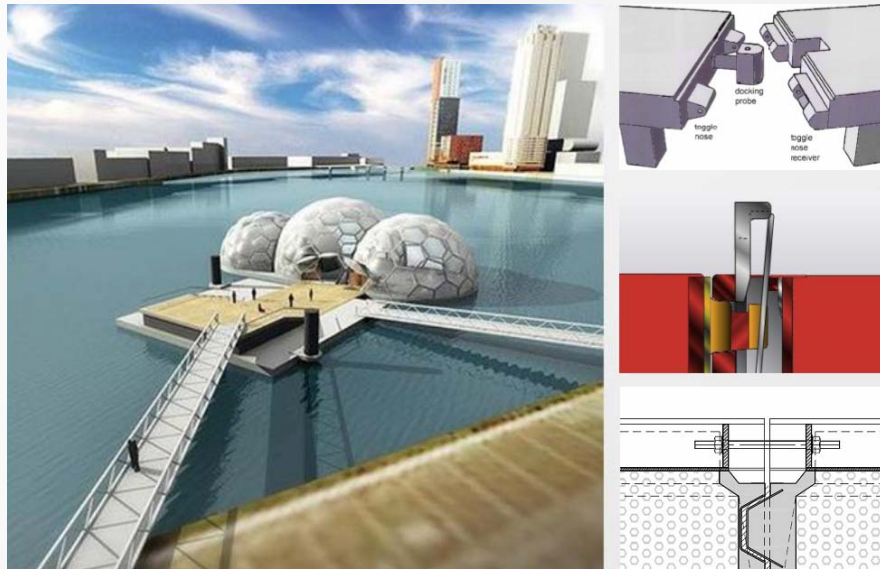


Master Thesis

# Connecting Modular Floating Structures

A General Survey  
and Structural Design of a Modular Floating Pavilion



Maarten Koekoek  
October 2010

## Master Thesis

# Connecting Modular Floating Structures

A General Survey and Structural Design of a Modular Pavilion

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*'Climate change is now forcing itself upon us: a new reality that cannot be ignored. The predicted sea level rise and greater fluctuations in river discharge compel us to look far into the future, to widen our scope and to anticipate developments further ahead.'*

Second Delta Committee [Commissie Veerman, 2008]



## Preface

This report is the graduation thesis of Maarten Koekoek, student at Delft University of Technology. This Thesis about floating structures, is written in order to receive the title of Master of Science in the field of Civil Engineering with specialisation Structural Design.

This thesis is about floating structures. For me this was, and still is, a very interesting subject in which all fields of the study Civil Engineering are combined. More over, it is a topic with a social interest, since building on water is a kind of climate adaptation. This is one of the reasons why this rarely used building form is increasing in the Netherlands and it has succeeded in attaining good prospects. With this thesis I hope to contribute to this sustainable building form and to increase the knowledge about one of the gaps in this field.

This thesis focuses especially on the connections between floating structures, but all important elements of floating structures are also treated in this thesis. This was partially done because it was necessary to construct a good structural design and also because I found it interesting myself. Moreover it became clear that the knowledge about building on water is yet quite scarce. To display this not well known general information, a comprehensive survey will be given to start this report. This survey of both floating structures and interconnections can be applied as a guide for prospective floating projects.

The city of Rotterdam has advanced ambitious plans for building on water in the old city harbours. The Floating Pavilion realised in the Rijnhaven is the pilot project for Rotterdam's floating ambitions. This pavilion is used as case study for this thesis. In this thesis a structural design for a modular floating pavilion and a design for the connections have been made.

This thesis is realised in cooperation with Engineering Company Public Works Rotterdam, one of the driving forces behind the floating pavilion. In cooperation with this engineering company it was possible to obtain the results the way they have been developed now. Publicworks Rotterdam provided me with a work space, where the greater part of this thesis has been accomplished. Therefore I would like to express my gratitude to Public Works Rotterdam and my colleagues employed there who shared their knowledge with me.

Graduating at Public Works Rotterdam also gave me the opportunity to attend the building meetings of the floating pavilion, where I have gained a large amount of knowledge. That is why I want to thank all companies and people involved with the floating pavilion who have provided me with vital information.

Of course this graduation would not have been possible without my supervisors, who helped me, corrected my errors and gave me much good advice. Special thanks go to my mother, sister and housemate who were very kind to proofread this thesis and corrected the multiple errors.

I hope the reader may find this thesis interesting, instructive and informative. I hope the information given in this thesis shall be used for a real building project!

Maarten Koekoek

Delft, October 2010

# Summary

## Introduction

Floating structures already exist for a long time, but recently building on water has taken flight. Two reasons behind the increased interest in floating structures are the climate change and the lack of available building space in the Netherlands.

Because of the climate change the earth is warming up. By this warming of the earth sea levels are rising and for the Netherlands this climate change results in more severe rainfall and higher river discharges on the one hand, and to longer periods of drought on the other. These phenomena result in water problematics; too much water at one moment and a water shortage at another moment. An answer to these water problematics is water storage, which could be realised with more surface water.

However, space for water conflicts with other interests. Already a shortage of land exists, building ground is scarce and costly and when creating space for water, there will be even less space for building. But, there is a solution to this problem: multiple space usage and more specific building on water.

These days more and more floating buildings are realized in the Netherlands. The expectation is that building on water will increase further, as there are already plans for living quarters existing of floating structures.

In this thesis first a general contemplation has been done, this resulted in general applicable information on floating structures and connections.

## Floating structures in general

In the past the legal status of floating structures was not clear in the Netherlands. It was doubted whether a floating house is real estate or a movable good. The legal status is very important for people who live in a floating house, because the status determines if they can have a mortgage and what insurances they need. But the legal status is also very important to the designers and engineers, since it determines if the building codes have to be applied or not. Nowadays all floating structures can be categorised as real estate, as long as they are meant to stay permanent, or long lasting, at a certain location. This means building codes should be applied on floating structures.

Most elements of floating structures can be judged and checked with the building codes the same way as buildings on firm land, but the building codes are not completely fit to be used for floating structures. There are some important aspects for floating structures, which are not mentioned in the building codes, like buoyancy and demanded freeboard. Also waves are not covered by the existing building codes. Waves appear to be a very important load case for floating structures.

A load case which is not mentioned in the building codes are waves. Waves appear to be a very important load case for floating structures. Waves are the main cause of undesired swell. They cause as well horizontal forces and vertical loading, which results in hogging and sagging moments. For Dutch inland waters the waves might be low, but these relative low waves can also cause moments in the floating body which are in the same range of moments by imposed loads.

To calculate the internal forces and deformations for floating structures, the water can be schematised as an elastic support. Water results however in a very soft elastic support with a very low  $k$ -value of only  $10\text{kN/m}^3$ . This means a structure on water will heavily subside in the water and it is strongly susceptible to vertical movement and tilting.

Stability is an important factor for floating structures. A rectangular horizontal section provides the best shape stability. It appears that for standard floating buildings the shape stability is the normative factor for the stability. This means that a wide body with a small draught gives the best static stability. With enlarging the width and length, the stability increases a lot.

In the last two decades, next to the standard concrete caissons also floating bodies constructed from a combination of EPS and concrete have been used. EPS is short for expanded polystyrene, which is a very light synthetic material which provides the floating body its buoyancy. The system with both EPS and concrete results in floating bodies with a low self weight and high buoyancy, which results in a low draught. Another advantage of this floating body is that it is unsinkable.

## Connections

In the last decades an increasing number of floating buildings has been constructed. But most of them are rather small individual floating buildings. Therefore connections in between floating structures are for the Dutch construction industry still a rather unmapped territory. Connecting floating structures is needed in the case of connecting separate floating structures in, for example a floating quarter or floating city, or when a floating structure is built up from several separate modules. With the development towards floating districts and floating cities and the tendency towards increasingly large floating structures, connections are becoming more important.

Using interconnections results in the following benefits:

- Flexibility for changing functions and future expansion
- Flexibility to control cost by building modular units
- Transportation and relocation possible
- Less use of material by reduction of internal forces

All functions and demands of connections are explored and mentioned in the thesis.

The main function of connections is connecting the floating bodies and thus restricting relative movements. Connections can prevent movement in all directions, but preventing movement results in increasing internal forces, while by allowing them, the forces can be decreased. Multiple connection types are possible, this way allowing or preventing every type of relative movement separately is possible.

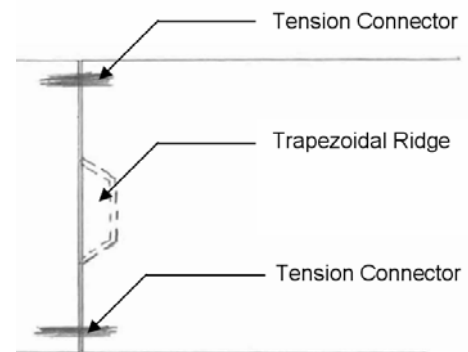
In part II these movement types have been differentiated and for all movement types possible connection alternatives are given. This is done for both of the two main families of interconnections; floating bodies with intermediate distance or without.

Without space in between was found a more interesting case, so the connection part mainly focuses on connection for floating bodies without intermediate space. It has been found that for Dutch inland water conditions it is not needed to allow some degrees of freedom for limiting the forces, thus the rigid connection is elaborated more extensively.

All possible basic connection options have been listed and examined. For all different connection types is explored what options are the best. For a rigid connection without intermediate distance it appeared that it can best consist of:

- trapezoidal ridges for self alignment and shear force.
- in the lower area a vertical pen as tension connector.
- on the topside a longitudinal bolt as tension connector.

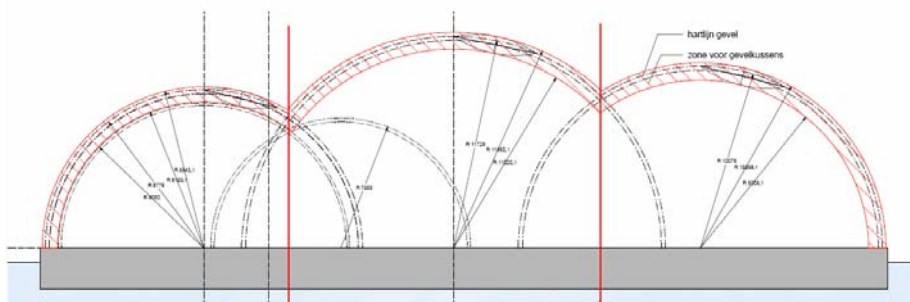
This connection has been elaborated and designed and dimensioned in the case study.



## Case study floating pavilion

For Rotterdam the option of floating buildings is very interesting. Since a large part of the old harbours will become available with the transfer of port activities towards Maasvlakte. As a result this area of inland water will become available, from which Rotterdam intends to use a part for floating buildings.

To give more publicity to these plans for the old ports, and to gain experience with floating structures, the municipality decided to construct a floating pavilion. The Floating Pavilion is a showcase and a test case for climate consciousness and building on water at the same time. The floating pavilion is the pilot project for Rotterdam's floating ambitions and has to function as an international icon where exhibitions and lectures about delta technology and climate adaptation can be given.





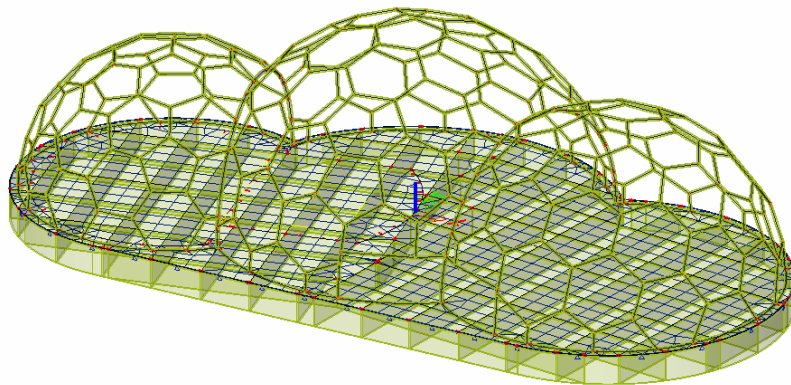
This floating pavilion, with dimensions of 24x46 metres, has already been constructed, but it has been constructed in one piece. In the initial design of the pavilion the separate spheres should be switchable and removable. In this way it would also be possible to attach more spheres in a later stage.

In a feasibility study done in the beginning of 2009, the realization of such a modular pavilion seemed difficult and time consuming and therefore appeared not feasible for this project. In particular, the reliability and feasibility of connections required too much research at that time. Therefore these issues have been investigated in this thesis.

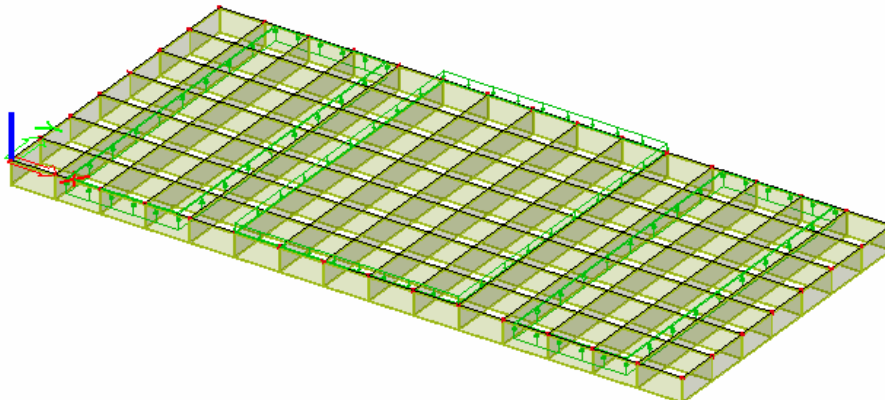
In the third part of this thesis a structural design is made for a modular pavilion, the structural design focuses on the floating body and mainly on the connections. But since the superstructure and the loads on the superstructure provide the boundary conditions for the floating body and connections, this superstructure has also been modelled and calculated.

A combination of EPS and a concrete framework has been found the best floating system for the pavilion, since this results in an unsinkable and light floating body with a low draught. The framework should have high beams for rigidity and to provide rigid connections between the modular parts. For the beams is chosen a center to center distance of three metres.

The best beam geometry is a rectangular beam geometry with beams perpendicular on the connection surfaces. It appears that the concrete beams and floor of the floating body result in the largest self weight by far, so these are the main determining factors of the draught of 0,88m. The total height of the floating body is the draught plus the demanded of freeboard of 800mm. This resulted in a height of 1,7m.



For determining the influence of waves and imposed loading a rectangular schematisation of the pavilion has been made. This rectangular schematization is more insightful than the complex real shape, so the effects of the loadings are more clear and the results can be checked with hand calculations. The rectangular shaped model also matches with the formula's for the motions and works better in the multiple computer programs.



The wave loading is schematised with static surface loads. First the wave load was calculated with wave properties which followed from the nomograms. This resulted in internal forces which were much bigger than the internal forces by imposed loading.

Subsequently the wave load and wave properties were examined more closely and the formula's of Bretschneider were used. This resulted in a normative waves with a design height ( $H_d$ ) of 1,51m and a length of 11,62m. These normative waves resulted in internal forces which are in the same range or a bit smaller than the internal forces by imposed loads. After the internal forces had been calculated, it appeared that the loads did not lead to too high internal forces; the floating body and beams can resist the loads. The floating body deforms only very little, unless it is loaded with torsion, than the deformations become large.

It is not necessary to decrease the internal forces by movement allowing connections, thus the connections between the beams can be executed as fully rigid connections.

Rigid connections are favourable from as well from functional demands as from viewpoint of movements. It appeared that the natural oscillation periods of the floating pavilion, are in the same range as the wave period. The natural oscillation periods for the modular parts are approximately equal as the oscillation periods for the whole pavilion, but when these modular parts are rigidly connected, the total floating body will have a length significantly larger than the wave length. As a result there will be less movements.

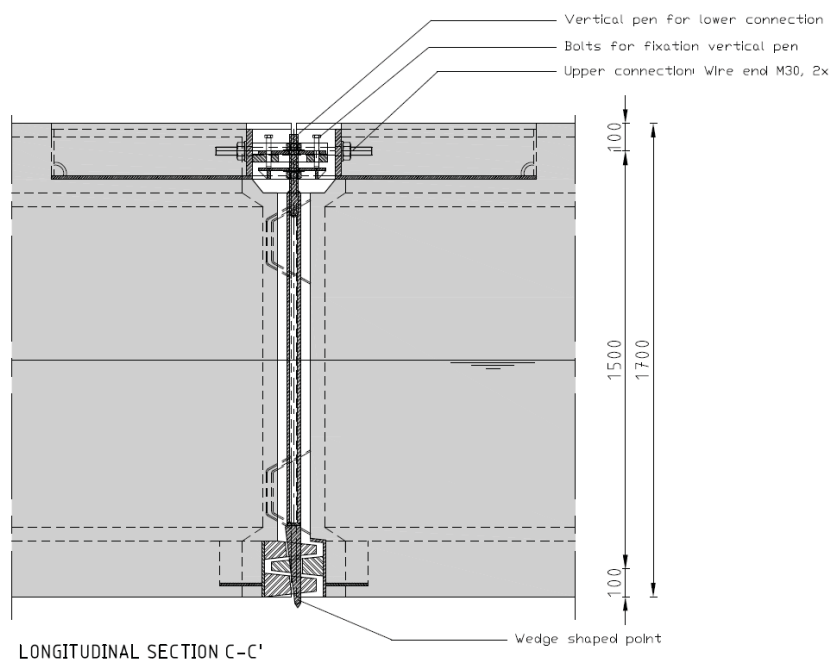
The rigid connection should be applied between the floating foundation only and the superstructure will not be part of this, because the dimensions of the elements of the superstructure should be highly increased if the super structure should also contribute to the rigid connection. Therefore a rigid connection between the beams is chosen.

The designed connection consists of the following:

- Trapezoidal ridges, for self alignment and shear forces
- A vertical steel pen with a wedge shaped point, for the tension connection at the bottom side. This pen will be easily to insert and to fix from the top of the floating bodies.
- A longitudinal bolt as tension connector on the top side and for pre-stressing the connection.
- Elastic material in between, for amongst others preventing small relative movement (which will exist by allowing deviations) and for impact damping.

This design has resulted in a connection easy in execution, since it provides self alignment and only actions from the top are needed. The connection does allow tolerances but will still provide a very rigid connection, since it can be tightened.

The designed connection is applicable for multiple floating systems and can be used for multiple projects, as long as the height of the floating body and the loads are in the same range as in this case study.



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## Thesis Structure

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# Introduction

## Floating structures

Floating structures are structures which rely on the buoyancy force of the water. Floating structures as meant in this thesis are used to station dwellings, utility buildings, roads, harbours or other (urban) functions on surface water. Floating structures are also used in offshore industry.

Floating structures are nothing new, in some places in the world people are living on water for centuries. In the last two decades however, and especially during the last few years building on water has taken flight.

## Building on water, answer to our problems

In this last decade the number of floating houses and floating utility buildings have strongly increased and have a more modern appearance. When this trend and the current plans and ideas are observed, it can be concluded that building on water is becoming more and more important. A reason behind this, is that floating structures can be a solution for two contemporary problems: water problematics and shortage of land in densely populated areas.

The climate is changing and the world is getting warming. By this warming of the earth sea levels are rising. This sea level rise will already cause water problematics for the Dutch coastal regions and on top of this the climate change will also result in more severe rainfall and higher river discharges on the one hand, and to longer droughts at the other. These phenomena result in water problematics; too much water at one moment and a water shortage at another moment. An answer to these water problematics is 'more space for water'. Water storage, which can be realised with more surface water, is one of the solutions.

More space for water conflicts with other interests though. The area claims in the Netherlands are increasing to a great extent. Already a shortage of land exists, building ground is scarce and costly and when creating space for water, there will be even less space for building. However, there is a solution to this problem: multiple space usage and more specific building on water.

Floating structures are more flexible than other forms of multiple space usage, because they move along with fluctuating water level and the structures itself can also be moved. These days more and more floating buildings are realized in the Netherlands. The expectation is that building on water will increase further, as there are already plans for whole quarters existing of floating structures. (Information of first paragraphs is elaborated in chapter 1)

## Building on water in Rotterdam

To Rotterdam the option of floating buildings is very interesting. Many port activities have already been transferred to the Maasvlakte or will move to the second Maasvlakte when this will be completed. With the shift of port activities towards Maasvlakte, 1600 hectares of former port sites will be available which can be developed for new destinations. Water plays an important role in the plans and also building on water plays a large role. in the plans. For the Maashaven a real 'Floating City' is planned.

## Floating Pavilion

To give more publicity to these plans for the old ports, and to gain experience with floating structures, the municipality decided to construct a floating pavilion. The Floating Pavilion is a showcase and a test case for climate consciousness and building on water at the same time. The floating pavilion is the pilot project for Rotterdam's floating ambitions and has to function as an international icon where exhibitions and lectures about delta technology and climate adaptation can be given.

## Subject thesis

The subject of this thesis is floating structures and the connecting of modular floating structures. The floating pavilion functions as case study for this subject. The pavilion has already been realised, but it has been constructed out of one part. In this thesis a new structural design is made for a pavilion constructed out of several modules.



## **Problem Analysis**

Building on water could be a solution to the combined issue of water problematics and the scarcity of building ground. With building on water several problems occur:

### **General problems with building on water**

With building on water several issues, complexities or problems occur which do not take place with building on firm land. Examples of this are:

- instability
- sinkability
- difficulties during execution by not having a firm work floor
- different and some times unclear regulations

Last two decades some experience has been gained with floating structures. A sparse but increasing amount of floating structures is realised every year. A large amount of research has been done in recent decades, mainly to the stability and preventing subsidence. This does not mean however, that the mentioned points do not result in problems anymore, but the knowledge to tackle these problems mostly prevails. However, there is no or hardly experience with connecting floating structures with each other.

### **Connections unmapped territory**

Most constructed floating structures are rather small individual floating buildings, and most studies also focus on behaviour of separate floating structures.

Therefore connections in between floating structures are for the Dutch construction industry and Dutch construction research institutes still a rather unmapped territory. Connecting floating structures is needed in the case of connecting separate floating structures in, for example a floating quarter or floating city, or when a floating structure is built up out of several separate modules.

At the moment the knowledge for creating a disconnectable and durable connection is still lacking.

### **Pavilion**

The initial idea for the Floating Pavilion was to create a modular pavilion. In this initial design the different spheres should be switchable and removable. In this way it would also be possible to attach more spheres at a later stage. In a feasibility study done in the start of 2009, in assignment of the commissioner of the pavilion, the realization of such a modular pavilion seemed difficult and time consuming and therefore appeared not feasible for this project, since there was little time for this project. In particular, the reliability and feasibility of connections required too much research. Therefore this lack of knowledge remained and is chosen as subject for this thesis.

### **Problem definition**

*At this moment there are no connection systems with accompanying floating bodies, which fit all demands to realise the floating pavilion out of modular parts.*

### **Sub questions:**

- Why building on water?
- What are the experiences and what is the current state of building on water?
- How does floating work and how will the stability be realised?
- What loads will act on a floating structure?
- What effects will waves have on a floating structure?
- Will using new and light materials be interesting in this design?
- Which criteria the floating body of the pavilion has to meet?
- What movements are allowable?
- What are the possible floating systems and what are their advantages and disadvantages?
- What are the possible connection options?
- How can the floating bodies be constructed and the connection be realised, so they can be achieved without great effort? (easy in execution)

## **Objective**

The objective of this graduation is:

*To realise a structural design for a modular separable floating pavilion, with the focus on the floating body and especially on the connections.*

Most important of the case study of the floating pavilion is to make a good structural design of the modular floating foundation, where the main focus of the research will be centred on the connections between the separate modules.

The goal of this study is to solve problems which occur in connecting floating structures by researching and designing these connections and in this way fill in the hiatus which exists in the knowledge of floating structures.

This expertise can thus be used for subsequent floating projects, amongst others the envisaged floating quarters in Rotterdam. The designed connections need to be easy to apply.

Floating structures will be placed in a broad perspective by including the history, current state and the necessity of building on water.



# Part I

## Floating Structures

*The first part of this thesis starts at the very start from floating structures and investigates the development of building on water from this beginning to the present. In the first chapter building on water is also placed in a broad social context, by treating the question 'Why building on water'. Current developments will also be discussed.*

*In chapter 2 "Basics of floating" some important aspects and essential knowledge of floating are treated. Subsequently the loads on floating structures are dealt with.*

*In chapter 4 the different types of floating bodies are considered and is analysed when what option is best.*

---



# 1. Introduction Floating Structures

What is meant by floating structures? Wang et al. (2008) gives the following definition of floating structures: *Floating structures rely on the buoyancy force of the water in order to support themselves. In the broadest sense, soft seabed contact structures where installations depend on a certain degree of buoyancy in order to reduce the reaction force on their supports, are considered floating structures.*

In this thesis, floating structures include all structures that float, starting from small structures of several metres, with water as their only or main vertical support, which have no own propelling and which mainly stay at the same location. So ships and rafts are not included in the term floating structures in this thesis.

Floating structures as meant in this thesis are used to provide space for dwellings, utility buildings, roads, harbors, infrastructure or other (urban) functions. Floating structures are also used in offshore industry.

## 1.1 From past to present

### 1.1.1 Floating dwellings

All over the world a very large majority of the people live, and have built a home on firm land. However there is also a small minority which is living on water, at some places even for centuries. In the following paragraphs this history will be described. More extensive information can be found in appendix 1.

#### ***Ancient history***

In multiple countries in south east Asia people have lived on water for more than thousand years . Examples are the floating villages in Cambodia. Not only in Cambodia there were, and still are, large floating communities, but also in Vietnam, Thailand, Indonesia and China. Where the Cambodian floating dwellings look like normal houses, the Chinese floating villages exist mostly out of small boats.



Figure 1: Floating village on the Tonle Sap, Cambodia

On the border of Peru and Bolivia the Incan tribe the Uros have lived and are still living on big rafts made of totora reeds. The Uros originally created their islands, many centuries ago, to prevent attacks from their more aggressive neighbours, the Incas and the Collas. References, more details and more examples of ancient floating communities can be found in appendix 1.

#### ***The beginning of floating in Western Europe and The Netherlands***

From the 17th century onwards people started to live in boats and ships in European cities as Amsterdam. [Kloos and De Korte, 2007] At the end of the nineteenth century the steel ship made it's entrance as cargo ship. The wooden ships could not compete with the steel ships and the steel ships replaced the wooden ships, so many wooden ships become useless and were given a new life as houseboat.

In 1918 the act for trailers and houseboats (Wet Woonwagens en Woonschepen, WWW) is introduced in the Netherlands. Then numerous houseboats are given a mooring permit. [Kloos and De Korte, 2007]

In 1922 the concrete ark, a hollow concrete foundation, made its introduction. In fact these arks were the first floating homes in Western Europe which were not houseboats in the strictest meaning of the word.

In contrast with the houseboat, which is a boat that is reconstructed as a place to live in, the concrete ark is a housing that is constructed on a floating foundation. This system is relatively cheap and technically robust and does not need much maintenance. [Spruyt Arkenbouw, 2006; Fit, 2006; de Graaf, 2009]

At the end of the 20th century the number of houseboats in the Netherlands is estimated to a number of 10,000.

### ***Last decades; floating houses***

In the last decades floating houses made their introduction. Floating houses are houses which have the appearance of normal houses which are constructed on a floating foundation. They are in a way stationary, because they are made for a certain location and are fixed to their mooring post with a firm construction [Fit, 2006; de Graaf, 2009]. Floating houses originated in North America. In the early 1980's International Marine Floatation Systems Inc. (IMF) introduced a new technology of constructing real estate on water [www.floatingstructures.com]. This system is based on a core of polystyrene foam (EPS, expended poly styrene, see appendix for information) and a concrete shell. This system gives the possibility to build on water and results in less draught so it can be used in more shallow waters. On top of these advantages the system is also unsinkable. (More information is given in chapter 4 *Floating bodies*.)

This development contributed to the formation of large floating quarters in the cities Seattle and Vancouver. In these quarters one can find large floating houses with the same appearance as normal houses and villas. A large part of these large floating houses are built with the 'IMF-method' and another part is built on concrete caissons.



Figure 2: Floating quarter in construction (www.floatingstructures.com)

In the Netherlands the first project with 'floating houses' was realised in 1992. In Marina Oolderhuske (next to Roermond, province of Limburg) 80 recreational villas were built in a gravel mining lake linked and next to the river Maas. After a few years of quietness in the 'living on water branch', Marina Oolderhuske was followed up in Maasbommel.

In 1999 the first floating house for permanent habitation in the Netherlands was constructed by the company Ooms. This was also the first house in the Netherlands which was constructed according to the 'Canadian System', which equals the 'IMF-method'. [Van Osch, 1999, Fit 2006]



Figure 3: Towing of the first water house van Ooms (www.ooms.nl)

The insight that 'living on water' had good prospects to become a successful form of 'multiple space usage' (see 1.2) made various contractors interested. Also the ark builders professionalize and numerous architecture firms start designing floating buildings. From then on the floating homes that are designed and built, get an appearance like normal houses and have a more modern look. In Figure 4 and Figure 5 artist impressions from the leaflet from 2001 of Ooms are shown. These designs, were examples for the successors of 'The Lighthouse'. In the years after this multiple individual floating houses are constructed. See appendix 2 *Reference projects*.

To avoid the negative associations with the messy arks and house boats, from that moment onwards the contractors and municipalities start talking about 'waterwoning' (water house) or watervilla. [Fit, 2006]



Figure 4: Artist Impressions of the Leaflet of Dura Vermeer, made by Architectenbureau Sytze Visser

### ***Floating Houses in plans local governments***

With the new type of floating houses, the image of living on water improves a lot. These houses have nothing to do with the messy image, moreover, they have a very exclusive image. So with this development the local governments become much less sceptical about floating buildings. With the water problematics (see paragraph 1.2) in mind, floating becomes a hot item with the national and local governments. Numerous municipalities make plans to integrate a large amount of floating houses in their new to be built 'Vinex-wijken' (Dutch new housing estates) to give them more exclusivity. Amsterdam was the first municipality to construct a large amount of floating buildings bundled in a new 'Vinex-wijk'. In 2002 was started with the new quarter IJburg with approximately 2000 residences, fully constructed in the IJmeer (IJlake). In this quarter 185 floating houses have been realized [www.steigereiland.com; Graaf, 2009].



Figure 5: Impression Waterbuurt IJburg [www.waterhuis.nl]

### Overview past to near future

The just sketched development of the floating houses is depicted in Figure 6. In this figure the history of floating dwellings in the Netherlands is shown with pictures on a timeline. The line starts in the beginning with house boats and a low density, then continues with individual, loose floating homes. Then the line goes up to the present times with an increasing density and bigger houses, from there on to the 'quarters' with connected floating houses. The line is lengthened to the future, where floating cities are expected. [Graaf, 2009; Kuijper 2006] (See paragraph 1.2 and appendix 3).



Figure 6: From History to Future [Kuijper, 2006]

At this moment floating houses are still a small luxury market. However, there is a strong market demand that could contribute to the availability of floating structures to the middle and lower end of the market. This is driven by a trend toward larger scale projects (for example 'Het Nieuwe Water' ) in which the lower segments of the housing market should also have a place. To realise this, also apartment buildings will be realised in these projects.

Figure 7 shows the steady increase of the number of floating houses in the Netherlands. In this figure the amount of floating houses that have been realized in large projects are depicted. The houseboats and the few individual floating houses are not taken into account in this graph.

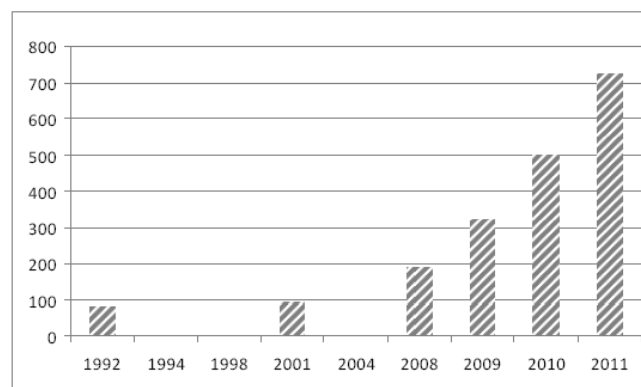


Figure 7: New floating houses in the Netherlands, including projections for 2009 to 2011 [Graaf, 2009, based on Czapiewka, 2008 and SEV, 2008]

In 1992 the first big project, Marina Oolderhuske, started. For the projects Den Ham en Maasbommel the year 2001 is taken. The amount of almost 200 floating houses in 2008 is from IJburg. The large amount of floating houses from the years 2009 to 2011 is coming from amongst others the project 'Het Nieuwe Water' in Naaldwijk, where 600 houses will be constructed. Municipalities of Almere and Den Bosch have also got advanced plans. The plans for 'Het Nieuwe Water' in Naaldwijk can be found in appendix 4. More examples of floating houses can be found in appendix 2 *Reference projects*.



### **1.1.2 Other floating structures**

Next to floating houses there are also floating structures with other purposes than residence purpose. Examples are floating utility buildings, floating infrastructure, structures for offshore industry and temporary floating structures for constructing other structures.

Floating bridges have been constructed for a long time. According to old Chinese books the first floating bridges have been built in ancient China by the Zhou Dynasty in the 11th century. Over the centuries, floating bridges, mostly small, have been built all over the world. Some history of floating bridges and other floating structures is given in appendix 1b. Examples from recent bridges, floating infrastructure and other floating structures are given in appendix 2 *Reference projects*.

## 1.2 Why Floating Structures

In the past people lived on water for mobility, safety and economic reasons. Nowadays, the main reason why people want to live on water, is because they like it for personal reasons: A floating house is attractive because of the space and view on the surroundings (see Figure 8), the living environment near the water and the feeling of freedom it gives. These motives followed from several research, amongst others from Fit in 2006, Heijmans in 2006, SEV in 2008.

But today there are more important motivations to build on water. Below the reasons for floating structures are listed:

### Reasons for floating structures:

- Personal reasons
- Climate change and water problematics
- Shortage of land
- Floating better than land reclamation
- Mobility of buildings
- Natural resources below sea level
- Exploiting new forms of Energy

For all in this paragraph stated information, more information can be found in appendix 2. Reasons 2 and 3 are two large problems where a large part of the world, and the Netherlands especially, has to cope with in the 21<sup>st</sup> century. These problems together can be a reason for building on water. Together they are one of the main reasons behind the strong increase of living on water in the municipal plans in the Netherlands.

This is explained in the next paragraphs.



Figure 8: View from a floating home in Maasbommel [Wilberink&Munster]

### **Climate change and water problematics**

The climate is changing, world wide it is getting warmer, weather changes all over the world and sea levels are rising (IPCC 2007, KNMI 2006). Worldwide this results in multiple problems; here the situation for the Netherlands is discussed, of course there are also other area's where similar problems arise.

Because of the climate change the Dutch summers will be hotter and dryer, but when it rains, the showers will be more severe. In the winter months it will rain more often, longer and more heavy. These points are part of the KNMI climate conclusions, which are based on the reports of the IPCC,



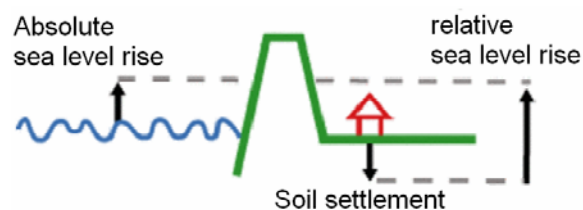
the most important KNMI climate conclusions are listed below. For more information and background see appendix 5b.

#### KNMI Climate Conclusions

- 1 to 6 degrees temperature raise ( 2 to 4 degrees is within bandwidth of 80%)
- 6 to 25 % more winter precipitation
- severe summer droughts but also 2 to 10 times higher chance at extreme summer showers.

The climate change also results in a rise of the sea level and changing river discharges, which will also contribute to waterproblematics (see appendix 5b). On top of this the Dutch soil is setting. All mentioned phenomena lead to, to put it simple, too much water at one moment and to a lack of water at another moment:

- The severe droughts in summer, small river discharges, salination and large water demand will result in water shortages.
- The high water levels and severe rains results in a surplus of water and results in flooding risks.



Figuur 9: Combination of sea level rise and soil settlement [KNMI, 2006]

#### **More space for water storage needed**

An answer to the mentioned water problems is water storage. In this way, the water can be stored when there is a surplus of water in the wet periods, and the stored water can be used in the dry periods.

In the policy and water management of the Netherlands water storage is getting more and more important. There has been a change in perception on water management. River flooding in the mid 90's and pluvial flooding at the end of 90's led to the establishment of a new government policy, 'dealing differently with water' (Tielrooij, 2000). Water retention increased in importance and there was a shift in approach from 'fighting the water' to 'living with water' [Graaf, 2009]. The 'Commissie waterbeheer 21st eeuw' (Committee Watermanagement 21st Century) [Tielrooij, 2000] concluded that it was not longer possible to suffice with technical measures as heightening the dikes and enlarging capacity of pumping stations. For a durable water policy more room for water is necessary. According to experts an area the size of the province of Utrecht. (Akker, 2004). In all recent Dutch water committees and spatial planning notes it is said that there needs to be more water storage and more space for surface water (see appendix 5b).

#### **Shortage of land: Multiple space usage**

In the Netherlands and also in other densely populated areas in the world, there is a shortage of land. The area-claims for living, working and recreation increase. When all land-claims are placed beside each other, the Netherlands appear too small, see appendix 5d. So this culminates in a conflict with the need of more space for water. A solution could be combining water storage with building. With multiple space usage, land surface is used for different functions at the same time. One form of multiple space usage is combining the water storage function with other functions. Dutch policy stimulates this multiple space usage, see paragraph 1.3.

#### **Floating most suitable form**

Combining water storage and building can be done in several ways. When much water storage is needed, or when an option is needed where the buildings can remain in a water surface for a longer time, floating buildings appear the best form of multiple space usage, see appendix 5d.

## 1.3 Floating Policy and Legislation

### 1.3.1 Policy

As said in the paragraph on history, the government was in the beginning not keen on floating homes. But this has changed. According to national policy more space for the water is needed (see appendix 5b). The national government has designated 15 locations for experimenting with multiple space usage. And several municipalities have adopted building on water. For example Naaldwijk and Rotterdam have major plans with floating structures (see appendix 4).

#### EMAB locations, 15 designated areas

The Ministry of Spatial Planning has designated 15 areas for multiple space usage in the floodplains of the rivers, the so called EMAB locations (Experimenting with Adjusted Buildings). [ministerie van Verkeer en Waterstaat, 2005]

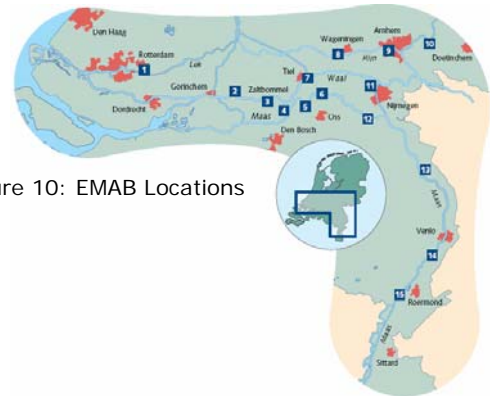


Figure 10: EMAB Locations

### 1.3.2 Legislation

For the new building form of floating buildings, the existing legislation in the Netherlands did not suffice. There was much ambiguity (onduidelijkheid) on the legal status of the new building form; is a floating house real estate or a movable good? It was also unclear if floating structures had to be reviewed and checked according to the 'Bouwbesluit' (Dutch building codes which have to be fulfilled with any building), as this resulted in several problems. New regulations about buoyancy, freeboard, tilt, stability, cultivable area on water, space underneath the structure, how to deal with fire, etc., were also needed. For this the Ministry of Housing, Spatial Planning and Environment (VROM) published the brochure *Drijvende woningen en de Bouwregelgeving. Handreiking voor ontwikkelaars, bouwers en gemeentelijke plandoelers.* (Floating homes and the Building Regulations. Guideline for developers, builders and local plan reviewers) in April 2009. This guideline has, at the time this thesis was written not any legal meaning yet, but as the title states, it can be used as a guideline. In the near future new legislation is expected.

In Canada, guidelines for floating structures have already been implemented.

#### **Real estate or movable goods**

In the past the legal status of floating structures was very unclear in the Netherlands. It was not clear if a floating house is real estate (onroerend goed) or a movable good (roerend goed). For people who want to live in a floating house this a very important question, because on movable good one cannot have a mortgage and more problems with insurances and higher insurance costs arise. So, this made the floating houses much less popular.

Another reason why this question is important is, that according to the Dutch law this classification decides if the floating structure is a 'bouwwerk in de zin van de Woningwet' (Building structure according to the Housing Act) or not. If it is a 'bouwwerk in de zin van de Woningwet' this means the 'Bouwbesluit', 'Gebruiksbesluit' and 'Gemeentelijke bouwverordening' (Dutch building and planning codes) are acting on the structure. If it is classified as 'woonark' (barge) these laws do not act on it.

#### **According to previous legislation**

In the past, the legal status of floating objects was regulated by the 'Wet op woonwagens en woonschepen' (law on caravans and houseboats) from 1918. Under this law, anything that floats and where is lived upon, is a houseboat. The 'Law on caravans and houseboats' was abolished in 1999 and the rules for placing and building a 'mobile home' were included in the Housing Act. Except for one article on mooring permits no federal regulations for floating homes were left. The government had the view that adequate regulations should be addressed to local governments. So no clear rules were developed.

Jurisprudence under the law on houseboats has been fully adopted by 'Article 88 of the Huisvestingwet'. So this legislation did not distinguish between types of houseboats, floating monuments, floating parks or new water housing. Also the external design, the architecture, did not matter according to this legislation. Everything that floats and where is lived upon, still had to be classified as a 'houseboat'.

However, during the last decade this issue was frequently judged differently. According to Stichting Urgenda (2009) court rulings showed no clear line at all. In similar cases floating objects were classified as well as real estate or movable goods. In one municipality they are assessed as real estate, in the other as "barge". The current projects are sold as floating real estate. [Stichting Urgenda (2009)]

#### **According to the guideline of 2009**

According to the guideline (VROM, 2009) the legal status can be chosen before the project. So Floating buildings can be as well real estate or movable good. But with this choice the jurisprudence has to be taken into account.

According to the guideline the numerous jurisprudence is clear about this case. In the guideline is written that 'De Raad van State' (Dutch judicial authority) applies every time to the definition of a 'bouwwerk' (building structure) according to the 'Model Bouwverordening'.

According to the 'De Raad van State' a floating structure is a 'bouwwerk' when there is "solid anchoring", whereby the structure can move only vertically and it is horizontally fixed, and the structure is intended to function permanently on one site and is also constructed in this way.

So, when one wants his floating structure to be accounted as real estate, it has to have a solid connection with the firm land and it has to be constructed for one place.

#### **Clarity in November 2009**

In November 2009 the minister of VROM, Mrs J.M. Cramer, proclaims that all floating structures which are meant to stay permanent, or long lasting, at a certain location, can be categorised as real estate [Cramer, 2009].

#### ***Verification structure according to Building Codes and Guideline VROM***

In the latter is said that when a floating structure is judged as real estate, it is judged as 'bouwwerk in de zin van de Woningwet' (Building structure according to the Housing Act). If it is a 'bouwwerk in de zin van de Woningwet' this means it has to fulfil the 'Bouwbesluit' (Dutch building codes).

Most elements of floating structures can be judged and checked the same way as buildings on firm land by the building codes. But there are also things which the building codes do not mention, like buoyancy, freeboard and space needed under the structure. Other parts of the building codes are not, or less applicable, as demands on tilt. The building codes should be used where ever they can be used. And when this is not the case the new regulations of the guideline VROM can be used, until there is new legislation. In any case the loads have to be applied according to the building codes (NEN 6702 or EC1991).

The regulation and verification for floating structures which is not covered by the Eurocode, is discussed in Appendix 6, Legislation and Verification. In this chapter the guideline from VROM is also extensively elaborated.

## 2. Basics of Floating

In this chapter the basics of floating will be discussed. The most important element with regard to floating structures is the buoyancy. The buoyant force and the weight define the draught and freeboard of the structure. These definitions will be explained. Buoyancy is a simple principle which is described by the principle of Archimedes.

Next to the buoyancy, tilt and stability of a floating structure are also very important. In this chapter is explained how a stable structure can be realised, what this means for the tilt and movements of the structure and how tilt can be countered.

### 2.1 Water pressure and buoyancy

#### Archimedes

An object thanks it's buoyancy on the hydrostatic water pressure. The principle of buoyancy can be described with Archimedes' principle see appendix 7. The Archimedes principle is depicted below.

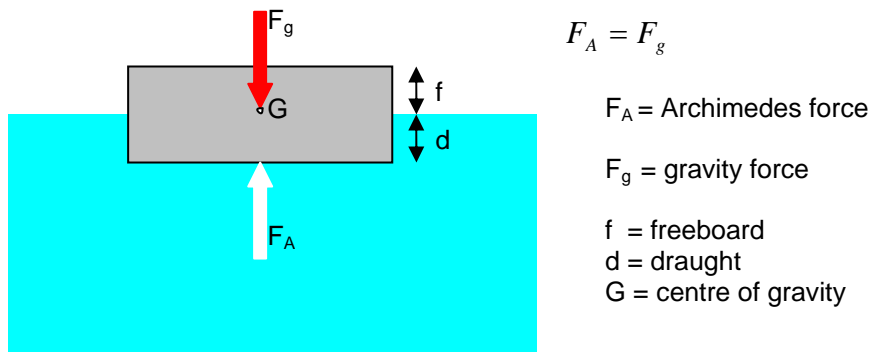


Figure 11: Principle of buoyancy

The real Archimedes formulas is given in appendix 7, but when for  $\rho g$  is taken a value of 10, this formula for the Archimedes force can be simplified to the following:

$$F_A \text{ (kN)} = 10 \cdot V$$

With:  $V$  = immersed volume

#### Draught

The draught of a floating structure is it's depth in the water (see Figure 11). In case of a floating body with a flat and level bottom plane and vertical walls (see also Figure 11) the draught is equal as the depth of it's bottom plane. In case of tilt, the term draught is used both for the average depth of the bottom of the structure, as well as for the largest depth of the floating body. In this thesis the difference is made clear by speaking about main draught and draught deepest point. Also just the word draught is used, than the meaning should be clear by the context.

With the given formula of Archimedes the main draught of a free floating body can be calculated. In case of a floating body with a flat and level bottom plane and vertical walls (see also Figure 11), the immersed volume of the floating body is the bottom surface times the main draught:

$$\text{Immersed Volume: } V = d \cdot A$$

So in this case this leads to the following simple formula for the draught:

$$\text{Draught: } d = \frac{F_g}{10 \cdot A}$$

With:  $d$  = (main)draught (m)  
 $V$  = Immersed volume ( $\text{m}^3$ )  
 $F_g$  = gravity force (kN)  
 $A$  = surface of bottom plane ( $\text{m}^2$ )

### Freeboard

The freeboard is equal to the height of the floating body minus the draught.

$$\text{Freeboard: } f = h - d$$

With:  $f$  = freeboard (m)  
 $h$  = height floating body (m)  
 $d$  = draught (m)

### Hydrostatic pressure

As said, the Archimedes principle makes use of the principle of hydrostatic pressure. The hydrostatic pressure gives pressure in every direction which increases with the width.

In a fluid at rest the pressure is isotropic, this means it acts with equal magnitude in all directions.

However, in a flowing fluid this doesn't hold. But the differences are small when the water isn't flowing very fast, so the hydrostatic pressure will also give a good approach in relative slow flowing fluids. Since floating structures are mainly situated in relative calm (slow flowing) water, the formula for hydrostatics will be used.

The principle of hydrostatics and it's resulting forces are discussed in appendix 7.

### Schematising water as elastic foundation

A floating structure supported by water can be seen as a structure on a base plate on an elastic foundation with water as continuous elastic foundation. The dimensioning and verification of the structure can be done using this schematisation. See appendix 7.3

### Calculating deformations and internal forces

The deformations, internal forces and moments of a beam on a elastic foundation can be calculated with the differential equations known from mechanics. But if the floating structure can be schematized as completely rigid, than the elastic support can be schematized as a linear load. This way the laborious differential equations have not to be used. The floating structure can be schematized as completely rigid if the structure itself has a high rigidity, for example by a large structural height, and if it has a small length compared to the height. The remark will be made that a structure on water can be schematized a lot sooner as rigid than a structure on another foundation, because of the low k-value of the water and the fact that water can't have internal friction.

## 2.2 Tilt and stability

### 2.2.1 Rotation/tilt

If an eccentric vertical load, a horizontal load, or an moment is acting on a floating structure this will cause rotation around the centre of buoyancy leading to a certain amount of tilt (scheefstand) of the floating structure.

A rotation will lead to one part of the floating structure getting more deep into the water and the other part less deep. According to the hydrostatics the deeper sunken part get more water pressure (see the figure below) and this way a higher buoyant force, so this way a righting moment will arise. This righting moment can bring the structure in equilibrium again.

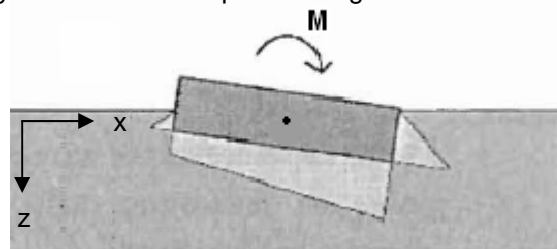


Figure 12: Rotated floating structure [van Winkelen, adjusted]

The amount of tilt, due to an static eccentric load or moment, can be calculated by calculating which rotation is needed to reach the righting moment which will equal the acting moment. This can be done by applying the hydrostatics. This is explained in appendix 8.

Clearly the draught of the deepest point and the freeboard change in case of rotation. Due to tilt the draught will increase. On the deep side the freeboard will get lower and on the high side higher. If these height changes are relevant they can be easily calculated by summing the initial values and the height changes due to rotation. In appendix 8 is given an example which illustrates this.

## 2.2.2 Stability

Stability is very important for floating structures. If the righting moment caused by water pressure (also called the hydrostatic restoring force) can bring the floating structure in equilibrium, and back to the original position after the imposed load is taken away, the structure is called stable.

Stability is about the following:

When a body in equilibrium is brought out of its initial equilibrium position by introducing a force or moment on it, and afterwards it is taken away again, three things can happen:

- the body will not return to its initial equilibrium position: unstable
- the body will return to its initial equilibrium position: stable
- the body will find a new equilibrium position: neutrally stable

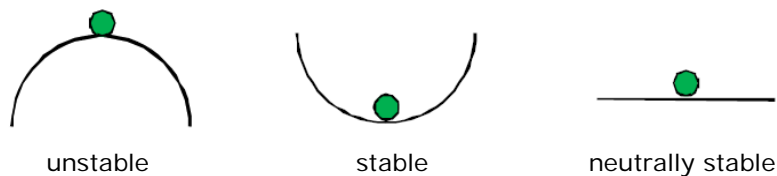


Figure 13: Principle of stability

An unstable floating structure will tilt, so a floating structure needs to be stable.

If a floating body rotates less than another body under influence of the same load, the structure is called more stable.

The stability of a floating structure is defined by the height between its centre of gravity and its metacentre; the metacentric height (GM). This is explained in appendix 8

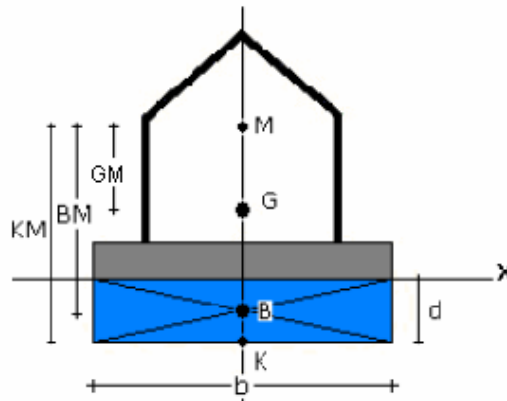


Figure 14: Metacentric height [Kuijper, 2006]

### Conclusions metacentric height

A large metacentric height means large stability. So from view of stability, first the point of gravity should be as low as possible (small KG), this is called weight stability. Second the metacentre should be as high as possible (large KM), this has to do with the shape of the floating body and so this is called shape stability.

### Weight stability

The lower the centre of gravity, the more stable the floating structure. If the centre of gravity is located lower than the hinge where the floating body tilts around, then the structure will always, like a tumbler, raise itself. This is also the way the principle of the keel of a sailboat functions.

Weight stability will only be effective with large rotations. The larger the rotation, the larger the uplifting moment. This means the stability increases with the rotation.

### Shape stability

The base of shape stability lies in the shifting of the centre of buoyancy, when the structure rotates. The more the centre of buoyancy shifts and can shift with a certain rotation, the higher the metacentre will be, so the more stable the floating structure will be.

A floating body with a in vertical direction rectangular section (as depicted in figure 11, 12 and 14) is clearly more stable than other shapes, like a cylindrical or triangular shape [Winkelen, 2007]. This can be explained by the following: a floating body with a rectangular vertical section has to displace the most water with a rotation. (For ships to much stability has disadvantages, that's one of the reasons ships have got a more round shape). For rectangular floating bodies the formula for the metacentric height has been given in appendix 8. From this formula can be seen that the width to depth ratio is decisive. The width of the floating bodies works quadratic in this formula, so with enlarging the width, the stability increases a lot. So, a wide body with a small draught gives the best stability.

This makes perfect sense, because with a larger width, the centre of buoyancy can shift more to the sides. With a wide, rigid, floating body the centre of buoyancy already shifts largely with a small rotation. So a wide body has a high initial stability. However, when the rotation becomes large the distance between G and M becomes smaller, so the uplifting moment becomes smaller, but with normal small rotation this can be neglected.

### Shape is decisive factor

A parameter study from Kuijper in 2006, with common values for floating bodies for low rise buildings, e.g. small draught and small occurring rotations (less than 10 degrees), shows that up to a width till 6 metres the position of the center of gravity is the determining factor for stability. Width a width from circa 9 metres, the position of the centre of gravity has little effect on the stability. With a width larger than 9 metres the width is very clear the decisive factor for the stability.

### High-rise is possible

With a high floating structure, like an apartment building on a floating body, the centre of gravity of the structure will go up, and the draught of the float easy becomes high. Both could be a problem for stability. But when the floating body is given a large enough width, also high-rise is possible [Winkelen, 2007].

## 2.2.3 Preventing tilt

Tilt can be caused by temporary loads and waves or by an permanent eccentric load, for example because there are more structures, or more heavy structures on one side.

Tilt by permanent loads can be countered with permanent ballast weight on the opposite side or changing the shape of the floating body. This changing of the shape could for example be done by increasing the height of the floating body (at the bottom) at the side where it is more heavily loaded, so this side will gain more buoyancy.

Tilt by not permanent loads can be countered with a large stability, as elaborated, this can be realised with a floating body with a vertical rectangular section and a large width with a small draught. Other ways to prevent tilt by non permanent loads are the following:

- movable ballast, for example with ballast tanks or water cellars.
- movable buoyancy, for example by 'balloons' or 'bags' with air under the floating body.

## 2.2.4 Dynamic stability

Next to static stability also the dynamic stability is important.

If a floating structure is positioned in waves or swell, the element can start to sway. If the frequency of the load is unfavourable, the sway can increase and increase, which leads to large movements [Baars et al., 2009]. Large movements are undesirable and could lead to serious damage. In order to prevent this, the natural oscillation period (eigenperiod) of the element has to be significantly larger than that of the waves or swell. The dynamic movement and dynamic stability is discussed in next paragraphs.

## 2.3 Movement Floating Structures

### 2.3.1 Six different movements

A completely free floating body has got six degrees of freedom. On the basis of the axis shown in Figure 15 the movements can be defined. The naval architecture defines the movements as follows [Journée, 2001]:

#### Translations

- In x-direction; *surge*
- In y-direction; *sway*
- In z-direction; *heave*

#### Rotations

- Around the x-axis; *roll*
- Around the y-axis; *pitch*
- Around the z-axis; *yaw*

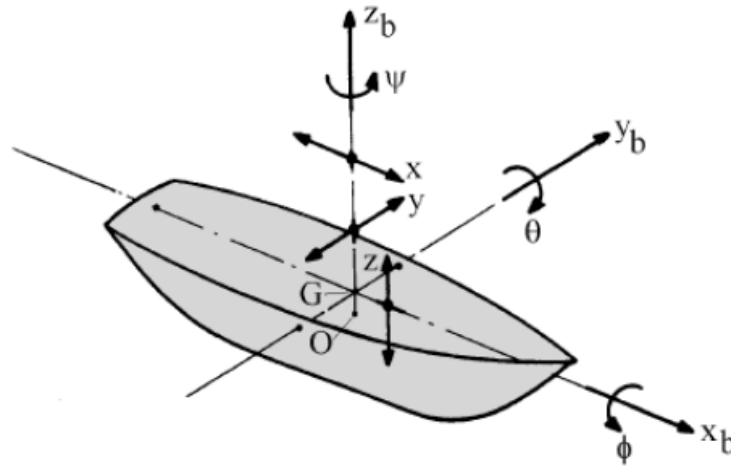


Figure 15: Movements [Journée, 2001]

A floating structure will tend to make the depicted movements if a load acts on it. The degrees of freedom will be less if some of the movements are prevented by mooring. Normally surge, sway and yaw will be prevented by the moorings. If some movements are prevented by mooring, of course these connections and moorings have to be calculated on the loads induced by prevented movement.

#### **Surge and sway**

Mostly the translation of floating structures in x- and y-direction, the surge and the sway, are prevented by the moorings. At least, this is the case when the floating structures are moored with the standard mooring construction on mooring poles. (for mooring options see chapter 10)

In case of mooring with tension cables to the bottom, a certain surge and sway will be possible.

#### **Heave**

Principally, floating structures can move freely in direction of the z-axis. \*

The buoyancy and the vertical load (including own weight) determine the height with respect to the water level (see 2.1). This draught can change if the vertical load changes. Free floating structures will follow the height change of the water level. So for these structures the movement in z-direction can be quite large in for example high tidal movement.

Waves can also put a floating structure in vertical motion. This will result in a heave oscillation.

(Also other dynamic loads can in theory put the floating structure in vertical heave motion, but normally this is less likely to happen)

#### **Dynamic motion: oscillation**

The dynamic heave motion of a floating structure will be a form of oscillation. The most important parameter for the oscillation, is the natural frequency of the floating structure. This is explained in appendix 9. In this appendix is also taken a look at the amount of oscillation and what influences it. There is also taken a quick look at the size of these movements.

To prevent resonance, the natural period should be significantly larger than the natural period of the waves. As can be seen from the resulting formulas of appendix 9, a large natural period can be reached with a large draught and width.

---

\* In this thesis floating structures which can move freely in z-direction are called free floating structures. There can also be chosen for completely fixed floating structures, which can not move in vertical direction. According to the definition from Wang, given in the introduction, these can still be accounted as floating structures. But in this thesis only the free floating structures are considered.



**Roll and pitch**

Roll and pitch are in fact the same, only pitch is rotating in the longest direction and roll is rotating in the less wide direction. So the way of calculating and the formulas are for both movements the same, only the width and the length has to be interchanged.

If a floating structure has a static rotation, this is called tilt in this thesis. This tilt can be caused by static eccentric loads or moments. This was discussed in 2.2.

Roll and pitch will be caused by dynamic eccentric loads and moments, and by waves.

These dynamic loads cause the dynamic roll and pitch movement of the floating structure.

**Dynamic roll and pitch movement**

The dynamic roll and pitch movement can be derived in the same way as the heave motion, see appendix 9.

It appears that the natural period depends mainly on the metacentric height. As stated in 2.2, from certain dimensions on, for normal floating structures the metacentric height depends mostly on the width. So, it might sound contradictory, but it can be concluded that large structures will have a low natural period for roll and pitch movement.

So, increasing dimensions result in shorter natural oscillation periods, but large dimensions will also result in less rotating of the floating structure. So large floating structure will oscillate fast with little movement.

The swell of a floating structure will be worst if the length and width of the structure are smaller than half of the wavelength.

**Yaw**

If the floating structure is moored on more than one mooring pole, these moorings will prevent the rotation around the z-axis.

**2.3.2 Allowable Movement**

Surge, sway and yaw will be, as stated before, usually be prevented.

Any amount of heave is allowable as long as the moorings allow them, usually this means the mooring piles have to be high enough. Heave oscillation is undesirable, from a comfort point of view, but there are no limitative criteria. Usually heave will not lead to damage, as long the aquifer openings or deck will not disappear in the water by the heave oscillation. The same accounts for roll and pitch.

These boundaries are discussed more elaborately in appendix 6 *Verification and Legislation*.

## 3. Loads on Floating Structures

In paragraph 1.3 it was mentioned that if a floating structure is classified as real estate, the loads that act on it, have to be determined according to the building codes (NEN 6702 or EC1991). All relevant loads which can act on floating structures are mentioned in this chapter.

Some loads have to be treated differently for floating structures than for landbased structures and there are also some loads which do not act on land based structures but do act on floating structures. Especially these two latter cases will be mentioned in this chapter. Loading by waves is a special load case, which is not mentioned in the building codes, and which can be very important for floating structures. Waves will be elaborated far more extensively in appendix 10.

### 3.1 Load factors

Load factors are used to bring the probabilistic characteristics of the structure and loads into account. Load factors depend on the chosen design working life and reliability class. According to the Eurocode loads have to be multiplied with load factors bigger than 1,0 when the structural safety is checked (ULS). In the serviceability state (SLS), for calculating the deformations, the load factors equal 1,0. But for floating structures this gives a problem, because large movements (sag and tilt) can occur which could lead to structural failure. But simply using the load factors from the ULS on these deformations is not the answer.

Using the ULS load factors on the self weight and imposed loads, or using the SLS load factors, results in large differences in freeboard and draught. Using the ULS load factors from the Eurocodes for calculating the buoyancy, draught and freeboard, will result in a very high necessary freeboard in the normal situation. How the load factors will be used is as follows:

- Calculating freeboard and draught will be done with all load factors 1,0 (this is seen as SLS)
- Load factors larger than 1,0 will only be used for failure. So, this means load factors will be used for determining structural failure of elements (as usual) and for determining if the floating structure will stay buoyant in the normative situation (this is seen as ULS).

In appendix 6, the verification and check of floating structures is discussed. In that appendix chapter is also stated what the limits of freeboard etc. are, and when what load with which factors has to be used.

### 3.2 Self weight and imposed loads

Self weight and imposed loads have to be determined according to the building codes (NEN 6702 or EC1991). The self weight and the imposed loads mainly determine the height of freeboard and draught and the buoyancy.

#### ***Imposed loads***

Imposed loads are an important factor for draught and tilt. So, with a modular floating structure, imposed loads also play an important role for the forces and movements in the connections.

For the imposed loads the specific use categories with their corresponding surface loads from Eurocode 1991-1-1 will be used. A table with the several specific use categories and their corresponding surface loads is attached in appendix 22.

#### **Reduction of imposed loads**

If the given surface loads will be used over the whole surface, or over a large part of the surface of the floating structure, this has got large consequences for draught and tilt. To forestall that the imposed load has got a disproportionate effect on the height of the needed freeboard, reduction factors will be used.

This will be done as follows: first the total maximum imposed load is examined and subsequently the amount of surface occupied by this load, if the surface load of the Eurocode is used. For example, if a floating structure with a congregate purpose has been given a limit to the amount of visitors, it may not be necessary to calculate the full surface load over the full surface, because this load can simply not be reached by the limited amount of visitors. Then a surface will be calculated which can be fully loaded by the limited amount of load. Over the rest of the surface there will be used the by the Eurocode given load, times a reduction factor. The reduction factors that will be used are the reduction factors given by Eurocode 1991-1-1. For an example of the just explained, see the case study Floating Pavilion.

### 3.3 Wind and snow

The wind and snow load on a floating structure will be determined just as described in Eurocode 1991. Wind on large water surfaces can be stronger than wind on land. In this thesis this is taken into account by taking the area around the floating structure as flat open country terrain which results in the highest wind pressure. Extra strong winds are in this thesis not taken into account, because most floating structures are situated inlands. But if the floating structure is situated at seas, an extra strong wind should be considered.

The wind is also the main cause of waves (see next paragraph). For both calculation the wind speed at a height of 10 meters is used (as starting point of the calculation. For structures the peak wind speed is most important, while for wind waves the average wind speed during a longer amount of time during a storm is important. So this may result in that for the wind, as cause for waves, may be taken a lower wind speed. However, for a short fetch the wind speed for calculating waves becomes closer to the peak velocity.

In practice the building codes uses the 10 minute mean wind velocity with an annual risk of being exceeded of 0.02 [Eurocode EN 1991-1-4]. So, if for the normative wave a longer lasting wind is needed, the wind speed that is used for calculating this normative wave can be lowered.

Snowload has also to be taken into account for calculating the buoyancy in extreme situations. Load factors have to be used in the same way as done for imposed loads and self weight.

For calculating the tilt of a floating body wind load could play an important role. When the tilt could lead to subsidence of the structure than load factors should be used, otherwise this is not necessary.

### 3.4 Waves

Waves are an important load case for floating structures. Waves will result in forces in horizontal and vertical direction and in movement of the floating structure. Wave can also cause large moment in the floating structure, this is depicted in Figure 16.

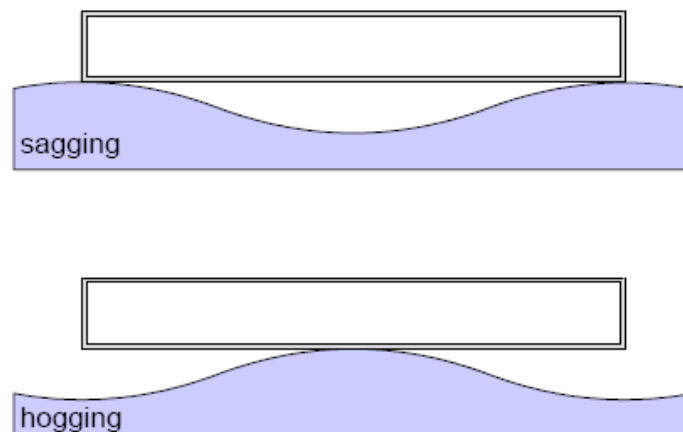


Figure 16: Wave causing sagging and hogging moment [Groenendijk, 2007]

In hydraulic engineering one can distinguish multiple types of waves. Tidal waves will result in large vertical movement of the floating structure (if allowed). As load case, the waves caused by wind and by boats are the most important. In most cases waves generated by wind will be normative. Waves by wind depend on the strike length (fetch), the wind velocity and the depth of the water.

When wind waves approach the coast, a number of changes occur, caused by the change of the water depth. Obstacles also influence the waves by amongst others diffraction and reflection. Information about waves, wave properties and how these are influenced can be found in appendix 10. In this appendix is also explained in what forces waves will result and how these forces can be calculated.

### 3.5 Current and Drag

If floating structures are placed into a flowing water, forces by current and drag have to be taken into account. This thesis focuses mainly on floating structures with a living or utility purpose and these structures are mostly placed in water with no real or small current. So in this thesis current and drag is not discussed, but if the floating structures will be placed in a floating water, this has to be considered. When for calculating the horizontal force by waves the rule of thumb is used, this mostly gives enough reserve to also include a small amount of drag.

With the Flexbase system (see appendix 12) the current and drag are more important, because this can cause EPS to come loose, since the bottom layer of EPS are not fixed within the concrete. This drag might become large when the structure is transported by towing, thus then this has to be taken into account.

### 3.6 Incidental Loads

#### ***Collision***

If the floating structure is situated in a water surface where ships navigate, a risk on collision may occur.

According to VROM, 2009 the water manager (Rijkswaterstaat, Waterschap) has to ensure that no big ships will be in the water of the floating structures, or there should be built good fenders around the floating structures or there should be taken other measurements to avoid a collision between big ships and floating structures (See also appendix 6. Legislation)

Collision is for connections not of much importance. When a major collision happens, this will result in large damage on the floating structure anyhow. If a connection will then succumb also, this is of less importance, because this won't result in increasing damage. (At least, this is the case if the superstructure than can and will be split up above the connection).

In case of collision with a big ship, failure of a connection can even be advantageous because then the collided part of the structure will be able to brake loose from the rest of the structure and this way it can move along with colliding ship. When a colliding part is able to move along, the impact, and this way also the damage, will be smaller.

So floating structures and connections will not be checked on collision in this thesis.

#### ***Ice***

When the water surface in which the floating structure is situated gets frozen, the expanding ice layer will introduce compressive forces on the floating body. This is why it is often seen that ice around houseboats gets cut away, to prevent leakage. If the water surface where the floating structure is situated in, can get frozen, this load case should be taken in account.

### 3.7 Loads caused by movements

If movements are prevented by mooring, the connections and moorings have to be calculated on the loads induced by the prevented movement. When the loads due to this movements have a dynamic character, the loads have to be multiplied. with a dynamic amplification factor.

## 4. Floating Bodies

As sketched in the chapter 1 *Introduction floating structures* and in the appendix *Reference projects*, in the last two decades, experience has been gained with multiple options for floating structures. In the offshore industry there is experience with large floating structures, mainly consisting of steel, in severe environments. For urban functions the floating bodies are mostly smaller and other materials are used. In paragraph 4.1 multiple possibilities for floating bodies are given and the bodies which are interesting for urban functions in shallow water, their advantages and disadvantages are mentioned. In paragraph 4.2 the shape of the floating body is discussed. In paragraph 4.3 is said when what options should be chosen.

### 4.1 Options floating bodies

The multiple options for floating bodies are listed below:

1. Concrete caisson
2. Concrete 'tray'
3. Steel floating bodies
4. Concrete-EPS bodies
5. Composite floating bodies
6. Natural materials
7. Pneumatic Stabilizing platform
8. Semi-Submerged
9. Tension-Leg structures

Below, the different floating bodies which are interesting for urban functions in shallow water are explained and elaborated. For floating systems which can be used in deep water this is done in appendix 12. Some examples are given and advantages and disadvantages are mentioned. (The given list is not completely complete, one could think of other possibilities of combinations, but the most used possibilities and the possibilities with the most potential are discussed.)

#### 4.1.1 Concrete Caisson

The name 'caisson' is French and has to be translated as 'large chest', which refers to the general shape of caissons. In civil engineering a caisson could be designated as a retaining watertight case or box. Floating caissons are used already for a long time in civil engineering, for constructing bridges, piers, jetties and last decades also for tunnels. The caissons are transported while they are afloat and when they arrive at the right location they are immersed. [Voorendt M.Z., et al. 2009]

Since the twentieth century concrete caissons are used as foundations for floating structures. In 1922 the concrete caisson made its introduction as foundation for a floating house and since then it is by far the most used foundation for floating structures.

There can be made a distinction between the standard caisson and the pneumatic caisson.

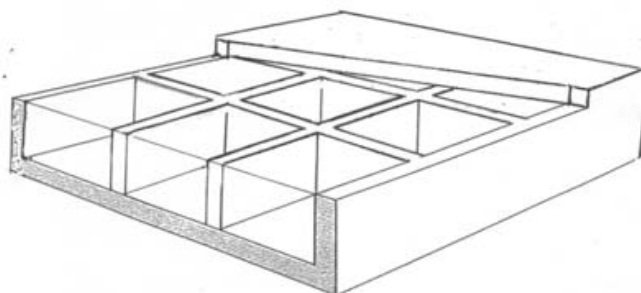


Figure 17: Standard concrete caisson

### Standard Caisson

The standard concrete caisson is a closed concrete box with concrete walls, bottom and top. Larger caissons have also got concrete inner walls. This has two reasons:

- decreasing the spans
- partitioning for safety in case of leakage

The enclosed air compartments give the concrete caisson it's buoyancy. On top of the floating body the structure can be placed, but the internal space can also be used.

The concrete caisson with it's thick bottom and walls results in a rather massive system, this results in high self weight, which is beneficial for weight stability, but also results in a large draught and small buoyant capacity, which strongly limits the weight of the superstructure.

The standard caisson is by far the most used floating foundation for floating houses and floating utility buildings. The floating houses in IJburg, the floating detention center in Zaandam, and the floating swimming pool in Paris, which were mentioned in chapter 1 and appendix *Reference projects*, are examples of floating structures on a concrete caisson.

#### Advantages:

- much experience
- large weight stability
- internal space
- relatively cheap
- high durability/low maintenance

#### Disadvantages:

- little buoyant capacity
- large draught
- sinkable

### Pneumatic Caisson

The difference between the standard caisson and the pneumatic caisson, is that the pneumatic concrete has no bottom. Now the buoyancy must come from the enclosed air between water and concrete top.

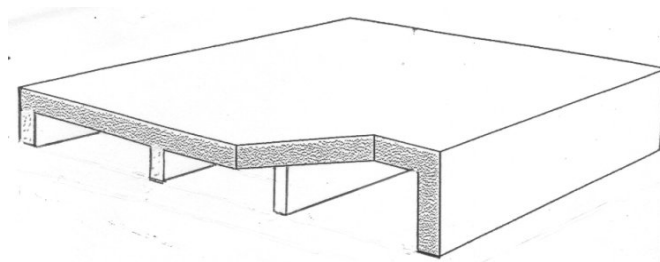


Figure 18: Pneumatic concrete caisson

Usually the air pressure is enlarged by high pressure air pumps. This system will fail if the enclosed air can escape, so airtightness is very important. This floating system is in fact not suitable as floating body for floating structures, since it has a very low buoyancy and it is a somewhat risky system.

### 4.1.2 Concrete 'tray' / Open caisson

A rectangular concrete tray is in fact the same as the standard caisson, but then without walls and top, see Figure 19.

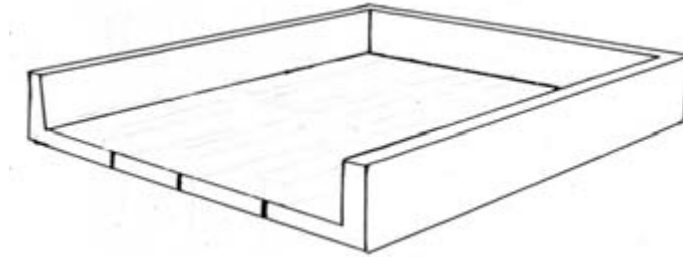


Figure 19: Rectangular tray / open caisson

In case of a large floating body, there can be built in the 'tray'. With smaller structures such as floating homes, there can be built on top of the walls. This is the system which is used for all standard 'woonarken' (barges). In Figure 20 and 5 a barge in production and a finished barge are depicted.



Figure 20: Concrete Casco [Arkenbouw Post]



Figure 21: 'Woonark' [www.remax.nl]

Also other shapes than a rectangular vertical section can be used (Figure 22), but this is less practical during construction, and in fact does bring hardly any advantages. Rectangular is also best for stability, as was stated in chapter 2.

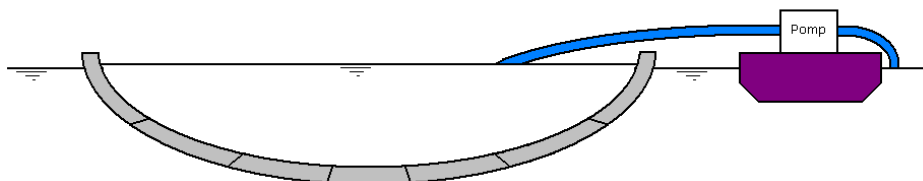


Figure 23: Round concrete 'tray' [Kuijper, 2006]

For the rectangular shape the same advantages and disadvantages hold as given for the concrete caisson.

### 4.1.3 Steel floating bodies

Steel is the most used material in offshore industry and ships. Floating bodies made of steel can have any shape. Rectangular steel pontoons are often used for temporary use, such as for maintenance of bridges, an example is given in Figure 24.



Figure 24: Crane on steel caissons [ww.Flexifloat.com]

With steel also other shapes are possible. In Figure 25 a floating house from Herzberger is depicted. This house is based on a hexagon of six large steel tubes welded to each other.



Figure 25: Floating house from Herzberger, Middelburg [www.woonen.nl]

Steel floating bodies can have small wall thicknesses, which results in a small self weight, which gives high buoyancy. This also results in small weight stability, but this can be simply counteracted by adding ballast weight, and as said weight stability is of less importance for large structures.

The big disadvantage from steel is that it is susceptible for corrosion, so it needs a lot of maintenance. This is the reason why the steel option is often crossed off. This was also the case for the floating pavilion.

#### Advantages:

- much experience
- internal space
- low self weight, high buoyant capacity
- small draught
- different shapes easily possible

#### Disadvantages:

- high maintenance
- relative expensive
- sinkable
- no insulation

Aluminium is another possibility, which is less susceptible of corrosion and it is even lighter, but also more expensive.

Subsidence can be prevented with filling the steel/aluminium bodies with EPS. This is done for the floating aluminium elements of the road in Hedel, see paragraph 1.3.1.



#### 4.1.4 EPS and Concrete

In the early 1980's International Marine Floatation Systems Inc. (IMF) introduced a new technology of constructing real estate on water by making use of the very light polystyrene foam (EPS, expanded poly styrene, see appendix 11 for information). The big advantage of this system is that no expensive dock or assembly hall is needed. This system is based on a core of EPS and a concrete shell. The system thanks it's buoyancy on the EPS, with a density of only  $20\text{kg/m}^3$ , which is 50 timer lighter than water. The concrete has a purpose for strength, stiffness and protection of the EPS.

The IMF system (see appendix 12) made construction on water possible. When the construction is finished using this method, it results in a reversed concrete tray which is in fact a pneumatic caisson (see Figure 18), but now the space is filled with EPS instead of air. Filling this space with EPS results in a lot more buoyancy and so in less draught. And on top of this, the structure becomes unsinkable, because there were there is EPS, there can be no water, so there are no possibilities of water accumulation.

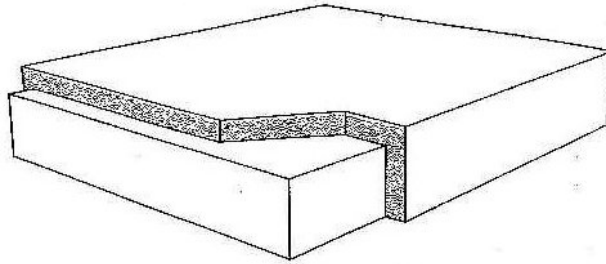


Figure 26: Reversed tray filled with EPS

#### Low self weight and draught

The EPS-concrete system also gives a much lighter structure than the standard caisson system, because far less concrete is necessary. This is because the EPS system does not need a concrete floor, unlike . the standard caisson system. On top of this, the walls can be a lot thinner too, because an eventual leakage will not result in water accumulation and sinking and there are no partition walls needed. The EPS supports the floor directly, so ,, the floor can also be thinner and by the direct support of EPS, less beams or inner walls for strength will be needed.

These savings on concrete results in a low self weight and draught. With this system a water depth of 1,5m can be enough [Kuijper (2006), Graaf (2009)]

#### Becoming popular and developments

Over the last decades the EPS-concrete system has become a popular system, which has already been used often. In the floating quarters in the United States and Canada a large part of the floating houses have the EPS system as foundation. A few examples of floating structures with this system were mentioned in appendix 2 *Reference projects*.

The relatively new EPS-concrete method has made some large developments last years. There have been made multiple new designs, which are based on the combination EPS and concrete. These systems are elaborated in appendix 14. The construction methods for the different systems is elaborated in appendix 15, since for the construction of the EPS-concrete system there are multiple different construction methods. An example of a recent design for this system is given in Figure 27.

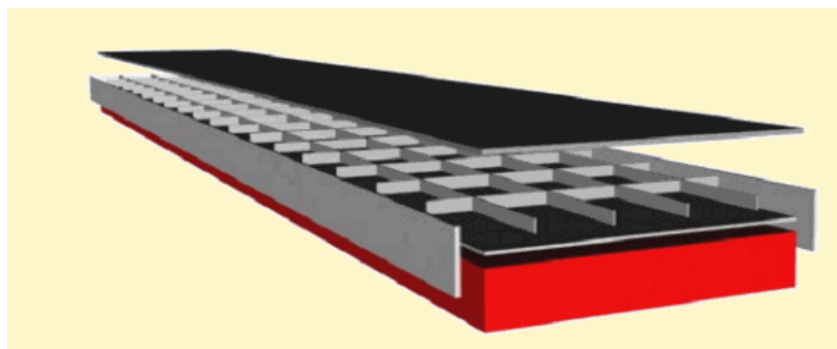


Figure 27: Floating body by Aquastrenda

### **Costs**

When the buildings are constructed on water an expensive dock or assembly hall is not necessary. Some systems make more efficient ways of constructing possible, for example if the EPS can be used as formwork.

Using EPS will mean extra costs. For a floating body of 100m<sup>2</sup>, with an EPS layer of 1 meter high, 100m<sup>3</sup> EPS is needed. At a price of 60 euro's per m<sup>3</sup> (see appendix EPS) this will cost approximately 6.000 euro's. These extra costs can completely or partly be recovered by using less concrete, by more efficient constructing and by saving on costs for a construction site.

### **Ad- and disadvantages**

For the multiple options from the EPS and concrete system the following advantages and disadvantages hold:

#### Advantages:

- unsinkable
- low selfweight, high buoyant capacity
- small draught
- high durability/low maintenance
- construction on water possible
- different shapes relative easy possible
- works insulating

#### Disadvantages:

- no internal space

In appendix 14 several different concrete-EPS are described.

## **4.1.5 Composites**

Composites can also be a very interesting material for floating bodies. The new composite materials are very strong and light, and can be shaped in any form. So floating bodies constructed out of composites can be very light.

Composites are currently also used for constructing bridges and therefore they have already proven them selves as construction material. However they have not yet been used as foundation for large floating structures yet.

The big disadvantage of composites is that they are still very expensive at the moment. The expectation is that they will become cheaper in the future. Now, the high costs is the main argument to cross of the composite alternative for this thesis, but with the remark that it still could be very interesting.

## **4.1.6 Air Cushions**

For floating there can be made use of air cushions. Generally these air cushions will be made of plastics. The great advantage of air cushions is their flexible buoyancy, but at the same time this is also a disadvantage. The lack of having a own shape of their own will make them less reliable, and the risk of leakage is also higher with cushions systems and the consequences will be more critical.

## **4.1.7 Natural materials**

Just as in the old days natural products as wood and reeds can be used. But these materials have a short durability in water, so in this thesis these are not regarded as options.

## 4.2 Shape Floating Bodies

As said in chapter 2, a rectangular shape (see Figure 28) is best from view point of stability. Most of the time this also holds from the viewpoint of execution, because a flat bottom is far more simple, especially in case of the concrete floating bodies, as no difficult formwork has to be used then. So for inland waters this shape should be opted for.

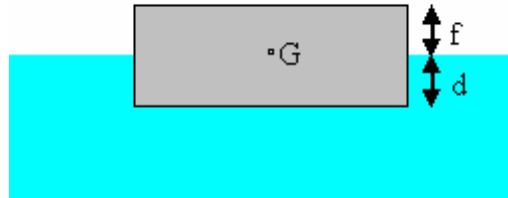


Figure 28: Floating structure with rectangular horizontal section

In case of severe wave conditions, other shapes can turn out to be more favourable. Other shapes, such as the semi-submerged floating body (see appendix 12), have less surface at water surface, the spot where the waves will influence the structure most and therefore they are less influenced by waves.

## 4.3 When to choose what option

For the use of floating structure a distinction can be made between situations:

- possible severe wave conditions; floating structures at deep seas and at oceans
- no occurrence of large waves; floating structures in inland waters or protected by wave attenuators

This thesis is aimed especially at the second situation.

### ***For severe wave conditions***

For the first mentioned situation, with possible severe wave conditions, the pneumatic stabilizing platform, the semi-submerged structure and the tension leg structure are good options, since these structures are less susceptible for waves and will behave well in severe wave conditions. These three options are not suitable for shallow innerwaters, so they can be disregarded for the second situation.

### ***For shallow innerwater***

For the second situation, floating structures situated in inland waters, four options are left:

- Concrete caisson
- Concrete 'tray' / open caisson
- Steel floating bodies
- Concrete-EPS bodies

The other options have been eradicated already .

The characteristics are given in the table below:

	Concrete	Steel	Concrete+EPS
Durability and maintenance	++	--	++
Draught	--	++	+
Safety (sinkable or not)	--	-	++
Costs	+	-	+
Internal Space	+	++	--
Weight Stability (only applicable for small structures <9m)	++	+	-

Table 1: Characteristics floating systems

As mentioned, steel has the big disadvantage that it is susceptible for corrosion and therefore maintenance will be needed. This is the reason why this option is often crossed of. This was done in the Business Case Drijvend Paviljoen (2009) and in the theses of Ling (2001), Kuijper (2006) and van Winkelen (2007). The corrosion/maintenance argument is also the main argument why almost all floating bodies of floating houses of last decades are made of concrete.

After eradicating the steel option, in fact only two options are left: the concrete option and the EPS-concrete system. In paragraph 4.1 were mentioned multiple concrete systems, but as described in 4.1, the pneumatic caisson is less interesting as floating foundation for houses or buildings, and the 'tray' system is for small bodies very similar to the standard caisson, so these systems are now treated as the same system, just called a concrete floating body. It should be mentioned that for a large body innerwalls are necessary for partitioning and limiting of spans. So now the choice has to be made between a full concrete system and the EPS-concrete system.

#### **Choice between full concrete and EPS-concrete**

From chapter 2 followed that for small floating bodies with dimensions below 9 metres, weight stability was important for stability. Therefore for floating bodies smaller than 9 meters should be chosen for a full concrete system, unless being unsinkable should be considered more important.

Also when the space in the floating body can be made useful and when this is believed to be vital important, the full concrete option should be chosen. This space could also be used for water cellars, which can be very useful in preventing tilt by not permanent loads.

When a small draught is needed, the EPS-concrete system should be chosen. Should being unsinkable be regarded as more important than internal space in the floating body the EPS system is the designated choice.

The just given situations or wishes and resulting choices are also given in Table 2.

In the theses of Ling, Rijcken, den Vijver, Kuijper and AquaStrenda and in the article of ir. Ties Rijcken and prof. ir. J.A. den Uijl in Cement, 2005 the choice has been made for the concrete EPS-system, mainly because of the two just mentioned reasons.

<b>Situation/wish</b>	<b>Choice</b>
Small structure (<10m)	Concrete
Internal space needed	Concrete
Low draught	Concrete+EPS
Unsinkable	Concrete+EPS

Table 2: Situations and choices

# Part II

## Connections

*This part is about the connections in between floating structures, also called interconnections in this thesis. First a review is made about why and when connections between floating structures have to be used and some reference projects are given. In chapter 6, the functions and criteria for interconnections are given. In chapter 7, the movements and forces of modular floating structures are discussed. All connection types and the basic possibilities are listed in Chapter 8. In chapter 9, the execution of connecting is considered. The final chapter investigates how a detachable rigid connection can be realised best. The other connection types are elaborated in the appendices belonging to this chapter.*

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## Part II

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## 5. Introduction Connections

Floating structures as defined in the first sentence of chapter 1, have to be moored, either temporarily or permanently. Thus, in fact all floating structures have mooring connections. Mooring separate floating structures is an old technology and has been done for as long as there were floating structures. However, floating structures can also be moored or connected with each other. In this thesis, these connections are called 'connections in between floating structures' or 'interconnections' and this is a far less known technology.

In the development towards floating districts and floating cities (see chapter 1), the connections between floating structures are becoming increasingly important, as in floating districts and cities, the floating structures have to be coupled with each other. Next to this, there is a tendency towards increasingly large floating structures. Large floating structures often have to be subdivided into separate parts. As a result, these interconnections will be used more and more.

Most frequently, this coupling of floating bodies has to be realised on water, which brings multiple difficulties along with it.

There is limited knowledge about the connection between floating structures, as sketched in the introduction of this thesis. The above mentioned tendencies make this subject very interesting. So this part of the thesis focuses on the connection between floating structures.

### Connections unmapped territory

Building on water is still not a usual practice. However, more and more experience is gained with floating structures in the last two decades according to chapter 1. An increasing number of floating structures is constructed every year and much research has been done. But most constructed floating structures are rather small individual floating buildings, and most researches also focus on behaviour of separate floating structures.

Therefore connections in between floating structures are still a rather unmapped territory, for the Dutch construction industry and Dutch construction research institutes. As DeltaSync and Urgenda put it: 'The technology for floating building is now focused on free floating buildings, not on floating districts or cities' [Stichting Urgenda and DeltaSync, 2008].

### Goal and focus

This part of the thesis tries to cover all possibilities for connection situations: the different connection types, the problems, criteria and the current experience. The rigid connection for floating bodies with no intermediate distance appeared to be most feasible for most modular floating structures in Dutch inner-water circumstances, therefore this part mainly focuses on that connection type. Next to this, this connection part focuses mainly on 'in line' lying connections.

Thus the focal point of this connection part lies on the rigid connection of floating structures 'in line'. In line means that the floating bodies will only have connections on the two opposite sides. However, most considered options and the connection designed in chapter 10 are applicable for floating bodies with connections in all directions.

### Structure Part II

This chapter presents when interconnections have to be used and a few examples of connected floating structures are given. In chapter 6, the functions and criteria for interconnections are listed. In chapter 7 the movements and forces of modular floating structures are discussed. Examining all the connection types and the list of the basic possibilities is the focus of chapter 8. In chapter 9, the execution of connecting is considered. The final chapter investigates how a detachable rigid connection can be realised best. The other connection types are elaborated in the appendices belonging to this chapter.

## 5.1 When to use interconnections

Connections between floating bodies have to be used in the following situations:

- Modular floating structures
- Coupling standard modules
- Connecting separate floating structures

### Modular floating structures

Modular floating structures are used in the following situations and for the following reasons:

- For very large floating bodies, it is not possible to be constructed all in one piece, so they have to be constructed out of multiple parts.
- Modular splitting because of transportation (into relative large pieces for transport over water, into small modules for transport over the road)
- Modular splitting because of opportunities for flexible use.
- Splitting for structural reasons, because the internal forces will increase with increasing width and length and might become too large.
- Splitting for structural reasons, if the superstructure is different, might be beneficial or necessary to make use of floating bodies with different heights and draught (see Figure 1). This will result in less material consumption and lower internal forces.



Figure 1: Floating structure with different superstructures and therefore different height floating body.

### Coupling standard modules

- Connecting of standard elements or pontoons to form larger surfaces. Often used for temporary floating platforms (see for example Figure 8)
- Interconnecting floating infrastructure
- Interconnecting floating breakwaters

### Connecting floating structures

- Connecting floating structures with floating infrastructure
- Coupling separate floating structures

Structural reasons for splitting up, because of high internal forces will be elaborated in chapter 7.

### **On water and detachable**

In most of the mentioned cases, the interconnections have to be realised on water. If the floating structure is modular because of structural reasons the connections can also be realised 'in the dry'.

In most of the cases mentioned here, the connections need to be detachable. In some cases the floating structure doesn't need to be sectional when it is completed. When the floating structure doesn't need to be sectional after completion, but only needed to be modular before or during construction, the connections don't need to be detachable. This could be the case when the structure needs only to be transported once to its final location, or it is split for reasons that only accounts during construction, for example if the floating structure is bigger than the dock where it is built in.

### **Conclusion**

It can thus be concluded that using interconnections results in the following benefits:

- Flexibility for changing functions and future expansion
- Flexibility to control cost by building modular units
- Transportation and relocation possible
- Less use of material by reduction of internal forces

Connections make it possible to create transportable and flexible floating structures, 'floating cities', floating infrastructure and floating mega structures.



## 5.2 Reference projects with interconnections

In this paragraph, a few examples of floating structures with interconnections are given.

### Large Modular floating structures

Below three examples of large modular floating structures is given.



Figure 2: Mega-Float Airfield, Japan

#### Mega-Float Floating Airfield, Japan

Dimensions: 1000x 60 – 121 m<sup>2</sup>  
4 large steel pontoons  
height pontoons: 3 m  
Completed: 2000  
[www.srcj.or.jp](http://www.srcj.or.jp)



Figure 3: Mega Platform, Singapore

#### Mega Floating Platform, Singapore

Dimensions: 120 x 83 m<sup>2</sup>  
15 steel pontoons with bolted connections  
Completed: 2006  
[Wang et al. 2008]

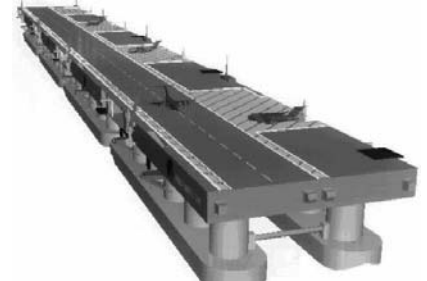


Figure 4: MOB

#### Mobile Offshore Base, US

Dimensions: min. 1500 x 150 m<sup>2</sup>  
+- 4 Semi-submersible pontoons  
Completed: Study  
[Den Vijver, 2003]

### Large Floating Greenhouse

A combination of several parties, including the province of South-Holland, a contractor, an engineering agency and the research agency TNO, is doing research to the feasibility of a floating greenhouse of 50.000 m<sup>2</sup> for a large polder in the province South-Holland. This will become the largest floating surface after the floating airfields. For such a surface, connections in between are inevitable.

### Coupled floating buildings

Below three examples are given where coupling of floating buildings is done. In these examples, floating bodies are coupled to become a larger surface. The examples in figure 5 and 6 were transported separately and connected at the site.



Figure 5: Sailing School Roy Heiner.

#### Sailing School Roy Heiner

Sailingschool with offices and conferencerooms.  
2 Modular concrete barges connected with steel rods.  
Dimensions: 24 x 12m<sup>2</sup>  
Transported from Urk to Lelystad in 2003  
[[www.arkenbouw.nl](http://www.arkenbouw.nl)]



Figure 6: Coupled houses IJburg.

#### Coupled houses IJburg

Examples of floating houses for infocentre IJburg.  
4 main modular concrete barges connected with steel elements with pin-hole connection, with small pontoons in between.  
Dimensions: 4x 9x6 m<sup>2</sup>  
Completed: 2000  
[own picture and [www.attika.nl](http://www.attika.nl)]



Figure 7: Info centre IJburg.

#### Info centre IJburg, Amsterdam

2 coupled floating structures with space in between.  
Dimensions: 19x37m<sup>2</sup> and 18x12 m<sup>2</sup>  
Completion: 2000  
Companies: Ooms BV, Attika, FDN Engineering  
[[www.attika.nl](http://www.attika.nl)]

### Coupling pontoons

Rectangular steel pontoons are often used for temporary use, such as during construction or maintenance. For larger surfaces the pontoons have to be coupled. Below two examples of coupled steel pontoons are given.



Figure 8: Left, Steel Flexifloat pontoons with crane on top (Flexifloat) Right: picture pontoons Heijhaven.

### Floating infrastructure

Below some examples of roads and bridges which are already realised are depicted. These examples are also given in appendix 2 with more information. These floating infrastructures consist of a lot of floating elements. Multiple connection types have been used, as well hinged and rigid. Information about the connections in the military bridges and the floating road is given in Appendix..



Figure 9: Highway bridge, US



Figure 10: Military Bridges



Figure 11: Floating Road, Hedel

### Floating Wave Attenuators

Floating wave attenuators, just as floating infrastructures, can be seen as floating line elements. Mostly the modular parts are connected with (semi)hinge connectors.

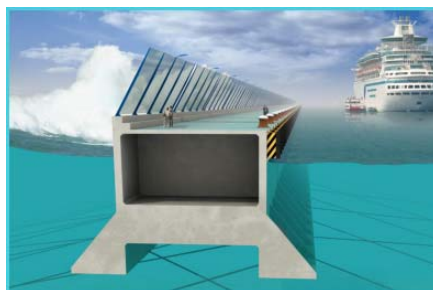


Figure 12: FDN Floating Breakwater



Figure 13: Floating Breakwater, IMF

These examples can also be found in appendix 2 together with some more information.

## 5.3 Developments/plans

### Floating building elements

There have been multiple plans, designs and researches on 'floating bricks' as they are called in the thesis of, amongst others Rijcken (2003) and Kuijper (2006). These 'floating bricks' are small floating elements from several square meters which can be coupled in any amount to large surfaces. Examples of these floating elements are the designs of Ties Rijcken and Maarten Kuijper, given in chapter 5. The idea behind these floating bricks is that they can be easily transported over the road and constructing floating structures out of standard small elements will be cheaper than producing new shapes every time.

### Floating cities

Plans for floating cities, exist already for a long time, but becoming more and more realistic (see chapter 1). Rotterdam and Naaldwijk have advanced plans (see appendix 4)

## 6. Functions and criteria of interconnections

Reasons why structures have to be connected or have to be separated in modular parts were mentioned in chapter 5. When connections have to be used they have to fulfil the functions and criteria given in this chapter.

### 6.1 Functions

Connections can or need to fulfil the following functions:

#### Functions:

- Restricting movements
- Allowing certain movements
- Transfer of forces:
  - Moments (can be divided in compression and tension component)
  - Tension
  - Compression
  - Shear force
- Transit of pipes and electrics
- Self-alignment and rapid engagement

The first mentioned option; restricting movements, is the main function of structural connections. The second option; allowing certain movement might sound contradictory, but in certain situations certain movements have to be restricted while others have to be allowed. This is explained in chapter 7. The function, self-alignment is important during construction on the water surface, this is being discussed in next paragraphs and in chapter 10.

### 6.2 Criteria / Fundamental Design Requirements

Connections have to fulfil the functions listed above. From these functions a list of criteria can be deduced. These criteria have to be completed with other criteria and design requirements, so that also the problems arising with connecting floating structures can be countered.

#### 6.2.1 Problems arising

Connecting floating structures on the water surface gives rise to several problems.

The first problem of connecting floating structures is the lack of accessibility to the connectors that are located between the elements and/or below the water level.

Another problem that could occur is the uneven draught or heel of the floating structures; this will result in connection elements not being in the same position. If these location/position inequalities are large, the draught and heel should be adjusted with ballasting and trimming of the floating structures. But small location deviations will remain. This should be covered by self alignment and accommodating of tolerances (explained later on in this paragraph).

According to Han (2007) the following (additional) problems arise with connecting and connections of floating structures and pontoons:

#### Problems [Han, 2007]

- Large relative motions
- Collision induced high acceleration
- Highly dynamic, small window (short time period) to secure
- Unstable floating platform due to flexible links/tolerances
- Strength/Fatigue failures due to high sea loadings

The first and third mentioned problem can be countered with self-alignment. For problems 2, 4 and 5 the use of pre-stress and using an elastic material in between the connected faces might be a solution. These solutions are explained later on in this paragraph, the function of an elastic material is discussed in paragraph 10.1.

## 6.2.2 Criteria and design requirements:

Below a list is given with criteria on which the connections can be assessed. With these criteria the mentioned functions and problems are taken in account. This list of criteria is completed with other generally applicable criteria. The list mainly accounts for interconnections which have to be constructed on water. The less obvious criteria are explained afterwards.

### Criteria and design requirements:

- Strength
- Rigidity / Restricting (or allowing) movements
- Pre-stress / pressing together
- Easy execution (Uitvoerbaarheid)
  - Accessibility connection
  - Amount of operations
  - Difficulty of operations
  - Risk on problems during execution
- Self-alignment
- Accommodating size/location tolerances
- Robustness (reliability / break / amount of moving parts)
- Durability and Maintenance (corrosion, painting/cleaning necessary, inspection possible)
- Proven in practice / experience
- Disconnectable (detachable)
- Impact damping
- Amount of material/costs

### **Easiness connecting**

Easy connecting during execution is important, because connecting floating elements on water will easily become difficult by the movement of the floating bodies and by the lack of accessibility of parts that are in between the floating elements and/or below the water surface. The demand for easy connecting is taken in account in the criteria practicability (with its sub-criteria), self alignment and accommodating size/location tolerances. Easy execution and self alignment is elaborated in chapter 9, *Execution of connection*.

### **Accommodating size/location tolerances**

Size and location deviations have to be taken into account. When these are not (sufficiently) taken into account, this may result in misfit of elements. The tolerances of the connection have to be large enough, because this will otherwise result in difficulties during coupling. Accommodating of size tolerances by allowing more space around the elements will however lead to loosely fitted connections. This will result in undesired movements if no precautions are taken. Two possible methods to counter this are pre-stressing of the connection and using elastic intermaterial.

### **Pre-stress of the connections / Pressing together**

It might be possible to design connectors which make the floating bodies be pressed together. This results in a form of pre-stress in the connectors which are pressing the bodies together. Pre-stress of the connections has several functions. One function is preventing the small relative movements caused by allowing tolerances as mentioned in the last paragraph. These relative movements are in many situations undesirable. On top of that, the small relative movements can also cause very unfavourable dynamic loading. With pressing the elements together with 'pre-stress' of the connections the relative movements can be prohibited. And the connection can be made more rigid.

Pre-stress can also work very effectively against fatigue: 'pre-stress' makes sure that certain spots will only be in tension and other spots only in compression, so stress changes from positive to negative, which will lead to fatigue, wont occur.

With certain ways of pre-stressing a form of self-alignment might also be possible. (When the floating bodies pull themselves together, they will move towards each other). This could work very beneficial when the floating bodies are heeling a little when they have to be coupled. When the connectors can tighten the joints, this relative heel can be undone.

When the connecting surfaces are pressed together, the mentioned trapezoidal ridges will also perform better for shear forces.

Summarizing the last mentioned: pre-stress has got the following functions:

- accommodating tolerances
- preventing small relative movement
- more rigid
- counteracting fatigue
- self-alignment
- countering relative heel

The remark will be made that the here sketched benefits of pre-stress in fact only account for a rigid connection. And normally, it is only usable for connecting floating structures without intermediate distance.

#### **Disconnectable or not**

As said in chapter 5, in most cases the interconnections need to be detachable. In some cases the floating structure doesn't need to be sectional when it is completed. When the floating structure doesn't need to be sectional after completion, but only needed to be modular before or during construction the connections don't need to be detachable. This could be the case when the structure needs only to be transported once to its final location, or it is split for reasons that only account during construction, for example if the floating structure is bigger than the dock where it is build in.

#### **Costs**

The amount of the costs will depend mainly on which materials will be used and the amount of materials. As long as this is the case and the cost do not increase by expensive operations or special parts, the costs of the connections will be relatively low compared to the costs of the floating structures which consist of much more materials. So, a little more costs for a better solution is quickly justified. But when movable or mechanic elements or other systems will be introduced in the connection, the costs will increase significantly.

It can be said that the costs depend mainly on the simplicity of the connection. The more simple the connection, the less it will cost.

## 7. Relative movement and forces

In chapter 6 the functions of the connections are given. The main purpose of structural connections is prevention of relative movements. This chapter discusses the movement between connected floating elements. The different movement allowing/prohibiting connection types which can be distinguished are given and the advantages and disadvantages of allowing or forestalling movements are given. Restriction of movement results in forces, this will also be treated in this chapter.

### 7.1 Relative movement

In chapter 4 the movements of single body floating structures are treated.

In the figure below the possible movements between floating elements are depicted, these are called relative movements.

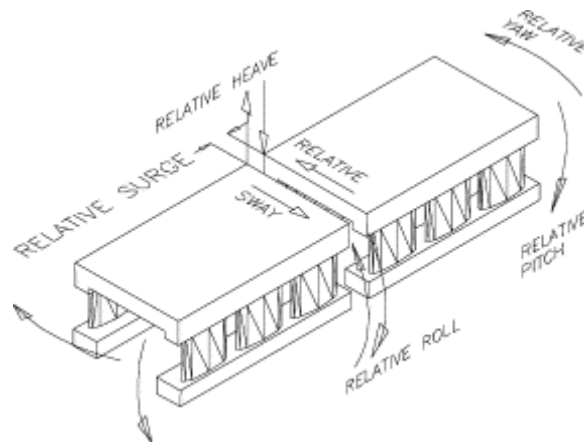


Figure 14: Relative movements between floating elements [Derstine, 2000]

### 7.2 Movement allowing/preventing options

As said in chapter four, moorings have to prevent surge and sway of a floating structure. When a modular part of a floating structure is not moored, but connected to another floating structure instead, this connection must prevent the sway and surge of the floating element. With the just given movement definitions: the interconnection must prevent relative sway and relative surge.

It is also possible that a interconnection prevents more movements, or no movements at all. In fact a wide spectrum of connector types is possible. The relative movement 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 different movement allowing or preventing options. [Rooij, G. de. (2006)].

But these 18 options can be decreased to a smaller amount of options, if the assumption is made that a structural interconnection must always prevent surge and sway. Below a list is given with the main possibilities for movement allowing or restricting options.

#### Possible connection types for preventing movement

- Fully flexible (non-structural)
- Vertical free, allowing relative heave
  - o allowing relative rotation (only preventing sway)
  - o preventing relative pitch (preventing sway and roll)
  - o preventing relative roll (preventing sway and pitch)
- Hinged connection
- Rigid connection

Combinations of these types are also possible: semi-rigid, semi-hinged and allowing partial rotation, see for example the non-linear MOB connector in Figure 16.



With preventing movements, large forces will be introduced in floating body and connection. So there always have to be managed between the degrees of freedom, the amount of rigidity and amount of forces. This will be elaborated in paragraph 7.3. But first the possibilities for movement allowing or restricting options will be discussed.

In chapter 10 is mentioned what connection alternatives are possible for the rigid connection without intermediate distance, for the other here given movement types this is done in appendix 17 and 18. In those chapters and appendices the connections will be elaborated and the best options will be chosen.

### 7.2.1 Fully flexible connection (Non-structural)

A fully flexible connection can be used when the connection is not needed to prevent sway or surge, because both structures are moored and the other relative movements are allowed. Then the connection is not a structural connection, but it is intended for transport, for example of cables, ducts, electricity or a bridge for pedestrians or cars.

This option is common for floating structures with intermediate distance.



Figure 15: Floating structures with a bridge in between (non-structural connection)

### 7.2.2 Connection flexible in vertical direction

In the case the connection must ascertain the location of the floating structure, the connection needs to prevent surge and sway. When there are no demands for the floating bodies to stay at the same height, the deck surface does not need to continue, a connection which allows relative heave can be used.

#### Advantages and disadvantages

Allowing relative heave will lead to a strong decrease of the internal forces in connections and beams, which can be caused by waves and unequal vertically imposed loads.

This allows the connection itself to be constructed lighter and this also accounts for the floating bodies, which do not have to be dimensioned on large vertical forces (which could be provided by a rigid connection).

A connection which does not allow relative heave results in more stability. And so this results next to less relative movement also in overall less movement of the floating structures, because of two reasons. First by increased stability and second because this way the combined length of the floating structure can be made longer than the order of the wave length.

This can be a good option for connections with an intermediate distance. For floating bodies without intermediate space this option seems less logic.

For the vertical free direction the choice can be made if this connection allows or doesn't allow relative rotation.

#### **Rotation allowing**

A vertically flexible connection, which will also allow relative rotation, shall except for resisting horizontal movement not contribute to preventing motions. If the connection will not prevent rotation, relative pitch and roll will occur, due to unequal loads or waves.

With this form of coupling, the stability barely increases compared to separate floating modules. The relative rotation, which can be quite large, certainly in relatively small floats, can be found undesirable.



### Rotation restricting

A vertical flexible connection can also be constructed in a way it prevents relative rotation.

A connection which allows no rotation ensures that the two floats must display the same rotation and thus provide more stability and this way the flotation will move much less.

#### Advantages rotation restricting

So, the advantages of rotation restricting are the following:

- No relative rotation
- More stability and less movement

No relative rotation is also beneficial for the connections with transport use in case of intermediate distance between the floating bodies. When the rotation is not restricted than at the bridges and the ducts there have to be made use of special couplings or flexible materials.

#### Disadvantages rotation restricting

Main disadvantage are the higher internal forces in the connectors and floating bodies.

When more than two spacers are used, the relative roll gives more difficulties, since by the change in rotation the distance between the floating bodies will change unequally. This is no problem when only two connectors are used, but with using more connectors this results in forced lengthening or shortening of the connectors which can cause high stresses and failure if the connectors cannot follow this deformation. To overcome this, elastic materials, shock absorbers and/or springs can be used.

The change of distance by relative roll for floating bodies with intermediate distance has been calculated for the connections between the floating pavilion an plaza. See Appendix 16, *Rotation causing change in offset*.

### 7.2.3 Hinged Connection

A hinged connection is used when relative heave is undesirable and when relative pitch is allowed.

With a pure hinge connection only shear and normal forces are transferred and not moments, so this means the moments in the floating bodies will be less when using hinge connections. A pure hinge connection only allows relative pitch (so it prevents all three relative translations, relative roll and relative yaw). But with a hinge connection with more degrees of freedom, it is also possible to allow a certain amount of roll or yaw because of limiting the internal forces. This can be realised with a pin or ball joint in the centre and connectors which can elongate at the sides. An example is the MOB connector see Figure 16.

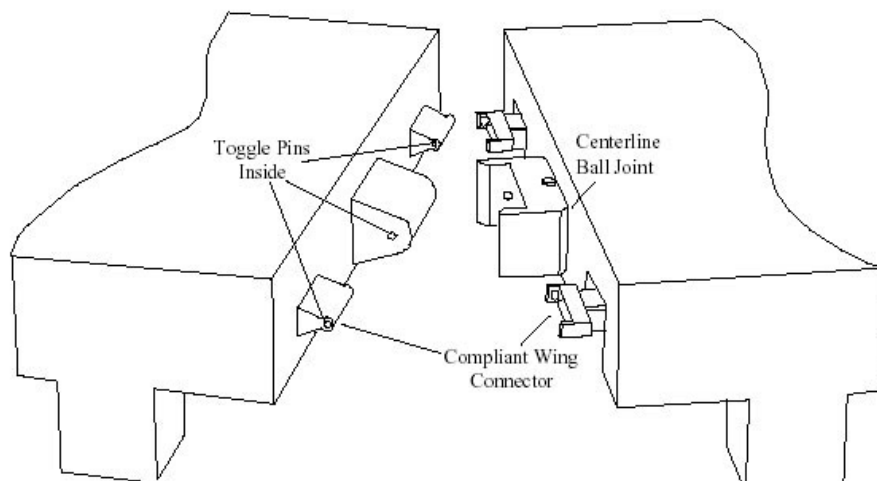


Figure 16: McDermott, nonlinear hinged connection for MOB.

The advantages and disadvantages for relative roll allowing types are the same as the advantages and disadvantages for allowing rotation mentioned in paragraph 7.2.2.

## 7.2.4 Rigid Connection

A rigid connection shall be used when all relative movements are undesirable. With rigid connections, not only the relative movements are prevented, but also the overall motions are decreased. This is because a floating body which consists of rigidly joint elements can be approximated as a single rigid body\*. With a rigid body the motions of the entire structure will also be the smallest, see paragraph 7.3. In the cases where the options with more degrees of freedom are chosen, this is done for limiting the internal forces. A rigid connection is possible as long as the internal forces stay within the limits of the strength of the floating structure. Only when the forces become too large or the dimensions threaten to become too large due to the internal forces, a connection is chosen with more degrees of freedom. The following two paragraphs elaborate on the choice between hinges and rigid connections.

## 7.3 Movement versus forces

### More stability with increasing dimensions

As stated in chapter 2, the static stability increases strongly with increasing the length and width. A large rigid floating body is by its high static stability less susceptible for movements by imposed loads. With increasing the length and width it can also be realized that the dimensions of the structure will be larger than the occurring wave lengths, decreasing the susceptibility of the structures to movement due to waves. So a large rigid structure will result in less movement.

### Larger forces with increasing dimensions

Unequally divided vertical loads on the body will result in internal forces and moments. As mentioned in chapter 3 *Loads*, waves and unequal imposed loads will cause internal forces. Internal forces can be calculated with the calculation methods mentioned in chapter 2 and appendices 7 and 10.

#### By waves

High waves result in large unequal water pressure, and when the waves will become longer, the internal moments caused by them will be larger. When the length of the structure is smaller than the wave length, the structure will tend to move along with the waves, which results in less internal forces. If the length of the structure is significantly larger than the wavelength the structure will not tend to move along resulting in internal forces and moments caused by the difference in water pressure (see appendix 10).

#### By imposed loads

Unequally divided imposed loads will also cause internal forces and moments. With a larger floating body, these forces and moments will be larger for two reasons:

- The body will move along less, so the imposed load is not equalised by the water pressure at the same spot only, so moments and shear forces will arise.
- With a larger floating body the moment arm can also be larger, so larger moments can exist

### Moving along means decreasing forces

High internal forces can be decreased in two ways:

- Moving along of the structure
- Smaller dimensions

Like with every structure, if a floating structure moves along with the load, this will decrease the internal loads. So if the floating structure can move along with waves, they cause less internal forces. Moving along with the waves will occur if the dimensions of the floating structures are smaller than the wavelength. If a line of coupled floating elements, with dimensions significantly smaller than the wave length is coupled with hinged connections which allow pitch with a large rotation, these line of coupled floaters can also move along with the waves and no large forces will be introduced.

---

\* Very large floating structures, also without interconnections, do in affect not behave as rigid structures anymore, since they will deform in severe wave conditions. This is for example the case with large oil tankers. But since this thesis is not about these very large floating structures this simplification is justified.

With smaller dimensions the internal forces by imposed loads will also be smaller, which was explained in last paragraph. So in fact smaller elements with elements the floating bodies will move along more easily and the internal forces will be less.

#### **Introducing degrees of freedom: less forces more movement**

With separating a large floating structure and introducing connections with degrees of freedom, the floating body will not be rigid any more and at the spot of these connections the body can perform relative movements. If the allowed relative movements are relative heave movements or pitch or roll rotations, this means the floating body is in a way separated in multiple floating bodies with smaller dimensions. As said, smaller dimension means better moving along and less internal forces.

Another way to put it, is mentioning that a hinged connection will not transfer bending moments, so this means the bending moments in the floating bodies will be smaller.

But better moving along also means more movement. So this is where have to be chosen between by picking a connection type: high forces and little movement or less forces with more movement.

### **7.4 When to use what connector type**

For relatively small structures in relatively calm water, rigid connections will not lead to too large internal forces. So in these cases rigid connections can be used. This will be the most occurring situation for floating structures in the Netherlands. For the case study of the pavilion, part 3 of this thesis, it appeared that a rigid connection is also the best solution.

For large structures in rough conditions or very large structures it is recommended to use hinged connections to limit the internal forces.

# 8. Connection types

In this chapter all possibilities for connections are given. This chapter starts with a scheme which gives all possibilities and depicts the location where the several options are treated in this thesis.

## 8.1 Connection types (scheme)

Below a scheme is given with all different types of connections. The first distinction that can be made is if the connected floating bodies have got a intermediate distance or not. This is the top layer in the scheme and this will be treated in paragraph 8.2.

Then the choice can be made what movements the connections can make or have to allow. This movement was elaborated in chapter 7. The bottom layer in the scheme gives all basic connection options. These basic options are discussed in paragraph 8.3.

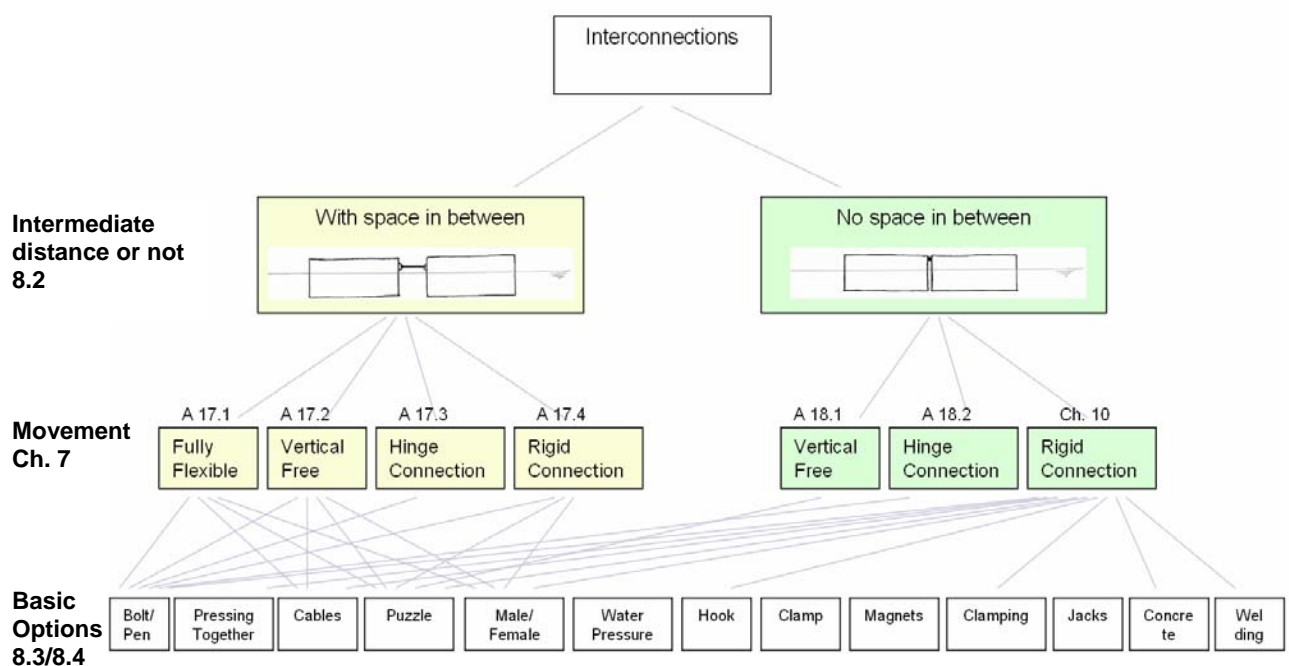


Figure 17: Scheme connection options

In this part only the rigid connection without intermediate distance is being elaborated, because this type was found most interesting and will be applied most in practice. Also for the other types is considered what connections would be best. The sketches and advantages and disadvantages are given in appendices 17 and 18.

## 8.2 With or without intermediate distance

When connecting floating structures they can be chosen to connect the floating bodies right against each other, with no space in between, or they can be connected with space in between.

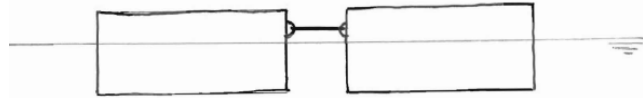


Figure 18: Floating bodies with intermediate distance



Figure 19: Floating bodies without intermediate distance

### Reasons for intermediate distance

Choosing for an intermediate distance in between two structures can be for aesthetic reasons. But it is also a logical choice if the two structures have got different owners. The choice for space in between can also have a structural background.

When the structures have got space in between them, the connections can be designed in a way that the structures are able to move in vertical direction independent from each other and won't influence each other.

When the two structures are allowed to move in vertical direction completely independent from each other, the connection doesn't have to transfer any vertical forces. So, then the connection can be dimensioned more lightly. In this case the beams or floating body can also be dimensioned more lightly.

The possible connections for floating bodies with intermediate distance will be elaborated in appendix 17.

### Reasons for no intermediate distance

If two floating infrastructure elements should be connected, these elements have to be placed against each other, otherwise they lose their functionality. The same holds if the elements are parts of one large floating structure.

In general, if the surface should continue, then the elements have to be placed together.

Also a connection that influences each other's movement in vertical direction gives more stability for the floating structures and leads to less movement of the structures. This is possible both with and without intermediate distance, but this is more easy to realize with no space in between.

When a rigid connection is desired, there could best be chosen for floating structures without intermediate distance, because then it is far easier to make a rigid connection. Then also pre-stress can be applied.

## 8.3 Basic connection alternatives

In the search and the design procedure for good connections between floating structures, first all basic possibilities for connections will be mapped.

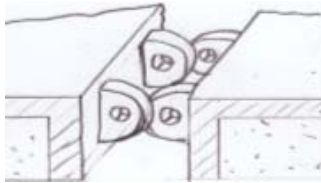
In this paragraph 8.3 the basic connection methods are given. Here combinations are not yet taken in account but only the basic principles. This paragraph focuses on possibilities to transfer powers and preventing movement.

On the next two pages all basic options are depicted and in 8.4 these options are elaborated.

**Basic connection alternatives**

**1. Bolt/pen/pins**

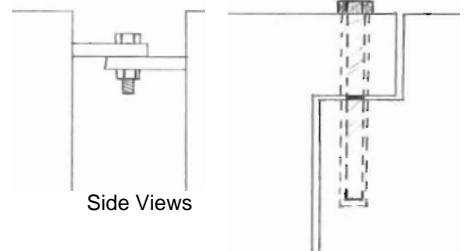
a. horizontal transverse



b. horizontal longitudinal



c. vertical (bolt and pen)

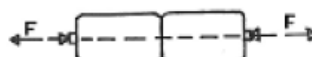


**2. Pressing together**

a. real prestressing cables



b. just pressing together (cables/rods)



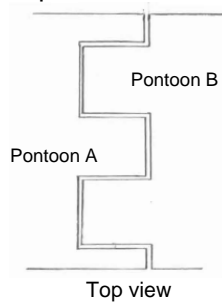
**3. Cables**



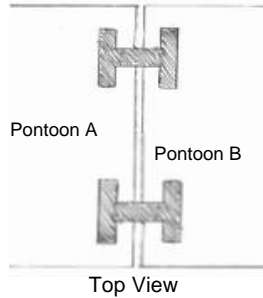
Side View

**4. Puzzle**

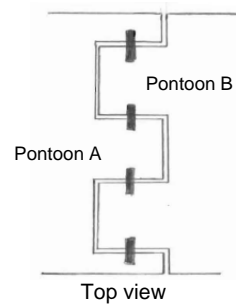
a. puzzle



b. separate puzzle pieces

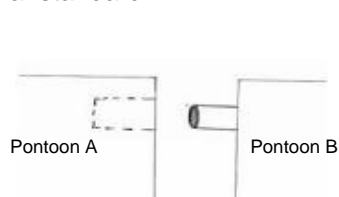


c. combination

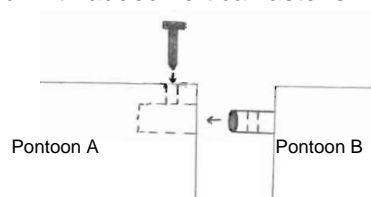


**5. Male/Female connectors**

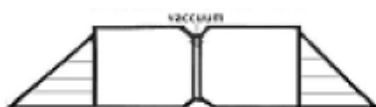
a. standard



b. with added vertical fastener

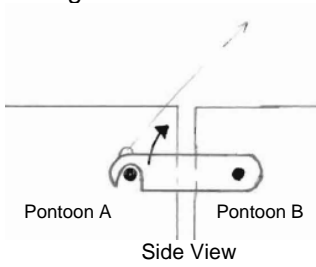


**6. Waterpressure**

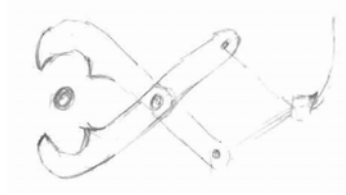


**7. Hook**

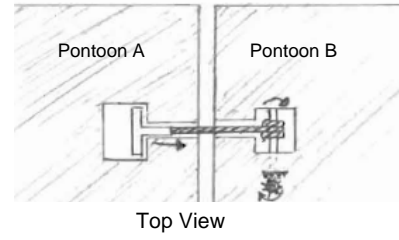
a. Single hook



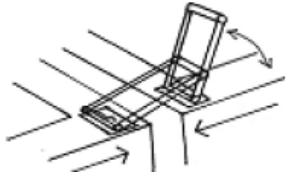
b. Double hook



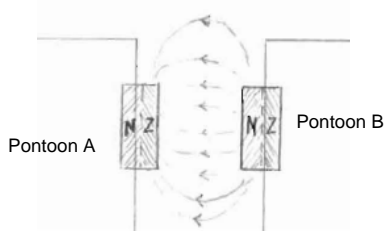
c. Improved ribbon bridge solution



**8. Clamp**

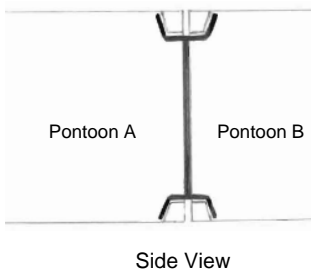


**9. Magnets**

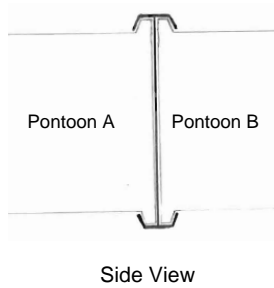


**10. Clamping connection (opsluiten/inklemmen)**

a. from center outwards

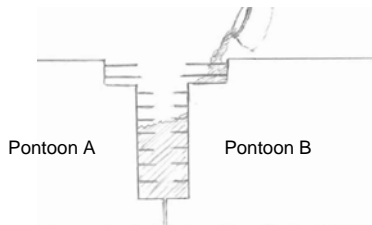


b. from the outside



**11. Pneumatic/Hydraulic Jacks**

**12. In-situ concrete**



**13. Welding**



## 8.4 Remarks on alternatives and examples

In this paragraph will be given some remarks and comments on the alternatives that were given in last chapter. Some alternatives are illustrated with examples out of practice. In this paragraph some alternatives will already be disregarded.

### 8.4.1 Bolt/pen/pins

For the connection with bolts or pens there are a lot of possibilities. As sketched in 8.1 these connectors can be used in the three different directions. This distinction is important, because this difference in orientation results in different degrees of freedom and difference in accessibility.

With the variant with the bolt or pen in transverse direction a form of pre-stress is possible, while this is not possible with the other alternatives.

The option with bolts in transverse direction is the most used connection option for the rigid connecting of concrete barges. An example is the sailing school of Roy Heiner (Figure 5). Also Nico Groenendijk chose for this connection variant in his design for a floating stadium.

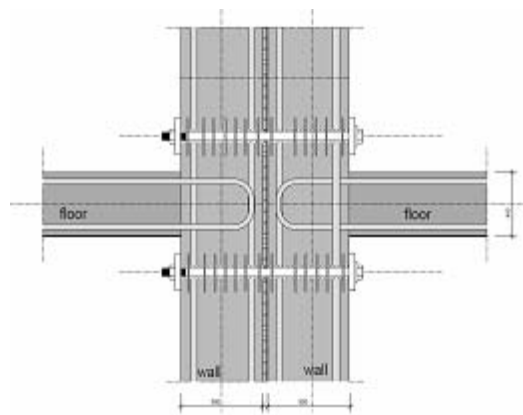


Figure 20: Cross section from the side transverse bolt connection for floating stadium, by Nico Groenendijk

This connection is especially convenient for connecting standard concrete floating bodies, because here the bolts remain easily accessible, because the floating body is empty. When using Concrete bodies filled with EPS, this variant is less good from accessibility perspective, at least for the connection that do not lie close to the surface.

In Figure 21 a connection with vertical pens is depicted. This option was designed by de Rooy (2006) for rigid connecting of floating elements for a floating container terminal. According to Groenendijk (2007) the pens in this design will deform too much, resulting in the connection not being disconnectable any more. When the bolt goes through more than two locking plates, the deformation will be less.

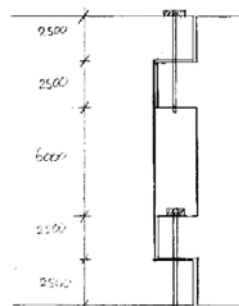


Figure 21: Design for connection for floating container terminal with vertical pens (Gina de Rooy)

In appendix15 several examples of connections with horizontal bolts or pens are given.

Vertical bolts or pens are also often used in practice, amongst others to couple pontoons. In Figure 22 an example is given; connections of the pontoons of the Heijsehaven.





Figure 22: Connection pontoons Heijsehaven, vertical element with double pen below, securing bolts above

Deviations in locations and sizes could result in problems with this alternative, but with a peak pointed bolt this can be partially overcome.

The connection option with bolts and pens can also be used for connections of floating bodies with intermediate distance and then it can be combined with steel sections or rods in between. This option can also be combined with ball joints or spherical bearings for extra degrees of freedom (see appendix 17)

The flex-connection of FDN Engineering, which is used in between their wave attenuators, can also be put in this family of alternatives. This option uses an elastic material in between and a pen in transverse direction with a kind of ball joint in the middle.



Figure 23: Rubber profile of FDN Flexconnection

#### 8.4.2 Pressing together

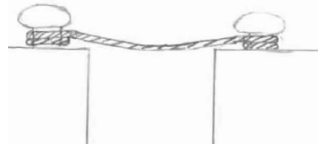


Floating elements can be laced up together by transiting cables or rods through to the floating body. By tensioning the cables the sections will be 'prestressed'. This method can be used as well with usual concrete pre-stress (post-tensioning) as well with just lacing up without grouting the ducts and making firm slots at the end of the cables.

In the case when multiple small elements are to be coupled this lacing up method seems a good option, since in fact only one connection has to be made for coupling multiple elements. This is one of main reasons why Rijcken (2003) and Kuijper (2006) choose this option (see appendix 14).

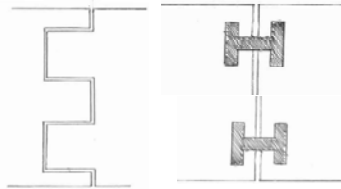
Using common concrete post-tensioning in the bottom of the floating bodies will result in a lot of difficulties, since the water (contained or salty) may not enter the ducts that will be below water level on the bottom of the floating bodies.

### 8.4.3 Cables



The big advantage of using cables is that they are flexible and dimensional deviations or movements won't result in any problems. A kind of pre-stress is also possible. The connection from the cables to the solid structure is mostly seen as less robust and permanent as some other alternatives.

### 8.4.4 Shape



With puzzle-shaped floating bodies, the floating bodies can be shaped in a way they will fit exactly into each other. Depending on the direction of the 'puzzle' shape, forces in all direction can be taken.

An example of a connection where the shape is used to form a rigid connection, and on top of this, also able to take tension forces and horizontal shear forces is Willy's design, see Figure 24.

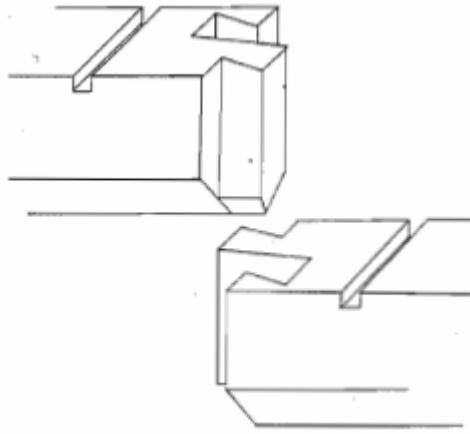


Figure 24: Willy's design, Patent No. BE904213

A big disadvantage of this design is, that during connecting, one of the floating bodies have to be lifted out the water or submerged.

A shape connection can also be very beneficial for the self alignment and shear force, an example of a shape with this advantages was given in Figure 33. This option can be very good combined with other alternatives. Such a shape in combination with separate 'puzzle pieces' is used in practice on the pontoons of Hann-Ocean, see picture below. This example is discussed more in detail in appendix 15.

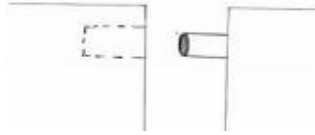


Figure 25: Hann-Ocean connector (hann-ocean.com)

Normally with this option it is not possible to have pre-stress in the connection, but with adjusting the shape in a certain way a certain amount of pre-stress can be realized. This principle is applied with the Hann-Ocean connector, see appendix 15.

The use of bulkheads (schotten) between the two floating bodies to pass on the shear force is also possible, this option is seen as the same as the 'puzzle-piece option'. When there is space in between the floating bodies, bulkheads could be more interesting than small 'puzzle-pieces'.

### 8.4.5 Male/Female



The male/female connection is in fact also a shape connection and this basic principle is used a lot in interconnections with no space in between. As said in last paragraph a shape connection can be very beneficial for the self alignment and shear force. Together with a fastener this connection is also able to take tension forces. In Figure 26 the Flexifloat connection is depicted, an example of this principle.

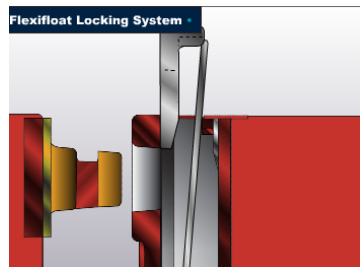
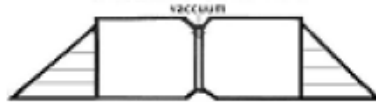


Figure 26: Flexifloat locking system

More information on this solution is given in appendix 15. The complete Flexifloat solution leads to connection which is very easy in execution.

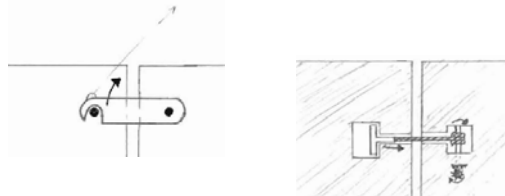
### 8.4.6 Waterpressure



In multiple works where connections between floating structures are discussed it is said that the separate floating elements will be pressed together because of the hydrostatic water pressure. In fact this is only the case if there is no water in between the floating structures. When there is space between the floating elements which is connected with the surrounding water, the hydrostatic water pressure in between the floating elements will be the same as the surrounding water pressure. Hence compressive connection between the floating elements by water pressure just doesn't exist.

This water pressure connection can only be realised if it can be guaranteed that the surrounding water can not flow between the elements and it can not built up the same water pressure. This can be done by sealing of the bottom and the sides of the gap between the floating elements. In this thesis this connection is seen to be vulnerable and so it is disregarded as a possible option.

### 8.4.7 Hook



A single hook as sketched in paragraph 8.1 can be a solution which can lead to very quick and easy connecting. But the single hook as sketched is in fact not fixed yet, with certain movements it can come loose easily, but this can be overcome by adding parts which closes the hook which fixes the pin. With the double hook (see 8.1) the hook and pin are also fixed.

With these options pre-stress is normally not possible. But when it is made possible that the axle or hook can move in horizontal direction, relative of the floating elements, the connection can be pre-stressed. This can be done by making use of tandracks or pneumatic cylinders. This however makes the connection less robust.

With the Improved Ribbon Bridge solution (see figure in 8.1 and Figure 27) the hook is attached to a steel cable which can be brought on tension by coiling the cable. With this method, size or location deviations will not result in problems and the connection can be brought in a range of pre-stress.



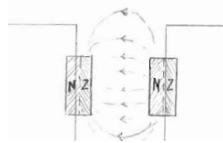
Figure 27: Connecting Improved Ribbon Bridge (IRB)

#### 8.4.8 Clamp



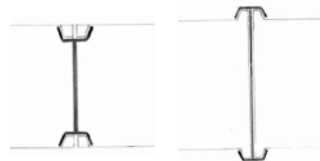
With a clamp, it is also possible to create a quick and easy connection. But this alternative is less robust, so therefore it is also not recommended.

#### 8.4.9 Magnets



Connections can also make use of electromagnetic force. This option is often proposed in harbour studies for mooring ships to the quays. Theoretically, electromagnetic connections work very well, but it costs a lot of energy and when the power goes down, the connection fails, so this is also a far less durable connection, so this option is also not recommended.

#### 8.4.10 Clamping connection



With a clamping connection is meant a connection which makes use of clamping parts of both floating elements together. As depicted in 8.1 it is shown that this can be done from the center outwards and from the outside inwards. The clamping from the center outwards is used for the floating road in Hedel, see Figure 28. For the clamping connection from the outside is made a design for connecting floating greenhouses see Figure 29 in a master thesis.

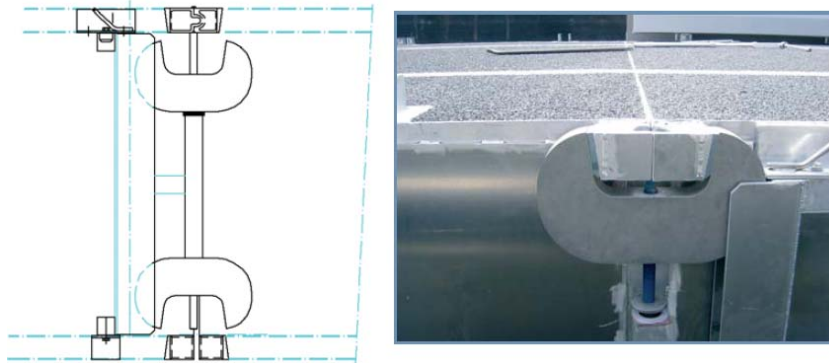


Figure 28: Clamping from center outwards, clamp connection floating road Hedel (Cement)

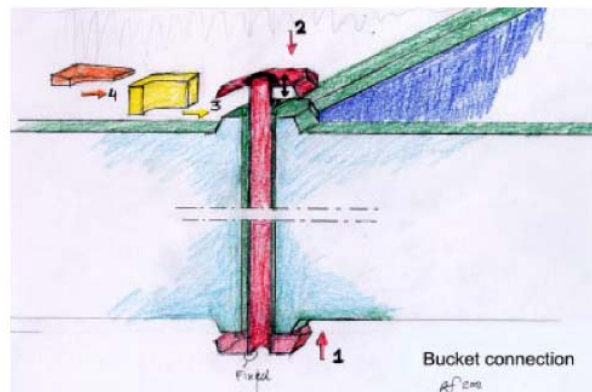


Figure 29: Design clamping connection from the outside: 'bucket connection'

Both connections can only be used for rigid connecting. Creating pre-stress in the connection is hardly possible.

#### 8.4.11 Pneumatic/Hydraulic jacks

Pneumatic or hydraulic jacks can be used when it is wished that the connectors can move to make the connection or for creating pre-stress in the connection. Jacks can well be used together with hooks, and by doing so, they can also be brought on pretension. Or jacks can be also be used just to create the connection itself. The company Mega-Float (the company which promotes the construction of very large floating structures as floating cities and floating airfields, mostly of steel, since the main Japanese steel constructor are the founders of this company. This company is also the main party behind the realised Mega-Float floating Airfield in Japan, mentioned in chapters 1 and 7) has made a design for a disconnectable joint which makes use of pneumatic- as well as hydraulic jacks. See Figure 30.

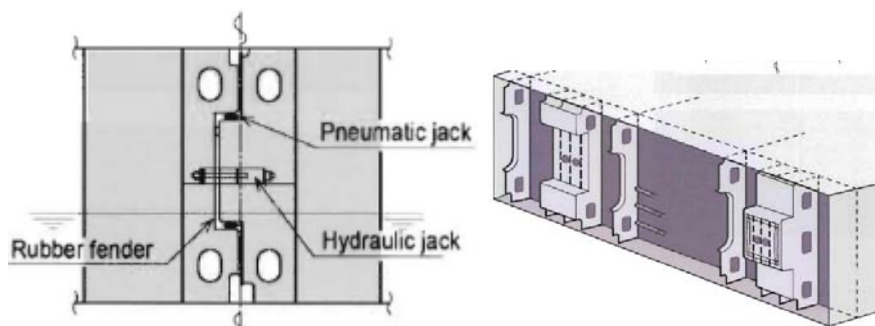
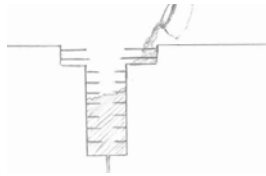


Figure 30: Disconnectable joint of Mega-Float, Side view and isometric view

Again here is made the remark that hydraulic and pneumatic jacks are found less robust than for example, a connection of only bolts.

### 8.4.12 In-situ concrete



In-situ concrete might be a good option for a rigid connection. But of course this lead to a connection which is not disconnectable.

### 8.4.13 Welding



For a rigid connection, steel floating bodies can also be welded together. Under water welding is also possible with costly underwater welding techniques. This has been done, for example for the Mega-Float Floating Airfield (see chapter 1 and 7). Welding also has the disadvantage that the connection is not disconnectable. Other disadvantages are the long process of joining, welding shrinkage and welding induced deformation and risks on fatigue.

## 8.5 Evaluation alternatives

The evaluating of the alternatives will be done for the different options for movement as given in paragraph 8.1 (hinged, rigid, etc.) and for structures with intermediate distance and structures without intermediate distance. For the rigid connector without intermediate distance this is done in chapter 10, the other situations are discussed in appendix 17 and 18.

### Combining Alternatives

All alternatives can be combined with each other to create an connection which combines all benefits. For example For floating elements which should be rigidly connected with no space in between it seems logical to make use of a shape connection, for self alignment and passing on shear forces. This 'shape connection' then has to be combined with a connection that is able to withstand tension

### Excluded alternatives

The following three alternatives are excluded, since they were considered as a considerably less robust than the other alternatives:

6. Waterpressure
8. Clamp
9. Magnets

When the connection has to be disconnectable, the following three options can also be disregarded:

- 2a. Real pre-stress
12. In-situ concrete
13. Welding

### Remaining alternatives

For a disconnectable connection, the following options remain:

1. Bolt/pen
  - a. horizontal transverse
  - b. horizontal longitudinal (zie voor voor en nadelen ook Hann-Ocean)
  - c. vertical
2. Pressing together (cables/rods)
  - b. just pressing together
3. Cables
4. Shape
  - a. puzzle
  - b. separate puzzle pieces
  - c. combination

5. Male/Female
  - b. standard
  - c. with added vertical fastener
7. Hook
  - a. single hook
  - b. double hook
  - c. Improved Ribbon bridge solution
10. Clamping connection (opsluiten/inklemmen)
  - a. from center outwards
  - b. from the outside
11. Pneumatic/Hydraulic Jacks

For the different specific possibilities, it will be decided what alternatives from the remaining possibilities will be a good option. If there are multiple good options, then there is made use of an MCA for the evaluation and choice for these alternatives. For the MCA the criteria of paragraph 6.2 are used.

The MCA is only used for the rigid connection for bodies without intermediate distance, see paragraph 10.3.

## 9. Execution of connecting

In this chapter the execution of connecting floating bodies will be discussed. This chapter is focussed on a rigid connection without intermediate distance, but most contents accounts for every connection as well with or without an intermediate distance.

The main demand for connecting is that it should be easy in execution. In this chapter is elaborated how the execution of connecting can be done best. The methodology of connecting is the focus of this chapter.

### Bring floating bodies together

The floating bodies have to be brought together. If the bodies are constructed somewhere else, they will be transported to the right location by ships. These ships will also bring the floating bodies also approximately in the right position for coupling. So provisions for the towing or pushing by ships to bring the bodies together have to be made.

The final part of this bringing together can best not be done by pushing or towing by ships, because the fine positioning will not go well with ships. Furthermore, it will be very hard to keep the ships and floating body at the exact location.

So this final bringing together can best be done by making use cables. So provisions for these cables should be made. With a block tackle the cables can be pulled and the gap between the bodies can be closed.

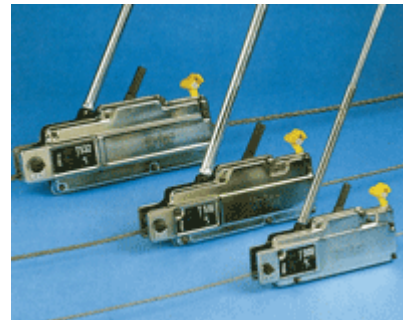


Figure 31: lever hoist [www.technolift.co.uk]    Figure 32: Tirfor tackles [www.ilsa.be]

### Make use of self alignment

Floating modules will easily move, especially when they are relatively small. This will especially cause difficulties in connecting on water with waves or current, since then the floating bodies will behave dynamically, so a small window to secure will exist. On the contrary, large floating bodies will not easily move along, but this means they are also hard to place exactly at the right spot. When the floating bodies align themselves in the correct position if they are being pressed together this makes connecting far more easy.

This self alignment can be done by making use of shaped edges which will interlock when pressed together (this can be applied for floating bodies with no space in between). An example of a shaped edge are trapezoidal ridges and cavities, see Figure 33.

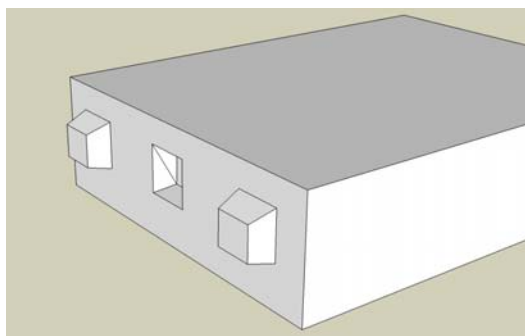


Figure 33: Trapezoidal ridges and cavities for self alignment



The self-alignment can also be beneficial when the separate bodies are heeling, or have a slight uneven draught.

### **Level the floating bodies out**

With a shape connection for self alignment the floating bodies can interlock also when they are not positioned exactly opposite each other or if they are a little tilted. But when the floating bodies are heeling too much or have a different draught, it might be possible that the shape connectors cannot interlock. Then the floating bodies have to be levelled out. This can be done by filling or emptying the watercellars or by using trim weights.

### **Firstly: Rough position coupling**

If the floating bodies are brought together, the connection can be realised. But when the floating bodies are not connected yet, the floating bodies will still perform relative movements. Even with shaped edge surfaces like the trapezoidal nocks this remains possible, since tolerances should be taken in account and the bodies can always perform free relative movement in one direction, most often this will be the relative surge direction and the gap between the bodies will not be completely closed. So the first connecting mechanisms have to allow some tolerances. Then it should be possible to tighten the joint with this realised connection, so relative movements will be prevented.

The best solution would be a two step connection, where the elements are coupled first in a rough positioning and then tightened exactly.

So there are two situations for the connection:

1. the just explained 'first connecting during execution' with tolerances.
2. the connection during usage with less relative movement

These two connection steps and two connection phases could be realised with the same connectors, but can also be realised with different connectors.

In fact the mentioned cables with block tackle can be seen as a connector for the first rough connection step. When the cables are placed only on top and tensioned by the block tackle, so the bodies are tensioned together, the only relative movement the bodies can perform are the relative movements allowed by the tolerances of the shape connection and relative pitch. With an elastic material in between the joint, this relative movement admitted by tolerances will not (or far less) occur.

### **Secondly: Couple exactly**

When a connection is realised and the joint is closed, subsequently the connection which is strong enough for during usage can be realised: the exact coupling. This can be done by tightening the first connection or realizing another connection.

Realisation of this connection is easy when following criteria are fulfilled:

- Limited amount of operations
- Easy operations
- Connectors well accessible, preferably from top
- Limiting risk on problems

Especially the bottomside connectors below water level are mostly hard accessible. It is best as all operations for the connections can be done from the top or from the inside of the floating body. Eventual problems should be avoided. An example of a problem during execution is water in a prestressing duct.

## 10. Design Disconnectable Rigid Connection

In chapter 7 all possible connection types are mentioned. One of those options is considered in this chapter; the rigid connection without intermediate distance is considered. The other connection types are elaborated in appendix 17 and 18 and are categorized with what relative movements they allow.



Figure 34: Floating bodies with no intermediate distance

The rigid connector is elaborated more extensively because this results in the least relative movements and the best hydrodynamic behaviour. It has been found that for the standard conditions for most floating structures in the Netherlands, which are situated in less severe wave conditions, there is no need to introduce more degrees of freedom if the connections and floating body are made strong enough.

### 10.1 Elements of rigid connection

#### Separate in several structural parts

The functions of the connector can be separated over several separate structural parts. This will lead to a connector easier to produce and construct. Every degree of freedom can be limited or restrained individually by an separate connector.

As said in chapter 6, there can be made use of shaped edge surfaces for easy alignment, for example trapezoidal ridges. Another function of these ridges is that they can pass on the shear force, in vertical as well as in horizontal direction. When these ridges are used, a rigid connection can be realised by adding tension members in top and bottomside area. The compression can be realised by the contact surfaces.

So the connection should consist of the following:

- trapezoidal ridges (nokken): for alignment and shear force
- contact surface: to transfer compression forces
- topside and bottomside connection: for transferring tension forces

This is depicted in Figure 35.

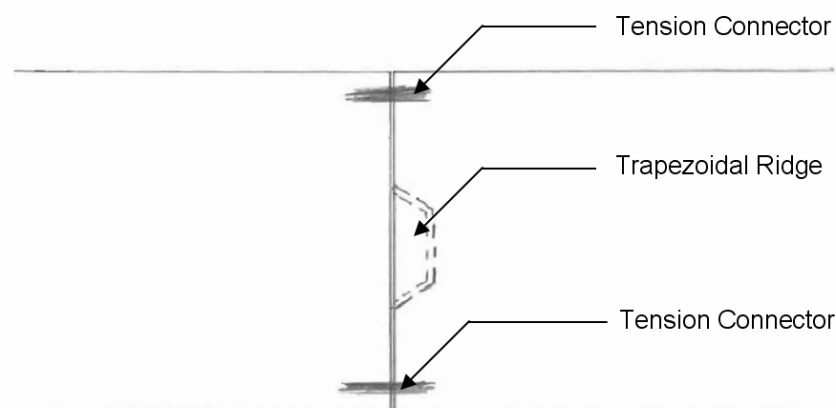


Figure 35: Basic principle of rigid connector with trapezoidal ridge and tension connectors topside and bottomside

With using this system, now in fact only the best tension connectors have to be chosen. For these topside and bottomside tension connectors, the possible alternatives are given in the next paragraph.

### Pre-stress of the connection and Elastic material in between

When the connection will be pressed together by prestressing the connection, the shear ridges will function better. On top of this, pre-stress of the connection gives a lot of other advantages (see 6.2.2), so if possible, pre-stress of the connection should be applied.

An elastic material in between can be easily prestressed if the edge surfaces are pulled together. This way the connection gets prestressed while the peak stresses will be divided over the surface. This way all advantages of pre-stress can be easily realised.

Most important benefits of using elastic intermaterial is the accommodating of tolerances and preventing relative movement and damping undesirable dynamic loading.

Another benefit is the impact damping during coupling.

### Pre-stress only in topside connector

If the connection consists out of two connectors on tension (an topside and an bottomside connection), then with the right contact shape or initial heel it is in fact only required to tighten one of the two connectors for prestressing both topside and bottomside connection, or in fact the hole connection for that matter. Since the topside connector is best accessible, it is most easy to apply the pre-stress in this topside connector.

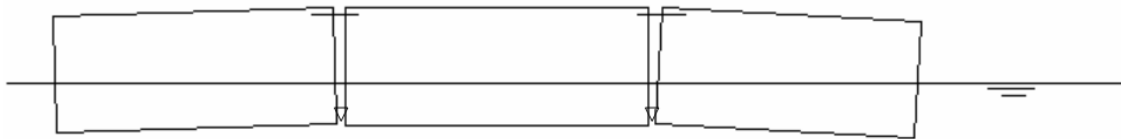


Figure 36: Modular floating structure with initial heel

In Figure 36 a modular floating structure with initial heel of the outer elements is depicted. When it is first interconnected at the lower parts, there remains a cavity between the upper parts. With applying the proper connection at the topside, this gap can be tightened.

In case an elastic intermaterial is used this should be pressed together when realising the bottomside connection. This can be realised quite easily, since at this stage only the lower area of the elastic material makes contact. Now if the space above will be tightened, the offset between the floating bodies underneath will tend to become larger if this is not prevented, since the floating bodies rotate around their centre of gravity. By the rotating around the centre of gravity and the pressing of the elastic material, the offset underneath will become larger, but when this is prevented by the connection, this connection will become under tension.

When multiple floating bodies have to be connected to each other, or when the coupling has to be done in all directions, this principle of pre-stress at the bottomside connector will not work. But in this situation it will also not be needed, because in that situation the floating bodies will not be able to perform any relative motions anymore if they are tensioned strongly together by the topside connection.

## 10.2 Connection alternatives

For the possible alternatives for tension members in the rigid connection, the basic options mentioned in paragraph 8.1 are considered. In chapter 8.5 is said which options were striped off, and which options remained. All remaining options can in principle be used. Illustrations and remarks on the remaining options are discussed in chapter 8. Selection is done in the next paragraph.

Again is mentioned that these connection will be combined with ridges for shear force.

When the shaped surface is used anyway, this means that option 4b automatically becomes equal with option 4c. So option 4b is omitted in the evaluation. Option 4c is in fact very similar with the Hann-Ocean connector (see paragraph 8.2 and appendix 15) so for evaluating this variant, the Hann-Ocean connector taken as example. Alternative 5, the male/female connection with vertical fastener, is used in the design of Flexifloat (see paragraph 8.2 and appendix 15), so in this evaluation in fact the Flexifloat option is evaluated.

### 10.3 Evaluation/choice of alternatives

The evaluation of the options is done by a multi criteria analysis. For the MCA the criteria of paragraph 6.2 are used.

The filled in MCA is depicted below. In next paragraph is explained how this is done and what assumptions are taken.

Alternatives	Criteria	Strength										Practicability										Total	
		Tension	Compression	Shearforce	Rigidity/Movements	Prestressing	below	above	Amount of operations	below	above	Risk on problems during execution	below	above	Self Alignment	Accommodating tolerances	Robustness	Durability and Maintenance	Experience	Impact damping	Costs	below	above
1 Bolt/pen		3	3	0	1	5	4	3	4	3	4	1	5	5	5	5	5	5	5	5	38	44	
a horizontal transverse		4	4	4	1	5	4	3	5	4	5	4	5	5	5	5	5	5	5	5	48	55	
b horizontal longitudinal		3	3	0	4	5	4	4	4	4	4	2	5	5	5	5	5	5	5	5	44	45	
c vertical		4	5	5	4	5	2	2	3	3	4	5	4	4	4	4	4	4	3	3	45	48	
2 Pressing together (cables/rods)		4	4	4	3	5	3	2	4	5	5	5	3	4	4	4	4	4	4	4	45	49	
b just pressing together		4	3	0	4	4	2	1	1	1	1	2	4	3	2	3	3	3	3	3	29	29	
3 Cables		4	3	1	4	5	4	4	5	5	5	4	3	4	3	4	3	3	3	42	44		
4 Shape		3	4	1	5	5	5	5	5	4	4	3	3	4	4	4	4	4	4	4	45	45	
a puzzle		2	2	2	5	5	4	4	5	4	4	1	1	4	2	4	2	4	4	35	36		
b separate puzzle pieces		2	2	2	5	5	4	4	5	4	4	1	1	4	2	3	2	3	3	34	35		
c combination		2	4	4	1	5	2	1	5	4	4	5	1	2	3	2	3	2	2	31	39		
5 Male/Female		3	3	0	5	4	3	4	3	4	2	3	4	4	3	4	4	3	3	39	35		
7 Hook		3	3	0	5	5	3	3	4	2	4	3	4	4	2	4	4	2	3	35	38		
a single hook		2	4	4	5	5	4	5	5	1	1	5	1	1	4	1	4	1	1	38	38		
b double hook		2	4	4	5	5	4	5	5	1	1	5	1	1	4	1	4	1	1	38	38		
c Improved Ribbon bridge solution		2	4	4	5	5	4	5	5	1	1	5	1	1	4	1	4	1	1	38	38		
10 Confinement (opsluiten/inklemmen)		3	3	0	5	4	3	4	3	4	2	3	4	4	3	4	4	3	3	39	35		
a from center outwards		3	3	0	5	5	3	3	4	2	4	3	4	4	2	4	4	2	3	35	38		
b from the outside		3	4	4	5	5	4	5	5	1	1	5	1	1	4	1	4	1	1	38	38		
11 Pneumatic/Hydraulic Jacks		3	4	4	5	5	4	5	5	1	1	5	1	1	4	1	4	1	1	38	38		

Figure 37: Multi Criteria Analysis for detachable rigid connection

#### Remarks on MCA

##### Score

Every criteria is given a value from 0 up to 5, where 5 is the best. At first, every criterion has got the same weight.

##### Difference topside and bottomside

For the topside and bottomside connector can be chosen for a different alternative. So, all alternatives are evaluated for both topside and bottomside conditions. So at the criteria where the position is important, the alternatives are given a value for both topside and bottomside position.

##### Omitted criteria

The criteria which are marked red, are not taken account in this MCA, because these are the criteria which are fulfilled by the trapezoidal connections and elastic material.

##### Not accessible caissons

For the point accessibility the remark is made that this analysis is done for connections with floating bodies which cannot, or can hardly be reached from the inside. This is for example the case for EPS filled bodies. So if the connections can be reached from the inside, for example hollow concrete or steel caissons, some options might get more points for accessibility.

More information

For the best ranked alternatives, some background is given in appendix 15 on values given here, especially why some options have got low values at some points. This is done for alternative 4c, which is as said approximately equal to the Hann-Ocean Connector, so this is treated in the appendix on the Hann Ocean Connector. The Male/Female connection with vertical fastener is treated under the Flexifloat connection, which is in fact the same.

**Topside connection**

First a look is taken at the topside connection. For the topside connection there is a clear winner. This is the horizontal longitudinal bolt. With the longitudinal bolt pre-stress is well possible and it results in a simple and robust connection. This solution in fact scores at every point very well, so it doesn't appear possible to improve it by combining it with other alternatives.

**Bottomside connection**

The connection at the lower area is the first firm connection that shall be realized during the execution process, so this connection should be easy to realise.

If the pre-stress will be realised with the topside connection, pre-stress has not to be realised with the bottomside connection (see 10.1). So the high scoring alternatives are recalculated, now without pre-stress as criteria, see the table below.

Alternatives	Strength			Practicability					Other Criteria					Total		
	Tension	Compression Shearforce	Rigidity/Movements	Prestressing	below Accessibility	Amount of operations	below Difficulty of operations	below Risk on problems during execution	Self Alignment	Accommodating tolerances	Robustness	Durability and Maintenance	Experience		Impact damping	Costs
1 Bolt/pen																
b horizontal longitudinal	4	4	4	1	4	3	4	4	4	5	5	5	5	5	5	44
c vertical	3	3	0	4	4	4	4	4	4	2	5	5	5	5	5	44
2 Pressing together (cables/rods)																0
b just pressing together	4	5	5	4	2	2	3	3	5	4	4	4	4	3	4	40
3 Cables	4	4	4	3	3	2	5	5	5	3	4	4	4	4	4	41
4 Shape																0
c combination	4	3	1	4	4	4	5	5	4	3	4	3	3	3	4	41
5 Male/Female	3	4	1	5	5	5	4	4	3	3	4	4	4	4	4	44

Table 1 Multi Criteria Analysis for detachable rigid connection, bottomside connection, without pre-stress

From the MCA it appears that the bolt/pen alternatives and the male/female (Flexifloat) alternative have got an equal amount of points. To take a decision on what alternative shall be chosen, there can be taken a look at what criteria are found more important.

Below the accessibility for placing the horizontal longitudinal bolt is very poor. This accessibility is found to be more important than some other criteria, so then the longitudinal bolt option can be omitted.

The two alternatives left, the vertical bolt and the male/female flexifloat alternative are in fact quite similar. In both alternatives there is a vertical element which in fact realizes the connection. It is possible to combine each other's positive points to an optimal connection. This is done in the case study in part III. First, a design with the short vertical bolt is made. This initial design is depicted in the next paragraph and the designed bottomside connection is evaluated here.

## 10.4 Resulting Disconnectable Rigid Connector

It was found that the optimal disconnectable rigid connector consists out of the following elements:

- trapezoidal ridges: for alignment and shear force.
- topside connection: horizontal longitudinal bolt: for tension, tightening the joint and pre-stress.
- bottomside connection: combination of vertical bolt and Flexifloat connection: for easy connecting and tension.

A sketch of the connection with a short vertical bolt as bottomside connection is given below. On the right, a 3D sketch of the topside bolt connection is given, designed in a way that it can be placed very easily. This is one of the possible options for the longitudinal bolt connection; in the case study this design is adjusted.

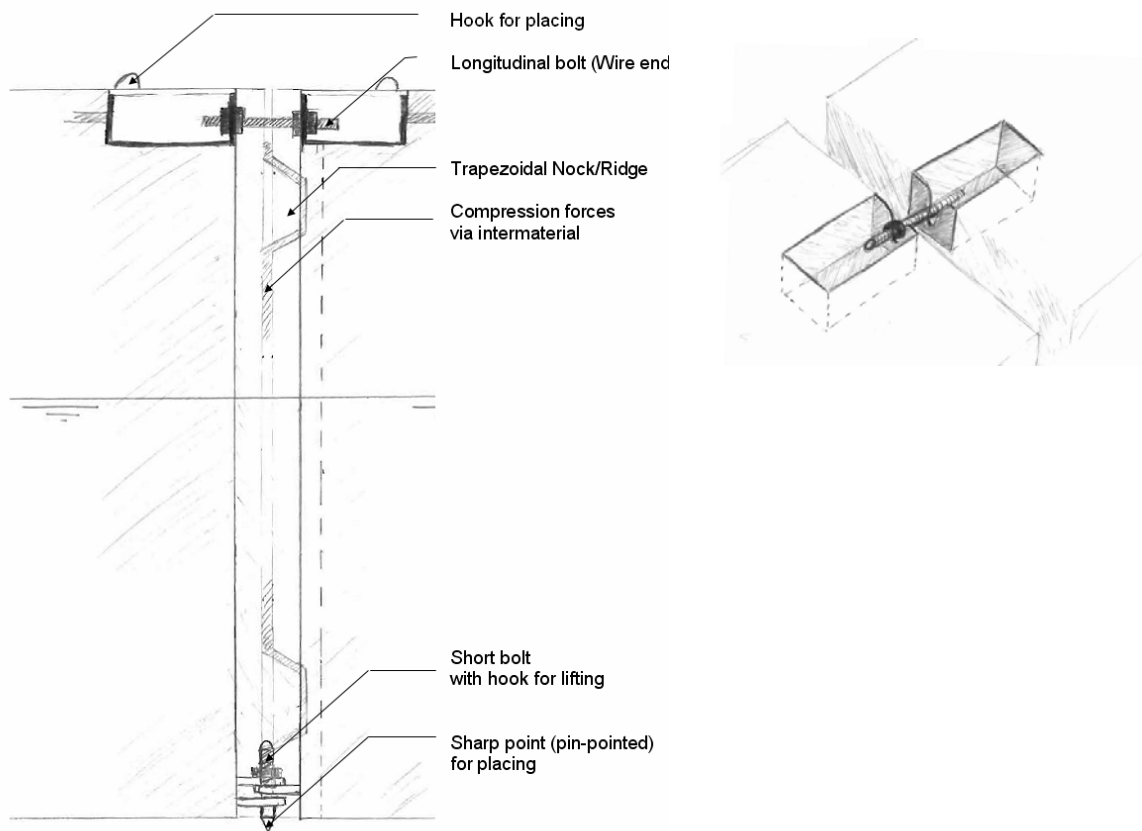


Figure 38: Sketch of connection with below vertical bolt, side view and sketch of topside detail

### Realizing bottomside connection

The problem for every low connection is that it is not easily accessible. Another problem is that the bottomside connection is below water surface, so the visibility will be poor.

But in this connection the bottomside connection can be realised quite easy (in comparison with some other alternatives) with the vertical pen. The pen will be pin-pointed so, it will 'find' the hole itself. With a pin-pointed pen a certain amount of location deviation doesn't matter. Multiple methods are thought of for placing, this won't be the problem; however fixating the short pen is a problem.

### Placing the low pen/bolt

The placing of the bolt can be done by placing it with a hollow cylinder. The pen or bolt that realizes the connection can be placed and fixed within the end of the cylinder.

This fixation of the bolt/pen in the cylinder can be done with several mechanisms:

- By a tight fit which holds the bolt as long as there no real tension on it. Then the bolt can be fixed in the connection by screwing or clicking. By subsequently pulling the cylinder up, the bolt will come loose out of the cylinder.
- By a screw connection. By continuing rotating the hollow cylinder when the bolt is fixed, it gets out.
- By a thin rope through the cylinder. Which has a small hook on it, which is hooked to an eye welded on the pen

### Problems with fastening short bolt

When the short vertical pen is placed it has to be fastened. Normally, fastening a pen through locking plates, would be easily done with a nut on both ends. But a nut on the underside is difficult to place, because it is hardly accessible.

### **Choose for long pen**

For this short pen connection multiple ways to make an easy fixation has been thought of. But all these connections are found less robust, so a long pen which can be connected on the topside might be better. This long pen brings along some extra material costs, but will be a lot more easy in execution. The long pen can also have a more wedge shaped long point, which is even better for allowing location and size deviations and can also apply some extra pre-stress in the bottomside connection (see case study for explanation and drawings).

In the case study the depicted short bolt connection and the Flexifloat connection are combined to an optimal connection. This connection makes use of a long vertical pen. This connection has been dimensioned and calculated for the situation of the case study and further elaborated in part 3. Below a preview of the final result is given:

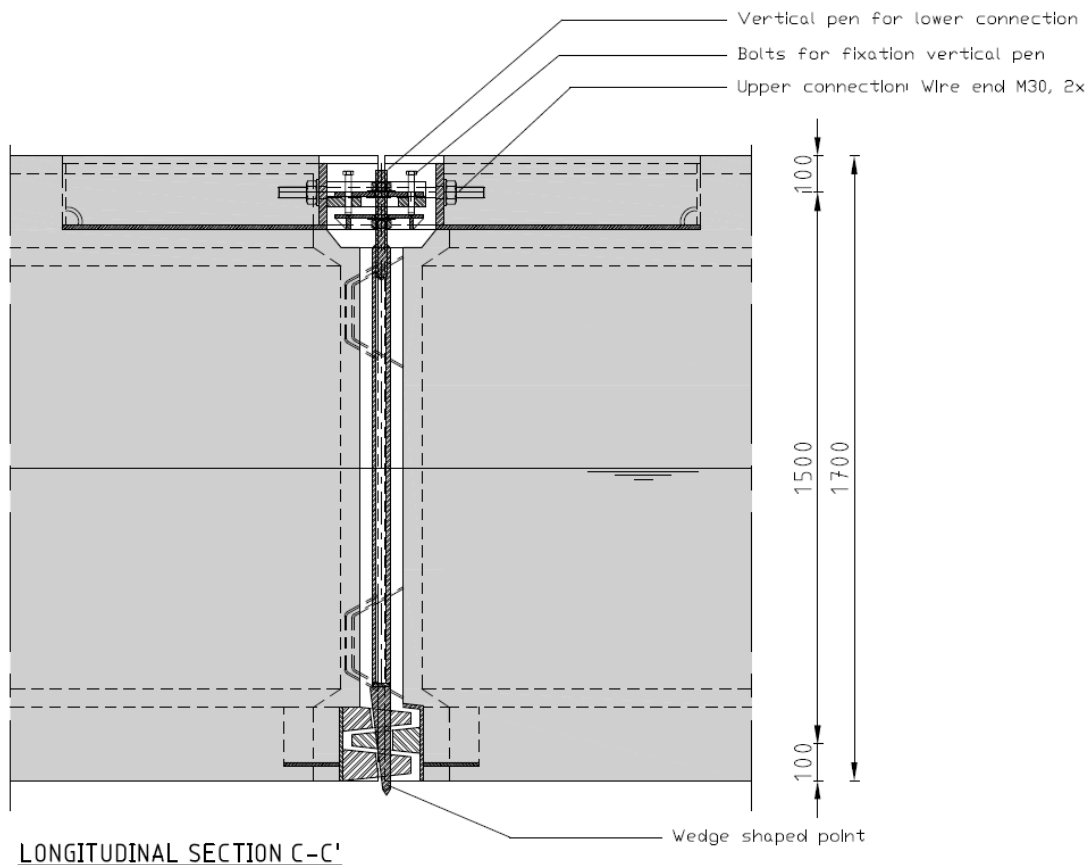


Figure 39: Rigid connection, result case study





**Part III**  
**Case Study**  
**Floating Pavilion**

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## Part III

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# 11. Floating Pavilion

## 11.1 Background

### Design pavilion

In 2007 Delta Sync won a design competition of Royal Haskoning with a design for a floating city. DeltaSync and the municipality of Rotterdam presented this design in a simpler form, in the form of a pavilion, to the organization of World Expo Shanghai 2010. But the organization in Shanghai had no interest.

Then the former mayor of Rotterdam, Ivo Opstelten, and Projectbureau Klimaat "adopted" the design, with the intention to realize the pavilion in Rotterdam, so they would also have a pavilion in Rotterdam, at the same moment they would participate with an the World Expo in Shanghai.

### Rotterdam climate initiatives and purpose pavilion

The city of Rotterdam is doing a lot to become an environmental friendly city and to adapt to the changing climate. They have the strive to be one of the leading cities on this field. Here to amongst others the Rotterdam Climate Initiative (RCI) has been formulated, an ambitious climate program, with the objective to reduce the CO<sub>2</sub> production with 50% by 2025 (compared to 1990). Next to this there is the program Rotterdam Climate Proof (RCP), to make Rotterdam fully water proof by 2025. [www.rotterdamclimateinitiative.nl]

Amongst others to show their efforts, plans and designs in the field of climate, delta and water technology Rotterdam takes part in the World Expo 'Better City, Better Life'. The city board also wanted to show their sustainable plans and their sustainable technology in Rotterdam.

Thus the purpose of the Floating pavilion is creating an information center and exhibition space which shows Rotterdam's present and future ambitions at the field of climate, delta and water technology, and gives an overview of what has been achieved in these areas. [RCI, 2010]

With events such as World Expo Shanghai 2010, Dutch Delta Design 2012 and the start of the Tour de France in Rotterdam, Rotterdam finds itself in the international spotlight. Therefore they wanted an eye-catching icon project, a project with which the city can show it self as climate-conscious city, with a lot of knowledge in the field of climate and water. So the floating pavilion has to act as this icon project, an international model of delta technology and climate adaptation, where VIPs can be received. [Gemeentewerken, et al., 2009]

### Floating in Rotterdam

The Floating Pavilion is a showcase and test case for climate consciousness and building on water at the same time. Floating is one of the water adaptive technologies which is part of Rotterdam Climate Proof. The floating pavilion is the pilot project for Rotterdam's floating ambitions.

To Rotterdam the option of building on water is very interesting. Because with the construction of the first and second Maasvlakte, many port activities have moved Maasvlakte or will shift towards the second Maasvlakte when this will be finished. This means 1600 hectares of former port sites will be available which can be developed for new destinations. The plans for building floating quarters in this old harbours are already in advanced state, see appendix 4.

### Delivery

In June 2009 the decision was made to build this pavilion, so at that time was started to develop the preliminary design and engineering.

The World Expo 2010 started on the first of may, so that date was the planned delivery date for the floating pavilion. This meant that the design and engineering had to be done in a very short period of time, because the building already had to start in September to meet the completion date. To gain time was decided to realise this project in a 'bouwteam' (building team alliance).

The planned delivery date of May 1 has not been met. But just before the start of the Tour de France on the 25th of June 2010, the pavilion was officially opened in presence of the minister of VROM. During the opening has been announced that the pavilion will be the paragon and centre of expertise of the new to be established National Water Center.

[Gemeentewerken, et al., 2009; Bestuursdienst, 2010]

## 11.2 Location

The pavilion will be built in the Heijsehaven and then towed to the Rijnhaven. There, the pavilion will be placed for the first few years near the Tillemakade, see Figure 1 and Figure 2. Thus the pavilion will be placed next to the new epitome area of Rotterdam with the 'Kop van Zuid' with it's new skyscrapers and close to the Erasmusbrug.



Figure 1: Location Heijsehaven and Rijnhaven (GoogleEarth)

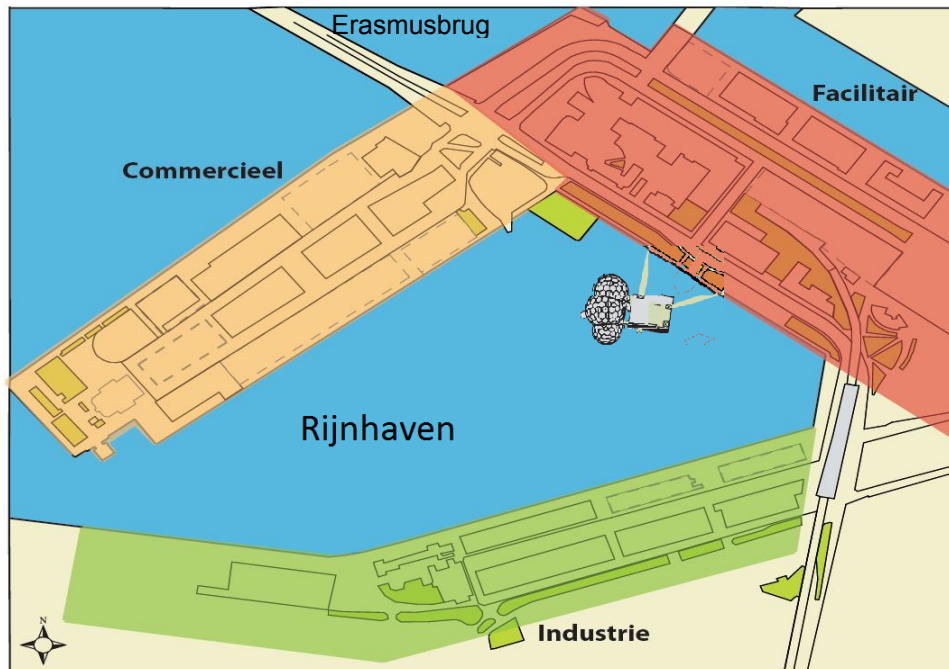


Figure 2: Location pavilion in Rijnhaven, with functions around Rijnhaven

### Depth and Water heights Rijnhaven

The Rijnhaven is in connection with the North Sea via the Oude Maas and Nieuwe Waterweg. So the Rijnhaven has tidal movement. The water heights are given below, they are derived from measurements of Havenbedrijf Rotterdam, Hydro Meteo informatieBundel nr. 3. The heights and wave levels are given related to NAP, Normaal Amsterdams Peil.

#### Height Quay Rijnhaven:

- Quay Height + 3,34 m NAP. [PDA et al., 2009]

#### Water heights

- Extreme	+ 2,60 m NAP.
- Average High Water Spring Tide (GHWS):	+ 2,03 m NAP.
- Average High Water level (GHW)	+ 1,31 m NAP.
- Average water level	+ 0,25 m NAP.
- Average Low Water level (GLW)	- 0,45 m NAP.
- Average Low Water Spring Tide (GLWS)	- 0,98 m NAP.

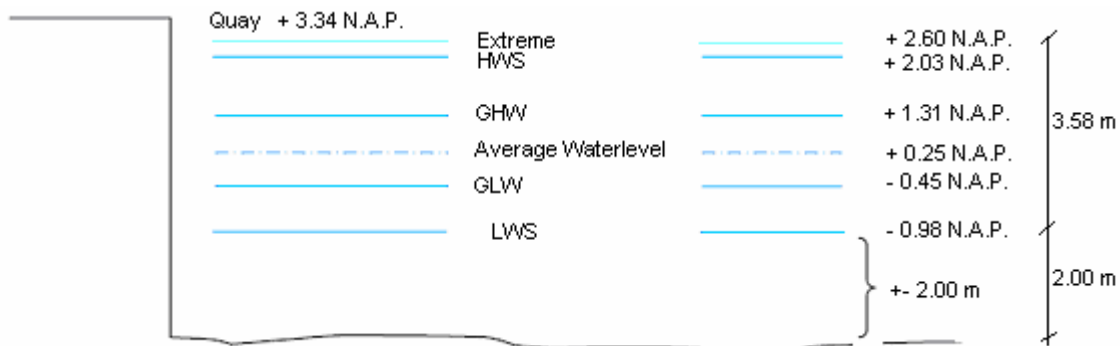


Figure 3: Waterheights Rijnhaven

#### Bottom level

The bottom level of the Rijnhaven once had a depth of at least – 5,00 m NAP., but there has been dumped a large amount of pumice (mining waste) in the harbour at the location where the pavilion will be located (see Figure 4), since this part of the harbour was not in use any more. So now the bottom level finds itself at self at a depth of 3 approximately metres below NAP, but at some spots it could be less than two metres. When this is necessary for the pavilion, the bottom can be equalised and deepened by shoving the pumice to the centre of the Rijnhaven where it has a larger depth. But this brings high costs with it.

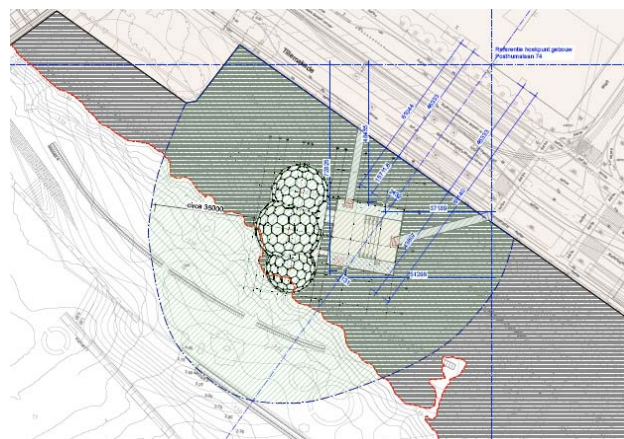


Figure 4: Location floating pavilion in Rijnhaven with depths Rijnhaven

## 11.3 Design criteria

Below a list of design criteria is given with the most important and most relevant wishes and demands of the client incorporated:

### Location

- first location will be the Rijnhaven
- after 5 years it shall probably be relocated to another site
- the orientation for the DeltaSync/PDA design is fixed, because of the internal placing of solar panels and building physical measures

### Freeboard, tilt, draught:

- maximum skewness: 5% [VROM Inspectie, 2009]
- preferred maximum skewness: 1%
- freeboard in normal use: 80 cm (for preventing ascending)
- minimum space underneath structure: 50 cm (for water current and preventing silting)
- preferably a draught of pavilion which gives no need for dredging

### Functional Requirements

- pavilion suitable for expeditions, conferences, lectures and meetings
- pavilion accessible for 250 persons
- auditorium for 150 persons
- multi purpose
- removable
- floating plaza accessible for water taxi
- two bridges for safety, one accessible for ambulance

### Low maintenance

- as low maintenance as possible

### Legislation

- classifying as real estate
- fulfil all building codes

### Model for climate awareness

- the pavilion has to become a model for climate awareness
- use of environmental friendly products
- small energy consumption
- self-sufficient

### Pilot project floating

- paragon for floating technology
- gaining floating knowledge

### Budget

- 5 million euro's

### When modular

- easy connecting and disconnecting
- design of connection preferably also usable for other floating structures and multiple floating systems

For this list is amongst others made use of the program of wishes composed by PDA (2009), information from the building team meetings and out of conversations with the project manager Birsen Hofmans.



## 11.4 Design

The design of the floating pavilion exists out of two floating isles: the pavilion isle and the plaza isle. The pavilion isle has a superstructure existing of three half domes, from which the center dome is the largest with a height of 12 meters. The idea of the architects behind this design that the domes look like soap bubbles on the water. The pavilion is 46 meters long and 24 meters wide, the floating plaza measures 24x24 meters. The pavilion and floating plaza are two separate floating structures connected with each other. The floating plaza is moored with 2 large mooring piles, the pavilion is on it's turn moored to the plaza. The floating plaza is connected to the quay with two steel bridges. Below two impressions are given and a top view are given.

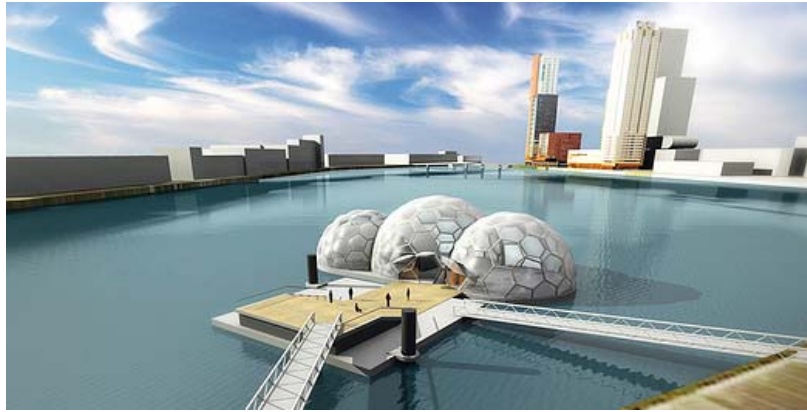


Figure 5: Impression floating pavilion with on the background the 'Kop van Zuid' (DeltaSync / PDA)



Figure 6: Impression floating pavilion with on the background the 'Kop van Zuid' (DeltaSync / PDA)

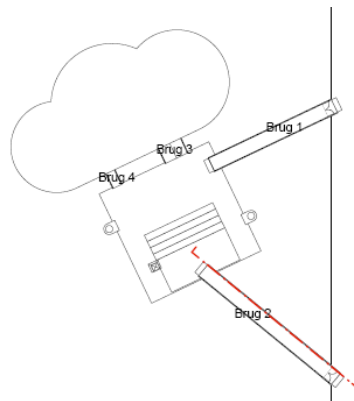


Figure 7: Top view pavilion, plaza and bridges to the quay

In the figure below a section in length direction of the pavilion is given. In this image the special sustainable installations and materials of the pavilion are mentioned, in paragraph 11.5 is mentioned what is done make the pavilion sustainable.

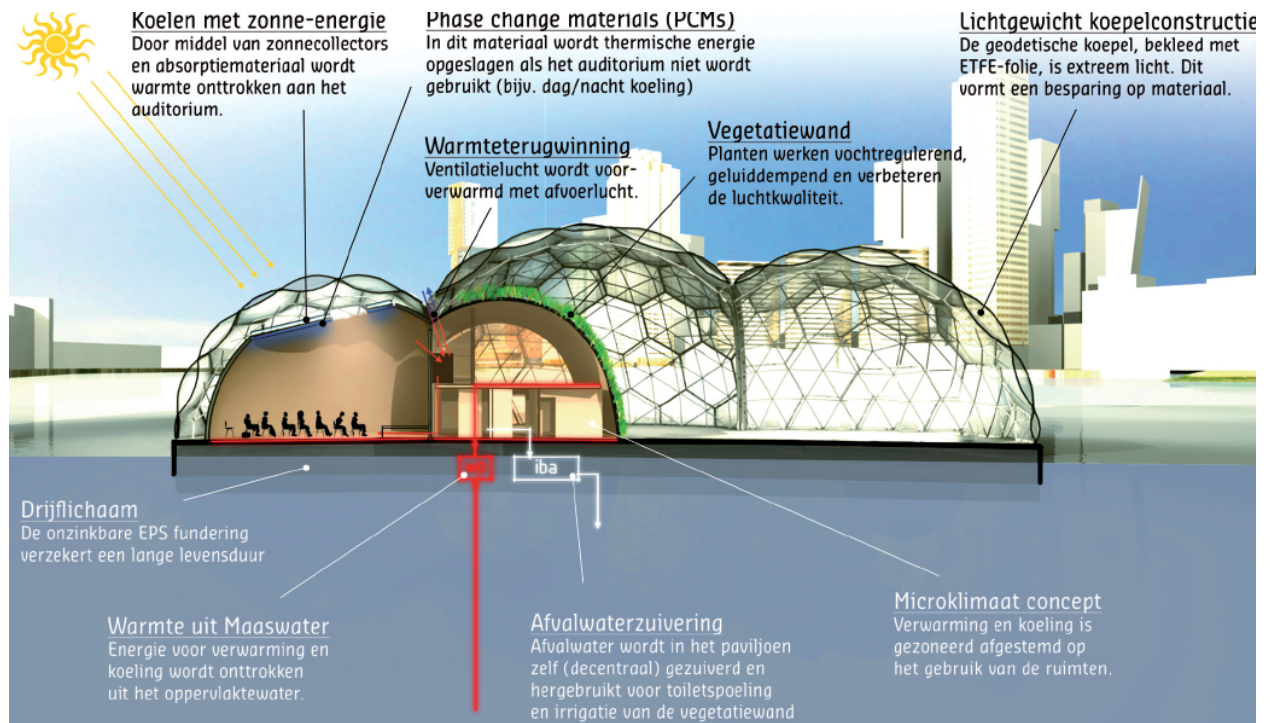


Figure 8: Image floating pavilion with it's sustainable specialties (PDA/DeltaSync/RCI)

### 11.4.1 Shape Description

#### Superstructure Pavilion

The superstructure of the pavilion consists of a dome structure composed out of three connected hemispheres. The diameters of the three domes (centrelines bearing structure) are: 17.1, 23 and 19.7 meters.

These hemispheres are slid together and partially slid in to each other. Where the spheres intersect, that part which falls within the other dome is cut off geometry. In the images below two section lines are depicted. The cutting lines shown here indicate where the domes are 'cut', so they are not complete hemispheres anymore. These cutting lines also seem logical section lines for when the pavilion will be divided into modules.

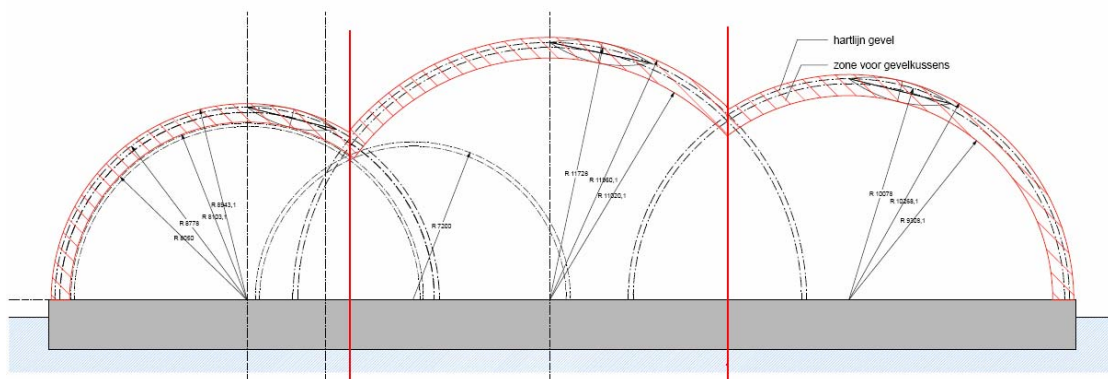




Figure 9: Length section pavilion, with section lines

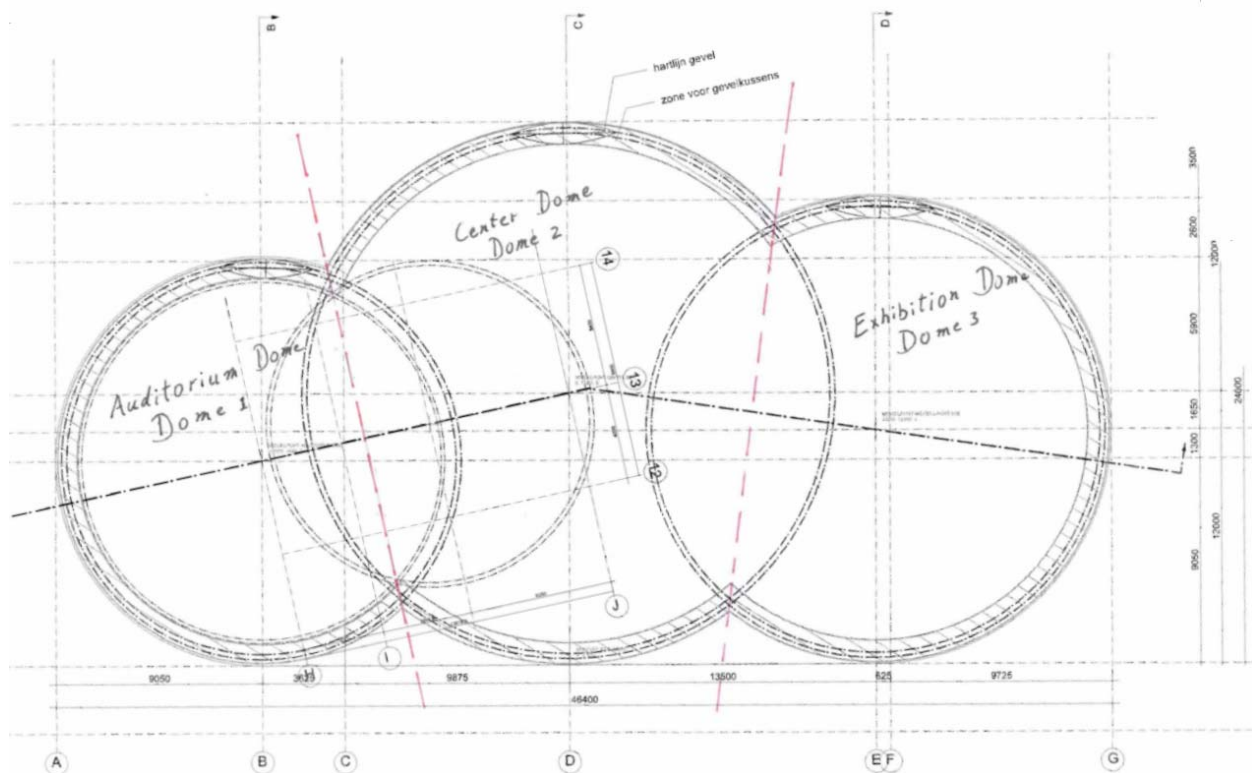


Figure 10: Top view, with circles for geometry and section lines

The bearing structure of the superstructure has a length of 45.4 metres and a width of 23 metres (the diameter of the center dome). The outer diameter of the facade will be somewhat larger.

### Shape Floating Body Pavilion

The surface area of the floating body float will have the same shape as the superstructure, but slightly wider. The supports of the steel structure will be 500mm inwards from the edges of the floating body. Thus the radii of the circles, where the geometry of the floating body exist of, are 1 meter greater than the radii of the circles of the super structure. This results in circles with radii of 18.1, 20.7 and 24.0 meters.

The total length of the float is 46.4. The width is 24.0 meters.

### Surfaces

For the calculating the freeboard, draught, etc. the exact surface of the floating body is very important.

The surfaces of the three circles, with their cut of part, which are the main part of the floating body are the following: (The surface area is calculated is calculated in appendix 20):

-Surface area dome 1, Auditorium Dome:	209,12 m <sup>2</sup>
-Surface area dome 2, Center Dome:	376,47 m <sup>2</sup>
-Surface area dome 3, Exhibition Dome:	276,24 m <sup>2</sup>

On the plaza side of the floating body has a straight edge. So except for the three parts of circles, the floating body has some additional surface.

-Extra surface (triangle 1) Auditorium Dome:	6,92 m <sup>2</sup>
-Extra surface Center Dome (triangle 2 and 3):	12,17 m <sup>2</sup>
-Extra surface (triangle 4) Exhibition Dome:	5,34 m <sup>2</sup>

### 11.4.2 Functional Description

Figure 11 shows the floor plan of floating pavilion. In this floor plan the functional use of the pavilion can be seen.

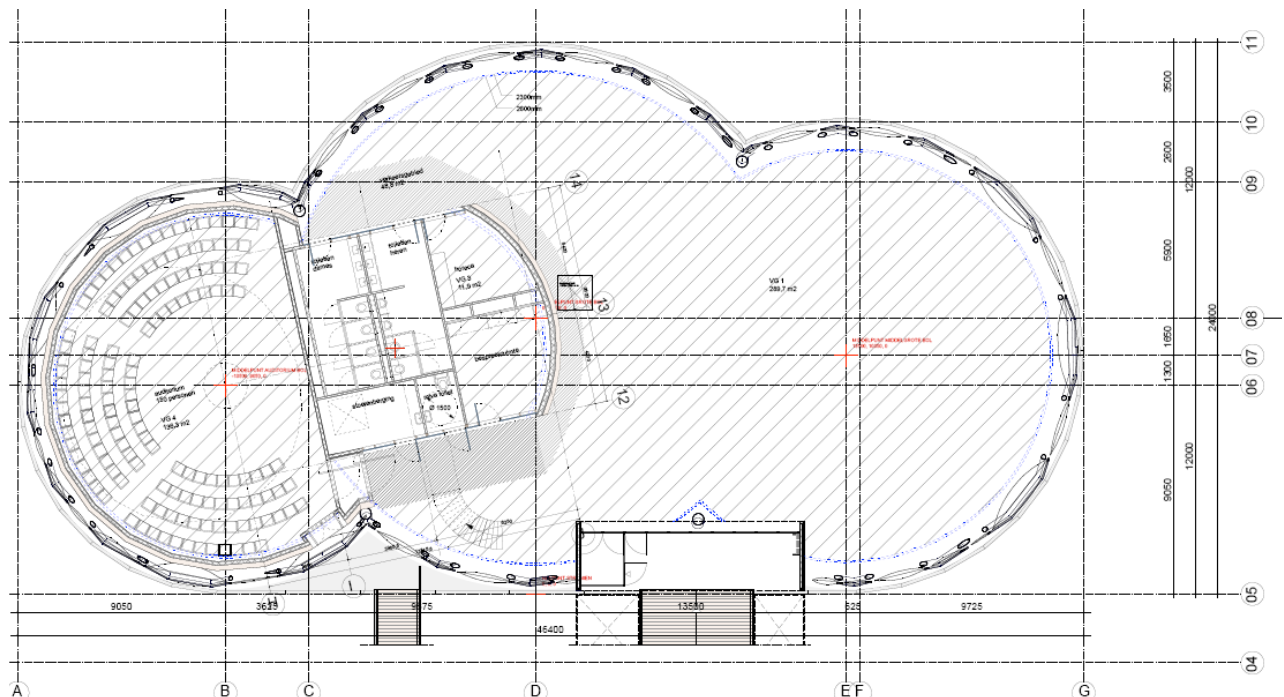


Figure 11: Floor plan Pavilion

#### Auditorium

In the left sphere, the auditorium dome, the chairs of the auditorium are depicted. This dome accommodates 150 people. This auditorium needs to have an indoor climate, while the 'exhibition' part of the pavilion can have a semi-outdoor environment. Therefore this auditorium will have an inner wall with isolation.

#### Internal buildings center dome

The center dome has built-in structures also. These structures house the restrooms and meeting rooms and exists out of two floors. These spaces also need to have an inner climate. The circular inner wall (which can be seen in Figure 11) will be executed as a 'green wall', a wall with vegetation. This green wall is for environmental exude, but it also functions in climate control. The central dome also houses a cellar. In this cellar and on a part the second floor, the installations will be situated.

#### Exhibition space

The space which is left, half of the center dome and the complete right dome (dome 3) will be exhibition space.

## 11.5 Sustainability and self sufficiency

The sustainability aspect of the pavilion can be found in the climate control, additional electric systems and water use. The pavilion will be powered by electricity only, which will be largely generated by the pavilion itself, by making use of amongst others photo-voltaic foil. This solar energy will be used for energy-efficient LED lighting and climate control. The climate control makes also use of surface water, thermal energy storage in phase change materials, use of demand-driven natural ventilation and a vegetation wall. The climate control is made more efficient by dividing the pavilion in separate climate zones. For the toilets will be made use of surface water.

By the self sufficiency of the pavilion the autonomy is increased, thereby it will be less dependent on systems on firm land, so it will also be more easy and cheaper to relocate the pavilion.

## 12. Modular Pavilion

### 12.1 Why Modular

The pavilion will be divided into modules. This has got several reasons:

- More flexible in use
- More easy for transport
- Advantages during execution

#### More flexible in use

The splitting up will be done in a way that the pavilion is more flexible than before. In the modular configuration parts of the pavilion can be disconnected and other parts can be attached, so that another, larger or smaller pavilion is created which can be used for other exhibitions or other purposes.

#### Transport

Another reason for dividing the pavilion is that the modular parts will be more easy to be transported if the pavilion will have to be transferred to a different location. The smaller parts will not only be more easy to handle, but when the parts will have smaller outer dimensions, they might be transported to places which would not be possible with larger sizes. With smaller outer dimensions it will be possible to pass certain bridges and sluices which can not be passed by the pavilion as a whole.

#### Advantages during execution

During the production phase splitting might also be beneficial. For smaller parts it will be more easy to be produced in assembly hall or dock. Especially in case of production in an assembly hall the small sizes can be advantageous, because smaller parts can be realised in smaller assembly halls. Also the doorways of assembly halls are often limited. Next to this, with producing the pavilion in an assembly hall it has to be displaced to outside with a crane or on a trailer if it is finished. With smaller sizes this will be a lot easier.

### 12.2 Splitting up

#### Splitting at intersection domes

As well from technical as from functional view the pavilion can best be split at the line where the domes intersect each other, since at these intersections the bearing structure is intersected anyways. This splitting of the domes will be done where the center lines of the main steel structure intersect with each other, see Figure 9, Figure 10 and Figure 12.

#### Splitting domes in pieces

The domes can also be split in pieces. As stated in chapter *Construction Method* dividing the structure in a lot of part is not advantageous, since this results in a lot of expensive connection with no functional use. But with dividing the domes in 2 parts, the maximum width of the domes will almost be halved. So this might also be an option, especially from transport view.

In the table below is mentioned what the maximum sizes will be, with three different ways of splitting the structure.

	Max width (m)	
	Center dome	Outer domes
<b>complete pavilion</b>	24	
<b>3 parts, 3 whole domes</b>	20	16
<b>4 parts, center dome through half</b>	10,5	16
<b>6 parts, all domes through half</b>	10,5	11

Table 1: Dimensions modular parts

The corresponding images are given on the next page.

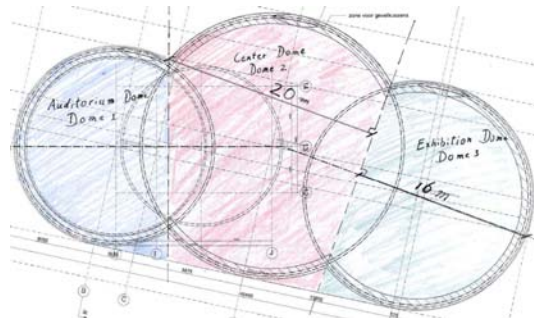


Figure 12: Pavilion split in 3 parts

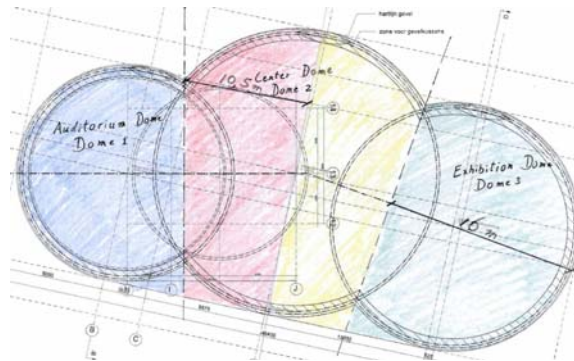


Figure 13: Pavilion split in 4 parts

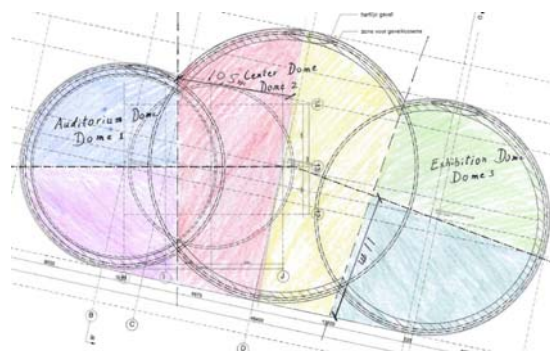


Figure 14: Pavilion split in 6 parts

The side view of splitting in three parts can be seen in Figure 9, for the other modular configurations this principle will be the same.

With the in chapter 15 chosen geometry all here mentioned configurations will be possible, but when is chosen for 4 or 6 modular parts, the geometry of the centre dome has to be adjusted somewhat, by placing beams directly next to the section lines, and rotating the perpendicular beams on this beams so they become really perpendicular. When a dome is parted, the steel bearing structure and facade does also need to have the ability to be parted, unless the splitting is done only for the floating body during construction and the superstructure will only be realised after coupling.

If the superstructure is also to be split, the bearing structure needs to be adjusted in a way it can be parted and than for the ETFE cushions it should be possible to be removed or parted, or the complete geometry should be adjusted. This option requires a lot of additional adjustments and therefore costs will be higher. Coupling these smaller structural parts will also be a lot more difficult because these not-complete dome parts will not be stable.

### Conclusion

Splitting the pavilion at the section lines of the domes brings multiple advantages with it. At this moment there are no overriding reasons for splitting the domes in smaller parts. The pavilion does not have to pass narrow bridges or sluices, and the pavilion could be built into an assembly hall or dock with doors with a width wider than 20 m. So the pavilion will be splitted in 3 parts.

## 13. Preliminary Structural Design Pavilion

In the first part of this chapter, the geometry of the superstructure is elaborated. The geometry of the here elaborated superstructure is the result of the design of DeltaSync/PDA and was adjusted by the German contractors VectorFoiltec and Teschner. The calculations and the modelling for this superstructure is completely redone for this thesis in paragraph 15.1 and here for is not made use of existing calculations or existing models.

In the second part of this chapter, the type and the height of the floating body and the materials will be determined. Second the geometry and dimensions of the concrete structure will be elaborated. The chosen floating body is completely different from the floating body which has been used for the real project, so the calculations and the modelling for that floating body could not be used. The chosen design for the floating body of the real floating pavilion is attached in appendix 18.

### 13.1 Superstructure

The superstructure of the pavilion consists of a steel dome structure composed out of three slit together hemispheres. The geometry of the domes are based on the geometry of the geodetic dome. The diameters of the three domes (centrelines bearing structure) are: 17.1, 23 and 19.7 meters.

#### 13.1.1 Geodetic Geometry

The principle of using geodetic geometry in domes constructed out of steel elements is used more often. A well known example is the Eden project in Cornwall, existing out of multiple geodetic domes.

The basic principle of the geodetic sphere is projecting a regular icosahedron (polyhedron existing out of twenty triangles) on a sphere. Each triangle in the icosahedron can subsequently be divided into a number of triangles. In Figure 15 this geometry, with the large and small triangles, is depicted on the three hemispheres of the pavilion.

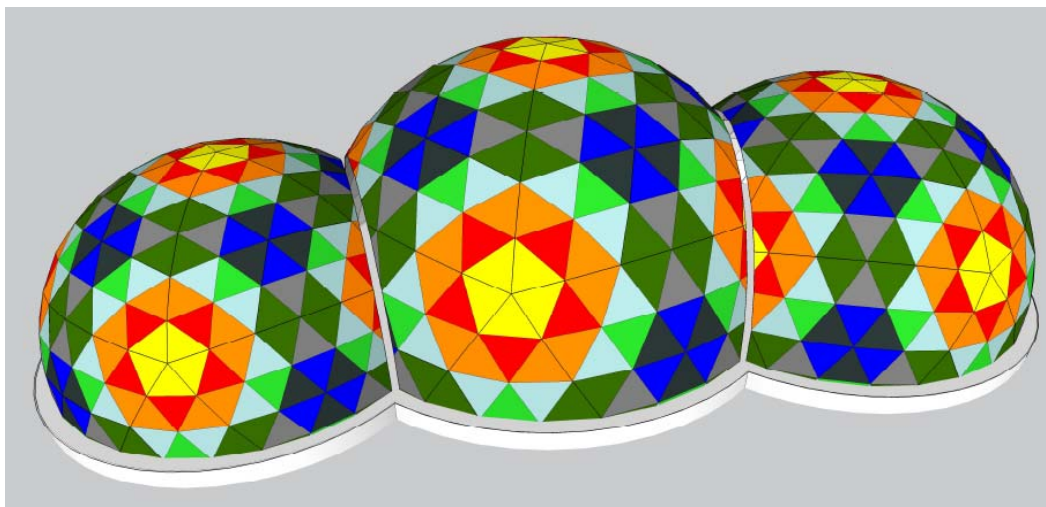


Figure 15: Geodetic geometry (proposition of Vector Foiltec), with clearly recognizable triangles of 20-plane

These small triangles can be combined to a hexagon and the edges of these hexagons will be the beam sections of the structure. Where these beam sections come together the vertices will be situated. In these nodes three beam sections come together. This leads to a geodetic geometry, to a dome mainly made up of hexagons and several pentagons. At the location of the initial vertices of the icosahedron the hexagons can be found, see Figure 15 and Figure 16.

The bold lines in Figure 16 indicate one cut of a geodetic sphere, this is dome 1, the auditorium dome. A cut from the geodesic sphere is self-supporting and. If the cuts are supported at the edges, no further support is needed.



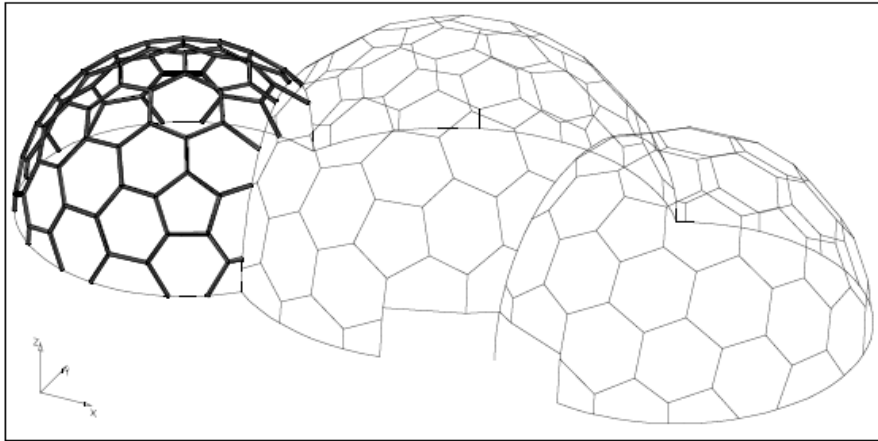


Figure 16: Geometry of superstructure, with highlighted dome 1

The center dome is called dome 2, and the right dome which is called the exhibition dome or dome 3.

### 13.1.2 Choice of material and profiles

#### Support structure

The architect opted for round tubes because this gives a smooth and modern appearance. Next to this, CHS profiles have a smooth outer surface with no edges where moisture and dirt will collect, so this is advantageous for durability and less maintenance will be necessary. On top of this, a round section which has the same properties in every direction simplifies the modelling and calculation. The disadvantage of CHS is that the connections are difficult, but there are connection methods which result in connections which are relatively easy in construction if there is made use of computerized cutting. For the pavilion is opted for such a relative easy connection, with computerized cutting and welding, this can be seen on picture in figure A19-2 in appendix 19.

For the material for the domes is chosen for steel S355. S355 is nowadays kind of the standard structural steel quality.

Aluminium could be a good alternative, because it's lighter and is less susceptible for corrosion. In this project is not chosen for aluminium, because it is usually more expensive. But for another project and certainly for floating structures aluminium could be a good alternative for steel.

#### Frameworks

On top of the steel bearing structure an aluminium framework will be assembled. This framework will consist of light aluminium profiles which will be exactly placed on top of the bearing structure. ETFE cushions will be clamped in between the aluminium profiles.

### 13.1.3 Dimensions

Per dome will preferably be chosen for one profile size only. With all similar outer diameters this results in a more aesthetic appearance. Using all similar profiles also gives advantages during manufacturing and prevents difficulties and errors at the construction site.

The required dimensions will be calculated after modelling the structure in Scia Engineer. This will be done in the next phase of the structural design. For now first will be proceeded with the estimated dimensions and weights chosen by of Vector Foiltec. These are given below:

#### Dimensions:

Dome elements, Dome 1:	CHS 193,7/ 6,3
Dome elements, Dome 2:	CHS 193,7/ 6,3
Dome elements, Dome 3:	CHS 193,7/ 6,3
Lower border elements:	CHS 193,7/ 6,3
Arches between domes:	CHS 244,5/ 8,0

So at first, the dimensions for all dome elements are chosen the same. Except for the arches (see Figure 18), because these will be loaded a lot more heavily.

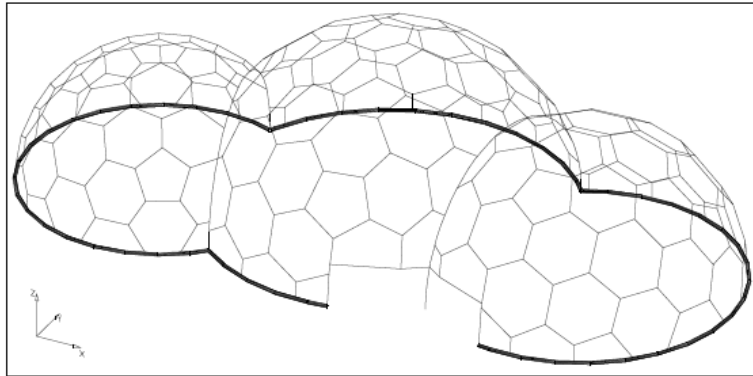


Figure 17: Geometry structure, lower border elements

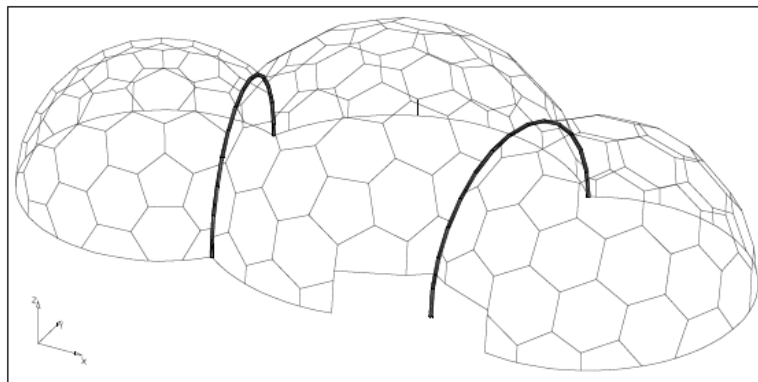


Figure 18: Geometry superstructure, arches

### Adjusting for modular pavilion

In the modular configuration the arches need to be split up in two separate elements. For now the dimensions will be estimated at two times a CHS 197,3/8,0.

The separate domes are also stable when they are not connected with each other. So in the modular configuration a structural connection is not strictly necessary.

When the superstructure is not one whole entity, the domes should be calculated as such, and some structural sections might have to be enlarged. Later on is researched how big the relative movements will be for the modular parts of the pavilion. Then is also chosen if the modular super structure should be structurally connected or not. Then will be decided if the superstructure should contribute to the rigidity of connections between modular parts. If the superstructure should contribute to the rigidity the sections will probably have to be enlarged.

Both with or without a structural connection, there could be made use of a sealing between the separate parts by making use of rubber profiles. These rubber profiles could be clamped in aluminium profiles which can be riveted or bolted to the steel arches. By using rubber profiles in between, a certain amount of deformations, which will occur when the separate modules will move relatively from each other, can be taken.

## 13.2 Floating Body

### 13.2.1 Choice Floating System

#### **Choice: EPS/concrete combination**

In chapter 4 of the general part the pros and cons of the different floating systems were appointed. This project will be loaded by a relatively small wave load only. So the options specifically suitable for large wave loads are not eligible for the pavilion.

The treatment of floating systems in part 1 showed that when a small depth is desired, the floating system of a combination of EPS concrete with the most appropriate. When the benefits of unsinkability and little required maintenance on the float are taken in account the choice will be even more clear..

An advantage of the concrete caisson, space in the floating body, has not much value if there is little or no use for this basement. So the choice is clear, and a floating system of EPS with concrete is selected.

- EPS/concrete combination best for pavilion:
  - o unsinkable
  - o small draught
  - o low maintenance
  - o for pavilion space in floating body not necessary

In chapter 2 was mentioned that floating structures with a relative light superstructure, with a low draught and with a width larger than 10 metres will be stable by shape stability. Since the pavilion fulfils this demands it will be stable independently of the choice of the floating system. The amount of stability will be checked in chapter 19.

#### **Countering rotation**

The EPS/concrete combination has got one disadvantage compared to the concrete caisson. In a concrete caisson water cellars can easily be realized. With water cellars the floating body can be trimmed flexible. In an EPS floating system these water cellars are difficult to achieve. Permanent deviations in loadings can be easily countered by increasing or decreasing the height of the EPS at certain locations.

When connecting the modular elements, it is important that the various floating bodies will the connectors will be at the same height. This can be achieved easily with water cellars, but as said, with the EPS system, these are hard to realize. So with EPS the trimming or adjusting the freeboard has to be done may with loose trim weights, such as concrete blocks or sand bags. This way the floating body not need to have an heavy water cellar, which also is beneficial to the depth.

#### **Structure floating body**

What the concrete structure of the floating body will look like depends for a large part on the construction method (see appendix 14b). High beams provide more rigidity and provide more possibilities for creating a fixed connection in between the floating bodies. Here for a construction method which enables high beams is preferred.

Only with the 'construction method in situ on water' the beams will be significantly less high. This is one of the main disadvantages 'in situ construction on water', but this method has got multiple disadvantages, thus in appendix 14 this method is crossed off. With the other construction methods beams as high as the floating body are possible. There will be chosen for the principle and the system of Maarten Kuijper (see appendix 14).

In this chapter will be taken a look at what geometry, thicknesses and material characteristics will be chosen and what kind of reinforcement will be used.

#### **Conclusion**

There have been chosen for a concrete/EPS system, based on Kuijper's system. Trimming during coupling can be achieved with single trim weights. The concrete beams can get a height up to as high as the float itself.



### 13.2.2 Materials

With the chosen EPS/concrete system, the floating body will consist out of a concrete floor and concrete 'walls' with Expanded Polystyrene (EPS) in between. The EPS provides the buoyancy and unsinkability and provides an optimal mold for the concrete beams, so formwork is only needed at the edges. The concrete provides the stiffness and strength of the floating body. In this paragraph the type of concrete and EPS is chosen and is taken a look at the aptitude of EPS and how it should be used.

#### Concrete

##### Demanded properties concrete in optimal EPS/concrete system

The self weight of the concrete will result in by far the largest vertical loads (see 14.3). So, the dimensions of the concrete floor and the concrete beams will be the determining factor for the draught. By limiting the draught, the pavilion can be used in more shallow waters. Next to this, limiting the draught also results in a smaller height of the floating body, which will save in costs because less EPS and concrete will be used. Here for the dimensions of the floor and beams should be limited as much as possible.

At first the amount of concrete will be decreased by using optimal (from limiting material point of view) shaped concrete beams. Second, a kind of concrete should be chosen which makes light beams or beams with small dimensions possible.

During construction the EPS will blocks will serve as formwork. This EPS formwork will be shaped in a way so that a an optimal shape for the concrete beams will arise. This construction system, with EPS as lost formwork and an optimized beam shape has however the disadvantages that using standard reinforcement cages is less good possible and compacting with a vibrator will be troublesome. The first disadvantage could be countered with a concrete with a high tension strength or fibre reinforcement, in fact in that case this is not a disadvantage at all since it brings multiple advantages. By using a self compacting concrete vibrating is not necessary. Since the floating body will be placed in salty water, it should not be susceptible to chloride ingress, here for the concrete should be dense.

Concluding can be said that a concrete should be chosen which has the following properties or with which following can be fulfilled:

- limiting weight
- self compacting
- no use of traditional reinforcement cages
- not susceptible to chloride ingress

These points will be elaborated below.

##### Limiting weight

Realizing smaller concrete elements can be done by making use of Fibre Reinforced High Strength Concrete in stead of standard concrete with traditional reinforcement. Using Fibre Reinforced High Strength results in smaller elements because of the following:

- less material needed because of higher strength
- no (or less) coverage needed

##### High strength

When the strength of the concrete is decisive, the dimensions of the beams and floors can be decreased with using a high strength concrete. Often the stiffness is normative for the beams, but with using the chosen system with optimal shaped high beams, or using pre-stress, the strength of the concrete can be normative. With high strength concrete as well the compression as tension stress increases.

##### No coverage needed

If there is no or less coverage needed, the beams can also be dimensioned much lighter. By making use of fibre reinforcement in stead of traditional reinforcement, there is no coverage needed at all. (Only the first centimetre of the beams can not taken in account in calculation [Grunewald, February 2010])

### Light weight concrete

Light weight concrete might also seem an option, but it appears it is not, since the strength of the concrete strongly decreases with light weight aggregate. As example is taken light weight concrete with a density of 1800kg/m<sup>3</sup>. For this LWC concrete accounts that the strength has to be equal or more than the strength of normal weight concrete times a factor of 0,75 to become a good alternative. Below this is depicted by calculating the strength per kg with an index strength of 100N/mm<sup>2</sup>.

$$\text{LWC: strength per kg} = 100(\text{N/mm}^2) \cdot 0,75 / 1800(\text{kg/m}^3) = 0,042 (\text{N/mm}^2) / (\text{kg/m}^3)$$

$$\text{NWC: strength per kg} = 100(\text{N/mm}^2) \cdot 1,0 / 2400(\text{kg/m}^3) = 0,042 (\text{N/mm}^2) / (\text{kg/m}^3)$$

A strength of 75% of the strength of normal weight concrete is for strong concretes with lightweight aggregate in practice quite difficult to reach, but it will be more expensive, so this means it is not a good option concrete with a relatively high strength is needed. High strength concrete is with lightweight aggregate not possible.

### **No use of traditional reinforcement cages**

With the chosen EPS/concrete system with optimized concrete beams, traditional reinforcement cages are, as said, less convenient. This can be countered with a concrete with a higher tension strength or fibre reinforcement. The increase in tensile strength, when using high strength concrete, makes it possible to construct with fewer or no additional reinforcement. The high strength concrete can be well combined with fibre reinforcement.

Not using traditional reinforcement results in multiple practical advantages:

Concrete braiding is not needed anymore, which saves a lot of labour costs. Secondly, by avoiding reinforcement, the coverage requirements have not to be met, which results in thinner beams as mentioned in last paragraph (no coverage needed).

### **Self compacting dense concrete**

Self compacting concrete and high strength concrete exist of a precisely composed grading curve with smaller particles which develops into a dense concrete. Dense concrete is less permeable and chloride ingress and carbonation will be reduced.

### **Choice concrete (Conclusion)**

From the points mentioned in this paragraph can be concluded that should be chosen for a dense self compacting fibre reinforced high strength concrete.

With a dense self compacting fibre reinforced high strength concrete the weight of the floating body can be strongly decreased. With the smaller particles of a self compacting high strength concrete, fibre reinforcement will be possible and the concrete will be less susceptible for chloride ingress. Because of the fibres no traditional reinforcement is needed, which results in no need for coverage, optimal shaped beams will be possible and less labour costs.

### **Choice Strength of Concrete**

For the choice of the fibre reinforced high strength concrete will be chosen for a strength as low as possible, because concretes stronger than C55/67 become far more expensive with increasing strength. C55/67 is these days a quite normal strength class, which can be purchased at a reasonable price, since this chosen strength is relatively easy to achieve, and can have a good workability without too many expensive admixture.

These days much higher strengths are possible. The company Mebin BV (market leader in concrete or mortar in the Rotterdam region) supplies concrete with a strength till C90/105. This strongest concrete of Mebin they call Starcrete. (<http://www.heidelbergcement.com/>)

In the thesis of Kuijper (2006), is chosen for a concrete with a compressive strength of 120 N/mm<sup>2</sup> and a tensile strength of 12N/mm<sup>2</sup>. Already in 2004 the firm Hurks produced concrete frame elements in concrete quality B160. [[www.hurksbeton.nl](http://www.hurksbeton.nl)]

For now will be aimed at fibre reinforced concrete C55/67. If this is needed for the strength this can be heightened.

The tension strength of the concrete is proportional to the compression strength; a higher compression strength also means a higher tension strength. If a higher tension strength is desired

than the concrete tension strength, there can be made use of fibre reinforcement, fibres behave best in a dense concrete with small particles, which is the case with high strength concrete C55/67. With fibre reinforcement tension stresses up to 18N/mm<sup>2</sup> can be reached. The needed amount of steel fibres can be calculated by calculating the amount of steel which is needed. With which the calculated strength amount has to be multiplied by a factor 1/2 or 1/3 because of the unequal orientation of the fibres. (For example: for a tension stress of 18N/mm<sup>2</sup> a reinforcement percentage of 7,5% is needed (calculated with a factor ½ for orientation)).

## EPS

### First selection: EPS 20

For the type of EPS is initially opted for EPS 20. The number behind the EPS type indicates the density of the EPS. Nowadays, the EPS is often indicated by the compressive strength at 10% distortion. Then EPS 20 equals EPS 100 in this other way of indicating. This report, however, uses the old way by indicating the specific gravity, as this is found more clear. Information of EPS is given in Appendix EPS 13, EPS.

### Price proportional to weight

The price of EPS is almost linear proportional to the weight. So the heavier the EPS is, the more there is to be paid. The lighter the EPS is the more buoyancy is generated. (But in fact this does not make much difference, since all kinds of EPS are many times lighter than water.)

So with a lighter EPS slightly more buoyancy is generated with even less costs. Therefore, initially is chosen for the most common and light EPS 20. When more strength or stiffness appears necessary, a heavier EPS type will be chosen.

EPS 20	
Weight	20 kg/m <sup>3</sup>
Long term compression strength	30.000 N/m <sup>2</sup>

### Water pressure resistance

The long term compressive strength of EPS 20 is, as can be seen from the table in Appendix 13, 30 k/Pa. This is equal to 30.000N/m<sup>2</sup> or 0.03 N/mm<sup>2</sup>.

The static water pressure can be calculated with  $p = \rho g z$ .

At one meter depth, the water will be approximately:  $p = 1000 * 10 * 1 = 10.000\text{N/m}^2$ .

This means that EPS 20 can withstand water pressure 3 meters and then it will still be within the elastic region. By the same kind of EPS and a higher draught than 3 meters the EPS will deform more.

### Moisture absorption and deformation

EPS absorbs moisture when submerged. For full immersion, the moisture absorption of EPS 20 will be 4%, with EPS 25 it is 3.8%. Creep deformation of EPS is less than 2% as listed in Appendix EPS. According to CEN under a load of approximately  $0.30\sigma_{\epsilon=10\% \text{ short duration}}$  an deformation of 2% should be taken in account. The elastic deformation at depth of 1 meter is only 0.125%.

### Protection EPS

EPS is well resistant against chemical degradation and environmental factors. EPS is not resistant against oily fluids, since EPS will solve in multiple oily materials. But oily materials will only be present at the water surface line, since oily fluids have a lower density than water. So in fact the EPS has only to be protected along the waterline. This can be done by a layer of concrete around it, the concrete edge beam. For protection of the EPS at the bottom, for example against intrusion and small animals, the EPS can be coated or wrapped in a foil.

### Sustainability

as mentioned in the design criteria, the sustainability of the various materials used in this project is also important, since this project should be a paragon for sustainability.

EPS is a mono material, which means that it exists out of one type of material . Therefore it is suitable for recycling and thus fit to be used in a sustainable building.

## **Conclusion**

The floating body will be realised from expanded polystyrene with a specific weight of  $20\text{kg/m}^3$  (EPS 20) and a dense self compacting fibre reinforced high strength concrete from strength class C55/67. With this materials it should be possible to realize an optimal floating body.

### **13.2.3 Height Floating Body**

#### **Required freeboard**

In the design criteria of 11.3 is stated that a freeboard of about 80 centimetres is demanded for preventing ascending. In chapter 4 and appendix 6, the current regulations for freeboard are discussed. As said, most regulations and requirement are primarily intended for hollow, sinkable caissons. For non sinkable floating bodies these are less relevant. Nevertheless, for making sure the inside of the pavilion will stay dry, in this structural design is taken as assumption that the freeboard should always be more than 10 centimetres (in the normative situation, Ultimate Limit State), which corresponds with requirements of the guideline of VROM (2009).

#### **Draught in rest**

The draught of the floating body depends on the design and the dimensioning of the floating body and vice versa. So this is a iterative process. But when some estimations are done for the average thicknesses of the beams the draught can be calculated. When the load and the surface of the bottom plane is known, the draught can be easily calculated with the formula for the draught given in paragraph 2.1. For the resulting floating body the draught have been calculated in appendix 20 Draught calculation. The average draught of the whole pavilion is approximately 0,88 meters (SLS). More information about the loads can be found in the next chapter, chapter 14.

#### Draught modular parts

In case of a modular pavilion, the modular parts will have a different draught if they are not coupled, since they are loaded differently. The central dome will be loaded most heavily and will have a draught of about 0,92 meters (see appendix 20).

#### **Maximum Draught**

The maximum main draught can be calculated as all loads have been determined, so in this stadium of the design this is in fact to early to do this, but again with some estimations this can be calculated. Since the maximum draught results in the minimum freeboard, it is important that the maximum draught will be calculated at this moment, so this was first done with some estimations. The calculation for the maximum draught for the resulting floating body is given in appendix 20. The maximum main draught of the floating body by self weight, imposed load and snow load is 1,34m.

#### **Height Floating Bodies**

The requirement of a freeboard of 80 cm appears normative for the height of the freeboard. In the ULS the maximum main draught is  $1,34 - 0,88 = 0,46$  meters more than the draught in unloaded situation in SLS. In unloaded situation in SLS the freeboard is 80 centimetres, so the freeboard in ULS with full load will be  $0,80 - 0,46 = 0,34$  meters, this is more than the required 10 centimetres.

The normative requirement freeboard of 80cm results in a height requirement for the floating body of the pavilion of  $800 + 880 = 1680\text{mm}$ . So for the height of the floating body is chosen a height of 1700mm.

#### Height modular parts

For the three modular parts (see chapter 12) can be chosen a different height, this way is made optimal use of materials. This way less trim weights will be necessary. In modular configuration the central dome will be chosen a height of 1750mm. The height of the auditorium float will be almost equal and the exhibition dome can have a smaller height.

#### **Varying height floating body**

Every floating body itself can also have varying heights if the construction will not be constructed in situ at the water. This way more buoyancy can be realised at the spots where the structure is loaded more heavily.

More heavily loaded at the edges

At the edges the floating bodies will be loaded more heavily because of two reasons:

- The superstructure is supported at this edges
- At the edges there are edge beams. These edge beams have only half of the upwards force compared to the other beams, because it has only EPS on one side (This is made clear in paragraph 15.2.1 Figure 38).

This more heavy load at the edges will cause that the edges will be suppressed more, so moments and deformations will arise. To counter this effect it might be a good idea to add some extra EPS below the edges for extra buoyancy.

First: no height differences

It will appear that the weight of the superstructure is low compared to the other loads as for example self weight of the floating body, so first is assumed that varying the height is not necessary.

After the stresses and deformations of the floating body have been calculated, this decision can be revoked if it appears necessary.

**13.2.4 Concrete Structure**

In the theses of den Vijver (2006) and Kuijper (2006) is stated that beams below the floor are necessary for the rigidity. These conclusions will be taken in account for the initial design, so concrete beams will be used.

The concrete beams give the floating body it's strength and rigidity/stiffness. The self weight of the concrete will result in by far the biggest vertical forces. So, the dimensions of the concrete floor and the concrete beams will be the determining factor for the draught. Here for, as said before, the dimensions of the floor and beams should be limited as much as possible.

***Beams*****Rectangular geometry beams**

Since the pavilion needs rigidity in two directions, the beams will be in a rectangular grid geometry. Maarten Kuijper has chosen for a center to center distance of the beams of 3 meters. This offset seems most favourable for the combined material use of floor and beams. So this distance will also be chosen for the floating pavilion. After some trial and error it seems to appear that the optimal center to center distance for the beams is approximately twice the height of the floating body.

Radial geometry: not good

A radial and tangential geometry is from structural view also a logical solution to pass on the lateral thrust of the superstructure to the opposite side, so both opposites will be in equilibrium. But here this is not favourable for four reasons:

- In this design there are no complete domes, so half of the supports of the steel structure have no equal support at the opposite site.
- The radial solution results in high beam density in the centre, which will result in extra stiffness which is not needed at this spot and to extra weight.
- With a radial structure a lot of different shaped EPS blocks will be necessary.
- With the radial geometry, the beams will make all different and unpractical angles with connection edge.

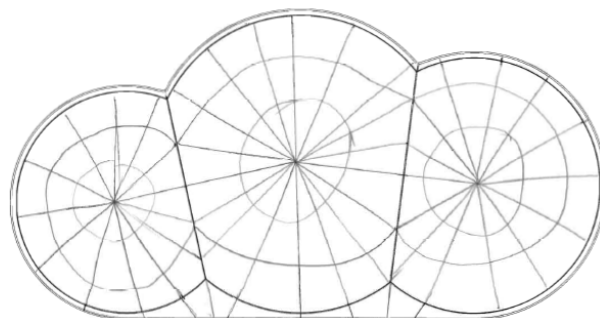


Figure 19: Radial beam geometry

### Beams rectangular on edges

The beam grid will be positioned in such a way that the beams will be exactly parallel and perpendicular to the edges of the connection sections, this will be structural best for the connection and it will also provide the most complete EPS blocks. When using pretensioning at the connection it will also be advantageous if the beams with pretensioning rods will be perpendicular to the connection edge.

Furthermore, the beam grid geometry is placed in a way that as many cut EPS parts will have the same shape. This is achieved by placing the middle beams exactly through the center point of the circle. This way both sides of the circles will consist of the same EPS blocks. (With this beam geometry it would also be possible to split the separate domes, see ch. 12).

From structural view it is also very useful to have the beams situated under the two arches at the intersections of the domes. At the supports of the arches the largest forces (also lateral trust) on the floating body will occur.

Below the configuration of the beams is depicted in the top view of the floating body. The thick line represents the circumference of the steel structure of the superstructure. The dashed-dot lines represent the lines where the pavilion will be intersected in the modular configuration.

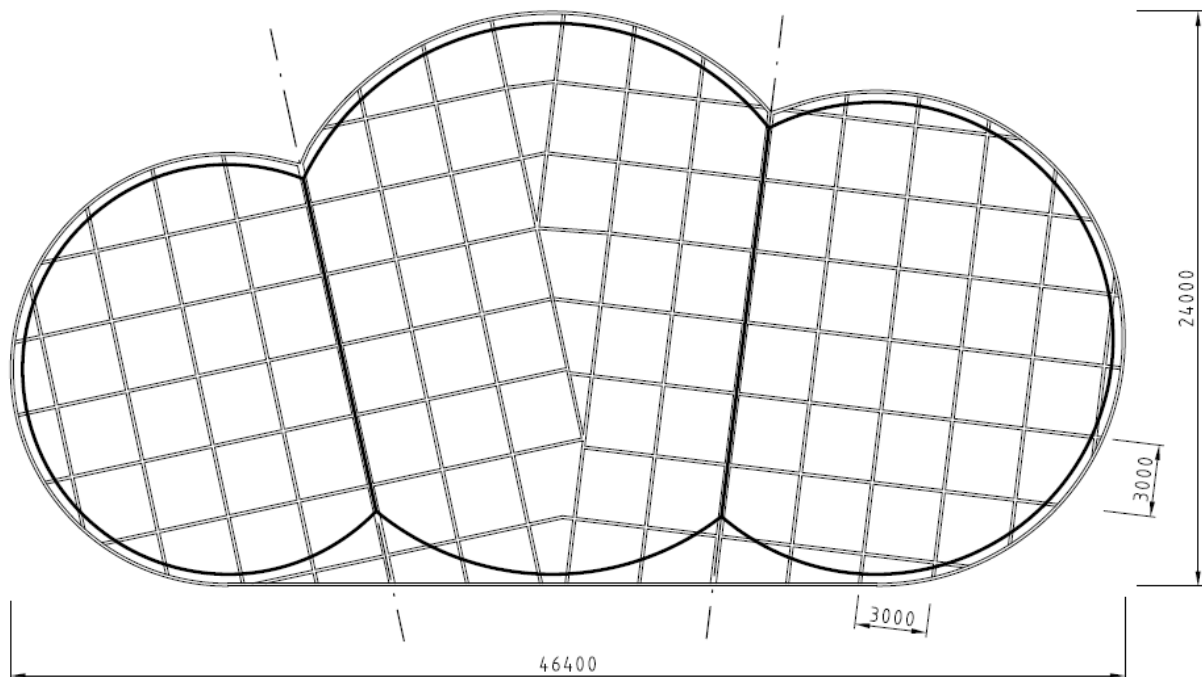


Figure 20: Top view floating body: Beams + section lines, configuration 1

The depicted configuration gives the largest amount of complete EPS blocks; the squares in between the beams.

### Problems in the center?

In the middle of the centre floating body, the beams come together under an angle. This means prestressing of the beams is not possible in this configuration (see also next paragraph). Because of this angle the internal forces of the beam can not be passed on to the other beam in the same direction. But the configuration is chosen in such a way that at the points where the beams come together this will be nodes where multiple beams come together, so the forces can be resolve in normal forces in other directions. At a few points in this configuration this is not completely the case, since there exists a small offset between the point of actions, so here large shear forces might occur. Thus here the configuration could be adjusted, but then more EPS has to be sawn. In the drawing in appendix 30 this slightly adjusted geometry is depicted.

### Adjust geometry for prestressing

Prestressing of the connections is possible with the mentioned rectangular configuration, because the beams are placed rectangular on the connection surfaces, but the prestress cables can not run through the center dome from the one connection to the other with this configuration, since there exist a twist in the beams in the middle of the center dome.

If prestress of the beams appears necessary the geometry have to be adjusted, because the beams come together under an angle in the center dome. The geometry has to be adjusted in a way the beams do not make a twist.

Here four alternatives are given which could be used when is chosen for prestressing of the beams. With the first two alternatives the beams are placed rectangular at connection surfaces, for the third this is partially the case and four alternative 4 this is not the case at all.

#### *Alternative 1*

Alternative 1 is avoiding the twist by a applying a slightly curved beam part in the middle. This advantages are the curved beams itself and the shear force which is introduced this way.

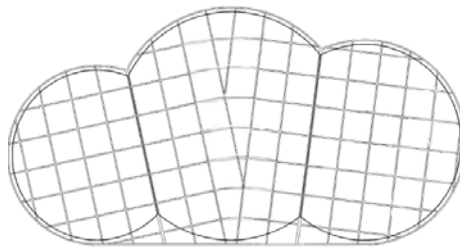


Figure 21: Beam geometry, alternative 1

#### *Alternative 2*

If the outer domes keep the same beam geometry the beams can be lengthened all the way through to the other side of the center dome. Prestressing of as well the beams as the connections will then be well possible, but this results in a less optimal diamond or triangular beam geometry, with a lot of different EPS blocks.

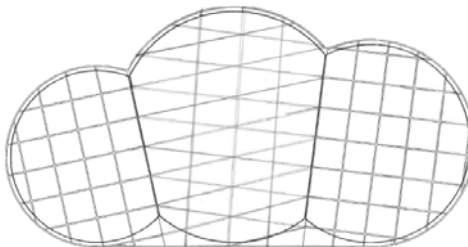


Figure 22: Beam geometry, alternative 2

#### *Alternative 3*

In alternative 3 the twist is avoided by using just one grid field is for the center dome. A slightly smaller grid is necessary to let the beams come exactly together at the connections. If the prestressing needs to run through at the connections, the tendons do need to make a small twist here. Might this result in too much difficulties or problems, than the geometry of the outer domes also have to be rotated, which leads to geometry 4.

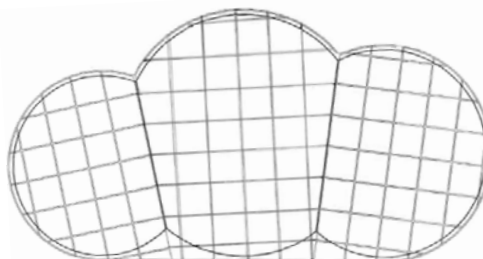


Figure 23: Beam geometry, alternative 3

**Alternative 4**

Alternative four exists out of one beam grid for the complete pavilion. This way the beams are not placed parallel and perpendicular on the connection surfaces.

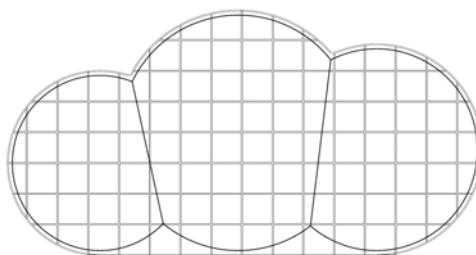


Figure 24: Beam geometry, alternative 4

**Conclusion beam geometry**

Prestressing of the beams will bring some problems with it and extra costs. So at first will be chosen for beams without prestress. In this case the first mentioned rectangular beam geometry is the best and there will be continued with this geometry.

If prestress of the beams appears necessary or advantageous during elaboration of the structural design, then one of the alternatives have to be chosen.

**High beams for rigidity**

The higher the beams will be, the more rigid they will be and the less width they need for moment resistance. High beams will also provide more possibilities for creating a fixed connection in between the floating bodies. Therefore is opted for beams high beams, with the full height of the floating body. This also provides better fixation of the EPS elements.

According to a structural rule of thumb the height of the beams should be equal to approximately 1/10 a 1/15 of the length [Marel, 2004]. This rule of thumb is in fact not meant for a supported beam, but it does help to realise that a beam with a height of approximately 1,6 metres is not very odd for a span between 20 and 45 metres.

**Optimal shaped beams**

By opting for EPS as formwork for concrete, an optimal design of the beam can be realized. This means that for resisting moments especially material at the top and bottom is required and in the centre of the beam less concrete is necessary. Kuijper's thesis showed that a kind of I shaped beam with openings (see figure 8 in appendix 14) is most favourable for forces and materials use.

For this graduation the optimal shape of the beam will not be investigated. The design of the beam will be based on the findings of Kuijper [2006]. So the by Kuijper designed beam will be used but a in a less slender shape. For the modelling the beams will be assumed as normal straight beams with the average thickness of the I shaped beam.

**Width beams**

Initially, an average thickness of 120 mm for the beams will be assumed. If later it appears that this too wide or too narrow, this can be adjusted.

If later it appears that with this beam configuration the beams are largely over dimensioned, the beams may have a smaller height. If with the now chosen beam configuration and dimensions the strength might not be fulfilled, this can at all times be taken care of by increasing the width. The bars will not have to come closer together and will never be higher than the floating body.

Figure 25 depicts beam sections with which an average thickness of less then 120 mm can be reached.



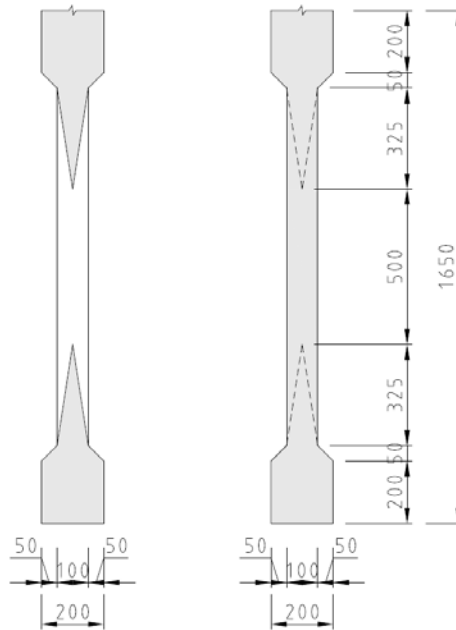


Figure 25: Beam section over cavity and section over full beam

$$\text{Average thickness beam at opening: } t_{ave,1} = \frac{A_1}{h} = \frac{2 \cdot (200 \cdot (200 + 30) + 325 \cdot 100)}{1650} = 75,45 \text{ mm}$$

$$\text{Average thickness over full beam: } t_{ave,2} = \frac{A_2}{h} = \frac{2 \cdot 200 \cdot (200 + 30) + 1150 \cdot 100}{1650} = 125,5 \text{ mm}$$

### Reinforcement/prestress

In this design is made use of Fibre Reinforced High Strength Concrete. The benefits of this concrete were mentioned in the beginning of this chapter. For now was made the assumption that prestressing is not necessary. In a later phase the stresses and crack width in the beams will be calculated and then can be decided if prestress is necessary.

### Edge beams

The sides of the floating body have to be protected against aggression of aggressive fluids, floating oil layer, vermin (ongedierte) and collision. To realize this protection there will be concrete around the floating body; the edge beams. These beams will also have structural function, they have to function as the other structural beams.

The edge beams will also have the same height as the floating body. They will have a homogeneous section with a width of 120 mm.

### Beams for support superstructure

If it appears necessary there will be beams underneath the supports of superstructure. This beam can be constructed by cutting a trench in the EPS.

The superstructure will also have a beam at the bottom of the structure (the lower border elements), which will also be able to pass on forces, so the spots where the forces are brought to the body don't have to be exactly at the point where CHS elements of the dome come down. So the supports can be shoven a small distance to the side alongside this bottom beam, so they will act on the big beams.

Furthermore, everywhere where it is needed, the floors, beams or ribs can be thickened by cutting pieces out of the EPS, which can filled with concrete.

### Floor

According to a structural rule of thumb the height of the floor should be equal to approximately  $1/35^{\text{th}}$  of the span [Marel, 2004]. This rule of thumb is in fact for an other situation than the situation in this case where the Floor is supported by the EPS. But this gives an idea. According to the rules of thumb the floor has to have a  $d$  height of  $3000/35 = 85 \text{ mm}$ .

### Waffle-slab floor

Kuijper (2006) concludes in his thesis that 50% percent of concrete (and thus weight) can be saved if a waffle-slab floor is used in stead of a massive concrete floor. A waffle-slab floor is easy to realise by creating slits in the top EPS layer, which will be filled with concrete while pouring the floor, see Figure 26.

In the case study of Kuijper a floating foundation is designed for a load of the same order of magnitude as the load at this pavilion. The beam distance chosen by him is the same as in this case study, so his findings can be applied in this design.

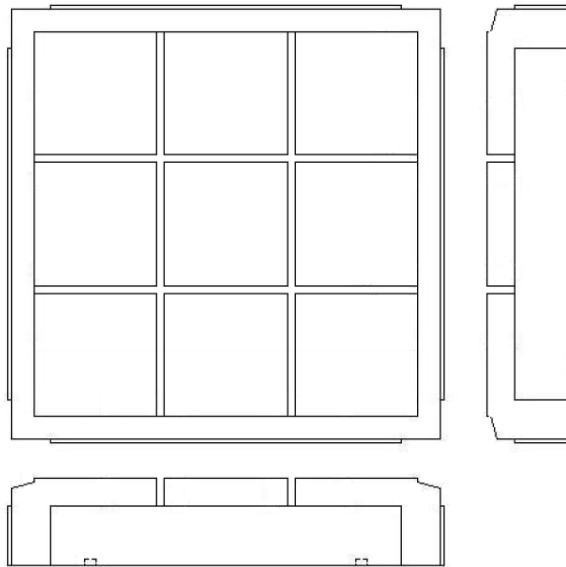


Figure 26: EPS block with contra shapes for waffle-slab floor [Kuijper, 2006]

His research shows that a flat plate floor (with fibre reinforcement and without additional reinforcement) needs to have a thickness of about 150mm .

The required slab thickness of a waffle-slab floor, Kuijper assumed at 50 mm. He found that the optimal distance between the ribs of a waffle slab is between 500 and 1000mm. He opted for a c.t.c. distance between the ribs of 0,75m and dimensions of 250 by 50 mm<sup>2</sup> (h x w). With further optimization Kuijper found an even thinner floor, with a thickness of 30 mm is possible (at the same c.t.c. distance of the ribs).

Because of the mentioned material savings, for this design will also be chosen for a waffle slab floor. In this structural design the just described waffle-slab floor, with a floor of 50 mm and ribs with a c.t.c. distance of 750mm, will be used. This is somewhat thicker than the by Kuijper found optimization, this way the floor will fulfil the strength and stiffness requirements.

### 13.2.5 Cellar

The cellar for the installations (ca. 9x3m) is not yet placed in this phase of the structural design. The choice of the location of the cellar will be done after the stresses in the beams are calculated, because then it can be decided which beams can be cut by the basement.

### 13.2.6 Composition EPS blocks

The spaces between the beams of 3 by 3 meters have to be filled with EPS blocks. It is advantageous to construct the cubic EPS block of several separate layers (see Figure 26). Standard layers with different heights, for example 15, 20 and 25 centimetres can be combined so that the total EPS block can have any height. This way floating bodies with different heights can be constructed using the same molds.

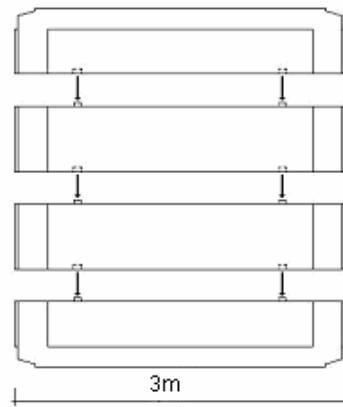


Figure 27: EPS block consisting out of multiple layers [Kuijper, 2006]

The blocks should have dimensions so they can be carried by one person. Layers with dimensions of 3 to 1 meter or 1,5 to 1,5 meters might be an option. Now is chosen for blocks of 1,5 x 1,5 meters, so each layer will consist of four identical pieces.

### 13.2.7 Result structural design floating body

The floating body is designed with a rectangular beam grid with a c.t.c. distance of 3 meters and a waffle-slab floor. The beam grid will be placed as shown in below in figure. This way the beams are orthogonal to the connection surfaces. Technical drawings can be found in appendix 31.

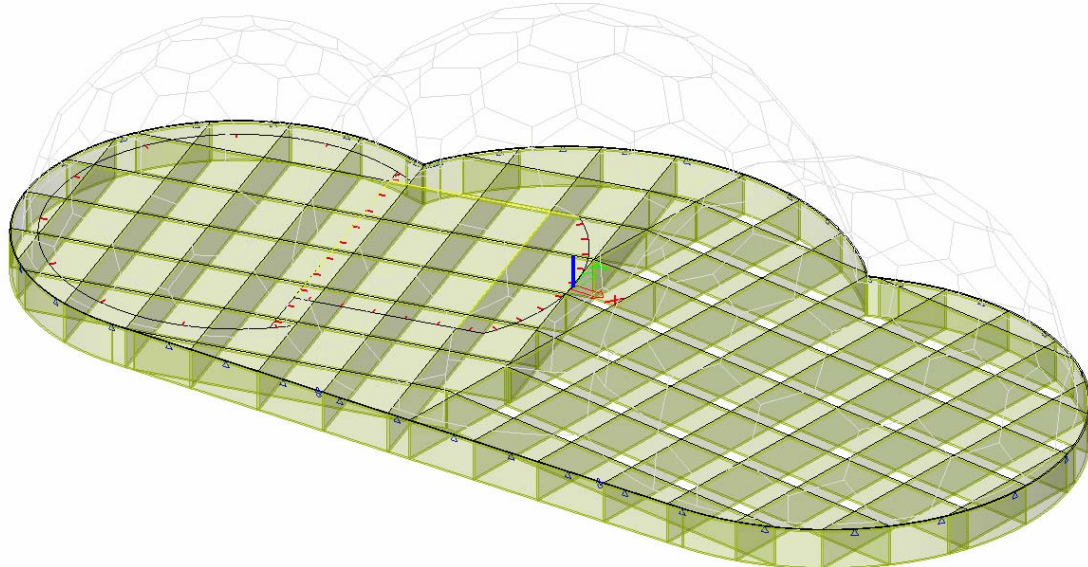


Figure 28: 3D image beam geometry floating body

For the float, the system designed by Kuijper can be used, which has all the components in it that just came along. Each of the three modular parts will be poured as one complete framework, these three modular parts shall be coupled at location at the water surface.

## 13.3 Construction Method

### 13.3.1 Choice Construction method

The multiple construction methods with their ad- and disadvantages are given in appendix 14. The floating body is made of high quality concrete, which makes controlling the circumstances of great importance. In a factory or a dock, the conditions can be guarded best, so there working with high-strength concrete is best possible.

With constructing in situ on water and construction at the waterside this is less good possible.

Production 'at the waterside' will be about the same as constructing in a dock or assembly hall if the contractor can ensure that the concrete poured outside will have the required quality will. However, it should be taken in account that the construction period might be longer because delay during bad weather might occur. But with this construction form the costs for dock or assembly hall will be saved. Building on water has got multiple disadvantages, only one of them is the limited beam height. This option is striped off in appendix 14b.

Constructed in a dock or assembly hall is preferred.

#### Dock or assembly hall

It does not really matter if the constructing of the floating body is done in a dock or an assembly hall. That choice will normally be up to the contractor. Costs will probably be the deciding factor. Production in a dock is from structural view a little better, because this way the structure will not be loaded by possible unfavourable loads which might occur by lifting with a crane or panning the structure out of the assembly hall. So construction in a dock has got a slight preference.



Figure 29: Large assembly hall at Heijplaat

Around Rotterdam multiple construction docks are situated, with widths till at least 40 metres. Docks are situated in the Eemhaven, Waalhaven and the Wiltonhaven (all close to present construction location Heijplaat. The Waalhaven is closer to the Rijnhaven than the present construction location, other locations are slightly further. In Barendrecht a very large dry dock is situated.

### 13.3.2 Realisation

The floating body will be produced as stated in the appendix Construction Method.

The square EPS blocks will be produced in ingenious molds, which results in an EPS Block which have got the exact cavities. The EPS blocks have to be put at very exact distances from each other. The blocks that do not have got the rectangular shape, need to be cut in the right shape. This can be very easy done with an filament (gloeidraad). This can be done as well at the construction site as in the EPS factory.

After the EPS elements are precisely put together the concrete can be poured.

It is wise to put a foil underneath the EPS before pouring the concrete. The EPS will normally will not be affected, but with relatively very cheap foil, it will be prevented that the EPS comes in contact with harmful substances or vermin and prevents pieces crumbling. This foil also protects the EPS during transport.

## 14. Loads on Pavilion

In paragraph 1.3 was mentioned that if a floating structure is classified as real estate, the loads that work on it, have to be determined according to the building codes (NEN 6702 or EC1991). The floating pavilion will be classified as real estate, so the loads will be determined according to the building codes. For this case study is made use of *Eurocode 1: Actions on structures* (EN1991-1-1 from 2002).

As said in chapter 1 and 3, certain relevant loadings for floating structures are not mentioned by the Eurocode, as for example waves, or have to be dealt with differently. These loadings and situations where mentioned and elaborated in chapter 3 of this thesis, for this case study chapter 3 is used as guidance.

In this chapter first something is said about the elements which play an important part by determining the acting loads: the failure methods, normative situations, the design working life and the reliability class. After this, the acting loads are mentioned.

### 14.1 Failure methods and normative situations

When the production of the floating body is done in a dock, the construction phase will not be normative, in contrast to when the floating body is produced on the water (the Flex Base / Dura Vermeer method). So, the normative loads will only occur during the usage phase.

Normative situations can be the maximum imposed load, the snow load, wind load and for the beams the wave loading.

Another normative load might occur when the floating bodies are towed during relocation. This towing introduces extra tow or pushing forces, possibly larger wave loads and hydraulic friction loads. If the floating structure is to be moved this preferably done by pushing and not by pulling, so the beams will only be loaded by compression. Moving the structure will be done in one day that there will be little waves. Under these conditions the movement will not be normative, so transportation does not have to be taken in account in the strength calculation.

For floating structures subsidence also accounts as structural failure. Subsidence can be caused by movements (tilting/sagging). This means tilting and sagging can also be failure methods. How this should be taken in account has been mentioned in chapter 3 and 6.

As mentioned in chapter 6 the guideline demands a maximum tilt of 5%. The wish of the is client to keep the tilt below 1% under normal conditions of use.

## 14.2 Design working life and reliability class

For determining of loads and the load factors, first the design life and reliability class are determined. The design working life will be 50 years and the demanded reliability class is RC2.

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures <sup>(1)</sup>
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.

Table 2: design working life (Eurocode EN1990)

Reliability Class	Minimum values for $\beta$	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

Table 3: Reliability classes with corresponding reliability factor (Eurocode EN1990)

Reliability class 2 with a reliability factor  $\beta$  of 3,8 corresponds with a failure probability of  $P < 0.0001$ . (Reliability class 2 corresponds with consequence class 2 from Eurocode 1990, which corresponds to safety class 3 according to NEN6700. According to the eurocode RC3 has to be taken for example with high-rise or large public buildings (NEN-EN1990/NB)

### Load factors

Load factors are used to bring the probabilistic characteristics of the structure and loads in account. They will be used for calculating the ultimate limit state. The load factors corresponding to the design working life of 50 years and reliability class 2 will be used:

	Load factor
Self weight	1,2
Imposed load	1,5
Beneficial working load	0,9

Table 4: Load factors for design life of 50 years and RC 2 (Eurocode EN1991-1-1)

## 14.3 Self weight

The self weight from the structural elements follow from the dimensions and density of the used structural components. The dimensions follow from the structural design. In Appendix 21 one can find a table with the weights of all structural components and the amount of draught this results in. Below is made a short list to give an idea in what way the elements contribute to the draught. This list gives the estimation which has been made after the preliminary structural design.

(For drawings of the floating body see appendix 31)

### Floating body

-	<u>EPS</u> 1,65m EPS (-2% deformation <sup>1</sup> ), 20kg/m <sup>3</sup> Moisture absorption EPS, 4% <sup>2</sup> , 1000kg/m <sup>3</sup>	1,65 x 0,20 kg/m <sup>3</sup> = 0,04 x 1,4m x 1000kg/m <sup>3</sup> =	0,33 kN/m <sup>2</sup> 0,56 kN/m <sup>2</sup>
-	<u>Slab Concrete waffle-slab Floor</u> 50mm concrete, 2450kg/m <sup>3</sup> ;	0,05 x 24,5 =	1,23 kN/m <sup>2</sup>
-	<u>Ribs waffle-slab Floor</u> 50x250mm, c.t.c. 750mm, 2450kg/m <sup>3</sup>		0,79 kN/m <sup>2</sup>
-	<u>Concrete beams</u> average 120mm x 1650mm, 2450kg/m <sup>3</sup>		3,46 kN/m <sup>2</sup>
-	<u>Concrete edge beams</u> 120mm x 1650mm, 2450kg/m <sup>3</sup>		0,52 kN/m <sup>2</sup>
-	<u>Trim weights</u> (evt.) concrete, average 10 mm	0,01 x 24,5 =	0,245 kN/m <sup>2</sup>
	<u>Total Floating body</u>		<b>6,94 kN/m<sup>2</sup></b>

### Cellar+Installations

-	Cellar+Installations : 336 kN [Sinnema, 2009]		<b>0,37 kN/m<sup>2</sup></b>
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### Floor

-	Top floor (50mm concrete+finishing)		<b>1,25 kN/m<sup>2</sup></b>
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### Super structure

-	Total self weight of the dome structure 484 kN (from Scia model 15.1)		<b>0,54kN/m<sup>2</sup></b>
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As can be seen, the self weight of concrete of the floating body is largely the biggest part of the total weight.

<sup>1</sup> Deformation: As said in 13.2, under a load of approximately  $0.30\sigma_{\epsilon}=10\%$  short duration an deformation of 2% should be taken in account. This is taken in account as 2% less buoyancy.

<sup>2</sup> Moisture absorption: As said in 13.2, EPS absorbs moisture when submerged. For full immersion, the moisture absorption of EPS 20 will be 4%. In the normative situation the draught of the floating body will be 1,4m

## 14.4 Imposed Loads

### Classes of specific use

The central dome and the exhibition dome need to be classed, according to Eurocode 1991-1-1, in specific use class C3 use (see appendix 22): areas without obstacles for people walking around, such as rooms in museums, exhibition rooms, etc. The auditorium dome can be classified according to Eurocode 1991-1-1 in use class C2 (see table below), but in the future, the pavilion may be arranged differently. So the entire pavilion will be classed within the high loading class C3.

The load which belongs to this specific use class is 5kN/m<sup>2</sup>.

### Reducing load Reduction factors

All structural elements loaded by the imposed load have to be strong enough to bear this imposed load. But for calculating the draught and freeboard of the floating pavilion, the imposed load does not have to be taken in account over the complete floor surface of the floating pavilion. The full load only has to be taken over a part the floor surface and over the rest of the floor surface the imposed load times a reduction factor will be used. Why and how this is done is elaborated in Appendix 22.

Appendix 22 gives as result that the maximal imposed load will only be used over 45m<sup>2</sup> at the time. Then the rest of the floor surface shall be loaded with the maximum imposed load times the reduction factor:

$$5 \times 0,25 = 1,25 \text{ kN/m}^2.$$

The maximum load will be put at different locations, so that the pontoon will be loaded as unfavourably as possible (for tilted and internal forces in the beams).

The mentioned 45 square meters can also be distributed across multiple surfaces. An unfavourable situation that will be considered is that the maximum load will be split over two different locations. For this purpose a surface made of 2x25m<sup>2</sup>.

## 14.5 Snow load

The snow loads on the superstructure are determined according to NEN 6702: 2007, Chapter 8.7. According to this code there can be differentiated between multiple snow loadings as a uniform, unbalanced an roof valley. These different snow loadings are applied on the dome and calculated in appendix 23. In the figure below the normative snow load on the Scia model is depicted, with 1kN/m in z direction on every beam on the highest part of the dome.

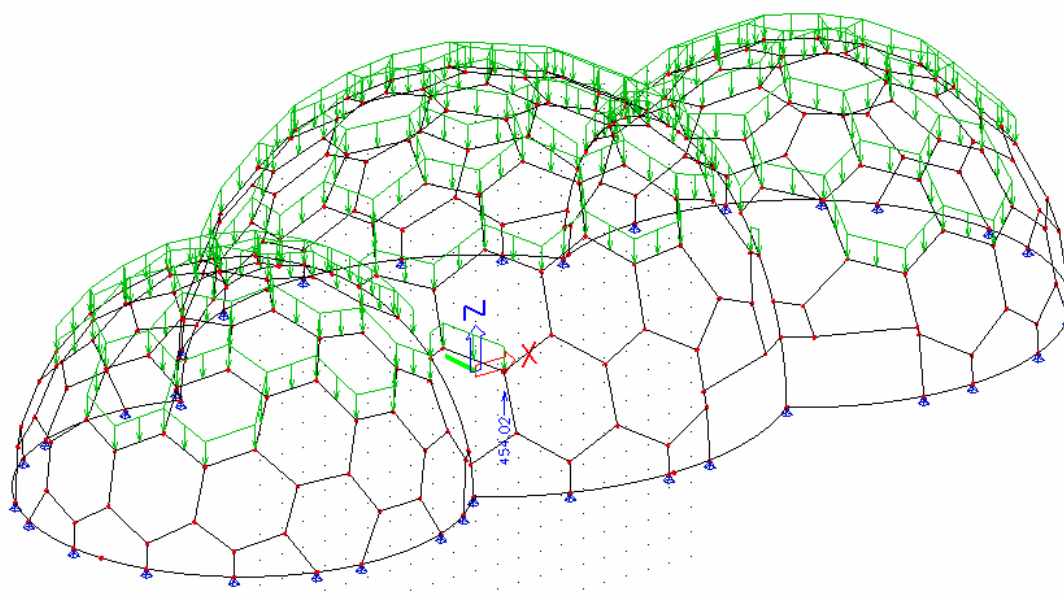


Figure 30: Snow load on domes

The total resulting vertical force of this snow load is 454,02 kN.



## 14.6 Wind load

The wind load is determined according to Eurocode EN 1991-1-4.

### Fundamental basic wind velocity

For calculating the wind pressure on structures, the fundamental basic wind velocity is one of the most important parameters.

According to the Dutch appendix of Eurocode EN 1991-1-4, in wind area II (most of the western part of the Netherlands, see Figure 31) has to be taken a fundamental basic wind velocity of 27 m/s. The definition of fundamental basic wind velocity is as follows: the 10 minute mean wind velocity with an annual risk of being exceeded of 0.02, irrespective of wind direction, at a height of 10 m above flat open country terrain and accounting for altitude effects (if required) [Eurocode EN 1991-1-4].

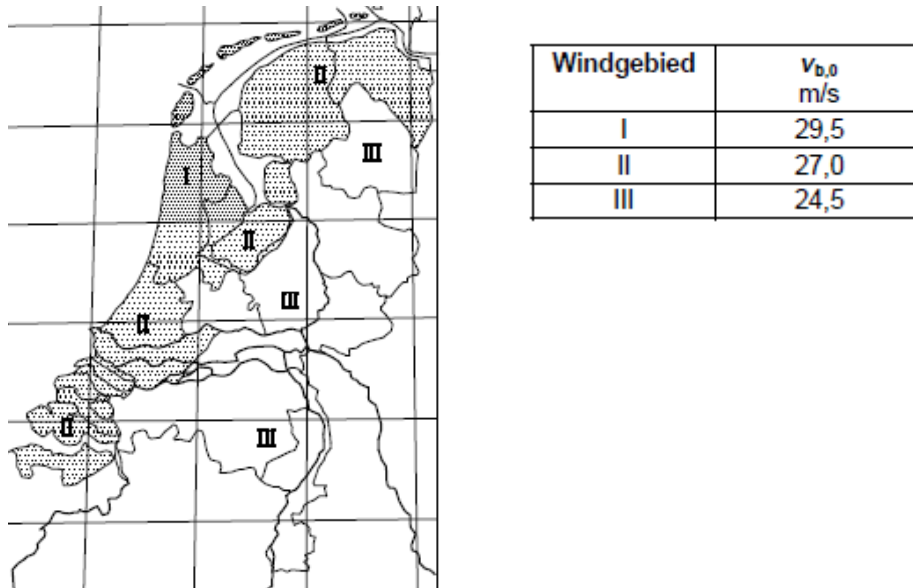


Figure 31: Wind area's and velocity according to NEN-EN1991-1-4NB

### Peak velocity pressure

The peak velocity pressure is needed for calculating the wind pressure and wind forces on the structure. The peak velocity pressure depends on the height of building, the area and the fundamental basic wind velocity. For the Netherlands the peak velocity pressure  $q_p$  can be taken from table NB4 in the Dutch national appendix of Eurocode 1991-1-4. For the Pavilion the Rijnhaven will be accounted as open country terrain, since in the west of the pavilion is the open water of the Rijnhaven (see Figure 32). For open country terrain in wind area II a value from  $q_p = 0,85 \text{ kN/m}^2$  is given for a height of 10 meters and  $0,98 \text{ kN/m}^2$  for a height of 15 metres. For a height of 12 the  $q_p$  can be interpolated, which results in a value of  $0,90 \text{ kN/m}^2$ .

The resulting forces on the pavilion by the peak velocity pressure are calculated in appendix 23. The resulting wind loads of these calculations are put on to the Scia model, this is elaborated in chapter 15.

## 14.7 Waves

The floating body will be loaded by waves. These waves will result in forces in both horizontal and vertical direction. For the principles of waves and how the wave forces act on the structure, see chapter 3 *Loads on Floating Structures* and appendix 10.

In this paragraph the wave characteristics and wave load for waves in the Rijnhaven are determined. In this paragraph is tried to determine the wave characteristics quite accurate, but with the thought kept in mind that the pavilion can also be situated at other locations and that the structural design done in this thesis, should also be valid for other locations and designs, so the wave characteristics are not determined as profound as possible. The wave characteristics should for a real project be determined more elaborately and than the specific harbour should be modelled and measurements should be done.

### 14.7.1 Wave Height according to the Clients Brief

According to the *Programma van Eisen*, composed by DeltaSync the maximum wave height in the Rijnhaven is 0,5 metres. How this value is derived is not clear, probably it was based on a advise by telephone of the *HavenBedrijf Rotterdam* (HbR, Harbour Company Rotterdam). According to the HbR this height will not be exceeded by waves induced by ships and waves during 'heavy weather'.

### 14.7.2 Determining wave properties

Since the origination of the wave height given in the clients brief is not clear, the wave heights will also be calculated with the methods mentioned in appendix 10. Calculating with this methods is needed anyway to determine the period and the length of the waves. First the wind velocity, fetch and depth have to be determined.

#### Wind Velocity

The Rijnhaven is situated from east to west, so the harbour is fully exposed to normative wind (see image Rijnhaven Figure 32). According to the Eurocode for Windload, EN 1991-1-4, the windpressure on buildings in wind Area II is calculated with a fundamental basis wind velocity of 27 m/s. In the paragraph wind the definition of the fundamental basis wind velocity had already been given; this was the peak velocity for 10 minutes. First this value of 27 m/s will also be taken for waves. When it appears the normative waves depend on a fetch with a wind durance over a longer time than 10 minutes, than the wind velocity can be lowered.

#### Strike Length (Fetch)

The length of the Rijnhaven from west to east is about 700 metres, see Figure 32.



Figure 32: Rijnhaven with arrow for fetch

For the strike length is taken a longer length than the length of the harbour, because the waves which enter from the Oude Maas will enter the harbour with a certain height, so for the strike length is taken a value of 1000 meters.

#### Depth Rijnhaven

The average depth of the in the Rijnhaven with high water is 7 metres. At the sides the water is less deep. At the location of the pavilion the depth at high water will be taken as 5,5 metres. (See paragraph 13.2 Location). For the calculations will be taken values varying between 4 and 7 metres.

#### ***Determining properties with nomograms***

First the maximum wave height was determined with the graphs of Groen&Dorrestijn published in the reader of Introduction to Hydraulic Engineering fig 3-43 (see appendix 11).

The qualification deep, transitional or undep water depends both on the depth of the water and the length of the waves. Since the depth changes quite a lot by the tidal movement in Rijnhaven the waves have to be calculated for different depths. According to the calculation in appendix 25 the Rijnhaven during high water may be accounted as deep water. Thus the nomogram and

corresponding formula's for deep water are used. If the nomogram for undep and transitional water is used, the check formulas give that the Rijnhaven can be calculated as transitional water. This difference in outcome is a result of the largely different outcome in wave lengths for both nomograms. (Calculation is done in Appendix 25)

Both nomograms result in a quite corresponding significant wave height. But the period already differs more than a factor 2 and the length differs largely, see Table 5.

	Deep water nomogram d = 6m	Transitional water nomogram d = 4m	Transitional water nomogram d = 6m
Wave height Hs (m)	0.9	1.12	1.32
Wave period (s)	2.5	5.2	6.1
Wave length (m)	9.74	29.3	41.87

Table 5: Wave properties according to Nomograms

So both the nomograms give a very different outcome. This is because the nomograms are less good usable for this area, and the outcome appear to be not very reliable. So now the more accurate Bretschneider method will also be used to determine the wave properties.

### ***Determining properties with Bretschneider***

With the Bretschneider method given in chapter 3 the wave heights and wave period can be calculated. Filling in the formula's of Bretschneider with the variables as mentioned below results in the wave properties mentioned in Table 6.

- F = strike length (fetch): 1000m
- U = wind velocity at a height of 10 meters: 25m/s
- d = water depth: 4 to 7 meters

Depth (m)	Wave height Hs (m)	Wave period (s)	Wave length (m)
4	0,64	2,66	10,80
5	0,65	2,69	11,20
6	0,67	2,71	11,45
7	0,67	2,73	11,62

Table 6: Wave properties according to Bretschneider

The wave length depicted in the table is calculated with the following formula which is given in Appendix 10:

$$\text{Wave length: } L = \frac{gT}{2\pi} \cdot \tanh\left(\frac{2\pi d}{L}\right) \cdot T$$

### ***Determining wave height***

#### **Significant wave height**

With Bretschneider the undisturbed significant wave height has been determined.

As explained in chapter 3 there multiple phenomena that influence the wave height and the other wave characteristics.

In the Rijnhaven will be two factors that influence the height of these wind waves; 'friction' and reflection. Because it is a relative narrow water surface the water will gain 'friction' from the sides. By this 'friction' the wave height will decrease. (In fact it is no friction but the effect looks like friction, because the waves will become less high close to the sides, because of reflection under an angle). The waves will bump at the end of the harbour into the vertical wall and than they will be reflected. This increases the wave height.

So one factor increases the wave height and the other decreases the wave height. At this stage it goes to far to investigate how big both influences will be, so for now the assumption is taken that the friction results in almost halving the wave height and the full reflection results in almost doubling the height. So, the assumption is made that both effects will be approximately equal.

Shoaling will in first not be taken in account, since the slope of the bottom does not give inducement to think that this will result in big differences.

So for this case the calculated significant wave height will not be changed and is kept at 0,67m. Here the remark is made that this significant wave height is far from certain, in the following calculations also different wave heights of the same range should be taken in account. For a real project it is strongly recommended to achieve the specific wave characteristics for the specific location with extensive measurements and/or extensive modelling.

### Design wave height

As stated in chapter 3 the wave height  $H_s$  is the average of the highest 1/3 of the waves. The assumption is made that the wave height is distributed according to a Raleigh distribution. With this assumption the maximum design wave height  $H_d$  can be calculated by a certain exceedance probability.

With an exceedance probability of  $\Pr(H > H_d) = 0,10$ , this gives the following relation for the wave heights:  $H_d = 2,25 H_s$

This results in a maximal design height of  $H_d = 2,25 \cdot 0,67 = 1,52$  m.

### Other characteristics

#### Length

The length and period will be taken just as calculated with Bretschneider. Shoaling is not taken in account, because the effects will in this situation only be small and shortening of the length works only beneficial.

#### Shape

According to **Fout! Verwijzingsbron niet gevonden.** of Appendix 10 the waves should be schematized according to the 3<sup>rd</sup> order theory. But as said in chapter 3 the formula's from the linear theory will be used, but for the schematization for vertical loads the wave shape may be adjusted. As explained in chapter 3, reflection under an angle will also result in shorter crests, this also results in less high wave forces.

In more shallow water and in harbours with a lot of reflection the waves will get a more cnoidal shape. If this appears necessary, or if it will appear more practical in calculation, this will be taken in account.

#### Direction

The flow direction of the normative waves is the same as the direction of the normative wind. This direction is depicted in Figure 32. For the structural design the wave direction shall not be taken in account, since the floating structure can be relocated, so the pavilion should be able to bear the normative wave from every direction.

### Conclusion Wave Properties

In the table below the wave characteristics which shall be used for calculation are given.

depth (m)	Wave height $H_s$ (m)	Design Wave height $H_d$ (m)	Wave period (s)	Wave length (m)
4	0,64	1,44	2,66	10,80
5	0,65	1,46	2,69	11,20
6	0,67	1,51	2,71	11,45
7	0,67	1,51	2,73	11,62

Table 7: Wave characteristics

With the largest water depth the highest wave height and length will occur, so in most cases this wave will be normative. But waves with shorter wavelengths and periods might in some situations be normative.

### 14.7.3 Resulting Horizontal forces

As stated in chapter 3 the horizontal forces caused by waves will be calculated with the following formula:

$$F_{\max} = \frac{1}{2} \rho g (2H_i)^2 + d_t \rho g (2H_i)$$

The figure below depicts how this force is induced.

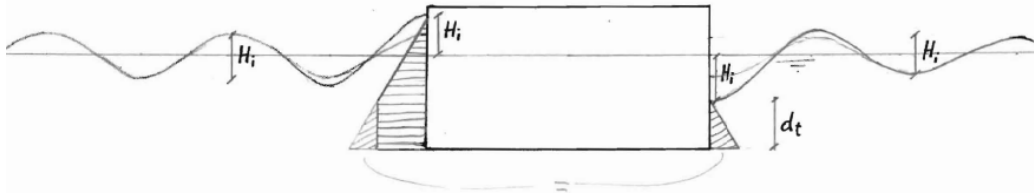


Figure 33: Horizontal pressure by waves

By filling in this formula for the normative wave which can occur with a water depth of seven meters (see Table 7) this gives the following horizontal wave force:

$$F_{\max} = \frac{1}{2} \cdot 10.000 \cdot (2 \cdot 1,51)^2 + d_t \cdot 10.000 \cdot (2 \cdot 1,51) = 45,6 \text{ kN} / \text{m} + d_t \cdot 15,1 \text{ kN} / \text{m}$$

Probably the draught of the floating pavilion will not or barely exceed the wave height, so then the maximal horizontal force will be less than  $45,6 \text{ kN} / \text{m}$ .

### 14.7.4 Resulting Vertical Pressure

In the figure below a floating body of 45 loaded with a wave of 30 metres is drawn. The drawn wave results in a sagging moment. In the figure is also the pressure caused by this wave drawn. This pressure is determined as stated in appendix 10.4. Here still a wave of 30 metres is depicted, because this length was a result of using the nomograms.

For two reasons will be chosen to schematise the wave more as a cnoidal wave (see chapter 3) for this situation:

1. This way the wave can be shoven somewhat more to the sides without immediately having to adjust the pressure value when a part of the wave will be located next to the floating body.
2. With schematisation the wave as a normal sinusoidal wave, the trough could be deeper than the draught of the floating body, this shall in fact not to quickly be the case, with schematizing the wave as a cnoidal wave this will happen less quick.

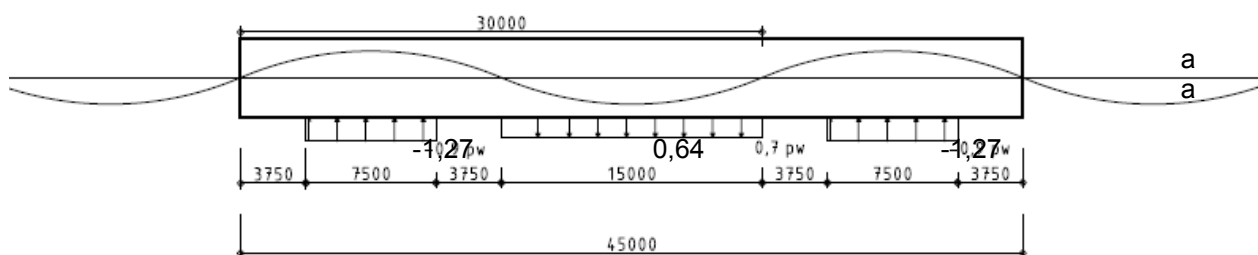


Figure 34: Wave with length of 30 metres with resulting pressure on structure

In the cnoidal wave shape the up pointing surface load will have a width of  $\frac{1}{4} L$ , and the down pointing surface load will have a length of  $\frac{1}{2} L$ .

By adjusting the sinusoidal wave to a cnoidal wave the value of the up and down pointing surface load also have to be adjusted in comparison with the given loads appendix 10.4.

For the up pointing surface loads applies:  $p = \frac{1}{4} / \frac{1}{3} \cdot 0,955 \rho g a = 1,27 \rho g a$

For the down pointing surface loads applies:  $p = \frac{1}{2} / \frac{1}{3} \cdot 0,955 \rho g a = 0,64 \rho g a$

(See appendix 10.4 for explanation).

For a design wave height of 1,51m and an amplitude a of 0,758m (see Table 7: Wave characteristics) this results in:

For the up pointing surface loads:  $p = 1,27 \rho g a = 1,27 \cdot 10.000 \cdot 0,758 = 9,59 kN / m^2$

For the down pointing surface loads:  $p = 0,64 \rho g a = 0,64 \cdot 10.000 \cdot 0,758 = 4,79 kN / m^2$

## 14.8 Incidental Loads

### 14.8.1 Collision

In the Rijnhaven there are still tall ships navigating their way to the wheat and flower processing company Codrico. Thus a kind of collision protection around the pavilion is necessary [Gemeentewerken et al. 2009]. It is assumed that an external party will take care of the protection against collision. This can be realised by the placing fenders or alarm buoys. Or the Rijnhaven will be closed for large ships.

Collision is for connections not of much importance, because of the reasons mentioned in paragraph 3.6. So floating structures and connections won't be calculated on collision in account in this thesis.

The watertaxi will keep coming in the Rijnhaven and will even moor at the floating plaza. So mooring on the plaza will be possible but building of the floating pavilion will be designed in a way that mooring will be impossible. The outside will be smooth and there will be no points are available where a boat or ship can moor. The pavilion will therefore not be dimensioned on the mooring of ships.

Because the ships may not moor on the pavilion the probability of collision will become even smaller. This is another reason that collision will not be included in this report.

### 14.8.2 Fire

In the pavilion there will only be very few combustible material, so the risk on a heavy fire is very little. Nevertheless, a fire is always possible and the EPS will melt with high temperatures. So heavy fires can be dangerous for the pavilion, but in this report fire load will not be taken in to account.

### 14.8.3 Ice

When the water surface where the floating structure is situated gets frozen, the expanding ice layer will introduce compressive forces on the floating body. This is why it is often seen that ice around houseboats gets cut away, to prevent leakage. With a floating body of EPS leakage is no problem, so ice results in a less serious thread, but the concrete still should be able to resist the compression forces. In this thesis this will not be checked.

## 15. Elaboration Structural Design

The designed structure is modelled in the program Scia Engineering and the in chapter 14 described loads are put on it. With Scia the internal forces, deformations and stresses in the elements are calculated. With the internal forces and deformations calculated, there can be checked if the chosen elements fulfil or if they have to be adjusted. Deformations, draught and tilt will also be checked with this model.

The program used is Nemetschek Scia Engineer, release 2009, version 9.0.325.

### 15.1 Superstructure

#### 15.1.1 Model in Scia

In Scia a model has been made of the super structure to extract the internal forces. The geometry of the superstructure from chapter 13 is inserted in this model, together with all it's loads. How this is done and how the model is made is listed below.

##### Structure

- The geodetic geometry is inserted from a 3D DWG file in Scia (this DWG was based on a Autocad drawing of DeltaSync/PDA and Teschner). This resulted in a line model in Scia.
- Every line has been given the properties of the steel sections that had been chosen in the preliminary structural design, these sections are adjusted after calculation if this appeared necessary
- Joints are taken as completely rigid (the structure will be welded together)
- Hinge supports are placed everywhere where dome elements are connected with the 'ground beam' (see chapter 13)
- The result is shown in Figure 35

##### Loads

- All loads from chapter 14 have been inserted (as different loadcases and loadings)
- Loads on the domes as wind and snow loads and self weight of all elements are inserted as line loads. Herefor the surface loads are recalculated to line loads. This is done by dividing the hexagonal surfaces in triangles and allocating the surface of these triangles to the closest beam. This is explained in appendix 23.

##### Calculation

- The internal forces are calculated with linear calculations

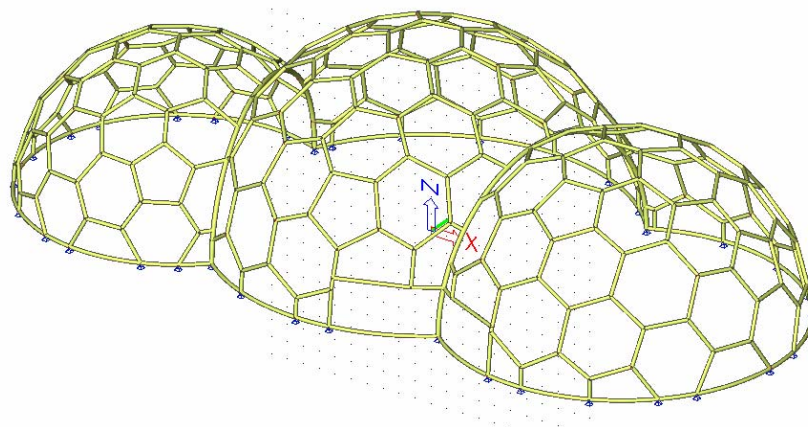


Figure 35: Dome structure, from Scia

### 15.1.2 Results model and choice elements

With Scia all internal forces have been calculated. The normal forces in the auditorium dome and arches are clearly bigger than the normal forces in the other elements. This can be seen in Figure 36.

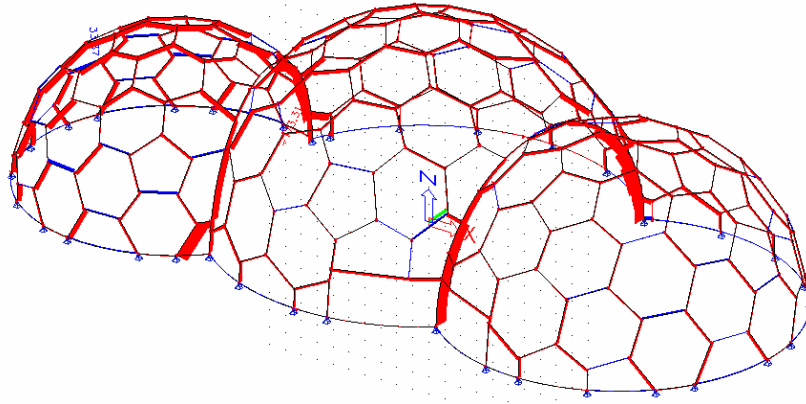


Figure 36: Normative: Normal forces by selfweight + permanent load + snow

The normative load case for almost every beam is the loadcase selfweight + permanent load + snow. A table with the normative forces can be found in appendix 24. From the Scia output could be noted that there occur large moments in several beams. This is especially the case close to the sections with the other domes. In the calculations appeared that these moments were normative for the dimensions of the beam elements.

With the by Scia calculated internal forces the dimensions of the profiles are calculated. These calculations and some iterations resulted in the dimensions given in Table 8. These dimensions are slightly smaller than the dimensions chosen by Teschner. The calculation for the heaviest loaded circular hollow section is attached in appendix 24.

Elements	Profiles	Length Longest Beams
Dome Elements, Dome 1 (Auditorium)	CHS 193,7/ 6,0	2,0
Dome Elements, Dome 2 (Center)	CHS 193,7/ 6,0	2,3
Dome Elements, Dome 3 (Exhibition)	CHS 193,7/ 5,0	2,2
Lower border elements:	CHS 193,7/ 6,0	ongoing, bended
Arches between domes:	CHS 244,5/ 8,0	ongoing, bended

Table 8: Beam characteristics

### 15.1.3 Optimisation

It appeared that the geodetic geometry is not optimal for this structure.

At the spots where the domes are intersected with each other, there appear larger normal forces, shear forces and larger moments. For optimal flow of forces the geometry should be adjusted close to the irregularities. This should be done in a way that in the nodes on the edges multiple beams come together, and not just one beam on the 'arch beam'.

In this project this optimization will not be chosen, because this results in beams with a lot of difference lengths and all different connections. With a normal geodetic geometry almost all nodes and beams can be the same, which saves a lot of costs.



## 15.2 Floating Body

The floating body will also be calculated with a 3D model in Scia Engineering. But since the floating body has a quite difficult shape, first a rectangular floating body is used to see if the model in Scia gives the correct results. In appendix 8 was already checked that Scia gives the correct results for draught and tilt. Some outcomes of the Scia calculations of this rectangular floating body are checked with hand calculations to verificate if the Scia output gives the correct results for the internal forces. This is done in appendix 27. It was found that Scia gives the correct results.

### 15.2.1 Rectangular floating body

The floating body with the difficult shape is schematised as a rectangular floating body.

#### Advantages schematization

A rectangular floating body is far more easy to work with in for example computer programs that can calculate the movement of floating bodies, so therefore this schematisation has also been used. It also works more handy in programs as Scia if the loads have to be inserted and all influences of the load will be more clear. So this schematization is used for determining the influence of waves and imposed load in the next chapter.

Third, with a rectangular shape verifying with hand calculation is possible. With the oblique placed beams and the curves, calculating by hand is hardly possible for the real shape. But out of a rectangular floating body can just be taken one beam, which can be checked by hand, with almost none simplifications.

#### Characteristics rectangular body

The schematised body that is used for this and coming chapters is the following, see Figure 37.

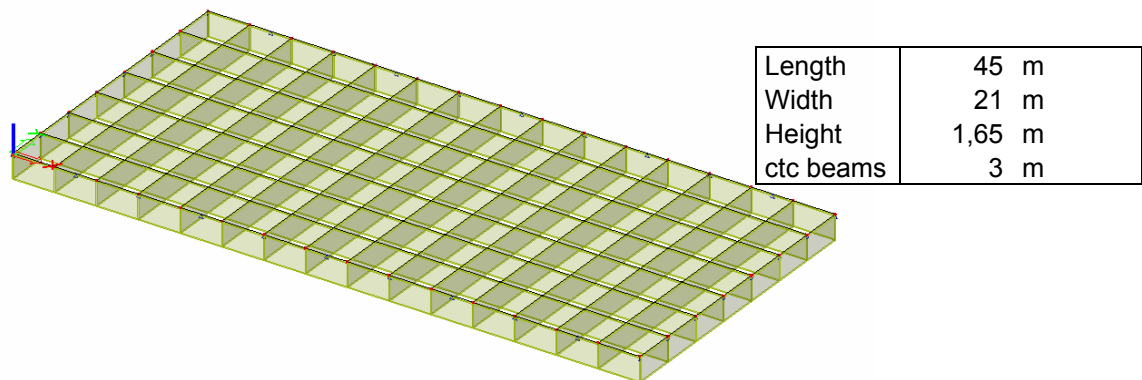


Figure 37: Rectangular floating body; 21x45 m<sup>2</sup> (Scia)

The dimensions have been chosen in a way that all dimensions and magnitudes are almost the same as the real pavilion. for the length and width is chosen a multiple of 3m, because this way the beam grid with the same center to center distances could still be easily used. The height is the chosen height of the structural design. The concrete beams and the floor have both a thickness of 10 centimetres.

How the modelling in Scia is done exactly is mentioned in appendix 26.

#### **Load**

Next to the self weight also a surface load of 3 kN/m<sup>2</sup> has been added. This surface load of 3 kN/m<sup>2</sup> is approximately the same load as the total surface load on the pavilion with all it's loads summed, so this way it gets also approximately the same value for it's draught, an average of 844mm.

### Outcome Scia (Verification)

First is taken a look at the results without self weight, but with the imposed load of  $3\text{kN/m}^2$ , because this way possible errors could be easily detected. subsequently is taken a look at the results with self weight.

#### With load of $3\text{kN/m}^2$ , without self weight

According to Scia:

- the deformation is everywhere  $300\text{mm}$  (so this is the draught): correct
- all internal forces in the beams are 0: correct
- the internal forces in the floor are 0: correct

#### With load of $3\text{kN/m}^2$ and selfweight

##### Deformation

The deformation calculated by Scia is depicted in Figure 38.

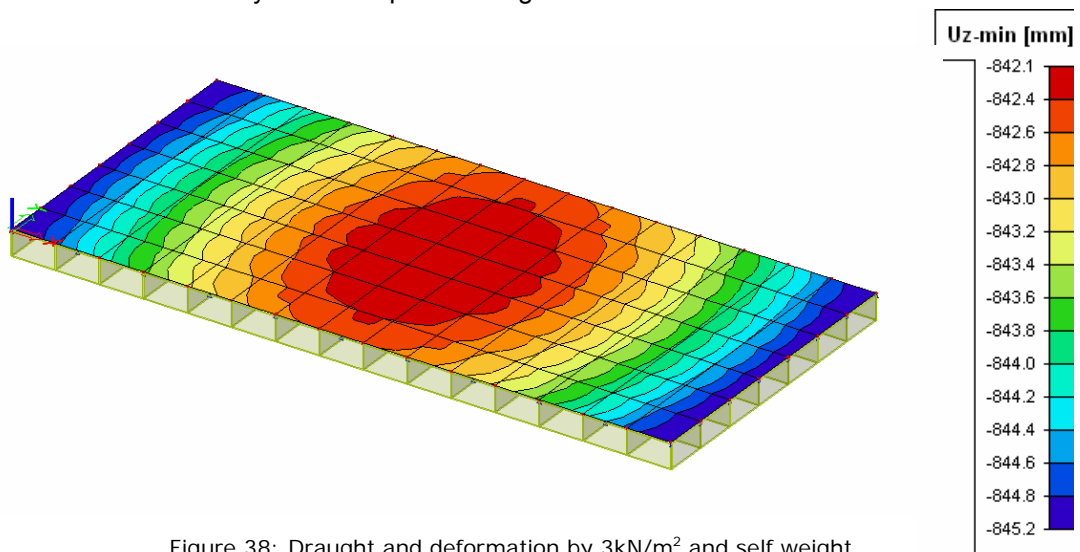


Figure 38: Draught and deformation by  $3\text{kN/m}^2$  and self weight

From Figure 38 can be seen that the edges are sagged deeper than the centre, this is because the edges are relatively more heavy, since there are edges beams, which only have, compared to the other beams, half of the volume of EPS to get buoyed up. But the difference is only  $3,1\text{ mm}$ , this is not much.

The average draught is  $845,5\text{ mm}$  and this corresponds with draught calculations done.

##### Internal Forces

##### Moments

The deformation by the edge beams also causes some internal forces. In Figure 39 the occurring moments are depicted. The maximum moment is  $55,92\text{kNm}$ .

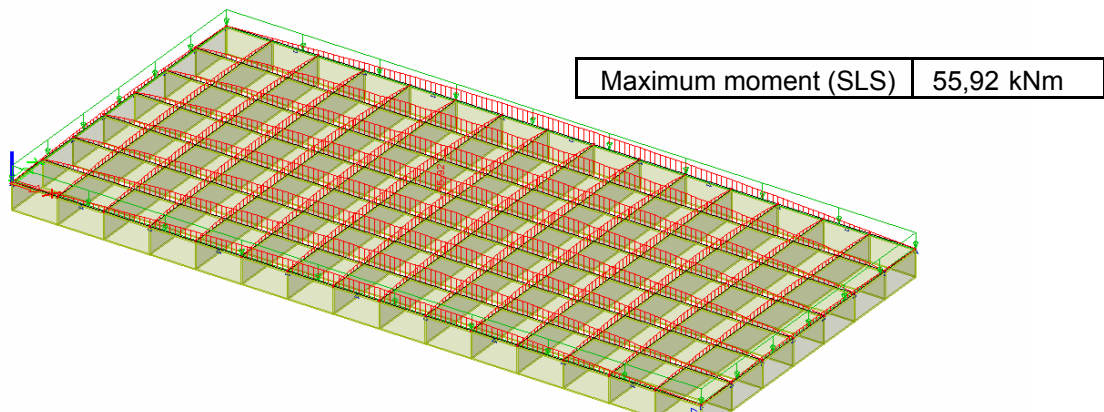


Figure 39: Moments caused by self weight (caused by edgebeams)

### Normal forces not caused by water pressure but by moments

There appear also some normal forces in the beam. At first might be thought that this is because of the hydrostatic water pressure on the sides of the pavilion. But in fact this is not the case, since this horizontal water pressure is not taken in account, because there has only been inserted a vertical elastic support. (Also, in the situation without the self weights the normal forces did not appear).

The normal force in the beam by hydrostatic water pressure equals  $F_H = \rho g d \cdot \frac{1}{2} d \cdot ctc\_beams$  (see appendix 7). With a draught of 84 centimetres this leads to a normal force 10,6 kN/m<sup>2</sup>.

This normal force of 10,6kN lies in the range of the maximum normal forces in the center of the floating body calculated by Scia, which have values of around 12kN (see Figure 40). But as said, these normal forces have got nothing to do with the hydrostatic pressure. In the case the normal force was caused by the water pressure the normal force would be equal through the whole beam.

Thus Scia doesn't take the water pressure component in account, so this has to be added afterwards.

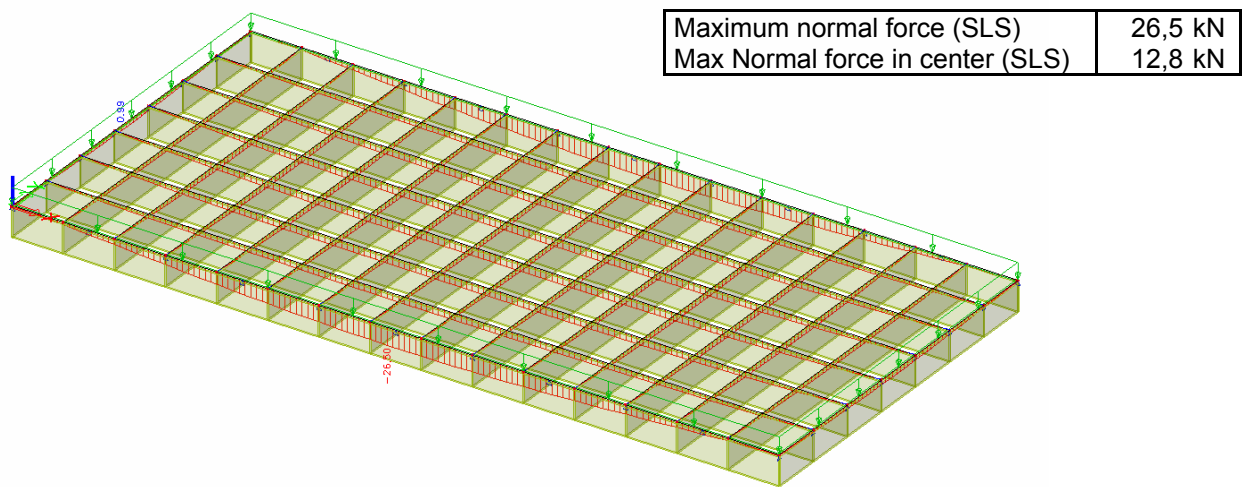


Figure 40: Normal forces by self weight and Imposed load of 3kN/m<sup>2</sup>

The large normal forces which occur in the beams in the center of the floating body appear to be a result of the acting moments. This is because Scia partly divides the hogging moments in a tension force in the floor and a compression force in the beam, for the sagging moments this is just the other way around. For the wave and imposed loads studied in the next chapter, hand calculations are done and it appeared that adding the by Scia calculated moment and the by Scia calculated normal force times half the height of the beam, resulted in the moments from the hand calculation. It was found that the effective width of the floor also plays a role in this. The role of the effective width is discussed in appendix 26.

The large compression forces in the edge beams are not a result of the moments in the beam, but are a result of the plate behaviour.

## 15.2.2 Scia model complete pavilion

In the study of the rectangular floating body has been found that the Scia model gives the correct results, so now the real shape of the complete pavilion is modelled. The Scia model of the complete pavilion is made to extract the internal forces and deformations of the floating body. First the superstructure from 15.1.1 is inserted in this model, together with all its loads. The floating structure has been modelled underneath the superstructure, thus all the loads of the super structure work correctly on the floating body. How the pavilion is modelled in Scia is described in appendix 26. In Figure 41 the result is depicted.

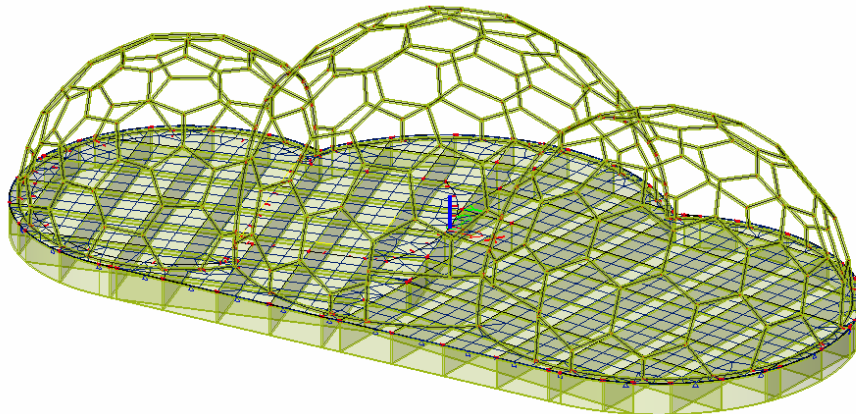


Figure 41: Image of the in Scia constructed complete structure

### 15.2.3 Output and calculation

In this paragraph is taken a look at the output of the Scia calculations for the floating body. First is taken a look if the model seems to give the right results. This is the case, so subsequently is continued with judging the internal forces and elaborating the structure.

#### Verification

First has been taken a look at the forces and deformations in the floating body when the loads of the superstructure and built-in structures do not act on it. Scia gives the expected results for the draught and also the moment diagram looks good, see Figure 42.

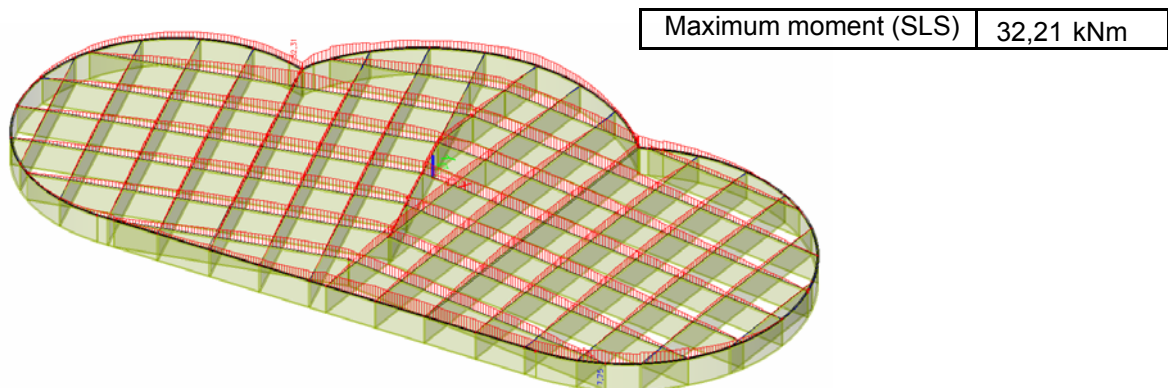


Figure 42: Moments in floating body by self weight concrete only

The normal forces in the beams of the pavilion are, with an maximum of 78,36kN, relative high, but no strange things occur; from the sides to the middle they increase gently, without leaps or anything so the model looks fine (see Figure 43)

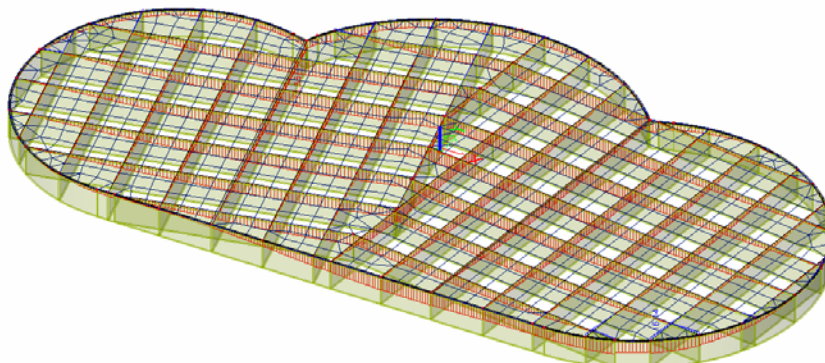


Figure 43: Normal forces in floating body by self weight concrete only



## Tilt

### Without trim weights

With no variable load acting and without trim weights and everywhere the same height of EPS, the floating pavilion heels towards the auditorium dome. This is because the auditorium has a built-in structure and the heavy cellar, installations and built-in structure of the center dome are also at the side of the auditorium dome. Without trim weights this results in a tilt of 25 centimetres, see Figure 44.

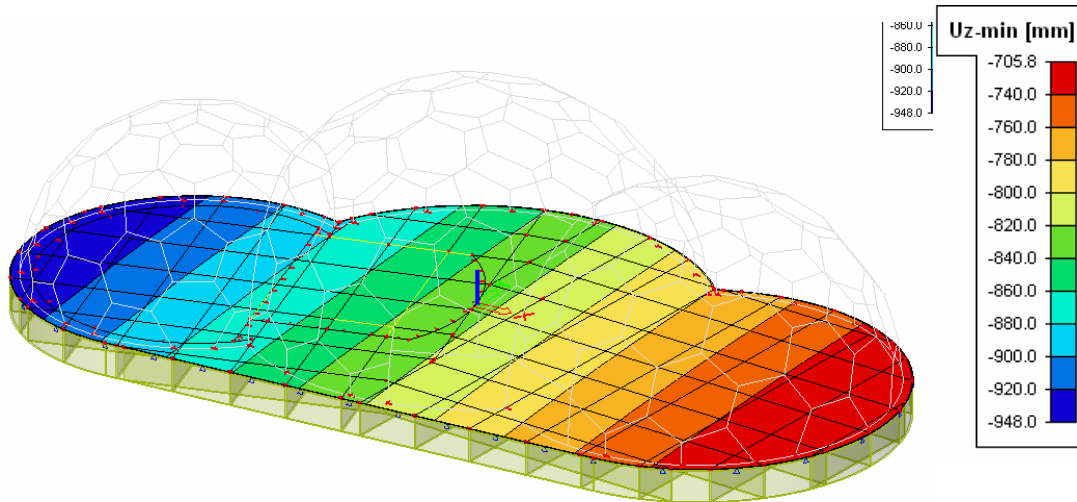


Figure 44: Tilt without trim weights

### With trim weights

The use of trim weights will result in reducing the tilt till almost none. See Figure 45. This countering of the tilt can also be done by increasing and decreasing the draught by adjusting the EPS, but here it is done with adding trim weights because this is more easy to simulate in Scia and gives a better insight.

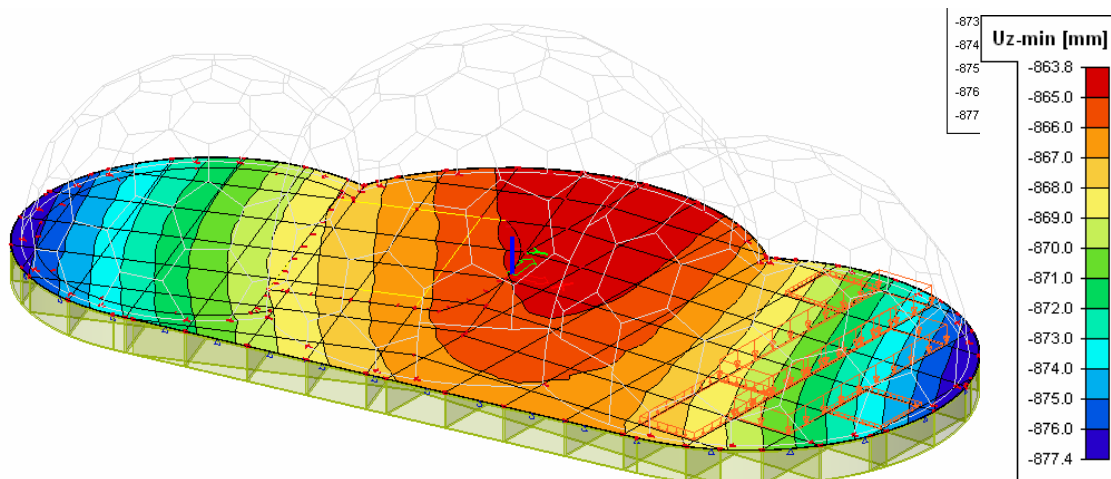


Figure 45: Using trim weights reduces the tilt till almost none

For the result given in the picture a total amount of 414 kN of trim weights is being used. This total weight is divided over 12 floor fields of 3x3 m, how this is done can be seen in Figure 45 and Table 9: These trim weights can be realized by increasing the floor depth at the bottom of the floor, by cutting away the top layer of the EPS, in this case with respectively 4, 8 and 20 cm of concrete.

	load in kN/m <sup>2</sup>	thickness of concrete (cm)	amount of fields	area floorfields (m <sup>2</sup> )	total weight (kN)
fields 5kN	5	20	8	72	360
fields 1kN	2	8	2	18	36
fields 0,5kN	1	4	2	18	18
<b>Total</b>					<b>414</b>

Table 9: Trim weights

The total difference in height is now only 14 mm. In Figure 45 it looks like the figure is tilted in the short direction, but this is less than 4 mm. The rest of the 14 mm of height difference is caused by a curvature in length direction.

The tilt and movements of the floating pavilion due to imposed loads will be treated in chapter 16.

### **Draught**

From Figure 45 also the sag can be seen. The average vertical displacement is 869mm, which is equal to the main draught. This is almost equal as the earlier in Excel calculated draught of 875mm (see appendix 21). The value calculated by Scia is more precise, because now all the exact amount of material have been taken in account. In the table below the draught is given for the load case with and without snow. The deepest draught gives the draught of the deepest point.

	Main Draught (mm)	Deepest Draught
Selfweight+trim	869	877
Selfweight+trim+snow	922	933

Table 10: Draught

### **Beams**

#### **Normative forces**

In Table 5 the normative internal forces in the beams are depicted.

	M <sub>y</sub> max (M <sub>zz</sub> , kNm) max	M <sub>y</sub> hogging (M <sub>zz</sub> , kNm) at support	Beams	N (kN) max	V <sub>z</sub> (kN) max
By self weight only	81,33	81,33	66		
By self weight +trim ULS	123,89	111,3	123,89	307,89	57,82
By self weight +trim + snow ULS	147,23	147,23	140,85	351,66	102,31
By self weight +trim + wind ULS	169,13	169,13		336,36	108,13

Table 5: Normative forces in beams

#### **Normal Forces by moments**

There appear to be large normal forces in most of the beams. In the centre of the pavilion mostly compressive forces, at the sides also a little tension, see Figure 46.

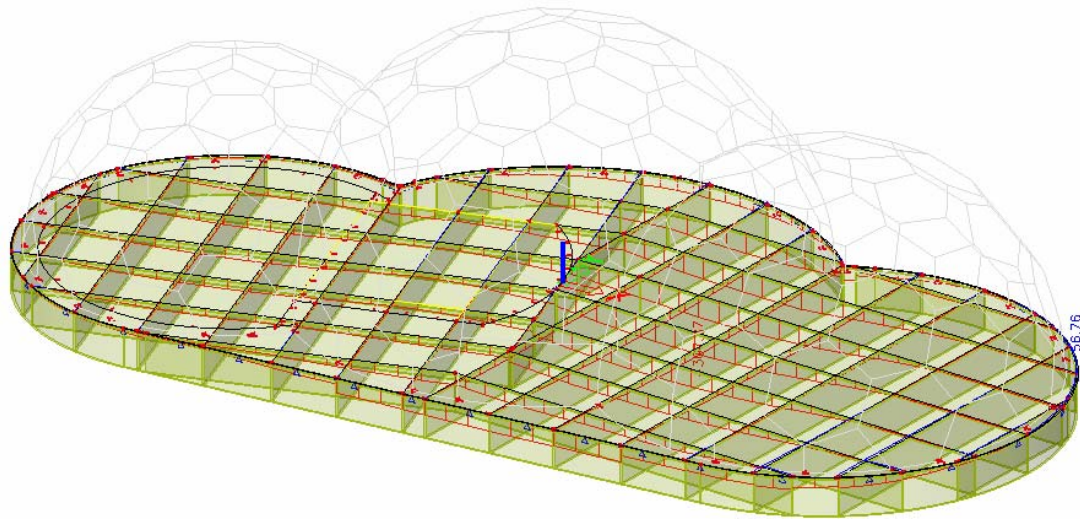


Figure 46: Normal forces (by self weight, snow and trim weights)

The maximum force caused by self weight, permanent loads and trim weight is 307,87kN (ULS) in a beam at the section of dome 2 and 3. The beams next to this beam have values close to this value. If the structure is loaded with snow the highest moment is reached: 351,66kN (ULS).

As mentioned in 15.2 these large normal forces in the beams appear to be a result of the acting moments. By this beam geometry these normal forces are heightened, but luckily the large normal forces are only compression forces, so this will not result in problems.

### Moments

The biggest moment occur at the same spots as the highest normal forces. The largest moment in the middle of the floating body moment occurs with snow load: a hogging moment of 147,23 kNm. The absolute maximum moment of 169,1kN occurs with wind load. It is a peak load at the side.

### Compression Stress

The maximum moment and the maximum normal force together cause the maximum stresses in the beam. The stresses in the concrete are calculated in appendix 28

The normal forces result in a maximum compression stress in the concrete of circa 2,4N/mm<sup>2</sup>. The acting moment results in a maximum compression stress in the concrete of about 2,0N/mm<sup>2</sup>. Together this gives a compression stress of only 4,4N/mm<sup>2</sup>. These stresses are very low, so this means that for this stress the beams are largely over dimensioned. So probably the beams will also fulfil the strength of the loads by waves and imposed load, these are elaborated in next chapter.

### Reinforcement

The moment causes tension stresses in one of the flanges. These tension stresses has to be taken by the reinforcement.

#### Normal Reinforcement

With normal reinforcement with a distance of 40 mm from the bottom of the beam, the moment arm  $z$  of the beam will be 1500 mm, as depicted in Figure 47 (the moments caused by self weight and snow will by hogging moments and no sagging moments, so in fact the tension will be in the upper flange, but the principle stays the same.)

The tension force caused by the normative moment of 169,13 kNm can be taken four bars with a diameter of 10mm ( $A_s$  with four bars  $d=10$  is 315mm<sup>2</sup>). This results in a reinforcement percentage of 0,16 percent, which is a lot lower than the 'economic reinforcement percentage of 0.8-1.2%. (the calculations can be found in appendix 28)

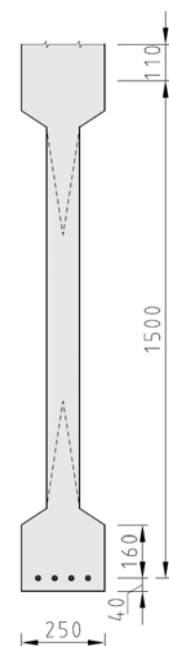


Figure 47: Beam Section With normal reinforcement

### Fibre Reinforcement

Earlier the choice has been made for fibre-reinforcement. With enough fibres, fibre reinforcement high strength concrete can take stresses up to  $18 \text{ N/mm}^2$ .

The amount of needed fibre reinforcement will be calculated as follows:

First a surface of the beam is assumed which will take the tension force. Here this is taken from a height from 2 to 20 centimetres between the bottom, see Figure 48.

This results in a working surface of  $0,25 \times 0,18 = 0,045 \text{ m}^2$

The required amount of steel  $A_s$ , will be calculated the same way as for normal reinforcement, but now with a lower  $z$  value. A  $z$ -value of  $1430 \text{ mm}$  will be used, see the figure on the right. This calculated surface  $A_s$  will be divided by three, for amongst others the irregular orientation of the fibres.

The required percentage of fibre reinforcement to resist the moment of  $169,13 \text{ kNm}$  is  $1,8\%$ . (the calculations can be found in appendix 28)

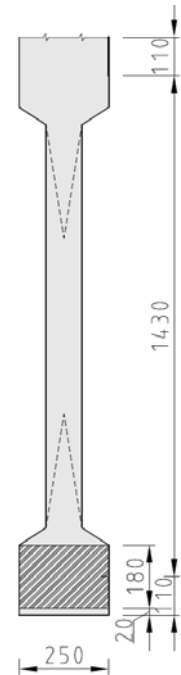


Figure 48: Beam Section With fibre reinforcement

### Combining normal and fibre reinforcement

The fibre reinforcement can be combined with the normal reinforcement. When using 2 bars of reinforcement with a diameter of  $10 \text{ mm}$  the amount of fibres can be reduced below 2 percent. Placing these two beams can be done relative simple and by using the fibre reinforcement other bar reinforcement like shear force reinforcement is not necessary.

### **Shear force**

The shear forces stay at most parts below  $30 \text{ kN}$ , but there are a few locations with a high value, these locations can be found primarily around the supports of the superstructure. These normative shear force is  $108 \text{ kN}$ , which leads to a stress of  $0,7 \text{ N/mm}^2$ . Concrete from strength class  $\text{C55/67}$  has a shear strength of  $0,8 \text{ N/mm}^2$ , so even without reinforcement the beams fulfil the strength. (See calculation in Appendix 28). If the fibre reinforcement is taken in account the beams have a far bigger shear capacity.

### **Deflection**

From Figure 45 could be seen that the total deflection is less than  $14 \text{ mm}$ . This is far below the allowed  $0,003L$ , which is  $135 \text{ mm}$ .

### **Floor**

Figure 147 shows the stresses in the floor caused by the acting hogging moment which already arises with only self weight (discussed before) are shown. The shown stresses are the tension stresses in  $x$ -direction (the normative direction) on the top of the floor (also normative). The stresses which appear with snow load and imposed load by persons are only slightly bigger.

The tension stresses are a result of the hogging moments in the floating body, which are divided in a compression component in the lower part of the beam and a tension component in the upper part of the beam and the floor. These stresses caused by this moments stay between  $-1 \text{ N/mm}^2$  and  $+1 \text{ N/mm}^2$ . This won't result in problems since the tension strength of the concrete is without reinforcement already  $1,9 \text{ N/mm}^2$ .

At the supports of the superstructure however, stress peaks appear, since the supports of the structure are not placed below the vertical elements of the superstructure in stead of upon the concrete beams. If the supports of the superstructure will be placed on the beams, these stress peaks will not occur.



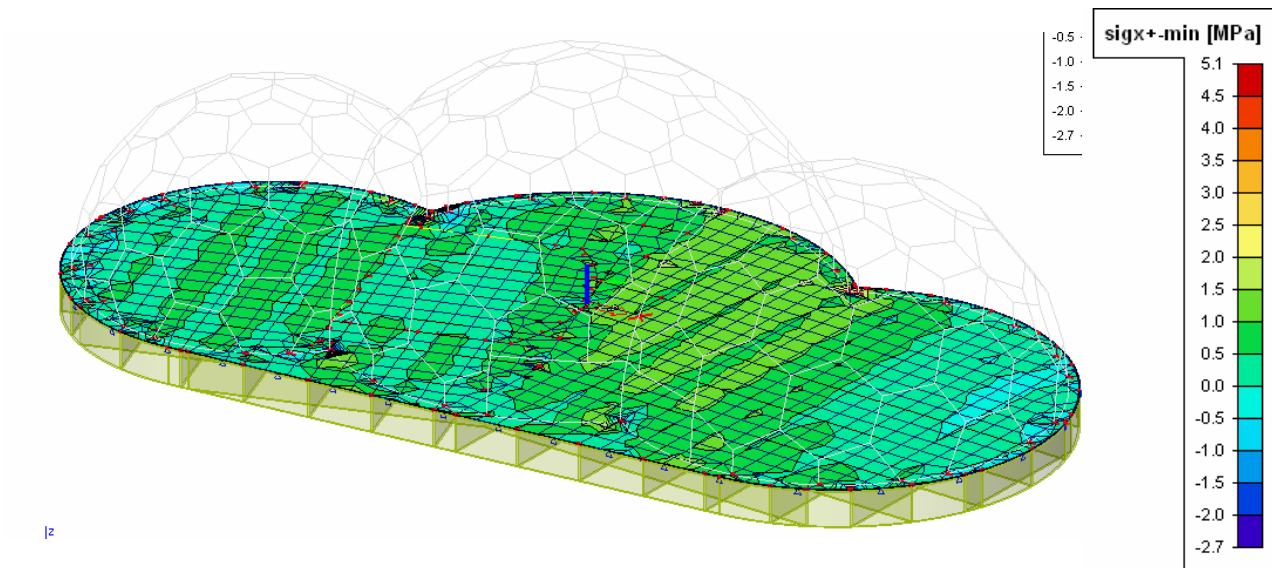


Figure 49: Stresses in floor caused by self weight

## Conclusions and Optimising Structure

### Chosen dimensions stay the same

The in chapter 13 chosen dimensions, do not need to be adjusted and stay the same:

- average width beams: 120mm
- height beams: 1650mm

### Trimweights

Trimweights are being used to prevent tilt in unloaded condition. This optimization has been done in 15.2.3 paragraph *Trim*.

### Modular Pavilion

With the modular pavilion it is not needed to have only one height for the floating body. The trimweights can be taken out of the exhibition dome and it can be given a lower height. This way the moments by self weight will be much lower.

### High strength and post-tension not necessary

Both high strength concrete and post-tensioning of the concrete appear not necessary, because the stresses remain rather low. The strength of a standard C37/45 appear even more than strong enough. Fibre reinforced concrete is still recommended.

## 16. Results Wave and Imposed loading

In last chapters the structural design was done still without taking the wave loading and imposed loading in account. In this chapter is taken a look at the consequences of the wave load and the imposed loads. The reason why the waves and imposed loading is not taken along straight away, had four reasons:

1. The wave load on the structure depends partially on the draught of the structure, which has been assured in last chapter.
2. The internal forces of the wave and imposed loads will be compared to the results of other load cases
3. For obtaining the forces by the waves and the imposed loads the floating body will be schematized as rectangular floating bodies, so another shape is used. This is the most important reason.
4. Now is already known where the internal forces by the other load cases are the biggest, so it can be seen which wave and imposed loading will increase this high internal forces.

This chapter is used to take a look at the impact of the imposed loads and wave loads. The normative loads will be used for calculating the beams of the floating structure and the connections will also be calculated with these forces. The loads in this chapter are taken without load factors in this chapter.

The rectangular floating body used for this chapter is the same as in paragraph 15.1

### 16.1 Imposed Loads

The maximum imposed load for specific use class C is  $5 \text{ kN/m}^2$ . As explained in the chapter loads, for the pavilion is kept a maximum surface where this loads acts on. For the pavilion this maximum surface is  $45 \text{ m}^2$ .

For this rectangular floating body is taken a some what larger surface of  $21 \times 3 = 63 \text{ m}^2$  meters.  $63 \text{ m}^2$  is the surface of 7 floor fields (of  $3 \times 3$  metres) next to each other on a gridline in the y-axis (for example see Figure 50).

In the program Scia this surface load is inserted at multiple locations. Below the most important and normative situations are treated.

First the load is placed completely on one edge, this is depicted in Figure 50.

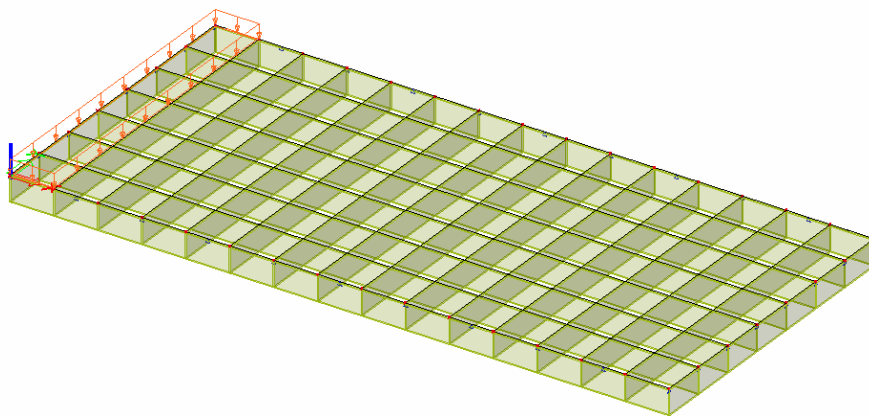


Figure 50: Rectangular floating body, loaded with  $5 \text{ kN/m}^2$  over  $3 \times 21 \text{ m}$ , completely on the left

The loads are also placed in the same way on the other gridlines lying next to depicted load, but the in Figure 50 depicted load appeared the normative 'line load'.

The surface load has also been divided over two different surfaces. The normative situation in this case is when the loads are placed on both ends, see Figure 51. In this figure both surface loads have a width of 1,5 metres.

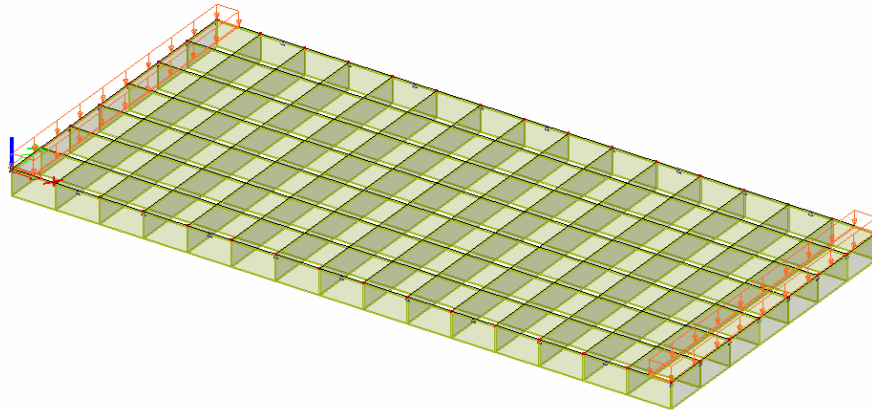


Figure 51: Rectangular floating body, loaded with  $5\text{kN/m}^2$  over  $2 \times 1,5 \times 21\text{m}$ , on the edges  
Next to these line loads the imposed loads are also applied as rectangular of 3 times 2 floor fields. The normative situation is depicted in Figure 52.

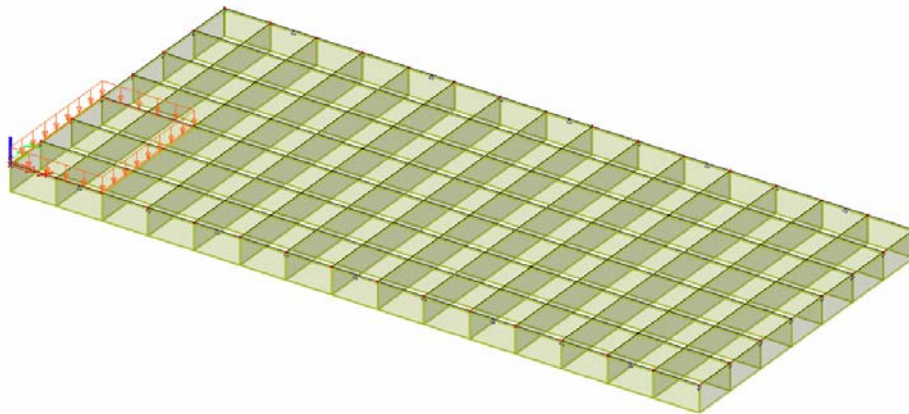


Figure 52: Rectangular floating body, loaded with  $5\text{kN/m}^2$  over  $6 \times 9\text{m}$ , on the corner

These loads can again also be divided over 2 corners; this is done for the straight opposite corners and for the diagonal opposite corners, see Figure 54 and Figure 53.

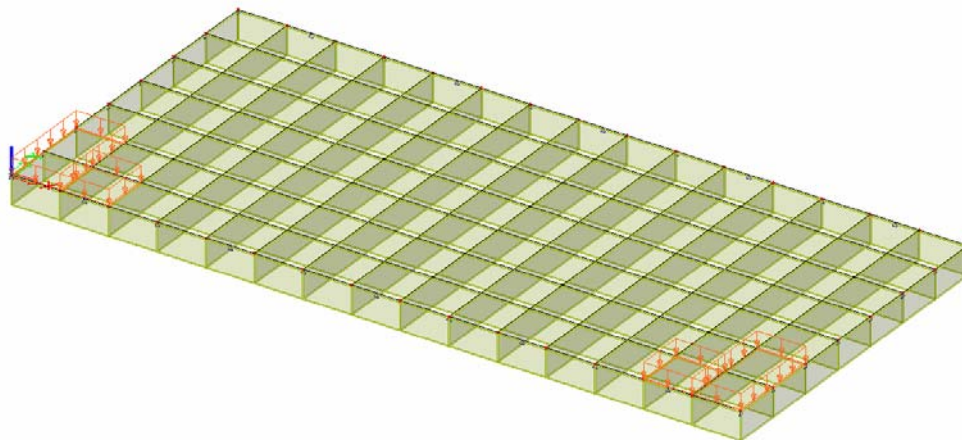


Figure 53: Rectangular floating body, loaded with  $5\text{kN/m}^2$  over  $2 \times 27\text{m}^2$ , on the straight opposite corners

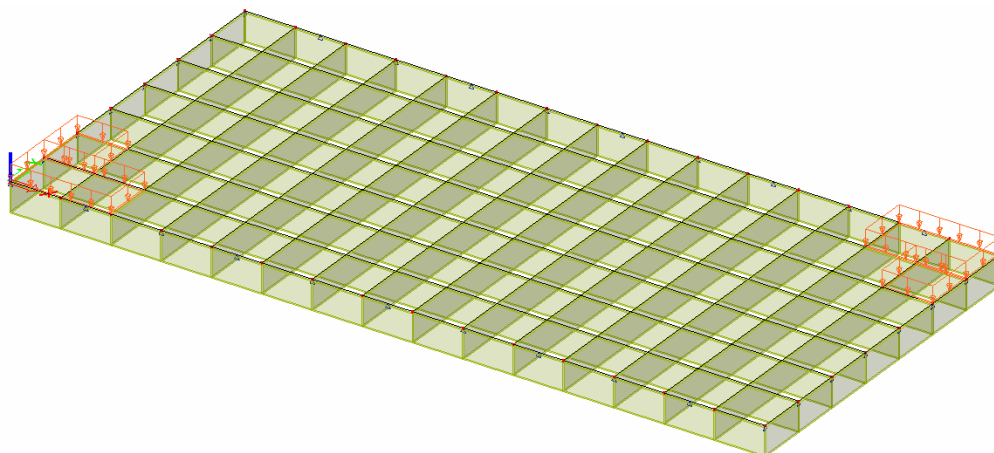


Figure 54: Rectangular floating body, loaded with 5kN/m<sup>2</sup> over 2x9m<sup>2</sup>, on the diagonal opposite corners

### Resulting moments

In the table below the resulting moments are given from the normative and most important load cases (given values are in SLS). Where these maximum moments are situated, is shown in Figure 55well and Figure 56. The resulting moment of load situation 4 (depicted in Figure 51) is checked with a hand calculation in appendix 27.

Load situations	M <sub>y</sub> max (M <sub>zz</sub> , kNm)	N (kN) At max M
	max	
1. By Self Weight only (see 15.2.1)	-55,93	-12,5
2. By Imposed, 5kN/m <sup>2</sup> , 'line' 3 m, 1 side (normative hogging M)	-199,76	-51,3
3. By Imposed, 5kN/m <sup>2</sup> , both sides 'line' 3 m	-339,36	-78,81
4. By Imposed, 5kN/m <sup>2</sup> , both sides 'line' 1,5 m	-182,45	-42,33
5. By Imposed, 5kN/m <sup>2</sup> , 'line' 3 m, center (normative sagging M)	191,02	45,5
6. By Imposed, 5kN/m <sup>2</sup> half floating body	-167,48	-45,05
7. By Imposed, 5kN/m <sup>2</sup> , rectangular 9x6 m <sup>2</sup> on 1 corner	-252,53	-39,28
8. By Imposed, 5kN/m <sup>2</sup> , 2x rectangular 27m <sup>2</sup> on opposite corner	-289,45	-31,42
9. By Imposed, 5kN/m <sup>2</sup> , rectangular 27m <sup>2</sup> on diagonal corner	-137,57	-26,34
10. By Imposed, 5kN/m <sup>2</sup> , rectangular 9x6 m <sup>2</sup> in center	172,89	56,06

Table 11: Moments by imposed loads (SLS)

Load situation 3, with on both sides an imposed load of 5kN/m<sup>2</sup> over 21x3 metres gives the highest moment, this load situation however will not be taken in account, because this results in more than twice the maximum surface. This load situation is depicted to illustrate the influence of a larger surface.

The load situations 2 and 4 (where depicted in Figure 50 and Figure 51) can occur, and are the normative load situations for the single and divided surface 'line' load. The outcome of load situation 4 has been checked with a hand calculation in appendix 27 and appeared correct.

The normative situation for the hogging moment is load situation 8.

Further it might be noted that a half loaded pavilion results in less moment than a imposed load of only three meters wide.

### Normative hogging moment

Load situation 8, the floating structure loaded on two opposite corners, depicted in Figure 53, results in the normative hogging line moment. In the figure below is shown where the moments occur for this load situation. The values of the moments can be read in Table 11.

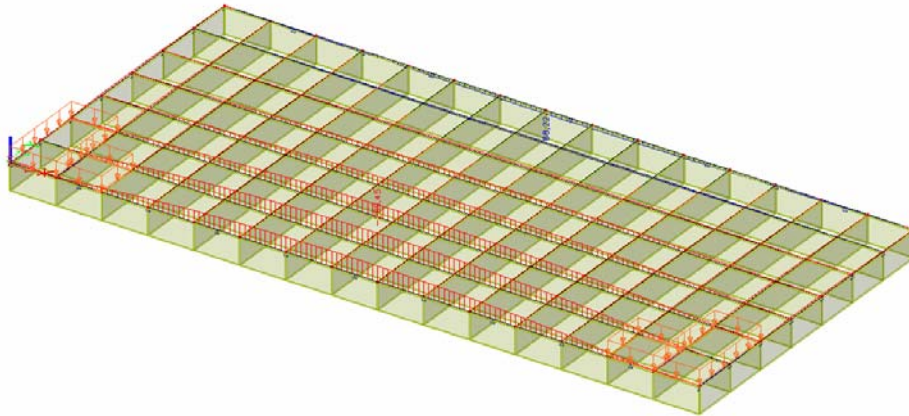


Figure 55: Moment by imposed load of 5kN/m<sup>2</sup>, 21x3m, on the left edge (LS2)

### Normative hogging moment

The normative sagging moment occurs with the imposed loading placed in the center, the normative value can be seen in the table. A sagging moment results in tension in the top and compression in the bottom. In the figure below the moments caused by the imposed load in the center is shown.

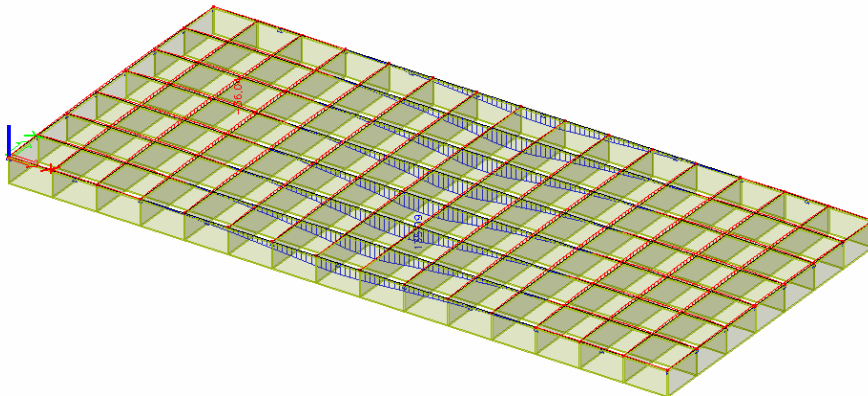


Figure 56: Moment by imposed load of 5kN/m<sup>2</sup>, 21x3m, in the middle (LS5)

The value of this load combination can be seen in Table 11: Moments by imposed loads.

### Shear forces

The largest shear forces occurs with load situation 3, with a maximum of 38,26kN. This is far below the shear force of 108kN which appeared in chapter 15, so this will not result in problems.



## Deformations and tilt

In the table below the draught, deformation and tilt is depicted for the same 'line load' situations as mentioned in last paragraph.

Load situations	deepest point (mm)	highest point (mm)	Tilt/ Deformation (mm)	Average Draught (mm)	Rotation (%)
1. By Self Weight only (BGT)	-845,2	-842,1	3,1	-843,7	0,0
2. By SW+Imposed, 5KN/m2, 3 m, 1 side	-977,3	-790,3	187	-883,8	0,4
3. By SW+Imposed, 5KN/m2, both sides 3 m	-922,5	-902,7	19,8	-912,6	0,0
4. By SW+Imposed, 5KN/m2, both sides 1,5 m	-884,3	-872,2	12,1	-878,3	0,0
5. By SW+Imposed, 5KN/m2, 3 m, center	-878,3	-874,7	3,6	-876,5	0,0
6. By SW+Imposed, 5KN/m2 half floating body	-1450,4	-710,2	740,2	-1080	1,6

Table 12: Draught, deformations and tilt (Scia results)

The fourth column of the table gives the difference between the highest and lowest point, so this is the tilt plus or minus the deformation for a tilted floating body, with no tilt it is the deformation.

### Tilt

How the tilt could be calculated is elaborated in chapter 2 and 4, with the example was shown that Scia gives approximately the right results.

Tilt does occur if the floating body is loaded on one side.

#### In length direction

In the case the imposed load only acts on the maximum surface, the tilt stays rather low, with only 187 mm height difference between both sides. This tilt is no risk for the buoyancy and is clearly below the demand of 5% of the Guideline from VROM (see chapter 6) and is with 0,4% still below the wished limit of 1% from the client.

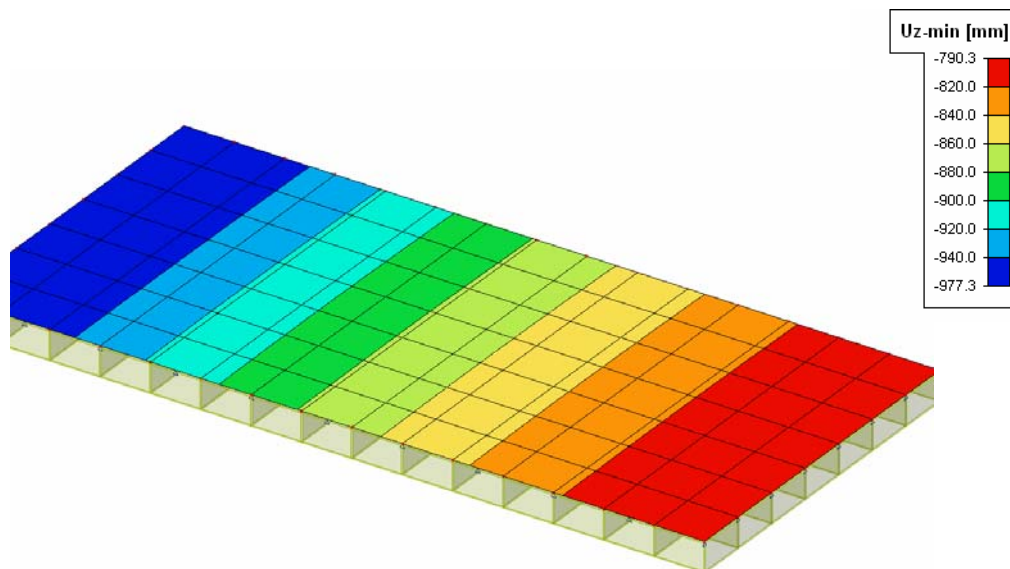


Figure 57: Tilt by imposed loading one side 3x21m

#### In short direction

If the pavilion is loaded on the two corners on the same long edge, the floating body will rotate around in the short direction. The resulting height difference is with 140mm less than with rotating in the length direction, but the rotation is larger. The rotation is with 0,7 percent still below the desired 1%.

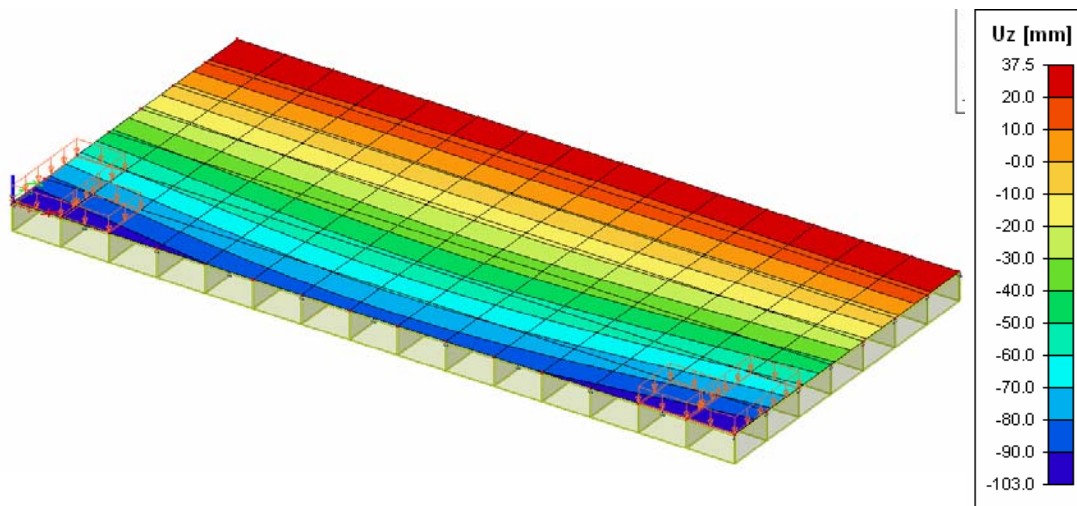


Figure 58 Rotation by load on straight opposite corners

#### Pavilion half loaded

By loading half of the pavilion with  $5\text{kN/m}^2$  the rotation will become critical, since it leads to a height difference of 710 mm. This loading is excluded for the pavilion but it is interesting to take a look at the consequences.

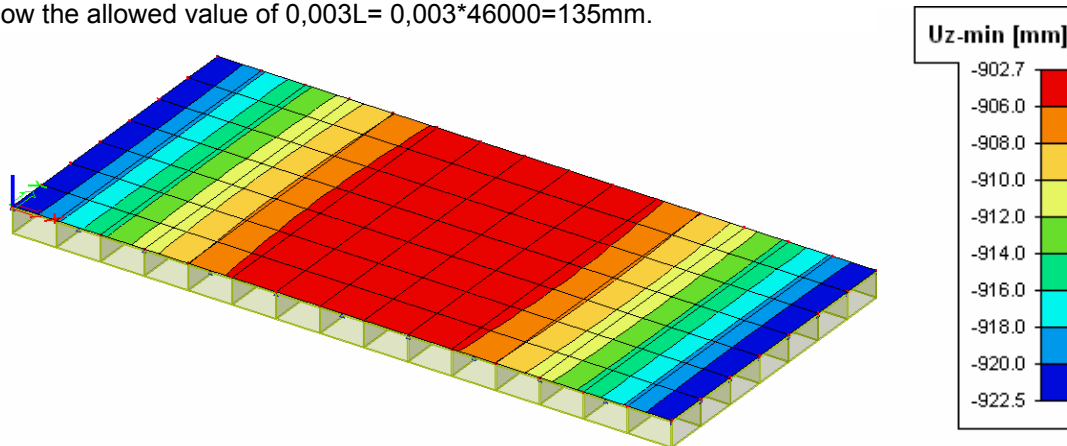
The rotation of 1,6% is still largely below the demanded 5% but larger than the desired 1%.

This load situation results in a freeboard of less than 20 cm on the deep side, this value will become even less if the structure is loaded by snow at the same time. This value becomes critical, but is still enough. Only if due to trim or wave overtopping the risk of subsidence occurs (this is not the case for an EPS body), the loads has to be heightened with load factors. If load factors are applied in this case, the tilt will be too high.

The rotation of 1,6% however is still largely below the demanded 5% but larger than the desired 1%.

#### **Deformation**

The largest deformations occur if both edges are loaded. The deformation of loading with load situation 3 (twice the maximum surface) is only 19,8 mm (see Figure 59), so this is quite low and far below the allowed value of  $0,003L = 0,003 \cdot 46000 = 135\text{mm}$ .

Figure 59: Deformation by both edge loaded by imposed load ( $2 \times 21 \times 3\text{m}^2$ )

It is difficult to see the pure deformation of the one side loaded floating body, but this will be close to the deformation of the two side loaded floating body, since the hogging moments are also in the same range.

### Large torsion by load on opposite corners

The load situation with both diagonal opposite corners loaded, which resulted in relative low internal forces does however result in very large deformation, see Figure 60.

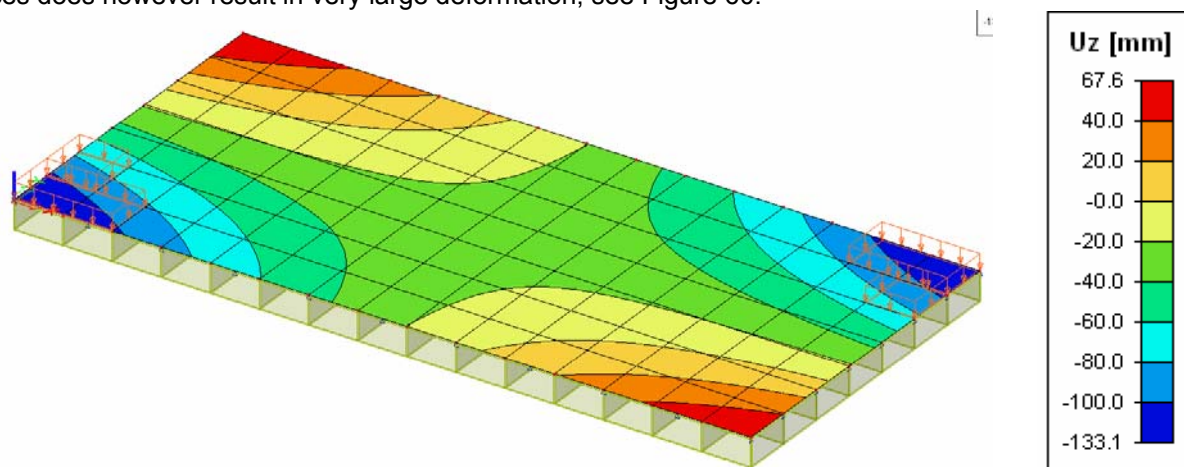


Figure 60: deformation by load on diagonal opposite corners

The deformation by this torsion is in total 200mm. This is quite much. The rotation that occurs at the edges is in this situation even larger than for tilt for all edges. This means the floating body is more susceptible for torsion than for tilt!

Thus this torsion loading results in large deformation, but it will not be a problem for the strength for the beams or for the connections, since the internal forces will stay rather small. The largest shear force that occurs by this loading is 32,74kN.

## Conclusions Imposed Loads

For the imposed loads the following conclusions are drawn:

- With 'line loads', loading on one side completely on the edge, gives the biggest moments in the floating body
- Half the load on both edges gives a little lower result, than the just mentioned situation
- Scia results can be used, they give the correct outcome (the moments have to be raised with the normal forces)
- With 'rectangular surface loads' the biggest forces are reached if these loads are placed on the corners of the same long edge.
- For normative sagging moments and tension forces in the lower connections, the imposed loads have to be placed on top of the connection.
- The moments by imposed loads are approximately twice as big as the outcome of the load situations calculated in the chapter before
- With the mentioned maximum surface for the imposed loads the bending and tilt stay relatively small. However, a quite large torsion can occur.



## 16.3 Wave Loads

### 16.3.1 Vertical loading, by long wave of 30m

In paragraph 14.7 is explained and calculated in what water pressure and vertical load the waves result. In Figure 61 the resulting figure of paragraph 14.7 is shown. The figure depicts a floating body of 45 metres, loaded with a wave with a length of 30 metres and resulting water pressure. The drawn wave results in a sagging moment. Here a wave of 30 metres is taken, because this length followed of using the nomograms for transitional water (see chapter 14.7).

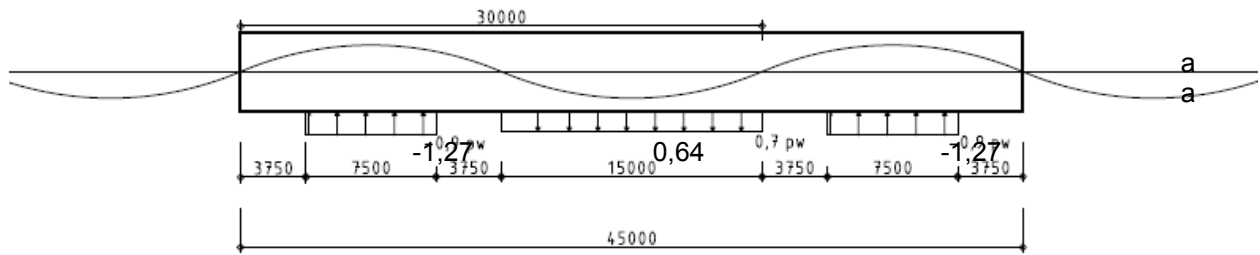


Figure 61: Wave with length of 30 metres with resulting pressure on structure

The up pointing surface load will have a length of  $\frac{1}{4} L$ , and the down pointing surface load will have a length of  $\frac{1}{2} L$ .

For a design wave height of 1,51m and an amplitude  $a$  of 0,758m (see Table 7: Wave characteristics) this results in an up pointing surface load of  $9,59 \text{ kN/m}^2$  and for the down pointing surface loads a value of  $4,79 \text{ kN/m}^2$  (see 14.7).

These loads are inserted over 7,5m and 15m on the rectangular floating body in Scia, see the figure below:

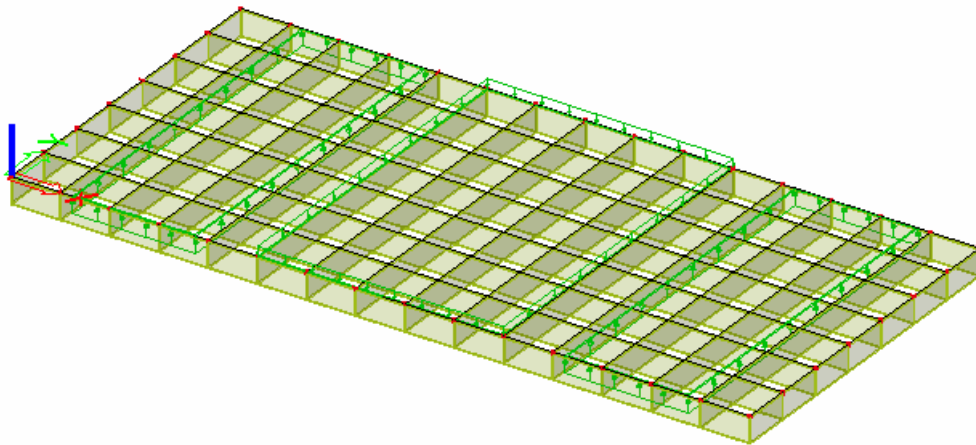


Figure 62: Rectangular floating body with load by 'sagging wave'

### Moments

This resulted in the following moments in the beams, see the figure below:

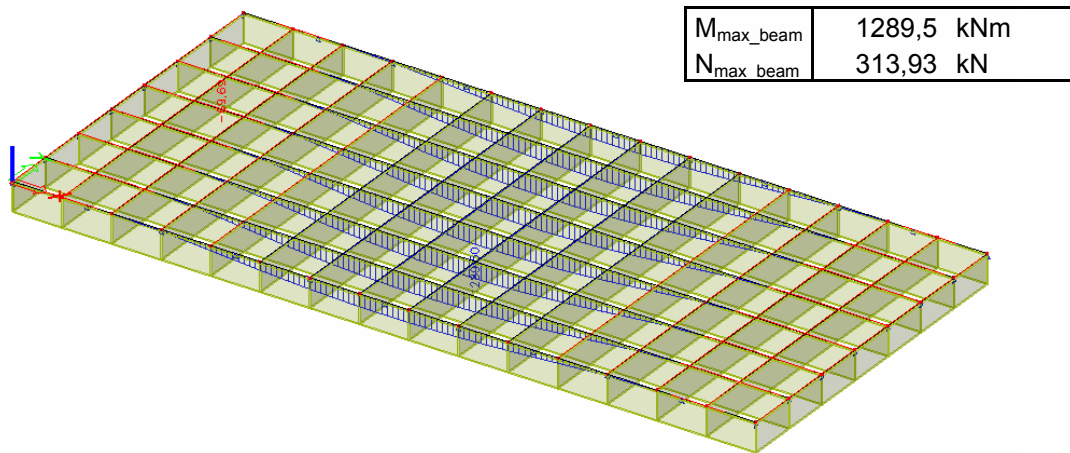


Figure 63: Moments in beams by 'sagging wave'

the depicted values for the maximum moment and normal force are the forces caused by the wave load only and not by the combination including self weight.

As mentioned more often before and calculated in appendix 27, Scia divides the moment in a moment in the beam and a normal force in beam and floor. For this wave load this results in the following total moment per beam:

$$M_{tot,beam} = 1289,5kNm + 313,9kN \cdot \frac{1}{2} \cdot 1,65m = 1548,5kNm$$

A hogging wave results in a total moment of 1751,2kNm. With a sinusoidal wave shape, both moments would have been equal, thus by changing the wave shape to a more cnoidal shape the moments change by approximately 12 percent.

### Deformation

The maximum deformation for the sagging wave is 45 mm and the maximum deformation by the hogging wave is 56 mm. This is still far below the allowed deformation of 135mm (0,003\*I).

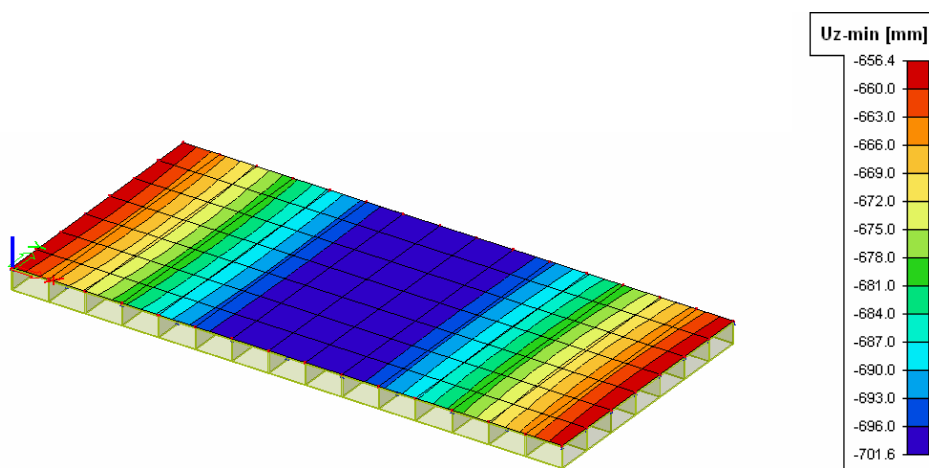


Figure 64: Deformation by sagging wave

### Conclusion

The moments by this wave load will be a lot bigger than by the imposed loads (more than a factor 5). Now it appeared this wave was normative, there is taken a closer a look at the wave loading. In fact

only after this, Bretschneider has been used and only after this was also taken a look at other effects that might change wave characteristics, which are all discussed in chapter 3. In the next paragraph will be looked at the effects of wave loading, by waves with a much shorter length, which is calculated with the formulas of Bretschneider.

Deformation was even by this severe loading not a problem, so it won't be a problem in any situation.

### 16.3.2 Vertical loading by shorter waves

The formulas of Bretschneider resulted in a much shorter wave. The wave characteristics of the longest and highest wave, which are calculated with Bretschneider are shown in the table below:

depth (m)	Wave height $H_s$ (m)	Design Wave height $H_d$ (m)	Wave period (s)	Wave length (m)
7	0,67	1,51	2,73	12

Table 13: Characteristics highest and longest wave according to Bretschneider

The wave length of 12 metres is a little longer than the calculated 11,62 metres, but a wave with a length of 12 metres makes inserting of the load and checking by hand much easier. The outcomes will only be a little over estimated.

Shorter waves can be normative too.

#### **Normative wave**

With the long wave, the normative wave for the maximum moment was quite easy to determine, it was a wave which started with a crest or trough at the beginning of the floating structures and also ended with a crest or trough (the same maximum at both ends). So than the normative wave had a length of 2/3 of the floating structure or just somewhat longer, as shown in last chapter.

The normative wave for the maximum moment is for the shorter waves much more difficult to determine. Several wave positions and wave lengths can be normative but it is logical to again assume that the maximum moment will occur with again with a same maximum at both sides. This can be the case with a wave with a length of 10 metres.

From trying some different wave loading in Scia it appeared that this wave is not normative for the internal moment. The longest wave length resulted in the largest moments, for this situation that is a wave with a length of 12 metres, see Figure 65 and the resulting moments in figure 66.



Figure 65: Example of wave load by wave of 12m

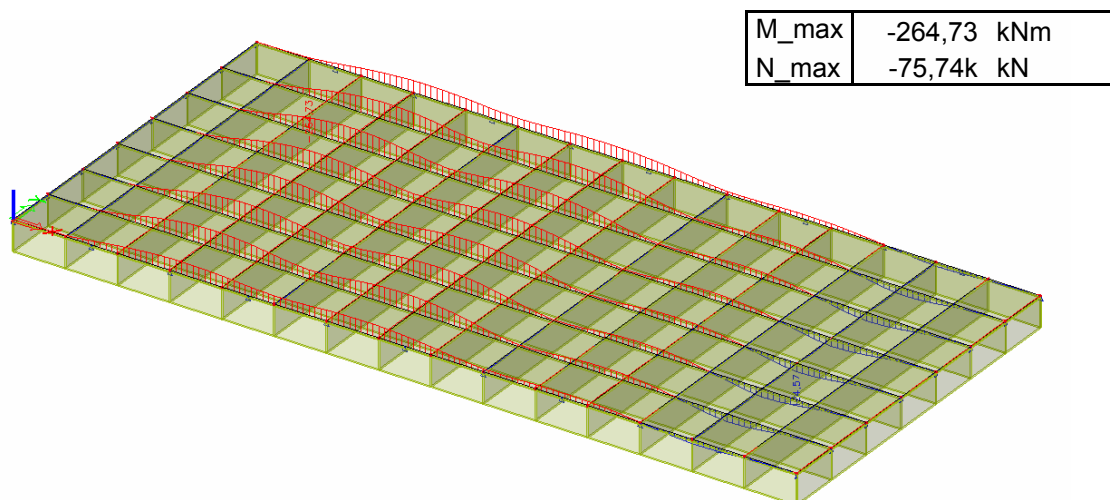


Figure 66: Moments by wave of 12 m

This resulting moment is only a little smaller than the maximum moment by the imposed load, which was 289,45kNm.

### 16.3.3 Diagonal Wave Field

Of course the waves will not only come perpendicular to the floating body. The wave will also ‘attack’ the floating body under an angle. These diagonal wave fields can also be translated to static surface loads. An example of the surface loads by a diagonal wave field is depicted in Figure 67. In this figure the floating body is loaded with the approximately normative wave for this floating body, since it corners on the same edges have got opposite loading. This wave was just taken to look at the consequences, according to the formula's of Bretschneider waves with this length will not occur in the Rijnhaven.

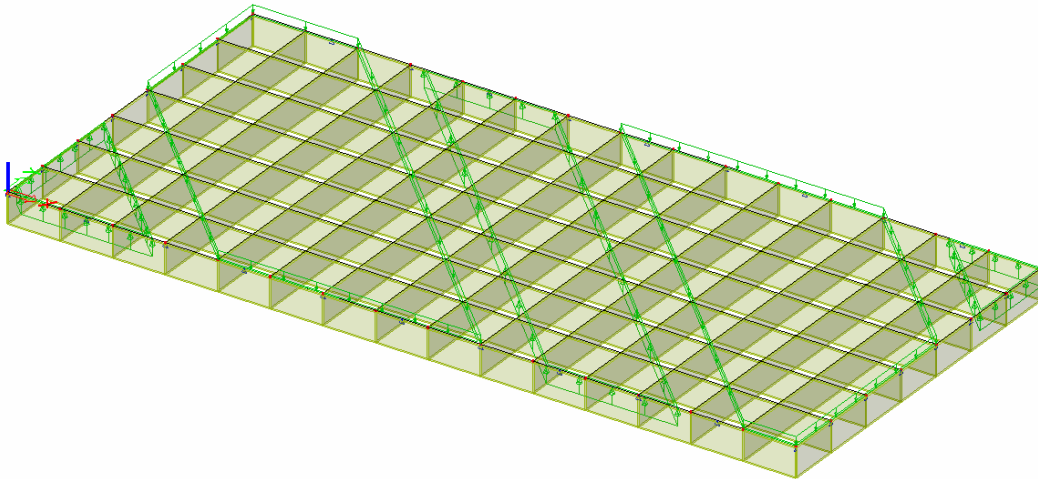


Figure 67: Diagonal wave field on floating body,  $l_{\text{wave}}=21,2\text{m}$

#### Moments

The internal forces caused by this diagonal wave loading are in the same range but a little smaller than the normative forces found by perpendicular wave loading of waves with the same wave length. (Compared to the perpendicular loading there is a factor which increases the internal forces and one factor which decreases the internal forces. Because of the diagonal wave field the, the wave in fact appears longer for one beam, which results in higher moments. But now the forces will be transferred in two directions which result in smaller internal forces)

#### Deformations

With the imposed loads it already appeared that the floating body was very susceptible to torsion. Thus also diagonal wave loads result in larger deformations than the deformations by the perpendicular wave fields, but the deformations by the waves are smaller than the deformations by the imposed loads on the diagonally opposite corners.

#### Shear forces

The just mentioned torsion causes (vertical) shear forces in the beams. The largest shear force caused by the normative wave of 21,6m depicted in Figure 67, has a value of 102,1kN. With the shorter wave lengths these values will be a lot lower. This force is however already lower than the shear forces caused by the superstructure. Thus diagonal wave loading will not cause problems for the shear forces.

### 16.3.4 Horizontal forces

The draught of the pavilion is less deep than the trough of a standing wave, so the formula and value for the horizontal force, given in paragraph 14.7.3 has in fact to be adjusted and lowered. In fact it was already on the high side, since a real standing wave will not occur in this situation. But if this value of paragraph 14.7.3 is used, this can be seen as an absolute upper limit.

The (horizontal) shear force caused by the horizontal wave loading, depends at where the moorings of the floating structure are situated. When this is done as depicted in chapter 13, the maximal horizontal shear force will be lower than  $15\text{m} \times 45,6\text{kN} / \text{m} = 684\text{kN}$ . This total amount will be taken mainly by the floor.

If the pavilion will be modular this force have to be passed on by the connections. These connections can be equally divided over the beams which take this horizontal force. If this force is taken by 6

beams (presumed there are no connections in the edge beams, the modular pavilion has also 6 connectors), the shear force per connector is maximal  $648/6 = 108$  kN

### 16.3.5 Conclusions

With the nomograms waves were found that resulted in largely bigger loads than the loads by all other loadings, so then there had to be looked closer into the wave loadings and then Bretschneider was used. Bretschneider gave waves with a largely shorter wave length which results in far smaller moments. The resulting moments of waves calculated with Bretschneider are in the same range as the moments caused by the imposed loads. The normative waves will only appear during heavy storms.

The normative load combination is a storm wave load + imposed load.

In chapter 15 was concluded that the beams had quite some reserve. In appendix 29 is calculated and concluded that the in this chapter found loads can be resisted. The normal linear deformation of the floating body is very small. The rectangular framework is however very susceptible to torsion, so torsion will result in large deformations.

## 17. Stability and Movement

The possible tilt of the pavilion has already been elaborated in chapters 15 and 16. This chapter is about the dynamic motions.

The movements surge, sway and yaw will be prevented by the moorings. The movements heave, roll and pitch can occur, the slow heave motion by the tidal movement doesn't give any problem. The quick heave, pitch and roll motions are undesirable. As explained in chapter 3, these movements depend on natural oscillation periods. To define the natural oscillation frequencies, again is made use of the schematisation as a rectangular floating body of 21x45m.

### 17.1 One piece Pavilion

#### 17.1.1 Stability

As explained in chapter 2 the stability depends on the height of the metacentre. The heights of the metacentres are calculated in appendix 29. The metacentre heights, measured from the keel, are given in the table below:

Height metacentre (short direction)	45,2 m
Height metacentre (long direction)	206,1 m

Table 14: Height metacentres

These meta center heights are far above the centre of gravity (1,7m), this means the pavilion is highly stable. This was already known, since in chapter two was already said that floating structures with a width larger than 9 metres and a low draught will in normal situation always be stable.

#### 17.1.2 Movement

##### Natural oscillation periods

For calculating the movements, the rectangular schematisation has been used again, because with the rectangular shape the in chapter four given formulas can be used.

The natural oscillation periods are calculated with the formulas given in appendix 9. The calculation sheet is depicted in appendix 29 . The outcomes are the following:

natural oscillation period heave short direction	5,69 s
natural oscillation period heave long direction	11,6 s
natural oscillation period roll	1,83 s
natural oscillation period pitch	1,81 s

Table 15: Natural oscillation periods

In the values of the table the added water mass is not yet taken in account. In appendix 9 was said that the natural oscillation period with the added water mass taken in account will be approximately  $\sqrt{1,5}$  (=ca. 1,2) times bigger.

##### Moving along

In chapter 15 was stated that the periods of the normative waves are around 2,7 seconds. So for the heave movement the floating pavilion has natural oscillation periods which are largely longer, so heave oscillation will not occur.

The roll and pitch natural oscillation periods however, are lower than the normative wave periods. This means resonance can occur!

Luckily, for the long direction (pitch), this won't give to much problems, since the pavilion is a lot longer than the occurring wave length, so a large pitch movement because of the waves is not likely to occur.

But for the short direction of the pavilion, there does exist a problem. It will have the tendency to move very easily along with the waves. This can be prevented by increasing it's width with a rigid

connection to the floating plaza before the pavilion, this will more than double the width, so than also in this way the length of the combined floating structure will be much longer than then wave length, so there is no reason left for large roll movements. But for the pavilion which is now being built, there has not be chosen for a rigid connection.

For the rest of this thesis, for looking at the movement of the modular pavilion and for designing the connections, the floating pavilion and the floating plaza will be considered as two separate floating structures, which can roll and pitch without influencing each other. So just as in the real situation in this thesis there is no rigid connection between pavilion and plaza.

After the designing of the connections between the modular pavilion parts is taken a look if this connection can also be used for the connection between pavilion and plaza. This connection can be better be done with another connection, or the both floating bodies have to be placed against each other.

## 17.2 Modular Pavilion

### 17.2.1 Schematisation modular pavilion

For calculating the movements the rectangular schematisation of chapter 18 has been used again. The schematisation is exactly the same as the rectangular floating body of chapter 18 (same dimensions, same weight, etc.). This floating body is separated in to three parts, in a way that the surfaces are almost equal to the real surfaces. There is chosen for lengths which are a multiple of three, so the same grid can still also be used for the modular parts, see Figure 68.

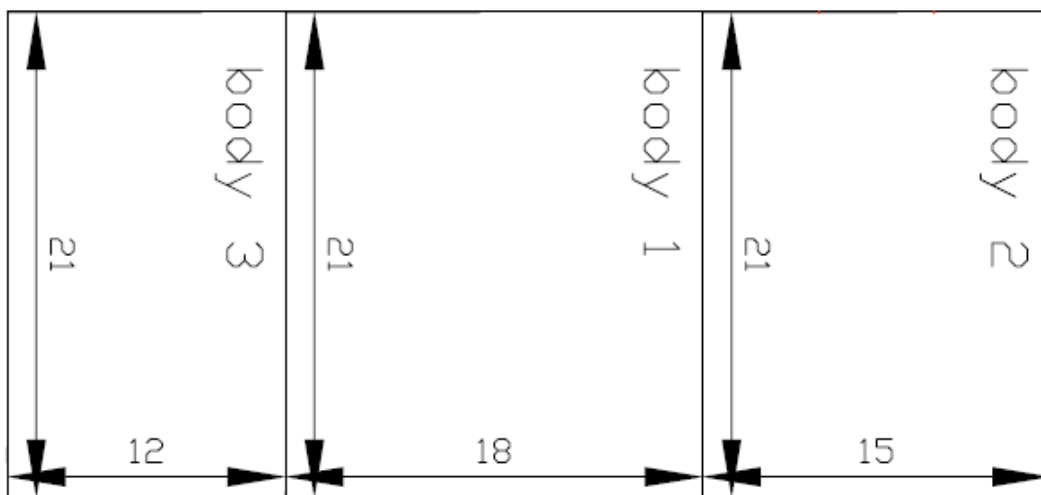


Figure 68: Schematised modular floating body, topview

### 17.2.2 Stability

The heights of the metacentres are calculated in appendix 29. The metacentre heights, from the keel up, are given in the table below:

	Body 1	Body 2	Body 3
Height metacentre, y-direction (m)	41,3	41,3	41,3
Height metacentre, x-direction (m)	30,5	21,3	13,8

Table 16: Height metacentres of modular parts

These meta center heights are far above the centre of gravity (1,7m), this means the pavilion is highly stable.



## 17.2.3 Movement

### Natural oscillation periods and corresponding movement

The natural oscillation periods for heave for the separate modules are smaller than for the one piece pavilion. But still larger than wave period. For roll and pitch the natural oscillation periods are a little larger, around 1,94 seconds (see Table 17 and appendix 29). This is again lower than the normative wave periods. The length of the modular parts are in the range of the normative waves: the normative waves have a length of around 11 metres. The modular parts have lengths of 12 to 18 metres. This fact, together with the fact that the natural oscillation frequency of the modular floating structures lie in the same range and are a little smaller as the wave frequency, will have as result that especially the smaller outer parts will have the tendency to move along with the waves, if they are disjointed.

		body 1	body 2	body 3	
<b>Dimensions floating body 1</b>					
width floating body	b	21,00	21,00	21,00	m
length floating body	l	18,00	15,00	12,00	m
height floating body	h	1,70	1,70	1,70	m
Area	A	378,00	315,00	252,00	m <sup>2</sup>
Draught	d	0,82	0,82	0,82	m
<b>Stability</b>					
$GM = KM - KG = \frac{b^2}{12d} + \frac{1}{2}d - KG$					
Height centre of gravity	KG	1,70	1,70	1,70	m
Height metacentre (short direction)	KM_1	41,28	41,28	41,28	m
Height metacentre (long direction direction)	KM_2	30,45	21,28	13,78	m
Metacentric height (short direction)	GM_1	39,58	39,58	39,58	m
Metacentric height (long direction)	GM_2	28,75	19,58	12,08	m
<b>eigenperiods, roll</b>					
eigenperiod, short directon (2pi/f)	T_0_y	1,92	1,92	1,92	s
eigenperiod, long directon	T_0_x	1,93	1,95	1,99	s

Table 17: Characteristics of modular parts

In the table above the colored fields are the entry fields. The values in the non-colored fields are calculated with the formula's given in appendix 8 and 9.

## 17.3 Conclusions for connections

In the last paragraph was concluded that the outer parts will have the tendency to move along easily with the normative waves.

A lot of movement in the pavilion is undesired. By connecting the modular parts the total length becomes larger and the total length of the jointed parts will not be in the range of the wavelengths anymore. This means less movements will occur.

But now, if the modular parts are connected with hinged connections, they will still have the possibility, to rotate free from each other. This is very undesirable for multiple reasons. For example form comfort point of view and another reason is because the superstructure will normally not be able to follow these large rotations.

So from out of view of movement, the modular parts should be connected in a way that shouldn't allow rotations. So a rigid connection is needed.



## 18. Choice connection system

### 18.1 Choice connection system

From chapter 17 followed that from viewpoint of movement should be chosen for connecting the modular parts in a way that doesn't allow rotation. This can be done in two ways:

- flexural connection between the floating bodies and also structural connection between superstructure.
- rigid connection between the floating bodies

If a rigid connection between the modular parts appear not to be possible, because of too high forces or other reasons, there will be taken a look at connections which will allow rotation.

#### 18.1.1 Rigidity via superstructure

One possibility of connecting the modular parts in a way that doesn't allow rotation is by a flexural connection between the floating bodies and a connection between the superstructure. The idea of this solution is that in this way a large height between the upper and lower connections exists, so the moment can be taken more easy. From the chapter Structural Design can however be concluded that the superstructure is light relative to the floating body. If the superstructure should be able to take the moment, which than will be imposed on it by waves and imposed loads, the superstructure have to become a lot more massive.

The internal loads where the superstructure is now dimensioned upon is quite low (maximal normal forces in the dome elements are now around 70kN (ULS), see appendix 24. The moments in the beams of the floating body exceed 800kNm(ULS), see paragraph 18.2. This means that with an average connection height of 8 metres and also a connection every 3 metres the normal forces in the superstructure will increase at least with  $M/h=800/8=100\text{kN}$ . This means more then a doubling, thus passing on the moment via the superstructure means a lot of extra amount of steelworks, which brings a lot of extra expenses. Here the remark has to be made that the costs of the steel superstructure of the pavilion are already higher than the costs of the floating body.

Next to this, this solution has got other disadvantages, for example that this high lying connections will also be difficult accessible.

#### **Conclusion: not via superstructure**

So first is chosen to see if the floating bodies can be rigid connected. If this is not possible, or brings also disadvantages with it, there will be again taken a look at this solution. In that case the option with adding a new structure of from example an lattice works will also be taken a look at. At first this is not done, because this brings restrictions to the functional use.

#### 18.1.2 Rigidity by connection between floating bodies

As said in the latter paragraph at first is chosen for a rigid connection between the floating bodies. From part 2 of this thesis can be concluded that this should be possible. From viewpoint of dividing the forces over the beams it seems logical to make a connection between every beam. this is depicted in Figure 69.

From the structural design from the one-piece pavilion could already be calculated that the beams can resist the loadings that belong to a floating body of 46 metres. Thus that will not result in problems. Further, for the beams have been chosen a quite large height so a connection with a certain moment capacity should be possible. The forces which act on the connection are determined in next paragraph.

## 18.2 Forces in modular pavilion and connections

In last paragraph is said that there will be placed a structural connection between every beam, see Figure 69. If the connections can be schematised as fully rigid, or having a rigidity in the same range of the beams, the forces on the connections can be taken directly from the Scia model from the one-piece pavilion, by taking a look at the normative forces in the beams.

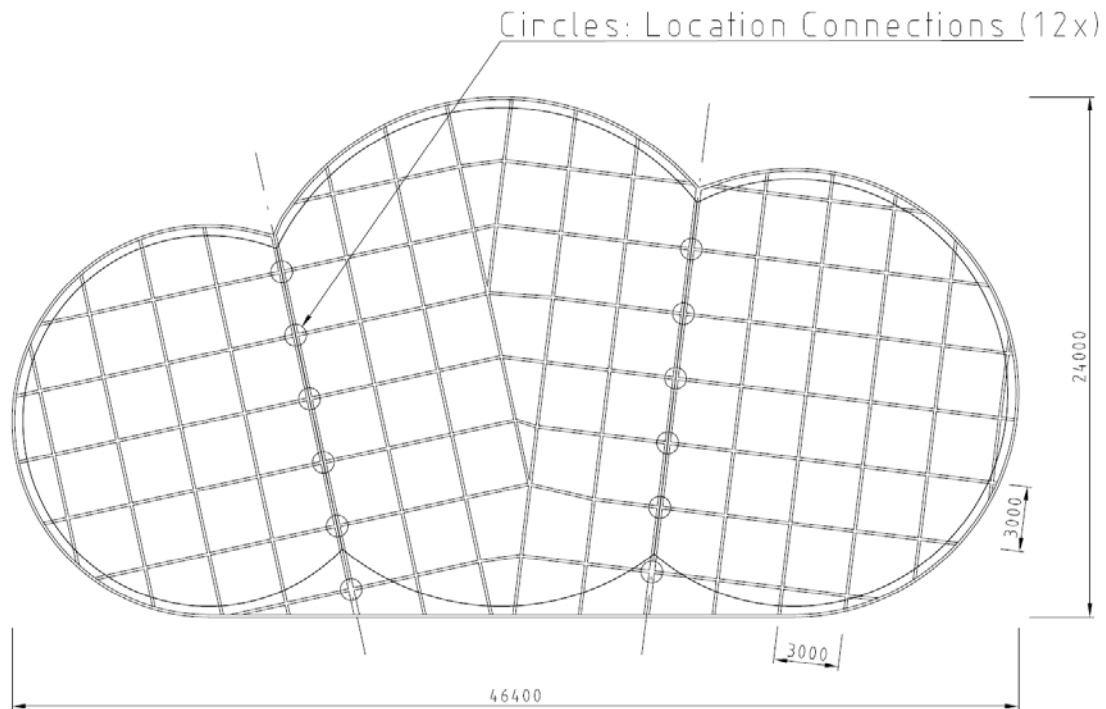


Figure 69: Location of connections

The forces by wave loading and imposed loading will be taken out of chapter 16. From the structural design from the one-piece pavilion and the rectangular schematisation could already be calculated that the rigid beams can resist the loadings. The calculations can be found in appendix 28.

### 18.2.1 Combinations

The design forces in the connections will be taken from both the real shape schematisation and the rectangular schematisation, thus the normative forces in the beams out of chapter 15 and 16 will be used. Combination of waves with imposed loading and other load cases will be taken in account. The normative forces of the wave will be summed with the normative forces by the imposed loads. For combinations the reduction factors are taken in account (as described in EC1991).

#### Reduction factors

The following combination reduction factors will be used:

- 1,0 and 0,6
- 0,8 and 0,8

The second mention option, 0,8 and 0,8 will in fact never be normative, since multiplying the largest load with 1,0 and the less high load with 0,6 results in a higher value.

## 18.2.2 Resulting Moments

In Table 18 the normative loads which results in the highest load are depicted:

Loadcase	Origin	Moment by SciaSLS	Heightened moment SLS	Load factor	Mom ULS	Red factor	Moment combination
<b>Selfweight+Snow Load</b>	Model Pavilion (ch 15)				147,2	1	147,2
<b>Imposed Load</b>	Rectangular Body (ch16)	289,45	314,59	1,5	471,9	1	471,9
<b>Wave Load</b>	Rectangular Body (ch16)	264,73	325,4	1	325,4	0,6	195,2
<b>Total moment</b>							<b>814,3</b>

Table 18: Calculation normative moment by combination, moments in kNm

Thus the total normative moment is 814,3kNm. For the calculation of the connections a value of 900kNm has been used.

### Remark: Heightened moment

The heightened moment is the moment including the moment caused by the normal force calculated by Scia.

### Remark: Load factor

On the wave load no extra load factor has been used, since there was already been made use of a probability factor to find the design wave height.

## 18.2.3 Normal forces

### **Normal forces by $M_y$**

The moments result in a tension and compression force in the connections:

$$N_m = \frac{M_y}{h} : \quad N_m = \frac{900kNm}{1,5m} = 600kN$$

This normal force by moments has to be added on the standard normal forces.

It was said before that the normal forces given by Scia where a result of dividing the moments over beams and floor. These normal forces, resulting from the imposed and wave loading, have already been taken in account in the heightened moment (for).

### **Normal forces by other factors**

#### Normal forces by horizontal wave forces

The horizontal wave loading does result in a compression force of maximum 108kN (see 16.3.4). The wave loads cause in principal no tension forces in the beams, but preventing relative surge may however cause some tension forces in the connectors.

#### Normal forces caused by wind

The wind on the superstructure will also cause some normal forces, mostly compression, in the connection. Wind in length direction results in compression forces in all directions with a value of ca.  $225/6=37,5kN$ . A wind perpendicular to the pavilion or a diagonal wind will also cause a moment around the vertical z-axis, so this results in both compression and tension forces. These by the moment caused forces will be less then 30kN.

#### Compression forces via contact surfaces

The compression forces will be of less importance for the connectors, since the compression will be transferred by the contact surfaces (see next chapter). The connectors only have to be calculated on

tension force. If the connection is pre-stressed this should also be taken in account in the normal forces.

#### Design load connections

All normal forces mentioned in last paragraph give for the compression forces a load of 170kN. As explained the tension force will be a lot smaller, but the connection is nevertheless calculated with a similar tension load, which is an upper limit. For calculating the connections a design normal force of 200 kN is taken.

So this results in a total normal force by moment and normal forces of:

$$N_d = N_{by\_moment} + N_{N\_combined} = 600 + 200 = 800kN$$

The normal compression force in the real shape model of the floating pavilion had a value of 351,66 kN. This value is used for checking the beams.

### **18.2.4 Horizontal shear force**

Horizontal wave and wind load together cause the normative horizontal shear force. The horizontal wind load on one dome is extracted from Scia and results in a value of ca. 225kN (ULS). This has to be divided by 6 connectors:  $225/6=37,5$  kN. The horizontal wave load resulted in a value of 108kN. With the mentioned reduction factors this results in a combined force of 145,5kN.

Since this shear force might not be equally divided over the connections, a shear force of 200kN is used for calculating the connections.

This horizontal shear force is largely bigger than the vertical shear force.

## 19. Structural Design Connections

Chapter 17 showed that from viewpoint of movement should be chosen for a rigid connection between the modular parts. Chapter 18 demonstrated that the best way to achieve this is by a rigid connection between the floating bodies and not via the superstructure. Earlier chapters found that one large rigid floating body will not result in problems; the beams are strong enough. An important demand is the disconnect ability of the connection.

So the design of the connection, for in between the modular parts of the pavilion, can be based on the disconnectable rigid connector, that was designed in paragraph 10.4. In this chapter the connection will be elaborated further, dimensioned and designed in detail.

The location of the connections can be found in the last chapter and in appendix 31.

### 19.1 Connection design

#### 19.1.1 Basic Design

In part 2, paragraph 10.4 was stated that a rigid connection can be best realised by the following:

- upper connection: horizontal longitudinal bolt: for tension, tightening the joint and pre-stress
- lower connection: vertical pen (combination vertical bolt and Flexifloat connection): for easy connecting and tension
- elastic material in between, for amongst others preventing small relative movement that will exist by allowing deviations and for impact damping.

Below the initial design sketches of the connections are presented, which contain the required principles that were just mentioned. The left sketch depicts the connection with the short bolt, the right sketch depicts the connection with a long vertical pen. These sketches are the base for the detailed design of the connection.

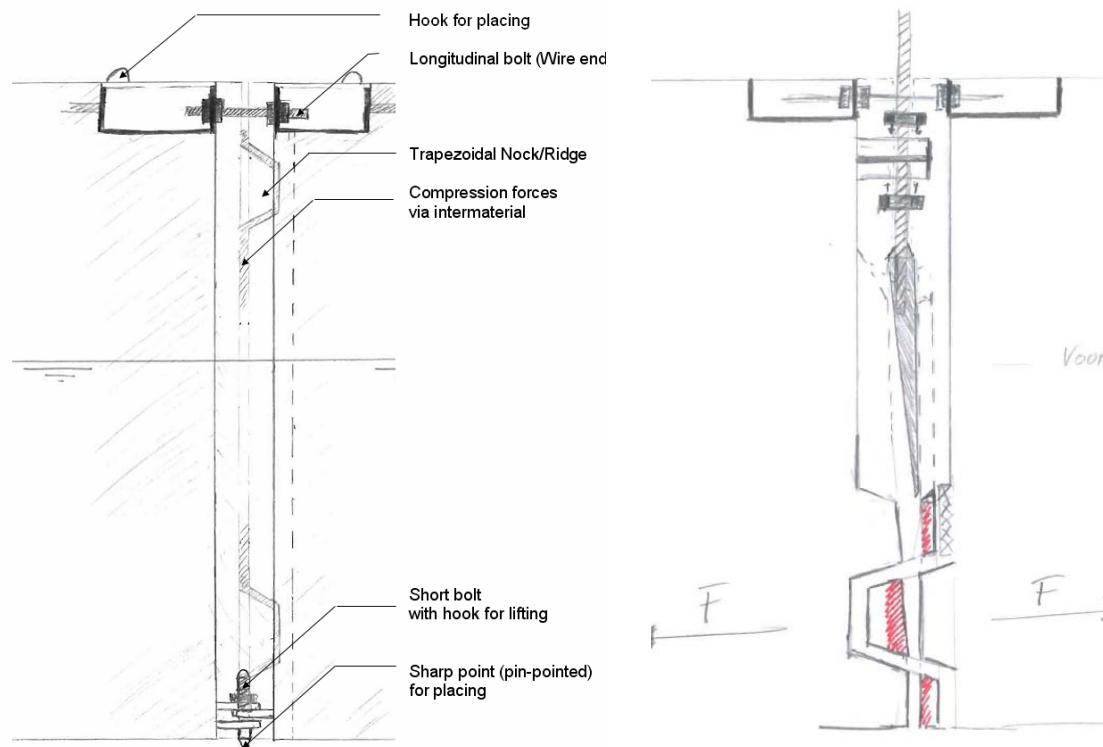


Figure 70: Initial design sketches of rigid connection, side view. Left: short bolt. Right: wedge shaped long pen.

## Materials

The connection will consist of the following materials:

- Upper and lower tension connection:
  - o Bolts and nuts: Steel strength class 8.8
  - o Pen: Steel strength class 8.8
  - o Plates: S355
- Elastic intermaterial: rubber/neoprene
- Trapezoidal ridges: Concrete with steel plate covering

All steel elements should be (hot dip) galvanized. This is one of the most durable protection methods. For the long pen, which is especially susceptible for corrosion, the corrosion protection is mentioned more in detail. For the lower locking plates account that the corrosion protection layer might be damaged while connecting. These parts will be below water level, so when the layer gets damaged the corrosion will not go too fast, maximal 0,1mm per year [Stigt Thans, 2010], so that is 5mm in 50 years. At every place where the steel connection parts might abrade with each other, the zinc layer should be protected with a layer of Teflon.

### 19.1.2 Design of lower connection

The key problem with the low connection is that it is not easily accessible. With a pinpointed vertical pen or bolt the placement is relatively easy, but as mentioned in part 2, especially the fixation is a problem. Therefore a long pen is chosen to realise the lower connection, with the fixation at the top of the floating bodies.

#### Long pen

With a long pen the execution is far more simple, however this solution has two disadvantages:

- A long pen brings along some extra material costs
- Corrosion around the waterline

#### Material Cost

Compared to the total costs, the material costs for the pen are low. Besides that, these costs can be earned back since the pen reduces total system cost, because of a less complex construction, lower risk for complications and a shorter construction time.

#### Corrosion Pen

Materials around the water surface are even more susceptible to corrosion. Without the long pen this problem does not occur because the low steel parts are always below the water and the high parts are always above the water surface. With the pen this is not the case, so if there are no precautions taken, the pen will corrode faster.

Corrosion can be prevented in the following ways:

- Pen of stainless steel
- Normal steel with layer paint
- Hot dip galvanizing (thermisch verzinken)

#### *Pen of stainless steel*

It might not be wise to use stainless steel and normal steel at the same time. This will cause galvanic corrosion, which will occur especially around the contact surfaces. Since this will normally be right at the normative spots of the connectors, this will be extra disadvantageous. Normally, this galvanic corrosion is prevented by adding a layer of synthetic material in between, but in this connection the specific contact surfaces are the most heavily loaded surfaces of the structure. So a synthetic layer is not likely to hold under those circumstances.

#### *Layer of corrosion protection*

The corrosion can also be countered by a protective layer of paint around the water surface, but these paint layers have often proved to not very durable.

#### *Galvanizing*

Galvanizing provides an impenetrable and rustproof layer, with a good durability.

Galvanizing is the selected solution for the long pen, since this is the most durable solution.

### Wedge shaped point for placing

The pen will be pinpointed for 'automatic alignment' with the hole. With a pinpoint the pen can be inserted through the connection when the holes are not exactly aligned. By pressing the pinpoint down, it will push the locking plates away, to enlarge the hole. (This principle will be explained for this specific case later on in this paragraph).

In the case of connecting two floating bodies with ridges for self alignment, the situation of the holes not being aligned, can in fact only happen in one direction. There can be an intermediate distance between the floating bodies, and then the holes will not be above each other. In Figure 71 the top view of the lower connection with different intermediate distances is depicted.

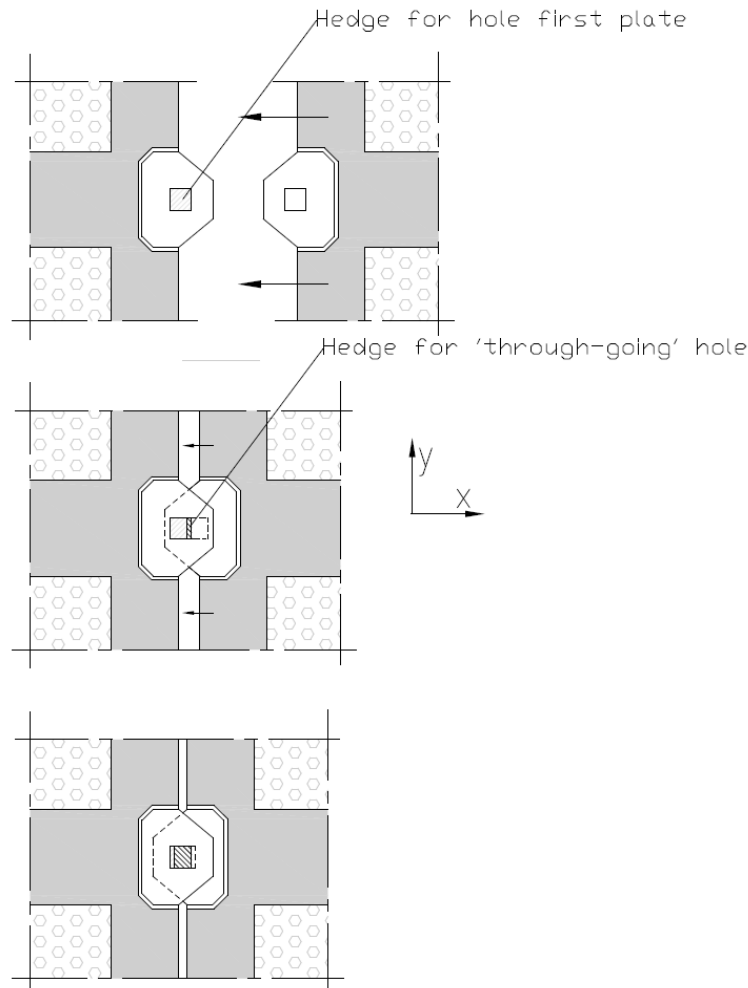


Figure 71: Top view lower connection, with different intermediate distances

Figure 4 shows that the intermediate distance can only occur in the positive x-direction (seen from the left floating body, the body with the upper plate). In the other directions, the y-directions and the negative x-direction, such a location deviation between the two connection-halves will not occur.

The intermediate distance in negative x-direction is not possible since the floating bodies can not move more towards each other then straight against each other. And because of the self alignment caused by the ridges, there can only be a small location deviation to the sides (y-direction).

The previous implies that the 'through-going hole' (the overlapping area of the holes through which the pen can be inserted) will always first appear on the right (positive x-direction) of the above lying hole. This is also shown in Figure 71. It is shown that the 'through-going hole', becomes larger when the two floating bodies are moving more towards each other. This means that for placing, the pinpoint needs to be primarily sloping to one side. This results in a 'wedge shape' point. This is depicted in Figure 72. (Former principle can also be seen in the side view 'cartoon' of the end result in Figure 85, this could provide more clarity.)



Figure 72: Side view wedge shaped point

### Wedge allows range of intermediate distances

When the wedge shape of figure 5 is used and the holes also feature sloping sides, the floating bodies can be brought closer towards each other by pushing the pen down. This principle is depicted in the figure below:

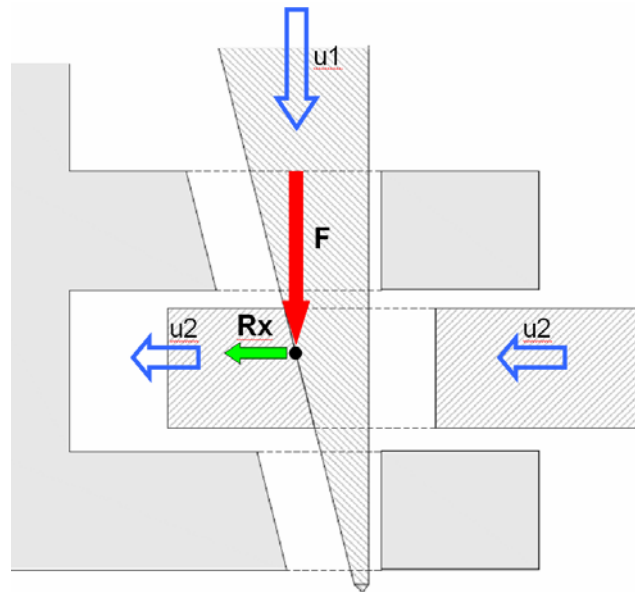


Figure 73: Bringing bodies closer together by pushing pen down

### Explanation of Figure 73

- The wedge is pushed or hammered down:  $u_1$
- This results in a force on the sloping side of the center plate:  $F$
- This force induces reaction forces, amongst others a reaction force in x-direction:  $R_x$
- This horizontal force results in a relative movement of the center plate, and attached floating body, to the left:  $u_2$

In Figure 73 can also be seen that this connection can bear the tension forces at a range of intermediate distances, depending on how deep the wedge is inserted.

These findings suggest that a wedge shaped pen point can be used for coupling floating bodies, even with larger intermediate distances. So with a wedge shape point, location deviations can be countered.

### Fixation pen

Because of the wedge shape, the pen can not drop out downwards anymore, but when the floating bodies tend to move away from each other, this will push the pen upwards. So the pen needs a good fixation at the top that keeps it down. It was already mentioned that this needs to be on the top of the floating body, because that way it will be easily accessible.

### Range of heights and driving force

It would be very beneficial if this fixation could be realised for different intermediate distances. It would be even better if with this fixation the pen can also be pressed down and moved downwards. This means this upper connection should be able to give the needed force at all possible height ranges of the pen, belonging to the range of intermediate distances of the floating body. It should not only give a passive reaction force, but also be able to give an action force to drive the pen in. With the connection drawn in Figure 70 this is possible, but the lower nut, which provides the force downwards, is in fact not accessible. The connection which realises the downward force should be very well accessible, preferably from the top.



### Reaction force

The normal force in the pen equals the tensile force in the lower connection divided by the slope of the wedge. Later on in this chapter the slope of the pen will be chosen, but here the force in the pen of the end result is already given, so one knows in what range of dimensions this is about:

$$N_{pen,d} = F_d \cdot slope\_wedge : N_{pen,d} = 800 * \frac{1}{8} = 100kN$$

### Two options

Below two options are given, at both options the pen will be pressed down by screwing the screws down, see Figure 74:

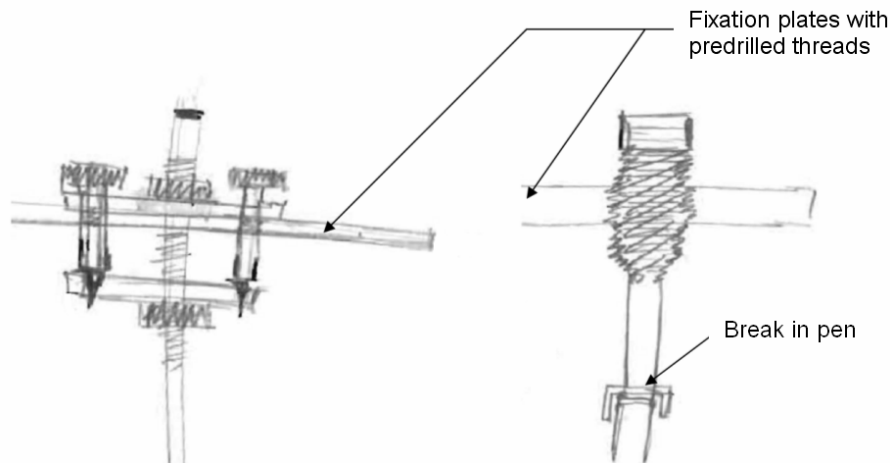


Figure 74: two options for easily accessible fixation pen, side view

Both options feature a horizontal plate which is attached to one of the floating bodies. The pen can then be fixed to this horizontal plate.

#### Left option

This option makes use of the following:

- two separate plates which should be placed after the pen has been inserted
- 2 or 4 bolts which will press the pen down via the lower plate and nut.

#### Right option

The right option makes use of one big 'bolt' which presses the pen down. This is done with a break in between, so the wedge shaped pin point will not rotate along.

The left option has been selected, because this was judged to be more easy by the people from practice. 'Push bolts' are in fact more used in construction practice. So bolts can be bought which have tip specially shaped for this function.

The fixation plate needs to have a relatively large height, since the strength of the predrilled thread is clearly lower than that of the bolts of strength class 8.8.

### **Lower locking plates**

The lower locking plates will also have a little trapezoidal shape, so the interlocking will run more smoothly. The plates need to be relatively solid and strong, so they will not deform when something collides against them.

### 19.1.3 Design upper connection

The upper connection exists out of a longitudinal bolt or wire end, which constitutes the tension connection. Using wire ends in combination with steel head plates will provide the strongest and most durable solution. After the wire ends and nuts have been placed the joint can be closed by tightening the nuts.

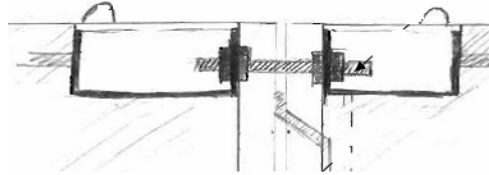


Figure 75: Initial sketch upper connection

Initially holes were chosen which were open at the top (see sketch in figure 38 in part 2). This is easy in execution, but it is structurally not a very good option. So other options have been generated:

#### Some options:

- Adding an extra head plate to the sketched initial design. The extra head plate can be slid in from above, after the wire end is placed (the added head plate has an opening at the bottom). Then this added head plate can be bolted down to the steel box.
- The wire end will be inserted into disjointed steel boxes. After the wire is inserted through the steel boxes, the steel boxes will be bolted down at the right position on the floating bodies.
- The same steel boxes as the initial design sketch, but now with normal round (closed) holes in the head plates. The holes will be larger, so there will be more hole clearance for easy placing.

The third option is selected: closed bolt holes with extra clearance for easy placement. This results in a structurally good solution with the least necessary acts. See the result paragraph and appendix 31 for drawings.

### 19.1.4 Design ridges

The ridges need to be big enough for self alignment and transferring shear forces. The ridges need to interlock earlier than the bulge of the lower connection, so they need to be more protruding. The trapezoidal ridges will consist of concrete with steel plate covering, against damaging the concrete ridge and at the same time this steel plates will be formwork during construction.

### 19.1.5 Elastic intermaterial

The function of the elastic material was 19.1.1 and more elaborately in part 2 of this thesis. The intermaterial can be made out of rubber profiles or neoprene. In this thesis there is no research done what elastic material can be used best, nor what the strength is, nor the needed amount of material. This study limits itself to the statement that it is best to use elastic intermaterial and that it should be possible to find a strong and durable enough material, since the mentioned materials are also used in the supports of bridges etc.

## 19.2 Dimensioning connections

The forces on the connections were calculated in previous chapter. These forces are used for dimensioning and checking the connections. In the table below the normative strengths are given:

Tension force upper and lower connection: $F_d$	800 kN	(ULS)
Shear force ridges: $V_d$	200 kN	(ULS)

Table 19: Forces in connections

Dimensioning has been done with simple dimensioning rules, which are meant only for dimensioning, these formulas are mostly not in the codes. The correct formulas are used for validation. Most important dimensioning is shortly described in this paragraph, without much explanation. In appendix 30 more elaborated dimensioning calculations and verifications can be found.

### 19.2.1 Upper connection

#### Bolt diameter

The tensile force in the upper connection is transferred by one or multiple longitudinal bolts or wire ends. The wire ends are loaded by pure tension. The required bolt surface,  $A_{bolt}$ , can be estimated

by: 
$$A_{bolt} = \frac{F_{t,d}}{f_y}$$

When filling in the appropriate parameters, this results in the following required bolt surface:

$$A_{bolt} = \frac{800 \cdot 10^3}{640} = 1250 \text{ mm}^2$$

To realise this required bolt surface, two bolts with a diameter of 30mm are chosen, this results in a surface area of 1414mm<sup>2</sup>.

#### Plate thickness

The plate thickness of the head plate is, according to the rule of thumb, chosen equal as the bolt thickness: 30mm. During validation this 30 mm was found to be largely over dimensioned, and the thickness could be reduced to 20mm.

### 19.2.2 Lower connection

#### Dimensioning pen

The vertical pen is loaded by shear. The pen thickness can be estimated by:

$$A_{pen} = \frac{F_{v,d}}{\tau}$$

The tension force in the lower connection is 800kN. For calculating the surface area of the pen, the shear design load has to be used. This shear design load is equal to the tension force divided by 2, because the shear force on the pen will be divided over 2 shear planes.

This results in the following surface:

$$A_{pen} = \frac{800 \cdot 10^3 / 2}{800 / \sqrt{3}} = \frac{400 \cdot 10^3}{462} = 866 \text{ mm}^2$$

This can be realized with a wedge shaped pen, with dimensions of 50x20 = 1000 mm<sup>2</sup> at the smallest section (the lowest). During validation it appeared that this section does not fulfil the requirement completely; the minimum shear surface needs to be 52x21mm<sup>2</sup>.

### 19.2.3 Ridges

The ridges transfer the shear force. If there is no extra reinforcement used for shear force, the dimensions of the ridges can be calculated as follows:

$$A_{ridge} = \frac{V_d / n}{\tau_c}$$

with: n = the amount of ridges

$\tau_c$  = shear strength of the concrete = 0,4\*tensile strength

$$\text{for } f_{ck}=55: f_{ctk}=3,0 \text{ N/mm}, \quad \tau_{cd} = \frac{0,4 \cdot 3,0}{1,5} = 0,8 \text{ N/mm}^2 \text{ [Eurocode 1992-1-1]}$$

$$A_{ridge} = \frac{200 \cdot 10^3 / 4}{0,8} = 62,5 \cdot 10^3 \text{ mm}^2$$

This required surface can be reached with 250x250mm<sup>2</sup>. The base of the ridges are chosen to be 250x250mm<sup>2</sup>. This means the surface area's which will be smaller more to the 'top' of the trapezoids, will not fulfil this demand by concrete only. But together with the steel plate covering and the steel fibres, the strength will be easily fulfilled.

## 19.3 Detailing connection

### 19.3.1 Upper connection

#### Bolts

Choice: 2 bolts M30. (M30 is largest standard, one size is bigger is M33).

Chosen for rolled screw thread (rolled screw thread is 15% stronger than cutted thread).

#### Hole

At first a hole, open at the top, was chosen for easy placement, but finally a closed hole with extra hole clearance (gatspeling) was preferred. Normally the hole clearance is 2 mm. A hole clearance of 6 mm was chosen here. For a bolt with a diameter of 30mm this leads to a hole of 36mm.

#### Nuts

The nuts have a height of 0,8 times the diameter of the bolt: 24mm

Normally the wrench width (sleutelwijdte) of the nut is 1,5 times the bolt diameter. For two reasons the width of the nuts is taken somewhat larger:

- When using prestressing bolts, the width of the nut has to be taken somewhat higher.
- The hole is taken somewhat larger than usual here.

For this connection the nuts have the outer dimensions of a standard M36 nut. Then the outer diameter is 66mm with a wrench width of 55mm. (NEN1560)

#### Tightening bolts

The tightening of the bolts will be done with a torque wrench, see figure 9.



Figure 76: Torque Wrenches: electronically readable, classic and electric torque wrench

### Space around bolt and nut

For placing and fixation there has to be enough space around the bolts and nuts. The demanded minimum space from the centre of the bolt to the side is 1,5 times the hole diameter (Overspannend Staal):  $1,5 \times 36 = 54\text{mm}$ . In the final design, with 90 mm around the center this demand is spaciouly fulfilled, see Figure 77 and drawings in appendix 31. This way there is enough space for an electric torque wrench.

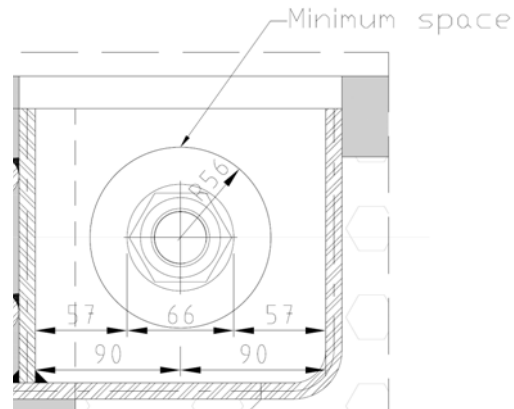


Figure 77: Space around bolt

### Steel boxes

The shape of the steel boxes will be as simple as possible. The design needs to result in enough space for the connectors and fixation of the connectors. The steel boxes will be realised with as less separate plates as possible and with as less welding as possible. If welds are necessary it needs to be preferably done with fillet welds.

### Anchorage

The anchorage of the steel boxes in to the concrete will be done by welding reinforcement bars to the steel boxes, see Figure 78 and drawings in appendix 31 This way the introduced tension force is passed on directly into the beams. This anchorage by bars also provides a more stiff solution than with dowels.

The bars will be placed in a way that they can be directly welded longitudinally to the steel 'boxes'. This is done in a way the welds will be easily accessible for the welder. From calculations it was found that this could be realised with 5 tension bars with a width of 24 mm (see validation in appendix 31).

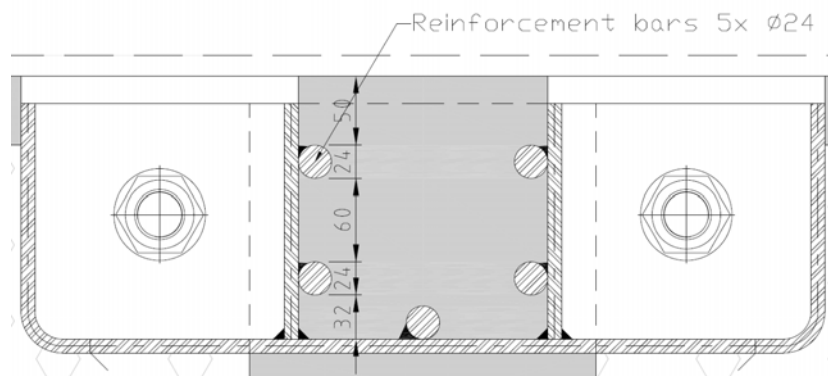


Figure 78: Section over upper connection: tension bars welded to plates

For a good joining with the edge beams, dowels are placed on the sides of the steel boxes (see drawing 4 and drawing 5 of appendix 31)

### 19.3.2 Lower connection

#### Wedge

The wedge is chosen to have an almost square topside and the tip should be as thin as possible.

#### Slope

The wedge has a sloping with a chosen slope of 1:8. A less steep slope (e.g. 1:5) results in more horizontal movement with the same vertical movement of the pen. With a steeper slope, (e.g. 1:12) the wedge needs to be longer for the same horizontal movement, or the allowable intermediate distance between the floating bodies will be smaller. A steeper slope results in decreasing the force needed for moving and less axial forces in the pen when connected. The chosen slope is a compromise between both.

#### Thickness tip

The wedge is eccentric and needs to have a quite sharp point, this way the connection can be realised when the intermediate distance between the two floating bodies is still relatively large. So, the wedge needs to have a sharp tip for placing, but not too thin, because that way it could deform too easily.

Figure 79 shows how these criteria are realized.

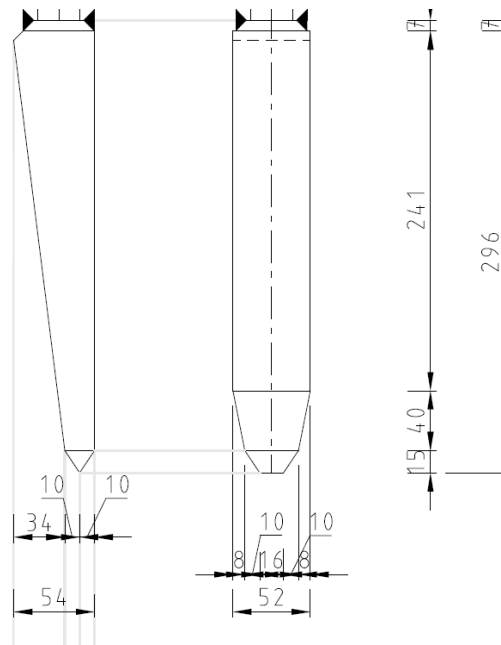


Figure 79: Wedge shaped pen point, side view (l) and front view (r).

At the utmost of the wedge, the wedge is pinpointed in all four directions, so the wedge will find the hole itself. At the spot where the wedge slope starts the thickness is 20mm, this is thick enough to prevent deformation.

During first dimensioning was found that when connected, at the lowest shear section the wedge needs to have a surface area of least 866 mm<sup>2</sup>. The validation showed that this has to be enlarged to 1045,2 mm. When the pen is inserted a few centimeters in the lower area this required shear area is fulfilled by 52x21mm.

With a width of 52mm and depth of 54mm the top is approximately square.

The possible intermediate distances for the connection with this wedge shape and the resulting vertical movement of the pen, will be discussed in the results paragraph.

#### Lower locking plates

The plates will also have a little trapezoidal shape, so the interlocking will run more smoothly. A plate with a small thickness is cheaper and beneficial for the location of the resulting tension force in the connection: If these plates are thin, the connection will have a low height, so then the resulting force can be located relatively low, and then the resulting force can be well transmitted to the center of the

thick part of the beam. (See drawings in appendix 31). But to keep some robustness they should not be too thin, because then they could deform easily when a collision occurs. At the spot where the pen goes trough, the thickness of the trapezoidal shaped plate is approximately the same as the top width of the pen: 50 mm.

The plates including the holes will be casted steel pieces. When the plates are as wide as the beams and as protruding as the ridges, the nett section of the plates will be strong enough. (see validation).

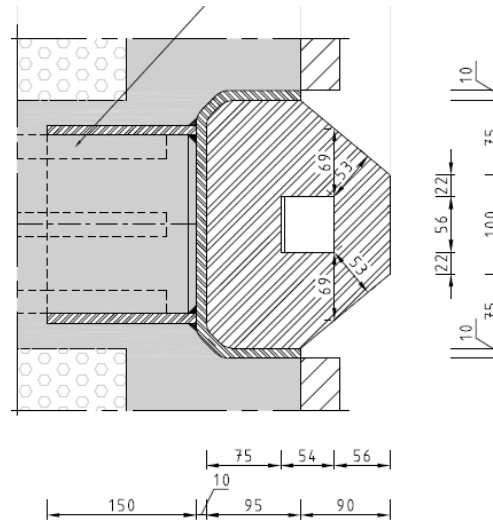


Figure 80: top view plate

#### Holes

The wedge has a width of 52mm, so the width of the holes needs to be somewhat wider, 56 mm is chosen here. The depth of the holes is increasing with the height, with the same slope as the wedge (see result).

#### Anchorage

The anchorage will be done in the same way as the upper connection, with the same amount of reinforcement bars, see Figure 81. (for more drawings see appendix 31)

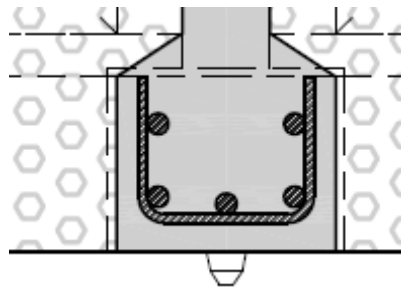


Figure 81: Section over lower beam: Anchorage lower connection with reinforcement bars

Welding the reinforcement bars directly to the trapezoidal steel plates seems easier, but the chosen option also has other benefits. The steel plates in the chosen option, which are placed around the locking plates, make sure that these plates will always be parallel (above each other) and by anchoring in this way, the resistance against rotation and deformation is also higher. On top of this, the steel plates around the locking plates form also automatically a formwork for the cavities for the opposite plates.

## 19.4 Result

### Overview

Below the end result is depicted in a few drawings. All drawings can be found in Appendix 31. The drawings below serve to show the main concept of the connection.

Section C-C' is the section exactly over the heart of the longitudinal beams and the vertical pen. Section D-D' is the section straight over the ridges. For the exact location of the section is also referred to Appendix 31.

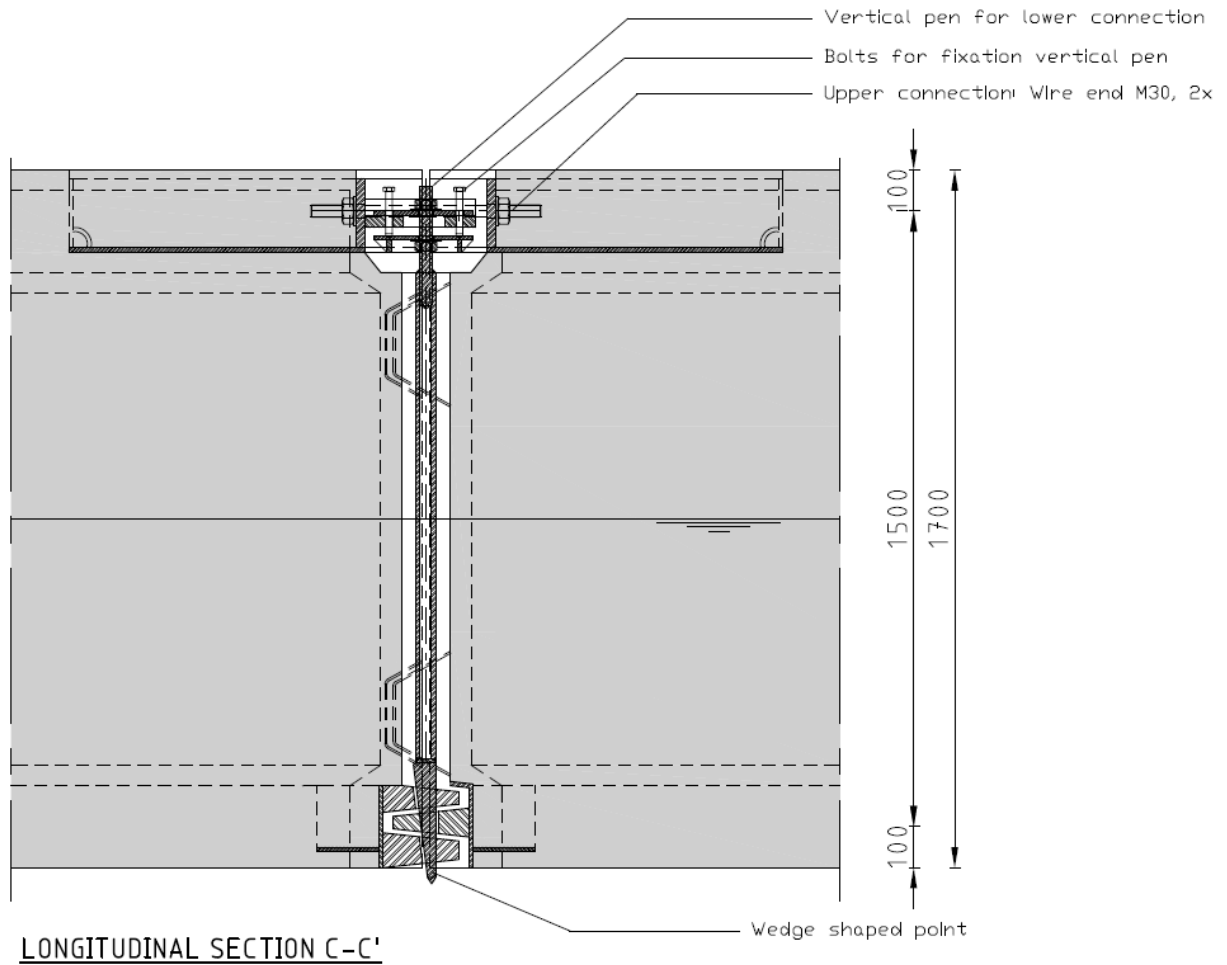
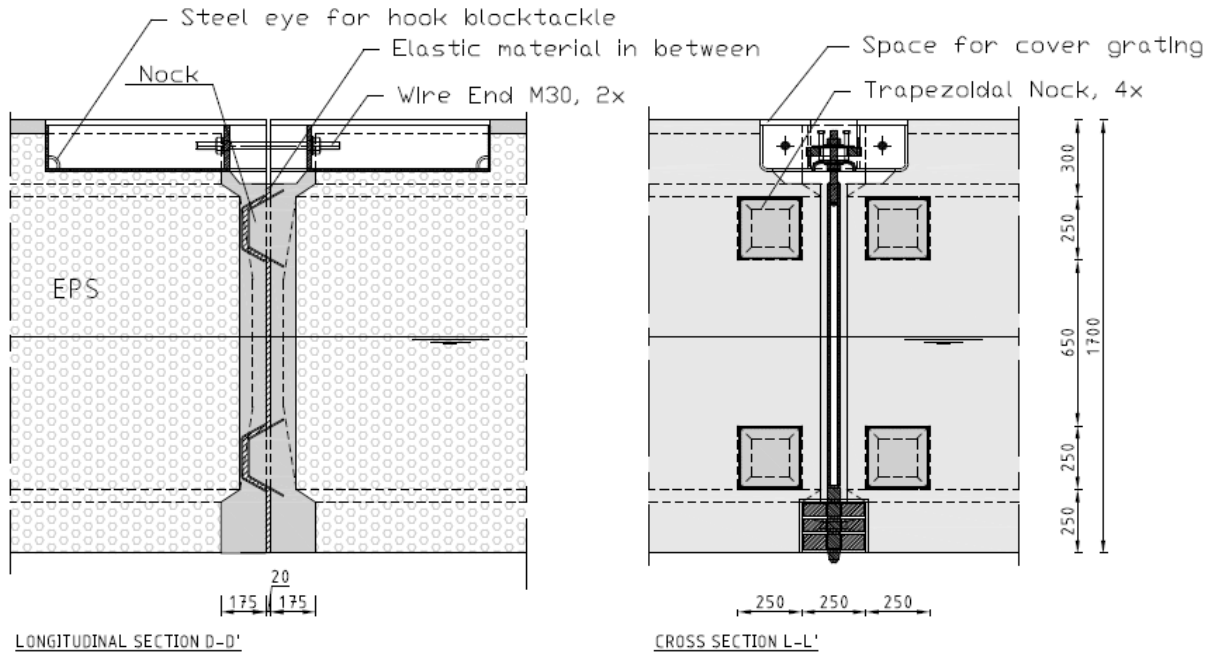


Figure 82: Rigid connection pavilion, section C-C'





Figur 83: Rigid connection pavilion, section D-D' and L-L'

In appendix 31 multiple sections can be found to show all connection elements and how they are positioned relative to the concrete and EPS lattice system. For all dimensions and details also see appendix 31.

### Fixation pen

In the figure below a detailed image of the fixation of the pen of the lower connection at the top of the floating body can be seen. With this solution the complete connection can be realised from the top of the floating body.

There is space left above and underneath the fixation to allow the pen to move 70 millimetres up or down ( $80 - 10 = 70\text{mm}$  and  $110 - 40 = 70\text{mm}$ ).

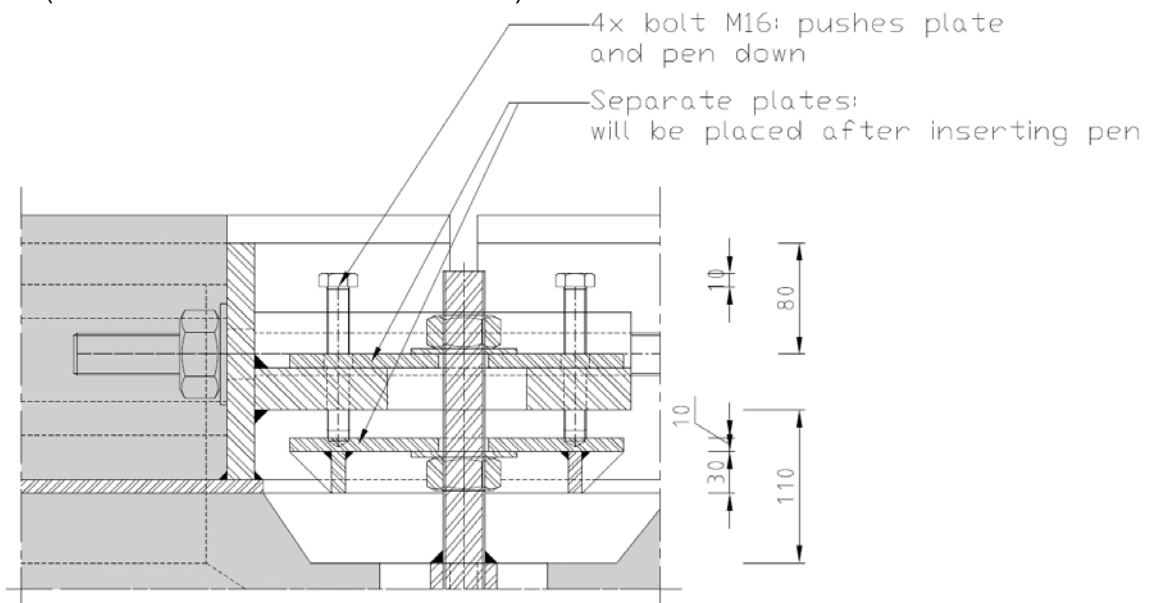


Figure 84: Fixation pen detail, side view

For the bolts special push bolts are available, with special push plates. In between the push bolts and plates there will be a layer of Teflon to protect the corrosion layer.

Stiffener plates are welded to the main fixation plate and lower fixation plate, to provide more rigidity. All welded elements will be welded all round.

## Characteristics connection

The design presented fulfils all given demands and functions:

### The main characteristics of this design:

- Rigid (prestressed) connection
- Self alignment
- Easy jointing, completely from the top of the floating body (see Figure 84)
- Simple connection, with low amount of actions, with well known simple and durable techniques
- Accommodating tolerances
- Durable
- Simple in production
- Low costs

All points listed above were mentioned before, and they are fulfilled in the ways that were suggested. Last four points are elaborated some more below:

### **Durable**

The connection is designed in a way it results in a durable connection, since there has not been made use of mechanic parts like jacks or chain wheels or what so ever, but there has only been made use of the simple and reliable bolt connections. However when the bolt and thread ends start to rust, the connection might become not detachable anymore. In the design precautions against this are taken, but the corrosion cannot be precluded after being in a wet environment for multiple years. This corrosion will not result in failure of the connection, since with the used corrosion preventing, the corrosion will be limited during a lifetime of several decades. The lifetime can be further increased by replacing the detachable elements as bolts, nuts, pen and thread ends.

### **Simple in production**

The demand to be simple in production has been realised by the following:

- No need for adjusting the beam system and sizes, no adjusting of the standard EPS blocks at the edges, since the connection is designed in a way that it will fit in the standard beams.
- No difficult formwork. One straight concrete edge, with the connectors which form almost completely their own formwork, since they are surrounded with steel plates.
- Relatively simple steel elements: consisting of relatively little different parts. Not much operations needed, the operations needed on the steel are simple:
  - o 90° plate bending
  - o only fillet welding (see details in appendix 31)

### **Low costs**

The connection is designed with as simple techniques as possible and it is simple in production as just mentioned. On top of this there is no need of adjusting the standard EPS/concrete system, thus the costs will be relatively low.

### **Self alignment and accommodating tolerances**

Accommodating tolerances is realised by choosing connectors which allow some size and location tolerances and different intermediate distances between the floating bodies.

Self alignment is realised by shear ridges and by connectors which close or reduce the intermediate distance in between by tightening them.

### Intermediate distance between the floating bodies (Location tolerance)

This connection is designed for a standard intermediate distance between the floating bodies of 20mm. In this space in between there will be elastic material. This elastic material will, in unloaded situation, have a thickness larger than 20 mm, so with a space in between of 20 mm it is pressed together so this results in a kind of pre-stress of the connection.

With 40 mm in between the two floating bodies, or less, the lower connection can be realised. This is because with this intermediate distance the openings of the lower connection are placed sufficiently over each other, so the pen point can be stabbed partially through the centre plate. See Figure 85c.

Because of the sloping point the floating bodies will be pressed together when the pen is pressed downwards.

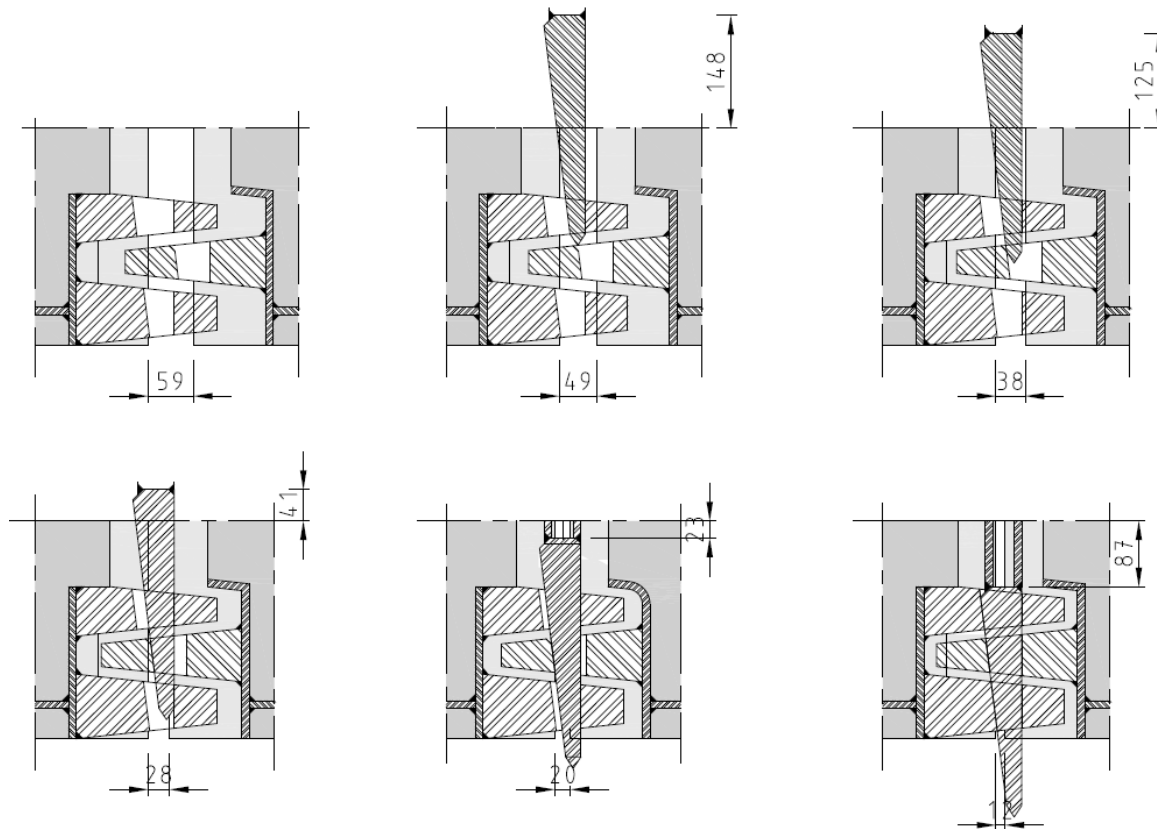


Figure 85: Inserting pen causing decreasing intermediate distance

With an intermediate distance of 28 mm the pen can be inserted deep enough in the lowest hole, so that the connection will be strong enough for the normative situation (see Figure 85). When the pen will be pressed completely down, the intermediate distance between the floating bodies will be 12 mm. This is the smallest intermediate distance possible. So the connection functions within an intermediate distance between 12 and 28 mm.

When the pen is fastened in the upper fixation, the height of the pen can be adjusted for 70 mm (see Figure 84). With a wedge with a slope of 1:8, moving the pen 70 mm up or down results in changing the intermediate distance between the floating bodies with 9 mm.

So the pen should first be inserted and hammered down till approximately the right height and when the fixation of the pen is realised, the height can be adjusted.

### Validation and adjustment

From the validation in appendix 28 can be seen that all elements fulfil. But where the upper and lower locking plate of the lower connection fulfil the demands widely, the center plate does fulfil the demands barely. So it is best to enlarge the center plate and reduce the upper and lower plate.

## 19.5 Execution

### Connecting

The execution of the connection can be realised in the following 10 steps:

- step 1: Bring floating bodies together with ships and block tackle (for example of blocktackle see chapter 10)
- step 2: Make sure the connectors are approximately at the same height, and have got the right heel. Otherwise adjust draught and heel by using trim weights.
- step 3: Bring floating bodies closer together with ships and block tackle, let them interlock and tighten cables with blocktackle so the gap between floating bodies becomes closed
- step 4: Realise lower connection by inserting pen
- step 5: Insert pen further (possibly with hammer)
- step 6: Add nut and plates in fixation
- step 7: Add vertical bolts and fixate lower connection
- step 8: Make upper gap smaller with block tackle
- step 9: Add longitudinal wire ends
- step 10: Close intermediate distance and bring connection on tension by tightening bolts and screwing down bolts for pushing down pen, screw top bolt on.

Done

### Disconnecting

Disconnecting of the connection has to be done just in the reverse order as the just described connecting. First as well the upper as the lower connection can be loosened. Then the push-bolts have to be removed and subsequently the vertical pen can be pulled out of the lower connection by tightening the top bolt of the pen. By tightening this top bolt, the bolt pulls the pen up. When it has come loose the pen can be removed, as well as the other elements.

## 19.6 Applicability Connection

This connection is designed especially for this floating pavilion with this floating system. But the designed connection could also be used in other situations.

Conditions that have to be fulfilled for using this connection are the following:

- The forces in the connections must be in the same range as in this design
- The structural height of the floating body should be in the same range or larger

Thus this means that this connection can be used for floating structures around the same dimensions, or for bigger structures with less severe wave and imposed load.

The connection can be used for the following systems:

- concrete caissons (all types)
- steel caissons
- EPS/Concrete combinations with high concrete beams

The connection can not be used for EPS systems with relative low beams such as the Flexbase system or the Ooms system, or there have to be made provisions in such a way that there is transversal strength in the lower parts of the body.

The connection has been designed for floating bodies in line, but can also be used for connecting in multiple directions.

# Conclusions

## Main conclusion:

### Resulting structural design of floating body and rigid connection

The main result of this thesis is a structural design for a modular floating pavilion. Findings show that a combination of EPS and a concrete framework are the best floating system for the pavilion, since this results in an unsinkable and light floating body. The concrete structure should have high beams for rigidity and for allowing rigid connections between the modular parts. It appeared that also with a rigid one-piece floating body, the internal forces caused by the wave and imposed loading can be resisted by the beams. This means, that the connections between the modular parts can be realised as rigid connections. A design of a detachable rigid connection consisting of steel elements has been made.

The designed connection consists of the following:

- Trapezoidal ridges, for self alignment and shear forces
- Vertical steel pen with wedge shaped point, for lower tension connection. Easily to insert and to fix from top of the floating bodies.
- Longitudinal bolt as upper tension connector and for pre-stressing the connection.
- Elastic material in between, for amongst others preventing small relative movement that will exist by allowing deviations and for impact dampening.

This design has resulted in a connection, which is easy in execution, since it provides self alignment and only actions from the top are needed. The connection does allow tolerances but will still provide a very rigid connection, since it can be tightened. The connection is designed in a way that it results in a reliable and durable connection, since there has only been made use of simple and reliable connectors. However, when the bolt and thread ends start to rust, the connection might become not detachable anymore. In the design, precautions to prevent this are taken, but the corrosion cannot be forestalled after being in a wet environment for multiple years. The connection consists of simple techniques and is simple in production, thus it results in a relatively cheap solution.

The designed connection is applicable for multiple floating systems and can be used for multiple projects, as long as the height of the floating body and the loads are in the same range as in this case study.

## List of minor conclusions

During this graduation multiple conclusions have been made, these minor conclusions are listed per subject on the next two pages:

### Research building on water in broad perspective:

- Building on water is a sparsely used building form, but it is strongly increasing. It is becoming more popular with local municipalities.
- Building on water is an answer to water problematics, since it is a form of multiple space usage, which provides water storage.
- Building on water results in new building space, which is especially of interest in densely populated areas with a large amount of surface water.
- Regulations about floating structures are improving, but are not sufficient yet.

### Floating Body

- The EPS/concrete system results in a floating body with the following characteristics:
  - o unsinkable
  - o small draught
  - o low maintenance
- For this reasons the combination EPS and concrete is best for the pavilion, as for the pavilion no internal space in the floating body is necessary.
- High beams are needed for rigidity (also for protection EPS)
  - o this not possible with Flexbase method
  - o choice for 'Maarten Kuijper system', but in principal without post/pre-tensioning

- post/pre-tensioning only if necessary, since this brings multiple disadvantages (see further below)
- In the case study was opted for self compacting fibre reinforced concrete C55/67, after elaboration the structural design, also a lower strength seems to fulfill, since the highest compressive strength found was  $13,6\text{N/mm}^2$ .
- The frame system appeared to be very susceptible to torsion. This torsion does not result in large stresses or forces for the connections, but does result in large deformation.

### Post/pretensioning

- More expensive
- Irregular shape gives more problems with post/pretensioning
- Beams with pretensioning can not be very small because of covering requirements and preventing 'buckling'
- Using Post/pretensioning in wet area is difficult, so preferably they should not to be constructed below water surface

For Pavilion:

- Not necessary for tension stresses, can be easily taken with normal reinforcement.
- Structure rigid enough

### Stability

- For floating bodies with a small draught and a width and length over 10 metres, shape stability will be normative over weight stability and they will easily be stable.

### Superstructure

- A dome structure gives other loads than a standard superstructure, but the superstructure is relative light, so this does not give any problems and the floating body does not have to be adjusted.
- A geodetic geometry without triangles, is from a structural viewpoint not a good geometry, as large moments will occur in the elements. With another geometry a lighter structure can be achieved.

### Loads

- For determining the internal forces caused by waves a cnoidal representation for the waves is taken. This schematization results in a difference for the internal forces of 12% with the linear schematization.
- For Pavilion: Waves and imposed loads result in internal forces which are approximately equal.
- The internal forces caused by the loads can be resisted by the floating body.

### Rectangular schematization

For determining the influence of waves and imposed loading a rectangular schematisation of the pavilion has been made. For the following reasons:

- This rectangular schematization gives more insight than the complex real shape, so the effects of the loadings are more clear and the results can be checked with hand calculations.
- The rectangular shaped model also matches with the formulas for the motions.
- Better application in the multiple computer programs.

### Movement

For Pavilion:

- Natural oscillation frequency of pavilion in range of waves
- Length of modular parts in range of wave length
- For not connected modular parts this results in tendency to move along easily

### Connections general

Using disconnectable connections in between floating structures can result in the following benefits:

- Flexible use
- Easier transportation floating structures
- Use floating bodies of different height, which can save a lot of materials, and makes trimming unnecessary
- Production inside possible
- Unlimited surfaces possible, also with the 'dry production methods' (not on water)

**Rigid connection needed and possible for pavilion**

- Because of easy moving along of modular parts, rigid connection necessary
- Rigid connection is possible concerning stresses

**Recommended general characteristics connection**

- connections need to allow size location deviations
- prestressing the connection is beneficial for multiple reasons
  - o preventing small relative movements
  - o beneficial against fatigue
  - o better self alignment possible and better functioning shear ridges
- self alignment results in easy execution

**Design rigid connection**

What the resulting rigid connection consists of is mentioned in the main conclusion.

The lower tension connection is difficult to access, therefore make use of long pen with fixation at the top. The pen should have wedge shaped point, this results in:

- o connecting possible at different offsets
- o prestressing of connection possible
- o with wedge shaped point tolerances and deviations can be countered

## Recommendations

Wave loading and hydrodynamic behaviour

- Determining the wave loads has in this thesis been done with some multiple simplifications and without real modelling and measurements. For a factual project which will be realised, determining the wave loads should always be done with wave measurements and/or the harbour should be modelled in a specialised wave computer program. Also the resulting internal forces should be extracted out of such a specialised wave computer program.
- In this research mostly the hydrostatical part of the behaviour of floating structures and the floating pavilion was covered. The wave loadings were translated into static loadings. The hydrodynamic part and especially the movements caused by waves should be further worked out with an advanced computer programs to predict the movements of (modular) floating structures. This should especially be done if there will be opted for non-rigid connections, because in that situation the relative motions can be very important.
- Further research to movements of floating structures by waves and the consequences for the comfort and well being of the people inside floating structures can be very useful

Floating system

- The floating system with EPS and relative slender concrete framework appears to be susceptible to torsion. This should be investigated. A research into the exact consequences and how this large torsional deformations can be countered could be an interesting topic for a next study.
- In this graduation no further research has been done to the concrete bearing structure. This means the recommendations for further research of the former graduations still remain. Further research for an optimal concrete/EPS floating foundation could consider the following:
  - o Optimal ECC mixture. A research into this optimal mixture could focus on many different criteria, as costs, execution, strength etc.
  - o Functioning of the EPS/concrete combination as well in executions as load bearing.
  - o Further optimization of the beam shape. With very slender designs, together with, as without tensioning, the local buckling effects should be taken into account. Other deformations should be studied with slender designs.
- Floating bodies completely out of synthetic materials, for example out of fibre reinforced plastics, could be interesting, since they are light and do not suffer from corrosion. This is an interesting graduation topic.

- Fibre reinforcement in concrete which is exposed to salt water, should not result in problems according to several sources. But in practice there are cases of some seriously deteriorated fibre reinforced floating bodies. Probably this is happened because of bad execution, but it would be wise to research the behaviour of fibre reinforcement in salt water conditions. Synthetic fibres might be a better option.

#### Stabilizing system

- This case study was elaborated with a large limitation in the number of visitors, this way the tilt stayed within limits. With a larger number of visitors or other ways of unequal loading a stabilizing system becomes more interesting. This could be done with water cellars, pumps and sensors. Research could be done into such a system.

#### Connection

- No research has been done to the elastic material in between. Before this connection will be used in practice this should be done.
- The designed connection should first be applied in limited use and tested. Then it can be adapted if needed. Subsequently there the durability of the connection can be studied.
- It could be very interesting to investigate when the rigid connection becomes less favorable in comparison with the hinged connection. There can be done further research to, and make elaborate designs of the other connection systems as hinged connection systems if their appear use for this systems.
- This connection has only be designed for inland water circumstances. Research for as well a hinged as rigid connection for high seas with large wave loadings can be useful.

#### Floating Structures in General

- The current building codes and regulations are not fit for floating structures. There need to be regulations which describe the demands on floating structures and to take all loads in account. New regulations and checks are expected, but probably the first regulations on this topic will still have multiple gaps. Thus researching will remain very useful. Suggestions out of this report could be used for it.
- As mentioned floating structures have not always been a success in the past, because of insufficient demand. With the more clear regulations and the better notoriety of floating structures this is becoming better, but for every project a research should be done if there is a market for the structures and what the demands are.

#### Economical Feasibility

- In this thesis nothing has been said about the economic feasibility. For floating structures the economic feasibility was not taken in account, because floating structures appear economically feasible since mostly the 'water lot' is a lot less expensive than a normal lot. The floating foundation does not have to be more expensive than a foundation for land based structures. There has been done some researches to the economic feasibility which gave a positive outcome, but it could be useful to map all the costs more in detail, so better based conclusions can be made.
- In this thesis is said when what floating system is best. Here for costs were taken in account but this was done without elaborating the costs. In earlier researches these costs are specified more in detail, but this can be done better.
- Also the costs for the connections have not been mapped. The connection is designed with as simple techniques as possible, and without the need of adjusting the system, thus it was concluded that the costs for this connection will not be too high, but before this design should be implement the entire building costs should be researched.



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