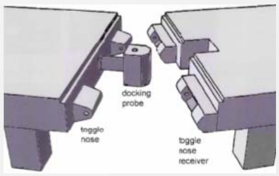
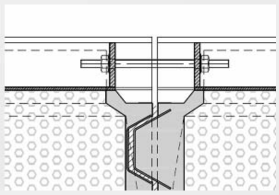
Master Thesis

Connecting Modular Floating Structures

Appendices

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Appendix 1a

History of living on water

A1.1 Ancient history

South-east Asia
An example of people who have been living on water for centuries are the fishermen in the Siem Reap province in Cambodia. The fishermen tribes have built complete floating villages at the edges, and also a few at the middle, of the lake of the combined river and lake 'Tonle Sap' (translation: Great Lake). Some of the villages move depending on the water level in the lake. These villages do not only move in height with changing water heights, as all floating villages, but they also change their location. The floating houses developed through the centuries. [*http://en.wikipedia.org/wiki/Kompong_Phluk nov 2009*]

Figure 1: Floating village on the Tonle Sap, Cambodia

In South-east Asia a lot of floating villages can be found. Not only in Cambodia, 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. These Chinese floating villages have already exist for over thousand years. The most famous floating village is Aberdeen floating village, located at the Aberdeen Harbour in the Southern District of Hong Kong.
The people living on boats in Aberdeen are mostly Tanka, a group of mainly fishermen which arrived in Hong Kong around the 7-9th century. The floating village increased enormously when the English built their main harbour and settlement, which they called Aberdeen, on the Island Hong Kong. At that time also a lot of Chinese traders settled in the floating village. The total population of boat dwellers in Hong Kong was estimated at 2,000 in 1841, peaked in 1963 with 150,000 and dropped to 40,000 in 1982. [*http://en.wikipedia.org/wiki/Aberdeen_floating_village, jan 2010]*


Figure 2: House boats Aberdeen, Hong Kong


Figure 3: Floating village Halong Bay, Vietnam

In Vietnam the floating villages exist out of small cots which are built on rafts. The rafts exist out of wooden planks, mostly supported by empty barrels and jerry cans as floaters.

South America
On the border of Peru and Bolivia is situated one of the highest navigable lakes of the world. This biggest lake by water volume of South America is inhabited by the pre-Incan people the Uros.

Figure 4: Lake Titicaca

The Uros lived and still live on big rafts made of totora reeds. The Uros originally created their islands, many centuries ago, to prevent attacks by their more aggressive neighbours, the Incas and the Collas The reeds at the bottoms of the islands rot away fairly quickly, so new reeds are added at the top constantly, about every three months. The islands last about thirty years. The rafts are anchored with ropes attached to sticks driven into the bottom of the lake.


Figure 5: Dwellings on rafts of totora reeds, Lake Titicaca, Peru

A1.2 The beginning of floating in Western Europe and The Netherlands

From the 17th century people started to live in boats and ships in European cities as Amsterdam. The publication Mooring Site Amsterdam [Kloos and De Korte, 2007] describes the history of living on water in the Netherlands, in particular in Amsterdam. Paintings and laws in the municipal archives show that already in the 17th century there were houseboats.
At the end of the nineteenth century the steel ship has it's entrance as cargo ship. The wooden ships can not concur with the steel ships and the steel ships displace the wooden ships, so a lot of wooden ships get out of use and get a new life as houseboat.

In 1918 de act for trailers and houseboats (Wet Woonwagens en Woonschepen, WWW) is introduced in the Netherlands. Then a lot of houseboats get a mooring permit.In 1922 the concrete ark, a hollow concrete foundation, made its introduction. In fact these arks where the first floating homes in Western Europe that weren't houseboats in the strict meaning of the word. Contrary to 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 doesn't need a lot of maintenance [Spruyt Arkenbouw, 2006], [Graaf, R. de].

The economic crisis around 1930 caused that a lot of people couldn't afford a normal house anymore, so they moved into the cheaper houseboats. After the second world war the amount of houseboats increased enormously because of the housing shortage after the second world war. On top of that, a lot of space became available. The old narrow harbours and canals got free, because they were not suitable for the modernising cargo ships. From that time on floating homes especially for living, and not for transportation, were built more and more.
At the end of the 20th century the number of houseboats in the Netherlands is estimated to a number of 10,000.

The messy image, the lack of regulations and policies, hasn't made the houseboat very popular with local governments in the recent decades. It has created a restrictive policy of the government. So the last decades hardly new moorings have been added. [Kloos and De Korte, 2007; Wilberink, & Munster, 2005]

A1.3 Last decades; floating houses

Last decades floating houses made it's 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 homes in North America

In the early 1980's the technology of floating houses had a very important development when 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)

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

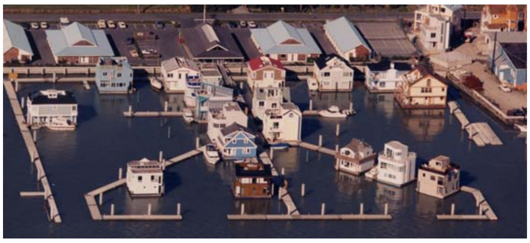


Figure 6: Floating quarter in construction (www.floatingstructures.com)

First floating houses in the Netherlands

Recreational floating houses
The first project with 'floating houses' in the Netherlands was constructed in 1992. In Marina Olderhuuske (next to Roermond, province of Limburg) 80 recreational villas were built in a gravel mining lake linked and next to the river Maas.

Figure 7: Floating recreational villa's in Marine Oolderhuuske (Richtlijn Levende Steden)

After a few years of quietness in the 'living on water branche', Marina Oolderhuuske got a follow up in Maasbommel. In Maasbommel (province of Gelderland) 14 floating houses for recreational use were realised. [Rutger de Graaf, 2009; JJ Fit, 2006; Stichting Reinwater, 2002]

Figure 8: Floating Houses in Maasbommel (Fit, J.J.) (www.drijvendestad.nl)

Floating houses for residential purpose
In 1999, Construction Group Ooms imported the Canadian system (referred to as IMF system at last page and in chapter 5) to the Dutch market. Ooms BV claims in an article in Elsevier [Van Osch, 1999] to have constructed the first real waterhouse [JJ Fit, 2006]. According to Ooms the villa's in Marina Oolderhuuske were not 'real houses' because they were meant for recreational use, and on top of that the villa's in Oolderhuuske only float when the water in the river Maas gets really high.

In June 1999, the three floors high water house from Ooms called ‘The Lighthouse’, was launched in the harbour of Hoorn in the presence of Secretary of State Remkes. At that time the contractor had plans to construct 200 of these big water houses every year. The company had contacts with the municipalities of, amongst others, Lelystad, Almere, Rotterdam and municipalities in Friesland and Groningen. According to Ooms they had ‘warm interest’ [Van Osch, 1999] [JJ Fit].

Figure 9: Dragging 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 made multiple contractors interested. Also the ark builders professionalize and multiple architecture firms start designing floating buildings. From now on the floating homes that are designed and built get an appearance from normal houses and get a more modern look. In Figure 9 and Figure 10 artist impressions from the leaflet from 2001 of Ooms are shown. These designs, made by Architectenbureau Sytze Visser, were examples for the successors of ‘The Lighthouse’.
To avoid the negative associations with the messy arks and house boats, from then on the contractors and municipalities start talking about ‘waterwoning’ (water house) or watervilla. (JJ Fit)

Figure 10: Artist Impressions of the Leaflet of Dura Vermeer, made by Architectenbureau Sytze Visser

From 2000 on, several floating houses are achieved for individual private clients. These homes are mostly located in aquatic areas around the Loosdrechtse- and Vinkeveense plassen, etc..

Floating Houses introduced in plans local governments
With the new type of floating houses, the image of living on water gets a lot better. 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 a lot less sceptical about floating buildings. With the water problematics (see paragraph 1.2) in mind, floating becomes hot with the rulers. Multiple municipalities make plans to integrate a large amount of floating houses in their new to build ‘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 they 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 11: Impression Waterbuurt IJburg (www.waterhuis.nl)

Figure 12: Overview IJburg (www.steigereiland.com) and pictures IJburg

The floating houses in IJburg where very popular and the demand was a lot bigger than the amount of houses available. In the past this has also been different, multiple projects had to suffer from a lack of demand. This had several reasons, these reasons are discussed in appendix 2 *Why Floating structures*.

Appendix 1b : History of non residence floating structures

Floating Bridges
Floating bridges have been constructed for a long time. According to old Chinese books the first floating bridge have been built in ancient China by the Zhou Dynasty in the 11th century. During the centuries, floating bridges, mostly small, have been built all over the world.

Floating bridges are usually temporary structures, some are used for long periods of time. Permanent floating bridges are useful for sheltered water-crossings where it is not considered economically feasible to suspend a bridge from anchored piers. Floating bridges were especially built by cultures who lacked the knowledge of building permanent bridges. Often small boats were used to construct the bridge on. The material mostly used is wood.

Figure 13: Pontoon bridge across the James River at Richmond, Virginia, 1865 [wiki]

Figure 13: The U.S. 9th Army crosses the Rhine on a temporary steel treadway pontoon bridge, 1945 [wiki]

Floating bridges were and are, especially popular by armies in wartime as crossings. Such bridges are usually temporary, and are mostly destroyed after crossing or collapsed and carried a long. [http://en.wikipedia.org/wiki/Pontoon_bridge]

Nowadays permanent floating bridges are still being built. Even highway bridges with a length of more than 2000 metres are constructed as floating bridges. Recent floating bridges and other structures are treated in appendix 2.

Offshore industry
Floating drilling platforms are in use since the 1970's. When the oil became more expensive it became feasible to win the oil out of deeper depths. [Barltop, 1999]

Other Structures
Structures with other functions than the few functions already mentioned, were hardly constructed as floating structures in the old days.
But temporary floating structures were also used in the old age. Floating caissons are for a long time for constructing bridges, piers, jetties and last decades also for tunnels. The caissons are transported while they are a float and when they arrive at the right location they are immersed. The first found example of this application was in 13 BC, at the port of Ceasara, Judea, where the mole was constructed using a wooden caisson. This method of building using caissons is still being used. [Voorendt et al., 2009]

A1-10

Appendix 2:

Reference projects

Appendix 2: Reference projects

In this paragraph examples of recent realized floating structures and some plans for floating structures are given.

Floating houses

Individual modern floating houses and dwellings can be found at multiple places in western Europe and North America.

Figure 14: Hamburg

Hamburg
B-type floating house from floatinghomes, 2007
www.floatinghomes.de

Figure 15: Seattle

Seattle
Classic Appearance Floating house, International Marine Floating Structures, 2000
www.floatingstructures.com

Figure 16: Blauwe Hart, Leeuwarden

Blauwe Hart, Leeuwarden
7 floating houses
Ooms BV, 2007
www.wonenopwater.nu

Until now only individual floating houses and a few coupled floating houses have been realized. According to van Winkelen (2007) it is possible to realise floating high-rise. There are no plans to realise high-rise yet, as far as research for this project turns out, but there are plans to construct floating apartment buildings of four storeys high. Ideas for a floating city are wide spread, see Developments/Plans for the future.

Floating utility buildings

Last years multiple floating utility buildings have been realized in the Netherlands. These utility buildings are build on floating bodies which are a lot larger than the floating bodies for houses. The floating bodies are single concrete caissons (fig. 25, 26, 27, 28) or made from a combination of EPS and concrete (fig. 24, 29)

Figure 17: Infocentre IJburg

Infocentre IJburg, Amsterdam
Dimensions: 19x37m^2
Completion: 2000
Companies: Ooms BV, Attika, FDN Engineering

Figure 18: Wijkcentrum Wessem

Wijkcentrum Wessem
Dimensions: ±1000m^2
Completion: 2006(?)
www.FDN-Engineering.nl

Figure 19: Detention centre, Zaandam

Detention centre Zaandam
Dimensions: 2x 100x22 m^2
Completion: 2007
Companies: Besix, Royal Haskoning

Figure 20: Firestation, Deventer

Figure 21: Floating Pool, Paris

Figure 22: Greenhouse, Naaldwijk

Fire Station, Deventer
Dimensions: 500 m^2
Completion: …
www.FDN-Engineering.nl

Piscine Josephine Baker, Paris
Roofed 25 meters pool
Dimensions: 90x20m^2
Completion: 2006
www.placesinfrance.com

Greenhouse Naaldwijk
Dimensions: 33,5x24,5m^2
Completion: 2005
Companies: Dura Vermeer, Flexbase. www.flexbase.eu

Nuclear power plants
Russia has plans to construct 10 floating nuclear power plants. These floating small nuclear Power plants produce 1/150 of the energy a standard Russian power plant produces. They will provide energy to regions with undeveloped infrastructure and can be transported to areas struck by natural disasters or other emergencies [Wang et al. 2007]. According to plans the first one will be completed in 2010. (http://www.bellona.org/english_import_area/international/russia/npps/27075)

Floating stadium
There have been made multiple designs for floating stadiums, but there are no concrete plans yet.

Floating bridges

Permanent floating bridges are most useful for sheltered water-crossings where it is not considered economically feasible to suspend a bridge from anchored piers. Floating bridges are even used for highways.

Figure 23: Highway bridge, US

Figure 24: Yumemai Bridge, Japan

Figure 25: Military Bridges

Lacey V. Murrow Memorial Bridge, Washington Lake, Seattle-Mercer
2018m long highway bridge
2nd longest floating bridge
Completed: 1993

Yumemai Bridge, Japan
Floating Pivot Bridge
Length: 410 m
On 2 megafloat pontoons
Completed: 2000

Military Floating Bridges
The military uses multiple floating bridges, most exist out of small steel pontoons. Modern example: Improved Ribbon Bridge

Floating Infrastructure

Floating infrastructure is almost inevitable for floating cities. In the Netherlands experience is gained with a floating road in Hedel. However, in fact the principle is the same as the long floating bridges like the Lacey V. Murrow Memorial Bridge.

Figure 26: Floating Road, Hedel

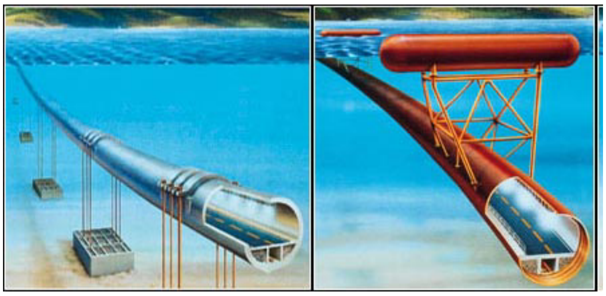


Figure 27: Impression HogSWjord Tunnel, Norway

Floating Road, Hedel, the Netherlands
Length: 70 m
Pontoons: Aluminium, filled with EPS
Dimensions pontoons: $5{,}25x3{,}5x1{,}0m^3$
Companies: Bayards Aliminium Construction, DHV, TNO, XX Architects
Completed: 2003

HogSWjord Tunnel, Norway
Length: 1400 m
25 metres below water; not in sight, potential cheaper than normal tunnel
Tube diameter: 9,5
Completed: Study NTNU
www.ntnu.no

Large Modular Structures

Figure 28: Mega-Float Airfield, Japan

Figure 29: Mega Platform, Singapore

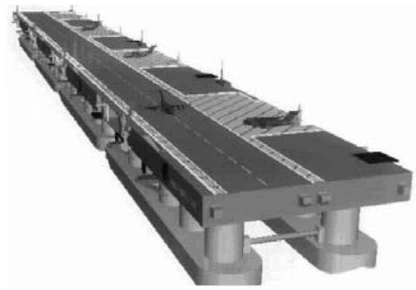


Figure 30: MOB

Megafloat Floating Airfield, Japan
Dimensions: 1000x 60 – 121 m^2
4 large steel pontoons
height pontoons: 3 m
Completed: 2000
www.srcj.or.jp

Mega Floating Platform, Singapore
Dimensions: 120 x 83 m^2
15 steel pontoons with bolted connections
Completed: 2006
[Wiki; Wang et al. 2008]

Mobile Offshore Base, US
Dimensions: min. 1500 x 150 m^2
+- 4 Semi-submersible pontoons
Completed: Study
[Den Vijver, 2003]

Large Floating Greenhouse
A combination of several parties, including province South-Holland, a contractor, an engineering agency and research office TNO, are doing research after the feasibility of a floating greenhouse of 50.000 m^2 for a large polder in the province South-Holland.

Floating Wave Attenuators

Floating breakwaters are able to absorb a lot of wave energy and become more economic than bottom based breakwaters in deep water. Floating breakwaters require less space. They have to be connected to the bottom, this is mostly done with cables.

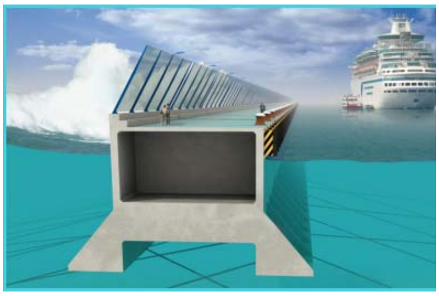


Figure 31: FDN Floating Breakwater Monaco

Figure 32: Floating Breakwater, IMF

Figure 33: Pier-Extension,

Floating Breakwater FDN
FDN-Engineering made two designs for floating Breakwaters. Their designs are made of concrete and one design makes also use of EPS. Their design is among others used in Bonaire.
www.FDN-Engineering.nl

Floating Breakwater IMF
International Marine Floating Structures has realized several floating breakwaters.
www.imf.com

Pier-Extension, Monaco
Enormous concrete breakwater with parking inside
Dimensions: $352x28x19m^3$
Completed: 2002
Concrete Technology, Vol.5, No.2, 2006, p.61

Offshore industry

In offshore engineering floating structures are used since the1970's. Most structures operate in very deep water with heavy wave conditions. Here for mainly semi-submersibles are used. Other floating platforms are the ship shaped platforms, tension leg platforms and spar platforms. The first semi-submersible was used in 1975 and since then a lot of semi-submersibles have been built. The semi-submersible crane vessel Heerema is also mentioned here because it is mainly used in offshore industry.

Figure 34: Petrobras P-51, Atlantic

Figure 35: Thialf Heerema

Figure 36: Brent Spar

The Brazilian Petrobras P-51
Semi-submersible oil platform
operating depth: 1700m
Constructed: 2006

Thialf, Heerema Marine Constructors
Largest Crane Vessel in the World
Dimensions: 201,6 x 88,4 m2
Depth to work deck: 49.5 m
capable of a tandem lift of 14,200t
hmc.heerema.com

Brent Spar, North sea
Oil Storage Buoy
Dimensions: 147m high, 29m diameter
Constructed: 1976
Worldwide publicity in 1995 when it got hijacked by Greenpeace, when Shell wanted to dump it in the sea.
[Rice&Owen, 1999]

Appendix 3:

Possible developments and future plans

Floating Cities

From the 1960's on multiple utopian ideas for floating cities arised (Wang C.M., 2008).
The futuristic looking Lilypad floating city concept is one of the most well developed ideas for a functioning sea community. Envisioned as a floating 'ecopolis' for climate change refugees, Vincent Callebaut's design resembles a water lily and would not only be able to produce its own energy through solar, wind, tidal and biomass technology but would also process CO_2 in the atmosphere and absorb it into its titanium dioxide skin. Each of these floating cities could hold as many as 50,000 people. http://blog.residesi.com/2009/05/12-fantastic-floating-cities-and.html

Figure 37: Lilypad floating city design

The idea of a floating city was also mentioned in the farewell speech of Frits Schoute in 2000. A floating city is, as the words says it; a floating city, a large builded area, floating on water with all different functions; living, working etc. In the farewell speech of Frits Schoute the landscape architect Jan van de Bospoort is quoted. According to him living on water becomes in the future a necessity, because of the climate change and lack of space. He sees a floating city arising in the Northsea in the near future. Several companies and students have worked out this idea of a floating city and think it is feasible. In 2006 students from Delft University won the Royal Haskoning Delta competition with an elaborated plan for a floating city in the IJmeer. Out of this prize winning studentgroup the company DeltaSync arised. DeltaSync has elaborated the plans for the floating city even more.

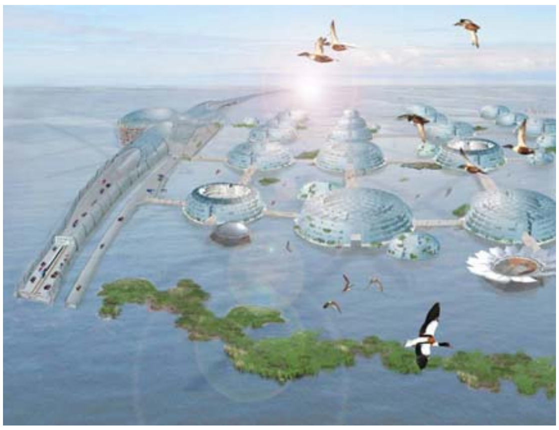


Figure 38: Artist impressions of floating city from DeltaSync

A floating city on high sea could be possible. The waves can be attenuated by floating wave attenuators and the city could be connected to the soil with cables.

Exploiting new forms of Energy

As the worldwide energy consumption increases and fossil fuels decrease, other forms of energy become more and more interesting and lucrative. Several durable energy forms can be gained at see, like solar, wind, algae, thermal, and wave energy.
Waves contain a lot of energy and this energy can be converted to electric energy. At the moment the technology of generating energy out of waves is still in his infancy, but the technology looks promising. The *Pelamis Wave Energy Plant* and *Point Absorbers* make use of floating devices. The availability of wave energy grows progressively as one harvests further offshore, so ocean colonization and developments in floating technologies play a key role in future developments in wave energy conversion. (Wang et al. 2008)

Floating Agriculture

Next to floating cities, floating agriculture could also be a possibility. Floating agriculture is an answer to the too big footprint of the western world. Floating grasslands can easy be realised with only relative thin layers of EPS and a small amount of a stronger material to give it its strength. An additional advantage of floating agriculture could be that the ‘floatlands’ can be moved during the season, so the floating agriculture can always have a ideal climate.

Appendix 4:
Advanced Plans for Floating Quarters

A4.1 Floating Plans Naaldwijk

In the Plaspoelpolder, next to Naaldwijk, the largest project of the Netherlands with floating buildings will be realised.
The Plaspoelpolder, measuring approximately 2,5km by 500m (70ha), is now still a mainly empty piece of grassland, but from 2010 on this will be converted in to the quarter 'New Water'.
The artificially maintained water level of this former polder, will be raised to bosom-level, creating a site that will not only act as a regional water storage area, but will also hostsa multitude of water-related developments including 1200 dwellings, from which 600 floating.
The zoning plan for this project is already approved. The new quarter will be constructed between 2010 and 2017.
The first project of New Water will be the apartment complex Citadel. [www.waterstudio.nl, de Volkskrant 9 sept. 2009]

Figure 39: Zoning Plan 'New Water', www.waterstudio.nl

Figure 40: Apartment complex 'Citadel'

Figure 41: Maquette 'New Water'

A4.2 Floating Plans Rotterdam

To Rotterdam the option of floating buildings is very interesting. Many port activities have moved already moved to the Maasvlakte or will move to the second Maasvlakte when this will be finished. With the shift of port activities towards Maasvlakte, 1600 hectares of former port sites will be available which can be developed for new destinations. Under current planning, these ‘city harbours’ will be developed with an emphasis on sustainable, innovative and modern working and living environments. Water plays an important role in the plans. In the Maashaven a real 'Floating City' is planned, with floating residential houses which generate energy from the tides. This ‘Floating City’ will be stepwise created up to the year 2030. [Gebiedsplan Stadshavens]

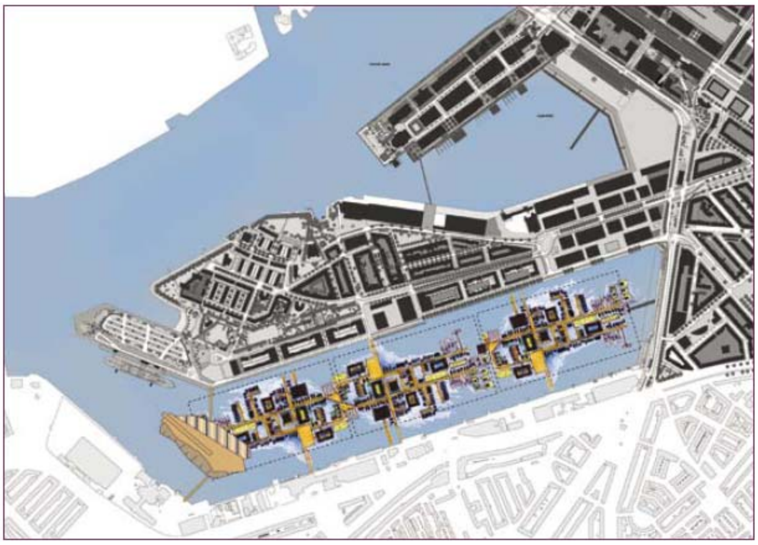


Figure 42: Exploratory Floating City Maashaven, From Gebiedsplan Stadshavens

The start with floating buildings is made with construction of a floating pavilion in the Rijnhaven. This is a pilot project for floating buildings in Rotterdam, this project is completed in june 2010. This is the pavilion which is used for the case study. Another plan that will be realized in the near future is the project ‘Werkeilanden’, which consists of multiple offices on two large floating bodies. This project will be realized in the Heijsehaven in 2011.

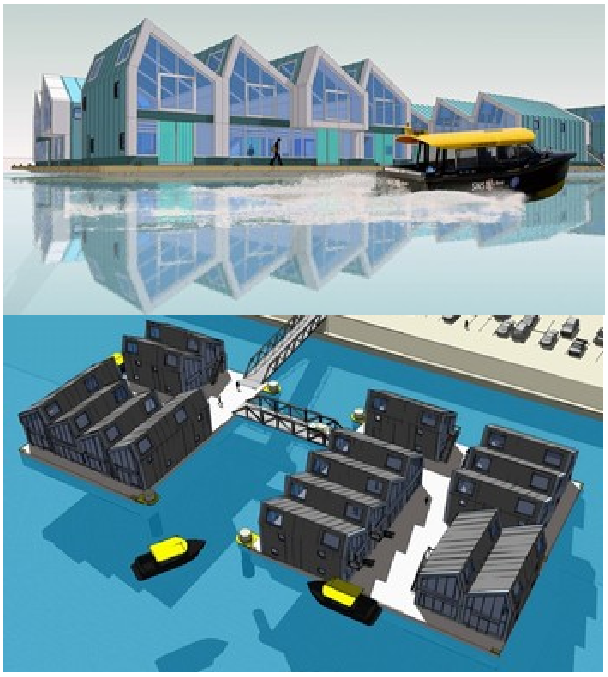


Figure 43: Impressions project Werkeilanden, Public Domain Architecten

Appendix 5: Why floating structures

A5-2

Appendix 5a: Personal wishes

In appendix 1, a few reasons were mentioned why people liked or wanted to live on water.
The fisherman in South-East Asia wanted to live close to where they fished. On top of that, this way they could keep an eye on their boats. The Uros on the lake Titicaca lived on the water because of their safety. Till half way the twentieth century, people who lived in houseboats in western Europe did this mostly out of economic motives.
Nowadays, the main reason why people want to live on water, it is because of other personal reasons.

Today, the main reason why people want to live next or on water, is because they like it. A floating house is attractive because of the space and view on the surroundings, the living environment near the water and the feeling of freedom it gives. These arguments are given in answers on an questionnaire with 112 respondents from Fit in 2006. Researches from Heijmans (2006) and SEV (2008) give these also as most the important reasons. According to Heijmans (2006) there is a market for 360,000 floating houses.

Figure 44: View from a floating home in Maasbommel [Wilberink&Munster, 2005]

Living on water therefore has the qualities of an attractive living environment, which has a big demand. According to Vos (2002) this remains in tact even when it is presented in large scale within a larger urban structure. [Vos, 2002]. A survey of Bervaes and Vreke (2004) shows that houses next to water are 8 to 16% more expensive compared to ordinary houses. According to the report 'Richtlijn levende Steden' the prices of houses with view over water can be 40% higher than comparable homes on land.
According to the researches, another reason why people like the floating houses, is because the house can be designed and constructed according to their own wishes.
A vast majority of respondents prefer the new water housing, such as built in Maasbommel, above the conventional ark. [Wilberink, 2005]

Reasons not to live on water
As sketched, there is a lot of demand to living on water. Nevertheless, in the past some projects also failed, for instance due to long walking distances to parking space and the lack of gardens. (Schuwer, 2007; Fit,2005). Other reasons had to do with the unclear legislation around the floating homes (see paragraph 1.3) and with fear for tilt or other technical issues.

Appendix 5b: Water problematics

The climate is changing, world wide it's getting warmer. 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. Sea levels are rising, while the soil is setting and maximum river discharges are getting higher. This results in large problems with water. To adapt to this water problematics, a large amount of water storage is needed in the Netherlands. According to the experts an area of the size of the province of Utrecht. [Akker, 2004]
In this appendix the situation for the Netherlands is discussed, but there are also places in the world where they suffer similar problems.

Climate Change

Climate Change is a hot topic all over the world in the recent years. It's a frequent occurring topic in newspapers and television, and there are held many conferences on the topic and many reports on the topic have been published. According to the IPCC, climate change will result in a hotter climate with the associated floodings or water shortages.

Climate Change Worldwide according to IPCC
According to the IPCC Fourth Assessment Report from 2007 the global surface temperature increased with 0.74 ± 0.18 °C between the start and the end of the 20th Climate model projections summarized in the latest IPCC report indicate that the global surface temperature is likely to rise a further 1.1 to 6.4 °C during the 21st century. For this predictions the IPCC uses multiple climate scenario's. These different climate scenario's differ in the amount of greenhouse gasses that are exhausted and the influence of these gasses. According to the IPCC these greenhouse gasses are the main reason for the global warming.

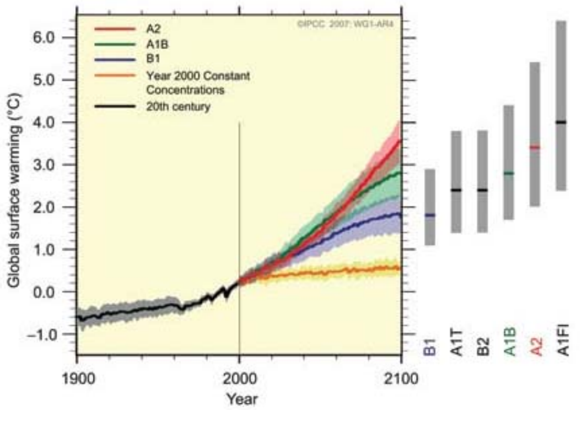


Figure 45: Climate scenario's according to IPCC 2007

IPCC
In 1988 two organizations of the United Nations established the Intergovernmental Panel on Climate Change (IPCC). The IPCC was established to provide the world with a clear scientific view on the current state of climate change and its potential environmental and socio-economic consequences [www.IPCC.com]. This scientific intergovernmental body does not carry out its own original research, nor does it do the work of monitoring climate or related phenomena itself., but it reviews and assesses scientific, technical and socio-economic information of climate change. A main activity of the IPCC is publishing reports to provide balanced scientific information to governments and decision makers. [IPCC, Wikipedia]
In the Netherlands the decision making bodies like VROM use the global climate scenarios of IPCC reports and regional climate scenario's of KNMI (which are again based on the IPCC reports) as base for their policy, together with the reports of multiples committees (like the Delta Committee and Commissie Waterbeheer), which are again based on figures of the IPCC and KNMI. The last official report of the IPCC is the Fourth Assessment Report from 2007.

Climate Change in the Netherlands
The Royal Dutch Meteorological Institute (KNMI) has made regional scenarios for the Netherlands based on the global scenarios of the IPCC. In 2006 the KNMI presented four different climate scenario's. [KNMI, 2006]

Four Different Climate Scenario's
Two main driving forces were selected to construct the scenarios, see Figure 46. The first driving force is global temperature change. For the low (G) scenarios, global temperature increase is +1 °C in 2050 and +2 °C in 2100. For the high (W) scenarios, global temperature increase is +2 °C in 2050 and +4 °C in 2100. Both temperature increases are within the range given by the IPCC.

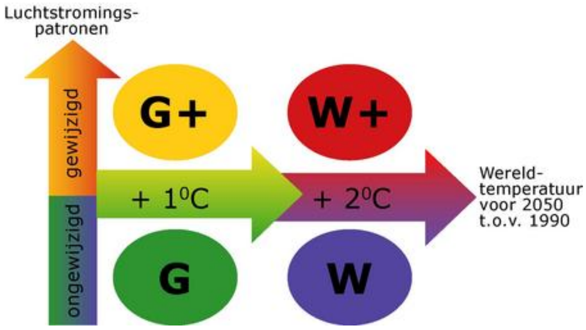


Figure 46: Four different climate scenarios

The second driving force is the change of the mean seasonal regional atmospheric circulation. This are the + scenarios. This seasonal regional atmospheric circulation determines the Dutch regional climate to a large extent. Predominantly western atmospheric circulation, very similar to the current situation, results in a relatively mild and temperate climate whereas a change towards more eastern circulation conditions would change the regional climate into a more continental climate with dry and hot summers. In this scenario summer droughts will occur more frequently, with less summer showers, but if they occur they will be more heavy.
In all scenario's there will be more winter precipitation.

The four scenarios were designed to span a large part of possible futures in order to deal with the uncertainty of future changes. Based on current insights that are derived from current climate models, the probability that the future climate will be within the range of the four scenarios is estimated at 80% (KNMI, 2006). Therefore, there is not a 'most likely' scenario and all four scenarios should be used to develop water management options.

KNMI Climate Conclusions
From the KNMI report the following can be concluded:
- 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.

Criticism on IPCC report
The IPCC exists out of a lot of renown scientists, but there is also a lot of criticism on the IPCC report and on it's conclusions. This criticism comes also from other scientists specialized in this field. The criticism is however mainly directed at the conclusion of the IPCC that the cause of the climate change is because of the greenhouse effect. The fact that the climate changes and worldwide it is getting warmer, is not or barely challenged by renowned scientists. [Multiple sources on the internet, newspapers, documentaries etc.]
At present, little is known about the specific effects of climate change. However, the size of changes and precise effects may be uncertain, the direction of change is not.

Change in river discharges
The climate change in the rest of Europe will affect the discharge levels of our big rivers. According to the KNMI the supply of especially the Rhine will become smaller in summer and bigger in winter. In a dry scenario the Rhine discharge in summer can decrease with up to 60%, while the winter discharge can increase with 30%.

Sea level rise
Global warming causes an worldwide sea level rise, by among others by thermal expansion and melting of land ice. The following predictions are made:

Sea level rise estimates for 2100

- IPCC WG1 Fourth Assessment Report, 2007 (Global):	18-59 cm
- Royal Dutch Meteorological Institute, KNMI, 2006 (North Atlantic):	35-85 cm
- "Al Gore" scenario:	50-150 cm
- GIEC (Intergovernmental Group on the Evolution of the Climate):	20-90 cm

Within the models of the IPCC and KNMI the uncertainties in changes in ice sheet flow are not taken fully in account. Greenland and Antarctica don't contribute to the sea level rise in these estimates. This is one of the reasons, as the IPCC says it in it's own report (AR4 Synthesis), the upper values of the ranges are not to be considered upper bounds for sea level rise [IPCC, 2007].
Here for the Delta Committee 2008 ordered further research to predict upper values for the sea level rise. In this research the ice outflow of Antarctica and Greenland is taken into account. The Delta Comittee uses the following upper level estimates:

Upper level estimate Dutch Coast, Delta Committee 2008
- 2100: 0,55 to 1,20 m
- 2200: 2,0 to 4,0 m

Consequences of sea level rise
See level rise causes big risks of flooding for many coastal area's. Many people are living in low lying delta areas. In 2030 it is predicted that 50% of the world population lives within 100 km of the cost. [IPCC, 2007]
According to the Delta Committee the risk of floodings in the Netherlands with the rising sea level can be kept sufficient small by increasing the threshold capacity by amongst others strengthening and raising the dikes, but against increasing costs. But the sea level rise gives also other problems.

Drainage will be harder
With a see level rise the draining of water towards the sea will be harder, so in high water periods this will also lead to a water level rise in rivers and other water outlets.

Salination
The sea level rise causes seepage of brackish groundwater into the Dutch polders under sea level. This will increase the demand for fresh water to flush the polder water systems.

Land Subsidence
In the North and West of The Netherlands the soil settles. In the west this is because of setting of peat because of oxidation of the peat. This subsidence has been going on for centuries, but it has increased disproportionate in recent decades by reducing the level of the ground water. In the peat lands the decrease may be up to a maximum of 1 meter per century.
In the North the soil settles by gaining natural gas. This settlement goes up to several decimetres per century.
This soil settlement will cause that the negative consequences by seawater level rise become even bigger.

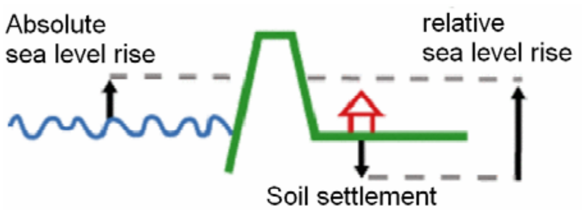


Figure 47: combination of sea level rise and soil settlement [KNMI, 2006]

Urbanisation
Traditional urbanisation also leads to load on the water system. An urbanised area has a large water demand, but usually it has only little own storage. And when the urbanised area is struck by a severe shower it can't coop with the water because most soil is paved and impenetrable by water why the soil has no storage function. Most of the times there is also to less surface water for water storage.
Last years it has occurred multiple times in the Netherlands that urban areas where flooded by a severe shower. Examples from last years are the city of Utrecht and Westland.

Resulting droughts and flooding risks
As made clear in previous paragraphs, climate change, sea level rise, land subsidence and urbanisation together results in, to put it simple, to 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 result in a water shortages.
- The high water levels and severe rains results in a surplus of water and results in flooding risks.

Solution: More space for water
An answer to the mentioned water problems is water storage. This way, when there is a surplus of water it can be stored in the wet periods and 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, R. de]
The Commissie waterbeheer 21st eeuw (Committee Watermanagement 21st Century) (Tielrooij, 2000) concluded that it was not longer possible to suffice with technical measuremenst as hightening the dikes and enlarging capacity of pumping stations. For a durable water policy more room for water is necessary.

The advised measurements from the Commissie waterbeheer 21st eeuw are the following:

- No further loss of space for water by introducing the water test
- Water storage in the form of retention areas
- Room for the rivers
- Appointing emergency overflow area's; 'calamiteitenpolders'

<u>Policy: Room for Water is needed</u>

- According to the Commissie Waterbeheer 21st Eeuw (Tielrooij, 2000) there would be an extra 60.000 acres of water surface needed to retain and store water till 2015, for 2050 110 thousand acres is needed.
- According to the 5e Nota Ruimtelijke Ordening (5th National Planning Note) from 2001 the extra demand for watersurface in 2030 is ha 490.000 (approximately four times the surface of the IJsselmeer) . This is divided over storage polders, inundation polders and other spatial measurements.
- In the new Nota Ruimte from 2006, (the successor of the 5th National Planning Note) in Chapter 3.2.3 the following is said: Water is the structuring principal for spatial planning. To avoid water nuisance, and to reduce desiccation (verdroging) the space will designed and used in a way, that water will be detained better. If necessary measurements will be taken to store water. Here to, the water surface will be enlarged, whether or not temporary, or the water levels can be heightened. Only in the last resort water will be drained away. This priority order (hold - store - drain) is called the "three step strategy Water '.
- The Delta Committee 2008 is mainly focused on improving threshold capacity. But in two of it's recommendations is stated that more space is needed for water; alongside the rivers and for local water storage in deep polders.

Appendix 5c: Shortage of land

Shortage in the Netherlands
In the Netherlands there is a big shortage of land. The area claims for living, working and recreation increase and increase. So this brings a conflict with the need of more space for water. When all land claims are placed beside each other, the Netherlands appear too small, see Table 1.

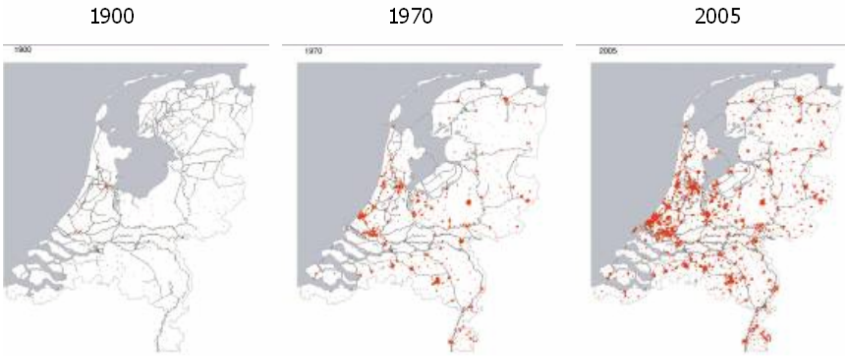


Figure 48: Urbanisation of the Netherlands in 20th century

Function	Space demand in 2030 (acres)
Living	286.231
Working	138.862
Infrastructure	211.548
Recreation	226.705
Water	1.255.269
Nature	794.427
Agriculture	2.028.307 +
Total	**4.941.349**
Surface The Netherlands	**4.152.800 -**
Shortage	**788.549**

Table 1: Space demand in 2030 according to CBS Bodemstatistiek in 1996

The shortage is largest in more heavily urbanized area's. The principle of multiple space usage could be the solution.

Shortage over the world
Also in other parts of the world there is a shortage of available building soil. This is especially the case in big cities at the coast. The urban population is expected to double from 2 billion to 4 billion in the next 30 to 35 years. (UNFPA, 2007). Urbanisation takes predominantly place in coastal and river areas.
Best examples of big cities with a shortage of available soil are Singapore and Hong Kong.

Appendix 5d: Multiple space usage

With multiple space usage land surface is used by different functions at the same time. One form of multiple space usage is combining the water function with other functions.

Different types
There are several options of combining water storage with for example residence buildings. There are types that can be combined with (semi)permanent surface water and there are options which can't. The options which can be combined with surface water result in a lot bigger water storage function. For the types that can't be combined with surface water there are a lot possibilities, here they are classified in two classes; 'damage control types' and 'rainbarrel types'.
The types that can be combined with open surface water are buildings on piles and floating buildings.

'Damage control types'
Several options, for example 'wetproof'- and 'dryproof' buildings, can be consisdered as the 'Damage control types'. These types can be built in an area where a flooding risk exists. With an eventual flooding these houses won't be damaged to much.
These types won't by severely damaged but they don't function during a flooding, so they are not usable when a flooding is often expected.

'Rain barrel types'
There are also several options that can be considered as the 'rain-barrel-type'. These houses have some water storage in or below them. Of course the soil below buildings but also below roads can be used for water storage. Water storage in the soil is also called infiltration type.

On Piles
The buildings are placed on piles high above the ground. This way there is room for water storage underneath the houses.

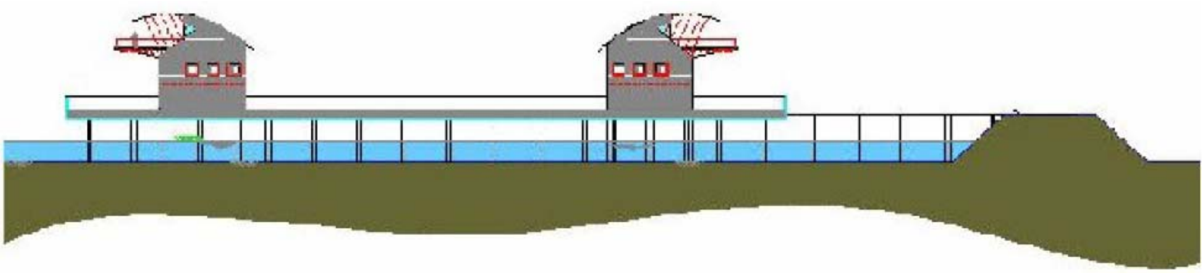


Figure 49: Houses on piles

Floating Structures
Floating structures can be realised on an open water surface. The structures can move along with the water level. This way any water height is possible, so the buildings are not at the expense of water storage.
The floating buildings can be permanent in water, but when the floating body and soil underneath is made suitable, the floating building can also rest on the ground when the water is low or when there is no water at all. This form is called amfibic and is used in Den Ham (see appendix references), there a concrete floating body is used which can rest on a concrete slab.

Most suitable form
When a large storage capacity is needed, or when an option is needed where the buildings can be in a water surface for a longer time, or the water should be able to flow freely around the buildings, the floating buildings and buildings on piles are best suitable. With these forms, as well the frequency of flooding as well as the water level variation can be high. Floating is best suitable when there is always a relatively high water level. With floating structures the variation of the water level can be unlimited high.
When the ground is also getting dry at some times the buildings on piles might be advantageous, because in that case the floating buildings need a double foundation; the floating body itself and a

foundation on the soil. Remaining soil can with this amfibic floating form also cause a problem concerning crookedness.

Appendix 5e: Additional reasons for floating

Floating better than land reclamation

Another solution for shortage of building land is creating new land in the sea.
According to Wang et al. (2008) in some cases floating structures may offer significant advantages over the traditional land reclamation technique for offshore colonization:

- **Possible at large depths**: Floating structures are little effected by water depth or the nature of the seabed. Land reclamation becomes uneconomical at depths bigger than 20 meters and scenarios with soft seabed.
- **No Foundation**: For floating structures there is no foundation for vertical forces needed.
- **Seismic shocks:** Floating structures are not or less sensitive to seismic shocks.
- **Shorter construction time:** Land reclamation takes several years because of consolidation, it took less than 4 months to construct the 1 km long Mega-Float, a floating test runway in the bay of Tokyo.
- **Advantageous for harbors:** Floating extra advantageous for harbors, because next to the land deep water is needed. With land reclamation using a sheet piling construction this is expensive.
- **Environmentally friendly:** Floating structures have less impact on environment, have negligible effect on tidal currents etc.

Mobility

A big advantage of floating structures is that they can be transported over water, so their location can be changed. When they are not of use anymore at a certain location, or of more use at another spot, they can be transported to this other spot, provided there are no major obstacles in the waterway. Buildings for which this can be really beneficial are for example; big stades for big tournaments as a world cup, prisons or exhibition buildings.

Offshore Industry

The floating structures in offshore industry are mostly very big structures, which are constructed for winning natural resources from the sea bottom. Twenty percent of the oil and gas production takes place in offshore areas. [Moan, 2003] At large depths floating structures become more economic than firm structures.

Exploiting new forms of Energy

As the worldwide energy consumption increases and fossil fuels decrease, other forms of energy become more and more interesting and lucrative. Several durable energy forms can be gained at see, like solar, wind, algae, thermal, and wave energy.

Appendix 6:

Verification and Legislation

A6.1 Verification structure

In paragraph 1.4 is said that when a floating structure is judged as real estate, it has to fulfil the 'Bouwbesluit' (Dutch building codes). But as said in paragraph 1.4 there are also subjects or parts where the building code can't by applied on floating structures, or for some subjects extra regulation is necessary. The regulation which is needed to (structurally) check the floating structure is treated in this paragraph. In this paragraph is also extensively discussed what is treated in guideline from VROM, 2009.

Again the remark is made that the existing building codes should be used where ever they can be used. In the cases they can not be used 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 guideline distinguishes between two types of floating structures for verification.

Two different types
The guideline from VROM distinguishes between two types of floating structures:
1. Floating body with one or multiple structures on it
2. Floating structure where the floating body is part of the building.

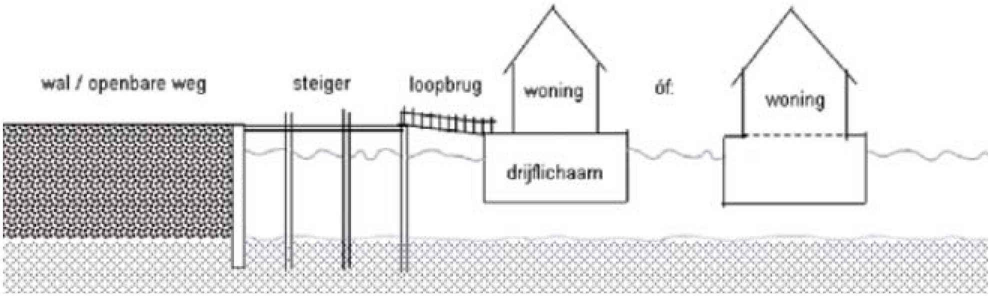
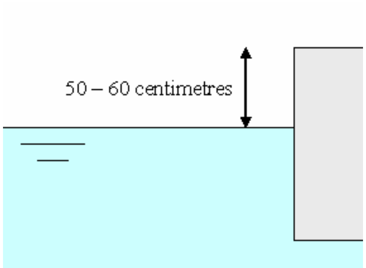


Figure 50: Two types of floating structures, according to guideline from VROM

According to the guideline, in case of floating houses, the floating body of the first mentioned option should not be characterized as a residence building, while in the second it should. This distinction is important for implementing the rules of the building codes.

Buoyancy
The building codes give rules for failure and collapse of the structure, but according to the guideline the rules of the building codes are not applicable on a floating structure which could sink without failure of the structure. According to the guideline in practice a guideline for freeboard is used to ensure the buoyancy.

<u>Minimum Freeboard</u>
To ensure the buoyancy, under maximal loads during normal usage, the minimal height difference between the lowest aquifer opening and the water surface level should be around 50 or 60 centimetres.
This height from 50 or 60 centimetres is to prevent wave overtopping. Wave overtopping could lead to water in the structure which leads to a higher load and this way increasing subsidence. When there is no risk of accumulating water in the structure by waves this height can be lower. This height between lowest aquifer opening and water surface level is in most cases equal as the freeboard (distance between water surface level and deck), at least when there are no windows underneath the deck.



In the guideline is not made clear of this freeboard height should be calculated with or without load factors. In this thesis is made the assumption that this height of 50 or 60 centimetres is a demand for the usability state, so it can be calculated with load factors of 1,0.
For calculating subsidence of the structure, what is seen as collapse, the ULS load factors have to be used. For this check on subsidence, the next mentioned demand is used.

Absolut minimum freeboard and demand residual buoyancy
In practice is made use of a demand for residual buoyancy. According to this demand the freeboard should be in any situation 10 centimetres. So also when the loads will be extreme during calamities, for example by extinguishing water and also with leakage of one or two compartments. The guideline confirms this demand.

Tilt and swell
Also tilt can be a risk for the buoyancy of a floating structure. Due to tilt a aquifer opening can sag under the water level, and this can cause water to enter, causing risks for the buoyancy. So such an amount of tilt may never occur.

According to the guideline from VROM the 'Bouwbesluit' (Dutch Building Code) of 2003 doesn't give any demands for the service limit state. So this means that according to the 'Bouwbesluit' any tilt and any swell (deining) may occur, as long it doesn't treaten the structure. So the guideline says that the demands for serviceability, so also for tilt and swell, has to be regulated by private law.

So according to the guideline, the Dutch building codes doesn't give any demands for the serviceability state, but in fact in the Dutch Code for foundations (NEN6740) there ís given a demand for rotation! In this code is stated that the rotation for residence buildings is not allowed to exceed 1:300. Here by NEN6740 refers to the figure depicted below. ω may exceed 1:300.

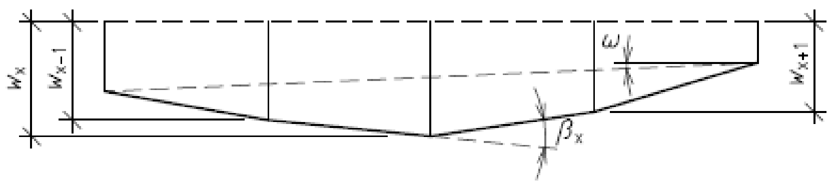
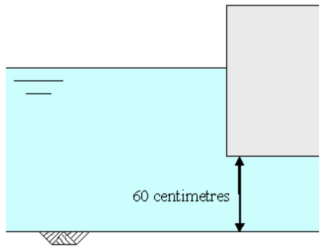


Figure 51: Demands on a foundation, according to NEN6740

Floating buildings can be constructed in a way that they are almost level in there 'initial-state', the state where no external loads are acting on them. But when a load will act on the structure, a rotation will very easily occur. In practice it is undoable to stay below a rotation of 1:300. Therefore in practice an maximum inclination of 5° or 50% from the freeboard is used. A smaller tilt can be demanded by the client. (VROM, 2009).
With placing the sewers and pipes the possible tilt has also to be taken in account.

Depth underneath structure
The guideline from VROM describes a depth from 0,6 meters underneath the floating structure. This to prevent the floating structure touching the soil, also when it is tilted. This depth is also needed to secure a certain flow capacity. The flow of water affects the water quality, particularly the amount of sediment and pollutants that is imported and exported. In addition flow is also needed to avoid oxygen shortage in the water around the floating buildings. (see Appendix 6.3 Water quality)



A6.2 Remaining legislation and recommendations

Low maintenance preferred
For maintenance in or above water surfaces there is severe legislation. Here for low maintenance of floating structures and low maintenance materials are preferred for floating structures. (VROM, 2009)

Fire safety
For the fire safety legislation the normal building code rules can be used, the guideline gives some advise how to interpret these rules for floating structures and gives a few additional rules.

Connections electro etc.
In the building codes electro is regulated up to the meter boxes on the firm landing stage, behind the meter box not. For regulations for the electro connection can be taken a look at NEN 2768, but this is not yet applicable for floating structures.

Collision
The building code doesn't demand that any buildings are calculated on collision with ships. This does count for quay or dike houses, and now also for floating structures. The watermanager (Rijkswaterstaat, Waterschap e.a.), have 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. Rijkswaterstaat uses the ROBK (Richtlijnen voor het ontwerpen van betonnen kunstwerken = Guidelines for the design of concrete works) for the design of their fenders.

'Waterwet'
In national waterways the floating structures have to fulfil all other additional legislation that accounts for these waters. All regulation for waterways and municipal waters can be found in the in December 2009 introduced Waterwet (Water Act).

Ice
The guideline mentions the building codes doesn't mention ice as a load, but that it can be a problem for floating structures, so the water should be hold free of ice or the structure should be designed in a way that it has resistance against ice load.

A6.3 Water quality and building density

The most important aspect for the water quality is the amount of oxygen in the water. Floating structures reduce the surface of the water, thereby reducing the entrance of daylight, and the production of oxygen from the air through emission. Lack of light means that plants do not survive, plants make oxygen using light. A shortage of oxygen in water causes that fish will avoid these areas or die. A lack of oxygen in water also causes a rotting smell.
Further it is important to avoid a large amount of algae, especially the arise of cyan bacteria (blue algae) should be avoided. Blue algae grow best at temperatures between 20 and 30°C, in low light conditions and mineral rich water.
Measures to maintain water quality are aimed at the causes of poor quality. But in the building codes and in the guideline here is not said anything about water quality. In Canada there do exist guidelines on this topic. The Canadian guidelines says that no more than 40% of the water should be cultivated. Another way to realise good water quality is making sure there is enough flow. Next to ensuring there is enough space underneath the structure this can also be done with fountains and such.

Appendix 7:

Buoyancy and Hydrostatic Pressure

A7-2

A7.1 Buoyancy and Archimedes

An object thanks it's buoyancy on the hydrostatic water pressure. The principle of buoyancy can be described with Archimedes' principle.

Archimedes principle:

Any object, wholly or partially immersed in a fluid, is buoyed up by a force equal to the weight of the fluid displaced by the object.

This upwards directed force is called the Archimedes force. As said this force is equal to the weight of the displaced water. This weight is equal to the density of water ρ (kg/m^3), times the gravitational acceleration g (m/s^2), times the volume of the displaced water V (m^3). In formula this gives:

$$F_A = \rho \cdot g \cdot V$$

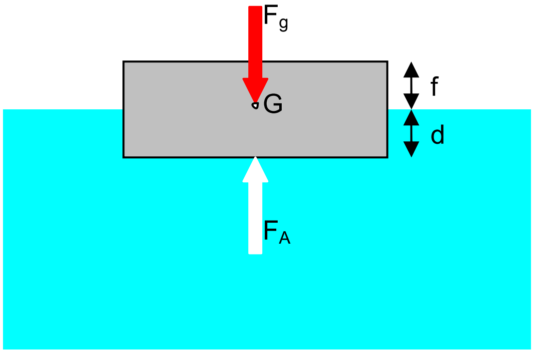


Figure 52: Principle of buoyancy

In case of a in vertical direction free floating structure in equilibrium situation, the weight of the floating structure is equal with the Archimedes force.

$$F_A = F_g$$

F_A = Archimedes force
F_g = gravity force

f = freeboard
d = draught
G = centre of gravity

Parameters ρg

Density water
The density of pure water is largest at a temperature of approximately four degrees Celsius. At this temperature the density is approximately 999,8 kg/m^3. Above and below this temperature the density gets lower.
Seawater has a larger density then fresh water. The water is more heavy with a higher salinity.

The density of water is given by the following formula:

$$\rho_w = 1000 + 0{,}805 * S - 0{,}0166 * (T - 4 + 0{,}212 * S)^{1{,}69} \text{ (kg/m}^3\text{)}$$

where: S = the salinity (kilograms/m^3)
T = the water temperature in C°

Example density Northsea: The water temperature of the Northsea is ca. 5 degrees Celsius in January and ca. 20 degrees in August. Than the density varies normally between 1016 kg/m^3 and 1024 kg/m^3.

Gravitational acceleration
The gravitational acceleration varies over the world, depending on the latitude, between 9,78m/s^2 and 9,83m/s^2. Below is said ρg will be taken 10 kN/m^3, when this simplification is used for the gravitational acceleration will be taken 10 m/s^2.

ρg = 10 kN/m^3
In most situations ρg can be seen, for simplicity reasons, as one parameter, the specific gravity γ_w, with a value of 10 kN/m^3. In this thesis this will also be done, because this way the calculations will still be exact enough and they do not depend on the location in the world and the temperature any more. Next to this the hand calculations will be more clear with calculating with this rounded of parameter.

Simplified formula
This simplification gives the simplified formula for the Archimedes force:

$$F_A(kN) = 10 \cdot V$$

A7.2 Hydrostatic pressure

Archimedes principle makes use of the principle of hydrostatic pressure.
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 hydrostatic pressure is linear with the depth of the water and depends on the density and gravitational acceleration.

$$p = \rho g z$$

With: p = waterpressure in Pa (N/m2)
ρ = density of the water (kg/m3)
g = gravitational acceleration (m/s2)
z = depth in meters

The schematization of a floating structure in the figure below will be used in this chapter to explain several definitions and principles of floating.
In this figure point G is the centre point of gravity of the combined floating body and structure.
Point B is called the center of buoyancy, this is the centroid of the displaced water volume. And the spot where the line of action of the buoyant force intersects.

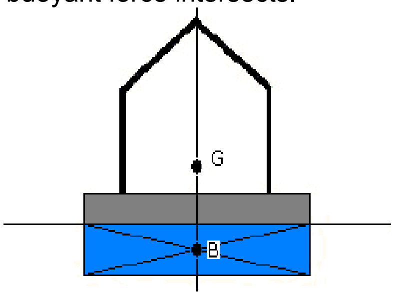


Figure 53: Floating structure with centre of buoyancy and centre of gravity[Kuiper, 2006]

The waterpressure results in forces on immersed surfaces. The force on a surface will be found by integrating the pressure over the surface.
In Figure 54 the hydrostatic waterpressure with resulting forces are drawn on the standard floating structure:

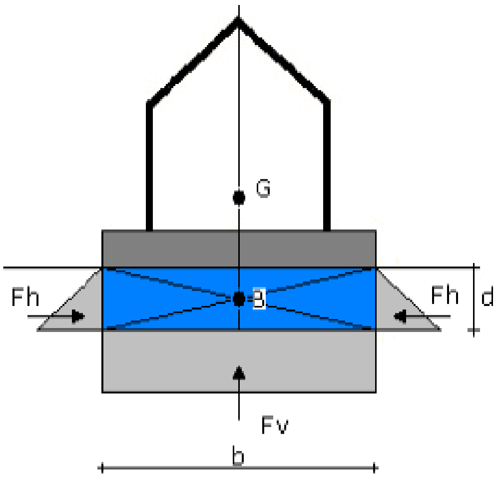


Figure 54: Hydrostatic pressure [Kuiper, 2006]

$$p_{\max} = \rho g d$$

Then for the horizontal and vertical forces this results in:

$$F_H = \rho g d \cdot \frac{1}{2} d \cdot l$$

$$F_v = \rho g d \cdot b \cdot l$$

With l = the length of the construction

The vertical force with this formulas calculated, is of course equal to the buoyant force according to Archimedes. Also with a certain rotation.
The net horizontal forces of the pressure figure, have got their line of action through the centroid of the figure, so in this case on 2/3 of the draught. The hydrostatic waterpressure will lead to stresses in the floating body, but they will not influence the forces on the moorings. With equal water height on both sides, the horizontal forces will cancel each other out. This also holds with a certain rotation.
With the occurrence of waves, water heights differ, so than then above mentioned doesn't hold anymore and forces on the mooring will be introduced.

A7.3 Schematising water as elastic foundation

A floating structure supported by water can be seen as a structure on a base plate on a elastic foundation with water as continuous elastic foundation.
The standard formula for stresses on an elastic support, as result of a deflection, is the following;

$$\sigma = k \cdot u$$

With: σ = stress in (N/m^2)
k = subgrade modulus (N/m^3)
u = deflection

For foundation on water applies: the stress σ is equal to the hydrostatic water pressure, the deflection u is equal to the depth (z). The subgrade modulus k is equal to ρg, which is 10.000 N/m^3.

k = ρg = 10.000 N/m^3 (10kN/m^3)

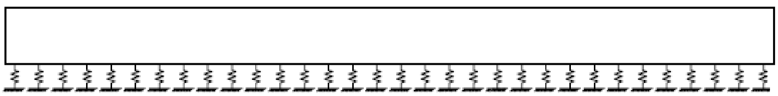


Figure 55: Schematisation structure on elastic foundation

A k-value of 10 kN/m^3 is very low compared to, for example the k-values of sand, which are normally more than 1000 times higher. This means a structure on water will heavily subside in the water and it is strongly susceptible to tilting and vertical movement.

Appendix 8:

Tilt and stability

A8-2

A8.1 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 contra action moment will arise. This contra action moment can bring the structure in equilibrium again.

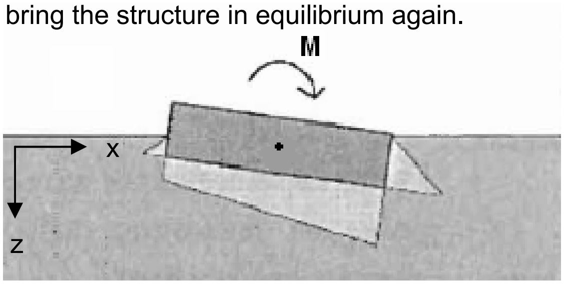


Figure 56: Rotated floating structure

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

In formula this gives:

$$M_S = M_{righting}$$

With: M_S = Moment due to solicitation (Acting moment)
$M_{righting}$ = Righting moment due to waterpressure

In the case the centre of gravity is situated at approximately the same spot as the centre of buoyancy, and the rotation is small, than the following formula holds for a rectangular floating body (later on is given a more general applicable formula):

$$M_{righting} = \varphi \cdot \rho g \cdot I_w$$

With: φ = rotation
I_w = Moment of inertia of the plane intersected by the waterline, around the y-axis

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.

A8.2 Stability

The stability of a floating structure is defined by the height between it's centre of gravity and it's meta centre; the metacentric height (GM).

Metacentric height

In Figure 57 two floating structures are depicted with an imposed rotation. Due to this imposed rotation, the centre of gravity and the net buoyant force have shifted to the right, see Figure 57.
The intersection point of the line of action of the net buoyant force and the centreline of the structure is called meta centre.

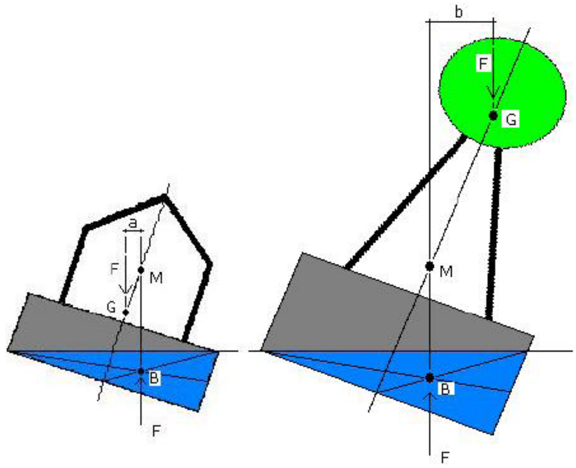


Figure 57: Rotated floating structures with high and low centre of gravity, with imposed rotation [Kuiper, 2006]

M = Meta centre
B = Centre of buoyancy
F = F_A (Archimedes force) = F_g (gravity force)

By the imposed rotation, both buoyant force (F) and centre of gravity (G) have shifted and so the lines of actions are not on the same line anymore, so moments arise.
In the left figure the meta centre is above the centre of gravity, and so an uplifting moment arises:

$$M_{up} = F \cdot a$$

This means the floating structure will return back in its original equilibrium position (stable).

In the right figure the meta centre comes above the centre of gravity, and so a heeling moment arises:

$$M_{heel} = F \cdot b$$

This means the floating structure will tilt (unstable).

With this examples is made clear that the meta centre (M) should always be above the centre of gravity (G).
The more G is above M, the higher the uplifting moment can be and the more stable the structure becomes. The height between M and G, is called the metacentric height (GM). So a higher metacentric height means more stability.

With the formula of Scribanti the height between centre of buoyancy and meta centre can be calculated:

$$BM = \frac{I_w}{\nabla}\left(1 + \frac{1}{2}(\tan\alpha)^2\right)$$

With: I_w = Moment of inertia of the water plane area (m^4)
∇ = Water displacement or Immersed volume (sometimes also mentioned with V, as done in 2.1 and 2.2) (m^3)
α = rotation

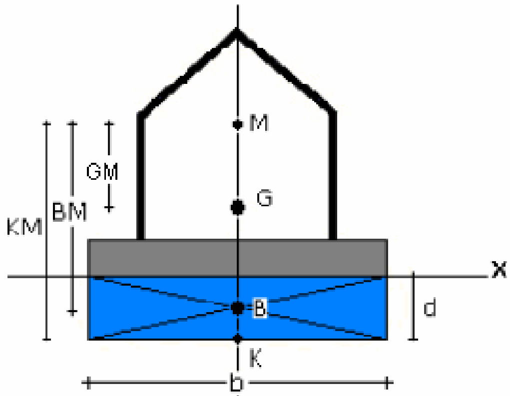


Figure 58: Metacentric height [Kuiper, 2006]

From the latter formula can be seen that the rotation plays a role in the height of the metacentre. But with small rotations (smaller than 10°), it appears that this rotation can be neglected.
So this gives the following simplified formula:

$$BM = \frac{I_w}{\nabla}$$

For a floating body with a rectangular shape, the following holds:

$$I_{w,rect} = \frac{1}{12} l b^3$$
$$\nabla = lbd$$

This results in:

$$BM = \frac{b^2}{12d}$$

In case of a rectangular floating body, of course the stability in both orthogonal directions is not equal if the width and length are not equal. In the direction of the smallest length the floating body is less stable. The formula's above and below the formulas are given for the least stable direction, with interchanging the length and the width in the formula's the stability for the more stable direction can be calculated.

The height of the point of buoyancy from of the keel is for a rectangular floating half the draught:

$$KB = \frac{1}{2} d$$

The height between meta centre and bottom floating body (keel) KM, is then given by:

$$KM = KB + BM = \frac{b^2}{12d} + \frac{1}{2} d$$

So when the location of the centre of gravity (KG) is known, this gives the following formula for the metacentric height (GM):

$$GM = KM - KG = \frac{b^2}{12d} + \frac{1}{2} d - KG$$

A8.3 Calculating rotation

When the metacentric height is determined, the rotation under a imposed load can be calculated.
It was already stated in figure 7 that the righting moment equals F·a:

$$M_{up} = F \cdot a$$

With: $F = F_A$ (Archimedes force) = F_g (gravity force)

The distance a depends on the rotation α and metacentric height:

$$a = \sin \alpha \cdot GM$$

This results for M_{up} in:

$$M_{up} = F \cdot \sin\alpha \cdot GM$$

When the rotation is not imposed, but a result of an acting moment, the righting moment (M_{up}) has to be equal with the acting moment (M_S). So the resulting rotation of this acting moment can be calculated:

$$\sin\alpha = \frac{M_{acting}}{F \cdot GM}$$

A8.4 Example

Here will be given an example of a floating body loaded by an imposed load on one side of the pontoon, which will cause rotation.

In the figure below a floating body is depicted with a height of 2 metres, a length of 30 metres and a width of 20 metres.

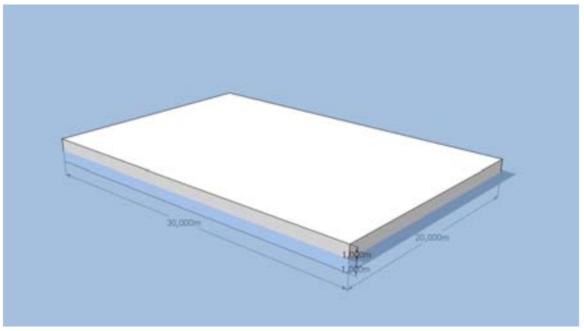


Figure 59: Rectangular floating body for example

Dimensions pontoon	
Length:	30 m
Width:	20 m
Heigth:	2 m

The load on the floating body and it's own weight is together 10 kN/m^2.
From the formula of Archimedes follows that the main draught will be 1,0 m. This leads to a remaining freeboard of 1 metre.

Load	
Self weight+ load	10 kN/m^2
Height centre of gravity (KG)	2 m

Resulting data	
Draught	1 m
Freeboard	1 m
Metacentre, short direction	31,8 m
Metacentre, long direction	73,5 m

Tilt due to imposed eccentric load

Now the above mentioned floating body will be loaded with an imposed load. This load will have a magnitude of 5 kN/m^2 and a width of 1m and will be placed at the edge of the floating body, see the figure depicted below:

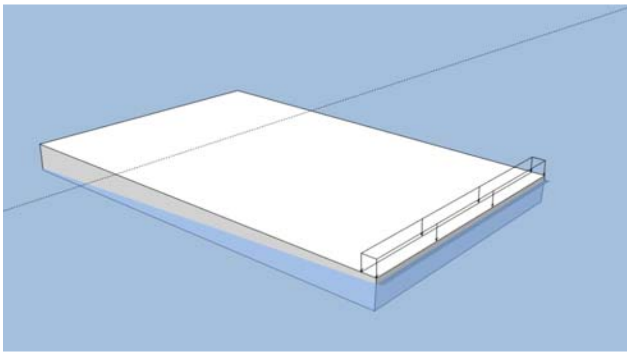


Figure 60: Skew floating body by eccentric imposed load

Imposed eccentric Load		
Magnitude load	5	kN/m
Surface	20x1	m
Resulting force	100	kN
Resulting Moment	1450	kNm

This imposed load will lead to a slight subsidence (increased main draught) and a tilt. (Also depicted in Figure 60).
The subsidence of the centre of gravity can again be calculated with Archimedes and will be 17 mm.
The rotation can be calculated with the following formula (see chapter 2):

$$\sin\alpha = \frac{M_{acting}}{F \cdot GM}$$

$$\alpha = \sin^{-1}\frac{1450}{6100 \cdot 73{,}5} = 0{,}185 \text{ degrees}$$

Resulting movements		
Sag centre of gravity	0,017	m
Rotation	0,185	degrees
	0,00323	rad
Sag by rotation	0,049	m

Check with other formula
In this example the centre of gravity is situated at the same spot as the centre of buoyancy and the rotation is small. This means the formula mentioned in paragraph 2.3.1 can also be used. This formula gives almost exactly the same rotation; 0,00322 rad in stead of 0,00323 rad.

Check program
This example is also imported in the computer program Scia Engineer. The water is schematised as an elastic foundation as explained in paragraph 2.3.
The output of the calculations of the program are almost exactly the same as the results from the hand calculation. There is a very small difference because the floating body is schematised as plate, which is not completely rigid. Because of this the floating body deforms also a little by the imposed load and therefore the sags are slightly different.
Now it is verified that the program Scia gives the good results, the program can be used more for calculating saggings.

Appendix 9:

Movement and Dynamic Stability

Six different movements

A completely free floating body has got six degrees of freedom. On the basis of the axis shown in Figure A9.61 the movements can be defined. The naval architecture defines the movements as follows [Journée, 2001]:

<u>Translations</u>
• In x-direction; *surge*
• In y-direction; *sway*
• In z-direction; *heave*

<u>Rotations</u>
• Around the x-axis; *roll*
• Around the y-axis; *pitch*
• Around the z-axis; *yaw*

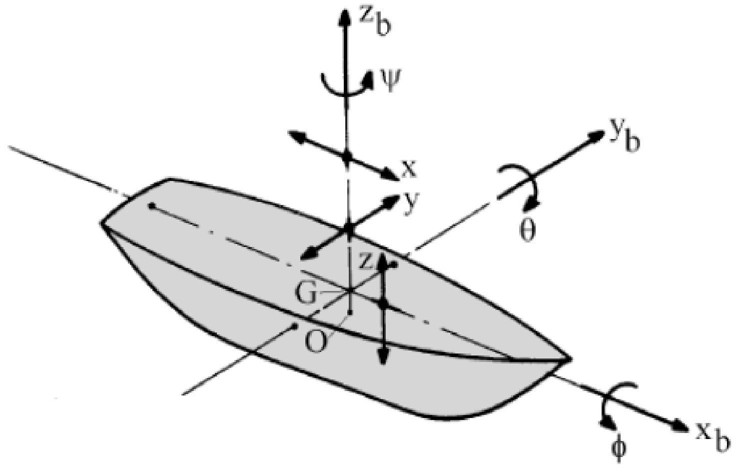


Figure A9.61: Movements [Journée, 2001]

Describing a moving floating body can be done by six coupled equations of motion. A system of 6 coupled movements leads to a stiffness matrix of 6 x 6. By hand such a system is very hard to solve. But there are several computer programs specialized in determining the movements of floating bodies. The degrees of freedom and so the amount of equations will be less if some of the movements are prevented by mooring. .

Dynamic oscillation movements

In this part the oscillation of floating structures will be explained. The formula's for the natural oscillation period will be given and the things that will affect these oscillation will be mentioned and in what way they will effect the oscillation.

Simplifications and Assumptions

For the hand calculations some simplification and assumption has to be made:

- For the calculations a rectangular floating platform is considered. In practice, floating platforms usually have a rectangular shape. For a manual calculation it is justified to also assume rectangular shaped floating bodies in case of divergent shaped bodies.
- There will be assumed a rigid body. For a relatively small area, this simplification is justified. For large surface areas, the elasticity does affect the movements.
- The wave propagation in x or y direction is considered. In the case of wave inflow perpendicular to the x direction from pitch, yaw and surge will occur. In the case of wave inflow perpendicular to the y direction the platform will perform heave, roll and sway motions. In short, with perpendicular inflow, the movements can be disconnected in two times three linked movements.
- It is assumed that the moorings doesn't influence the heave and roll movement. Formally, the mooring will influence the motions. The rotation of the floating body will however only be small. Hereby the influence of the moorings will be neglected.
- The movements are assumed to be completely decoupled. This results in treating the individual movements in an insightful way. For a manual calculation this is justified.

Dynamic heave movement

The dynamic heave motion can be schematised with a damped linear mass system, where the water acts as well as spring and damper. The mass in the system is the mass of the floating structure plus the mass of the along moving water.
The movement equation can be given by the following formula:

$$(m+a)\ddot{x} + b\dot{x} + cx = F$$ (equation A9.1)

In this formula (m+a) is the total moving mass, b is the damping constant and c is the spring constant.

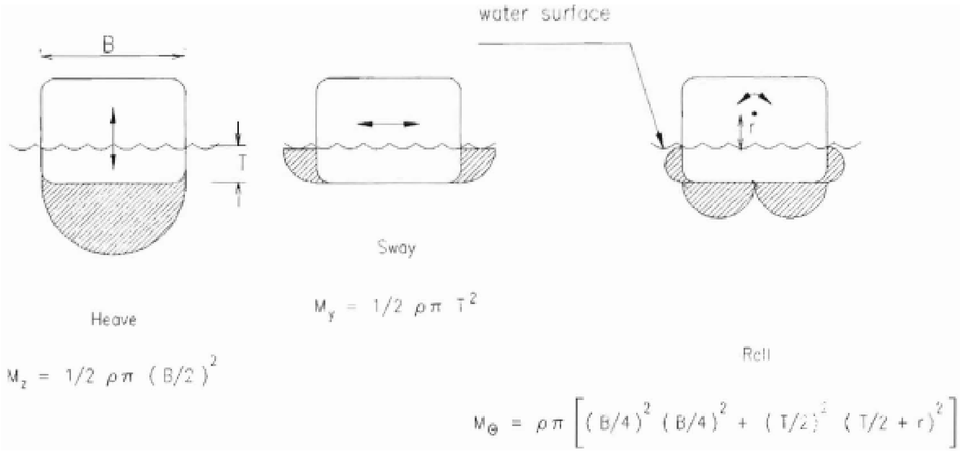


Figure 62: Added mass for the three movements [Barltrop, 1998]

The mooring connection can influence the damping, but for now the assumption is made that the mooring connection doesn't influence the movement at all.
F is the external force, for example the wave force. The wave force is a function of the wave frequency.

Wave force: $F = F_A \cdot \sin \omega t$ (equation A9.2)

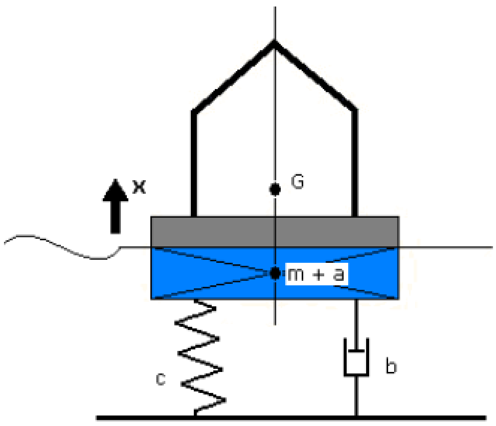


Figure 63: Damped linear mass system [Kuijper,2006]

Now the assumption is made that the structure performs a movement with the same frequency:

$$x(t) = \hat{x} \sin(\omega t + \varphi)$$ (equation A9.3)

The natural oscillation frequency (eigenfrequentie) of the floating structure plays a very important role in the dynamic heave motion. If this natural oscillation frequency is in the range of the load frequency, this can lead to resonance. This resonance could lead to large movements, which are undesirable.
The natural oscillation frequency can be found by equating the undamped linear mass system to zero.

$$(m+a)\ddot{x} + cx = 0$$

When equation A9.3 is substituted in this equation this equation can be solved to the oscillation frequency:

$$\omega_0^2 = \frac{c}{(m+a)}$$

Determining natural oscillation for rectangular floating body
Now the values for the spring constant (c), the mass (m) and the added mass (a) will be determined for a rectangular floating body.
The spring constant for water; *k*, has already been determined in paragraph 2.3, the spring constant which is used here, is *k* (*k* = *ρg*) times the waterplane area of the structure (horizontal section).

$$c = \rho g A_w$$

For the added mass has been derived an approximation formula:

$$a = \frac{1}{2} L \rho \pi \left(\frac{B}{2}\right)^2$$

with: B = width of the floating body in wave propagation direction
L = length of the floating body

The mass of the structure is equal to mass of the displaced water:

$$m = \rho L B d$$

When above mentioned formulas are substituted in the formula for the natural frequency, this results in the following:

$$\omega_0^2 = \frac{\rho g A_w}{\rho L B d + \frac{1}{2} L \rho \pi \left(\frac{B}{2}\right)^2} = \frac{g}{d + \frac{1}{8}\pi B}$$

With the standard goniometric relations: $\omega = f \cdot 2\pi$ and $T = \frac{1}{f}$. This results in the following equation for the natural heave period:

$$T_0 = \frac{2\pi}{\sqrt{\frac{g}{d + \frac{1}{8}\pi B}}}$$

To prevent resonance, the natural period should be significantly larger than the natural period of the waves. As can be seen in the given formula, a large period can be reached with a large draught and width.

Height of heave oscillation
With the natural frequency of the structure and properties of the waves known, also the size of the heave can be calculated. This however takes many calculations making use of response functions of the waves and floating structure. For this thesis this calculations will not be done. If it is necessary to calculate the amount of the heave, it is best to use a specialised computer program.

For now will be sufficed with the comment that the heave could be in the range of the wave height if the natural oscillation period frequency of the structure is about the same as period of the waves. If the natural oscillation period of the floating structure is largely bigger than the period of the load, the heave movement will be small.

Of course the damping plays also a roll in the height of the heave. The damping of a floating structure can be given by:

$$b = 2\xi \cdot \sqrt{(m+a)\cdot c}$$

In this formula ξ is the dampingratio. For ships is usually taken a value of 0,1, for rectangular floating structures this damping will be higher. The dampingratio can also be validated with a computer program. In the given formula one can see that the damping depends especially on the total moving mass and with the dampingratio on the shape.

Dynamic roll and pitch movement

The dynamic roll and pitch movement can be derived in the same way as the heave motion. The dynamic roll and pitch heave motion can also be schematised with a damped linear mass system.

The dynamic movement equation can be given by the same formula as the dynamic movement equation for heave (equation 4.1), but now *x* is the amount of rotation. The parameters a and c will have other values and have to be derived again, which is not done in this thesis, only the result is shown. The damping constant b stays the same.

<u>Calculation natural pitch and roll oscillation</u>

The natural oscillation frequency of the structure plays again a very important role, if the natural oscillation is in the same range as the period of the waves, again resonance can occur. When the dynamic movement equation is substituted and elaborated in the same way as done for heave, one can find the following formula for the natural oscillation frequency of a rectangular floating body:

$$\omega_0^2 = \frac{gBd\left(\frac{B^2}{12d} + \frac{d}{2} - KG\right)}{Bd\left(i_y + i_z\right)^2 + \pi\left(\left(\frac{B}{4}\right)^2 + \left(\frac{d}{2}\right)^2 + \left(\frac{d}{2} + r\right)^2\right)}$$

If the additional water mass will be ignored this formula can be simplified to the following formula:

$$\omega_0^2 = \frac{g \cdot GM}{j^2}$$

With: GM = metacentric height (m)
j = polar inertia radius of the element (m)

$$j = \sqrt{\frac{I_{polar}}{A_w}}, \qquad I_{polar} = I_y + I_z$$ (Dutch notation: $I_{polar} = I_{zz} + I_{xx}$)

This leads to the following formula for the natural oscillation period:

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

(When de addititional water massa is taken in account, with standard dimensions the mass m+a will become around 1,5 times as big, then ω becomes √1,5=1,2 times as small, T becomes √1,5=1,2 times bigger)

<u>Result of formula/sensitivity analysis</u>

In this formula, above the partition line the polar radius is given and below the metacentric height. As stated before, from certain dimensions on the metacentric height depends mostly on the width. So this means in this formula as well above and below the fraction line a factor is given that depends mostly on the width and the length.

With a very small value of the width and length the metacentric height gets really low, and however this sounds contradictory, this results in a large natural oscillation period and so a high dynamic stability. So increasing dimensions, when they are still small, results in decreasing the natural oscillation period so this leads to less dynamic stability. When the dimensions become larger, increasing or decreasing the dimensions do not play a role anymore for the oscillation period. (see Table 2).

For an elongated floating body the just mentioned means that the dynamic stability in the short direction is better. This can also be seen in Table 2.

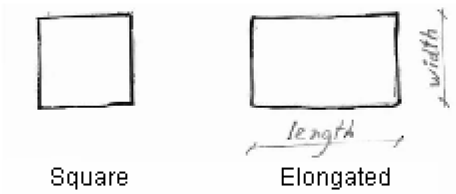


Figure 64: Top view square and elongated pontoon

			from small width to large width					
			6,5x10	8x12	10x15	20x30	40x60	1000x1500
<u>Dimensions floating body 1</u>								
width floating body	b	m	6,5	8	10	20	40	1000
length floating body	l	m	10	12	15	30	60	1500
height floating body	h	m	1,7	1,7	1,7	1,7	1,7	1,7
draught	d	m	1	1	1	1	1	1
Height centre of gravity	KG	m	3	3	3	3	3	3
<u>Calculated meta heights</u>								
Height metacentre (short direction)	KM_1	m	4,02	5,83	8,83	33,83	133,83	83333,83
Height metacentre (long direction)	KM_2	m	8,83	12,50	19,25	75,50	300,50	187500,50
Metacentric height (short direction)	GM_1	m	1,02	2,83	5,83	30,83	130,83	83330,83
Metacentric height (long direction)	GM_2	m	5,83	9,50	16,25	72,50	297,50	187497,50
<u>Calculated eigenperiods</u>								
eigenperiod, short directon (2pi/f)	T_0_y	s	3,73	2,75	2,39	2,07	2,01	1,99
eigenperiod, long directon	T_0_x	s	2,39	2,24	2,14	2,02	2,00	1,99

Table 2: Natural oscillation period for increasing widths

As stated before the height of the center of gravity has from certain dimensions small influence on the metacentric height, so the height of the centre of gravity has also small influence on the oscillation period.

An increasing draught has a positive influence on the natural period. The eigenperiod will be higher with a deeper draught, this is shown in Table 3.

			d=0,1	d=0,5	d=0,7	d=0,9	d=1,2	d=1,5	d=2	d=10
Dimensions floating body 1										
width floating body	b	m	20	20	20	20	20	20	20	20
length floating body	l	m	20	20	20	20	20	20	20	20
height floating body	h	m	1,7	1,7	1,7	1,7	1,7	1,7	1,7	1,7
draught	d	m	0,1	0,5	0,7	0,9	1,2	1,5	2	10
Height centre of gravity	KG	m	3	3	3	3	3	3	3	3
Calculated meta heights										
Height metacentre (short direction)	KM_1	m	333,38	66,92	47,97	37,49	28,38	22,97	17,67	8,33
Height metacentre (long direction)	KM_2	m	333,38	66,92	47,97	37,49	28,38	22,97	17,67	8,33
Metacentric height (short direction)	GM_1	m	330,38	63,92	44,97	34,49	25,38	19,97	14,67	5,33
Metacentric height (long direction)	GM_2	m	330,38	63,92	44,97	34,49	25,38	19,97	14,67	5,33
Calculated eigenperiods										
eigenperiod, short directon (2pi/f)	T_0_y	s	0,63	1,44	1,71	1,96	2,28	2,57	3,01	5,55
eigenperiod, long directon	T_0_x	s	0,63	1,44	1,71	1,96	2,28	2,57	3,01	5,55

Table 3: Natural oscillation period for increasing draught

Conclusion

- With increasing the width the natural oscillation period decreases slowly
- The height of the center of gravity does not really matter for large floating bodies
- A deeper draught as favourable for the dynamic stability

From both tables together can be concluded that with a draught of around 1 metre and with standard dimensions, the natural oscillation period will be between 1,5 and 3 seconds.
These natural oscillation periods are in the same range as the periods of wind induced waves caused by a fetch between 100 and 10.000 metres. This means floating structures can be brought in natural oscillation.

Size of roll and pitch

With the natural oscillation period of the structure and properties of the waves known, the size of the pitch and roll can be calculated. With heave was already said that this calculating of the movement takes a lot of calculations, making use of response functions of the waves and floating structure. For roll and pitch this response functions will be even more complex and extensive. So this calculating should be done with a specialised computer program.

Increasing dimensions result in shorter natural oscillation periods, but large dimensions will also result in less rotating of the floating structure.
The swell of a floating structure will be worst if the length and width of the structure are smaller than half of the wavelength.

Appendix 10:

Waves

A10-2

A10.1 Wave properties

In the figure below a sinusoidal wave is depicted with the basic concepts of waves.

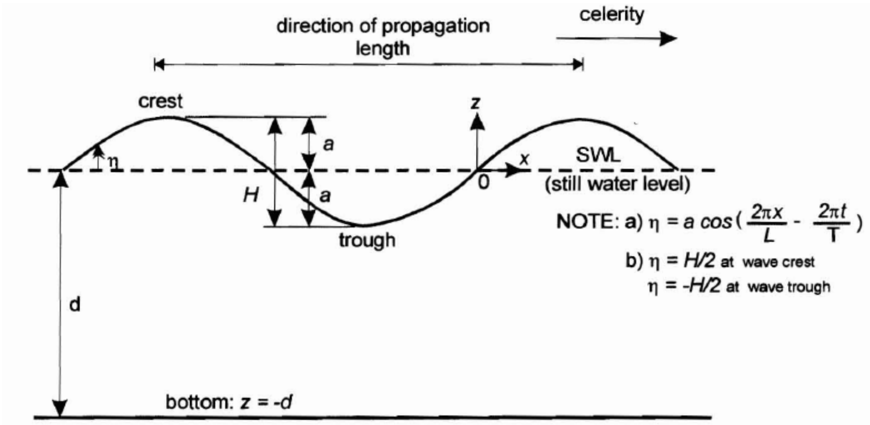


Figure 65: Elementary understandings of waves [Baars, et al., 2009]

In real the sinusoidal wave as given in the figure above, defined by a waveheight H and period T doesn't occur, except for in a laboratory situation. In nature one will find a more irregular wave field, see Figure 66.

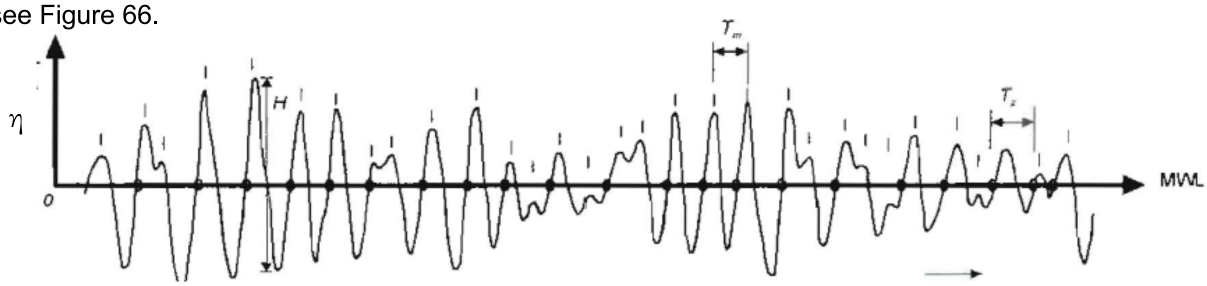


Figure 66: Irregular wave representation, from reader Introduction Hydraulic Structures

This irregular wave field can be analysed in multiple ways and the significant and design waveheights can be extracted, as well as the corresponding wave periods and lengths. For explanation how this can be one should see a wave handbook, for example Holthuijsen, (2007).

Linear/Airy wave theory

time

In practice the linear wave theory, which assumes regular sinusoidal waves (as depicted in Figure 65), is used a lot for modelling and calculation. The approximation of waves by a simple sine function is in fact a crude simplification, only for small, undisturbed waves in deep water it lies close to reality (another name for this theory is the small-amplitude wave theory). In Figure 67 can be seen when the in which circumstances this schematization lies close to reality.

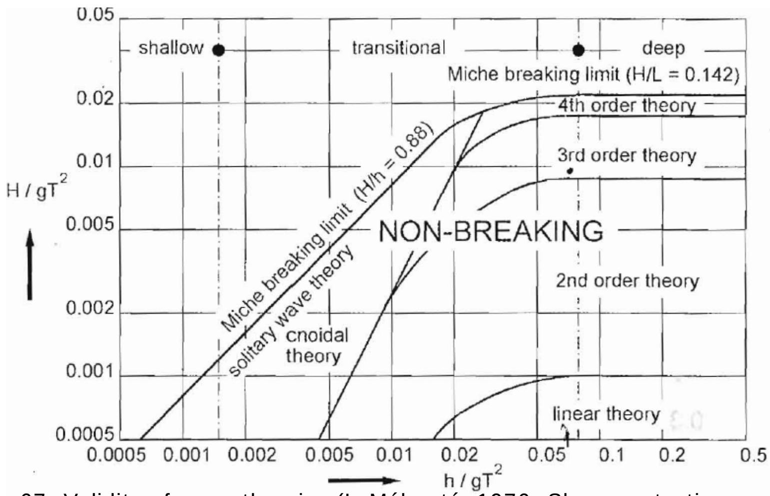


Figure 67: Validity of wave theories (LeMéhauté, 1976; Shore protection manual, 1984)

Most situations where floating structures will be used, inlands or close to the shore, will not be in the area of the linear wave theory. But for an adequate understanding however, the linear theory is very useful and attractive, since it gives a simple but complete description of waves. [United States Army Corps of Engineers, 1984]

Also in this thesis the linear wave theory will be used. Where necessary, other methods are used on top of this linear wave theory, to represent the irregular and probabilistic behaviour of waves. The calculation methods and formulas given in this chapter are based on the linear wave theory. (For detailed description and explanation of waves see a referencebook, for example Holthuijsen (2007))

The most important terms for waves are listed below (most can also be found in Figure 65):

L	Wave length; the horizontal distance between two successive wave crests (or troughs)
H	Wave height; the difference between the highest and lowest point
a	Wave amplitude, H =2a
T	Wave period; the time which passes between two consecutive wave crests
f	Wave frequency; f =1/ T
c	Wave velocity (celerity) with which the wave crests travel
η	Displacement of the water level

The displacement of the water surface in the sinusoidal approximation can be given with the following formula:

$$\eta(x,t) = a\sin(\frac{2\pi}{T}t + \frac{2\pi}{L}x)$$

With: t = a point in time
x = the x-coordinate

Also a few formulas to the describe some wave characteristics will be given:

$$L = c \cdot T$$

$$c = \frac{gT}{2\pi} \cdot \tanh(\frac{2\pi d}{L})$$

Wave height
In hydraulic engineering is made a lot of use of the term H_s. The significant wave height H_s, is the average of the highest 1/3 of the waves. This wave height occurs regularly and is therefore a lot lower than the design wave height H_d. If the wave heights can be assumed as a Raleigh distribution, the design wave height can be calculated if the significant wave height is known. In the ultimate limit state has to be calculated with the design height H_d. An example how this done is given in paragraph Wind waves.

Wave Energy
For waves it is also usual to take the wave energy and energy specter in account. This has not been necessary for this thesis, and so the wave energy is not elaborated.

Assuring specific characteristics
For a project where waves will be normative and which will be really realised, it is strongly recommended to achieve the specific wave characteristics for the specific location with extensive measurements and/or extensive modelling.

A10.2 Wind waves

Wind is the main cause of waves. The height of wind induced waves depends on the wind velocity, the strike length of the wind (fetch), the durance of the wind and the depth of the water.
If no measurements are done, the significant wave height, wave period and wave length can be estimated using the formulas from Bretschneider. Next to the formulas of Bretschneider the renowned nomograms of Groen and Dorrestein can also be used (the nomograms of Groen and Dorrestein can be found in appendix 11). The Bretschneider method and the nomograms will not give the same result, because Groen and Dorrestein used other datasets. Next to this, the nomograms are more just an approach, with Bretschneider can be calculated more exactly. Bretschneider gives good agreement with reality.
Together with the given standard formula's all properties for waves in free water surfaces can be calculated using the nomograms or the Bretschneider method.

Bretschneider
The formulas from Bretschneider are given below.

$$\bar{H} = 0,283 \tanh(0,53\bar{d}^{0.75}) \tanh\left(\frac{0,0125\bar{F}^{0,42}}{\tanh(0,53\bar{d}^{0,75})}\right)$$

$$\bar{T} = 7,54 \tanh(0,833\bar{d}^{0,375}) \tanh\left(\frac{0,077\bar{F}^{0,25}}{\tanh(0,833\bar{d}^{0,375})}\right)$$

With:

$$\bar{H} = \frac{gH_s}{U^2}, \qquad \bar{T} = \frac{gT_p}{U}, \qquad \bar{F} = \frac{gF}{U^2}, \qquad \bar{d} = \frac{gd}{U^2}$$

F = strike length (fetch)
U = wind velocity at a height of 10 meters
d = waterdepth

Below the results of Bretschneider will be given with different strike lengths and depths. The graphs are from the thesis of Maarten Kuijper. He has taken a wind velocity at a height of 10 meters of 32 m/s. This is a quite high value, but nevertheless this graphs give a good view on wind waves and how they are influenced. In the case study for the pavilion is taken a lower wind velocity.
As said in paragraph 3.3, in case of a long fetch, the wind velocity for calculating waves can be taken somewhat lower than for the wind on structures. This is because for wind waves the average wind speed during a longer amount of time during a storm is important, while for structures the peak wind speed is most important . However, for a short fetch the wind speed for calculating waves becomes closer to the peak velocity.

Height

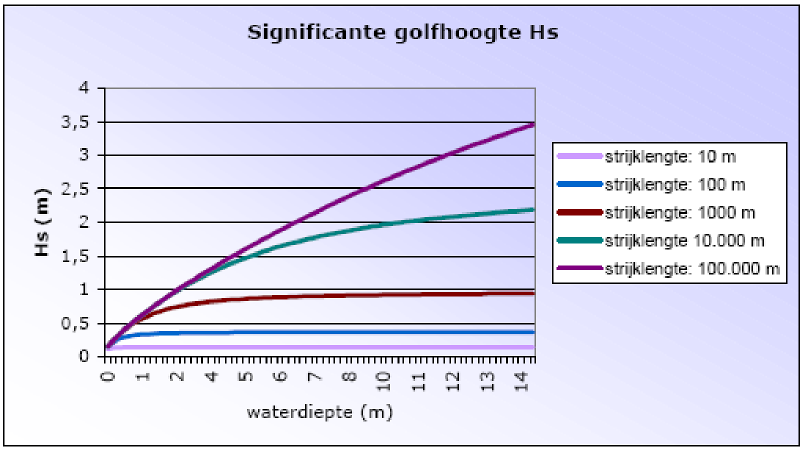


Figure 68: Significant Wave height according to Bretschneider, U = 32 m/s [Kuijper, 2006]

The depicted wave height is the significant wave height H_s. As mentioned before, the significant wave height H_s, is the average of the highest 1/3 of the waves, which is clearly lower than the wave height that can occur.

<u>Calculating design height</u>
If the assumption is made that the wave heights are distributed according to a Raleigh distribution, the design wave height can be calculated.
If the assumption is made that the peak of a storm lasts two hours and the wave period is 3 seconds, the amount of waves (N) during this storm is 7200/3 = 2400.

If one now determines a certain exceedance probability, from the following formula the ratio H_d/H_s can be calculated:

$$\Pr(H > H_d) = 1 - \exp\left(-N \cdot e^{-2\left(\frac{H_d}{H_s}\right)^2} \right)$$

With an exceedance probability of 0,10, this results in the following:

$$H_d = 2{,}25 H_s$$

An exceedance probability of 0,10 might seem quite big, but one has to keep in mind that this windspeed also already has a very small chance of occurance.

<u>Period</u>

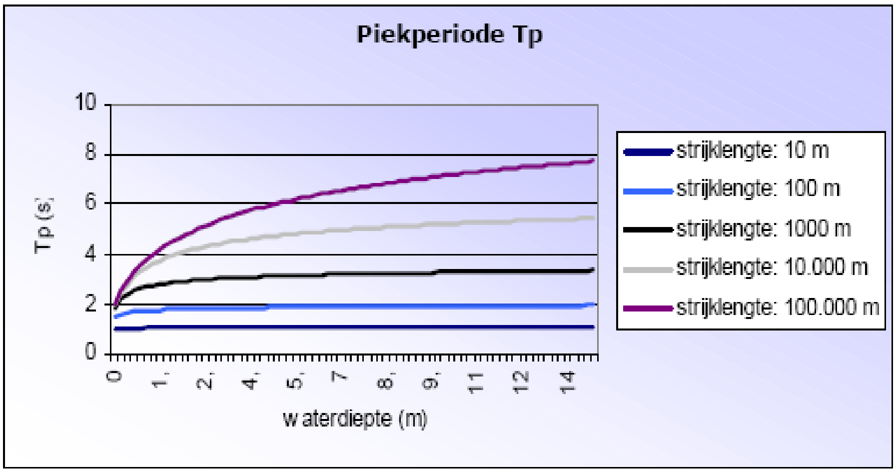


Figure 69: Peak wave period according to Bretschneider U = 32 m/s [Kuijper, 2006]

<u>Length</u>
With the given formula's for the wave length and wave velocity the wave length can be calculated with the following formula:

Wave length: $$L = \frac{gT}{2\pi} \cdot \tanh\left(\frac{2\pi d}{L}\right) \cdot T$$

Since the length is in this formula also on the right side of the equal sign, a few iteration steps may be necessary.
For a fetch of 1000 metres and a depth of 5 metres this formula gives a wave length of 11,2 metres. A bigger depth gives a slightly larger wave length and a longer fetch can give quite an increase of length; a fetch of 10.000m gives a length of 22 metres with a same depth.

A10.3 Influence of coast and obstacles

Decreasing draught

When wind waves approach the coast, a number of changes occur, caused by the change of the water depth. The three principles influencing waves approaching the coast are:

- Refraction
- Shoaling
- Breaking of waves

Due to the smaller depth towards the cost, the wave velocity decreases and the wave front turns, so it runs in to the cost (refraction).

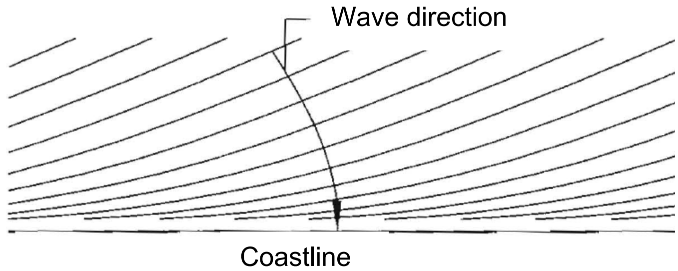


Figure 70: Refraction [d'Angremond, et al., 2003]

As a result, the wave crests become narrower, the wave becomes more concentrated an the wave height increases. At the same time, the wave velocity decreases, thereby reducing the wave length, causing a further increase of the wave height (shoaling). So the wave height increases and the wave length reduces. At a certain point, the waves are so steep, that they break.

If floating structures are used at a location where the depth decreases fast, these effects should be taken in account. Shoaling could be a problem due to the higher getting crests, but on the other hand, the decreasing of the length is mostly beneficial (see case study). Breaking waves on a floating structures should be avoided if possible.

As said earlier, the linear wave theory, with the schematised sinusoidal shape wave, is best suitable for deep water with small waves. In shallow water waves will get a more cnoidal shape.
A cnoidal wave is characterised by sharper crests and flatter troughs than the normal sine wave, see Figure 71.

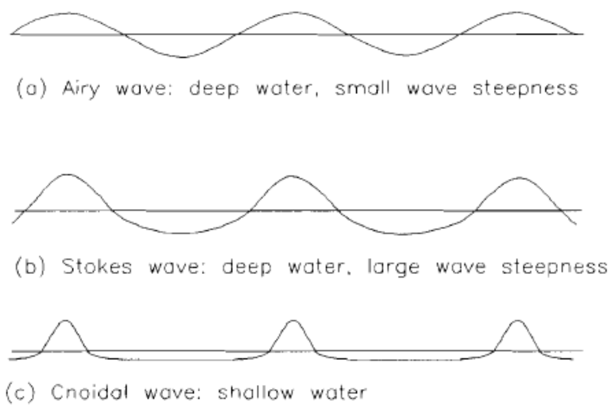


Figure 71: Different wave shapes [Barltrop, 1998]

For these other wave shapes also other schematizations account, see Figure 67.

Obstacles

Obstacles also influence the waves by amongst others diffraction and reflection.

Reflection

Reflection can be very important for the wave load, because of the following:
If waves run into a wall or a structure, they can break or reflect. The reflection can be partial or complete. If the reflection is partial in stead of complete the wave looses energy.
In the case if a wave bumps orthogonal on to a vertical wall and it is fully reflected, a standing wave will occur with twice the normal height.

With reflection under an angle the following will occur, see figure:

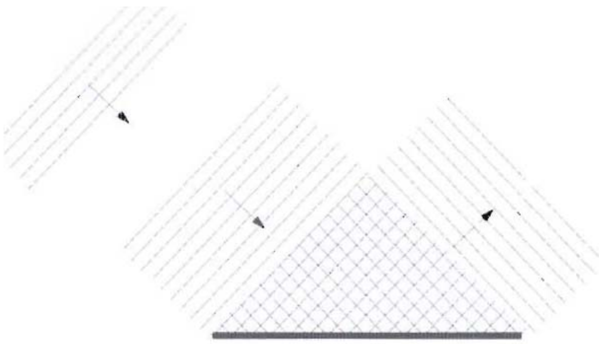


Figure 72: Reflection under an angle [Molenaar, et al. 2009]

This will have as result that the wavefield after the reflection will be less regular and the waves after the reflection will become less long crested. The crests will become less wide, short high peaks in more a ‘chessboard configuration’ will occur.

Diffraction

Diffraction occurs if obstacles are placed within the wave field. In Figure 73 is an example depicted which illustrates how this principle works.

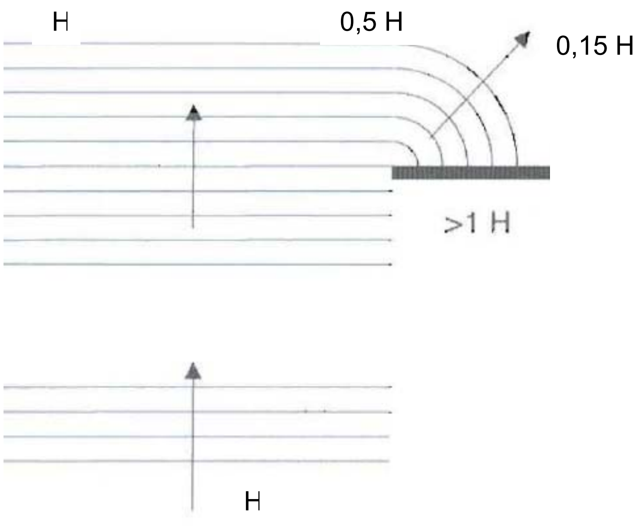


Figure 73: Diffraction [Molenaar, et al. 2008]

A10.4 Forces caused by waves

Waves will result in forces in horizontal and vertical direction.

Horizontal Forces

Waves result in horizontal forces on the floating structure in the direction of the wave field. Forces due to waves will evolve in two ways:

- difference in waterheight (non-breaking waves)
- wave force (breaking waves)

Non-breaking waves

In the case of non-breaking waves, forces are caused by the difference in water height and the hereby occurring difference in water pressure.

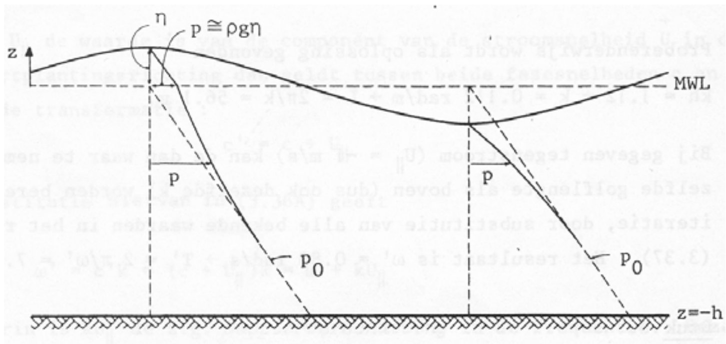


Figure 74: Water pressure by waves [Battjes, 1999]

There are several methods to determine the maximum horizontal force by waves and corresponding water pressure. The most easy method is the rule of thumb:

Standard Rule of thumb: $F_{\max} = \frac{1}{2}\rho g H_i^2 + d\rho g H_i$

With: F_{max} = Maximum horizontal wave force per meter
H_i = Height incoming wave, for strength equal to H_d
d = depth floating structure

This is depicted in Figure 75

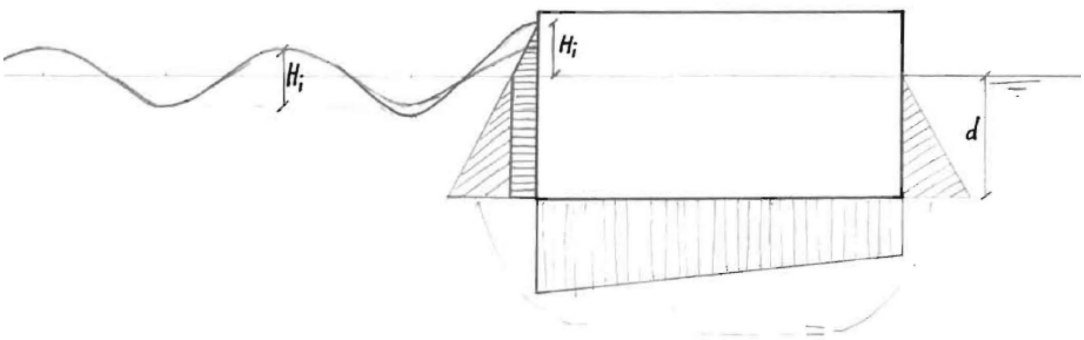


Figure 75: Difference in horizontal water pressure by waves

H_i is the height of the incoming wave (for strength H_i is equal to H_d). According to the linear theory for non-breaking waves against a vertical wall, the wave height H in front of the structure is double the incoming wave height H_i, in case of total reflection. In standard rule of thumb formula this effect is already taken in account.

The standard Rule of Thumb formula is meant for structures only loaded by waves at one side. This rule assumps a structure on one side loaded with a crest and on the other side no water or water without waves. So if on the other side of the floating structure will be a through in stead of normal water level, the horizontal force will be higher. This situation is depicted in Figure 76.

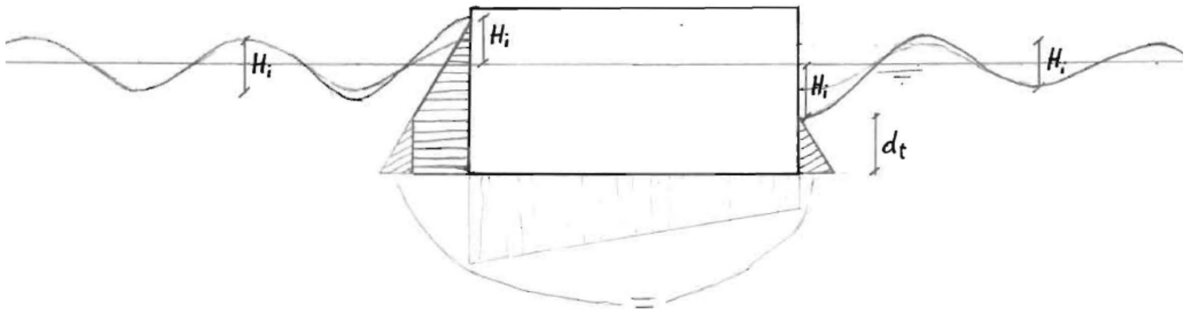


Figure 76: Difference in horizontal water pressure by waves both sides

The formula has to be adjusted to this situation:

Rule of thumb with one side crest and other side through: $F_{\max} = \frac{1}{2}\rho g(2H_i)^2 + d_t \rho g(2H_i)$

With: F_{max} = Maximum horizontal wave force per meter
H_i = Height incoming wave, for strength equal to H_d
d_t = draught in wave through

The rule of thumb method gives an over estimation of the wave force, because with this method the water pressure below the crest is taken as linear with the height all the way down. (In this schematization the pressure line in Figure 74 will stay straight, just as the striped line). This means that with this method, even on the bottom there still is a rather large water pressure difference under a crest and under a through. In reality this is not the case, other methods, for example the linear theory and Sainflou's method do take this in account.
For structures with a small draught the difference between the rule of thumb and the other methods is very small. So for floating structures with a small draught the rule of thumb can be used.

Breaking Waves
The load due to breaking waves has a very important dynamic character. Due to the collision between the wave and the structure, a transfer of impulse takes place. At the moment of impact a relatively high pressure occurs, which only lasts a very short time (in the order of 1/100s). Because of the short time span, this pressure is not representative for the stability of the structure (due to the inertia of mass). This pressure can be of importance for the strength of the structure (partial collapse) and can be quite big [Molenaar et al., 2008]. Therefore it is best to prevent the breaking of waves on a structure. Of course this is not always possible but in most cases, where floating structures are used in small inner waters, floating structures are not or hardly loaded with breaking waves. So, breaking waves will not be taken in account in this thesis

Vertical Forces
Waves will also result in vertical forces due to differences in water pressure. This will result in mainly moments in the floating body and forces in the connections in the case of a modular floating structure. Examples of two wave situations which cause a sagging and a hogging moment are depicted in Figure 77.

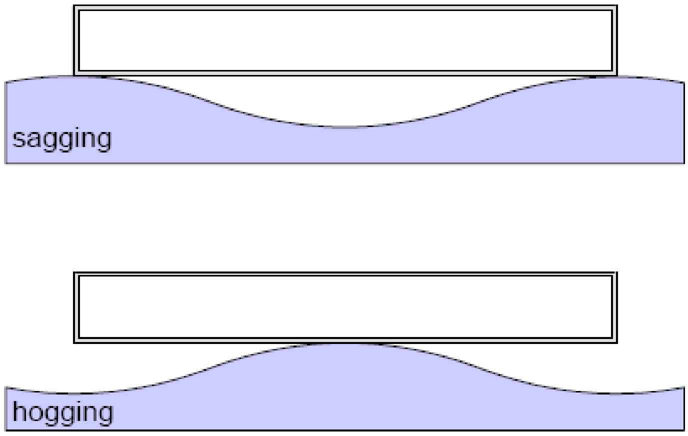


Figure 77: Wave causing sagging and hogging moment [Groenendijk, 2007]

The resulting vertical forces of the waves at the bottom of the floating structure will be taken in account by adding an extra upgoing water pressure at the crests and decreasing the neutral water pressures at the troughs. In Figure 78 is given an example of how this can be done.
When it is done as depicted in Figure 78, with two surface loads of 1/3L, there has to be added one uppointing load field and one downpointing loadfield (on top of the average water pressure due to the draught).

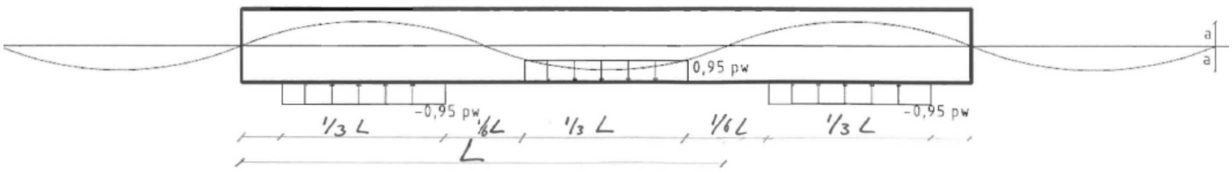


Figure 78: Resulting vertical forces by waves

The value of the schematized up and down pointing water pressure can be found by integrating the sine function. The surface underneath the standard half sine is 2:

$$A = \int_0^{\pi} \sin x dx = \cos 0 - \cos \pi = 1 - -1 = 2$$

So this results in the following for the total force for one crest or one through:

$$F_{vert} = 2 \cdot \frac{L}{2\pi} \rho g a = 0{,}32 L \rho g a$$

With a schematized block length of the surface load of 1/3 L, this gives:

$$p_{wave} = \frac{F_{vert}}{x} = \frac{0{,}32 L \rho g a}{\frac{1}{3} L} = 0{,}955 \rho g a$$

This p_{wave} can also be referred from the hydrostatic water pressure by the wave amplitude (water pressure by wave, p_w, so here the p stands for pressure and not for the p from the line load as it does in p_{wave}) as is done in the picture: $p_{wave} = 0{,}96 p_w$.

When the waves will have a more cnoidal form (see Figure 71) the schematization will be changed in a form which fits better with the cnoidal shape.

A10.5 Movement due to waves

Waves will also cause movement of the floating structure, as well heave as roll and pitch. This is discussed in chapter movement. The amount of the movement depends on the dynamic stability of the structure and the amount of freedom of the connections and moorings.
When the movements are prevented by connections or moorings this will also result in forces.
Movement due to waves will be discussed in the chapter 4 Movement Floating Structures

Sources used for this complete appendix:
Baars, et al. (2009), d'Angemond, et al. (2003), Holthuysen, (2007), Molenaar, et al (2008).

Appendix 11: Wave Nomograms

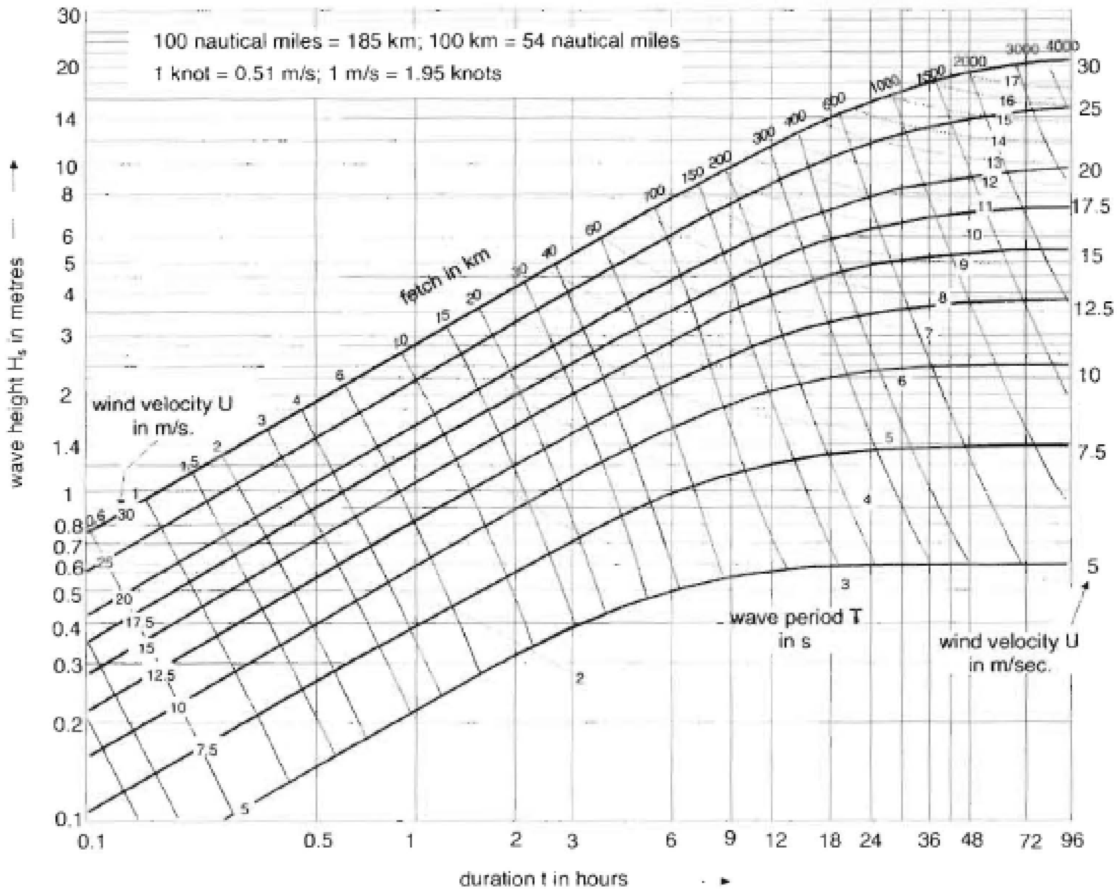


Figure 79: Nomogram for deep water

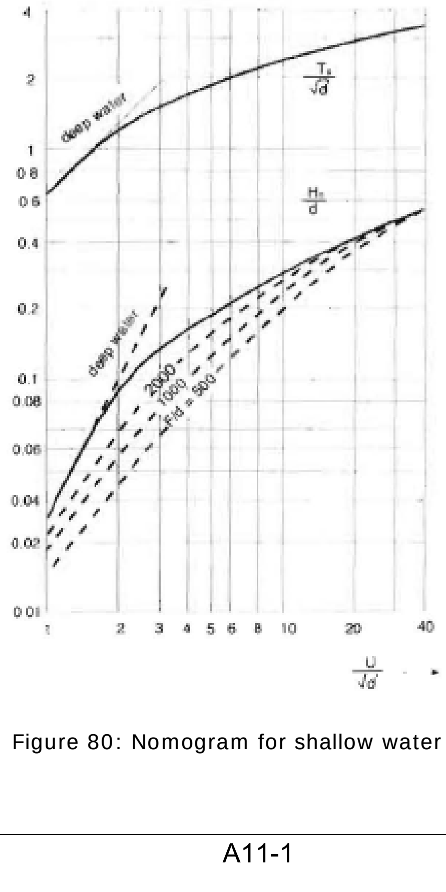


Figure 80: Nomogram for shallow water

Appendix 12:

Floating bodies for deep water

A12-2

Pneumatic stabilizing platforms

A PSP platform consists of multiple cylinders, made of steel or concrete . For small structures, the cylinders have a diameter of several decimetres, with large structures the cylinders can also have a diameter of several meters and a length of several tens of meters. At the top, the cylinder is closed by a steel or concrete deck.
The air, which is enclosed in the cylinder by the deck on the top side and on bottom side by the water, gives the platform its buoyancy. The air pressure can be adjusted by air pumps. The cylinders are interconnected with each other, so that the air can still flow from cylinder to cylinder. By waves the water in the cylinders oscillates. Then the air is put in motion between the cylinders but structure remains in place. So a pneumatic stabilizing platforms gives a high stability in waves.

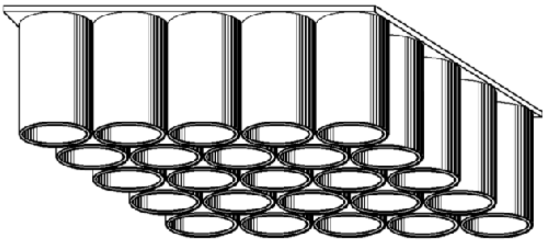


Figure 81: PSP (den Vijver, 2003)

<u>Advantages:</u>
- hardly influenced by waves
- adjustable for changing loads, by adjusting air pressure
- high durability with use of concrete
- gaining energy by wave motion possible

<u>Disadvantages:</u>
- deep draught by the large pipes, only possible with deep water
- sinkable
- expensive
- low experience

This system is only an interesting option at situation with large waves.

Semi-Submerged

Semi-submerged structures finds its pioneers in the oil drilling rigs developed over the last 25 years in the North Sea and the Gulf of Mexico. These floating structures are designed to minimize the effect of waves on the structure. (Wang, 2007) In Figure 82 the principle of semi-submerged is depicted. In chapter one are also some examples given.

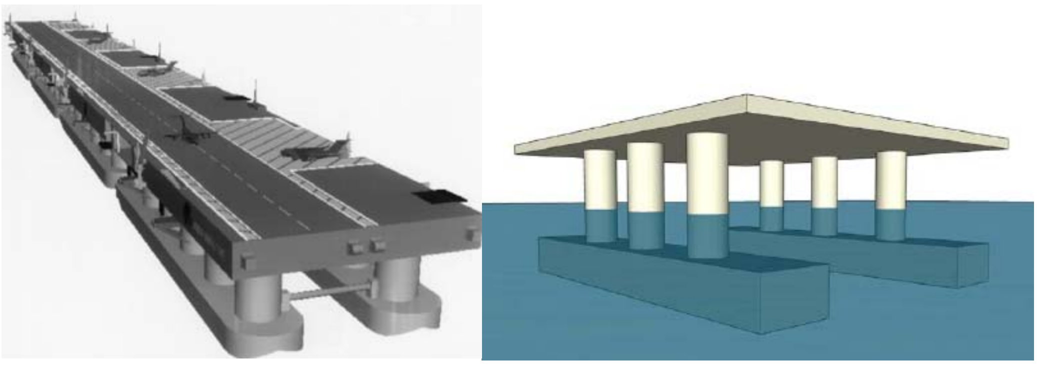


Figure 82: design for semi-submersible offshore base and principle of semi-submerged floating body

Semi-submerged structures have got less surface around the height of the water surface, the spot where the waves will result in the largest forces and movement. For this reason the hydromechanic behavior of a semi-submerged structures is a lot better. They are much more stable. This is also beneficial for coupling.

The floating element deep under the water surface gives a better hydromechanic behavior but also results in the use of more material and a lot deeper draught than with other systems.
So this solution is only suitable at places where there act big waves at the structure.

Advantages
- very stable
- hardly influenced by waves

Disadvantages
- deep draught and high freeboard
- a lot of materials, so very expensive

Tension-leg platforms

A Tension-leg platform is a vertically moored floating structure, normally used for the offshore production of oil or gas. The platform is permanently moored by means of tethers or tendons grouped at each of the structure's corners. A group of tethers is called a tension leg. A feature of the design of the tethers is that they have relatively low elasticity, such that virtually all vertical motion of the platform is eliminated. Tension leg floating structures need a permanent structure (heavy body or tension poles) in, or at the sea bottom.

Figure 83: Tension leg platform (www.sbmatlantia.com)

A tension leg structure is in fact only an interesting option with large depths and when heave motion is undesirable.

Appendix 13:

EPS

A13-2

This appendix gives a lot of information about EPS. This appendix is in Dutch and is copied out of the Master Thesis of B.G. den Vijver, *Detaillering Ponton Drijvende Stad,* Augustus 2003. Only a few small textual changes has been made.

Inleiding
Steeds vaker wordt geëxpandeerd polystyreen (EPS) gebruikt als isolatiemateriaal, licht ophoogmateriaal in de Grond, Weg- en Waterbouwsector en voor vele andere toepassingen, waaronder verpakkingen. Voor een goed begrip van de eigenschappen van EPS is het van belang eerst te weten wat de typische kenmerken van het materiaal zijn en hoe het wordt gemaakt.

Het Basismateriaal

Kenmerken
De afkorting EPS staat voor 'geëxpandeerd polystyreen', een karakteristieke en vrijwel altijd witte kunststof, die al 40 jaar voor diverse doeleinden wordt toegepast. EPS (vroeger ook piepschuim, tempex of PS-hardschuim genoemd) is van oorsprong bedoeld als isolatiemateriaal en kent daarin nog steeds haar grootste toepassing. Iedere kubieke meter EPS bevat ongeveer 10 miljoen bolletjes, ook wel parels genoemd. Elke parel telt ca. 3000 gesloten cellen die met lucht gevuld zijn. Concreet bestaat EPS qua volume slechts voor ongeveer 2% uit polystyreen en voor 98% uit lucht. Deze celstructuur met stilstaande lucht, de beste thermische isolator, maakt EPS bijzonder geschikt als isolatiemateriaal. EPS is licht van gewicht en daardoor eenvoudig te verwerken. Het is daarnaast duurzaam en degenereert niet in de loop der tijd.

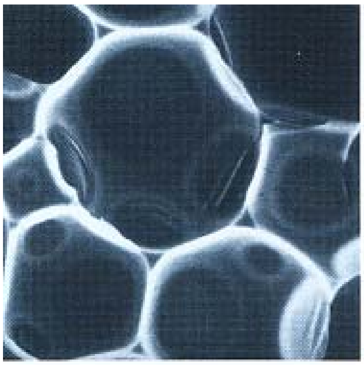


Figure 37 Celstructuur EPS

Productie van EPS
In de eerste productiefase wordt uit de fossiele grondstof aardolie via raffinage nafta verkregen. Van een fractie van nafta wordt benzeen en etheen gevormd die tezamen monostyreen vormen. Door polymerisatie en het toevoegen van het blaasmiddel pentaan ontstaat de grondstof expandeerbaar polystyreen: kleine, harde bolletjes (polystyreen-beads). Bij de tweede productiefase van EPS behoeft er niet anders gebruikt te worden dan stoom. Stoom van 105°C verwarmt en verzacht het expandeerbaar polystyreen. Daardoor wordt het pentaan gasvormig, en expandeert de polystyreen-bead zowel onder invloed van pentaan als van de stoom. Hierdoor ontstaat een gesloten celstructuur in elke geëxpandeerde EPS-parel, waar bij afkoeling het pentaan inwendig condenseert en vervolgens door de ontstane onderdruk 98% holle ruimte in de parel treedt. Voor het overgrote deel worden ze echter vervolgens met stoom van 115-125°C in blok-, band- of vormautomaten aaneengesloten tot het homogene EPS-bouwmateriaal.

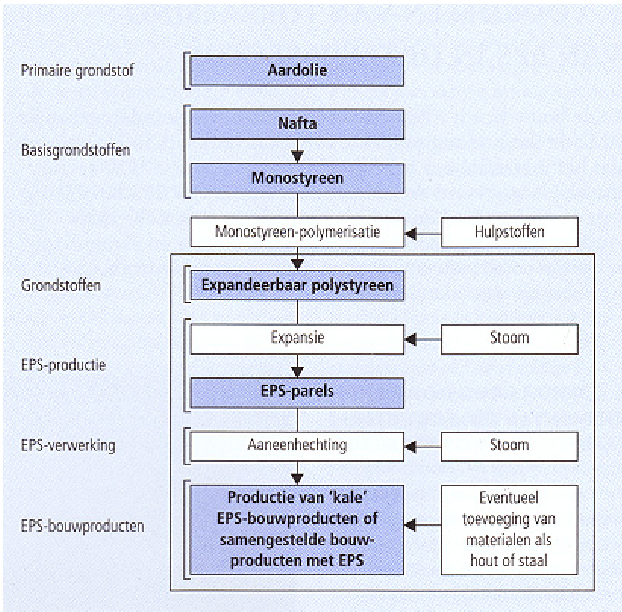


Figure 38 EPS productieschema

Duurzaamheid
Zowel in het toepassingsgebied isolatie in de woning- en utiliteitsbouw als bij gebruik in de GWW-sector is gebleken dat EPS niet veroudert. Zelfs na lange tijd behoudt het materiaal zijn specifieke eigenschappen. Doordat EPS een zuivere polymeer is zonder anorganische en vrije bestanddelen vormt het geen voedingsbodem voor planten, grassen en mossen. Het is ook schimmelbestendig, rotvrij en degenereert niet onder invloed van 'natuurlijke' oorzaken. Bovendien is EPS bestendig tegen UV-componenten uit het zonlicht. De chemische resistentie tegen niet-aardoliederivaten is zeer goed te nomen; bij gebruik in de GWW-sector is afdekking met vloeistofdichte folie een vereiste. De volgende tabel geeft een overzicht van de chemische resistentie van EPS.

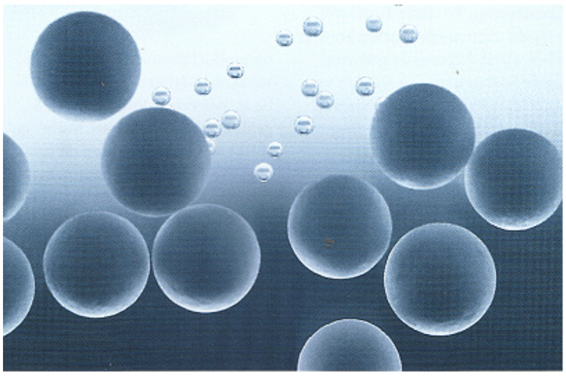


Figure 39 EPS structuur, luchtbellen

Stof	Bestendigheid			Stof	Bestendigheid		
	Bestand tegen	Voorwaardelijk bestand tegen	Niet bestand tegen		Bestand tegen	Voorwaardelijk bestand tegen	Niet bestand tegen
Aceton			•	Micro-organismen	•		
Alcohol	•			Oplosmiddelen 3)			•
Ammonia	•			Paraffineolie			•
Anhydride	•			Pentachloorphenol 2)		•	
Asfaltbitumen 1)		•		Salpeterzuur 50%	•		
Benzine			•	Spijsolie			•
Cement	•			Teeroliën			•
Chloor	•			Terpentine			•
Creosootolie 2)		•		UV-straling 4)	•		
Dieselolie			•	Vaseline			•
Gips	•			Verf 2)		•	
Kalk	•			Waterstofperoxide	•		
Kunststoffen 5)	•			Zeep	•		
Lijm 2)		•		Zoutzuur 35%	•		
Magnesiet	•			Zwavelzuur 95%	•		
Metalen	•						

1) gedurende zeer korte tijd, wanneer de contacttemperatuur niet hoger is dan 110°C
2) wanneer het oplosmiddel geheel verdampt is
3) zoals bijvoorbeeld in asfaltbitumenoplossing
4) bij niet-permanente blootstelling
5) zonder weekmakers

Tabel 4 Chemische resistentie EPS

Mechanische eigenschappen

De meeste producteigenschappen van EPS zijn gerelateerd aan de EPS-aanduiding in volumieke massa. Die volumieke massa is echter geen producteis, maar een hulpmiddel ter identificatie.

Kruip en relaxatie

Kruip is het verschijnsel waarbij de vervorming toeneemt in de tijd als gevolg van belasting. Dit blijkt voor EPS afhankelijk te zijn van vijf onafhankelijke variabelen: volumieke massa, spanning, vervorming, tijd en temperatuur. Dit zelfde geldt ook voor relaxatie, het verschijnsel waarbij onder een opgelegde vervorming de (inwendige) spanning in de tijd afneemt. Voor de berekeningen wordt aanbevolen het concept CEN-norm voor EPS aan te houden. Daarin wordt gesteld, bij een belastingniveau van circa 0.30 $\sigma_{\varepsilon=10\%\ \text{korte duur}}$ (zie volgende alinea) een totale vervorming van 2% aan te houden voor een levensduur van 50 jaar. Recent onderzoek gaf aan dat de te verwachten kruip na 1 jaar, bij een belastingniveau van ca. 25% van de korte-duur druksterkte (σ_{10}) minder dan 0.2% bedraagt. De helft van die kruip treedt al op na 1 dag Het kruipgedrag op een logaritmische schaal is lineair te noemen.

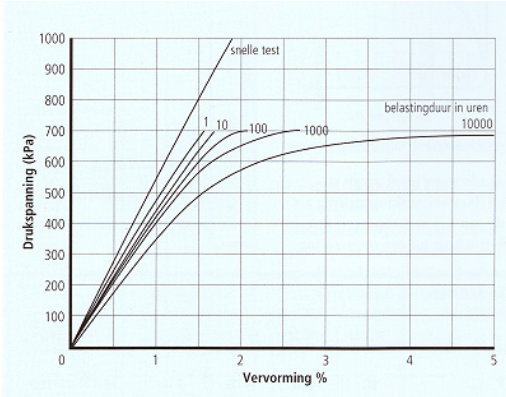


Figure 40 Spanning/vervormingsrelatie

Korte-duur druksterkte

De spanning/rek-relatie van EPS heeft een bijzondere vorm. De lineair-elasticiteitsgrens ligt bij circa 1 tot 1.5% vervorming. De druksterkte, zoals in NEN-EN 826 gedefinieerd, is arbitrair bepaald als de spanning bij een vervorming van 10% van het proefstuk. Deze $\sigma_{\varepsilon=10\%}$ wordt ook in het kader van kwaliteitsbewaking als productvariabele gecontroleerd. De belastingssnelheid bedraagt hierbij ongeveer 10% per minuut. Deze initiële lineair-elastiche vervorming wordt gebruikt voor de bepaling van de Elasticiteitsmodulus E_t (kPa). In het tweede trajectdeel is dus sprake van een niet-lineair elastisch spanning/rek-verloop. De vloeigrens bedraagt circa 75% van de druksterkte voor alle densiteiten EPS.

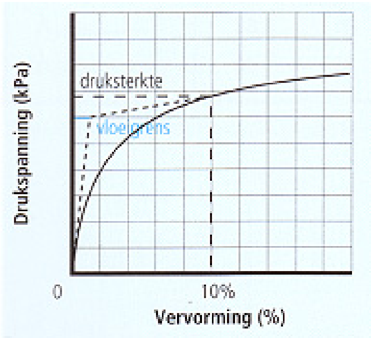


Figure 41 Spanning/rek-relatie

Lange-duur druksterkte

De lange-duur druksterkte is het toelaatbare continue belastingniveau gedurende 50 jaar, teneinde ongewenste kruipeffecten te beperken. Op basis van de aangegeven kruiprelaxatie effecten wordt de lange-duur druksterkte van EPS gesteld op circa een vierde van de korte-duur druksterkte.

Wrijvingscoëfficiënt

Voor berekeningen kan een veilige waarde voor de wrijvingscoëfficiënt van 0.5 worden aangehouden, als de wrijvingshoek zich beperkt tot circa 30°.

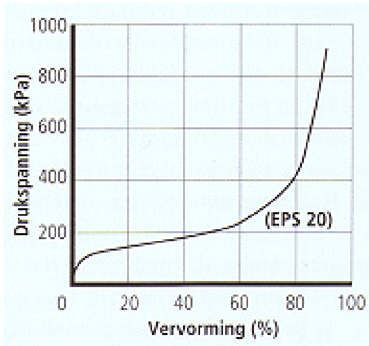


Figure 42 Last vervormingsdiagram

Gegevens

Eigenschappen	Type EPS					
	EPS 15	EPS 20	EPS 25	EPS 30	EPS 35	EPS 40
Druksterkte ($\sigma_{10,\ \text{kort}}$) (kPa)	80	120	170	210	260	300
Druksterkte ($\sigma_{2,\ \text{lang}}$) (kPa)	20	30	40	50	60	70
Elasticiteitsmodulus (E_t) (kPa)	4000	6000	8000	10000	12000	14000
Treksterkte (σ_t) (kPa)	200	280	360	440	520	600
Buigtreksterkte (σ_b) (kPa)	190	270	360	460	570	670
Afschuifsterkte (τ) (kPa)	50	75	135	175	235	310

Tabel 6 Eigenschappen EPS

Bestand tegen waterdruk
De lange duur druksterkte van de meest voorkomende EPS soort, EPS 20, is zoals valt af te lezen uit bovenstaande tabel 30 k/Pa. Dit is gelijk aan 30.000N/m^2 of 0,03N/mm^2.

De statische waterdruk is te berekenen met p=ρgz.
Op één meter diepte bedraagt de waterdruk ongeveer: p=1000*10*1=10.000N/m^2.

Dit betekent dat EPS 20 bestand is tegen 3 meter waterdruk en dan nog binnen het elastische gebied zit. Bij dezelfde EPS soort en een hogere waterdruk zal het EPS meer ingedrukt worden/gaan vervormen.

Thermohygrische eigenschappen

Warmtegeleidingscoëfficiënt
Dankzij de unieke parelstructuur bevat EPS ca. 98% lucht, de beste isolator, en daarmee heeft EPS zeer goede warmte- en koude-isolerende eigenschappen. Deze komen tot uitdrukking in de warmtegeleidingcoëfficiënt (λ-waarde).

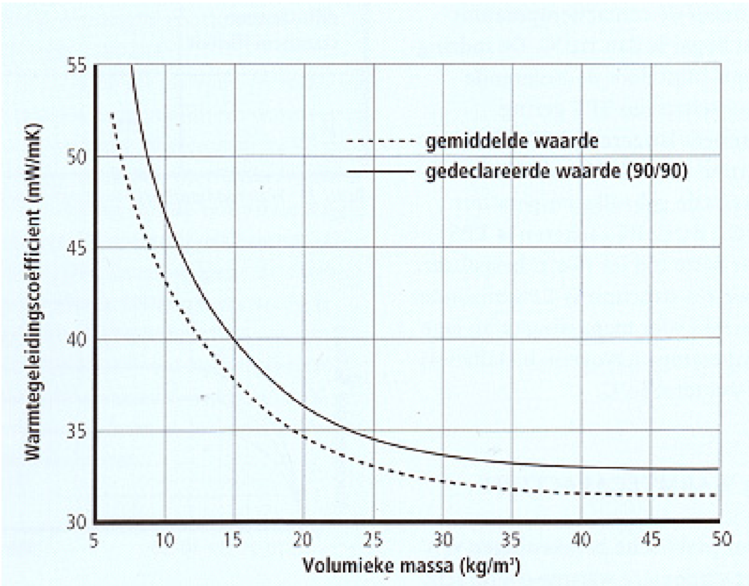


Figure 43 Warmtegeleidingscoëfficiënt

Temperatuurbestendigheid
EPS is een thermoplastische kunststof en daarmee bestand tegen hitte gedurende een zeer korte tijd, wanneer de contacttemperatuur niet hoger is dan 110°C. De indringdiepte blijft door de isolerende eigenschap van EPS gering. Wanneer langere tijd EPS door warmte wordt belast, dan is de maximale gebruikstemperatuur 75°C à 85°C. Bij cacheren is EPS zeer korte tijd tot 160°C belastbaar. Door z'n structuur is EPS bijzonder geschikt voor toepassingen bij lage temperatuur (cryogene installaties) en wel tot –180°C.

Warmtecapaciteit
Voor praktische berekeningen van niet-stationaire warmte-overdracht kan voor de warmtecapaciteit een waarde van 1500 J/kg K worden aangehouden. Deze waarde wordt grotendeels bepaald door het gegeven dat EPS voor 98% uit lucht bestaat.

Uitzettingscoëfficiënt
Voor de uitzettingscoëfficiënt kan voor praktische berekeningen voor alle typen EPS een waarden van 7 * 10^{-5} m/mK worden aangehouden.

Wateropname bij onderdompeling
Als gevolg van de fysische opbouw van EPS die tot een vorm zijn geschuimd neemt EPS in de parels geen water op. Het mogelijke aanwezige water bevindt zich in de ganglionnen tussen de parels. Bij gedraineerde EPS in de GWW-sector reduceert de wateropname met 50%.

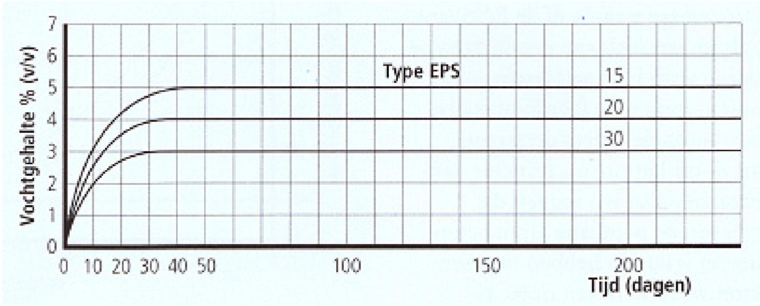
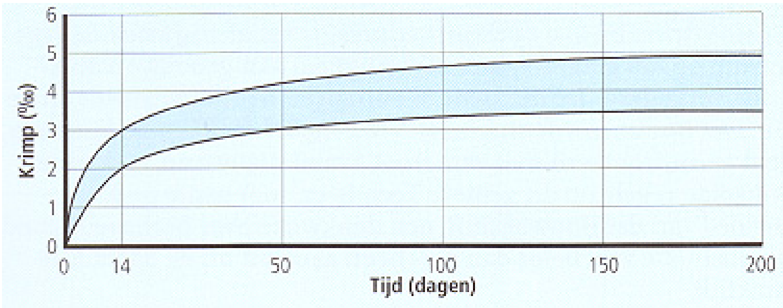


Figure 44 Wateropname EPS (%) in dagen

Capillariteit
Door de mechanische opbouw van EPS is de capillariteit vrijwel nihil en in de praktijk verwaarloosbaar.

Geboortekrimp
Als gevolg van de fabricage met stoom zal door de uitwisseling van drukverschillen juist na de blokvorming de grootste krimp optreden, dit wordt 'geboortekrimp' genoemd. Voor praktische toepassingen is deze in de meeste gevallen verwaarloosbaar, gelet op de tijd die er ligt tussen productie en uiteindelijke toepassing op de bouw.



Voor berekening kan eenvoudigheidshalve met nakrimp van ca. 2 $^0/_{00}$ worden gerekend tenzij de vervormingen door opgelamineerd materiaal verhinderd worden.

EPS Materiaal eigenschappen

Materiaal eigenschappen	Grootheid		Type EPS					
	notatie	Eenheid	EPS15	EPS20	EPS25	EPS30	EPS35	EPS40
Volumieke massa (nominaal) ter identificatie	ρ	Kg/m^3	15	20	25	30	35	40
Warmtegeleidingscoëfficiënt (rekenwaarde)*	λ	W/m.K	0.040	0.038	0.036	0.036	0.036	0.036
Diffusieweerstandsgetal	μ	-	20	30	40	60	90	120
Druksterkte bij 10% vervormingseis	σ_{10}	kPa	60	100	130	165	200	240
Korte-duur druksterkte	σ_{10}	kPa	80	120	170	210	260	300
Lang-duur druksterkte	σ_{10}	kPa	20	30	40	50	60	70
Buigsterkte	σ_t	kPa	190	270	360	460	570	670
Treksterkte	σ_t	kPa	200	280	360	440	520	600
Vochtopname bij volledige onderdompeling na 7 dagen	%	-	1.70	0.60	0.55	0.50	0.45	0.40
Vochtopname bij bolledige onderdompeling na 1 jaar	%	-	5.0	4.0	3.8	3.5	3.3	3.1
Lineaire uitzettingscoëfficiënt	α	m/m	$7*10^{-5}$	$7*10^{-5}$	$7*10^{-5}$	$7*10^{-5}$	$7*10^{-5}$	$7*10^{-5}$
Wrijvingscoëfficiënt	c	-	0.5	0.5	0.5	0.5	0.5	0.5
Warmtecapaciteit	C	J/Kg K	1500	1500	1500	1500	1500	1500
Temperatuurbestendigheid (min/max)	T	-	-110/70	-110/70	-110/70	-110/70	-110/70	-110/70
Tijdelijk bestand tegen een maximale temperatuur	T_{max}	-	110	110	110	110	110	110
E-modulus	E	kPa	4000	6000	8000	10000	12000	14000

Tabel 6 EPS-eigenschappen

Kosten

Voor een bepaling van de kosten voor EPS wordt uitgegaan van de prijslijst van Unidek EPS. Alle prijzen zijn in euro's exclusief b.t.w.

ALGEMENE KENMERKEN	
lengte	500 tot 8000 mm (oplopend met 500 mm)
breedte	1000 – 1200 – 1220 –1250
dikte	10 – 15 – 20 mm (vanaf 20 mm per 10 mm oplopend tot 250 mm)
randafwerking	recht
verpakking	in rekfolie met etiket (max. pakkethoogte 500 mm): max. 2500 mm
levertijd	5 werkdagen – franco vanaf 10 m^3

Tabel 7 Algemene Kenmerken EPS.

PRIJS PER M^3 standaardmaten			
EPS 10.0	45.00	EPS 25.0	72.00
EPS 12.5	47.00	EPS 27.5	78.00
EPS 15.0	51.00	EPS 30.0	85.00
EPS 17.5	54.00	EPS 32.5	89.00
EPS 20.0	62.00	EPS 35.0	95.00
EPS 22.5	67.00	EPS 40.0	108.00

Tabel 8 Prijs per m^3 EPS.

TOESLAG afwijkende maten in % (op basis standaard maten)*					
	> 2 m^3**	> 5 m^3	> 10 m^3	> 40 m^3	> 80 m^3
afwijkende lengte	20	10	5	3	2
afwijkende breedte	40	20	10	6	3
afwijkende dikte	8	4	2	1	0
* Verpakking; los, rekfolie, tape of PE-zak van 330L ** Afname minder dan 2 m^3: prijs op aanvraag					

Tabel 9 Toeslag op Prijs per m^3 , afwijkende maten EPS.

BRUTO PRIJSTOESLAGEN M^3	
Randafwerking 2-zijdig	4.00
Randafwerking 4-zijdig	6.00
Haaklasverbinding rondom	2.00
EPS-OK-kwaliteit (opslag 4 weken)	2.00
EPS-DR t.b.v. drainage	5.00
Geschuurde platen (15 t/m 220 mm)	10.00

Tabel 10 Prijs per m^3 , extra's EPS.

TOEBEHOREN	
Unidek PU-lijm type PU112 (1 kg flacon)	6.40
Unidek PU-lijm type PU112 (10 kg jerrycan)	54.25
Unidek PU-lijm type PU112 (210 kg drum)	1055.00

Tabel 11 Prijs per toebehoren EPS.

Conclusie

In het algemeen is de duurzaamheid van EPS redelijk hoog, want het ondervindt geen invloed van uv-componenten uit het zonlicht, niet-aardoliederivaten en is rottingsvrij en schimmelbestendig. Alleen tegen aardoliederivaten zoals benzine en oliën is EPS niet bestand.

De verschillende soorten EPS hebben een heleboel overeenkomstige eigenschappen, zoals warmtegeleidingcoëfficiënt, lineaire uitzettingscoëfficiënt, wrijvingscoëfficiënt, warmtecapaciteit en temperatuurbestendigheid. De EPS types verschillen voornamelijk in elasticiteitsmodulus en sterkte-eigenschappen en niet te vergeten de prijs.

Een EPS type met een grotere volumieke massa heeft dus een grotere elasticiteitsmodulus en sterkte, maar is ook weer duurder.

Materiaal eigenschappen	**Grootheid**		**Type EPS**					
	notatie	Eenheid	EPS15	EPS20	EPS25	EPS30	EPS35	EPS40
Druksterkte bij 10% vervormingseis	σ_{10}	kPa	60	100	130	165	200	240
Korte-duur druksterkte	σ_{10}	kPa	80	120	170	210	260	300
Lang-duur druksterkte	σ_{10}	kPa	20	30	40	50	60	70
Buigsterkte	σ_t	kPa	190	270	360	460	570	670
Treksterkte	σ_t	kPa	200	280	360	440	520	600
E-modulus	E	kPa	4000	6000	8000	10000	12000	14000
Prijs per m^3	€/m^3	€	51,--	62,--	72.--	85.--	95.--	108.--

Tabel 12 Verschillen van de types EPS

A13-10

Appendix 14:

EPS-concrete systems

A14-2

Appendix 14a: EPS-concrete systems

In this Appendix several EPS-concrete systems which have been used and several new EPS-concrete principles are elaborated.

IMF or Ooms method

As mentioned in paragraph 5.1, in the early 1980's International Marine Floatation Systems Inc. (IMF) introduced constructing real estate on water with using EPS. In 1999 Ooms introduced this system in the Netherlands. The big advantage of this system is that that no expensive dock or assembly hall is needed. The construction method of IMF and Ooms is depicted in Figure 84.

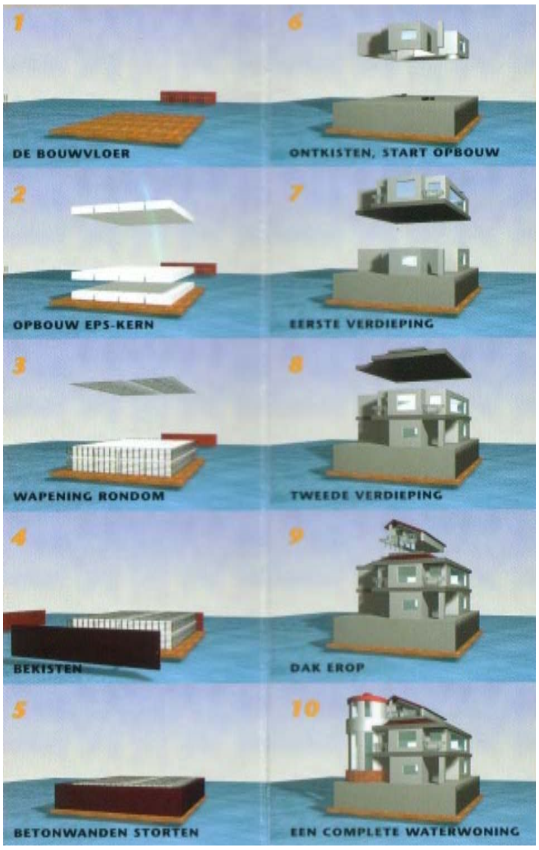


Figure 84: IMF or Canadian construction method waterhouse (Ooms BV.)

This system results in a thick core of EPS with a thin concrete layer on top of it, and concrete side walls for stiffness and protection of the EPS.
This system gives the possibility to construct on water at any sheltered location.

Flexbase method

Flexbase also developed a concrete-EPS system which can be used for constructing on the water. (Flexbase is a combination of contractor Dura Vermeer and polystyrene manufacturer Unidek.) Thos method has the same advantage as the IMF/Ooms system: no expensive dock or assembly hall is needed. This system is based on putting multiple layers of polystyrene elements on the water. When the demanded height of the EPS is reached, the concrete beams and floor are being poured on it. This is all done at the water surface. In Figure 85 can be seen how the first layer of EPS is placed on the water.
This system was applied as a pilot in a floating greenhouse in Naaldwijk in 2005 (see also paragraph 1.3), and it again used for the floating pavilion. The floating bodies constructed with this system are fully constructed on the water.

Figure 85: Putting in layer the first layer of EPS, Naaldwijk [Flexbase]

To realize a one-piece EPS surface, the separate EPS elements are connected with an so called 'haaklas'. This 'haaklas' is depicted in figure 93.

Figure 86: Connecting first layer of EPS with 'haaklas', Greenhouse Naaldwijk [Flexbase]

This 'haaklas' has got a little strength, but is not yet strong enough to walk over, here for planks are placed on top of the first layer during construction, see Figure 87.

Figure 87: First layer of EPS of the floating pavilion, with planks on it

A second layer of EPS, is placed orthogonal on top of this first one. This layer also has got an 'haaklas' and with this two layers glued together, it is strong enough to walk over.
For the pavilion, 3 layers of EPS with a thickness of 20 cm and a fourth layer with a thickness of 40 centimetres are used, till the bottom of the beams, this can be seen in Figure 88.

Figure 88: Pouring the beams of the pavilion

In Figure 88 can also be seen how the concrete beams are casted.
With this method there is no limitation on dimensions, which would be the case with a construction dock. The system therefore offers more degrees of freedom with regard to form and size. Dimensions of the floating body are only limited by the control of the concrete during the pouring and stiffening.
By ‘glowing’ the EPS (cutting with a hot wire) any shape of floating body is possible.

Disadvantages
Constructing on the water give also problems. Before the beams have hardened, the structure is not rigid at all. This means it will easily deform, this has also happened when the beams of the pavilions were casted, which had as a result that after the casting the beams were already deformed. Casting have to been very cautious.
Another disadvantage of this method is that the height of the beams is limited, because it needs a certain amount of EPS below them. This means the structure will have a relative low rigidity. Another disadvantage is that the EPS is not fully protected by the EPS so, the risk exists that the EPS can break loose of the structure, during transport or by waves.

First TU master thesis’s on EPS bodies

Dennis Ling designed in 2001 a hexagonal floating body made of EPS with thin toplayer of concrete. So in fact this was the same solution as the result of the IMF method. Only Dennis Ling designed a very large ‘building stone’ for his floating city, where multiple buildings should be constructed up on. He did his calculations for a square floating body width a large width of 100m, existing out of blocks of EPS with a thickness of 2 metres, and an outer concrete wall of 0,2 m. On top of the EPS body was also placed concrete layer with a thickness of 20 centimetres. (Dennis Ling, 2001)
In 2002 Den Vijver continued with this design since it was not fully calculated. He did the calculations for hexagonal floating body and designed a hexagonal pontoon of 50 metres. He concluded that beams were necessary for the rigidity and stiffness of the floating body. Then he redesigned the hexagonal body, now with concrete beams, with a center to center distance of 10 metres. In fact this design still didn’t fulfil the demands of rigidity, for the high loads he wished to use (multiple storey buildings), so than he lowered his loads somewhat. (Den Vijver, 2003) From this thesis’s could be concluded that beams are necessary for rigidity of the structure.

Ties Rijcken’s system

In 2003 Ties Rijcken designed a much smaller float, called floating brick. Initially, he designed a hexagonal building block that part could form part of a large 'honeycomb'. This hexagonal building block has a diameter of 3 meters. Ties Rijcken concluded that making cavities in the EPS could form an optimal formwork for fibre-reinforced high strength concrete. The combination of EPS and an optimised shaped high-strength concrete frame, would lead to a very light and strong unsinkable body. Using this system, combined with light timber housing would result in a draught of only 40 centimetres.

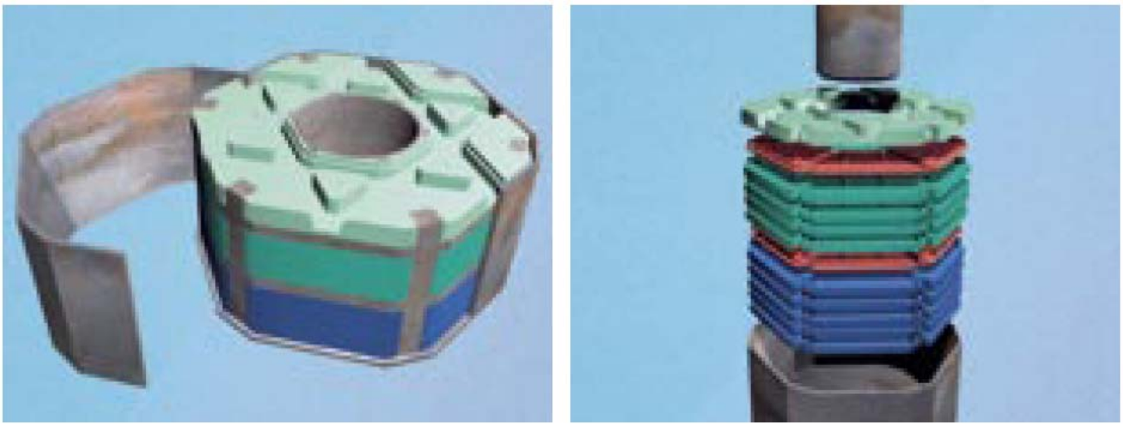


Figure 89: Honeycomb building brick of EPS and UHPC by Ties Rijcken

In Figure 89 can be seen that the EPS top layer has an different shape, so there remain cavities for pipes and electricity. Later on Ties Rijcken also designed a rectangular EPS modular building brick. The second concept is orthogonally oriented and consists of only one element of EPS. The initial costs are lower so the system is more suitable for a pilot project than some more complex honeycomb system. This orthogonal system is depicted in Figure 90.

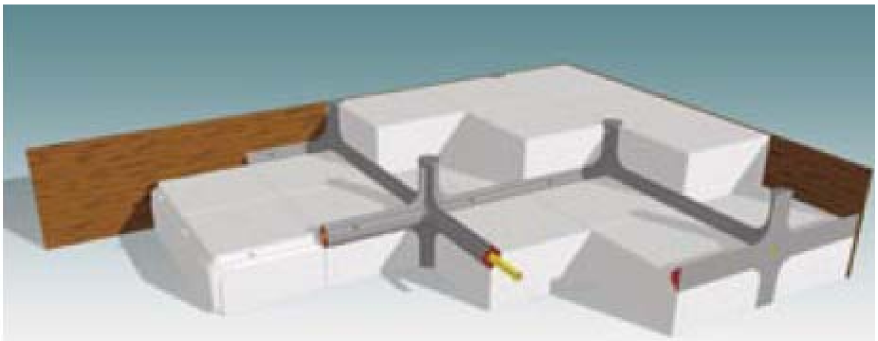


Figure 90: Orthogonal design with EPS and UHPC by Ties Rijcken

The separate building blocks can be connected to each other with pouring in anchors on the corners, or by linking them with tension rods.
[T. Rijcken and prof. ir. J.A. den Uijl in Cement, 2005, T. Rijcken, Neerlands H_2Oop, producten voor waterwijken. TU Delft, september 2003]

Maarten Kuijper's system

In 2006 Maarten Kuijper graduated on 'a floating foundation'. He also designed an EPS combined with fiber reinforced concrete building block, which was based upon the design of Ties Rijcken. Kuijper optimised the shape of the beams (see Figure 91) and also adjusted a 'cassette vloer' which also saves a lot of weight. The design of Kuijper is depicted in Figure 92 and Figure 93.

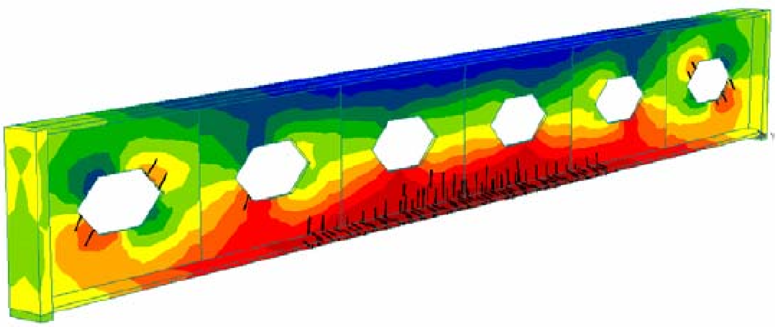


Figure 91: Optimised fibre reinforced beam, with cavities, by Kuijper (2006)

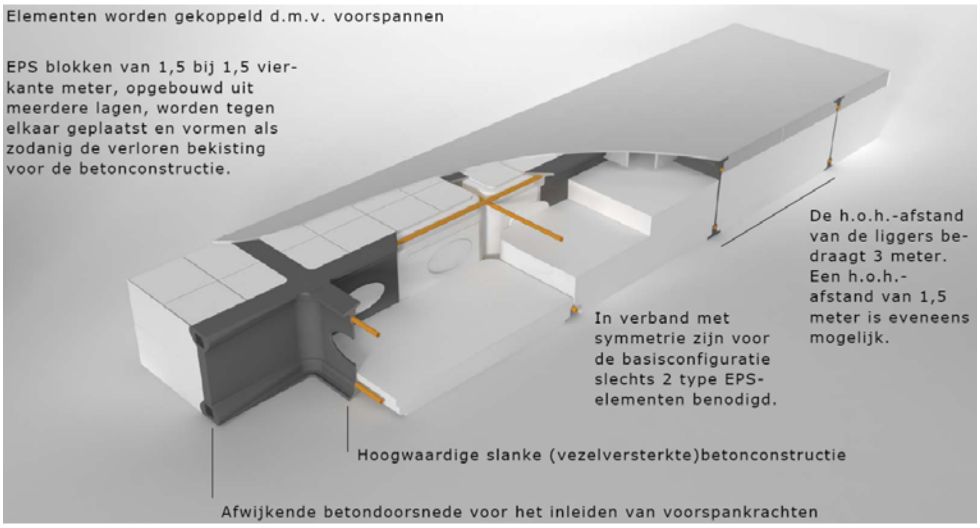


Figure 92: Floating Element from Kuijper, EPS and UHPC

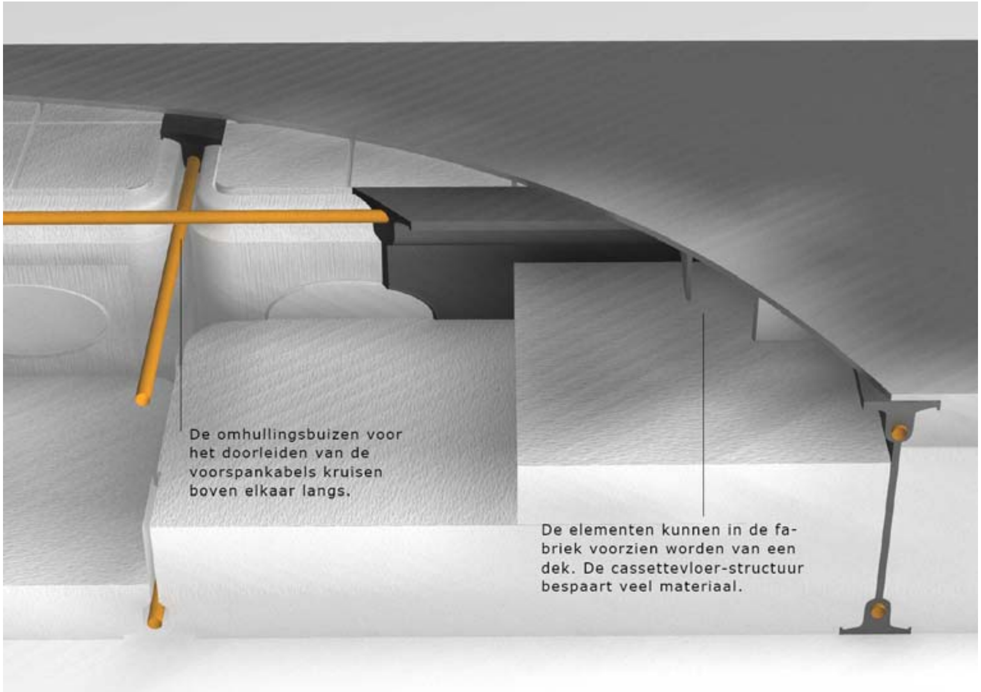


Figure 93: Floating Element from Kuijper, EPS and UHPC

Aquastrenda system

In 2004 a group of 6 students of the 'hogeschool Zuyd' in Heerlen graduated on a floating living quarter. The group existed out of 3 civil engineering students and 3 building engineering students, they called themselves 'Aquastrenda'. They also designed a floating body of concrete and EPS, see Figure 94. With their graduation result they have won the Kivi Niria HBO price.

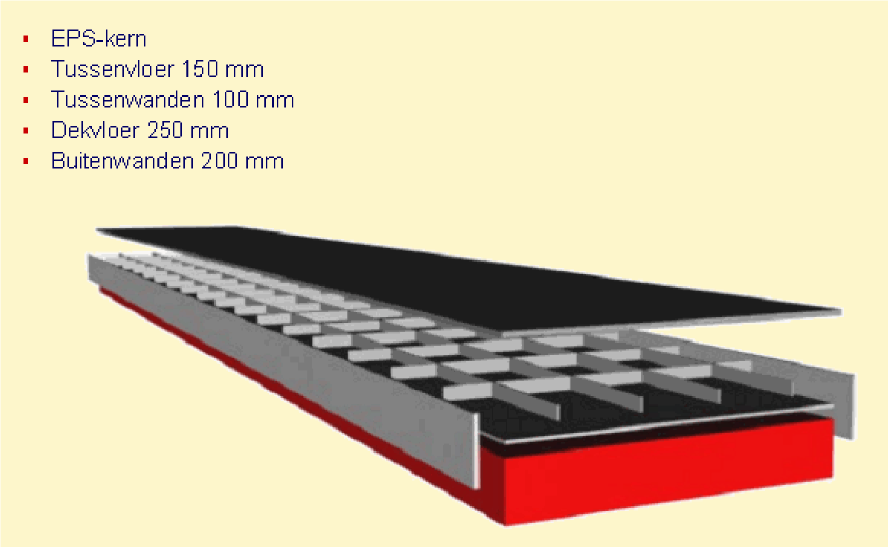


Figure 94: Floating body of Aquastrenda

The group describes their result as follows:
'Het ontwikkelde drijflichaam onzinkbaar en is er de mogelijkheid om in het drijflichaam voorzieningen ten aanzien van de koppeling en installaties aan te brengen. Het drijflichaam is opgebouwd rondom de EPS-kern die de onzinkbaarheid garandeerd, hierop is een betonnen tussenvloer (150 mm) voorzien, hierop worden tussenwanden van 100 mm dik geplaatst die 600 mm hoog zijn en daarmee de kruipruimte vormen. Op deze tussenwanden rust de betonnen dekvloer van 250 mm dik voor de drijflichamen van de infrastructuur De constructie is bijzonder stijf door de werking van de tussenwandjes in combinatie met de dek- en tussenvloer waardoor een soort van kokerprofiel ontstaat.' [Aquastrenda, 2004]

Appendix 14b: Construction methods EPS-Concrete systems

5 construction methods for EPS floating body

The floating bodies can be constructed in five different ways at five different spots
- In situ at the waterside
- In situ floating on water (Flexbase Method)
- In situ in a dock
- Prefab elements made in factory, assembly on water
- Production in big assembly hall near water.

The advantages and disadvantages of each method will be discussed below. In part 3 the best method will for the pavilion will be selected.

At the waterside

Somewhere near the project site a construction site is realised. The site is provided with a level horizontal formwork floor. The EPS blocks will be placed on this floor on. next to the side edges vertical wooden formwork is placed. These can be kept in place by straps. Where necessary reinforcement and sheaths can be placed. When this is done the concrete frame can be poured. After the concrete has hardened a crane can lift the floating body into the water or it can be pushed into the water.

Advantages
- Space for construction of large elements
- Inspection is well possible

Disadvantages
- There has to be found a suitable site on the waterfront (with additional costs)
- Outside, so conditions building conditions will not be good with bad weather (negative impact on concrete quality etc.)

On water (Flexbase Method)

This method is already explained in the last paragraph, illustrated by some pictures.
First, a layer of EPS is placed on the water , simply by sliding the EPS elements into the water. Normally these EPS elements will have a width of 1.5 meters, a length of 5 metres and a height around 20-25 centimetres. The elements are connected with each other by means of a so-called 'haaklas'. A picture of a 'haaklas' was shown in Figure 86 in appendix 14a.

On top of this layer another layer of EPS is glued, with the through going seems perpendicular to the seems of the first layer. On these two bottom layers the next layers of EPS are placed. Subsequently, the platform is made in a similar way as 'at the waterside'. This method was also used for the Floating Greenhouse constructed by Dura Vermeer/Flex Base. For the realised floating pavilion this option has also been chosen.

Advantages
- No construction site needed
- The floating body does not have to be hoisted into the water, and it does not have to be transported when it is built at the deployment location.
- Large floating bodies can be produced in one piece, since when it is produced at the destination location it does not have to be transported anymore. Then mutual connections are not necessary.
- More floats can be constructed simultaneously.

Disadvantages
- Application of the construction floor of EPS reduces the overall height of the concrete frame. Not good conditions for pouring concrete.
- Inspection of concrete construction is not good possible because the structure is already in the water.
- Applying prestressing difficult or impossible.

- The stiffness of the water is low. This ensures that the not yet hardened concrete may accumulate at certain places which results in deformation of the platform, or in the worst case failure of this foundation (can be compared to water accumulation on soft flat roofs).
- The hardening of the concrete will be problematic because the floating body and the concrete will be constantly in motion by waves, so the concrete will not achieve a good quality.
- With this method only a flat bottom is possible. There cannot be varied with depth of the float. With uneven load/structure this is unfavourable and results in tilt or large internal forces.
- Inconvenient during construction, because it's more difficult to reach certain points. Constructing on the water results in more risks and so on.

In a Dock
With construction in a floating dock, the floating body can be constructed in the same way as *in situ at the waterside*. When the construction is finished the dock will be filled with ballast water. Then the floor of the dock sinks below the water so the element starts to float. The element can then be navigated out of the dock. The complete construction including superstructure can be done in the dock.
In a dock on land the fabrication is done in the same way as in a floating dock. After the construction is finished the dock will be filled with water.

Around Rotterdam are multiple construction docks, with widths till at least 40 metres. In Barendrecht a very large dry dock is situated.

Advantages
- Large floating bodies can be produced in one whole, since the floating structures do not need to be transported over the road or be lifted into the water.
- Inspection well possible
- Good pouring conditions (a dock can be roofed, this results in a floating factory)
- Superstructure can then also be constructed inside, so no influence of bad weather.

Disadvantages
- Renting a dock is costly
- High cycle time, with production of multiple large floats
- Production can take place far form deliver location. The floating dock with significant sizes, can not operate anywhere.

Prefab
Elements with a maximum size of 12 by 3 meters, are manufactured at a concrete factory. By truck, the elements are transported to the construction site. On the water, the elements are linked together. Optionally, the pouring of the floor happen after the coupling, thus in situ in the water.

Advantages
- Not affected by weather conditions during manufacture elements for optimal concrete quality
- Inspection well possible
- Optimal working conditions
- Modular parts give more flexibility and better recyclable
- Will be most favourable with an automated production process for large production(eg Stadshavens)
- For large production, the decoupling of manufacturing and construction will also shorten the construction time

Disadvantages
- Small size of individual elements
- Many connections needed
- Transport distance can be significant.

Production in big assembly hall near water

Production in a big assembly hall will give the same construction method as construction at the waterside and in a dock. Only difference with construction at the water side is that now the construction can be done inside, so it has not to suffer from bad weather conditions.
Only difference with construction in a dock is that now the structure can not be sailed out of the production place. So it has to be shoven outside, or a crane could be use for this purpose.
At Heijplaat there are big assembly halls with doors more than 20 metres wide, and with a runway (kraanbaan) to outside.

Figure 95: Large assembly hall at Heijplaat

<u>Advantages</u>
- Ideal working and pouring conditions
- Inspection well possible
- Superstructure can then also be constructed inside, so no influence of bad weather.

<u>Disadvantages</u>
- Renting a assembly hall is costly
- High cycle time, with production of multiple large floats

Choice Construction method

If the floating body is made of high quality concrete, controlling of circumstances is 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. Beside this disadvantage, building on water has got multiple disadvantages as mentioned, so this is not a good option.

A14-12

Appendix 15:

Realised connection systems

In chapter 8 is given an overview of possible connections.
In this Appendix is given some more information about several existing connection methods.

The connection systems that will be treated in this appendix:
1. Flexifloat
2. Hann-Ocean
3. MOB
4. Improved Ribbon Bridge
5. Pontoon Connection Heijsehaven
6. Floating Road Hedel

A15-2

A15.1 Flexifloat connection system

'The Flexifloat Construction System is a combination of portable, interlocking modular steel barges and auxiliary attachments. All components of the sectional barge system are portable and are designed for road transport by standard highway trucks and trailers. Flexifloats can be quickly connected into larger assemblies of various sizes and shapes without the need for special tools or equipment. The resulting platforms are capable of supporting all types of heavy equipment loads.' [www.flexifloat.com]

Description

- rigid disconnectable system.
- consisting out of steel elements
- up and below male/female connection with vertical fastener, both at the same time fixated by hammering down top fixation pen.

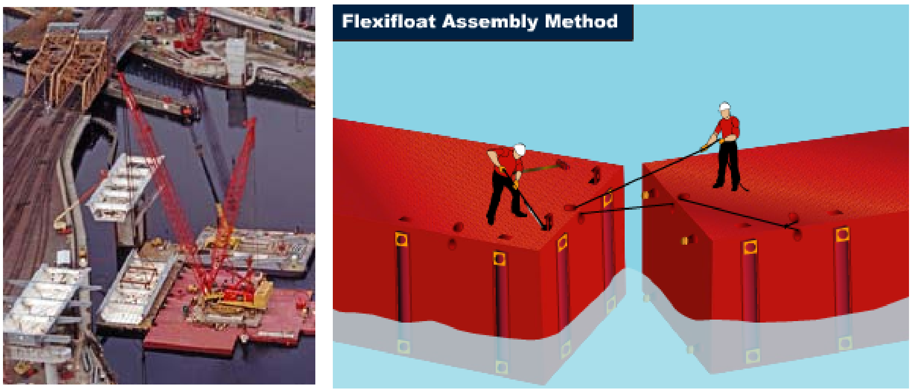


Figure 1: Flexifloat steel pontoons [www.flexifloat.com]

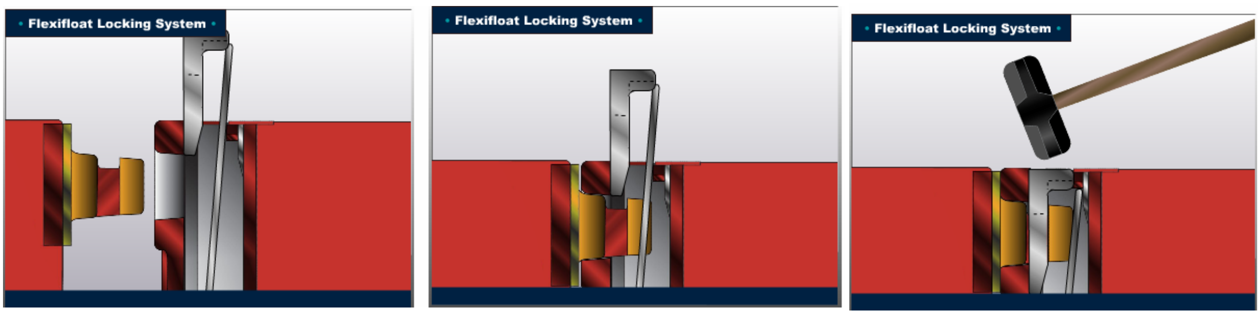


Figure 2: Connectionsystem Flexifloat [www.flexifloat.com]

Advantages

- Connection system easy in execution. For connecting in fact only one action needed: hammering vertical fastener down.

Disadvantages

- Very precise size and location tolerances and uprightness required. Coupling in wave circumstance might be difficult or impossible. The same holds for an uneven draught of the floating bodies.
- Pre-stressing not possible
- The vertical fastener is not being locked or secured: Less secure
- Large amount of steel around water surface

Additional remarks

- Applying a certain amount of prestress could be possible by applying a wedge shaped fastener.

MCA Score

This connection scores especially good on practicability. It scores bad on prestressing, since this is hardly possible. Since the vertical connector goes through the horizontal main connection part, the surface for transferring tension is relative small, this is why it only got 3 points for the criteria tension.

A15.2 Hann-Ocean connection system

The firm Hann-Ocean Technology Pte Ltd has designed an easy ‘drop-n-lock’ rigid pontoon locking system. ‘Hann-Ocean Technology is a world leader in floating platform connector technology and its related engineering solutions. Mr. Henry Han is the founder of Hann-Ocean Technology and also the inventor of Han's Fender Connector (Patent No.: 109504, 10/527,901) and Han's Frictional Locking Connector (Patent No.PCT/SG2006/000008) resolving the major challenges faced in the joining of large floating platforms in rough sea.’ [hann-ocean.com]
The Hann-Ocean Rigid Pontoon Connector connection system results in a rigid disconnectable system. This connection system is elaborated below.

Description

- Rigid disconnectable system.
- Structural part of connection consisting out of in both pontoons an slightly sloping elongated cavity (shaft) and two steel ‘puzzle pieces’ per connection
- Up and below tension connection with ‘puzzle piece’ in elongated cavity. Both can be dropped in the shaft from the top of the floating body.
- Self-alignment and impact damping by integrated rubber/plastic ridges and their contra shape.

Figure 3: Hann-Ocean rigid pontoon connector

Figure 4: Hann-Ocean Rigifloat containerized pontoons

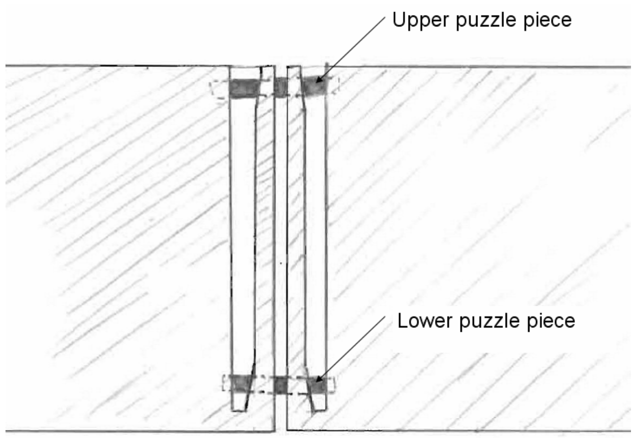


Figure 5: Sketch longitudinal section Hann-Ocean Rigifloat

Connector Properties	
Length (mm)	< 520
Width (mm)	< 250
Thickness (mm)	50
Material	Mild Steel
Weight (kg)	< 27

Table 1: Connector properties Hann-Ocean rigid connector [hann-ocean.com]

Advantages
According to hann-ocean.com (2009)
- Rigid Connection - Firm engagement with self-tensioning
- Simple yet Robust - Only one pair of movable solid keys for superior strength
- 'Drop-n-Lock' - 20 seconds to join with self-alignment feature
- Modular Interface - Easy integration with pontoons

Disadvantages
- The vertical fastener is not being locked or secured properly, while it will be pressed up by the sloping shaft edges (see Figure 5). If this is done with plastic band from below (properly the chosen solution by Hann Ocean) this has two major disadvantages: At first this will need more actions. Second: the plastic band is not very durable) Less secure
- One structural disadvantage is, that the 'puzzelpieces' are larger than for example a vertical pen, so the cavities for placing this pieces are also bigger. This means if this principle is used for concrete floating bodies, this large cavities have to be placed in the edgebeams, which will probably cause that the edgebeams have to be thickened a lot.

Additional remarks
- It is not clear how the relative heave is prevented and how the vertical shear forces are passed on in this connection. If this is only done by the small plastic ridges, than this shear capacity is very low.

MCA Score
This connection scores especially good on practicability, just as the Flexifloat connection. It scores bad on prestressing, since this is hardly possible. It scores also less good on rigidity and movements since there is no reason for the puzzle pieces to stay down and don't come up a little. if that happens small movements will be possible again.

A15.3 McDermott MOB connection system

The McDermott MOB connection system is an example of an semi hinged connection system. This connection is used for the Mobile Offshore Base mentioned in paragraph 5.2. This system is especially interesting for large structure in severe wave circumstances. More information about this connection can be found amongst others in den Vijver (2006).

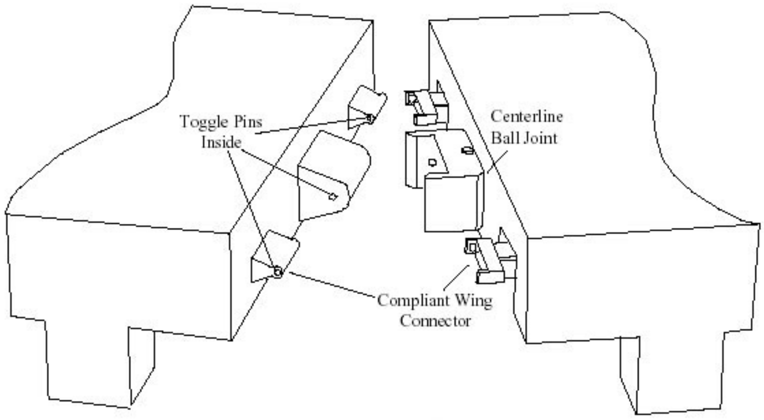


Figure 6: McDermotts Nonlinear Movement allowing MOB Connection

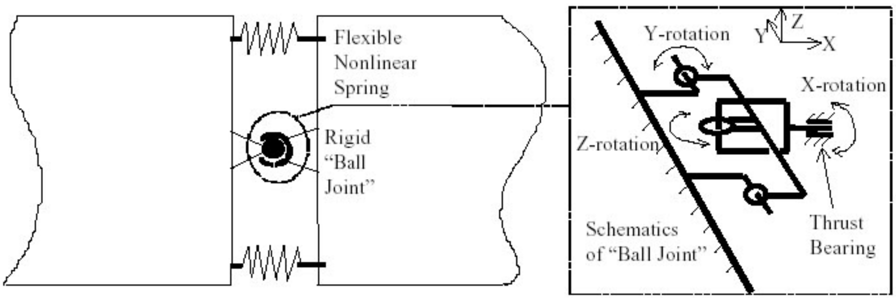


Figure 7: Schematization of McDermotts MOB connector

A15.4 Improved Ribbon Bridge

The Improved Ribbon Bridge, the mobile bridge of the United States Army, exists out of steel pontoons which will be coupled to each other. The main element of the connection is a steel hook on a cable. By winding the steel cable the gap between the steel elements is closed and the connection is tensioned.

Figure 8: Tightening connection Improved Ribbon Bridge

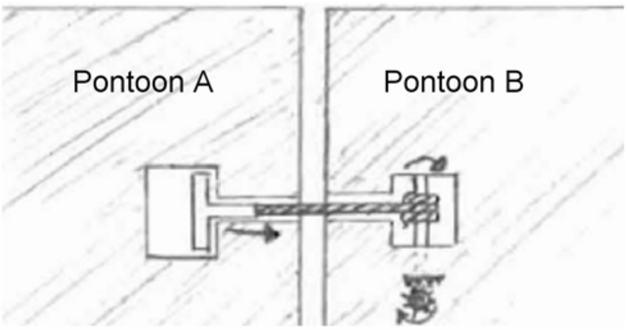


Figure 9: Sketch of top view connection improved ribbon bridge

A15.5 Pontoon Connection Heijsehaven

Steel pontoons are often used for temporary floating surfaces during construction. During construction of the floating pavilion in the Heijsehaven, there was also made use of steel pontoons. The pontoons are connected rigidly with each other. Below is made use of a double vertical pen and on the top connection is realised with a half steel arc and steel plates with vertical bolts, see

Figure 10: Picture of connection Heijsehaven

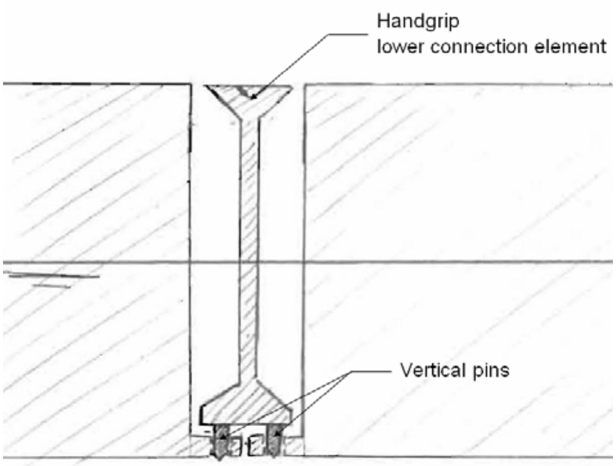


Figure 11: Pontoon connection Heijsehaven: Lower connection with vertical pins

A15.6 Floating Road Hedel

TNO, Bayard Aluminum Structures, DHV and XX-architects have developed a floating road for vehicles with a speed up to 80 km/h. This floating road which is constructed out of aluminium and EPS is quick to assemble and easy to transport. The prototype of this floating road is tested on location in Hedel, Noord Brabant. The floating road is particularly suitable for temporary diversions during maintenance of bridges or roads alongside a canal.

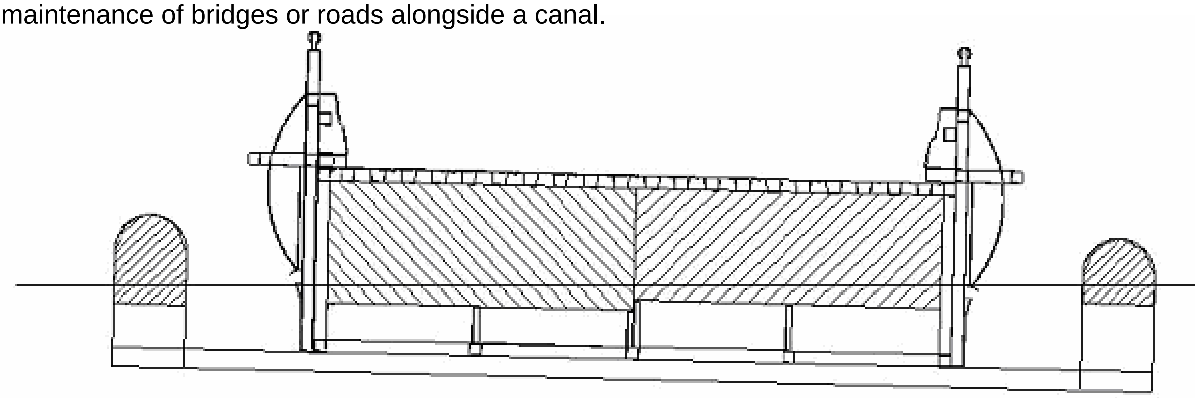


Figure 12: Cross section Floating road Hedel

The aluminium floating bodies are in longitudinal direction rigidly coupled with each other. The coupling between the longitudinal sections is provided by extruded shape- and contra shape aluminium profiles, see Figure 13. This way a flat transition is obtained between the two sections, without hindrance for the traffic. Then the connection between the two sections is fixated by a ‘claw connection’, see Figure 14. This claw connection’ is from tightened from the road surface. This results in pre-tensioning of the connection.

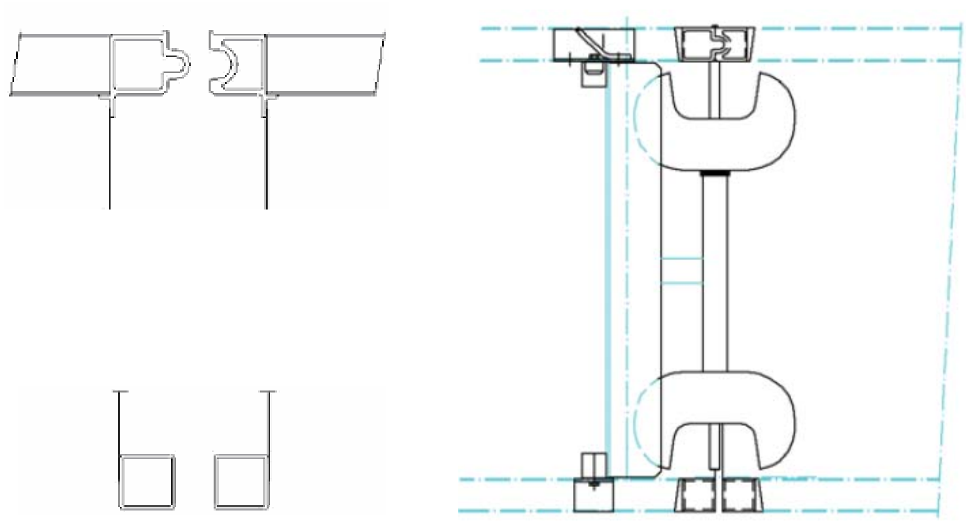


Figure 13: Longitudinal connection Floating Road. Left: shape and contra shape. Right: Clamping connection

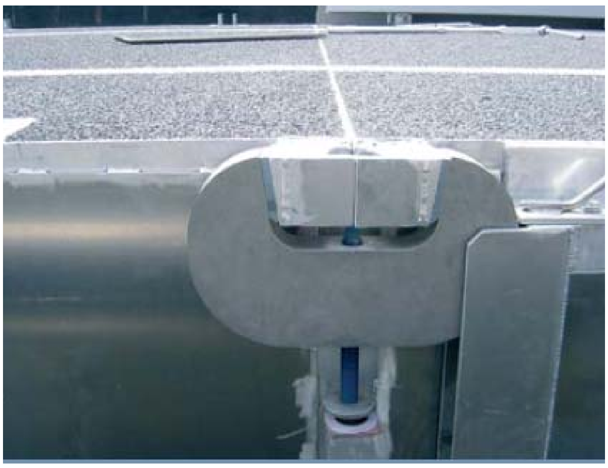


Figure 14: Rigid connection by clamping claw

[Maljaars, et al., 2005]

A15-10

Appendix 16:

Rotation causing change in offset

A16-2

Rotation causes change in offset

If two floating bodies are connected with each other by distance keepers and one of them performs a roll rotation, then the distance between the two floating bodies will get less, since the distance keepers will keep the same length. The change in intermediate distance by rotating of the distance keepers will not be linear, but parabolic. The floating bodies will be rigid, so the distance in between the floating bodies will stay linear and the bodies will not be able to follow the parabolic intermediate distance that the distance keepers want to perform. This means that if more than two distance keepers are used, the distance keepers have to perform an imposed length change. Below this is calculated and depicted in a graph for two floating bodies with a length an intermediate distance of 3 metres.

Characteristics	
2 floating bodies	
width (y-coordinate)	24 m
freeboard in rest	950 mm
initial intermediate distance	3 m
body A loaded on one side	
tilt	1 m height difference

Floating body is loaded on one side, which has caused an tilt with a height difference of 1 m. This has caused a change in intermediate distance by rotating of the distance keepers. This is shown in the graph below.

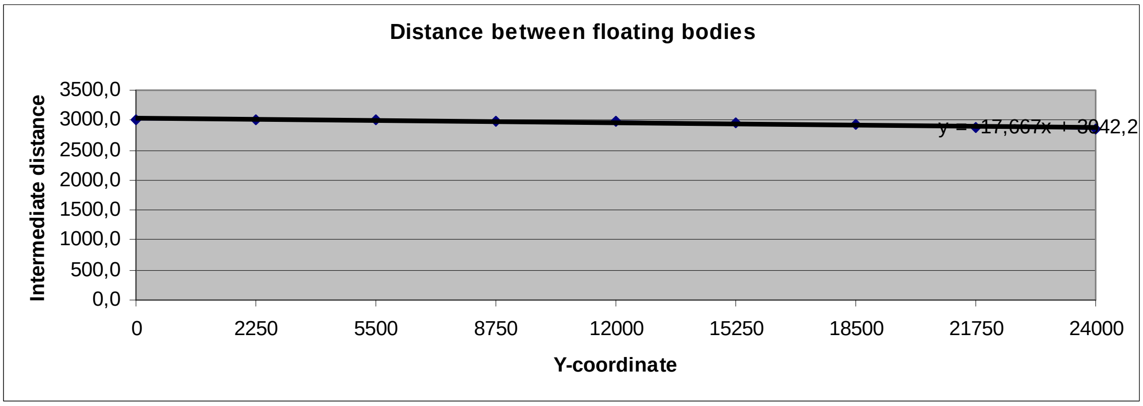


Figure 15: Distance between floating bodies by rotation

This change in intermediate distance is calculated in the table on the next page.

The change in intermediate distance by rotating of the distance keepers is calculated with the following formula:

$$d = \sqrt{l_{distance\_keeper}{}^2 - \Delta h^2}$$

With: d = intermediate distance
Δh = height difference

This outcome of this formula are given in column 6. This results in a parabolic line. This line is depicted in figure A15-2. Through this line a drawn a linear line, which depicts the intermediate distance between the two rigid floating bodies. On grid 2 and 8 the intermediate distance between the floating bodies is taken approximately the same as the length of the distance keepers. This means that if distance keepers are placed at these locations, the distance keepers don't have to perform deformations.

gridline	y-coordinate	height body A relative to watersurface	height body B relative to watersurface	height difference	intermediate distance floating bodies, wuithout deforming distance keepres	delta l	linear intermediate distance	required length distance keepers	length change	relative length change	stress
	mm	mm	mm	mm	mm	mm	mm	mm	mm	delta l/l	N/mm2
1	0	1050,0	950,0	100,0	2998,3	1,7	3017	3018,7	-18,7	-0,0062	-1305,98
2	2250	956,3	950,0	6,3	3000,0	0,0	3003,5	3003,5	-3,5	-0,0012	-245,455
3	5500	820,8	950,0	-129,2	2997,2	2,8	2984	2986,8	13,2	0,0044	924,4009
4	8750	685,4	950,0	-264,6	2988,3	11,7	2964,5	2976,3	23,7	0,0079	1660,142
5	12000	550,0	950,0	-400,0	2973,2	26,8	2945	2972,0	28,0	0,0093	1957,162
6	15250	414,6	950,0	-535,4	2951,8	48,2	2925,5	2974,1	25,9	0,0086	1813,583
7	18500	279,2	950,0	-670,8	2924,0	76,0	2906	2982,4	17,6	0,0059	1230,315
8	21750	143,8	950,0	-806,3	2889,6	110,4	2886,5	2997,0	3,0	0,0010	211,024
9	24000	50,0	950,0	-900,0	2861,8	138,2	2873	3010,7	-10,7	-0,0036	-746,844

Table 2: calculation intermediate distance by rotation

In the figure below column 6 and 8 are plotted.

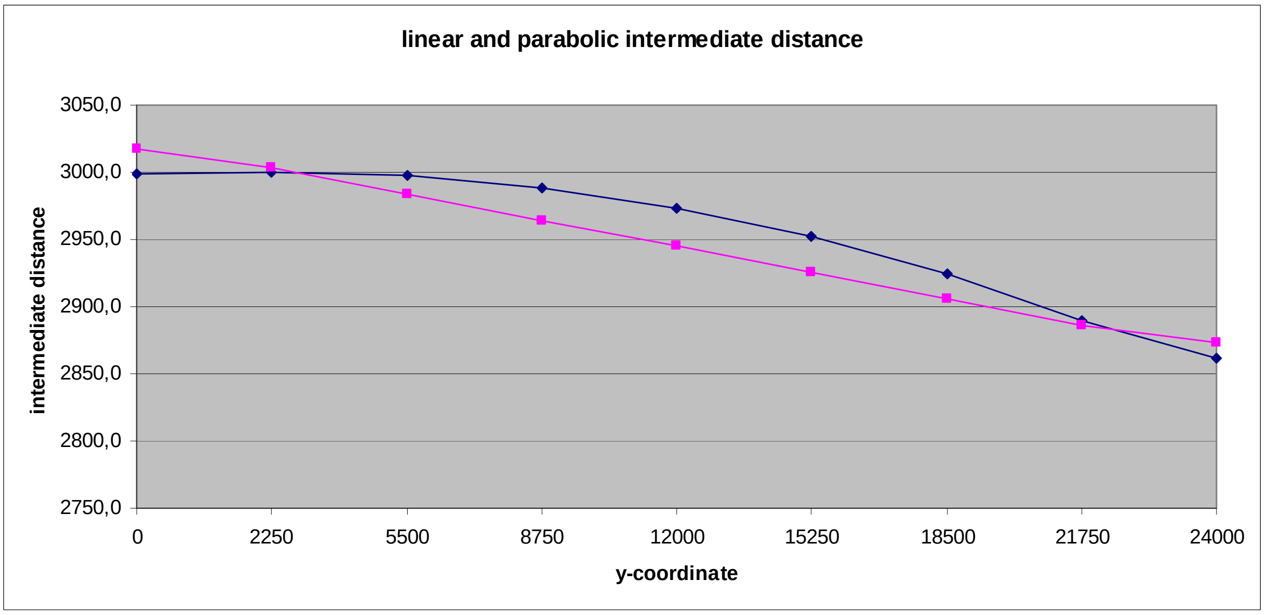


Figure 16: linear and parabolic intermediate distance

In Figure 16 can be seen that if more than 2 fixed distance keepers will be used, they have to perform an imposed length change.

Appendix 17:

Connections for intermediate distance

In this appendix the possible connections for floating structures with space in between are mentioned. They are categorized with what relative movements they allow.

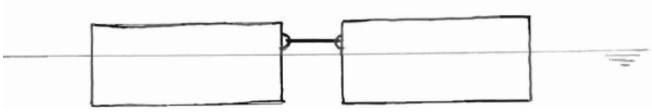


Figure 17: Floating bodies with intermediate distance

<u>Possible connections</u>

- fully flexible (non-structural)
- vertical free, allowing heave
    - o and allowing relative rotation
    - o and preventing relative pitch
    - o and preventing relative roll
- hinge connection
- rigid connection

A17.1 Fully flexible (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. This connection, is meant for transport reasons, for example of cables, ducts, electricity or a bridge for pedestrians or cars.

Figure A17-18: Floating structures with a non-structural connention

17.2 Vertical free

This connection is used when the connection must ascertain the location of a floating structure, so it needs to prevent surge and sway. This results in a ‘distance keeping’ construction with spacers.

17.2.1 Rotation allowing connection

If the connection does not prevent rotation, relative pitch and roll will occur due to unequal loads or waves. The difference in rotation can be quite large, certainly in relatively small structures.
The spacers need to have connection which will be able to follow these of rotations. This can be achieved with spherical bearings or with normal pen connections in combination with glands (glands make twisting of elongated element possible).

When more than two spacers are used, the relative roll will cause problems for the spacers, because by this rotation, the intermediate distance will change unequally with the distance the spacers want to to keep between both floating modules. This is no problem with two connections, but with more spacers this results in a forced change in length. This problem could be overcome by using pistons with springs or jacks, but this will result in considerable additional costs. (The change in intermediate distance and length change of the spacers is calculated for the connections between the floating pavilion and floating plaza. See Appendix 15 Rotation causing change in offset)

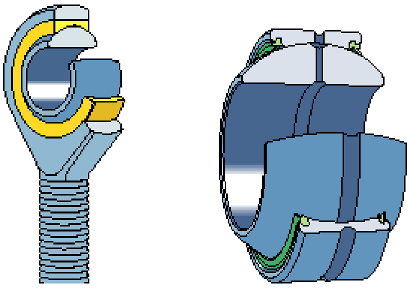


Figure A17-19: Spherical plain bearing and spherical rod bearing (www.skf.com)

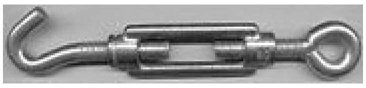


Figure A17-20: cable gland (kabelwartel)

Connection possibilities:
The three following connections are possible, based on the remaining possible basic options from chapter 10. VFRF is short for Vertical Free, Rotation Free.

- VFRF 1. Tubes and rods, with horizontal transverse bolts/pens, with fixed holes (variation on variant 1b. from par. 8.3)
- VFRF 2. Tubes and rods, with horizontal transverse bolts/pens , with vertical slots (variation between variant 1b. and 4 from par. 8.3)
- VFRF 3. Fenders and cables (variation on variant 3 from par. 8.3)

Remarks on possibilities

<u>VFRF 1. Tubes and rods, with horizontal transverse bolts/pens, with fixed holes</u>
With clamping the connection elements the sway will be prevented. This can also be realized with horizontal diagonals (see figure 6). Ofcourse without diagonals in vertical direction, because this would make it rigid in vertical direction. Spherical plain bearings or glands are necessary with this option. Except for the spherical bearings this results in a quite easy connection. With the separate parts is a lot experience in building industry etc. (This option is chosen for the case study for the connection between floating pavilion and floating plaza)

Figure A17-21: Side View VRFR 1

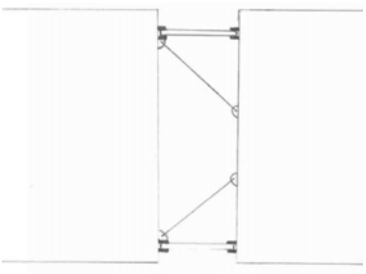


Figure A17-22: Top View VRFR1 with diagonals

Figure A17-23: Example of Mooring system with rods and cables (ABC Arkenbouw)

Alternative 7 from paragraph 8.3, the hook can be a sub-option of this alternative, which might result in slightly more easy coupling, but as said in chapter 10.2, this option is less durable and there is space enough for connecting with bolts or pens, so this option is less interesting.

VFRF 2. Tubes and rods, with horizontal transverse bolts/pens, with vertical slots
This option is almost the same as last option, but in stead of fixed holes can be made use of vertical slots.

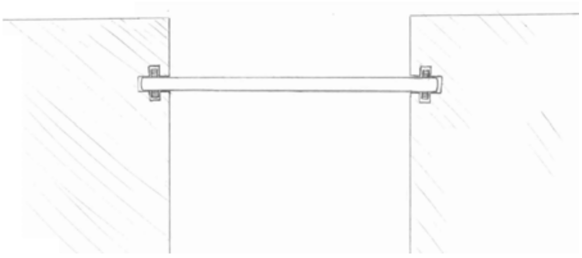


Figure A17-24: Top view option VFRF 2

VFRF 3. Fenders and cables (variation on variant 3 from par.8.3)

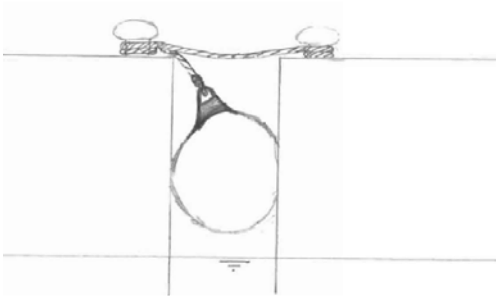


Figure A17-25: Connection with cables and fenders

Choice connection
All three options can be good options, but option VFRF1 seems most robust.

A17.2.1 Rotation restricting connection

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

Connection possibilities:
The three following connections are possible, based on the remaining possible basic options from chapter 10. VRFR is short for Vertical Free, Rotation Resistant.

- VFRR 1. Tubes and rods, with horizontal transverse bolts/pens, with vertical slots (variation on variant 1b. from par.8.3)
- VFRR 2. Bulkheads with vertical slots (variation on variant 4 from par.8.3)

Remarks on possibilities

VFRR 1. Tubes and rods, with horizontal transverse bolts/pens, with vertical slots
Now a vertical diagonal is necessary to prevent the rotation. If now in combination with this vertical diagonal fixed wholes would be used, a free vertical movement is not necessary anymore. This is why now vertical slots in stead of fixed holes have to be used.

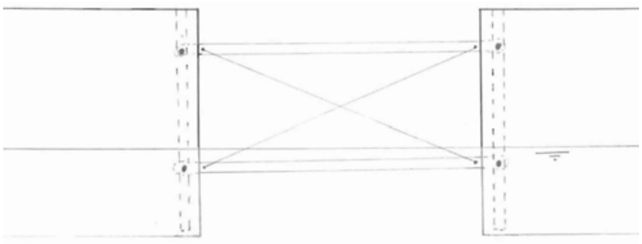


Figure A17-26: Connection VFRR 1, side view

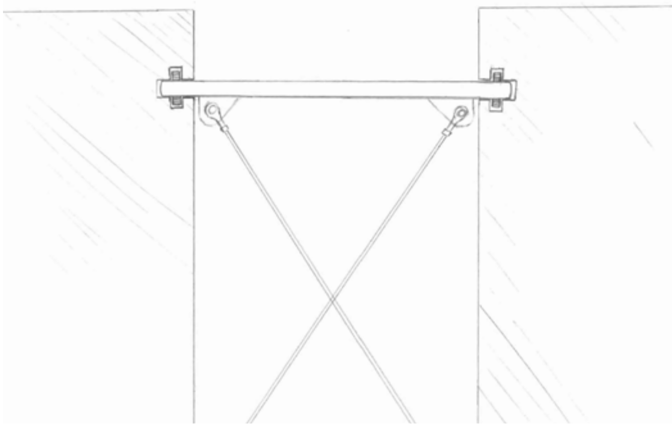


Figure A17-27: Connection VFRR 1, top view

VFRR 2. Bulkheads with vertical slots (variation on variant 4 from par.8.3)
The option with bulkheads is in fact the same as VFRR1, but with is stead of the separate rods and cables a bulkhead is used.

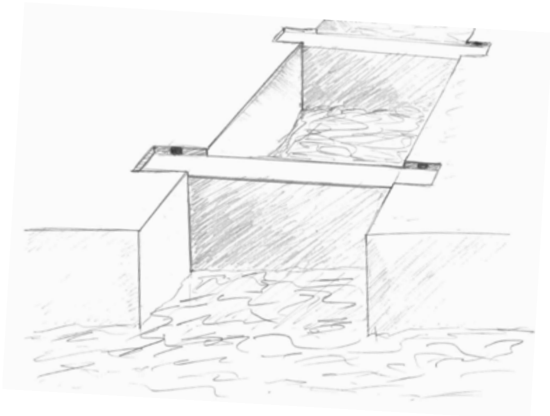


Figure A17- 28: VFRR2: Bulkheads

Choice connection
Both options are possible. But this option shall not be chosen very quickly anyway. There shall be sooner made a choice for allowing rotation or completely rigid, with also no allowing of relative heave. The only reason why this option should be choosen, is that this solution results in more stability compared to the rotation allowing option. But this solution has got quite an disadvantage compared to the rotation allowing option and a full rigid option: there will occur large torsion forces in the connectors. And with this solution the longitudinal and vertical forces and moments in the connections and beams will appear almost as big as with rigid option. So then it would not be logical to chose for this solution.

A17.3 Hinge Connection

For the hinge connection there are the following possibilities:

H1. Horizontal transverse bolt/pen connection
H2. Options with spherical bearings

These two options are in fact approximately the same, with the only difference that spherical bearings result in more degrees of freedom.

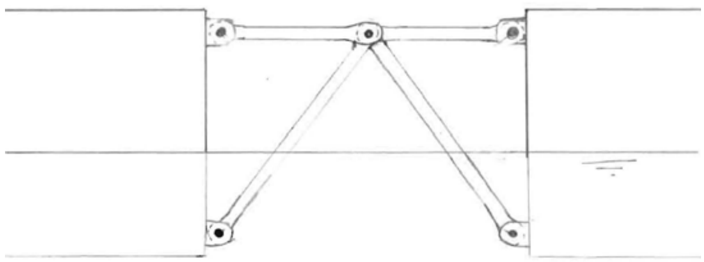


Figure A17- 29: Hinge Connection

The FDN Flexconnection and MOB options are in fact also possible (semi) hinge connections. These options are used in structures without space in between, so these options shall be elaborated in the next chapter. This is a little arbitrary, since a connection can only perform some rotation when it has some space in between, but in this situations where these examples are used the deck of the floating elements continues, so here they are categorised under the option without space in between. These options could also be used for connections without space in between however.
But in case of structures with space in between. there is enough space to place the horizontal bolt, so there for the mentioned options becomes more attractive.
When is wished for a semi-hinge connection then these other options might become interesting again, but here for is referred to appendix 17.2. The remark will be made that the MOB and MegaFloat option are used for very large floating structures and they will be less interesting for small structures.

A17.4 Rigid Connection

Also with an intermediate distance it is possible to realise a connection which prohibits vertical relative movement and roll and pitch. This might be desirable for functional reasons and it will also contribute to the stability and leads to smaller movements.

This connection can be realised in the following ways:

Connection possibilities:

RC 1. Tubes and rods, with horizontal/vertical bolts/pens, with fixed holes (variation on variant 1b. from par.8.3)
RC 2. Bulkheads (variation on variant 4 from par.8.3)
RC 3. The first two variants combined with a male/female connection (based on on variant 5 from par.8.3)

RC 1 and RC 2 can also be combined with other fastener options from 8.3, but this will lead to basicly the same results. A benefit of option RC 1. is for this option the least material is needed.

A17-8

Appendix 18:

Connections without intermediate distance

A18-2

In this appendix the possible connections for floating structures with no space in between are mentioned. They are categorized with what relative movements they allow. The disconnectable rigid connector is elaborated in chapter 10. Advantages and disadvantages are given.

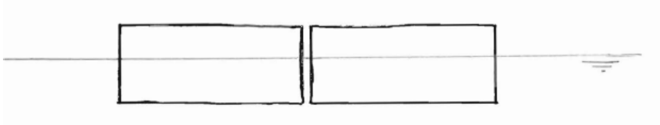


Figure 30: Floating bodies with no intermediate distance

Possible connections
- hinge connection
- semi-hinge connection, non linear
- vertical free, allowing heave
- rigid connection

When structures with no space in between have to be coupled, there can be made use of shaped edge surfaces of the floating bodies for self alignment, see chapter 9.

Vertical free

For the vertical free connection there are the following possibilities:

VF1. Puzzle
VF2. Bulkheads (puzzle pieces)
(both based on alternative 4 of paragraph 8.3)

These vertical free options are not very likely to be used. With almost the same effort a completely rigid solution can be realised. The here presented option will also not lead to large benefits for the forces compared to the rigid solution, because the relative rotation will also lead to large forces.

Hinged connection

For the hinge connection there are the following possibilities:

H1. Horizontal transverse bolt/pen connection (or spherical bearing)
H2. Options with spherical bearings

These two options are in fact approximately the same, with the only difference that spherical bearings give more degrees of freedom.

Semi-hinged connection

The FDN Flexkoppeling, MOB options are examples of semi hinge connections. The remark will be made that the MOB option is used for very large floating structures and they will be less interesting for small structures. Information about the MOB connector system is given in appendix 16.

Non-detachable rigid connection

The disconnectable rigid connection is elaborated in chapter 10.
For the non detachable solution the following extra options are also possible:

2a. Real prestressing
12. In-situ concrete
13. Welding

These options can provide very rigid connections, with a strength which can be equal or stronger as the rest of the floating body. The concrete options can be quite easy in execution.

Appendix 19:

Drawings Realised Floating Pavilion

Realised Floating Pavilion

As mentioned the geometry of the superstructure of the realised floating pavilion is the same as the geometry of the superstructure in this thesis, only the dimensions differ a little, since the calculations were done again.
The floating body has a complete different geometry. In the figures below can be seen that the beams have less height, since the body was constructed on water. The height of the beams is 750mm, the width 250mm, the c.t.c. distance is 325mm.

Cross sections

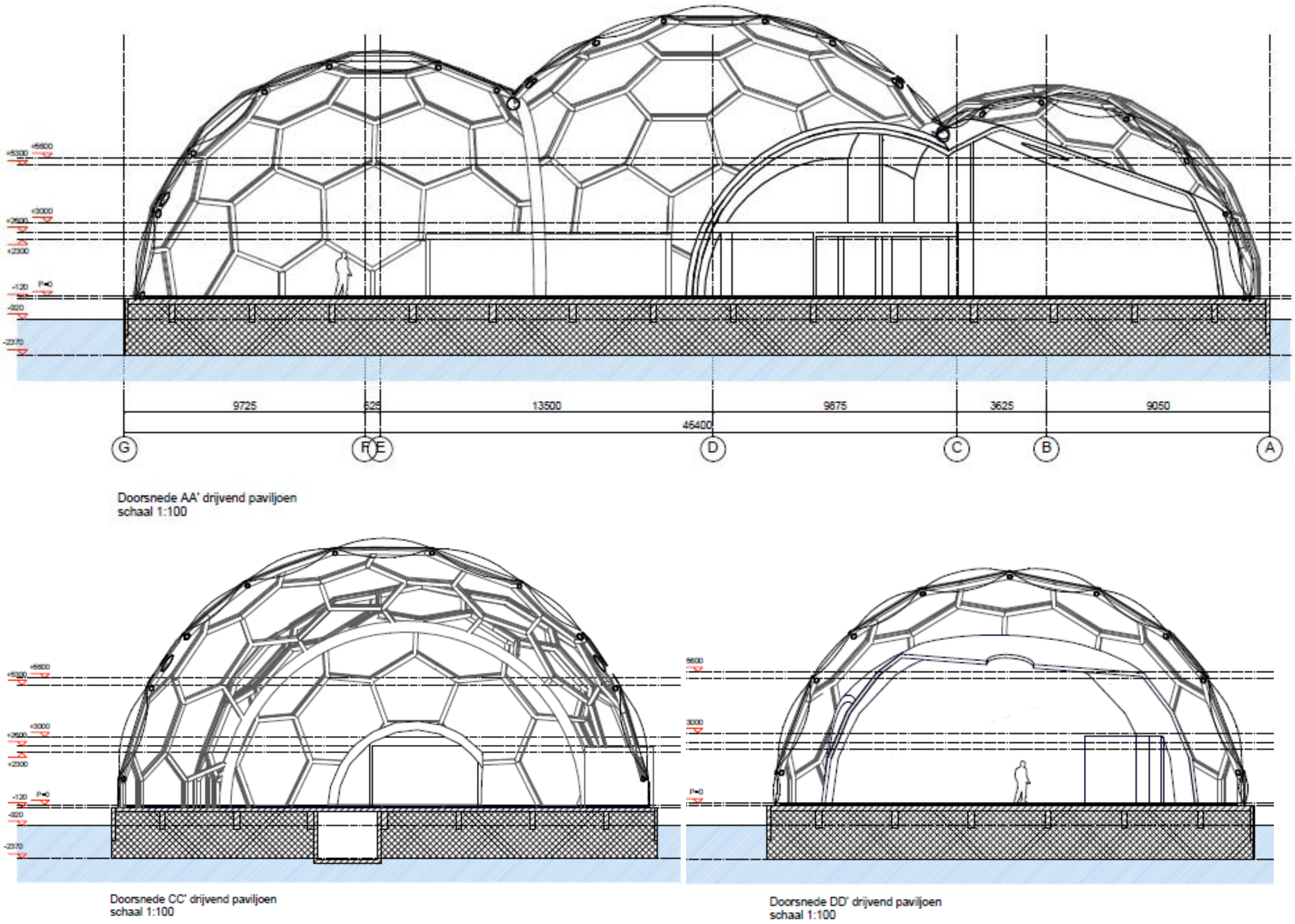


Figure A19-1: Cross sections floating pavilion [PDA,2009]

Figure A19-2: Connections superstructure

Realised beam geometry

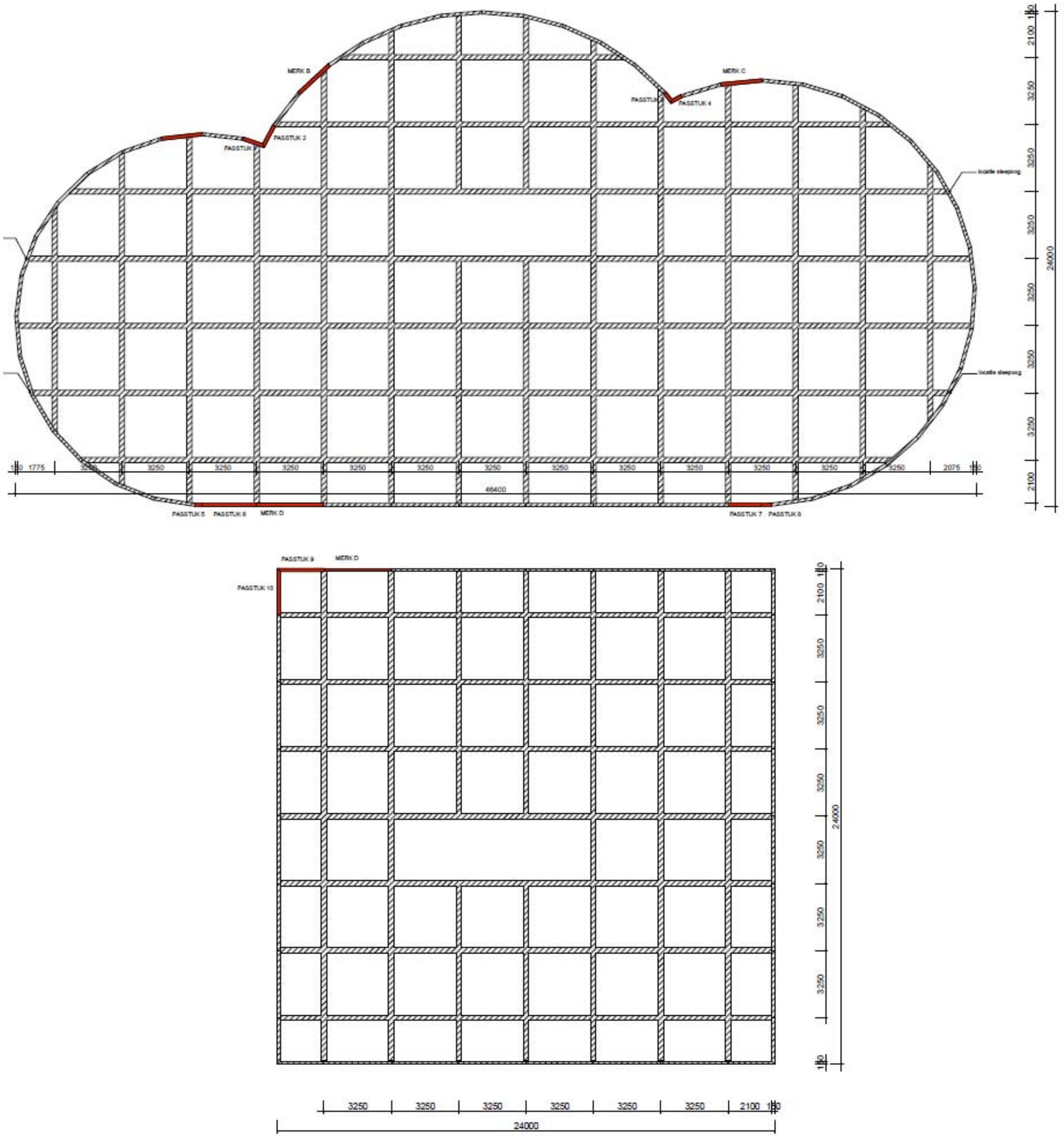


Figure A19-3: Top view beam geometry realised floating body (designed and calculated by Advin and Flexbase)

Appendix 20:

Calculation surface area

Surface area Floating body

Auditorium Koepel, Dome 1

cirkeldeel1					driehoekdeel1			Extra stukje1		
d1	18,1	m			b1	4,661095	m	b1	5,382	m
r1	9,05	m			h1	7,757364	m	h1	2,574	m
alpha1	59	grad	1,029744259	rad						
beta1	242	grad	4,22369679	rad						

Opp_cirkel1	172,9657	
Opp_driehoek1	36,15781	209,1
Opp_Extra_	6,926634 +	

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje1 **216,0501** m2

Center Dome, Dome 2

cirkeldeel2_1					driehoekdeel 2_1			Extra stukje 2_1		
d2	24	m			b2_1	9,192533	m	b2_1	4,563	m
r2	12	m			h2_1	7,713451	m	h2_1	2,574	m
alpha2_1	40	grad	0,698131701	rad						
alpha2_2	47	grad	0,820304748	rad	driehoekdeel 2_2			Extra stukje 2_2		
beta2	186	grad	3,246312409	rad	b2_2	8,18398	m	b2_1	5,382	m
					h2_2	8,776244	m	h2_1	2,34	m

Opp_cirkel2	233,7345	
Opp_driehoek2_1	70,90616	
Opp_driehoek2_2	71,82461	376,5
Opp_Extra_stuk 2_1	5,872581	
Opp_Extra_stuk 2_2	6,29694 +	12,17

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje1 **388,6348** m2

Exhibition Dome, Dome 3

cirkeldeel3					driehoekdeel3			Extra stukje3		
d3	20,7	m			b1	5,484664	m	b1	4,563	m
r3	10,35	m			h1	8,777298	m	h1	2,34	m
alpha3	58	grad	1,012290966	rad						
beta3	244	grad	4,258603375	rad						

Opp_cirkel3	228,0961	
Opp_driehoek3	48,14053	276,2
Opp_Extra3	5,33871 +	

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje1 **281,5754** m2

Total Surface Area Floating Body

Totaal Oppervlak Drijflichaam = Opp1+ Opp2 + Opp3 **886,2603** m2

Surface area internal space

Auditorium Koepel, Dome 1

cirkeldeel1					driehoekdeel1		
d1	17,1	m			b1	4,53081	m
r1	8,55	m			h1	7,250811	m
alpha1	58	grad	1,012291	rad			
beta1	244	grad	4,258603	rad			

Opp_cirkel1	155,6573	
Opp_driehoek1	32,85205	
Opp_Extra_	0	+

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje **188,5093** m2

Center Dome, Dome 2

cirkeldeel2_1					driehoekdeel 2_1		
d2	23	m			b2_1	8,809511	m
r2	11,5	m			h2_1	7,392058	m
alpha2_1	40	grad	0,698132	rad			
alpha2_2	47	grad	0,820305	rad	driehoekdeel 2_2		
beta2	186	grad	3,246312	rad	b2_2	7,842981	m
					h2_2	8,410568	m

Opp_cirkel2	214,6624	
Opp_driehoek2_1	65,12041	
Opp_driehoek2_2	65,96392	
Opp_Extra_stuk 2_1	0	
Opp_Extra_stuk 2_2	0	+

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje **345,7467** m2

Exhibition Dome, Dome 3

cirkeldeel3					driehoekdeel3		
d3	19,7	m			b1	5,219705	m
r3	9,85	m			h1	8,353274	m
alpha3	58	grad	1,012291	rad			
beta3	244	grad	4,258603	rad			

Opp_cirkel3	206,5902	
Opp_driehoek3	43,60162	
Opp_Extra3	0	+

Opp1 = Opp_cirkel1 + Opp_driehoek1+ Opp_Extra_stukje **250,1918** m2

Total Surface Internal space

Totaal Oppervlak Drijflichaam = Opp1+ Opp2 + Opp3	**784,4479**	m2	without first floor
Totaal Oppervlak Drijflichaam = Opp1+ Opp2 + Opp3	**877,9479**	m2	with first floor

A20-4

Appendix 21:

Draught calculation

A21-2

Loads								Draught		
			height (m) / Volume/m3	density (kg/m3)	g	Load in kN (/m2)	Total Load	Draught SLS (m)	Load Factor	Draught ULS (m)
Self Weight										
Floating body										
EPS Drijflichaam: 1,65m (-2% vervorming), 20kg/m3			1,65	20	9,81	0,32373 kN/m2	286,9089 kN	0,032373		
Vochtindringing, 4%*, alleen voor gedeelte onder wateroppervlak 0,04			0,04	1000	9,81	0,3924 kN/m2	347,7684 kN	0,03924		
Betonnen dek/vloer 50 mm, 2450kg/m3 ;			0,05	2450	9,81	1,201725 kN/m2	1065,041 kN	0,1201725		
Ribben onder vloer 50x250mm, h.o.h. 750										
A ribben= 0,0125 m^2, 6 ribben van 2,9 m per 9m2										
6*2,9*0,0125/9=			0,024166667	2450	9,81	0,580834 kN/m2	514,7697 kN	0,058083375		
Betonnen balken 200 tot 0 mm, gem 120mm x 16500mm, 2450kg/m3 ;										
hoogte balken		1,65 m								
breedte balken		0,12 m								
A balken= bxh		0,198 m2								
2 ribben van 3 m per 9m2, 2*3,0*A_balk/9=			0,132	2450	9,81	3,172554 kN/m2	2811,708 kN	0,3172554		
Randbalken 120mm x 1800mm, 2450kg/m3 ;										
hoogte balken		1,65 m								
breedte balken		0,12 m								
A balken= bxh		0,198 m2								
1m per 10m2, A_balk/10=			0,0198	2450	9,81	0,475883 kN/m2	421,7562 kN	0,04758831		
Evt. Trimgewichten, beton gem 10 mm			0,01	2450	9,81	0,240345 kN/m2	213,0082 kN	0,0240345		
Total floating body						**6,387471 kN/m2**	**5660,96** kN	**0,638747085**	**1,2**	**0,766496502**
Cellar+Installations										
Afmetingen kelder (uitwendig) = 9,45 x 2,95 x 2,0 m^3 (l x b x h) (met aan alle zijden 2,5 cm speling)										
wanden										
dikte wanden		0,12								
lengte wanden	lange wand = 9,45 - 0,175	9,275								
	korte wand = 3,0 - 0,175 = 2,825	2,775								
hoogte wanden:h_drijflichaam – vloer drijflichaam – vloer kelde		1,85								
volume:		(9,325 x 2 + 2,825 x 2) x 1,85 x 0.12 = 5,39 m^3								
gewicht:		volume	5,39	2450	9,81	129,546 kN				
vloer										
dikte vloer	= 0,15m	0,15								
oppervlakte	= lengte x breedte	27,878								
volume:	= opp x dikte vloer	4,1816								
gewicht:	volume x 2450 kg/m3		4,181625	2450	9,81	100,5033 kN				
Total cellar:						230,0492 kN				
Installaties	(afkomstig uit berekening Advin)					80 kN				
Total Cellar+Installations						**310,0492 kN**	**310,05 kN**	0,03498409	1,2	0,041980909
Floor										
- Afwerking vloer										
- isolatielaag										
- betonnen afwerkvloer 5 cm beton			0,05	2450	9,81	1,201725 kN/m2				
- Polyurethaanvloer										
together:						**1,25 kN/m2**	**1107,825 kN**	0,125	1,2	0,15
Dome Structure										
- Staalconstructie						257,63				
- ETFE folie										
- Plafonds + Installaties aan gevel						226,3				
Total dome structure						**483,93 kN**	**483,93 kN**	0,054603615	1,2	0,065524338
Built-in Structure										
- Wandjes + Stalen en Houten constructie verdieping						1,5 kN/m2				
Oppervlakte inbouw:	lengte	11								
	breedte gem.	8,5								
	oppervlakte	93,5					**140,25 kN**	0,015824927	1,2	0,018989913
Installations first floor										
- Installaties verdieping						**48 kN/m^2**	**48 kN**	0,005416018	1,2	0,006499221
Total Self Weight							**7751 kN**	**0,874575736**	**1,2**	**1,04949088**
Imposed Loads										
Veranderlijke belasting personen etc.										
- Opgelegde Belasting gebruiksklasse C3		5 kN								
	Momentaanfactor	0,25				**1,25 kN/m2**	**1224,7 kN**	**0,138187439**	**1,5**	**0,207281159**
External Actions										
Wind										
Only large upward forces		600 kN upwards								
Snow										
Uniform Snowload							359 **kN**		1,5	
Unbalanced Snowload X/Y							**477 kN**	**0,053821678**	**1,5**	**0,080732516**
Unbalanced Snowload Roof/Valley							410 **kN**		1,5	
Combinations								Diepgang BGT	Belastir	Diepgang UGT
Total SW + Imposed							**8975,715 kN**	**1,012763175**	**1,2 en 1**	**1,256772042**
Total SW + Snow							**8228,015 kN**	**0,928397413**	**1,2 en 1**	**1,130223399**
Total SW + Imposed + Snow							**9452,715 kN**	**1,066584853**	**1,2 en 1**	**1,337504558**

A22-1

Appendix 22:

Imposed loads

A22-3

This appendix is still in Dutch:

Gebuiksklassen
De middenkoepel (koepel 2) en de tentoonstellingskoepel (koepel 3) dienen volgens Eurocode 1991-1-1 ingedeeld te worden in gebruiksklasse C3; ruimten zonder obstakels voor rondlopende mensen, bijv. ruimten in musea, tentoonstellingsruimten enz. (zie Table 1: Opgelegde belastingen volgens table 6.2 of EC1991-1-1) De auditoriumkoepel kan volgens Eurocode 1991-1-1 ingedeeld worden in gebruiksklasse C2. Maar in de toekomst kan het paviljoen mogelijk anders ingericht worden. Hierom zal voor het hele paviljoen met de hoge gebruiksbelasting van klasse C3 gerekend worden.

<u>Belasting bij gebruiksklassen</u>
De belasting behorend bij deze gebruiksklassen is gegeven in table 6.2 uit Eurocode 1991-1-1, zie Table 1.

Table 1: Opgelegde belastingen volgens table 6.2 of EC1991-1-1

Klasse van belaste oppervlakte	q_k kN/m^2	Q_k kN
Klasse A (wonen en huishoudelijk gebruik)		
A-vloeren	1,75 [a]	3 [a]
A-trappen	2,0	3
A-balkons	2,5	3
Klasse B (kantoorruimten)		
B-kantoorruimten	2,5	3
Klasse van belaste oppervlakte	q_k kN/m^2	Q_k kN
Klasse C (bijeenkomstruimten)		
C1-tafels	4,0	7
C2-vaste zitplaatsen	4,0	7
C3-zonder obstakels voor rondlopende mensen	5,0	7
C4-fysieke activiteiten	5,0	7
C5-grote mensenmassa's	5,0	7
Klasse D (winkelruimten)		
D1-kleinhandel	4,0	7
D2-warenhuizen	4,0	7
[a] Deze waarden moeten ook zijn gebruikt voor constructies van ondergeschikte betekenis.		

Momentaanfactoren
De vloeren en balken die direct belast worden moeten de opgegeven maximale belasting zonder meer kunnen dragen. Maar bij elementen die belast worden door belasting van meerdere vloeren (bijvoorbeeld de onderste kolommen in een flatgebouw) of grotere velden mag de belasting verminderd worden, omdat nooit alle vloervelden tegelijk maximaal belast zullen worden. Hiervoor worden momentaanfactoren gebruikt.
Dus wanneer men bij het paviljoen wil kijken naar het gedrag van de gehele constructie samen, bijvoorbeeld voor zakking of scheefstand, mag de gegeven maximale belasting vermenigvuldigd worden met een momentaanfactor. De gehele belasting zal alleen meegenomen worden over een bepaald gebied waarvan de grote afhangt van het gebruik. De momentaanfactoren staan gegeven in onderstaande table.

Action	ψ_0	ψ_1	ψ_2
Imposed loads on buildings, category			
Category A: domestic, residential areas	0,4	0,5	0,3
Category B: office areas	0,5	0,5	0,3
Category C: congregation areas	0,25	0,7	0,6
Category D: shopping areas	0,4	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area, vehicle weight ≤ 30 kN	0,7	0,7	0,6
Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,7	0,5	0,3
Category H: roofs	0	0	0
Snow loads	0	0,2	0
Wind loads	0	0,2	0
Temperature (non-fire)	0	0,5	0

Table 2: Values for reductionfactors, Dutch National Annex Eurocode 1991

φ_0 is de combinatie reductie factor, dus dit is de momentaanfactor die gebruikt moet in combinatie worden met andere lasten, bijvoorbeeld met een deel van de vloer volledig geladen en de rest geladen met de reductiefactor.
Bij gebruiksklasse C φ_0 = 0,25.

<u>Oppervlakte maximale belasting</u>
In totaal zullen er maximaal 250 personen tegelijk toegelaten worden in het paviljoen. (Zie Programma van Eisen)
Met deze maximaal hoeveelheid personen, zal door personen de gegeven belasting nooit gehaald worden over het gehele paviljoen.

Voor gebruiksklasse C3 is een maximum opgegeven van 5 kN/m^2. Deze 5 kN/m^2 kan door 250 personen maar op 40m^2 behaald worden met 6,25 personen per vierkante meter (als wordt uitgegaan van gemiddeld 80kg, = 0,8kN per persoon). De overige oppervlakte is dan volledig vrij van personen.

Dat al deze 250 personen zich op 40m^2 bevinden is niet waarschijnlijk, onder andere omdat een dichtheid van 6,25 personen normaliter niet haalbaar is. Maar onder deze hoge piekbelasting kunnen ook andere zaken dan personen vallen.
Er wordt hier nu van uit gegaan dat de maximale belasting kan bestaan uit 5 personen per vierkante meter (nog steeds erg hoog) en dan nog 1 kN/m2 door andere zaken. Op deze manier wordt de 5kN/m^2 gehaald. Als er nu van uit wordt gegaan dat 225 personen bij elkaar zullen staan, resulteert dit op deze manier op 45m^2 maximale belasting. 45m2 komt overeen met 5 belaste vloer velden van 3x3 meter.
Het deel van de bezoekers dat ergens anders staat valt onder de belasting op het overige oppervlak die vermenigmuldigt is met de momentaanfactor.

De gegeven maximale belasting van 5 kN/m^2 zal dus over 45 vierkante meter meegenomen.
Over de rest van de vloeroppervlakte zal de belasting maal de momentaanfactor gezet worden:

$$5 \times 0{,}25 = 1{,}25 \text{ kN/m}^2.$$

De maximale belasting zal op verschillende plekken gezet worden, zodat het ponton zo ongunstig mogelijk (bijvoorbeeld voor scheefstand of krachten in de balken) belast wordt.

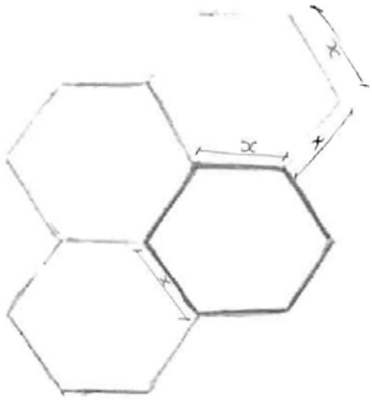
Deze 45 vierkante meter kan dus ook over meerdere oppervlaktes verdeeld worden.
Een andere ongunstige situatie waar mee gerekend zal worden is dat de maximum belasting op twee verschillende plekken zal werken. De oppervlaktes waarop de maximale belasting zal werken, zal dan wel kleiner zijn. Hiervoor wordt een oppervlakte gesteld van 2x25m^2.

Appendix 23:

Loads on superstructure

A23.1 Dividing surface loads as line loads

The loads on the superstructure act as surface load on the cushions. These surface loads has to be translated to lineloads on the structural beams, so the these lineloads can be inserted on the Scia model. Here to has to be calculated what surface belongs to the beam. This is done as showed below:



On the left a top view of the hexagonal configuration is depicted, all beams have the same length x.
To find the adjacent area which belongs to the beams the hexagonals are divided in triangles. This can be seen in Figure 1.

$$h = \cos 30° \cdot x = 0{,}86x$$

$$A_{adjacent,tot} = 2 \cdot A_{\Delta}$$
$$A_{adjacent,tot} = 2 \cdot \tfrac{1}{2} h \cdot x = 0{,}86x^2$$

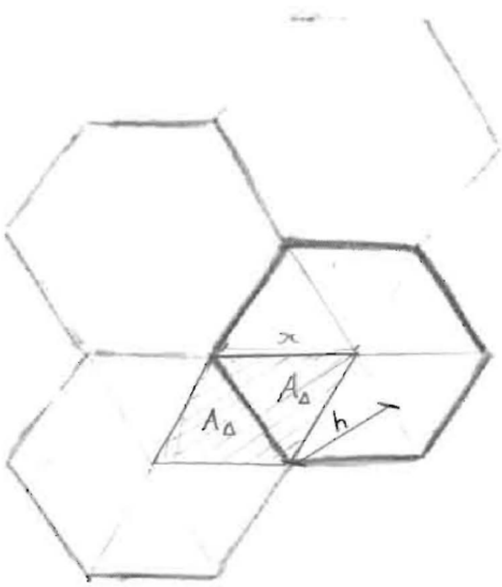


Figure 1: Top view hexagonal beam structure with adjacent surface Area

A23.2 Snow load

The snow loads are determined according to NEN 6702: 2007, Chapter 8.7.

Snow load on the roof: $p_{rep} = p_{sn,rep} \cdot C_i$

With: $p_{sn,rep}$ = Snow load on the ground = 0.70 kN/m2 (NEN6702)

C_i = values for the snow load can be found in the figures below

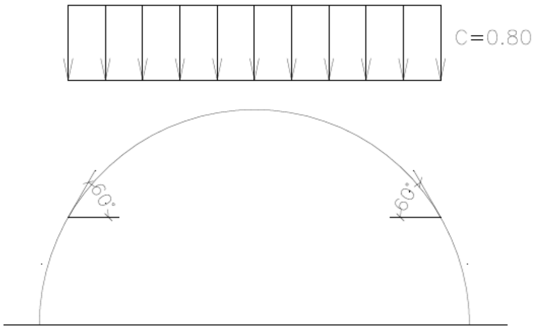


Figure 2: Uniform snow load

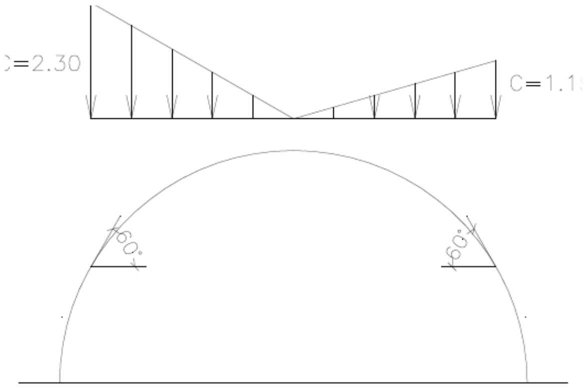


Figure 3: Unbalanced snow load

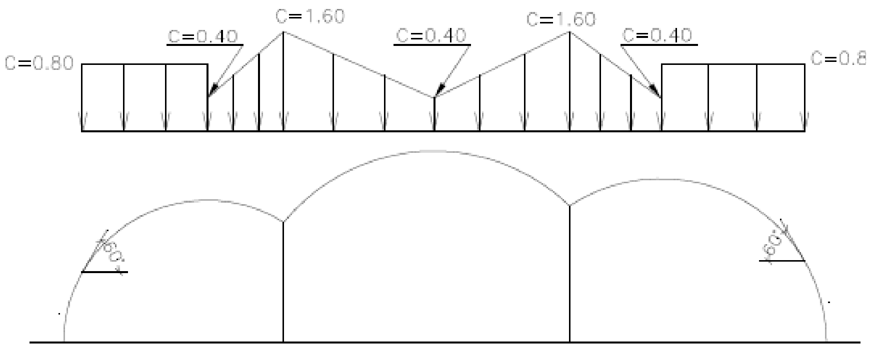


Figure 4: Unbalanced snow load roof/valley

For the uniform snow load, the C_i value equals 0,8 as given in Figure 2. This results in:

$$p_{rep} = p_{sn,rep} \cdot C_i$$

$$p_{rep} = 0{,}7 \cdot 0{,}8 = 0{,}56 kN / m^2$$

The value of p_{rep} is a surface load and this surface load has to be translated to a line on the structural beams. In A23.1 was mentioned how this should be done. These findings are used, but for the snow load it is applied a little different, because all the multiple different snow loads will be taken together as one snow load. Below is briefly explained how this is done.

<u>All load cases in one</u>
First is taken a look at the beams which do not lie close to the section lines of the domes, eg. the beams on top. As calculated in A23.1 all beams in a hexagonal configuration have got a adjacent surface of 0,86 l^2. And with p_{rep} calculated the line load can be calculated. With a beam length of 2m this gives:

$$A_{adjacent} = 0{,}86 \cdot 2^2 = 3{,}44 m^2$$

$$q = \frac{p \cdot A_{adjacent}}{l}: \qquad q = \frac{0{,}56 \cdot 3{,}44}{2} = 0{,}96 \approx 1{,}0 kN/m^2$$

So this means that for the level hexagonals the surface load of 0,56 on the cushions equals a line load of approximately 1kN/m on every beam.
Where the domes become steeper, there is less flat horizontal surface relative to the beams. So here the line load could be taken lower. But in the unbalanced case the C_i values become higher at the steeper parts, so therefore the snow load becomes larger in these areas. So therefore at the sides is also taken a value of 1 kN/m.
In the valleys between the domes the density of beams is larger, so if on every beam is put the line load of 1 kN/m this also results in larger snow loads which corresponds with the higher valley load. So with this schematising of 1kN/m on the beams all possible snow loads are taken in account at the same time.

In the figure below the schematised snow load is given, with 1kN/m in z direction on every beam on the highest part of the dome.

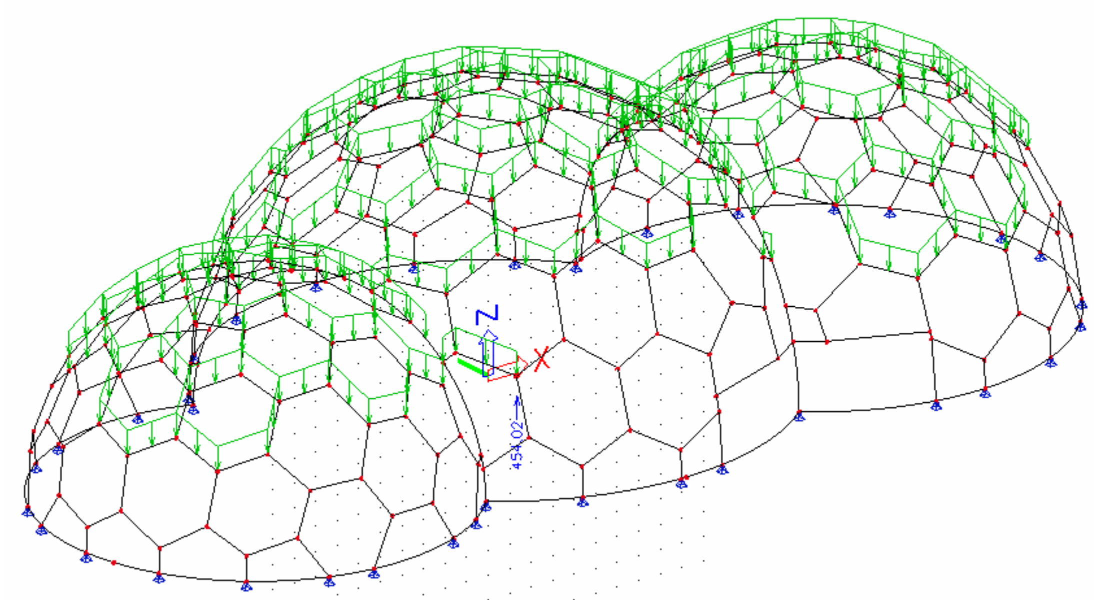


Figure 5: Snow load on domes

The total resulting vertical force of this snow load is 454,02 kN.

<u>Check schematization snow load</u>
Check with resulting force: The total surface of the domes is 784m^2 (see appendix..). Uniform snow load on a level surface of 784 m^2 gives a total load of 784*0,8*0,7 = 439,04 kN. The resulting load of the schematised snow load as given in Figure 5 gives a load of 454,02 kN which is higher than 439,04 kN, so this schematization is on the safe side.

A23.3 Wind load

For calculating the wind pressure is made use of EN1991-1-4, chapter 5, 6 and 7. All given formula's are from those parts.
According to EN1991-1-4, H5.2, the total wind pressure can be found by summing the internal, external pressure and the friction. For this thesis the internal pressure is not taken into account, because this will very low and not relevant. The friction component is for the domes already taken in account in the calculations for the outer pressure because it is accounted in the coefficients. The wind pressure on the external surfaces on height z can be calculated with the following formula:

Wind pressure: $w_e = q_p(z) \cdot c_{pe}$

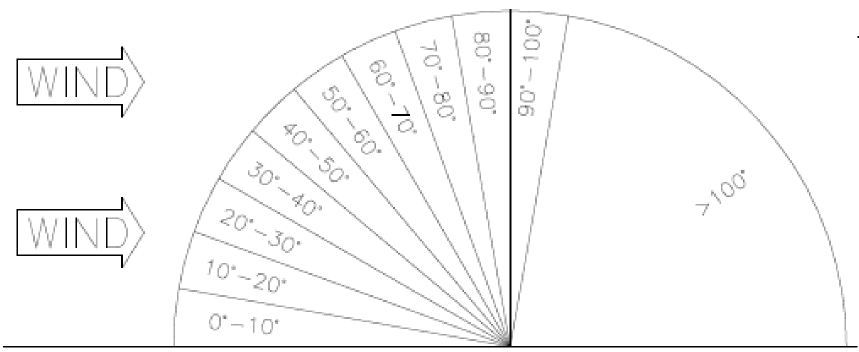
With: $q_p(z)$ = the peak velocity pressure, in this case 0,9 (wind area II, open terrain area, height of 12 meters, see paragraph 14.5)
c_{pe} = the pressure coefficient for the external pressure.

The c_{pe} are calculated with the following formula:

$$c_{pe} = c_{p,0} \cdot \psi_{\lambda\alpha}$$

With: $\psi_{\lambda\alpha}$ = end effect factor
$c_{p,0}$ = constants which can be read out of the table below

The values of $c_{p,0}$ and $\psi_{\lambda\alpha}$ are given in paragraph 7.9 of EN1991-1-4 and are also depicted in the table below.



α	$c_{p,0}$	$\psi_{\lambda a}$	c_p
0–10	0,97	1,00	0,97
10–20	0,76	1,00	0,76
20–30	0,38	1,00	0,38
30–40	–0,12	1,00	–0,12
40–50	–0,64	1,00	–0,64
50–60	–1,09	1,00	–1,09
60–70	–1,39	1,00	–1,39
70–80	–1,50	1,00	–1,50
80–90	–1,41	0,95	–1,34
90–100	–1,15	0,83	–0,95
>100	–0,80	0,66	–0,53

Figure 6: Angles in a dome with corresponding c- and lambda values

For wind in positive x-direction is shown how this works:

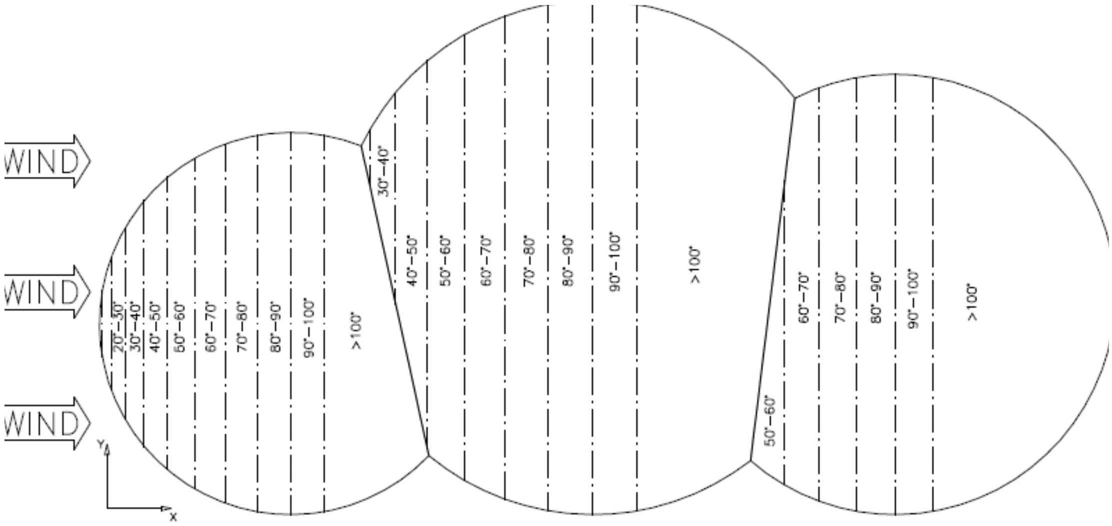


Figure 7: Wind in x-direction

With the given formulas and values from the table, now for every part of the surface and for every direction the wind pressure can be calculated.

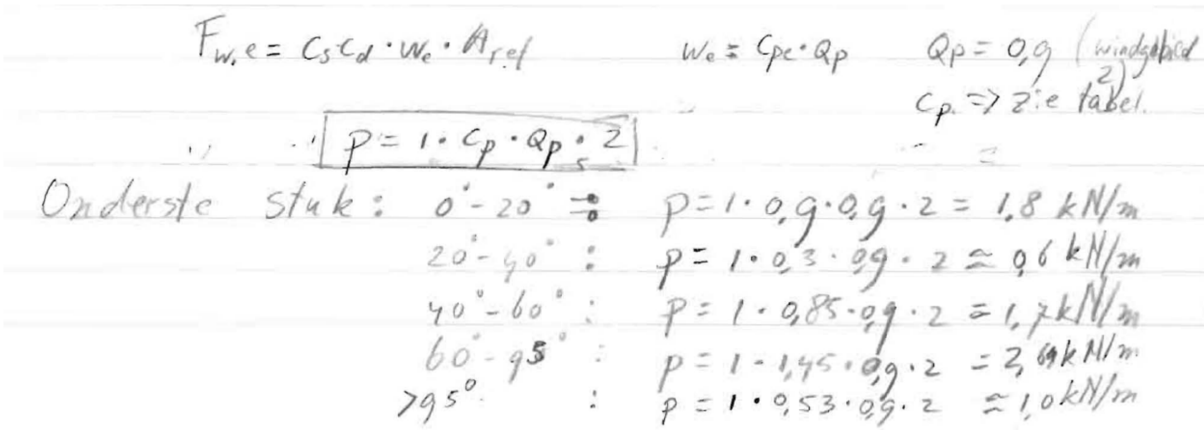
Subsequently, the wind force F_w acting on a structure, or a structural component can than be determined by using the following expression:

Wind force: $F_w = c_s c_d \cdot \sum w_e \cdot A_{ref}$

With : $c_s c_d$ = the structural factor, which may be taken 1 for structures lower than 15 meters

A_{ref} = reference area of the structure or structural element

On the model line loads have to be inserted in stead of forces, so A_{ref} has to be divided by the beam length. This gives a facor of approximately 2 (A23.1 and A23.2). In the hand calculations below the line loads are calculated for the different heights.



$F_{w,e} = c_s c_d \cdot w_e \cdot A_{ref}$ $w_e = c_{pe} \cdot Q_p$ $Q_p = 0,9$ (windgebied 2)

$c_p \Rightarrow$ zie tabel.

$\boxed{p = 1 \cdot c_p \cdot Q_p \cdot 2}$

Onderste stuk: 0°-20° : $p = 1 \cdot 0,9 \cdot 0,9 \cdot 2 = 1,8$ kN/m

20°-40° : $p = 1 \cdot 0,3 \cdot 0,9 \cdot 2 \simeq 0,6$ kN/m

40°-60° : $p = 1 \cdot 0,85 \cdot 0,9 \cdot 2 = 1,7$ kN/m

60°-95° : $p = 1 \cdot 1,45 \cdot 0,9 \cdot 2 = 2,6$ kN/m

>95° : $p = 1 \cdot 0,53 \cdot 0,9 \cdot 2 \simeq 1,0$ kN/m

Figure 8: Hand calculation line load

The wind load is brought on the structure in Scia. For every height the wind pressure load is calculated with the formula's given in this paragraph and than this wind pressure is divided over the structural beams and inserted as line loads as mentioned in A23.1. The outcome for wind in the y-direction is depicted below:

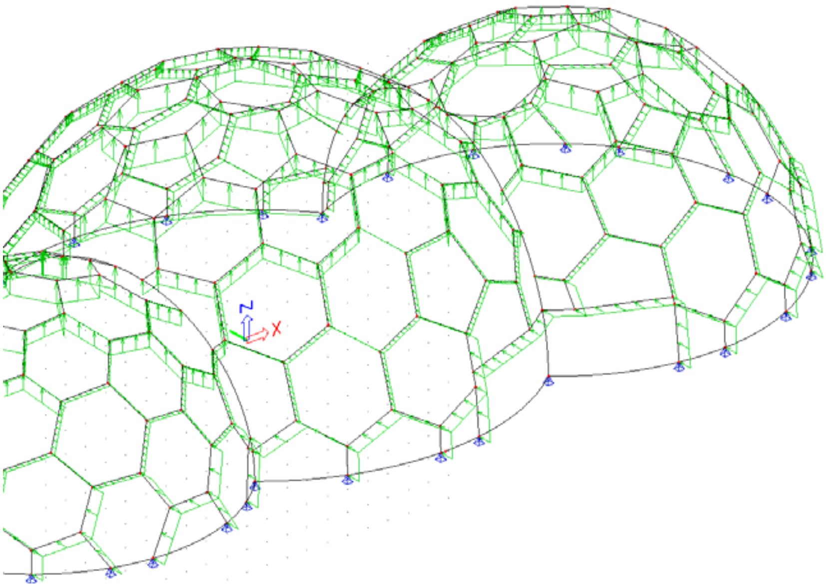


Figure 9: Domes with wind load (Scia)

Appendix 24:

Calculation Superstructure

A24-2

Material

The superstructures exists completely out of Circular Hollow Section, steel class S355.

Steel:		S355		
Tensile strength		$f_{t;p;d}$	490	N/mm2
Yield strength		$f_{y;p;d}$	355	N/mm2
Allowable shear strength	=f_tbd/√3	$\tau_{t;p;d}$	282,9	N/mm2
Shear yield strength	=f_ybd/√3	$\tau_{y;p;d}$	205,0	N/mm2

Loads

See appendix 23, Loads on superstructure.

Maximum internal forces

With Scia all internal forces have been calculated (in chapter 15 the model was described). The maximum internal forces are given in the table below.

		SW+permanent load UGT		**SW+perm+snow UGT**	
Beams, dome1	M_y	25,5	kNm	36,32	kNm
	M_z	27,77	kNm	33,21	kNm
	N	-55,58	kN	-116,31	kN
	V	26	kN	36,26	kN
Beams, dome2	M_y	18,71	kNm	25,85	kNm
	M_z	16,63	kNm	31,33	kNm
	N	-39,69	kN	-68,08	kN
	V	29,95	kN	52,3	kN
Beams, dome3	M_y	7,34	kNm	23,6	kNm
	M_z	-5,14	kNm	18,3	kNm
	N	-13,63	kN	-44,63	kN
	V	8,41	kN	24,12	kN
Arch	M_y	19,9	kNm	40,31	kNm
	M_z	27,9	kNm	37,48	kNm
	N	-87,47	kN	-173,31	kN
	V	29,95	kN	45,22	kN

Table 3: Maximum internal forces from Scia

The normative load case for almost every beam is the loadcase SW + permanent load + snow.

Verification normative beams

Here the calculation for the normative beam is shown. As said, per dome is taken one profile only, so the most heavy loaded beam will be normative. The elements of the auditorium dome are loaded most heavily, because there the inner structure is hanging on the roof.
The calculation shown here is done for the beams which are most heavily loaded with the dimensions which sufficed.

Check on Buckling

In the calculation for the stresses is taken a buckling length equal to the system length, see table 8 of part III.

Normal force		N_d	116,3	kN
Outer diameter		d_{CHS}	193,7	mm
outer radius		r_u	96,85	mm
wall thickness		t	6	mm
inner radius		r_i	90,85	mm
Surface area	$= (\pi r_u)^2 - (\pi r_i)^2$	A_{CHS}	3538,1	mm2
Moment of Inertia	$= \frac{1}{4}\pi(r_u - r_i)^4$	I_y	15597230,8	mm4
polar inertia radius	$= \sqrt{\frac{I}{A}}$	i	66,4	mm
slenderness	$= \frac{l_{buc}}{i}$	λ	30,1	
specific slenderness	$= \pi\sqrt{\frac{E}{f_y}}$	λ_e	76,4	
relative slenderness	$= \frac{\lambda}{\lambda_e}$	λ_{rel}	0,39	

With the calculated relative slenderness, the buckling factor can be determined with the buckling curves given in NEN and Eurocodes. Curve A can be used. From curve A follows that a relative slenderness of 0,39 results in a buckling factor of 0,94.

Verification: $$\frac{N_{c;s;d}}{\varpi_{buc} N_{c;el;d}} \leq 1$$

$$N_{c;el;d} = A_{pen} \cdot f_y : \qquad N_{c;el;d} = 3538{,}1 \cdot 355 = 1256{,}0 \cdot 10^3\, N$$

Check: $$\frac{116{,}3}{0{,}94 \cdot 1256{,}0} = 0{,}10$$

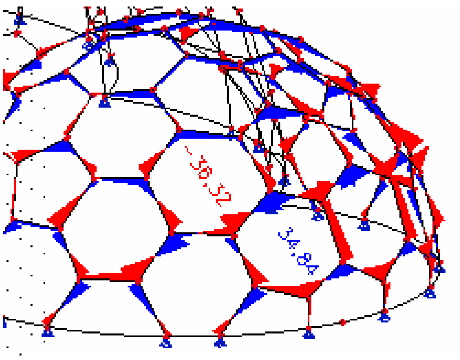
Check on moment

In order to consider the limitation of the stresses at the welding seams, the stress ratio of the cross sections is limited to 0.80. This was not necessary for buckling since in that case the middle of the beams are normative, but here the edges are normative.

$$\sigma_{m,y,d} = \frac{M_{y,d} \cdot r}{I_y}$$ [1]

$$\sigma_{m,y,d} = \frac{36{,}3 \cdot 10^6 \cdot 96{,}85}{13{,}2 \cdot 10^6} = 266{,}4 N/mm^2$$

Check: $$\frac{\sigma_{m,y,d}}{0{,}8 \cdot f_{y;s;d}} = \frac{225{,}5}{0{,}8 \cdot 355} = 0{,}79$$



[1] In Dutch notation $M_y = M_{zz}$, $I_y = I_{zz}$,

Check on shear
The shear stress is calculated with the following formula:

$$\tau_{1,\max,d} = \frac{\pi}{2} \cdot \frac{V_d}{A_{CHS}}$$

$$\tau_{1,\max,d} = \frac{\pi}{2} \cdot \frac{36{,}26 \cdot 10^3}{3538{,}1} = 16{,}1 N/mm^2$$

Check: $\frac{\tau_{1,\max,d}}{0{,}8 \cdot \tau_{y;b;d}} = \frac{15{,}4}{0{,}8 \cdot 355/\sqrt{3}} = 0{,}057$

Check on combination
For the check on combination element S315 is taken, see figure. Element S315 is the beam loaded by the largest moment and so this beam appeared also normative for combination. The internal forces of this beam are given below:

Internal forces normative beam:S315		
My	36,32	kNm
Mz	18,32	kNm
N	-57,94	kN
Vy	23,63	kN
Vz	13,1	kN

Table 4: Internal forces beam S315

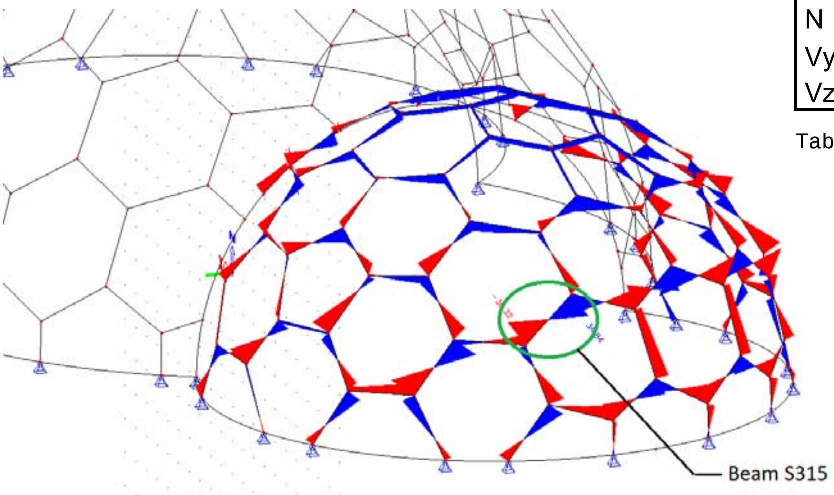
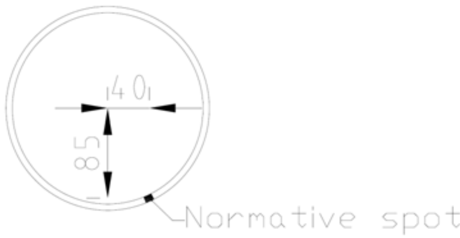


Figure 10: Moment in beams auditorium dome, S315=Normative beam



check stress combination:

stress by N: $\sigma_{s,n,d} = \frac{N_d}{A}$

$$\sigma_{s,n,d} = \frac{-57{,}94 \cdot 10^3}{3538{,}1} = -32{,}9 N/mm^2$$

stress by M_y: $\sigma_{M_y,d} = \frac{M_{y,d} \cdot e_z}{I_y}$

$$\sigma_{M_y,d} = \frac{36{,}32 \cdot 10^6 \cdot -85}{15{,}6 \cdot 10^6} = -197{,}9 N/mm^2$$

stress by M_z: $\sigma_{M_z,d} = \frac{M_{z,d} \cdot e_y}{I_z}$

$$\sigma_{M_z,d} = \frac{18{,}32 \cdot 10^6 \cdot -40}{15{,}6 \cdot 10^6} = -47{,}0 N/mm^2$$

stess by V_y $$\tau_{y,\max,d} = \frac{V_{y,d} S_y}{2 \cdot t \cdot I_z}$$

$$\tau_{y,\max,d} = \frac{23{,}6 \cdot 10^3 \cdot (\frac{1}{3} A \cdot (40 + 36{,}7))}{2 \cdot 6 \cdot 15{,}6 \cdot 10^6} = 11{,}5 N/mm^2$$

stess by V_z $$\tau_{z,\max,d} = \frac{V_{z,d} S_z}{2 \cdot t \cdot I_y}$$

$$\tau_{z,\max,d} = \frac{13{,}1 \cdot 10^3 \cdot (\frac{2}{3} A \cdot (85 + 5{,}9))}{2 \cdot 6 \cdot 15{,}6 \cdot 10^6} = 3{,}8 N/mm^2$$

check combination stress: $\sqrt{(\sigma_{n,d} + \sigma_{My,d} + \sigma_{Mz,d})^2 + 3\tau_{y,\max,d}{}^2 + 3\tau_{z,\max,d}{}^2} \leq f_{y;s;d}$

$$\sqrt{(-197{,}9 - 47{,}0 - 32{,}9)^2 + 3 \cdot 11{,}5^2 + 3 \cdot 3{,}8^2} = 278{,}8 \ N/mm^2$$

Check: $\frac{\sigma_{compare,s,d}}{f_{y;s;d}} = \frac{278{,}8}{0{,}8 \cdot 355} = 0{,}98$

Deformation
The maximum deformation occurs with snow load. According to the Scia model the maximum deformation is 34,2 mm in the arch between dome 2 and 3. This will not result in any problems.

Choice Elements

Now it has checked that a CHS with dimensions 193,7/6,0 suffices. For the center dome will be taken domes with the same dimensions, since the moments appeared normative for the dome elements and the center domes is loaded with moments of the same range. For the exhibition dome is chosen for elements with a smaller wall thickness. For the arches is taken a somewhat larger profile.

Appendix 26:

Scia Input

A26-2

In this appendix the input for the Scia model is given.
The program used is Nemetschek Scia Engineer, release 2009, version 9.0.325.

A26.1 Rectangular Floating Body

Structure

Below is mentioned how the structure is modelled in Scia.

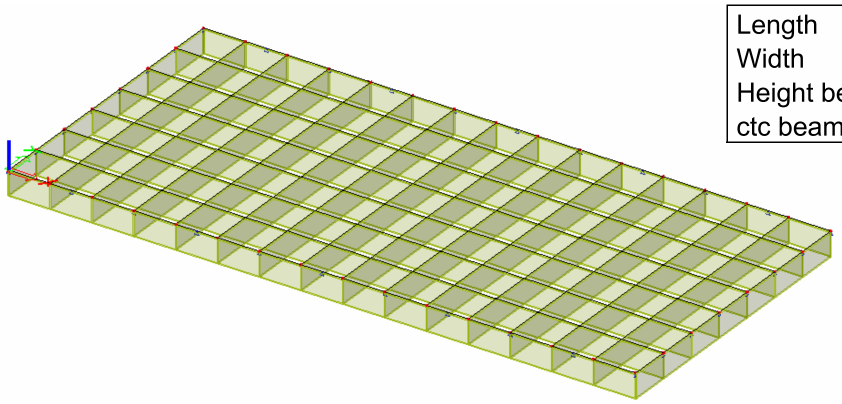
- The geometry depicted in **Fout! Verwijzingsbron niet gevonden.** was used. This beam grid and floor are converted as a 2D plate with beams.
- The water is schematized as an elastic support with a spring constant of 10kN/m2. In chapter 4 has already been checked that Scia give the correct results for rotation with this method.
- The elastic support acts directly on the concrete floor, which is schematised is 2D plate, with a thickness of 100mm. In real, the water pressure is also passed on to the floor via the EPS. This mean the beams mainly hang on the floor.
- The net for calculation of the 2D elements has a grid of 1m.
- The beams are schematised as normal straight ribs with dimensions of 1650x100mm^2.
- The effective width appeared very important in the Scia calculation for the moments and normal forces in the beams and in the floor. The effective width is been put on 'standard', because this resulted in the largest forces and the least forces in the floor, which means the beams itself will take the largest part of the moments, which is expected to be the case in the real situation, because the beams are much more rigid.
  (The input screens for the plate and ribs are depicted in Figure 13 and Figure 14.

Load

- The self weight has been automatically generated
- Next to the self weight also a surface load of 3 kN/m^2 has been added. This has been inserted as surface load over the complete 2D-slab
- Wave and imposed loading has also been inserted as surface load on the 2D-slab. This is done with the magnitude of the load and the surface area as mentioned in chapter 16.

Calculation

- The internal forces are calculated with linear calculations



Length	45	m
Width	21	m
Height beams	1,65	m
ctc beams	3	m

Figure 11: Rectangular floating body; 21x45 m2 (Scia)

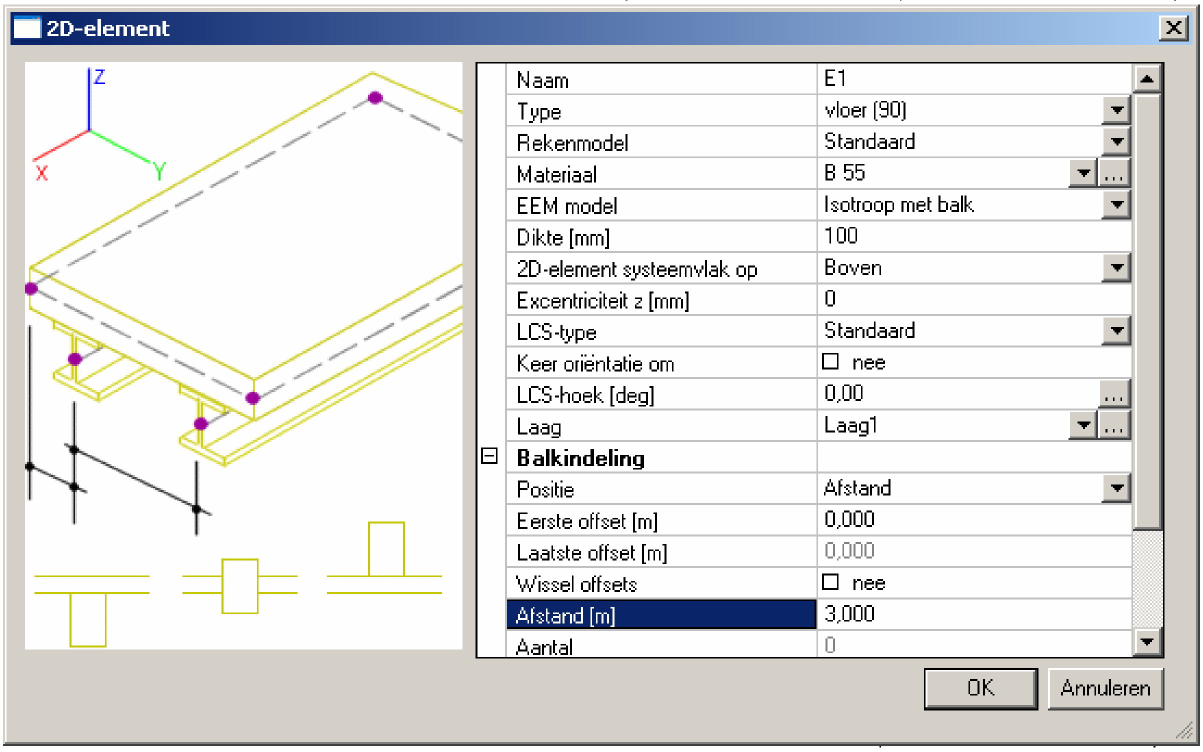


Figure 12: Entry screen plate with ribs (1)

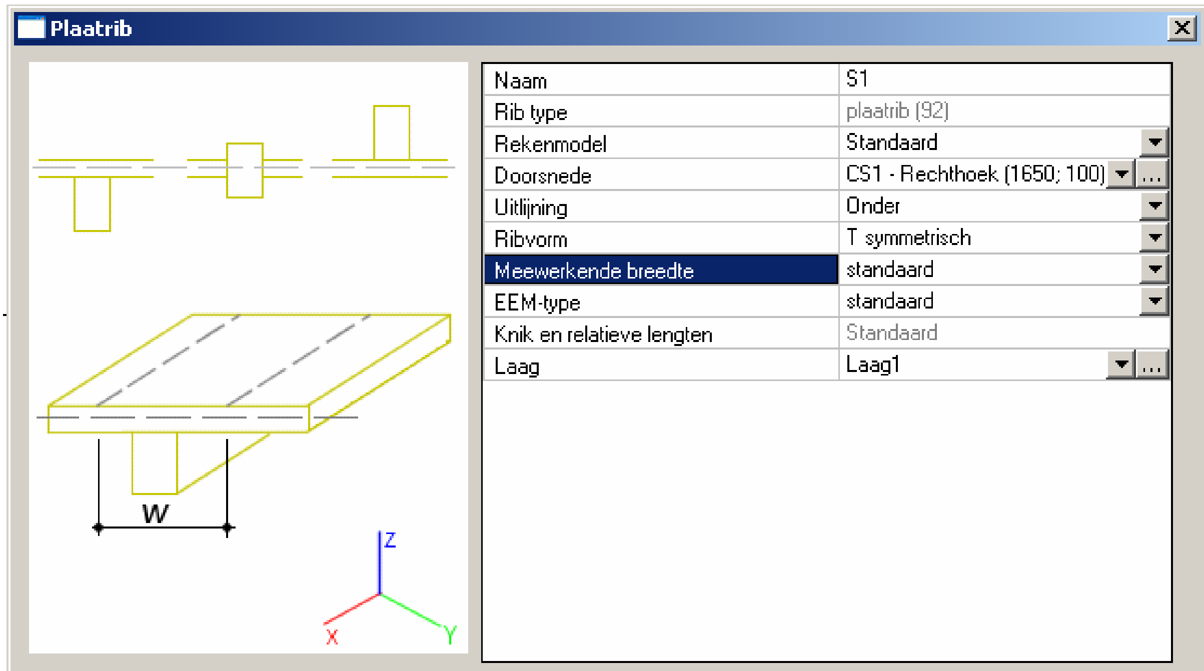


Figure 13: Entry screen ribs (2)

A26.2 Scia model floating pavilion

Structure

- The top view of the floating body depicted in **Fout! Verwijzingsbron niet gevonden.** in chapter 14 drawn is used. This beam grid and floor are converted as a 2D plate with beams.
- The water is schematized as an elastic support with a spring constant of 10kN/m2. In chapter 4 has already been checked that Scia give the correct results for rotation with this method.
- The elastic support acts directly on the concrete floor, which is schematised is 2D plate, with a thickness of 100mm. In real, the water pressure is also passed on to the floor via the EPS. This mean the beams mainly hang on the floor.
- The beams are schematised as normal straight ribs with dimensions of 1650x100mm^2.
- The effective width appeared important in the Scia calculation for the moments and normal forces in the beams and in the floor. The effective width is been put on ‘standard’, because this resulted in the largest forces and the least forces in the floor, which means the beams itself will take the largest part of the moments, which is expected to be the case in the real situation, because the beams are much more rigid.
- The dome elements have the earlier chosen dimensions
- The lowest beams of the dome structure are in the same level as the plate
- The cellar has not yet been modelled in the structure. The place where the cellar can best be situated is appointed after the stresses and deformation had been viewed. The load of the cellar and its installation are inserted at the location where the cellar is chosen.
- The built-in structures are not modelled because these are not important for this thesis. There has been made an estimation of the loads of these structures and these are inserted as line loads (1,5 kN/m) on the plate.
- The result is depicted in Figure 14.

Loads

- Self weight and equal imposed loading are taken in account
- The self weight has been automatically generated
- The internal structures, as walls etc., have been inserted as line loads
- All load situations on the superstructure are taken in account
- In this chapter wave loading and loading by unequal imposed loads is not yet taken in account. this is done in the next chapter.

Calculation

- The internal forces are calculated with linear calculations

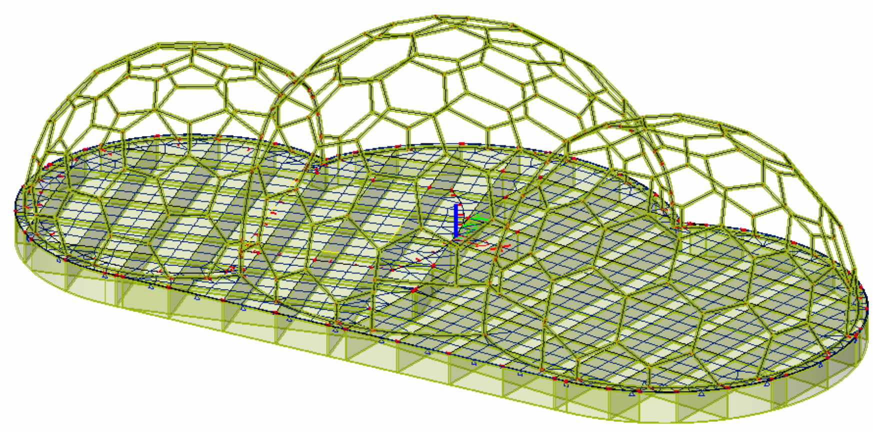


Figure 14: Image of the in Scia constructed complete structure

A26-6

Appendix 27:

Hand calculation

A27-2

A27.1 Check resulting moment Imposed Load

To see if Scia gives the correct outcome, a hand calculation is done for the load situation depicted in Figure 15.

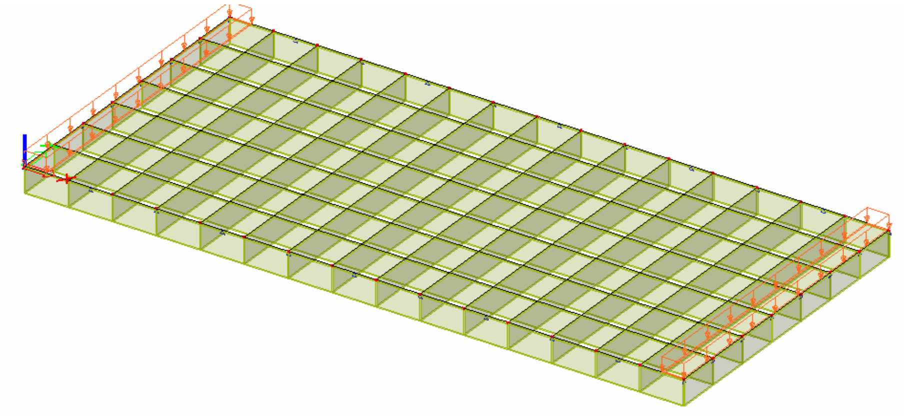
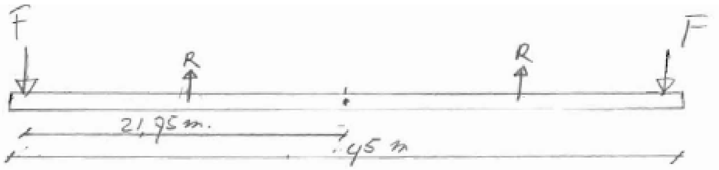


Figure 15: Rectangular floating body, loaded with 5kN/m^2 over 2x 1,5x21m, on the edges

The hogging moment by the load divided over both sides at the edge, is checked with a hand calculation. This is done according to the method sketched in paragraph 2.3. Here for the calculation is simplified by assuming the floating body is completely rigid.
For calculation the next shown schematisation is used:



$$F = 1{,}5m \times 5kN/m^2 \times ctc\_beams = 1{,}5 \cdot 5 \cdot 3 = 22{,}5kN$$

The depicted R's are the resultants of increased vertical water pressure:

$$R = F$$

The maximum moment in the middle can be calculated as follows:

$$M = -21{,}75 \cdot F + \tfrac{1}{2} \cdot 21{,}75 \cdot R = \tfrac{1}{2} \cdot 21{,}75 \cdot 22{,}5 = -244{,}68kNm$$

This calculated value is a less than the value which followed from Scia 182,45kNm. This is the result of, that the program Scia divides the internal moment over both the beam and the plate, this remark is already made before, but now will be calculated if adding them will result in the correct result.

Raising moments of Scia with normal force
Scia divides the internal moment over both the beam and the plate. In both the beam and the plate is an opposite directed normal force, which together also make a moment. So if the moment calculated by Scia is summed with the moment caused by the normal force this should result in the real moment. The moment by the normal force is found by multiplying it by half the beam height:

$$M_{by\_N} = N \cdot \tfrac{1}{2} \cdot h_{beam} = -42{,}23kN \cdot \tfrac{1}{2} \cdot 1{,}65m = -34{,}84kNm$$

For this situation this leads to the following total moment:

$$M_{tot} = -182{,}45kNm - 42{,}23kN \cdot \tfrac{1}{2} \cdot 1{,}65m = -217{,}23kNm$$

This is again a little lower than the calculated value of -244,68 kNm. This difference can explained by the fact that the body is in fact not completely stiff, so the resulting forces of the elastic support will be located nearer to the resulting forces of the acting imposed load, this results in a lower moment.
So the summed moment based on the values of Scia are a little lower than the calculated value with the simplified hand calculation. The total moment based on the values of Scia will be taken as the real internal moment, and will be used for the checks.

A27.1 Check resulting moment Wave Load

The resulting moment by the sagging wave is checked with a hand calculation. The figure below is used for calculating the occurring moment:

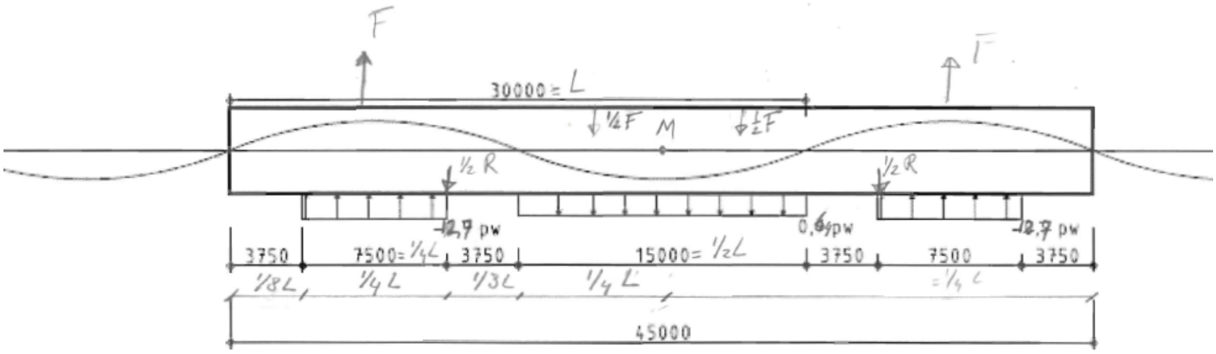


Figure 16: Sagging wave on floating body, with resulting forces of water pressure

Here F is the resulting force of the uppointing wave pressure:

$$F = \tfrac{1}{4} L \cdot p \cdot ctc\_beams = \tfrac{1}{4} L \cdot p \cdot 3 = \tfrac{3}{4} pL$$

With: $p =$ uppointing surface load by wave = $9{,}59 kN / m^2$ (see 2 pages ago)
$L =$ Length wave = 30 m

Again the depicted R's are the resultants of increased vertical water pressure: $R = F$

The maximum moment in the middle can be calculated as follows:

$$M = \tfrac{1}{2} L \cdot F - \tfrac{1}{8} L \cdot \tfrac{1}{2} F - \tfrac{1}{2} \cdot \tfrac{3}{4} L \cdot \tfrac{1}{2} R = \tfrac{8}{16} FL - \tfrac{4}{16} FL = \tfrac{1}{4} FL$$

With substituting F, this results in the following:

$$M = \tfrac{1}{4} (\tfrac{3}{4} pL) L = \tfrac{3}{16} pL^2$$

By filling in p and L the moment is known:

$$M = \tfrac{3}{16} 9{,}59 \cdot 30^2 = 1618 kNm$$

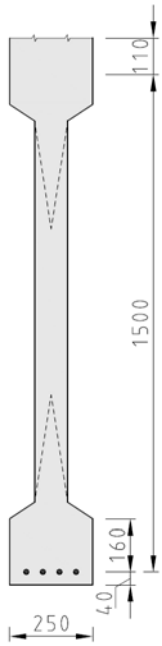
This calculated value is again slightly bigger than M_{tot} of 1548,49kNm which followed from Scia. This slightly bigger value is also in this case amongst others the result of not being completely rigid of the floating body. But the values are close enough together that can be concluded that this Scia model also gives the good result for this wave loading. So the total moment based on the values of Scia will be taken as the real internal moment, and will be used for the checks.

Appendix 28:

Calculation stresses floating body

A28-2

Check floating body



Schematisation/Assumptions

- The internal moment is expected to be taken only by the 'upper and lower flanges'. The stresses in these flanges are assumed to be equal in these flanges.
- The normal forces are equally divided over the complete section surface.
- The maximum stresses are calculated over the sections over the cavities, because this will be normative
- The floor will also contribute in the bearing of the forces, but this is not taken in account

Characteristics

Surface area beam section over the cavity:

$$A_{total,cavity} = 2 \cdot (250 \cdot (200+30) + 2 \cdot \tfrac{1}{2} \cdot 325 \cdot 100) = 147{,}5 \cdot 10^3\, mm^2$$

Surface area flange section:

$$A_{flange} = (250 \cdot (200+30) = 57{,}5 \cdot 10^3\, mm^2$$

Length moment arm: $l_{arm} = 1500mm$

Load

Moments

In the table below the combined normative moment is calculated, see chapter 18 for information.

Loadcase	Origin	Moment by SciaSLS	Heightened moment SLS	Load factor	Mom ULS	Red factor	Moment combination
Selfweight+Snow Load	Model Pavilion (ch 15)				147,2	1	147,2
Imposed Load	Rectangular Body (ch16)	289,45	314,59	1,5	471,9	1	471,9
Wave Load	Rectangular Body (ch16)	264,73	325,4	1	325,4	0,6	195,2
Total moment							**814,3**

Table 5: Calculation normative moment by combination, moments in kNm

For calculating the connections a value of 900kNm has been taken. $M_d = 900kNm$

Normal Force

The design normal tension force for the beam is taken the same as for the connection, see 18.2.3.

$$N_{d\_tension} = 200kN$$

For compression is taken a higher value, since a high value of 351,66 kN appeared from the real model schematisation (see table 11 on p III-46)

$$N_{d\_compression} = 351{,}66 + 200 = 551{,}66kN$$

Shear Force

The maximum shear in the beams follow from the real shape model and was 108,13 kN (see 18.2.3.). This load appeared not at the spot where the highest loads by waves and imposed loads where found (for the beams the load is lower then for the connection, because the floor mainly takes the horizontal shear forces).

$$V_d = 108{,}13kN$$

Compression stresses

Compression stress by combined normal forces + moment:

$$\sigma_{c;c;d} = \frac{N}{A} + \frac{M/l}{A_{flange}}$$

Normative situation:

$$\sigma_{c;c;d} = \frac{551{,}66 \cdot 10^3}{147{,}5 \cdot 10^3} + \frac{836{,}3 \cdot 10^6 / 1500}{57{,}5 \cdot 10^3} = 3{,}7 + 9{,}7 = 13{,}4 N/mm^2$$

Tension stress and reinforcement

Normal reinforcement

Tension by moment only:

$$N_s = \frac{M}{z} = \frac{814{,}3 \cdot 10^3}{1500} = 557{,}3 kN$$

$$A_s = \frac{N_s}{f_y} = \frac{557{,}3 \cdot 10^3}{500/1{,}15} = 1281{,}2 mm^2$$

With standard reinforcement steel FeB 500, a surface of 1281,2mm^2 will be needed.

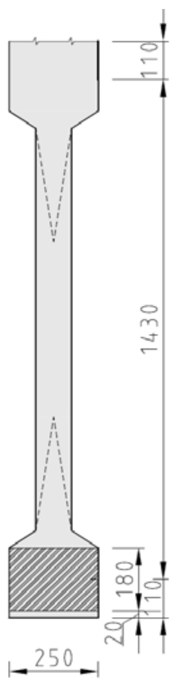
This can be taken by four bars with a diameter of d=22, or five bars with a diameter of 20mm

A_s five bars d=20: $A_s = 5 \cdot \pi 10^2 = 1570 kN$

Tension by tension force only:

$$A_s = \frac{N}{f_y} = \frac{200 \cdot 10^3}{500/1{,}15} = 459{,}8 mm^2$$

Tension force and moment combined:
The needed steel surface for tension and moment combined = 1281,2+459,8=1741,0mm^2. This can be taken with 5 bars with a diameter 22mm.



Fibre reinforcement

First the amount fibre reinforcement is calculated for the moment of 169,1 kNm from model for the real shape of the pavilion. 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 1430mm will be used, see the figure on the right.

Load of real shaped model

$$N_s = \frac{M}{z} = \frac{169{,}1 \cdot 10^3}{1430} = 117{,}4 kN$$

$$A_s = \frac{N}{f_y} = \frac{117{,}4 \cdot 10^3}{500/1{,}15} = 269{,}9 mm^2$$

This calculated surface A_s will be multiplied by three, for amongst others the irregular orientation of the fibres.

$$A_{s,\,fibres} = A_s \cdot 3 = 269{,}9 \cdot 3 = 809{,}7mm^2$$

This results in the required percentage of fibres of 1,8%:

$$\varpi = \frac{A_{s,\,fibres}}{A_{working\_surface.\,flange}} = \frac{809{,}7mm^2}{0{,}045m^2} = 1{,}8\%$$

<u>Normative load</u>
If the normative load as calculated with the normal reinforcement is used, this results in the following:

$$A_{s,\,fibres} = A_s \cdot 3 = 1749{,}8 \cdot 3 = 5249{,}4mm^2$$

This results in the required percentage of fibres of 11,7%:

$$\varpi = \frac{5249{,}4mm^2}{0{,}045m^2} = 11{,}7\%$$

Shear force

The shear forces stay at most parts below 30kN, but there are a few locations with a high value, these locations can be found primarily around the supports of the superstructure.

Normative shear force = 108,13kN

$$V_d = 108{,}13kN$$

Surface area beam section over the cavity:

$$A_{total,cavity} = 2 \cdot (250 \cdot (200 + 30) + 2 \cdot \tfrac{1}{2} \cdot 325 \cdot 100) = 147{,}5 \cdot 10^3\, mm^2$$

Allowable shear strength C55/67:

τ_c = shear strength of the concrete = 0,4*tensile strength

for f_{ck}=55: f_{ctk} =3,0 N/mm, $\tau_{cd} = \frac{0{,}4 \cdot 3{,}0}{1{,}5} = 0{,}8$ N/mm^2 [Eurocode 1992-1-1]

Total allowable shear strength without shear reinforcement:

$$V_{u,d} = A \cdot \tau_{cd} = 147{,}5 \cdot 10^3 \cdot 0{,}8 = 118 \cdot 10^3 = 118kN$$

Check: $\frac{V_d}{V_{u;d}} = \frac{108}{118} = 0{,}92$ So, the strength for shear force is already fulfilled without shear force reinforcement. There is made use of fibre reinforcement so the beams have in fact a larger shear force capacity.

A28-6

Appendix 29:

Calculation meta centres and natural oscillation periods

For complete one-piece pavilion
The pavilion is schematized as a rectangular floating body, from 45x21.
The colored fields in the table are the entry fields. The other values are calculated with the formula's given in appendix 8 and 9.

		Complete rectangular pavilion	
Dimensions floating body			
width floating body	b	21,00000	m
length floating body	l	45,00000	m
height floating body	h	1,70000	m
Area	A	945,00000	m2
Draught	d	0,82	m
Stability $GM = KM - KG = \frac{b^2}{12\,d} + \frac{1}{2}d - KG$			
Height centre of gravity	KG	1,70000	m
Height metacentre (short direction)	KM_y	45,21561	m
Height metacentre (long direction direction)	KM_x	206,14969	m
Metacentric height (short direction)	GM_y	43,51561	m
Metacentric height (long direction)	GM_x	204,44969	m
Dynamic stability, roll			
short direction			
moment of inertia Ixx	I_xx	633,00	m^4
moment of inertia Izz	I_zz	0,97	m^4
polar moment of inertia	I_polar	633,96	m^4
polar inertia radius j=I/A	j	6,07	
eigen frequency (hoeksnelheid), short direction	ω_0_y	3,44	rad/s
long direction			
moment of inertia Ixx	I_xx	6228,48	m^4
moment of inertia Izz	I_zz	2,07	m^4
polar moment of inertia	I_polar	6230,55	m^4
polar inertia radius j=I/A	j	12,99	
eigen frequency, long direction	ω_0_x	3,48	rad/s
eigenperiods			
eigenperiod, short directon (2pi/f)	T_0_y	1,83	s
eigenperiod, long directon	T_0_x	1,81	s
Dynamic stability, heave			
eigen frequency (hoeksnelheid), short direction	ω_0_heave_y	1,102913723	rad/s
eigen frequency, long direction	ω_0_Heave_x	0,54078403	rad/s
eiegnperiod, short directon (2pi/f)	T_0_heave_y	5,696896483	s
eigenperiod, long directon	T_0_heave_x	11,61865912	s

For modular pavilion
The modular pavilion is schematized as given in paragraph 17.2.
The colored fields in the table are the entry fields. The other values are calculated with the formula's given in appendix 8 and 9.

		body 1	body 2	body 3	
Dimensions floating body 1					
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	m2
Draught	d	0,82	0,82	0,82	m
Stability					
$GM = KM - KG = \frac{b^2}{12d} + \frac{1}{2}d - KG$					
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
short direction					
moment of inertia Ixx	I_xx	694,58	694,58	694,58	m^4
moment of inertia Izz	I_zz	1,28	1,28	1,28	m^4
polar moment of inertia	I_polar	695,85	695,85	695,85	m^4
polar inertia radius	j	6,07	6,07	6,07	
eigen frequency , short direction	ω_0_y	3,28	3,28	3,28	rad/s
long direction					
moment of inertia Ixx	I_xx	437,40	253,13	129,60	m^4
moment of inertia Izz	I_zz	1,09	0,91	0,73	m^4
polar moment of inertia	I_polar	438,49	254,04	130,33	m^4
polar inertia radius	j	5,20	4,34	3,47	
eigen frequency, long direction	ω_0_x	3,26	3,23	3,16	rad/s
eigenperiods					
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
Dynamic stability, heave					
eigen frequency, short direction	ω_0_heave_y	1,09	1,09	1,09	rad/s
eigen frequency, long direction	ω_0_Heave_x	1,25	1,47	1,78	rad/s
eiegnperiod, short directon (2pi/f)	T_0_heave_y	5,75	5,75	5,75	s
eigenperiod, long directon	T_0_heave_x	5,01	4,27	3,53	s

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Appendix 30: Dimensioning and check connections

Forces

Tensile force upper and lower connection	F_d	800	kN	tensile	(ULS)
Shear force ridges	V_d	200	kN	shear	(ULS)

Table 6: Forces in connections

Materials

Bolts, wire ends and nuts:

Steel:		quality 8.8		
Tensile strength		$f_{t;b;d}$	800	N/mm2
Yield strength		$f_{y;b;d}$	640	N/mm2
Allowable shear strength	=f_tbd/√3	$\tau_{t;b;d}$	461,9	N/mm2

Steel plates etc.

Steel:		S355		
Tensile strength		$f_{t;p;d}$	490	N/mm2
Yield strength		$f_{y;p;d}$	355	N/mm2
Yield strength (for 40<t<100)		$f_{y;p;d}$	325	N/mm2
Allowable shear strength	=f_tbd/√3	$\tau_{t;p;d}$	282,9	N/mm2
Shear yield strength	=f_ybd/√3	$\tau_{y;p;d}$	205,0	N/mm2

Welds

Strength weld	$= \frac{f_{t;p;rep}}{\beta\gamma}$	$f_{w;u;d}$		
Strength weld for S355	$= \frac{510}{0,9 \cdot 1,25}$	$f_{w;u;d}$	436	N/mm2

Reinforcement steel

Steel		FeB 500		
Material Factor		γ_s	1,15	
Yield strength		$f_{y;s;d}$	435	N/mm2

Concrete

Concrete		C55/67			
characteristic strength		f_ck	55	N/mm2	
design strength	=0,85*f_ck/1,5	f_cd	31,17	N/mm2	
charac. tensile strength		f_ctk	3	N/mm2	(EC 1992-1-1)
Design shear strength	=0,4*f_ctk/1,5	τ_cd	0,8	N/mm2	

Upper connection

Dimensioning Bolts

The bolts are loaded by tension. The estimation of bolt thickness is done with the following formula:

$$A_{bolt} = \frac{F_{t,d}}{f_{y;b;d}}: \qquad A_{bolt} = \frac{800 \cdot 10^3}{640} = 1250 mm^2$$

This will be realised with 2 bolts:

Needed surface per bolt:	A_bolt / 2	A_bout_min	625	mm2
Minimum radius:	√(A_b_min/π)	r_min	14,1	mm
Chosen radius		r	15	mm
Chosen diameter		d	30	mm
surface area chosen bolt		A_b;s	706,9	mm2

Check bolts tensile strength

Tensile strength bolt: $F_{t;u;d} = 0{,}72\alpha_{red;2} f_{t;b;d} A_{b;s}$

$F_{t;u;d} = 0{,}72 \cdot 1 \cdot 800 \cdot 706{,}9 = 407{,}15$ kN

Tensile strength connection: $F_{t;u;d;c} = 2 \cdot F_{t;u;d} = 2 \cdot 407{,}15 = 814{,}3$ kN

Check: $\frac{F_d}{F_{t;u;d;c}} = \frac{800}{814{,}3} = 0{,}98$

Check punch head plate (pons)

Punch strength plate: $F_{t;u;d} = 0{,}48\pi d_m t_p f_{t;p}$

$F_{t;u;d} = 0{,}48\pi \cdot 66 \cdot 20 \cdot 490 = 975{,}4$ kN

Punch strength connection: $F_{t;u;d;c} = 2 \cdot F_{t;u;d} = 2 \cdot 975{,}4 = 1950{,}7$ kN

Check: $\frac{F_d}{F_{t;u;d;c}} = \frac{800}{1950{,}7} = 0{,}41$

For strength still largely over dimensioned, but this is done to prevent deformation.

Anchorage

Dimensioning anchorage:

Required steel surface: $A_s = \frac{F_{t,d}}{f_{y;s;d}}: \ A_s = \frac{800 \cdot 10^3}{435} = 1840 mm^2$

chosen: 5 rods Ø24mm.

surface chosen rods: $A_s = 5 \times 452{,}4\ mm^2$

Check Anchorage

Tension strength anchorage: $F_{t;u;d} = A_s \cdot f_{y;s;d}: \qquad F_{t;u;d} = 5 \cdot 452{,}4 \cdot 435 = 984{,}0 kN$

Check: $\frac{F_d}{F_{t;u;d}} = \frac{800}{984{,}0} = 0{,}81$

Check sidewalls steel box

Tension strength sidewalls: $F_{t;u;d} = 2 \cdot A_{sw} \cdot f_{y;s;d}$

$$A_{sw} = h \cdot t_{pb}$$

Thickness plate box	t_{pb}	10	mm
Hight box	h	190	mm

$$A_{sw} = 190 \cdot 10 = 1900 mm^2$$

$$F_{t;u;d} = 2 \cdot 1900 \cdot 355 = 1349 kN$$

Check: $\frac{F_d}{F_{t;u;d}} = \frac{800}{1349} = 0{,}59$

Lower connection

Pen point

Dimensioning pen point

The vertical pen is loaded by shear. The estimation of the pen thickness is done with the following formula:

$$A_{pen} = \frac{F_{v,d}}{\tau_{t;b;d}}$$

$$A_{pen} = \frac{800 \cdot 10^3 / 2}{800/\sqrt{3}} = \frac{400 \cdot 10^3}{462} = 866 mm^2$$

This can be realized with a wedge shaped pen, with dimensions of 50x20 = 1000 mm^2 at the smallest section (the lowest).

Check pen point on shear

The just chosen dimensions of 50x20mm^2 will not fulfil the checks, so the pen point has been altered and the check here is given for the new pen point, the new dimensions are 52x21mm^2.

Width wedge	w1	52	mm
Normative depth wedge	w2	21	mm
Surface normative area (=w1xw2)	A_bolt_min	1092	mm2

Shear strength, 1 shear surface: $F_{v;u;d} = 0{,}48 \cdot f_{t;b;d} \cdot A_b$

$$F_{v;u;d} = 0{,}48 \cdot 800 \cdot 1092 = 419{,}3 kN$$

Shear strength bolt, 2 surfaces: $F_{v;u;d;c} = 2 \cdot F_{t;u;d} = 2 \cdot 401{,}4 = 838{,}7 kN$

Check: $\frac{F_d}{F_{v;u;d;c}} = \frac{800}{838{,}7} = 0{,}95$

Normal force in pen
In the pen there will be a normal force, because by the slope of the wedge point the gets pushed up when a tension force arises in the lower connection. This normal force equals the tension force times the slope:

$$N_{pen,d} = 800 * \tfrac{1}{8} = 100kN$$

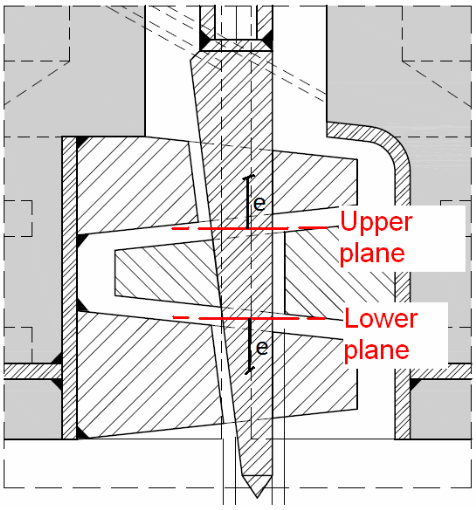
This force will act in the pen itself, on the lower locking plates and on the fixation of the pen.

<u>Check normal force</u>
Compression strength pen point: $F_{n;u;d} = f_{y;b;d} \cdot A_b$

$$F_{n;u;d} = 640 \cdot 1092 = 699kN$$

Check: $$\frac{N_{pen,d}}{F_{n;u;d}} = \frac{100}{699} = 0{,}14$$

Check pen on combination stress
For the combination stress, 2 planes in the pen point might be normative: between top and center plate and between center and lower plate, see figure on the right. The lower plane is smaller, but here there is no normal force. There also exists a small moment by the eccentricity *e*.



Moment in planes pen: $M_d = \tfrac{1}{2} \cdot F_d * e$

$$M_d = \tfrac{1}{2} \cdot 800 \cdot 10^3 \cdot 40 = 16 \cdot 10^6\ Nmm$$

<u>Check upper plane</u>
Surface upper plane: $A_{pen,1} = 52 * 40 = 2080mm^2$

compression stress by N_{pen} : $$\sigma_{s,n,d} = \frac{N_{pen,d}}{A_{pen,1}}$$

$$\sigma_{s,n,d} = \frac{100 \cdot 10^3}{2080} = 48{,}07\ \ N/mm^2$$

max shear stress by F_d (in center): $$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{F_d/2}{A_{pen,1}}$$

$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{800 \cdot 10^3/2}{2080} = 288{,}5\ \ N/mm^2$$

max shear stress by moment (side): $$\sigma_{s,m,d} = \frac{M \cdot 1/2d}{I_{pen,1}}$$

$$\sigma_{s,d} = \frac{16 \cdot 10^6 \cdot 1/2 \cdot 40}{\tfrac{1}{12} \cdot 52 \cdot 40^3} = 32{,}05\ N/mm^2$$

It can be seen that the stresses by the acting moment due to the eccentricity are low compared to the stress by the shear force, so the stress at the sides by the moment will not be normative.

check stress combination (center) : $$\sqrt{\sigma_{s,n,d}{}^2 + 3\tau_{1,\max,d}{}^2} \leq f_{y;b;d}$$

$$\sqrt{48{,}07^2 + 3 \cdot 288{,}5^2} = 501{,}9 \ \ N/mm^2$$

Check: $\dfrac{501{,}9}{640} = 0{,}78$

<u>Check lower plane</u>

Surface lower plane: $A_{pen,2} = 52 * 32 = 1664mm^2$

max shear stress by F_d (in center): $\tau_{1,\max,d} = \dfrac{3}{2} \cdot \dfrac{F_d/2}{A_{pen,2}}$

$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{800 \cdot 10^3/2}{1664} = 360{,}6 \ \ N/mm^2$$

max shear stress by moment (side): $\sigma_{s,m,d} = \dfrac{M \cdot 1/2d}{I_{pen,2}}$

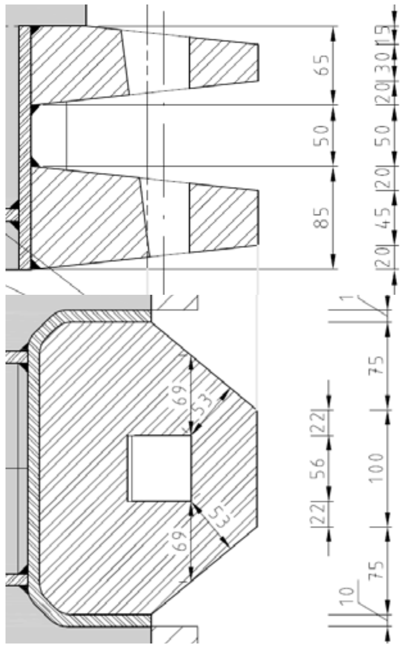
$$\sigma_{s,d} = \frac{M \cdot 1/2d}{\frac{1}{12} \cdot 52 \cdot 32^3} = 50{,}1 \ N/mm^2$$

It can be seen that the stresses by the acting moment due to the eccentricity are low, compared to the stresses by the shear force, so the stress at the sides caused by the moment will not be normative. The stress by shear force in the center is clearly normative. In the center of the lower plane there is only shear force. (The shearforce has already been checked for the capacity of the full surface area, below can be seen that this is also fulfilled for the peak stress in this plane).

check shear stress (center) : $\tau_{1,\max,d} \leq \tau_{y;b;d}$

Check: $\dfrac{\tau_{1,\max,d}}{\tau_{y;b;d}} = \dfrac{360{,}6}{640/\sqrt{3}} = 0{,}97$

Check on upsetting/compression (stuik)



Thickness locking plates

Thickness at hole	t_hole	50	mm
thinnest thickness	t_thinnest	30	mm
Average thickness over normative area	t_ave1	40	mm

Compression strength: $F_{t;u;d} = 2\alpha_c \alpha_{red;1} f_{t;d} d_{b,nom} t$

alpha stuik (smallest)	alp_c	0,3589744
alpha red	alp_red	1

$$F_{t;u;d} = 2 \cdot 0{,}36 \cdot 1 \cdot 800 \cdot 52 \cdot 50 = 1493\text{kN}$$

Check: $\dfrac{F_d}{F_{c;u;d}} = \dfrac{800}{1493} = 0{,}54$

Check locking plates
Center plate is normative (only one plate, in stead of two)
The small moments which will occur by the shear forces are not taken in account because these will be zero at the spots where the shear stresses are maximal. The small moments will result in tension stresses with values below 20N/mm^2, an example of stress by moment is calculated below:

Maximal moment in plane e1 (see edge distance below): $M_d = \frac{1}{2} \cdot F_d * e_y$

$$M_d = \tfrac{1}{2} \cdot 800 \cdot 10^3 \cdot 75 = 30 \cdot 10^6\, Nmm$$

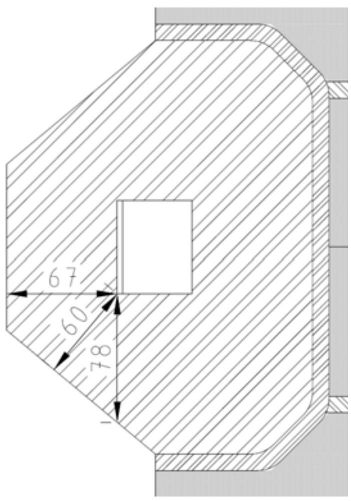
Stress by moment: $\sigma_{s,m,d} = \dfrac{M \cdot 1/2d}{I_{xx}}$

$$\sigma_{s,d} = \frac{30 \cdot 10^6 \cdot 1/2 \cdot 78}{\frac{1}{12} \cdot 48 \cdot 75^3} = 17{,}12\, N/mm^2$$

The locking plates are calculated linear elastic, but when necessary the tension will be redistributed. This means if the plates will fulfil elastically, they have got quite some reserve.

Edge distances center locking plates

Orthoganal edge distance 1 (tension)	e1	78	mm
Orthoganal edge distance 2 (shear)	e2	67	mm
Smallest edge distance	x_s	60	mm



Check smallest orthogonal nett surface on tension

Strength nett surface: $F_{t;u;d} = \sum A_{np} \cdot f_{y;s;d}$

Nett surface plate: A_{np} =2*e$_1$*t$_{ave}$ = 2*78*48 = 7488 mm2

$$F_{t;u;d} = 7488 \cdot 325 = 2433{,}6kN$$

Check: $\dfrac{F_d}{F_{t;u;d}} = \dfrac{800}{2433{,}6} = 0{,}33$

<u>Check smallest edge distance</u>
The smallest edge distance, x_s, will be loaded by tension and shear in horizontal direction, because of the tension force in the connection. It will also be loaded by a vertical shear force, because of the pen pushing against the centreplate. In the center center of this section the combination stress will be largest, because here the shear stresses will have their maximums.

Smallest surface plate $A_{\min,p}$ =2*x_s*t_{ave} = 2*60*45 = 5400 mm2

Decomposing F_d in tension stress and shear stress:

tension stress by F_d:
$$\sigma_{s,d} = \frac{F_d/2 \cdot \cos\alpha}{A_{\min,p}/2}$$
$$\sigma_{s,d} = \frac{800 \cdot 10^3/2 \cdot \cos 39{,}8}{5400/2} = 113{,}8 \;\; N/mm^2$$

max shear stress by F_d (in center):
$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{F_d/2 \cdot \cos(90-\alpha)}{A_{sp}/2}$$
$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{800 \cdot 10^3/2 \cdot \cos(90-39{,}8)}{5400/2} = 142{,}24 \;\; N/mm^2$$

max shear stress by N_{pen} (in center):
$$\tau_{z,\max,d} = \frac{3}{2} \cdot \frac{N_{pen}/2}{A_{sp}/2}$$
$$\tau_{z,d} = \frac{3}{2} \cdot \frac{100 \cdot 10^3/2}{5400/2} = 27{,}8 \;\; N/mm^2$$

check stress combination:
$$\sqrt{\sigma_{s,d}{}^2 + 3\tau_{1,\max,d}{}^2 + 3\tau_{z,\max,d}{}^2} \leq f_{y;s;d}$$
$$\sqrt{113{,}8^2 + 3 \cdot 142{,}24^2 + 3 \cdot 27{,}8^2} = 275{,}6 \;\; N/mm^2$$

Check: $$\frac{\sigma_{compare,s,d}}{f_{y;s;d}} = \frac{275{,}6}{325} = 0{,}85$$

<u>Check smallest shear surface:</u>

Shear surface plate A_{sp} =2*e_2*t_{ave} = 2*67*40 = 5360 mm2

max shear stress by F_d (in center):
$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{F_d/2}{A_{sp}/2}$$
$$\tau_{1,\max,d} = \frac{3}{2} \cdot \frac{800 \cdot 10^3/2}{5360/2} = 223{,}9 \;\; N/mm^2$$

max shear stress by N_{pen} (in center):
$$\tau_{z,\max,d} = \frac{3}{2} \cdot \frac{N_{pen}/2}{A_{sp}/2}$$
$$\tau_{z,\max,d} = \frac{3}{2} \cdot \frac{100 \cdot 10^3/2}{5360/2} = 28{,}0 \;\; N/mm^2$$

check stress combination: $\sqrt{3\tau_{1,\max,d}{}^{2} + 3\tau_{z,\max,d}{}^{2}} \leq f_{y;s;d}$

$$\sqrt{3 \cdot 223{,}9^{2} + 3 \cdot 28{,}0^{2}} = 390{,}8 \ N/mm^{2}$$

Check: $\dfrac{\sigma_{compare,s,d}}{f_{y;s;d}} = \dfrac{390{,}8}{325} = 1{,}20$

So, this check is not fulfilled. This means this section doesn't fulfil elastically. This means in the centre the steel will start to yield, then the stresses in the centre will be redistributed somewhat to the sides. Plasticly this suffices, because when this combination stress is spread out over the complete section, it results in a value of 390,8/1,5 =260,5N/mm^2.
But this centreplate can best be enlarged, while the upper and lower plate can be reduced.

Check welds
The plate is welded with two fillet welds to the anchorage plate, one above and one below, both take half F_d and half N_{pen}. This forces in the welds are beared at the back (l=220mm, basic weld load situation 1 and 2 [CL2B]), and on the sides (l=90mm, basic weld load situation 2 and 3 [CL2B]). The load is assumed to be divided 50/50.
The stresses caused by the moment (resulting of the distance between pen and weld) are approximately 20 times as low, so these are neglected.

<u>Weld at the back (basic situation 1 and 2):</u>

Combined weld stress: $\sigma_{w;s;d} = \dfrac{1}{2}\sqrt{2}\,\dfrac{\frac{1}{2}F_{s;d}}{a \cdot l} + \dfrac{1}{2}\sqrt{2}\,\dfrac{\frac{1}{2}N_{pen;d}}{a \cdot l} \leq f_{w;u;d}$

$$\sigma_{w;s;d} = \frac{1}{2}\sqrt{2}\,\frac{\frac{1}{2} \cdot 800 \cdot 10^{3}}{5 \cdot 220} + \frac{1}{2}\sqrt{2}\,\frac{\frac{1}{2} \cdot 100 \cdot 10^{3}}{5 \cdot 220} = 289{,}3$$

Check: $\dfrac{\sigma_{w;s;d}}{f_{w;u;d}} = \dfrac{289{,}3}{436} = 0{,}66$

<u>Welds at the sides (basic situation 2 and 3):</u>

Combined weld stress: $\sigma_{w;s;d} = \dfrac{\sqrt{3}}{2}\,\dfrac{\frac{1}{4}F_{s;d}}{a \cdot l} + \dfrac{1}{2}\sqrt{2}\,\dfrac{\frac{1}{4}N_{pen;d}}{a \cdot l} \leq f_{w;u;d}$

$$\sigma_{w;s;d} = \frac{\sqrt{3}}{2}\,\frac{\frac{1}{4} \cdot 800 \cdot 10^{3}}{5 \cdot 90} + \frac{1}{2}\sqrt{2}\,\frac{\frac{1}{4} \cdot 100 \cdot 10^{3}}{5 \cdot 90} = 231{,}7$$

Check: $\dfrac{\sigma_{w;s;d}}{f_{w;u;d}} = \dfrac{231{,}7}{436} = 0{,}53$

Check Anchorage
This is exactly the same as for the upper connection.

Check Pen on buckling

The pen will pass its compression force of 100kN on to the upper fixation. The pen has a buckling length equal to it's length: 1,4 m.

Force in pen	=1/8*H	N_p	100	kN
length pen			1,4	m
buckling length			1,4	m
Chosen pen				
Outer diameter		d_p	48	mm
outer radius		r_u	24	mm
wall thickness		t_p	10	mm
inner radius		r_i	14	mm
Surface area	$= \pi r_u^{\ 2} - \pi r_i^{\ 2}$	A_p	1193,8	mm2
Moment of Inertia	$= \frac{1}{4}\pi(r_u - r_i)^4$	I_y	230404,4	mm4
polar inertia radius	$= \sqrt{\frac{I}{A}}$	i	13,9	mm
slenderness	$= \frac{l_{buc}}{i}$	λ	100,8	
specific slenderness	$= \pi\sqrt{\frac{E}{f_y}}$	λ_e	76,4	
relative slenderness	$= \frac{\lambda}{\lambda_e}$	λ_{rel}	1,32	

With the calculated relative slenderness, the buckling factor can be determined with the buckling curves given in NEN and Eurocodes. Curve A can be used. From curve A follows that a relative slenderness of 1,32 results in a buckling factor of 0,50.

buckling factor (curve a) ω 0,84

Verification: $$\frac{N_{c;s;d}}{\varpi_{buc} N_{c;el;d}} \leq 1$$

$$N_{c;el;d} = A_{pen} \cdot f_y : \qquad N_{c;el;d} = 1193{,}8 \cdot 355 = 423{,}8 \cdot 10^3\, N$$

Check: $$\frac{100}{0{,}50 \cdot 423{,}8} = 0{,}47$$

Check Fixation pen

Small fixation plates

Check punch lower fixation plate

Punch strength plate for a complete plate: $F_{;u;d} = 0{,}48\,\pi d_m t_p f_{t;p}$

The lower plate has a recess (sparing) for placing, so the strength is multiplied by an extra factor 0,6, which is on the safe side.

$$F_{c;u;d} = 0{,}60 \cdot 48\pi \cdot 60 \cdot 10 \cdot 490 = 266kN$$

Check: $\dfrac{F_d}{F_{c;u;d}} = \dfrac{100}{266} = 0{,}38$

Punch top fixation plate

The 10 mm thick top fixation plate is in fact only loaded when the connection is being demounted. Then the top nut is tightened for ejecting the pen. The force needed to do this is far lower than 100kN, so punch will not be a problem at all.

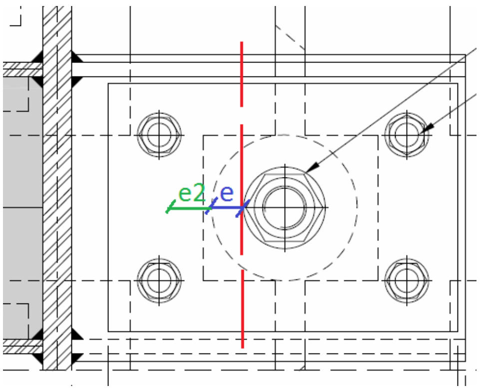
Check moment capacity fixation plates

The fixation plates will be loaded by a moment around the y-axis. The lower plate by the up pressing pen, and the upper plate by the down pointing force by demounting the connection.
If the strengthening elements below the lower plate are not taking in account, both plates have the same moment capacity:

Elastic moment capacity: $M_{y;el;d} = f_{p,y,d} \cdot W: \quad M_{y;el;d} = 355 \cdot \frac{1}{6} bh^2$

$$M_{y;el;d} = 355 \cdot \tfrac{1}{6} \cdot 170 \cdot 10^2 = 1{,}48 \cdot 10^6\, N/mm^2$$

The normative maximal moment will appear directly next to the large nut and washer; see the section line in the figure on the right. The excentricity *e* is also given. For the upper plate *e* is the distance from the nut to the edge where the plate is supported. The resulting support force will be somewhat further away, this is given with e+e2. For the lower plate e+e2 is approximately the distance from edge nut to heart stiffener plate.
The assumption is made that the load is equally distributed to all directions. So every edge takes ¼ of the load.



Maximal appearing moment: $M_{s;d} = \frac{1}{4} N_{pen} \cdot (e + e2)$

$$M_{s;d} = \tfrac{1}{4} 100 \cdot (20 + 25) = 1{,}1310^6\, N/mm^2$$

Check: $\dfrac{M_{s;d}}{M_{y;el;d}} = \dfrac{1{,}13 \cdot 10^6}{1{,}48 \cdot 10^6} = 0{,}76$

Main fixation plate

The main fixation plate is first calculated without the stiffening strips above and below the fixation plate.

<u>Check on shear</u>

Shear strength:

$$V_{z;el;d} = \frac{2}{3} \cdot b \cdot h \frac{f_{p;y;d}}{\sqrt{3}}$$

$$V_{z;el;d} = \frac{2}{3} \cdot 210 \cdot 30 \frac{355}{\sqrt{3}} = 860{,}8kN$$

Check: $\dfrac{N_{pen,d}}{V_{z;el;d}} = \dfrac{100 \cdot 10^3}{860{,}8 \cdot 10^3} = 0{,}12$

<u>Check on moment</u>

Elastic moment capacity:

$$M_{y;el;d} = f_{p,y,d} \cdot W$$

$$M_{y;el;d} = 355 \cdot \tfrac{1}{6} bh^2$$

$$M_{y;el;d} = 355 \cdot \tfrac{1}{6} \cdot 210 \cdot 30^2 = 11{,}2 \cdot 10^6 \, Nmm^2$$

Maximal appearing moment:

$$M_{s;d} = N_{pen} \cdot e$$

$$M_{s;d} = 100 \cdot 10^3 \cdot 150 = 15 \cdot 10^6 \, Nmm^2$$

Check: $\dfrac{M_{s;d}}{M_{y;el;d}} = \dfrac{15 \cdot 10^6}{11{,}2 \cdot 10^6} = 1{,}34$

The plate does not fulfil on elastic moment capacity, but it does fulfil plastically, since the plastic moment capacity is a factor 1,5 bigger. Stiffening strips are added to counter the large deformation which will occur when the acting moment will be larger than the elastic moment capacity.

<u>Check on moment with stiffening strips</u>

When the stresses in the fixation (caused by the moment) are calculated with the stiffeners as drawn in the appendix, the stresses in the main fixation plate will stay below the yield strength, but the stresses in the stiffeners will exceed the yield strength. So for a better deformation resistance, these stiffeners can best be enlarged.

With dimensions of 60x20mm the maximum stress in the top fibre of the stiffener will be reduced to 318N/mm2, which is below the yield strength of 355N/mm2 . The calculation is depicted in the table on the next page. For calculation, the fixation plate plus stiffeners are split in two, where both halves take half of the load. This schematisation is on the safe side.

Calculation stress fixation plate by Moment: with stiffeners				
height stiffener plates		h_sp	60,0	mm
width siffener plates		b_sp	20,0	mm
surface area stiffener plates	=hxb	A_sp	1200,0	mm2
position point of gravity of stiffener	=1/2h_sp+h_fp	y_sp	60,0	mm
height fixation plate		h_fp	30,0	mm
width ½ plate		b_fp	105,0	mm
surface area ½ fixation plate	=hxb	A_fp	3150,0	mm2
position point of gravity of fp	=1/2h_fp	y_fp	15,0	mm
Total surface area (1/2sp+fp)	=A_sp+A_fp	A_tot	4350,0	mm2
position of combined centre of gravity	=(A_sp*y_sp+A_fp*y_fp)/(A_sp+A_fp)	y_tot	27,4	mm
Iy stiffener	=1/12*b*h^3	I_sp	360000,0	mm4
Iy stiffener steiner	=1/2*A*(y_sp-y_nc)^2	I_sp,st	637116,5	mm4
Iy 1/2 plate	=1/12*b*h^3	I_fp	236250,0	mm4
Iy 1/2 plate steiner	=1/2*A*(y_fp-y_nc)^2	I_fp,st	242711,1	mm4
Iy Total (1/2plate+stiffener)	sum of above components	I_0,5t	1476077,6	mm4
stress by M, highest fibre stiffener	=M*(y_tot+h_sp)/(I_0,5t)	$\sigma_{M,sp,up}$	318,0	N/mm2
stress by M, lowest fibre	=M*y_tot/(I_0,5t)	$\sigma_{M,fp,up}$	139,3	N/mm2
stress by M, highest fibre above	=M*(h_fp-y_tot)/(I_0,5t)	$\sigma_{M,fp,up}$	13,1	N/mm2

With the larger dimensioned stiffener plates the welds can also be fulfilled more easy. The stiffener plates will be welded all round. With a throat thickness of 7 mm these welds will be fulfilled.

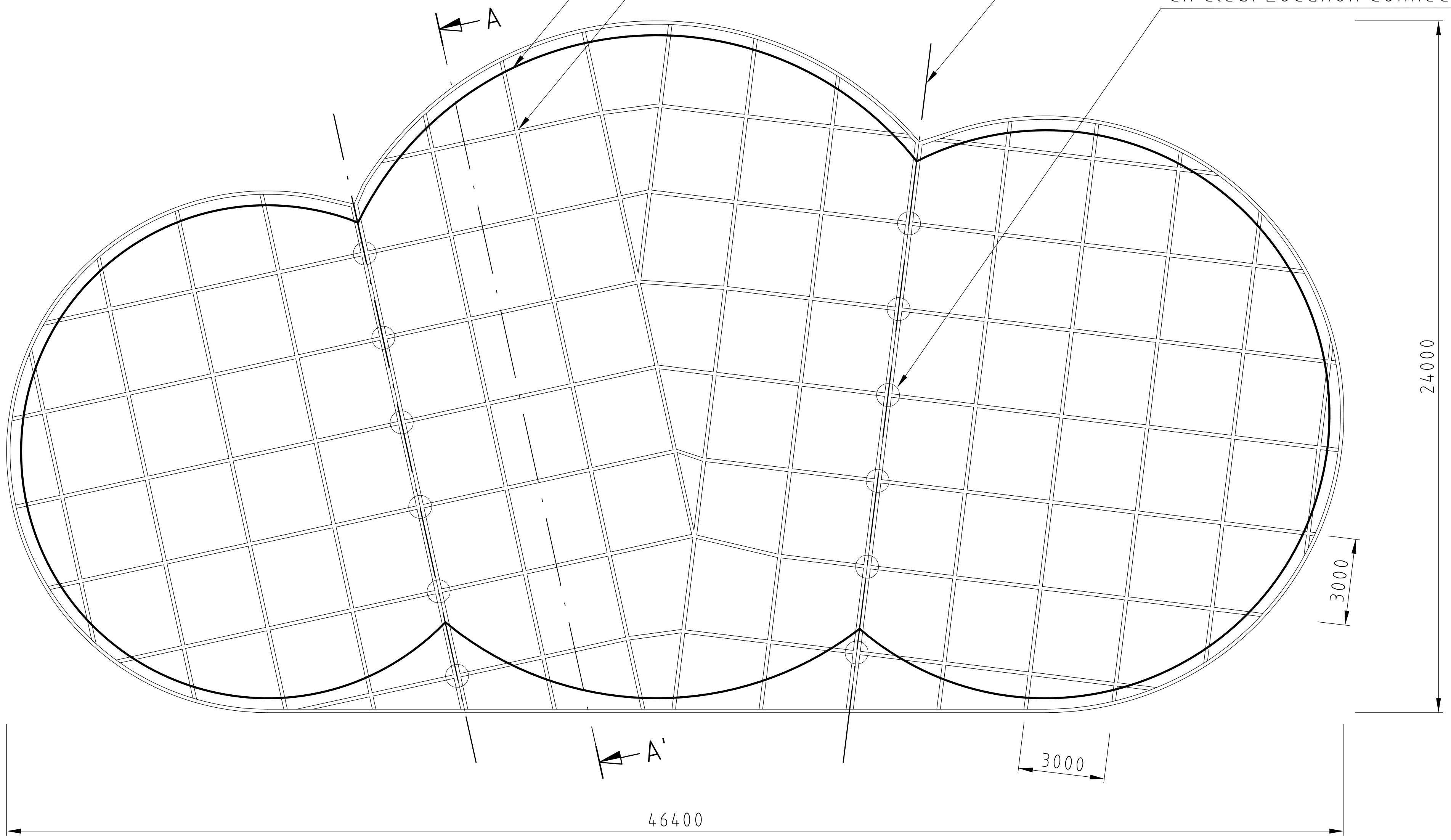
Appendix 31:

Drawings:

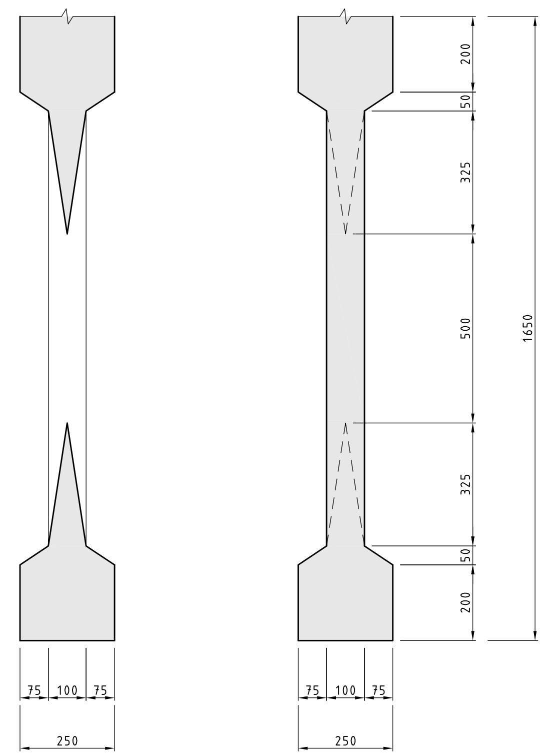
Drawing Overview Floating Body

Drawings Rigid connection floating body

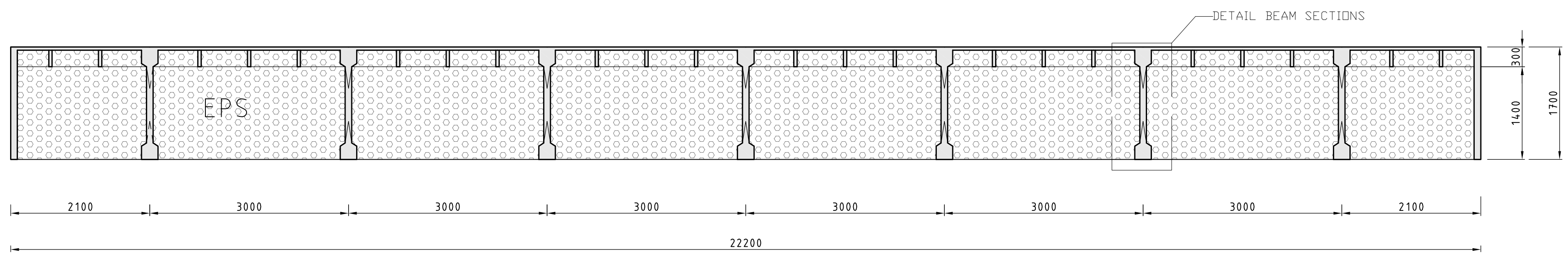
A31-2



TOP VIEW FLOATING BODY 1:200

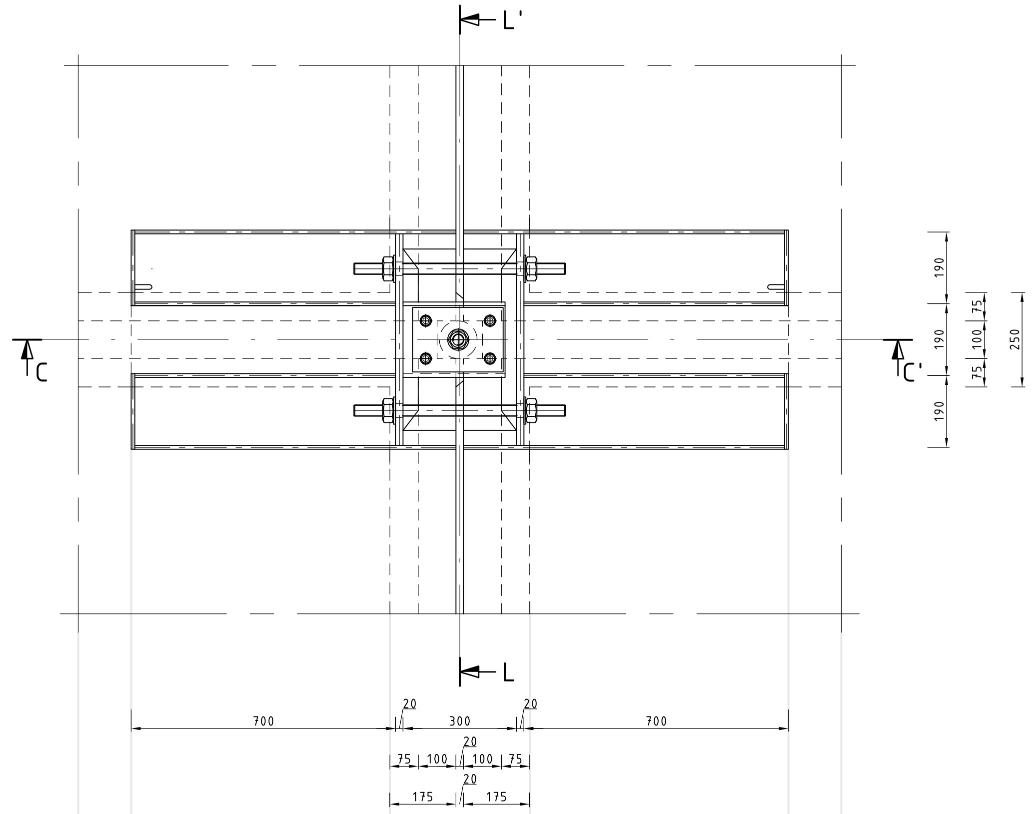


DETAIL BEAM SECTIONS 1:20

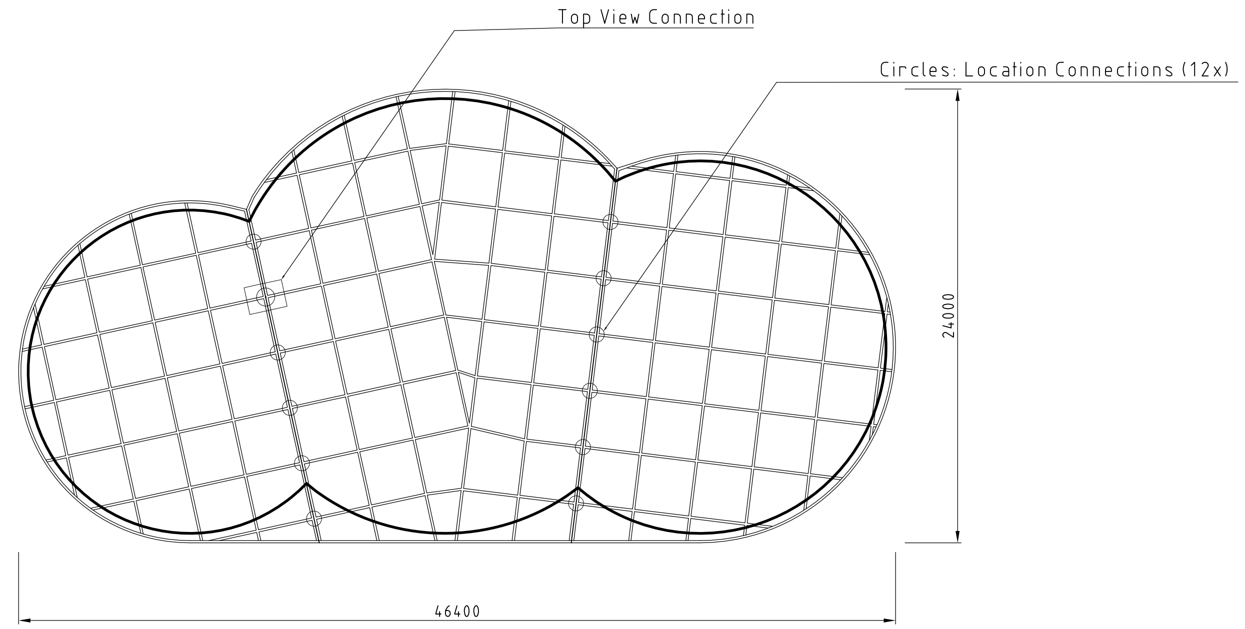


SECTION A-A' 1:100

Bestandsnaam : MIJN PAVILJOEN, COMPLEET, VOOR IN VERSLAG.DWG			
Gemeente Rotterdam Gemeentewerken Ingenieursbureau		Galvanistraat 15 Postbus 6633 3002 AP ROTTERDAM Telefoon : 010 489 NR1 Telefax : 010 489 NR2	
Floating Body Floating Pavilion Rotterdam			
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	Pavilion	Dimensions :	MM

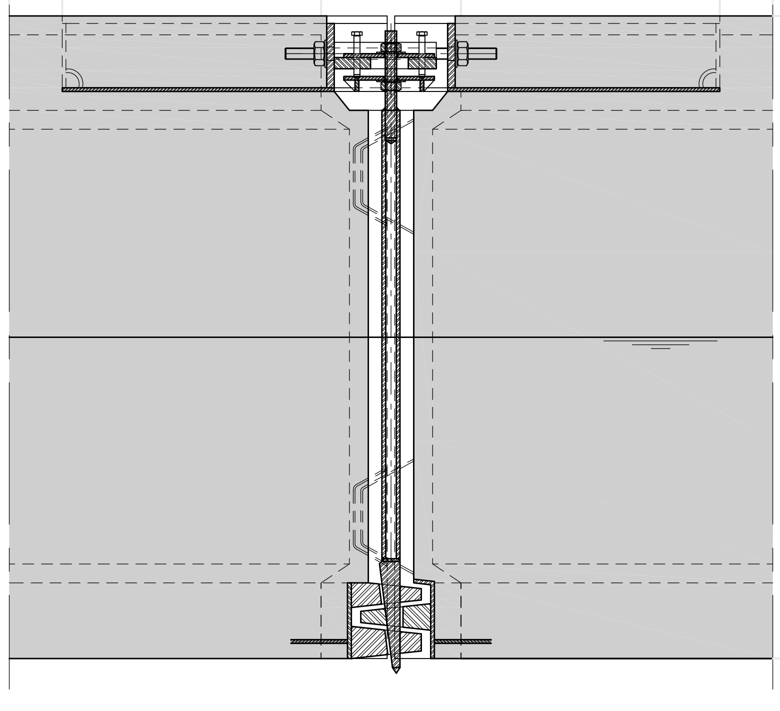


TOP VIEW CONNECTION

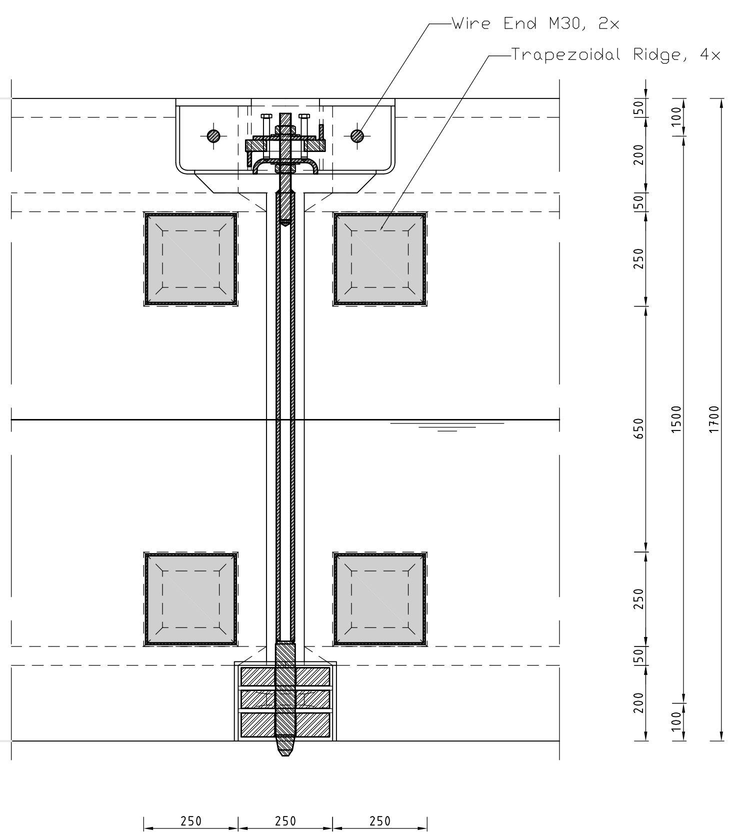


TOP VIEW FLOATING BODY PAVILION 1:500

- IN DRAWING 2 THE CONNECTION IS ALSO DRAWN SEPARATED
- MORE LONGITUDIANAL SECTIONS ARE GIVEN IN DRAWING 3
- MORE CROSS SECTIONS ARE GIVEN IN DRAWING 4
- MORE TOP VIEW SECTIONS ARE GIVEN IN DRAWING 5
- DETAILS ARE GIVEN FROM OF DRAWING 6, THERE THE DIMENSIONS OF THE CONNECTORS ARE GIVEN



LONGITUDINAL SECTION C-C'



CROSS SECTION L-L'

LEGEND
EPS
Concrete
Steel
Elastic material

Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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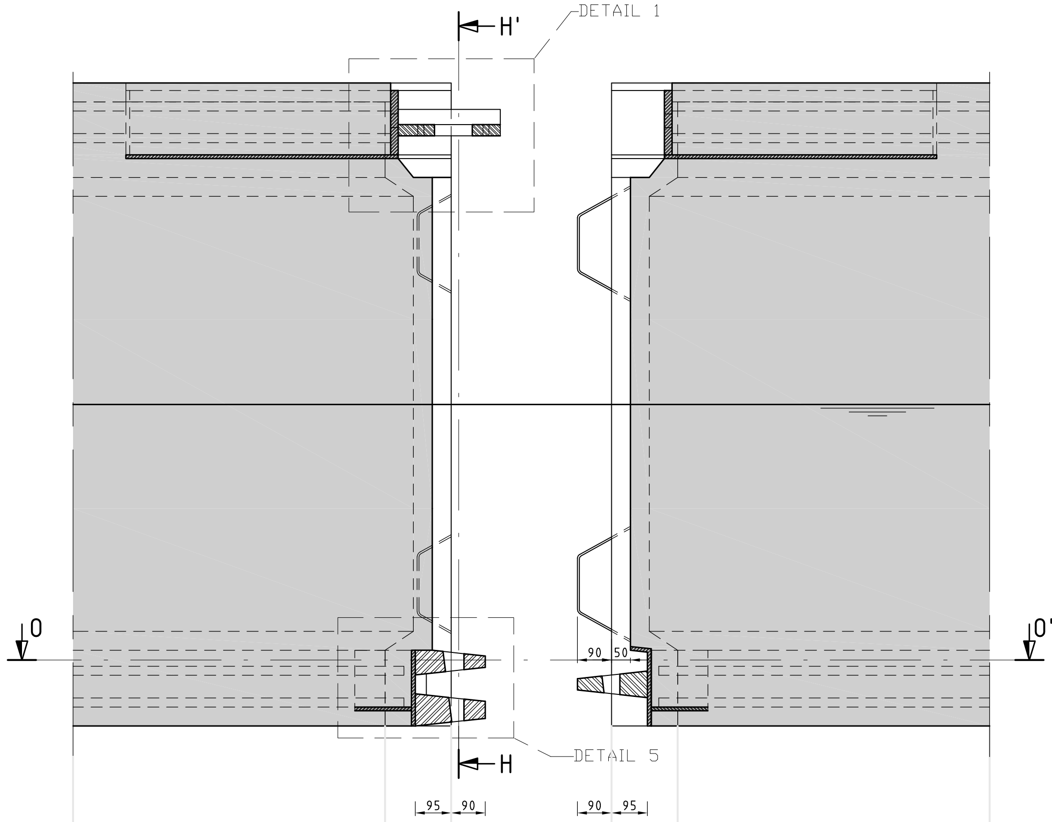
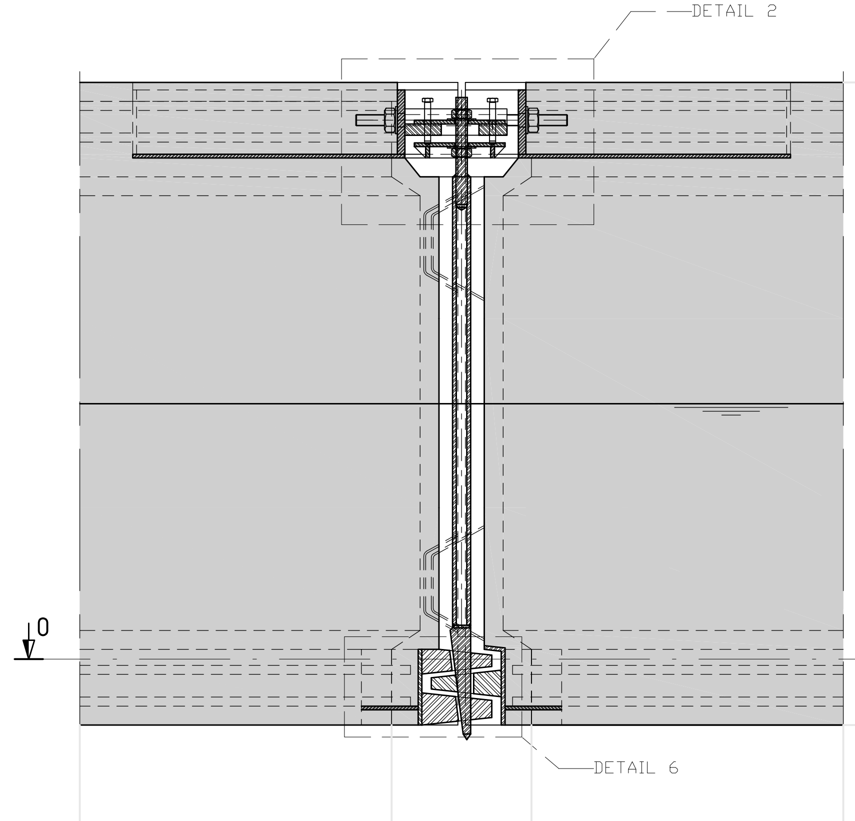
Rigid Connection Floating Pavilion

Drawing 1: Overview Connection

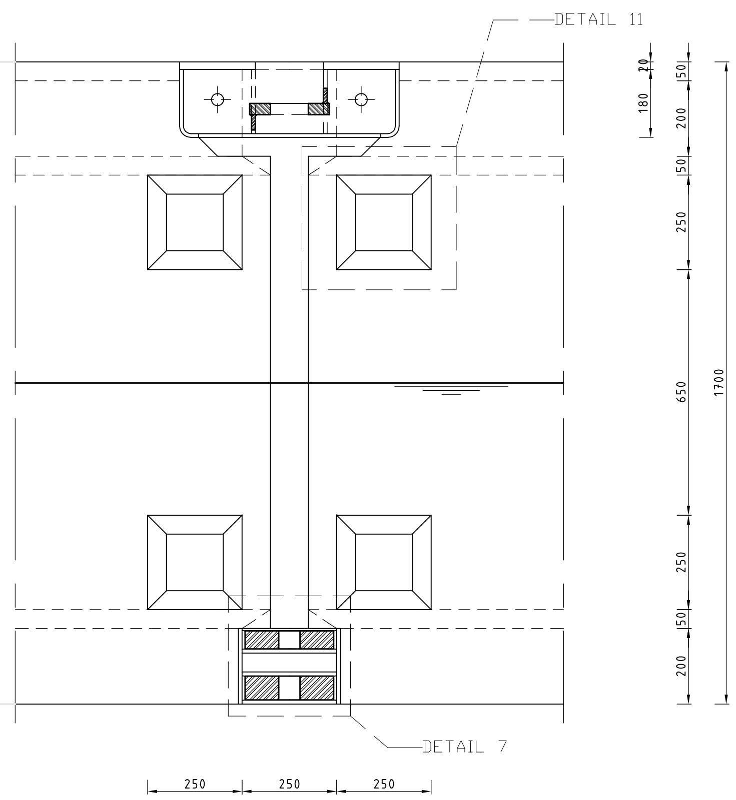
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:20
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	1	Dimensions :	MM



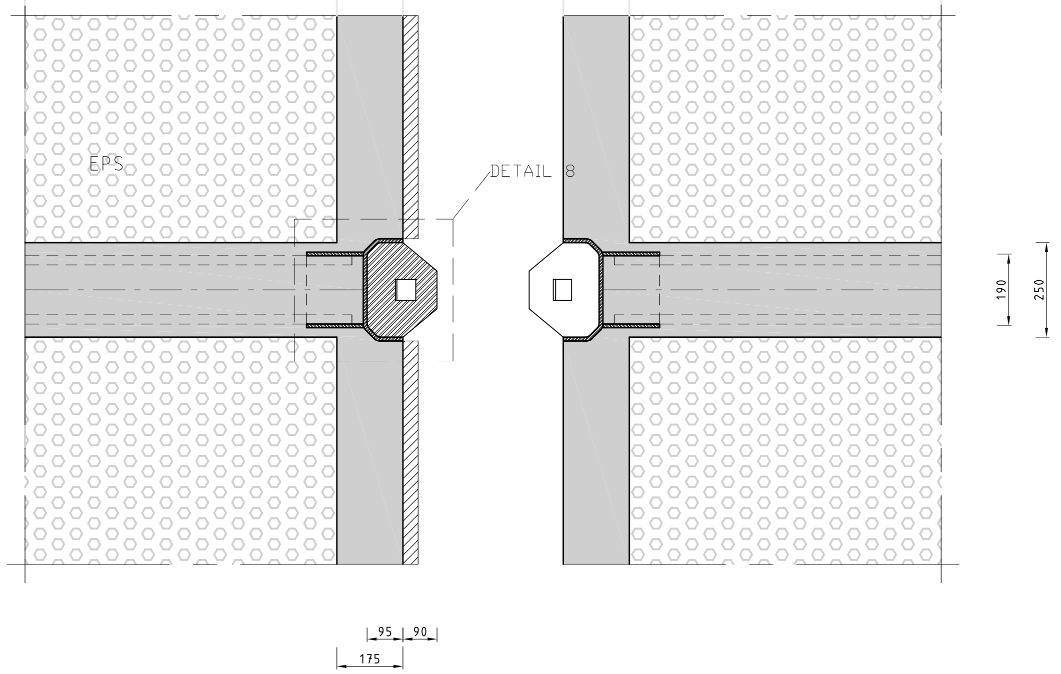
SECTION C-C' NOT CONNECTED



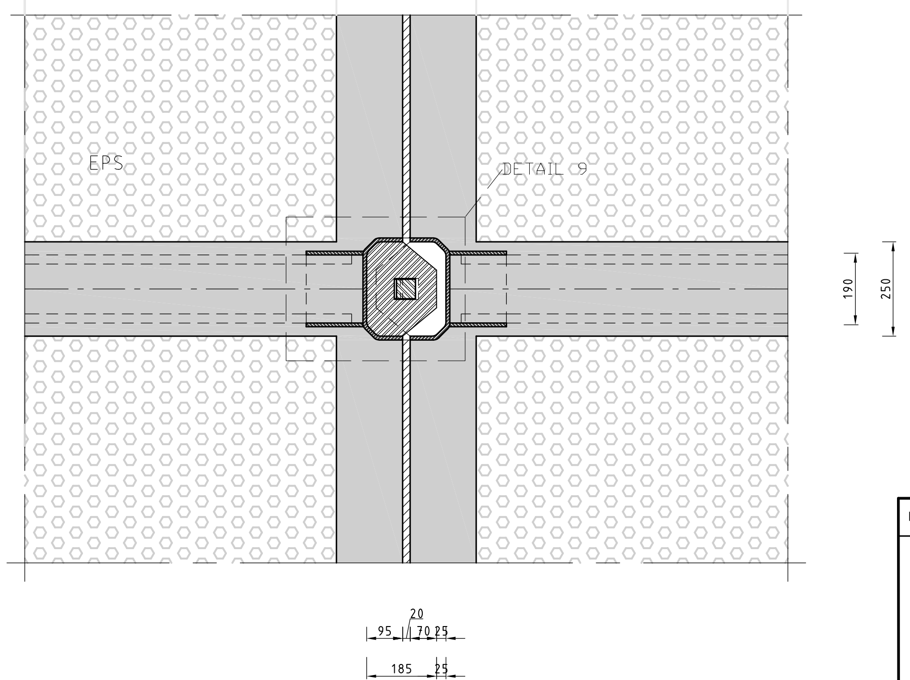
SECTION C-C' CONNECTED



SECTION H-H' NOT CONNECTED



SECTION O-O' NOT CONNECTED



SECTION O-O' CONNECTED

LEGEND
- EPS
- Concrete
- Steel
- Elastic material

Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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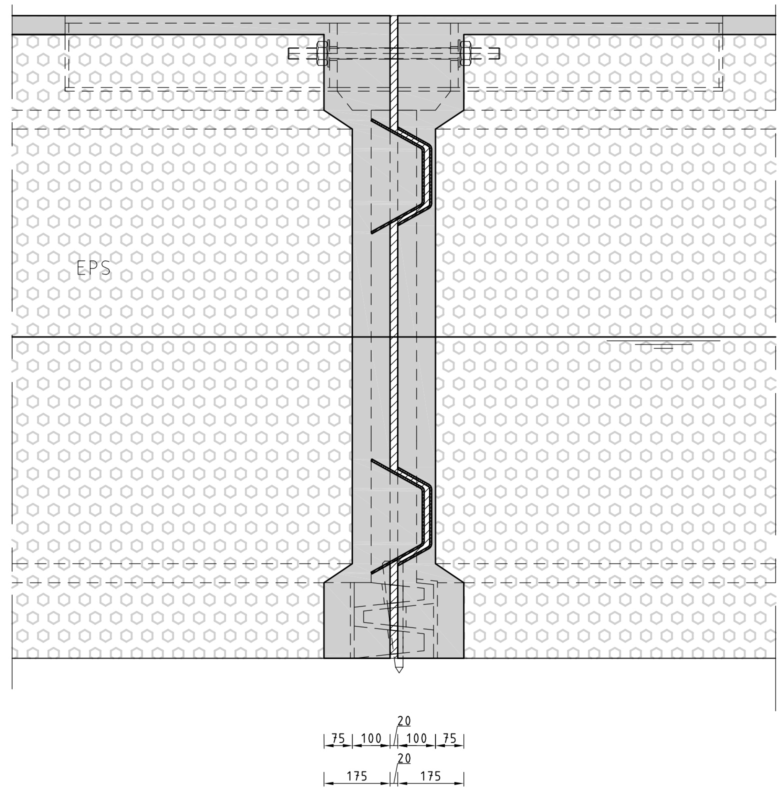
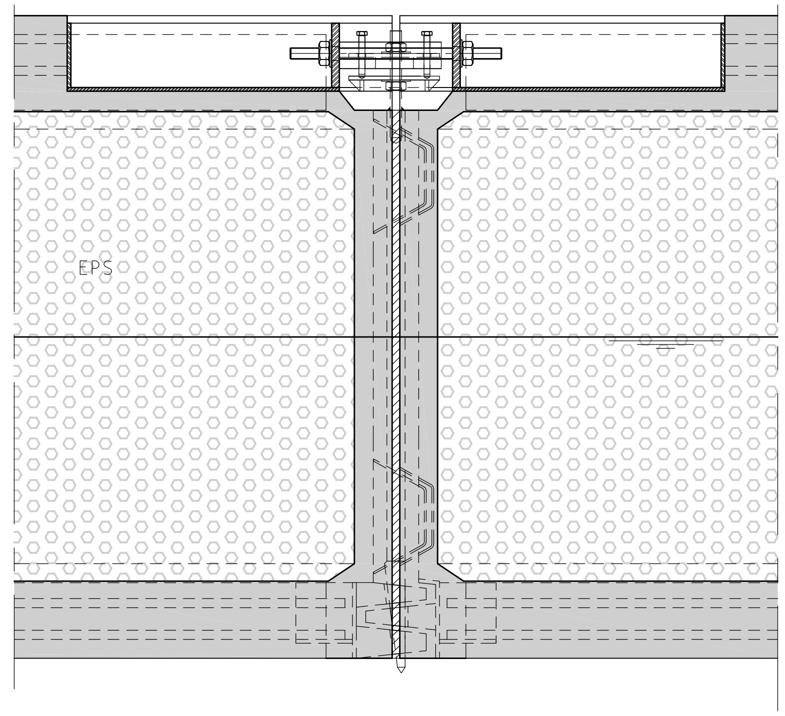
Rigid Connection Floating Pavilion

Drawing 2: Separate and connected

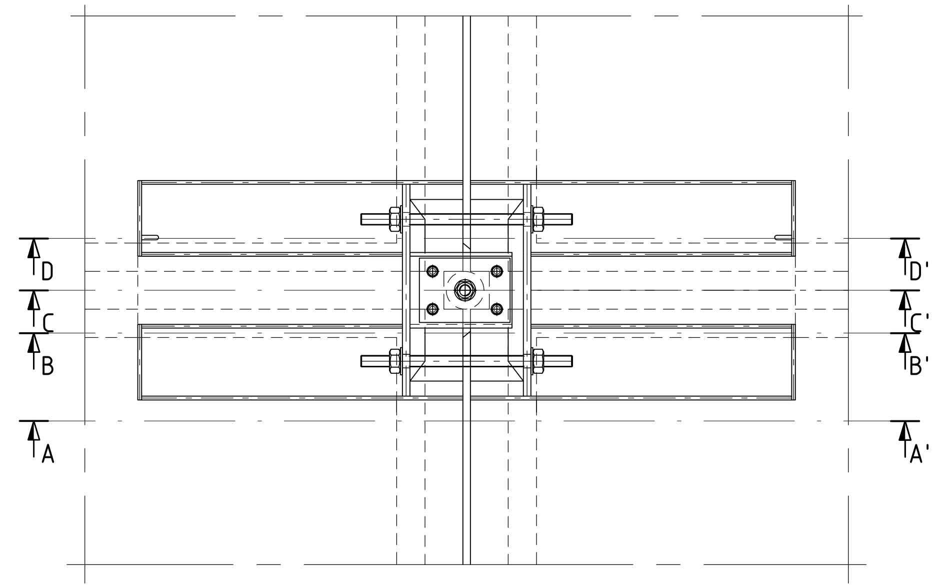
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:20
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	2	Dimensions :	MM



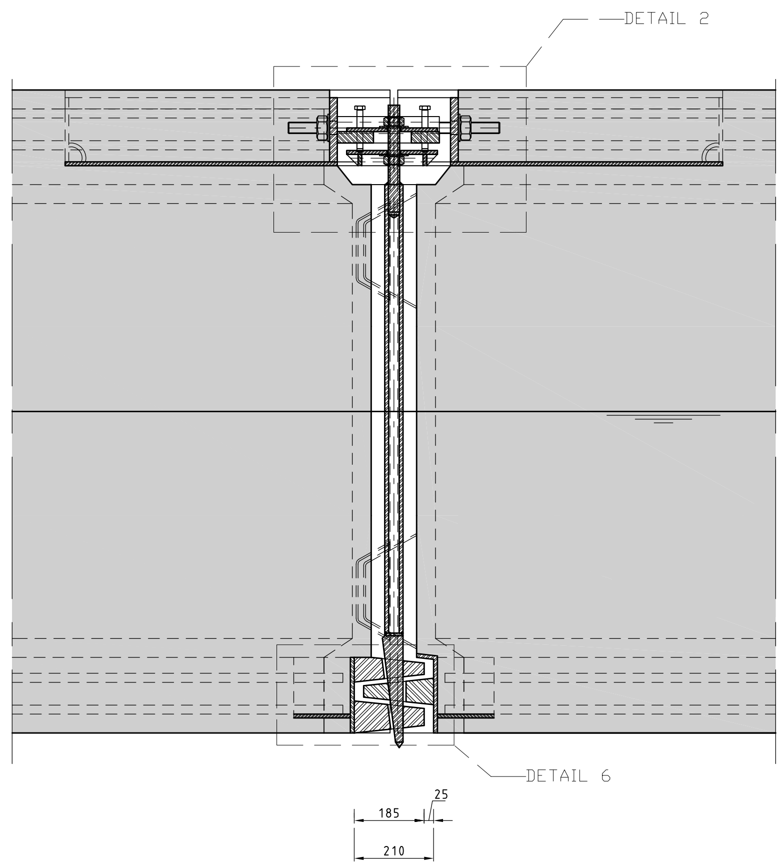
SECTION A-A'



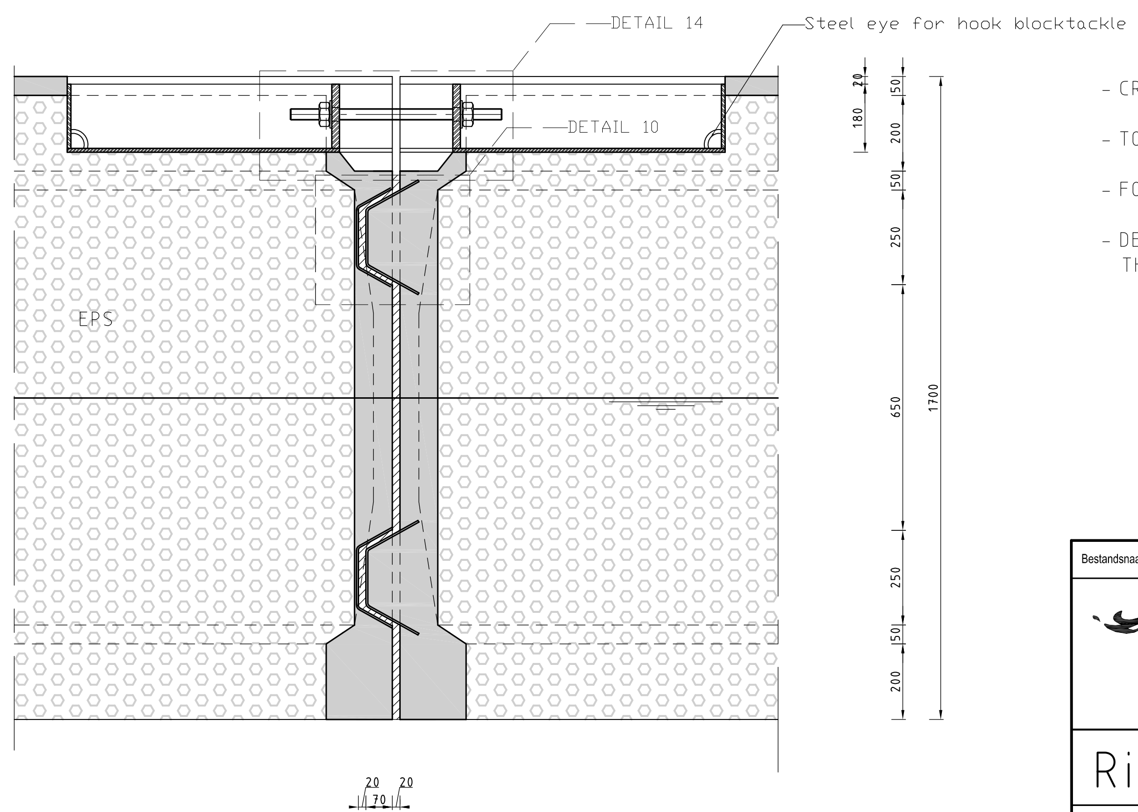
SECTION B-B'



TOP VIEW



SECTION C-C'



SECTION D-D'

- CROSS SECTIONS ARE GIVEN IN DRAWING 4
- TOP VIEW SECTIONS ARE GIVEN IN DRAWING 5
- FOR COMBINATION OF TOP, LONGITUDINAL AND CROSS SECTION SEE DRAWING 1
- DETAILS ARE GIVEN FROM OF DRAWING 6,
  THERE THE DIMENSIONS OF THE CONNECTORS ARE GIVEN

LEGEND
- EPS
- Concrete
- Steel
- Elastic material

Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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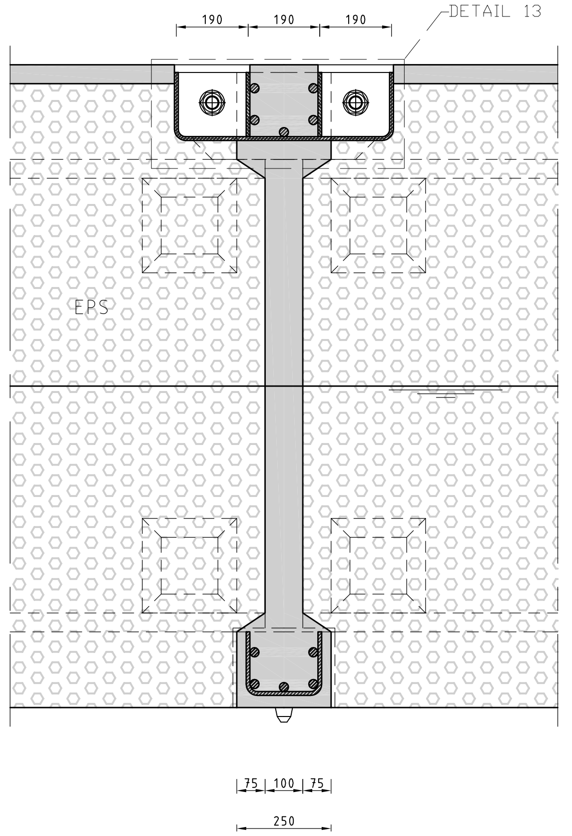
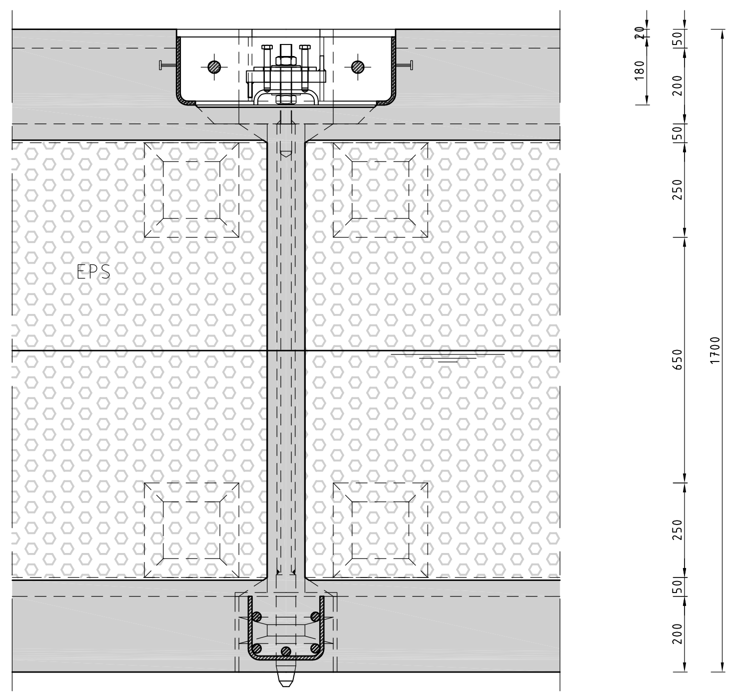
Rigid Connection Floating Pavilion

Drawing 3: Longitudianal Sections A to D

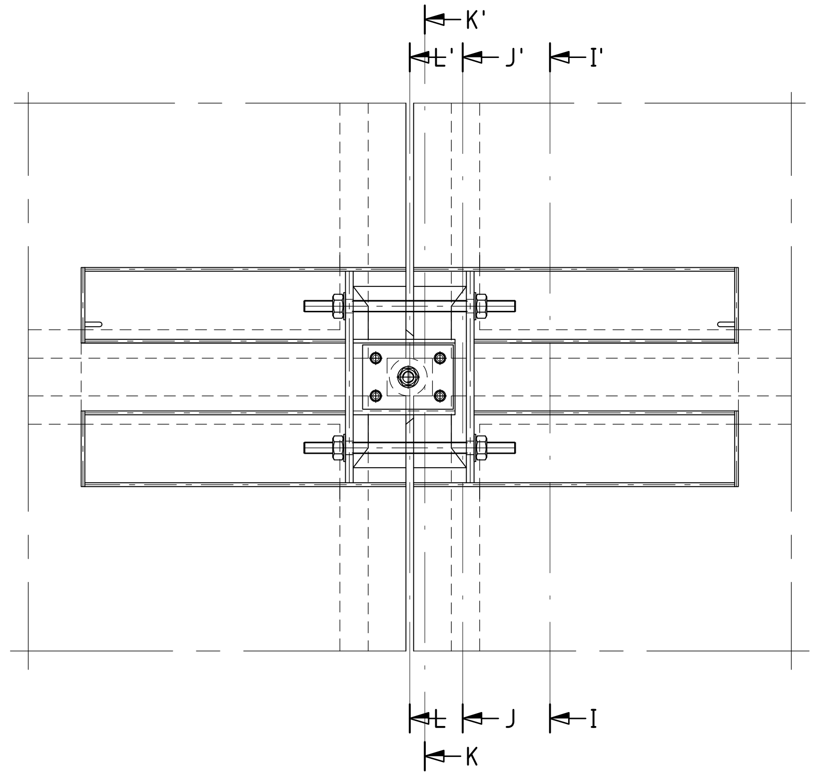
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:20
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	3	Dimensions :	MM



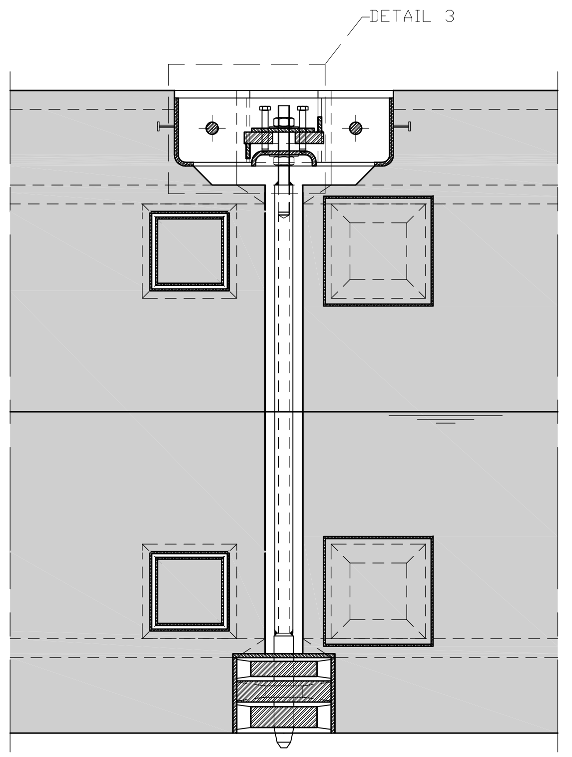
SECTION I-I'



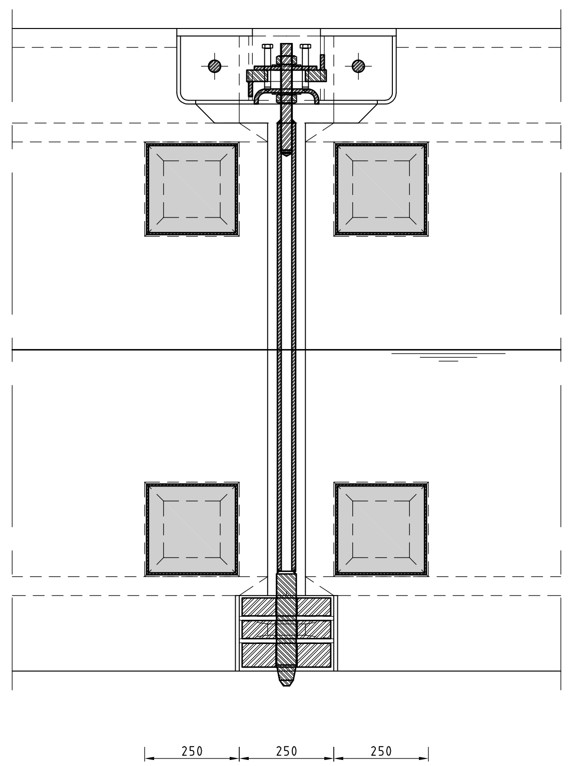
SECTION J-J'



TOP VIEW



SECTION K-K'



SECTION L-L'

- LONGITUDIANAL SECTIONS ARE GIVEN IN DRAWING 3
- TOP VIEW SECTIONS ARE GIVEN IN DRAWING 5
- FOR COMBINATION OF TOP, LONGITUDINAL AND CROSS SECTION SEE DRAWING 1
- DETAILS ARE GIVEN FROM OF DRAWING 6, THERE THE DIMENSIONS OF THE CONNECTORS ARE GIVEN

LEGEND

EPS
Concrete
Steel
Elastic material

Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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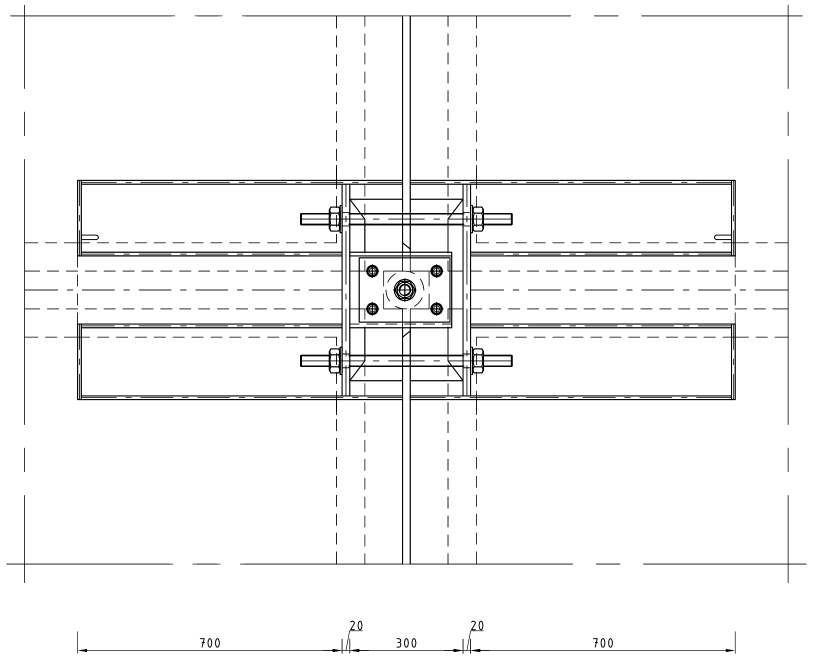
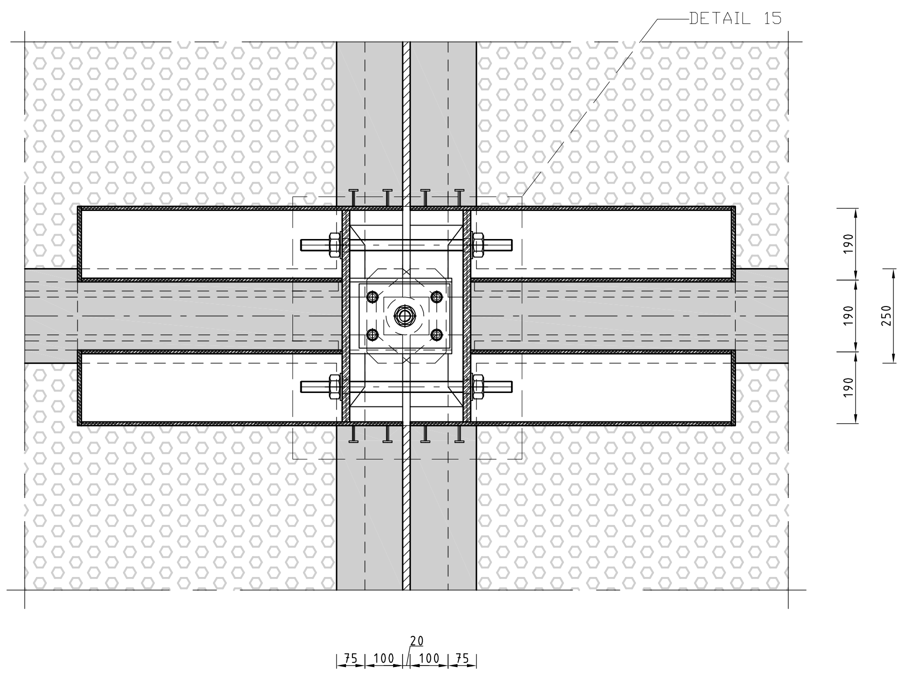
Rigid Connection Floating Pavilion

Drawing 4: Cross Sections I to L

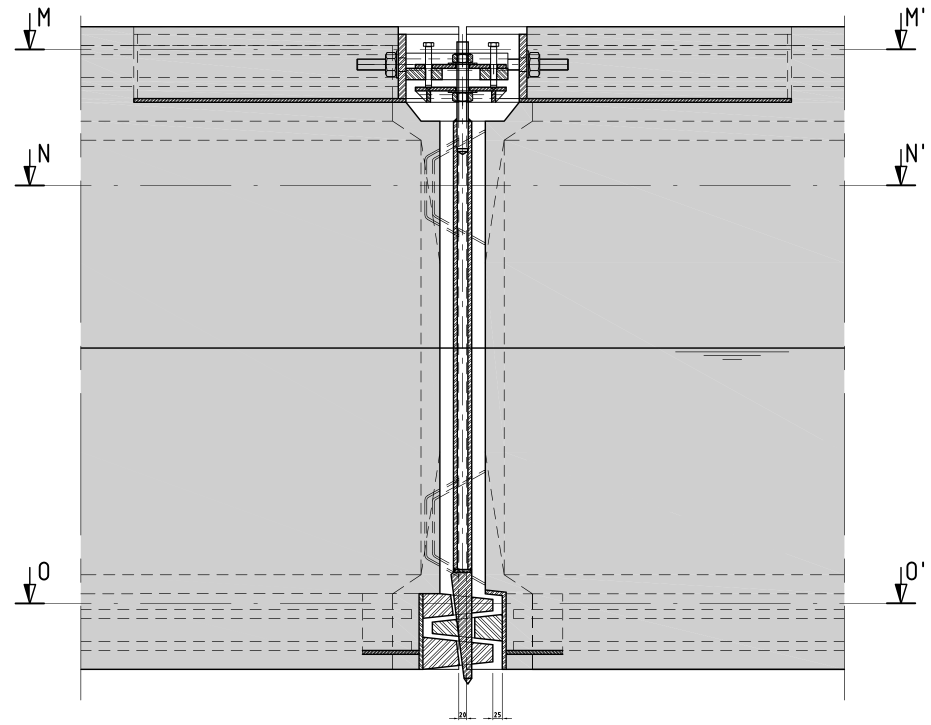
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:20
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	4	Dimensions :	MM



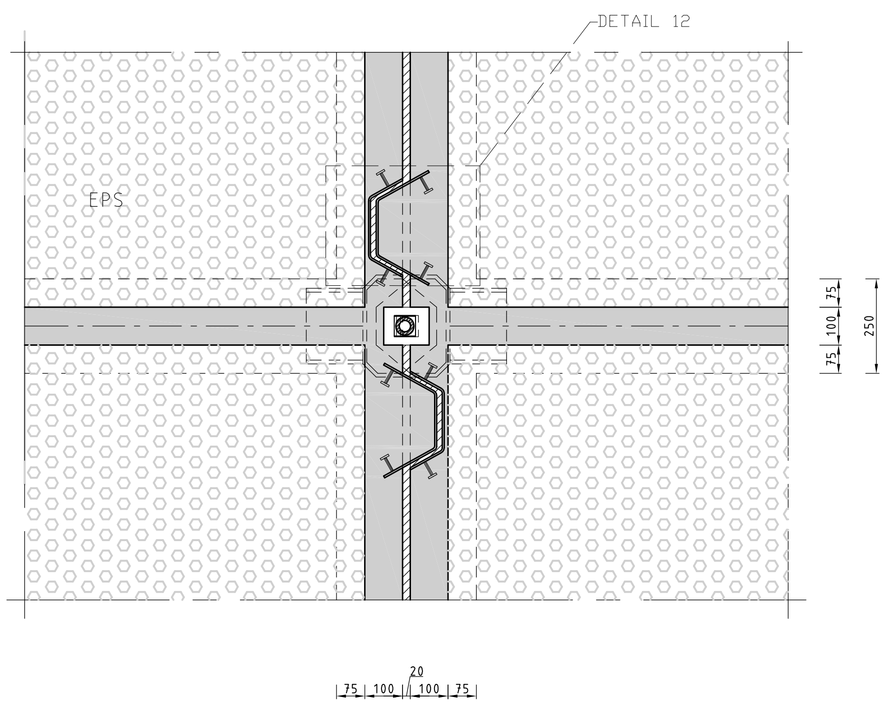
TOP VIEW



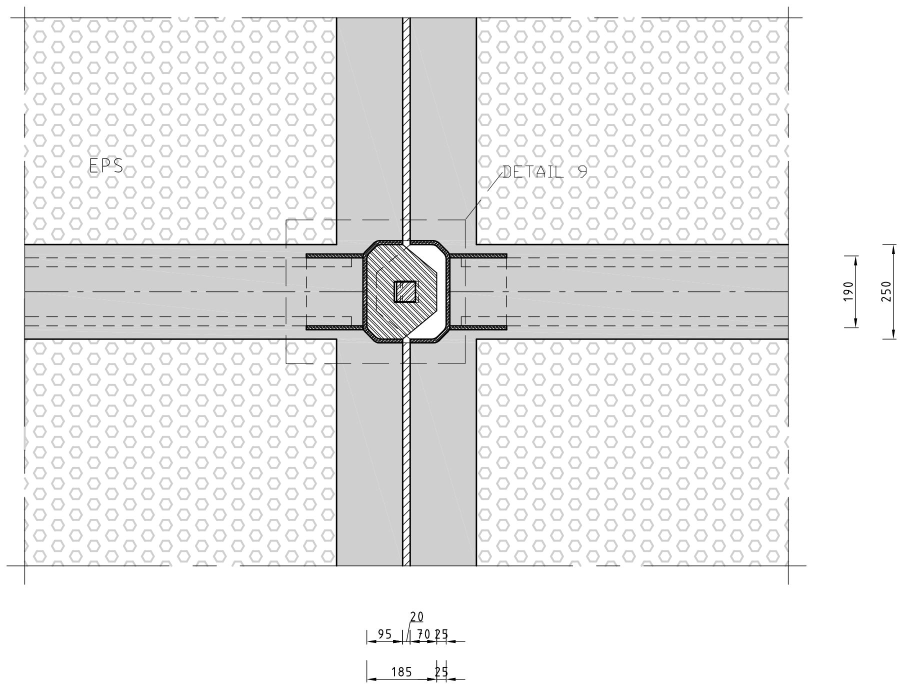
SECTION M-M'



SECTION C-C'



SECTION N-N'



SECTION O-O'

- LONGITUDIANAL SECTIONS ARE GIVEN IN DRAWING 3
- CROSS SECTIONS ARE GIVEN IN DRAWING 5
- FOR COMBINATION OF TOP, LONGITUDINAL AND CROSS SECTION SEE DRAWING 1
- DETAILS ARE GIVEN FROM OF DRAWING 6,
  THERE THE DIMENSIONS OF THE CONNECTORS ARE GIVEN

LEGEND

- EPS
- Concrete
- Steel
- Elastic material

Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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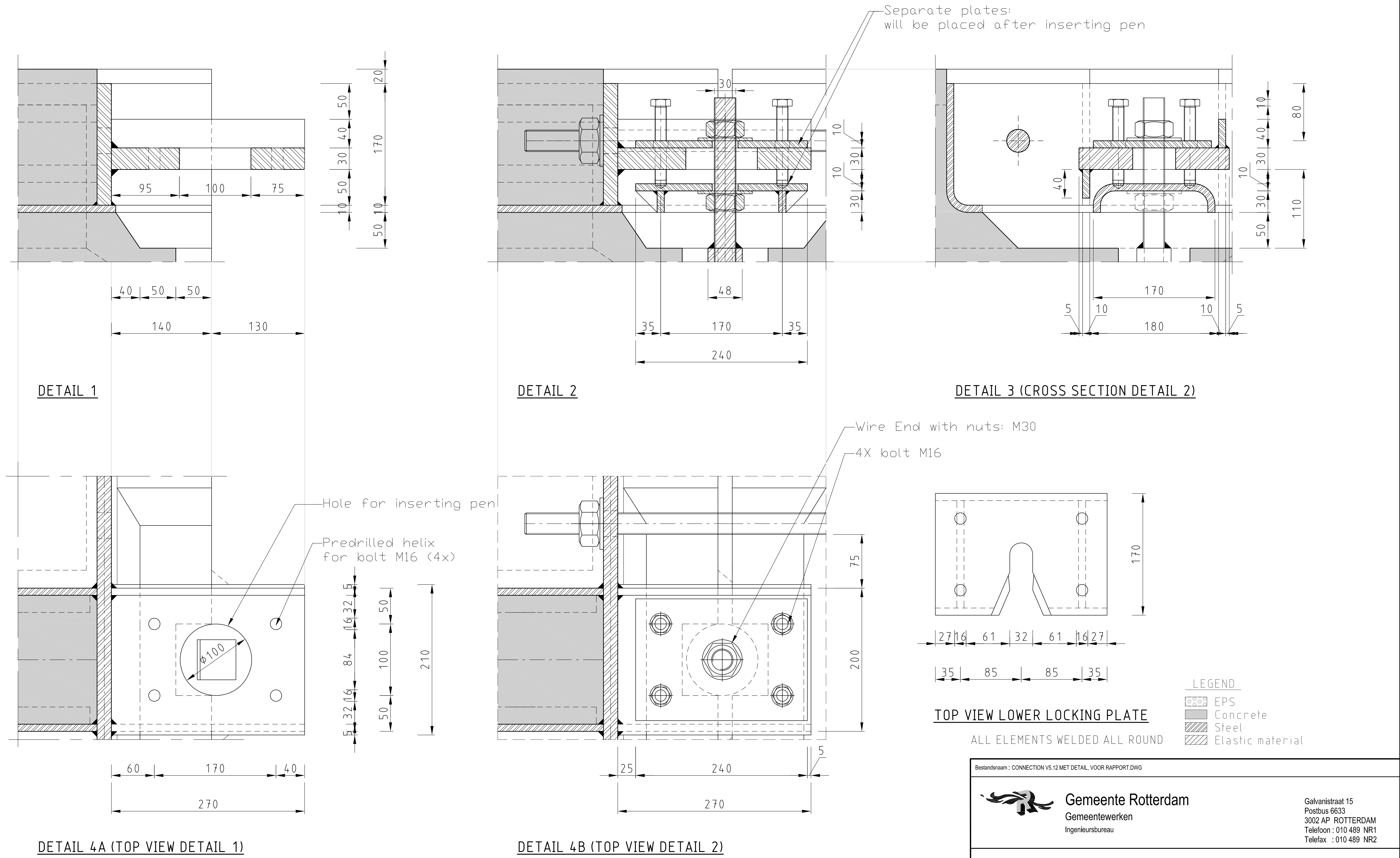
Rigid Connection Floating Pavilion

Drawing 5: Top View and Sections M to O

Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:20
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	5	Dimensions :	MM



Bestandsnaam : CONNECTION V5.12 MET DETAIL, VOOR RAPPORT.DWG

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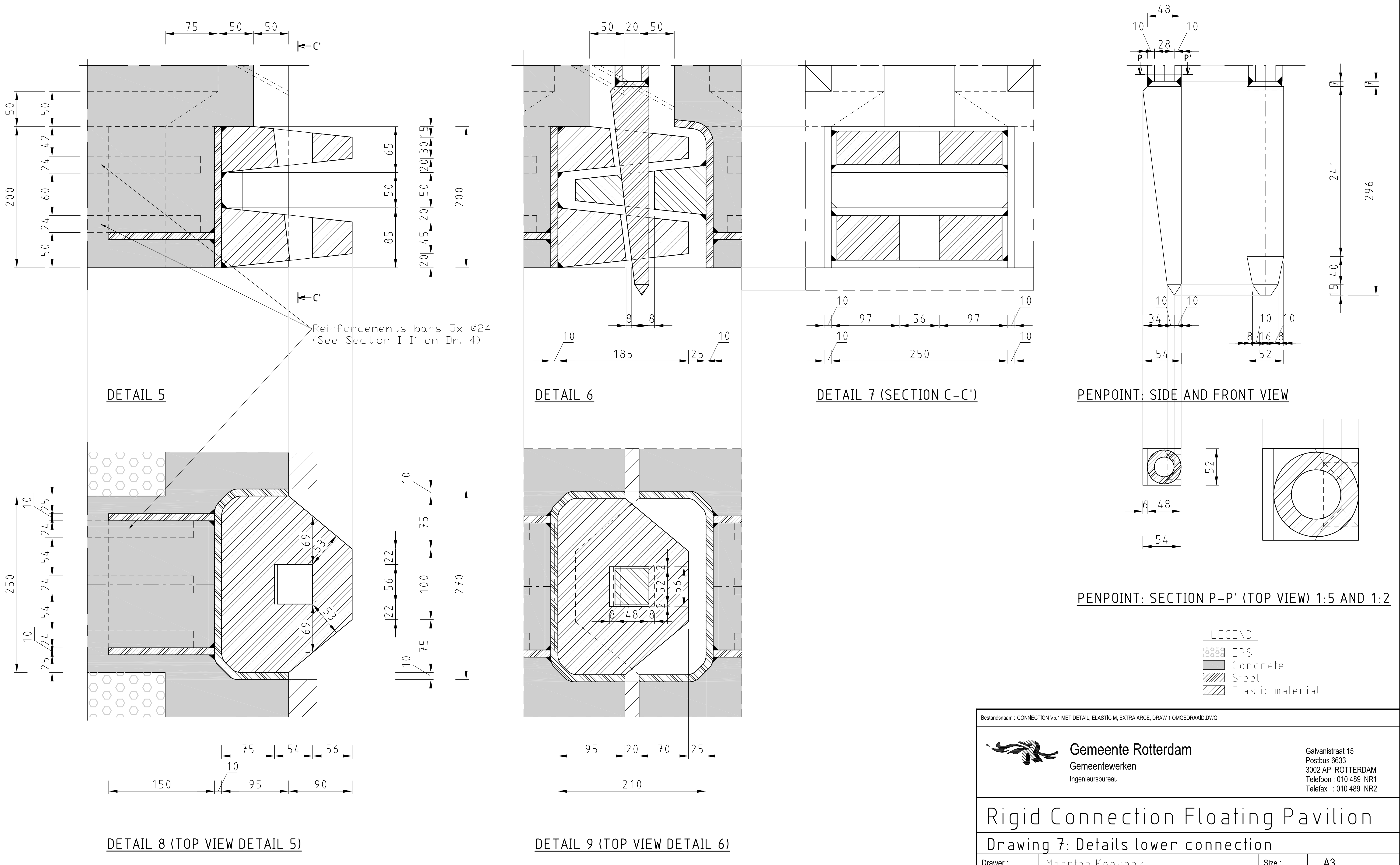
Drawing 6: Fixation of pen for lower connection

Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:5
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	6	Dimensions :	MM



Bestandsnaam : CONNECTION V5.1 MET DETAIL, ELASTIC M, EXTRA ARCE, DRAW 1 OMGEDRAAID.DWG

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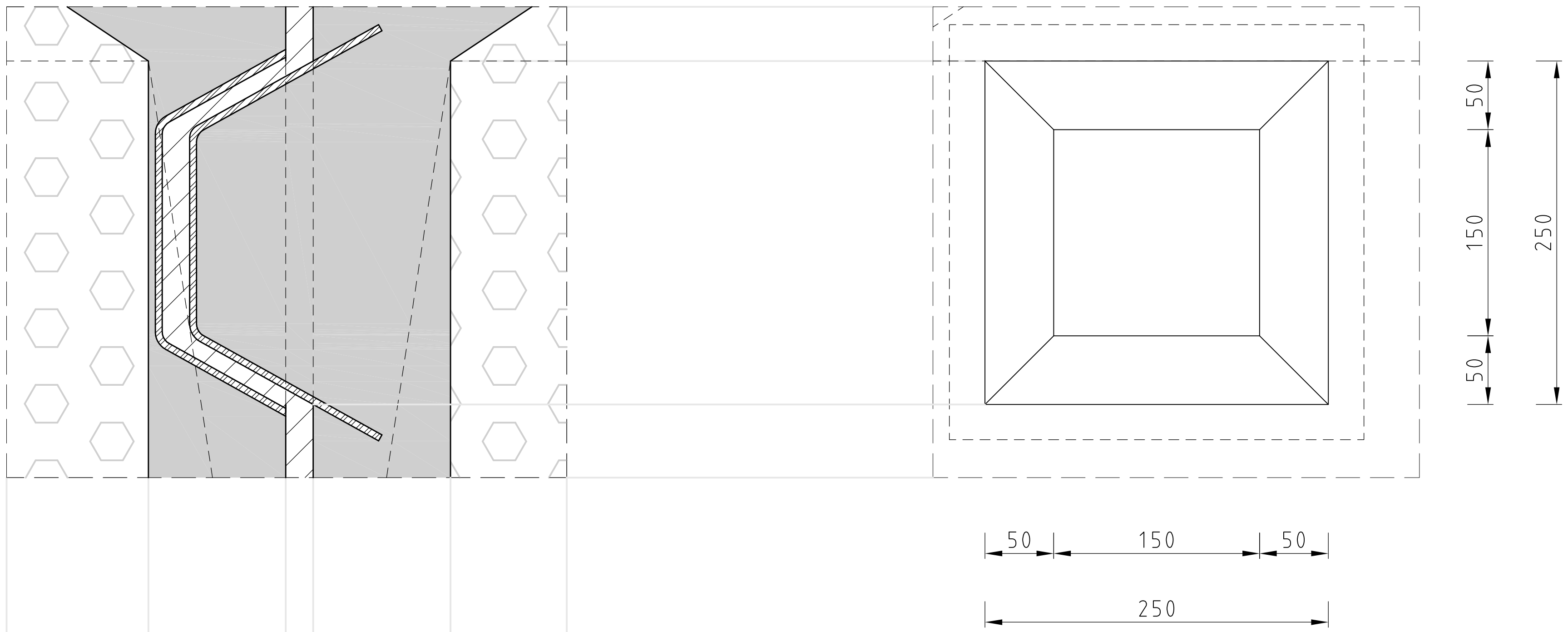
Rigid Connection Floating Pavilion

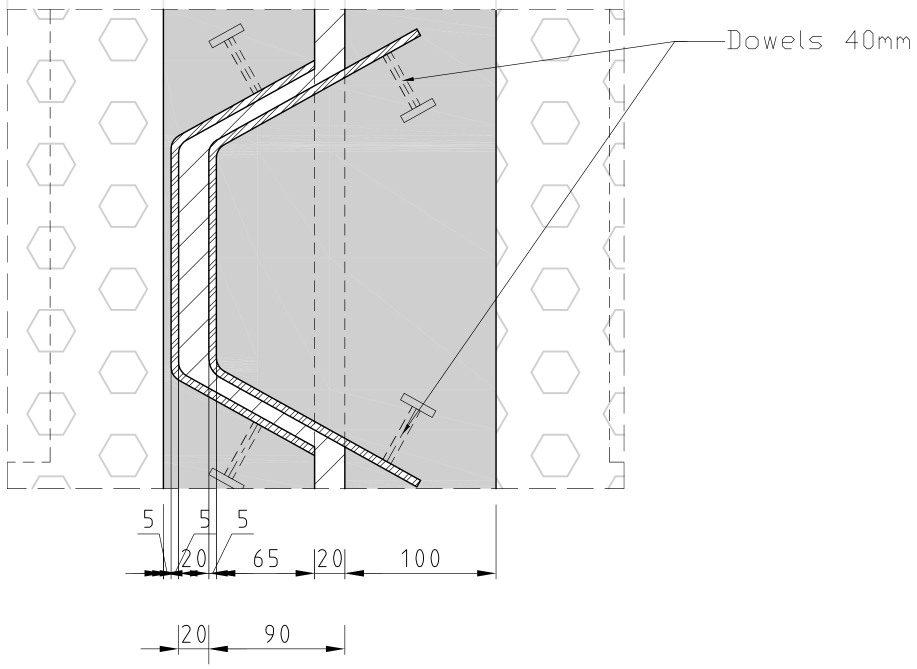
Drawing 7: Details lower connection

Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:5
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	7	Dimensions :	MM



DETAIL 10 (SECTION D-D')

DETAIL 11 (SECTION L-L', NOT CONNECTED)



DETAIL 12 (TOP VIEW SECTION DETAIL 10)

LEGEND

EPS
Concrete
Steel
Elastic material

Bestandsnaam : CONNECTION V5.12 MET DETAIL VOOR RAPPORT.DWG

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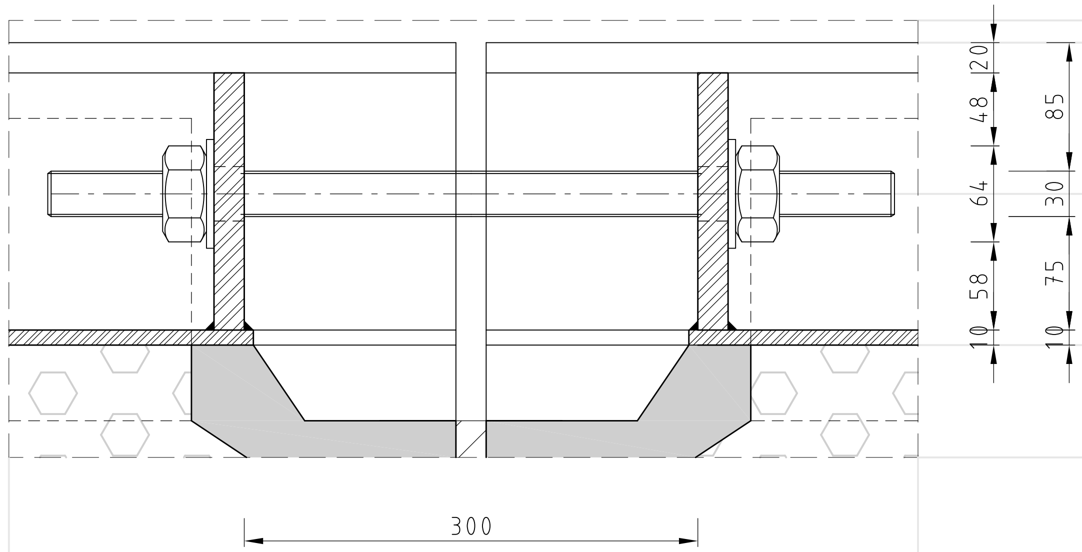
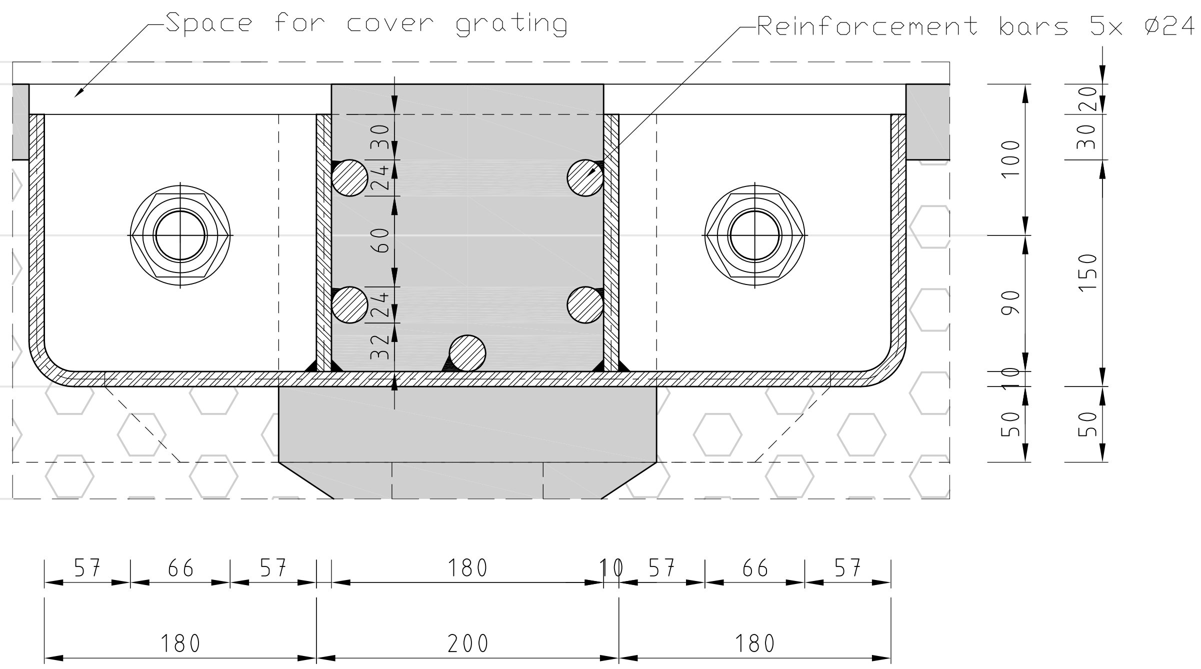
Rigid Connection Floating Pavilion

Drawing 8: Details Trapezoidal Ridges

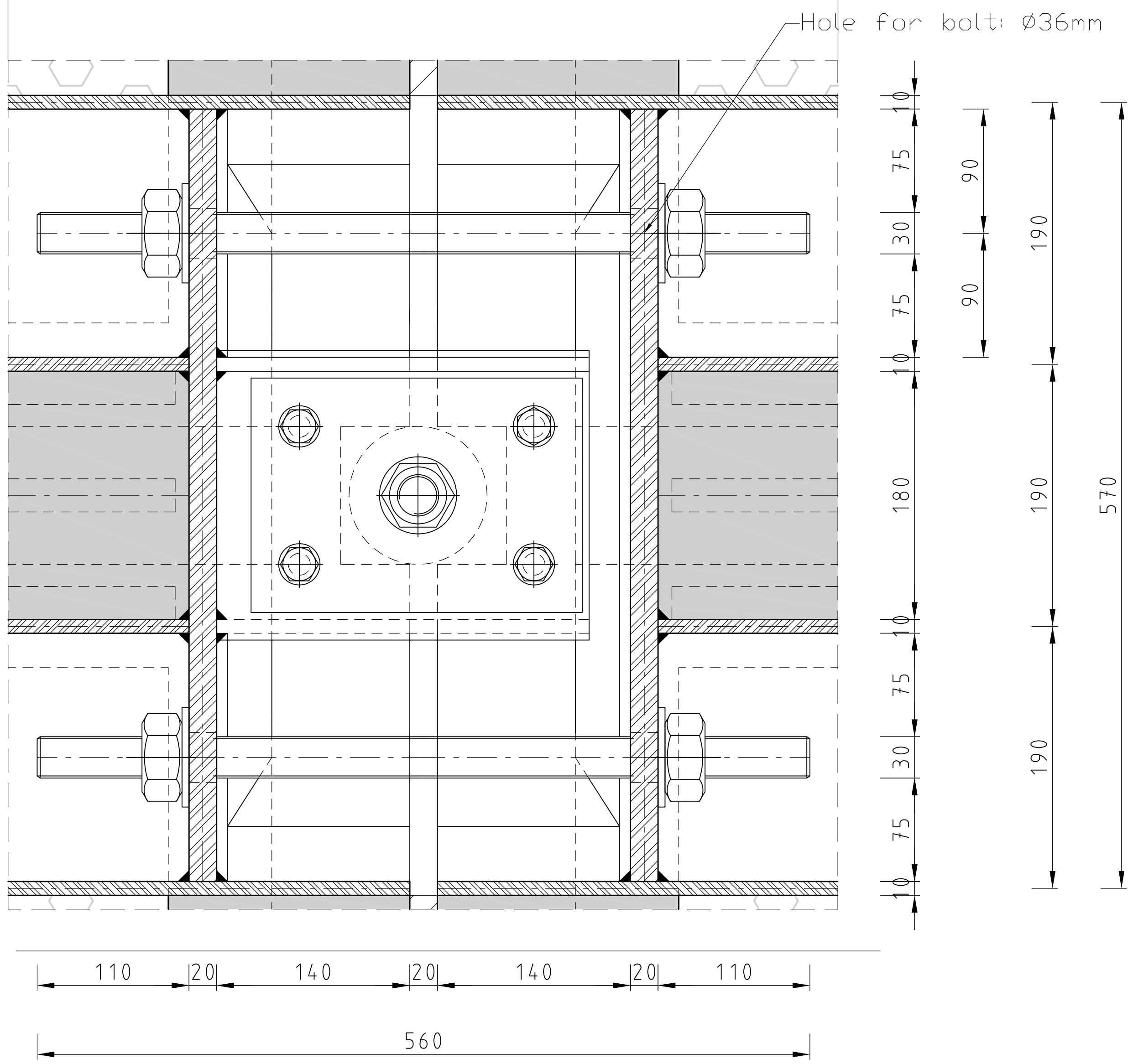
Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:5
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	8	Dimensions :	MM



DETAIL 13



DETAIL 14



DETAIL 15 (TOP VIEW DETAIL 13)

ALL ELEMENTS WELDED ALL ROUND

LEGEND
EPS
Concrete
Steel
Elastic material

Bestandsnaam : CONNECTION V5.12 MET DETAIL VOOR RAPPORT.DWG

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Drawing 9: Details Upper bolts

Drawer :	Maarten Koekoek	Size :	A3
Date :	19-7-2010	Scale :	1:5
Project :	Master Thesis Floating Structures, Case Study		
Drawing :	9	Dimensions :	MM