.

3. Western Scheldt Container Terminal

According to the existing design of the WCT, the terminal can be created by large land reclamation in the Western Scheldt of 120 ha, directly bordering on the existing port area.

The space available is nearly 3 km in length, between the eastern harbour dam and the cooling water discharge canal from the nuclear plant at Borssele. The exact direction and shape of the terminal can be changed if necessary, in relation to the environmental conditions of the location.

There is sufficient area available in the Western Scheldt and the location is easier to navigate for ULCS coming from the North Sea, than the other alternatives. The dimensions and capacity at this location can meet the requirements of the new container terminal. Another advantage of the location is that it is not an existing harbour area, so the filling in of the area is still open.

The terminal would be rather exposed to waves coming from sea.

However, it also lies next to a deep navigational channel and still rather near to the river mouth.

Total area 120-150 ha + 30 ha existing harbour area on land

Deep-sea quay length 2615 m to be created in the Western Scheldt

Depth NAP -19,0 m

Inland shipping quay length 350 m + dock (harbour basin)

Construction time 4-6 years

Complication: European environmental rules and regulations on nature reserves.



Figure 1.1: Location of the Western Scheldt Container Terminal

The research on the traditional design of the Western Scheldt Container Terminal is based on a few important documents:

- Startnotitie Western Scheldt Container Terminal
- Integrale eindnotitie
- Variantnota Containers in Vlissingen PMR- Zuidwest Nederland by Rijkswaterstaat, directie Zeeland
- Program of Requirements by Zeeland Seaports

2. Summary of Program of Requirements

Throughput and shipping traffic
A minimum design throughput of 150 million containers per year is expected.
Number of Twenty feet Equivalent Unit boxes : $75*10^6$
Forty feet Equivalent Unit boxes: $75*10^6$ FEU (= 2TEU*$75*10^6$ = $150*10^6$ TEU)
This leads to a total design throughput of $225*10^6$ TEU /year.

$$\text{Teu-factor } f = \frac{N_{20'} + 2.N_{40'}}{N_{tot}} = 1,5$$

Functional requirements
Area requirements:
Area terminal (storage yard) : 136 ha. (in total ca. 150 ha) by land reclamation
Total terminal area : 180 ha. [Hesse-Noord Natie]
Including internal transport infrastructure, without harbour dock
Emergency storage should be possible on the pier.

Berth facilities:
number of deep-sea berths : 6
Number of barge berths : 5 feeders/inland ships + 1-2 tugboats

Quay dimensions:
Sea quay
$L_{sea\ pier}$: 2615 m
From which ULCS (deep-sea): 2500 m
Depth requirement ULCS : LLWS – 17,5 m
Bottom level : NAP – 20,0 m

Inland shipping harbour
L_{pier} barge quay : 300 m at LLWS – 9,30 m
Bottom level : NAP – 12,0 m
L_{pier} barge quay : 440 m at LLWS – 5,70 m

L_{pier} tug boat quay : 190 m (variable depth)
Bottom level : NAP – 12,0 m

General configuration of the quay:
Existing ground level : NAP+5,20 m
Topside quay wall : NAP+7,25 m
Lifetime : minimum 50 years and safety class 3

Equipment:
Number of container cranes: 16 (super-post Panamax) with lift capacity of 80 tons = 800 kN

Operational and technical requirements
Motions, accelerations etc. are dealt with in another chapter.

Economic requirements
- Full exploitation, optimise revenues
- Possibility for future expansion

Site location requirements

- The conditions on site should not lead to unacceptable downtime of the terminal.
- Economic potential
- Available area should be sufficient
- Minimal dredging, maximal water depth

The floating terminal locations have already been discussed.

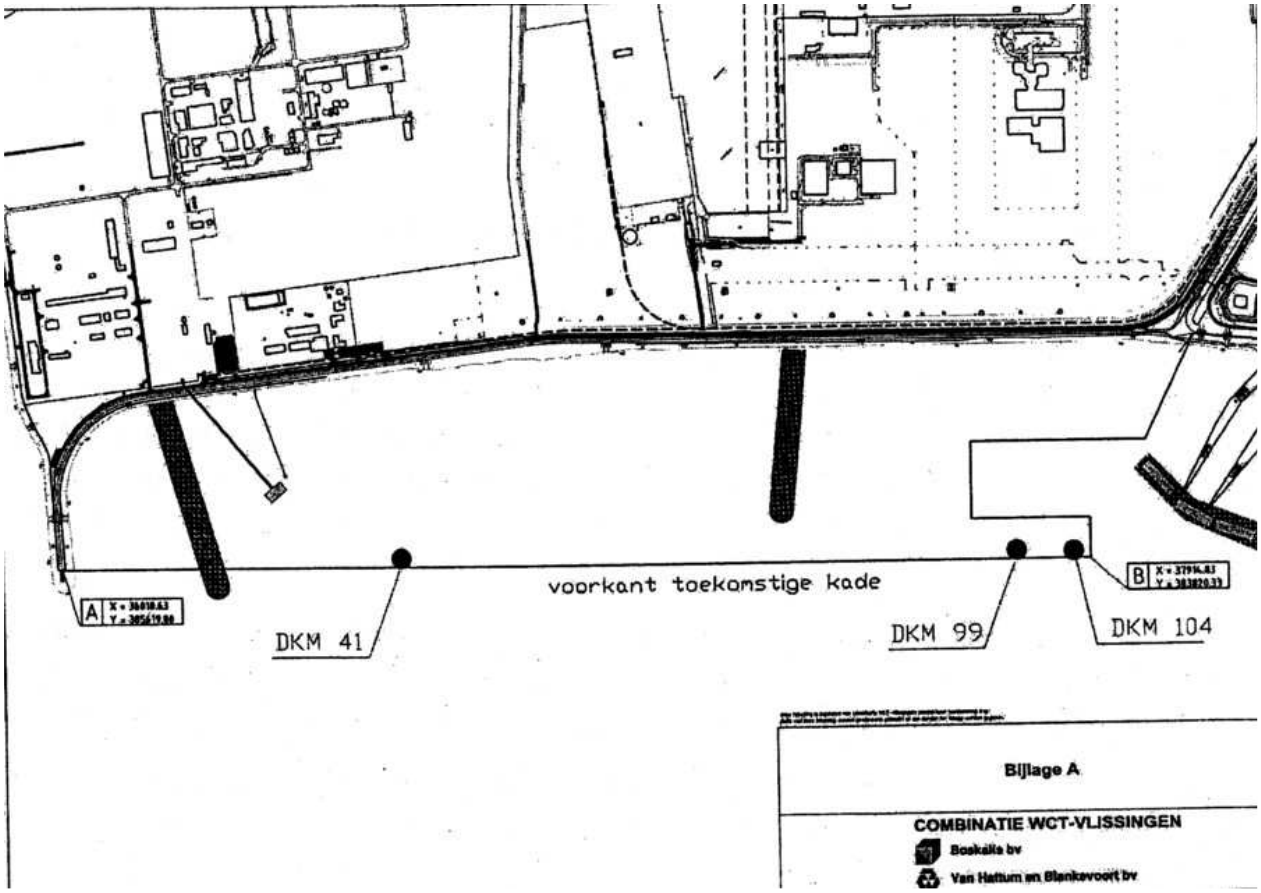


Fig 2.1: Plan view of land-based WCT-design [Program of Requirements]

2.1 Design ship data

Short summary from *[Program of requirements WCT, 2003]*:

Ship	Capacity	L_{OA} (m)	Beam	Max draught	Actual draught
Design ship	12500 TEU	381	57	17	14.2
Post Panamax	6700 TEU	300	42.8	14	12
Panamax	4200 TEU	294	32.2	12.6	10

Future Ultra Large Container Ships (ULCS) :

Capacity	:	12500 TEU
Length b.p.	:	381 m
Length o.a.	:	400 m
Beam	:	57 m (22 boxes)
Actual depth	:	14,2 m (normal; Lloyds)

Max. depth	:	17 m
Height incl. boxes	:	54 m
Water moved	:	220 000 mtons
Block coefficient	:	0,686
Design speed	:	23-25 knots
Machinery	:	single / hybrid
Berth depth [ref.27]	:	NAP – 20 m by Lloyds register [ref. 27, appendix 3]

Largest Feeder:

Capacity	:	9700 dwt
Length	:	109,60 m
Beam	:	18,60 m
Actual depth	:	8,30 m
Height incl. boxes	:	m
Water moved	:	11000 mtons
Block coefficient	:	
Design speed	:	knots
Machinery	:	
Berth depth [ref.27]	:	NAP – 12,0 m

Largest inland ship:

JOWI Capacity	:	4600 t
Length	:	134,16 m
Beam	:	16,84 m
Actual depth	:	3,0 m (normal; Lloyds)
Height	:	5,5 m
Water moved	:	mtons
Block coefficient	:	
Design speed	:	knots

Tug boats:

Kotug Rotor Tug

Bollard pull	:	75 t
Length	:	31,63 m
Beam	:	12,00 m
Max. depth	:	6,57m
Height	:	4,40 / 5,50 m
Water moved	:	907 t

The WCT includes a terminal for ULCS with four to six berths and a quay and apron area terminal for inland ships and feeders, the “Lichterhaven” with two piers; north and south.
The Lichterkade will have to have a length of 300 m with a contract depth of 12 m, an extra head pier has a length of 100 m where small inland ships and tug boats can berth.
If the layout of the floating terminal is different from the fixed terminal, a solution should be found for these three berths along the pier.

3. Positioning of the terminal

Mooring systems can generally be divided into the following systems *[from Journee,J. and Massie, W.; lecture notes Offshore hydromechanics, TU-Delft 2001]:*

- Single point mooring (SPM):
  Vessels are secured by a single line or structure.
  The floating object is allowed to weathervane; that is, swing around in order to align itself with prevailing wind, wave and current conditions. This alignment tends to reduce the load on the mooring system. However the mooring forces enter the structure at one point, which will have to endure a very large force. An SPM requires a lot of space and as such, is mostly preferred at an offshore location.
- Multibuoy moorings (MBM) or Spread moorings:
  An MBM holds a vessel in a relatively fixed position, and the vessel cannot turn head into the prevailing waves. As a result, an MBM can experience relatively high loads if wind, currents or waves act at an angle to the mooring. The floating body cannot move and rotate freely, but the mooring forces are distributed better over the structure. Smaller forces are easier to introduce into a structure.
- Dynamic Positioning System:
  Next to a mooring system a Dynamic Positioning System could be employed, mainly to relieve the structure of some of the loads by turning it head into the waves [ref. 22]. This system would only be effective when the terminal can rotate freely, such as when located offshore. A fixed location near the shore would not permit such movements; therefore a DPS will be useless. The current velocities in the Western Scheldt should be taken into account.

Mooring system components include anchors, sinkers, anchor chains, buoys and mooring lines. The type of mooring to be used depends on the location, the water depth, the type of structure, current and wave loads and other influencing factors.

1.1 Alternative mooring systems

A multibuoy system will be applied at the Western Scheldt Container terminal.

I. Three to five large dolphins along the side of the terminal without berths
II. Large piles or dolphins with a sliding connection in the centre longitudinal axis of the terminal
III. Many small (suction) piles along the sides of the terminal without berths
IV. A large pile with a sliding connection for every pontoon element of the terminal, all piles in the centre of the pontoon

Alternative 3 can either be constructed with tubular piles and a sliding connection or with tubular suction piles and anchoring cables. Tubular piles can be used as mooring piles, but tubular suction piles can be removed and re-used again.
Their diameter is often larger and through a sucking mechanism, the pile can be easily drawn from the soil again.

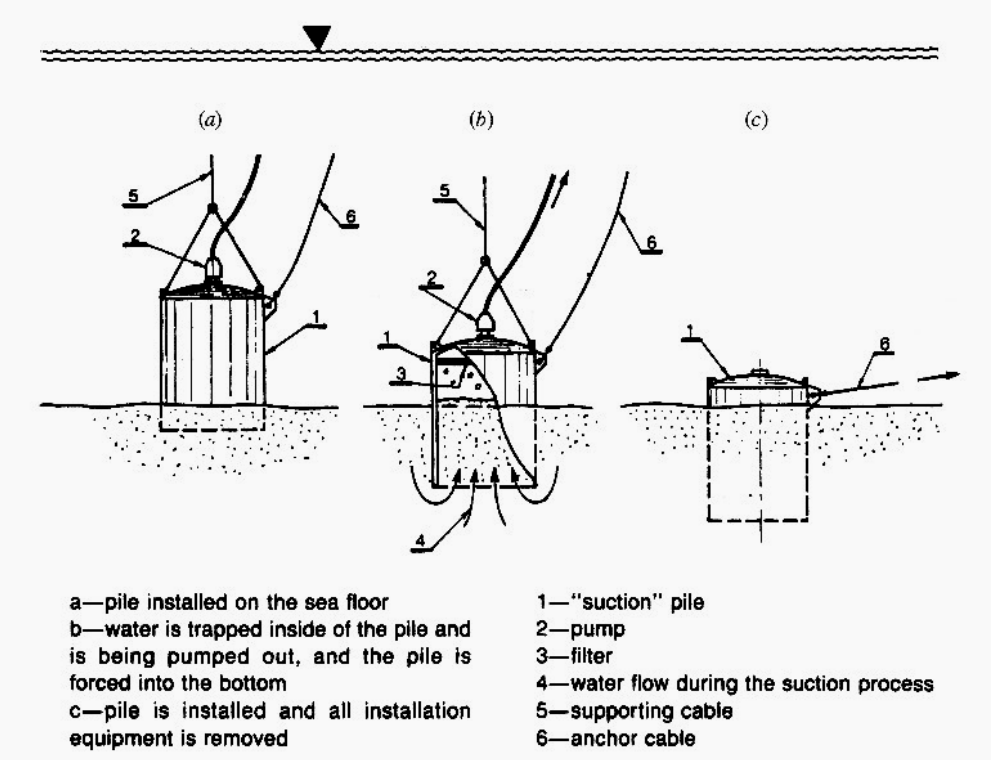


Figure 3.1: Principle of tubular suction piles

Choice aspects	Alternative I	Alternative II	Alternative III	Alternative IV
Constructability	+	+	+	-
Loads on terminal	-	-	+	++
Space required	+	+-	-	-
Difficulty installing elements	+	--	+-	-
Moving terminal + station keeping system	+-	-	+	-
Costs	--	-	+	-
Total	2nd	3rd	1st	4th

++ very good
+ good
+- average
- bad
-- very bad

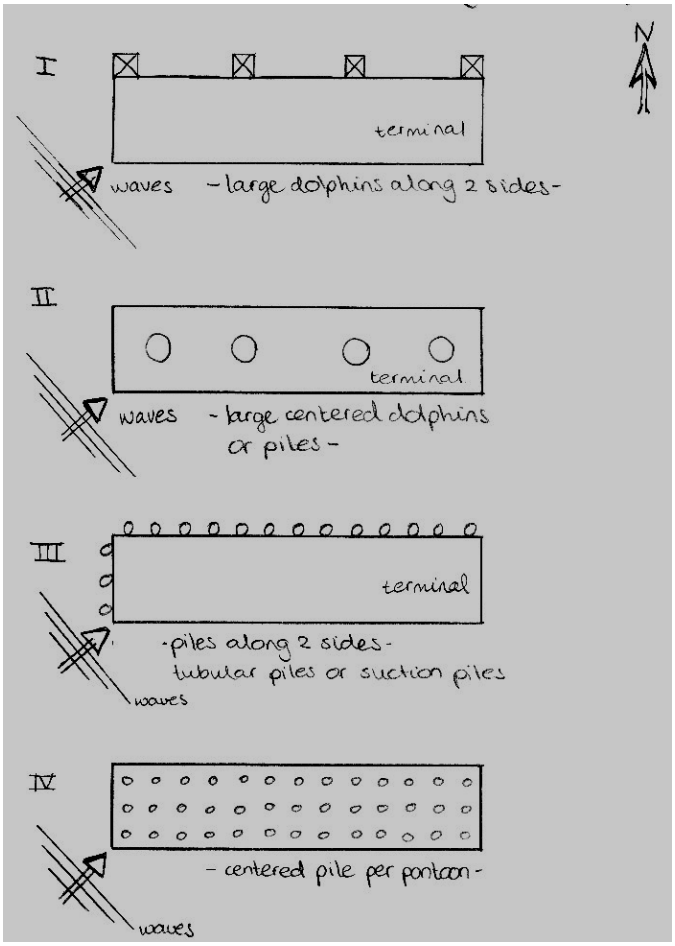


Figure 3.2: Alternative mooring systems

Important aspects per alternative are:

I. A few large dolphin structures are expensive, but very well constructable. They may be moved if kept separate from its foundation (or founded on suction piles). They generate large point loads in the connecting element and the attachment detail may be complex.

II. All space along the ship berths is available for shipping, the station keeping system is centred in the structure, favourable to the force distribution. Installing the elements will be difficult and the design of the structural detail of the connection is complex.

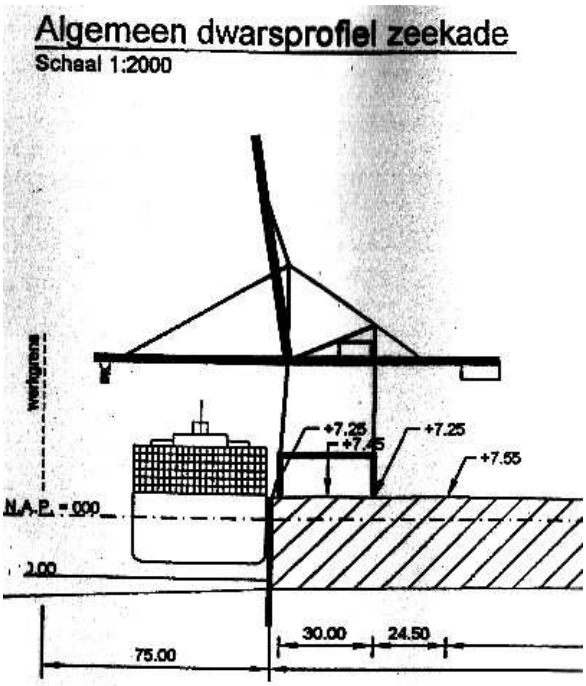
III. The amount of piles is decisive for this alternative. If few large suction piles are needed, then sufficient space at the berths and sides will remain. Piles and anchor cables are mainly a low-cost and well constructable alternative, unless the number of piles grows too large. Suction piles can be removed and installed again when the terminal is moved.

IV. This alternative includes a large pile in the centre of every pontoon. The deck will have to be continuous. Therefore, the construction of the connection detail and installing the elements will be difficult. All space at the berths remains free. However, removing the terminal and the large number of piles will be slow and expensive.

Alternative III is selected as the best applicable mooring system, with multiple tubular piles.

4. Loads on the terminal

A number of drawings and tables show the configuration of the crane and traffic loads:



Crane loads:

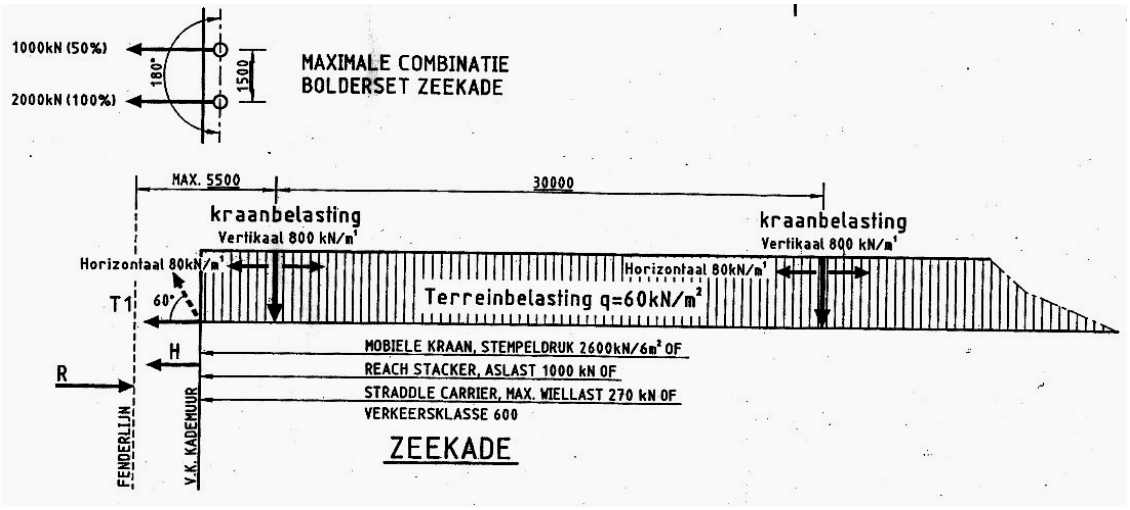
The following tables and figures show the crane loads of the WCT super Post Panamax crane. Also the configuration of the wheel system and of other mobile cranes etc are included *[Combinatie WCT, 2003].*

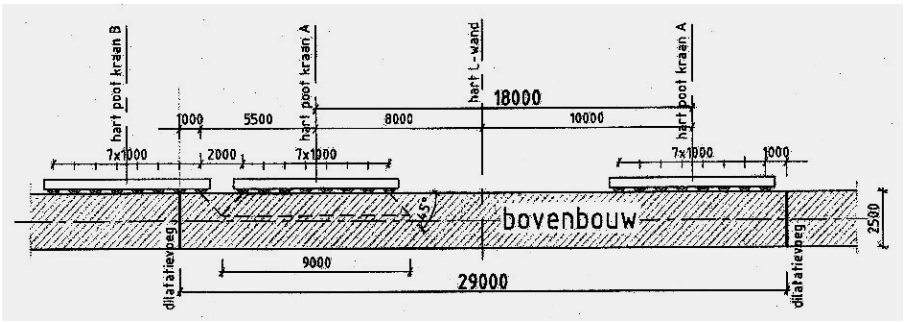
Tabel 13: Bepaling maatgevende verticale kraanbelasting

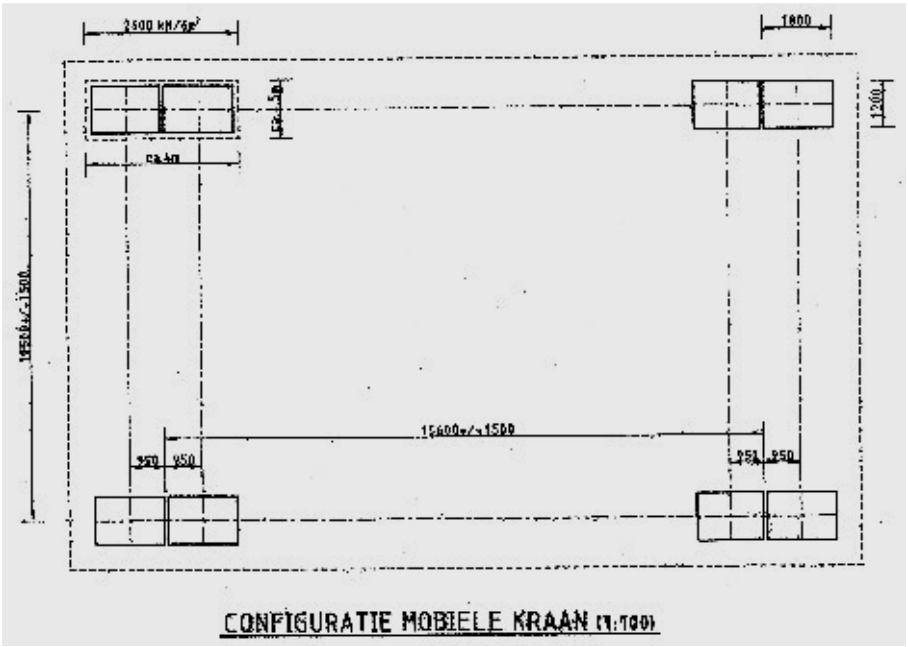
Scenario			Wielbelasting		Pootbelasting		Som	Blokbelasting	
	Maximaal	Wind	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]	[kN/m]
			links	rechts	links	rechts		links	rechts
1	WZ	Geen	675,1	712,1	5401	5697	**11098**	600	633
2	LZ	Geen	288,9	304,8	2311	2438	4749	257	271
3	WZ	Aflandig	620,3	800,1	4962	**6400**	**11362**	**551**	**711**
4	LZ	Aanlandig	347,9	197,3	2783	1578	4361	309	175
5	WZ	Parallel	754,3	632,9	6034	5063	11097	670	563
6	LZ	Parallel	194,4	399,4	1555	3195	4750	173	355
7	Geen	Aflandig	186,3	668,9	1490	5351	6841	166	595
8	Geen	Aanlandig	520,6	78,7	4165	630	4795	463	70
9	Geen	parallel	92,3	635,0	738	5080	5818	82	564

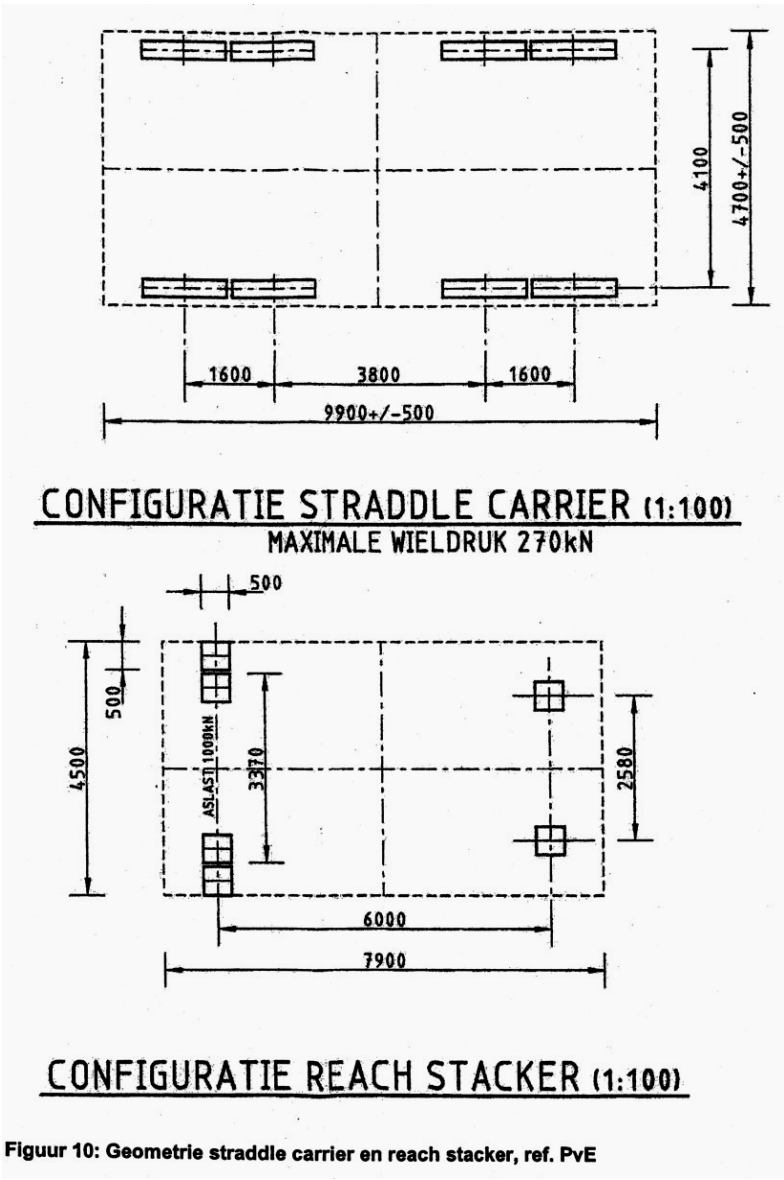
Tabel 14: Blokbelasting WZ kraanbaan

scenario	blokbelasting WZ kraanbaanbelasting [kN/m']	
	links	rechts
Aanlandige wind	600	633
Wind parallel en aflandig	551	711


1000kN (50%)
2000kN (100%)
180°
1500
MAXIMALE COMBINATIE
BOLDERSET ZEEKADE
MAX. 5500
30000
kraanbelasting
Vertikaal 800 kN/m'
kraanbelasting
Vertikaal 800 kN/m'
Horizontaal 80kN/m'
Horizontaal 80kN/m'
T1
60°
Terreinbelasting q=60kN/m²
H
R
MOBIELE KRAAN, STEMPELDRUK 2600kN/6m² OF
REACH STACKER, ASLAST 1000 kN OF
STRADDLE CARRIER, MAX. WIELLAST 270 kN OF
VERKEERSKLASSE 600
FENDERLIJN
V.K. KADEMUUR
ZEEKADE


hart poot kraan B
hart poot kraan A
hart L-wand
hart poot kraan A
18000
1000
5500
8000
10000
7x1000
2000
7x1000
7x1000
1000
bovenbouw
2500
dilatatievoeg
9000
29000
dilatatievoeg


2500 kN/6m²
1800
1200
ca.4m
15600+/-1500
950
950
950
950
CONFIGURATIE MOBIELE KRAAN (1:100)



Figuur 10: Geometrie straddle carrier en reach stacker, ref. PvE

Appendix Part 2:

Overall Design of the Terminal

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2. Conceptual design calculations of the terminal ____ *23*

3. Conceptual design pontoon 180x50x16 m ____ *29*

1. Conceptual design calculations of the pontoons

All of the following are the calculations made in a spreadsheet of the element 420x50x15,5m:

Gegevens:

γ_{water}	10	kN/m^3			
γ_{beton}	25	kN/m^3			
$\gamma_{ballast}$	18	kN/m^3			
L_{bak} =	420,9	m	Vbak buiten =	332069,1	m^3
B_{bak} =	50,9	m			
H_{bak} =	15,5	m			
D_{vloer} =	0,9	m			
D_{dak} =	0,9	m			
$D_{buitenwand}$ =	0,9	m			
$D_{binnenwand}$ =	0,5	m			
H_{binnen} =	13,7	m			
aantal comp. In lengterichting =	28		aantal comp=	140	
aantal comp. In breedterichting	5				
$L_{compartiment}$ =	15	m	V per comp. =	1887,175	m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	264204,5	m^3
L_{binnen} =	14,5	m	A per comp =	137,75	m^2
B_{binnen} =	9,5	m	Abinnen tot =	19285	m^2

Beton:

Vvloer =	19281,429	m^3
Vdak =	19281,429	m^3
Vbuitenwanden =	11590,2	m^3
Vbinnenwanden =	20194,485	m^3
Vtotaal =	70347,543	m^3
Gewicht beton = Fneerw	1758688,58	kN

Belastingen:

Kraan 2 x 80 ton =		kN
Verkeer =		kN
verkeer aslast =		kN
Overige kranen =		kN
Opslag =		kN
	0	
Totaal gewicht = Fneerw	1758688,58	kN

diepgang excl belasting =	8,21		Hs =	2,39	m
vrijboord excl belasting =	7,29				
diepgang incl belasting =	8,21	m			
vrijboord incl belasting =	7,29	m	vrijboord voldoende		
	47,0%				
Fopw excl belasting =	1758688,58	kN			
Fopw incl belasting=	1758688,58	kN			
vereiste speling diepgang - waterdiepte	1	m			
waterdiepte locatie	21	m	waterdiepte voldoende		

Ballasten t.b.v. Transport (leeg)

						G tijdens tranport			
gewenst vrijboord	1	m				1758688,6	kN	7,75	m	13629836,5
factor ballasten =	1,0		$\gamma_{water\ bij\ ballasten}$	10,25	kN/m^3	1425425	kN	2,95	m	4209505,2
Fneerw benodigd	3184114	kN				3184114	kN			17839341,7
benodigd gewicht ballast	1425425	kN								
benodigd volume ballastzand	79190	m3				KG transport =		5,60	m	
Hoogte zand over A binnen tot	4,11	m	voldoende ruimte binnen							

APRON ELEMENT

Gegevens element:

γ_{water}	10	kN/m^3
γ_{beton}	25	kN/m^3
$\gamma_{ballast}$	18	kN/m^3
L_{bak} =	420,9	m
B_{bak} =	50,9	m
H_{bak} =	15,5	m
D_{vloer} =	0,9	m
D_{dak} =	0,9	m
$D_{buitenwand}$ =	0,9	m
$D_{binnenwand}$ =	0,5	m

Platform

L =	2104,5	2525,4	2650	m
B=	509	509	500	m
Vbak buiten =	332069,1	m^3		
n elem =	110	stuks	allen gelijk	
nzeekade =	11	stuks		

Compartimenten

H_{binnen} =	13,7	m		
aantal comp. In lengterichting =	28			
aantal comp. In breedterichting	5		aantal comp=	140
$L_{compartiment}$ =	15	m	V per comp. =	1887,175 m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	264204,5 m^3
L_{binnen} =	14,5	m	A per comp =	137,75 m^2
B_{binnen} =	9,5	m	Abinnen tot =	19285 m^2

Beton:

Vvloer =	19281,429	m^3	
Vdak =	19281,429	m^3	
Vbuitenwanden =	11590,2	m^3	
Vbinnenwanden =	20194,485	m^3	
Vtotaal =	70347,543	m^3	
Gewicht beton = Fneerw	1758688,575	kN	4178,4 kN/m1

Betoneigenschappen

	f'ck	f'b	fb	fbm	(N/mm^2) E'b	(N/mm^2) Eizz
B45	45	27	1,65	3,3	33500	1,91E+20
B55	55	33	1,9	3,8	36000	2,05E+20
B65	65	39	2,15	4,3	38500	2,20E+20
S355					2,10E+05	

Belastingen:

Kraan 2 x 80 000 kN =	160000	kN	
Verkeer =	64271,43	kN	
verkeer aslast =	600	kN	
Overige kranen =	14560	kN	
Opslag =	0	kN	
	239431,43		568,8559 kN/m1
Totaal gewicht = Fneerw	1998120,005	kN	4747,256

8,679 m CG ok 6,821 m bk ponton 0,648 m ow CoG

diepgang excl belasting =	8,21			
vrijboord excl belasting =	7,29			
diepgang incl belasting =	9,33	m		
vrijboord incl belasting =	6,17	m	Hs = 2,06 m	gamma= 3,00
	39,8%		vrijboord voldoende	

Fopw excl belasting =	1758688,575	kN
Fopw incl belasting=	1998120,005	kN

STORAGE ELEMENT

Gegevens element:

γ_{water}	10	kN/m^3	
γ_{beton}	25	kN/m^3	
$\gamma_{ballast}$	18	kN/m^3	
L_{bak} =	420,9	m	
B_{bak} =	50,9	m	
H_{bak} =	16	m	
D_{vloer} =	0,9	m	
D_{dak} =	0,9	m	
$D_{buitenwand}$ =	0,9	m	
$D_{binnenwand}$ =	0,5	m	

Platform

L =	2104,5	2525,4	2650	m
B=	509	509	500	m
Vbak buiten =	342781	m^3		
n elem =	110	stuks	allen gelijk	
nzeekade =	11	stuks		

Compartimenten

H_{binnen} =	14,2	m		
aantal comp. In lengterichting =	28			
aantal comp. In breedterichting	5		aantal comp=	140
$L_{compartiment}$ =	15	m	V per comp. =	1956,05 m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	273847 m^3
L_{binnen} =	14,5	m	A per comp =	137,75 m^2
B_{binnen} =	9,5	m	Abinnen tot =	19285 m^2

Beton:

Vvloer =	19281,429	m^3		**Betoneigenschappen**					(N/mm^2)	(N/mm^2)
Vdak =	19281,429	m^3			f'ck	f'b	fb	fbm	E'b	Elzz
Vbuitenwanden =	12013,2	m^3		B45	45	27	1,65	3,3	33500	2,98E+20
Vbinnenwanden =	20931,51	m^3		B55	55	33	1,9	3,8	36000	3,20E+20
Vtotaal =	71507,568	m^3		B65	65	39	2,15	4,3	38500	3,42E+20
Gewicht beton = Fneerw	1787689,2	kN	4247,301 kN/m1	S355					2,10E+05	

Belastingen:

Kraan =	0	kN						
Verkeer =	8492,4	kN						
verkeer aslast =	600	kN						
Overige kranen =	14560	kN						
Opslag =	1117740,6	kN						
	1141393		2711,791 kN/m1					
Totaal gewicht = Fneerw	2929082,2	kN	6959,093	11,117 m CG ok	4,883 m bk ponton	2,555 m ow	CoG	

diepgang excl belasting =	8,34				
vrijboord excl belasting =	7,66				
diepgang incl belasting =	13,67	m			
vrijboord incl belasting =	2,33	m	Hs =	2,06 m	gamma= 1,13
	14,5%		vrijboord voldoende		

Fopw excl belasting =	1787689,2	kN
Fopw incl belasting=	2929082,2	kN

Statische stabiliteit van het drijflichaam tijdens transport **element 420x50x15,5 m**

Kantelen mag niet worden ingeleid door troskrachten, golfbewegingen etc.
Het drijflichaam moet zodanig worden ontworpen, dat bij een extern opgelegde hoekverdraaiing (golf)
een corrigerend, tegenwerkend koppel (moment) ontstaat dat de bak weer naar zijn rusttoestand brengt.

Bewegingsrichting is in delengterichting van de bak,
dus de berekening van het metacentrum wordt in de dwarsdoorsnede uitgevoerd.
Metacentrum = punt M
Massazwaartepunt element = punt G
Drukpunt = massazw. water = punt B

Aanvangsstabiliteit DWARS:

diepgang bak	8,21	m
vrijboord	7,29	m
KB	4,10	m
KG	7,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	4625418,4	m4
V = volume waterverplaatsing	175868,86	m3
afstand BM = I / V	26,30	m
KM= KB+BM	30,40	m
GM=hm= KM-KG	22,65	m

Controle: De eis voor de statische stabiliteit is, dat M boven G ligt,
ofwel dat hm groter is dan 0,5 m.
Aan deze eis wordt voldaan. Het drijflichaam is statisch stabiel.
Conclusie:
De constructie is zwaar overgedempt. Gm >> 1,0 m.
Dit heeft tot gevolg dat de bak niet zal gaan zwaaien,
maar kan gaan trillen met een hoge frequentie en kleine amplitude

Aanvangsstabiliteit LANGS:

diepgang bak	8,21	m
vrijboord	7,29	m
KB	4,10	m
KG	7,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	316281153,1	m4
V = volume waterverplaatsing	175868,86	m3
afstand BM = I / V	1798,39	m
KM= KB+BM	1802,50	m
GM=hm= KM-KG	1794,75	m

GM is zeer groot!

Stabiliteit na ballasten tijdens transport DWARS

diepgang bak	14,50	m
vrijboord	1,00	m
KB	7,25	m
KG	5,60	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	4625418,4	m4
V = volume waterverplaatsing	310645,25	m3
afstand BM = I / V	14,89	m
KM=KB+BM	22,14	m
GM=hm= KM-KG	16,54	m

Conclusie: Diepgang 6,29 m groter; KB 3,15 m groter;
KG 2,15 m lager. Echter BM 11,41 m lager, dus GM daalt.

Stabiliteit tijdens transport LANGS

diepgang bak	14,50	m
vrijboord	1,00	m
KB	7,25	m
KG	5,60	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	316281153,1	m4
V = volume waterverplaatsing	310645,25	m3
afstand BM = I / V	1018,14	m
KM=KB+BM	1025,39	m
GM=hm= KM-KG	1019,79	m

Stabiliteit DWARS operationale fase:
(Eindfase incl belasting)

diepgang bak	13,67	m
vrijboord	2,33	m
KB	6,84	m
KG	11,12	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	4625418,4	m4
V = volume waterverplaatsing	292908,22	m3
afstand BM = I / V	15,79	m
KM=KB+BM	22,63	m
GM=hm= KM-KG	11,51	m

Controle: De eis voor de statische stabiliteit is, dat M boven G ligt, ofwel dat hm groter is dan 0,5 m.
Aan deze eis wordt voldaan. Het drijflichaam is statisch stabiel.

Stabiliteit LANGS operationale fase:

diepgang bak	13,67	m
vrijboord	2,33	m
KB	6,84	m
KG	11,12	m
hoogte bak	15,5	m
halve hoogte bak		m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	316281153,1	m4
V = volume waterverplaatsing	292908,22	m3
afstand BM = I / V	1079,80	m
KM=KB+BM	1086,63	m
GM=hm= KM-KG	1075,51	m

Stabiliteit transport DWARS		
incl waterballast		
diepgang bak	14,50	m
vrijboord	1,00	m
KB	7,25	m
KG	5,60	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I = Iu - sigma Ii	4480379,2	m4
V = volume waterverp	310645,25	m3
afstand BM = I / V	14,42	m
KM=KB+BM	21,67	m
GM=hm= KM-KG	16,07	m

Conclusie: Diepgang 6,29 m groter; KB 3,15 m groter;
KG 2,15 m lager. Echter BM 11,41 m lager, dus GM daalt.

Stabiliteit transport LANGS		
incl waterballast		
diepgang bak	14,50	m
vrijboord	1,00	m
KB	7,25	m
KG	5,60	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I = Iu - sigma Ii	315943263,9	m4
V = volume waterver	310645,25	m3
afstand BM = I / V	1017,05	m
KM=KB+BM	1024,30	m
GM=hm= KM-KG	1018,70	m

Stabiliteit DWARS operationale fase:		
incl waterballast		
diepgang bak	13,67	m
vrijboord	2,33	m
KB	6,84	m
KG	11,12	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	4480379,2	m4
V = volume waterverp	292908,22	m3
afstand BM = I / V	15,30	m
KM=KB+BM	22,13	m
GM=hm= KM-KG	11,01	m

Stabiliteit LANGS operationale fase:		
incl waterballast		
diepgang bak	13,67	m
vrijboord	2,33	m
KB	6,84	m
KG	11,12	m
hoogte bak	15,5	m
halve hoogte bak		m
Lengte bak	420,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	315943263,9	m4
V = volume waterver	292908,22	m3
afstand BM = I / V	1078,64	m
KM=KB+BM	1085,48	m
GM=hm= KM-KG	1074,36	m

	γ_{water}	10	kN/m^3
	γ_{beton}	25	kN/m^3
zand, te verpompen	$\gamma_{ballast}$	18	kN/m^3
Alternatief?	L_{bak} =	420,9	m
Breedte	B_{bak} =	50,9	m
Hoogte	H_{bak} =	15,5	m

	D_{vloer} =	0,9	m
	D_{dak} =	0,9	m
	$D_{buitenwand}$ =	0,9	m
	$D_{binnenwand}$ =	0,5	m
	Dtussenschot =	0,5	m
	M ponton =	1758688,58	kN
	Mcomp =	**12562,06**	kN

Massieve (=gesloten) drijvende constructie: tpv tussenschot

A tussenschot		0	m^2
1/12bh^3	Iy = Izz	15795,44	m^4
1/12hb^3	Iz = Iyy	170334,96	m^4
	Ip = Iy + Iz =	186130,40	m^4

Centre of Gravity	CG (x, y, z) :	210,45	25,45	7,75	m
traagheidsstraal	i= sqrt Iy/A = Iy/M		m		
i^2 = Iy/A	i= sqrt Iz/A = Iz/M		m		

Doorsnede met compartimenten

	Ix =	2,615E+10	kNm^4
	Iy =	4,68E+09	kNm^4
	Iz =	4,58E+09	kNm^4
	Ip = Iy + Iz =	9,26E+09	kNm^4

Iy = som 1/12 bh^3 + A*z^2 in m4

PONTON					**1/12Mx^2 x=l**			**1/12My^2 y=b**			**1/12Mz^2 z=h**		
Deel i	**M i (kN)**	**xC (m)**	**yC (m)**	**zC (m)**	**Ixx eigen kNm2**	**Isteiner**	**Ixx**	**Iyy eigen kNm2**	**Isteiner**	**Iyy=Iz**	**Izzeigen kNm2**	**Isteiner**	**Izz = Iy**
vloer	482035,7	0	0	-7,75	7,12E+09	2,90E+07	7,15E+09	1,04E+08	2,90E+07	1,33E+08	3,25E+04	0,00E+00	3,25E+04
dak	482035,7	0	0	7,75	7,12E+09	2,90E+07	7,15E+09	1,04E+08	2,90E+07	1,33E+08	3,25E+04	0,00E+00	3,25E+04
buitenwand a langs	129742,425	0	25	0	1,92E+09	8,11E+07	2,00E+09	8,76E+03	0,00E+00	8,76E+03	2,03E+06	8,11E+07	8,31E+07
binnenwand b	70674	0	15	0	1,03E+09	1,59E+07	1,05E+09	1,47E+03	0,00E+00	1,47E+03	1,11E+06	1,59E+07	1,70E+07
binnenwand c	70674	0	5	0	1,03E+09	1,77E+06	1,04E+09	1,47E+03	0,00E+00	1,47E+03	1,11E+06	1,77E+06	2,87E+06
binnenwand d	70674	0	-5	0	1,03E+09	1,77E+06	1,04E+09	1,47E+03	0,00E+00	1,47E+03	1,11E+06	1,77E+06	2,87E+06
binnenwand e	70674	0	-15	0	1,03E+09	1,59E+07	1,05E+09	1,47E+03	0,00E+00	1,47E+03	1,11E+06	1,59E+07	1,70E+07
buitenwand f langs	129742,425	0	-25	0	1,92E+09	8,11E+07	2,00E+09	8,76E+03	0,00E+00	8,76E+03	2,03E+06	8,11E+07	8,31E+07
buitenwand 1 dwars	15135,075	210	0	0	2,23E+08	0,00E+00	2,23E+08	3,27E+06	6,67E+08	6,71E+08	2,37E+05	6,67E+08	6,68E+08
binnenwand 2 dwars	8225,875	-195	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	3,13E+08	3,14E+08	1,29E+05	3,13E+08	3,13E+08
binnenwand 3	8225,875	-180	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,67E+08	2,68E+08	1,29E+05	2,67E+08	2,67E+08
binnenwand 4	8225,875	-165	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,24E+08	2,26E+08	1,29E+05	2,24E+08	2,24E+08
binnenwand 5	8225,875	-150	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,85E+08	1,87E+08	1,29E+05	1,85E+08	1,85E+08
binnenwand 6	8225,875	-135	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,50E+08	1,52E+08	1,29E+05	1,50E+08	1,50E+08
binnenwand 7	8225,875	-120	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,18E+08	1,20E+08	1,29E+05	1,18E+08	1,19E+08
binnenwand 8	8225,875	-105	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	9,07E+07	9,23E+07	1,29E+05	9,07E+07	9,08E+07
binnenwand 9	8225,875	-90	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	6,66E+07	6,83E+07	1,29E+05	6,66E+07	6,68E+07
binnenwand 10	8225,875	-75	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	4,63E+07	4,79E+07	1,29E+05	4,63E+07	4,64E+07
binnenwand 11	8225,875	-60	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,96E+07	3,13E+07	1,29E+05	2,96E+07	2,97E+07
binnenwand 12	8225,875	-45	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,67E+07	1,83E+07	1,29E+05	1,67E+07	1,68E+07
binnenwand 13	8225,875	-30	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	7,40E+06	9,06E+06	1,29E+05	7,40E+06	7,53E+06
binnenwand 14	8225,875	-15	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,85E+06	3,50E+06	1,29E+05	1,85E+06	1,98E+06
binnenwand 15	8225,875	0	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	0,00E+00	1,65E+06	1,29E+05	0,00E+00	1,29E+05
binnenwand 16	8225,875	15	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,85E+06	3,50E+06	1,29E+05	1,85E+06	1,98E+06
binnenwand 17	8225,875	30	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	7,40E+06	9,06E+06	1,29E+05	7,40E+06	7,53E+06
binnenwand 18	8225,875	45	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,67E+07	1,83E+07	1,29E+05	1,67E+07	1,68E+07
binnenwand 19	8225,875	60	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,96E+07	3,13E+07	1,29E+05	2,96E+07	2,97E+07
binnenwand 20	8225,875	75	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	4,63E+07	4,79E+07	1,29E+05	4,63E+07	4,64E+07
binnenwand 21	8225,875	90	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	6,66E+07	6,83E+07	1,29E+05	6,66E+07	6,68E+07
binnenwand 22	8225,875	105	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	9,07E+07	9,23E+07	1,29E+05	9,07E+07	9,08E+07
binnenwand 23	8225,875	120	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,18E+08	1,20E+08	1,29E+05	1,18E+08	1,19E+08
binnenwand 24	8225,875	135	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,50E+08	1,52E+08	1,29E+05	1,50E+08	1,50E+08
binnenwand 25	8225,875	150	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	1,85E+08	1,87E+08	1,29E+05	1,85E+08	1,85E+08
binnenwand 26	8225,875	165	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,24E+08	2,26E+08	1,29E+05	2,24E+08	2,24E+08
binnenwand27	8225,875	180	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	2,67E+08	2,68E+08	1,29E+05	2,67E+08	2,67E+08
binnenwand28	8225,875	195	0	0	1,20E+08	0,00E+00	1,20E+08	1,65E+06	3,13E+08	3,14E+08	1,29E+05	3,13E+08	3,13E+08
buitenwand 29 dwars	15135,075	210	0	0	2,23E+08	0,00E+00	2,23E+08	3,27E+06	6,67E+08	6,71E+08	2,37E+05	6,67E+08	6,68E+08
Ponton	**1758621,1**						**2,615E+10 kN m^2**			**4,684E+09 kNm^2**			**4,577E+09**

Er geldt:
Ixx = 1/12yx^3
Iyy = 1/12zy^3
Izz = 1/12yz^3
lees voor x, y, z dan l, b, h

Ip 9,260E+09 **kN m^2**

volgens die benaderingsmethode van MT geldt de volgende Steiner-regel:
Ixxtotaal = Ixx + M*(y^2 + z^2)
Iyytotaal = Iyy + M*(x^2 +z^2)
Izztotaal = Izz + M*(x^2 +y^2)

Ponton:	**Kxx^2 = Ixx/M**	kxx =	122 m
	Kyy^2 = Iyy/M	kyy =	52 m
	Kzz^2 = Izz/M	kzz =	51 m

Dynamische stabiliteit voor slingeren (roll)

Bewegingsrichting is in de lengterichting van de bak,
dus de berekening van het metacentrum wordt in de dwarsdoorsnede uitgevoerd.
(transversal static stability)

Metacentrum =	punt M
Massazwaartepunt element =	punt G
Drukpunt = massazw. water =	punt B

Tijdens transport

waterverplaatsing	V=	310645,25	m^3	
gewicht betonconstructie	G =	1758688,58	kN	excl. ballast
afstand onderkant - B		7,25	m	
afstand onderkant - G		5,60	m	
afstand	BM =	14,89	m	I / V
hoogte metacentrum GM	hm =	16,54	m	
Ipolair = Iyy + Izz =	Ip =	9,26E+09	kNm2	
A dwarsdoorsnede	A =	21423,81	m^2	
M per m lengte	M =	1758689	kN	
polaire massatraagheidsstraal van het element	k =	72,56	m	sqrt (Ipolair / A dwarsdrsn)
versnelling zwaartekracht	g =	9,81	m/s^2	
eigenperiode van het element	To =	35,80	sec	(2*pi*j) / (sqrt(hm*g))
eigenfrequentie van het element	fo = 1 / To	0,0279	Hz	laag genoeg

Tijdens Operatie

waterverplaatsing	V=	175868,858	m^3	
gewicht betonconstructie	G =	1910617,35	kN	incl. ballast
KB =		6,84	m	
KG =		11,12	m	
afstand	BM =	15,79	m	I / V
hoogte metacentrum GM	hm =	11,51	m	
Ipolair = Iyy + Izz =	Ip =	9,26E+09	m^4	
A dwarsdoorsnede	A =	21423,81	m^2	
M per m lengte	M =	1910617,35	kN	
polaire massatraagheidsstraal van het element	k =	69,62	m	sqrt (Ipolair / A dwarsdrsn)
versnelling zwaartekracht	g =	9,81	m/s^2	
eigenperiode van het element	To =	41,17	sec	(2*pi*j) / (sqrt(hm*g))
eigenfrequentie van het element	fo = 1 / To	0,0243	Hz	laag genoeg

Dynamic stability roll in operation

rho	1,025
g	9,81
nabla	292908,22
GMt	11,51
Ixx	26154457087
m phi	1,00E+08
omega phi	0,036

Dynamic stability heave in operation

rho	1,025
g	9,81
Aw	21423,81
M	1998120,005
m zz	1,00E+07
omega z	0,134

Dynamic stability pitch in operation

rho	1,025
g	9,81
nabla	292908,22
GMl	1075,51
Iyy	4683786771
m pitch	1,00E+11
omega pitch	0,174

The following results are from spreadsheets-calculation of the 180x50x15,5 element
(summary of the calculations, as most are the same as for 420x50x15,5-element)

STORAGE ELEMENT

Gegevens element:

γ_{water}	10	kN/m^3	
γ_{beton}	25	kN/m^3	
$\gamma_{ballast}$	18	kN/m^3	
L_{bak} =	180,9	m	
B_{bak} =	50,9	m	
H_{bak} =	16	m	
D_{vloer} =	0,9	m	
D_{dak} =	0,9	m	
$D_{buitenwand}$ =	0,9	m	
$D_{binnenwand}$ =	0,5	m	

Platform

L =	2170,8	2160	2650	m
B=	509	500	500	m
Vbak buiten =	147325	m^3		
n elem =	120	stuks	allen gelijk	
nzeekade =	12	stuks		

Compartimenten

H_{binnen} =	14,2	m		
aantal comp. In lengterichting =	12			
aantal comp. In breedterichting	5		aantal comp=	60
$L_{compartiment}$ =	15	m	V per comp. =	1956,05 m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	117363 m^3
L_{binnen} =	14,5	m	A per comp =	137,75 m^2
B_{binnen} =	9,5	m	Abinnen tot =	8265 m^2

Beton:

Vvloer =	8287,029	m^3		**Betoneigenschappen**					(N/mm^2)	(N/mm^2)
Vdak =	8287,029	m^3			f'ck	f'b	fb	fbm	E'b	Elzz
Vbuitenwanden =	5878,8	m^3		B45	45	27	1,65	3,3	33500	2,06E+20
Vbinnenwanden =	8764,95	m^3		B55	55	33	1,9	3,8	36000	2,21E+20
Vbinnenwanden =	8764,95	m^3		B65	65	39	2,15	4,3	38500	2,36E+20
Vtotaal =	31217,808	m^3		S355					2,10E+05	
Gewicht beton = Fneerw	780445,2	kN	4314,235							

Belastingen:

Kraan =	0	kN					
Verkeer =	4172,4	kN					
verkeer aslast =	600	kN					
Overige kranen =	14560	kN					
Opslag =	471180,6	kN					
	490513		2711,515	kN/m1			
Totaal gewicht = Fneerw	1270958,2	kN	7025,75	11,088 m CG ok	4,912 m bk ponton	2,716 m ow	CoG

diepgang excl belasting =	8,48					
vrijboord excl belasting =	7,52					
diepgang incl belasting =	13,80	m				
vrijboord incl belasting =	2,20	m	Hs =	2,06 m	gamma=	1,07
	13,7%		vrijboord voldoende			

Fopw excl belasting =	780445,2	kN
Fopw incl belasting=	1270958,2	kN

ELEMENT WOTHOUT LOADS

Gegevens:

γ_{water}	10	kN/m^3			
γ_{beton}	25	kN/m^3			
$\gamma_{ballast}$	18	kN/m^3			
L_{bak} =	180,9	m	Vbak buiten =	147325	m^3
B_{bak} =	50,9	m			
H_{bak} =	16	m			
D_{vloer} =	0,9	m			
D_{dak} =	0,9	m			
$D_{buitenwand}$ =	0,9	m			
$D_{binnenwand}$ =	0,5	m			
H_{binnen} =	14,2	m			
aantal comp. In lengterichting =	12		aantal comp=	60	
aantal comp. In breedterichting	5				
$L_{compartiment}$ =	15	m	V per comp. =	1956,05	m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	117363	m^3
L_{binnen} =	14,5	m	A per comp =	137,75	m^2
B_{binnen} =	9,5	m	Abinnen tot =	8265	m^2

Beton:

Vvloer =	8287,029	m^3
Vdak =	8287,029	m^3
Vbuitenwanden =	5878,8	m^3
Vbinnenwanden =	8764,95	m^3
Vtotaal =	31217,808	m^3
Gewicht beton = Fneerw	780445,2	kN

Belastingen:

Kraan 2 x 80 ton =		kN
Verkeer =		kN
verkeer aslast =		kN
Overige kranen =		kN
Opslag =		kN
	0	
Totaal gewicht = Fneerw	780445,2	kN

diepgang excl belasting =	8,48		Hs =	2,37	m
vrijboord excl belasting =	7,52				
diepgang incl belasting =	8,48	m			
vrijboord incl belasting =	7,52	m	vrijboord voldoende		
	47,0%				
Fopw excl belasting =	780445,2	kN			
Fopw incl belasting=	780445,2	kN			
vereiste speling diepgang - waterdiepte	1	m			
waterdiepte locatie	21	m	waterdiepte voldoende		

Ballasten t.b.v. Transport (leeg)

gewenst vrijboord	1,5	m			
factor ballasten =	1,0		$\gamma_{water\ bij\ ballasten}$	10,25	kN/m^3
Fneerw benodigd	1368510,761	kN			
benodigd gewicht ballast	588066	kN			
benodigd volume ballastzand	32670	m3			
Hoogte zand over A binnen tot	3,95	m	voldoende ruimte binnen		
water ballast	58806,56	7,12	m		

Zwaartepunt tijdens transport ballast

G*x beton	6243561,60
G*x ballast	1162267,61
zw ligging KZ	5,41

Ballasten t.b.v. Stabiliteit en lastspreiding:

Fopw hele constructie onder water =	1473249,6	kN			
factor ballasten =	1,2		$\gamma_{water\ bij\ ballasten}$	10,25	kN/m^3
Fneerw benodigd		kN			
benodigd gewicht ballast	0	kN			
benodigd volume ballastzand	0	m3			
Hoogte zand over A binnen tot		m3			

APRON ELEMENT

Gegevens element:

γ_{water}	10	kN/m^3
γ_{beton}	25	kN/m^3
$\gamma_{ballast}$	18	kN/m^3
L_{bak} =	180,9	m
B_{bak} =	50,9	m
H_{bak} =	15,5	m
D_{vloer} =	0,9	m
D_{dak} =	0,9	m
$D_{buitenwand}$ =	0,9	m
$D_{binnenwand}$ =	0,5	m

Platform

L =	2170,8	2160	2650	m
B=	509	500	500	m
Vbak buiten =	142721,1	m^3		
n elem =	*120*	stuks	allen gelijk	
nzeekade =	12	stuks		

Compartimenten

H_{binnen} =	13,7	m		
aantal comp. In lengterichting =	12			
aantal comp. In breedterichting	5		aantal comp=	60
$L_{compartiment}$ =	15	m	V per comp. =	1887,175 m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	113230,5 m^3
L_{binnen} =	14,5	m	A per comp =	137,75 m^2
B_{binnen} =	9,5	m	Abinnen tot =	8265 m^2

Beton:

Vvloer =	8287,029	m^3		**Betoneigenschappen**					(N/mm^2)	(N/mm^2)
Vdak =	8287,029	m^3			f'ck	f'b	fb	fbm	E'b	EIzz
Vbuitenwanden =	5671,8	m^3		B45	45	27	1,65	3,3	33500	2,06E+20
Vbinnenwanden =	8456,325	m^3		B55	55	33	1,9	3,8	36000	2,21E+20
Vtotaal =	30702,183	m^3		B65	65	39	2,15	4,3	38500	2,36E+20
Gewicht beton = Fneerw	767554,575	kN	4242,977 kN/m1	S355					2,10E+05	

Belastingen:

Kraan 2 x 80 000 kN ton =	160000	kN	
Verkeer =	27623,43	kN	
verkeer aslast =	600	kN	
Overige kranen =	14560	kN	
Opslag =	0	kN	
	202783,43		1120,97 kN/m1
Totaal gewicht = Fneerw	970338,005	kN	5363,947

9,370 m 6,130 m bk ponton 1,169 m ow

diepgang excl belasting =	8,34				
vrijboord excl belasting =	7,16				
diepgang incl belasting =	10,54	m			
vrijboord incl belasting =	4,96	m	Hs =	2,37 m	gamma= 2,09
	32,0%		vrijboord voldoende		

Fopw excl belasting =	767554,575	kN
Fopw incl belasting=	970338,005	kN

Doorsnede met compartimenten

Ix =	2,234E+09	kNm^4	
Iy =	6,08E+08	kNm^4	Iy = som 1/12 bh^3 + A*z^2 in m4
Iz =	5,61E+08	kNm^4	
Ip = Iy + Iz =	1,17E+09	kNm^4	

PONTON					**1/12Mx^2 x=l**			**1/12My^2 y=b**			**1/12Mz^2 z=h**		
Deel i	**M i (kN)**	**xC (m)**	**yC (m)**	**zC (m)**	**Ixx eigen kNm2**	**Isteiner**	**Ixx**	**Iyy eigen kNm2**	**Isteiner**	**Iyy=Iz**	**Izzeigen kNm2**	**Isteiner**	**Izz = Iy**
ribber	207175,7	0	0	-8	5,65E+08	1,33E+07	5,78E+08	4,47E+07	1,33E+07	5,80E+07	1,40E+04	0,00E+00	1,40E+04
dak	207175,7	0	0	8	5,65E+08	1,33E+07	5,78E+08	4,47E+07	1,33E+07	5,80E+07	1,40E+04	0,00E+00	1,40E+04
buitenwand a langs	57797,55	0	25	0	1,58E+08	3,61E+07	1,94E+08	3,90E+03	0,00E+00	3,90E+03	9,71E+05	3,61E+07	3,71E+07
binnenwand b	31446,5	0	15	0	8,41E+07	7,08E+06	9,11E+07	6,55E+02	0,00E+00	6,55E+02	5,28E+05	7,08E+06	7,60E+06
binnenwand c	31446,5	0	5	0	8,41E+07	7,86E+05	8,48E+07	6,55E+02	0,00E+00	6,55E+02	5,28E+05	7,86E+05	1,31E+06
binnenwand d	31446,5	0	-5	0	8,41E+07	7,86E+05	8,48E+07	6,55E+02	0,00E+00	6,55E+02	5,28E+05	7,86E+05	1,31E+06
binnenwand e	31446,5	0	-15	0	8,41E+07	7,08E+06	9,11E+07	6,55E+02	0,00E+00	6,55E+02	5,28E+05	7,08E+06	7,60E+06
buitenwand f langs	57797,55	0	-25	0	1,58E+08	3,61E+07	1,94E+08	3,90E+03	0,00E+00	3,90E+03	9,71E+05	3,61E+07	3,71E+07
buitenwand 1 dwars	15687,45	90	0	0	4,28E+07	0,00E+00	4,28E+07	3,39E+06	1,27E+08	1,30E+08	2,64E+05	1,27E+08	1,27E+08
binnenwand 2 dwars	8590,25	75	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	4,83E+07	5,00E+07	1,44E+05	4,83E+07	4,85E+07
binnenwand 3	8590,25	60	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	3,09E+07	3,27E+07	1,44E+05	3,09E+07	3,11E+07
binnenwand 4	8590,25	45	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	1,74E+07	1,91E+07	1,44E+05	1,74E+07	1,75E+07
binnenwand 5	8590,25	30	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	7,73E+06	9,46E+06	1,44E+05	7,73E+06	7,88E+06
binnenwand 6	8590,25	15	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	1,93E+06	3,66E+06	1,44E+05	1,93E+06	2,08E+06
binnenwand 7	8590,25	0	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	0,00E+00	1,73E+06	1,44E+05	0,00E+00	1,44E+05
binnenwand 8	8590,25	-15	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	1,93E+06	3,66E+06	1,44E+05	1,93E+06	2,08E+06
binnenwand 9	8590,25	-30	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	7,73E+06	9,46E+06	1,44E+05	7,73E+06	7,88E+06
binnenwand 10	8590,25	-45	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	1,74E+07	1,91E+07	1,44E+05	1,74E+07	1,75E+07
binnenwand 11	8590,25	-60	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	3,09E+07	3,27E+07	1,44E+05	3,09E+07	3,11E+07
binnenwand 12	8590,25	-75	0	0	2,30E+07	0,00E+00	2,30E+07	1,73E+06	4,83E+07	5,00E+07	1,44E+05	4,83E+07	4,85E+07
buitenwand 13 dwars	15687,45	-90	0	0	4,28E+07	0,00E+00	4,28E+07	3,39E+06	1,27E+08	1,30E+08	2,64E+05	1,27E+08	1,27E+08
Ponton	**781600,2**						**2,234E+09 kN m^2**			**6,085E+08 kNm^2**			**5,609E+08**

Er geldt:
Ixx = 1/12yx^3
Iyy = 1/12zy^3
Izz = 1/12yz^3
lees voor x, y, z dan l, b, h

Allen volgens die benaderingsmethode van MT geldt de volgende Steiner-regel:
Ixxtotaal = Ixx + M*(y^2 + z^2)
Iyytotaal = Iyy + M*(x^2 +z^2)
Izztotaal = Izz + M*(x^2 +y^2)

Ponton:	**Kxx^2 = Ixx/M**	kxx =	54 m
	Kyy^2 = Iyy/M	kyy =	28 m
	Kzz^2 = Izz/M	kzz =	27 m

Aanvangsstabiliteit DWARS:

diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	8	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	1987973,9	m4
V = volume waterverpl	78044,52	m3
afstand BM = I / V	25,47	m
KM= KB+BM	29,71	m
GM=hm= KM-KG	21,71	m

Controle: De eis voor de statische stabiliteit is, dat M boven G ligt, ofwel dat hm groter is dan 0,5 m.
Aan deze eis wordt voldaan. Het drijflichaam is statisch stabiel.

Conclusie:
De constructie is zwaar overgedempt. Gm >> 1,0 m.
Dit heeft tot gevolg dat de bak niet zal gaan zwaaien, maar kan gaan trillen met een hoge frequentie en kleine amplitude

Aanvangsstabiliteit LANGS:

diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	8	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	25110319,4	m4
V = volume waterverp	78044,52	m3
afstand BM = I / V	321,74	m
KM= KB+BM	325,98	m
GM=hm= KM-KG	317,98	m

Stabiliteit na ballasten transport DWARS

diepgang bak	15,00	m
vrijboord	1,50	m
KB	7,25	m
KG	8,00	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	1987973,9	m4
V = volume waterverpl	138117,15	m3
afstand BM = I / V	14,39	m
KM=KB+BM	21,64	m
GM=hm= KM-KG	13,64	m

Stabiliteit tijdens transport LANGS

diepgang bak	15,00	m
vrijboord	1,50	m
KB	7,25	m
KG	8,00	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	25110319,4	m4
V = volume waterverp	138117,15	m3
afstand BM = I / V	181,80	m
KM=KB+BM	189,05	m
GM=hm= KM-KG	181,05	m

Stabiliteit DWARS operationale fase:		
(Eindfase incl belasting)		
diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	5,76	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	1987973,9	m4
V = volume waterverpl	78044,52	m3
afstand BM = I / V	25,47	m
KM=KB+BM	29,71	m
GM=hm= KM-KG	23,95	m

ofwel dat hm groter is dan 0,5 m.
Aan deze eis wordt voldaan. Het drijflichaam is statisch stabiel.

Stabiliteit LANGS operationale fase:		
diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	5,76	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	25110319,4	m4
V = volume waterverp	78044,52	m3
afstand BM = I / V	321,74	m
KM=KB+BM	325,98	m
GM=hm= KM-KG	320,22	m

comp 15x15x16 m

Ii T waterplane	1035,99	m4	sigma Ii T	62159,69	m4
Ii L waterplane	2413,49	m4	sigma Ii L	144809,69	m4

I = Iu - sigma Ii

Stabiliteit transport DWARS		
incl waterballast		
diepgang bak	15,00	m
vrijboord	1,00	m
KB	7,25	m
KG	8,00	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I = Iu - sigma Ii	1925814,2	m4
V = volume waterverpl	138117,15	m3
afstand BM = I / V	13,94	m
KM=KB+BM	21,19	m
GM=hm= KM-KG	13,19	m

Stabiliteit transport LANGS		
incl waterballast		
diepgang bak	15,00	m
vrijboord	1,00	m
KB	7,25	m
KG	8,00	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I = Iu - sigma Ii	24965509,7	m4
V = volume waterverplaa	138117,15	m3
afstand BM = I / V	180,76	m
KM=KB+BM	188,01	m
GM=hm= KM-KG	180,01	m

Stabiliteit DWARS operationale fase:		
incl waterballast		
diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	5,76	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	1925814,2	m4
V = volume waterverpl	78044,52	m3
afstand BM = I / V	24,68	m
KM=KB+BM	28,91	m
GM=hm= KM-KG	23,15	m

Stabiliteit LANGS operationale fase:		
incl waterballast		
diepgang bak	8,48	m
vrijboord	7,52	m
KB	4,24	m
KG	5,76	m
hoogte bak	16	m
halve hoogte bak	8	m
Lengte bak	180,9	m
Breedte bak	50,9	m
I dwarsdoorsnede	24965509,7	m4
V = volume waterverplaa	78044,52	m3
afstand BM = I / V	319,89	m
KM=KB+BM	324,13	m
GM=hm= KM-KG	318,37	m

Tijdens transport				
waterverplaatsing	V=	138117,15	m^3	
gewicht betonconstructie	G =	767554,58	kN	excl. ballast
KB		7,25	m	
KG		8,00	m	
afstand	BM =	14,39	m	I / V
hoogte metacentrum GM	GM =	13,64	m	
Ipolair = Iyy + Izz =	Ip =	1,17E+09	kNm2	
A dwarsdoorsnede	A =	9207,81	m^2	
M per m lengte	M =	767555	kN	
polaire massatraagheidsstraal van het element	k =	39,03	m	sqrt (Ipolair / A dwarsdrsn)
versnelling zwaartekracht	g =	9,81	m/s^2	
eigenperiode van het element	To =	35,80	sec	(2*pi*j) / (sqrt(hm*g))
eigenfrequentie van het element	fo = 1 / To	0,0279	Hz	laag genoeg

Tijdens Operatie				
waterverplaatsing	V=	78044,52	m^3	
gewicht betonconstructie	G =	970338,01	kN	incl. ballast
KB =		4,24	m	
KG =		5,76	m	
afstand	BM =	25,47	m	I / V
hoogte metacentrum GM	GM=	23,95	m	
Ipolair = Iyy + Izz =	Ip =	1,17E+09	m^4	
A dwarsdoorsnede	A =	9207,81	m^2	
M per m lengte	M =	767555	kN	
polaire massatraagheidsstraal van het element	k =	68,08	m	sqrt (Ipolair / A dwarsdrsn)
versnelling zwaartekracht	g =	9,81	m/s^2	
eigenperiode van het element	To =	40,25	sec	(2*pi*j) / (sqrt(hm*g))
eigenfrequentie van het element	fo = 1 / To	0,0248	Hz	laag genoeg

Dynamic stability roll in operation

rho	1,025
g	9,81
nabla	78044,52
GMt	13,64
Ixx	2234070131
m phi	1,00E+07
omega phi	0,069

Dynamic stability heave in operation

rho	1,025
g	9,81
Aw	9207,81
M	970338,005
m zz	1,00E+06
omega z	0,217

Dynamic stability pitch in operation

rho	1,025
g	9,81
nabla	78044,52
GMl	320,22
Iyy	4683786771
m pitch	1,00E+08
omega pitch	0,229

2. Conceptual design calculations of the terminal

The following are the results of calculations of a rigid single-body terminal, comprised of elements of 420 x 50 x 15,5 m.

Gegevens element:

γ_{water}	10	kN/m^3
γ_{beton}	25	kN/m^3
$\gamma_{ballast}$	18	kN/m^3
L_{bak} =	2100	m
B_{bak} =	500	m
H_{bak} =	15,5	m
D_{vloer} =	0,9	m
D_{dak} =	0,9	m
$D_{buitenwand}$ =	0,9	m
$D_{binnenwand}$ =	0,5	m

Platform

L =	10500	12600	2650	m
B=	5000	5000	500	m
Vbak buiten =	16275000	m^3		
n elem =	110	stuks	allen gelijk	
nzeekade =	11	stuks		

Compartimenten

H_{binnen} =	13,7	m		
aantal comp. In lengterichting =	140			
aantal comp. In breedterichting	50		aantal comp=	7000
$L_{compartiment}$ =	15	m	V per comp. =	1887,175 m^3
$B_{compartiment}$ =	10	m	Vbinnen tot =	13210225 m^3
L_{binnen} =	14,5	m	A per comp =	137,75 m^2
B_{binnen} =	9,5	m	Abinnen tot =	964250 m^2

Beton:

Vvloer =	945000	m^3		
Vdak =	945000	m^3		
Vbuitenwanden =	64071,612	m^3		
Vbinnenwanden =	1155294,285	m^3		
Vtotaal =	3109365,897	m^3		
Gewicht beton =50*Gelem	87934429	kN	41873,54	kN/m1

Betoneigenschappen

	f'ck	f'b	fb	fbm	(N/mm^2) E'b	(N/mm^2) Elzz
B45	45	27	1,65	3,3	33500	9,44E+20
B55	55	33	1,9	3,8	36000	1,01E+21
B65	65	39	2,15	4,3	38500	1,09E+21
S355					2,10E+05	

Belastingen:

Kraan10 x 25600 kN =	256000	kN		
Kraan 6 x 17600 kN =	105600	kN		
Verkeer =	453000	kN		
verkeer aslast =	30000	kN		
Overige kranen =	728000	kN		
Extra achterlandinfra+faciliteiten	125000	kN		
Opslag =	48000000	kN		
	49697600		23665,52	kN/m1
Totaal gewicht = Fneerw	137632029	kN	65539,06	12,751 m CG ok 2,749 m bk ponton 0,357 m ow CoG

diepgang excl belasting =	8,37					
vrijboord excl belasting =	7,13					
diepgang incl belasting =	13,11	m				
vrijboord incl belasting =	2,39	m	Hs =	2,06 m	gamma=	1,16
	15,4%		vrijboord voldoende			

Fopw excl belasting =	87934429	kN
Fopw incl belasting=	137632029	kN

	γ_{water}	10	kN/m^3
	γ_{beton}	25	kN/m^3
zand, te verpompen	$\gamma_{ballast}$	18	kN/m^3
Alternatief?	L_{bak} =	420,9	m
Breedte	B_{bak} =	50,9	m
Hoogte	H_{bak} =	15,5	m

	D_{vloer} =	0,9	m
	D_{dak} =	0,9	m
	$D_{buitenwand}$ =	0,9	m
	$D_{binnenwand}$ =	0,5	m
	Dtussenschot =	0,5	m
	M ponton =	1758688,58	kN
	Mcomp =	**12562,06**	kN

Massieve (=gesloten) drijvende constructie: tpv tussenschot

A tussenschot		0	m^2
1/12bh^3	Iy = Izz	15795,44	m^4
1/12hb^3	Iz = Iyy	170334,96	m^4
	Ip = Iy + Iz =	186130,40	m^4

Centre of Gravity	CG (x, y, z) :	210,45		25,45	7,75 m
traagheidsstraal	i= sqrt Iy/A = Iy/M		m		
i^2 = Iy/A	i= sqrt Iz/A = Iz/M		m		

Doorsnede met compartimenten

	Ix =	2,615E+10	kNm^4
	Iy =	4,68E+09	kNm^4
	Iz =	4,58E+09	kNm^4
	Ip = Iy + Iz =	9,26E+09	kNm^4

Iy = som 1/12 bh^3 + A*z^2 in m4

PLATFORM					1/12Mx^2 x=l			1/12My^2 y=b			1/12Mz^2 z=h		
Deel i	M i (kN)	xC (m)	yC (m)	zC (m)	Ixx eigen kNm2	Isteiner	Ixx	Iyy eigen kNm2	Isteiner	Iyy=Iz	Izzeigen kNm2	Isteiner	Izz = Iy
ponton a1	1758688,58	-840	225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,33E+12	1,33E+12
ponton a2	1758688,6	-420	225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,99E+11	4,04E+11
ponton a3	1758688,6	0	225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	0,00E+00	4,68E+09	4,58E+09	8,90E+10	9,36E+10
ponton a4	1758688,6	420	225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,99E+11	4,04E+11
ponton a5	1758688,6	840	225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,33E+12	1,33E+12
ponton b1	1758688,6	-840	175	0	2,62E+10	5,39E+10	8,00E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,29E+12	1,30E+12
ponton b2	1758688,6	-420	175	0	2,62E+10	5,39E+10	8,00E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,64E+11	3,69E+11
ponton b3	1758688,6	0	175	0	2,62E+10	5,39E+10	8,00E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	5,39E+10	5,84E+10
ponton b4	1758688,6	420	175	0	2,62E+10			4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,64E+11	3,69E+11
ponton b5	1758688,6	840	175	0	2,62E+10	5,39E+10	8,00E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,29E+12	1,30E+12
ponton c1	1758688,6	-840	125	0	2,62E+10	2,75E+10	5,36E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,27E+12	1,27E+12
ponton c2	1758688,6	-420	125	0	2,62E+10	2,75E+10	5,36E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,38E+11	3,42E+11
ponton c3	1758688,6	0	125	0	2,62E+10	2,75E+10	5,36E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	2,75E+10	3,21E+10
ponton c4	1758688,6	420	125	0	2,62E+10	2,75E+10	5,36E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,38E+11	3,42E+11
ponton c5	1758688,6	840	125	0	2,62E+10	2,75E+10	5,36E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,27E+12	1,27E+12
ponton d1	1758688,6	-840	75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,25E+12	1,26E+12
ponton d2	1758688,6	-420	75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,20E+11	3,25E+11
ponton d3	1758688,6	0	75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	9,89E+09	1,45E+10
ponton d4	1758688,6	420	75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,20E+11	3,25E+11
ponton d5	1758688,6	840	75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,25E+12	1,26E+12
ponton e1	1758688,6	-840	25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,24E+12	1,25E+12
ponton e2	1758688,6	-420	25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,11E+11	3,16E+11
ponton e3	1758688,6	0	25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	1,10E+09	5,68E+09
ponton e4	1758688,6	420	25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,11E+11	3,16E+11
ponton e5	1758688,6	840	25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,24E+12	1,25E+12
ponton f1	1758688,6	-840	-25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,24E+12	1,25E+12
ponton f2	1758688,6	-420	-25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,11E+11	3,16E+11
ponton f3	1758688,6	0	-25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	1,10E+09	5,68E+09
ponton f4	1758688,6	420	-25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,11E+11	3,16E+11
ponton f5	1758688,6	840	-25	0	2,62E+10	1,10E+09	2,73E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,24E+12	1,25E+12

ponton g1	1758688,6	-840	-75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,25E+12	1,26E+12
ponton g2	1758688,6	-420	-75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	3,10E+11	3,15E+11	4,58E+09	3,20E+11	3,25E+11
ponton g3	1758688,6	0	-75	0	2,62E+10	9,89E+09	3,60E+10	4,68E+09	0,00E+00	4,68E+09	4,58E+09	9,89E+09	1,45E+10
ponton g4	1758688,6	420	-75	0	2,62E+10	9,89E+09	3,60E+10	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,20E+11	3,25E+11
ponton g5	1758688,6	840	-75	0	2,615E+10	9,89E+09	3,60E+10	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,25E+12	1,26E+12
ponton h1	1758688,6	-840	-125	0	2,615E+10	2,75E+10	5,36E+10	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,27E+12	1,27E+12
ponton h2	1758688,6	-420	-125	0	2,615E+10	2,75E+10	5,36E+10	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,38E+11	3,42E+11
ponton h3	1758688,6	0	-125	0	2,615E+10	2,75E+10	5,36E+10	4,684E+09	0,00E+00	4,68E+09	4,577E+09	2,75E+10	3,21E+10
ponton h4	1758688,6	420	-125	0	2,615E+10	2,75E+10	5,36E+10	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,38E+11	3,42E+11
ponton h5	1758688,6	840	-125	0	2,615E+10	2,75E+10	5,36E+10	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,27E+12	1,27E+12
ponton i1	1758688,6	-840	-175	0	2,615E+10	5,39E+10	8,00E+10	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,29E+12	1,30E+12
ponton i2	1758688,6	-420	-175	0	2,615E+10	5,39E+10	8,00E+10	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,64E+11	3,69E+11
ponton i3	1758688,6	0	-175	0	2,615E+10	5,39E+10	8,00E+10	4,684E+09	0,00E+00	4,68E+09	4,577E+09	5,39E+10	5,84E+10
ponton i4	1758688,6	420	-175	0	2,615E+10	5,39E+10	8,00E+10	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,64E+11	3,69E+11
ponton i5	1758688,6	840	-175	0	2,615E+10	5,39E+10	8,00E+10	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,29E+12	1,30E+12
ponton j1	1758688,6	-840	-225	0	2,615E+10	8,90E+10	1,15E+11	4,684E+09	1,24E+12	1,25E+12	4,577E+09	1,33E+12	1,33E+12
ponton j2	1758688,6	-420	-225	0	2,615E+10	8,90E+10	1,15E+11	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,99E+11	4,04E+11
ponton j3	1758688,6	0	-225	0	2,615E+10	8,90E+10	1,15E+11	4,684E+09	0,00E+00	4,68E+09	4,577E+09	8,90E+10	9,36E+10
ponton j4	1758688,6	420	-225	0	2,615E+10	8,90E+10	1,15E+11	4,684E+09	3,10E+11	3,15E+11	4,577E+09	3,99E+11	4,04E+11
ponton j5	1758688,6	840	-225	0	2,62E+10	8,90E+10	1,15E+11	4,68E+09	1,24E+12	1,25E+12	4,58E+09	1,33E+12	1,33E+12
totaal 50 pontons	87934429												
PLATFORM	**87934429,0**						**3,041E+12 kN m^2**			**3,126E+13 kNm^2**			**3,307E+13**

Ponton:

Kxx^2 = Ixx/Mplatform	kxx =	186	m
Kyy^2 = Iyy/M	kyy =	596	m
Kzz^2 = Izz/M	kzz =	613	m

Ix =	3,041E+12	kNm^4
Iy =	3,13E+13	kNm^4
Iz =	3,31E+13	kNm^4
Ip = Iy + Iz =	6,43E+13	kNm^4

Statische stabiliteit van het drijflichaam

platform 2100x500x15,5 m

Aanvangsstabiliteit DWARS:

diepgang bak	7,37	m
vrijboord	8,13	m
KB	3,69	m
KG	7,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	2104,5	m
Breedte bak	509	m
I dwarsdoorsnede	2,313E+10	m4
V = volume waterverplaatsing	7,895E+06	m3
afstand BM = I / V	2929,41	m
KM= KB+BM	2933,09	m
GM=hm= KM-KG	2925,34	m

Controle: De eis voor de statische stabiliteit is, dat M boven G ligt, ofwel dat hm groter is dan 0,5 m.
Aan deze eis wordt voldaan. Het drijflichaam is statisch stabiel.
Conclusie:
De constructie is zwaar overgedempt. Gm >> 1,0 m.

Aanvangsstabiliteit LANGS:

diepgang bak	7,37	m
vrijboord	8,13	m
KB	3,69	m
KG	7,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	2104,5	m
Breedte bak	509	m
I dwarsdoorsnede	3,954E+11	m4
V = volume waterverpl	7,895E+06	m3
afstand BM = I / V	50077,43	m
KM= KB+BM	50081,11	m
GM=hm= KM-KG	50073,36	m

GM is zeer groot!

Stabiliteit DWARS operationale fase:
(Eindfase incl belasting)

diepgang bak	13,11	m
vrijboord	2,39	m
KB	6,55	m
KG	12,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	2104,5	m
Breedte bak	509	m
I dwarsdoorsnede	2,313E+10	m4
V = volume waterverplaatsing	7,895E+06	m3
afstand BM = I / V	2929,41	m
KM=KB+BM	2935,96	m
GM=hm= KM-KG	2923,21	m

Stabiliteit LANGS operationale fase:

diepgang bak	13,11	m
vrijboord	2,39	m
KB	6,55	m
KG	12,75	m
hoogte bak	15,5	m
halve hoogte bak	7,75	m
Lengte bak	2104,5	m
Breedte bak	509	m
I dwarsdoorsnede	3,954E+11	m4
V = volume waterverpl	7,895E+06	m3
afstand BM = I / V	50077,43	m
KM=KB+BM	50083,98	m
GM=hm= KM-KG	50071,23	m

Dynamische stabiliteit van het drijflichaam

Tijdens Operatie				
waterverplaatsing	V=	13763202,9	m^3	
gewicht betonconstructie	G =	87934429	kN	incl. ballast
KB		6,55	m	
KG		12,75	m	
afstand	BM =	2929,41	m	I / V
hoogte metacentrum GM	hm =	2923,21	m	
Ipolair = Iyy + Izz =	Ip =	6,43E+13	kNm2	
A dwarsdoorsnede	A =	1050000	m^2	
M per m lengte	M =	87934429	kN	
polaire massatraagheidsstraal van het element	k =	855,12	m	sqrt (Ipolair / A dwarsdrsn)
versnelling zwaartekracht	g =	9,81	m/s^2	
eigenperiode van het element	To =	31,73	sec	(2*pi*k) / (sqrt(hm*g))
eigenfrequentie van het element	fo = 1 / To	0,0315	Hz	laag genoeg

Een grote traagheidsstraal veroorzaakt een grote eigenperiode en dus een kleine eigenfrequentie,
die ruim lager moet zijn dan de deiningsfrequentie van de golven.
Het platform als geheel wordt niet getransporteerd+ alleen de elementen (deels over zee).
In een haven komt een breed golfspectrum binnen, waarvan de korte golven overheersen.
Van deze korte golven wordt een periode van 3 tot 10 seconden geschat,
welke een frequentie opleveren van respectievelijk 0,7 tot 2,0 Hz.
Langere golven hebben een nog veel lagere frequentie.
Deze worden inprincipe gebroken bij binnenkomst in de Westerschelde.
De eigenfrequentie ligt ver buiten dit bereik- dus veilig.
De zeer grote hm-waarde veroorzaakt een kleinere To dan voor het ponton!

3. Conceptual design pontoon 180x50x16 m

An optimisation sheet has been made to give a clear overview of the conceptual design to be made in the next part of this thesis. The design parameters, dimensions and loads are presented here.

Design parameters

Parameter	Symbol	value	Unit
rho		1025,00	kg/m^3
g		9,81	m/s^2
γ_{water}		10,06	kN/m^3
γ_{beton}		25	kN/m^3
$\gamma_{ballast}$		18	kN/m^3
General			
Sea bottom level at site			
	LLWS	-2,07	m
	MHWS	2,83	m
	HHW 1/1000	5,3	m
required freeboard	F	1	m
req. Underkeel clearance		2	m
trim water ballast			ton
slope of seabed		0	deg
gravity constant	g	9,81	m/s2
Waves			
significant wave height at max WL	Hs max	2,37	m
significant wave height at min WL	Hs min	1,78	m
significant wave height offshore	Hs sea	7,07	m
reflection coefficient	Kr	2	
transmission coefficient		0,3	
angle of wave incidence		165	deg
Peak period	Tp	10	s
peak wave frequency	w p	0,628	
Wind			
Max wind velocity	Uw	31,2	m/s
wind coefficient	Cw	1	
density air	r air	1,23	kg/m3
fetch	f	500	m

Parameter	Symbol	value	Unit
Swell (assumption)			
range wave height		0,2-0,5	m
significant wave height	Hs	0,5	m
smalle period	T	8	s
peak period	Tp	12	s
Current			
max flood current velocity		1,65	m/s
max ebb current velocity		1,35	m/s
Density sea water		1030	kg/m3
Passing ships			
passing speed		15	knots
		7,5	m/s
produced wave height		0,5	m
water level depression		0,48	m
Structural material			
Concrete quality		B65	
steel quality		S355	
reinforcement steel quality		FeB500	
prestressing steel quality		FeP1860	
Density reinforced concrete		2500	kg/m3
Density prestressed reinforced concrete		2650	kg/m3
Density reinforcement steel		7850	kg/m3
Density prestressing steel		7850	kg/m3
acceptable amount of reinforcement		300	kg/m3
Ballast			
Density ballast water	r water	1000	kg/m3
density ballast wet sand	r sand	1800	kg/m3
density ballast wet iron ore	r ore	3500	kg/m3
angle of internal friction	f ballast	25	deg
max percentage of comp filled		50	%

Dimensions:

Parameter	Symbol	value	Unit	
Platform terminal:				stramien
Length	L	2168,4	m	2160
Width	B	507	m	500
Height	H	16	m	
nr of pontoons l	nl	12		
nr of pontoons b	nt	10		
Total nr of pontoons	n	120		
radius of gyration x	kxx =	146	m	
radius of gyration y	kyy =	622	m	
radius of gyration z	kzz =	638	m	
Pontoon element:				
L_{outer} =	L	180,7	m	180
B_{outer} =	B	50,7	m	50
H_{outer} =	H	16	m	
Draught	T		m	
D_{floor} =		0,7	m	
D_{roof} =		0,7	m	
$D_{outer\ wall}$ =		0,7	m	
$D_{inner\ wall}$ =		0,5	m	
Compartments				
H_{inner} =		14,6	m	
nr of comp. longitudinal =	nl	12		
nr of comp. transversal =	nt	5		
total nr of compartments =	n	60		
$L_{compartment}$ =		15	m	
$B_{compartment}$ =		10	m	
L_{inner} =		14,5	m	
B_{inner} =		9,5	m	
radius of gyration x	kxx =	54	m	
radius of gyration y	kyy =	28	m	
radius of gyration z	kzz =	27	m	

Ix =	2,23E+09	kNm^4
Iy =	6,08E+08	kNm^4
Iz =	5,61E+08	kNm^4
Ip = Iy + Iz =	1,17E+09	kNm^4

Beton:		
Vvloer =	6413,043	m^3
Vdak =	6413,043	m^3
Vbuitenwanden =	4701,2	m^3
Vbinnenwanden =	9033,75	m^3
Vtotaal =	26561,036	m^3
Gewicht beton = Fneerw	664025,9	kN

Zwaartepunt tijdens transport ballast

G beton	664025,9	kN
KZ eigen	8	m
G*x beton	5,31E+06	kN*m
G ballast	5,88E+05	kN
KZ ballast	2,00E+00	m
G*x ballast	1,18E+06	kN*m
zw ligging KZ	5,18	m

Zwaartepunt tijdens operatie

G beton	664025,9	kN
KZ eigen	8	m
G*x beton	5,31E+06	kN*m
G dekbelasting of ballast	2,03E+05	kN
KZ dekbelasting	17,00	m
G*x ballast	3,45E+06	kN*m
zw ligging KZ	10,11	m

Loads on the terminal:

Parameter	Symbol	value	Unit			
Self weight						
γ_{beton}		25,00	kN/m^3			
Gbeton incl comp		3674,74	kN/m1 l			
rustende bel		1,50	kN/m2			
Gantry crane facilities						
combination factor	psi	0,60				
vertical wheel load		800,00	kN			
vertical line load		800,00	kN/m1			
wheel system load		6400,00	kN			
total crane load		25600,00	kN			
horizontal perpendicular		80,00	kN/m1			
Storage						
combination factor		0,67				
stacked 5 levels high P=		60,00	kN/m^2			
emergency storage on apron		40,00	kN/m^2			
Traffic					total	
combination factor		0,50				
VK 60 p=		3,00	kN/m^2			
axes loads 3x		200,00	kN/axis	3	600,00	kN
						kN
Straddle carrier 2x4x270 kN		270,00	kN/wheel	8	2160,00	kN
reachstacker/heftruck axis		1000,00	kN/axis	2	2000,00	kN
Mobile crane 2600 kN/strut	2600	433,33	kN/m^2 over 6m^2	4	10400,00	kN
Ballast load						
rho		1025,00	kg/m^3			
g		9,81	m/s^2			
$\gamma_{ballast}$		18,00	kN/m3			
$\gamma_{ballastwater}$		10,06	kN/m3			
h water ballast		7,20	m	max at transport		
h sand ballast		4,00	m			
floor load water ballast		72,40	kN/m2			
floor load sand ballast		72,00	kN/m2			

Parameter	Symbol	value	Unit	
Hydrostatic load				
on outer walls				
max draught		13,44		
rho		1025,00	kg/m^3	
g		9,81	m/s^2	
pressure rho*g*h		135,14	kPa=kN/m2	
F = 1/2*rho*g*h^2		908,16	kN	
Water level depression-passing ship				
delta operational phase =		0,47	m	locally?
Fopw =		927061,30	kN	
Lokaal minder Fopw=buiging +trek				
Wave loads (non breaking on wall)				
[S. van Baars; Handbook hydraulic engineering ct 3330]				
g		9,81	m/s^2	
rho		1025,00	kg/m^3	
water depth h		20,00	m	
shortest wave T		3,10	s	
Longest wave operational phase Tp		6,20	s	
omega short		2,03	rad/s	
omega long		1,01	rad/s	
Lo		60,02	m	
L short		15,00	m	
Wave length operational phase L		59,40	m	
Hs		2,37	m	
T		13,11	m	
rule of thumb:				
F1		28,24	kN/m1	
F2		312,42	kN/m1	
F total		**340,66**	kN/m1	
Sainflou:				
k short		0,11		
k long		0,42		
p1		23,83	kN/m2	
p0		5,66	kN/m2	
		0,01	kN/m2	
Ftotal		**323,20**	kN/m1	
Linear Theory:				
-d<z<0:				
p		7,24	kN/m2	
0<z<Hi:				
pmax at z=0		23,83	kN/m2	
(at z=Hi, p=0)				
Ftotal		222,65	kN/m1	

Parameter	Symbol	value	Unit	
Wind load				
During operation: ULS survival				
wind speed max 1/1000 yrs		31,20	m/s	
wind direction at max speed		270,00	deg N	
pw at 30 m area2 in NL		1,24	kN/m2	
pw at <7 m area2 in NL		0,78	kN/m2	
Current load				
during transport:				
flood current u =		1,65	m/s	
ebb current u =		1,35	m/s	
Out at sea? nearshore u =		3,00	m/s	
During operation:				
Max flood current u =		1,50	m/s	
Max. ebb current u =		1,20	m/s	
F resulting=1/2rho.u^2		1395,28	N/m2	
Fdrag		19207,00	kN	
Flift		3658,00	kN	
Drift forces	*[Journee and Massie; Offshore Hydromechanics]*			
g		9,81	m/s^2	
gamma		1025,00	kg/m^3	
water depth h		20,00	m	
Hs		2,37	m	
H1/3		2,37	m	
m0_dzeta (JONSWAP)		0,37	m2	
reflection coefficient R		2,00		
significant amplitude		1,19	m	
regular waves: infinitely long cylinder				
Maruo [1960]				
mean wave drift force F		28,24	kN/m1	
Total anchorage force		59303,30	kN	2100 m
Wave drift force from irregular waves:				
F = rho.g.m0		3,72	kN/m1	
Total anchorage force		7812,93	kN	
F= 1/16.rho.g.H1/3		3,53	kN/m1	
Total anchorage force		7412,91	kN	2100m
Fd=P1+P2/2 * cos((w1-w2)*t)	to be extended			

Parameter	Symbol	value	Unit	
Froude-Krylov force	(as a check on other calculations)			
Ffk = c*dzeta*				
dzeta*=effective wave elevation				
Lponton		180,00	m	
Bponton		50,00	m	
c=rho.g.L.B		90497,25		
deep water:				
dzeta*		0,85		
Ftotal		77259,39	kN	
distributed evenly over pontoons		643,83	kN	(minimum)
to be extended &				
To be compared with wave forces from Delfrac				
mooring forces from fixation system				
piles along long side of terminal				
centre to centre		90,00	m	
number		24		
per pontoon		2		incl 2 * 1/2 = gedeeld met volgend ponton
piles along head short side terminal				
centre to centre		50,00	m	
number		10		
estimated force per pile		1732,33	kN	
max			kN	
estimated force per adjoining pontoo		3464,65	kN	
max (corner pontoon)		0,00	kN	
apron pontoons (min load)		643,83	kN	
pile in ground at point A and point load from pontoon in point P				
distance AP =	a	23,00	m	
distance P to top =	b	2,00	m	
length pile above bottom	l	25,00	m	
Shear force Va=F				
Moment Ma = F*a				
Tube D	D	2,20	m	
tube steel thickness	t	0,05	m	
I = 0,4D^3*t	I	0,21	m^4	
W = 0,8D^2*t	W	0,19	m^3	
yield stress	fy;d	355,00	N/mm^2	
Mu = fy * W	Mu	68728,00	kNm	
Fu = Mu / a	Fu	2988,17	kN	
safety factor gamma		1,50	-	
	Fd <	1992,12	kN	> estimated force per pile
Per pontoon		3465	kN	sufficient

Parameter	Symbol	value	Unit	
distribution of mooring forces from fixation system				
distributed load over pontoon length		19	kN/m^1	
introduced via stiff element connectors				
number of rows of elements		10		
av 50% of load reaches front elemer		19	kN/m^1	
Bolder load (troskracht)				
per bolder		2000,00	kN	
centre to centre		30,00	m	critical distance
		66,67	kN/m1	
Fender load (aanvaring)				
Per fender set		5500,00	kN	
centre to centre		15,00	m	critical distance
per toggle hole		150,00	kN	
centre to centre		15,00	m	
mobile cranes and transport means				
Point/Force loads Mv max:				
Traffic		200	kN	
		200	kN	
		200	kN	
Stackers: 2 axes		1000	kN	
		1000	kN	
Staddle carrier: 4x2 wheels		540	kN	
		540	kN	
		540	kN	
		540	kN	
Mobile crane; 4 struts		2600	kN/6m^2	
		2600	kN/6m^2	
		2600	kN/6m^2	
		2600	kN/6m^2	

Appendix Part 3:

Hydrodynamics of the Floating Terminal

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1. Short manual on input Delfrac

This manual has been written with the help of Ir. J. van Kessel, during the making of several models for Delfrac. It is still in Dutch:

HANDLEIDING INPUTMODEL GENEREREN VOOR DELFRAC - door Gina de Rooij

Een tekstbestand *.inp wordt als input gebruikt voor het programmaatje haven3.
Dit bestand wordt door haven3 verwerkt tot een *.dat bestand,
Dat als input voorde viewer en voor DELFRAC kan dienen.

Open MS-DOS prompt:
C:\>DIR
C:\>folder naam>dir p (alles tonen)
type file.inp [enter]

In MS dos zijn alle bewerkingen en foutmeldingen zichtbaar die anders niet zichtbaar zijn
bij dubbelklik-openen vanuit Windows.

MANUAL door Jan van Kessel:

haven3.exe = maken van model
input file: [test2.inp]

[TEST2.DAT] wordt gemaakt door het programma

[TEST2.DAT] kan je vervolgens in de viewer stoppen...

########
Viewer: GFAWIN37.EXE
file --> load: MULTVW3.GFW
run
number of bodies? {vooralsnog 1} [enter]
TEST2.DAT (aanklikken)

Uiteindelijk komt het model op het scherm.
met volgende toetsen kan je wat spelen:
y=yaw
p=pitch
r=roll
pijltjes
+/- = zoom
f = model vullen met panelen. Kan je zien of normalen in juiste richting staan: wit moet buitenzijde van je ponton zijn & grijs moet binnenzijde zijn (gelinkt aan draairichting)
q=quit

#######
Nu zou het model klaar moeten zijn voor Delfrac....

#####
overige info:
- aantal panelen max. 600 nemen (later is dit verhoogd tot 1000 en tot 4 x 2500)
- zie ook "TEST2 met info.DAT" dit is hetzelfde bestand als TEST2.DAT, maar hier heb ik wat dingen bijgezet
- ca. (min)5 panelen in de lengte van de kortste golf; Lgolf uit vuistregel berekenen; dan Lgolf/5 = ca. Lpaneel; dan bepaal n panelen over Lponton

---

Het model *.DAT is input voor DELFRAC en bevat:
Vorm van 1 element, in Delfrac wordt het gehele platform uit verschillende typen elementen opgebouwd.
Elementen in blokvorm met alle zijden en bv midden in platform met minder wanden?
Verder ook hydraulische rvw: golfrichtingen, stromingssnelheid, golffrequenties.
In principe wordt het onderwaterschip gemodelleerd. Op z=0 bevindt zich automatisch de waterlijn.
Houd hierbij rekening met de waterstanden en diepgang van het element in verschillende situaties.

NB In het *.DAT bestand kunnen ook nog wijzigingen aangebracht worden als .txt bestand
(wel bij opslaan de juiste extensie opgeven)
VB: Wijzigingen in massa-stijfheidsmatrix, traagheidsstralen k, golfrichtingen, stroomsnelheden etc.
NB golfkrachten bepaald door extrapolatie door Delfrac boven waterlijn. Totale hoogte van de constructie is daarbij belangrijk.
Vraag is voorlopig nog, of de diepgang gemodelleerd moet worden of de constructiehoogte.
Zie diktaat Offshore Hydromechanics H5.2.8 Theory "Airy" voor golfkrachten.

Het .INP bestand modelleert een bak met maximaal 5 zijden, dus hol en open aan 1 zijde (bijvoorbeeld de bovenkant). Een gesloten bak kan verkregen worden door een kwart te modelleren (aan te geven in .DAT) met 2 zijkanten missend; een lange en een korte. De gehele constructie is dan aan alle zijden gesloten met panelen.

VOORBEELD TEST2.INP:

```
Dit is een testje
1  1  1   schaalfactoren (bv, slanker+langer maken zonder coo wijzigen)
3            aantal delen
1            deel zijkanten
-1           draairichting (neg) (ivm richting normalen naar buiten)
1  1         startpnt  startpaneel; beide nummeringen starten bij hoekpnt 1
6    np             nr of points; altijd 8 nodig voor rechthoekige gesloten bak
1    0    8    0.225 (x,y,z) z meestal<0
2    0    8   -0.225
3    11   8    0.225   3   1                Vast op deze regel:aantal elem X; aantal elem z
4    11   8   -0.225                       (elk 1e punt van lijnstuk 2 punten)
5    11  0     0.225   2    1               Vast op deze regel:aantal elem y; aantal elem z
6    11  0    -0.225
2     deel bovenkant                       - 0, 1 of 2: symm? 1/2 modelleren?
+1    draairichting  (ivm normalen)
4    np
1    0    8    0.225
2    0    0    0.225
3    11   8    0.225   3    2               bovenvlak; n elem x;y
4    11   0    0.225
2     deel onderkant                       deel zijkant kan ook gedefinieerd worden
-1    draairichting   (ivm normalen)
4    np
1    0    8    -0.225
2    0    0    -0.225
3    11   8    -0.225   3   2               ondervlak; n elem x;y
4    11   0    -0.225
```

NB Dit voorbeeldbestand levert een ponton dat 1 zijkant mist.
one body kan maar 5 zijden aan, dus meestal bovenzijde weglaten en onderzijde en zijkanten definiëren. Als een kwart van het geheel gemodelleerd wordt (zie .DAT-file), moet 1 lange zijkant en 1 korte zijkant weg worden gelaten en de boven- en onderzijde worden gedefinieerd.
elementen zijn rechthoekig, dus 1 extra element betekent minimaal 2 extra hoekpunten
Aantal elem opgeteld geeft 17, hieronder zijn 17 elementen in .DAT aangegeven:

VOORBEELD TEST2info.DAT:

```
Dit is een testje
 0   0   0              verschuiving assenstelsel
  38    17   1   4      aantal (hoek)punten, aantal elementen(panels), symm; modelleer 1/2 of 1/4
                         constructie, nauwkeurigheid=4
 2                      aantal golffrequenties door te rekenen
 .20  .30               golffrequenties; geef intervallen op bv 0,1 – 2,0
 1                      aantal golfrichtingen door te rekenen
 180                    golfrichtingen (graden)
 -1                     waterdiepte; -1 betekent diep, absolute diepten positief in m aangeven
 0.0                    Verticale Zwaartepuntsligging tov waterspiegel z=0
 1.025   9.81           dichtheid water en zwaartekrachtversnelling
 0.0                    Dummy
  1.0    0    0    0    0    0   massamatrix Mxx
    0  1.0    0    0    0    0   massamatrix Myy
    0    0  1.0    0    0    0   massamatrix Mzz
    0    0    0  1.0    0    0   massamatrix kxx          zie diktaat Offshore Hydromechanics H8.2
    0    0    0    0  1.0    0   massamatrix kyy          voor schepen.  Analytisch (bak) ; k = √I/A of
    0    0    0    0    0  1.0   massamatrix kzz          √I/M
    1          0.000    8.000    0.225   Punten x,y,z in meter
    2          0.000    8.000   -0.225
    3          3.667    8.000    0.225
    4          3.667    8.000   -0.225
    5          7.333    8.000    0.225
    6          7.333    8.000   -0.225
    7         11.000    8.000    0.225
    8         11.000    8.000   -0.225
    9         11.000    8.000    0.225
   10         11.000    8.000   -0.225
   11         11.000    4.000    0.225
   12         11.000    4.000   -0.225
   13         11.000    0.000    0.225
   14         11.000    0.000   -0.225
   15          0.000    8.000    0.225
   16          0.000    4.000    0.225
   17          0.000    0.000    0.225
   18          3.667    8.000    0.225
   19          3.667    4.000    0.225
   20          3.667    0.000    0.225
   21          7.333    8.000    0.225
   22          7.333    4.000    0.225
   23          7.333    0.000    0.225
   24         11.000    8.000    0.225
   25         11.000    4.000    0.225
   26         11.000    0.000    0.225
   27          0.000    8.000   -0.225
   28          0.000    4.000   -0.225
   29          0.000    0.000   -0.225
   30          3.667    8.000   -0.225
   31          3.667    4.000   -0.225
   32          3.667    0.000   -0.225
   33          7.333    8.000   -0.225
   34          7.333    4.000   -0.225
   35          7.333    0.000   -0.225
   36         11.000    8.000   -0.225
   37         11.000    4.000   -0.225
   38         11.000    0.000   -0.225
    1      1   3   4   2   elementen: el 1 bestaat uit punten 1,3,4,2
    2      3   5   6   4              volgorde is van belang, ivm richting van de normalen
```

3	5	7	8	6	rechtsom
4	9	11	12	10	
5	11	13	14	12	
6	16	19	18	15	
7	17	20	19	16	
8	19	22	21	18	
9	20	23	22	19	
10	22	25	24	21	
11	23	26	25	22	
12	27	30	31	28	
13	28	31	32	29	
14	30	33	34	31	
15	31	34	35	32	
16	33	36	37	34	
17	34	37	38	35	

2 1 2 Voor berekening driftkrachten; watersnelheden x,y,z
1 1 2 vaak alleen x bekend(stroomsnelheden; gebruik max waarde)
2 3 4 Wanneer geen snelheden opgegeven; 2x0 eronder = 2
dummies
1 1 2

2. Input Delfrac pontoons

ELM18050.inp:

```
WCT ponton 180x50x16 m (D=13.80) met <600 panelen
1  1  1   schaalfactoren
3          aantal delen
1            deel zijkanten
-1           draairichting (neg)
1  1         startpnt  startpan
8    np
1    0       50.9  0      5    2
2    0       50.9 -13.80
3    180.9   50.9  0      18   2
4    180.9   50.9 -13.80
5    180.9   0     0      5    2
6    180.9   0    -13.80
7    0       0    0       18   2
8    0       0    -13.80
2     deel onderkant
-1    draairichting
4    np
1    0       50.9 -13.80
2    0       0    -13.80
3    180.9   50.9 -13.80  18   5
4    180.9   0    -13.80
2     deel  ene zijkant
1     draairichting
4    np
1    0       50.9  0
2    0       50.9 -13.80
3    0       0     0      5    2
4    0       0    -13.80
```

ELM18050.DAT:

```
WCT ponton 180x50x16 m (D=13.80) met <600 panelen
 0   0   0
  264   182   1   4
 20
 .10  .20  .30  .40  .50  .60  .70  .80  .90  1.0  1.1  1.2  1.3  1.4  1.5  1.6  1.7  1.8  1.9  2.0
 7
 180   90   135   165   64   101   119
 20
 -2.716
 1.025   9.81
 0.0
  1.0     0     0     0     0     0
    0   1.0     0     0     0     0
    0     0   1.0     0     0     0
    0     0     0    57     0     0
    0     0     0     0    30     0
    0     0     0     0     0    29
     1            0.000    50.900     0.000
     2            0.000    50.900    -6.900
     3            0.000    50.900   -13.800
     4           10.050    50.900     0.000
     5           10.050    50.900    -6.900
  ….
    260            0.000    10.180    -6.900
    261            0.000    10.180   -13.800
    262            0.000     0.000     0.000
    263            0.000     0.000    -6.900
    264            0.000     0.000   -13.800
     1        1    4    5    2
     2        2    5    6    3
     3        4    7    8    5
     4        5    8    9    6
     5        7   10   11    8
     6        8   11   12    9
     7       10   13   14   11
     8       11   14   15   12
     9       13   16   17   14
    10       14   17   18   15
….
    180      258  261  260  257
    181      260  263  262  259
    182      261  264  263  260
 2   1   2
 1   1   2
 2   3   4
 1   1   2
 0
 0
```

ELM42050.inp:

```
WCT ponton 420x50x16 (D=13.67) m met  <600 panelen
1  1  1   schaalfactoren
3          aantal delen
1            deel zijkanten
-1           draairichting (neg)
1  1         startpnt  startpan
8    np
1    0     50.9  0      5    2
2    0     50.9 -13.67
3    420.9 50.9  0      42   2
4    420.9 50.9 -13.67
5    420.9 0     0      5    2
6    420.9 0    -13.67
7    0     0     0      42   2
8    0     0    -13.67
2     deel onderkant
-1    draairichting
4    np
1    0     50.9 -13.67
2    0     0    -13.67
3    420.9 50.9 -13.67  42   5
4    420.9 0    -13.67
2     deel  ene zijkant
1     draairichting
4    np
1    0     50.9  0
2    0     50.9 -13.67
3    0     0     0      5    2
4    0     0    -13.67
```

ELM42050.DAT:

```
WCT ponton 420x50x16 (D=13.67) m met  <600 panelen
 0   0   0
  552   398   1   4
 20
 .10  .20  .30  .40  .50  .60  .70  .80  .90  1.0  1.1  1.2  1.3  1.4  1.5  1.6  1.7  1.8  1.9  2.0
 7
 180   90   135   165   64   101   119
 20
 -2.555
 1.025   9.81
 0.0
  1.0     0     0     0     0     0
    0   1.0     0     0     0     0
    0     0   1.0     0     0     0
    0     0     0   122     0     0
    0     0     0     0    52     0
    0     0     0     0     0    51
     1            0.000    50.900     0.000
     2            0.000    50.900    -6.835
     3            0.000    50.900   -13.670
     4           10.021    50.900     0.000
     5           10.021    50.900    -6.835
     6           10.021    50.900   -13.670
     7           20.043    50.900     0.000
     8           20.043    50.900    -6.835
     9           20.043    50.900   -13.670
    10           30.064    50.900     0.000
….
   550            0.000     0.000     0.000
    551           0.000     0.000    -6.835
    552           0.000     0.000   -13.670
     1       1    4    5    2
     2       2    5    6    3
     3       4    7    8    5
     4       5    8    9    6
     5       7   10   11    8
….
395      545  548  547  544
   396      546  549  548  545
   397      548  551  550  547
   398      549  552  551  548
 2   1   2
 1   1   2
 2   3   4
 1   1   2
 0
 0
```

3. Input Delfrac rigid terminal

```
 WCT kwart 2100x500x15.5 (D=13.11) m met<2500 panelen
 0   0   0
 2745  2482   4   4
 10
 0.50  0.60  0.70  0.80  0.90  1.00  1.10  1.20  1.30  1.40
 7
 180   90   135   165   64   101   119
 20
 -0.359
 1.025   9.81
 0.0
  1.0     0     0     0     0     0
    0   1.0     0     0     0     0
    0     0   1.0     0     0     0
    0     0     0   186     0     0
    0     0     0     0   596     0
    0     0     0     0     0   613
1  0,000  254,500  0,000
2  0,000  254,500  -6,555
3  0,000  254,500  -13,110
4  15,474  254,500  0,000
5  15,474  254,500  -6,555
6  15,474  254,500  -13,110
7  30,949  254,500  0,000
8  30,949  254,500  -6,555
9  30,949  254,500  -13,110
10  46,423  254,500  0,000
11  46,423  254,500  -6,555
12  46,423  254,500  -13,110
13  61,897  254,500  0,000
14  61,897  254,500  -6,555
15  61,897  254,500  -13,110
16  77,371  254,500  0,000
17  77,371  254,500  -6,555
18  77,371  254,500  -13,110
19  92,846  254,500  0,000
20  92,846  254,500  -6,555
21  92,846  254,500  -13,110
22  108,320  254,500  0,000
23  108,320  254,500  -6,555
24  108,320  254,500  -13,110
25  123,794  254,500  0,000
26  123,794  254,500  -6,555
27  123,794  254,500  -13,110
28  139,268  254,500  0,000
29  139,268  254,500  -6,555
```

30 139,268 254,500 -13,110
….

2725 1036,776 29,941 -13,110
2726 1036,776 14,971 -13,110
2727 1036,776 0,000 -13,110
2728 1052,250 254,500 -13,110
2729 1052,250 239,529 -13,110
2730 1052,250 224,559 -13,110
2731 1052,250 209,588 -13,110
2732 1052,250 194,618 -13,110
2733 1052,250 179,647 -13,110
2734 1052,250 164,676 -13,110
2735 1052,250 149,706 -13,110
2736 1052,250 134,735 -13,110
2737 1052,250 119,765 -13,110
2738 1052,250 104,794 -13,110
2739 1052,250 89,824 -13,110
2740 1052,250 74,853 -13,110
2741 1052,250 59,882 -13,110
2742 1052,250 44,912 -13,110
2743 1052,250 29,941 -13,110
2744 1052,250 14,971 -13,110
2745 1052,250 0,000 -13,110

1	1	4	5	2
2	2	5	6	3
3	4	7	8	5
4	5	8	9	6
5	7	10	11	8
6	8	11	12	9
7	10	13	14	11
8	11	14	15	12
9	13	16	17	14
10	14	17	18	15
11	16	19	20	17
12	17	20	21	18
13	19	22	23	20
14	20	23	24	21
15	22	25	26	23
16	23	26	27	24
17	25	28	29	26
18	26	29	30	27
19	28	31	32	29
20	29	32	33	30
21	31	34	35	32
22	32	35	36	33
23	34	37	38	35
24	35	38	39	36
25	37	40	41	38

….

Reference points:

17 17 4
1 -1052.5 -283
2 -631.35 -283
3 -210.45 -283
4 210.45 -283
5 631.35 -283
6 1052.5 -283
7 -1052.5 -254.5
8 -631.35 -254.5
9 -210.45 -254.5
10 210.45 -254.5
11 631.35 -254.5
12 1052.5 -254.5
13 -1052.5 -203.6
14 -1052.5 -101.8
15 -1052.5 101.8
16 -1052.5 203.6
17 -1052.5 254.5
1 1 1
2 2 2
3 3 3
4 4 4
5 5 5
6 6 6
7 7 7
8 8 8
9 9 9
10 10 10
11 11 11
12 12 12
13 13 13
14 14 14
15 15 15
16 16 16
17 17 17
0
0

4. Input Delfrac connection forces

4	number of construction parts
Pontoon I centre of pier	
0.02	Weight of pontoon in relation to wieght of terminal
0 229.05 -0.359	centre of gravity of pontoon
210.45 203.6 0	reference axes; location of connection forces
122 0 0	kxx pontoon
0 52 0	kyy pontoon
0 0 51	kzz pontoon
2	number of intervals of panel ranges
2209 2320	panel numbers of pontoon in model of terminal
4691 4802	panel numbers of pontoon in model of terminal
Pontoon II 420x50 end of pier	
0.02	
841.800 229.05 -0.359	
631.35 203.6 0	
122 0 0	
0 52 0	
0 0 51	
2	
55 114	
2321 2482	
Pontoon III 180x50 end of pier	
0.009	
961.800 229.05 -0.359	
871.35 203.6 0	
54 0 0	
0 28 0	
0 0 27	
3	
85 114	
2366 2401	
2447 2482	
Pontoon VI 180x50 end of pier 4th quadrant	
0.009	
961.800 -229.05 -0.359	
871.35 -203.6 0	
54 0 0	
0 28 0	
0 0 27	
3	
7531 7560	
9812 9847	
9893 9928	

NEW
panels pontoon a:centre pier
2209-2320 = 112 st klopt

panels pontoon B:420x50 end pier
55-114 = 60
2321-2482 = 162
total 222 st klopt

panels pontoon c: 180x50 end pier
2366-2401
2447-2482 total=102 klopt
85-114

2nd quadrant

c old		c new	
85	114	2567	2596
2366	2401	4848	4883
2447	2482	4929	4964

3rd quadrant

c old		c new	
2567	2596	5049	5078
4848	4883	7330	7365
4929	4964	7411	7446

4th quadrant

c old		c new	
5049	5078	7531	7560
7330	7365	9812	9847
7411	7446	9893	9928

2nd quadrant

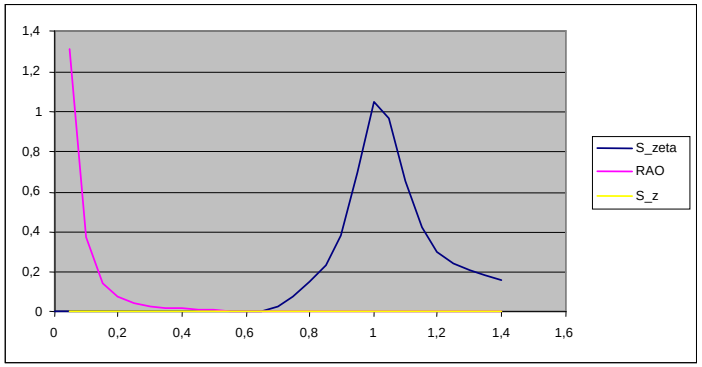
a old		a new	
2209	2320	4691	4802

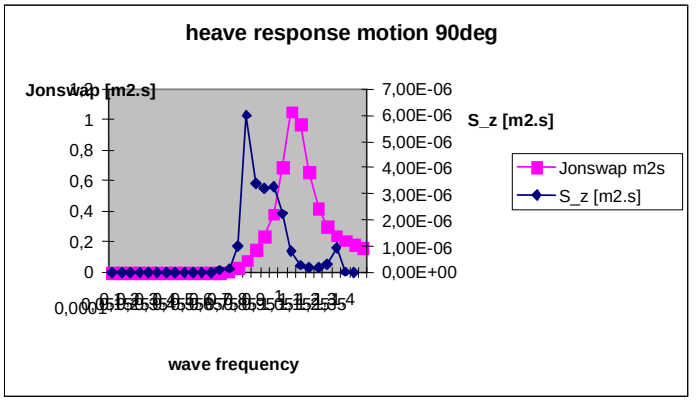
5. Spectral analysis

Response motions:

Computation amplitude response motions from RAOs

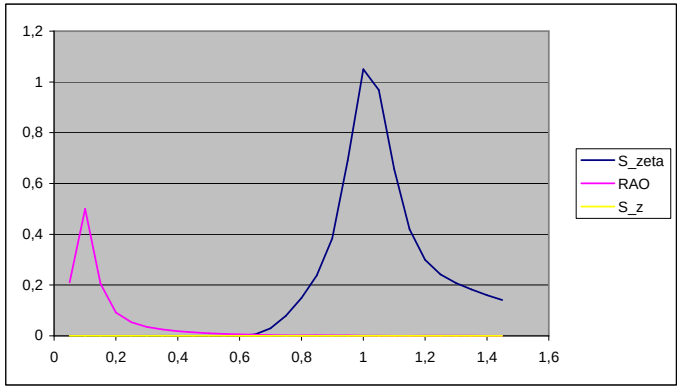
					RAO		Simpson Integration		
Hs	2,37								
Tp	**6,2**								
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Heave (90)**	**S_z**	**Sf**	**J*L**	
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[m/m]**	**[m2.s]**	**[-]**		
		0,0001	4,9315E-45	0					
y0	ordinaatafstand=	0,05	8,8975E-41	0	1,31E+00	0,00E+00	1	0,00E+00	
		0,1	9,9666E-37	0	3,72E-01	0,00E+00	4	0,00E+00	
n ordinaten>21		0,15	6,7932E-33	0	1,42E-01	0,00E+00	2	0,00E+00	
nauwkeurig genoeg		0,2	2,8174E-29	0	7,72E-02	0,00E+00	4	0,00E+00	
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	4,82E-02	5,50E-147	2	1,10E-146	
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	3,20E-02	8,98E-72	4	3,59E-71	
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	2,27E-02	7,68E-40	2	1,54E-39	
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	1,69E-02	1,39E-24	4	5,54E-24	
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	1,27E-02	1,12E-16	2	2,24E-16	
		0,5	4,2244E-12	2,6313E-08	9,73E-03	2,49E-12	4	9,95E-12	
		0,55	5,4111E-10	1,31823E-05	7,43E-03	7,27E-10	2	1,45E-09	
		0,6	4,2175E-08	0,000591608	5,52E-03	1,80E-08	4	7,20E-08	
		0,65	2,0002E-06	0,006453889	3,86E-03	9,62E-08	2	1,92E-07	
		0,7	5,7724E-05	0,029687796	2,28E-03	1,54E-07	4	6,17E-07	
		0,75	1,0136E-03	0,079237255	3,53E-03	9,86E-07	2	1,97E-06	
		0,8	1,0831E-02	0,149970469	6,33E-03	6,01E-06	4	2,40E-05	
		0,85	7,0416E-02	0,237992026	3,78E-03	3,40E-06	2	6,80E-06	
		0,9	2,7858E-01	0,384391531	2,89E-03	3,20E-06	4	1,28E-05	
		0,95	6,7060E-01	0,692667608	2,17E-03	3,27E-06	2	6,54E-06	
		1	9,8227E-01	1,050203761	1,46E-03	2,23E-06	4	8,93E-06	
		1,05	9,2271E-01	0,968406146	9,20E-04	8,20E-07	2	1,64E-06	
		1,1	6,3726E-01	0,656279016	6,46E-04	2,74E-07	4	1,10E-06	
		1,15	3,2587E-01	0,419635266	6,32E-04	1,67E-07	2	3,35E-07	
		1,2	1,2339E-01	0,299749077	7,68E-04	1,77E-07	4	7,07E-07	
		1,25	3,4591E-02	0,241942631	1,14E-03	3,14E-07	2	6,28E-07	
		1,3	7,1805E-03	0,208169947	2,10E-03	9,17E-07	4	3,67E-06	
		1,35	1,1036E-03	0,182571285	3,44E-04	2,17E-08	2	4,33E-08	
		1,4	1,2560E-04	0,160440775	1,25E-04	2,52E-09	4	1,01E-08	
y28		1,45	1,0583E-05	0,140796905	7,69E-05	8,33E-10	1	8,33E-10	
								7,01E-05	
						m0		1,17E-06	m^2
						Z_1/3		**2,16E-03**	m
						H_1/3		4,32E-03	m
					Statistisch:	**Z_max**		**4,02E-03**	m

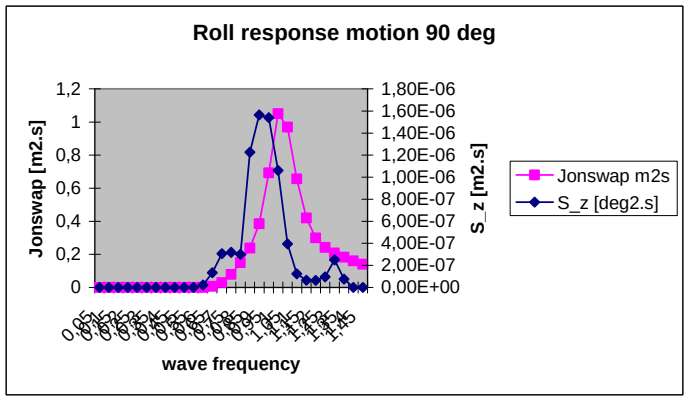




Computation amplitude response motions from RAOs

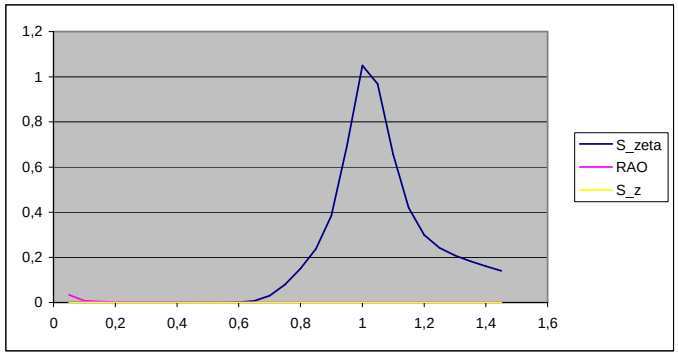
Hs	2,37							
Tp	**6,2**				**RAO**		**Simpson Integration**	
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Roll (90)**	**S_z**	**Sf**	**J*L**
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[deg/m]**	**[deg2.s]**	**[-]**	
		0,0001	4,9315E-45	0				
	ordinaatafstand=	0,05	8,8975E-41	0	2,11E-01	0,00E+00	1	0,00E+00
		0,1	9,9666E-37	0	5,01E-01	0,00E+00	4	0,00E+00
n ordinaten>21		0,15	6,7932E-33	0	2,06E-01	0,00E+00	2	0,00E+00
nauwkeurig genoeg		0,2	2,8174E-29	0	9,22E-02	0,00E+00	4	0,00E+00
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	5,40E-02	6,91E-147	2	1,38E-146
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	3,59E-02	1,13E-71	4	4,51E-71
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	2,48E-02	9,16E-40	2	1,83E-39
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	1,80E-02	1,57E-24	4	6,27E-24
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	1,36E-02	1,28E-16	2	2,56E-16
		0,5	4,2244E-12	2,6313E-08	1,03E-02	2,80E-12	4	1,12E-11
		0,55	5,4111E-10	1,31823E-05	7,92E-03	8,28E-10	2	1,66E-09
		0,6	4,2175E-08	0,000591608	6,10E-03	2,20E-08	4	8,80E-08
		0,65	2,0002E-06	0,006453889	4,56E-03	1,34E-07	2	2,68E-07
		0,7	5,7724E-05	0,029687796	3,21E-03	3,07E-07	4	1,23E-06
		0,75	1,0136E-03	0,079237255	2,01E-03	3,19E-07	2	6,39E-07
		0,8	1,0831E-02	0,149970469	1,42E-03	3,01E-07	4	1,20E-06
		0,85	7,0416E-02	0,237992026	2,27E-03	1,23E-06	2	2,45E-06
		0,9	2,7858E-01	0,384391531	2,02E-03	1,56E-06	4	6,26E-06
		0,95	6,7060E-01	0,692667608	1,49E-03	1,54E-06	2	3,08E-06
		1	9,8227E-01	1,050203761	1,01E-03	1,06E-06	4	4,24E-06
		1,05	9,2271E-01	0,968406146	6,38E-04	3,94E-07	2	7,88E-07
		1,1	6,3726E-01	0,656279016	4,35E-04	1,24E-07	4	4,97E-07
		1,15	3,2587E-01	0,419635266	3,99E-04	6,68E-08	2	1,34E-07
		1,2	1,2339E-01	0,299749077	4,61E-04	6,37E-08	4	2,55E-07
		1,25	3,4591E-02	0,241942631	6,28E-04	9,53E-08	2	1,91E-07
		1,3	7,1805E-03	0,208169947	1,10E-03	2,51E-07	4	1,01E-06
		1,35	1,1036E-03	0,182571285	6,45E-04	7,60E-08	2	1,52E-07
		1,4	1,2560E-04	0,160440775	4,97E-05	3,96E-10	4	1,58E-09
		1,45	1,0583E-05	0,140796905	7,89E-05	8,76E-10	1	8,76E-10
								2,25E-05
						m0		3,75E-07 deg^2
						Z_1/3		**1,22E-03** deg
						H_1/3		2,45E-03 deg
					Statistisch:	**Z_max**		**2,28E-03** deg

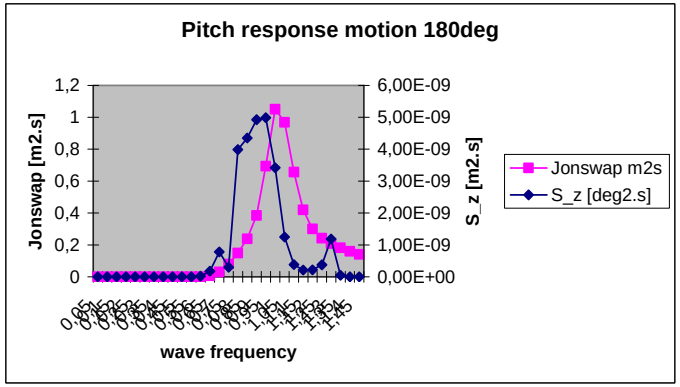




Computation amplitude response motions from RAOs

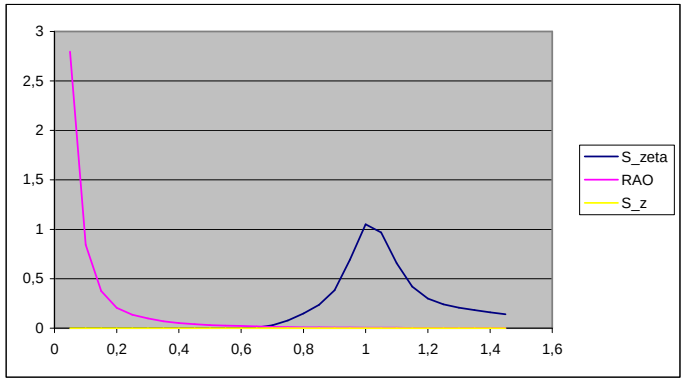
					RAO		Simpson Integration	
Hs	**2,37**							
Tp	**6,2**							
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Pitch(180)**	**S_z**	**Sf**	**J*L**
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[deg/m]**	**[deg2.s]**	**[-]**	
		0,0001	4,9315E-45	0				
	ordinaatafstand=	0,05	8,8975E-41	0	3,44E-02	0,00E+00	1	0,00E+00
		0,1	9,9666E-37	0	7,31E-03	0,00E+00	4	0,00E+00
n ordinaten>21		0,15	6,7932E-33	0	3,71E-03	0,00E+00	2	0,00E+00
nauwkeurig genoeg		0,2	2,8174E-29	0	2,25E-03	0,00E+00	4	0,00E+00
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	1,37E-03	4,43E-150	2	8,85E-150
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	8,96E-04	7,03E-75	4	2,81E-74
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	6,57E-04	6,42E-43	2	1,28E-42
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	4,93E-04	1,18E-27	4	4,73E-27
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	3,71E-04	9,55E-20	2	1,91E-19
		0,5	4,2244E-12	2,6313E-08	2,92E-04	2,24E-15	4	8,94E-15
		0,55	5,4111E-10	1,31823E-05	2,27E-04	6,77E-13	2	1,35E-12
		0,6	4,2175E-08	0,000591608	1,85E-04	2,03E-11	4	8,12E-11
		0,65	2,0002E-06	0,006453889	1,67E-04	1,79E-10	2	3,59E-10
		0,7	5,7724E-05	0,029687796	1,63E-04	7,84E-10	4	3,14E-09
		0,75	1,0136E-03	0,079237255	6,18E-05	3,02E-10	2	6,05E-10
		0,8	1,0831E-02	0,149970469	1,63E-04	3,98E-09	4	1,59E-08
		0,85	7,0416E-02	0,237992026	1,35E-04	4,35E-09	2	8,70E-09
		0,9	2,7858E-01	0,384391531	1,13E-04	4,93E-09	4	1,97E-08
		0,95	6,7060E-01	0,692667608	8,49E-05	4,99E-09	2	9,97E-09
		1	9,8227E-01	1,050203761	5,71E-05	3,42E-09	4	1,37E-08
		1,05	9,2271E-01	0,968406146	3,59E-05	1,25E-09	2	2,49E-09
		1,1	6,3726E-01	0,656279016	2,42E-05	3,85E-10	4	1,54E-09
		1,15	3,2587E-01	0,419635266	2,23E-05	2,09E-10	2	4,19E-10
		1,2	1,2339E-01	0,299749077	2,69E-05	2,17E-10	4	8,68E-10
		1,25	3,4591E-02	0,241942631	3,96E-05	3,80E-10	2	7,60E-10
		1,3	7,1805E-03	0,208169947	7,54E-05	1,18E-09	4	4,73E-09
		1,35	1,1036E-03	0,182571285	1,66E-05	5,01E-11	2	1,00E-10
		1,4	1,2560E-04	0,160440775	2,62E-06	1,10E-12	4	4,40E-12
		1,45	1,0583E-05	0,140796905	3,75E-06	1,98E-12	1	1,98E-12
								8,31E-08
						m0		1,38E-09 deg^2
						Z_1/3		**7,44E-05** deg
						H_1/3		1,49E-04 deg
					Statistisch:	**Z_max**		**1,38E-04** deg

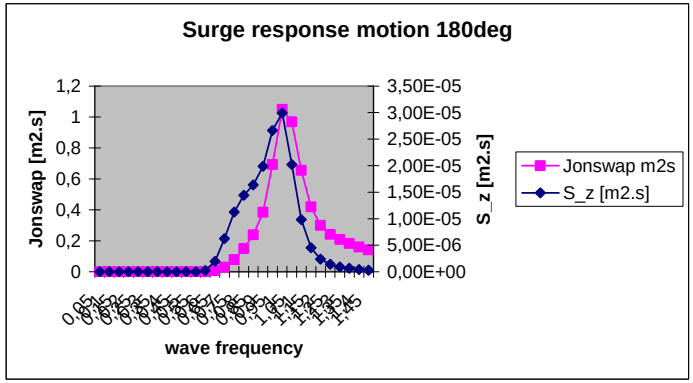




Computation amplitude response motions from RAOs

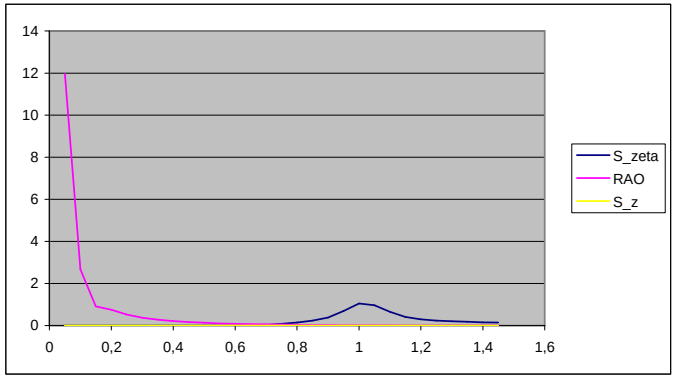
Hs	**2,37**				**RAO**		**Simpson Integration**	
Tp	**6,2**							
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Surge (180)**	**S_z**	**Sf**	**J*L**
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[m/m]**	**[m2.s]**	**[-]**	
		0,0001	4,9315E-45	0				
	ordinaatafstand=	0,05	8,8975E-41	0	2,80E+00	0,00E+00	1	0,00E+00
		0,1	9,9666E-37	0	8,43E-01	0,00E+00	4	0,00E+00
n ordinaten>21		0,15	6,7932E-33	0	3,75E-01	0,00E+00	2	0,00E+00
nauwkeurig genoeg		0,2	2,8174E-29	0	2,05E-01	0,00E+00	4	0,00E+00
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	1,36E-01	4,39E-146	2	8,79E-146
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	9,73E-02	8,29E-71	4	3,32E-70
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	6,90E-02	7,07E-39	2	1,41E-38
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	5,22E-02	1,32E-23	4	5,30E-23
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	4,10E-02	1,17E-15	2	2,34E-15
		0,5	4,2244E-12	2,6313E-08	3,21E-02	2,71E-11	4	1,08E-10
		0,55	5,4111E-10	1,31823E-05	2,60E-02	8,88E-09	2	1,78E-08
		0,6	4,2175E-08	0,000591608	2,14E-02	2,70E-07	4	1,08E-06
		0,65	2,0002E-06	0,006453889	1,75E-02	1,97E-06	2	3,95E-06
		0,7	5,7724E-05	0,029687796	1,45E-02	6,27E-06	4	2,51E-05
		0,75	1,0136E-03	0,079237255	1,19E-02	1,12E-05	2	2,25E-05
		0,8	1,0831E-02	0,149970469	9,81E-03	1,44E-05	4	5,77E-05
		0,85	7,0416E-02	0,237992026	8,30E-03	1,64E-05	2	3,28E-05
		0,9	2,7858E-01	0,384391531	7,19E-03	1,99E-05	4	7,95E-05
		0,95	6,7060E-01	0,692667608	6,20E-03	2,66E-05	2	5,33E-05
		1	9,8227E-01	1,050203761	5,34E-03	2,99E-05	4	1,20E-04
		1,05	9,2271E-01	0,968406146	4,57E-03	2,02E-05	2	4,04E-05
		1,1	6,3726E-01	0,656279016	3,87E-03	9,83E-06	4	3,93E-05
		1,15	3,2587E-01	0,419635266	3,29E-03	4,55E-06	2	9,10E-06
		1,2	1,2339E-01	0,299749077	2,81E-03	2,36E-06	4	9,44E-06
		1,25	3,4591E-02	0,241942631	2,41E-03	1,41E-06	2	2,81E-06
		1,3	7,1805E-03	0,208169947	2,11E-03	9,25E-07	4	3,70E-06
		1,35	1,1036E-03	0,182571285	1,91E-03	6,65E-07	2	1,33E-06
		1,4	1,2560E-04	0,160440775	1,62E-03	4,21E-07	4	1,68E-06
		1,45	1,0583E-05	0,140796905	1,43E-03	2,87E-07	1	2,87E-07
								5,04E-04
						m0		8,39E-06 m^2
						Z_1/3		**5,79E-03** m
						H_1/3		1,16E-02 m
					Statistisch:	**Z_max**		**1,08E-02** m

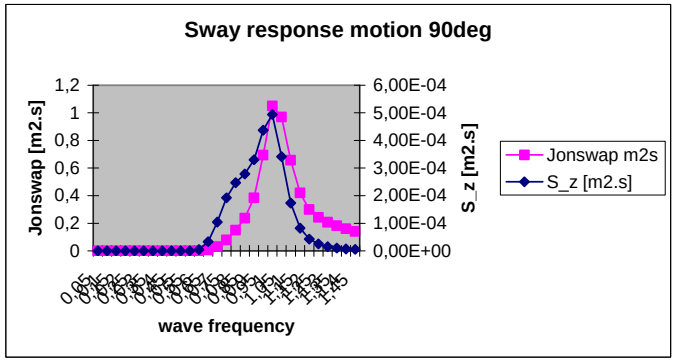




Computation amplitude response motions from RAOs

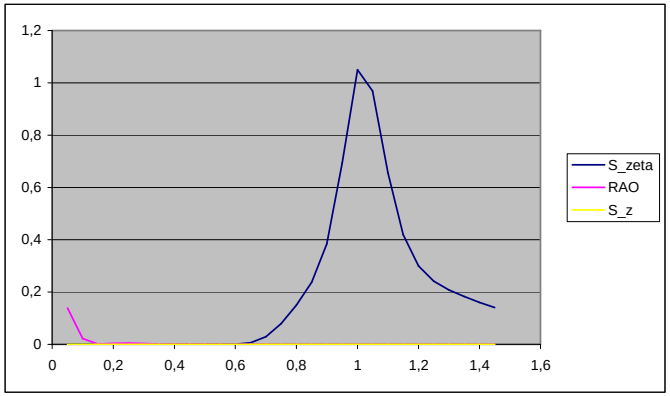
					RAO		Simpson Integration	
Hs	**2,37**							
Tp	**6,2**							
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Sway (90)**	**S_z**	**Sf**	**J*L**
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[m/m]**	**[m2.s]**	**[-]**	
		0,0001	4,9315E-45	0				
	ordinaatafstand=	0,05	8,8975E-41	0	1,20E+01	0,00E+00	1	0,00E+00
		0,1	9,9666E-37	0	2,67E+00	0,00E+00	4	0,00E+00
n ordinaten>21		0,15	6,7932E-33	0	9,14E-01	0,00E+00	2	0,00E+00
nauwkeurig genoeg		0,2	2,8174E-29	0	7,63E-01	0,00E+00	4	0,00E+00
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	5,30E-01	6,64E-145	2	1,33E-144
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	3,76E-01	1,24E-69	4	4,97E-69
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	2,82E-01	1,18E-37	2	2,36E-37
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	2,16E-01	2,26E-22	4	9,03E-22
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	1,67E-01	1,94E-14	2	3,87E-14
		0,5	4,2244E-12	2,6313E-08	1,33E-01	4,68E-10	4	1,87E-09
		0,55	5,4111E-10	1,31823E-05	1,07E-01	1,52E-07	2	3,04E-07
		0,6	4,2175E-08	0,000591608	8,72E-02	4,50E-06	4	1,80E-05
		0,65	2,0002E-06	0,006453889	7,20E-02	3,35E-05	2	6,69E-05
		0,7	5,7724E-05	0,029687796	5,93E-02	1,04E-04	4	4,17E-04
		0,75	1,0136E-03	0,079237255	4,93E-02	1,93E-04	2	3,85E-04
		0,8	1,0831E-02	0,149970469	4,06E-02	2,47E-04	4	9,87E-04
		0,85	7,0416E-02	0,237992026	3,42E-02	2,79E-04	2	5,58E-04
		0,9	2,7858E-01	0,384391531	2,93E-02	3,30E-04	4	1,32E-03
		0,95	6,7060E-01	0,692667608	2,51E-02	4,37E-04	2	8,74E-04
		1	9,8227E-01	1,050203761	2,17E-02	4,94E-04	4	1,98E-03
		1,05	9,2271E-01	0,968406146	1,88E-02	3,42E-04	2	6,83E-04
		1,1	6,3726E-01	0,656279016	1,63E-02	1,74E-04	4	6,94E-04
		1,15	3,2587E-01	0,419635266	1,40E-02	8,25E-05	2	1,65E-04
		1,2	1,2339E-01	0,299749077	1,20E-02	4,31E-05	4	1,72E-04
		1,25	3,4591E-02	0,241942631	1,02E-02	2,50E-05	2	4,99E-05
		1,3	7,1805E-03	0,208169947	8,50E-03	1,50E-05	4	6,01E-05
		1,35	1,1036E-03	0,182571285	7,54E-03	1,04E-05	2	2,07E-05
		1,4	1,2560E-04	0,160440775	6,50E-03	6,78E-06	4	2,71E-05
		1,45	1,0583E-05	0,140796905	6,05E-03	5,16E-06	1	5,16E-06
								8,48E-03
						m0		1,41E-04 m^2
						Z_1/3		**2,38E-02** m
						H_1/3		4,76E-02 m
					Statistisch:	**Z_max**		**4,42E-02** m

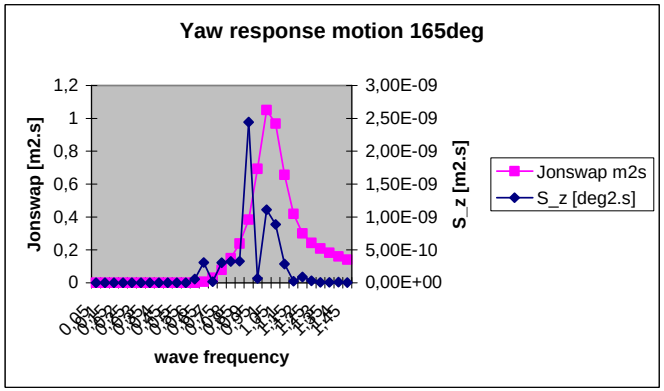




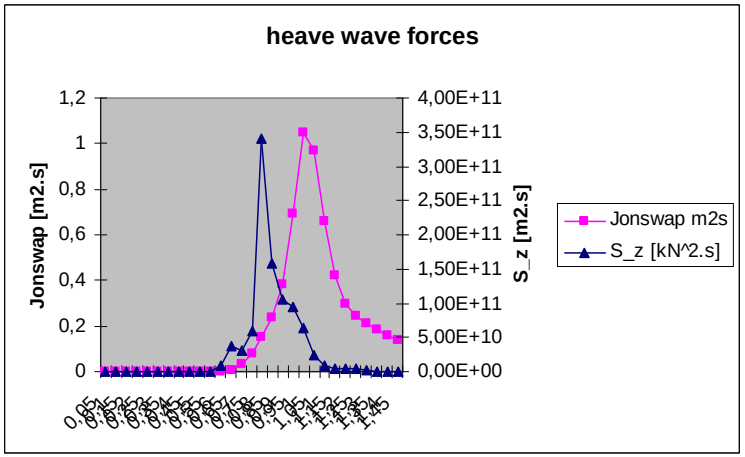
Computation amplitude response motions from RAOs

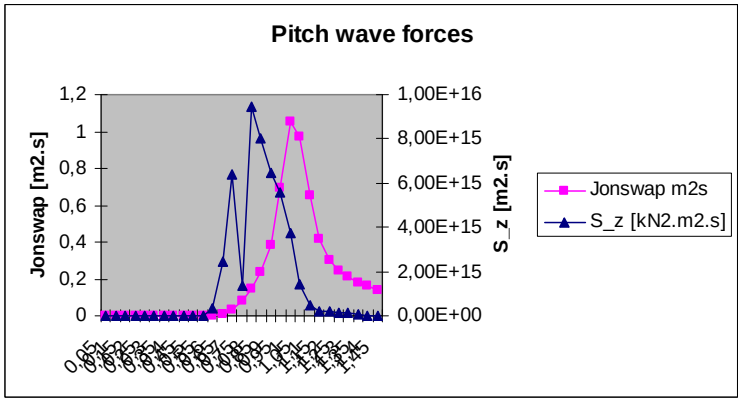
Hs	**2,37**				**RAO**		**Simpson Integration**	
Tp	**6,2**							
gamma	**3,3**	**omega**	**A (Exp)**	**Jonswap**	**Yaw (165)**	**S_z**	**Sf**	**J*L**
omega_p	**1,013417**	**(rad/s)**		**m2s**	**[deg/m]**	**[deg2.s]**	**[-]**	
		0,0001	4,9315E-45	0				
	ordinaatafstand=	0,05	8,8975E-41	0	1,39E-01	0,00E+00	1	0,00E+00
		0,1	9,9666E-37	0	2,26E-02	0,00E+00	4	0,00E+00
n ordinaten>21		0,15	6,7932E-33	0	1,33E-03	0,00E+00	2	0,00E+00
nauwkeurig genoeg		0,2	2,8174E-29	0	4,42E-03	0,00E+00	4	0,00E+00
ordinaat=interval		0,25	7,1102E-26	2,3683E-144	4,55E-03	4,90E-149	2	9,79E-149
n=28; 29 grenzen van x*0.05		0,3	1,0918E-22	8,76683E-69	3,54E-03	1,10E-73	4	4,40E-73
dus even aantal ordinaten;		0,35	1,0202E-19	1,48618E-36	2,00E-03	5,93E-42	2	1,19E-41
1e Simpsonregel + -factoren		0,4	5,8004E-17	4,8634E-21	6,64E-04	2,14E-27	4	8,57E-27
zijn toepasbaar.		0,45	2,0067E-14	6,95663E-13	9,52E-04	6,31E-19	2	1,26E-18
		0,5	4,2244E-12	2,6313E-08	6,81E-04	1,22E-14	4	4,88E-14
		0,55	5,4111E-10	1,31823E-05	2,98E-04	1,17E-12	2	2,35E-12
		0,6	4,2175E-08	0,000591608	3,05E-04	5,49E-11	4	2,20E-10
		0,65	2,0002E-06	0,006453889	2,19E-04	3,09E-10	2	6,19E-10
		0,7	5,7724E-05	0,029687796	2,26E-05	1,51E-11	4	6,05E-11
		0,75	1,0136E-03	0,079237255	6,20E-05	3,05E-10	2	6,10E-10
		0,8	1,0831E-02	0,149970469	4,65E-05	3,24E-10	4	1,30E-09
		0,85	7,0416E-02	0,237992026	3,71E-05	3,27E-10	2	6,54E-10
		0,9	2,7858E-01	0,384391531	7,97E-05	2,44E-09	4	9,77E-09
		0,95	6,7060E-01	0,692667608	9,75E-06	6,58E-11	2	1,32E-10
		1	9,8227E-01	1,050203761	3,25E-05	1,11E-09	4	4,45E-09
		1,05	9,2271E-01	0,968406146	3,03E-05	8,87E-10	2	1,77E-09
		1,1	6,3726E-01	0,656279016	2,08E-05	2,84E-10	4	1,14E-09
		1,15	3,2587E-01	0,419635266	7,26E-06	2,21E-11	2	4,43E-11
		1,2	1,2339E-01	0,299749077	1,70E-05	8,69E-11	4	3,48E-10
		1,25	3,4591E-02	0,241942631	1,13E-05	3,08E-11	2	6,16E-11
		1,3	7,1805E-03	0,208169947	5,95E-06	7,36E-12	4	2,94E-11
		1,35	1,1036E-03	0,182571285	6,14E-06	6,89E-12	2	1,38E-11
		1,4	1,2560E-04	0,160440775	7,85E-06	9,88E-12	4	3,95E-11
		1,45	1,0583E-05	0,140796905	3,78E-06	2,01E-12	1	2,01E-12
								2,13E-08
						m0		3,54E-10 deg^2
						Z_1/3		**3,76E-05** deg
						H_1/3		7,53E-05 deg
					Statistisch:	**Z_max**		**7,00E-05** deg

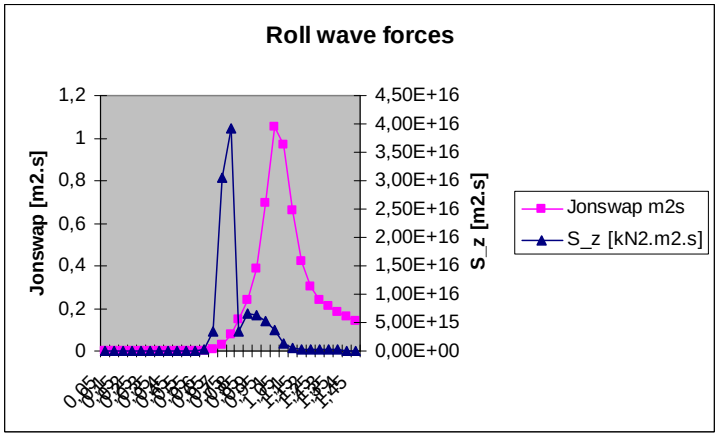


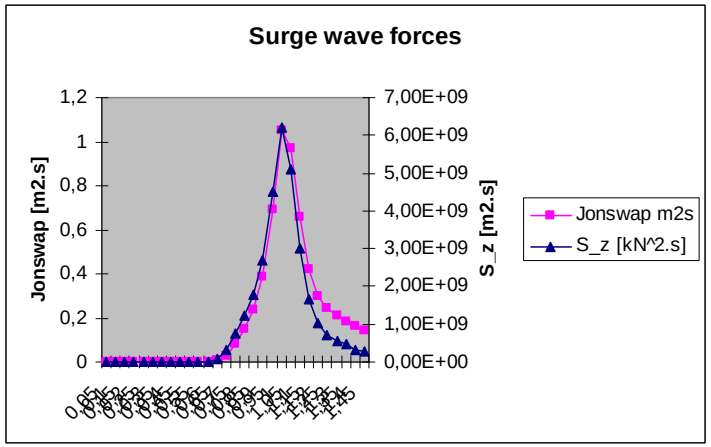


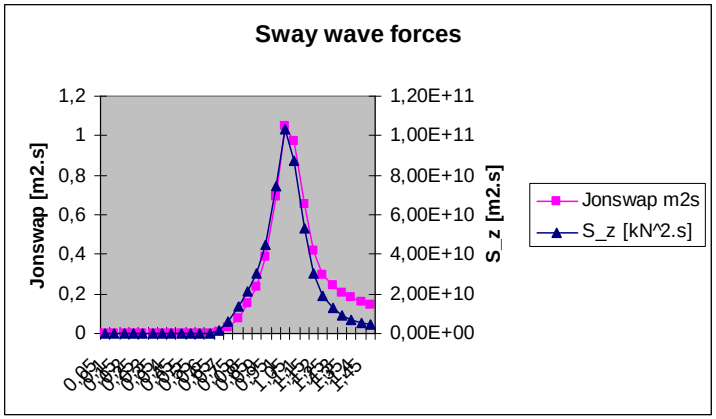
Wave forces:

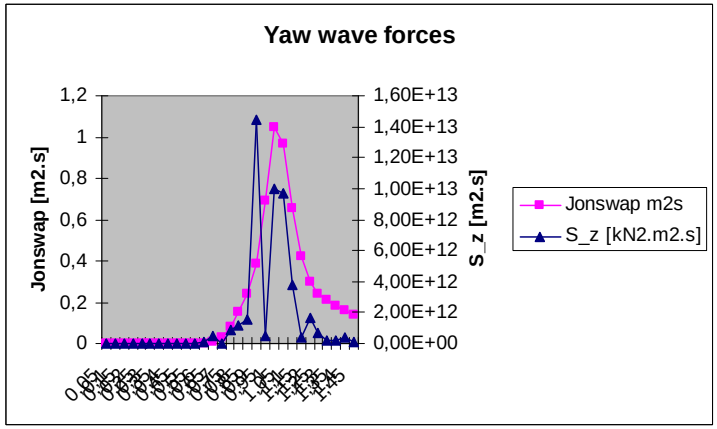

heave wave forces
Jonswap [m2.s]
S_z [m2.s]
Jonswap m2s
S_z [kN^2.s]


Pitch wave forces
Jonswap [m2.s]
S_z [m2.s]
Jonswap m2s
S_z [kN2.m2.s]

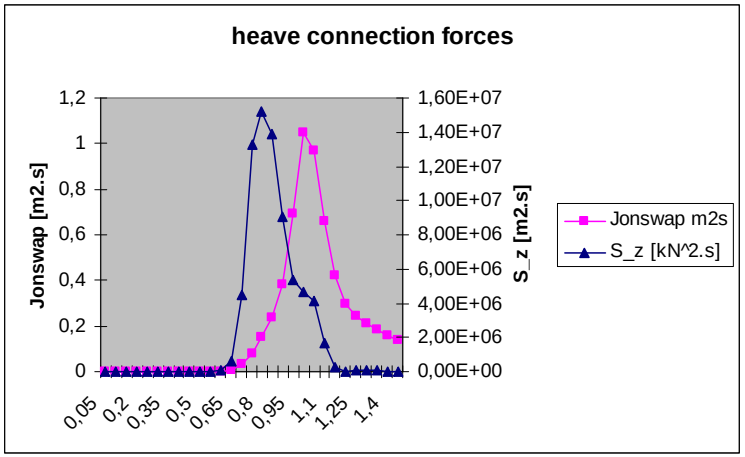

Roll wave forces
Jonswap [m2.s]
S_z [m2.s]
Jonswap m2s
S_z [kN2.m2.s]

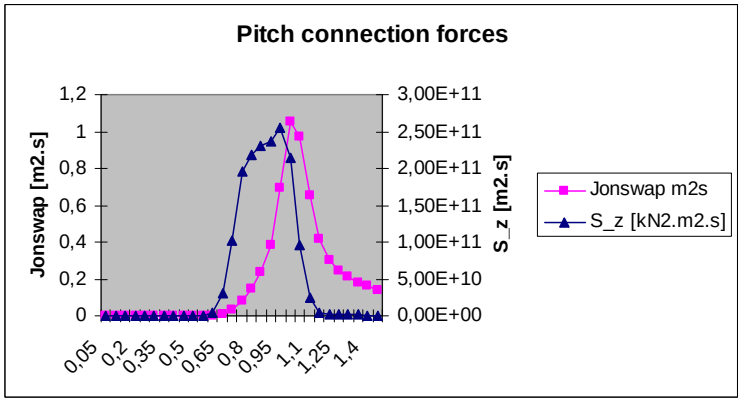

Surge wave forces
Jonswap [m2.s]
S_z [m2.s]
Jonswap m2s
S_z [kN^2.s]

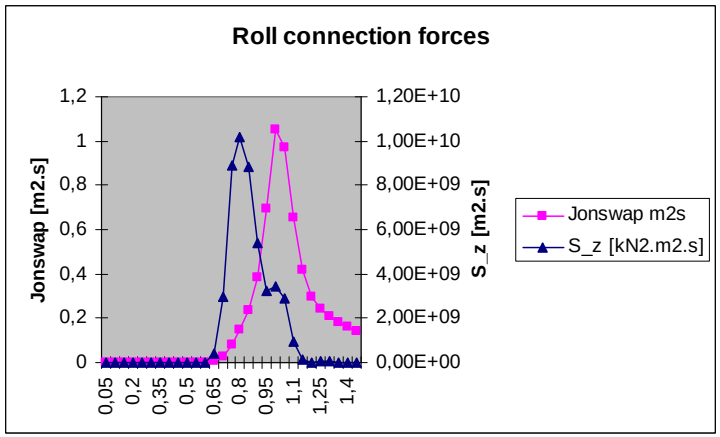

Sway wave forces
Jonswap [m2.s]
S_z [m2.s]
1,2
1
0,8
0,6
0,4
0,2
0
1,20E+11
1,00E+11
8,00E+10
6,00E+10
4,00E+10
2,00E+10
0,00E+00
Jonswap m2s
S_z [kN^2.s]

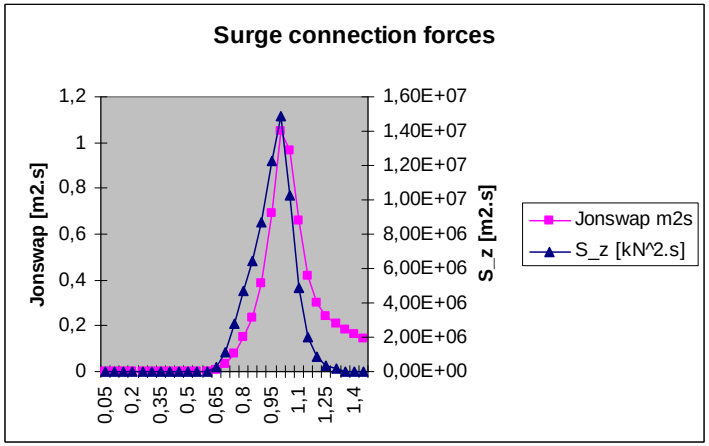

Yaw wave forces
Jonswap [m2.s]
S_z [m2.s]
1,2
1
0,8
0,6
0,4
0,2
0
1,60E+13
1,40E+13
1,20E+13
1,00E+13
8,00E+12
6,00E+12
4,00E+12
2,00E+12
0,00E+00
Jonswap m2s
S_z [kN2.m2.s]

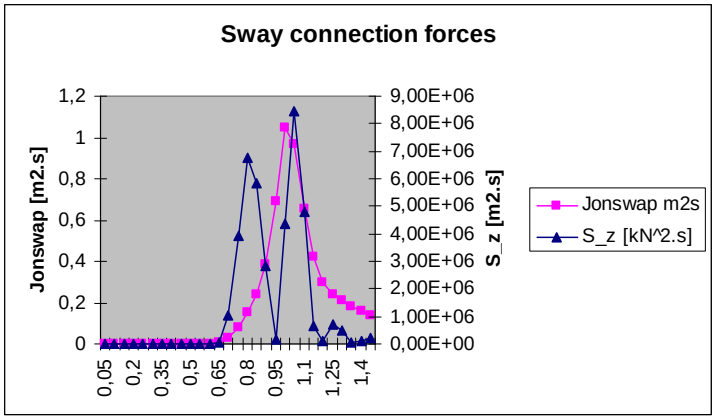
Connection forces:

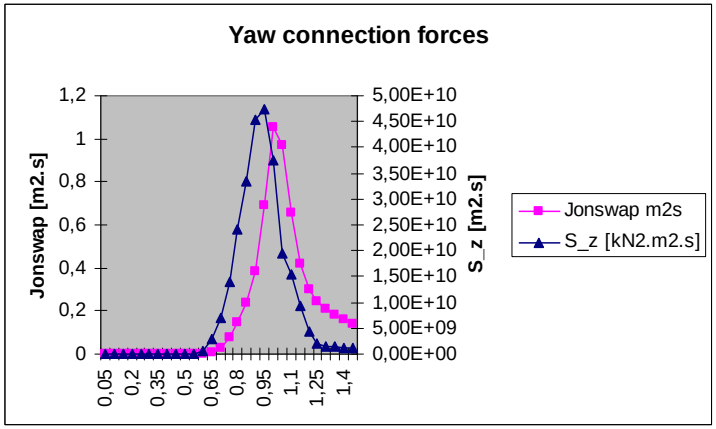









Sway connection forces
Jonswap [m2.s]
S_z [m2.s]
1,2
1
0,8
0,6
0,4
0,2
0
9,00E+06
8,00E+06
7,00E+06
6,00E+06
5,00E+06
4,00E+06
3,00E+06
2,00E+06
1,00E+06
0,00E+00
0,05
0,2
0,35
0,5
0,65
0,8
0,95
1,1
1,25
1,4
Jonswap m2s
S_z [kN^2.s]


Yaw connection forces
Jonswap [m2.s]
S_z [m2.s]
1,2
1
0,8
0,6
0,4
0,2
0
5,00E+10
4,50E+10
4,00E+10
3,50E+10
3,00E+10
2,50E+10
2,00E+10
1,50E+10
1,00E+10
5,00E+09
0,00E+00
0,05
0,2
0,35
0,5
0,65
0,8
0,95
1,1
1,25
1,4
Jonswap m2s
S_z [kN2.m2.s]

Relative wave elevations around vessel

version 14

REF. POINT no.	X in m	Y in m	Wave direction : 180.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 90.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 135.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 165.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.
			Wave frequency: 0.700			Wave frequency: 0.700			Wave frequency: 0.700			Wave frequency: 0.700		
1	-1052.5000	-283.0000	1	0.4938	25.3919	1	0.5591	255.8647	1	1.2453	246.4920	1	2.1756	14.6178
2	-631.3500	-283.0000	2	0.5085	25.8857	2	0.1574	354.8123	2	1.0809	174.1730	2	2.0575	328.3187
3	-210.4500	-283.0000	3	0.5660	28.4408	3	0.1421	356.3200	3	1.1249	117.8313	3	2.0354	287.0521
4	210.4500	-283.0000	4	0.6650	30.0491	4	0.1421	356.3200	4	1.1231	62.9500	4	1.9121	245.4061
5	631.3500	-283.0000	5	0.8082	31.3694	5	0.1574	354.8121	5	1.0482	8.2909	5	1.6649	200.4932
6	1052.5000	-283.0000	6	0.8818	3.2608	6	0.5591	255.8646	6	0.7709	4.3150	6	0.8528	137.2631
7	-1052.5000	-254.5000	7	0.2689	19.9567	7	1.3717	153.3799	7	1.4362	229.1528	7	1.2608	6.9782
8	-631.3500	-254.5000	8	0.3944	27.7034	8	1.8763	159.0738	8	1.9796	176.2171	8	1.7282	329.3817
9	-210.4500	-254.5000	9	0.4328	29.1030	9	1.8720	159.0779	9	1.9752	118.5669	9	1.6722	286.8610
10	210.4500	-254.5000	10	0.5091	31.2086	10	1.8720	159.0779	10	1.9645	61.4920	10	1.5791	243.9996
11	631.3500	-254.5000	11	0.6148	31.4060	11	1.8763	159.0737	11	1.9448	4.2433	11	1.4195	197.4386
12	1052.5000	-254.5000	12	1.3514	12.6103	12	1.3717	153.3797	12	1.3917	295.0396	12	1.3447	118.8324
13	-1052.5000	-203.6000	13	0.0807	197.7014	13	0.9315	349.8658	13	0.4728	35.0714	13	0.3447	165.4097
14	-1052.5000	-101.8000	14	0.0579	219.6578	14	0.7565	2.3369	14	0.2616	42.9721	14	0.1750	176.3964
15	-1052.5000	101.8000	15	0.0579	219.6579	15	0.6659	33.2702	15	0.1342	71.9236	15	0.0933	207.1024
16	-1052.5000	203.6000	16	0.0807	197.7017	16	0.6121	44.3487	16	0.0997	84.6162	16	0.0724	213.6996
17	-1052.5000	254.5000	17	0.2689	19.9568	17	0.4247	231.4634	17	0.0630	262.0261	17	0.0835	23.1872
18	1052.5000	-283.0000	18	0.8818	3.2608	18	0.5591	255.8646	18	0.7709	4.3150	18	0.8528	137.2631
19	631.3500	-283.0000	19	0.8082	31.3694	19	0.1574	354.8122	19	1.0482	8.2909	19	1.6649	200.4932
20	210.4500	-283.0000	20	0.6650	30.0491	20	0.1421	356.3200	20	1.1231	62.9500	20	1.9121	245.4061
21	-210.4500	-283.0000	21	0.5660	28.4408	21	0.1421	356.3200	21	1.1249	117.8313	21	2.0354	287.0521
22	-631.3500	-283.0000	22	0.5085	25.8857	22	0.1574	354.8123	22	1.0809	174.1730	22	2.0575	328.3187
23	-1052.5000	-283.0000	23	0.4938	25.3919	23	0.5591	255.8647	23	1.2453	246.4920	23	2.1756	14.6178
24	1052.5000	-254.5000	24	1.3514	12.6103	24	1.3717	153.3797	24	1.3917	295.0396	24	1.3447	118.8324
25	631.3500	-254.5000	25	0.6148	31.4060	25	1.8763	159.0737	25	1.9448	4.2433	25	1.4195	197.4386
26	210.4500	-254.5000	26	0.5091	31.2086	26	1.8720	159.0779	26	1.9645	61.4920	26	1.5791	243.9996
27	-210.4500	-254.5000	27	0.4328	29.1030	27	1.8720	159.0779	27	1.9752	118.5669	27	1.6722	286.8610
28	-631.3500	-254.5000	28	0.3944	27.7034	28	1.8763	159.0738	28	1.9796	176.2171	28	1.7282	329.3817
29	-1052.5000	-254.5000	29	0.2689	19.9567	29	1.3717	153.3799	29	1.4362	229.1528	29	1.2608	6.9782
30	1052.5000	-203.6000	30	2.1364	17.8427	30	0.9315	349.8657	30	1.7680	188.1739	30	2.0703	80.5688
31	1052.5000	-101.8000	31	2.0825	16.8232	31	0.7565	2.3368	31	2.1558	294.5476	31	1.8532	347.4928
32	1052.5000	101.8000	32	2.0825	16.8232	32	0.6659	33.2701	32	1.9684	159.9818	32	1.9608	161.9532

33 1052.5000 203.6000	33 2.1364 17.8427	33 0.6121 44.3486	33 1.8994 271.8704	33 2.0602 71.9952
34 1052.5000 254.5000	34 1.3514 12.6103	34 0.4247 231.4633	34 1.5177 147.5014	34 1.4391 24.5265
35 1052.5000 283.0000	35 0.8818 3.2608	35 0.3398 122.5711	35 1.2717 68.2913	35 1.0421 346.1862
36 631.3500 283.0000	36 0.8082 31.3695	36 0.0970 325.0184	36 0.1467 121.8809	36 0.3635 12.9102
37 210.4500 283.0000	37 0.6650 30.0491	37 0.1033 333.5721	37 0.0749 127.3432	37 0.2047 8.4906
38 -210.4500 283.0000	38 0.5660 28.4408	38 0.1033 333.5718	38 0.0479 130.8271	38 0.1371 4.9542
39 -631.3500 283.0000	39 0.5085 25.8858	39 0.0970 325.0192	39 0.0348 137.3921	39 0.1001 0.9297
40-1052.5000 283.0000	40 0.4938 25.3920	40 0.3398 122.5712	40 0.0858 155.1097	40 0.1079 344.2358
41 1052.5000 254.5000	41 1.3514 12.6103	41 0.4247 231.4633	41 1.5177 147.5014	41 1.4391 24.5265
42 631.3500 254.5000	42 0.6148 31.4061	42 0.0727 64.4931	42 0.1177 125.5931	42 0.2790 14.8018
43 210.4500 254.5000	43 0.5091 31.2086	43 0.0798 65.3810	43 0.0586 130.3228	43 0.1597 9.2353
44 -210.4500 254.5000	44 0.4328 29.1030	44 0.0798 65.3807	44 0.0362 133.8869	44 0.1078 5.7335
45 -631.3500 254.5000	45 0.3944 27.7035	45 0.0727 64.4936	45 0.0246 142.7058	45 0.0817 1.8692
46-1052.5000 254.5000	46 0.2689 19.9568	46 0.4247 231.4634	46 0.0630 262.0261	46 0.0835 23.1872
47 1052.5000 203.6000	47 2.1364 17.8427	47 0.6121 44.3486	47 1.8994 271.8704	47 2.0602 71.9952
48 1052.5000 101.8000	48 2.0825 16.8232	48 0.6659 33.2701	48 1.9684 159.9818	48 1.9608 161.9532
49 1052.5000 -101.8000	49 2.0825 16.8232	49 0.7565 2.3368	49 2.1558 294.5476	49 1.8532 347.4928
50 1052.5000 -203.6000	50 2.1364 17.8427	50 0.9315 349.8657	50 1.7680 188.1739	50 2.0703 80.5688
51 1052.5000 -254.5000	51 1.3514 12.6103	51 1.3717 153.3797	51 1.3917 295.0396	51 1.3447 118.8324
52-1052.5000 283.0000	52 0.4938 25.3920	52 0.3398 122.5712	52 0.0858 155.1097	52 0.1079 344.2358
53 -631.3500 283.0000	53 0.5085 25.8858	53 0.0970 325.0192	53 0.0348 137.3921	53 0.1001 0.9297
54 -210.4500 283.0000	54 0.5660 28.4408	54 0.1033 333.5718	54 0.0479 130.8271	54 0.1371 4.9542
55 210.4500 283.0000	55 0.6650 30.0491	55 0.1033 333.5721	55 0.0749 127.3432	55 0.2047 8.4906
56 631.3500 283.0000	56 0.8082 31.3695	56 0.0970 325.0184	56 0.1467 121.8809	56 0.3635 12.9102
57 1052.5000 283.0000	57 0.8818 3.2608	57 0.3398 122.5711	57 1.2717 68.2913	57 1.0421 346.1862
58-1052.5000 254.5000	58 0.2689 19.9568	58 0.4247 231.4634	58 0.0630 262.0261	58 0.0835 23.1872
59 -631.3500 254.5000	59 0.3944 27.7035	59 0.0727 64.4936	59 0.0246 142.7058	59 0.0817 1.8692
60 -210.4500 254.5000	60 0.4328 29.1030	60 0.0798 65.3807	60 0.0362 133.8869	60 0.1078 5.7335
61 210.4500 254.5000	61 0.5091 31.2086	61 0.0798 65.3810	61 0.0586 130.3228	61 0.1597 9.2354
62 631.3500 254.5000	62 0.6148 31.4061	62 0.0727 64.4931	62 0.1177 125.5931	62 0.2790 14.8018
63 1052.5000 254.5000	63 1.3514 12.6103	63 0.4247 231.4633	63 1.5177 147.5014	63 1.4391 24.5265
64-1052.5000 203.6000	64 0.0807 197.7016	64 0.6121 44.3487	64 0.0997 84.6162	64 0.0724 213.6996
65-1052.5000 101.8000	65 0.0579 219.6579	65 0.6659 33.2702	65 0.1342 71.9236	65 0.0933 207.1024
66-1052.5000 -101.8000	66 0.0579 219.6577	66 0.7565 2.3369	66 0.2616 42.9721	66 0.1750 176.3964
67-1052.5000 -203.6000	67 0.0807 197.7015	67 0.9315 349.8658	67 0.4728 35.0714	67 0.3447 165.4097
68-1052.5000 -254.5000	68 0.2689 19.9567	68 1.3717 153.3799	68 1.4362 229.1528	68 1.2608 6.9782

REF. POINT	X in	Y in	Wave direction : 180.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 90.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 135.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.	Wave direction : 165.000 REF. POINT no.	WAVE AMPL. m/m	PHASE ANGLE degr.
			Wave frequency: 1.000			Wave frequency: 1.000			Wave frequency: 1.000			Wave frequency: 1.000		
1	-1052.5000	-283.0000	1	0.5607	179.9811	1	1.4200	271.2269	1	0.5363	45.9821	1	2.0899	63.9609
2	-631.3500	-283.0000	2	0.5643	195.7585	2	1.9278	275.7409	2	0.8205	26.6559	2	1.8782	352.2350
3	-210.4500	-283.0000	3	0.6279	205.2054	3	1.8949	276.6598	3	0.8348	17.3898	3	2.0320	277.4195
4	210.4500	-283.0000	4	0.7174	213.9512	4	1.8949	276.6597	4	0.8433	8.2708	4	2.1276	209.3842
5	631.3500	-283.0000	5	0.8524	219.7270	5	1.9278	275.7408	5	0.8692	359.9874	5	1.8723	143.2427
6	1052.5000	-283.0000	6	0.8679	219.0056	6	1.4200	271.2267	6	1.1975	293.4306	6	0.9534	88.9015
7	-1052.5000	-254.5000	7	0.2573	184.8739	7	1.3877	89.2672	7	1.3910	210.5768	7	1.2753	68.2154
8	-631.3500	-254.5000	8	0.3876	199.2723	8	1.8769	98.6331	8	1.8783	206.7822	8	1.6986	355.0123
9	-210.4500	-254.5000	9	0.4285	207.1452	9	1.8488	98.2774	9	1.8436	196.7715	9	1.7840	279.0186
10	210.4500	-254.5000	10	0.4965	214.1404	10	1.8488	98.2773	10	1.7988	186.5276	10	1.8450	208.0148
11	631.3500	-254.5000	11	0.6143	218.1127	11	1.8769	98.6330	11	1.8171	176.9207	11	1.6923	137.3508
12	1052.5000	-254.5000	12	1.4355	211.6658	12	1.3877	89.2671	12	1.4473	156.8493	12	1.4324	29.3889
13	-1052.5000	-203.6000	13	0.0652	210.3855	13	0.8897	150.1150	13	0.4228	235.2708	13	0.3192	88.4403
14	-1052.5000	-101.8000	14	0.0177	276.0312	14	0.7423	262.4517	14	0.2106	339.5374	14	0.1590	196.0735
15	-1052.5000	101.8000	15	0.0177	276.0319	15	0.6395	132.2070	15	0.1111	210.5351	15	0.0830	65.6973
16	-1052.5000	203.6000	16	0.0651	210.3829	16	0.5799	237.0300	16	0.0813	316.3288	16	0.0601	181.5948
17	-1052.5000	254.5000	17	0.2573	184.8727	17	0.4627	296.9955	17	0.0631	27.9595	17	0.0766	244.8994
18	1052.5000	-283.0000	18	0.8679	219.0056	18	1.4200	271.2267	18	1.1975	293.4306	18	0.9534	88.9015
19	631.3500	-283.0000	19	0.8524	219.7270	19	1.9278	275.7408	19	0.8692	359.9874	19	1.8723	143.2427
20	210.4500	-283.0000	20	0.7174	213.9512	20	1.8949	276.6597	20	0.8433	8.2708	20	2.1276	209.3842
21	-210.4500	-283.0000	21	0.6279	205.2054	21	1.8949	276.6598	21	0.8348	17.3898	21	2.0320	277.4195
22	-631.3500	-283.0000	22	0.5643	195.7585	22	1.9278	275.7409	22	0.8205	26.6559	22	1.8782	352.2350
23	-1052.5000	-283.0000	23	0.5607	179.9811	23	1.4200	271.2269	23	0.5363	45.9821	23	2.0899	63.9609
24	1052.5000	-254.5000	24	1.4355	211.6658	24	1.3877	89.2671	24	1.4473	156.8493	24	1.4324	29.3889
25	631.3500	-254.5000	25	0.6143	218.1127	25	1.8769	98.6330	25	1.8171	176.9207	25	1.6923	137.3508
26	210.4500	-254.5000	26	0.4965	214.1404	26	1.8488	98.2773	26	1.7988	186.5276	26	1.8450	208.0148
27	-210.4500	-254.5000	27	0.4285	207.1452	27	1.8488	98.2774	27	1.8436	196.7715	27	1.7840	279.0186
28	-631.3500	-254.5000	28	0.3876	199.2723	28	1.8769	98.6331	28	1.8783	206.7822	28	1.6986	355.0123
29	-1052.5000	-254.5000	29	0.2573	184.8739	29	1.3877	89.2672	29	1.3910	210.5768	29	1.2753	68.2154
30	1052.5000	-203.6000	30	1.7770	215.2069	30	0.8897	150.1149	30	1.9083	319.8513	30	1.9666	314.1912
31	1052.5000	-101.8000	31	1.9508	216.9139	31	0.7423	262.4515	31	1.8909	233.5564	31	1.8603	159.4264
32	1052.5000	101.8000	32	1.9508	216.9139	32	0.6395	132.2069	32	1.9871	92.3345	32	1.9277	205.6790

33	1052.5000	203.6000	33	1.7770	215.2066	33	0.5799	237.0299	33	1.9856	20.1809	33	1.7878	46.9256
34	1052.5000	254.5000	34	1.4355	211.6655	34	0.4627	296.9955	34	1.4877	163.0567	34	1.3895	322.5919
35	1052.5000	283.0000	35	0.8679	219.0051	35	0.3620	116.9848	35	1.1445	28.3150	35	0.8601	273.9838
36	631.3500	283.0000	36	0.8524	219.7266	36	0.0639	227.0962	36	0.1332	121.0337	36	0.3158	292.0388
37	210.4500	283.0000	37	0.7174	213.9510	37	0.0524	223.7592	37	0.0699	121.6410	37	0.1770	284.9990
38	-210.4500	283.0000	38	0.6279	205.2053	38	0.0524	223.7593	38	0.0450	115.4700	38	0.1178	276.2049
39	-631.3500	283.0000	39	0.5643	195.7579	39	0.0639	227.0954	39	0.0311	104.6582	39	0.0863	266.4417
40	-1052.5000	283.0000	40	0.5607	179.9806	40	0.3620	116.9850	40	0.0532	162.1044	40	0.0395	273.6089
41	1052.5000	254.5000	41	1.4355	211.6655	41	0.4627	296.9955	41	1.4877	163.0567	41	1.3895	322.5919
42	631.3500	254.5000	42	0.6143	218.1121	42	0.0364	310.1294	42	0.0963	125.2294	42	0.2223	295.2531
43	210.4500	254.5000	43	0.4965	214.1403	43	0.0327	323.3422	43	0.0485	125.1336	43	0.1213	288.2840
44	-210.4500	254.5000	44	0.4285	207.1451	44	0.0327	323.3423	44	0.0306	120.7655	44	0.0795	280.4185
45	-631.3500	254.5000	45	0.3876	199.2715	45	0.0364	310.1319	45	0.0208	112.5056	45	0.0583	271.3249
46	-1052.5000	254.5000	46	0.2573	184.8727	46	0.4627	296.9955	46	0.0631	27.9595	46	0.0766	244.8994
47	1052.5000	203.6000	47	1.7770	215.2066	47	0.5799	237.0299	47	1.9856	20.1809	47	1.7878	46.9256
48	1052.5000	101.8000	48	1.9508	216.9139	48	0.6395	132.2069	48	1.9871	92.3345	48	1.9277	205.6790
49	1052.5000	-101.8000	49	1.9508	216.9139	49	0.7423	262.4515	49	1.8909	233.5564	49	1.8603	159.4264
50	1052.5000	-203.6000	50	1.7770	215.2069	50	0.8897	150.1149	50	1.9083	319.8513	50	1.9666	314.1912
51	1052.5000	-254.5000	51	1.4355	211.6658	51	1.3877	89.2671	51	1.4473	156.8493	51	1.4324	29.3889
52	-1052.5000	283.0000	52	0.5607	179.9806	52	0.3620	116.9850	52	0.0532	162.1044	52	0.0395	273.6089
53	-631.3500	283.0000	53	0.5643	195.7579	53	0.0639	227.0954	53	0.0311	104.6582	53	0.0863	266.4417
54	-210.4500	283.0000	54	0.6279	205.2053	54	0.0524	223.7592	54	0.0450	115.4700	54	0.1178	276.2049
55	210.4500	283.0000	55	0.7174	213.9510	55	0.0524	223.7592	55	0.0699	121.6410	55	0.1770	284.9990
56	631.3500	283.0000	56	0.8524	219.7266	56	0.0639	227.0962	56	0.1332	121.0337	56	0.3158	292.0388
57	1052.5000	283.0000	57	0.8679	219.0051	57	0.3620	116.9848	57	1.1445	28.3150	57	0.8601	273.9838
58	-1052.5000	254.5000	58	0.2573	184.8727	58	0.4627	296.9955	58	0.0631	27.9595	58	0.0766	244.8994
59	-631.3500	254.5000	59	0.3876	199.2715	59	0.0364	310.1319	59	0.0208	112.5057	59	0.0583	271.3249
60	-210.4500	254.5000	60	0.4285	207.1451	60	0.0327	323.3423	60	0.0306	120.7655	60	0.0795	280.4185
61	210.4500	254.5000	61	0.4965	214.1403	61	0.0327	323.3422	61	0.0485	125.1336	61	0.1213	288.2840
62	631.3500	254.5000	62	0.6143	218.1121	62	0.0364	310.1294	62	0.0963	125.2294	62	0.2223	295.2531
63	1052.5000	254.5000	63	1.4355	211.6655	63	0.4627	296.9955	63	1.4877	163.0567	63	1.3895	322.5919
64	-1052.5000	203.6000	64	0.0651	210.3828	64	0.5799	237.0300	64	0.0813	316.3288	64	0.0601	181.5947
65	-1052.5000	101.8000	65	0.0177	276.0315	65	0.6395	132.2070	65	0.1111	210.5350	65	0.0830	65.6972
66	-1052.5000	-101.8000	66	0.0177	276.0312	66	0.7423	262.4517	66	0.2106	339.5374	66	0.1590	196.0736
67	-1052.5000	-203.6000	67	0.0652	210.3855	67	0.8897	150.1150	67	0.4228	235.2708	67	0.3192	88.4403
68	-1052.5000	-254.5000	68	0.2573	184.8739	68	1.3877	89.2672	68	1.3910	210.5768	68	1.2753	68.2154

6. Overview coordinates data Delfrac

An overview was made of coordinates on the terminal and on the lee-side of the terminal, as input for Delfrac. At these coordinates, data is wanted on section forces (connection forces), motions at the lee-side of the terminal and wave transmission (motions behind the terminal). This was done for the rigid terminal model (no elements).

There are two different types of output of Delfrac at reference points"

- The (transmitted, reflected and/or diffracted) wave height
- The relative motions (of the apron pontoons)

The first two of the above types of output have already been generated. The third requires more work and will shortly be produced. The output of the reference point is a reasonable check of the accuracy of the output in general, as especially the wave heights should lie within an narrow expected range of values. The output at reference points is included in the other output of the terminal model and is not further discussed here.

Overzicht coördinaten in te voeren in Delfrac

Terminal: 2104,5 x 509 x 15,5 m

Met 5x10 pontons: 420,9 x 50,9 x 15,5 m

Vereenvoudiging

De volgende coördinaten zijn voldoende om een beeld te krijgen van de verbindingskrachten in een stijve terminal. Dit is exclusief de randen van de terminal (?):
- z-coördinaten:
z = -13.11 (onderkant)
z = 0 (bovenkant, want z = 2,39 valt buiten het model?)
z = -5,36 (midden)
Deze z-coördinaten gelden voor alle (x,y)-coördinaten in de onderstaande tabel.

	Xp	Yp	Zp
Intern in terminal	-631.35	-203.6	0 -5.36 -13.11
	-631.35	-101.8	0
	-631.35	0	0
	-631.35	101.8	0
	-631.35	203.6	0
	-210.45	-203.6	0 -5.36 -13.11
	-210.45	-101.8	0
	-210.45	0	0
	-210.45	101.8	0
	-210.45	203.6	0
	210.45	-203.6	0 -5.36 -13.11

	210.45	-101.8	0
	210.45	0	0
	210.45	101.8	0
	210.45	203.6	0
	631.35	-203.6	0 -5.36 -13.11
	631.35	-101.8	0
	631.35	0	0
	631.35	101.8	0
	631.35	203.6	0

Coördinaten achter de terminal t.o.v. de inkomende golven.
(x,y,z) met y ½ * $B_{ontwerpschip}$ buiten terminal en z op waterspiegel.
Gevraagd zijn hier de bewegingen (en golfhoogten)

	X	Y	Z
Lange zijde extern	-1052.5	-283	0
	-631.35	-283	0
	-210.45	-283	0
	210.45	-283	0
	631.35	-283	0
	1052.5	-283	0

Coördinaten aan de rand van de terminal.
Gevraagd zijn de bewegingen/krachten van de achterste rij pontons. Wederom met y negatief?

	X	Y	Z
Lange zijde	-1052.5	-254.5	0
	-631.35	-254.5	0
	-210.45	-254.5	0
	210.45	-254.5	0
	631.35	-254.5	0
	1052.5	-254.5	0
Korte zijde	-1052.5	-203.6	0
	-1052.5	-101.8	0
	-1052.5	101.8	0
	-1052.5	203.6	0
	-1052.5	254.5	0

Appendix Part 4 - A:

Structural design pontoon

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1 Structural design of pontoon

1.1 Approach of the conceptual design

1.1.1 Load distribution on the structure of the terminal

The distribution of the deck load of the terminal depends on the stiffness of the elements and their connections. An entirely rigid homogeneous structure with infinite stiffness in every direction, will have an evenly distributed load over all structural members. The load will not be distributed to other structural members, if the structural members are all very slack. Rigid structural parts “attract” a larger parts of the total load on the structure than slack structural parts.
In reality, the pontoon elements have a defined bending stiffness EI, which is not infinite. Only when compared with the waves, the elements seem to be infinitely stiff. The connections can only be considered as entirely stiff, when the pontoons are connected to each other by prestressing elements.

The terminal built up from its elements will have to be modelled to consider the load distribution over the separate elements. There are several ways to model the terminal in this respect, with different accuracy.

Model of the terminal:

- An integrated body, with infinite stiffness in every direction
- A beam grid with defined bending stiffness EI per beam
- A body consisting of several infinitely stiff elements, rigidly connected to each other
- A body consisting of several infinitely stiff elements, flexibly connected to each other

The last model type is the most accurate, but it is also influenced by the wave forces and the response motions of the elements. It is very difficult to combine these dynamic features and static deck load in a relatively simple conceptual design. The worst case scenario from dynamics should actually be used in a static design. The beam grid model may be appropriate.
At present, it seems impossible to give a good estimate of the stiffness of the connectors. It is therefore assumed that the connections are entirely rigid, as the crane track demands very stiff connections between the pontoons at the quayside.
For the conceptual design of the rigidly connected pontoons, it will be assumed that the terminal is an infinitely stiff platform. The load is evenly distributed.

1.1.2 Calculation method

As a first estimate, the pontoon is considered as a beam supported on the waves.
The maximum bending moment in the hull is approximated by the Still Water Bending Moment, mostly used for ships.
Next, the governing cross sections are chosen. The governing cross sections are considered as a framework and thus, the internal forces are determined. Then the bending moments in the longitudinal section are determined according to the section method, the pontoon still modelled as a beam, but supported at the connectors and lifted by the buoyancy force of the water, including waves. The following models have been considered using a Finite Element Program (because of the repetition of the work):

- Longitudinal section with b = 50 m, still water or passing wave
- Transversal section with l = 180 m, passing wave crest and wave trough
- Transversal section with b = 1m, still water
- Transversal section with b = 1m, passing wave trough
- Transversal section with b = 1m, passing wave crest; this proved to be the governing situation.

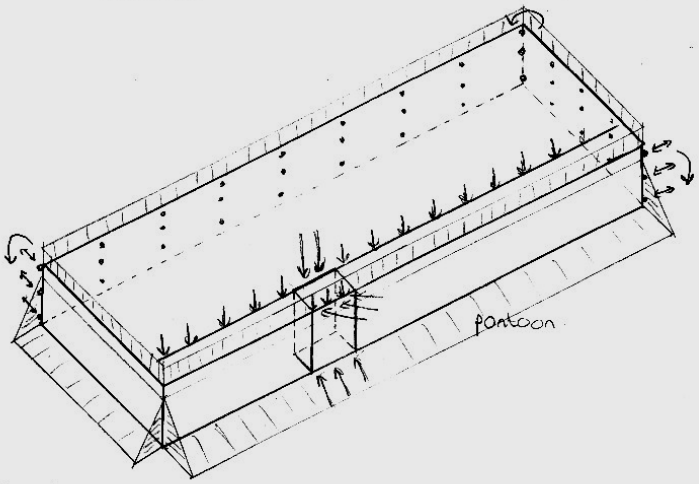
As the wave length is much shorter than the length of the pontoon, all passing waves will create both wave tops and wave troughs underneath the pontoon, therefore these do not have to be modelled separately.

1.2 Conceptual manual design calculations

Van Hattum en Blankevoort

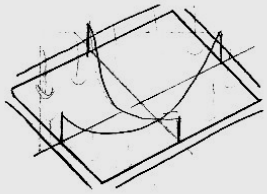
werknr.: .....
betreft: .....
.....

datum: .....
naam: .....
bl. nr.: ..... van .....



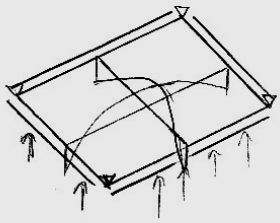
1 Rough force distribution of pontoon
Structural models:

1 deck



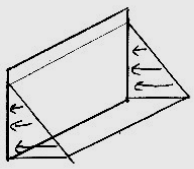
deck load (storage etc)
supported by walls

2 floor



resulting upward buoyancy load
(?- ballast)

3 Outer walls



hydrostatic water pressure +
wave load

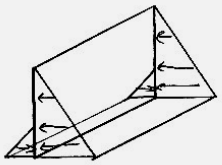
Van Hattum en Blankevoort

werknr.: ..... datum: .....

betreft: ..... naam: .....

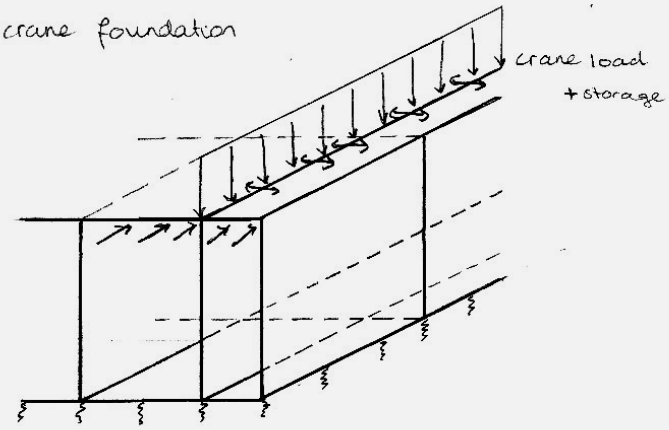
..... bl. nr.: ..... van .....

4. Inner walls



ULS: survival condition of one flooded compartment
hydrostatic water pressure from flooded compartment & ballast

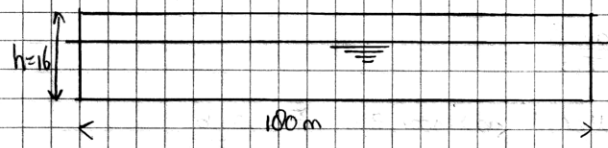
5 crane foundation

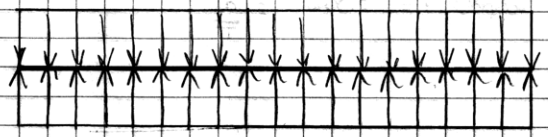


6 longitudinal and transversal cross section ⇒ general force distribution

Golfbelasting op ponton

a) Geen golven:





$q_{ponton} = q_{eg} + q_{ballast} + q_{var}$

$q_w = 10 \cdot T = 136$ kN/m'

$q_{ponton} = q_{water} \Rightarrow q_{resulterend} = 0 \Rightarrow \begin{cases} V = 0 \\ M = 0 \end{cases}$

$q_{eg} = \frac{4314,24}{50\,m} = 86,28$ kN/m' $\quad b = 1$ m

$q_{storage} = 40$ kN/m' $\quad b = 1$ m

$q_{crane} = 76800 / (100 \times 50) = 8,53$ kN/m' lengte $\quad b = 1$ m

2 pon x 2 wielstellen x 8 wielen

1 kraan = (16m x 800 kN/m') x 2 pon = 25600 kN

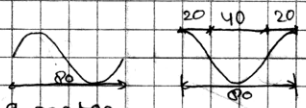
3 kranen = 76800 kN

$q_{ballast} = q_{water} - q_{eg} - q_{storage} - q_{crane}$

$= 136 - 86,28 - 40 - 8,53 = 1,19$ kN/m' vol beladen

b) Golven:

* Transport: L = 80 m



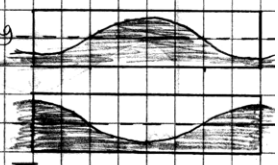
$q_{resulterend} = q_{water} - q_{ponton}$

$q_{ponton} = q_{eg} + q_{ballast}$

$q_{water} = 10 \cdot T + q_{golf}$

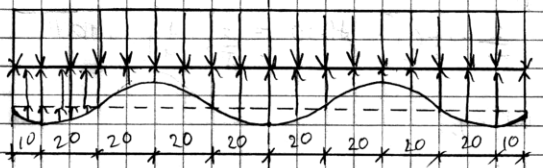
I hogging

II sagging



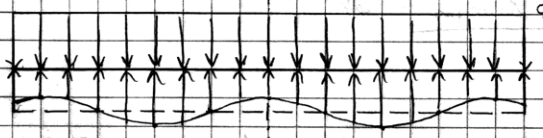
$q_{ponton} = q_w = 10 \cdot T$

I hogging



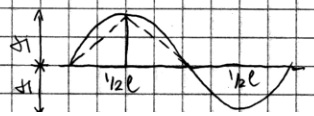
$q_w = 136$ $\quad q_{w\,min} = 112,3$ kN/m' $\quad q_{w\,max} = 159,7$ kN/m'

II sagging

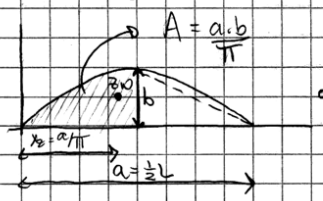


$q_{ponton} = q_w = 10 \cdot T = 136$ kN/m'

Ten gevolge van de kleine golflengte tov de lengte vh ponton, zijn de verschillen tussen the hogging and the sagging condition minimaal. Slechts de locatie van het min en max moment verschillen



Belasting uit sinusvormige golf



a = ½L
b = ½ . H . 10
opp $A = \frac{a.b}{\pi}$
$x_{zw} = \frac{a}{\pi}$

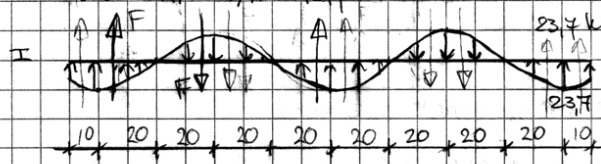
sinusfunctie $y = b \cdot \sin \frac{\pi}{a} x$

Resulterend gaat door zw $F = \frac{a.b}{\pi}$

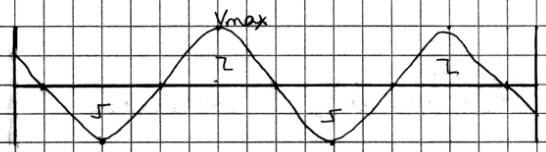
arm koppel $t = 2 . x_{zw} = 2 . \frac{a}{\pi}$

Transport

L = 80 m
a = 40 m = ½L
b = 2,37 . 10 = 23,7 kN
$x_{zw} = a/\pi = 40/\pi = 12,7$ m



q-lijn resulterend

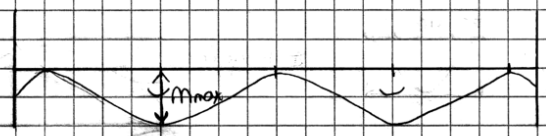


$V_x = -q \frac{d}{dx}$

$V_{max} = A = \frac{a.b}{\pi}$

$V_{max} = \frac{40 . 23,7}{\pi}$

$V_{max} = 301,8$ kN /m lengte



m

$m_{max} = F . 2 . x_{zw} =$ koppel

$m_{max} = \frac{a.b}{\pi} \cdot 2 \cdot \frac{a}{\pi} = \frac{2a^2.b}{\pi^2} = \frac{2}{\pi^2} \cdot (\frac{1}{2}\ell)^2 . H . 10 = \frac{2}{\pi^2} \cdot \frac{1}{4}\ell^2 . 10.H$ met $\pi^2 \approx 10$

$m_{max} = \frac{1}{20} . H . 10 . \ell^2 = 7584$ kNm/m

$m_{max} = \frac{ab}{\pi} . 2 . \frac{a}{\pi} = 7684$ kNm/m lengte

Gebruiksfase

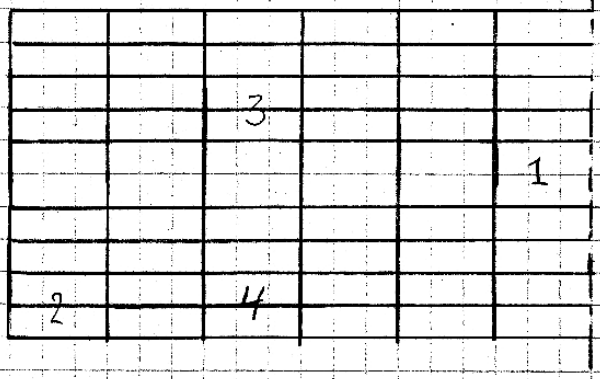
L = 60 m → a = ½L = 30 m
b = 2,37 . 10 = 23,7 kN
$x_{zw} = a/\pi = 9,55$ m

$V_{max} = A = \frac{a.b}{\pi} = 226,3$ kN/m lengte

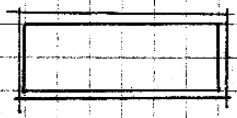
$m_{max} = \frac{ab}{\pi} . 2 . \frac{a}{\pi} = 4322,4$ kNm/m lengte

Platen

* Dek



* Beschouw platen
Allen star opgelegd, lijnvormige opleggingen



$L_4/L_x = 15/10 = 1.5$

$q = 40\ kN/m^2$
$g_{eg} = 22.5\ kN/m^2 + 62.5\ kN/m^2$

Lineaire elasticiteitstheorie

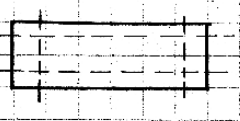
ⓐ plaat type I-2 NEN 6720

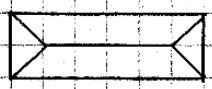
Zie verder

aanname: kraanbelasting gaat voor 90% direct de wand in

ⓑ Evenwichtsmethode (strip methode)

Verdeling in stripen → verdeling vindt via dwarskrachtnullijnen

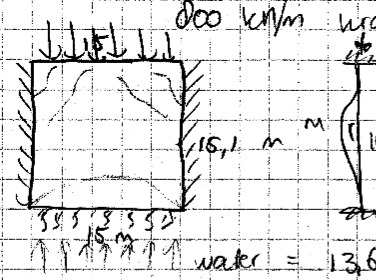




Beschouw krachtsverdeling in "liggertjes" binnen stripen

* Buitenwand met waterdruk ⊥ vlak
* Buitenwand met connectiekrachten ⊥ vlak // vlak etc
* Binnenwand met kraanbelasting in // zijn vlak
* Vloer met ballast belasting + opwaartse druk

* Binnenwand met kraanbelasting



N = 936 kN $\sigma_n = \frac{F}{A} = \frac{936 \cdot 10^3}{500 \cdot 1000} = 1,87\ N/mm^2$ druk

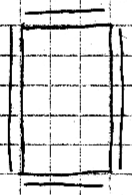
voldoet

water = 13,6 m * 10 = 136 kN/m'
ballast leeg

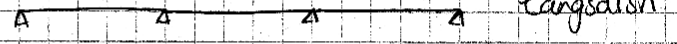
M berekenen volgens stabiliteit

* Vloer met ballast en opwaartse druk

meest ongunstig = géén ballast tijdens operatie
(tijdens transport is vrijboord kleiner, maar dan is ruim (8 m) ballast aanwezig).

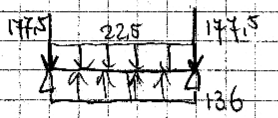


$l_y/l_x = 15/10 = 1,5$
Plaattype I-2



wand gewicht = 0,5 x 25 x 14,2 x 1 = 177,5 kN/m' ↓
(↑ h binnen)

restgewicht = 81,5 g kN/m' langsrichting ↓ hele pontondrsn
vloer gewicht = 0,9 x 25 x 1 = 22,5 kN/m' ↓



177,5 - 136 = 41,5 kN/m'
136 - 22,5 = 113,5 kN/m'



Schematisatie voor standaardgeval p_d = 113,5 kN/m' ↑

M_{vx} A = 0,046 $p_d l_y l_x^2$ = 7831,5 kNm A=C
B = 0,019 " = 3234,8 kNm
C = 0,046 " = 7831,5

M_{vy} I=III = 0,010 $p_d l_x l_y^2$ = 2553,8 kNm
II = 0

Parameter	Symbol		Unit

Structural Mechanics

During operation **LC II** excl connection forces

gamma = 1,5

Lwave		60,00	m
Load combi: self weight + water + waves + equipment + ballast			
draught	T	13,60	m
wave height 1/1 yrs	H	2,37	m
q water		136,00	kN/m^2
q waves		286,00	kN/m^2
q sw =		86,30	kN/m^2
q ballast =		2,00	kN/m1
q traffic =		0,00	kN/m1
q crane 2xtrack =		800,00	kN/m1
		80,00	kN/m1
q storage =		40,00	kN/m1

Manual calculations

Longitudinal direction,

complete pontoon section

complete pontoon section	4322,40	kNm/m pontoon section
Ms =	0,00	kNm/m
Vmax =	226,30	kN/m

NEN 6720 Plate/ slab I-2; theory of elasticity **VBC table 18 & GTB 8.4.b** **III**

pd = 40 + (0,8*25*1) =	pd	60	kN/m2	top slab			E B65	3,85E+07	kN/m^2						
buoyancy	pd	111	kN/m2	bottom slab											
compartment 1: 15 * 10 m							**compartment 2: 15 * 5 m**		outer compartment						
length	ly	15	m				length	ly	15	m					
width	lx	10	m				width	lx	5	m					
plate thickness	d	0,7	m				plate thickness	d	0,7	m					
	ly / lx	1,5	[-]					ly / lx	3	[-]					
slab stiffness	EI	1,1E+16	Nmm^2	=	1,10E+07	kNm^2	slab stiffness	EI	5,50E+15	Nmm^2	=	5,50E+06	kNm^2		
mid section	0,5 lx	5	m	0,5ly	7,50	m	mid section	0,5 lx	2,5	m	0,5ly	7,50	m		
edge section	0,25lx	2,5	m	0,25ly	3,75	m	edge section	0,25lx	1,25	m	0,25ly	3,75	m		

Top slab	coefficient			gamma	design values	deformation d [m]
Mv;x	0,034	204,00	kNm	1,5	306,0	
Mv;y	0,011	66,00	kNm	1,5	99,0	
Ms;x	-0,075	-450,00	kNm	1,5	-675,0	1,0E-04
Ms;y	-0,055	-330,00	kNm	1,5	-495,0	
ax/lx	0,19	1,90	m			
ay/ly	0,14	2,10	m			
m*xx4=6	-0,068	-408,00	kNm	1,5	-612,0	
m*xx5	0,031	186,00	kNm	1,5	279,0	
m*yyd=f	-0,047	-282,00	kNm	1,5	-423,0	
m*yye	0,016	96,00	kNm	1,5	144,0	

Top slab	coefficient			gamma	design values	deformation d [m]
Mv;x	0,043	64,50	kNm	1,5	96,75	
Mv;y	0,01	15,00	kNm	1,5	22,5	
Ms;x	-0,083	-124,50	kNm	1,5	-186,75	1,3E-05
Ms;y	-0,049	-73,50	kNm	1,5	-110,25	
ax/lx	0,19	0,95	m			
ay/ly	0,06	0,90	m			
m*xx4=6	-0,083	-124,50	kNm	1,5	-186,75	
m*xx5	0,041	61,50	kNm	1,5	92,25	
m*yyd=f	-0,044	-66,00	kNm	1,5	-99	
m*yye	0,015	22,50	kNm	1,5	33,75	

Bottom slab	coefficient			gamma	design values	deformation d [m]
Mv;x	0,034	377,40	kNm	1,5	566,1	
Mv;y	0,011	122,10	kNm	1,5	183,2	
Ms;x	-0,075	-832,50	kNm	1,5	-1248,8	1,9E-04
Ms;y	-0,055	-610,50	kNm	1,5	-915,8	
ax/lx	0,19	1,90	m			
ay/ly	0,14	2,10	m			
m*xx4=6	-0,068	-754,80	kNm	1,5	-1132,2	
m*xx5	0,031	344,10	kNm	1,5	516,2	
m*yyd=f	-0,047	-521,70	kNm	1,5	-782,6	
m*yye	0,016	177,60	kNm	1,5	266,4	

Bottom slab	coefficient			gamma	design values	deformation d [m]
Mv;x	0,043	119,33	kNm	1,5	178,9875	
Mv;y	0,01	27,75	kNm	1,5	41,625	
Ms;x	-0,083	-230,33	kNm	1,5	-345,4875	2,4E-05
Ms;y	-0,049	-135,98	kNm	1,5	-203,9625	
ax/lx	0,19	0,95	m			
ay/ly	0,06	0,90	m			
m*xx4=6	-0,083	-230,33	kNm	1,5	-345,4875	
m*xx5	0,041	113,78	kNm	1,5	170,6625	
m*yyd=f	-0,044	-122,10	kNm	1,5	-183,15	
m*yye	0,015	41,63	kNm	1,5	62,4375	

Summary FEM; slab calculation VI

hydrostatic pressure	pd	150	kN/m2	bottom slab	E B65	3,85E+07	kN/m^2	
incl wave load		24	kN/m2					

Outer wall: 15 * 16 m

length	ly	16						
width	lx	15						
plate thickness	d	0,8						
	ly / lx	1,07						
EI		2,464E+16	Nmm^2	=	2,46E+07	kNm^2		

slab	coefficient		gamma	design values	deformation d [mm]
Mv;x =	338,00	kNm	1,5	507	4,0E+00
Mv;z =	345,60	kNm	1,5	518,4	
Ms;x =	-838,20	kNm	1,5	-1257,3	
Ms;z =	-1037,90	kNm	1,5	-1556,85	
Vmax x =	456,90	kN	1,5	685,35	
V max z =	605,10	kN	1,5	907,65	
Mt =	155,50	kNm	1,5	233,25	

MxD+ ULS (span)	509,90	kNm
MyD+	535,80	kNm
MxD- ULS (support)	1264,20	kNm
MyD-	1563,90	kNm

Inner wall: 15 * 15 m

length	ly	15						
width	lx	15						
plate thickness	d	0,7						
	ly / lx	1						
EI		1,6507E+16	Nmm^2	=	1,65E+07	kNm^2		

slab	coefficient		gamma	design values	deformation d [mm]
Mv;x =	309,00	kNm	1,5	463,5	9,8E+00
Mv;z =	347,40	kNm	1,5	521,1	
Ms;x =	-801,80	kNm	1,5	-1202,7	
Ms;z =	-1025,50	kNm	1,5	-1538,25	
Vmax x =	452,30	kN	1,5	678,45	
V max z =	610,50	kN	1,5	915,75	
Mt =	149,5	kNm	1,5	224,25	

MxD+ ULS (span)	469,2	kNm
MyD+	537,6	kNm
MxD- ULS (support)	1204,4	kNm
MyD-	1542,9	kNm

Summary results from FEM calculations **II**

Transversal cross section

Top slab			
Mvy =	1755,00 kNm	1,5	2632,5 kNm
Msy =	-2203,10 kNm	1,5	-3304,65 kNm
Nmax =	159,40 kN	1,5	239,1 kN
Nmin =	-1280,00 kN	1,5	-1920 kN
Vmax =	677,10 kN	1,5	1015,65 kN
Bottom slab			
Mvy =	2364,70 kNm	1,5	3547,05 kNm
Msy =	-1352,80 kNm	1,5	-2029,2 kNm
Nmax =	0,00 kN	1,5	0 kN
Nmin =	-1280,00 kN	1,5	-1920 kN
Vmax =	827,70 kN	1,5	1241,55 kN
Outer walls			
Mvz =	2359,80 kNm	1,5	3539,7 kNm
Msz =	-1031,30 kNm	1,5	-1546,95 kNm
Nmax =	233,30 kN	1,5	349,95 kN
Nmin =	-541,80 kN	1,5	-812,7 kN
Vmax =	924,90 kN	1,5	1387,35 kN
Inner longitudinal walls			
Mvz =	1423,80 kNm	1,5	2135,7 kNm
Msz =	-1446,70 kNm	1,5	-2170,05 kNm
Nmax =	0,00 kN	1,5	0 kN
Nmin =	-1379,50 kN	1,5	-2069,25 kN
Vmax =	12,00 kN	1,5	18 kN

Longitudinal cross section

Top slab	1/50*			
Mv =	73815,80	1476,32 kNm	1,5	2214,474 kNm
Ms =	-132816,50	-2656,33 kNm	1,5	-3984,495 kNm
Nmax =	0,00	0,00 kN	1,5	0 kN
Nmin =	-39840,90	-796,82 kN	1,5	-1195,227 kN
Vmax =	36072,20	721,44 kN	1,5	1082,166 kN
Bottom slab				
Mv =	132264,6	2645,29 kNm	1,5	3967,938 kNm
Ms =	-63199,5	-1263,99 kNm	1,5	-1895,985 kNm
Nmax =	0	0,00 kN	1,5	0 kN
Nmin =	-35343,4	-706,87 kN	1,5	-1060,302 kN
Vmax =	39239,1	784,78 kN	1,5	1177,173 kN
Outer walls				
Mv =	82134,50	1642,69 kNm	1,5	2464,035 kNm
Ms =	-82691,30	-1653,83 kNm	1,5	-2480,739 kNm
Nmax =	0,00	0,00 kN	1,5	0 kN
Nmin =	-35738,10	-714,76 kN	1,5	-1072,143 kN
Vmax =	37743,60	754,87 kN	1,5	1132,308 kN
Inner transversal walls				
Mv =	68905,60	1378,11 kNm	1,5	2067,168 kNm
Ms =	-68809,90	-1376,20 kNm	1,5	-2064,297 kNm
Nmax =	0,00	0,00 kN	1,5	0 kN
Nmin =	-67188,00	-1343,76 kN	1,5	-2015,64 kN
Vmax =	8840,80	176,82 kN	1,5	265,224 kN

1.3 Design calculations [spreadsheets] of pontoons

Parameter	Symbol	value	Unit					
material properties								
	f'ck	f'b	fb	fbm	fbr	fbrk	E'b	
B45	45	27	1,65	3,3			33500	N/mm^2
B55	55	33	1,9	3,8			36000	N/mm^2
B65	65	39	2,15	4,3	5,76	4,12	38500	N/mm^2
FeB500							2,10E+05	N/mm^2
S355							2,10E+05	N/mm^2

Deck plates				
H deck		700	mm	
B65				
concrete cover	c	50	mm	
cover on prestressing	cp min	55	mm	
	cp max	305	mm	
Assumption reinforcement regarding concrete cracking				
FeB500				
diameter reinforcement		16	mm	
diameter reinforcement		20	mm	
As: d 16-100		2011	mm^2	
As: d 20-100		3142	mm^2	
internal leverage arm	ds	626	mm	ds=h - c - 1/2*diam;s
	dp max	614	mm	
	dp min	350	mm	
Prestressing steel FeP1860				
FeP1860	f p;rep	1600	N/mm^2	
	f p	1450	N/mm^2	
	f pu;rep	1860	N/mm^2	
	f pu	1690	N/mm^2	
	Ep	2,0E+05	N/mm^2	
max initial stress	sigma pi	1350	N/mm^3	
max stress during tensioning	sigma p0	1450	N/mm^4	
Initial prestressing65%	sigma p0	1217,5	N/mm^2	
relaxation 1000 hrs	dsigma p;rel	24,35		

Van Hattum en Blankevoort | werknr.: | datum: | betreft: | naam: | bl. nr.: van top slab

Top slabs : Statically indeterminate structure

$A_b = 700 \cdot 1000 = 7 \cdot 10^5 \ mm^2/m'$
$W = \frac{1}{6} b h^2 = 8{,}167 \cdot 10^7 \ mm^3$
B6s prefabricated slabs connected by seams casted in situ

Possible structures:
1. central continuous tensioning, installed through all prefabricated deck slabs at post-tensioning of the deck
2. Pre-tensioned prefabricated deck slabs connected by anchors / reinforcement in the seams

ad 1. Self weight of deck slab should be carried only by reinforcement during construction

 or suspended from crane

$g_{sw} = 0{,}7 \times 25 = 17{,}5 \ kN/m^2$

GTB 1990 – 8.2 Plate type III-1  $l_y/l_x = 1{,}5$

L = 15 m
B = 10 m
h = 0,7 m
$W = 8{,}167 \cdot 10^7 \ mm^3$
$I = 2{,}858 \cdot 10^{10} \ mm^4$

Reinforcement moments:

$m_{ux,max}^* = 0{,}101 \cdot 17{,}5 \cdot 10^2 = 316{,}8 \ kNm$
$m_{sx,max}^* = 0{,}106 \cdot 17{,}5 \cdot 10^2 = 185{,}5 \ kNm$

$m_{uy,max}^* = 0{,}266 \cdot 17{,}5 \cdot 10^2 = 465{,}5 \ kNm$
$m_{sy,max}^* = 0{,}111 \cdot 17{,}5 \cdot 10^2 = 194{,}3 \ kNm$

$\delta = 0{,}0085 \frac{17{,}5 \cdot 10^4}{EI} = 0{,}0135 \ m = 13{,}5 \ mm$ ok.

$EI = 38500 \cdot \frac{1}{12} \cdot 1000 \cdot 700^3 = 1{,}1005 \cdot 10^{15} \ Nmm^2$
$= 1{,}1005 \cdot 10^6 \ kNm^2$

$m_{d,max} = 465{,}5 \ kNm$
$\frac{m_d}{f'_c b d^2} = \frac{465{,}5}{39 \cdot 1 \cdot 0{,}621^2} = 30{,}95$

$\sigma = m/W = \frac{465{,}5 \cdot 10^6}{8{,}17 \cdot 10^7} = 5{,}7 \ N/mm^2 > f_b$

$\phi_{b\,min} = 0{,}27\ \%$
$A_{s\,min} = 0{,}27 \cdot b \cdot h \cdot 10^{-4} = 0{,}27 \cdot 1 \cdot 0{,}7 \cdot 10^4 = 1890 \ mm^2/m'$
Apply $\phi 16 - 100 = 2011 \ mm^2$ (practical reinforcement)

Conclusion: Only small reinforcement is needed during construction (transport, installing)
Central post-tensioning can be applied later connecting all prefabricated slabs

Van Hattum en Blankevoort

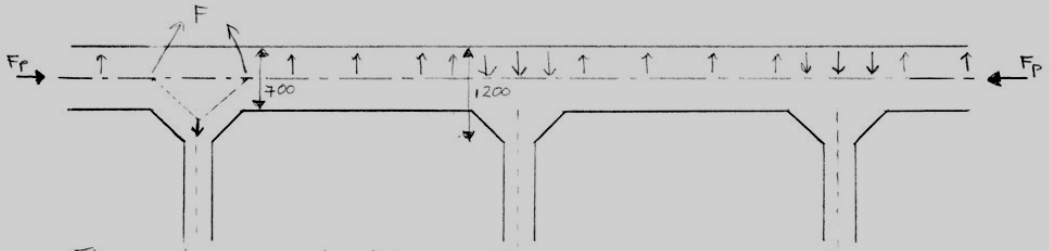
werknr.: ..... datum: .....

betreft: ..... naam: .....

..... bl. nr.: 2 van top slabs

Top slabs

During operation:
- Continuous deck
- Statically undetermined structure (multiple plate fields)
- post-tensioning with attachment



The system's centre line differs from the course of the prestressing elem

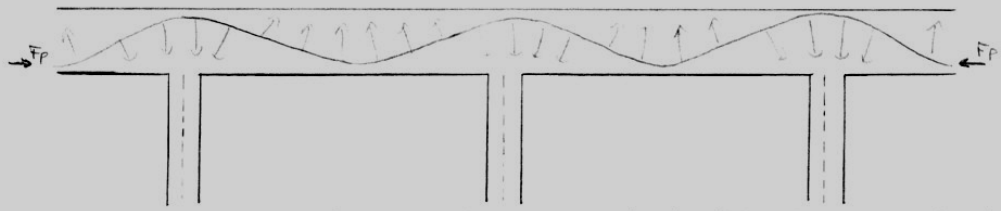
Locally: $e_p = \frac{1200}{2} - \frac{700}{2} = 250$ mm

$R = \infty$ straight cables

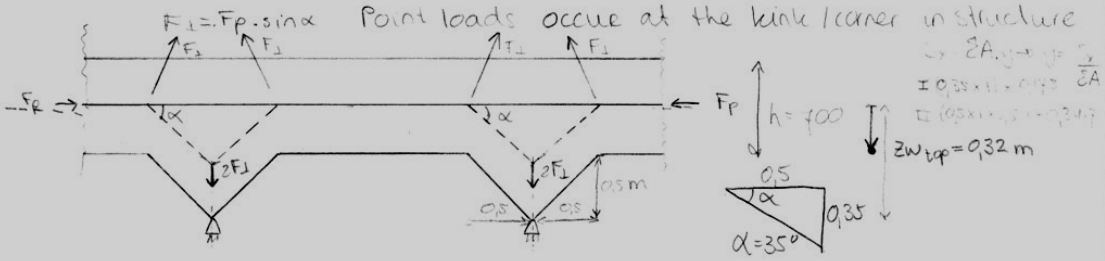
$q_p = 8 F_p \cdot f$

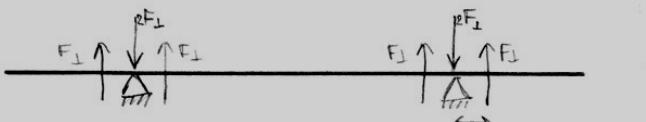
$q = F_p / R = 0$ curvature $\frac{1}{R} = 0$

The above is a practical approach of the following principle:

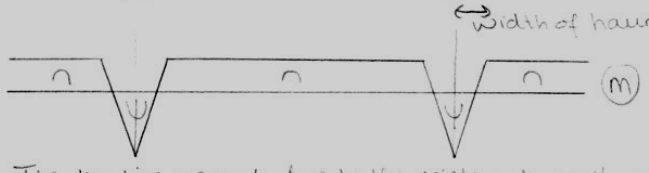


$F_\perp = F_p \cdot \sin\alpha$ Point loads occue at the kink / corner in structure





2. $F_\perp$ goes directly into the supporting wall



The bending moments due to the point loads partly compensate M_{sw}, M_g from LC 2

Properties of top slab B65						
x*y = 10*15 m	h	700	mm			
	b	1000	mm			
	Ab	700000	mm^2			
	Lx	10	m	Ly	15	m
	W	8,17E+07	mm^3			
	mvx	306,0	kNm	mvy	144,0	kNm
	msx	-675,0	kNm	msy	-495,0	kNm
	vx	721,4	kN	vy	677,1	kN
stresses from loads	sigma v;x	3,75	N/mm^2	sigma v;y	1,76	N/mm^2
	sigma s;x	-8,27	N/mm^2	sigma s;y	-6,06	N/mm^2
Type of prestressing	straight cable			1 strand includes 7 threads		
diametre strand I	d	12,9	mm			
	Ap	100	mm^2			
diametre strand II	d	15,7	mm			
	Ap	150	mm^2			
nr of strnds	n	14	12-37			
nr of tubes /m		1		hoh 1000 mm		
cover on prestressing	ap	67	mm	ap=cp+diam;s-1/2*diam;p		
	Ap per m1	2100	mm^2			
	Fpi	-2835	kN/m			
prestressing level	sigma p	-4,05	N/mm^3			
haunches	b	0,5	m			
	h	0,5	m			
haunches on support	t	0,5	m			
delta system line	z	0,18	m			
	tan alpha	0,240				
	alpha	13	degrees			
		0,227	rad			
Point load from prestressing cable	F	637,7	kN/m	fp*sin(alpha)		
distance point load to support	ax	1,25	m	ay	1,25	m
distance F to other support	bx	8,75	m	by	13,75	m
	Msupport	698	kNm	Msupport	731	kNm
	sigma supp	8,5	N/mm^2	sigma supp	8,9	N/mm^2
	Mspan	-99,65	kNm	Mspan	-66,43	kNm
	sigma span	-1,22	N/mm^2	sigma span	-0,81	N/mm^2
Total stress in cross section	sigma v;x	-6,58	N/mm^2	top		
		-1,52	N/mm^2	bottom		
	sigma s;x	-4,33	N/mm^2	top		
		-3,77	N/mm^2	bottom		
	sigma v;y	-5,00	N/mm^2	top		
		-3,10	N/mm^2	bottom		
	sigma s;y	-6,94	N/mm^2	top		
		-1,16	N/mm^2	bottom		
	Fpw	-2409,75	kNm/m			

Properties of bottom slab B65							
x*y = 10*15 m	h	800	mm				
	b	1000	mm				
	Ab	800000	mm^2				
	l	15	m				
	W	1,07E+08	mm^3				
	mvx	566,1	kNm	mvy	255,4	kNm	
	msx	-1248,8	kNm	msy	-915,8	kNm	
	vx		kN	vy		kN	
stresses from loads	sigma v;x	5,31	N/mm^2	sigma v;y	2,39	N/mm^2	
	sigma s;x	-11,71	N/mm^2	sigma s;y	-8,59	N/mm^2	
Type of prestressing	straight tendon		1 strand includes 7 threads				
diametre strand I	d	12,9	mm				
	Ap	100	mm^2				
diametre strand II	d	15,7	mm				
	Ap	150	mm^2				
nr of strnds	n	19	12-37				
nr of sleeves /m		2		hoh 500 mm			
cover on prestressing	cp	67	mm	ap=cp+diam;s-1/2*diam;p			
	Ap per m1	5700	mm^2				
	Fpi	-7695	kN/m				
prestressing level	sigma p	-9,62	N/mm^3				
haunches	b	0	m				
	h	0	m				
haunches on support	t	0	m				
delta system line	z	0	m				
	tan alpha	0,000					
	alpha	0	degrees				
		0,000					
Point load from prestressing steel	F	0	kN/m	fp*sin(alpha)			
distance point load to support	a	0	m				
distance F to other support	b	0	m				
	Msupport	0	kNm				
	sigma supp	0	N/mm^2				
	Mspan	0,00	kNm				
	sigma span	0,00	N/mm^2				
Total stress in cross section	sigma v;x	-14,93	N/mm^2	top	ULS		
		-4,31	N/mm^2	bottom	ULS		
	sigma s;x	2,09	N/mm^2	top	ULS		
		-21,33	N/mm^2	bottom	ULS		
	sigma v;y	-12,01	N/mm^2	top	ULS		
		-7,22	N/mm^2	bottom	ULS		
	sigma s;y	-1,03	N/mm^2	top	ULS		
		-18,20	N/mm^2	bottom	ULS		

Berekening van een gescheurde I-vormige doorsnede met voorspanning en normale wapening (met oplosser)

Invoercellen | Iteratiewaard | Overschrijding

Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Beton		Ligger			Belastingen			Berekeningen		
Sterkteklasse =	C53/65	Totale hoogte =	800 mm		Uitwendige (SLS)normaalkracht op de doorsnede =	0 kN		Hoogte drukzone BGT =	781,5 mm	
$f`_{ck}$ =	65 MPa	A_b =	800000 mm^2		Uitwendige (ULS) normaalkracht op de doorsnede =	0 kN		ε_b =	-0,00069	
$f`_b$ =	39 MPa	zwpt tov boven z_{bb} =	400 mm		Normaalkracht door voorspanning $-F_p$ =	-6298,5 kN		Verschil M (of N bij N_{SLS}=0) =	0 kNm (of kN bij N_{SLS}=0)	
ε_{bpl} =	0,00175	zwpt tov onder z_{bo} =	400 mm		Normaalkracht (SLS) inclusief voorspanbelasting =	-6299 kN		Hoogte drukzone UGT =	291,3 mm	
ε_{bu} =	0,00350	I_b =	4,27E+10 mm^4		Normaalkracht (ULS) inclusief voorspanbelasting =	-6299 kN		Constante deel drukzone =	145,6 mm	
E_b* =	22286 MPa	W_{bb} (boven) =	1,07E+08 mm^3		Moment (SLS) inclusief voorspanbelasting =	832,5 kNm		Verschil N =	0 kN	
α =	0,75	W_{bo} (onder) =	1,07E+08 mm^3		Moment ULS inclusief voorspanbelasting =	1249 kNm		M_u =	2624 kNm	
β =	0,389	k_{bb} (boven) =	133 mm		Betonspanning bovenzijde SLS σ_{bb} =	-15,68 MPa				
		k_{bo} (onder) =	133 mm		Betonspanning onderzijde SLS σ_{bo} =	-0,07 MPa				

							BGT											UGT					
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\,rechthoek}/\varepsilon_b$	$N`_{b\,driehoek}/\varepsilon_b$	$e_{b\,rechthoek}$	$e_{b\,driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\,rechthoek}$	$N`_{b\,driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma'_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	800	1000	8,0000E+05	400	3,20E+08	1,7067E+11	1,000	ONWAAR	WAAR	0	8708682	0	-139	-1214721	0	-6044	843	-15,5	0,00350	39,0	ONWAAR	WAAR	145,644282	ONWAAR
Laag 2			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	800	0,00E+00	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00611	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	800		8,0000E+05		3,20E+08	1,7067E+11				0	8708682			-1214721	0	-6044	843			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2000	1	624	250	435	0,202	80634	224	18062	-56	-13	-28	0,00400	435	870	224	194,782609
Laag 2	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Totaal			2000						80634		18062	-56	-13				870		195

	Voorspanstaal						BGT										UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	5700	0,5	390	6298,5	1691	1522	1105	0,501	285565	-10	-2856	-198	2	-35	1070	0,00119	0,0067	237	1342	1352	-10	-14
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Totaal			5700			6298,5					285565		-2856	-198	2							1352		-14

Berekening van een gescheurde I-vormige doorsnede met voorspanning en normale wapening (met oplosser)

Invoercellen | Iteratiewaard | Overschrijding

Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Beton			Ligger			Belastingen			Berekeningen		
Sterkteklasse =	C53/65		Totale hoogte =	800	mm	Uitwendige (SLS)normaalkracht op de doorsnede =	0	kN	Hoogte drukzone BGT =	786,4	mm
$f`_{ck}$ =	65	MPa	A_b =	800000	mm^2	Uitwendige (ULS) normaalkracht op de doorsnede =	0	kN	ε_b =	-0,00051	
$f`_b$ =	39	MPa	zwpt tov boven z_{bb} =	400	mm	Normaalkracht door voorspanning $-F_p$ =	-4641	kN	erschil M (of N bij N_{SLS}=0) =	0	kNm (of kN bij N_{SLS}=0)
ε_{bpl} =	0,00175		zwpt tov onder z_{bo} =	400	mm	Normaalkracht (SLS) inclusief voorspanbelasting =	-4641	kN	Hoogte drukzone UGT =	246,7	mm
ε_{bu} =	0,00350		I_b =	4,27E+10	mm^4	Normaalkracht (ULS) inclusief voorspanbelasting =	-4641	kN	Constante deel drukzone =	123,4	mm
E_b* =	22286	MPa	W_{bb} (boven) =	1,07E+08	mm^3	Moment (SLS) inclusief voorspanbelasting =	610,7	kNm	Verschil N =	0	kN
α =	0,75		W_{bo} (onder) =	1,07E+08	mm^3	Moment ULS inclusief voorspanbelasting =	916	kNm	M_u =	2372	kNm
β =	0,389		k_{bb} (boven) =	133	mm	Betonspanning bovenzijde SLS σ_{bb} =	-11,53	MPa			
			k_{bo} (onder) =	133	mm	Betonspanning onderzijde SLS σ_{bo} =	-0,08	MPa			

							BGT											UGT						
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma`_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	800	1000	8,0000E+05	400	3,20E+08	1,7067E+11	1,000	ONWAAR	WAAR	0	8762628	0	-138	-1208104	0	-4490	619	-11,4	0,00350	39,0	ONWAAR	WAAR	123,374337	ONWAAR
Laag 2			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	800	0,00E+00	0,0000E+00	-0,017	ONWAAR	ONWAAR	0	0	400	395	0	0	0	0		-0,00785	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	800		8,0000E+05		3,20E+08	1,7067E+11				0	8762628			-1208104	0	-4490	619			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2000	1	624	250	435	0,207	82600	224	18502	-42	-9	-21	0,00535	435	870	224	194,782609
Laag 2	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Totaal			2000						82600		18502	-42	-9				870		195

	Voorspanstaal						BGT										UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	4200	0,5	390	4641	1691	1522	1105	0,504	211706	-10	-2117	-108	1	-26	1079	0,00203	0,0076	406	1511	1707	-10	-17
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Totaal			4200			4641					211706		-2117	-108	1							1707		-17

Outer walls without haunches! :steunpuntmoment verticaal
Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Invoercellen | Iteratiewaarde | Overschrijding

Beton		Ligger		Belastingen		Berekeningen	
Sterkteklasse =	C53/65	Totale hoogte =	800 mm	Uitwendige (SLS)normaalkracht op de doorsnede =	0 kN	Hoogte drukzone BGT =	747,4 mm
$f`_{ck}$ =	65 MPa	A_b =	800000 mm^2	Uitwendige (ULS) normaalkracht op de doorsnede =	0 kN	ε_b =	-0,00084
$f`_b$ =	39 MPa	zwpt tov boven z_{bb} =	400 mm	Normaalkracht door voorspanning $-F_p$ =	-7293 kN	Verschil M (of N bij N_{SLS}=0) =	0 kNm (of kN bij N_{SLS}=0)
ε_{bpl} =	0,00175	zwpt tov onder z_{bo} =	400 mm	Normaalkracht (SLS) inclusief voorspanbelasting =	-7293 kN	Hoogte drukzone UGT =	316,1 mm
ε_{bu} =	0,00350	I_b =	4,27E+10 mm^4	Normaalkracht (ULS) inclusief voorspanbelasting =	-7293 kN	Constante deel drukzone =	158,1 mm
E_b* =	22286 MPa	W_{bb} (boven) =	1,07E+08 mm^3	Moment (SLS) inclusief voorspanbelasting =	1043 kNm	Verschil N =	0 kN
α =	0,75	W_{bo} (onder) =	1,07E+08 mm^3	Moment ULS inclusief voorspanbelasting =	1564 kNm	M_u =	2750 kNm
β =	0,389	k_{bb} (boven) =	133 mm	Betonspanning bovenzijde SLS σ_{bb} =	-18,89 MPa		
		k_{bo} (onder) =	133 mm	Betonspanning onderzijde SLS σ_{bo} =	0,66 MPa		

						BGT											UGT					
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma`_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	800	1000	8,0000E+05	400	#########	1,7067E+11	1,000	ONWAAR	WAAR	0	8328395	0	-151	-1256422	0	-6975	1052	-18,7	0,00350	39,0	ONWAAR	WAAR	158,0661	ONWAAR
Laag 2			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	800	#########	0,0000E+00	-0,070	ONWAAR	ONWAAR	0	0	400	382	0	0	0	0		-0,00536	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	800		8,0000E+05		#########	1,7067E+11				0	8328395			-1256422	0	-6975	1052			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2011	1	628	250	435	0,160	64262	228	14652	-54	-12	-27	0,00345	435	874	228	199,35
Laag 2	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Totaal			2011						64262		14652	-54	-12				874		199

	Voorspanstaal						BGT											UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	6600	0,5	390	7293	1691	1522	1105	0,478	315615	-10	-3156	-264	3	-40	1065	0,00082	0,0063	164	1269	1080	-10	-11
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Totaal			6600			7293					315615		-3156	-264	3							1080		-11

Properties of inner walls B65

	h	800	mm			
	b	1000	mm			
	Ab	800000	mm^2			
	l	15	m			
	W	1,07E+08	mm^3			
	mvx	375,36	kNm	mvz	430,05	kNm
	msx	-963,50	kNm	msz	-1234,32	kNm
stresses from loads	sigma v;x	3,52	N/mm^2	sigma v;z	4,03	N/mm^2
	sigma s;x	-9,03	N/mm^2	sigma s;z	-11,57	N/mm^2
				sigma v;c	0,00	N/mm^2
Type of prestressing	straight tendon		1 strand includes 7 threads			
diametre strand I	d	12,9	mm			
	Ap	100	mm^2			
diametre strand II	d	15,7	mm			
	Ap	150	mm^2			
nr of strnds	n	19	12-37			
nr of sleeves /m		2		hoh 500 mm		
cover on prestressing	ap	67	mm	ap=cp+diam;s-1/2*diam;p		
	Ap per m1	5700	mm^2			
	Fpi	-7695	kN/m			
prestressing level	sigma p	-9,62	N/mm^3			
haunches	b	0	m			
	h	0	m			
haunches on support	t	0,5	m			
delta system line	z	0,722	m			
	tan alpha	2,888				
	alpha	0	degrees			
		0,000				
Point load from prestressing steel	F	0	kN/m	fp*sin(alpha)		
distance point load to support	a	1,25	m			
distance F to other support	b	13,75	m			
	Msupport	0	kNm			
	sigma supp	0	N/mm^2			
	Mspan	0,00	kNm			
	sigma span	0,00	N/mm^2			
Total stress in cross section	sigma v;x	-13,14	N/mm^2	outside	ULS	
		-6,10	N/mm^2	inside	ULS	
	sigma s;x	-0,59	N/mm^2	outside	ULS	
		-18,65	N/mm^2	inside	ULS	
	sigma v;z	-13,65	N/mm^2	outside	ULS	
		-5,59	N/mm^2	inside	ULS	
	sigma s;z	1,95	N/mm^2	outside	ULS	
		-21,19	N/mm^2	inside	ULS	

Inner walls: steunpuntmoment verticaal

Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Invoercellen Iteratiewaard Overschrijding

Beton		Ligger			Belastingen			Berekeningen		
Sterkteklasse =	C53/65	Totale hoogte =	800 mm		Uitwendige (SLS)normaalkracht op de doorsnede =	0 kN		Hoogte drukzone BGT =	780,4 mm	
$f`_{ck}$ =	65 MPa	A_b =	800000 mm^2		Uitwendige (ULS) normaalkracht op de doorsnede =	0 kN		ε_b =	-0,00069	
$f`_b$ =	39 MPa	zwpt tov boven z_{bb} =	400 mm		Normaalkracht door voorspanning $-F_p$ =	-6196,5 kN		erschil M (of N bij N_{SLS}=0) =	0 kNm (of kN bij N_{SLS}=0)	
ε_{bpl} =	0,00175	zwpt tov onder z_{bo} =	400 mm		Normaalkracht (SLS) inclusief voorspanbelasting =	-6197 kN		Hoogte drukzone UGT =	287,7 mm	
ε_{bu} =	0,00350	I_b =	4,27E+10 mm^4		Normaalkracht (ULS) inclusief voorspanbelasting =	-6197 kN		Constante deel drukzone =	143,8 mm	
E_b* =	22286 MPa	W_{bb} (boven) =	1,07E+08 mm^3		Moment (SLS) inclusief voorspanbelasting =	822,88 kNm		Verschil N =	0 kN	
α =	0,75	W_{bo} (onder) =	1,07E+08 mm^3		Moment ULS inclusief voorspanbelasting =	1234,32 kNm		M_u =	2610 kNm	
β =	0,389	k_{bb} (boven) =	133 mm		Betonspanning bovenzijde SLS σ_{bb} =	-15,46 MPa				
		k_{bo} (onder) =	133 mm		Betonspanning onderzijde SLS σ_{bo} =	-0,03 MPa				

							BGT											UGT					
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma`_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	800	1000	8,0000E+05	400	#########	1,7067E+11	1,000	ONWAAR	WAAR	0	8695744	0	-140	-1216282	0	-5958	833	-15,3	0,00350	39,0	ONWAAR	WAAR	143,8469	ONWAAR
Laag 2			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	800	#########	0,0000E+00	-0,025	ONWAAR	ONWAAR	0	0	400	393	0	0	0	0		-0,00623	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	800		8,0000E+05		#########	1,7067E+11				0	8695744			-1216282	0	-5958	833			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2011	1	628	250	435	0,195	78538	228	17907	-54	-12	-27	0,00414	435	874	228	199,35
Laag 2	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Totaal			2011						78538		17907	-54	-12				874		199

	Voorspanstaal						BGT										UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	5400	0,5	390	6196,5	1691	1522	1148	0,500	270134	-10	-2701	-185	2	-34	1113	0,00124	0,0070	249	1396	1344	-10	-13
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Totaal			5400			6196,5					270134		-2701	-185	2							1344		-13

Inner walls: steunpuntmoment horizontaal
Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Invoercellen Iteratiewaard Overschrijding

Beton		Ligger			Belastingen			Berekeningen		
Sterkteklasse =	C53/65	Totale hoogte =	800 mm		Uitwendige (SLS)normaalkracht op de doorsnede =	0 kN		Hoogte drukzone BGT =	781,4 mm	
$f`_{ck}$ =	65 MPa	A_b =	800000 mm^2		Uitwendige (ULS) normaalkracht op de doorsnede =	0 kN		ε_b =	-0,00054	
$f`_b$ =	39 MPa	zwpt tov boven z_{bb} =	400 mm		Normaalkracht door voorspanning $-F_p$ =	-4819,5 kN		erschil M (of N bij N_{SLS}=0) =	0 kNm (of kN bij N_{SLS}=0)	
ε_{bpl} =	0,00175	zwpt tov onder z_{bo} =	400 mm		Normaalkracht (SLS) inclusief voorspanbelasting =	-4820 kN		Hoogte drukzone UGT =	248,5 mm	
ε_{bu} =	0,00350	I_b =	4,27E+10 mm^4		Normaalkracht (ULS) inclusief voorspanbelasting =	-4820 kN		Constante deel drukzone =	124,3 mm	
E_b* =	22286 MPa	W_{bb} (boven) =	1,07E+08 mm^3		Moment (SLS) inclusief voorspanbelasting =	642,33 kNm		Verschil N =	0 kN	
α =	0,75	W_{bo} (onder) =	1,07E+08 mm^3		Moment ULS inclusief voorspanbelasting =	963,50 kNm		M_u =	2389 kNm	
β =	0,389	k_{bb} (boven) =	133 mm		Betonspanning bovenzijde SLS σ_{bb} =	-12,05 MPa				
		k_{bo} (onder) =	133 mm		Betonspanning onderzijde SLS σ_{bo} =	0,00 MPa				

							BGT											UGT					
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma`_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	800	1000	8,0000E+05	400	#########	1,7067E+11	1,000	ONWAAR	WAAR	0	8707047	0	-140	-1214919	0	-4664	651	-11,9	0,00350	39,0	ONWAAR	WAAR	124,2585	ONWAAR
Laag 2			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	800	#########	0,0000E+00	-0,024	ONWAAR	ONWAAR	0	0	400	394	0	0	0	0		-0,00777	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	800		8,0000E+05		#########	1,7067E+11				0	8707047			-1214919	0	-4664	651			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2011	1	628	250	435	0,196	78958	228	18002	-42	-10	-21	0,00534	435	874	228	199,35
Laag 2	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-400	0	0	0	0	-0,00350	-435	0	-400	0
Totaal			2011						78958		18002	-42	-10				874		199

	Voorspanstaal						BGT										UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	4200	0,5	390	4819,5	1691	1522	1148	0,501	210377	-10	-2104	-113	1	-27	1121	0,00199	0,0077	375	1523	1575	-10	-16
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-400	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-400	0
Totaal			4200			4819,5					210377		-2104	-113	1							1575		-16

2 Concrete technology

Concrete is a homogeneous material. The material properties can be strongly influenced by the concrete composition. To the main components of cement, aggregates and water, additives and fillers can be added for a special purpose regarding material properties.
The main properties of concrete are:

- A large compression and small tensile strength
- The deformability of concrete influenced by internal and external forces; elasticity, shrinkage, creep, relaxation
- The important influence on concrete properties of the development of warmth (at hydration)
- The reasonably high durability and stiffness (E) of concrete

2.1 Components of concrete

A concrete composition contains:

1. Cement
2. Aggregates (pebbles and sand)
3. Water
4. Additives
5. Fillers

Ad 1. Types of cement

CEM I	:	portland cement
CEM II	:	composed portland cement (S, P, V, T, L, M)
CEM III	:	blast furnace cement
CEM III/B:		20-34% Portland cement clinker 66-80% granulates blast furnace slag
CEM IV	:	puzzolane cement
CEM V	:	composite cement

R	:	rapid
LH	:	low heat
HS	:	higher sulphate resistance
Plus	:	higher final strength

S	:	slag cement
P	:	puzzolane
V	:	fly ash (filler)
T	:	schist
L	:	lime stone
M	:	composite cement

Each cement type has its own characteristics that can make the particular type of cement less vulnerable to a certain failure mechanism. These are not discussed here, as it is beyond the scope of this thesis. The requirements and operational conditions of the pontoon shaped element lead to a dense cement type with maximum durability, especially regarding chlorides. . For this application, Portland cement and blast furnace cement have the best properties. The main differences between Portland cement and blast furnace cement are:

properties	CEM II-V Portland cement	CEM III Blast furnace cement
Development Initial strength	Faster	Slower
Development Initial strength when heated	Slower	Faster
Development end strength	higher	lower
Durability	High resistance against frost-thaw actions and against carbonation, possibly sensitive to ASR	High resistance against sulphate attack, chloride ingress (due to dense pore system) and ASR
Influence surrounding temperature on hardening	Smaller	Larger; slow hardening in winter and fast in summer
Colour	Dark, stained	Lighter and even; bluish at first
Post-treatment	Shorter duration	Longer duration
Heat/warmth development	higher	Lower and therefore less sensitivity to cracking due to temperature

Blast furnace cement has a high durability and resistance against chlorides. It ensures a dense pore structure and will therefore be very well suitable for the pontoon shaped elements, providing that the ultimate concrete strength is sufficient. In practice, concrete with blast furnace cement will reach a maximum quality of B65. High strength concrete of a higher quality is made with portland cement and fillers.The concrete strength can be approximated according to the next formula:

$$f'_{cm}(n) = a * N_n + \frac{b}{wcr} - c$$ *[ref. Betonpocket 2003 by Enci and Mebin]*

For CEM II-V:
a = 0,85
b = 33
c = 62
B100: f'_{cm} (28 days) = 0,85*100+(33/0,4) – 62 = 105.5 N/mm^2
B65: f'_{cm} (28 days) = 0,85*65+(33/0,4) – 62 = 75,8 N/mm^2

For CEM III/B
a = 0,75
b = 18
c = 30
B100: f'_{cm} (28 days) = 0,75*100+(18/0,4) – 30 = 90 N/mm^2
B65: f'_{cm} (28 days) = 0,75*65+(18/0,4) – 30 = 63,8 N/mm^2
Application of additives allows a lower water cement ratio. As such, a concrete strength of 65 N/mm^2 can be realised. B100 however is not considered to be feasible when applying blast furnace cement.

Ad 2. Aggregates

The aggregate part in concrete consists of sand and pebbles or (broken) stone. The characteristics of these aggregates vary in strength, colour, dimensions, shape and chemical composition, depending on the location of reclamation. Chemical reaction within the concrete due to the presence of salts or lime within the aggregates should be prevented. The aggregate to be applied should be tested on the presence of lime, which can trigger an Alkali Silica Reaction (ASR) in the concrete. This leads to expansion and consequently cracking of the concrete.

Concrete with a high strength should be very dense. The composition should include all sizes of aggregates and fillers. Aggregates with a rough surface and a sharp-edged or angular shape are better to compact while the grains experience a lot of friction. Therefore sharp-edged aggregates are

mostly used in high strength concrete and smooth sea, dune or river sand and pebbles can be applied in normal concrete.

Ad 4. Application of additives

Properties for application	Type of additive
Easy processing (fluid)	Plasticizer, superplasticizer, hyperplasticizer
Casting in or under water	Colloidal additive
Longer processing time/delayed hydration	decelerator
Shorter processing time / earlier hydration	accelerator
Casting on diagonal plane without upper formwork	Thixotropic additive
Self compacting or easy compacting concrete	hyperplasticizer
Freeze and thawing salt resistant wcr<0,45	Plasticizer, superplasticizer, hyperplasticizer
Freeze and thawing salt resistant wcr>0,45	Air entraining agent, MHK
Increasing water resistance of concrete	Air entraining agent, MHK, crystal growth increasing powder

Important aspects at application of additives are:

- Air bubble formers reduce concrete strength.
- Thixotropic additives cause compacting to be non-optimal, therefore the concrete cover should be increased.
- Accelerators increase the chance of initiation of corrosion and should therefore only be used in concrete without reinforcement.
- Due to the use of a decelerator, water will not be used for hydration during the delay-time. Bleeding will occur and continue for a longer time than usual; a longer concrete curing time should be taken into account to decrease bleeding

Ad 5. Fillers

Fine materials are used to fill the empty pores in concrete, this fine material has smaller diameters than sand. Larger parts of these fillers can occupy larger voids, while very small parts will fill the space between the former grains. The more voids are filled, the more dense the concrete will become and the porosity and permeability will decrease. The concrete strength will increase.

As fillers can be used:

- Fly ash
- Tras (crunched, ground tuff)
- Stone wheat/grain
- Silica fume (very small, for very high strength concrete)

2.2 Types of concrete

Concrete types can be classified into groups regarding their main characteristics as a result of the concrete composition.

1. Strength:
   - Low Strength Mortar (LSM) ($f'_b < 25$ N/mm^2)
   - Normal Strength Concrete
   - High Strength Concrete ($f'_b > 55$ N/mm^2)
   - Very High Strength Concrete ($f'_b > 200$ N/mm^2)
   - Fibre reinforced concrete has a slightly increased tension strength and strongly increased toughness

2. Density:
   - Light concrete (8 – 20 kN/m^3)
   - Normal concrete (20 – 26 kN/m^3)
   - Heavy concrete (26 – 35 kN/m^3)

3. Processing characteristics:
   - Normal concrete (with our without additives)
   - Very fluid or self compacting concrete
   - Self levelling concrete

4. Application:
   - Colloidal concrete with a high resistance against scour (wash out)
   - Coloured concrete
   - Underwater concrete
   - Cellular concrete
   - Shotcrete
   - Fibre reinforced concrete
   - Fluid-resistant or watertight concrete
   - Construction/ work floor concrete

Characteristics of High Strength Concrete composition compared to normal concrete are:
- High cement content
- A low water/cement factor (which increases strength)
- Quite a large amount of fillers such as fly ash or silica fume is used (which will decrease the percentage of empty pores/spaces in the hardened concrete
- A high dose of additives such as a decelerator or superplasticizer
- Broken, hard aggregates are used

Concrete types can be classified into groups regarding their main characteristics such as strength, density, processing characteristics and application as a result of the concrete composition.

The terminal is a large marine structure, heavily loaded. The structure has to bear the loads and remain afloat. The operational phase of the structure therefore requires an optimum in high strength, low self weight and off course costs. The volume of concrete used per pontoon is large and the costs of high strength concrete per m^3 are high. However, high strength concrete structural members may often have a smaller thickness. Therefore these types of concrete will be most interesting for this thesis. The construction phase will generate requirements regarding the processing of the concrete. These requirements will be turned into measures or additives, depending on the type of concrete needed for the operational phase. The elements will be (at least for the main part) constructed in a dry building dock; therefore underwater concrete is not necessary. From the different applications, only fibre concrete or colloidal concrete may present an alternative.

Characteristics of High Strength Concrete composition compared to normal concrete are:
- High cement content
- A low water/cement factor (which increases strength)
- Quite a large amount of fillers such as fly ash or silica fume is used (which will decrease the percentage of empty pores/spaces in the hardened concrete
- A high dose of additives such as a decelerator or superplasticizer
- Broken, hard aggregates are used

Main properties of High Strength Concrete are:
- High compression strength but only slightly increased tensile strength.
- Large durability.
- Liquid, thixotropic behaviour of the cement mortar; strong cohesion and viscosity or ”stickiness” of the mortar.
- Easy to compact.
- A low water/cement ratio and high part of fine materials (fillers) cause the concrete in the plastic phase to be sensitive to dehydration. Therefore a thorough and long post-treatment is necessary and the concrete surface should be covered to prevent drying out. Curing compound will help, but it is not sufficient.

- A low water/cement ratio as in High Strength Concrete leads to autogeneous shrinkage during the early hardening of the concrete. This may cause serious cracking. Autogeneous shrinkage may be reduced by applying porous aggregates, filled with water.
- High Strength Concrete should especially be checked on possible Alkali Silica Reactions when there is only a small (and therefore insufficient) part of blast furnace cement in the composition.
- High strength concrete has a fast hydration with a higher temperature development in the hardening concrete structure.
- High Strength Concrete has a large part of fine materials, which causes a larger density γ. During a fire, this larger density may cause parts of the concrete cover to crackle off from the reinforcement bars, which increase in volume due to the heat.
- Uncertainty regarding brittle behaviour at certain failure mechanisms.

2.3 Durability and governing failure mechanisms

Damage to a structure is done by the following actions
[Ref. Durability of engineering structures by Jan Bijen] :
1. Mechanical actions, e.g. impact, overloading, movements or a blast.
2. Chemical and biological actions from the environment.
3. Physical actions such as a frost-thaw action, thermal cracking, moisture movement, salt crystallisation and erosion.

Failure mechanisms can be divided in failure mechanisms of the concrete material, of the reinforcement or prestressing or of the connections between the pontoons.
These are the dominant failure mechanisms of concrete structures:
1. Degradation of cement stone by acids.
   Cement stone is mainly composed of lime and silica. As silica forms a very weak acid in water and lime is a strong base, a weak acidic reaction can concert the cement paste into calcium carbonate and silica-gel. The deterioration rate is determined by the aggressiveness of the water, soil or air (pH). Sulphates are mainly found in domestic waste water. Concrete is generally not resistant to an acid environment with a pH < 4. *[Ref. Durability of engineering structures by Jan Bijen]*
   Mineral acids are strongest and they can be expected in the water near to the terminal, generated by industry. However, the concentration will probably not be strong and this degradation mechanism will not be governing.
2. Degradation of cement stone by salts.
3. Degradation by expansive salts.
4. Degradation by expansive alkali-silica reactions.
5. Degradation of the reinforcement.

The last mechanism can have multiple causes:
- Passivation of the alkaline concrete environment.
- Chloride ingress causing pit corrosion.
- Carbonation causing uniform corrosion.
- Corrosion due to stray currents.
- Stress corrosion (in case of high tension stresses, such as in prestressing steel).

Every concrete component has its (chemical and physical) properties and during the hardening process, all components will react together in stead of only the cement and water reacting. Requirements from the structure on building material may be based on the operational phase or the construction phase and they will lead to design choices. Together with a maintenance program, these design choices lead to a well thought-over combination of concrete components.

Durability of marine concrete structures

Marine structures mainly come in contact with salt, water and air. This combination can cause multiple forms of degradation. Most deterioration mechanisms were found not to determine the length of the service life of the structure. According to *[CUR-report 215: Duurzaamheid mariene betonconstructies]*

the most potential deterioration mechanism in concrete marine structures is chloride ingress, as was concluded from test results from several Dutch marine structures.
Expectable deterioration mechanisms are carbonation, leaching, alkali-silica reactions, freeze-thaw action, erosion or salt crystallisation. These seemed virtually absent from the structures that have been researched.
The most severe deterioration of concrete is expected around the waterline and above it, in the splash zone, because the structure will be changing between wet and dry at periods of time. Carbonation is strongly reduced when concrete is wet for long periods of time; from the waterline downwards. Differences in exposure will cause a large amount of scatter in the chloride profiles of different areas on one structure.

Most deterioration mechanisms and in this case especially carbonation and chloride ingress and the following corrosion initiation can be reduced due to the choice of materials. The following aspects should be kept in mind when considering the choice of material:

- An Alkali Silica Reaction (ASR) can be suppressed for the main part by using little or no reactive aggregates.
- The application of blast furnace cement (CEM III/B) causes very dense concrete that is quite resistant against chlorides and it suppresses alkali silica reactions. However, the critical threshold of chloride content for initiation of corrosion of the reinforcement in blast furnace cement is as yet unknown in practice. [*Ref. CUR-report 215]*
- A low water/cement ratio prevents freeze-thaw damage.
- High Strength Concrete develops into a dense mature concrete with high strength. Dense concrete is less permeable and chloride ingress and carbonation may be reduced.
- The carbonation depth should be reduced to less than the concrete cover on the reinforcement. Therefore a large concrete cover in marine structures is beneficial to the length of service life.
- The concrete cover should not crackle from the reinforcement in case of heat (fire) or collision.
- As the influence of early contact with chloride at concrete is not known *[Ref. CUR-report 215],* this contact may be useful to avoid, especially when concrete elements are prefabricated in a building dock anyway.

2.4 Alternatives for concrete composition

Requirements of the building material follow from the environmental and structural conditions of the Western Scheldt Container Terminal:

- The concrete composition should produce a dense and strong structural concrete.
- The concrete should have a minimum permeability because of possible chloride ingress, but it does not have to be watertight/water resistant.
- The concrete composition should ensure that the concrete, which is casted in situ, is well processible.
- The hardened concrete should have a small sensitivity to chloride ingress.
- The hardened concrete should not experience Alkali Silica Reactions.
- The hardened concrete shout not be brittle at collision.
- The hardened concrete shout not be brittle at fire load (less important in the open air).
- Prestressed concrete needs a better moisture control because of the sensitivity to oxidation of prestressing cables (bars) and anchorage mechanisms (Very small cracks are allowed in the concrete hull).

The alternatives for the concrete composition are based on strength, durability and self weight. As blast furnace cement produces a dense concrete with small sensitivity to chlorides, the alternatives are considered with sufficient part blast furnace cement in the concrete composition, when this is possible for the specified material. This is valid for alternatives I, III and V.

1. Normal Concrete of B35 or B45 (NSC)
2. Light Concrete (LC)
3. High Strength Concrete B65 (HSC), blast furnace cement
4. High Strength Concrete B100 (HSC), Portland cement
5. Fibre-reinforced concrete

6. Very High Strength Concrete B200 (VHSC)

Choice of alternatives by choice matrix:

	Alternatives					
Aspects	NSC B35	LC	HSC B65	HSC B100	Fibre	VHSC
Strength	-	-	+	++	++	++
Self weight	0	++	0	0	-	-
Permeability & Density	-	--	+	++	0	++
Processing	+	0	+	+	0	-
Hardening& temperature	+	0	0	-	0	--
Water/cement ratio	0	-	+	+	+	+
Cracking	--	-	-	-	++	--
Chlorides	-	-	+	++	-	++
Brittle	++	+	+	-	+	--
Investment Costs	++	-	+	-	--	--
Maintenance	-	-	0	+	0	++
Total	0	--	++	+	+	-

++ very good
+ good
0 neutral
- bad
-- very bad

Normal concrete can have a maximum compression strength of 55 N/mm^2, but B35 or B45 are more common. It is not as expensive as high strength concrete and less brittle, and it can be made very well processible. It does however have a lower density and larger permeability than high strength concrete. The strength and density of normal concrete is thought to be too small for the structure at hand.

Light concrete can limit the self weight of the pontoon, providing smaller achievable dimensions.
The amount of concrete determines largely the costs of a pontoon. However, the concrete strength in this structure is of higher importance than the self weight, because of the large loads on the structure.
The wall thickness is limited by the application of prestressing.
The fibre concrete is thought to be an option to combine with one of the other alternatives.

Very high strength concrete is thought to be too expensive considering the large amount of concrete needed. It also requires extra measures during construction and hardening.
The best option seems to be high strength concrete with all the advantages of high strength, density and durability and reasonable costs. The choice between B65 and B100 is based on the resistance of the concrete against degradation mechanisms and on costs.
There are two questions regarding costs:

1. Would the extra costs of B100 be justified by the extra durability compared to B65?
2. Would it be financially feasible to add steel or plastic fibres to the concrete composition and will that influence the durability of the concrete?

Fibre reinforced concrete is assumed to be too expensive in large amounts.
B100 will have a denser pore system than B65, which is preferable regarding chloride ingress. At the same time, the extra structural strength can be used to optimise the pontoon design.

However, as blast furnace cement can not be applied in a B100 concrete composition, Portland cement will have to be used. Blast furnace cement has a higher resistance against chloride ingress, which is the most important degradation mechanism. Even with Portland cement and additives, it will prove to be difficult to reach a concrete quality of B100, when the concrete is casted in situ outdoors in stead of in a factory with climate control etc.

B65 with CEM III/B, rough sand, broken stone and small fillers (fly ash or silica fume) is chosen as the best concrete composition for the pontoons. A low water cement ratio is necessary. Additives such as a plasticizer can be used to ensure a processible mortar on site.

2.5 Protection and maintenance

The following is a summary of protection and repair measures of concrete structures.
Types of protection are:

- Additives in the concrete composition ensuring a large density, small permeability and small sensitivity to deterioration mechanisms of the concrete
- A coating on the structure surface protecting the concrete
- Sacrificial anodes preventing corrosion of the reinforcement
- Plastic or steel cover on the concrete around the water line

Repair principles and methods are:

- Protection against ingress of adverse agents (reduce or prevent)
- Moisture control within concrete
- Concrete restoration (to the originally specified shape and function)
- Structural strengthening; increase or restore the load-bearing capacity of an element
- Increase physical resistance to physical or mechanical attack
- Increase the resistance of the concrete surface to chemicals

[Ref. Durability of engineering structures by Jan Bijen]

3 Construction alternatives regarding the pontoons

Several types of formworks are listed, which may provide a solution for all walls of the pontoon.
Possible formworks at site:

1. The inner and outer walls can be constructed in a distinct framework for each wall. The necessary struts will require valuable work space.
2. Several walls can be placed as prefabricated slabs; struts are necessary until the joints are casted in concrete, connecting the slab to the adjoining walls.
3. The inner and outer walls can be constructed as compartments, using a specially designed formwork with dimensions equal to those of the compartments.
4. A tunnel framework (including the deck) can be applied to construct several corridors. However it is not likely to be used, as the transversal inner walls would still have to be constructed afterwards. The transversal inner walls can either be casted in-situ or be placed as prefabricated slabs, which still have to be connected to the other walls on site. Consequently, there are many undesirable construction joints.
5. A sliding shuttering could be used to construct the entire pontoon from the bottom slab upwards. It is not likely to be applied, because the pontoon height is too small.

The best alternative is considered to be the compartment formwork, which can be hoisted to every next compartment.

The deck structure can be cast in-situ, though this would mean a large amount of “lost framework”, struts and a lot of labour etc. It can also be partly prefabricated and partly constructed at site:

1. Prefabricated T or L beams can be combined with a casted concrete top layer. The beams are expensive and the amount of framework for the deck layer is large; it will increase building time. However, the construction height of the deck can be smaller than that of a rectangular slab, because the moment of inertia I (and the bending stiffness EI) and the moment of resistance W of the cross section is larger. The deck structure has a smaller dead load than a solid top slab.
2. Prefabricated concrete box girders are assumed to be a rather expensive solution, considering the surface area of the top slab of the pontoon, but a concrete top layer is easy to construct; there is no lost formwork.
3. Prefabricated compartment deck plates can be hoisted upon the walls. These will be rigidly connected to form one deck by casting the joints between the plates (upon the walls). There is much space for ducts in the solid slabs for prestressing steel.
4. Half-prefabricated composite slabs can be installed, on which a concrete deck layer is casted. As the prefab deck plates provide the formwork for the deck layer, this is a fast construction method. The prefabricated slabs that have to be transported and hoisted into place have a smaller dead load. The top slab can easily be finished while the pontoon is already floating. Prestressing steel can easily be installed.

The prefabricated slabs, whether totally or only half prefabricated seem to offer the best solution.

4 Internal forces in cross sections

A longitudinal and a transversal cross section and several slabs (top slab, outer wall, inner wall)have been calculated using a Finite Element Methods Program; ESA Prima Win. This appendix includes the results of those calculations.

Appendix Part 4 - B:

Structural design connections

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1 Compliant or flexible connections

Hinged or flexible connections have one or several unrestricted degrees of freedom. Mostly the rotations are allowed, while translations are restricted; thus ensuring a continuous deck.
The relative motions between two pontoons are assumed to exceed the limits regarding crane track performance. Therefore, compliant connectors will only be applied between elements large enough for one entire ship berth. The connection forces decrease when the connector gives freedom to more motions.. Mainly, pitch will be allowed, because to restrain pitch means exerting large connection forces on the elements.

Several practical hinge principles are used to develop a hinged connector that will meet the requirements of the floating terminal.

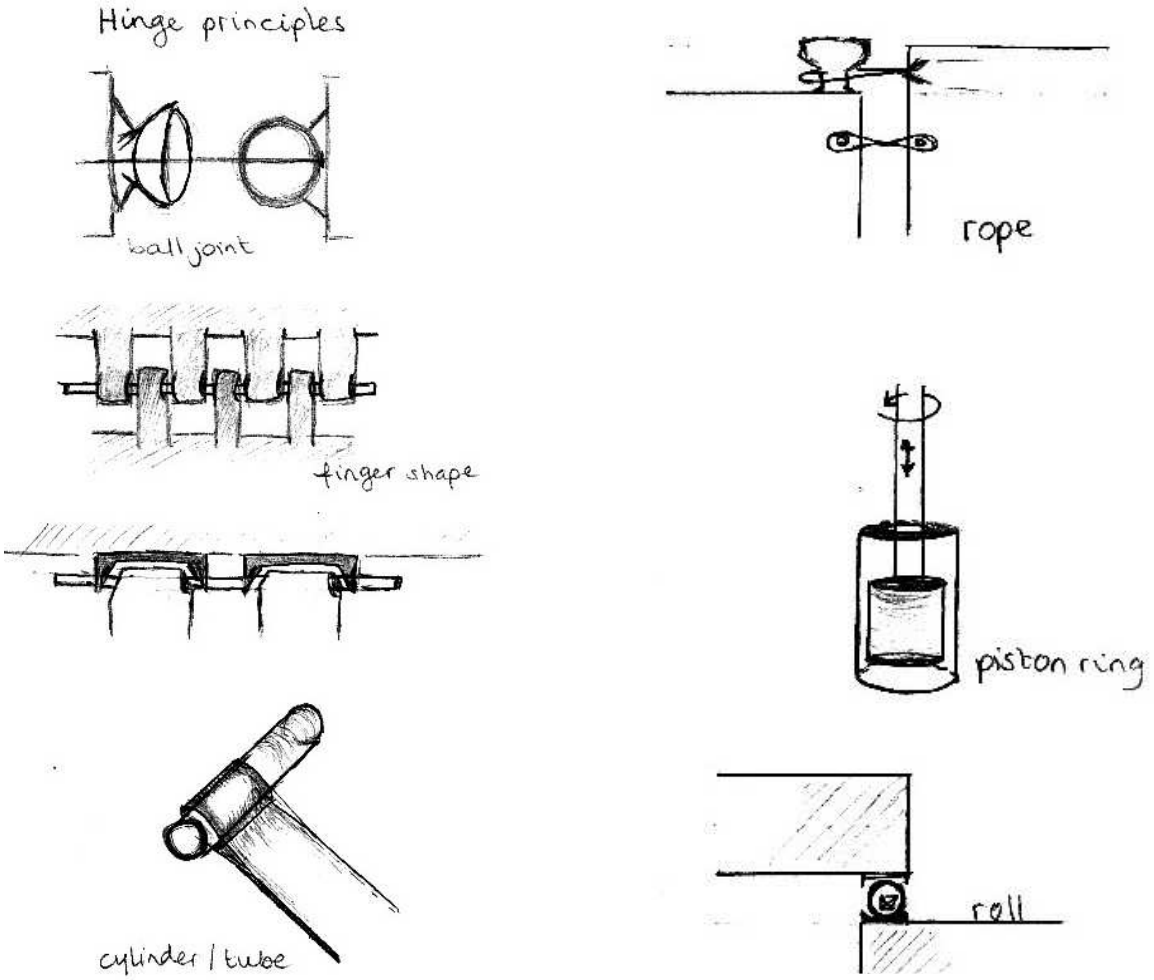


Fig. Practical small hinge principles

Different applicable principles in hinge connectors are:
- Allow relative rotations, but fix all relative translations
- Allow only pitch (by applying several of the above hinge connectors, especially connecting face to top)
- Allow relative rotation and horizontal bending of the entire structure (damping the dynamic responses rather than designing with a natural frequency different from the occurring wave frequencies)

A Dynamic positioning system for every separate pontoon would be too expensive for a semi-permanent structure. Besides, inner transportation between elements would hardly be possible. As such, the concept is not applicable (only as auxiliary concept) to a floating terminal.

There are 6 alternatives for flexible connections:
1. A compliant ball-joint (such as the Monaco semi-floating breakwater and quay wall)
2. A horizontal hinge with studs/cams
3. A horizontal piano hinge and centre joint, such as the compliant MOB McDermott connector
4. A hinged bridge between decks with multiple roller bearings
5. An vertical piano hinge system (based on the McDermott connector)
6. Piston ring/damper model in surge and sway direction

The figure on the next page shows the design alternatives.

Ad 1. A large steel ball and socket joint have to be fabricated at a specialised manufactory, such as NFM Technologies, which produced the ball and socket joint for the Monaco breakwater and the Maasvlakte storm surge barrier. These two examples concerned individual connections. The size and production of a large number of these connections will make them very expensive, though a small number of connections per pontoon are necessary. The joint would be attached to the hull of the pontoon and anchored in the concrete. This connection would allow the most relative motions, but all connection forces will be concentrated at the connector and thus difficult to introduce in the pontoon.

Ad 2. This design alternative is based on the original McDermott MOB rigid hinge connector with a horizontal piano hinge. Originally, only one piano hinge is designed, at the top side of the face plate (waterline). A second piano hinge at the bottom side of the faceplate would stiffen the connection and will therefore not be applied. The connection allows mainly pitch, because pitch generates the largest connection forces.

Ad 3. This design alternative is based on the original McDermott MOB rigid hinge connector with a horizontal piano hinge and a centre ball joint. The docking centre of the ball joint in the middle of the faceplate (or with a certain distance centre-to-centre) restricts roll, sway and pitch motions only little. The connection will allow mostly pitch, when the faceplate is mainly attached to the top of the other faceplate. The location of the joint and the number of connectors can influence the stiffness and allowed relative motions of the connection. According to the more recent MOB hinged connector, the centre ball joint may be replaced by a stud and docking centre, with a rubber cone installed on the stud. The horizontal hinges can be divided in parts; two or more outboard compliant hinges restrict the relative motions of surge, roll, heave and yaw. The docking centre of the ball joint in the middle of the faceplate (or with a certain distance centre-to-centre) restricts roll, sway and pitch motions only little. A collapsible rubber cone can be installed on the ball-joint. In a normal situation the cone is not collapsed and has a large stiffness. When the cone collapses due to large (surge) forces during a storm, the stiffness is reduced. Therefore, the motions will increase while the forces on the hull of the pontoon are kept beneath a design maximum.

Ad 4. A truss bridge above water level would connect the two deck topsides of pontoons. The bridge can be constructed as a rigid or a very compliant structure. The bridge will be supported by multiple roller bearings. This alternative does affect the continuous deck of the terminal.

Ad 5. This design alternative is an alternative on number 2. It comprises one or multiple piano hinges. The location of the joint and the number of connectors can influence the stiffness and allowable relative motions of the connection. A vertical connection is easier to couple, when the pontoons are subject to heave due to waves. Vertical hinges will however restrict the pitch motion, which will cause large connection forces on the pontoon.

Ad 6. The pontoons can be connected with large size piston rings. These restrict 4 of the 6 degrees of motion. The connection forces will have to be carried by the slender piston structure, which may cause problems.
This alternative can also be constructed with a continuous deck on top of the pontoons, supported by piston rings (as springs), which can control the horizontal position of the deck, even though the pontoons are moving in different response motions from each other. In that case, the pontoons can also be connected by cables (like rope), as the problem of fairly independent motions per pontoon is accounted for.

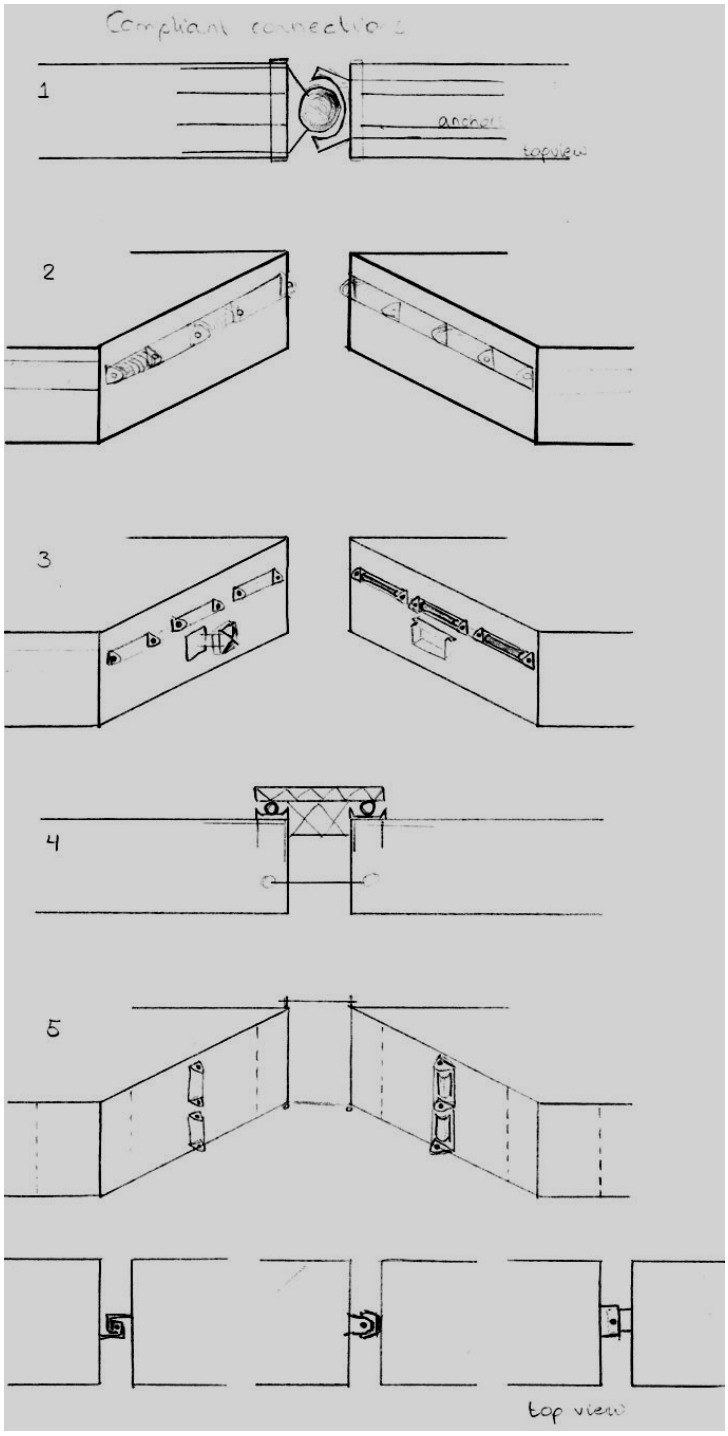
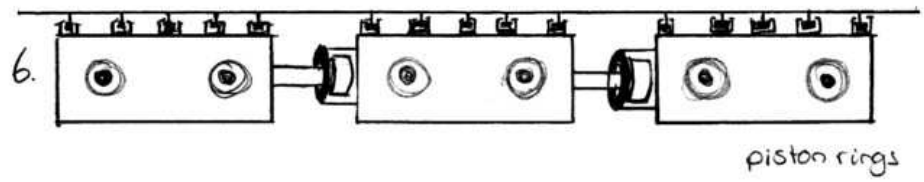


Fig. Design alternatives hinged connections.



Choice of alternative

	Alternatives					
Aspect	***I***	***II***	***III***	***IV***	***V***	***VI***
Nr of d.o.f. unrestricted or with margin	3	1	1	3	1	2
Restricted d.o.f. causing largest load	heave	yaw	yaw, roll	yaw	pitch	pitch
size of connection force	++	0	+	+	--	--
Distribution of connection forces	--	++	-	-	+	-
Peak stresses	-	+	-	-	0	--
Complexity connector	-	+	-	+	+	--
constructability	o	+	+	++	+	-
Joining technique	++	--	+	0	++	+
Easy decoupling	++	0	+	0	++	+
Motions allowed	++	+	++	+	-	-
Durability	+	-	-	-	-	-
Maintenance and repair	0	-	+	+	-	--
Fatigue	+	-	-	0	-	0
Material use	-	-	0	--	+	--
Location of connectors	-	++	+	+	0	-
Number of connectors	+	-	-	0	-	0
costs	--	+	0	--	+	-
Total	+	++	0	-	0	--

++ very good
+ good
0 neutral
- bad
-- very bad

Alternatives I, II and III seem feasible connections. The generally largest connection force due to pitch is eliminated. The three alternatives vary in costs an in feasibility of the coupling method and they generate a different load distribution on the hull structure of the element.

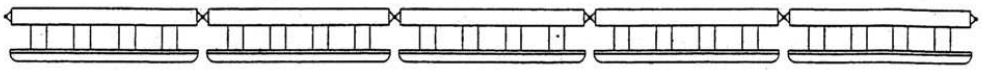


Figure 1: McDermott MOB Concept

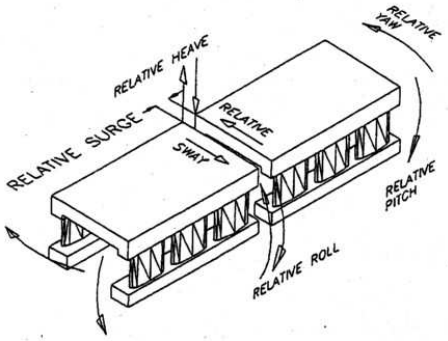


Figure 4. MOB Joint Degrees of Freedom

Table 2. MOB Joint Requirements

MOB Joint DOF	Requirement
Relative Surge	Soft
Relative Heave	Stiff
Relative Sway	Stiff
Relative Pitch	Soft
Relative Yaw	Stiff
Relative Roll	Stiff

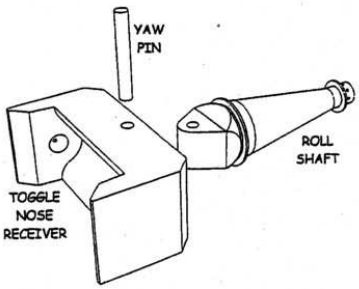


Figure 5: Centerline Ball Joint Detail

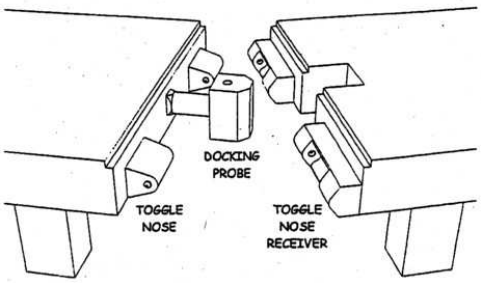


Figure 2: Simple Hinge Concept

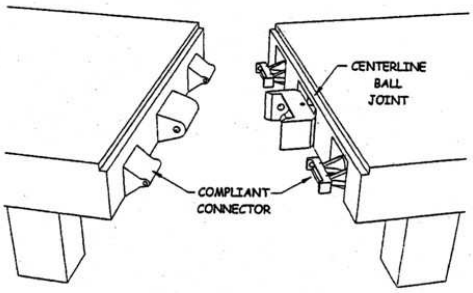


Figure 4: Compliant Connector Arrangement

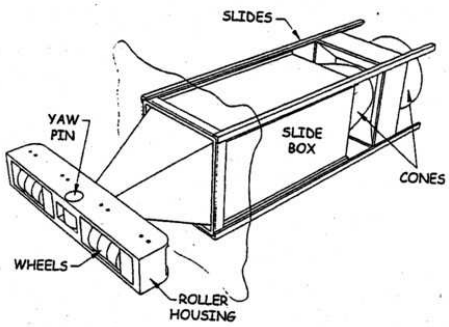


Figure 9: Roller Connector

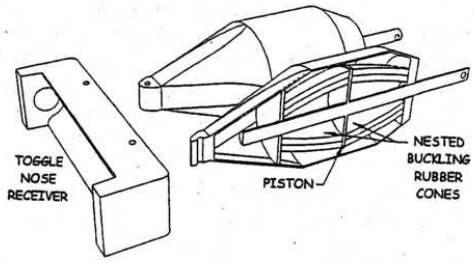
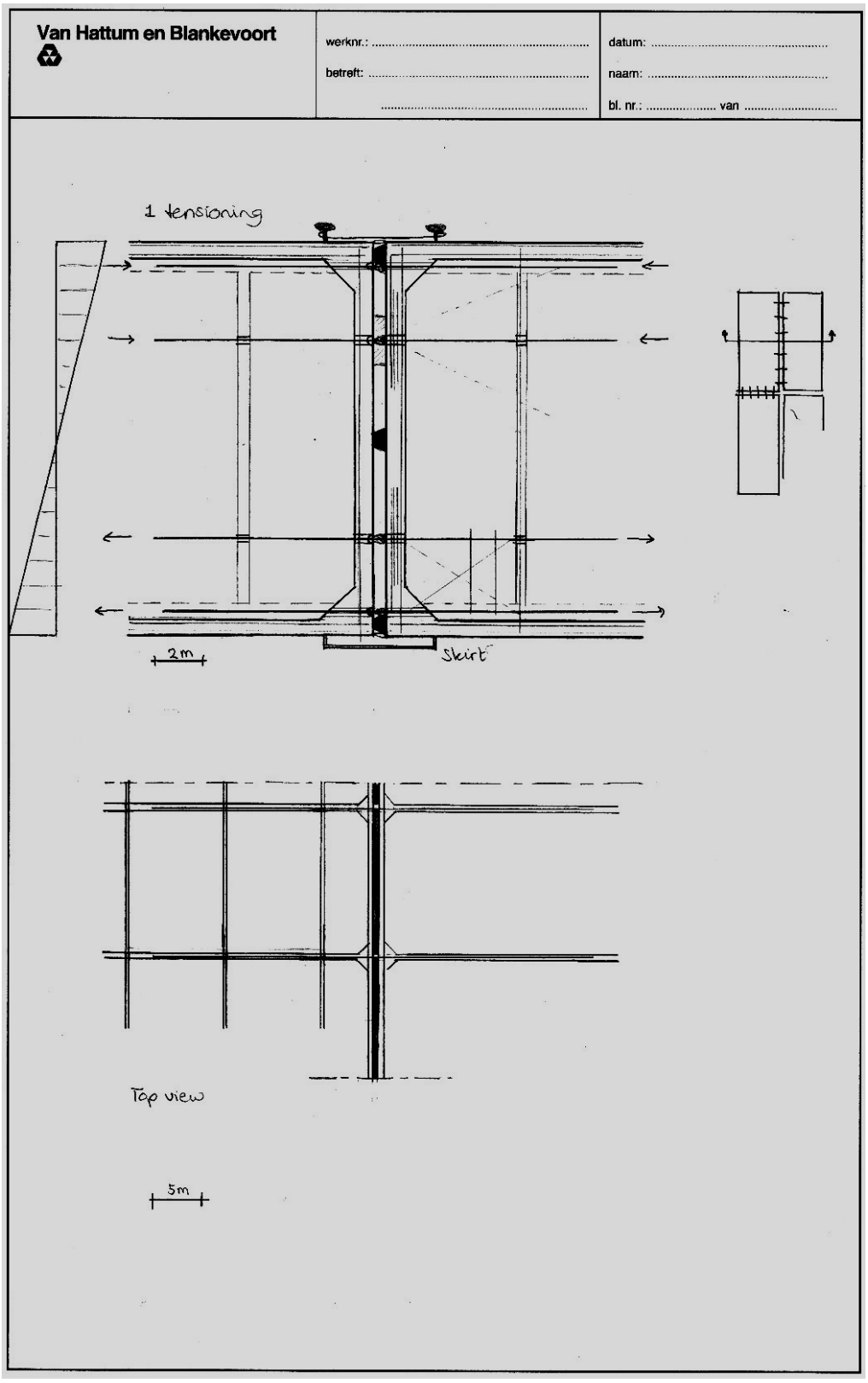


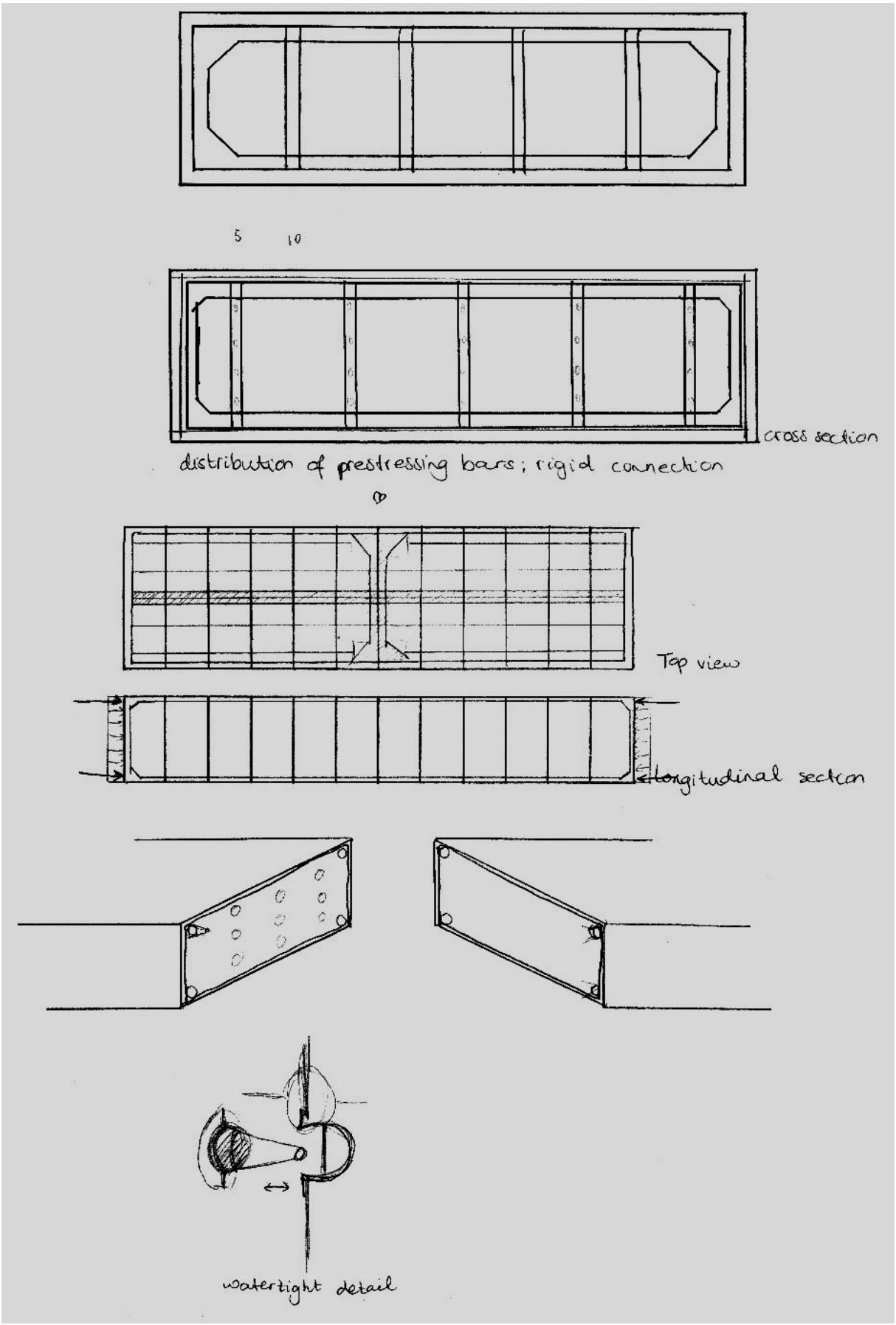
Figure 6: Compliant Connector Detail

Figure 1: MOB connectors

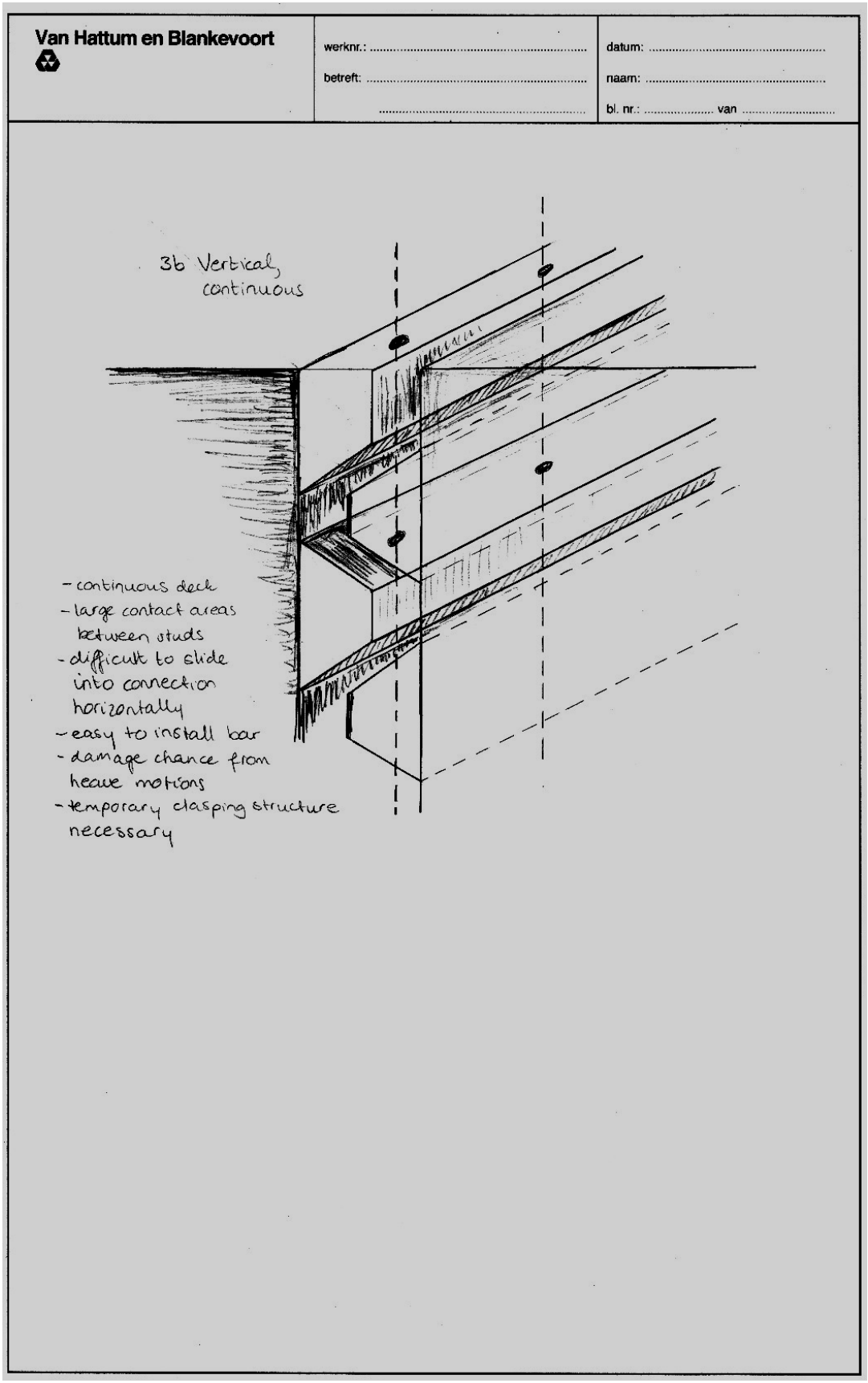
2 Elaboration of 2 connector alternatives

2.1 Tensioning the pontoons together with prestressing bars




5
10
cross section
distribution of prestressing bars; rigid connection
Top view
longitudinal section
waterstight detail

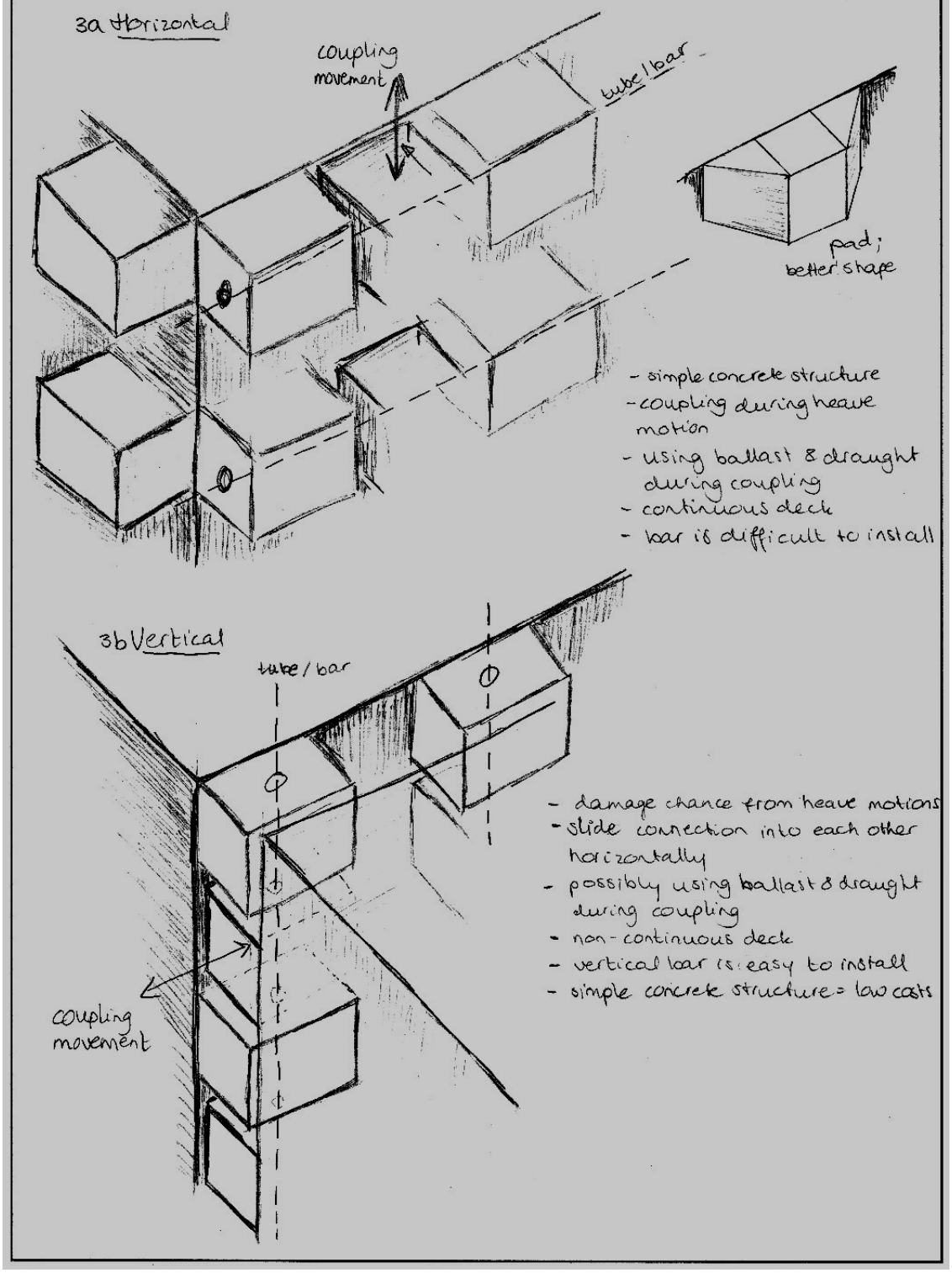
2.2 Concrete stud systems



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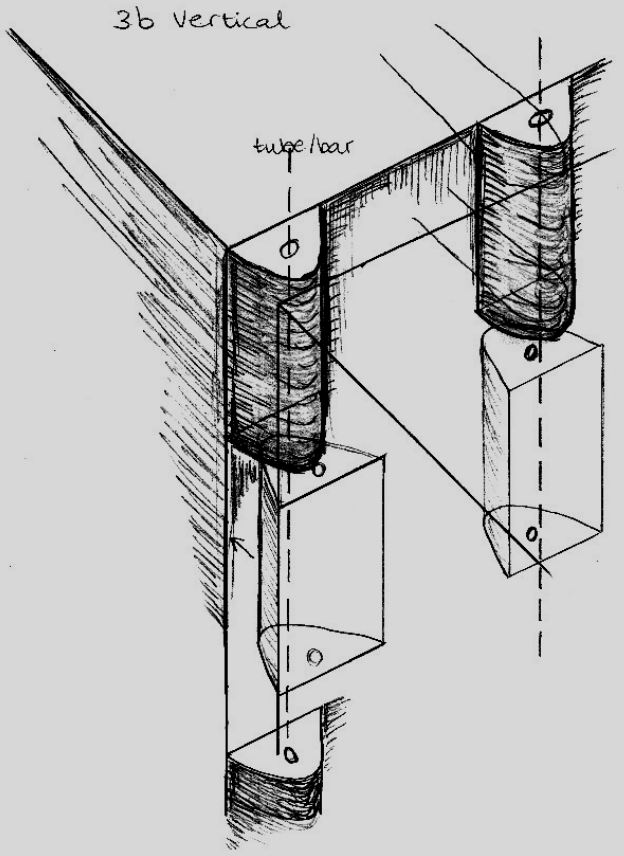
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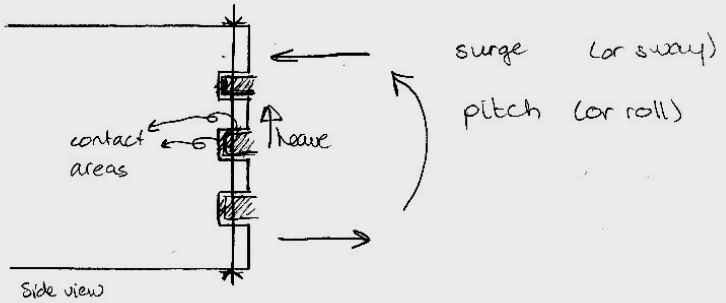
- steel or concrete parts
- rounded structure: smaller damage chance (expensive)
- bar is easily installed
- non-continuous deck
- use ballast & draught to obtain right vertical position; then slide horizontally into place
- prestressing cables with minimum radius! → bent through concrete studs

2.3 Introduction of connection forces

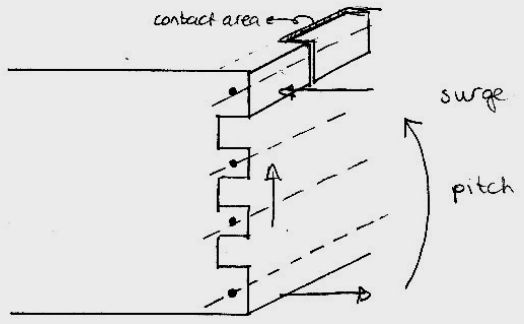
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- vertical force causes pressure in concrete studs : contact area
- vertical force cause normal force in bar ; preferable compared to shear
- bending moment is introduced into concrete as compression and tension force (via contact area and via bending/shear in bar)



- vertical force causes unfavourable shear in bar ; NO contact area of studs
- bending moment causes bending in bar
- bending moment is introduced as normal forces via shear in bar and via contact area

3 Design of rigid connector

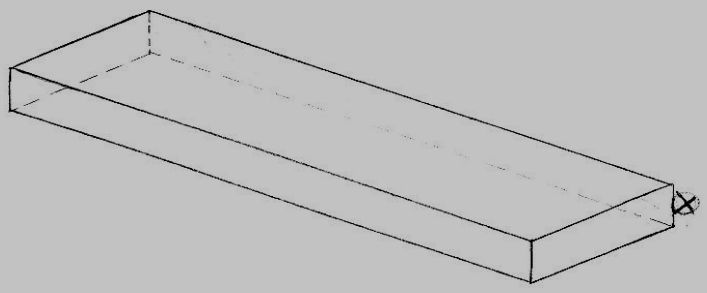
3.1 Connection forces

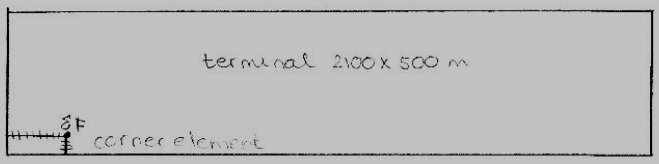
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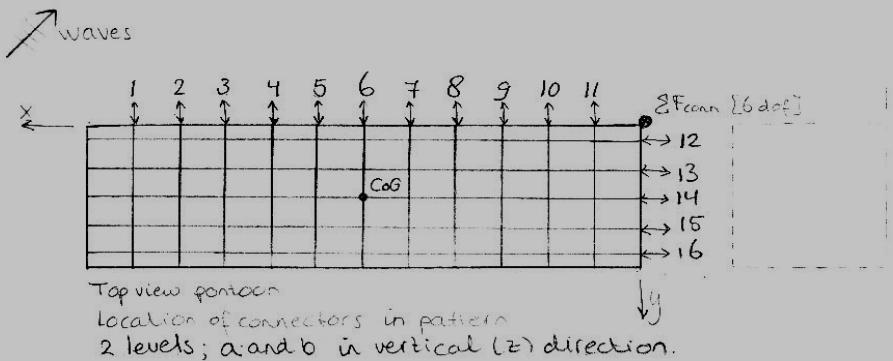
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Top view pontoon

location of connectors in pattern

2 levels; a and b in vertical (z) direction.

coordinates Distance Δ CoG - ΣFc = 93.41 m

corner ΣF conn = (0,0) CoG = (90,25)

1 (165,0)	9 (15,0)
2 (150,0)	10 (30,0)
3 (135,0)	11 (15,0)
4 (120,0)	12 (0,5)
5 (105,0)	13 (0,15)
6 (90,0)	14 (0,25)
7 (75,0)	15 (0,35)
8 (60,0)	16 (0,45)

Total pattern of 16 strands

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Connections are divided in :
- connections at long side (large pontoon face ⊥ y-axis)
  These restrict mainly sway, roll
- connections at short side (small pontoon face ⊥ x-axis)
  Heave and yaw are restricted by all connectors (evenly distributed)

The connector force distribution is dependant on the location of the particular connector regarding the distance to Centre of Gravity.

I Principle of moving a load:



all connection loads have been concentrated by Pelfiks in resultant ΣFconn
1 pontoon → 20 walls = 20 connections (symmetry)
distance Fconn ↔ CoG = $\sqrt{90^2 + 25^2}$ = 93,4 m.

$\frac{F}{20}$ = F per wall connector incl. distance a.

In general:
$F_{in\ CoG}: \Sigma F_{conn} + \frac{m}{93,4} = \Sigma F_i . x_i$

motion type	ΣFconnect		FCoG
Surge	6962,42	kN	
Sway	5796,13	kN	
Heave	1056,40	kN	
Roll	108685,9	kNm	
Pitch	1057547,33	kNm	
Yaw	432154,21	kNm	

- surge: $F_{CoG}: \Sigma F_{conn} + \frac{m}{25}$
- sway: $F_{CoG}: \Sigma F_{conn} + \frac{m}{25}$
- heave: $F_{CoG} = \Sigma F_{conn} + \frac{m}{93,4}$
- Moments can be displaced/moved without change
  - surge ↔ yaw
  - sway ↔ yaw
  - heave ↔ roll, pitch

The resultant forces are a sommation of all the forces on the parts of the dynamic model
The resultant forces are located in the Centre of Buoyancy which lies on the vertical z-axis through the Centre of Gravity of the pontoon. The forces are displaced/moved to the corner point

The connector forces will be calculated in relation to their distance to the centre of Gravity (CCoG).
Therefore the ΣFconnect have to be moved (back) to the CoG.

II even distribution of translation forces

	ΣFconn	nr of conn	Fxi	
surge	6962,42	2 x 5 = 10	696,3 kN	normal force
sway	5796,13	2 x 11 = 22	240,7 kN	normal force
heave	1056,40	2 x (5+11) = 32	220,51 kN	shear

III The 6 degrees of freedom are considered independently, as this is a combination of 6 maximum forces / motions originated from different location on the vessel

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Distribution of connection forces

motion	$\Sigma F_{connect}$	nr of conn	F_{xi}	Nr of studs	F_{stud}
Surge	6962,42 kN	5	1392,5 kN	2	696,3 kN
Sway	5296,13 kN	11	481,5 kN	2	240,8 kN
Heave	705640 kN	5+11=16	441,0 kN	2	220,5 kN
Roll	180.685,31 kNm	16		2	
Pitch	1057.547,35 kNm	16		2	
Yaw	432.154,21 kNm	16		2	

Studs are not placed on the corners of the pontoon, because the connection forces can not be distributed well into the pontoon at that location

* Roll :

Distances to centre of gravity a

$a_{total} = (11 * 25) + (2 * 20) + (2 * 10) + 0 = 335$ m (long side)

$M_{i;max} = \frac{20}{335} * 180\,685,31 = 10787,2$ kNm

$N_i = \frac{M}{h} = \frac{10787,2}{11} = 980,7$ kN



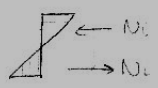
* Pitch : governing bending moment

$a_{total} = (5 * 90) + (2 * 75) + (2 * 60) + (2 * 45) + (2 * 30) + (2 * 15) + 0$

$= 900$ m

$M_{i;max} = \frac{90}{900} * 1057.547,35 = 105.754,7$ kNm

$N_i = \frac{M}{h} = \frac{105.754,7}{11} = 9614,1$ kN



* Yaw: in horizontal plain

$a_{total} = 2 * (75 + 60 + 45 + 30 + 15) + 2 * (20 + 10) + 0 + 0$

$= 510$ m

$M_{i;max} = \frac{75}{510} * 432.154,21$ kNm $= 63552,1$ kNm

$a_i * F_i = a_{total} * M_{yaw}$

$F_i = \frac{63552,1}{75} = 847,4$ kN

$F_{stud} = \frac{847,4}{2} = 423,7$ kN

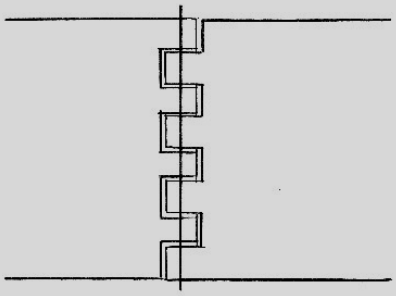
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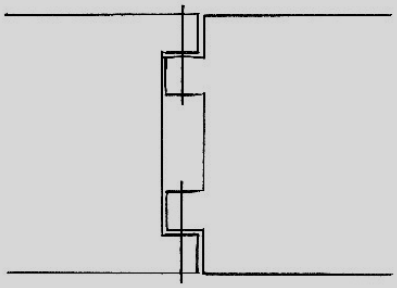
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Stud systems



A. Basic Principle : studs + knitting needle

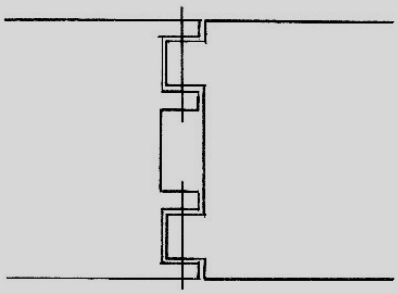
disadvantages :
- small studs where forces are large (outer studs)
- uncertainty of activity / bearing capacity of inner studs



B. Large outer studs

advantages :
- clear force distribution
- large studs at location of large forces

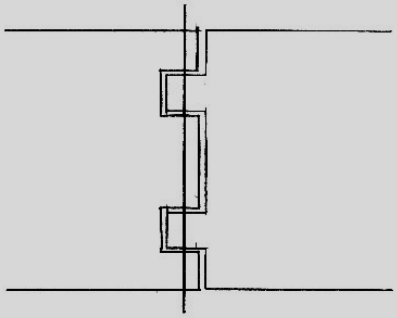
disadvantage :
- 1 (shear) section of steel tube



C. Large outer studs with double section

advantages :
- double (shear) section of tube
- large outer studs

disadvantage :
- less clear force distribution
- smaller section at out fibre (space for prestressing etc.)



D. Large outer studs with 1 inner stud

advantages :
- double (shear) section of tube
- only one tube, 16 m length

disadvantages :
- less clear force distribution
- very large material use

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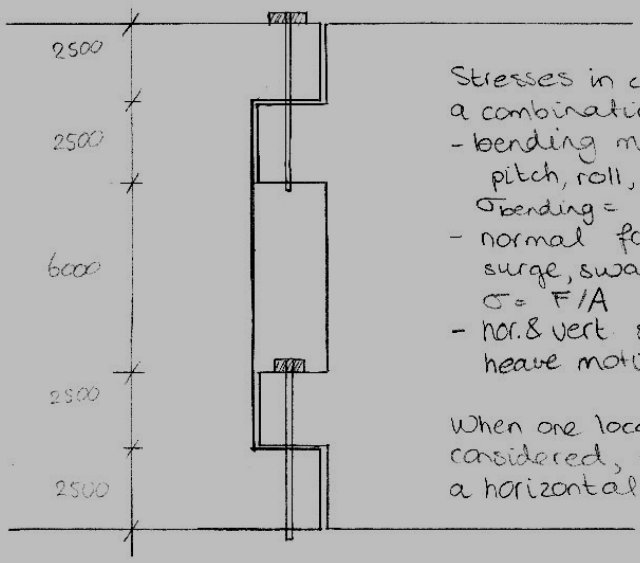
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System of 2 large studs

* more space to distribute and introduce the connection forces
* The studs at the top side and bottom side bear the entire load; no uncertainty regarding the load upon middle studs

Estimated dimensions

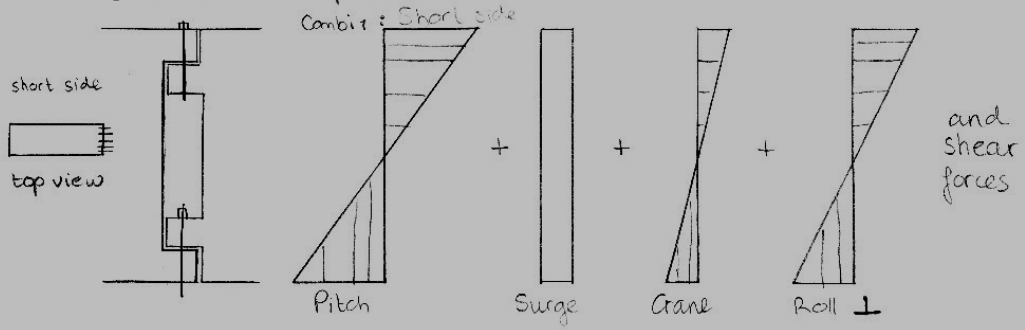
L: 6 x 4 2000 x 1000 = 250 x 150 m (x, y, z of pontoon)

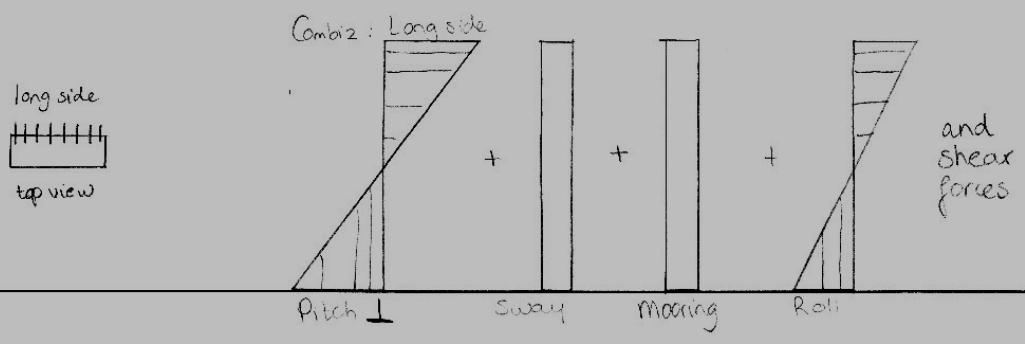


Stresses in connection are a combination of :
- bending moments
  pitch, roll, yaw motions
  $\sigma_{bending} = m/W$
- normal forces
  surge, sway motions
  $\sigma = F/A$
- hor.& vert shear forces
  heave motions

When one local connection is considered, yaw motions cause a horizontal shear force (⊥ drawing)

* Continuous system :

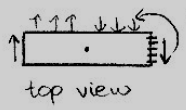




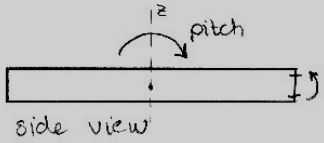
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* Continuous to discrete system

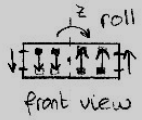
Combi 1 : Short side connection (governing connection)



$M_{yaw} = -\Sigma F.a$



$M_{pitch} = -\Sigma M_{com}$



$M_{roll} = -\Sigma F.a$

* The bending moment from yaw is modelled as a horizontal shear force on the local connector

* The bending moment from pitch is modelled as a bending moment ← compression force / a / → tension force on the local connector

* The bending moment from roll is modelled as a vertical shear force on the local connector
capacity of short side = 60/335 * M_{roll} = 32361,55 kNm

Combi 2 : Long side connection

The same modelling is valid:
* Yaw ⇒ horizontal shear force on local connector
* Pitch ⇒ vertical shear force on local connector
* Roll ⇒ bending moment on local connector

Bending moments are translated into a torque F*a
The arm a between the stud connectors = 16 − (2×2,5) = 11 m

Short side : $N_i\,pitch = \frac{M_i\,pitch}{11} = \frac{105754,7}{11} = 9614,1$ kN

Long side : $N_i\,roll = \frac{M_i\,roll}{11} = \frac{13484}{11} = 1225,8$ kN

Short side : $F_{i\,roll;max} = \frac{M}{40m} = 809$ kN (outer compartments)

F per stud = 404,5 kN

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Combi 1 : Short side

- $M_{pitch;max}$ = 105754,7 kNm ⇒ N_i = 9614,1 kN
- M_{roll} = $\frac{20}{335}$ * 180685,31 = 10787,2 kNm

$N_i = \frac{M}{11}$ = 980,7 kN

- N_{surge} = $\frac{1392,5}{2}$ = 696,3 kN
- M_{crane} = $\frac{1}{10} q l^2$ = $\frac{1}{10} \cdot 800 \cdot 8^2$ = 5120 kNm

$N_i = \frac{M}{11}$ = 465,5 kN



Total N_{stud} = 10775,9 kN

Combi 2: Long side

- M_{pitch} = $\frac{75}{900}$ * 105754,35 = 8813,9 kNm

$N_i = \frac{M}{11}$ = 8011,7 kN

- $M_{roll;max}$ = 13476,6 kNm ⇒ N_i = 1225,8 kN
- N_{sway} = $\frac{481,5}{2}$ = 240,8 kN
- $N_{mooring}$ = $\frac{1}{11}$ * (19 kN/m' * 180 m) = $\frac{3420}{11}$ = 310,9 kN

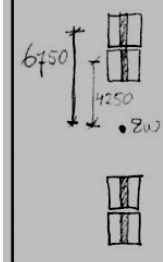
$N_{per\,stud}$ = $\frac{310,9}{2}$ = 155,5 kN

Total N_{stud} = 1622,1 kN

$A_b = b \times h = 3000 \times 2500 = 7,5 \cdot 10^6$ mm²

$W = I_{/y}$ with y = 8000 mm.

$I_{eigen} = \Sigma I_i + A.a^2$



1 stud Ø d tube = 1000 mm

$A_0 = 2 \times (1000 \times 2500) = 2 * 2,5 \cdot 10^6 = 5 \cdot 10^6$ mm² excl hole

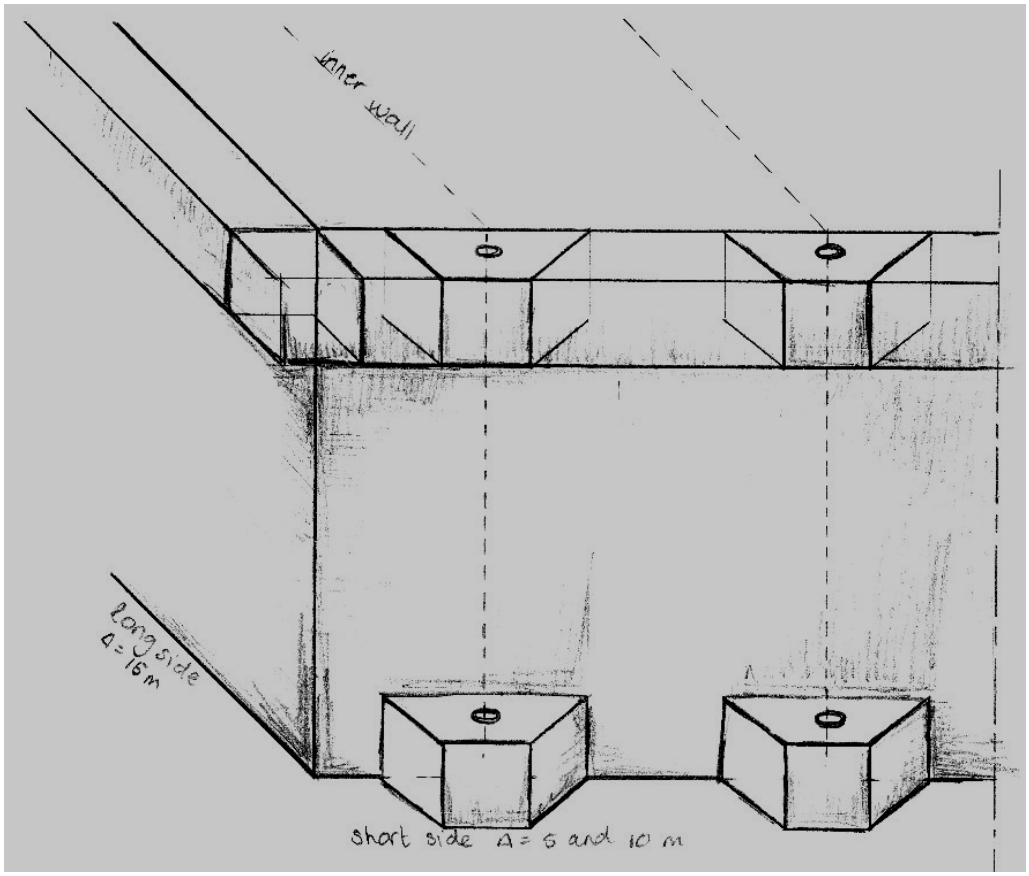
$I_i = \frac{1}{12} * 2000 * 2500^3 = 2,6 \cdot 10^{12}$

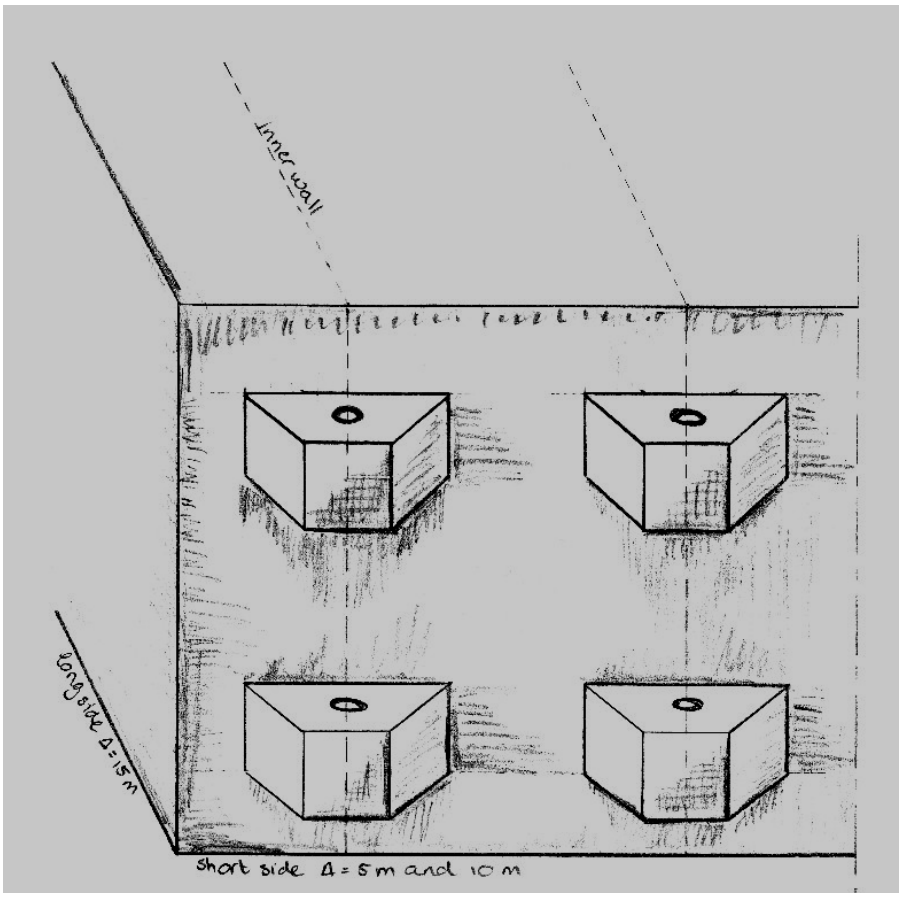
$I_{eigen} = 2 \times (2,6 \cdot 10^{12} + (5 \cdot 10^6 \times 6750^2)) + 2 \times (2,6 \cdot 10^{12} + (5 \cdot 10^6 \times 1250^2))$

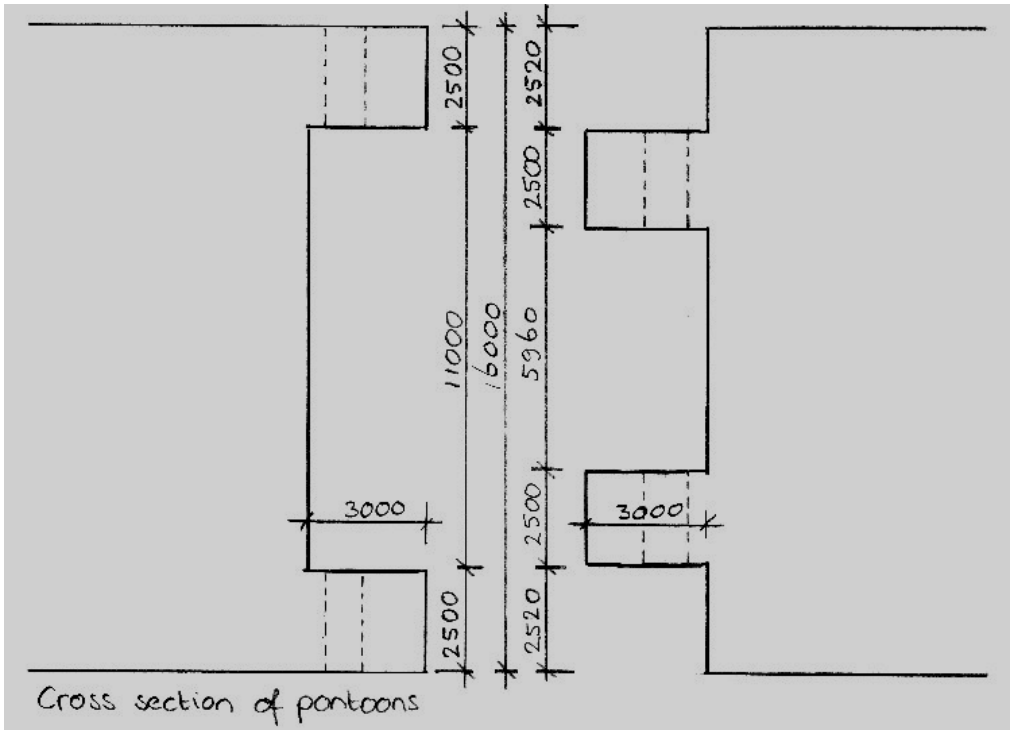
$= 6,47 \cdot 10^{14}$ mm⁴

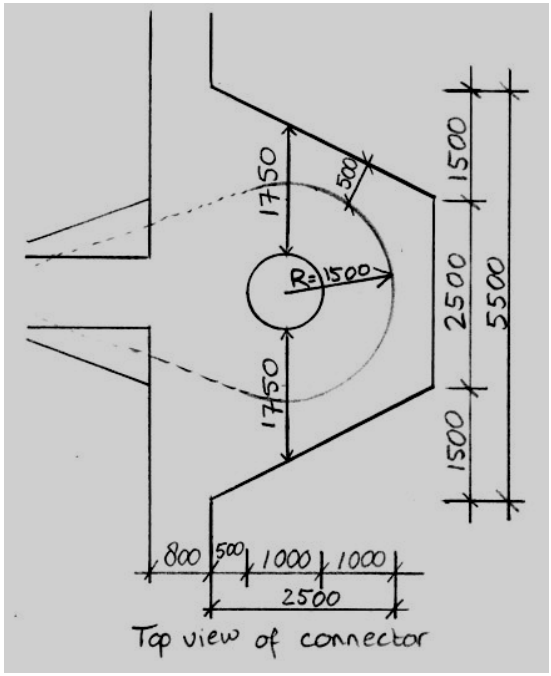
$W_{top} = W_{bottom} = \frac{6,47 \cdot 10^{14}}{8000} = 8,09 \cdot 10^{10}$ mm³

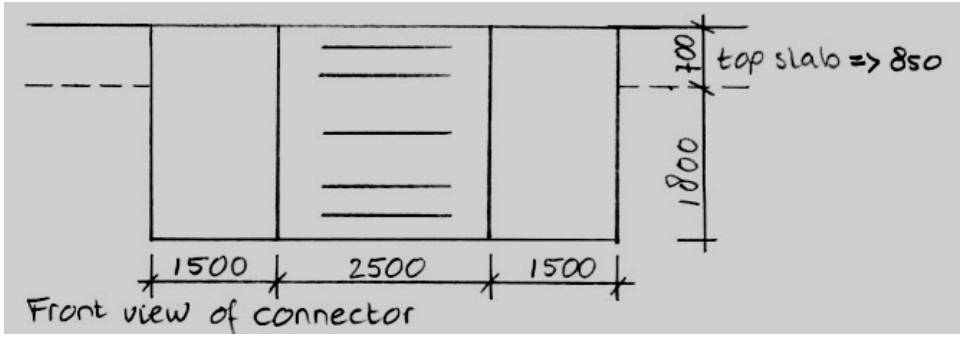
3.2 Design of connector






2500
2520
2500
11000
16000
5960
2500
2520
3000
3000
2500
Cross section of pontoons


1500
1750
500
R=1500
2500
5500
1750
1500
800
500
1000
1000
2500
Top view of connector


top slab => 850
1800
1500
2500
1500
Front view of connector

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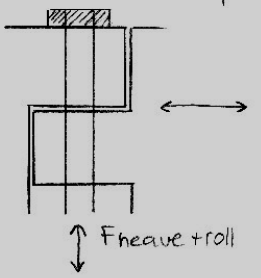
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Discrete system Combi 1: Short side

Side view : Top side stud



Max heave = $\frac{7056,4}{5}$ = 1411,3 kN ⇒ Fstud = 705,6 kN

Fsurge + Fpitch + Fcrane

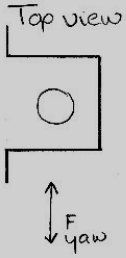
Governing combination = max tension + max shear



Fs+p+c ; tension = 10775,9 kN

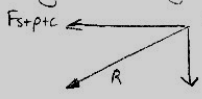
Fh+r = 705,6 + 404,5 = 1110,1 kN shear

Top view : Top side stud



$F_{surge} + F_{pitch} + F_{crane}$

Governing combination =



compression Fs+p+c = 10775,9 kN

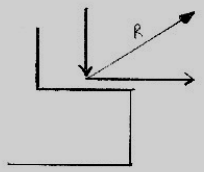
F yaw = 423,7 kN

yaw capacity short side = 450/510 * 432.154,21 = 381 312,5 kNm

Fi;yaw;max = $\frac{M}{90*5}$ = 847,4 kN

Fper stud = 423,7 kN

Side view : Bottom side stud



Governing force combination:
max tension + max shear

Top view situation is equal to the top side stud.

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Design top stud - short side of pontoon

* Normal forces:
Compression or tension due to surge+pitch+crane $N_i = 10775.9$ kN
Resulting force in vertical plane:
$R = \sqrt{10775.9^2 + 1110.1^2}$ kN $= 10832.9$ kN (ULS)

$A_{concrete,\ average} = 2 \times 1650 \times 2500 = 8{,}25 \cdot 10^6$ mm² $\perp$ normal force
$A_{concrete,\ minimum} = 2500 \times 2500 = 6{,}25 \cdot 10^6$ mm²
Introduction of forces:
normal stresses $\sigma = \frac{F}{A} = \frac{10832.9 \cdot 10^3}{6.25 \cdot 10^6} = 1{,}73$ N/mm² $< f_b$ $< f'_b$
Requirement $\sigma_{tension} < 0{,}5 * f_b = 1{,}08$ N/mm²

* Shear forces

$\tau_d = \frac{V.S}{b.I} \Rightarrow \tau_d = \frac{Vd}{b.d}$ rectangular cross section

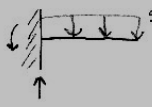
vertical: $\tau_d = \frac{1110{,}1 * 10^3}{2 * 1650 * 2300} = 0{,}15$ N/mm²

$\tau_d \leq \tau_u$
$\tau_u = \tau_1 + \tau_s \not> \tau_2$
$\tau_1 \not< 0{,}4 * f_b = 0{,}4 * 2{,}15 = 0{,}8$ N/mm² B65
$\tau_2 \not> 0{,}2 * f'_b * k_n * k_\theta = 0{,}2 * 39 * 1 * 1 = 7{,}8$ N/mm²

$\tau_d = 0{,}15 < \tau_u = 0{,}8 + \tau_s$ ok

$\tau_s = \frac{V_s}{b.d} = \frac{A_{sv} * z * f_s * \sin\alpha(\cot g\theta + \cot g\alpha)}{b * d}$ with $\alpha = 90°$

Dead load of stud: $A_{top} * h * \gamma_{concrete}$
$A_{top} = (2{,}5^2 + 2 * \frac{1}{2} * 1{,}5 * 2{,}5) - \pi * 0{,}5^2 = 9{,}21$ m²
$Q = 9{,}21 * 2{,}5 * 26$ kN/m³ $= 598.7$ kN ↓



$\tau_{d\ max} = \frac{(1110{,}1 + 598{,}7) \cdot 10^3}{2 * 1650 * 2300} = 0{,}23$ N/mm² $< 0{,}8$

Select practical reinforcement
$A_{sv\ min} = \phi 16\text{-}150 = 1340$ mm²/m

horizontal: $\tau_d = \frac{4237 * 10^3}{2500 * 2300} = 0{,}1$ N/mm² $< \tau_u = 0{,}8 + \tau_s$
Select $A_{sv\ min} = \phi 16\text{-}150 = 1340$ mm²/m

NB forces from steel tube in concrete contact area (within curve)



$A_{concrete} = 1000 * 2500 = 2{,}5 * 10^6$ mm²
$\sigma = \frac{F}{A} = \frac{10832.9 \cdot 10^3}{2{,}5 \cdot 10^6} = 4{,}33$ N/mm²

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Prestressing elements : tendons

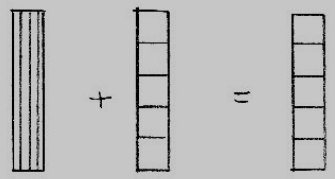
* Curved tendons in the connector causes a compression force in the stud and a distributed load due to the curvature. M=0.
* Smooth steel sleeve in curve ϕ 130 - 135 mm; n = 5 or 6
* Large losses through friction
  Estimation $F_{pw} = 0{,}5\ F_{pi}$
* requirement $\sigma \leq 1{,}08\ N/mm^2$ small tension allowed
* needed . $\sigma_{pw} \approx 3-4\ N/mm^2$
  $\sigma_{po} = 6\ N/mm^2$ incl g_p

$F_{pi} = \sigma_{pi} \times A_p \Rightarrow$ estimate $F_{po} = 6 * 7{,}5.10^6 * 10^{-3}$
$F_{pi} = 1300 * A_p$ $= 45000$ kN (excl g_p)

Assume $27\ \phi 15{,}7 * 6 = 24300\ mm^2$

$F_{pi} = 1300 * 24300 * 10^{-3} = 31590$ kN

Selection : use 6 tendons, with 19-27 strands



$\sigma_p = \frac{-31590 * 10^3}{7{,}5 * 10^6} = -4{,}2\ N/mm^2$

(outside curved tendons) ok

$A_p\ \phi 15{,}7 = 150\ mm^2$ n = 19, 25, 31, 37, 55 $\phi_{max} = 165$ mm
$A_b = 2 * 1500 * 2500 = 7{,}5 * 10^6\ mm^2$

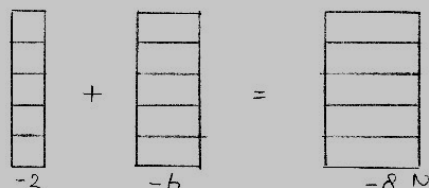
Selection: Type 27K15 of Freyssinet $\phi_d = 120-125$ mm
Requirements: $R_{min} = 1250$ mm < 1500 ok
anchorage : centre-to-centre distance x,y = 460 mm
dimensions bxh = 252 x 252 mm

* Requirements regarding curved tendons: NEN 6720
$r \geq 20 * \phi_k = 20 * 130 = 2600$ mm at maximum cracking (breaking) force
$\max \frac{N}{n} = 1$

ϕD tendons = 1000 + 2 * 1000 = 3000 mm
r = 1/2 * 3000 = 1500 mm < 2600 mm

Assumption : 50 % of cracking force can be exploited, instead of 100 % in ULS
$f'_b * 50\% = 19{,}5\ N/mm^2$ compression allowable ULS
$f_b * 50\% = 1{,}08\ N/mm^2$ tension stress allowable ULS

most unfavourable situation: (inside curved tendons).



< 19,5 ok

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prestressing level = $\frac{Fpw}{Ab}$

100% of Fpw is beared by the concrete within the curve of the tendons.



ep = R curved tendons = 1500 mm

$qp = \frac{8 \cdot Fp \cdot f}{l}$

$q = \frac{Fp}{R}$

$O = 2\pi R = 9425 * 75\% = 7069$ mm

curvature = $\frac{1}{R} = \frac{1}{1500}$ mm^{-1}

* 1 tendon: $Ap = 27 * 150 = 4050$ mm^2

$Fp = Ap * \sigma pi = 1300 * 4050 * 10^{-3} = 5265$ kN

$Ab = \pi \cdot 1500^2 - \pi \cdot 500^2 = 6{,}3 \cdot 10^6$ mm^2

contact area between curved tendon and steel tube

* 1 tendon:

$\sigma = -2 - \frac{5265 * 10^3}{6{,}3 * 10^6} = -2{,}8$ N/mm^2 > -19 N/mm^2 ok

* 6 tendons:

$\sigma = -2 - \frac{31590 * 10^3}{6{,}3 * 10^6} = -5{,}0$ N/mm^2 > -19 N/mm^2 ok

qp 1 tendon = $\frac{5265}{1{,}5} = 3510$ kN/m' qp causes a compression force, not a bending moment

qpw 1 tendon = $\frac{0{,}5 * 5265}{1{,}5} = 1755$ kN/m' → $\sigma = 2{,}0$ N/mm^2

$\sigma = \frac{7{,}069 * -3510 * 10^3}{6{,}3 * 10^6} = -3{,}9$ N/mm^2 extra compression stress (max.)

* Check of tension stress: governing situation: Fpw

Location: within curved tendons

$\sigma = +4{,}3 - (0{,}5 * 4{,}2) - 2{,}0 = 0{,}2$ N/mm^2 < $0{,}5 * fb = 1{,}08$ N/mm^2 ok

(Fpw; qpw)

* Check of compressive stress: governing situation: Fpi

Location: within curved tendons

$\sigma = -4{,}3 - 4{,}2 - 3{,}9 = -12{,}4$ N/mm^2 > $0{,}5 * f'b = 19$ N/mm^2 ok

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Check maximum stresses (optimisation of number of tendons)

Governing location: outside of steel tube
$A_{concrete}$ side view = $1000 * 2500 = 2{,}5 * 10^6$ mm²
$A_{concrete}$ top view = $1500 * 1000 - \frac{1}{2}(2\pi 500) = 1{,}5 * 10^6$ mm²

$|\sigma_{max}| = 4{,}33$ N/mm²

Selection: 6 tendons, 30 strands Ø15,7 FeP1860
$30 * 150$ mm² = 4500 mm² per tendon ⇒ Fpi = 5850 kN

Governing situation: at Fpi not Fpw
including q_p due to the curvature of the tendon

$F_{pi} = 1300 * 6 * 4500 * 10^{-3} = 35100$ kN
$\sigma_p = \frac{35100 * 10^3}{7{,}5 * 10^6} = 4{,}7$ N/mm² prestressing level whole stud
$q_p = \frac{F_p}{R} = \frac{4500 * 1300 * 10^{-3}}{1{,}5} = 3900$ kN/m'
q_p causes radial compressive stress
$l = 75\% * 2\pi R = 7069$ mm
$\sigma = \frac{F}{A} = \frac{7{,}069 * -3900 * 10^3}{6{,}3 * 10^6} = -4{,}4$ N/mm²

$\sigma_{min} = -4{,}3 - 4{,}7 - 4{,}4 = -13{,}4$ N/mm² > -19 compression ok
$\sigma_{max} = +4{,}3 - (0{,}5 * 4{,}7) - (0{,}5 * 4{,}4) = -0{,}25$ N/mm² < +1,08 tension ok

max compression occurs at initial tensioning; Fpi
max tension occurs at max load and Fpw (incl losses) = 0,5 Fpi

Check space for tendons and anchorage

6 tendons distributed over a height of 2500 mm
Selected tendon type: 27 K15
Ød = 120 - 125 mm
Rmin = 1250 mm < 1500 mm ok ok
Anchor b * h = 252 x 252 mm l = 581 mm
spiral reinforcement Ø560 mm < ½ 252 + ax = 586 mm
centre-to-centre distance x,y ≥ 460 mm
edge/rim - anchor distance x,y ≥ 330 mm
Working space for tensioning l = 2m < 5m compartment ok

Anchorage: 2 rows of 3 anchors
$\frac{2500 - 2 * 330 - 2 \cdot \frac{1}{2} \cdot 252}{5} = 318 < 460$ mm not sufficient space for 6 tendons

$\frac{2500 + 330 - 700 - 2 * \frac{1}{2} * 252}{2} = 939$ mm > 460 mm ⇒ optimisation in drawing

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Splitting reinforcement

* A splitting tensile force perpendicular to the concentrated prestressing load is necessary for the equilibrium of a truss model. The concentrated load works in the direction of the tendons. The distributed load q_p is a result of the curvature of the tendon.

$$A_{spl} = \frac{N_{spl}}{\sigma_s}$$

Assume $\sigma_s = \frac{1}{2} f_s = \frac{1}{2} * 435 = 218\ N/mm^2$

According to NEN 6720, art 9.13 : $f_s = \frac{3000}{\phi_k} \Rightarrow 360\ N/mm^2$ FeB500

$$f_s = \frac{3000}{16} = 188\ N/mm^2$$

Inner wall

$N_{spl} = 2 * 1/4 * F_{pw} * (1 - h_1/h) = 2/4 * 2925 * (1 - 1/2500) = 901$ kN

$F_{pw} = 5850 * 0.5 = 2925$ kN

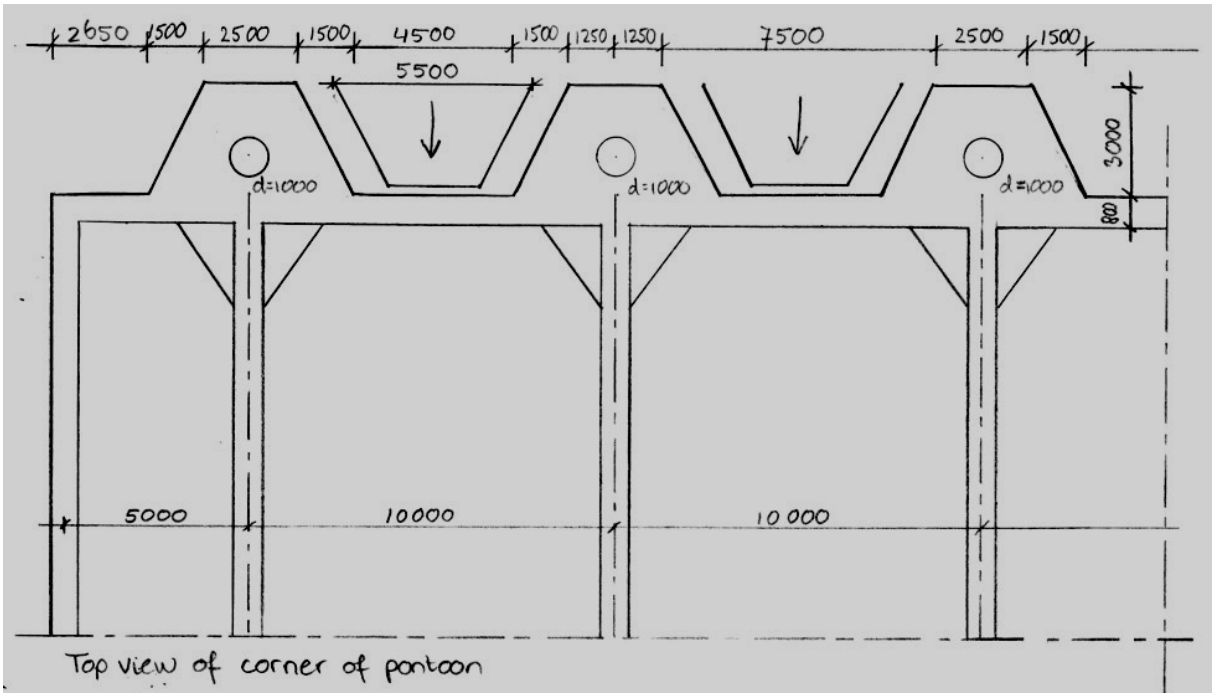
$A_{spl} = \frac{1275 \cdot 10^3}{188} = 4792\ mm^2 \Rightarrow$ Stirrups 2 sections $A_s = \frac{4792}{2} = 2396\ mm^2$

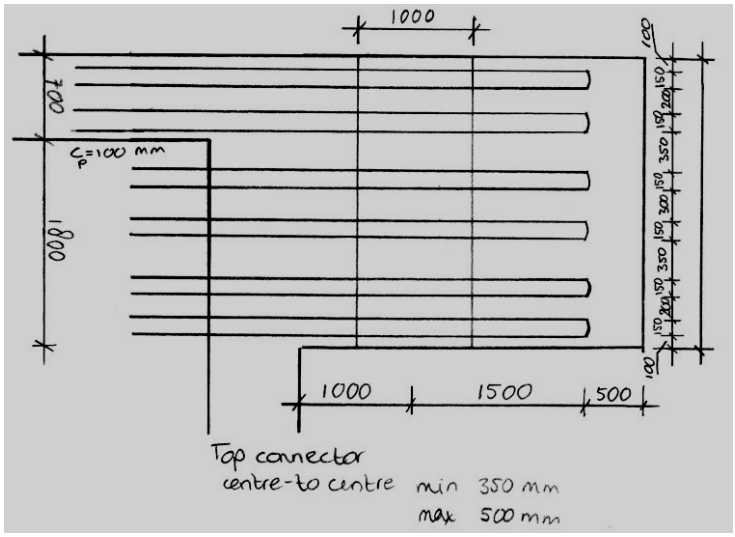
Near to anchorage : $\phi 20 - 100 = 3142\ mm^2$: 2 m

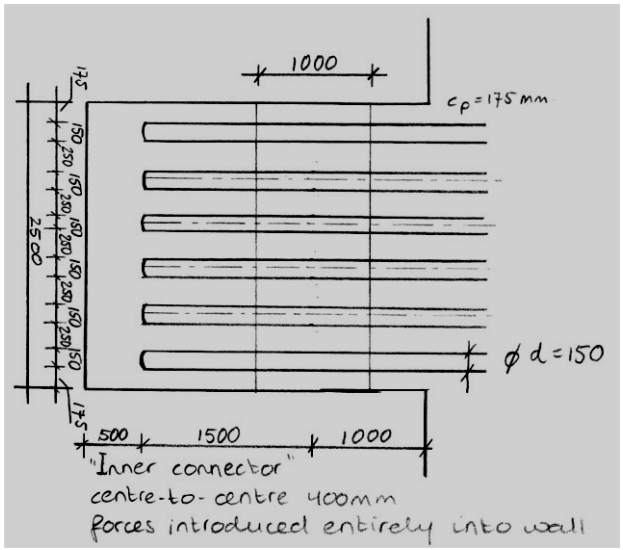
Centre of inner wall of outer compartment : $\phi 20 - 100$:

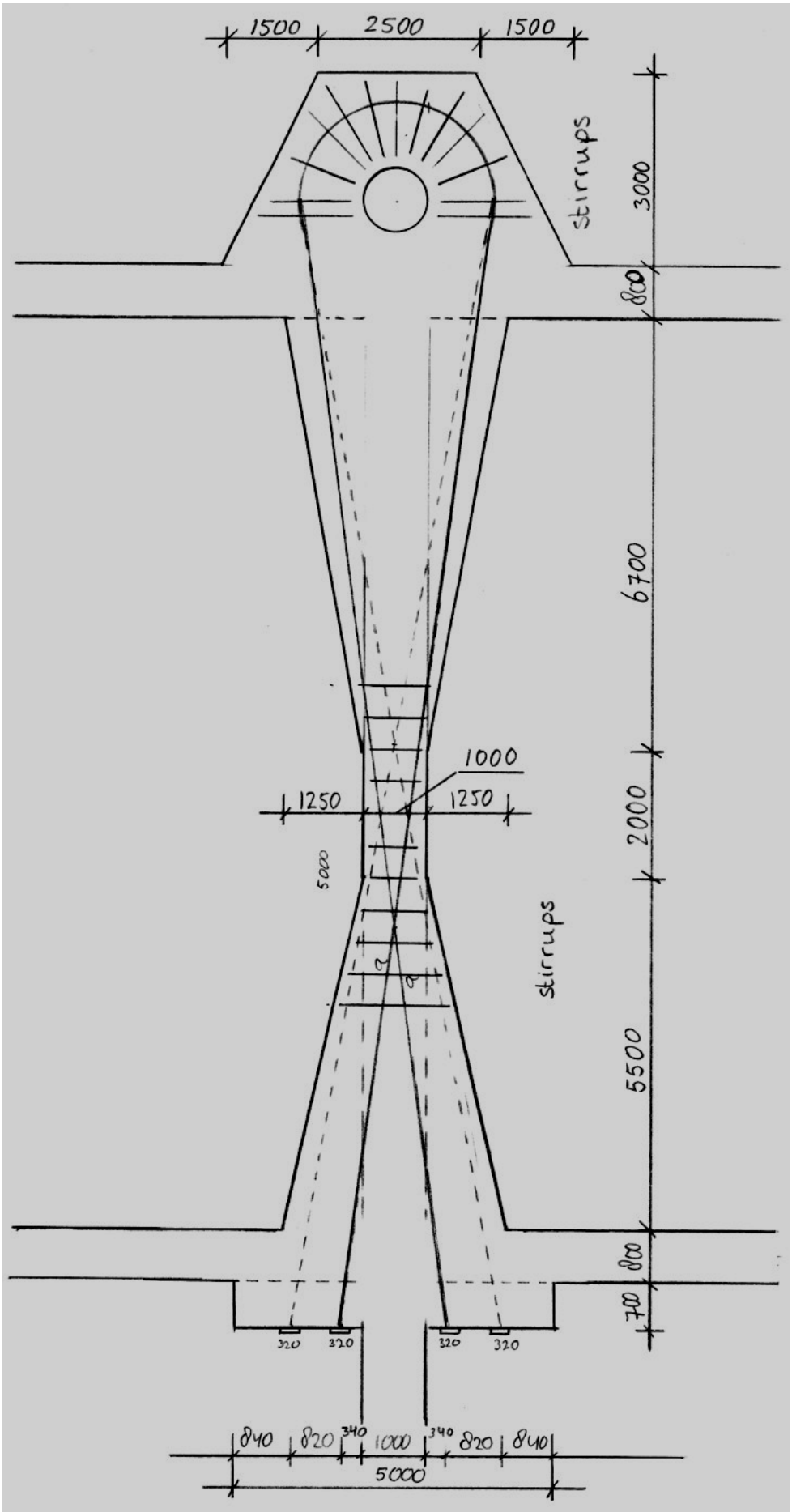
* Practical reinforcement is chosen next to splitting reinforcement:
  $\phi 16 - 100$ 2 directions top and bottom
  Stirrups because of shear forces :
  $\phi 16 - 100$ vertical and horizontal
  placed radially around steel tube

* 3 locations with splitting reinforcement :
  1. around curved tendons in connector
  2. In inner wall, where tendons come together
  3. Behind anchorage


2650
1500
2500
1500
4500
1500
1250
1250
7500
2500
1500
5500
d=1000
d=1000
d=1000
3000
800
5000
10000
10 000
Top view of corner of pontoon


1000
700
c_p=100 mm
1800
1000
1500
500
Top connector
centre-to centre min 350 mm
max 500 mm


1000
c_p=175 mm
2500
ϕ d=150
500
1500
1000
"Inner connector"
centre-to-centre 400mm
forces introduced entirely into wall


1500
2500
1500
stirrups
3000
800
6700
1000
1250
1250
2000
5000
stirrups
5500
800
700
320
320
320
320
840
820
340
1000
340
820
840
5000

Van Hattum en Blankevoort

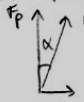
werknr.: ..... datum: .....
betreft: ..... naam: .....
..... bl. nr.: ..... van .....

Structural details

* The 2 ends of the curved tendon cross each other.
* At this section, 2 times the splitting force is taken into account, because of the curved profile of both ends of the tendon
* The tendons are not installed in the direction of the inner wall. Therefore, Fpi is the resultant of 2 forces, and thus larger than the necessary prestressing force.

$\tan\alpha = 2250/17500 \Rightarrow \alpha = 7°$

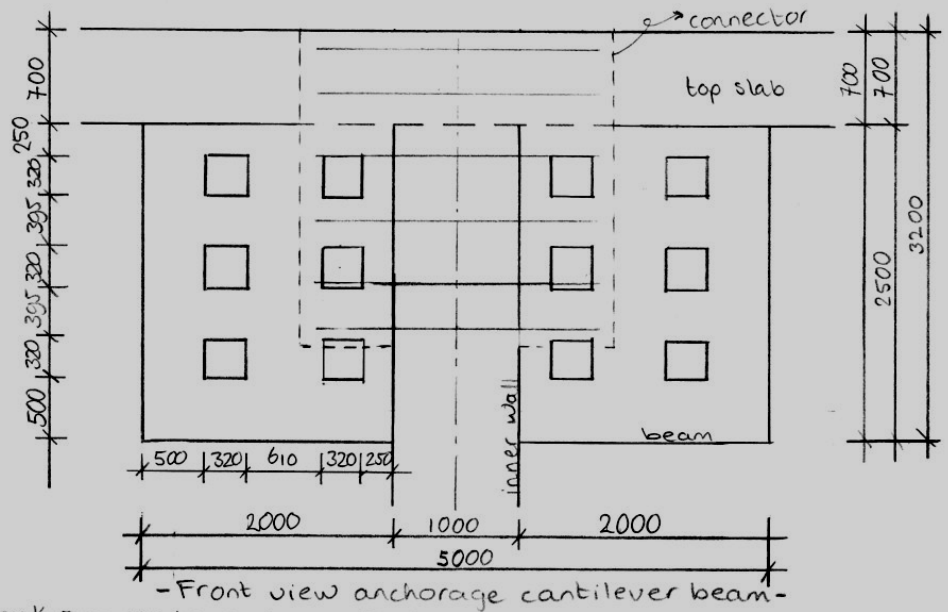
Since α is small, the difference between R and $F_{p\,necessary}$ is also small.



$R = \frac{F_p}{\cos\alpha} = 1{,}007\,F_p$

$\Delta = 0{,}7\,\%$

* The anchors are divided in 2 rows of 3 anchors at either side
  For 3 tendons, α is bigger than for the other 3 tendons
* 3 sleeves with 29 strands are selected and 3 sleeves with 31 strands are selected (max 37)
  Both of type 35 K 15 Freyssinet.
* The anchors are placed as close to the inner wall as possible



-Front view anchorage cantilever beam-

37 K 15: centre-to-centre = 590 > 540 mm
edge-to-anchor = 500 ≥ 380 mm
anchor max 320 × 320 mm

$\frac{2500 - 2 \times 500 - 320}{2} = 590$

3.3 Design of steel bar

Van Hattum en Blankevoort

werknr.: ..... datum: .....

betreft: ..... naam: .....

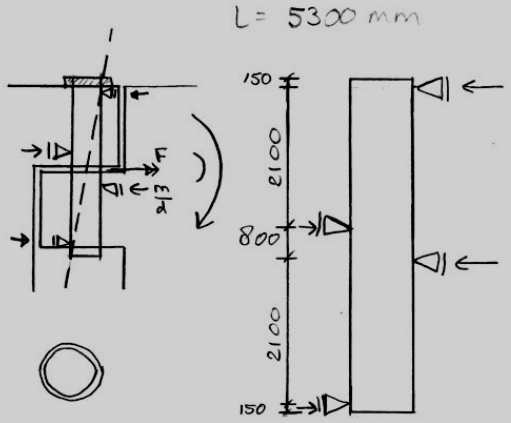
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Steel tube / bar

Assume : D=Ø 1000 mm outer diameter t=150 mm

Steel S355

L= 5300 mm



d = distance between tube sections

d between ♡ connectors = 11m

F = M/d compression / tension

$V_d = \frac{M}{d}$

$a * \frac{M}{d} = b * F_b$

$a * \frac{M}{d} = M$

a = 400 mm b = 2300 mm

2a = 800 mm

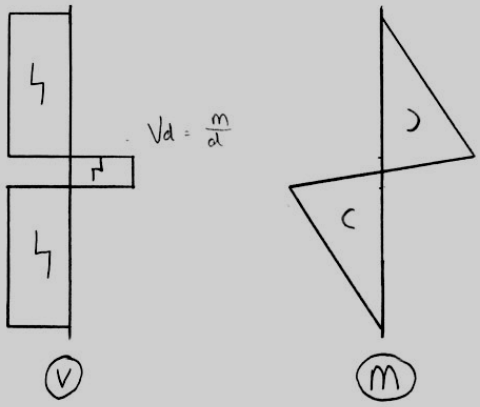
M max = 110874.7 kNm

$V = \frac{M}{11}$ = 10.079,5 kN

V + Nsurge = 10.775,9 kN

$F_b = \frac{1}{2} . \frac{M}{d}$

$2 * V = M/d$



$M_d = b * F_b$

$= 2,1 * 0,5 * \frac{M}{d}$ = 10583,5 kNm

Vd = 10079,5 kN

$I_{tube} = 0,4 * D^3 * t = 6.10^{10}$ mm⁴

$W_{tube} = 0,8 * D^2 * t = 1,2.10^8$ mm³

$M_u = f_y * W$

$M_{u\ S355}$ = 42600 kNm

$\tau_d = \frac{V_d . S}{b . I}$

$V_d \leq 0,25 . V_{pl;d}$

circular, thin walled tube: $\tau_{max} = 2 * \tau_{average}$

$\tau_{average} = \frac{V_d}{A}$

$\tau_{max} = \frac{2 * 10079,5 * 10^3}{(\pi . 500^2) - (\pi . 350^2)} = 50,3\ N/mm^2 < \frac{f_{y;d}}{\sqrt{3}}$

$A = 4,006 . 10^5$ mm²

Van Hattum en Blankevoort | werknr.: | datum:
betreft: | naam:
bl. nr.: van

Steel tube /bar

*Combination of bending and shear (N=0) requirements:

$M_d < M_u$? $V_d < V_u$ } Interaction formulas for thin walled tubes: $V_d \leq 0{,}25\, V_{pl} \Rightarrow \frac{M_d}{1{,}04\, M_{pl}} \leq 1$

$M_u = f_{yd} * A = 42600$ kNm $> M_d = 10583{,}5$ kNm

$V_{u;pl} = \frac{f_{y;d}}{\sqrt{3}} * A = 82107$ kN $> V_d = 10079{,}5$ kN $\leq 0{,}25\, V_{pl}$

* Check of $t = 100$ mm

$W = 0{,}8 * 10^6$ mm^3

$M_u = 28400$ kNm $> M_d$ ok

$\tau_{max} = 2 * \frac{10079{,}5 * 10^3}{(\pi \cdot 500^2) - (\pi \cdot 400^2)} = 71{,}3$ N/mm^2 < 205 N/mm^2 ok

$V_u = \frac{f_{y;d}}{\sqrt{3}} * A = 57951$ kN $> V_d$ ok

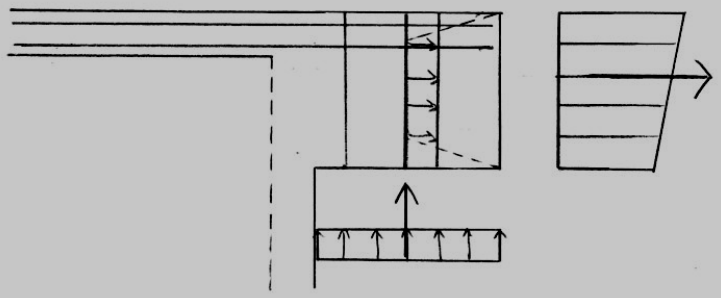
4 Introduction of connection forces

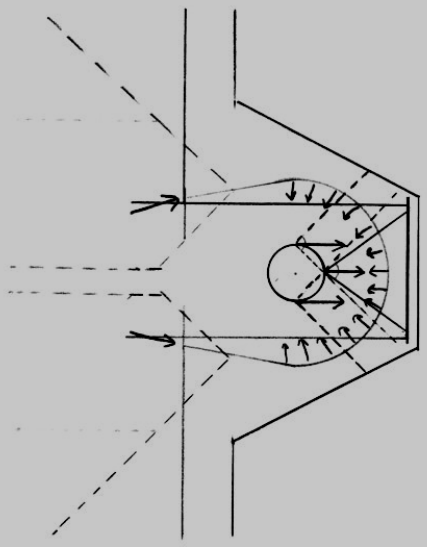
Van Hattum en Blankevoort

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naam: .....
bl. nr.: 2 van .....

Introduction connection forces





Force introduction in connector and top slab

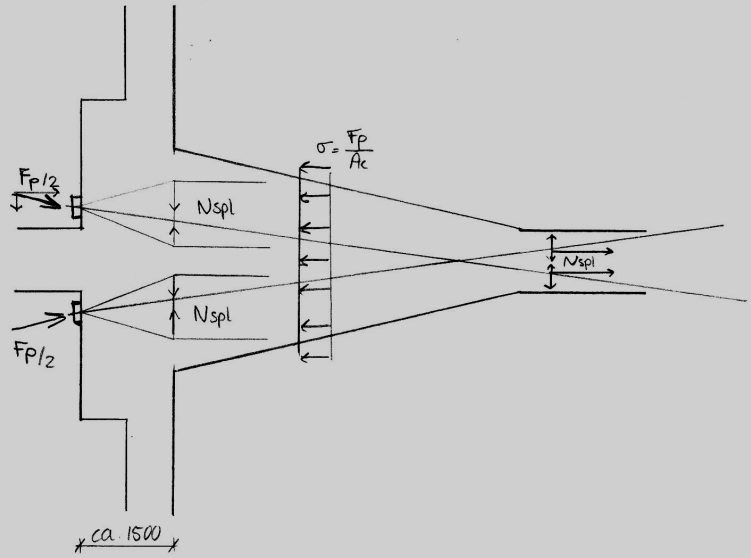
Van Hattum en Blankevoort

werknr.: .....
betreft: Force introduction
.....

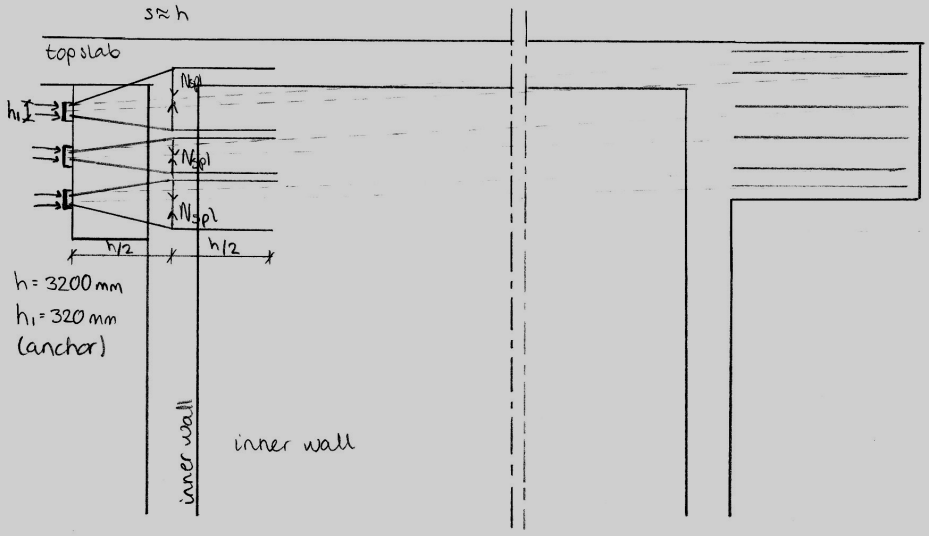
datum: .....
naam: .....
bl. nr.: 1 van .....

Introduction of prestressing force

* Top view:



* Side view:



Van Hattum en Blankevoort | werknr.: | datum: | betreft: | naam: | bl. nr.: 3 van

Introduction of connection forces

The loads from the top and bottom studs are transferred to the inner walls and the top or bottom slab respectively.

The loads on the "inner studs", nearer to the centre of the height of the pontoon, are transferred only to the inner walls.

① Inner wall of outer compartment:
The governing situation: all connection forces introduced into wall
t = 1000 mm
haunches on either side, which compensate each other.

* horizontal prestressing steel:

20 ϕ15,7 - 1000 mm => sufficient space for anchorage and for extra curved tendons from studs: 6 * 2 ϕ 15,7 top and bottom
ϕd = 150 mm (outer)

Anchorage of curved tendons: at cantilever beam behind 1st inner wall

Anchorage of horizontal prestressing in the wall: at the ends of the pontoon (including couplers)

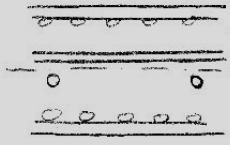
Thus, there should only be sufficient space for the tendons in the wall section, the anchors are placed in different locations.

According to NEN6720
the distance in between tendons a> D = ca. 40 mm
> 25 mm
> 0,5 ϕk = max 75 mm

The centre-to-centre distance in the wall of 500 mm will meet the requirements

* Vertical prestressing steel:

14 ϕ 15,7 - 500 mm at the supports (3 m on both sides) ϕd = 105
10 ϕ 15,7 - 1000 mm in the middle ϕd = 85 mm



$\Delta y = \frac{1000 - 2*50 - 4*16 - 105 - 150 - 80}{2} = 250$ mm

ok

Van Hattum en Blankevoort | werknr.: | datum: | betreft: | naam: | bl. nr.: 4 van

Check of stresses in inner wall of outer compartment

Stresses are caused by

* bending moment in wall LC II
* prestressing force in wall LC II
* connection forces transferred by studs: normal force and shear
* curved prestressing

Shear forces are checked separately.

*Bending moments:

$M_{x;0} = 375,4$ kNm $M_{z;0} = 430,1$ kNm
$M_{s;x;0} = -963,5$ kNm $M_{s;z;0} = -1234,3$ kNm
* $F_{p;x} = 3900$ kN (constr dock) $F_{p;z} = 1950$ kN $F_{pw;z} = 1658$ kN
$F_{pw;x} = 3315$ kN (corrected) $F_{ps;z} = 5460$ kN $F_{pw;sz} = 4641$ kN

* Governing inner wall in outer compartment = underneath crane track
Normal force due to crane track:
$q = 800$ kN/m
$\sigma = \frac{F}{A} = \frac{-800 * 10^3}{1000 * 1000} = -0,8$ N/mm² < $f'b = 39$ N/mm²

* Normal force from connection:
$N_{total} = 10832,9$ kN (resultant)
Introduced completely into wall: $\sigma = \frac{F}{A} = \frac{10832,9 * 10^3}{1000 * 3500} = 3,1$ N/mm²
distributed over height 2500 + 2*500

* Normal forces from prestressing:
6 * 29 Ø15.7 => $F_{pi} = 26100$ kN $F_{pw} = 13050$
$\sigma_p = \frac{26100 * 10^3}{1000 * 3500} = 7,5$ N/mm² $\sigma_{pw} = 3,7$ N/mm²

wall Fp:
$\sigma_x = 3900.10^3/1000^2 = 3,9$ N/mm² $\sigma_{z;max} = 5460\,10^3/1000^2 = 5,5$
$\sigma_{xw} = 3,3$ N/mm² $\sigma_{zw;max} = 4,6$ N/mm²

* Maximum compressive stress
horizontal
$\sigma_{min} = \frac{-5,8}{M_{sx}} - 3,3 - 3,1 - 3,7 - 0,8 - 4,6 = 21,3 < 29$ N/mm² $= 0,75 * f'b$

vertical
$\sigma_{min} = \frac{-7,4}{M_{sz}} - 4,6 - 0,8 - 3,3 - 3,1 - 3,7 = 22,9 < 29$ N/mm² $= 0,75 * f'b$
Ok, design meets the requirements

Van Hattum en Blankevoort

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* Maximum tensile stress

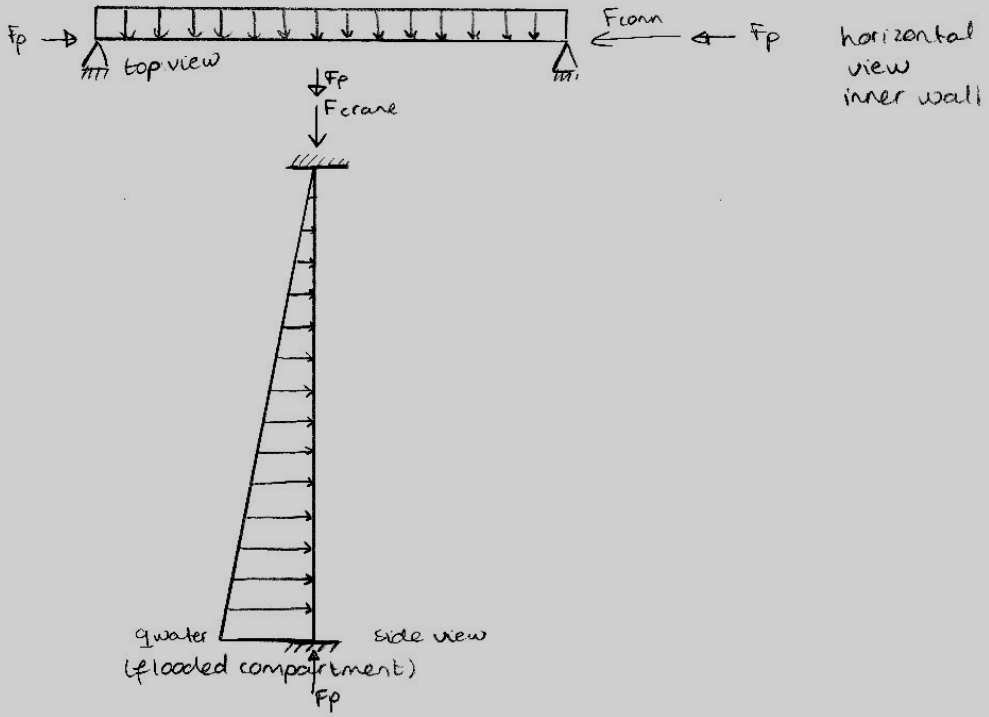
horizontal

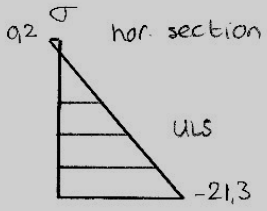
$\sigma_{max} = +\underset{M_{SX}}{5,8} - 3,3 + 3,1 - 3,7 - \underset{F_{pw;VB}}{1,7} = +0,2\ N/mm^2 < 0,5 \times f_b = 1,08\ N/mm^2$

vertical

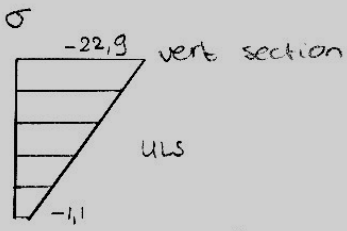
$\sigma_{max} = +7,4 - 4,6 + 3,1 - 3,7 - 3,3 = -1,1\ N/mm^2 < 0,5 \times f_b = 1,08\ N/mm^2$

ok, the design meets the requirements.





σ as a result of:
$M_{SX} + F_{pw,x} + F_{pw,z} + N_{conn} + F_{pw,conn} + F_{crane}$



σ as a result of:
$M_{SZ} + F_{pw,z} + F_{pw,x} + N_{conn} + F_{pw,conn} + F_{crane}$

Inner walls of outer compartment: steunpuntmoment horizontaal

Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Invoercellen | Iteratiewaard | Overschrijding

Beton		Ligger			Belastingen			Berekeningen		
Sterkteklasse =	C53/65	Totale hoogte =	1000	mm	Uitwendige (SLS)normaalkracht op de doorsnede =	0	kN	Hoogte drukzone BGT =	922,0	mm
$f`_{ck}$ =	65 MPa	A_b =	1000000	mm^2	Uitwendige (ULS) normaalkracht op de doorsnede =	0	kN	ε_b =	-0,00033	
f_b =	39 MPa	zwpt tov boven z_{bb} =	500	mm	Normaalkracht door voorspanning $-F_p$ =	-3442,5	kN	erschil M (of N bij N_{SLS}=0) =	0	kNm (of kN bij N_{SLS}=0)
ε_{bpl} =	0,00175	zwpt tov onder z_{bo} =	500	mm	Normaalkracht (SLS) inclusief voorspanbelasting =	-3443	kN	Hoogte drukzone UGT =	188,0	mm
ε_{bu} =	0,00350	I_b =	8,33E+10	mm^4	Normaalkracht (ULS) inclusief voorspanbelasting =	-3443	kN	Constante deel drukzone =	94,0	mm
E_b* =	22286 MPa	W_{bb} (boven) =	1,67E+08	mm^3	Moment (SLS) inclusief voorspanbelasting =	642,33	kNm	Verschil N =	0	kN
α =	0,75	W_{bo} (onder) =	1,67E+08	mm^3	Moment ULS inclusief voorspanbelasting =	963,50	kNm	M_u =	2412	kNm
β =	0,389	k_{bb} (boven) =	167	mm	Betonspanning bovenzijde SLS σ_{bb} =	-7,30	MPa			
		k_{bo} (onder) =	167	mm	Betonspanning onderzijde SLS σ_{bo} =	0,41	MPa			

							BGT											UGT					
	Hoogte	Breedte	Oppervlak	zbb	S	I	$\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	ε_{top}	$\sigma'_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	1000	1000	1,0000E+06	500	#########	3,3333E+11	1,000	ONWAAR	WAAR	0	10273893	0	-193	-1979382	0	-3352	646	-7,3	0,00350	39,0	ONWAAR	WAAR	93,9987	ONWAAR
Laag 2			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	1000	#########	0,0000E+00	-0,085	ONWAAR	ONWAAR	0	0	500	474	0	0	0	0		-0,01512	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	1000		1,0000E+06		#########	3,3333E+11				0	10273893			-1979382	0	-3352	646			0				

	Wapeningsstaal						BGT								UGT				
	FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2011	1	628	250	435	0,319	128255	128	16417	-42	-5	-21	0,00819	435	874	128	111,92
Laag 2	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Totaal			2011						128255		16417	-42	-5				874		112

	Voorspanstaal						BGT											UGT						
	FeP	E_p	A_p	ξ_p	d_p	Fp	f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	$\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	3000	0,5	460	3442,5	1691	1522	1148	0,501	150328	-40	-6013	-49	2	-16	1131	0,00506	0,0108	394	1542	1182	-40	-47
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Totaal			3000			3442,5					150328		-6013	-49	2							1182		-47

Inner walls outer compartment: steunpuntmoment verticaal

Aanname: bovenzijde van de ligger is de gedrukte zijde van de ligger

Invoercellen | Iteratiewaard | Overschrijding

Beton

Sterkteklasse =	C53/65	
$f`_{ck}$ =	65	MPa
$f`_{b}$ =	39	MPa
ε_{bpl} =	0,00175	
ε_{bu} =	0,00350	
E_b* =	22286	MPa
α =	0,75	
β =	0,389	

Ligger

Totale hoogte =	1000	mm
A_b =	1000000	mm^2
zwpt tov boven z_{bb} =	500	mm
zwpt tov onder z_{bo} =	500	mm
I_b =	8,33E+10	mm^4
W_{bb} (boven) =	1,67E+08	mm^3
W_{bo} (onder) =	1,67E+08	mm^3
k_{bb} (boven) =	167	mm
k_{bo} (onder) =	167	mm

Belastingen

Uitwendige (SLS)normaalkracht op de doorsnede =	0	kN
Uitwendige (ULS) normaalkracht op de doorsnede =	0	kN
Normaalkracht door voorspanning -F_p =	-4819,5	kN
Normaalkracht (SLS) inclusief voorspanbelasting =	-4820	kN
Normaalkracht (ULS) inclusief voorspanbelasting =	-4820	kN
Moment (SLS) inclusief voorspanbelasting =	822,88	kNm
Moment ULS inclusief voorspanbelasting =	1234,32	kNm
Betonspanning bovenzijde SLS σ_{bb} =	-9,76	MPa
Betonspanning onderzijde SLS σ_{bo} =	0,12	MPa

Berekeningen

Hoogte drukzone BGT =	971,8	mm
ε_b =	-0,00043	
erschil M (of N bij N_{SLS}=0) =	0	kNm (of kN bij N_{SLS}=0)
Hoogte drukzone UGT =	248,5	mm
Constante deel drukzone =	124,3	mm
Verschil N =	0	kN
M_u =	2871	kNm

	Hoogte	Breedte	Oppervlak	zbb	S	I	BGT $\varepsilon_{top}/\varepsilon_{bb}$	geheel in drukzone	deels in drukzone	$N`_{b\ rechthoek}/\varepsilon_b$	$N`_{b\ driehoek}/\varepsilon_b$	$e_{b\ rechthoek}$	$e_{b\ driehoek}$	$M`_b/\varepsilon_b$	$N`_{b\ rechthoek}$	$N`_{b\ driehoek}$	M_b	$\sigma`_b$	UGT ε_{top}	$\sigma`_{b,top}$	geheel in constante drukzone	tussen constante en lineaire	h in constante deel	geheel in lineaire drukzone
Laag 1	1000	1000	1,0000E+06	500	#########	3,3333E+11	1,000	ONWAAR	WAAR	0	10828333	0	-176	-1906604	0	-4650	819	-9,6	0,00350	39,0	ONWAAR	WAAR	124,2585	ONWAAR
Laag 2			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 3			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 4			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 5			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 6			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 7			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 8			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 9			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Laag 10			0,0000E+00	1000	#########	0,0000E+00	-0,029	ONWAAR	ONWAAR	0	0	500	491	0	0	0	0		-0,01058	0,0	ONWAAR	ONWAAR	ONWAAR	ONWAAR
Totaal	1000		1,0000E+06		#########	3,3333E+11				0	10828333			-1906604	0	-4650	819			0				

	Wapeningsstaal FeB	E_s	A_s	ξ_s	d_s	Gewenste spanning σ_s	BGT f_s	$\varepsilon_s/\varepsilon_b$	N_s/ε_b	e_s	M_s/ε_b	N_s	M_s	σ_s	UGT ε_{su}	σ_s	N_{su}	e_s	M_{su}
Laag 1	500	200000	2011	1	628	250	435	0,354	142282	128	18212	-61	-8	-30	0,00534	435	874	128	111,92
Laag 2	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 3	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 4	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 5	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 6	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 7	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 8	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 9	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Laag 10	500	200000	0	1	0	435	435	1,000	0	-500	0	0	0	0	-0,00350	-435	0	-500	0
Totaal			2011						142282		18212	-61	-8				874		112

	Voorspanstaal FeP	E_p	A_p	ξ_p	d_p	Fp	BGT f_{pu}	0,9 f_{pu}	σ_{pw}	$\Delta\varepsilon_p/\varepsilon_b$	$\Delta N_p/\varepsilon_b$	e_p	$\Delta M_p/\varepsilon_b$	ΔN_p	ΔM_p	$\Delta\sigma_p$	σ_{pe}	UGT $\Delta\varepsilon_{pu}$	ε_{pu}	$\Delta\sigma_{pu}$	σ_{pu}	ΔN_{pu}	e_p	ΔM_{pu}
Laag 1	1860	200000	4200	0,5	390	4819,5	1691	1522	1148	0,599	251442	-110	-27659	-108	12	-26	1122	0,00199	0,0077	375	1523	1575	-110	-173
Laag 2	1860	200000	0	0,5	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 3	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 4	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 5	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 6	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 7	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 8	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 9	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Laag 10	1860	200000	0	0,75	0	0	1691	1522	0	1,000	0	-500	0	0	0	0	0	-0,00350	-0,0035	-700	-700	0	-500	0
Totaal			4200			4819,5					251442		-27659	-108	12							1575		-173

Van Hattum en Blankevoort

werknr.: ..... datum: .....

betreft: ..... naam: .....

..... bl. nr.: 7 van .....

Selected solution

top and bottom slab will have thickness t = 850 mm
Distribution of tendons over connecter height is symmetrical.

Since the connection force of 2 tendons was not too much for the top/bottom slab, 1 tendon will not cause a problem
The design of the top and bottom slab including prestressing, meets the requirements.